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FOREWORD

Symposia on themes related specifically to surface water phenomena have been common on the Australian water resources scene for many years. However, the opposite has been the case in the rapidly developing groundwater field. To remedy this situation it seemed more than timely for the School of Civil Engineering of the University of New South Wales to organize a symposium with particular emphasis on contemporary groundwater studies.

The Groundwater Symposium was held at the University on 28th and 29th August, 1969 and was attended by more than one hundred engineers, geologists and others interested in groundwater from all states in the Commonwealth of Australia and from overseas. The symposium programme was centred on four study areas namely :

* Groundwater from unconsolidated sediments
* Groundwater recharge
* Digital and analogue models in groundwater investigations
* Conjunctive use of surface water and groundwater.

Papers were invited from authors having specialized knowledge in a particular study area and the resulting papers form the substance of these proceedings. In addition, written discussion on several papers has been included in this volume. It should be noted that this discussion represents only a small part of that presented from the floor during the open discussion periods.

This opportunity is taken to thank all those who contributed to the success of the Symposium, particularly the authors of papers for their stimulating contributions and ready co-operation. The production of the symposium preprints and these proceedings would not have been possible without the able assistance of Mrs. Ruth Rogan, Miss Elaine Adams, Mr. F. Stein and Mr. M. Spurge.

K. K. Watson,

Editor

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PAPER No. 1

GROUNDWATER IN UNCONSOLIDATED SEDIMENTS

RECENT DEVELOPMENTS IN NEW SOUTH WALES

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SUMMARY

There has been a marked increase in the development of the groundwater resources of unconsolidated sediments in New South Wales in recent years. A review is made of the causative factors, including the discovery of hitherto unknown resources and the trend to large diameter bores. The problems associated with these developments are discussed in relation to construction and testing of high yielding bores and to regional aspects. Attention is drawn to the need for long term systematic investigations to achieve the ideal of operational conjunctive use of surface water and groundwater resources.

1. INTRODUCTION

The importance of unconsolidated sediments as aquifers is well known to students of groundwater. Indeed, Todd (1959) states "Probably 90% of all developed aquifers consist of unconsolidated rocks, chiefly gravel and sand". In New South Wales, it is certainly the case that such sediments are the main source of groundwater suitable and sufficient for irrigation and town water supply purposes. While coastal sand areas of aeolian deposition are locally significant, e.g. the Tomago Sandbeds with regard to augmenting Newcastle's town water supply, and the Botany Sandbeds with regard to industrial use in the Sydney area, the most important sediments are the alluvial formations, particularly those of the inland drainage systems.

There have been some spectacular groundwater developments in the latter system in recent years and it is proposed to consider here the factors leading to this situation and some of the problems associated with it or arising from it.

2. FACTORS LEADING TO RAPID INCREASE IN GROUNDWATER DEVELOPMENT IN NEW SOUTH WALES

At a National Symposium in 1963, the author outlined the factors governing the occurrence, quality and yield of water in alluvial formations, and gave examples of some of the variations encountered in both humid and arid areas throughout Australia (Williamson, 1965). In relation to New South Wales, reference was made to the results of investigations which were in progress in the Lachlan Valley by the Water Conservation and Irrigation Commission and which had revealed hitherto unknown highly productive formations. These were at depths ranging to over 400 feet, much greater than the 100 feet or so then normally explored by landholders seeking groundwater for stock, domestic or irrigation purposes in this valley. From examples given of the yield - drawdown relationships in 8 - inch and 10 - inch diameter screened and developed bores, in which there had been selective screening of only part of the available aquifers, it was clear that there was ample potential for very much higher yields than the 50,000 or so gallons per hour then being obtained from these deeper formations. In point of fact, the limitation on discharge was commonly one imposed by the limitation of the casing diameter on the pump size.

In 1963, too, the Water Conservation and Irrigation Commission commenced a similar investigation in the Namoi Valley, and from the initial results, and those being obtained by landholders encouraged to test bore to greater depths than was their normal practice, it was again evident that there was potential for higher yields than could be obtained from bores with casing diameter then generally used, for irrigation bores - mostly 8 - inch, sometimes 10 - inch, and, rarely, 12 - inch.

Further impetus towards gaining increased knowledge of these groundwater resources was given by the State Grants (Water Resources) Act, 1964, by which the Commonwealth provided a subsidy to the States in order that they might accelerate investigations of their water resources, both groundwater and surface water.

1964 also heralded a series of severe drought years, and nothing seems to promote interest in groundwater so much as drought. Thus, the stage was set.

In the Narrabri - Wee Waa area of the Namoi Valley, cotton - growing had developed rapidly. Commencing with 97 acres in 1959, some 15,000 acres had been planted with cotton by the 1964-65 season. Its irrigation was based on water from the Namoi River, on which a regulating storage, Keepit Dam, had been completed in 1960. However the heavy demands and the severe drought conditions depleted the storage to the extent that there was intense activity by cotton growers to obtain groundwater as an alternative source of irrigation supply. In the flurry of drilling that followed, inexperienced contractors, inadequate equipment and bad construction techniques all caused their toll of abandoned or inefficient bores, but many were successful in obtaining yields of up to about 60,000 gallons per hour. Although useful, these yields were hardly adequate for the irrigation requirements, and there were still the limitations imposed by the diameter of the casing available, as well as by the boring equipment. The drought situation showed no sign of easing, and in 1966 an American cotton-growing interest brought out a contractor from the United States with appropriate equipment and experience to construct large diameter bores. The first of these was completed in September 1966 near Wee Waa, and yielded 3,000 gallons per minute. It is 314 feet deep, and of 20-inch diameter, with gravel envelope to 14-inch diameter slotted casing, admitting water from all of the five aquifers penetrated. Aggregate aquifer thickness is 130 feet.

With this demonstration that such large yields could be obtained, and with the drought situation persisting, many landholders had bores constructed using the same technique, and the number of bores of 20 to 24-inch diameter, with gravel envelope to 14 or 16-inch diameter slotted casing, grew rapidly. Yields ranged up to 4,000 gallons per minute. Furthermore, with evidence of similar groundwater conditions in inland valleys other than the Namoi, such bores have been constructed in the Macquarie, Lachlan and Murrumbidgee Valleys with similar results.

In light of the marketing situation, it is difficult to say for how long this trend will continue, but the evident potential for irrigation from such bores has led some interests to consider relatively large scale development such as individual schemes of a thousand or more acres.

There was no real statistical information on the acreage irrigated with groundwater in New South Wales until 1966-67, when the Bureau of Census and Statistics incorporated this item in its rural questionnaire. It is known that the acreage irrigated with groundwater has expanded rapidly, particularly since about 1964. It is significant that the 1966-67 statistics (considered by the Bureau to be not yet fully reliable) show some 88,000 acres, while those for 1967-68 show some 97,000 acres, an increase of about 12% in one year. The 1968-69 figures are not yet to hand, but a substantial increase is anticipated.

These developments have not been without their teething problems. Indeed, they are still being experienced. Furthermore, they raise questions as to their long term effect. The former problems largely relate to individual bores, while the latter are of more regional consequence. These aspects will be considered separately.

3. PROBLEMS RELATING TO INDIVIDUAL BORES

Prior to the advent of large diameter gravel - envelope bores, irrigation bores in New South Wales were mostly selectively screened. Particularly in the finer formations, the relationship between the size and grading of the aquifer materials and the screen aperture is critical. If the screen aperture is too small, the bore will be inefficient, if it is too large the life of the bore and pump will be endangered by "sand-pumping". The need for careful formation sampling is obvious. Similarly, when a "gravel - envelope" ("filter - pack" is a better term) is used, the proper relationship between the aquifer material, pack material, and slot-size or screen-aperture size, must be observed to achieve formation stability and optimum efficiency. Johnson (1966) gives an excellent treatment of the principles and practices involved in determining the specifications for the pack and screen.

In the initial rush to obtain groundwater supplies in the Wee Waa area in 1964-65, the many abortive results of screened and developed bores were due to the contractors being mainly too inexperienced in this type of bore construction, and/or their plant being inadequate for the work. When the large-diameter gravel-envelope bore came in vogue, in some respects the situation became worse. The construction technique was to use a heavy-duty cable tool rig to construct a bore-hole 20 to 24 inches in diameter without casing, the hole being supported by keeping it filled with mud. As can be readily visualized, it is impossible to obtain proper formation samples or water samples with this technique, though from the rough strata samples, penetration rates and general drilling performance, the experienced driller can usually determine that he is penetrating an aquifer. The usual practice was to then insert and centralize casing of 14 to 16 inch diameter, with a series of ¼ inch wide slots opposite the aquifers (some slotted all of the casing, right to the surface!) and to introduce into the annulus a gravel pack which had the general specification of passing ½ inch mesh and retained on ¼ inch. The bore was then cleaned up as much as possible by bailing, and final development was by pumping and back-washing.

The fact that the same pack material was used for all bores had the inevitable result that where it provided a reasonably appropriate relationship to the grain size and grading of the aquifer material, the bore could perform satisfactorily, but where, as was often the case, some of the aquifers were of materials too fine to be stabilised by the pack, a "sand-pumper" resulted. Damaged pumps, sand-choked irrigation channels and abandoned bores were not uncommon. On the other hand, it was also instructive to find in a number of cases that where 8-inch or 10-inch diameter selectively screened bores with developed natural pack had yielded perhaps 500 to 800 gallons per minute, large-diameter gravel-envelope bores subsequently constructed adjacent to them yielded 2,000 or more gallons per minute. This is probably the result of all or most of the aquifers being allowed to contribute to the bore, but the efficiency of the bores is also higher.

The ideal, of course, is to construct a small diameter test bore to obtain information on the depth, thickness, grain size and grading of aquifer materials, the quality of the water contained in them, and, desirably, to carry out pumping tests of the aquifer to determine at least their transmissivity. By assuming storage coefficient, it would then be feasible to calculate the order of yield that could be expected from a production bore at specified drawdown limits, and what diameter casing would be required to comfortably accommodate a pump to give this or the required discharge. The information on the aquifer materials would allow proper design of the production bore in terms of appropriate filter-pack specifications, and appropriate location and aperture size of slotted sections of casing or of bore screens. Few landholders will go to the expense of pumping tests in test bores, and in the general run of cases this omission is probably justified. However, most of those now proposing large diameter bores accept the logic of first constructing a test bore rather than starting on a very costly large diameter bore (about $1 per inch diameter per foot) and hoping for the best. Apart from the obvious advantages of having reliable information to design the production bore prior to its construction, in many cases test bores have revealed that suitable aquifers are not present at the particular site.

Mud-rotary drilling techniques have the great advantage of speed in test-boring, but sampling and interpretation are more difficult than for testing by cable-tool methods. In a few cases, production bores constructed on the basis of favourable indications from rotary drilled test bores gave very poor yields. The danger of assuming good water quality where testing by mud-rotary method prevents ready water sampling was also demonstrated by a production bore yielding water too saline for irrigation. Contractors now run casing in the rotary-drilled bore holes to obtain water samples. The problem of strata sampling is not easy to overcome, since the samples must be extracted from the drilling mud. The care and experience of the driller are of critical importance. Variations in penetration rates are a valuable guide and automatic time-penetration recording could be utilized here. There is also scope for contractors to use electric and/or gamma-radiation logging equipment, but so far none do so in New South Wales.

The use of slotted casing instead of bore screens is largely governed by economics, since good quality screens are very expensive, e.g. 10-inch diameter screen is about $60/foot. Screen manufacturers have attempted to bridge the cost-gap by producing a cheaper screen with round wire instead of using a specially drawn section to give an aperture that has a sharp outer edge and widens inwards to avoid undue clogging. The former screen type is primarily designed for use with a filter-pack, and an increasing number of landholders are now prepared to incur the additional expense of using such screens. Slotted casing, of course, is more prone to corrosion or encrustation, depending on the characteristics of the water, whereas screens are not only corrosion-resistant but have a greater proportion of open area, with consequent lower entry velocity of the water for a given discharge and thus less likelihood of encrustation. However, in relation to yielding ability the lower percentage of open area of slotted casing is often not as important an aspect as it may appear. This is because the permeability of the filter pack is, or should be, very much higher than that of the aquifer, and, assuming there is sufficient effective open slot area to transmit the yield of the aquifer without significant head loss, the performance of the slotted casing will be satisfactory. (This does not apply where slotted casing is used without a pack, for here the low percentage of open area can lead to poor bore efficiency). Slotted casing can also make development of a bore more difficult, especially with regard to removing the "mud-cake" from the periphery of the original hole behind the filter pack. Development techniques employing high-pressure jetting in conjunction with sodium polyphosphates (e.g. Calgon) to disperse clay offer great promise but are severely handicapped by the limited open area of slotted casing as compared with screens.

Another matter worthy of mention is the testing of yield. The situation is gradually improving, but there are still contractors whose equipment and/or knowledge is inadequate for satisfactory pumping tests. A few years ago, when contractors commenced constructing the large diameter bores previously referred to, it was not uncommon to find that a bore with an obviously high potential had been pump tested at perhaps 500 to 1,000 gallons per minute whereas the owner wished to take advantage of the potential and equip it to pump say 2,000 to 3,000 gallons per minute. Where data from a multiple-stage pumping test were available, in some cases a reasonable, though somewhat strained, estimate could be made of performance at the much higher discharge. However, a common difficulty was that the required discharge would cause the drawdown level to fall below one or more of the aquifers exposed to the bore, and since there was normally no information on the relative contributions of the aquifers, or of the effect of allowing such aquifers to change from their confined state, it was not possible to give a reliable assessment of the performance at higher discharges. There are some landholders, too, who feel that it is too costly to test-pump a bore for more than 8 or 10 hours, even though when they equip it they expect it to pump for 20 or more hours per day for months on end during an irrigation season!

When the Water Conservation and Irrigation Commission constructs an irrigation bore, it normally carries out a multiple-stage test (3 or 4 stages of 2 hours each, at progressively increased pumping rates), allows the bore to recover overnight, and then conducts a 24-hour test at the maximum practicable rate, followed by recovery measurements for a similar period. (The aquifers concerned are almost invariably confined). If the Commission is financing a bore it usually insists on similar tests. In other cases, the Commission recommends that such tests be undertaken, and a brochure outlining the procedures and information required in pumping tests is made available to landholders.

There is obviously much to be done in educating and encouraging contractors and landholders to take advantage of the present-day knowledge and technology of design, construction, development, and testing of high-yielding bores. Indeed, we have much to learn ourselves. From the problems being experienced there is evidently need for national standards on these matters, but it is equally evident that this would take considerable time to effect. As an interim measure, the practicability of incorporating appropriate specifications as a condition of bore license is under consideration. (In New South Wales, the Water Act requires that a license be obtained before a bore may be commenced, altered or deepened).

4. PROBLEMS OF A REGIONAL NATURE

The results of the investigations and developments previously described have also raised problems of a regional nature. The high discharges, the relatively intensive groundwater use in some areas, and the trend towards similar use in other areas, have caused many to express concern as to the long-term potential of the resources. It is evident, too, that many expect that in a relatively short time it should be possible to specifically determine the potential of the groundwater resources of an area or drainage basin. Such people are probably using too direct an analogy between surface water and groundwater reservoirs, and think that it should be almost as easy (or not much more difficult!) to operate a groundwater reservoir as it is a surface storage.

We recognise the much greater difficulty and complexity involved in determining the storage, recharge and discharge of a groundwater reservoir, and are conscious of additional limitations, such as water quality, the danger of inducing saline intrusion, problems of mutual interference between extraction points, economic pumping costs, and so on. Nevertheless, our ultimate aim must be to obtain enough data so that we can, in fact, effectively operate and manage our groundwater reservoirs - hence the need for comprehensive investigation programmes.

Unfortunately, at this stage much more data are required before even a broad quantitative assessment can be made. In fact, in spite of the oft-quoted statement that we live in the world's driest continent, and in view of the history of recurrent droughts and the dependence placed on groundwater, particularly in drought periods, it is rather surprising to find that systematic investigations of groundwater resources of inland drainage systems in New South Wales did not commence until 1957. Prior to this, any regional groundwater studies in these systems were limited to geological mapping and the collection and collation of existing data, so that knowledge of groundwater conditions was effectively restricted by the depths explored by landholders in seeking water supplies. Even in the more heavily populated coastal zone there had been only a few investigations relevant to groundwater prior to this date. (For an historical review of the development of the water resources of New South Wales, see Williamson, 1968). Consequently, in light of the developments in recent years, we are now faced with the mammoth task of investigating and assessing the groundwater potential of the unconsolidated sediments in both inland and coastal drainage systems as quickly as possible.

In the inland drainage systems, investigation programmes are currently in progress in the Gwydir, Namoi, Castlereagh, Macquarie, Lachlan and Murrumbidgee Valleys, but in view of their length, the area and depth of their sediments, the complexities involved, and the available staff and finance, it is obvious that they must be considered as long term projects. However, from the information from investigations and developments, a general picture of the groundwater situation in these valleys is gradually emerging. They may be illustrated by the Lachlan, since this is probably best known. Details of the investigation in progress and its findings are given by Williamson (1967) and only some of the salient points are given here.

Some 5,000 square miles of the Lachlan catchment is in highlands which were uplifted in late Tertiary time, and the rejuvenated drainage system retains little of its original alluvium. Downstream of Cowra, which is on or near the western margin of the uplift block, extensive alluvium occurs, ranging in depth from about 240 feet near Cowra to about 480 feet near Condobolin, and in width from a few miles to ten miles or more. The alluvium consists of two major formations. The upper or Cowra Formation is usually less than 120 feet thick and contains extensive gravel aquifers but the yields from them normally do not exceed about 30,000 gallons per hour. The lower or Lachlan Formation contains more highly productive aquifers, in some cases up to 150 feet thick. "Valley-in-valley" structure is common, with the Lachlan Formation occurring in the entrenched section, and for this reason much use is made of seismic refraction surveys in the course of investigations. With distance downstream the water quality gradually deteriorates, the limit of salinity less than 1,000 parts per million being about 15 miles upstream of Condobolin (over 100 miles down-valley from Cowra). The water becomes progressively more brackish with further distance downstream, but an intriguing aspect is that about 100 miles or so downstream of Condobolin, between Lake Cargelligo and the Lachlan Range, the water quality again becomes good, evidently due to another set of intake conditions. The location and controlling factors of the latter situation are currently under investigation. The good quality water then persists in a large fan-shaped area, attaining a width of about 40 miles and extending for some 20 miles or more beyond Hillston, but again the water gradually deteriorates in quality, both laterally and with distance westward. Bores with potential of up to 4,000 gallons per minute have recently been constructed in this area, and they are still not utilizing all of the available aquifers.

The conditions as described for the Lachlan have been found to apply to a greater or lesser degree to most of the other major inland drainage systems in New South Wales, but, so far as is known, none of the others have an intervening reach of their valley with poor quality groundwater. It is interesting, too, to reflect on the extent to which the widely distributed relatively shallow aquifers have retarded knowledge of the groundwater potential in the sense that they effectively masked the underlying formations for so long.

The large fan-shaped areas in which there is substantial potential for groundwater supplies for irrigation purposes are of particular interest. They occur where the valley loses its defined character and its alluvial formations extend out into the extensive inland riverine plains (Figure 1). At the outlet of the valley, the static level of the groundwater usually approximates river level, but with distance downstream it becomes relatively much lower than river level e.g. 70 - 100 feet or more. In the downstream areas, the river bed usually appears to be more or less impervious, and there is probably little hydraulic connection with the underlying aquifers. Taking into account the relatively low rainfall of these areas, their groundwater levels, and their size and consequent remoteness of the greater part of them from the river, it seems unlikely that appreciable localized recharge could occur, or that the aquifers acting as conduits from the outlet of the confined valleys could transmit sufficient water to keep pace with substantial groundwater withdrawals. Hence, it is expected that intensive development would lead to "mining" of the water, and controlled development may be necessary to ensure reasonable economic life of these resources.

The limits of these fans are only broadly known, but some idea of the task involved in investigating them is given by the size of the areas where on present indications, there is potential for irrigation from groundwater, e.g. (a) Gwydir : Moree area, downstream of Gravesend - 600 square miles (b) Namoi : Narrabri - Wee Waa - Burren Junction area - 600 square miles (c) Lachlan : Hillston area, downstream of Lachlan Range - 1,200 square miles (d) Murrumbidgee : downstream of Narrandera - 2,700 square miles. In the latter area (historically served by surface water for Irrigation Areas and Districts, but now found to be underlain by the largest good quality groundwater resource in the State), the sediments are known to be about 600 feet deep at Narrandera and to thicken westwards into the Murray Basin. The depths of production bores to date usually do not exceed about 450 feet, but aquifers up to 200 feet thick have been penetrated and yields range to 4,000 gallons per minute. So far, of the fan areas, only the Narrabri - Wee Waa - Burren Junction area, in the Namoi Valley, has experienced substantial groundwater development, and this is being kept under close observation to determine its effect.

Considering the overall situation with respect to groundwater in the unconsolidated sediments of the major inland drainage systems, it is evident that in the tributaries and upper reaches where the alluvium is relatively shallow and has good hydraulic connection with the stream, there will be a close inter-relationship between the surface water and groundwater. At the other extreme, in the fan areas, perhaps some 100 to 200 miles downstream, it is probable that the inter-relationship is so restricted that the surface water and groundwater regimes beyond a certain point may be considered as separate entities. In the intervening main or "trunk"

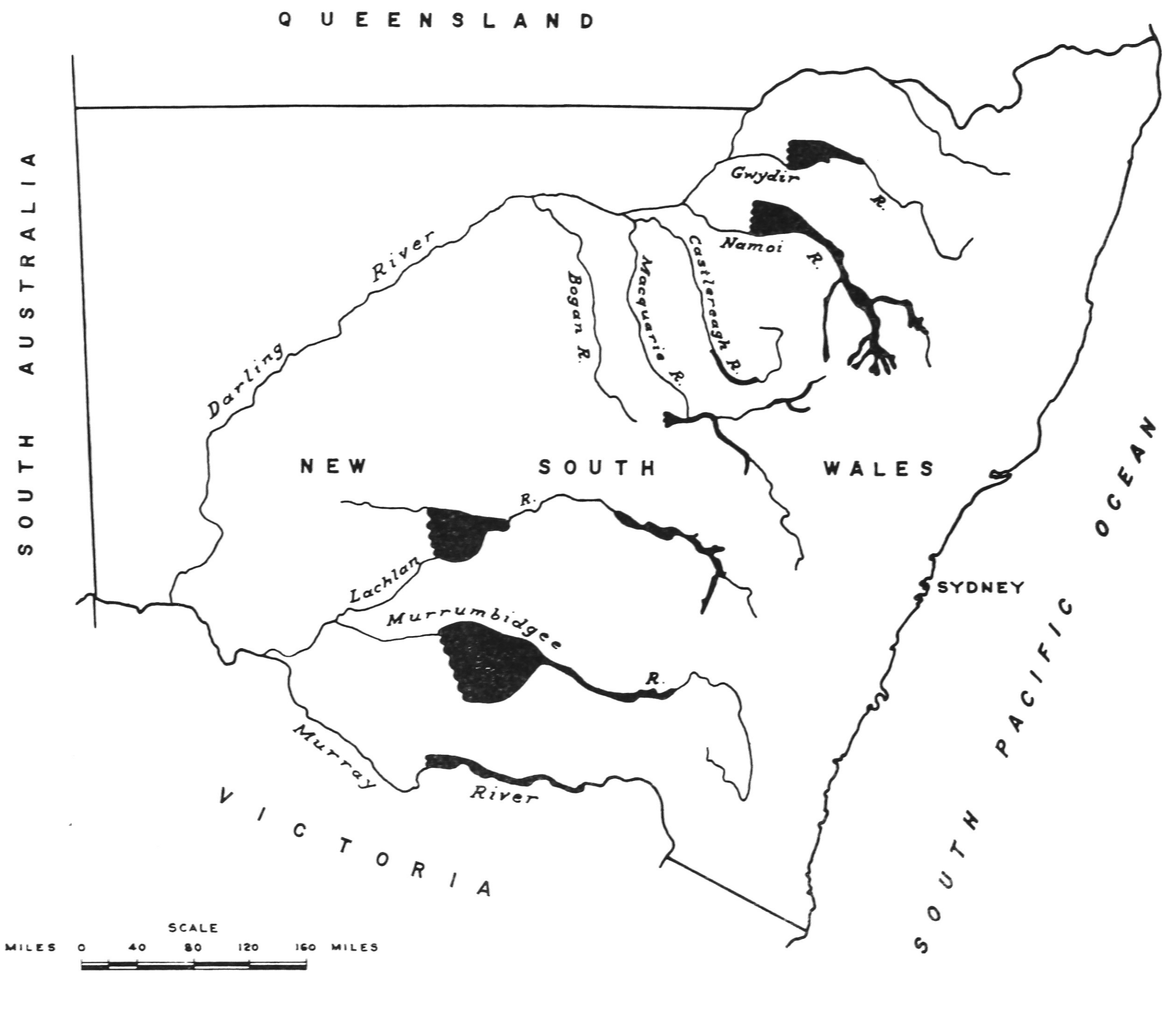

Fig. 1: Areas of unconsolidated sediments in which there is substantial groundwater potential for irrigation purposes in inland drainage systems of N. S. W. Note the extensive fan-shaped areas where the alluviated valleys lose their defined character and enter extensive inland riverine plains.

valley, where there may be zones of very extensive alluvium in which the major groundwater producing areas may be remote from the river, there will be a tendency to overlap or even alternation of these conditions, especially on a relatively short term basis.

5. CONCLUSION

Looking ahead to the ideal of operative conjunctive use of surface water and groundwater resources of a drainage basin, it is not difficult to visualize some of the difficulties involved. To achieve the goal of optimum development requires adequate knowledge of the water resources and the factors governing their behaviour and potential. Present knowledge is far from adequate and the need for considerable expansion of investigation of water resources is self-evident. It will also be impossible to achieve optimum development without appropriate legislation to effect such controls as are necessary, and this aspect also warrants considerable attention.

Within the limits of space and time for this paper it has been possible to give no more than a brief outline of the complexities and problems we are facing in determining the extent and potential of groundwater resources of unconsolidated sediments in New South Wales, and in guiding their development. They are but some of the challenges to present and future workers in the absorbing field of groundwater.

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PAPER No.2

NON-DARCY FLOW IN THE VICINITY OF WELLS

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SUMMARY

This paper deals with the velocity - hydraulic gradient relationships which are applicable to non-Darcy flow in the vicinity of wells and their application in determining flow rates and drawdowns. Methods available for computing non-Darcy flow nets are discussed and practical difficulties involved in determining the constants in the head loss equation are considered.

1. INTRODUCTION

An abundance of published information is available on the subject of flow to wells for cases in which Darcy's law is valid. The simplicity of the linear relationship between hydraulic gradient and flow velocity has encouraged a great deal of mathematical analysis of particular geometrical configurations of aquifers and wells.

There is, in contrast, little information available on non-Darcy flow which may, under certain conditions, occur in the vicinity of a well. It is only over recent years that the hydraulics of non-Darcy flow through granular materials has been reasonably well understood. The lack of finality in this understanding is, however, exemplified by the fact that a quarter of the subject matter of the forthcoming 13th Congress of the International Association of Hydraulic Research is devoted to non-Darcy flow through porous media.

Despite the increased amount of research effort which has been put into non-Darcy flow, a number of major problems still confront anyone who wishes to determine flow rates and hydraulic gradients outside the limit of validity of Darcy's law. Among these are the problems of defining the upper limit at which this law ceases to apply, determining the constants in the velocity-hydraulic gradient relationship for the particular granular material under consideration, and computing non-linear flow nets for the particular problem in hand.

It is the purpose of this paper to outline the methods and data which are currently available for application to well flow calculations and draw attention to those areas in which further research is required.

2. VELOCITY - HYDRAULIC GRADIENT RELATIONSHIP FOR NON-DARCY FLOW THROUGH GRANULAR MATERIALS

2.1 Straight Parallel Flow

Flows which obey Darcy's law have insignificant inertial effects and the complication of convergent, divergent or curved flow affecting the velocity-hydraulic gradient relationship does not arise. The possibility of this relationship being affected by these flow properties does arise when inertial effects are significant or turbulence occurs so the discussion in this section assumes that the flow is straight and parallel if the flow is outside the Darcy regime.

Darcy's law is frequently written in the form

$$V = kS$$

where V = flow velocity = $\dfrac{\text{discharge}}{\text{gross cross sectional area}}$

S = hydraulic gradient

k = coefficient of permeability

It may be re-written as

$$S = aV^n$$

when $a = \frac{1}{k}$ and depends on the properties of the fluid as well as of the porous medium, as does k.

Experimental investigations have shown that the linear relationship expressed above, ceases to hold at high flow velocities. Figure 1, which is a log-log plot of flow velocity versus hydraulic gradient for a number of rounded and angular materials shows that at sufficiently low velocities a linear relationship between S and V occurs but that at higher velocities the gradient increases at a greater rate for a given increase in velocity.

The exact nature of the line joining experimental points for a given granular medium is the subject of debate between various researchers. Theoretical considerations suggest that the line should begin as a straight line of 1:1 slope and gradually curve upwards until a slope of 2:1 is reached when flow becomes fully turbulent. However, careful experimental work reported independently by a number of authors[(1)-(3)] seems to indicate that the line is actually made up of a series of straight lines (probably four including the Darcy regime) with relatively abrupt charges from one to the other.

Equations which are commonly used to describe the relationship are:

$S = aV^n$ with a and n constant for particular straight line sections on the log-log plot or varying continuously along a curved line,

and $S = a\ V + bV^2$.

The physical interpretation of the velocity-hydraulic gradient relationship based on visual observation of dye streaks[(4)]

and measurements of fluctuations in velocity[5] and pressure[2] is as follows. In the linear or Darcy regime, laminar flow is present throughout and inertial effects are negligible. As the velocity increases, inertial effects begin to become significant and the hydraulic gradient required to cause a given flow velocity is greater than predicted by Darcy's law. At higher velocities still, eddy shedding occurs in the wake of individual grains and the rate of head loss becomes even higher. Finally, as the velocity is increased, turbulent spots appear and eventually the whole flow becomes turbulent, with laminar sublayers around the particles except for extreme velocities or roughnesses.

If the experimental evidence showing a number of straight line sections on a log-log plot is accepted, it would appear that the range of velocity over which the changes from one regime to the other occur is small compared with the range over which a particular regime remains established.

Wright[5] named the various types of flow net as "laminar", "steady inertial", "turbulent transition" and "fully turbulent" respectively.

The experimental data obtained from flow tests is frequently expressed in terms of a friction factor - Reynolds number relationship. For example, the data of Figure 1 is reflected in this form in Figure 2. The Darcy friction factor, f, is applied in the equation

$$S = \frac{fV^2}{2gd}$$

where g = gravitational acceleration

d = particle "diameter" or some other length characterizing the geometry of the particles or pores.

The difficulty of describing the geometry of granular media has so far prevented the development of a generally applicable friction factor - Reynolds number chart and further research is required in this field. At present it is not possible to make a reliable prediction of the upper limit of the Darcy, or any other, regime on the basis of a selected Reynolds number. A wide range of Reynolds number has been reported for the upper limit of applicability of Darcy's law. The data of Figure 2 alone show a variation from approximately 2 to 60.

Until generally applicable friction factor - Reynolds number charts become available for particular geometrically similar series of granular media there is little to be gained by using this

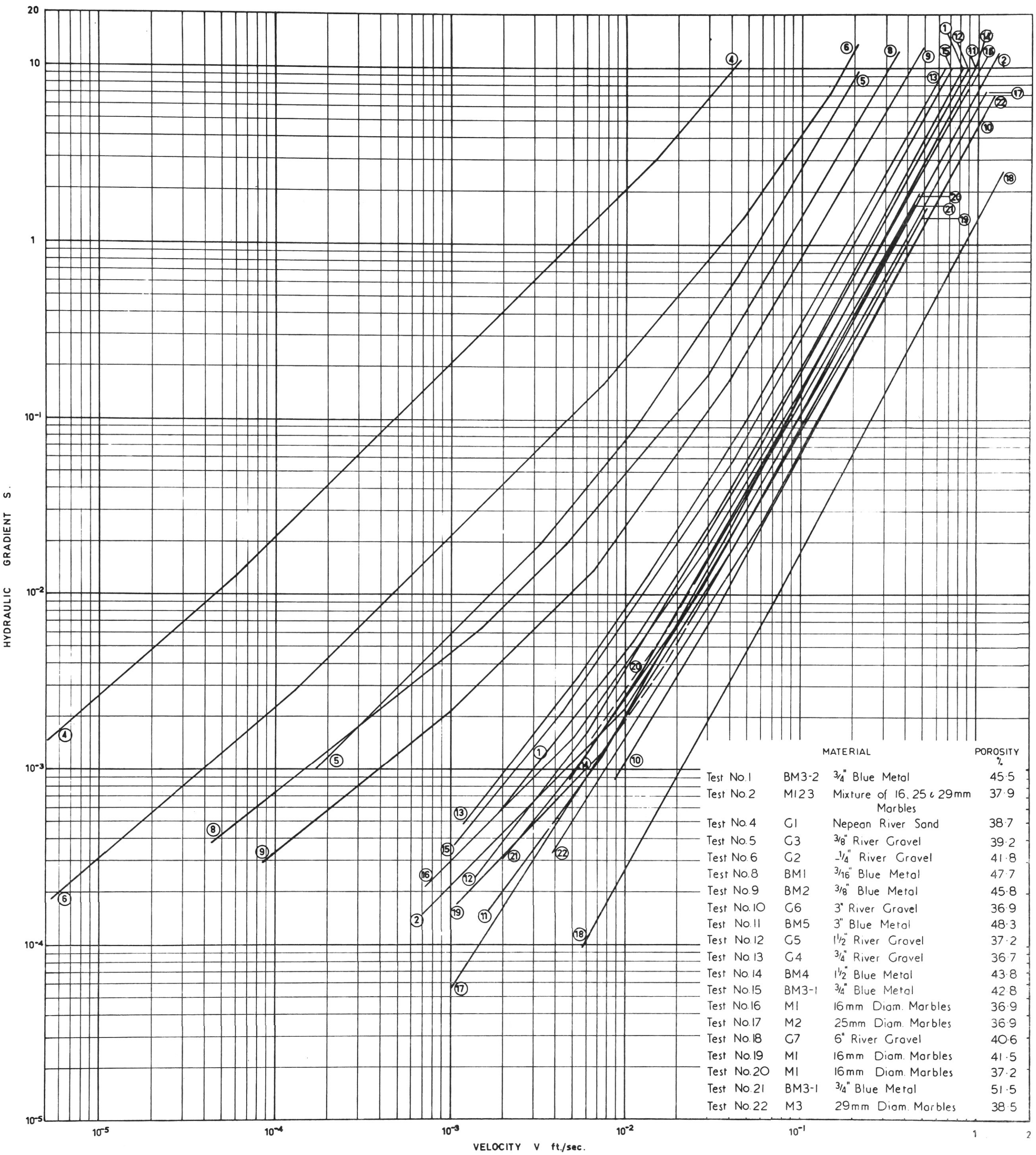

Fig. 1: Velocity-Hydraulic Gradient Lines from Permeability Tests.

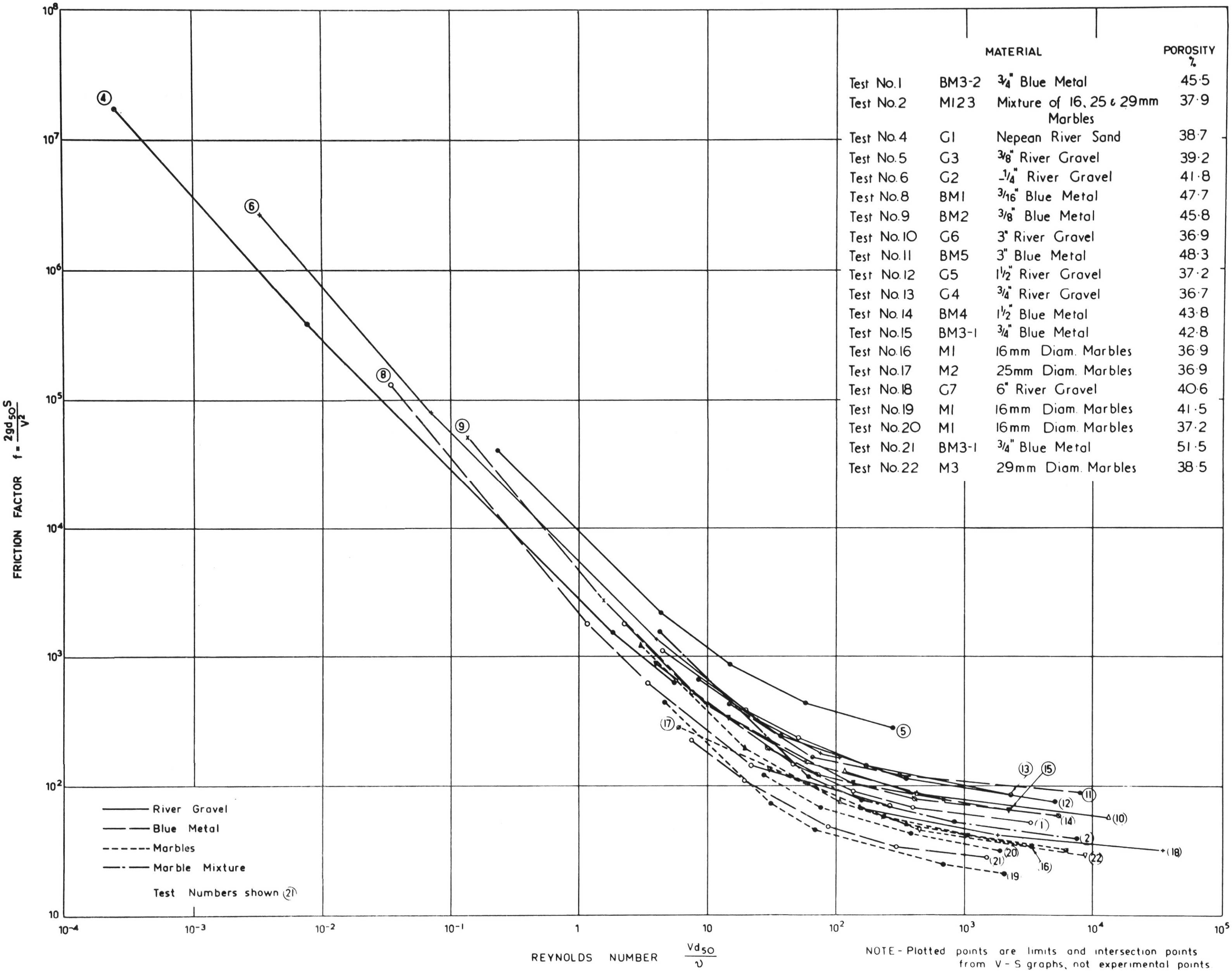

Fig. 2 FRICTION FACTOR VERSUS REYNOLDS NUMBER

form of expression. The direct S versus V relationships are easier to use provided the constants a and n are adjusted for viscosity change as necessary. For calculations of flows in shallow unconsolidated aquifers the effect of viscosity change is usually small since the range of likely water temperatures is restricted.

2.2 Converging Flow

Almost all of the experimental work aimed at determining velocity - hydraulic gradient relationships has been carried out in parallel flow permeameters. Wright[5] used a converging flow permeameter as well as a parallel flow permeameter and found that for the same material, porosity and flow velocity, head losses were lower in converging flow than in parallel flow. The effect was attributed to a reduction in the areas of the wakes behind particles and to a delay in the onset of turbulence.

Since flow in the vicinity of small diameter wells converges rapidly and it is in close proximity to such wells that non-Darcy flow is likely to occur, the effect of convergence on the constants in the velocity-hydraulic gradient equation requires further research. At present there is insufficient data to enable its effect in particular circumstances to be evaluated.

3. PREDICTION OF NON-DARCY FLOW IN THE VICINITY OF WELLS

Non-Darcy flow may occur in either the aquifer material near the well, the gravel pack material or perforated screen if velocities exceed a limiting value.

Since perforated screens are usually quite thin, non-linear losses through them are best treated by a loss coefficient with an abrupt loss assumed from one face of the screen to the other. Non-Darcy flow through gravel packs of finite thickness and aquifer material may not be treated as abrupt losses. The determination of the free water surface profile and flow rate requires the computation of non-linear flow nets. A discussion of methods available for doing this will be delayed until later in the paper.

The question of sampling the aquifer material in the vicinity of the well must necessarily be considered. This is not such a serious problem as sampling the aquifer in general because there is unlikely to be marked variations in the aquifer characteristics within the smaller zone in which non-Darcy effects are likely to occur. The greatest difficulty lies in determining the undisturbed state of the aquifer material from the samples and the final developed state and also the actual as opposed to intended state of gravel packs as placed. Small changes in porosity have such a large effect on the constants in the velocity-hydraulic gradient equation that accurate knowledge of the actual conditions in gravel packs and the developed aquifer material are necessary if gradients in the vicinity of the well are to be calculated accurately.

At present, if one has reliable samples from the vicinity of the well and the state of compaction is known, the only accurate way of determining the head loss-velocity characteristics is to carry out permeameter tests. The only accurate method of determining the additional drawdown due to the non-Darcy flow is to compute a non-linear flow net for the particular case in hand. This non-linear net will be asymptotic to the linear net which is applicable at greater distances from the well.

4. COMPUTATION OF FLOW NETS FOR NON-DARCY FLOW

None of the analytical or graphical procedures applicable to Darcy type flow can be applied to the non-Darcy type flows. Only a few attempts have been made to develop a field equation suitable for non-Darcy flow such as the equation suggested by Parkin (6) and later on extended by Kirkham (7). Parkin performed work on flow through rockfill embankments in the University of Melbourne.

Exponential formula of the form

$$S = aV^n \tag{1}$$

has long been used to describe non-Darcy flow in porous media. On the basis of friction-factor Reynolds number, or head loss-velocity correlations from laboratory permeability tests (2) it can be shown that the constants a and n are constants for a particular homogeneous, isotropic medium and also vary with the velocity of flow. Parkin, by combination of a vectorial form of the equation (1) with the vector equation of continuity, derived the resulting expression

$$(\phi_{xx} + \phi_{yy})(\phi_x^2 + \phi_y^2) + (N-1)(\phi_x^2 \phi_{xx} + 2\phi_x \phi_y \phi_{xy} + \phi_y^2 \phi_{yy}) = 0 \tag{2}$$

in which

$\phi = \frac{1}{a}$ x piezometric head and $N = \frac{1}{n}$ and subscript x and y refer to differentiation in the x and y directions respectively.

Kirkham [7] solved equation (2) for a turbulent flow around a 90^{o} metre bend taking into account variations in a and n in the exponential energy loss equation. Curtis and Lawson [8] applied the same equation successfully to describe the seepage pattern through the bank of rock fill dams. While the above mentioned workers all made use of the well known finite difference technique in the computer solution of the field equation, Fenton [9], by extending the basic method of finite elements and applying it to the turbulent flow field has achieved significant savings in calculation time.

The above description on the work on rock-fill dams suggests a possible method of solution in the investigation of the non-Darcy flow field in the vicinity of wells. By combining a suitable energy loss equation, such as equation (1), with the continuity equation a general field equation can be derived and by applying computation techniques the non-linear flow net could well be solved.

It is of interest to remark that while the well-known finite difference method represents a direct approximation approach to the differential equation type of formulation, the finite element procedure is an approximation applied to the variational formulation terms [10]. The freedom of choice of the shape of elements, and the ease of treatment of non-homogeneous situations make the later method more versatile and hence is expected to be more suitable to the solution of non-linear Darcy flow field in the vicinity of wells.

Zienkiewicz and Cheung [10] have discussed the application of the finite element method to the solution of differentiation equations of the type

$$\frac{\partial}{\partial x}\left(K\frac{\partial\phi}{\partial x}\right) + \frac{\partial}{\partial y}\left(K\frac{\partial\phi}{\partial y}\right) - q\,(x,y) = 0$$

Zienkiewicz, Mayer and Cheung [11] used the finite element method to solve problems with anisotropic seepage. Finn, [12], Taylor and Brown [13] applied the method to free surface Darcy flow. McCorquodale and Ng [14] applied the method to solve the non-Darcy flow through a rectangular section. All the above computations were done on high speed digital computers.

Currently, a research programme has been planned in the Water Research Laboratory, N.S.W., in order to develop a sounder theoretical basis for the flow of water through the developed aquifer, gravel pack, screen and other elements in the vicinity of the well. This will include analytical work and large scale well model testing. Following development of a sound theoretical basis, criteria could be established to enable the designer to select the optimum geometrical requirements for the developed aquifer, gravel pack, screen etc. and design them accordingly.

5. CONCLUSIONS

1. The head loss - flow ratio relationship for flow through granular materials may be represented as

$$S = aV^n \text{ with a and n varying with the type of flow regimes}$$

or $$S = aV + bV^2$$

Both may be made to fit closely to experimental functions.

For simplicity in computation techniques, the former is preferable.

2. Determination of drawdowns in wells where non-Darcy flow occurs requires the computation of non-linear flow nets. This may be done on the digital computer using finite difference or finite element methods.

3. Accurate knowledge of the type of material and state of compaction of acquifer and gravel pack materials close to the well is required for the constants in the head loss equation to be accurately determined.

4. The only reliable way at present of determining the constants in the head loss equation is by carrying out permeameter tests. The effect of convergence of the flow on the constants requires close attention.

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DISCUSSION

K.P. Stark, University College of Townsville.

I must congratulate the authors on their summary of the treatment of non-Darcy flow in the vicinity of wells. I would like, however, to add a few comments which are relevant to this non-linear flow regime and particularly to the work we, in Townsville, have done in this area over the last few years.

In particular I would support more emphasis being given to the Forchheimer relation (which is given in the paper as $S = aV + bV^2$) for a number of reasons.

Irmay (1958) showed that there is a rational basis for considering a Forchheimer relation, where 'a' and 'b' depend on the medium and fluid properties. However, Stark (1969), has shown more accurately that, in fact, both 'a' and 'b' are also dependent on Reynolds number (RN, defined in terms of seepage velocity and particle size) and, further, that this dependence must be considered if the change in RN is great enough to alter significantly the streamline and vorticity (or alternatively velocity) profiles of the flow. Definitive expressions are given for 'a' and 'b' in terms of the properties of the fluid and medium and the velocity distributions. These velocity distributions are, of course, dependent upon the boundary configurations of the actual flow path and RN. We are then in a somewhat similar situation to that used in the traditional acceptance of Darcy's equation for low RN. Thus, providing the velocity of flow is small the second term of the Forchheimer relation is negligible compared with the first and the Forchheimer relation reduces to Darcy's equation but, for Darcy's law to hold over a range of small Reynolds numbers (e.g. $RN < 1$ as often assumed) it is also necessary to assume that 'a' is independent of RN and this assumption is only valid if the velocity distributions of the actual flow between the medium particles remains constant. Experimental evidence by literally hundreds of workers has supported this assumption over limited ranges of RN and, indeed, this evidence plus the numerical solutions of the Navier-Stokes equations we have obtained suggests that the same assumption (VIZ: the velocity profiles do not change radically with RN) is valid for quite large ranges of RN. Thus the values of 'a' and 'b' in the Forchheimer equation whilst being dependent upon and calculable from the velocity distribution for a given RN can, indeed, be assumed as constants, within experimental accuracy, over quite wide ranges of RN (e.g. 0-50).

With this in mind, although our experimental permeameter results, Stark and Volker (1967), appeared to give the kinked straight line patterns referred to by the authors, a closer consideration of the deviations involved in the fitted relation showed that a Forchheimer relation can be fitted to the results at least as accurately and generally more accurately than the exponential relations. This is true also of the experimental results previously reported by Dudgeon and referred to by the authors. Indeed Dudgeon's experimental results would appear to be the most reliable results available.

Further the use of the non-linear Forchheimer relation does not involve any particular problems which would warrant its abandonment when solving field problems by either the finite difference or finite element method and we have obtained excellent agreement with experimental results of flow to a well in a coarse medium in an experimental tank 20' diameter using both analytical techniques.

Finally it must be stressed that although the accuracy obtained using a Forchheimer relation appears to be marginally better than that obtained from other relations which fit experimental data it should be favoured in use wherever possible because of its more rational derivation, its better fit to experimental data and because its use does not impose any additional analytical problems.

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C.F. Forbes, Water Conservation and Irrigation Commission, N.S.W.

The authors must be congratulated on a most interesting paper. It was most pleasing to read their "physical interpretation of the velocity - hydraulic gradient relationship" and most interesting to note that exponents for the equation

$$S = aV^n \qquad \ldots\ldots\ldots \text{(D1)}$$

were found to be less than 2 even for 6" river gravel (n = 1.5 approx.).

The stepped nature of the data as shown in the graph for hydraulic gradient v velocity is most interesting. This phenomenom was also obtained by Mogg (1959) (see Figs. D1 and D2), although it would not require too much imagination to fit a curve through some of his data as presented in figure D2. However, not all investigators have found such abrupt changes, especially in the finer materials. Fancher, Lewis and Barnes ((Muskat, 1946) and (Machis, 1946)) present friction factor curves (Figs. D3 and D4) which are distinctly curved over fairly wide ranges of Reynolds number. Results such as the authors' would plot as segmental lines on such diagrams. The inference of the curved lines, for the test results plotted, is that either the exponent or coefficient of V or both vary continuously with increasing velocity (Reynolds number). Although not precisely clear Mogg (1959) in referring to the work of Lockman and Hudson seems to infer the presence of continuous variation in their results. He also reports a stepped result for the coarser material in Lockman's work.

Whilst the position is certainly not clear it does appear that, at least in the finer and well graded material, continuous variation may occur over a much wider range than has been found in coarse material.

The equation $S = aV + bV^2$ (D2)

implies that inertial forces arise as V^2 forces only and that a transition as suggested by the formula for flow in non-linear regions

$$S = aV + bV^n \qquad \text{(D3)}$$

(Muskat, 1946) does not really exist.

Muskat (1946) gives equations D1 and D3 for non-linear flow. Referring to the problem he says " - - - - About all that appears to be definitely established is that the empirical data can be expressed in all cases by an equation" of the form of D3, "with n having a value in the neighbourhood of 2 - - - - ". He further, describes work by Lindquist, and Muskat and Botset in which n was found to be almost 2 for coarse materials. In Lindquist's work the transition from linear to non-linear flow was very sharp. Work by Chalmers, Taliaferro and Rawlins again described by Muskat (1946) resulted in values for n in equation D3 of from 1.7. to 2. This work was carried out on 16 different materials.

Do the authors think that continuous variation over a fairly wide range of Reynolds number in the values of a and n in equation D1 and a, b and n in equations D3 is possible in finer and well graded materials where flow passage sinuousity is of a much higher order than in coarser and uniform rounded materials?

Further, in view of their interpretation of the velocity - hydraulic gradient relationship do they think that inertial effects can arise which are proportional to V^n where n varies between one and two or are inertial effects always proportional to V^2?

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The Authors in Reply

The comments of Dr. Stark and Mr. Forbes centre on the equation relating flow velocity and hydraulic gradient. It is the authors' opinion that there is a considerable amount of research yet to be done to reconcile apparent discrepancies between the experimental results reported and theoretical solutions of the Navier-Stokes equations for relatively simple flow geometries. It is to be hoped that theoretical solutions for real porous media geometries will eventually become available and that more experimental data for a wide Reynolds number range will be forthcoming. A great deal more attention in experimental work needs to be paid to the flow at the pore level, rather than just to the overall macroscopic flow picture.

The senior author has had constructed a new permeameter and preliminary results confirm the belief that there is not a smooth progressive change from Darcy flow to turbulent flow. Further studies will be under way soon. The authors have not had the opportunity to study the fitting of the Forchheimer equation to the senior author's results but hope to do so in the near future. It would appear difficult to do this for a particular "regime" without having all experimental points lie on one side of the fitted curve. If a curve is being fitted over a number of regimes it would be possible to distribute the points on both sides of the line but there would be some interesting groupings of points which would require more than a simple least squares fitting process.

In answer to Mr. Forbes query regarding velocity exponents and inertial effects, we feel that the distinction between local velocity and the flow velocity should be emphasized. It might be possible to have the local inertial effect proportional to the square of the local velocity and yet, because of changing flow geometry or surface effects not have the overall hydraulic gradient - flow velocity relationship of the form $S = aV + bV^2$. Such things as the uniformity of grain size of the material,as Mr. Forbes suggests, might be involved in this way.

In conclusion we would emphasize that there will be no significant difference between non-Darcy flow nets determined on the basis of the Forchheimer or exponential equations. Experimental results would agree equally well if the exponential equations were used for well flow analysis since, over the relatively small range of Reynolds number involved, either equation can be fitted quite well to experimental permeameter results. The problem in practice is not to distinguish between these equations but to obtain even approximate data for in situ porous materials such as water bearing sands and gravels of interest in groundwater extraction.

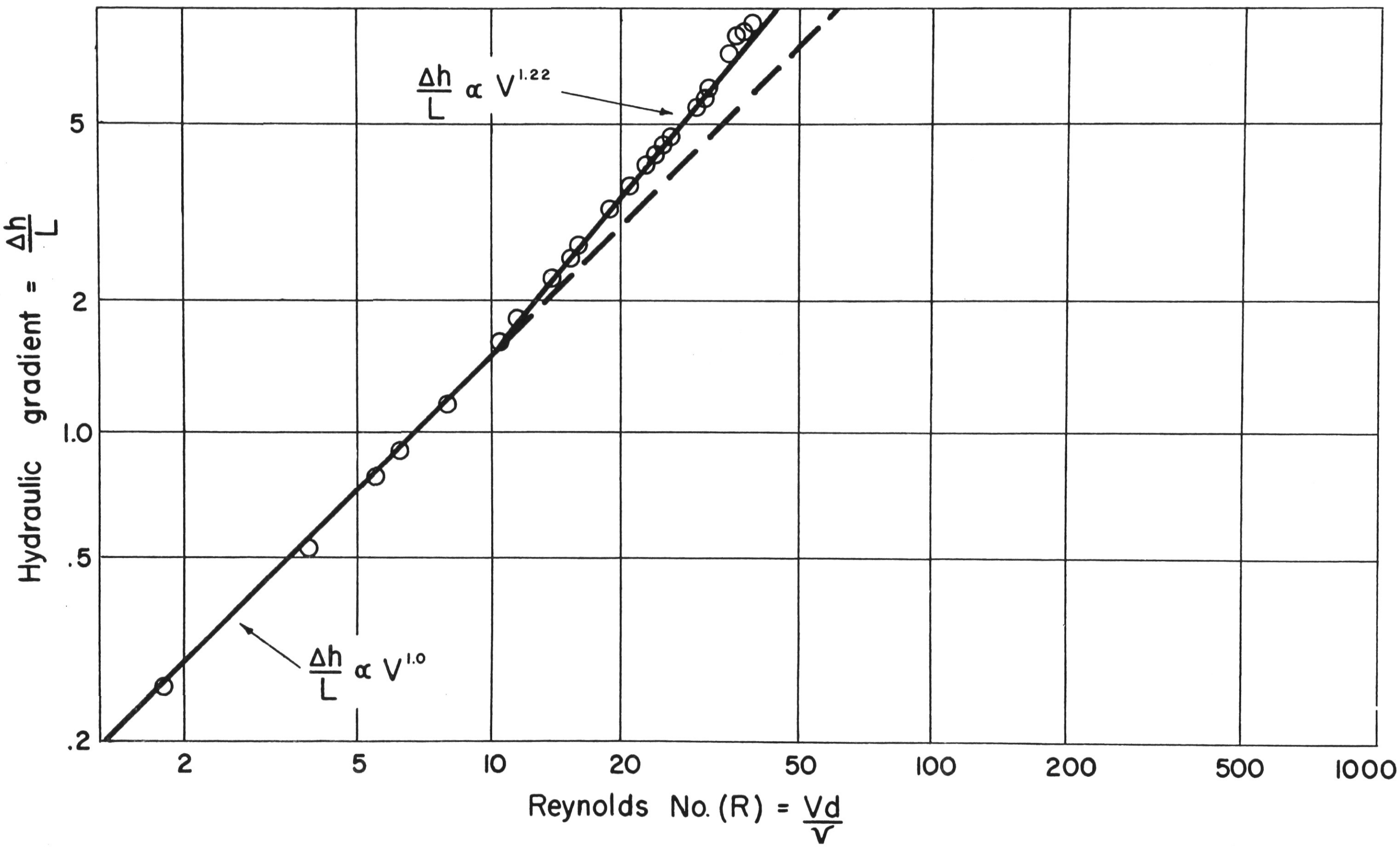

Fig. D1: Relationship between hydraulic gradient and Reynolds number for sample A. (After Mogg).

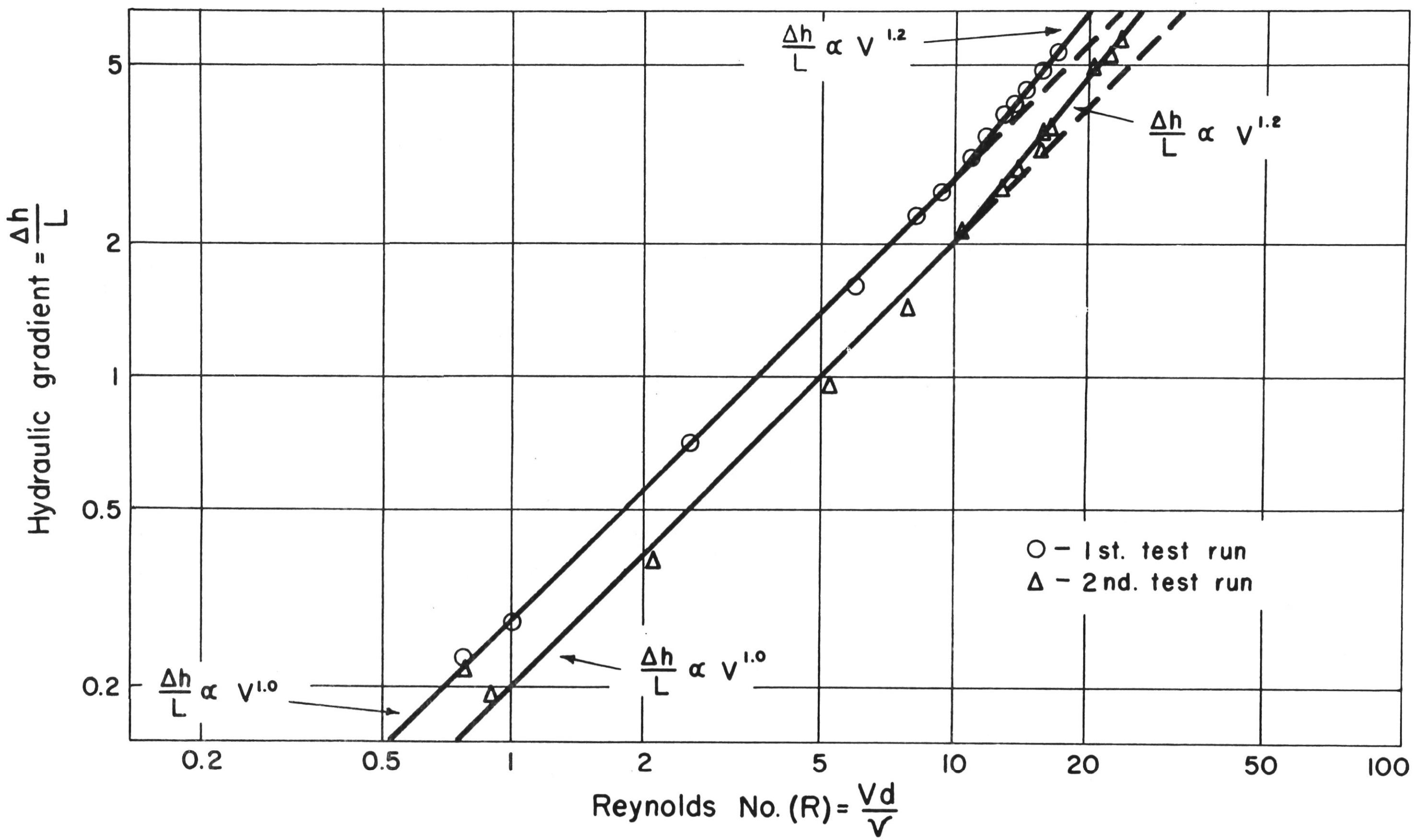

Fig. D2: Relationship between hydraulic gradient and Reynolds number for sample B. (After Mogg).

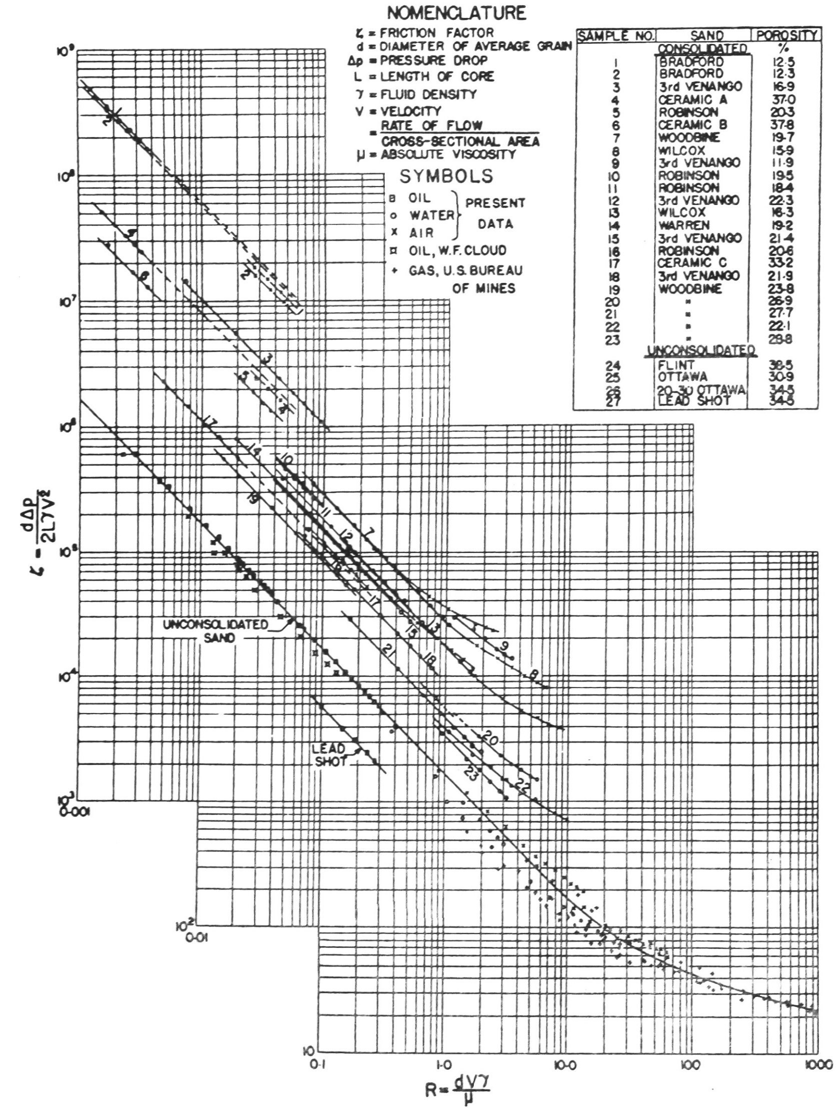

Fig. D3: Friction-factor chart for flow of fluids through sands. (After Muskat).

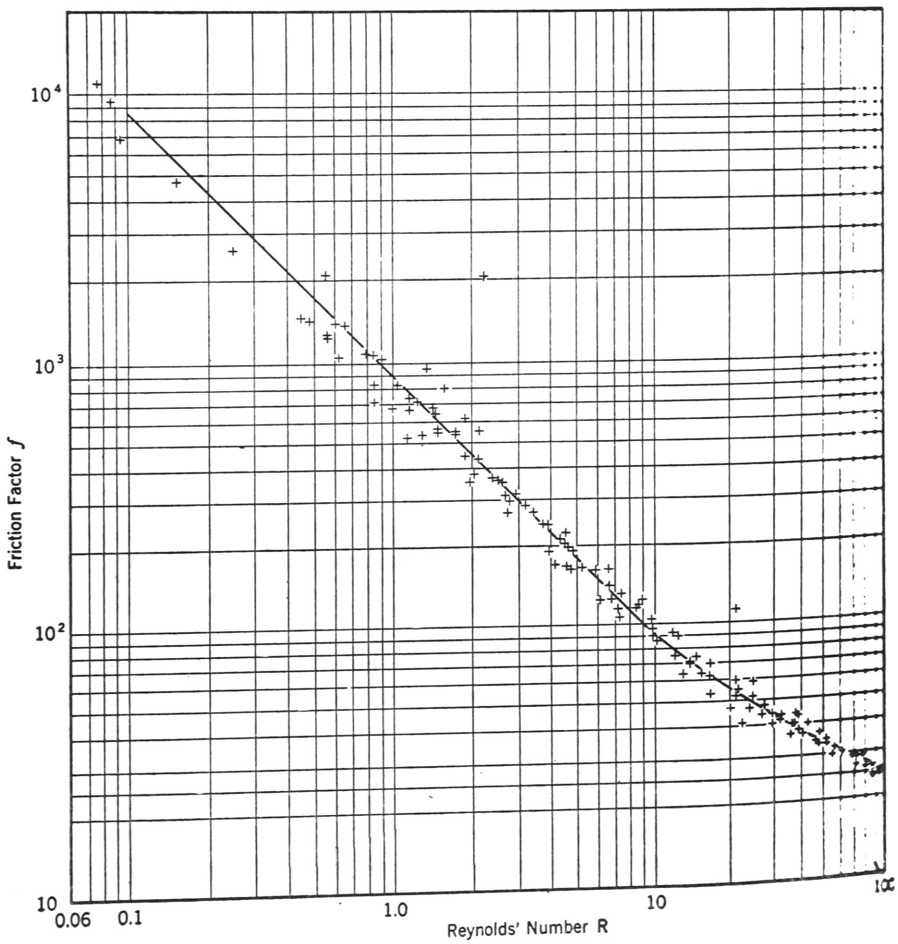

Fig. D4: Reynolds number - friction factor relationship.
(After Machis).

PAPER No.3

THEORY AND APPRAISAL OF HYDRAULIC CONDITIONS IN WELLS

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SUMMARY

Equations of flow into wells are considered and a new equation proposed.

The use of the standard well tests in relation to drawdown prediction, indicating state of development and suitability of design and construction is discussed.

Notions of well efficiency are considered and a standard suggested. The use of Jacob's "C" as a measure of well efficiency is discussed in relation to the problems raised by Mogg.

1. INTRODUCTION

A water well is a structure constructed in permeable and impermeable earth materials for the purpose of producing water from selected permeable water bearing formations. The structure consists of a central clear space provided by a supporting tube, a screen and a filter pack which retains the aquifer in place and should increase the permeability of the structure in an increasing amount as the well screen is approached. The supporting tube and the well screen can be of any one of a number of designs. The filter pack may be made of materials already forming part of the aquifer (as in a screened and developed well) or it may be formed from "artificial" (non insitu) material introduced to the annulus between the screens and supporting tube and the wall of the bore hole.

In this paper the outer boundary of the well in the water producing zone is considered to be the surface beyond which the aquifer is structurally undisturbed. This boundary is approximated by the well radius (r_w) which is defined as the average radius beyond which the aquifer is undisturbed.

It is generally assumed that flow in granular permeable aquifers follows Darcy's law. However, it is not inconceivable (although highly improbable in properly designed and constructed wells) that non-linear flow may occur in the undisturbed aquifer i.e. beyond r_w.

The notion that non-linear flow does not extend very far from the well screen is based on the fact that the value of Reynolds number (R_n) is inversely proportional to distance from the centre line of the well with an exponent which varies between 1 and 2 depending upon the grain size distribution outwards from the screen.

The validity of this reasoning is of course relative to the degree of compaction of the materials under consideration, as the onset of non-Darcy flow may occur between values of R_n from 1 to 10 for fully compacted and unconsolidated materials respectively. (Forbes (1))

Muskat (2) described the basic physical laws which govern the flow of water through porous media. In 1946 Jacob (3) proposed the first model of flow into a well incorporating non-linear flow.

Jacob postulated a flow model wherein the laws governing aquifer flow govern right up to a boundary swept out by what he called the effective well radius. Inside this radius it was assumed that fully turbulent flow existed. Jacob's equation of flow was thus given as

$$s = BQ + CQ^2 \quad . . . . \qquad (1)$$

where s = drawdown measured inside the well

Q = discharge

B = a time variable "constant"

C = a constant

BQ was assumed to be the head loss caused by the aquifer and is generally known as the aquifer loss. CQ^2 was assumed to be the head loss caused by flow in the well and is generally known as the well loss.

This theory and equation were found to fit many actual well examples; however, significant departures were soon discovered and in 1953 Rorabaugh (3) proposed a slightly different equation and new methods for graphical and analytical analysis of Jacob's step drawdown test.

Rorabaugh postulated the equation

$$s = BQ + CQ^n \quad . . . . \qquad (2)$$

to describe flow into a well, where n could be greater than 2.

He was concerned that he found values of n greater than 2 and explained this by suggesting that it was caused by errors in the value of the effective radius. His basic model was however the same as Jacob's i.e. a zone of linear flow existing outside the effective radius and fully turbulent flow inside the effective radius.

Muskat (2) shows that values of n in excess of 2 are physically unrealistic.

Jacob's equation suggests that there is no well loss at low values of R_n.

These theories were doubted and criticised by Mogg (4) who suggested from consideration of calculated values R_n in wells and experimental determination of onset of non-linear flow that the role of turbulent flow may have been over emphasized by previous workers.

He worked with unconsolidated material and apparently was unaware that in consolidated formations non-linearity can commence at Reynolds numbers much lower than in unconsolidated material. As defined, Reynolds number relates to the diameter of flow passages; however, in this work it is calculated using average grain diameter in place of flow passage diameter, hence these values of R_n can be seriously in error as flow regime indicators. For example compaction can alter flow path diameter without affecting grain size.

Subsequently Mogg (5) further criticised the step drawdown test and the practice of using Jacob's "C" as a criterion for well efficiency.

2. PROPOSED MODEL

Forbes (1) proposed another model for well flow and this is summarized in this section.

It is partly based on a consideration of Muskat's equation for flow through porous material

$$\frac{\Delta P}{\Delta s} = av + bv^n \quad \ldots\ldots\ldots\ldots \quad (3)$$

where ΔP = change in pressure over length Δs.

Δs = length (distance) between points of reference.

a and b = constants (for a further consideration of the nature of these see Forbes (1))

n = an exponent greater than 1 and less than 2.

v = velocity.

Forbes (1) defines the well as existing from the well centre to the average radius beyond which there was no aquifer disturbance. This average radius is called the well radius to distinguish it from Jacob's effective well radius defined above. Forbes (1) suggests that flow through the well is given by an equation of the form

$$s = C(Q)Q + \phi(Q)Q^{f(Q)} \quad \ldots\ldots \quad (4)$$

and that total drawdown in a discharging well is given by

$$s_w = BQ + C(Q)Q + \phi(Q)Q^{f(Q)} \quad \ldots \quad (5)$$

where s = drawdown caused by the well structure

s_w = total drawdown in the discharging well

BQ = aquifer loss up to the well radius or the reference radius where non-linear flow exists outside the well radius.

$C(Q) + \phi(Q)Q^{f(Q)}$ = well loss

$C(Q)$ = coefficient of linear well loss

$\phi(Q)$ = coefficient of non-linear well loss

($C(Q)$ and $\phi(Q)$ are both functions of Q)

B = time variable constant of form and value determined by the aquifer model

Q = discharge

$f(Q)$ = the non-linear exponent of flow. This is a function of Q with a probable lower limit of 1 and an absolute upper limit of 2.

In non-algebraic language the zones of flow in and around a well in which non-linear flow has been developed, are:-

(1) A non-linear flow zone in which there is both a linear and non-linear component of flow according to the equation 3.

(2) A linear flow zone existing from the outer boundary of the non-linear zone to the well radius.

(3) A linear zone obeying the law of flow for the aquifer beyond the well radius, or the reference radius when non-linear flow invades the undisturbed aquifer.

A further discussion of these zones and the nature of flow in the regions are given by Forbes (1) but those are beyond the scope of this paper. Suffice to say that the values of $f(Q)$, $C(Q)$ and $\phi(Q)$ are in dynamic balance for any given value of Q and for any given geometry and are definitely not time dependent.

Three regimes of flow will be considered:-

(1) At low Reynolds numbers non-linear flow is not developed and the value of $\phi(Q) \to 0$ and the well equation reduces to

$$s_w = BQ + C(Q)Q \quad \ldots\ldots\ldots\ldots \quad (7)$$

(2) At higher Reynolds numbers a non-linear flow zone is developed and well flow is described by equation 5 and $f(Q) < 2$.

(3) At moderate Reynolds numbers with a badly clogged or inadequate screen and an adequate filter, fully turbulent flow develops at the well screen and purely linear flow occurs throughout the filter. The flow equation becomes

$$s_w = BQ + C(Q)Q + \phi(Q)Q^2$$

This reduces to

$$s_w = (B + C(Q)\ )\ Q + \phi\,(Q)Q^2 \quad . \ . \ . \ . \ . \ . \ . \ . \ . \qquad (8)$$

which is an equation of the form of Jacob, $(B + C(Q))$ and $(\ \phi(Q))$ being equivalent to Jacob's B and C respectively.

It should be noted that if the flow rate is increased and inertial flow occurs in the pack $f(Q)$ will reduce from 2 to an average value less than 2, discrete for each flow rate. Appendix A gives some interesting examples and further discussion.

Forbes (1) also shows that Jacob's 'C' is proportional to permeability and hence of course grain size and compaction as had been suspected by Mogg (5).

According to Mogg (5) and Lennox (6), Jacob introduced the concept of well efficiency. Rorabaugh (3) further developed the concept. Mogg (5) proposed an additional definition.

The various definitions of well efficiency are given below.

(1) Jacob (after Mogg (5)) inferred that the efficiency of a well as an hydraulic structure could be inferred from a comparison of the effective well radius with the physical well radius i.e. the radius of the screen.

(2) Rorabaugh (1953) defined well efficiency as the ratio of the theoretical drawdown (computed by assuming that a logarithmic distribution of head is applicable all the way to the well screen face) to the drawdown in the well.

(3) Mogg (5) defined well efficiency as the ratio of the actual specific capacity at the well design rate after 24 hours pumping to the maximum specific capacity possible, calculated from formation characteristics and well geometry after 24 hours continuous pumping again at the design discharge.

It is well known that except for steady state conditions drawdown and hence specific capacity is proportional to a function of time and to a function of discharge and hence it is proper when well efficiency is being discussed that both the value of discharge and the time after pumping has commenced should both be stated.

This observation also partly applies to Jacob's effective radius as this quantity is proportional to discharge. Mogg's and

Rorabaugh's definitions are really identical if the same time of reference is used. Rorabaugh's definition being simpler, and also prior, has more appeal and, in the author's opinion, should be used as standard.

Thus well efficiency for any given time (24 hours is suggested) and discharge is given by

$$\eta_{t/Q} = \frac{s_1}{s_w} \times 100$$

where $\eta_{t/Q}$ = well efficiency (per cent) at time t and discharge Q

s_1 = formation (aquifer) loss for Q at time t at the well screen radius

s_w = measured drawdown in the well at time t and discharge Q

Efficiencies in excess of 100% can be obtained and indeed should be looked for; increased permeability in the well structure can lead to smaller head losses between the well radius and the well screen than would be obtained through the aquifer.

In single well installations where the aquifer storage coefficient is not known difficulty has been experienced in obtaining realistic values of this aquifer property and hence in calculating aquifer drawdown. This has invalidated the method of using well efficiency as a method for well evaluation and comparison.

The well known method for obtaining storage coefficient by obtaining t_o and assuming a value for well radius invariably leads to very large values of Storage Coefficient being computed even when allowance can be made for the non-linear well loss.

This is because of the hitherto unsuspected linear well loss which displaces the drawdown curve downwards an unknown amount causing an overlarge value of t_o to be obtained. Appendix B shows how errors in assumed values of well radius and Storage Coefficient affect the calculated value of drawdown.

Forbes (7) describes a method for obtaining storage coefficient in the pumped well free of this difficulty. The method has yielded reasonable values for storage coefficient for the non-leaky artesian case.

During well construction and commissioning, the groundwater technologist frequently asks questions like these. Is the screen clogged? Is the pack clogged? Would further development be beneficial? If so, which sort? At what rate can the well be pumped for the given production schedule? To answer these questions information about the aquifer and the well is required.

Information about the aquifer is obtained from the long term single rate pump test. This test is well known. The length of the test should be determined after considering the following factors.

(1) Nature of the aquifer. Involved here are considerations such as the degree of confidence in available knowledge concerning its extent, homogeneity and continuity, the proximity of suspected boundaries, pollutants etc.

(2) Use to which the water is to be put.

(3) Source of recharge to the aquifer.

Forbes (1) discusses this problem further and gives some suggestions for lengths of tests and a number of suitable references. The rate of discharge used in the long term single rate test should approximate the anticipated production rate wherever possible.

The step drawdown test was introduced by Jacob to enable determination of the constants B and C in his equation 1.

$$s = BQ + CQ^2$$

We have seen that this equation can be used to describe flow into a well with clogged screens at moderate rates of discharge and also in wells for low discharge rates where $C \to 0$. Even so it is never physically valid.

It is suggested that an equation of the form of equation 5 more closely describes flow into a well where functions $C(Q)$, $\phi(Q)$ and $f(Q)$ are of an unknown form, although it may be said that $C(Q)$ and $\phi(Q)$ approach Q as lower limiting values and $f(Q)$ is greater than 1 and has 2 as an absolute upper limiting value.

The present author feels that the determination of B and C in Jacob's equation is a worthless and unnecessary exercise but that the graphical analysis of the step drawdown test leads to the development of worthwhile relationships for quantitative and qualitative use in answering the questions referred to above.

When the test is used as a construction aid in determining if further development is necessary or as a routine maintenance feature it is suggested that the length of stages can be short, possibly no longer than 10 minutes; however, where the test is used for determining well characteristics longer stages may be used.

Because the test is of quite short duration in terms of long term tests or eventual pumping duties discharge rates far in excess of the design rate can and should be employed to explore the maximum possible range of flow regimes that the well may be subjected to in practice.

In general there are two ways of plotting the data and these are illustrated in Fig.1. In the first method, described by Williamson (8), the raw data is plotted on semi-log paper. In this method the first stage plots as a straight line but owing to time displacement of their origins the second and subsequent stages are curved. In the second method, proposed independently I presume, by Lennox (6) and Colville (9) each stage is plotted using zero time as the time origin and naturally the stages plot as straight lines.

Where the second method is used stages of equal length are suitable. However, where the first method is used for convenience of plotting and analysis it is advisable to extend the length of the second stage. In a three stage test if the first stage is "a" minutes the second stage should be "3a" and the third "a" minutes respectively. Where more than three stages are employed equal stages and the second plotting method should be used.

From the above plot two graphs are drawn in Fig.2 one showing drawdown for a given time (s_t) against discharge and the other s_t/Q against discharge.

Providing a sufficient range of discharge has been tested the graph of s_t v Q can be used for accurately predicting drawdown for the given time within the practical range of discharges regardless of the regime or how closely Jacob's law is approximated. Drawdown for different times can then be obtained in the usual manner. (Williamson (8)).

The second graph indicates the flow regime in the well and with the aid of well efficiency calculations and aquifer drawdown at the well radius, gives a clue to the internal condition of the well.

For example if we determine the efficiency of the well for the design discharge, for a variety of discharges and observe, firstly, that the efficiency is in excess of 100% and there is very little variation in efficiency for various discharges it can be assumed that the well has been expertly investigated, designed and constructed. However if efficiency is much less than 100% and there is considerable variation in efficiency for various discharges, a

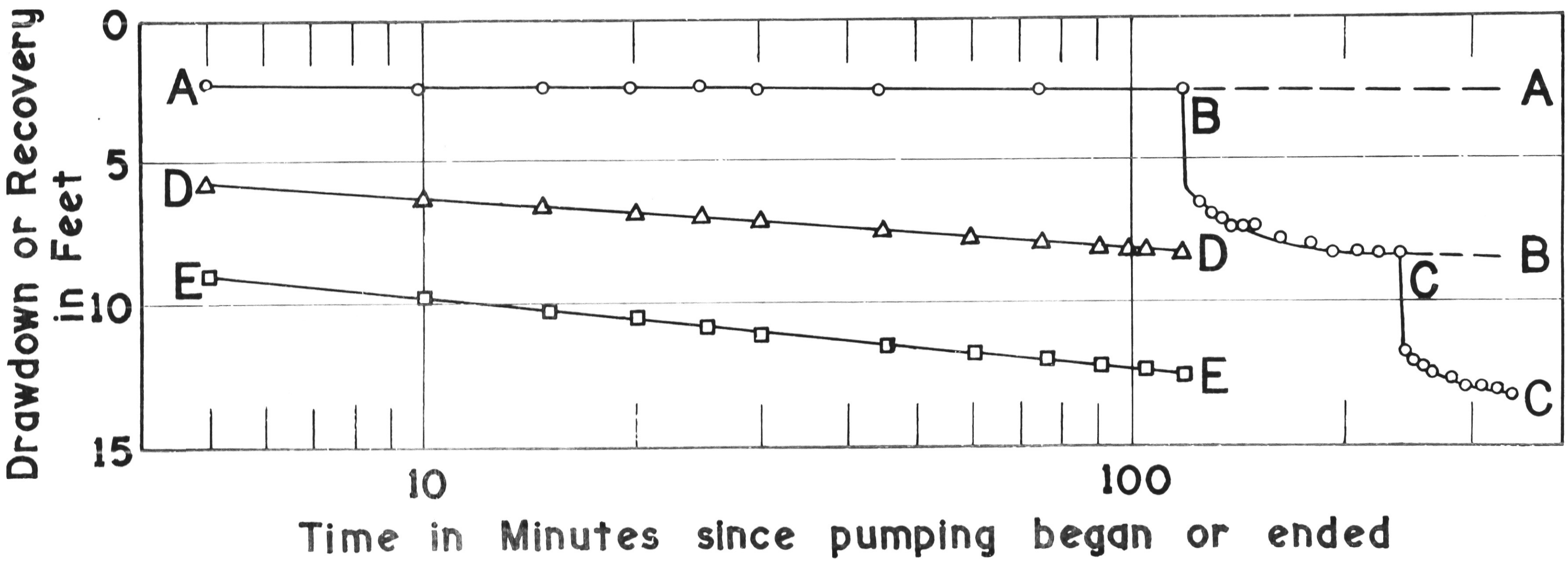

Fig. 1: Drawdown - time relationship for multi-stage step drawdown test.

A-B-C represents the raw data plot showing the different time origins for each stage.

AA, DD, EE have been compiled from the same data using the same time origin for each stage. It should be noted that the curved portions of the raw data plot become straight lines when the same time origin is used.

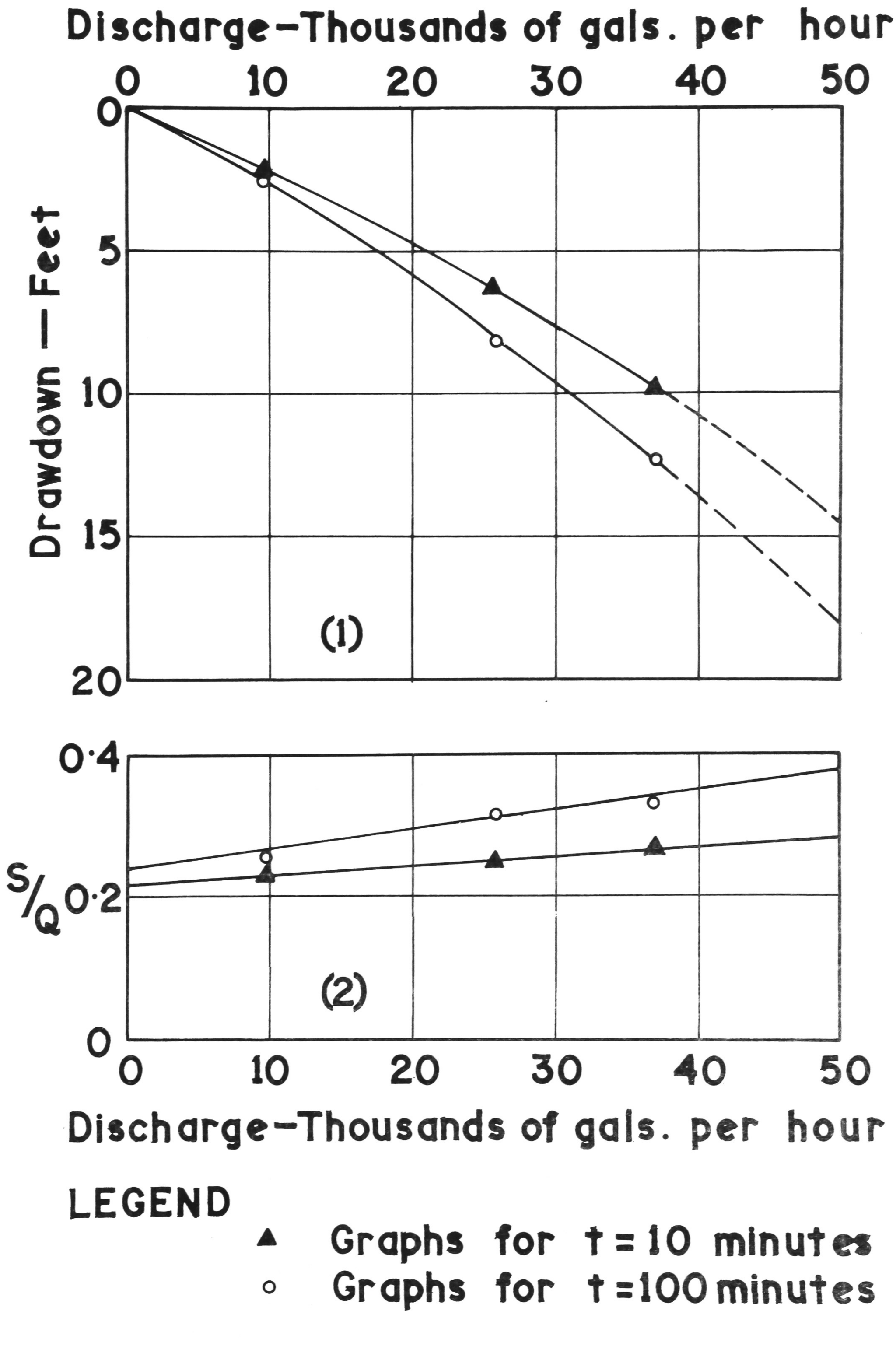

Fig. 2: Drawdown - discharge and s/Q - discharge relationships for the data given in Fig. 1 and compiled from the Colville - Lennox plot.

clogged screen or one with inadequate open area should be suspected. This would be confirmed if the s_t/Q v Q plot closely approximated a straight inclined line at low to moderate discharge rates. Further development work especially jetting with chemicals if clay was suspected could be expected to improve the hydraulic characteristics of the well (if adequate open area was present in the screens). On the other hand if efficiency was low and variation in efficiency at different discharge rates was slight, then clogging of the pack or hole-pack interface might be expected; this would be confirmed by a s_t/Q v Q plot showing a horizontal straight line or a horizontal straight line passing into a curve at moderate to high discharge rates. In this case high linear well loss is causing the inefficiency and the well should be treated by surging, valve and reverse valve surging with or without chemicals,to destroy wall cake etc. should this be felt necessary.

The presence of a high non-linear loss does not mean that a considerable linear well loss may not also be present. These well losses may be calculated by remembering that Jacob's BQ is equivalent to Forbes' (BQ + C(Q) Q). Hence, provided that non-linear loss can be obtained approximately and subtracted from the well drawdown to give (B + C (Q))Q the linear loss may be obtained by subtracting the aquifer loss at the well radius. If this linear well loss is compared with the aquifer loss between the well radius and the screen the effect of clogging can be readily observed.

Part of enlightened well operation should include periodical, say 6 monthly, step drawdown tests of short duration, say 10 minute steps, in order to assess deterioration of hydraulic condition. This would form a sound basis on which to plan any remedial work. For a much fuller discussion of pump tests with suitable references see Forbes (1).

3. CONCLUSION

(1) Experimental work suggests that flow departs from Darcy's law at Reynolds numbers of about 10 in unconsolidated formations and 1 in consolidated formations. It is thought that departure from linearity at such low values of R_n is caused by a rise in accelerative forces, over viscous forces caused by discontinuities and roughness in the flow passage and an impressed flow passage sinuosity.

The extremely low value of R_n at which flow departs from the linear law in consolidated material is probably due to an increase in impressed sinuosity, relative roughness and discontinuity brought about by the consolidation process itself whereby interpenetration of the grains is brought about with possibly little change in grain size.

(2) It should be possible to describe well flow by an expression of the form of equation 5. Equation 1 is an approximation which fits special cases of the above law but which is never physically valid.

(3) Well testing should be employed for:

(a) Drawdown prediction using graphical techniques only. The tested range of discharge should adequately cover the range in which drawdown prediction is to be made.

(b) Provision of an assessment of the well efficiency for stated discharges and times. Wells may be compared on the basis of efficiencies determined for their design discharge for the same time interval.

(c) Assessment of the condition of the well as an aid to construction and maintenance.

(4) The Coefficient C in Jacob's equation is a variable depending among other things upon the average grain size or interconnected pore size of the media $[C \propto 1/d^2$ to $1/(d^{3-n})]$ and hence should not be used as a criterion of well efficiency.

4. ACKNOWLEDGEMENT

I am grateful to the New South Wales Water Conservation and Irrigation Commission which has made this work possible and to my literary mentors, C.E. Jacob, M. Muskat, W.C. Walton, M.I. Rorabaugh, J.L. Mogg , D.K. Todd, D.H. Lennox and all other workers from whose writings I have had the privilege to profit.

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6. APPENDIX A

THE GRAPHICAL FORM OF WELL EQUATIONS

Unfortunately the results of extended pump tests covering the complete range of flow in a particular bore are not available. However, conditions are easily visualised and two models will be considered.

Model 1. A well in which turbulence is developed at the screen before non-linear flow arises in the pack.

Model 2. A well in which the screen is not an early control and where non-linear flow develops in the pack before full turbulence develops at the screen.

Let equation (A1) describe the flow

$$\frac{s_t}{Q} = B + CQ^{n-1} \qquad \text{(A1)}$$

and sets of values for Q, n, B be assumed.

Values for Model 1

Q	B	C	n-1	$\frac{s_t}{Q}$
5	0.8	0	0	0.8
10	0.8	0	0	0.8
15	0.8	0	0	0.8
20	0.8	0	0	0.8
25	0.6	0.0124	1	0.82
30	0.6	0.0124	1	0.98
35	0.6	0.0124	1	1.04
40	0.6	0.0124	1	1.11
50	0.55	0.0162	0.95	1.22
60	0.5	0.021	0.9	1.32
80	0.4	0.0318	0.8	1.48

Values for Model 2

Q	B	n-1	C	$\frac{s_t}{Q}$
5	.8	0	0	.8
10	.8	0	0	.8
15	.8	0	0	.8
20	.8	0	0	.8
25	7.75	0.15	0.022	0.81
30	7.5	0.18	0.0035	0.82
35	7.5	0.2	0.0145	0.84
40	.72	0.3	0.055	0.90
50	.66	0.42	0.070	1.02
60	.58	0.57	0.11	1.72
80	.48	0.72	0.15	4.00

The curves obtained by plotting these values are shown in Figs. 3 and 4 respectively.

Physical Significance

Model 1 In this case at low discharge, flow is purely linear, this changes quite abruptly to fully turbulent flow at the screen and completely linear flow elsewhere in the well region. During this regime the flow fits Jacob's equation. With increasing discharge rate non-linear flow begins to be exhibited in the pack and since "n" is an "average" value discrete only for a particular rate of flow, its value falls and will continue to fall as the volume of the non-linear flow region increases with increasing discharge. Probably becoming asymptotic to some fixed value whose magnitude ($1<n<2$) would be determined by the conditions in the well. The values of B and C will of course vary in accordance with the relative values of the flow region devoted to linear and non-linear flow.

It should be noted that formation loss is not included in equation (A1). B is not time dependent.

Model 2 In this model the screen is not a control and the onset of non-linear flow is gradual. Thus even when (if) full turbulence is developed in the screen and (or) in the pack close to the screen, because non-linear flow has preceded this and exists in regions beyond those displaying full turbulence "n" never achieves the value 2, although again it may be asymptotic to some value (magnitude determined by conditions) less than 2 as an upper limit.

Other models can of course be envisaged but the author feels that the shapes of the curves, Figs. 3 and 4, can be used to appreciate most of the "part" curves found as a result of pump testing procedure on real wells, where sections of the relationship may be curved, horizontal, concave upward, concave downward or inclined straight. Figs. 5, 6, 7, 8, 9, 10 and 11 show typical experimental curves exhibiting some of these features.

Figs. 5 and 6 could be interpreted as either model 1 or model 2 type curves - a further point at either end of the range would probably clarify the position.

Fig. 7 is a flat curve which could be the start of a curve of either model.

Figs. 8 and 9 are both model 1 curves.

Figs. 10 and 11 are both probably portions of model 2 curves.

APPENDIX B

ERRORS IN COMPUTED VALUES OF DRAWDOWN DUE TO ERRORS IN ASSUMED VALUES OF WELL RADIUS AND STORAGE COEFFICIENT (NON-LEAKY ARTESIAN CASE).

Errors in assumed or computed values for storage coefficient and well radius produce errors in u, W(u) and hence computed values of drawdown.

The error in u caused by errors in the assumed values of well radius and storage coefficient may be obtained approximately by the usual method which is outlined below.

The following equations are used in calculating drawdown in the non-leaky artesian case:

$$s = \frac{114.6Q\, W(u)}{T} \qquad \text{(B1)}$$

$$u = \frac{2247\ r^2\ S}{Tt} \qquad \text{(B2)}$$

Where s = drawdown (feet)

Q = discharge (Imp. gals/minute)

T = transmissivity (gals/day/ft.)

$W(u)$ = the well function of u.

r = well radius (ft.)

S = storage coefficient

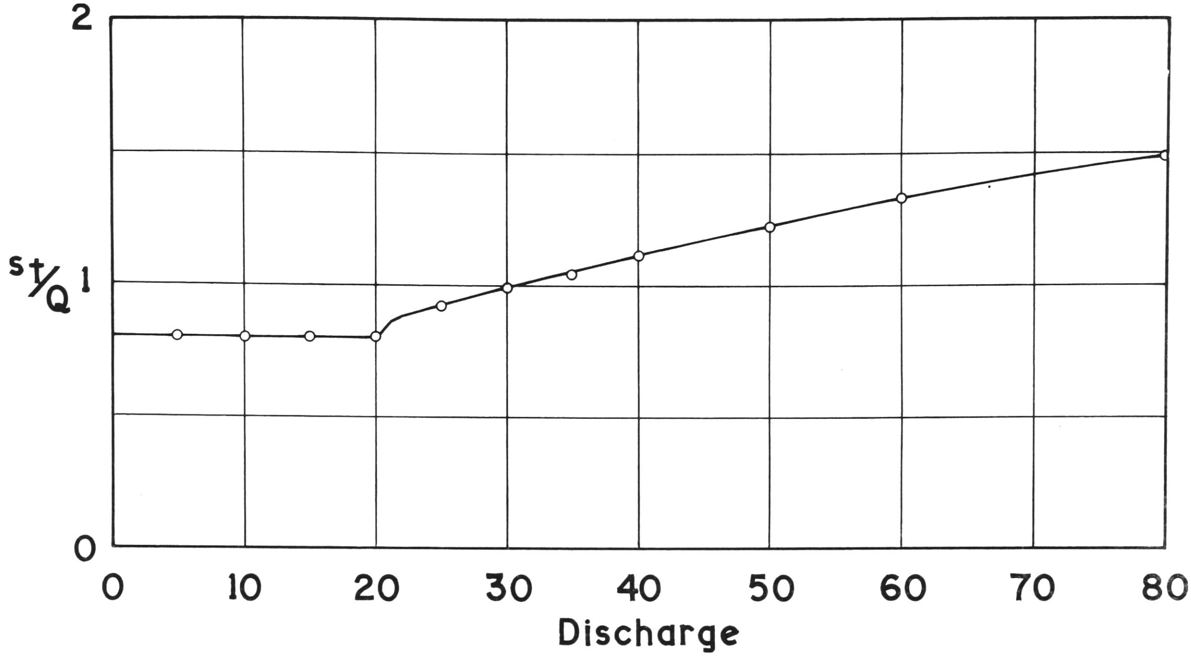

Fig. 3: s_t/Q - discharge relationship for Model 1 (assumed data).

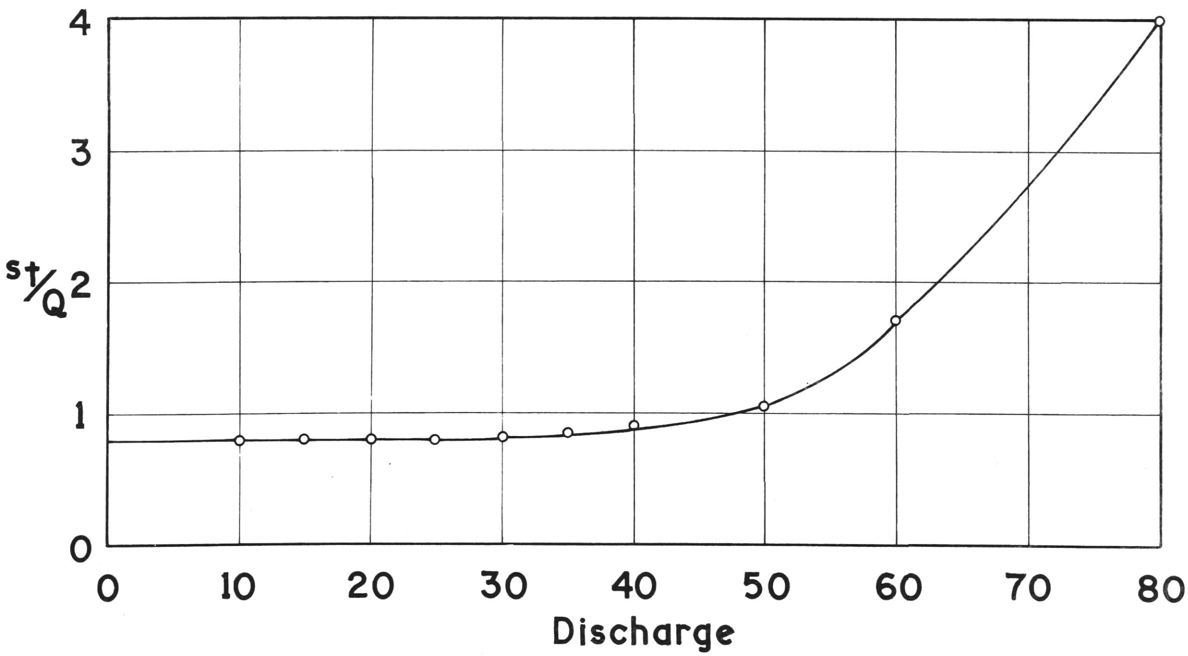

Fig. 4 s_t/Q - discharge relationship for Model 2 (assumed data).

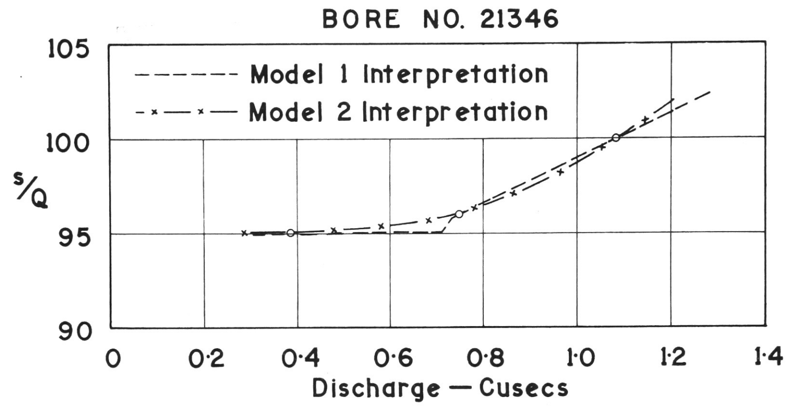

Fig. 5: s_{100}/Q- discharge relationship for Bore No. 21346.

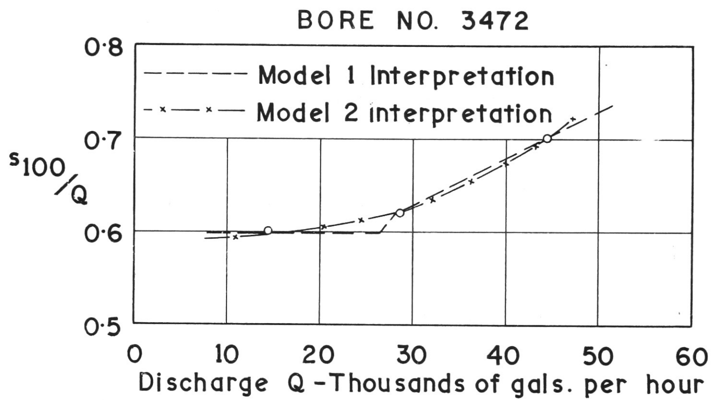

Fig. 6: s_{100}/Q- discharge relationship for Bore No. 3472.

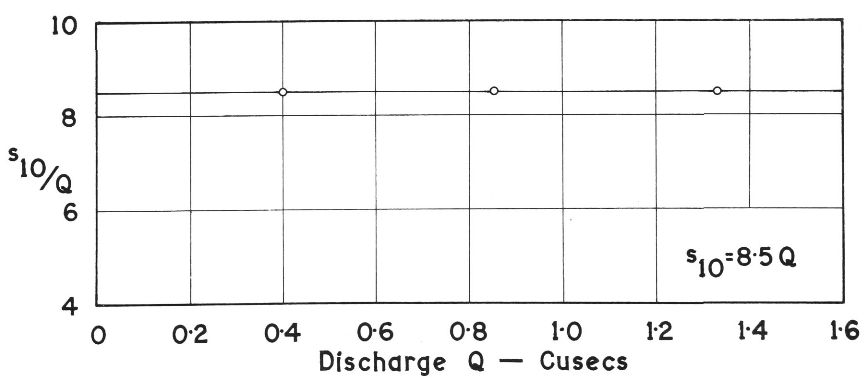

Fig. 7: s_{10}/Q- discharge relationship.

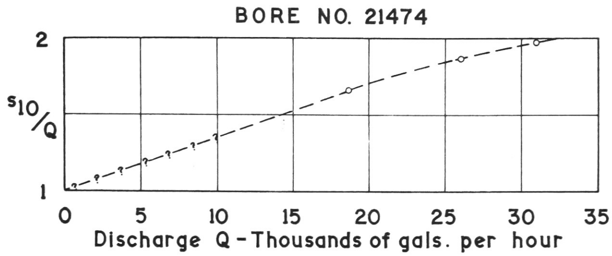

Fig. 8: s_{10}/Q - discharge relationship for Bore No. 21474.

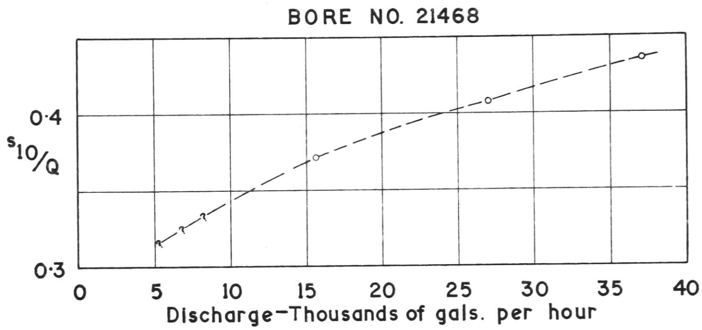

Fig. 9: s_{10}/Q- discharge relationship for Bore No. 21468.

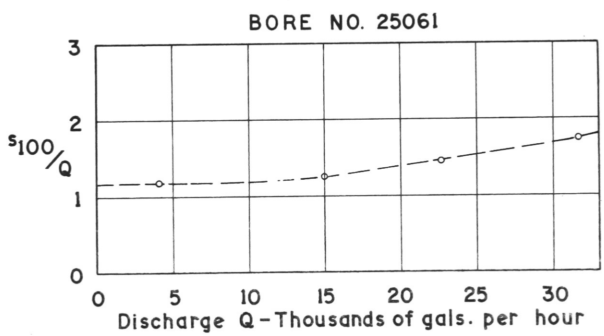

Fig. 10: s_{100}/Q- discharge relationship for Bore No. 25061.

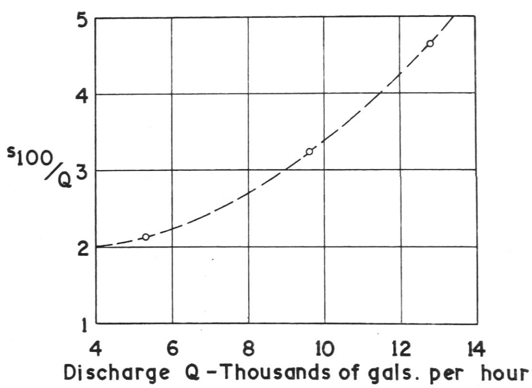

Fig. 11: s_{100}/Q- discharge relationship.

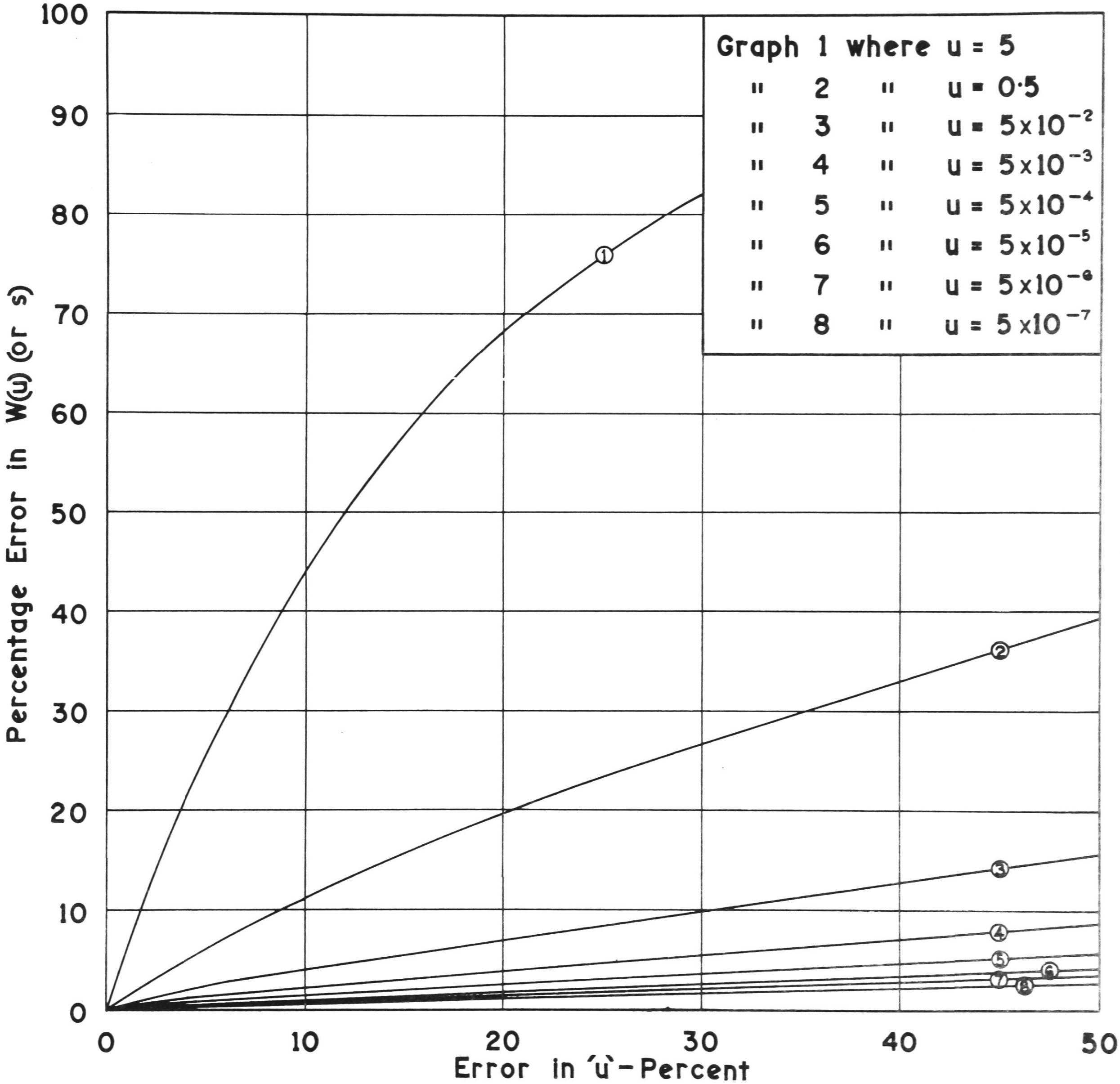

Fig. 12: Relationship between error in u(pc) and error in W(u) (pc)

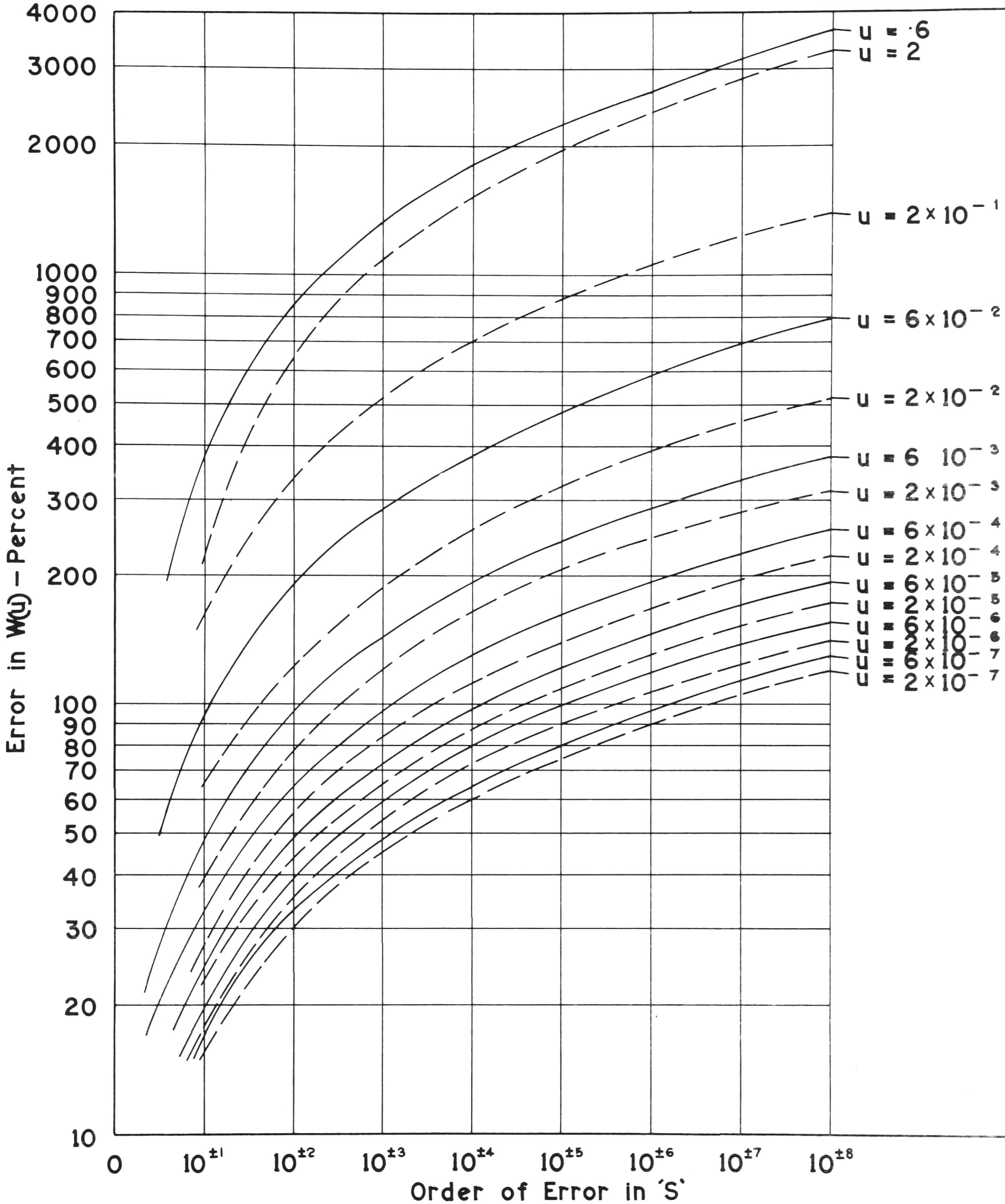

Fig. 13: Effect of gross error in storage coefficient (S) on the computed values of W(u).

t = time since pumping commenced (minutes)

Δu = percentage error in u.

Δr = percentage error in r.

ΔS = percentage error in S.

Then the maximum error in u for errors in r and S is given by:-

$$\Delta u = \pm (\Delta r) \times 2 + (\pm \Delta S) \; . \; . \; . \; . \; . \; . \; . \qquad \text{(B3)}$$

Thus for an error of either $\pm$ 10 per cent in r and S the corresponding error in u can be:-

$$\Delta u = \pm (10 \times 2) \pm 10$$

$$= \pm 30\%$$

The effect of such errors in u on corresponding values of W(u) has been obtained from the published tabulations of these parameters. The results for small (up to 50%) and large errors in u are shown in the graphs Figs. 12 and 13 respectively.

Obviously from equation B1 any percentage error in W(u) results in a similar percentage error in values of drawdown.

Fig. 12 and 13 show that considerable errors in drawdown can arise from large errors in u and this is especially so for the larger values of u.

DISCUSSION

W.H. Williamson, Water Conservation and Irrigation Commission, N.S.W.

The following comments are submitted by way of discussion on the subject paper:-

Mention is made of the method described by me at a Groundwater School in Adelaide, 1965 for plotting the data from a multiple stage pumping test. Since the reference given to this was not published, it is recorded here that the method is that by Bruin & Hudson (1955). At the aforementioned Groundwater School, Mr. J. Colville pointed out that the method was not strictly valid. He proposed that the data for the various incremental discharges should be individually plotted from the same time origin in each case, and provided the mathematical analysis. However, although urged to do so, Mr. Colville did not publish this. Subsequently, Lennox independently published the same method and analysis.

In practice, results obtained using Bruin & Hudson's method and Lennox's method normally do not vary significantly, especially for aquifers of high transmissivity. Consequently, because of its greater convenience, Bruin & Hudson's method is still used by the W.C. & I.C. for analysing multiple stage tests in the general run of cases. However, as well as being more correct, Lennox's method has the big advantage of being able to utilize early time - drawdown data in each stage and this is sometimes a drawback of Bruin & Hudson's method. Therefore, when circumstances indicate the need for a more rigorous solution, Lennox's method is used. It is generally considered desirable that it be used in cases where transmissivity is relatively low (in which case difficulty may be experienced in fitting a line of correct slope in extrapolating drawdown for second and subsequent stages by Bruin & Hudson's method), or where it appears that early time - drawdown data should be utilized to aid interpretation. In brief, if there is any doubt, Lennox's method is used.

To overcome one of the problems inherent in Bruin & Hudson's method (as referred to above in relation to the effect of relatively low transmissivity) Mr. Forbes has proposed that in a three stage test in which the duration of the first stage is "a" minutes, the duration of the second stage should be "3a" minutes. I do not see the necessity for this. If, as is desirable, the time - drawdown data are plotted as the test proceeds, it will be apparent whether they are conforming to the requirements for the application of the Bruin & Hudson method, but if they are not it is simpler to use Lennox's method for the subsequent analysis rather than incur the time and expense of extending the duration of the stage.

With regard to the duration of individual stages of multiple-stage or step-drawdown tests, I consider 100 minutes to be desirable, and would urge particular caution against very short duration stages such as 10 minutes. It is true that 10 minute stages should theoretically give valid results, but ideal conditions would be required and, unfortunately, these are not easily obtained. Good results from short stage tests are again more likely when the aquifer transmissivity is high. Apart from other problems, one of the bugbears of short tests is determining the effective discharge, and this is particularly difficult in the case of low transmissivities because of the rapidly changing head conditions in the early minutes of pumping. In the subsequent analysis, the plot of $^s/_Q$ against Q is quite sensitive and relatively small errors in data could give misleading indications. For the same reason I would be dubious about drawing curves to three points in a plot of $^s/_Q$ against Q from 10 minute data, and inferring hydraulic conditions from such curves, as has been done in figures 8 and 9 (Appendix A). Consequently, for the general run of cases I feel that short stages, such as 10 minutes, are best avoided. If they are used it should only be under very well controlled conditions and preferably for aquifers of high transmissivity.

I do not altogether agree with Mr. Forbes' contention that "the determination of B and C in Jacob's equation is a worthless and unnecessary exercise". While it is true that the values of these factors are not essential to assessing the performance of a bore if data is available from a range of discharges extending beyond the required or ultimate design rate, it is by no means uncommon to be presented with data for analysis and to find that the desired or optimum yield is in excess of the maximum yield that could be achieved by the test - pump unit. The obvious moral, of course, is for contractors to have test - pumps capable of exceeding the optimum yield of all bores they construct, but until this happy day arrives I see no alternative to having recourse to Jacob's equation for extrapolation. I think it is now generally recognised that the specific values of C proposed by Walton (1962) are not valid indications of the efficiency of a bore or the effectiveness of development. (In discussion with Mr. Prickett at Illinois State Water Survey, he stated that Walton (formerly of that Survey) concedes that the values he proposed were based on particular cases which were too few in number and insufficient in range). However, bearing in mind the difficulties and limitations well brought out by Mr. Forbes, if the data conform to Jacob's equation I feel that it is still useful to have some indication of the "well - loss" component of the total drawdown, and this involves determination of values for B and C.

It is true that anomalous aspects sometimes occur, and Mr. Forbes' valuable exposition on Jacob's equation, could well account for some of these. Other aspects of course, are the assumptions of the aquifer being infinite, homogeneous, and so on. However, the results of analysing numerous multiple stage pumping tests over the years, commonly involving extrapolation to assess the effect of pumping at higher rates than the maximum test rate for production purposes, have given rise to remarkably few problems from subsequent bore behaviour.

There is still the problem of determining the true efficiency of a bore, in relation to the formation. The logical approach is through the aquifer parameters, but this is beset by the difficulties in determining the true storage coefficient, especially if only the pumped bore is available. I fear that the methods proposed by Mr. Forbes for the latter will have limited application in the general run of cases because of the extreme accuracy required in the determination of Δs and the need to assume effective well radius.

However, it will be interesting to hear of the experience and views of others on this matter.

In conclusion may I congratulate Mr. Forbes on the thought and effort he has given to his paper and the insight he has given to some of the problems of well hydraulics.

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The Author in Reply

Concerning the multiple stage pumping test it was not realised that the method referred to was originally devised by Bruin and Hudson and the author appreciates the clarification of this point. The reason for the inaccuracy of the method lies in the fact that after the first stage the graph should theoretically always remain curved. Hence, extension by a straight line to obtain values of incremental drawdown after the first stage is incorrect. Certainly a straight line with a slope computed from the slope of the first stage multiplied by the ratio of the discharges should not be used.

The technique of extending the time duration of the second stage in a three stage test was obviously suggested for tests with stages of short but adequate time duration, or for analysts not wishing to use the longer Colville-Lennox method. It was suggested as a technique which could be found useful and not necessarily as a method to be always adopted. It is the author's opinion that step drawdown tests should always have more than three stages.

The duration of individual stages of multiple-stage drawdown tests is an important question. The length of the stages should be determined by the condition of the aquifer and by the information required from the test. From experience over many years it appears that where the aquifer conforms to the non-leaky infinite artesion case and the well is stable with a well-controlled test being performed, reliable information for most purposes can be obtained for the 10 minute interval.

With regard to the reliability of early information, calculations on a well with a transmissivity of approximately 20,000 gals/day/foot indicated that whilst information for 1, 2 and 3 minutes was slightly in error estimates of drawdown based on analyses at the 10 minute interval were just as reliable as estimates at 20, 30, 50 and 100 minutes. Aquifers of low transmissivity are usually pumped at quite low discharge rates hence drawdowns are rarely much greater than those encountered in wells constructed in aquifers of much higher transmissivity.

Figures 1 and 2 in the original paper and figure D1 represent a typical case in point. The transmissivity is of the order of 56,000 gals/day/ft - a moderately high transmissivity. Results have been plotted according to both the Bruin and Hudson (Fig. D1) and the Colville-Lennox methods (Fig.2).

It will be noted that the data from 5 minutes plots precisely on the straight line in Fig. 1 thus the analysis for any time greater than 5 minutes would result in acceptable estimates of future drawdowns being made. Additional calculations indicate that no advantage was obtained in testing this well beyond 10-20 minutes per stage.

With regard to the problem of establishing constant pumping rates when the water level is falling rapidly, an analysis of commercial borehole turbine pumps has indicated very effectively that discharge regulation need be no problem providing a suitable pump is used (see Forbes (1969)).

Problems encountered in analysing the results of multiple-stage tests usually arise due to the non-compliance of the aquifer with the non-leaky artesion case, in which event the aquifer may follow the water table model or the leaky artesion case or be affected by unsuspected boundary conditions. Other problems may be due to instability of the well and an inability to control discharge.

instability of the well and varying discharge should normally invalidate the test and suitable remedial steps should be taken and the well retested.

Aquifer problems require different treatments. In the water table case it should be possible, by making each stage of sufficient length, to ensure approximation to the artesion case for the maximum discharge rate and to employ the test to predict future drawdowns.

Where boundaries exist close to the well suitable results can often be obtained by reducing the length of each stage sufficiently to ensure that the whole test is complete before boundaries influence the drawdown curve. Other aquifer peculiarities may not be so easily overcome and each case must be considered on its merits.

With regard to the use of three points in a plot of $^s/_Q$ v Q for ten minute data, only three stage tests were available in this work. It is not considered that the use of 10 minute data is detrimental in any way in this case.

Prediction of drawdown for discharges in excess of tested rates can be made by extension of the s_t v Q curve beyond the last point with at least the same accuracy as the use of Jacob's formula. The results for Figures 1, 2 and D1 should be studied carefully. Type curves for such extension would be fairly easy to prepare.

The author does not agree with Mr. Williamson's statement that it is now generally recognised that specific values of C are not valid indications of the efficiency of a bore.

The determination of Jacob's C from Figs. 2 and D1 for times of 10 minutes and 100 minutes respectively reveals that, in each case, the value of C at 100 minutes is almost double that at 10 minutes. Bearing in mind the degree of fit of the data to the experimental curve, this is surprising as Jacob's theory demands that C be a oonstant. Superficially it may be argued that this change in C is due to compaction of the pack and aquifer close to the well. However, if this were the case the data would not plot as straight lines but as curves. Similar results have been observed in other wells where determinations for C have been made for different time intervals. This is possibly what was referred to in the discussion on the paper when it was suggested that well loss appeared to be time dependent (although progressive compaction could also explain this). These results are difficult to explain but they do suggest that undue arithmetic manipulation of pump test results is not justified at this stage.

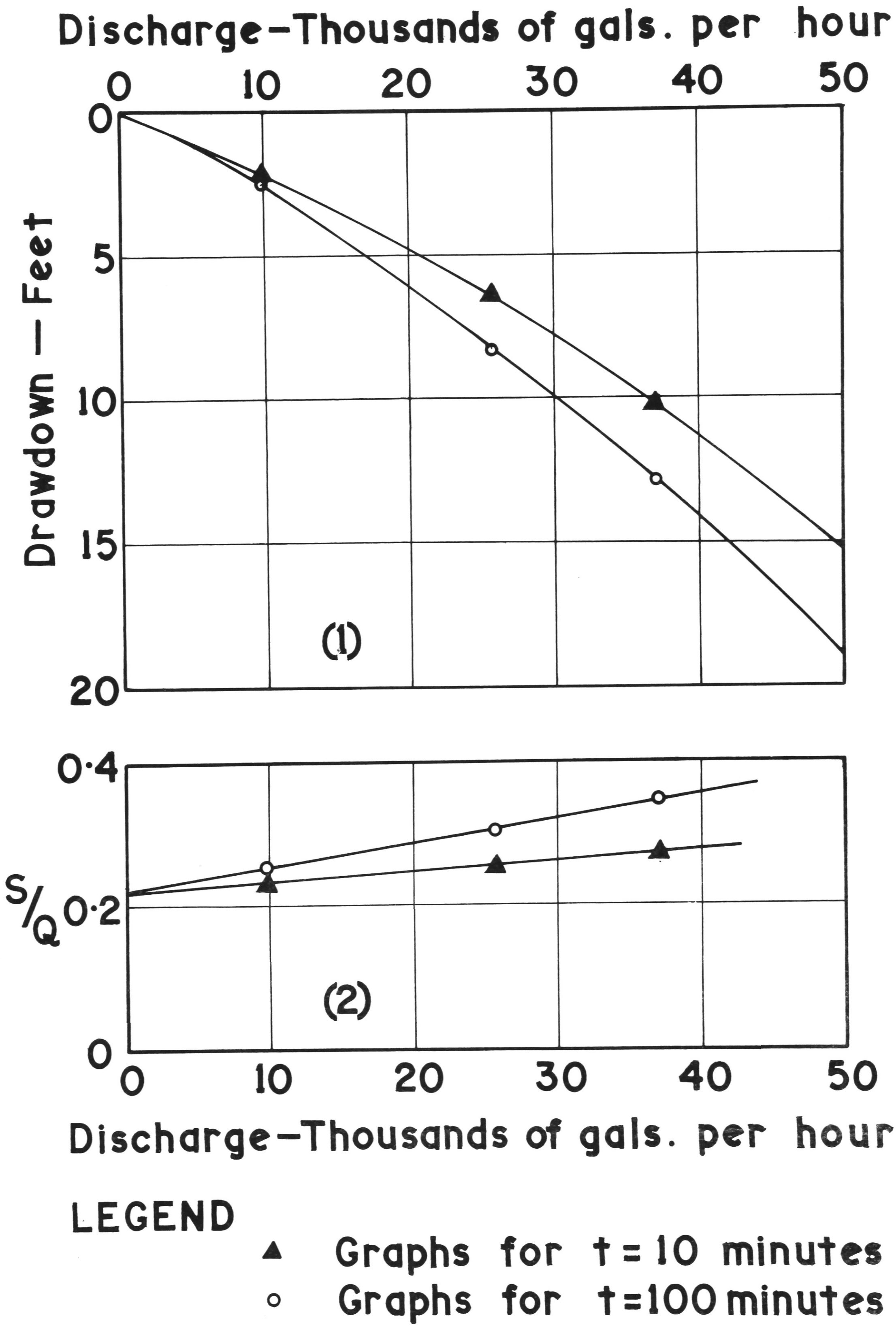

Fig. D1: Drawdown - discharge and s/Q - discharge relationships for the data given in Fig. 1 and compiled using the Bruin and Hudson Method.

Mr. Williamson refers to an unpublished paper of the author (Forbes (1969)). Before discussing the accuracy of the method it is necessary to consider the order of accuracy required in estimates of storage coefficient to ensure accurate estimates of drawdown. Appendix A in the paper shows that errors in u of 50%, or even 100%, do not seriously affect derived values of drawdown even when u is as large as 10^{-4}. The size of u and the effect of errors in S and r (see Eq. B2) can be controlled very effectively by selecting t sufficiently great to ensure that u is always less than say 10^{-5}. Thus one would probably determine well efficiency at 24 hours instead of 1 minute if S or u were rather large. Extreme accuracy in estimating storage coefficient is normally not required.

The method suggested in the above paper gives progressively better estimates of S as its value increases and as the value of well radius increases. The method is based upon the slope of the drawdown or recovery curve which can normally be obtained to a far higher degree of accuracy than that of the individual points making up the graph. In any event it is probable that the degree of accuracy required to give acceptable values of S is not so critical when S is greater than 10^{-4} for wells with a radius of 0.5 ft or greater.

As suggested above, values of S smaller than 10^{-4} need not be determined to a very high order of accuracy because of the insensitive nature of u when small values are involved. Admittedly care must be taken if good results are to be obtained and, in order to perfect the method, more accurate values for the standard constants found in current formula have been computed.

Effective well radius as defined by Jacob is not used in the calculations; the well radius is defined as in the paper and should be capable of evaluation to a suitable degree of accuracy.

It is felt that the method has merit and will be found useful.

I wish to thank Mr. Williamson for taking the time to read and comment on the paper and for raising so many interesting points.

PAPER No.4

ONE-DIMENSIONAL GROUNDWATER RECHARGE

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SUMMARY

The hydrologic significance of water movement in the unsaturated zone is discussed with particular reference to one-dimensional groundwater recharge. The relationships which characterize the hydrologic behaviour of unsaturated porous materials are then briefly described and recent instrumentation techniques detailed. A numerical solution of the differential flow equation is then outlined and its application to finite depth problems discussed.

1. INTRODUCTION

This decade has seen the coming of age from an analysis viewpoint of an important and often neglected part of the hydrologic cycle - namely the entry and movement of water in the soil profile. Due to the formidable nature of satisfactory physical analysis and the inherent problems of measurement, both in the field and laboratory, the movement of water in the unsaturated zone was for many years approached from an empirical viewpoint and could only be classed as a poor relation within the family of hydrologic studies. Intensive effort was concentrated on surface water research and research involving saturated aquifers with little detailed study on the physics of flow in the unsaturated zone, although that zone formed the important hydrologic link between the relatively stable hydraulic state in the groundwater aquifers and the almost continuously changing conditions at the soil surface. This lack of concentrated research has been discussed by Linsley (1964).

The growth of the subject has been marked by the development of a valuable conceptual framework to explain the physical processes involved in flow in unsaturated soils, by significant instrumentation improvements and by the ready availability of high speed digital computers. Such developments are of basic importance to the agriculturalist and irrigation engineer but, in addition, must become of increasing importance to research workers involved in a more general way with water resources planning, particularly in relation to the present study area of groundwater recharge, whether natural or artificial. The term 'soil-water hydrology' is becoming more common in describing processes in the unsaturated zone such as infiltration, drainage, evaporation from bare soil surfaces, soil water removal by plants and the transport of soluble material in soils. However, it should be stressed that the use of such a term should not encourage the development of rigid subject compartments. The hydrologic processes involved are dynamic and continuous and these must be paralleled, whenever possible, in our analysis by an integrated approach.

This paper has been restricted to a consideration of the one-dimensional groundwater recharge process. The principal reason for this is that an understanding of the physical processes involved can best be obtained using a one-dimensional system. In addition, the limitations of space make a more comprehensive study impossible. Baumann (1965) has recently given a review of technical developments in groundwater recharge with emphasis on two-dimensional flow and mound development.

The aim of this paper is to present a short summary of the current position from both the analytical and experimental viewpoints for the system under study. Only the physical aspects of the system have been considered, although it is realized that, in such cases as artificial recharge using low quality water, the chemical and biological factors have significant effects on the flow processes.

2. HYDROLOGIC CHARACTERISTICS

Two relationships are required to characterize the hydrologic behaviour of unsaturated porous materials and these are now generally well known. The first reflects the fact that water in an unsaturated soil is retained with considerable force. The energy of retention has been variously described but in this paper the term pressure head (h) will be used with units, cm. head of water. For an unsaturated soil the pressure head is characteristically negative relative to atmospheric pressure. The pressure head is plotted against the water content (θ) usually expressed on a volumetric basis (cm^3/cm^3). The form of the relationship is strongly hysteretic and depends on the sequences of wetting and draining. The boundary curves in the relationship are defined by draining from saturation and wetting from a very dry condition. In certain rather idealized situations where the hydrologic process is either a completely wetting one or a completely drying one the unique boundary curves in $h(\theta)$ only need by used. However, most systems which simulate field conditions have a complex wetting and draining history and require analysis using the scanning curve data within the hysteresis loop.

The second requirement is the relationship between the hydraulic conductivity (K,cm/min) and the water content. This could alternatively be expressed in terms of K(h) but the former is preferable since there is little, if any, hysteresis in $K(\theta)$ whereas K(h) reflects the hysteresis pattern of $h(\theta)$. The $K(\theta)$ relationship reveals a very marked decrease in K from the saturated value K_{sat}, for small decreases in water content.

Experimental techniques are now available (see next section) enabling the above relationships to be determined under laboratory conditions. However,the position for field measurement is less promising. In general the boundary $h(\theta)$ curves present no problem but the scanning curves and $K(\theta)$ are difficult to measure satisfactorily, particularly in a non-homogeneous profile.

3. INSTRUMENTATION

The technique of measuring the pressure head in unsaturated soils using a tensiometer is now well known. The range of the tensiometer in practice is limited to -850cm of water and the principal development in recent years in their use has been to incorporate pressure transducers in the system so that rapid response measurement together with the facility of convenient recording can be achieved. Rapid response and nondestructive measurement of both pressure head and water content are necessary to determine the hysteresis pattern in $h(\theta)$. By using miniature differential strain gauge pressure transducers it is possible to design tensiometer-pressure transducer units for both laboratory and field use (Watson (1967)). Early equipment in the laboratory used up to twenty tensiometers connected to one pressure transducer through a hydraulic selector valve. Such an arrangement was generally satisfactory but had shortcomings when very rapid changes of pressure occurred (e.g. at early times in the drainage of a saturated profile) and when the water content was low, making it impossible to switch satisfactorily from one tensiometer to another. This latter difficulty was due to the low conductivity of the material and the lack of available water to meet the volume displacement requirements of the pressure transducer. In recent equipment this difficulty has been satisfactorily overcome by using sufficient transducers to have one mounted at each measuring point. These can then be scanned by electronic means with the voltage output from the pressure transducers being measured by a digital voltmeter and then recorded on printed paper tape. The printed paper tape output is necessary for control purposes during an experiment. However, it is not a convenient method for computer input and it is hoped shortly to record the output in serial form on magnetic tape for direct input to the computer. It is possible to scan the tensiometer-pressure transducer units at a rate of ten per second.

Although the range of the tensiometer is satisfactory for measurements of pressure head with coarser porous materials it is limited when used with finer materials and soils which exhibit much lower values of soil water pressure. Two instruments are now available to overcome this problem. The first is the osmotic tensiometer (Peck and Rabbidge (1969)). This instrument makes use of a confined aqueous solution as the reference state in measuring the soil water potential. By such a depression of the reference level it is possible to measure potentials throughout the range 0 to -15 bar. Instruments of this type are now available commercially in Australia under the trade name "Aquapot".

The second method makes use of a thermocouple psychrometer. A suitable instrument has been described by Rawlins and Dalton (1967). In this instrument, which was designed to meet certain boundary conditions, the thermocouple is inserted in a thin walled ceramic sphere approximately 2 cm in diameter. The thermocouple wires were 0.0025 cm dia. Chromel-P and constantan and particular care was taken with heat sinks. The method has become feasible for field use because of the availability recently of portable nanovoltmeters having a minimum range of 0.01 μV.

The nondestructive measurement of water content for rigid porous materials in the laboratory is now usually carried out using gamma ray techniques. In this method the water content is inferred from density changes measured by gamma ray attenuation using a Cs-137 source which has an energy of 0.66 MeV. For soil with a pore geometry which changes with water content change it is necessary to use an additional isotope having a different energy and different mass absorption coefficients. The attenuation equations can then be solved simultaneously for both water content and bulk density. A suitable second source is Am-241 which has an energy of 0.06 MeV. Although the gamma ray method is now fairly common in laboratory studies it can also be used for field measurements. Its use in the field has the advantage of obtaining a generally planar water content whereas the alternative and far more common nuclear method using the neutron scattering technique measures the average water content of a sphere of soil often up to ten inches in diameter.

4. NUMERICAL ANALYSIS

Darcy's Law has been found to be applicable to flow in unsaturated soils (Childs and Collis George (1950), Watson (1966)) if the hydraulic conductivity is considered as water content (or pressure head) dependent. Hence

$$v = -K(\theta) \frac{d\phi}{dz} \tag{1}$$

where

$$v = \text{flux (cm/min)}$$

and

$$\phi = h + z \text{ (cm)}$$

Since steady state problems are rare in water movement studies it is usual to combine Darcy's Law with the equation of continuity

$$\frac{\partial \theta}{\partial t} = -\frac{\partial v}{\partial z} \tag{2}$$

where t is time and obtain the differential flow equation

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z}\left[K(\theta)\frac{\partial \phi}{\partial z}\right] \tag{3}$$

where z is measured +ve upwards. This reduces to

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z}\left[K(\theta)\frac{\partial h}{\partial z}\right] + \frac{\partial K}{\partial z} \tag{4}$$

For analytical purposes it is convenient to introduce a diffusion coefficient D where $D = K\frac{\partial h}{\partial \theta}$ so that equation (4) reduces to

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z}\left[D(\theta)\frac{\partial \theta}{\partial z}\right] + \frac{\partial K(\theta)}{\partial z} \tag{5}$$

In equation (5) D is dependent on the history of wetting and draining and accordingly its use has been restricted to cases where $D(\theta)$ is unique.

Equation (5) has been studied in detail and solved by quasi-analytical methods for infiltration into infinitely deep profiles. Philip (1969) has given a detailed review of these methods. These have been particularly valuable in giving an understanding of the physics of the infiltration process.

In groundwater recharge problems we are usually more concerned with finite depth solutions where the entire profile from natural surface to water table must be included in the analysis. The boundary conditions relating to this type of problem do not lend themselves to quasi-analytical solution. However they can be conveniently solved by numerical methods using high-speed digital computers.

A convenient form of equation (3) for drainage and recharge work is the pressure head form which for vertical isothermal flow in a homogeneous porous material is

$$C(h)\frac{\partial h}{\partial t} = \frac{\partial}{\partial z}\left[K(h)\frac{\partial h}{\partial z}\right] + \frac{\partial K(h)}{\partial z} \tag{6}$$

where $C(h) = \frac{d\theta}{dh}$ is the volumetric water capacity.

To solve equation (6) numerically a grid of points was superimposed on the region $t \geq 0$, $-L \leq z \leq 0$ where L is the depth of the profile. The z axis was divided into N intervals (N=100). The mesh points are defined by:

$$t_m = m\Delta t \quad m = 0, 1, 2 \,\ldots\ldots\ldots \tag{7}$$

$$z_n = -(N-n+1)\Delta z \quad n = 1,2 \ldots N+1 \tag{8}$$

$$\Delta z = L/N \tag{9}$$

The partial derivatives in (6) were approximated by the finite differences:

$$\frac{\partial}{\partial z}\left[K(h)\frac{\partial h}{\partial z}\right] \approx \frac{1}{2(\Delta z)^2}\Big[K_{n+\frac{1}{2},m-\frac{1}{2}}\left(h_{n+1,m} + h_{n+1,m-1} - h_{n,m} - h_{n,m-1}\right)$$

$$- K_{n-\frac{1}{2},m-\frac{1}{2}}\left(h_{n,m} + h_{n,m-1} - h_{n-1,m} - h_{n-1,m-1}\right)\Big] \qquad (10)$$

$$\frac{\partial h}{\partial t} \simeq \frac{h_{n,m} - h_{n,m-1}}{\Delta t} \qquad (11)$$

$$\frac{\partial K(h)}{\partial z} \simeq \frac{K_{n+\frac{1}{2},m-\frac{1}{2}} - K_{n-\frac{1}{2},m-\frac{1}{2}}}{\Delta z} \qquad (12)$$

where arbitrarily

$$K_{n+\frac{1}{2},m-\frac{1}{2}} = \frac{K_{n,m-1} + K_{n+1,m-1} + K_{n+1,m} + K_{n,m}}{4} \qquad (13)$$

and a similar definition is true for $K_{n-\frac{1}{2},m-\frac{1}{2}}$.

The finite difference approximations (10) through (13) were substituted in eq.(6) to obtain N-3 algebraic equations of the form

$$E_n\, h_{n-1,m} - F_n\, h_{n,m} + G_n\, h_{n+1,m} = -H_n \qquad (14)$$

By invoking the boundary conditions two other equations were obtained giving N-1 nonlinear equations in N-1 unknowns. To remove this nonlinearity an iteration process was used. Further details of the numerical analysis may be found in Whisler and Watson (1968).

The simplest finite depth problem is the drainage to a stationary water table of an initially saturated homogeneous profile. The boundary conditions for such a system are

$$\text{Upper boundary:} \quad -K(h)\left[\frac{\partial h}{\partial z} + 1\right]_{z=0} = 0 \qquad t > 0 \qquad (15)$$

$$\text{Lower boundary:} \quad h(z,t)_{z=-L} = 0 \qquad t > 0 \qquad (16)$$

The computed pressure head profiles for a 120 cm depth of Botany sand draining to a water table are given in Figure 1. Botany sand has a

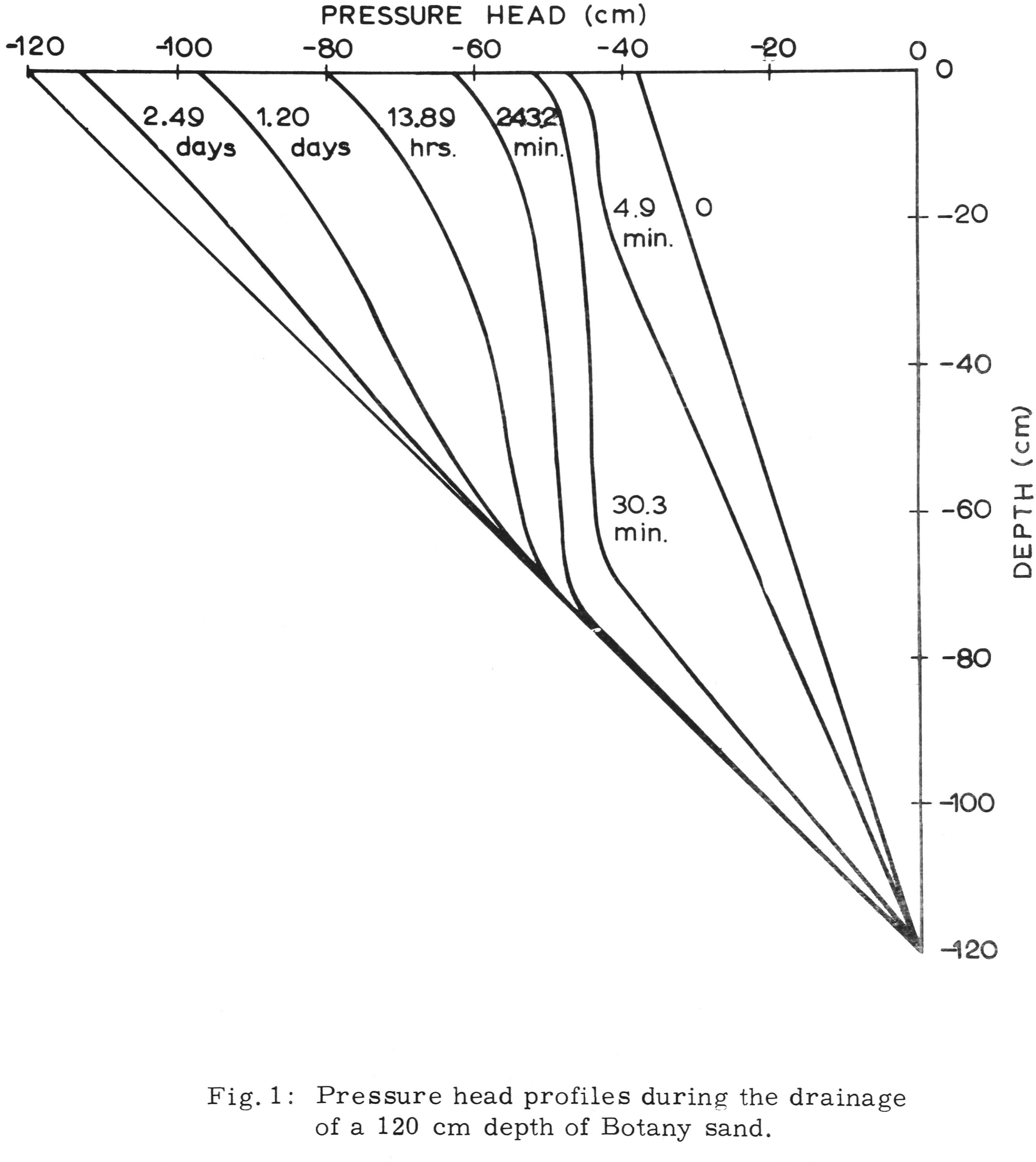

Fig. 1: Pressure head profiles during the drainage of a 120 cm depth of Botany sand.

K_{sat} value of 1.116 cm/min and an air entry value of -38 cm of water. Since the system is a completely draining one the boundary draining curve of the $h(\theta)$ relationship is used. The computed time for the 120 cm profile to reach the linear equilibrium condition was 8.19 days. The equivalent water content profiles are given in Figure 2.

The draining analysis can be extended to include drainage to a falling water table. This particular condition has relevance in studying delayed yield effects in pumping tests in unconfined aquifers. Another numerical extension which is as yet unpublished is the drainage of a stratified profile where a coarse material overlies a finer material. Any number of layers can be considered in the analysis but the coarse over fine sequence must always be a boundary condition at any interface. A fine over coarse sequence at an interface introduces problems of limited air access to the bottom layer and has not yet been analysed in detail.

In the analysis of certain groundwater recharge operations the drainage of the saturated profile forms a suitable initial condition prior to the establishment of some intermittent pattern of water application to the surface. Upon rewetting, the boundary draining relationship between water content and pressure head is no longer applicable and the primary hysteresis scanning curves must be used. Whisler and Watson (1969) have analysed ponded infiltration into a draining profile.

The boundary conditions for the system are

$$\text{Upper boundary:} \quad h(z,t)_{z=0} = h_T \qquad t > 0 \tag{17}$$

$$\text{Lower boundary:} \quad h(z,t)_{z=-L} = 0 \qquad t > 0 \tag{18}$$

where h_T is the depth of ponded water during inundation.

The initial condition is

$$h(z,t) = h(z) \quad t = t_o, \; 0 \geq z \geq -L \tag{19}$$

The system is illustrated numerically by considering a 400 cm depth of Botany sand which is draining to a water table. The initial conditions for the infiltration phase are the pressure head profiles at times of 31.1 min and 1.04 days after the start of drainage. The depth of ponding h_T is 15 cm. The pressure head profiles after 25.2 min of wetting are given in Figure 3. The continuing drainage of the 31.1 min initial profile is clearly seen during the wetting up period in contrast to the 1.04 day initial period where the drainage is negligible over 25.2 minutes.

The next phase in the cycle of intermittent recharge occurs when the ponded infiltration is terminated. Redistribution of the infiltrated water then occurs often while the lower section of the profile is continuing to drain. A computer programme for the solution of this problem has also been prepared but as yet it is unpublished pending experimental confirmation.

The above pattern of wetting and redistribution can be repeated either regularly or randomly in time. The degree of complexity involved in the analysis will depend principally on the timing of the events. As the history of wetting and draining becomes more complicated the handling of the hysteresis information becomes more formidable.

Another complication that may occur involves the change in pore air pressure during the wet front descent. In addition, a profile may exhibit one dimensional heterogeneity when $K(\theta)$ and $h(\theta)$ will vary with depth. However, these added complexities can be readily included in the numerical analysis and present no major difficulties in solving the systems.

5. ACKNOWLEDGEMENT

The support of the Australian Research Grants Committee in providing funds for the purchase of equipment and for computer time is warmly acknowledged.

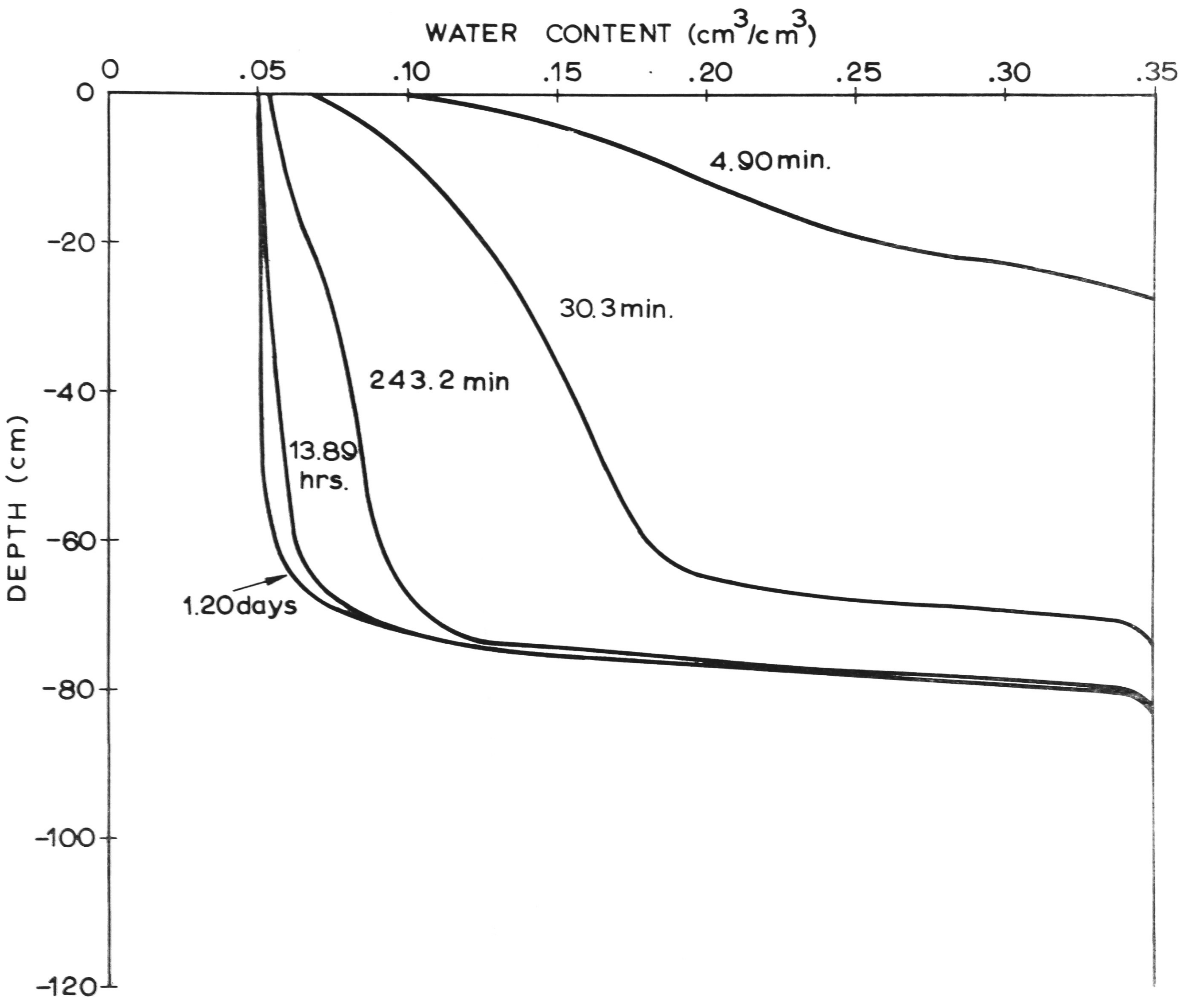

Fig. 2: Water content profiles during the drainage of a 120 cm depth of Botany sand.

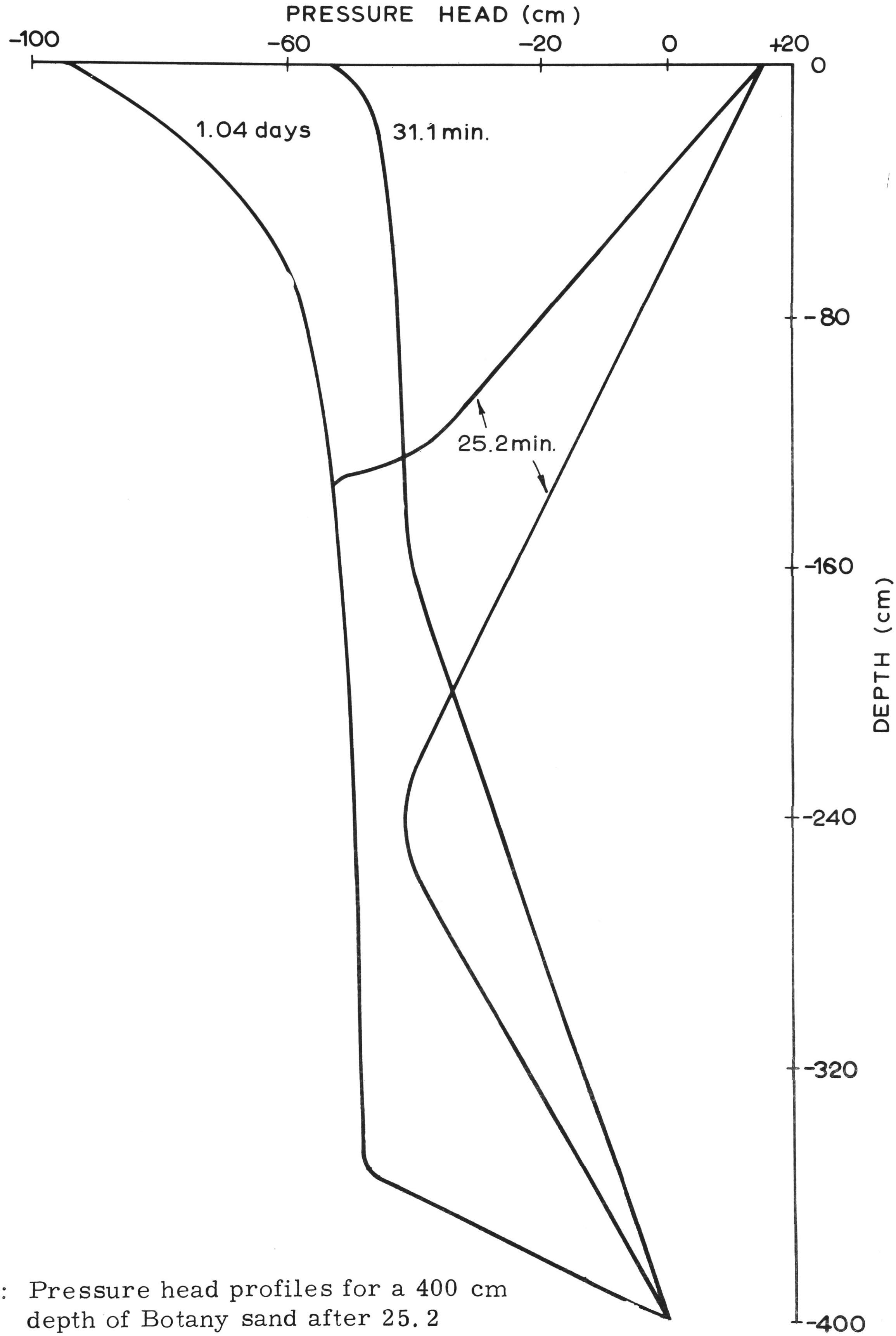

Fig. 3: Pressure head profiles for a 400 cm depth of Botany sand after 25.2 minutes of surface inundation under 15 cm of water.

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PAPER No.5

ARTIFICIAL GROUNDWATER RECHARGE - OPERATIONAL FEATURES IN THE BURDEKIN DELTA AND INVESTIGATION PROGRAMME IN THE BUNDABERG AREA

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PART A

GROUNDWATER RECHARGE - BURDEKIN DELTA

1. DESCRIPTION OF AREA

The Burdekin Delta is a 240 square mile area on the east coast of Queensland, and contains the towns of Ayr and Home Hill. It consists of a body of deltaic sediments overlying an old granite surface. Ground levels in the area range from R.L.5 state datum near the coast to R.L.70 on both banks on the western edge of the delta. Average annual rainfall for the area is 43 inches.

Chief industry on the delta is sugar cane growing. Cane assignments for selected years are given in Table I and areas irrigated over the last eight years are given in Table II.

TABLE I

CANE ASSIGNMENTS

Year	1940	1950	1963	June 1964	Dec. 1965	1966	1967	1968	1969
Area in Acres	31,000	35,000	56,000	56,000	72,000	72,000	72,000	72,000	72,000

TABLE II

AREA IRRIGATED

Year	1960/61	1961/62	1962/63	1963/64	1964/65	1965/66	1966/67	1967/68
Area in Acres	45,600	48,300	49,500	53,800	62,000	69,000	69,000	70,000

2. INVESTIGATION PROGRAMME

This was carried out during the period 1959-1964 and provided the following information.

The main water bearing material was found to consist of coarse sand whose total thickness occupied about one third to one half of the depth from ground level to bedrock. (Bedrock contours obtained from drilling are shown on Fig.1). The average water bed thickness in the area is 30 to 40 feet and an average transmissivity of 33,600 ft^2/day was determined from pump tests. Values of storage coefficient for the area ranged from .02 to .20 and the average is approximately 0.16. Water quality was generally in the range 140 to 2000 parts per million of total solids.

Groundwater level contours drawn for late January 1963 and early February 1964 showed that over this period a net loss occurred in the underground storage of the delta. In addition rainfall which was known to be the major factor in natural replenishment, was above average during the climatic year 1962/63 and river flow, the minor factor, was also better than average. Therefore it was concluded that the water use in 1963, estimated to be 45 ins. on 53,800 acres or 202,000 acre feet, exceeded the long term safe yield of the basin and that further increases in the area under cane would increase the demand and hence the over-draft. An analysis of water level behaviour over the period 1927 to 1963 indicated that annual deficiencies in the basin were 64,000 acre feet in the short term, and 30,000 acre feet in the long term based on the area irrigated in 1963/64.

3. REPLENISHMENT SCHEME

Following an investigation into possible methods of artificial recharge, including field and laboratory tests, a recharge scheme was implemented involving the construction of three pump stations on the Burdekin River to provide recharge water for the following replenishment channels.

(a) North Side

Sheepstation Creek natural water course - pump station capacity 30 cusecs.

Plantation Creek - natural water course - pump station capacity 2 x 30 cusecs.

Approval for a 30 cusec relift station from Plantation Creek to Lilliesmere Lagoon was obtained in January 1969, and approval for an additional 30 cusec pumping capacity into Plantation Creek from the Burdekin River was obtained in April, 1969.

(b) South Side

Warrens Gully into Sandpit Line - improved natural watercourse and artificial channel - pump station capacity 2 x 30 cusecs.

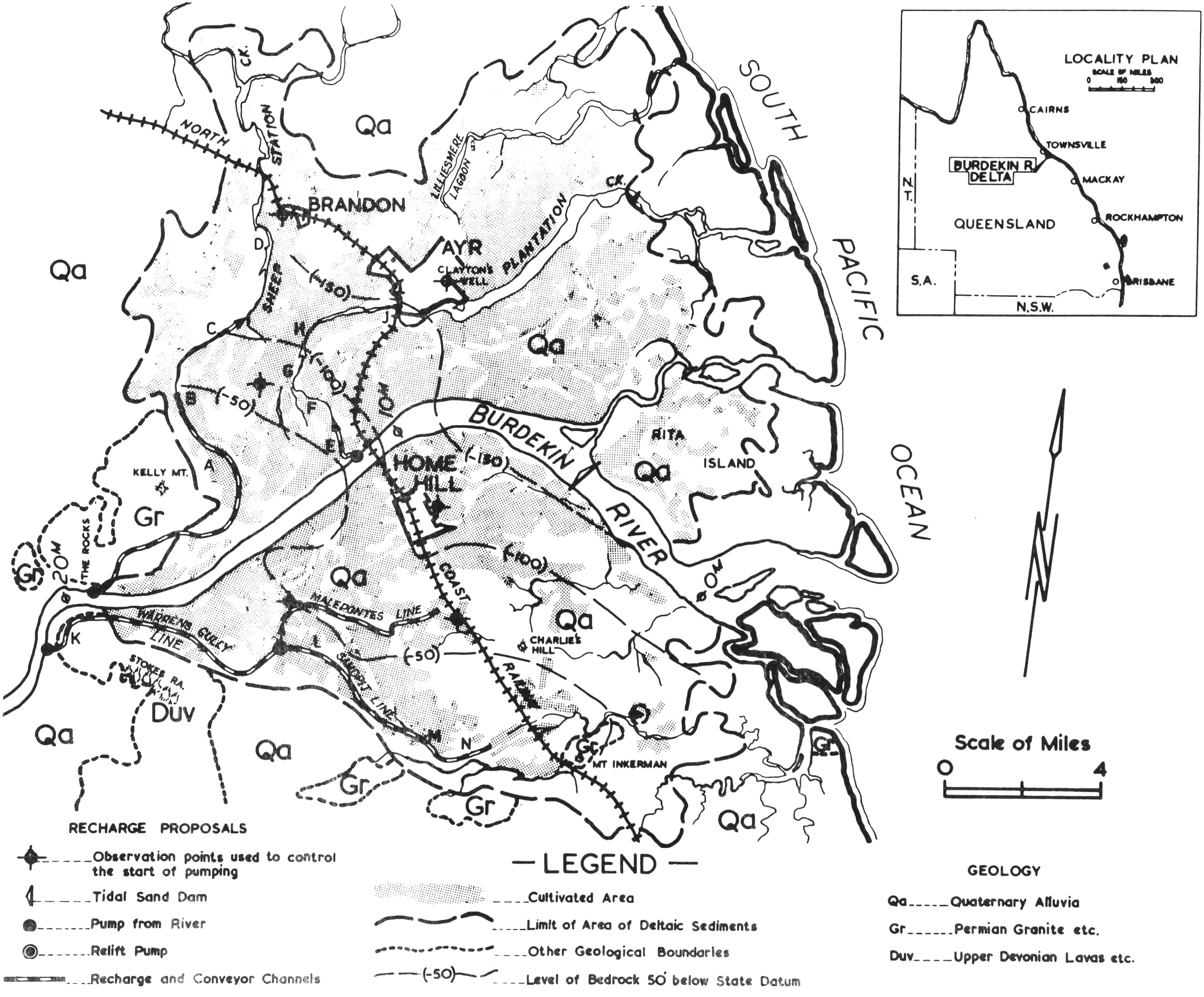


Fig. 1: The Burdekin Delta area.

Warrens Gully into Malepontes Relift Line - improved natural watercourse and artificial channel - relift pump station capacity 30 cusecs.

4. OPERATIONAL FEATURES

4.1 Administration

Control of the scheme is vested in two boards which were formed in May 1965, (North side) and March 1966, (South side). Some delay was experienced through failure of interested parties to reach agreement on method of financing the scheme.

4.2 Construction

North side construction began July 1965. The pump station for Plantation Creek began operation on 12.2.1966 and for Sheepstation Creek on 2.11.1966. South side construction began in September, 1966. The pump station at Warrens Gully began operation on 8.2.1967 and the relift pump for Maleponte's Lagoon on 16.6.1967.

4.3 Water Entitlement and Availability

The boards are entitled to divert portion of the unregulated flow in the Burdekin River and the maximum quantities were stated in the respective Orders in Council which established the boards.

Maximum Annual Quantity (a) N.B.W.B. = 61,000 ac.ft. (increased to 68,000 ac.ft. in April 69).

(b) S.B.W.B. = 42,000 ac.ft.

Pumping is carried out, where necessary, when the flow in the Burdekin River is less than 30,000 cusecs. With a pumping capacity of 150 cusecs the annual average quantity of water available is 64,000 ac.ft. while the maximum available (1921 to 1963) is 98,000 acre feet.

4.4 Artificial Replenishment to Date

Table III below shows quantities of water pumped for recharge to date.

TABLE III

ACRE FEET PUMPED FROM RIVER

(Figures rounded to nearest hundred acre feet)

PERIOD	NORTH			SOUTH	TOTALS
	Plant C.	Sheep C.	Sub-Total	Warren's G.	
1965 July-Aug		1,100	1,100	Nil	1,100
1965/66 Dec.-May	6,700	4,000	10,700	Nil	10,700
1966/67 Sept-Aug	27,700	16,600	44,300	14,000	58,300
1967/68 Nov.-Sept	9,800	12,800	22,600	14,500	37,100
1969 Jan.-Apr	11,700	6,300	18,000	12,300	30,300
TOTALS			96,700	40,800	137,500

4.5 Performance of Recharge Channels

Losses measured in the various recharge channels during pumping operations are given in Tables IV, V and VI. Experience has shown that maintenance of gauging sections is difficult and a number of sections have had to be abandoned because of the following changes in conditions -

(a) Increasing depth of water;

(b) Installation of drop boards;

(c) Growth of vegetation.

It is considered that suitable gauging facilities should be incorporated in the design of similar replenishment schemes for the purpose of recording performances. Inaccuracies in assessment of losses also occur because of private pumping from the recharge channels. Losses due to evaporation on the south side have been estimated at 1000 acre feet per annum assuming continuous operation.

Turbidity measurements taken on the recharge water in Sheepstation Creek and the Sandpit Diversion Line have recorded values of up to 3,000 p.p.m. Sheepstation Creek, a natural water course, tends to have an efficient settling action probably due to lagoons and grass growth. Low to medium turbidity water (100 - 1000 parts per million) appears acceptable for replenishment in this case. By contrast there is a very limited settling action in the Warrens Gully channel system and high turbidities in the recharge water have a marked deleterious effect on intake. The use of clean water is probably the most important single factor in maintaining a high intake rate.

4.5.1 North Side.

The quantity of water diverted in 1966/67 was above average and both Plantation Creek and Sheepstation Creek showed sufficient absorption capabilities. Unseasonable rain in June 1967, (8.32 inches at Ayr) and further heavy falls in early 1968 (January 19.32 inches and February 23.53 inches at Ayr) caused considerable natural replenishment and filled both creek channels. For the remainder of 1968 the replenishment capabilities of the creeks were lowered and pumping was resumed in April 1968, on a reduced scale. Pumping in 1969 has been limited by river flows to the period January to April. The creeks have shown a high absorption capacity and pumping has been almost continuous.

Additional proposed works for this area include the provision of two temporary 10 cusec pump units which can be moved from one locality to another and a relift station of some 30 cusec capacity on Plantation Creek in order to divert some of the replenishment water to Lilliesmere Lagoon.

TABLE IV (Refer to Fig. 1)

REPLENISHMENT IN SHEEPSTATION CREEK

Date	Section	Loss in Cusecs	Distance in Miles	Loss per Mile in Cusecs
12.1.1967	A to B	9	2	4.5
28.3.1967	A to B	3	2	1.5
18.8.1967	A to B	2	2	1
13.11.1967	A to B	6	2	3
1.12.1967	A to B	9	2	4.5
12.1.1968	A to B	5	2	2.5
	B to C	6	2	3
8.5.1968	A to B	3	2	1.5
	B to C	4	2	2
	C to D	15	3.5	4
5.7.1968	A to B	4	2	2
	B to C	1	2	0.5
	C to D	9	3.5	2.5
30.1.1969	A to B	7	2	3.5
	B to C	10	2	5

TABLE V (Refer to Fig. 1)

REPLENISHMENT IN PLANTATION CREEK

Date	Section	Loss in Cusecs	Distance in Miles	Loss per Mile in Cusecs
28.3.1966	F to H	40	2	20
13.4.1966	F to H	27	2	13
13.1.1967	E to J	44	7	6.5
10.4.1967	E to J	30	7	4
28.3.1966	G to H	24	1	24
30.1.1969	G to H	16	1	16

4.5.2 South Side

In order to obtain effective replenishment in this area it was considered necessary to improve sections of the former river and creek channels in the area by stripping the overlying and less permeable material from the creek beds and, where artificial channels were required, to construct sand filled trenches in the bottom of these along certain lengths of the channel. Recommendations to this effect were made in the design of the scheme.

However, the sand filled trenches were not incorporated in the construction of the scheme and because of the poor intake areas available, the south side channels have not been able to absorb 60 cusecs and the pumping station has been virtually restricted to one river pump since diversion began in February, 1967.

There is virtually no intake between the rising main outlet (K) and Malepontes relift pump station (L) - see Fig.1. The area between M and N proved a very favourable recharge area with an intake rate of 8 cusecs/mile increasing to 16 cusecs/mile with clear water and increased head. When recharge began considerable benefit accrued to farmers in this vicinity. Their underground supply had been critical and they faced severe losses. As a direct result of this replenishment, their crops were saved.

Malepontes diversion line has shown very low intake values. However, it is expected that the construction of sand filled trenches in the bed of the channels and the use of off-channel pits and lagoons in the area in the near future will allow full absorption of the designed 30 cusecs flow.

TABLE VI (Refer to Fig. 1)

REPLENISHMENT IN SANDPIT DIVERSION LINE

Date	Section	Loss in Cusecs	Distance in Miles	Loss per Mile in Cusecs
22.2.1967	K to L	2	8	0.25
	L to M	22	4.7	4.8
13.3.1967	K to L	1	8	0.15
27.6.1967	L to M	14	4.7	3
	M to N	6	1	6
29.11.1967	L to M	10	4.7	2
	M to N	4	1	4
8.12.1967	K to L	2	8	0.25
	L to M	23	4.7	5
	M to N	16	1	16
15.12.1967	K to L	1	8	0.15
	L to M	3	4.7	0.6
	M to N	4	1	4
5.1.1968	L to M	17	4.7	3.6
	M to N	3	1	3

4.6 Effect on Water Levels

Water level contours for the periods September 1966, August 1967, and August 1968, are shown on Figs. 2, 3 and 4. Annual rainfalls and 3 and 5 year moving averages are shown on Figure 5. It is considered that, in view of the below average rainfall obtained in the years 1964, 1965, 1966 and 1968 and the favourable water level situation occurring as at August 1968, the quantity of water pumped for artificial recharge over the period February 1966 to September 1968, (107,000 acre feet) has had a substantial effect in maintaining ground water levels throughout the area.

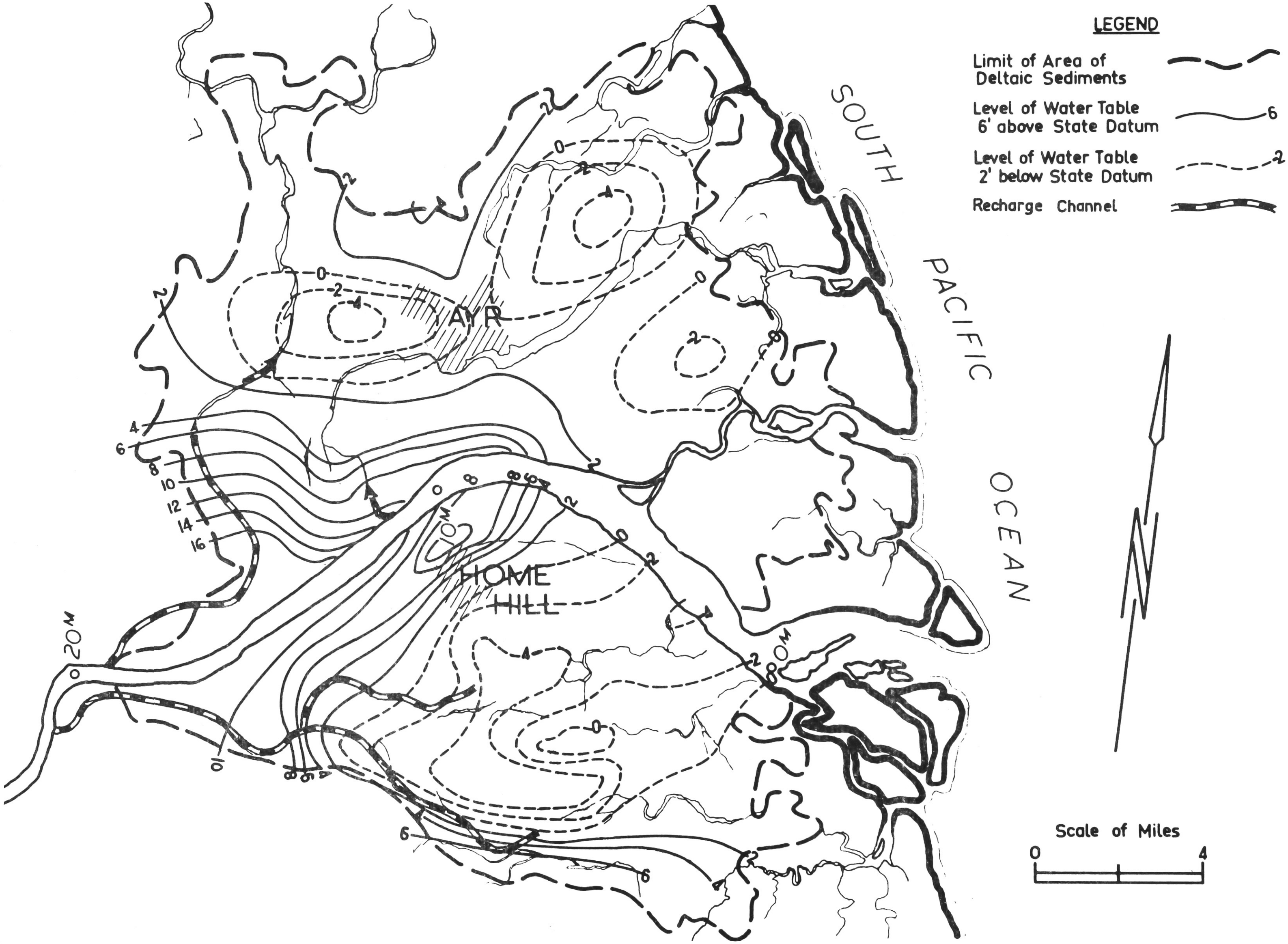
4.7 Effect on Water Quality

Measurements on water quality are taken at two monthly intervals from approximately 190 bores throughout the area. The effect of artificial recharge on water quality is not fully known at this stage. An analysis of the measurements obtained is complicated by variations in depths of sampling points and the fact that quality is influenced by both the strata composition and salt water intrusion. There is no evidence over recent years that either artificial or natural replenishment has improved water quality to any significant degree and a slow deterioration in quality appears to be taking place.

4.8 Operating Costs

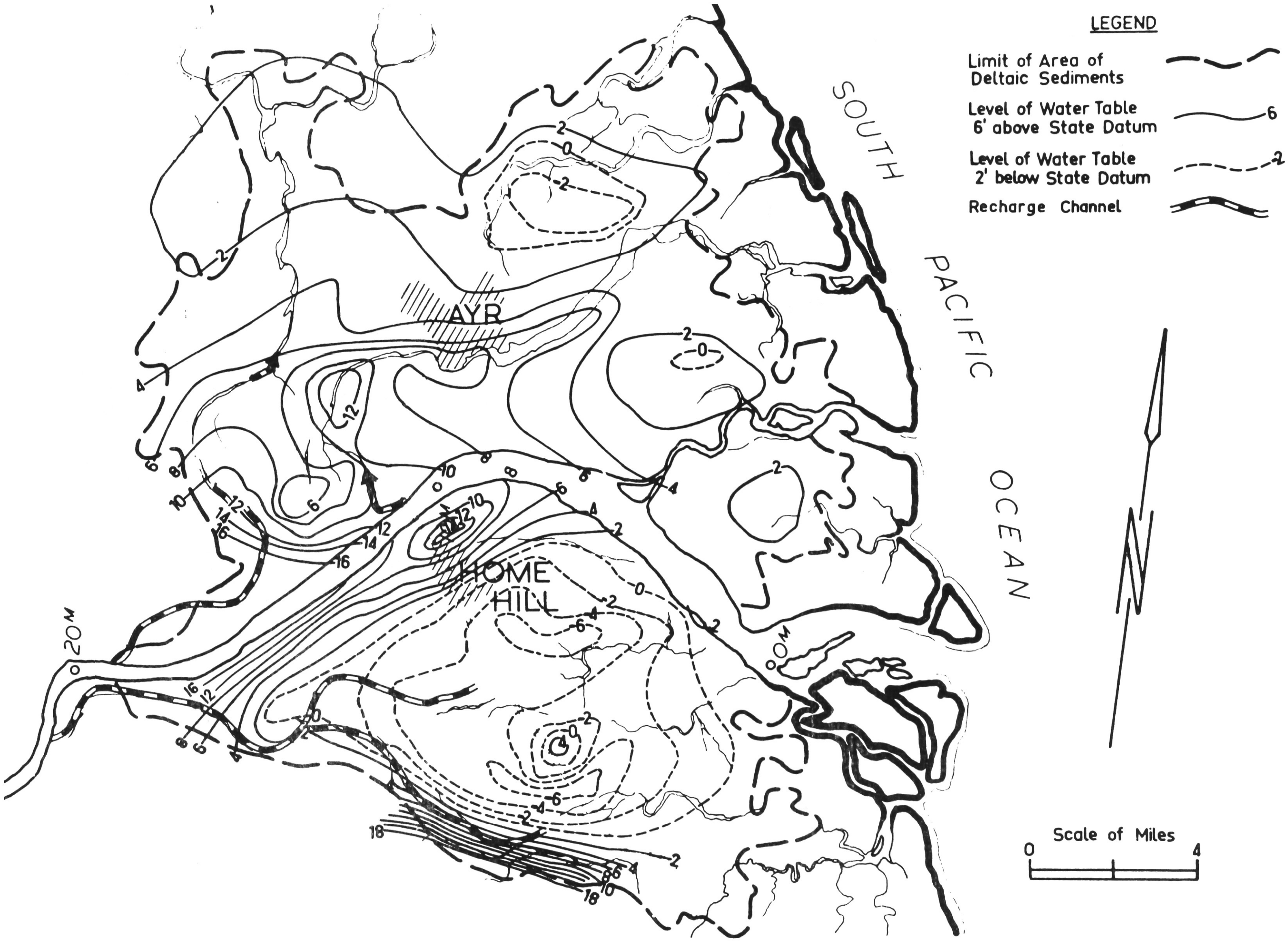
Total operating costs (including interest and redemption) for the financial years 1966/67 and 1967/68 together with quantities of water pumped are given below -

Year	NORTH SIDE			SOUTH SIDE		
	Total Acre Feet	Cost	Cost per Acre Foot	Total Acre Feet	Cost	Cost per Acre Foot
1966/67	37,889	$62,000	$1.64	11,315	$30,320	$2.68
1967/68	23,055	$82,000	$3.56	15,157	$68,000	$4.49



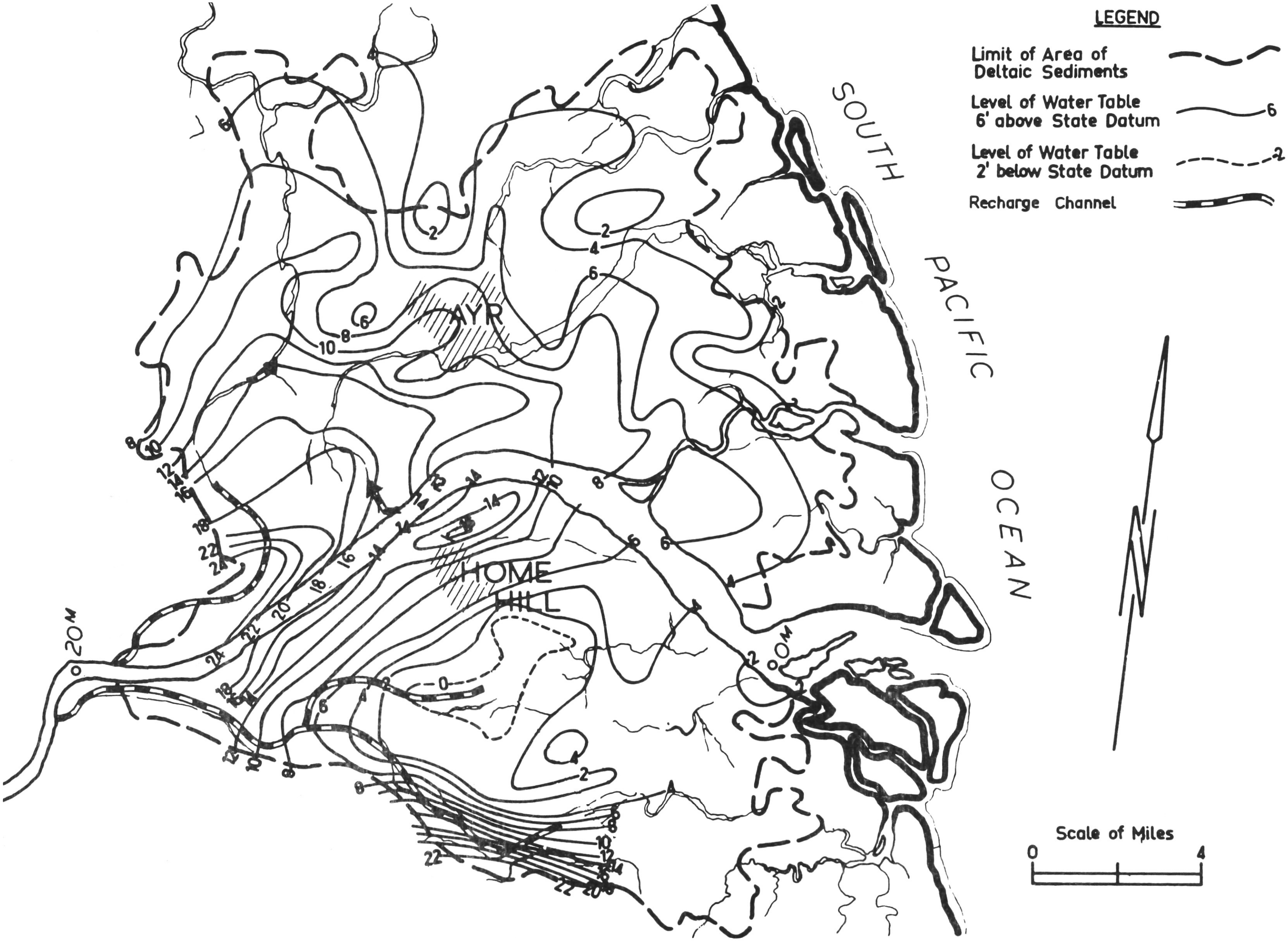
WATER LEVEL CONTOURS—SEPTEMBER 1966

Fig. 2: Burdekin Delta water level contours - September 1966.



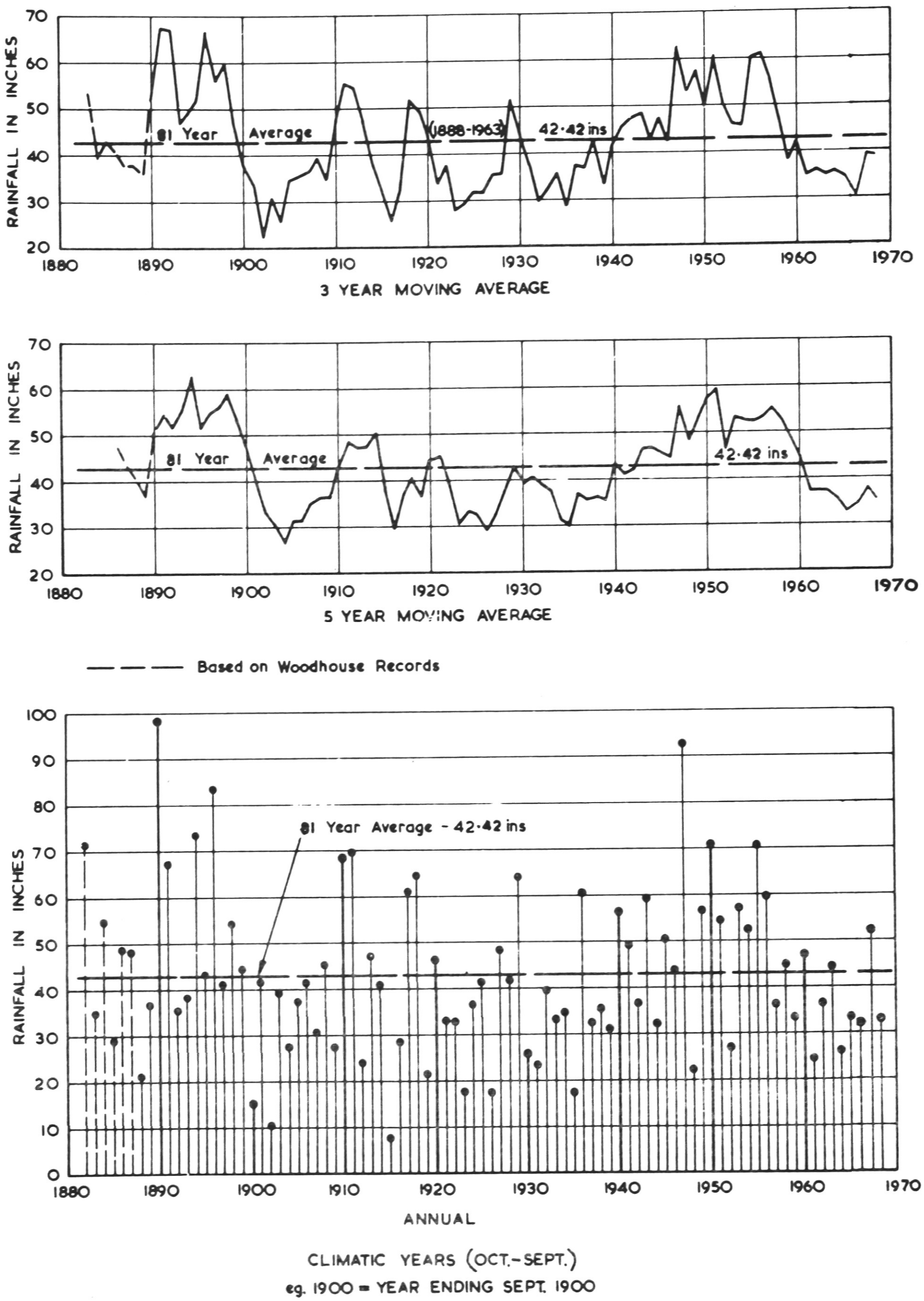
WATER LEVEL CONTOURS—AUGUST 1967

Fig. 3: Burdekin Delta water level contours - August 1967.



WATER LEVEL CONTOURS—AUGUST 1968

Fig. 4: Burdekin Delta water level contours - August 1968.



AYR RAINFALL

Fig. 5: Rainfall at Ayr.

PART B

GROUNDWATER INVESTIGATIONS - BUNDABERG AREA

1. ROLE OF GROUNDWATER SUPPLIES IN THE AREA

Within the main areas of underground water supply delineated on Fig.6, supplies in the range of 10,000 gallons per hour to 50,000 gallons per hour are generally available and have been developed for irrigation purposes, for the Bundaberg City Council and for the three sugar mills situated close to Bundaberg and for miscellaneous purposes.

2. ASSESSMENT OF ANNUAL DRAFT

Annual use of water for irrigation appears to be in the range of 1.5 to 2 feet of water per acre irrigated and the present irrigation demand is therefore in the range of 52,000 acre feet per annum to 70,000 acre feet per annum. This water is pumped from some 900 bores with a combined output of 900 cusecs, or 20 million gallons per hour. Other annual demands for water include the Bundaberg City which currently uses some 4,000 acre feet and Millaquin, Quanaba and Fairymead Sugar Mills (estimated combined use 2,000 acre feet). This represents a total quantity pumped from the groundwater reservoir in the range of 58,000 acre feet to 76,000 acre feet per year.

3. INVESTIGATION PROGRAMME

3.1 Scope of Investigations

The Irrigation and Water Supply Commission commenced investigations in 1953 with a view to determining the location, extent and safe supply of the groundwater resources. This investigation work as well as drought years in 1956/57, 1960/61 and 1964/65, and the opportunity for greatly increased sugar production resulted in rapid expansion of the use of irrigation from underground sources in the area to some 35,000 acres irrigated at present.

This rapid increase in irrigation areas and indications of substantial falls in water levels in 1960/61 and again subsequent to 1963 indicated the need to ascertain the safe available yield for the basin. The continual fall in water levels from early 1963 until the end of 1966 combined with evidence of salt water intrusion have indicated also the need to examine the possibilities of artificial recharge.

Since 1964, the investigations have been intensified and the programme has included -

(a) Drilling of 205 test bores to obtain strata information, depth to bedrock, available yield from bores and hydraulic properties of aquifers.

(b) The establishment of a network of some 240 observation bores in which water level measurements are taken to determine fluctuations in the water levels and from which water samples are taken to maintain records of water quality.

(c) The collection and assembly of data such as strata, yield and quality from some 2,252 Commission and private bores, which are most of those established to date in the area.

(d) Topographic surveys, both ground and aerial to provide information on the ground surface in the area and to relate all Commission and private bores from which information is available to a common datum.

(e) The construction of a geological model showing the data on bores, natural surface levels, water levels and bedrock levels to facilitate the study of the underground water system and its behaviour.

(f) Implementation of a large number of recharge trials throughout the area involving construction of two pumping stations on the Kolan and Elliott Rivers, two relift stations, eleven miles of asbestos cement pipeline and five miles of earth channels. The recharge experiments involved seventeen spreading trials with 150 observation bores, four absorption trials on creeks in the area and construction of seventeen injection bores.

(g) Additional investigations into the progress of natural recharge viz -

   (i) Drilling of shallow bores into the upper strata to detect any differences in standing water level between upper strata and the aquifer proper;

   (ii) Field permeability tests in both auger holes and excavated trenches;

   (iii) Laboratory permeability tests of undisturbed samples of the clay layers overlying the aquifers;

   (iv) An analysis of results of water quality samples with a view to detecting areas with ready access to the aquifer.

(h) Hydrological investigations to determine quantities of water available for recharge purposes and engineering investigations to determine the location, nature and cost of works required to convey water to possible areas of recharge.

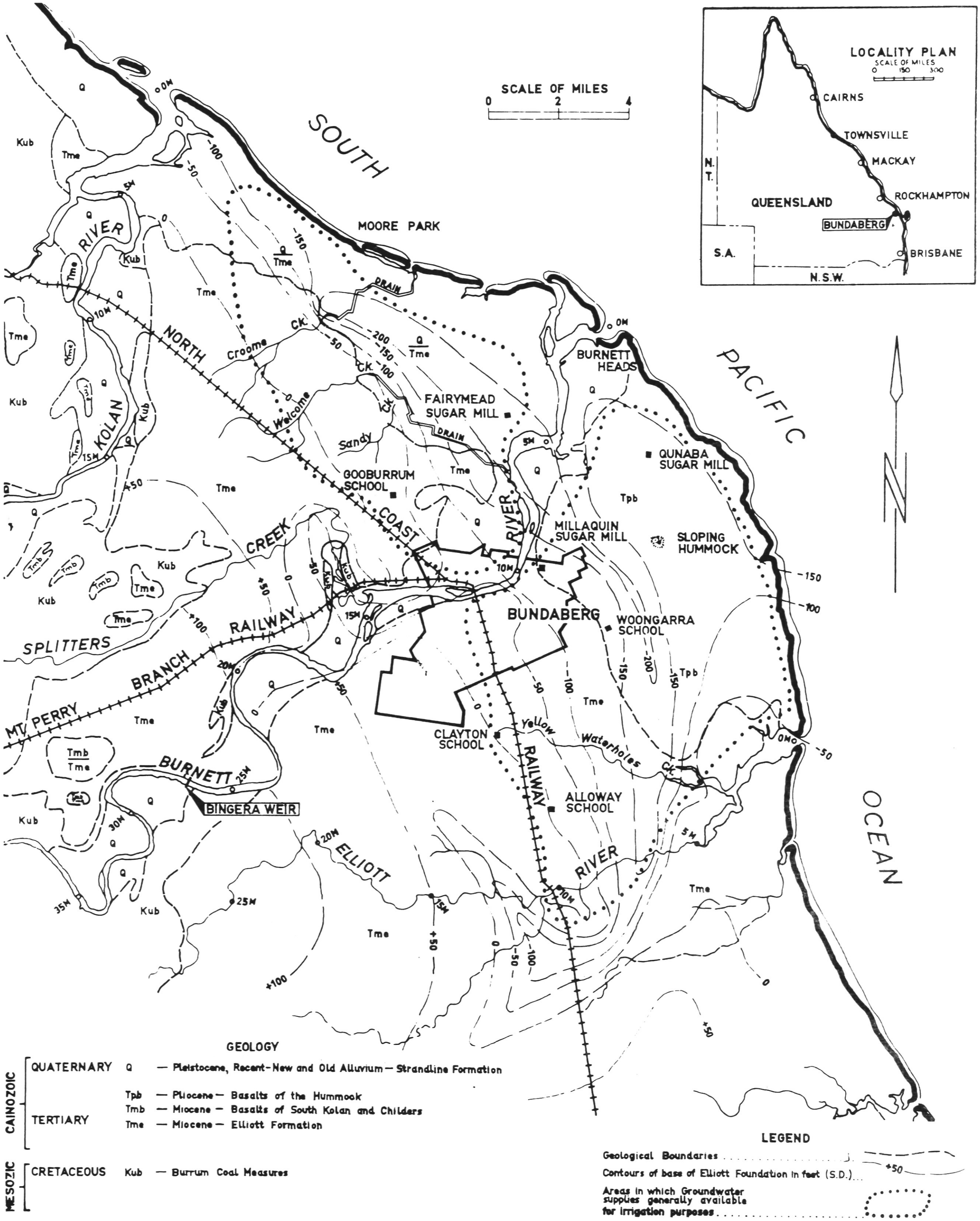


Fig. 6: The Bundaberg Area.

3.2 Assessment of Supplies

3.2.1 Nature of Aquifer System.

(a) Geological Properties - The groundwater supplies are obtained from unconsolidated and semi-consolidated horizons of sand, gravels and clayey sands and gravels which occur in the Elliott formation of Tertiary age. Geological boundaries are shown on Fig.6. Detailed investigations by the Irrigation and Water Supply Commission have shown that the main sand and gravel zones are associated with a sub-surface trough-like structure some 20 miles long by 8 miles wide which runs roughly parallel to the coast and which contains the greatest depth of Elliott formation.

In the western area, bedrock occurs at depths generally from 50 to 80 feet, and slopes to the east at about 10 feet to the mile. From the vicinity of the railway line to the east, the slope of the bedrock floor increases to as high as 100 feet per mile and depth to bedrock from surface exceeds 200 feet in places. Throughout most of the area, the aquifers appear to be overlain by relatively thick clay layers, 20 to 40 feet thick, the depth to top of these layers varying from surface level up to 40 feet. Where these layers are substantially below the surface the material above them varies through silts, loams and sandy loams to very sandy porous material in some areas.

(b) Hydraulic Properties - Values of transmissivity of the aquifer have been compiled from pump tests on some 70 bores both Commission and private throughout the area. Generally the values range from 3,200 ft^2/day up to 110,000 ft^2/day. However, these figures are based on successful production bores in the area and may not be indicative of the whole aquifer system. In addition the value of 110,000 $ft.^2$/day is probably too high and is caused by water leaking into the aquifer from the higher confining clay layers during pumping. There have only been a few cases where storage coefficient for the area have been calculated. These are of the order of 10^{-4} and are typical of a confined aquifer.

3.2.2 Water Quality.

Underground water quality in the area is observed by periodic sampling from some 410 bores. Surface water quality is also being observed at some 47 sampling points.

Analysis of these samples can be summarized as follows -

(a) The best quality water in the area exhibits a very low conductivity (generally less than 500 micromhos per cm. at 25°C) and is found at all depths throughout the Tertiary sediments, in the Clayton, Alloway and Thabeban areas south of the Burnett River and Meadowvale, Gooburrum and Welcome Creek areas to the north of the Burnett River.

(b) During the period 1953 to 1963, the quality of the water in the zones commonly tapped for irrigation purposes remained remarkably stable but since then significant increases in conductivity have been detected in bores up to five miles from the sources of salt water intrusion. The recent analyses have shown that extensive salt water intrusion is appearing particularly in the South Bundaberg area. A number of facilities have been abandoned and the water quality situation in this area is steadily deteriorating.

3.2.3 Water Level Behaviour.

(a) Natural Recharge - At present water level measurements are taken at two monthly intervals in some 240 bores throughout the district, and during periods of significant rainfall measurements are taken every couple of days in selected key bores in the area. From each set of water level measurements a contour map of groundwater levels for the district is produced. These contours show that the groundwater flow is directed towards the coast and also to the major drainage lines in the area. These are the Burnett, Elliott, and Kolan Rivers and Splitters and Yellow Waterholes Creeks. This situation, along with the fact that the Burnett and Kolan Rivers are tidal (and therefore normally salty) throughout the area of groundwater supplies shows that these two streams do not contribute freshwater to the underground beds under normal flow conditions. Furthermore contours of the base of the Elliott formation show that it is only in the lower fifteen miles of the Burnett and five miles of the Kolan where its base is below sea level. Under flood conditions when water levels in these rivers might be perhaps 10 feet to 20 feet above sea level, the contribution if any, must be small because of the short length of river where the water level would be higher than that in the adjoining aquifer and the brief period for which these conditions occur. Therefore there is no doubt that local rainfall is the source of natural recharge for the area.

(b) Correlating Water Levels with Rainfall - An examination has been conducted of rainfall data and groundwater levels for 16 selected bores in the area for the period 1957-1966, in order to find a rainfall recharge relationship. The rainfall used was the average for Bundaberg and Fairymead. The analysis showed that rises in water levels do not generally occur unless rainfall is received in a concentration of approximately 6 inches or more per month.

3.2.4 Need for Artificial Recharge.

When examining rainfall records over a long term period and comparing recent yers with those of the past, it is desirable to isolate and consider only that rainfall which is contributing to recharge and not the total rainfall for the year. Using this information, the "recharge" rainfall per year can be calculated and a 2, 3 and 5 year moving average plotted.

From an examination of "recharge" rainfall in conjunction with water level behaviour, it became apparent that a serious overdraft situation had developed in the area. These findings were confirmed from studies of water quality variation, where it was noticed that extensive salt water intrusion was occurring along the coast - particularly in the South Bundaberg area. From a consideration of water level and quality behaviour, "recharge" rainfall and annual draft, a safe yield of 20,000 acre feet per annum was adopted for the aquifer on each side of the river.

3.3 Recharge Trials

The following factors were considered in selecting the type of trial to be conducted -

(a) Availability of land and water supply;

(b) The surface infiltration rate together with infiltration rate of strata from below surface to the aquifer;

(c) The standing water level in the area in relation to the level in areas which it was desired to benefit;

(d) The ability of the aquifer to accept the rate of infiltration and to transmit it under the gradient established.

A summary of the results obtained from the recharge trials is given in Figure 7.

4. CONCLUSIONS

4.1 Feasibility of Artificial Recharge

4.1.1 Spreading Methods

Results of the 17 spreading trials conducted on both the North and South sides of the Burnett indicate that an overall infiltration rate of approximately ½" per day could be expected from spreading trials.

On the North side an annual requirement of 16,000 acre feet could be supplied at a rate of 30 cusecs for 267 days of the year. Even allowing for losses on a long term basis of some 5 cusecs in Sandy Creek and a loss/storage capacity of 5 cusecs in the Waimea Area, the total area required to absorb the remaining 20 cusecs at a rate of ½" per day is 1,000 acres. The operational area of such a scheme would probably need to be double this to allow spelling in alternate bays. Cost of resumption of land for this purpose together with an evaporation loss of some 5,000 acre feet per year would make this scheme economically unfeasible.

The annual requirements for the south side are 25,000 acre feet per year and for the same reasons as given previously for the north side, recharge by spreading is not economically feasible in this area.

4.1.2 Injection Methods

The trials indicated that there would be serious long term difficulties associated with the operation of such injection wells and there is very little evidence of them having been used anywhere in the world without considerable operation and maintenance costs. The most important factor is the need for a high quality recharge water and this would involve construction of a filtration plant and treatment works for any water supply available for recharge.

A preliminary estimate for a recharge scheme for the South side using injection bores and replenishing at the rate of 25,000 acre feet per annum involved a capital cost of $3,000,000 and an annual cost of $576,000.

Therefore it appears that artificial recharge of the groundwater resources of the area by injection methods as a means of supplementing present supplies is not economically attractive at this stage.

4.2 Remedial Measures

As the investigations have shown that artificial recharge of the groundwater supplies is not feasible it is considered that one or both of the following measures must be implemented in order to prevent ultimate failure of the groundwater supply.

(a) Restrictions on withdrawals from the groundwater reservoir;

(b) Provision of an alternative source of water to relieve the overdraft on groundwater resources.

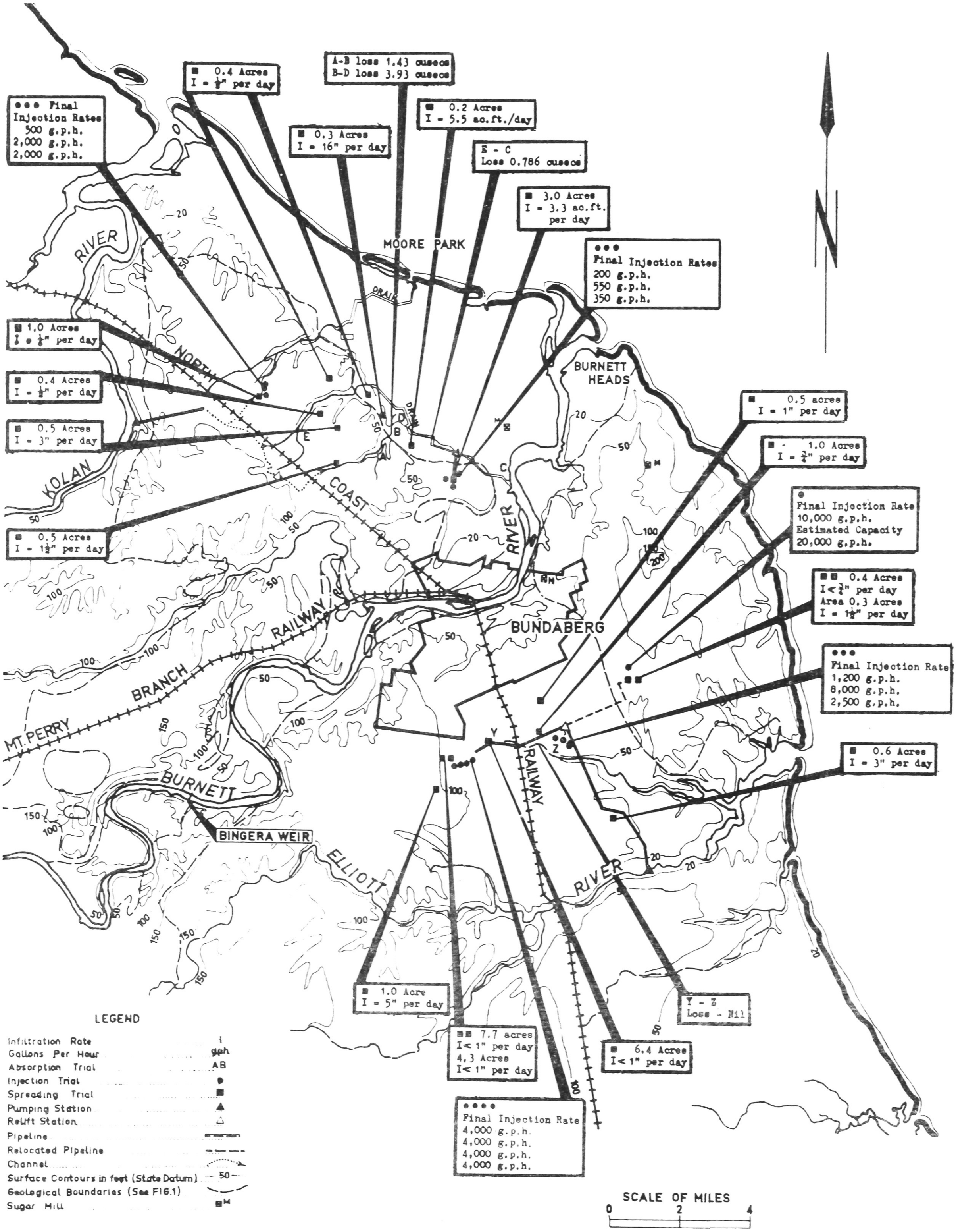


Fig. 7: Summary of results of recharge trials in the Bundaberg area.

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PAPER No.6

WASTEWATER RECLAMATION USING GROUNDWATER RECHARGE

A REVIEW

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SUMMARY

In some places where local fresh water supplies are limited, and water can be brought to the area only at considerable cost, wastewater is being reclaimed for reuse. The use of wastewater to recharge aquifers is a reclamation method which has a number of technical and aesthetic advantages, and has been the subject of laboratory, field, and full-scale investigations in America, Israel, and Holland.

When wastewater percolates through soil, natural physical, chemical, and microbiological processes not only can cause large reductions in the amount of putrescible organic material, and in the number of bacteria, but also can remove suspended material, and reduce the amount of nutrients which could cause eutrophic conditions.

For injection wells, water should be treated to a fairly high degree of organic purity; but seepage basins operated on a suitable intermittent schedule under good conditions can produce potable groundwater even from sewage which has had only primary treatment.

This paper reviews some of the research work and reports about wastewater reclamation using aquifer recharge.

1. INTRODUCTION

The aim of this paper is to provide a general review of wastewater reclamation using groundwater recharge. The length of the paper does not permit detailed review, but rather a general indication of the principles involved. There are a number of apparent anomalies in the literature on the subject, and much is yet to be learned before definite design procedures may be set out. The reader who desires more detailed information about research and development in this field in regard to any particular topic is referred to the list of references cited at the end of the paper.

2. RECLAMATION OF WASTEWATERS

When engineers begin to plan water supply or irrigation schemes, they normally seek first a source of water which is of good quality, requiring the minimum amount of treatment.

But in populous areas where the amount of water needed is constantly increasing, and the amount of fresh, or unused, water available at reasonable cost is limited, it becomes necessary to look towards reclaiming and re-using of wastewaters rather than discarding them.

Whetstone in his report to the Texas Water Development Board said that it is evident that "re-use of water through many cycles will be routine practice" in fifty years, and that the luxury of discarding once-used water will become but "a bitter memory of ancestral squandering". The two major forces which he claims will be responsible for this development are water economics, and improvements in sewage treatment methods (Whetstone, 1965).

In areas of intensive urbanization, the demand for water increases as the population of the area grows. The rate of increase in the demand for water is greater than the population increase because of a number of factors including:-

(a) technological developments which encourage people to use more water than they need (e.g. automatic washing machines, rotary lawn mowers).

(b) formation of wasteful habits in the use of a relatively cheap commodity.

(c) increasing industrialization.

Where growing urban complexes use water only once, they are generally faced, as time goes by, with the necessity of exploiting water resources situated further and further from the centre of use.

Not only does this result in increasing costs of water collection and transmission, but sometimes results in political wrangles between different authorities who may lay claim to the same water resource. For example, if the Melbourne and Metropolitan Board of Works were to take water from some catchments north of the Dividing Range, it would reduce the amount of water available to established irrigation systems serving drier parts of the state, creating a difficult political situation.

In some localities there is no longer enough natural surface and ground waters which can be collected at a reasonable cost. As a result, more attention is being paid to the reclamation of wastewater. In some places, such as at Windhoek in South Africa, the wastewater is reclaimed by extensive artificial treatment and pumped directly into the water supply mains, but in most other places the reclamation programme includes percolation through natural soil, to recharge an aquifer with water which may be recovered for re-use.

Where water has been recently used to carry liquid and solid domestic wastes, many people are naturally reluctant to drink it, or wash in it - regardless of any subsequent treatment it may have received.

Reclaimed wastewater which has been recovered from recharged aquifers has not only been subjected to some natural purification in seeping through the soil, but has also had an opportunity for dilution with natural groundwaters, and has psychologically lost its identity as sewage. Therefore, the public seem to be more ready to accept it than they would be to accept 'artificially' treated wastes which retain their identity during treatment and re-introduction into the supply mains, as at Windhoek.

Santee in California provides an example of public acceptance of reclaimed sewage for fishing, boating, and bathing. The highly treated effluent is percolated through infiltration beds and recovered for use in recreational lakes and a swimming pool (Merrell et al., 1967).

The economics of reclamation of wastes has been studied. A 1951 report from the University of California (California State Water Poll.Cont.Bd., 1951) stated that planned reclamation works in water deficient areas were competitive in cost with other methods of obtaining additional water. When estimates were prepared for reclamation works on Long Island they showed that the expected additional costs for reclaiming effluent, after it has undergone secondary treatment, are less than those for developing the most favourable alternative supply (Peters et al., 1968).

Even small towns have reclaimed wastewater by sewage treatment and groundwater recharge. In 1962 the township of Rancho

Santa Fe, California, with a population of only 530, installed an extended aeration sewage treatment plant. The effluent was spread in a dry river bed to recharge an aquifer which was used for agricultural and domestic well supplies (Oldsen et al., 1962).

3. AQUIFER RECHARGE METHODS

The two main methods of adding recharge water to an aquifer are:-

(a) direct injection through wells into the aquifer,

and (b) percolation to the aquifer from surface spreading basins.

The test injection well on Long Island is 500 ft. deep, and was drilled 36 inches in diameter (Baffa et al., 1967; Peters et al., 1968). A 60 ft. long x 16 inch diameter stainless steel screen was placed near the bottom of the well, and the annular space around it was filled with graded sand. The top of the screen was fastened to an 18 inch diameter fibreglass pipe which extended to the surface, the annular space around it being filled with cement grout. With such arrangements the water can be injected directly into the aquifer below the water table.

In surface spreading operations wastewater is admitted to shallow ponds or spreading basins from which it percolates through the soil to join the groundwater. There is usually an unsaturated zone between the bottom of the spreading basin and the groundwater table. This type of recharge is in use at Whittier Narrows (Parkhurst et al., 1964), Holland (Baars, 1957, 1964), and other places.

Another method of aquifer recharge by surface spreading, is the over-irrigation of crops with waste waters, so that the excess water becomes available for replenishment of groundwater after it has percolated through the soil.

4. DEVELOPMENT OF RECLAMATION PRACTICE USING AQUIFER RECHARGE

Reclamation of wastewater by percolation through the ground is not new. Incidental, or unplanned, recharge of groundwater has occurred from septic tank seepage pits, sewage farms, and the like for many years. In some cases effects have been detrimental because the groundwater table has been raised too much.

In Europe, where there are substantial concentrations of population along rivers like the Rhine, the rivers are used as both wastewater disposal channels and sources of raw water for water supply schemes, for city after city. However, Public Health authorities in Holland, Germany, and Sweden, require that even where river water is to be treated and pumped into a supply it must first be percolated through the soil to ensure adequate natural purification (Griffiths, 1964).

In some places the river water is reclaimed by construction of either wells or infiltration galleries adjacent to the river, so that the river water can be used directly for recharging the aquifers when the groundwater level is lowered by pumping (Taylor, 1964). At some locations (e.g. Dortmund) the natural infiltration from the river is supplemented by engineered infiltration basins.

In Holland the polluted waters from the Rhine are piped up to 30 miles to spreading basins on the sand dunes, and after percolating through the dune sand into "dune storage" are used to supply Amsterdam, The Hagüe, and Leydon (Baars, 1957, 1964; Taylor, 1964).

In South Staffordshire (U.K.), an average of two to three million gallons a day of sewage effluent have been infiltrated for many years into the Bunter sandstone. Public Supplies of water of the 'highest purity' are obtained from wells less than one mile away in the direction of groundwater flow (Buchan, 1964).

In Los Angeles, California, during the early 1930's, the authorities began a planned operation for spreading highly treated effluent on soil to increase the groundwater supplies (Goudey, 1931). In 1954, Bush and Mulford listed 112 places where sewage effluent was used for planned aquifer recharge in the State of California (Bush et al., 1954). A plant, which can process 12-14 million U.S. galls of sewage a day to produce an effluent for aquifer recharge through spreading basins, has been operated at Whittier Narrows, California, since 1962 (Anon, 1964; Parkhurst et al., 1964).

Aquifer recharge has been used for more than one object. Firstly, at Whittier Narrows and in Israel the object is to supplement an existing supply. Secondly, recharge water may be used to form a hydraulic barrier against intrusion of saline water. Thirdly, it may be used to create a supply in an aquifer where the natural groundwater is too saline for agricultural or domestic use - as proposed at Flushing Meadows, near Phoenix, Arizona.

The Flushing Meadows project is particularly interesting because it is proposed to perch a zone of reclaimed sewage within an aquifer containing naturally saline groundwater. By controlling the relative rates of recharge and pumping, it is expected that it will be possible to prevent both excessive spread, or outflow, of reclaimed water, and the intrusion of saline water into the recovery wells.

Brooklyn, New York, is situated on the western end of Long Island. The New York metropolis is spreading over Long Island to such an extent that the population of Nassau County, near the middle of the island, doubled in the period 1950 to 1965, to an estimated 1,450,000. Nassau County relies on groundwater as its principal source of supply. Until recent years, much of the water pumped from the ground was returned to it by seepage from septic tanks and sullage pits; but now the denser housing development has warranted the installation of sewerage systems which discharge the effluents into the sea.

As a result of the increasing use of water from the aquifer, because of increasing population to be served, coupled with decreasing recharge, the groundwater levels are dropping, and the rate of fall in the levels will accelerate as development occurs - unless some action is taken to avoid it.

The possible courses of action are:

a. to increase the rate of recharge of the aquifer
b. to decrease the rate of removal of water from the aquifer.

The removal rate could be reduced by obtaining at least some of the water supply for the county from some other sources, or by reducing the losses from the aquifer.

Other possible sources of supply investigated included:

a. bulk supply from New York City.
b. development of a mainland catchment beyond the New York catchments, and piping the water over a long distance.
c. desalting of seawater.
d. reclamation of wastewater.

The plan which appeared to be the most acceptable, after investigation, was intensive artificial treatment to reclaim the wastewater which would then be used for direct recharge of the aquifer. This recharge is planned in such a way that the water will not only augment the supply, but also form a barrier against salt water intrusion, permitting higher pumping rates.

A pilot plant, for advanced treatment of 576,000 U.S. galls. a day, has been constructed so that the feasibility of recharge may be studied (Baffa et al., 1967; Peters et al., 1968; Stevens et al., 1966).

Israel is a semi-arid country where particular attention has been paid to the conservation and re-use of water. Experiments have been conducted to establish design parameters for recharge basins to accept the wastewater from Tel Aviv which is expected to have a population of over one million by 1990. It is proposed that water from the recharged aquifers be pumped out into a pipeline leading to the south and Negev of Israel, for general use in those areas (Amramy, 1965, 1964; Nevo et al., 1967; Shuval, 1962).

5. WASTEWATER QUALITY DEFICIENCIES

Effluents from conventional sewage treatment plants are not suitable for direct re-use in domestic or industrial supplies (with a few exceptions) because of:-

(a) Oxygen Consuming Material

Most wastes contain an appreciable concentration of oxygen consuming materials in solution or in suspension. This material usually consists of organic compounds which can be broken down into simple compounds by bacteria and other micro-organisms (i.e. it can be biodegraded). If oxygen be present the bacteria use it in the biodegradation process and produce compounds such as carbon dioxide, nitrates, sulphates. If the oxygen supply be exhausted, however, the 'reducing" environment results in putrefaction, with products like methane, ammonia, and foul-smelling compounds such as methylamine (stale fish), sulphides (rotten eggs), mercaptans (stale cauliflower, skunk), and the like.

The potential for a waste to use up available oxygen, is usually described quantitatively by its Biochemical Oxygen Demand (B.O.D.), or by its Chemical Oxygen Demand (C.O.D.).

The Biochemical Oxygen Demand during 5 days at 20°C. (i.e. B.O.D.$_5^{20}$) of effluents from conventional sewage treatment works is of the order of 10 to 40 milligrams per liter (1 mg/l$\approx$1 part per million). Because this demand corresponds to a total oxygen requirement of perhaps 20 to 60 mg/l, it must be greatly reduced before the water could be used as a domestic supply without becoming putrescent during prolonged storage and transmission.

(b) Presence of Nutrients

Nitrogen and phosphorus are two of the important essential elements required as nutrients for the growth of cells. Nitrogen is a component of the amino acids which are necessary for protein formation; and phosphorus compounds are essential for the enzyme-controlled energy transformations which take place within cells and maintain life processes by converting the chemical energy contained in food into other forms of chemical energy, or to mechanical, electrical, or heat energy. If the wastewaters are used immediately for irrigation of some forms of crop or pasture, the nutrient content normally is of advantage. If, however, the wastewater is to be stored, or transported by pipes or channels, the presence of nutrients in significant concentrations may result in nuisance growths of algae and water weeds. These may be undesirable because of interference with the flow of water in channels or pipes, or because of undesirable tastes and odours, or even because of toxicity.

Domestic water supplies usually contain some dissolved oxygen, and under this condition most of the nitrogen present would be in the form of nitrates. Water containing too much nitrogen as nitrate can cause severe illness and death in young babies whose stomach juices are not sufficiently acid to inhibit the bacteria which convert nitrates to nitrites; the nitrites enter the bloodstream and, like carbon monoxide, combine with the haemoglobin causing cyanosis. Infant cyanosis has been reported to have been caused by a water containing 116 mg./l. of nitrate. In a number of places, water with up to 80 mg./l. is used without any apparent ill effects. The usually accepted limit of nitrate for potable waters is about 45 mg./l. of nitrate (10 mg./l. of nitrate nitrogen).

(c) Possible Presence of Pathogens

(i) Danger of Pollution

Organisms which cause disease are called pathogens. Although sewage treatment normally removes all but a few percent of the bacteria originally in the sewage, this small residuum still amounts to large numbers of bacteria -- one percent of one million is 10,000, and some sewage may contain a very large number of pathogens in each litre.

Pathogenic species include some types of bacteria, protozoa, fungi, helminths, and viruses. Because helminths and protozoa are more reliably removed by filtration, the greatest health hazards in treated wastewaters are likely to be caused by bacteria and viruses.

Large numbers of pathogens are passed - often only intermittently - in the faeces of persons who are infected by, or who are 'carriers' of certain diseases transmitted by the faecal-oral route. Some of these pathogens are capable of being carried in water and may sometimes be transmitted eventually to food or to drinking water, whence they may infect some other person. This is the commonest route for spreading such diseases.

(ii) Indicator of pollution

Coliform bacteria are relatively harmless organisms which normally inhabit the bowels of man and other warm-blooded animals, and are used as indicators of faecal pollution. Certain strains of coliform bacteria are always passed in great numbers in the faeces of man and warm-blooded animals, and their presence in water indicates the possible presence of faecal pollution - the probability that the water may contain pathogens of faecal origin. The concentration of coliform bacteria in a water is therefore regarded to be an indication of the magnitude of the probable health hazard associated with drinking the water. Coliforms are generally more resistant to most treatment processes and other unfavourable environments than pathogenic bacteria of enteric origin - e.g. typhoid; absence of coliforms generally indicates a water safe from such contamination. Viruses, however, are not so readily inactivated, and may persist in water although coliforms have been eliminated. Hence reclaimed water which may be bacterially safe cannot necessarily be assumed to be free from harmful viruses.

(d) Toxic Materials

Some wastewaters, especially those containing certain trade wastes, may contain materials which are injurious because of their toxic action (e.g. cyanide, lead, arsenic, boron, selenium, zinc). The permissible concentration of these toxic materials depends on the proposed use of the water.

(e) Dissolved Salts

Conventional water and wastewater treatment does not remove ions of sodium, chloride, sulphate, nor potassium. Some other ions may be partially removed under suitable conditions. The progressive build-up of salinity can limit the number of times a water may be re-used in the water-sewage-water cycle. Increments of common ions during one cycle of use have been published (Calif. State Water Poll. Cont. Bd., 1954) and the following list gives the cited 'normal' ranges in parts per million:

Nitrate	0 - 18
Phosphate	20 - 40
Sodium	40 - 70
Chloride	20 - 50

The 'normal' range quoted for increment of total dissolved salts was 100 - 300 parts per million. Up to ten cycles of water-sewage-water usage could possibly be obtained under favourable conditions, without having to resort to desalting.

One feature of the Californian report was the big difference between the increments as noted at different places. Local data is essential before any reliable assessment can be made about the limitations of water re-use possibilities.

(f) Detergents

Detergents in reclaimed water can cause trouble because of both foaming and physiological effects. Some detergents are difficult to biodegrade, and are known as 'hard' detergents. In some countries where trouble has been experienced, hard detergents have been banned in favour of 'soft' detergents, which are more readily biodegraded.

6. TREATMENT OF WASTEWATER BY SEEPAGE

Many of these unfavourable qualities of wastewater may be removed or reduced by percolation through soil. Investigations of percolate from soil have been made in laboratories, in pilot plants, and in full-scale installations, to study the effect of percolation on quality.

Apart from the head losses obtained when clean water percolates through soil, polluted water induces other effects such as the filtering out of particulate matter which is held in the soil pores, the biochemical degradation of organic matter, and chemical reactions between the percolating water and the soil, or natural groundwater.

The food contained in treated sewage is enough to support a strong growth of micro-organisms in the soil. The biological activity results in a layer of microbiological aggregates somewhat similar to the Schmutzdecke of a slow sand filter. The time taken for the establishment of this layer, its effectiveness, and its location, all vary widely depending on the soil, the waste, the temperature, and the amount of oxygen and nitrate present.

The activity of this layer:

hastens the degradation of organic material,

assists in the destruction of bacteria of enteric origin,

increases filtration efficiency,

decreases the rate of infiltration.

The mechanism for the formation of this layer includes physical, chemical, and biological processes which retard the transfer of pollution through the soil. It has been referred to as a 'defense' mechanism - a principle of fundamental importance in control of ground water pollution (Caldwell and Parr, 1937).

Stabilization of Oxygen Consuming Wastes

There are two distinct types of condition under which pollution may be present in the soil:-

(i) Percolating through an unsaturated zone above the ground water table;

and (ii) Moving with the groundwater below the ground water table.

Recharge water from spreading basins or water from excess irrigation usually spends some time in each of the two conditions; but water which is directly injected into an aquifer usually has no chance to seep through an unsaturated zone because it is usually injected below the groundwater table. Under these conditions, the transformation of organic matter is very slow, or even negligible.

The bacteria adsorbed to the soil grains in the 'defense' layers can "mineralise" organic matter, provided there is sufficient oxygen present as either free oxygen or nitrate oxygen. If the water is excessively polluted, and all the free oxygen and nitrate oxygen is used, anaerobic conditions will occur, bringing mineralisation of organic matter to a virtual standstill. Once the organic compounds enter the saturated groundwater zone they may remain in it for a very long time without much change.

Reports from Israel (Amramy, 1965) indicate the removal of about 75 percent of the BOD from the effluent of oxidation lagoons when it has percolated through sand over a distance of about eight metres, and of 90 percent in 30 metres. With intermittent operation, only the top 10 cm of sand in the spreading basins received an increment of organic matter, and there was no indication of a progressive accumulation of organic matter with time.

At Flushing Meadows, Arizona, effluent from a sewage treatment works in Phoenix has been experimentally infiltrated in field tests. The applied effluent had an average BOD of about 30 ppm. After percolating through about 30 feet of sand into a well, the percolate had a BOD of only 0.2 ppm. (Bouwer, 1968).

Near Lodi, California, Greenberg et al. made some tests in which effluents from both activated sludge treatment, and primary sedimentation only, were applied to experimental infiltration areas consisting of in-situ Hanford fine sandy loam. The activated sludge plant effluent had a BOD of about 10 ppm, and the settled sewage had an average BOD of about 100 ppm. The infiltration rates were in the range of 0.15 to 0.5 feet per day. Size analysis of the loam showed an effective size of 0.002 to 0.006 mm. and a uniformity coefficient of from 50 to 116.

Although some of the applied wastewater had a BOD of 100 ppm, none of the samples of percolate collected from a depth of one foot during a period of about 18 months had a BOD of more than 5 ppm (Greenberg & McGauhey, 1955).

In subsequent research, Butler et al. used soil from the same site, along with four other types of soil. For this work they placed each soil type in several lysimeters with a three feet depth of percolation to the sampling trays (Butler et al., 1954). They observed that in all samples most of the suspended organic matter was collected in the top half centimetre of the soil, and that prolonged spreading did not increase the depth of penetration. However, they also observed that when waste with a BOD of about 170 ppm was applied the BOD removal did not confirm the indications of the Lodi tests.

The general pattern of BOD removal was similar in all five types of soil, used in Butler's experiments regardless of grain size. At the end of one day the BOD of the percolate was about 5 ppm., the value increased slowly to about 100 ppm at the end of four weeks, and then started to decline until it stabilised at about 20 ppm after 25 weeks. These results indicated that under some circumstances it is possible for a considerable amount of organic pollution to percolate into the groundwater; and that there can be large discrepancies between field studies and lysimeter studies.

The initial high BOD reduction was attributed to adsorption of organic matter to the soil particles. During the period of increasing pollution in the percolate, the adsorptive capacity of the soil was nearing saturation, while a biologically active zone was becoming established. During this investigation the biologically active zone, or limiting zone, was between 10 cm. and 50 cm. below the surface.

Attention has been drawn to the importance of 'seeding' a soil to hasten the establishment of a suitable biological environment for stabilization of organic matter (Robeck et al., 1964).

There was a marked discrepancy between the behaviours of Hanford fine sandy loam packed in lysimeters, and nominally the same soil when tested in situ. This discrepancy could have been due to the existence of certain soil bacteria in the surface of the natural soil, and some differences of grading and stratification of the layers of the natural soil as compared with the disturbed soil packed in a lysimeter.

Nitrogen and Phosphorus Transformations

The results of the tests near Lodi (Greenberg & McGauhey, 1955) indicate that approximately one quarter to one half of the total nitrogen was removed whilst percolating through a depth of 13 feet, and most of the nitrogen which was left, was converted to nitrate.

Potassium decreased by about 50 percent during percolation, the greater portion of the removal occurring below 7 feet. Phosphates disappeared during the first foot of vertical travel. This may have been due to isomorphous replacement of hydroxyl ions in the clay lattice, or to biological removal of dissolved phosphates in the area of concentrated biological activity.

At Flushing Meadows, Arizona, two types of inundation schedules were tried: (wet schedule - e.g. 5 days wet, 2 days dry: and dry schedule - e.g. 1 day wet, 3 days dry). With the dry schedule there was sufficient free oxygen in the soil for complete aerobic digestion of the waste, and all the nitrogen was converted to nitrate. With the wet schedule, there was not sufficient free oxygen to oxidise the waste and the nitrate oxygen was required for completion of the oxidation. The result was that nitrates were denitrified to give molecular nitrogen which escaped to the atmosphere. Nitrate content of percolate from spreading basins can therefore be controlled by variations in the schedule for operating the basins.

Phosphorus removal at the Flushing Meadows site was about 75 percent when percolating through about 30 feet of sand.

Amramy (1965) has reported 90 percent phosphate removal, and 73 percent total nitrogen removal, in percolation of oxidation pond effluent through 30 metres of Israeli dune sand.

Removal of Bacteria and Viruses

Bacterial movement in the ground depends on whether the soil is saturated or not, the size of the soil grains, the state of any biological 'defense layer', and possibly on other factors.

The travel of bacteria in soil is greatest in coarse grained soil which is below the groundwater level. Where there are definite cavities or fissures which carry groundwater, bacterial contamination can travel for long distances.

Under the conditions of the Lodi experiments (Butler et al., 1954) bacterial disappearance in the percolating water was "tremendously rapid" in the first foot of soil, and only very occasionally were coliforms found at a depth of more than four feet, even though the applied wastes at times had coliform counts of over 5 million per 100 ml.. Furthermore, the number of coliform organisms penetrating one foot or more, as measured by tests on the liquid, was essentially independent of the intensity of the pollution of the waste water applied.

Further experiments at the University of California with five types of soil in lysimeters three feet deep showed the effect of formation of the 'defense' layer, and differences between different soils. The influent was settled sewage with a BOD of about 170 ppm, and a coliform concentration of over one million per ml.. Oakley sand with an effective size of 0.015 mm, subject to an infiltration rate of 0.1 feet per day, produced an effluent with a fairly constant coliform concentration of about 2,400 per 100 ml. for the first six weeks, after which time the concentration dropped to about ten percent of this value for the remainder of the study. The Yolo sandy loam performed differently: it had a similar effective size (0.0155 mm), and was subjected to an infiltration rate of 0.3 feet per day; it produced an effluent with a coliform concentration which fluctuated between 24,000 and 4,800 per 100 ml., with the observations tending generally towards the higher value. There was no indication in this case of the formation of a defense layer. The three other soils all yielded effluent coliform concentrations of less than 45 per 100 ml., with only a little variation, except that an occasional extremely high count appeared. If this indicates a periodic unloading of organisms in the soil, it could be of considerable interest.

Investigators in Arizona, Israel, and Holland have made observations relating to the travel of coliforms through an unsaturated seepage zone (Bouwer, 1968,1968: Amramy, 1965, 1964:4, Baars, 1957, 1964).

Bouwer has reported on field studies of recharge from spreading basins at Flushing Meadows, Arizona, where the soil profile at the test site consisted of about three feet of sandy loam, underlain by sand and gravel layers, with the water table at a depth of about 15 feet. The applied waste was effluent from an activated sludge plant, and it had a coliform concentration of about one million per 100 ml.. After percolating into the test well which was ten feet from the spreading basins, and 30 feet deep, the coliform concentration had been reduced to 0 to 5 when operating the basins on a 'dry' schedule (one day wet, three days dry); but the concentration increased to the range 8 to 33 per 100 ml. when operating on a 'wet' schedule (such as five days wet, two days dry). Under the wet schedule the soil became more saturated, and conditions tended toward anaerobic, causing denitrification of the nitrates; these conditions were not so favourable for the removal of coliforms as were the conditions existing when the dry schedule was used. However, under both schedules there was a highly efficient removal of coliform bacteria.

The spreading basins in Israel were fed with the effluent from oxidation ponds. Test wells were dug at a distance of 30 metres from the pond, and it was noted that "the bacterial removal was highly efficient".

The infiltration basins on the Dutch sand dunes receive water which has a coliform concentration of 1000 to 2000 per 100 ml. The sand has an effective size of 0.15 mm. The water percolates down to the ground water table, and stored water seeps to the collection channels. The samples taken from the collection channel which is 460 feet from the spreading basins regularly shows no coliforms in 100 ml.

Investigations showed that under a 15 years old spreading basin the bacterial penetration was only about half as much as it was under a spreading basin only one year old; the apparent reduction, with time, of the penetration of bacteria, was most likely due to the establishment of a 'defense' layer in the soil. Bacterial adsorption was greatest in the first 25 feet of travel of seepage water; this 25 feet was also the zone of active bacterial decomposition of organic wastes.

Coliforms have been observed to travel further in groundwater than they do in percolating through an unsaturated zone.

Some tests were performed in California to assess the travel of coliforms and other pollutants when sewage was injected into an aquifer (26, Krone et al., 1957). The injection wells were 100 ft. deep, leading to an aquifer which was from 3 feet to 7 feet thick and consisted of a mixture of sand and pea gravel with an effective size of 0.2 to 0.3 mm. The first waste injected was a mixture of ten percent raw sewage and 90 percent water, having a coliform concentration of 2.4 million per 100 ml.. With an injection rate of 37 U.S. gallons per minute, coliforms were detected at a distance of 100 feet in the direction of groundwater flow after a period of 33 hrs. but they were not detected at a distance of 200 feet, even after 46 days. Bacterial pollution was greatest near the beginning of the period, and subsequently became less.

Neither prolonged recharge, nor increased concentration of organic matter,nor greater injection rates caused the bacterial pollution to extend beyond the initial distance of travel.

At Santee, where channel alluvium was relatively uniform and consisted of silty fine to coarse sand with gravel, cobbles, and stone boulders, it was found that after percolation through imported alluvial sand and gravel, and travelling through the channel alluvium for 200 feet, the water had a coliform concentration in the range of 1,000 to 5,000 per 100 ml., and after further travel to a total of 2,500 feet, it still had a coliform concentration in the range of 5 to 20 per 100 ml. (Merrell et al., 1967).

In Tieton, Washington, there were complaints of well contamination up to 1500 ft. from a non-overflow lagoon which was infiltrating 3 inches to 15 inches per day. Faecal coliforms were obtained from samples collected at a distance of 250 feet from the lagoon (39, Bogan, 1961).

Reports of epidemic surveys show that in many cases bacteria have travelled further than in experiments (Mallman et al., 1961). It has been postulated that travel rate and distance may be increased by the lowering of surface tension produced by the use of synthetic detergents.

Except for investigation of suspected water supplies after the occurrence of epidemics, there is practically no information available about field studies of the movement of viruses in groundwater. For most epidemics in which viruses appeared to have travelled through the soil, it was suggested that the water had passed through fissures or fractures in the substrata, rather than that it had seeped through the soil itself (Drewry and Eliasson, 1968).

In laboratory experiments, water containing radioisotope-tagged viruses was passed through columns of nine different soils (Drewry et al., 1968). The results showed that virus retention by soils was an adsorption process characterized by linear adsorption isotherms. The adsorption was greatest at pH of 7.0 to 7.5, and decreased significantly at higher pH values. The tests indicated substantial inactivation of adsorbed viruses. It was concluded that virus movement through saturated soils should present little hazard to underground water supplies provided that soil strata are continuous, and that the usual practices are followed for separation of disposal systems and water supply wells.

Toxic Materials

Until more is known about the removal or change of toxic substances during seepage through sand or soil, toxic materials - both organic and inorganic - should be removed from the water before it is used for recharge. Under suitable conditions, toxic materials such as cyanides and organic pesticides may be biodegraded by a suitably acclimatized bacterial population, although under these conditions the B.O.D. removal and denitrification may be less efficient because of the absence of certain micro-organisms which may be susceptible to the toxicity.

Dissolved Salts

Observations made during the Lodi tests are indicative of the expected changes in dissolved salts as wastewater percolates through soil (24, Greenberg & McGauhey, 1955). In these tests, calcium, magnesium, and sodium ion concentrations remained relatively constant to a depth of 13 feet. Potassium ions decreased by about 50 percent during percolation, the greater proportion of the removal occurring below 7 feet. Ammonia was completely removed within 4 feet. of the anions, chlorides remained unchanged, sulphates and bicarbonates increased by roughly 30 percent, presumably due to oxidation of organic sulphur and carbon; and nitrates increased greatly because of the organic and ammonia nitrogen which was available for oxidation.

Percolation through soil does not normally cause an appreciable reduction in the amount of total dissolved salts in the water. As a result, groundwater recharge of wastewaters has little, if any, ultimate effect on the number of times a water may be re-used without desalination.

Detergents

Tests at the Robert A. Taft Sanitary Engineering Centre (33, Robeck et al., 1964) have shown that Alkyl Benzene Sulphonate (ABS), a 'hard' detergent, can be biodegraded by soil bacteria after a suitable period of acclimatization. Despite such degradation possibilities, it was found to be desirable to remove some of the ABS from the sewage effluent at Whittier Narrows before it was sent to the spreading basins (30, Parkhurst and Garrison, 1964); the concentration was reduced from about 4 mg./l to 1.7 mg./l by 7 minutes aeration with fine bubble diffusers, thus concentrating much of the offending ABS in a foam which was removed.

With a change-over to 'soft' detergents in U.S.A., it is expected that any detergent problems will be greatly reduced, because the new 'soft' detergents are more readily biodegraded.

Chemical Reactions with Soil or Groundwater

It is possible that reactions may occur between the recharge water and either the minerals in the soil, or dissolved matter in the groundwater, or both.

Where organic material is oxidised, the carbonaceous portion produces carbon dioxide, which, in water, forms bicarbonate and lowers the pH. The solubility of calcium, magnesium, and aluminium are affected by the pH, and therefore pH changes may cause leaching or deposition of the salts of these metals. The oxidation-reduction potential (ORP) is also affected by biological reactions. The solubility of some materials, including iron, manganese, and copper, are related to the ORP of the system, the reduced forms (ferrous, manganous and cuprous), whilst the oxidised forms (ferric, manganic, and cupric) are generally much less soluble.

Studies in Israel have shown that low solubility salts can be leached from layers of dunesand, and that some of the leached salts may tend to redeposit elsewhere (e.g. in lower layers where conditions may be different). It is believed that future changes in permeability may occur; increased permeability in the zones from which material is leached, but decreased in the zones which become clogged by redeposition.

Groundwaters in Long Island aquifers contain up to three mg/l of iron in the ferrous state. It is feared that any oxidation of this iron by chlorine or dissolved oxygen would convert it to the ferric state which then would be precipitated, causing permanent progressive blocking of the aquifer in the vicinity of the injection well (Peters and Rose, 1968).

Sulphides, which may be produced by anaerobic decomposition will combine with heavy metals to form insoluble metallic sulphides, in this way tending to reduce permeability, or even to cause practically complete blockage.

It is possible for phosphate to replace hydroxyl ions isomorphically in clay lattices. Short-term tests therefore could yield over-optimistic estimates of long-term phosphate removals in seepage treatment of wastewater.

Each project may have some specific problem which can be finally resolved only by the operation of a pilot plant for a sufficiently long time to indicate how the actual operating conditions are likely to change from time to time.

7. QUALITY OF WATER FOR RECHARGE

The required quality of water for any aquifer recharge project will depend not only on the method of recharge to be used, but also on the proposed use of the water to be taken from the aquifer. In all cases, toxic substances should be removed before waste is used for recharge.

When water is to be percolated intermittently from spreading basins into the groundwater through an unsaturated zone of adequate depth, the processes of natural treatment are brought into play. Many basins may be so favourably placed in such circumstances that they can be used safely to accept sewage which has been subjected to only primary treatment, even where the water is subsequently to be reclaimed for domestic use (Butler et al., 1954).

At the Whittier Narrows Plant, which has been operating since 1962, and at the Flushing Meadows experimental site, the water admitted to the basins is sewage which has undergone primary and secondary treatment. Amramy has reported on studies in Israel where spreading basins receive effluent from oxidation lagoon treatment of sewage.

In contrast, the earlier experiments in California treated the sewage to a high degree of purity by conventional treatment followed by chemical coagulation, and sand and carbon filtration, before applying it to spreading basins (Goudey, 1931).

The degree of treatment before spreading affects the rates of infiltration. In general, more highly purified effluents can be infiltrated both at a greater rate and for longer periods, before serious clogging occurs.

Where water is injected directly into an aquifer through deep wells, removal of bacteria is slower than in percolation from basins, and stabilization of organic material occurs at a negligible rate. In western U.S.A. direct injection is frequently used to form a barrier against intrusion of salt water. If the injected water be not reclaimed the quality of injected water is not so important as in the proposed Long Island groundwater replenishment scheme where the recharged water is expected to provide a substantial proportion of the domestic water supply.

In California there were direct injection experiments using mixtures of domestic supply water with up to 27 per cent raw sewage (Krone et al., 1957). When the aquifer became clogged it was cleared by pumping water from the well, thus reversing the flow. Extensive experimentation is being done in connection with direct injection. On Long Island elaborate precautions are being taken to ensure that the recharge water will not be detrimental to the groundwater quality, and that it will be compatible with the existing groundwater. The preliminary treatment of the waste consists of conventional primary treatment followed by activated sludge biological treatment, chemical coagulation with alum and coagulant aids, mixed media filtration, activated carbon adsorption, chlorination, pH adjustment, degasification, and final addition of sodium sulphite as a scavenger of oxygen and chlorine.

The pilot plants have produced effluents which more than meet the requirements for potable municipal water supply, and is not expected to oxidise the natural iron in the groundwater to cause precipitation of the iron as insoluble ferric salts, or to cause clogging in the aquifer, which lies several hundreds of feet below the surface.

The costs of the pretreatment and deep well construction for direct injection will restrict its use to those areas where land is too expensive for spreading basins, or where some other special conditions exist.

8. SPREADING BASIN OPERATION

The spreading basins on dune sand in Holland, using Rhine water, and the basins at Los Angeles where the sewage effluent is first coagulated and filtered, can be operated on a continuous basis (Baars, 1957,1964; Goudey, 1931). But where sewage with a lesser degree of preliminary treatment is used there is usually enough organic matter present to cause progressive clogging of the soil, so that the rate of infiltration gradually decreases. However, after a resting period, during which the soil surfaces of the basins are dried out, infiltration rates are increased sufficiently for re-loading of the basins, on an intermittent loading schedule.

Clogging has been claimed to be closely associated with polysaccharide and polyuronide production in the soil. Polysaccharides accumulate during the clogging process. It has further been shown that no polysaccharides form in the complete absence of oxygen, and that under highly aerobic conditions the polysaccharides are decomposed by fungi (Nevo & Mitchell, 1967). Anaerobic clogging is believed to be produced by the surface growth of bacterial slimes, coagulation and flocculation of dissolved organic matter, and possibly by the formation of ferrous sulphides in the soil.

Resting of the soil sets in motion the mechanisms of cleaning or "unclogging", which are believed to involve:-

a. Decomposition of the polysaccharides.

b. Desiccation or oxidation or both of bacterial slimes.

c. Drying of the soil surface, and its reconstitution by wind action.

d. Changes in soil structure due to drying and reactions with organic matter.

e. Oxidation of insoluble metallic sulphides to soluble sulphates, which may either be leached away, or act as an oxygen carrier to be again reduced to sulphide.

f. Oxidation and mineralisation of organic matter held in the filtering zone.

The effect of the length of the wetting and drying portions of the cycle has already been discussed in relation to the removal of nitrogen. If the wet portion of the cycles is too long when using polluted water the infiltration rate will decrease greatly. In addition, if the wet portion is kept shorter than about 7 to 10 days there will not be sufficient time for midge flies and mosquitoes to develop from eggs to maturity in the water, and insect nuisances thus may be avoided.

Experience with effluents from activated sludge plants, and lagoon treatment, suggests that the best infiltration cycle to adopt is a period of from three to six days wet, followed by a period of from six to twelve days dry.

Rates of infiltration depend on the soil type, grading of soil, the applied waste, and the management of the basin soil structure. Infiltration rates cited in the literature vary from about 0.1 feet per day to 1 million galls (U.S.) per acre per day (approximately 4 feet per day). The more common range of infiltration rates is from 0.3 feet per day to 2.0 feet per day.

In some laboratory lysimeter tests the growth of paddy rice resulted in both higher infiltration rates, and longer periods of wetting, whilst maintaining acceptable infiltration rates (Nevo et al., 1967). It was postulated that the oxygen exuded from the roots of the rice prevented reducing conditions from developing in the sand, and kept infiltration rates high.

At Flushing Meadows (Bouwer, 1968), it was found that Giant Bermuda Grass was effective in increasing the infiltration rate to over one third more than the next best infiltration rate observed.

9. ADVANTAGES OF USING A GROUNDWATER STAGE IN RECLAMATION

1. The treatment by percolation through the soil is generally efficient and relatively inexpensive.

2. The substantial time lag between use and re-use of the water provides additional opportunity for removal from the water of organisms, and destruction of chemicals which otherwise may be resistant to many artificial or high-rate natural treatment methods.

3. Because there is commonly very substantial underground storage and dilution opportunities in the receiving aquifers, the consequences of the short-term failure of preliminary treatment processes would be very much less serious than if direct re-use only were practised,

(e.g. Holland had sufficient 'emergency supplies' when the Rhine was polluted earlier this year, to obviate any noticeable consequences which may otherwise have resulted).

4. There is an opportunity for dilution with natural groundwaters.

5. The use of groundwater from recharged aquifers may not cause so much public distaste as might arise from direct re-use, even after advanced treatment to a high quality.

6. Water supplies stored underground are not as vulnerable as are surface waters to radioactive pollution, whether air-borne or from accidental discharge of wastewater.

7. In areas of water deficiency, reclaiming of wastewaters by groundwater recharge can be much more economical than importing water from an alternative source, or than reclaiming water directly from wastewater merely by using advanced waste treatment methods.

10. ACKNOWLEDGEMENTS

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DISCUSSION

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The importance of the subject of this paper can be gauged from the fact that if only 50% of our city wastewater could be reused, it would about double our water supply!

However, as the author points out, there are many problems. When there is maximum wastewater flow available, during the wet weather, the soil is usually saturated so that recharge by spreading basins is impossible while recharge under pressure by well injection reduces the advantage of purification by soil filtration. If we store the wastewater (or treated effluent) till a suitable season in surface reservoirs there is growth of algae and consequent deterioration in quality besides the cost of such storage.

On the other hand treated wastewater can be successfully used directly for crop irrigation as is being done at Werribee, Victoria. The question then arises of the merits of direct surface irrigation relative to aquifer recharge. The author has listed advantages of the latter use but not the disadvantages. The danger of "clogging" was mentioned in the paper and there is also the fact that a "time lag" exists between recharge and discovery of pollution in the water "recovered" from the aquifer. The persistence of viruses is well known and their detection is difficult. Their presence in a recharged aquifer may be discovered only after the source has been rendered pathogenic and an epidemic has broken out.

Other possible solutions are costly "tertiary" treatment of the effluent to render it suitable to be added directly to the city water supply, or, alternatively it can be disinfected and then piped separately round the city for restricted use. In Bahrein, where the writer worked for some years, each house had three separate water supplies: the first was "sweet" water for drinking, from an "Ionics" plant; the second was slightly saline bore water for washing and bathing; while the third was a poorer quality bore water for gardens. The danger of accidentally drinking the wrong water was slight as it was unpalatable, so that the danger was no greater than in using gas or electric supplies in the home.

The author mentioned the Melbourne & Metropolitan Board of Works in his paper and in this regard the report by Bird and Lang (1968) is valuable since it makes a similar review of overseas experience specifically related to the areas south east of Melbourne. It is of interest that these writers agree substantially with one conclusion by the author in his paper: "Investigations carried out elsewhere into the recharge of groundwater basins -

particularly when it is desired to use effluent water - have made it clear to the writers that before firm conclusions can be made on the practicability of an extensive and costly program of groundwater recharge, pilot studies must be made in the basin itself using water similar in all respects to that proposed for recharge".

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The Author in Reply.

The author thanks Mr. Lennie for his contribution to the discussion of this paper.

Mr. Lennie suggests that saturation of the soil during the wet season, makes recharge impossible. This depends, of course, on the type of soil in which the basins are sited. A statistical summary of reported infiltration rates (Koenig, 1964) indicates that in 80 per cent of installations where treated sewage is spread, the infiltration rates are between 0.4 feet per day and 1.1 feet per day, and in 50 per cent of installations the infiltration rate exceeds 1.0 feet per day. Under these circumstances, it would require a very wet season to prevent recharge by spreading. Thirty inches of rainfall in a month would reduce wastewater infiltration capacity by less than ten per cent.

Spreading basins used continuously for treated wastewater do suffer from clogging. But operational experience in most installations indicates that unclogging is readily accomplished by allowing the spreading basins to "rest" for a period of time. Koenig lists "experienced operating factors" (ratio of wetted time to cycle time) varying from 50 per cent to 97 per cent.

The time-lag between recharge and water recovery provides an opportunity for bacterial die-off, which gives an additional margin of safety over and above the destruction of pathogens by the microbiological "defence layer". Some of the research which has been done in regard to virus removal in soil seepage indicates that adsorption of the viruses on the soil is very effective, and that viruses can be held by adsorption until they are inactivated (Drewry and Eliassen, 1968). A very detailed study of viruses and their inactivation was made in connection with the Santee project (Merrell et al., 1967). Drewry and Eliasson concluded, from their experiments on nine soil types, that virus movement through saturated soils should present little hazard to underground water supplies, provided that the soil strata are continuous, and that the usual practices of separation of wells and disposal systems are followed.

The use of multiple water supply systems, as at Bahrein, is usually brought about as a result of a local economic situation. At Bahrein, the treatment of desalted water for drinking is expensive, and it is a proposition to reduce, by having a separate supply for drinking, the required amount of expensive water. In some towns in New South Wales - Dubbo, Bourke, Walgett, Warren, Brewarrina - dual systems have been approved, and some are under construction. In these towns the amount of garden water used is relatively large in proportion to water required for domestic purposes. Treated water will be supplied for domestic use, and chlorinated raw water will be supplied for gardens. In some dry areas, reclaimed wastewater may be suitable for the non-domestic supply, and would possibly permit re-use of the water without creating public distaste.

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PAPER No. 7

GROUNDWATER ANALOGUES

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1. PREAMBLE

As background to a discussion of groundwater analogues, the point should be made that any attempt to analyse a naturally occurring groundwater flow field invariably involves an idealisation (with some gross assumptions) of the soil properties, the flow characteristics, and the geometry of the soil region considered. Seldom, for example, are soils homogeneous and isotropic (even for the various layers in stratified soils); the boundaries between soil regions of different permeability straight and parallel to one another; flows steady and saturated; the media incompressible. The value of field tests in groundwater studies should be obvious.

Fortunately, in the vast majority of natural groundwater flows, Darcy's law (to be defined later) is justifiably assumed to be satisfied, and flow field analysis involves solving Laplace's equation with appropriate (but often ill-defined) boundary conditions for the region concerned. In passing, it should be noted that Darcy's law is unlikely to be applicable when flow velocities are high, as may occur with flow through coarse porous media (e.g. gravel beds), or flow adjacent to the screens of a pumped or recharged well. In these cases, a non-linear field equation applies, and although the analysis is more complex than for Darcy flow, methods of solution are available.[1,2]

In general, mathematical solutions of groundwater flow regions are impossible due to the complexity of the governing field equation and the usually complicated boundaries of the region to be analysed. However, with the present availability of high speed, large storage digital computers, numerical solutions of flow fields are readily attainable. By subdividing a two dimensional flow field into a large number of computational units (usually squares or rectangles), the field equation may be solved by finite-difference methods for each node of the grid. Difficulties arise if the boundaries of the flow field do not coincide with the nodes and sides of the computational units. To overcome these and other problems, a recent powerful numerical technique using finite elements has been developed, mainly by Zienkiewicz,[3,4,5] for solving Laplace's equation and, more recently, for solving the non-linear field equation[2]. This technique of dividing the flow field into elements (usually triangular for simplicity) is superior to the finite difference method because node points (vertices) may be located to coincide exactly with boundaries, which may be free surfaces, impervious or irregular. Inhomogeneity or anisotropy of the medium can be treated without difficulty,[4] and the technique can be extended to three-dimensional fields.[5]

In view of the capacity and versatility of the digital computer to numerically deal with complicated groundwater flow systems, the question might well be posed "What is the need or future for the various analogues that have been used in the past?" At a symposium[6] on "Artificial Recharge and Management of Aquifers" held at Haifa, Israel in March 1967, several papers were concerned with the use of sophisticated electrical-network analogues of both confined and unconfined groundwater systems. Aquifer properties and boundary parameters could be electrically simulated and varied, and function generators were available to provide the effects of discharge and recharge wells, spreading grounds, springs, and net infiltration from rainfall in aquifers of varying transmissibility. Towards the end of the symposium, the author of the only paper which dealt with the same type of groundwater system using a digital computer suggested that if a similar symposium was held five years later, the majority of papers would be about digital methods of solution! This prophecy could well be true, having regard to the recent advances in numerical methods abovementioned.

Despite this present advocacy of digital computers and numerical methods for groundwater studies, there are, for certain types of problem, advantages to be derived from the use of analogues, not the least being a better visualisation of the region under study because of the physical resemblance between the field situation and its analogue. To illustrate this with perhaps the simplest-to-use analogue (the electrical conducting (Teledeltos) paper technique) applied to two-dimensional flow fields in isotropic soils, the field boundaries on the conducting paper and the flow pattern within these boundaries are immediately meaningful as geometrically similar, scaled down versions of the actual region.

Three different types of analogue are discussed in this paper, namely membrane, electrical network, and Hele-Shaw analogues. Applications of the analogues will be presented, together with comments on the advantages and limitations of the three techniques.

It should at this stage be made clear, however, that an analogy is complete only when the characteristics of the physical system it describes can be expressed in identical mathematical form. With reference to the first paragraph of this paper, the accuracy of representation of any field situation by an analogue is primarily influenced by the accuracy of the field data, the tenability of the theory describing the flow of groundwater, and the compatibility of the field conditions with the assumptions inherent to the analogue.

2. THEORY OF GROUNDWATER FLOW

Quantitative studies of flow through porous media originate with the experiments on water flow through beds of sand carried out by H. Darcy and published in 1856. Darcy's law, an empirical relationship for flow in one direction, states

$$v = k\ i \qquad \ldots.(i)$$

where v = discharge or effective velocity = Q/A = total discharge across any plane of total area A.

k = coefficient of permeability or hydraulic conductivity, with dimensions of velocity, and a function of the properties of both fluid and porous medium.

i = gradient of piezometric head in the flow direction.

Generalized to describe three-dimensional flow, Darcy's law has the form

$$v_x = k_x \frac{\partial h}{\partial x} \qquad \ldots.(ii)$$

with similar equations (with x replaced with y and z) for the y - and z - directions.

In these equations, h = piezometric head = $(\frac{p}{\gamma_w} + y)$

where p = fluid pressure

γ_w = specific weight of water

y = elevation head above an arbitrary datum.

Now, the equation of continuity applied to a prism of soil of volume dx dy dz through which flow is taking place states[7]

$$\frac{\partial}{\partial x}(\gamma_w v_x) + \frac{\partial}{\partial y}(\gamma_w v_y) + \frac{\partial}{\partial z}(\gamma_w v_z)\ dx\ dy\ dz =$$

$$\frac{\partial W}{\partial t}\ dx\ dy\ dz \qquad \ldots.(iii)$$

where $\frac{\partial W}{\partial t}$ = rate of change of storage per unit volume due to compression of the soil, water, and gas within the prism.

If the soil is completely saturated (that is, air volume may be neglected), and unless the aquifers considered are hundreds of feet thick (for example, problems involving deep well pumping), compressibility of soil and water are negligible and the continuity equation, now a steady-state equation, becomes

$$\frac{\partial}{\partial x} v_x + \frac{\partial}{\partial y} v_y + \frac{\partial}{\partial z} v_z = 0 \qquad \text{....(iv)}$$

Substituting in equation (iv) for v_x, v_y and v_z from Darcy's law (equations (ii)), and assuming that the directional permeabilities are invariant during the flow process

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0 \qquad \text{....(v)}$$

If, using classical potential flow theory, the velocity potential ϕ is defined as

$\phi = k \left(\frac{p}{\gamma_w} + y\right)$, then equation (v) becomes

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = \nabla^2 \phi = 0 \qquad \text{....(vi)}$$

This equation, <u>the Laplace equation</u>, is a linear field equation applicable to the steady-state laminar Darcy flow of an incompressible fluid through a homogeneous incompressible porous medium. If the medium is isotropic, having the same permeability in all directions, the piezometric head can replace the velocity potential, and the Laplace equation simplifies to $\nabla^2 h = 0$ ....(vii)

<u>NOTE</u>. Identical equations apply for electrical current and heat flow with the analogous terms - electrical potential (voltage) and thermal potential (temperature) - replacing the hydraulic potential (piezometric head or ϕ).

In most groundwater studies it is necessary to simplify the physical problem by assuming it to be two-dimensional. Furthermore, when concerned with flows from wells, it is usually convenient to transform the two-dimensional Laplacian equation from cartesian to radial co-ordinates. Thus, for the case of radially-symmetrical, confined well flow through homogeneous isotropic media, the Laplace equation takes the form

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} \cdot \frac{\partial h}{\partial r} = 0 \qquad \text{....(viii)}$$

where h = piezometric head
r = radial distance from the well axis, taken as the origin of co-ordinates.

An implicit assumption in the use of the Laplace equation, in any of its various forms, to solve groundwater problems is that 'v' and 'i' in Darcy's equation (i) are linearly related. The limit of applicability of Darcy's law is commonly based on a Reynolds Number (R_N) criterion, with one definition of R_N being

$$R_N = \frac{vd}{\nu} \qquad \text{....(ix)}$$

where v = discharge velocity
d = 'representative' diameter of the porous medium
ν = kinematic viscosity of water.

Above a critical Reynolds Number (reported values vary from 0.1 to 75), the flow is no longer laminar-linear; inertia forces (causing non-linearity) become large compared with viscous forces, leading ultimately to turbulent flow. In these cases, a typical energy-loss equation is

$$i = av^n \qquad \text{....(x)}$$

where 'a' and 'n' are constants, and by analogy with pipe flow, 'n' could be expected to vary from 1 for laminar-linear flow conditions (Darcy's law) to 2 for fully turbulent flow conditions.

By rearranging equation (x), putting N = 1/n and re-defining $\phi = h/a$, and combining this energy-loss equation with the continuity equation for two-dimensional flow, the resultant non-Darcy flow field equation in cartesian co-ordinates takes the form[8]

$$(\phi_{xx} + \phi_{yy})(\phi_x^2 + \phi_y^2) + (N-1)(\phi_x^2\,\phi_{xx} + 2\phi_x\phi_y\phi_{xy} + \phi_y^2\phi_{yy}) = 0 \qquad \text{....(xi)}$$

where subscripts x and y refer to partial differentiation in the x - and y - directions.
On substituting N = 1, corresponding to Darcy flow conditions, it reduces to Laplace's equation

$$\phi_{xx} + \phi_{yy} = \frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0$$

Further, if 'a' can be considered constant, then 'ϕ' can be replaced by piezometric head 'h'.

Although practically all groundwater analogues are based on the applicability of Darcy's law and Laplace's field equation, the non-linear field equation (xi) is capable of solution by numerical methods as stated previously.

Detailed treatment of non-Darcy flow, and the use and definition of Reynolds number as a yardstick for the type of porous media flow, is available in the literature[9,8].

3. MEMBRANE ANALOGY.

The surface of a uniformly-tensioned membrane sheet, when deflected by a cylindrical probe, is defined by the Poisson equation (Prandtl 1903)

$$\frac{\partial^2 Z}{\partial r^2} + \frac{1}{r}\frac{\partial Z}{\partial r} = -\frac{\gamma_m t_o}{T_o} \qquad \text{....(xii)}$$

where Z = probe deflection
r = radial distance
γ_m = specific weight of membrane
t_o = thickness of tensioned membrane
T_o = uniform tension in the stretched sheet in its undeflected form.

By comparing this equation with Laplace's equation (viii), the analogy is complete when the term $\gamma_m t_o / T_o \to 0$. Using a thin, isotropic membrane of 'dental dam' rubber clamped under uniform tension, typical values of 'γ_m' and 't_o' are respectively 3.6×10^{-4} lb per sq. in. and 0.010 inch. A tension 'T_o' of 40 lb per inch produces an increase in membrane area of 35 per cent. With these values, $\gamma_m t_o / T_o = 9 \times 10^{-8}$. In any case, it has been shown[10] that the Laplacian solution may be determined by adding to the actual membrane deflections, the 'sag' deflections present (due to $\gamma_m t_o / T_o$) when the probe is at zero level. Alternatively, but with some experimental inconvenience the membrane may be placed in a vertical plane and the 'sag' deflections ignored.

The membrane analogy is useful for studying the drawdown patterns surrounding discharging and/or recharging wells. An appraisal of the technique[10] indicates that it is particularly suited to investigation of multiple well systems with irregular aquifer boundaries. However, application of the analogue to the two classifications of the physical system (confined or unconfined aquifers) is restricted to steady incompressible laminar flow through saturated homogeneous isotropic media to fully penetrating wells.

Well Theory

For the case of confined flow to or from a well with radial symmetry, the drawdown satisfying the Laplace equation is given by

$$h_2 - h_1 = \frac{Q}{2\pi kb} \ln \frac{r_2}{r_1} \qquad \text{....(xiii)}$$

with $r_w < r \leq r_o$

where h = piezometric head at a point on the drawdown curve,
r = radial distance from well to a point,
r_w = radius of well,
r_o = radius of influence of a well,
Q = steady flowrate to or from a well,
k = coefficient of permeability,
b = constant thickness of confined aquifer,
or kb = T = transmissibility of the aquifer,
ln denotes hyperbolic logarithm (base e)

Since the Laplace equation is linear in 'h' in this case, the effect of multiple wells is the sum of the individual well influences under the same boundary conditions.

For a well in an unconfined aquifer, a rigorous solution to the shape of the phreatic drawdown surface is not available. Adopting the Dupuit assumptions,[10] the comparable expression to equation (xiii) is

$$h_2^2 - h_1^2 = \frac{Q}{\pi k} \ln \left(\frac{r_2}{r_1}\right) \qquad \text{....(xiv)}$$

To make this linear-logarithmic, and thus permit superposition of individual well influences in a multiple well system, the following equation has been found acceptable[10] to define the phreatic surface

$$h_o - h = \frac{0.69Q}{\pi k h_o} \log \left(\frac{r_o}{0.1\, h_o}\right) \log \left(\frac{r_o}{r}\right) \qquad \text{....(xv)}$$

where h_o = undisturbed piezometric height above an arbitrary datum and log denotes common logarithm (base 10)

Membrane Technique

The membrane is stretched and clamped to a frame providing, in the case of the University of Melbourne apparatus, a 36 inches square working area. By marking a 1 inch square grid on the unstretched sheet, acceptable extension and uniformity in tension is indicated by the change in grid size. Known constant potential boundaries are simulated by imposing the shape of the boundary on the membrane such that the membrane is deflected about it. Wells in the field are represented by adjustable-height cylindrical probes, deflections "up" corresponding to discharge wells and deflections "down" to recharge wells (that is, an inverted picture of the actual drawdown pattern is produced). Deflections of the membrane may be measured by micrometer connected to a pantograph for transferring the co-ordinates of any point on the membrane directly to plotting paper. Alternatively, stereoscopic photography and photogrammetric plotting of membrane deflections may improve the accuracy and reduce the tedium.

A variety of fictitious, though possible, well systems has been investigated.[10] These include combinations of several discharge and recharge wells with unequal flow rates, wells in unconfined aquifers with initially sloping phreatic surfaces, wells near straight impermeable (barrier) boundaries (utilizing image concepts), and wells adjacent to irregular potential boundaries such as bends in rivers or channels.

In any of these systems, alternative requirements may exist. Either the flow rate in each well is known and the drawdown pattern is needed or vice versa or, perhaps, the drawdown in some and flow rate in others is prescribed. Although similar procedures for analysis and interpretation apply, the question of scaling becomes important. The horizontal scale for the analogue is chosen with regard to the well-spacings and distances to the principal defining boundaries. The vertical scale is then determined by utilizing either equation (xiii) for a confined aquifer or equation (xv) for the unconfined case to find what drawdown or discharge corresponds to an acceptable probe deflection (for the Melbourne frame, a deflection as great as 0.6 inch, corresponding to a membrane slope of about 30° at the probe, produced a linear-logarithmic solution). It should be realized that the probe radius (r_p) cannot be reduced below about 0.125 inch without it puncturing the membrane, so that it is disproportionately large compared with the well radius. This means that a circular drawdown contour is assumed at a distance of (r_p x horizontal scale) from the well centre of the prototype.

The usefulness of the analogy has been checked by applying it to a particular field situation for which drawdown data were available. The prototype comprised a well, located 206 feet from a lake, pumping at a steady rate of 99 gallons per minute from a

20 feet thick unconfined aquifer with k = 2900 gallon/day/square foot. The agreement between membrane deflections and measured drawdown in field observation bores was satisfactory, thus providing confidence for the use of the technique for well systems for which confirmatory field data is not available.

One such application of the analogy to an actual problem concerns the drawdown pattern due to pumping from a confined strip aquifer such as a prior-stream deposit.[11,12]. A strip aquifer is defined as one in which the width is small relative to its length. An axisymmetric well and multiple centrally located wells (five) between parallel impermeable side boundaries, as well as multiple centrally located wells (five) between parallel potential side boundaries have been treated, using the analogy and a rigorous mathematical analysis. The agreement between theoretical and experimental drawdowns was good,[11,12] despite some limitations. For aquifers with impermeable boundaries, an infinite number of image wells are required to represent the drawdown pattern. The theoretical analysis can account for this but the analogue, with a limited area of membrane, can represent only a finite number of image wells (five on each side of the strip aquifer in this case). A second limitation to both methods of analysis lies in the selection of the distance along the length of the aquifer at which the drawdown is effectively zero. An arbitrary large distance was chosen for the theoretical analysis and, of course, the membrane frame boundary fixed the distance in the analogy. It should be noted that the radius of influence or, in this case, the length of influence of a well pumping at a steady rate from an infinite aquifer without infiltration will be infinite and, strictly, a steady state condition will never exist.

To summarise, although analytical methods, solved mathematically or numerically with a computer or by conformal mapping techniques[10] are, in general, equally applicable to most field problems, the membrane analogy offers some advantages. The analogy provides a rapid solution, is ideally suited for irregular boundaries, and gives a three-dimensional picture of a drawdown pattern which remains constant for recording or can be manually modified to show the effects of changes in well operation. However, the restrictive assumptions inherent to the analogue, impose severe limitations on its use in many instances.

4. ELECTRICAL NETWORK ANALOGY

Since transmission and storage govern the movement of water over, into, through and out of soils or porous media, they can be conveniently represented by electrical resistance and capacitance, respectively. Combinations of these two types of element constitute an electrical analogue, and enable solutions to be obtained for groundwater problems, particularly when there is a linear response. Field situations can be simulated relatively quickly, for a wide range of accuracy depending on the number of elements used, the sensitivity of the measuring instruments, and the junction losses at the nodes.

Because there is no counterpart to gravity force in electrical analogues using these elements, phreatic surfaces have to be obtained by trial and error procedures. However this disadvantage can be minimised and need not cause much inconvenience.

Two main groups of electrical analogues used for research purposes are resistance networks and resistance-capacitance networks. Resistance networks are particularly suited to steady-state problems, whereas capacitance networks are designed to cope with transient phenomena. The general properties of both types of analogue have been described by Bouwer.[13]

The resistance network depends on the similarity of form of Darcy's and Ohm's laws, and the finite-difference form of the Laplace equation to Kirchoff's law.

The finite-difference two-dimensional form of equation (vi) is

$$\phi_0 = (\phi_1 + \phi_2 + \phi_3 + \phi_4)/4 \qquad \text{....(xvi)}$$

where the values of ϕ are respectively those at the mid-points of the sides of a square mesh, and ϕ_0 is the potential at the centre of the mesh. For a grid of equal resistors to be analogous

$$V_o = (V_1 + V_2 + V_3 + V_4)/4 \qquad \text{....(xvii)}$$

where V = voltage potential.

From Kirchoff's law for a set of four conductors entering a node

$$i_1 + i_2 + i_3 + i_4 = 0 \qquad \text{....(xviii)}$$

and therefore from Ohm's law

$$\frac{V_1 - V_o}{R_1} + \frac{V_2 - V_o}{R_2} + \frac{V_3 - V_o}{R_3} + \frac{V_4 - V_o}{R_4} = 0 \qquad \text{....(xix)}$$

Solving equation (xix) for V_o gives

$$V_o = \frac{V_1/R_1 + V_2/R_2 + V_3/R_3 + V_4/R_4}{1/R_1 + 1/R_2 + 1/R_3 + 1/R_4} \qquad \text{....(xx)}$$

In the special case when $R_1 = R_2 = R_3 = R_4$, the potential V_o is given by equation (xvii), hence V_o is analogous to the potential ϕ_o of equation (xvi).

Just as the accuracy of the relaxation solution of equation (xvi) can be improved by decreasing the size thus increasing the number of "squares", so the accuracy of a resistance grid can be increased by increasing the number of resistors used, hence effectively reducing the size of the "squares".

Since the charge E of a capacitor varies linearly with voltage (E = CV), capacitors can simulate the change in storage involved in falling water tables and varying moisture contents, provided the water stored is linearly related to piezometric head; this is fortunately true for most problems of interest.

In a resistance-capacitance network, an earthed capacitor is tied to certain nodes of a resistance grid. For a study of phreatic surfaces, for example, the capacitors would be tied to the appropriate nodes of the resistance grid. For such a system, equation (xvi) is re-written

$$\frac{\phi_1 + \phi_2 + \phi_3 + \phi_4 - 4\phi_o}{a^2} = S/_T \frac{\partial \phi_o}{\partial t} \qquad \text{...(xxi)}$$

where a = the side of a square,
S = the storage coefficient of the aquifer,
T = transmissibility of the aquifer

For the electrical counterpart

$$\frac{V_1 - V_o}{R_1} + \frac{V_2 - V_o}{R_2} + \frac{V_3 - V_o}{R_3} + \frac{V_4 - V_o}{R_4} = C\frac{\partial V_o}{\partial t} \qquad \text{...(xxii)}$$

To operate the analogue, R is made equivalent to the reciprocal of T, and C is chosen to be equivalent to the product a^2S.

Resistance Network. The earlier form of this analogue was made of conducting paper, as referred to previously. This paper enabled two-dimensional studies to be made of anisotropic homogeneous media. The method cannot readily cope with differences in the hydraulic conductivities of horizons in layered soils. Nevertheless the work of Childs, as reported in Luthin,[14] enabled the design of sub-surface pipe drains to be firmly based on theory.

In contrast to the limitations of conducting paper, resistance grids enable non-homogeneous media to be studied, with the added advantage that the grid size can be reduced for zones where streamlines converge. Hence accuracy in these zones is increased, but the physical appearance of the model is then distorted. In practice, about 500 variable resistors are used for most studies, resistors being pre-set to values corresponding to their places in the grid lattice, and the horizontal and vertical hydraulic conductivities 'k_h' and 'k_v' of the medium. The details for this procedure are well set out by Vimoke and Taylor[15]. Values of potential are measured at the nodes, so that equipotential lines can then be plotted. For the more symmetrical cases, the sources and sinks can be reversed so that streamlines can also be obtained directly; on the other hand streamlines must be drawn by eye for the more complicated patterns.

Curved boundaries such as phreatic surfaces are represented by their best-fitting "staircase" function, and are obtained by adjusting the boundary until its nodes indicate the desired piezometric condition. To allow for unsaturated flow in the capillary fringe above a phreatic surface, it is usual to provide an extra layer of equivalent saturation; the thickness of this layer is based on the shapes of the curves relating pressure-moisture content, and conductivity-moisture content respectively. Although specifically designed to cope with steady-state conditions, the analogue can give solutions for those transient problems which can be treated over a succession of specified time-intervals.

The analogue built at the University of Melbourne has about 1400 resistors in a number of resistance ranges. The difficulty in obtaining phreatic surfaces has been greatly reduced by the use of constant-current sources through which the inputs are fed to the required nodes. These sources allow any current between 10^{-5} and 10^{-3} amp. to be obtained and held constant to within 3% for load variations of 0-40,000 ohm. Without such sources much adjustment of the phreatic surface would be necessary. The equipment is described by Mein and Turner,[16] and has been used for a study of the complex drainage of large irrigated sand-dunes in Coleambally, N.S.W., which are underlain by either horizontal or sloping impermeable layers. For the equipment and procedures described, the error due to contact resistance at the nodes was about 0.1%, that due to the finite mesh size 1%, the resistor settings 0.5%, and the input sources less than

2%. The analogue solutions were therefore thought to be accurate to within 4%. About five to seven hours were required to design and set up a grid, and one to two hours for adjustment of the network and measurement of the node voltages.

Resistance-Capacitance Network. Most analogues of this type are made to simulate conditions in particular aquifers; hence fixed resistors and capacitors are used throughout. Once the grid is established, it is used to study the effects of pumping rates, new draw-off points, leakage, and lateral inflow, on piezometric levels. For studies that involve discontinuous periods of pumping, the schedules are simulated by wave-form generators. The general requirements of such analogues are described by Walton and Prickett.[17]

In addition to these applications to specific aquifers, this type of network can also be used to study processes such as infiltration into dry soils. Neglecting the effects of gravity, the following equation can be developed

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial y} \left(D \frac{\partial \theta}{\partial y}\right) \qquad \text{...(xxiii)}$$

where θ = moisture content,
D = diffusivity = $k . \frac{\partial \chi}{\partial \theta}$
χ = negative pressure potential

The electrical counterpart is

$$E = \frac{1}{RC} \cdot \frac{\partial^2 E}{\partial y^2}, \qquad \text{...(xxiv)}$$

where D is analogous to $1/_{RC}$. Finite-difference solutions can be obtained by using resistors in series and capacitors at the nodes.

An analogue has been built by the Department of Mines, South Australia to simulate conditions within the aquifer under the Northern Adelaide Plains. From a brief report,[18] it is expected that the analogue will enable estimates of the future behaviour of water levels in this aquifer to be made with greater confidence.

5. HELE-SHAW MODEL OR VISCOUS FLOW ANALOGY

The viscous flow analogy, due to Hele-Shaw (1898), is based on the finding that, for viscous flow of a fluid between closely spaced plates, the streamlines (indicated by non-diffusing dye lines) closely resemble the computed streamlines for a two-dimensional irrotational inviscid flow (that is, the governing equation of flow is represented by the Laplace equation for potential flow). The analogy is useful for a wide range of physical flow systems[19] including axisymmetric flows, compressible flows, magnetohydrodynamics, flows with rotation, and those obeying Poisson's equation. In a groundwater context, the analogy is particularly suited to transient flow problems, including those which involve movement of a phreatic surface.

It would appear paradoxical that the flow of a viscous fluid is used to represent an irrotational inviscid flow theory. Further, how can the analogy or the theory apply to porous media in which the flow between soil particles is both viscous and three-dimensional? By way of explanation, the following brief background is presented.

Viscous Analogy Theory as Applied to Potential and Seepage Flow.

The dynamic behaviour of a real fluid is expresssd by the complex Navier-Stokes equations of motion for which general solutions are not available because of the non-linear, second-order nature of the partial differential equations. Most modern texts dealing with Fluid Dynamics derive the Navier-Stokes equations and introduce simplifications to permit tractable solutions for particular classes of flow phenomena. One such text,[20] especially suited to the present discussion, simplifies the Navier-Stokes equations for steady two-dimensional, laminar incompressible flow between closely-spaced parallel plates to the form

$$\frac{\partial p}{\partial x} = \mu \frac{\partial^2 u}{\partial x^2} \qquad \text{...(xxv)}$$

and

$$\frac{\partial p}{\partial y} = \mu \frac{\partial^2 v}{\partial y^2} \qquad \text{...(xxvi)}$$

where p = fluid pressure
μ = fluid dynamic viscosity

In this case, the non-linear inertia force terms are neglected and the z - direction is normal to the plates, spaced apart a small distance 'm'.

Integrating these equations, imposing the boundary conditions for a viscous fluid that u and v = 0 (no-slip) at z = 0 and z = m, and adopting the velocity distribution between the plates as parabolic, gives

$$u_{mean} = \frac{m^2}{12\,\mu} \cdot \frac{\partial p}{\partial x} \qquad \text{...(xxvii)}$$

or, in a gravitational field,

$$u_{mean} = \frac{m^2 \gamma}{12 \mu} \cdot \frac{\partial h}{\partial x} \qquad \text{...(xxviii)}$$

with similar equations for v_{mean} in the y - direction

where γ = fluid specific weight
h = piezometric head

Now, replacing $\frac{m^2 \gamma h}{12 \mu}$ in equation (xxviii) with the velocity potential 'ϕ' in irrotational motion, it may be shown that the equations for two-dimensional viscous flow between closely-spaced plates are identical with those for potential flow of an ideal fluid.

For <u>seepage</u> flows, generally involving laminar flow through small irregular pores, the interest is usually in average discharge-velocities rather than local pore-velocities (that is, we are concerned with a macroscopic scale which is large compared with the grain or pore size, in the same way as the average velocity between parallel plates was considered rather than the velocity of individual filaments). If the local-average velocity (defined as the local flow rate averaged over a finite area of the porous medium) is used in the flow equations, the physical system can be replaced by a mathematical continuum.[21] To account for the flow geometry (density, shape, arrangement of pores) and fluid density and viscosity, the factor 'k' (coefficient of permeability) is introduced to the equations. If the medium is incompressible and isotropic, 'k' is a constant for a given fluid as stated previously. Thus, for laminar flow through porous media, with inertia forces negligible

$$u_{mean} = k_x \frac{\partial h}{\partial x} \text{ (c.f. equations (ii) and (xxviii))} \qquad \text{...(xxix)}$$

In summary, then, for laminar incompressibe flows

$$\phi \quad = \quad \frac{m^2 \gamma}{12 \mu} \left(\frac{p}{\gamma} + y \right) \quad = \quad k \left(\frac{p}{\gamma} + y \right) \qquad \text{...(xxx)}$$

(potential flow) (viscous flow between parallel plates) (seepage flow)

All three expressions satisfy the two-dimensional Laplacian field equation.

Viscous Analogy Technique. Two parallel plates, usually of transparent perspex or glass for visual reasons, are spaced some 3 mm or less apart (to ensure viscous flow), and orientated in either a vertical or horizontal plane. A viscous fluid (for example water, oil or glycerine) is introduced between the plates and its motion studied under the boundary conditions applicable to the field problem. The permeability of the model (from equation (xxx)) is $k_m = m^2 \gamma/12\,\mu$, so that regions of different permeability can be reproduced simply by changing the plate spacing 'm' by inserting thin laminae. Impermeable lenses and strata are often represented by paraffin wax. It should be noted that, although wider gaps are possible with highly viscous fluids, constant viscosity is assumed in the model. In the use of undiluted glycerine or glycerine-water mixtures, experimental difficulties are likely to be encountered as the viscosity is very temperature-dependent.

The vertical Hele-Shaw model is particularly suitable for studying two-dimensional, saturated unsteady flows in unconfined aquifers with complex geometry. Applications include the movement of the salt-water/fresh-water interface in a coastal aquifer due to tides and a changing phreatic surface resulting from abstraction of fresh water by well pumping and/or infiltration from rainfall, irrigation or artificial recharge. An engineering application of the technique is in the study of the time-variable flow pattern in a zoned-earth dam following rapid reservoir drawdown, with the permeabilities of the various zones represented by appropriate gap widths between the plates.

An interesting groundwater application of the horizontal Hele-Shaw model is to study the mixing phenomenon of two waters of different quality. In this case, the plate spacing is adjusted to simulate the variable transmissibility; the storage capacity of the fluctuating phreatic surface is simulated by adding storage vessels at various points (c.f. use of capacitors in an electrical network); and the groundwater is represented by a suitable oil. Into this system, a different coloured oil (representing a different quality water) is injected in one well, with another well abstracting the mixture. In this type of study and for salt-water interface problems involving two liquids, some hydrodynamic dispersion takes place in the prototype, particularly at higher flow rates. This cannot be simulated in the viscous flow analogy.

Like the capacitance network analogue, this analogy can readily cope with unsteady or transient flows. A suitable time-scale can be selected by appropriate choice of fluid and plate spacing 'm'. The problem of correct scaling becomes rather complicated, particularly when dealing with nonhomogeneous anisotropic aquifers, specific storage coefficients for confined aquifers, and two-liquid flows. Space does not permit an elaboration of the scaling procedure and in any event, the scaling laws are fully treated in the literature.[22,23] It is of interest to note that the similitude requirements are derived without recourse to the Dupuit assumptions, so that the viscous flow analogue can be used to check on errors resulting from use of these and other simplifications.

A typical vertical Hele-Shaw model is used extensively by Tahal Water Planning for Israel Ltd., Consulting Engineers in Tel Aviv, Israel, to study the movement of the interface in the mostly unconfined aquifers along Israel's Mediterranean Coast. It consists of two perspex plates 4 metre long by 30 cm high, spaced 1.5 mm apart, with the profile geology appropriately simulated. Heavy green oil is used to represent salt-water, and a lighter oil to represent fresh-water. Tellus 72 oil with colouring dye is commonly used - the viscosity can be changed from about 750 to 425 centistoke at 20°C by introducing 4.5 per cent naphthalene. Infiltration, without air entrainment, is reproduced by feeding in oil from a line source along the top of the model. In one such model profile of an anisotropic unconfined aquifer, the scales adopted were:- horizontal length = 1:3000, vertical height = 1:500, and time = 14.2 minute: 1 year.

Another local groundwater application of the viscous flow analogy is the salt-water intrusion problem due to pumping in the Botany Basin, New South Wales.[24] The study, although limited by lack of adequate geological data, discusses the theory and scaling requirements for Hele-Shaw models in some detail.

Finally, despite the several advantages of the viscous flow analogy there are some limitations to its use. The analogy assumes laminar flow conditions; wells and drains are incorrectly represented as two-dimensional sources and sinks; and capillary effects above the phreatic surface have to be (and can be) allowed for in the model. Hydrodynamic dispersion in two-liquid systems cannot be simulated in the analogy.

6. CONCLUSIONS

There are, of course, other analogues available for groundwater investigations which have not been mentioned in this paper because they are thought to be not so widely applicable.

In general, the appropriate method of solution (mathematical, numerical, analogue, model or field testing) is likely to depend on the particular problem. In some instances, a groundwater system may best be studied by using different techniques for its various parts. Tahal Water Planning for Israel Ltd., for example, makes extensive integrated use of a sophisticated electrical analogue, Hele-Shaw models, and digital computer. The operation of the groundwater basin, in plan, is simulated by electrical analogue, with data transferred to Hele-Shaw models to investigate vertical profiles under transient conditions.

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PAPER No.8

HYDROGEOLOGIC ELECTRIC ANALOGUE MODELS CONSTRUCTED BY THE SOUTH AUSTRALIAN MINES DEPARTMENT

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1. INTRODUCTION

Physical systems which obey similar mathematical relations between cause and effect can provide analogies, which may be used to enable one system to be understood in terms of a more easily manipulated system.

Relations between water flow in a porous medium and pressure differences within that medium may therefore be simulated by the flow of electricity in a resistive medium, by heat flow through conductive media or by the stretching of membranes since in each case there is a linear relation between cause and effect.

Electrical models are probably the easiest to control; a simple model where only constant flow considerations are involved can be constructed using a sheet of conductive paper (e.g. Teledeltos paper) and coating portions of the surface with conductive paint.

Our experience, however, has been that most problems involve the estimation of conditions after some change has occurred. This required a dynamic (i.e. changing with time) response.

Electrical models which allow results to be displayed on a suitable oscilliograph have been the only practical solution we have found to this problem.

It would be possible to add electrical capacity to a paper model to simulate storage by insulating the paper from a metal backing plate by a thin film of material, but this would, unless the film was very thin, require the use of higher frequency response equipment with the attendant problem of stray capacitance effects from probes used to make measurements.

For this reason our models to date have been based on those constructed of discrete resistors and capacitors assembled in a regular network on a pegboard base as described by W.E. Walton and T.A. Prickett (1963). The representation of a continuous aquifer by discrete components in such a network can be proved to be a very close approximation. The resistors are selected so that their values are inversely proportional to the transmissivity of the portion of the aquifer they represent. Values are chosen so that there is a relation between the electrical current changes within the model and the voltage changes produced such that a satisfactory order of response can be expected from the measuring equipment used and the electronic auxilliary equipment can supply the required voltages and currents without overload.

Once the above conditions have been satisfied the choice of time scale for the model (1 second represents N years) fixes the size of the capacitor which must be used at each resistor junction to produce the electrical equivalent of storage factor.

2. NORTHERN ADELAIDE PLAINS MODEL

The first of two models that have been constructed in our workshops deals with a pressure aquifer situation that exists just north of Adelaide.

Water taken annually from this aquifer has increased over the last decade to the point where further increase in the withdrawal would appear to hazard the livelihood of market gardeners relying on this source of water.

The summer level in bores in the less saline parts of the aquifer has fallen by more than one hundred and fifty feet during this period; from above sea level to a point where the aquifer nearly ceases to function as a pressure aquifer. The main aquifer is a limestone aquifer about two hundred feet thick approximately 200 feet from the surface.

Besides adding considerably to the expense of pumping equipment and power costs, the difficulty exists that both above and below the low salinity beds there are parallel aquifers containing water of poorer quality.

Reduction of pressure within the main supply aquifer can cause increased amounts of saline water to be introduced and contaminate the better quality water.

The first model was constructed to test the possibility that the main aquifer was effectively separated from the more saline waters and that lateral movement of water in this aquifer alone could meet the demand.

Transmissivity and storage coefficient values from a limited number of pump tests were used to control the selection of the values of resistivity and capacity necessary to simulate the main aquifer.

Prior to the large scale use of water the aquifer which is bounded by a major fault line (Para Fault) to the east and the sea to the west had a piezometric gradient from East to West of approximately eleven feet per mile. Current sources entering the model near the Eastern Margin in points lying along small rivers rising in the uplands east of the fault together with leakage paths to the sea in the west were provided to simulate the piezometric gradient. To the north the beds of the main aquifer are thinned with decreasing transmissivity, so the resistivity values were progressively increased in this part of the model to simulate this effect. To the south the aquifer beds continue although the salinity of the water increases so that the demand is low. To achieve a model boundary the model was continued for some distance beyond the pumped area and then terminated in resistor capacity network designed to simulate pressure storage from the remainder of the aquifer.

The Department conducted a census of individual users estimated water consumption and these statistics were used as a basis for determining the pattern of withdrawal.

The model was constructed on the basis of one node or junction point for each quarter square mile over the main area of interest with an increase to one mile centres outside this area.

At this scale it is impossible to accurately represent the position of each of the operating bores so a different method of arriving at the withdrawal pattern had to be used.

This was achieved by adding adjacent landholders estimates of water consumption up to an amount equal to 2½% of the total estimated withdrawal, and taking 2½% of the total current from a point on the model corresponding to the centre of the area from which water was pumped. Any variations introduced by the use of this 2½% or by arbitrary spacing of pick up points on peg board holes will be less than errors in the estimates submitted by landholders etc.

Since the greatest demand is mainly during the summer months, a square wave type of pumping pattern was employed in which current was withdrawn for a period corresponding to seven summer months and the system spelled for the remaining five months. The amount of annual simulated withdrawal was automatically increased until a level corresponding to the withdrawal estimates was reached, and then the withdrawal was held steady. We hope that the growth pattern of water usage has been stabilised by restrictions imposed on drilling within the area. After the required number of annual cycles have been displayed the pumping is suspended for a sufficient period to allow the model to return to its pre-pumped condition when the pumping cycle is recommenced. This repetition pattern is necessary for display on standard oscilloscopes. However, in future we will be able to use one shot type of operation as we have recently taken delivery of a storage type of cathode ray oscilliograph.

By selecting a fixed time reference along the cathode ray trace the expected water level at the time at any selected point may be found and equipotential lines may be drawn for the area for the period selected from results accumulated from a series of point observations.

Our first attempts to draw equipotential lines from this model suggested that the geological model previously assumed could not give a satisfactory explanation of the variations of water level actually measured in observation bores.

The changes in observed level would have been produced with less than one third of the estimated consumption, and even if we assumed that the consumption estimates were high to this extent or alternatively the transmissivity figures used were low, the high potential gradient on the edge of the area of maximum demand could not be reproduced.

If, however, we assumed that the water in the aquifers above the main aquifer could supply water through the intervening low permeability material another source of water supply could be added to our geological model. This could also give an explanation of the origin of the water being pumped from the area as the majority of the water used could then be replaced by water from non pressure gravity storage, the drainage coefficient of which could be as high as 10 to 20 per cent. This is two orders higher than the pressure storage coefficient of 1.0×10^{-3}.

A system of capacitors corresponding to a storage coefficient of 10%, and resistors whose values corresponded to an average vertical permeability of one half of one percent of the permeability of the main aquifer were added to the model. These components were introduced in series between the resistor junctions and a constant voltage reference level.

The equipotential contours obtained then were much closer to the observations of water levels made in the field since the withdrawal estimates produced variations of simulated water level of the correct order between summer and winter and the potential gradient condition at the edge of our maximum demand area could be explained.

Further matching of the field data to our model at this stage depends on more information of the shallow aquifers, both as to their horizontal extent and to the thickness of water bearing sediments.

Some evidence has been produced that some of the water table aquifers are becoming dry, which could lead to discontinuities being introduced. These would produce a faster rate of fall of the water pressure than at present in evidence.

To produce additional information geophysical surveys, drilling and collection of information on private shallow bores is proceeding, and this information will be used to modify the model as it becomes available.

3. CHOWILLA DAM MODEL

The first problem dealt with the influence on the water level in bores as a result of removal of water from an aquifer, our next problem is the inverse one; what happens to the groundwater flow when excess water pressure is applied to an aquifer in the form of water stored in a large dam directly over material having considerable permeability.

It was proposed to construct a dam across the River Murray, upstream from Renmark in South Australia to ensure a continuous supply of low salinity water within South Australia. This would result in the inundation of more than one thousand square miles of mainly scrub country.

The groundwater in the area is saline with salt concentrations of 20,000 to 30,000 parts per million. The purpose of our investigation was to estimate the increase in salt water return to the river below the proposed dam.

When the dam was designed it was recognised that a considerable volume of water would pass under the main dam wall and a system of drains was incorporated to intercept this flow and transfer it to evaporative storage. Due to the short distance of travel and the relatively small volume of water present under the dam wall it was expected that the flow into the drains would in time become fresh enough to return to the river.

The consultants engaged in studying all salinity aspects of the scheme were interested in knowing the volume of water, which would bypass these drains, and also the increase in this flow which would occur with time.

This model was constructed of discrete components in a similar manner to the earlier version of the Northern Adelaide Plains Model, except that points corresponding to the river were interconnected by lengths of tinned copper wire. These lengths were divided so that sections of the river controlled by separate locks could be represented on the model by a series of junctions each having a voltage, the magnitude of which corresponds to the level of water in a particular section.

The voltage corresponding to each of the river levels below the proposed dam was regulated by means of integrated circuit regulators to within 0.1% of the required voltage corresponding to fixing the water level with 0.01 feet. The regulators were arranged so that any addition to, or withdrawal of, current from its represented section of the river could be measured by means of a differential amplifier connected to the appropriate regulator.

Voltages to the river sections in the inundation area were derived from a single stabilised voltage supply by means of voltage dividing potentiometers.

This model could now be set up as a static model of the existing river system and equipotential lines plotted and compared with the readings of water levels made in observation bores within the area.

Adjustment of flow conditions at the boundaries enabled the pre-inundation pattern of potential to agree closely with the pattern expected from field data.

To achieve the effect of inundation all the junctions corresponding to the boundary of the flooded area now had to be raised to a potential corresponding to the expected level of water in the dam. This was done by connecting each of these points by means of semi-conductor diodes to a square wave type voltage generator. These diodes were normally held in a non-conducting state until the simulated dam filling by raising the voltage brought them to a conducting level and allowed a constant voltage to be applied to the junctions.

A similar system operated by a reduction of voltage allowing diodes to conduct brought the points on the model used to simulate the drains across the downstream side of the dam to their correct potentials.

The additional current flow to each of the stabilised sections representing river stretches controlled by locks could then be followed by connecting an oscillioscope to the differential amplifier used to monitor the current changes in the appropriate voltage regulator.

Photographs were taken of the oscillioscope traces and these were used to estimate the saline flow to the chosen section of the river at various times after inundation. Total increased flow was obtained by adding the flows for individual sections. Time to reach 70% of steady state conditions was of the order of 500 years for the transmissivity and storage coefficients employed. This was the main purpose for which this model was built as no other method of estimating this factor was available to the consultants.

4. CONCLUSIONS

Analogue models produce a simple non-mathematical approach to the study of hydrological problems. Results are capable of direct interpretation in problems which would require complex mathematical treatments to arrive at a theoretical solution. They are, however, only as accurate as the data from which the model is designed. Where insufficient information is available they may, however, assist in selecting which of several geological hypotheses may be nearer the correct solution, and to suggest where further confirmatory field evidence may be obtained.

Our Department has found that such models do perform a useful aid to their hydrogeology studies and in addition to modifications to the Adelaide Plains Model two additional studies of areas of South Australia which contain major water resources are proposed. These are in the South Eastern Portion of South Australia and in the Polda Basin Area on Eyre Peninsula.

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PAPER No.9

DIGITAL MODELS IN REGIONAL GROUNDWATER STUDIES

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SUMMARY

Digital models have been developed overseas in the last decade for analysis and simulation of groundwater flow. The general approach and computational procedures used in these models are outlined, and applications are reviewed. Only studies concerned with conditions over an aquifer or groundwater basin rather than with the hydraulics of single bores are considered. Comparison of the digital and electric analogue methods shows that the former has several advantages, and that these should be enhanced by future developments. More widespread observation well records are desirable if digital models are to be applied in Australia.

1. INTRODUCTION

With the growing understanding and development of underground water resources, the groundwater hydrologist is increasingly concerned with the quantitative description of aquifers and their response to development. Proper planning requires estimation of the way in which water levels or hydraulic heads will be affected throughout an aquifer by a particular programme of utilization, the evaluation of alternative schemes of utilization, and the development of an optimum programme of pumping and recharge. This represents an ideal approach which has not generally been possible to attain in the past in the Australian environment, but analytical tools have now been developed overseas to the stage where thorough aquifer investigation is possible.

When flow conditions are considered for a whole aquifer rather than the drawdown round a single bore or well, and particularly where the real boundary conditions are complex, closed mathematical or analytical solutions are not generally possible. Recourse must then be made to other methods. Analogues of various types have been most widely used. For general aquifer studies, electric resistance - capacitance networks are usual, and these have recently been reviewed by Bouwer (1967). For relatively simple cases, analogues of the membrane, conducting paper and Hele-Shaw types have found wide use.

The development of large high-speed digital computers during the last decade has led to a growing interest in numerical methods of analysing the flow of underground water. Rapid development of digital models and many practical applications have been reported in the last few years. The stage has now been reached where numerical or digital models are at least as useful as electric analogues, and it is certain that their advantages will increase in the future.

The scope and applications of digital models will be reviewed, and their advantages and limitations will be discussed. No applications to Australian studies have been reported, but requirements for practical use in Australia are briefly considered.

As implied in the title, application of numerical or digital methods to regional groundwater studies only will be considered. While it is difficult to define precisely what is meant by a region, the studies dealt with here consider conditions within a relatively large-scale unit such as a complete aquifer or groundwater basin. This may contain many bores, recharge areas, and areas of effluent flow. The scope of this paper does not cover either analysis of the hydraulics of single bores or micro-scale analysis of flow through the pores of an idealised porous medium by means of the Navier-Stokes equations. Numerical solutions of the latter have been published recently by Stark (1968).

2. APPLICATION OF DIGITAL MODELS

2.1 General Approach

The model used to investigate a particular problem is simply the set of mathematical equations adopted for relating the various variables which are considered as representing the inputs, outputs, and states of the aquifer and its groundwater body, and the manner in which these equations are applied to represent the continuous nature of the aquifer and groundwater in space and time. In general, Darcy's Law and the law of conservation of mass are combined to give a nonlinear partial differential equation describing the flow conditions at any point in an aquifer. The actual form of this equation depends on the assumptions made regarding the aquifer and the flow conditions. In all analogue and numerical solutions, the continuum making up the aquifer field and its boundaries is replaced by a finite set of points arranged over the region, and properties of the areas surrounding these points are considered to be localised in the points or nodes. This is generally referred to as discretization. The arrangement of the nodes is discussed in section 2.3. For digital models, continuous time is also replaced by finite time intervals. The partial differential equation model is thus generally approximated by a finite difference equation model by subdividing the aquifer and the variation of conditions with time into nodes, paths and time increments. If the region is represented by N nodes, a set of N finite difference equations can be written, one for each node. For each time increment these equations must be solved simultaneously for the flow conditions. If required, answers will then be available for every node in the aquifer at each increment of time. With a large and complex aquifer, the number of calculations and amount of data involved require the use of a high-speed computer with large storage to make the method practicable.

The nodes of the model can be arranged in one, two or three-dimensional grids to represent the flow conditions assumed in any particular problem. Various aquifer properties are required for the model. The boundaries must be known to define the extent of the grid of nodes, and values of storage coefficient and hydraulic conductivity or transmissivity are required at each node. The latter can have different values in different directions in an anisotropic medium. In addition, aquifer properties can change from point to point to represent a heterogeneous aquifer.

The computational procedure can be divided into two phases, namely identification of the model parameters and prediction or simulation. In the former, the best values of storage coefficient and hydraulic conductivity or transmissivity must be found. Often, these will simply be values supplied by the results of pump tests and other investigations of the aquifer. If records are available of water levels or hydraulic heads at one or more points in the aquifer and of inflows and outflows, the model can be run for the period for which records are available, and the aquifer parameters adjusted successively in some systematic fashion to give the best possible fit of the calculated and observed data. These adjusted values of the aquifer parameters are then used for subsequent calculations. This procedure is justified on the basis that the values supplied by the aquifer tests are really only estimates, and are samples of the properties which actually vary from point to point. Unfortunately, data for verification and adjustment of models as described above are rare in Australia.

After identification of the model parameters, the second phase is prediction or simulation of the conditions in the aquifer corresponding to a selected input and withdrawal pattern. The inputs and withdrawals may include pumping, natural outflow, natural recharge from streams and rainfall, artificial recharge, or any other situation that may affect conditions. The prediction may be carried out for one or several alternative utilization programmes, possibly over a period of critical natural conditions. Alternatively, calculations may be carried out for a large number of input sequences selected on a probabilistic basis as a sample of possible performance in the future. Once a digital model is programmed and verified, a large number of calculations can be carried out with little trouble and at relatively low cost.

2.2 Review of Digital Models

The first digital modelling of flow in porous media related to the drainage of oil reservoirs. Numerical methods were introduced into groundwater hydrology by Stallman (1956), who described a procedure for determining the transmissivity of homogeneous and non-homogeneous aquifers by analysis of unsteady flow conditions. He used an explicit solution with a rectangular grid of 21 nodes. No further work of this nature was published for several years. However, Walton and Neill (1960), Walton and Walker (1961) and Trelease and Bittinger (1963) described numerical models of a different type. They assumed that the aquifers considered could be idealised to fit elementary geometrical forms such as wedges and infinite or semi-infinite rectilinear strips. Image well theory and non equilibrium formulae were then used to develop a mathematical model. An aquifer in Illinois considered in the first two papers was 84 miles wide and 1000 feet thick. Computed results gave only a reasonable fit to observed water level data, probably as a result of the oversimplified boundary conditions.

A more concerted approach to the application of digital models based on finite difference equations applied at a grid of nodes began with the work of Fayers and Sheldon (1962). They considered steady-state flow in three dimensions in a nonhomogeneous aquifer. Tyson and Weber (1964) and Remson et al. (1965) considered steady-state two-dimensional flow, and Bittinger et al. (1967) used a model for transient two-dimensional flow. Chun et al.(1964, 1967) and Weber et al. (1968) described in very general terms digital models developed by the California Department of Water Resources. One application was to a 480 square mile aquifer in the Los Angeles area, but no details are given of the models. A long term model of a rather different nature was described by Fiering (1965) for investigation of the drainage and salinity control of the Indus plain in West Pakistan.

Several more complex models have been reported in recent years. Freeze and Witherspoon (1966, 1967, 1968) developed a numerical model for steady-state three-dimensional flow in non-homogeneous anisotropic aquifers, and gave several examples of its application. Pinder and Bredehoeft (1968) and Prickett and Lonnquist (1968) considered unsteady three-dimensional flow in nonhomogeneous anisotropic aquifers with vertical leakage. In the latter paper, only a general description is given, but Pinder and Bredehoeft describe their model in detail together with its application to an aquifer in Nova Scotia. Bittinger et al. (1967) and Longenbaugh (1967) described a transient two-dimensional model of an aquifer in the Arkansas River Valley in Colorado where interchange of water between the river and aquifer was possible along their entire length. Bittinger et al. also incorporated a salt budget in a model, enabling the simulation of salinity control or salt water intrusion.

Considerable work has been carried out on flow conditions in oil-bearing beds, and many of these methods would have potential application to the analysis of groundwater flow.

Several numerical solutions employing the finite element method have also been published, but these are described separately in section 2.5.

2.3 Finite Difference Grid

Several types of grids have been used for the discretization of the continuous aquifer in the application of finite difference equation models. An asymmetric grid has been used in a few models, and a typical node and the area surrounding it are illustrated in Fig. 1. Nodes can be located at pumping wells, observation wells and other control points, and also along the periphery of the aquifer to define the boundary adequately. Thiessen polygons are constructed round each node to define the area represented by each node, and the transfer surface for the flow of water between each zone. Asymmetric grids of this type have been used by Tyson and Weber (1964), Vemuri and Dracup (1967), and Weber et al. (1968).

The more usual grid is square or rectangular with nodes at the intersections. A three-dimensional rectangular grid can be used if desired. Fig. 2 shows the two-dimensional grid used by Pinder and Bredehoeft (1968) to represent an aquifer in Nova Scotia. The aquifer was nonhomogeneous and the nodes are grouped in zones with different transmissivities. The zone boundaries and transmissivities were based on both borehole and field test data and on the results of parameter identification computations to reproduce observed water levels.

Variations of the rectangular grid have been described by Bittinger et al. (1967). One of these is the use of variable grid spacing, as shown in Fig. 3. This allows "focussing" of the computations on areas of particular interest within a large aquifer. Another variation is the use of a curvilinear grid where this suits the configuration of the aquifer. Figure 4 shows part of a grid that was used for an aquifer along the Arkansas River in Colorado (Bittinger et al. 1967; Longenbaugh, 1967).

Boundary conditions must be simulated in the grid network. Impermeable boundaries such as the edge of the aquifer are modelled by assigning zero hydraulic conductivity or transmissivity to all nodes or grid cells outside the boundary, as shown on Figs. 2 to 4. Hydraulically connected surface water sources such as a river or lake can be represented by a constant or time-varying water surface elevation at the appropriate nodes. Infiltration rate restrictions may have to be imposed. Similarly vertical and horizontal inflow boundaries can be modelled where necessary.

Very large grids have been used in several studies, allowing either a large area to be modelled, or a smaller aquifer to be analysed in considerable detail. Remson et al. (1965) used a 50 x 50 grid of 2500 nodes to model an area of 8 x 6 miles. Prickett and Lonnquist (1968) used up to 1500 nodes and Pinder and Bredehoeft (1968) 1145 nodes. Even larger grids will be possible with future increase in the storage and speed of computers.

2.4 Methods of Solution of the Finite Difference Equations

A full discussion of the various methods of solution of the set of nonlinear finite difference equations comprising a numerical model is outside the scope of this review, but the various computational techniques will be referred to briefly. The techniques differ only in the method by which the derivatives are approximated.

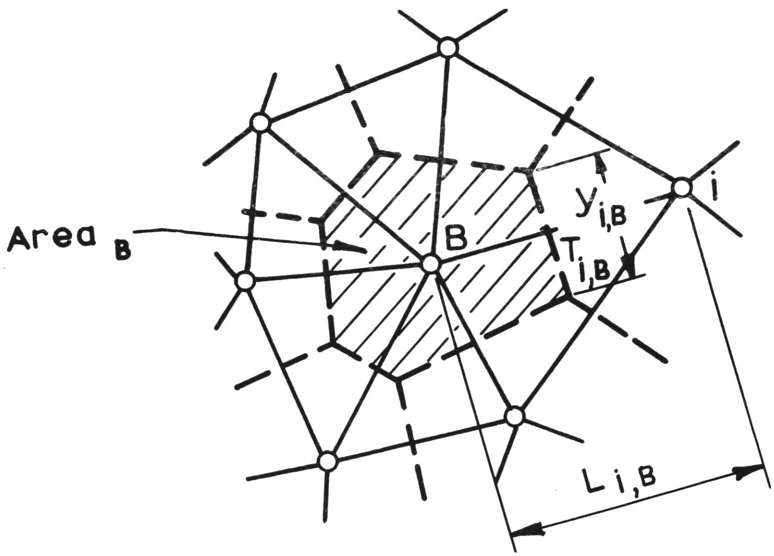

Fig. 1: Typical node in an asymmetric grid.

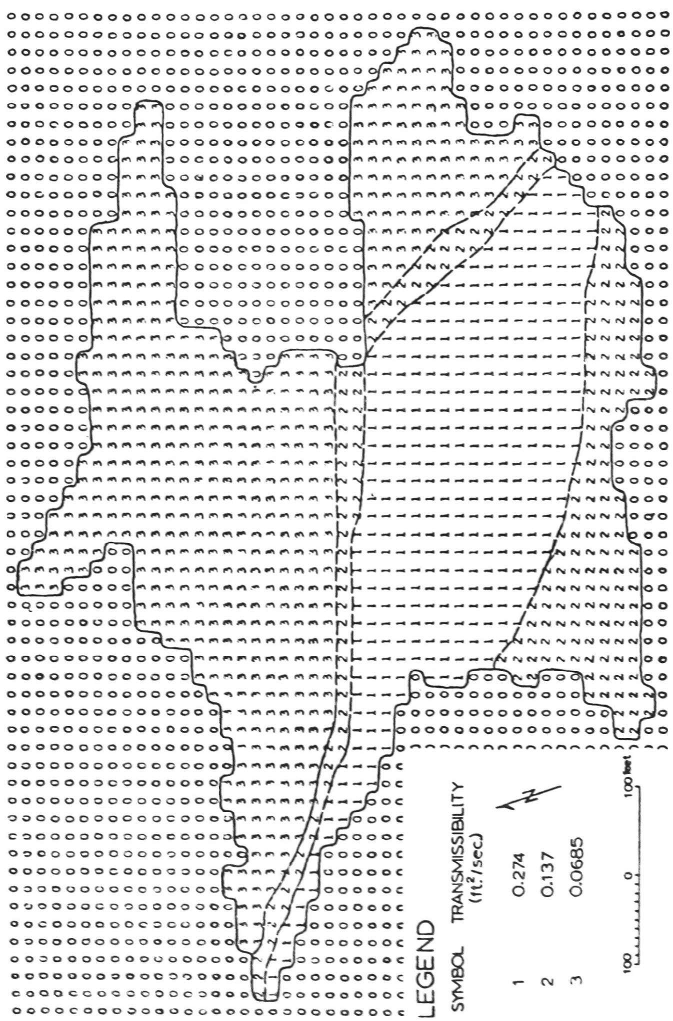

Fig. 2: Two-dimensional grid with transmissivity varying with area used by Pinder and Bredehoeft (1968) for a complex aquifer in Nova Scotia.

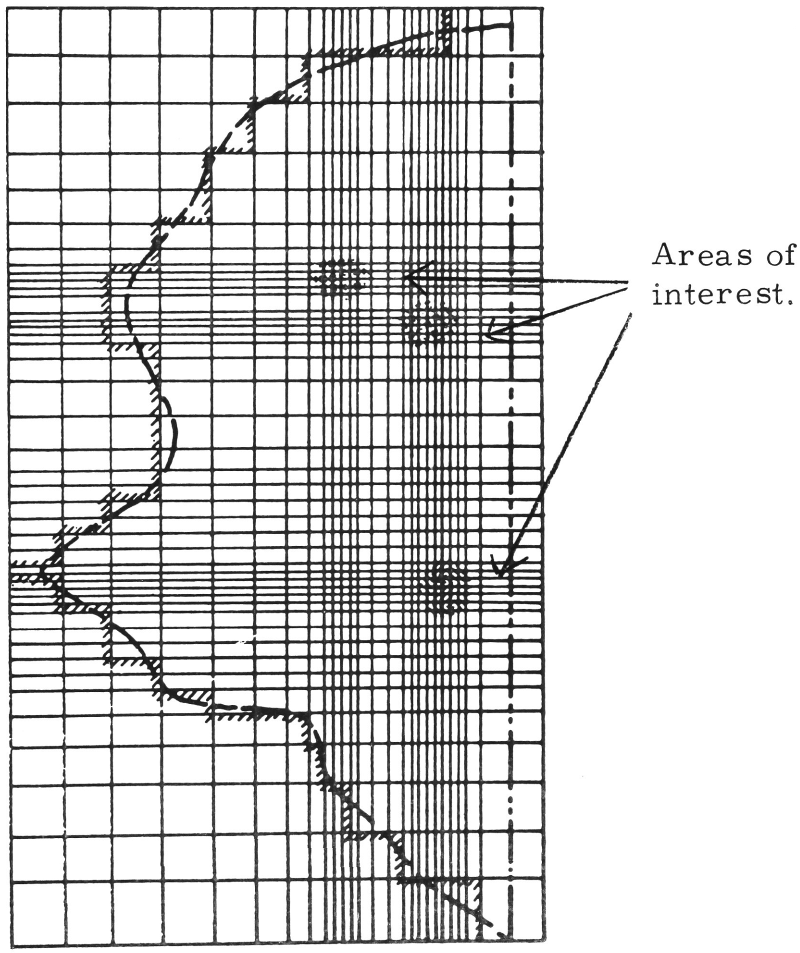

Fig. 3: Rectangular grid with variable spacing focussed on areas of interest (after Bittinger et al 1967).

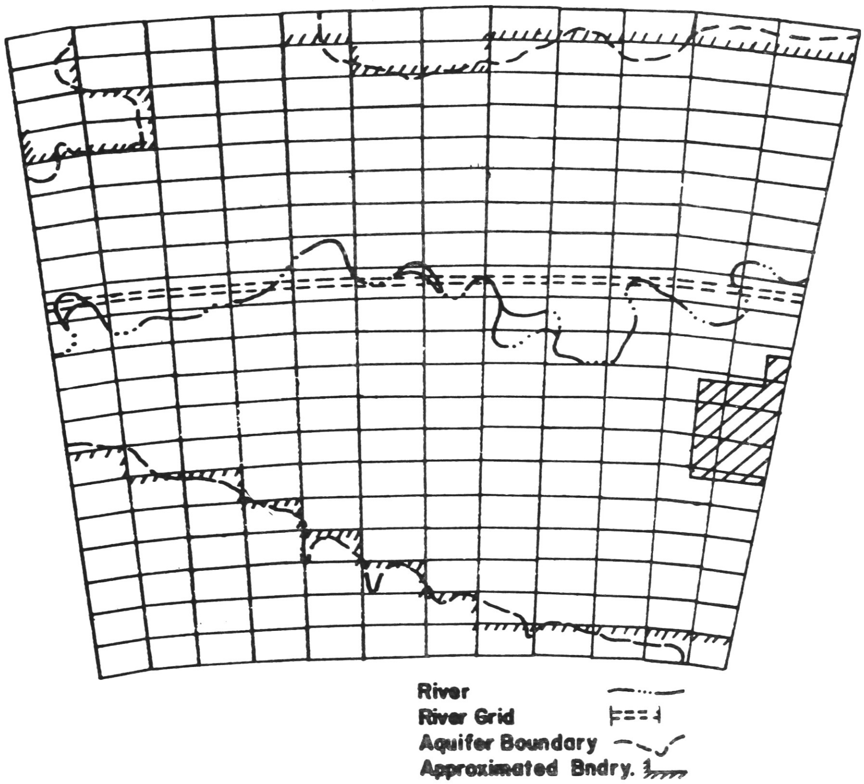

Fig. 4: Curvilinear grid network to suit aquifer configuration (after Bittinger et al 1967).

Explicit solutions are basically very simple and economical in computation time. However, the size of the grid step and time increment must be severely limited to prevent instability or divergence of the computations. Even with small increments, loss of accuracy is not always easy to detect. Kleinecke (1968) suggests that as a result of simplifications made in groundwater models, explicit methods will probably fail with time steps greater than about one tenth of those given by classical stability criteria. In general, the resulting very small time increments make explicit solutions impracticable in terms of computer time requirement.

Various implicit methods are available for solution of the equations. These are unconditionally stable, but generally require the inversion of an N x N matrix, where N is the number of nodes. Solutions for large grids thus require large computer storage and relatively long computation times. Over-relaxation techniques have also been used, and these are discussed by Fayers and Sheldon (1962), Pietraru (1968), and Freeze and Witherspoon (1966). The last authors report stability problems when using an irregular grid spacing.

Another method which has been used in several studies is the alternating direction method, which combines some of the advantages of both the explicit and implicit methods. Although this procedure has not been as well explored as the implicit methods, it appears to minimize both the number of calculations and the computer storage required. Pinder and Bredehoeft (1968) report studies indicating that the method may not be entirely valid when applied to the nonhomogeneous, anisotropic case.

Detailed discussions of computational techniques are given by Fayers and Sheldon (1962) Bittinger et al. (1967), Shamir and Harleman (1967) and Pinder and Bredehoeft (1968).

2.5 Finite Element Method

An alternative formulation to the finite difference equations discussed in the preceding sections has been developed in recent years for approximating the partial differential equations of flow. This is the finite element method, which is particularly suited to digital computation and provides a powerful method of solution. The technique has been developed for elasticity and structural problems, but is applicable to analysis of variations of potential over a field such as occur in the flow of groundwater. It is based on the variational principle and the calculus of variations. In place of solving the set of partial differential equations, the problem is to find a function minimizing a certain specified functional expression over the aquifer. The region is subdivided into elements as before, and the sum of the functional expression over the region is minimized by direct methods of the calculus of variations. This minimization over the region results in a series of simultaneous equations, the solution of which gives an approximate solution to the original problem.

In the finite element method, the nodes are at the corners of the elements into which the aquifer is divided. The basic elements are generally triangular, and complex boundaries can be readily modelled with irregular subdivision into these elements. Boundary conditions are also handled more easily than with finite difference methods.

Steady state finite element solutions have been described by Zienkiewicz and Cheung (1965), and these have been extended to nonhomogeneous and anisotropic media by Zienkiewicz et al. (1966). Three dimensional steady flow is considered by Zienkiewicz et al. (1967). Taylor and Brown (1967) give solutions for flow with a free surface, and Witherspoon et al. (1968) treat transient flow in a nonhomogeneous aquifer with vertical leakage.

Use of the finite element method in groundwater studies is still relatively new, but it appears that this technique may prove the most useful numerical approach in the future.

3. COMPARISON OF DIGITAL AND ANALOGUE SIMULATION OF AN AQUIFER

Only one study (Prickett and Lonnquist, 1968) has been reported with the aim of directly comparing digital models and electric analogues for simulation of aquifer conditions. Pinder and Bredehoeft (1968) used electric analogues to check their digital solutions, and comparison of the methods was a secondary benefit. Both studies involved complex aquifers with a large number of nodes, and considered unsteady three-dimensional flow in nonhomogeneous anisotropic aquifers with vertical leakage. These comparisons, with information in the other studies reviewed, provide sufficient evidence for a fairly firm assessment of the relative merits of the two approaches at the present time. It should be noted that the present section only deals with the modelling and simulation of flow in a given aquifer situation. There are several aspects relating to consideration of aquifer studies in their wider setting as an integral part of water resources investigations that make digital models advantageous, but these aspects will be dealt with separately in section 4.

The digital method does not require as much time for model construction and data readout. It is also more convenient than the equipment manipulation needed in the analogue technique, especially with complex boundary conditions and inputs.

Where boundary conditions are nonlinear the digital method is more versatile. Variation of permeability or other aquifer properties with time can also be handled more easily in digital models. With relatively simple boundary conditions and homogeneous aquifers, analogues may be preferable.

Digital equipment is generally much more accessible than analogue. Most organisations have access to a large modern computer. Special equipment is required for analogues, especially with complex conditions. Special skills are also required for assembling and manipulating the analogue equipment. Most analogue studies have been carried out in a relatively small number of centres where the equipment and special skills necessary have been developed.

Digital programming is now a common skill, and the programmes involved are generally not very complex. The digital method also enjoys the benefits of very large-scale research into numerical methods. Development of the finite element method is an example of this. Rapid developments in computers and in computer technology in the future will probably have no counterpart in analogues, although development of analogue computers, as distinct from analogues, will occur.

The analogue method at present has an advantage with very large problems. Digital models are limited in size by computer storage and computation time, but some large problems have been treated in the studies reviewed in section 2.2. Improvements in computers should reduce or eliminate this disadvantage in the not-distant future.

It has often been claimed that analogues have a great advantage in that they give a simple and rapid visual picture of the performance and dynamics of the aquifer system. In addition, the initial acquisition of analogue equipment provides the ready means for visualizing further aquifer problems as they arise. However, these advantages appear doubtful. The flexibility of most digital models and their rapid recall from library storage also provide a ready means of investigating continuing problems. Visual representation of aquifer conditions can readily be provided by on-line outputs of maps, charts, and shadow diagrams. Examples of outputs of this type designed to aid visualization are given by Bittinger et al. (1967), Longenbaugh (1967), Chun et al. (1967) and Prickett and Lonnquist (1968). Parameter sensitivity analysis, which can be carried out with either digital models or analogues, can also aid in presenting a picture of the nature of the response of an aquifer.

Determination of best values of aquifer parameters to reproduce observed data in the identification phase of investigation has generally been carried out on analogues. This appears to have been largely an intuitive procedure. However, it appears probable that parameter optimization procedures could be applied to digital models. Experience with models of the rainfall-runoff process indicates that much has still to be learnt before really satisfactory procedures are developed, but considerable advances have already been made. The only attempt at parameter optimization reported for the digital method illustrates these difficulties. Kleinecke (1968) used a simple least squares fitting procedure, but about 40% of the derived transmissivities and storage coefficients were negative, and many storage coefficients were greater than one. This indicates that constraints are necessary in such a procedure, and a nonlinear programming approach may prove useful. The only thorough investigation of parameter optimization was by Vemuri and Karplus (1969) using a hybrid computer, as discussed further in section 5. However, digital models appear to have potential in this regard.

The weight of evidence indicates that digital models at present have a slight advantage over analogues. This would not apply to authorities with analogue equipment and experience in its operation. The only major disadvantage of the digital method at the present time is the limitation on the size of the problem that can be handled. This limit will be raised in the future as computers are improved. In the more fundamental points of comparison, digital methods appear advantageous, and have greater potential for the future.

4. DIGITAL GROUNDWATER MODELS IN WATER RESOURCES STUDIES

As indicated in the previous section, the digital method has several further advantages when analysis of an aquifer is considered in its wider setting as an integral part of a water resources study.

In a particular region, groundwater is part of the overall water resource which should be utilised and developed in an optimum economic manner. Alternative designs and development programmes must be assessed to determine optimum utilization. Results of the aquifer model provide some of the inputs to the economic analysis. Techniques for this analysis have received considerable attention in recent years, and optimization procedures using digital computers have been developed. Digital models of aquifer hydrology thus offer the important advantage of being capable of direct coupling with the economic analysis.

Digital models are also very useful where probabilistic or stochastic data are involved. In simulation of the future performance of an aquifer, inputs such as rainfall and flow in an influent stream may be required. Alternatively, pumping may be required to supplement the flow of a stream in a conjunctive use scheme. Future rainfalls and streamflows are unknown, and values may be selected randomly from a probability distribution for use in the model. Many different random samples may be applied as input to the model. This adds realism to the simulation by considering a wide range of probable situations rather than a single predetermined situation, and provides a much firmer basis for design and decision making. Probabilistic or stochastic considerations of this type can readily be incorporated in a digital model.

It would also be desirable in many situations to couple models of surface hydrology and of groundwater, and possibly also of flow in the unsaturated zone, to form an overall model of the hydrology of a catchment or region. While analogue models of the rainfall-runoff process have been built, digital models are much more common and have undergone considerable development. Bell (1966) has reviewed several of these models. While the coupling of surface and subsurface models would be difficult with computer storage presently available, studies of this type are almost certain to be made in the future, and digital models will have an obvious advantage in such applications.

A further potential advantage of digital models is that they are compatible with computer oriented methods of data storage and retrieval. As further groundwater data are collected in the future, it is highly probable that computer oriented storage will be used for most types of data. Australian rainfall and runoff data are currently being translated into this form, and in the United States most of these data are now being recorded in computer compatible form. McNellis et al. (1968) have described a computer oriented storage system for groundwater data being used by the Kansas District of the U.S. Geological Survey. Computer programmes have also been developed to provide regular up-dated tabulations, maps, graphs and standard analyses of the data to aid checking, interpretation, and planning of field operations.

As noted earlier, it is also possible to include a salt budget or other multi-phase considerations in a digital model. This may enable the simulation of the movement of pollutants in groundwater, salt water intrusion, and salinity control.

The points discussed in this section are largely potential future advantages of digital models. However, when they are considered in conjunction with section 3, it is apparent that digital models are already capable of useful application, and that their usefulness and advantages over electric analogues should continue to increase.

5. USE OF HYBRID COMPUTERS

Although they cannot be classed as digital methods, it is of interest to note that two studies utilising hybrid computers have been reported. Vemuri and Dracup (1967) and Vemuri and Karplus (1969) describe work at the University of California, Los Angeles, and Shane et al. (1968) studied an aquifer in the Ohio River basin. These methods involve a combination of digital and analogue techniques. Data storage and inputs and outputs are handled by the digital computer, utilizing its storage capabilities. The analogue computer is used for inverting and solving the set of implicit finite difference equations, taking advantage of its speed. This also removes the limitation on the size of grid that can be analysed. Interface equipment is also required for the transfer of data between the two types of equipment. Vemuri and Dracup used the analogue computer for optimising transmissivity values in the identification phase of modelling. This is the only serious application of systematic parameter optimisation for groundwater models that has been reported to date. Shane et al. used the digital computer in the simulation phase for Monte Carlo generation of random natural recharge and of random streamflow deficits to be supplied from the aquifer.

Although the use of hybrid computers has technical advantages, specialised equipment and skills are required. In Australia, studies of a research nature only would appear possible. Practical studies would be limited to purely digital or analogue methods. Future improvements in digital computers and computer technology will probably reduce some of the advantages claimed for the hybrid approach.

6. DATA REQUIRED FOR DIGITAL MODELLING

Various types of data are required for satisfactory digital modelling of groundwater flow over a region. Boundaries and configuration of the aquifer must be known, together with estimates of the values of storage coefficient and hydraulic conductivity or transmissivity. The variation of these values over the aquifer is also required. Generally, data of this type are reasonably available in Australia or can be obtained by normal methods of investigation. If parameter identification computations are possible, initial estimates of the aquifer constants may be adjusted to produce a better fit of calculated to observed data.

Data on pumpage are also required as output from, or really negative input to the model. Records over a period of time are required for verification or identification of the model, and forecasts based on present and planned pumpage are required for the prediction or simulation stage of application of the model. At least approximate data on pumpage are generally available. Artificial recharge would also require consideration, but is rarely used in Australia. Streamflow and possibly precipitation records may also be required where natural recharge of the aquifer occurs or where deficiencies of flow are to be supplemented by pumpage. Records are normally available or can be estimated. For prediction of future performance, synthetic records of streamflow or rainfall may have to be generated. These are based on the sample recorded in the past, and various probabilistic or stochastic procedures are available for this synthesis. Chow (1964) has briefly reviewed these methods, and a more detailed discussion is given by Whitehouse (1967).

As with models of all types, digital models should be verified if at all possible. For this, the model should be run over a period for which observed data are available. Computed values can then be compared with the observed data to ensure that the model reproduces field conditions with acceptable accuracy. Water level or hydraulic head records from several observation wells over a period of five or more years would be desirable in the case of regional groundwater models. Such records are also essential to any parameter identification or optimisation studies. Unfortunately, records of observation wells are rather lacking in the Australian environment. There is an urgent need for expansion of the systematic collection of these records. The urgency of this need is increased by the fact that long records must already be available when an aquifer study is to be carried out.

7. CONCLUSION

Although digital models of regional groundwater have undergone only a few years of serious development, complex field situations can now be simulated with quite good accuracy. At the present time, digital models appear to have several advantages over electric analogues for regional studies, and these advantages are certain to increase in the future. Development of digital models has thus provided a very useful tool for the investigation of optimum utilization of groundwater resources. Interest in model capabilities is reflected by the many descriptions of work of this nature published in the last few years. Only equipment and skills commonly available are required for their use.

No Australian applications of digital groundwater models have been reported, but there is no reason why they could not be used in this country at the present time. However, there is a general lack of records from observation wells for verification of models. These measurements will need to be made on a much expanded scale if the potential value of digital models is to be fulfilled in the development of Australia's groundwater resources.

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DISCUSSION

R.J. Dickson and K.P. Stark, University College of Townsville.

We should like to add a few remarks to Dr. Pilgrim's paper regarding the finite element and finite difference methods of analysis of groundwater flow.

Both methods involve the specification of an unknown function throughout the field at a number of discrete points or nodes, and the subsequent solution of the same number of simultaneous equations. It is of interest to note that while the well known finite difference method represents a direct approximation to the differential equation type of formulation, the finite element procedure is an approximation applied to the variational formulation term. Hence, these methods of analysis result in equivalent formulations of the problem although they are derived by different techniques.

The development of these digital models, although equivalent in formulation, was entirely independent. The finite difference approach was first discussed in 1928 in a paper by Courant, Friedricks and Lewy, but until 1943, was employed only in small problems where hand calculations were possible.(1) Under the stimulus of wartime technology and with the aid of the first automatic computers, the finite difference method grew to be a powerful tool in numerical analysis. With a sound mathematical basis and numerous practical uses, this technique became extremely popular and is now widely recognised as a valuable computational aid.

The finite element method however, was developed by Clough et al. in 1956 for use with digital computers in the analysis of stress. It was not until the early 1960's that Zienkiewicz interpreted this method in terms of variational procedures and used it to solve boundary value field problems.(2) Since this method is still relatively new, and since it is only an alternative to the finite difference method it has not yet found the widespread use that the finite difference technique has for the solution of field problems.

It is evident therefore, that any continuum problem can be tackled by either method to obtain essentially the same result. There appears to be little benefit in using one method in preference to the other. However, more work has been done in the field of finite element methods towards systematizing and generalizing the computer programmes used for finite element methods. Boundary conditions, anisotropy, non homogeneity, and irregular boundary conditions are readily handled by finite element programmes without the necessity of any preliminary calculations. Finite element programmes have also been generalized to accommodate both continuum and structural problems with one, two, three or more degrees of freedom at each node, in two or three dimensions. The particular advantages of the finite element method lie in the technique used for the assembly of the equations, the systematic processing of data and the generalization of the programme system.

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The Author in Reply:

The author would like to thank Mr. Dickson and Dr. Stark for their remarks elaborating the comparison of the finite difference and finite element methods of analysing groundwater flow. Most analyses to date have utilised the well-developed finite difference approach, but as outlined in section 2.5 of the paper, several applications of the newer finite element method have been made in recent years to groundwater problems. With the rapid development of the finite element method, particularly in the field of structural mechanics, it is agreed that this should prove a valuable method of analysing groundwater problems in the future. As noted in the paper, it appears that this technique may become the most useful numerical approach.

PAPER No. 10

SYSTEM MODELLING FOR CONJUNCTIVE USE OF A GROUNDWATER AQUIFER AND A SURFACE RESERVOIR

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SUMMARY

The paper outlines some of the decisions to be made in designing for the conjunctive use of a reservoir and aquifer. Some of the methods available for analysing system models and making these decisions are then described. Details of a particular study using data from the Upper Hunter River serve to illustrate some of these methods and point out the difficulties involved in their application.

1. INTRODUCTION

Aquifers associated with rivers offer valuable reservoir capacity for regulation of river flow. When a dam is to be or has been built across a river, it may be possible to achieve much greater economic benefits by operating it in conjunction with an aquifer, if there is one, than by operating it on its own. The larger total storage capacity used can possibly not only increase the reliability of the regulated yield of the system, but also increase the yield and thus the benefits which flow from its use.

In the general problem of the design of such a system there are a number of both long-run and short-run decisions to be made. For the situation where there is an already existing dam across a river which provides a particular storage capacity, and where the water is to be used only for irrigation, there are long-run decisions still to be made as to:-

(i) the pumping capacity to be installed for pumping from the aquifer;

(ii) the area of land to be provided with irrigation distribution facilities;

(iii) the capacity, if any, of artificial aquifer recharge facilities.

Short run decisions include:-

(i) the proportion of irrigation demand to be met at any particular time from the aquifer;

(ii) the size of releases from the dam for aquifer recharge.

If the objective of the design is economic efficiency, then these decisions will be made according to some economic efficiency criterion or criteria. A criterion, in this context, usually means some quantifiable expression which is to be maximized or minimized. Examples of criteria could be the maximization of average annual net benefits, or the minimization of average annual costs to meet a specified demand for irrigation water. This latter example of a criterion often needs to be applied before the first one can be met.

An aquifer can function both as an additional storage provided free by nature and as a conduit, again provided free, but with a considerable lag, usually, in delivering its water. While the aquifer may deliver its water to the location where it is to be used, it still has to be pumped up, usually at a considerably greater cost (both greater fixed and greater variable costs) than with diversion or pumping from the river. Thus cost minimization will often favour minimizing use of the aquifer. On the other hand, if full use is to be made of the available resource, frequent recourse to pumping from the aquifer will be made, and in many cases expensive recharge facilities will be justified.

2. APPROACHES TO THE PROBLEM

The type of problem involved is an example of obviously complex decision making. It has been studied in a number of contexts, notably in Israel and the United States (References 1,2,3,4,5).

One approach is to use dynamic programming, as a sequential decision making tool, to devise the operating policy, i.e. to help make the short-run decisions, for a number of combinations of the long-run decisions, so that a best design may be found. There are difficulties with dynamic programming because of the large number of variables required to characterize the state of the system, and sometimes it is necessary to have recourse to the use of simulation and response surface sampling. Often, the choice of a suitable criterion for comparing designs proves difficult. A productive approach is to attempt to minimize the cost of achieving a particular target, and then to examine a number of target levels.

Whether dynamic programming, linear programming, simulation or any other technique is used, a system model has to be developed which incorporates the essential features of the system sufficiently accurately for sensible decisions to be made, but not too complex for it to overflow computer storage and consume excessive computer time.

It helps an examination of the problem of system modelling to divide the variables involved somewhat arbitrarily into a number of classes. There are the exogenous variables, or uncontrolled inputs, like rainfall, streamflow and evapotranspiration, successive values of which come from outside the system. There are endogenous variables, like rate of natural recharge from the river and leakage from the aquifer which characterize processes within the system. There are state variables like reservoir level and water table level which characterize the state of the system as time goes on. We can distinguish control or design variables, like reservoir releases or pumping rates from the aquifer, values of which we can decide on. All these variables will affect each other according to physical relationships, called "feedback relationships" in some contexts, which have to be understood and quantitatively evaluated to make the system model work. Out of all these relationships will emerge a so-called "dynamical equation" which will relate successive values of the state variables as time goes on.

Difficulties arise mainly in the identification of the system elements and in making the decision as to how severe lumping can be, e.g., how many distinct aquifers are there. Next, the quantification of the feedback relationships often involves the expensive gathering of data that are not already available. At least, system modelling along with sensitivity studies can pinpoint which relationships need to be known accurately and which are not crucial. A considerable difficulty often remains in establishing a suitable criterion or "objective function".

There follows a more detailed discussion of both the theoretical and practical aspects of modelling a system and implementing it for design. The discussion is based on work in progress at The University of New South Wales with financial assistance from the Australian Water Resources Council.

3. A PARTICULAR APPROACH

The object of the project is to develop a procedure for determining the optimum development of surface water and groundwater in a valley, i.e., determining the storage capacity of the surface reservoir, required pump capacity for river diversions and aquifer withdrawals and capacity of recharge facilities. The project is related to the Upper Hunter River in that data for that river will be used in the development of the procedure. Optimum development is taken to mean that development which satisfies a defined demand for water as much as possible at minimum cost. As pointed out in a previous section, this procedure is often a part of the work necessary to plan the overall development of a system with maximization of net benefits as the objective.

The system to be analysed is shown in Fig.1, and the flow diagram for the developed system is shown in Fig.2.

The word "demand" is taken in this context to mean a yearly pattern of monthly demands. When reference is made to different "demands", this implies the same pattern of monthly demands with the ordinates multiplied by an appropriate factor.

Imagine for the moment that we already have a system as in the above figures, with a reservoir of a given size, and we want to supply a given demand. We can find the reliability with which this demand will be met and the cost per unit of water as follows:-

(i) determine the operating policy for the system, i.e., the reservoir releases and aquifer pumpages per month for given initial stored contents (with the object of meeting the demand at minimum cost).

(ii) route a long period synthetic record of inflows through the system and operating policy, keeping an account of the shortfalls on demand, total water supplied, and accumulating the operating costs. From these accounts, and adding in fixed costs, find the average reliability and cost per unit volume of water supplied.

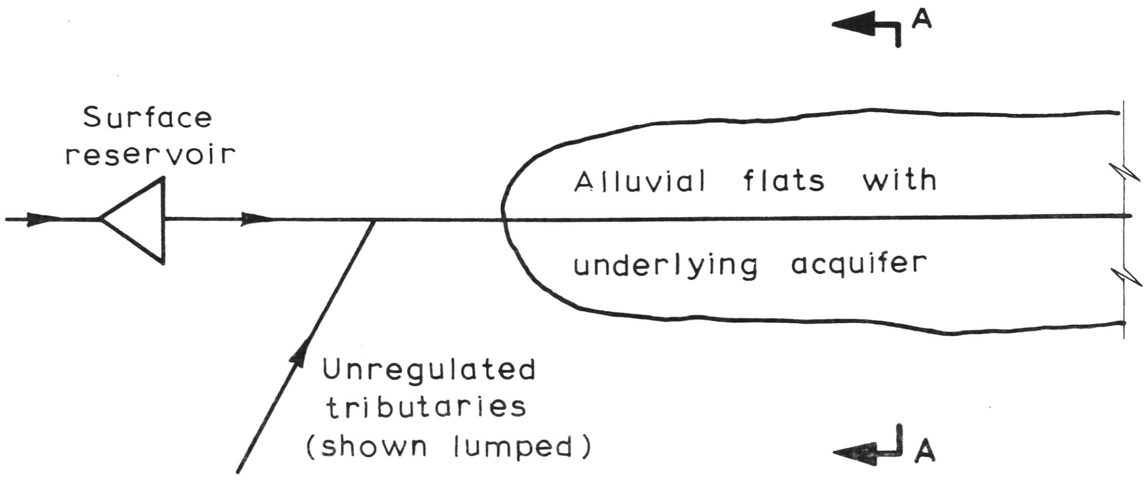
For section (i), the determination of the system operating policy, use is made of the technique known as dynamic programming, which will be described in more detail later.

In order to design a system for meeting a defined demand for water at minimum cost, the above procedure is repeated for a range of different possible reservoir sizes. For each size, the associated cost and reliability is found, and that size is selected which gives a minimum cost at the required reliability. From the above analysis the operating rules for the selected design are already known.

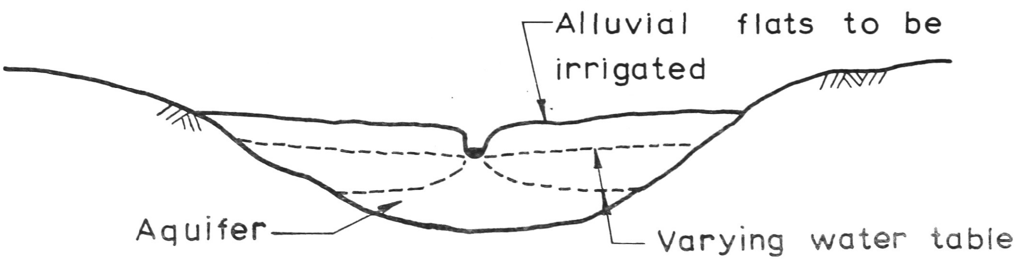
The procedure may be used to design a system with the objective of maximising net benefits in the following way. For a given reservoir size, find the cost and reliability for a number of different demands by applying the procedure for each of the demands. Then repeat this for different reservoir sizes, so that the cost is known for a number of combinations of demand and reservoir size.

For these combinations of reservoir size and demand for water, we presume that it is possible to obtain an estimate of the corresponding net benefits without the cost of water being counted. Determination of these benefits would involve finding the appropriate area to be irrigated. As water costs are now known, net benefits are easily found , and that combination with the highest net benefit is selected as representing the optimum development. For this combination, we already know the size of reservoir to be built, required pump capacities, system operating rules, and area to be irrigated.

We can now consider the analysis for any one of the above combinations in further detail.



SCHEMATIC PLAN



SECTION A-A

Fig. 1: System for which design for optimum development is required.

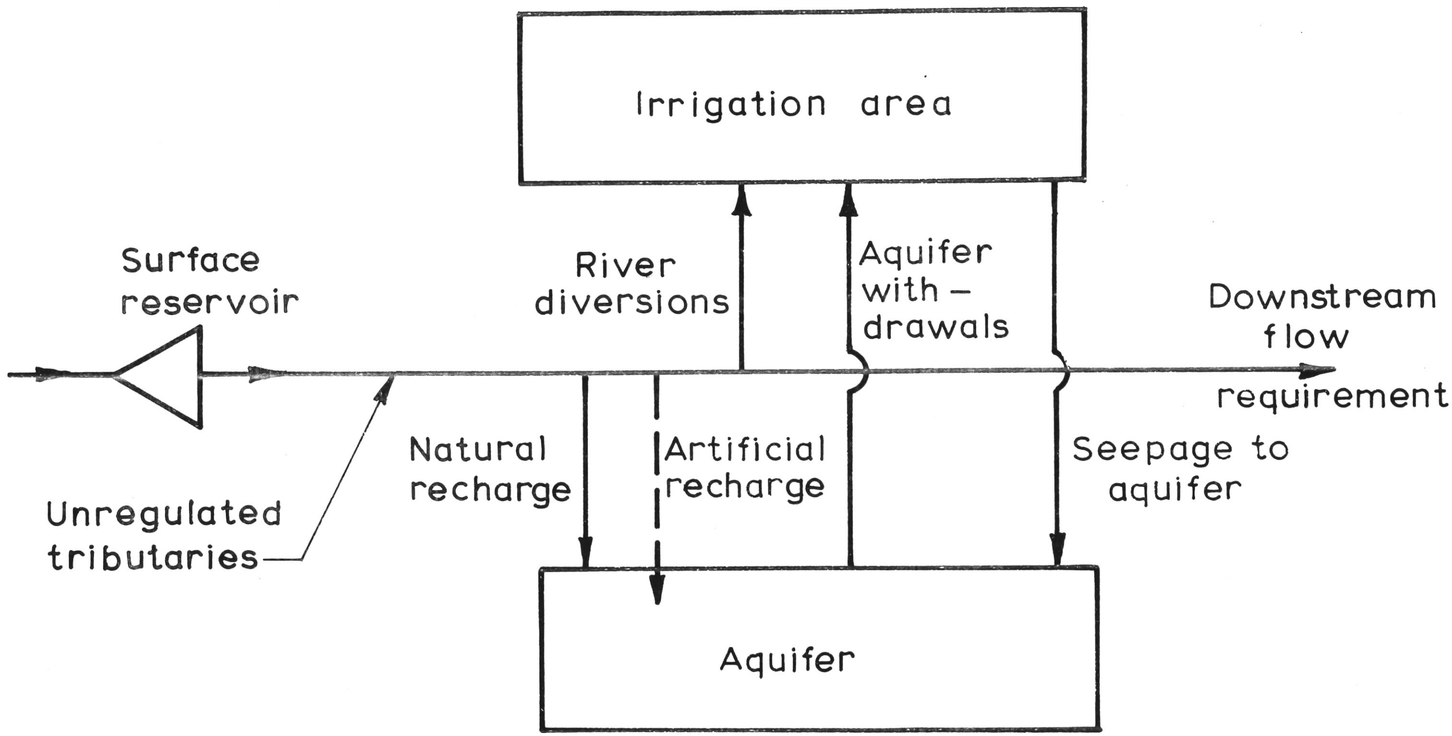


Fig. 2: Flow Diagram for the Developed System.

4. METHOD OF DYNAMIC PROGRAMMING

Imagine that we are at the start of the last month of the last year for which the water resource system will be operated.

Both the reservoir and aquifer could have quantities in storage ranging from empty to full. An increment in storage volume is selected, e.g. 10,000 ac.ft., and an operating policy is found for each possible combination of reservoir and aquifer storage contents. Consider the combination of 50,000 ac.ft. contained in the reservoir and 80,000 ac.ft. contained in the aquifer. The demand for the last month could be met by (i) releasing all required water from the reservoir and not pumping from the aquifer, (ii) pumping all required water from the aquifer and not making a reservoir release, or (iii) a range of combinations of reservoir releases and aquifer pumpages between the above extremes. The operating policy selected is that combination of reservoir release and aquifer pumpage with the lowest operating cost.

Having found operating policies for each possible combination of reservoir and aquifer contents for the last month of system operation, we then move backwards in time to the start of the second last month. Once again, an operating policy is found for each possible combination of reservoir and aquifer contents. For the combination of 50,000 ac.ft. in the reservoir and 80,000 ac.ft. in the aquifer, there is again a range of possible operating policies, with their associated operating costs. Each possible policy leads to a different combination of reservoir and aquifer contents at the start of the last month, but we already know the operating policies and costs for the last month, so it is possible to find the total cost for two months of operation for each of the possible policies being considered. The best policy is selected as the one which has the lowest total cost for two months of operation.

The above procedure is then repeated for the third last month, and operating policies selected which minimize costs for three months of operation, then the fourth last month, etc., etc..

This analysis is continued until the policies found for all months of a given year are the same as for the next year, at which point the optimum operating policy has been found.

5. DETAILS AND CONSTRAINTS ON THE DYNAMIC PROGRAMMING

The requirement to meet the demand as much as possible imposes a complication on the above procedure.

When the storage volumes being considered are low, it may not be possible to meet the demand with some or all of the possible operating policies. Those policies which involve a shortfall are neglected, and the best policy selected from those remaining. If all possible policies involve a shortfall, that which gives the smallest shortfall is selected regardless of cost. It is necessary to accumulate shortfalls from month to month in the same way as costs are accumulated, and the requirement for minimising shortfalls sometimes overrides the requirement of minimum cost.

The largest monthly aquifer withdrawal and monthly river diversion called for by the optimum operating policy determines the pump capacities to be installed.

The physical properties of the system influence the analysis in the following way. Suppose we are considering the nth last month of operation, and reservoir contents of 50,000 ac.ft. with aquifer contents of 80,000 ac.ft. If the demand to be met is, say 30,000 ac.ft., then two possible operating policies are:-

(i) reservoir release = 10,000 ac.ft. aquifer withdrawal = 20,000 ac.ft.

(ii) reservoir release = 20,000 ac.ft. aquifer withdrawal = 10,000 ac.ft.

For policy (i), the reservoir contents at the start of the (n-1)th last month will be -

$$R(n-1) = 50{,}000 - 10{,}000 + \text{inflow}\ (n) - \text{evaporation}\ (n) \tag{1}$$

and the corresponding aquifer contents will be -

$$A(n-1) = 80{,}000 - 20{,}000 + \text{recharge}\ (n) \tag{2}$$

For the combination of R(n-1) and A(n-1) the operating cost for the last n-1 periods is known. For policy (ii) a different combination of R(n-1) and A(n-1) will be found, with a different cost.

The inflow (n) is the average or expected inflow to the reservoir for month n, found from the historical streamflow records.

The evaporation (n) can be estimated. It will vary from month to month and will also be a function of the stored volume.

The recharge (n) depends on the river-aquifer gradient, i.e., it is a function of the ground water level and therefore of quantity in storage in the aquifer. The recharge is extracted from the reservoir release and is not available for pumping from the river for irrigation.

Other systems would probably require additional or different allowances to be made for the existing physical conditions. An example is for a case where the aquifer is a deep aquifer not connected directly to the river, but receives recharge from an area where the aquifer material strikes the surface, possibly many miles away. A second example could be an existing situation where irrigators near the river draw their supplies mainly from the river, while those at some distance from the river draw their supplies from groundwater only. In this case, the range of possible operating policies is restricted. If artificial recharge is to be considered, reservoir releases for this purpose could be included in the possible operating policies being considered at each stage of the analysis.

The above three conditions are included in the analysis either by changing or restricting the range of possible operating policies which may be considered, or by including them in equations (1) and (2) above, or both of these. Other possible constraints are included in the same way.

The numerical work involved in the analysis is simple but voluminous and it is necessary to employ a digital computer for this work.

6. DIFFICULTIES

Many practical difficulties have to be overcome to allow full implementation of the procedure. The most important of these are concerned with the physical properties of the aquifer. For the Upper Hunter area, it is not yet possible to define the amount of recharge that will occur in a given time for a given river-aquifer gradient. At present, no records are kept of quantities pumped from the aquifer, so the amounts of recharge that have occurred in the past cannot be found.

Allowance should also be made for the extent to which rainfall satisfies the irrigation demand, but the inclusion of this in the procedure outlined above has not yet been considered.

7. CONCLUSION

In spite of the difficulties involved, it is believed that the procedure outlined above is an example of a satisfactory method for planning the overall development of a conjunctive use system like the one being considered. The practical difficulties involved can be largely overcome by field testing of the aquifer in the investigation stage of a project, or by a more refined application of the procedure several years after the start of implementing a project, when more data has become available.

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PAPER No.11

THE CONJUNCTIVE USE OF WATER RESOURCES - A RESEARCH PROJECT IN THE UPPER HUNTER VALLEY

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1. INTRODUCTION

The purpose of this paper is to describe a water resources research project which has been initiated by the Hunter Valley Research Foundation in the Kingdon Ponds - Dartbrook catchments of the Upper Hunter Valley.

The project is still in its formative stages and it is not possible to report significant research results. However, because the work is unique in Australia, we believe that the planning which has already been carried out will be of interest and value to others.

The conjunctive use of water resources is simply the use of all water supply sources in a combination which is most effective for the attainment of a specific objective. Generally, considerable emphasis is given to groundwater supplies, although the role of surface supplies must always be taken into account.

It was concern for the exploitation of the Hunter Valley's groundwater supplies that prompted the Hunter Valley Research Foundation to recommend in 1965 (Ref. 1) that an intensive programme of investigation and study of these resources be commenced. The estimate of the amount of groundwater used in the Hunter Valley is approximately 40 percent of total consumption (Ref. 2), and there are sufficient instances of supply failures and quality deterioration for us to be concerned for the future of these sources (Refs. 3 and 4). The Foundation believes that the further exploitation of the Valley's water resources needs to be based on an adequate understanding of the inter-relationships which exist between surface and sub-surface sources of supply as well as the requirements of various user groups. This concern has led to the conjunctive use research project now underway.

Limitations of financial resources and manpower have made it necessary to confine the research work to a sub-catchment of the Hunter Valley. The study area covers the catchments of Kingdon Ponds and Dartbrook which are located in the Upper Hunter Valley (Fig. 1). There is no regulation of stream flows in the area, and in terms of present day developments, groundwater is the predominant source of supply for both municipal and rural uses.

The Kingdon Ponds - Dartbrook catchments were selected for the study for several reasons, the most important of which were:

* Most of the alluvium is outside the influence of the Hunter River which is regulated by releases from Glenbawn Dam.

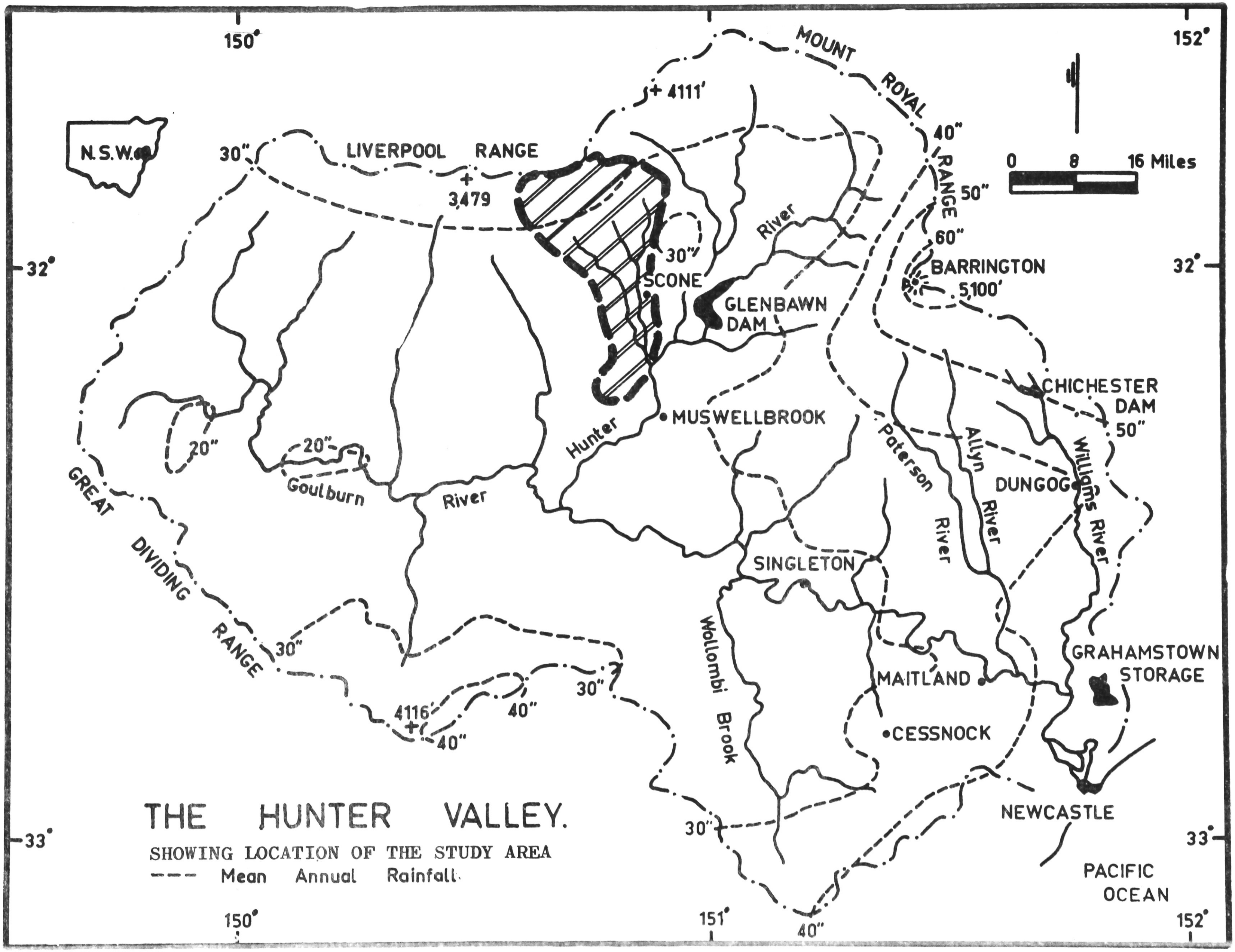

Fig. 1: The Hunter Valley showing location of the study area.

* With a catchment area of 320 square miles, it is of convenient size.

* It has municipal and rural water requirements.

* There is some information on the groundwater resources of the area already available from previous studies (Refs. 5 and 6).

* Many wells are located in the area.

* There are likely to be short term local benefits from the project because of the intensive agricultural land use and associated dependence on groundwater supplies.

The objective of the research programme is to formulate a suitable approach to the study of the conjunctive use of water resources. This broad-based objective has several requirements, including the following:

* The identification of suitable objectives for the conjunctive use of water resources in an area where resource developments rest largely with investments by individual landholders (as distinct from co-operative groups organised by government). For example, should developments be based on the maximisation of benefits to the total area or the maximisation of benefits to each individual user?

* The formulation of a mathematical model capable of simulating the water supply and withdrawal pattern and which could be used to determine operating rules for the attainment of a specified objective.

* To provide experience which could eventually be used in the planning of water resources development projects in other parts of the Hunter Valley.

* Additionally, it is expected that there will be short term gains to the study area which will allow plans to be formulated for the logical exploitation of water resources in that area.

The following sections of this paper describe the project work which has been done so far and outline some of the difficulties which must be faced. One of the long range requirements of the project is the extrapolation of methodology and techniques to other parts of the Hunter Valley and a brief review of the water resources structure of the Hunter Valley precedes the details of the research project.

2. WATER IN THE HUNTER VALLEY

The Hunter Valley is a coastal river catchment of about 8,500 square miles (Fig. 1) completely surrounded by mountain ranges varying from 1,500 feet high in the west to 5,000 feet high in the north.

Mean annual rainfall in the Hunter Valley varies from over 60 inches in the Barrington Tops area to less than 20 inches in some parts of the Valley west of Denman. The high rainfall at Barrington yields a large proportion of the total surface runoff in the Valley which on the average ranges from over 40 inches per annum at Barrington to less than one inch for much of the western part of the Valley. Table 1 lists values of mean annual runoff for several sites in the Hunter Valley.

Groundwater resources with significant yields are limited mainly to alluvial deposits adjacent to the main streams and to deposits in coastal sand beds.

Regulation of surface water supplies is provided by Glenbawn Dam located on the Hunter River, Chichester Dam on the Chichester River, Grahamstown Storage near the Williams River and Liddell Dam on Gardiners Creek. Glenbawn Dam has a storage capacity of 293,000 acre feet with 108,000 acre feet allocated to flood control and 185,000 acre feet to irrigation. Chichester Dam has a storage capacity of 19,000 acre feet with all water allocated to urban supplies for the Hunter District Water Board. Grahamstown is an off-channel storage of 150,000 acre feet capacity in which water diverted from the Williams River is held for use by the Hunter District Water Board. Liddell Dam provides a storage of 128,000 acre feet all of which is allocated to industrial use at Liddell Power Station. A dam is now under construction on the Paterson River at Lostock to provide a storage of 16,000 acre feet for irrigation purposes in the Paterson Valley.

Water use in the Valley was estimated to be approximately 121,000 acre feet in 1965 with allocations to various uses in the proportions shown in Figure 2. This is expected to increase to 216,000 acre-feet in 1975 and 383,000 acre feet by the year 2000. The 1975 demand represents an increase of 79 percent over the 1965 requirement (Ref. 8).

In 1965 irrigation requirements in the Hunter Valley were estimated to be 65,000 acre feet. Domestic and industrial needs totalled 56,000 acre feet. These values represented respectively 54 percent and 46 percent of the total. By 1975, irrigation demand is expected to fall to 51 percent of total requirements and remain

TABLE 1: MEAN ANNUAL PRECIPITATION AND RUNOFF
(Water Years, 1941-1958)

River	Catchment - Gauging Station	Catchment Area (sq. mls)	Mean Annual Precipitation (inches)	Mean Annual Runoff (acre feet)	Mean Annual Runoff (inches)
Hunter R.	Glenbawn	500	34.1	186,500	7.0
Hunter R.	Muswellbrook	1,630	32.7	402,000	4.6
Goulburn R.	Kerrabee	1,850	26.1	148,000	1.5
Hunter R.	Singleton	6,170	29.1	953,000	2.9
Paterson R.	Lostock	107	48.6	106,100	18.6
Allyn R.	Halton	79	51.6	100,300	23.8
Williams R.	Tilligra	75	50.7	92,000	23.0
Chichester R.	Chichester	76	58.7	132,000	30.7
Williams R.	Mill Dam Falls	374	47.1	397,000	19.9

Source: Reference 2.

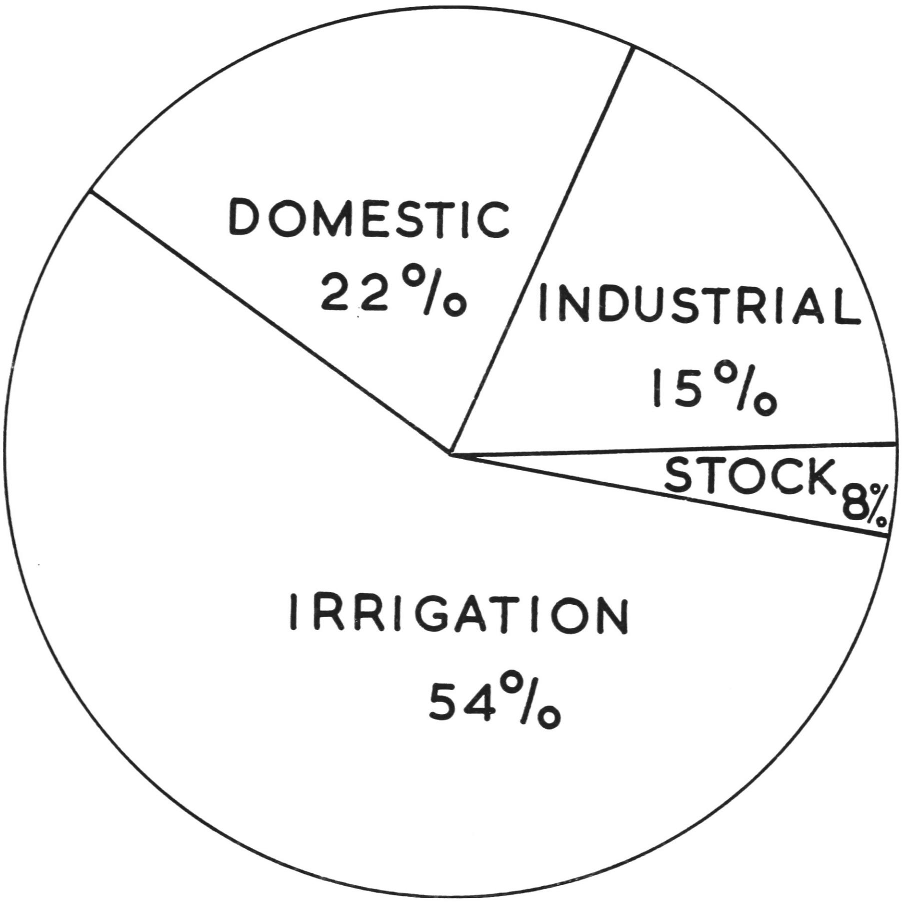

Figure 2: Proportions of water use in the Hunter Valley.

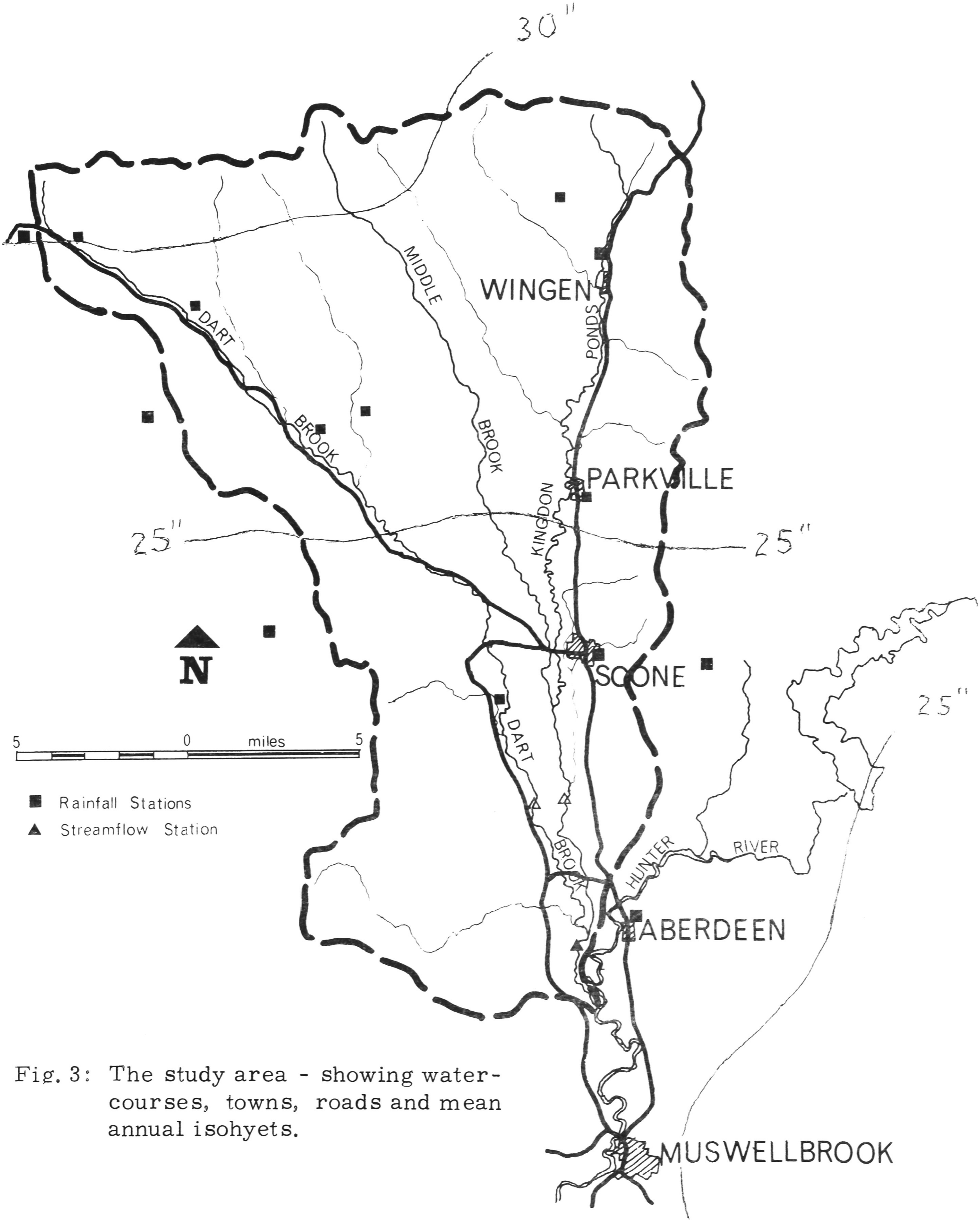

Fig. 3: The study area - showing water-courses, towns, roads and mean annual isohyets.

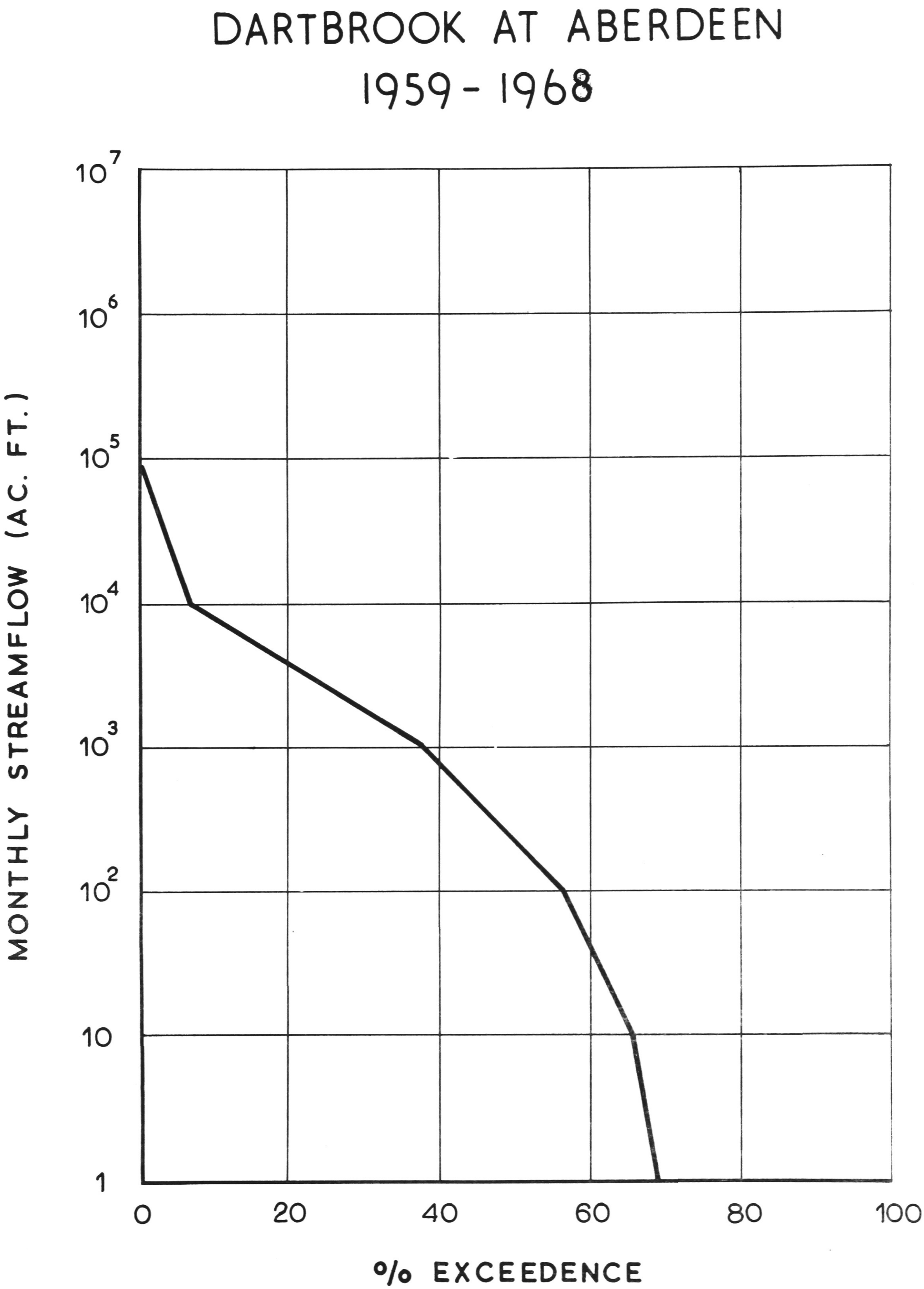

Fig. 4: Monthly flow duration curve for Dartbrook at Aberdeen.

at that value until the year 2000. Conventional domestic and industrial demands are expected to total only 38 percent of the 1975 total but will rise to 43 percent in 2000. The development of the Liddell Power station will create a special industrial demand by 1975 and at that time the requirement will represent 12 percent of the total. By the year 2000, this proportion will fall to seven percent (Ref. 9).

3. THE STUDY AREA

The area under study is the catchment area of Kingdon Ponds and Dartbrook located in the Upper Hunter Valley (Fig. 3). The creeks are minor tributaries of the Hunter River, joining it near Aberdeen. The catchment area is approximately 320 square miles. Elevations of the catchment range from 4,000 feet to less than 550 feet above sea level. The alluvium area has elevations ranging from 800 feet to less than 550 feet over a distance of approximately 15 miles. The most intense agricultural activities in the area and the greatest use of water take place on the alluvium which has an area of approximately 25 square miles.

The main population centre of the area is Scone (2,600) and the water supply for the town is obtained from groundwater sources although an augmentation scheme to import water from the Hunter River at Aberdeen by pipeline is now nearing completion. Other minor population centres are Wingen and Parkville. Another centre, Aberdeen, although outside the actual study area, serves the lower end of the area.

Water Resources

Rainfall ranges from a mean annual value of 35 inches on the northern rim of the catchment to less than 25 inches in the lower parts of the area (Fig. 3). Less than 10 percent of the valley receives over 30 inches on the average. Mean monthly rainfalls at Scone vary between 1.60 in winter and 2.90 inches in summer. Temperatures at Scone reach a mean monthly maximum value of 91^{o}F in summer falling to 62^{o}F in winter. Mean monthly minimums lie between 61^{o}F and 37^{o}F. Maximum occurrence of frosts at Scone is in July at a mean rate of seven days per month.

Streamflow in the area is variable and unreliable. The main water courses are Dartbrook, Middlebrook and Kingdon Ponds. Middlebrook is a tributary of Kingdon Ponds and joining it near Scone. Kingdon Ponds joins Dartbrook just above Aberdeen to flow into the Hunter River two miles further downstream. Ten years of streamflow records at the catchment exit show that more than 30 percent of months were without streamflow (Fig. 4) and that a flow of 250 acre feet per month has been exceeded on only 50 percent of occasions. Yet significant quantities of water are available on some occasions with recorded daily flows of up to 11,000 acre feet. In the drier months it is normal to observe small flows in each of the creeks in the upper reaches, but only scattered pools or long stretches of dry beds in the lower reaches.

Currently the main source of water in the area is groundwater. This is used mainly for irrigation of fodder crops for dairying and other activities, for municipal water supplies for the town of Scone and also for stock watering.

Extraction of groundwater is by pumping from wells located on individual properties. The wells are circular and lined with concrete cylinders between four and six feet in diameter, although wells constructed more than about 15 years ago are usually square in section and lined with timber. Wells are now generally 30 to 45 feet deep, although prior to the 1965-66 drought, depths were usually less than 30 feet. Pumps are usually electrically powered centrifugal pumps ranging from two to four inches diameter. During drought conditions in 1947, the water table fell considerably. More recently, in the severe drought of 1965-67, the available water on many properties was barely sufficient to sustain any irrigation.

Water Use

About 120 separate properties lie wholly or partly on the alluvium. Their mean area is about 350 acres. Approximately half of these properties are less than 200 acres and 75 percent are less than 500 acres. The total area of these properties is 60 square miles of which nearly 25 square miles form the flat alluvial land.

At present fieldwork is in progress to obtain some basic data on irrigation practice and water use on each property. To date, 58 properties have been visited. Mean area of the surveyed properties is 480 acres and 70 percent of them are less than 500 acres. Ten of the properties are operated in conjunction with properties in other areas - usually this second property is a large area of dry pasture (ranging up to 15,000 acres) for beef or fat lamb raising and the alluvial property is used for fattening of stock prior to marketing.

Based on the surveyed properties, it is estimated that the current total area under irrigation in the catchment is 8,000 acres which are used mainly for crops or pasture for grazing. This amounts to approximately 50 percent of the alluvial area, and allowing for roads, houses and other buildings, creek beds and land which may be unsuitable for crops, it can be seen that the area has reached an intensive level of development.

The irrigated area is supplied with water from about 250 irrigation wells and a few surface diversions on some upstream properties. Installed pumping capacity is estimated at 3500 HP of electric pumps and 1600 HP of diesel pumps. Of the surveyed properties 15 percent do not irrigate either because the type of operation does not justify irrigation or because the owner is not prepared to incur additional investment and work.

Estimated details of mean monthly irrigation requirements are given in Table 2. On the basis of the current acreage being irrigated, it is estimated that the mean annual demand on groundwater in the area is 7000 acre feet for irrigation. It is probable that on 30 percent of occasions the demand exceeds 10,000 acre feet per annum.

Another aspect of water use in the area is the municipal supply for the town of Scone. This is provided by six wells close to Kingdon Ponds which supply a 350,000 gallon reservoir for reticulation. It was estimated that the annual consumption in 1965 was 300 acre feet. At the high reliability level required for a town water supply system, the demand on the groundwater system by the town is quite significant.

At present an augmentation scheme is being constructed to import water from the Hunter River at Aberdeen where river flows are regulated by releases at Glenbawn Dam. The maximum capacity of this system will be 50,000 gallons per hour and it is likely that the demand on groundwater supplies by the town of Scone will be significantly reduced. However, pumping costs will require the Shire Council to give careful consideration to the proportion of the supply taken from each source.

4. CONJUNCTIVE USE OF WATER RESOURCES

Conjunctive use of water resources has been previously defined as the use of all water supply sources in a combination which is most effective for the attainment of a specific objective. This implies an allocation problem involving both physical and economic restrictions and two types of relationships must be determined to enable the best allocation to be made.

The required relationships are best dealt with in the form of two models - a physical (or water system) model and an economic (or conjunctive use) model. The former enables the physical quantities of available water under varying conditions to be determined, and the latter allows the allocation of this water to be made to various demands subject to given costs and an optimum allocation determined in the light of a specific objective.

Survey of Literature

The concept of conjunctive use of all sources of water in an area has been developed in recent years mainly on the west coast of the United States. In California, over-development of large underground basins together with the massive investments already made in plant and equipment has necessitated the study of conjunctive use of water resources using locally available water resources together with imported water. These studies enabled the formulation of a logical plan of management for the area.

The requirements for such a study are set out by Fowler (Ref. 10). They include an intensive investigation of all aspects and characteristics of the groundwater supply, assessment of the characteristics of all other sources of water and existing distribution and consumption system and finally the development of co-ordinated operation of these supply sources. The co-ordinated operation of water resources requires determination of inflow-outflow equations for surface and sub-surface storages and the determination of supply and utilisation constraints and the costs of operation and maintenance. Fowler notes that one authority must exercise control over a complete hydrologic unit to make practical use of a conjunctive use study. At present, this is a situation not commonly encountered in Australia.

The problem of predicting the operation of a groundwater basin is best tackled by simulation as first described by Tyson and Weber (Ref. 11). For the present state of knowledge in the field of groundwater basins, there are too many variables to model all processes within the basin. Tyson and Weber sub-divided their study basin (about 480 square miles) into approximately 80 Thiessen-type polygonal areas around node points, with the assumption that all activity within the area took place at the node and that all characteristics of the groundwater supply were constant throughout the sub-area. They developed difference-differential equations which predicted changes of level of groundwater at each node for various movements of water. The general form of the equations was as follows:-

$$\sum_i (h_i - h_b) \quad Y_{i,b} = A_b \, S_b \, \frac{dh_b}{dt} + A_b \, Q_b$$

$$\text{and } Y_{i,b} = \frac{J_{i,b} T_{i,b}}{L_{i,b}}$$

where $(h_i - h_b)$ = difference in head between nodes i and b in feet,

$Y_{i,b}$ = conductance of path between nodes i and b in acre feet per year feet,

TABLE 2: ESTIMATED IRRIGATION NEED AT SCONE

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Irrigation Requirements (70% effic.)	Current Area (acres)	Irrigation Requirement (ac. feet)
Mean Rainfall (80 years)	2.89	2.73	2.14	1.78	1.60	1.87	1.68	1.65	1.71	1.93	2.04	2.73	24.75			
Mean Evaporation (10 years)	5.57	4.06	3.94	2.88	2.22	1.60	1.49	2.11	2.68	3.96	5.24	5.81	41.56			
Lucerne																
Req'd Water	4.67	3.27	3.18	2.32	1.72	1.0	1.02	1.70	2.16	3.20	4.40	5.35				
Req'd Irrigation	1.78	0.54	1.04	0.54	0.12			0.05	0.45	1.27	2.36	2.62	10.77	15.40	3,200	4,100
Oats																
Req'd Water			2.76	2.02	1.49	0.96	0.89									
Req'd Irrigation			0.62	0.24									0.86	1.23	2,000	205
Fodder																
Req'd Water	4.06								1.88	2.78	3.82	4.65				
Req'd Irrigation	1.17								0.17	0.85	1.78	1.92	5.89	8.42	550	386
Pasture																
Req'd Water	4.06	2.85	2.76	2.02	1.49	0.96	0.89	1.48	1.88	2.78	3.82	4.65				
Req'd Irrigation	1.17	0.12	0.62	0.24					0.17	0.85	1.78	1.92	6.87	9.82	2,500	2,040
															8,250	6,731

Note: Unless indicated otherwise, all values are in inches of water

Sources: Commonwealth Bureau of Meteorology data
Soil Conservation Station at Scone data
Burton, J.R.: Water Storage on the Farm, Water Research Foundation, Bulletin No. 9, 1965.

A_b = area associated with node b in acres,

S_b = storage coefficient of polygonal zone associated with node b,

Q_b = volumetric flow rate per unit area in acre feet per year acre,

$J_{i,b}$ = length of perpendicular bisector associated with nodes i and b in feet,

$T_{i,b}$ = transmissibility at midpoint between nodes i and b in acre feet per year feet,

$L_{i,b}$ = distance between nodes i and b in feet,

and i represents in turn all nodes adjacent to node b, the node being evaluated.

A typical node point and its associated polygonal area and adjacent node points is shown in Fig. 5.

An electrical analogue computer was used for a trial and error determination of transmissibility and storage coefficients in each area by fitting the predicted water level curves obtained from input of basic historic data to the actual recorded curves. Records for well heights at a large number of wells were available for a period of approximately 30 years. Following fitting of the model, operational studies were made by using relaxation techniques on a digital computer.

The work of Tyson and Weber developed one of the inputs required for a conjunctive use study and details of the conjunctive use model for the California study are given by Chun, Mitchell and Mido (Ref. 12). This paper describes the process outlined by Fowler; namely, the assessment of all sources of supply, the formulation of alternative plans of operation and the estimation of future demands and supply of water. Optimisation was carried out when costs of each plan of operation had been determined. A flow chart of the process is given in Figure 6.

For the study made by Chun, Mitchell and Mido, the alternative plans were formulated to meet an estimated schedule of future water demands from 1964 to 1990. These demands could have been met entirely through an all-surface distribution system or by an all-groundwater system, or by a combination of these.

The costs of each project alternative considered in the study were calculated on an annual basis for the projected time period and were reduced to a total present worth for economic comparison. The plan with least total present worth was selected as the optimum project.

Chun, Mitchell and Mido concluded that provided computing facilities were available to cope with the large volume of computations, the method described should provide much of the data required to solve current water management problems.

In addition to scientific and economic problems in conducting a conjunctive use project, Valantine (Ref. 14) has described the legal and management problems associated with it. He concludes that only by legislation giving overall control of an area to one authority, can effective use be made of the results of a conjunctive use study.

For another California study Dracup (Ref. 15) has used a different approach to that previously described. He depends on the work of Chun, Mitchell and Mido for methodology and some data, but he has extended its application by using linear programming approach to identify the optimum operating plan for the area.

The main difference between this work and Chun, Mitchell and Mido is the method of allocation. Dracup's linear programming approach allowed a true optimum to be located.

Dracup did not formulate a groundwater basin model but instead listed all the variables which affect the volumes of available groundwater and determined an annual value for each year of the 30 year projection period. The variables accounted for in his groundwater basin calculations were as follows:-

Local surface water,

Artificial recharge,

Natural recharge components,

- sub-surface inflow,
- sub-surface outflow,
- extraction by phreatophytes,
- percolation from precipitation,
- percolation from streambeds,
- percolation from irrigation,

Groundwater storage capacity,

Safe yield criteria,

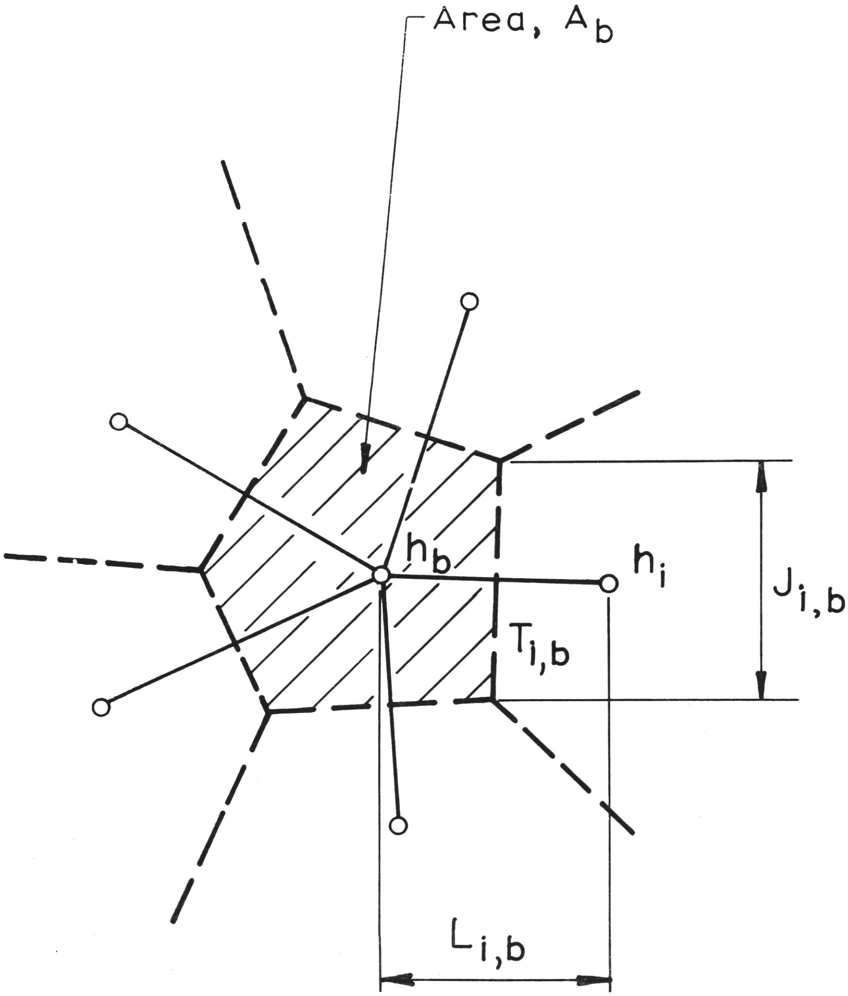

Figure 5: Diagram of typical node point of groundwater basin model used by Tyson and Weber (Ref. 11).

Waste water reclamation.

Some of these were given intrinsic values while others were accounted for as maximum values.

The allocation problem was initially solved as a simple linear programming problem of the general form

$$\text{Minimise } Z = \sum(\Sigma c_j)\, Q_i \qquad 0 < j \leq p, \quad i = 1,2, \ldots m$$

subject to several constraints of the general form,

$$\sum Q_k \geq \sum Q_n \qquad 0 < k \leq p, \quad 0 < n \leq p$$

where Z = total annual cost,

c = cost coefficients in dollars per acre foot, (e.g. cost of importing water from various sources, cost of pumping between various points, etc.)

Q = quantities of water in acre feet per year (e.g. quantities of water used for various activities, quantities of water available from various sources, etc.)

p = number of cost coefficients

m = number of identified quantities of water

The cost coefficients were known and some of the quantities were known and the rest existed as variables.

A different problem in conjunctive use has been studied by Domenico, Schulke and Maxey (Ref. 16), in Nevada. Here a surface supplied area experienced a severe drought and was forced to utilise groundwater supplies whose costs were significantly greater than those of surface supplies. Since the end of the drought, the wells have been used as supplementary supplies late in the season of a normal or near normal period. The study demonstrated that increased development of groundwater in conjunction with surface water could provide more reliable irrigation supplies at a lower cost providing individual control was replaced by centralised control. In the absence of such centralised control, cost of groundwater utilisation could often make crops unprofitable compared with other locations in Nevada. With such control, the authors contend that it would be possible to extend the benefits of conjunctive use of surface and underground supplies of water to virtually all landholders in the valley under study.

This economic study was based on assessment of various physical characteristics of sources of water supply and no attempt was made to model the operation of the groundwater basin. However, studies were made which demonstrated that sufficient groundwater is available for present utilisation. The authors recognised that future development will pose further economic problems when lowering of the water table causes higher pumping lifts. No optimisation was required either, as surface supplies were significantly cheaper than ground supplies but the study showed that reduced costs of utilising groundwater would provide land holders with water supplies that would allow profitable production in most years.

A more theoretical approach has been proposed by Buras (Ref. 17) specifically for the problem of conjunctive use of surface storages and aquifers. He uses a dynamic programming approach to solve the conjunctive use system shown in Figure 7 for the establishment of design criteria and operating rules. This includes design of the optimum size of the surface reservoir and recharge facility in addition to rules for water release from both surface and sub-surface reservoirs. Optimisation is carried out on the basis of maximisation of benefits from irrigation and prevention of floods downstream.

This study is again an economic study of the optimum allocation of several water resources to several demands. No attempt is made to simulate the various hydrologic processes associated with a groundwater basin in a water system model. Many physical processes that would appear in such a model appear as constraints in his conjunctive use model and estimates are made on an annual basis of such variables as sub-surface inflow and maximum recharge rate.

The Department of Water Resources of the State of California has now published the results of its conjunctive use study of the Los Angeles area (Ref. 13). This has been a comprehensive survey using the methodology proposed earlier (Refs. 10, 11, 12 and 14). The final report issued in 1968 was preceded by special studies for the determination of model parameters. Reports on this work were issued progressively as appendices to the final report.

The Los Angeles study was able to draw on large resources of data, manpower and facilities and it has set a pattern and standard for the conjunctive use approach to water resource problems.

5. THE WATER SYSTEM MODEL

A water system model is required to describe the inter-relationships of all water resources in the study area and further, with the conjunctive use model, to enable estimates to be made of the cost of various operating plans. Thus the output from this

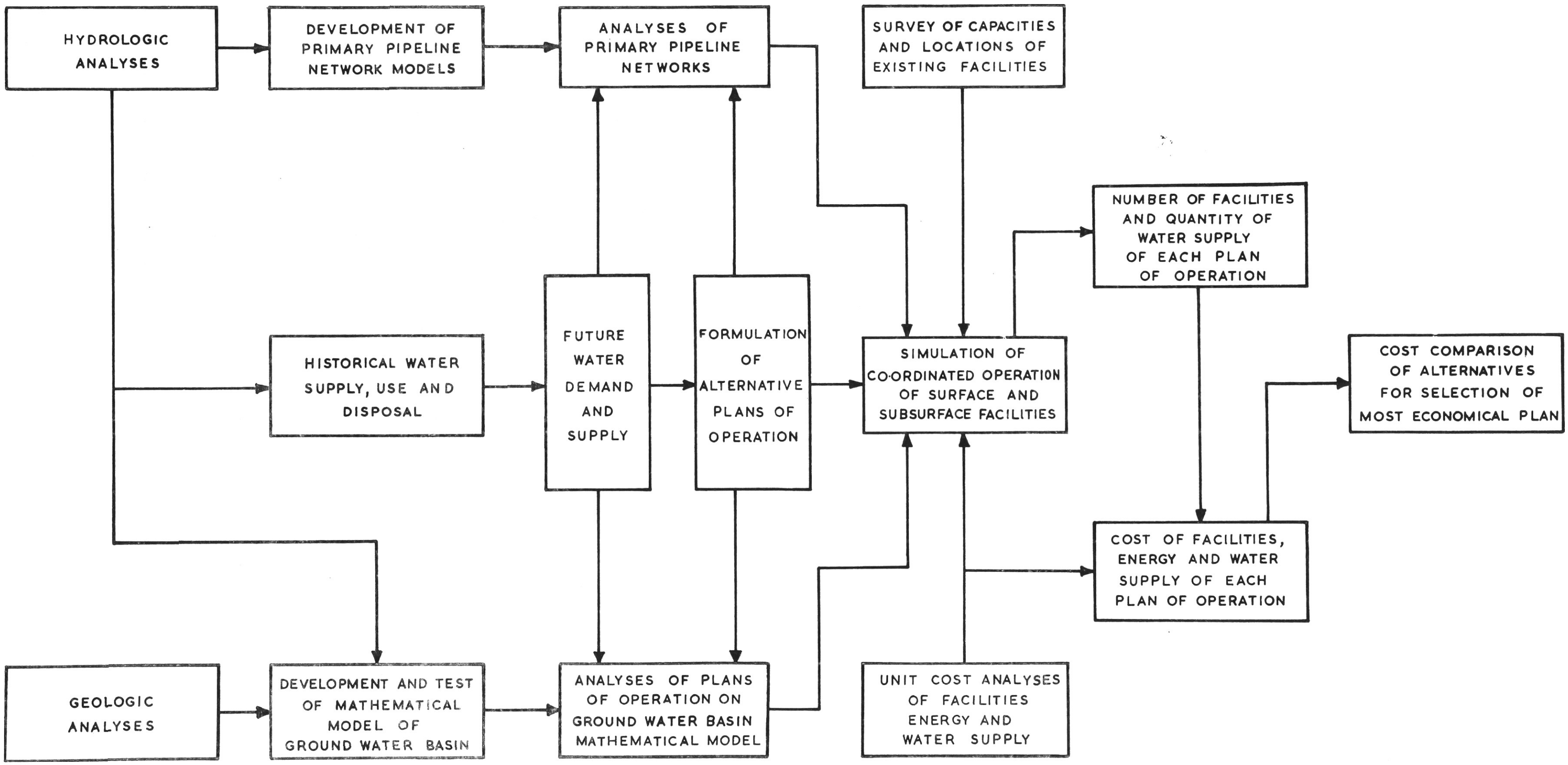

Fig. 6: Flow chart used by Chun, Mitchell and Mido (Ref. 12) for conjunctive use study.

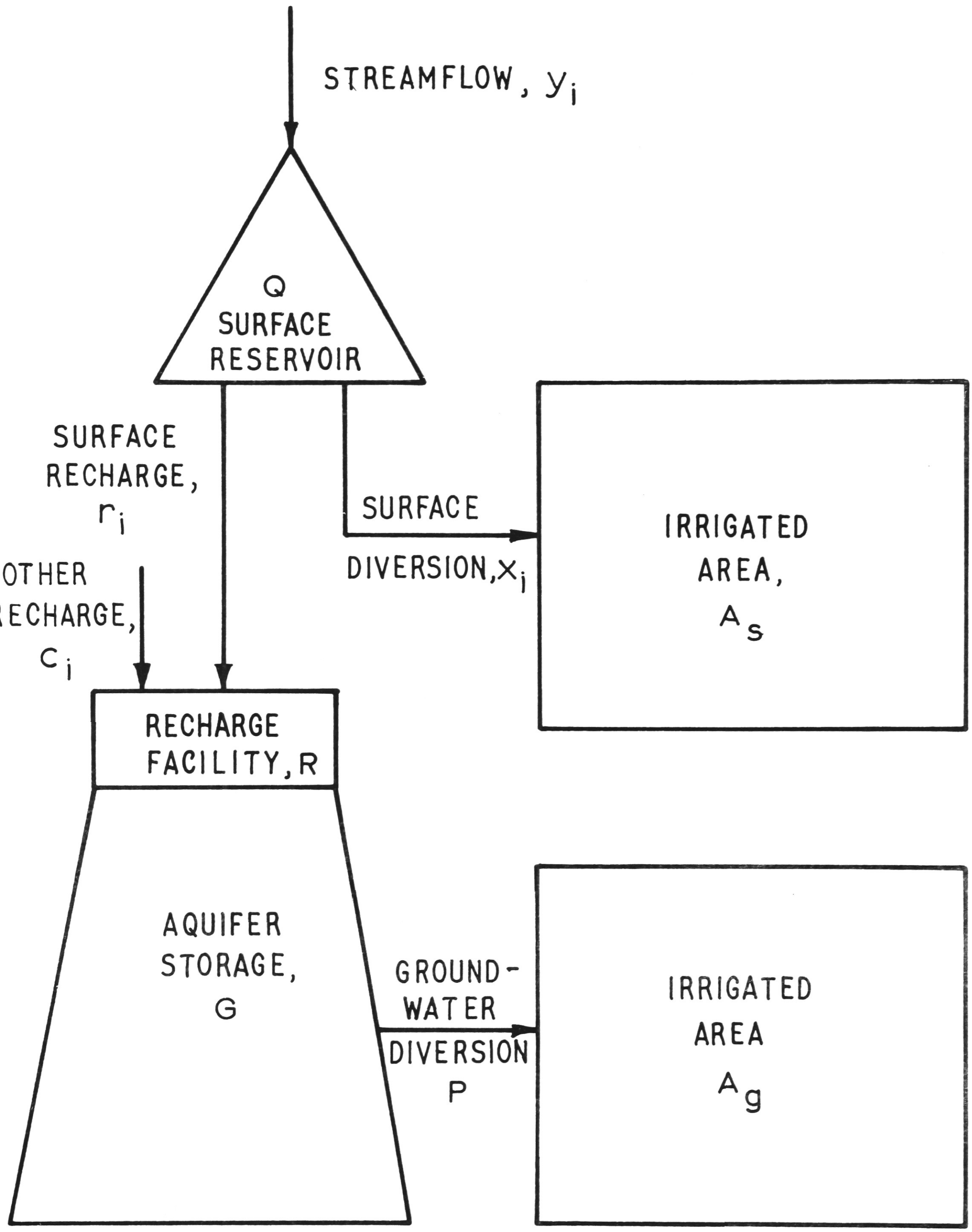

Fig. 7: Flow chart of conjunctive use model used by Buras (Ref. 17).

model will be one of several inputs to the conjunctive use model. A flow diagram of the proposed water system model is shown in Figure 8.

The most suitable application of the model would be to apply it to small subdivisions of the alluvium such that the model parameters are constant for each sub-area. Unfortunately, the availability of data will limit the number of subdivisions for the Kingdon Ponds-Dartbrook study to about five or six. Each area will be defined on the basis of topographical features and will take into account the main concentrations of irrigation. The model will be used to estimate the volume of water stored in the ground and which can be extracted from each sub-area of alluvium. At this stage a monthly accounting procedure is envisaged.

The data requirements for such a model are as follows:

* Precipitation (for each sub-area and on a basin-wide basis)
* Streamflow (at each sub-area)
* Geometry of aquifers
* Location of major recharge areas
* Storage coefficients and transmissibility of aquifers at several sections
* Gradient of water table for each sub-area
* Infiltration characteristics
* Soil moisture changes
* Details of aquifer recharge by percolation
* Evapotranspiration
* Records of water use
* Irrigation requirements

This is an extensive list of requirements and it is likely that approximations for some data will be required. The data presently available is described below.

Precipitation

Details of the location of rain gauges in the area are shown in Figure 3. There are two official rain gauges in the catchment, at Wingen and Scone, both with almost 100 years of records. In addition there are six official rain gauges located just outside the region, all with records of eight years or more. Of these, Aberdeen provides an estimate of rainfall at the lower end of the catchment area and several of the others will be used to estimate rainfall values on the catchment boundaries. In addition to these gauges, several privately owned gauges which have records available for periods between five and ten years have been located.

Streamflow

The only stream gauging station in the catchment is located on Dartbrook at Aberdeen (Fig. 3). Daily records are available for about ten years, and at present it is rated up to 9,500 c.f.s. Prior to the establishment of this station in 1958, two other stations were operated on Dartbrook and Kingdon Ponds upstream of the present station. Each of these controlled about one third of the area of the current station but only five years of records are available at the station on Dartbrook.

It is planned to synthesize additional streamflow records using a rainfall-runoff model similar to that of Boughton (Ref. 18).

Geometry of Aquifers

The water in the alluvium of the valley occurs in a complex pattern of shoestring aquifers (Ref. 19). It is expected that the aquifer geometry will be defined from geophysical investigations now in progress in which seismic and resistivity traverses are being made at several cross-sections in the alluvium. A linear distribution of aquifer properties will be assumed between each section.

Location of Recharge Areas

Initially, recharge from streamflow will be assumed to be uniform along the creek beds in the alluvium. The geophysical exploration together with field inspection is likely to be of value in determining the recharge areas. To date, recharge areas have been indicated in a qualitative manner by plotting variations of water quality in wells in the area (Ref. 19).

It is expected that further estimation of quantities of recharge under various conditions will be available as a residual in a water balance.

Storage Coefficients and Transmissibilty of Aquifers

It is planned to conduct several pumping tests in the area on the cross-sections between adjacent sub areas in order to measure aquifer transmissibility and storage coefficients.

There are some records of water level fluctuations at about five wells at the catchment outlet and at three others near Scone. It is expected that these data will be of use in providing

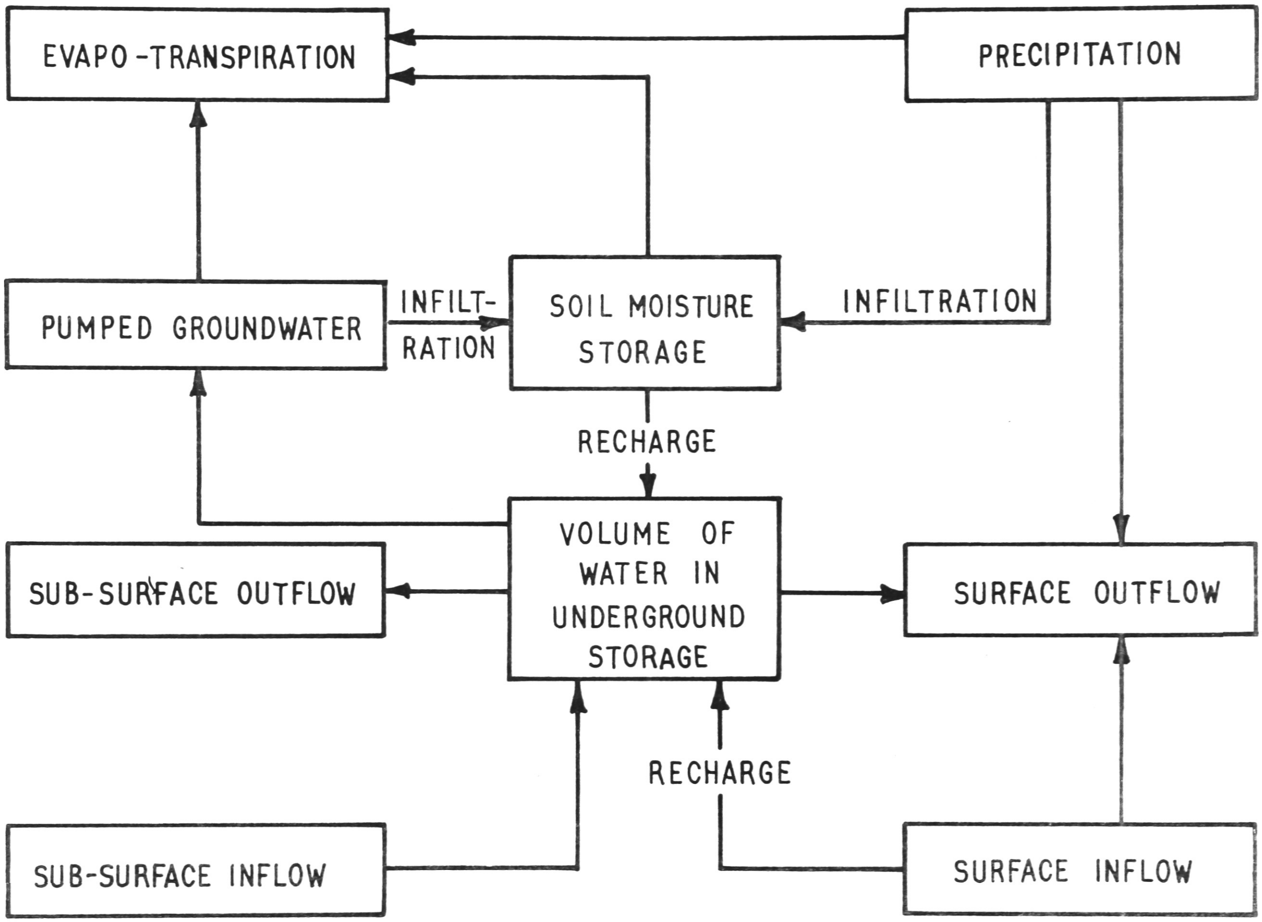

Figure 8: Flow diagram of proposed water system model.

some check on estimated coefficients when the model is being verified. Some data on water levels in wells in the area are now being regularly collected and it is hoped to extend this collection at a later date.

Water Table Gradient

Water table gradients will be required at sub-area boundaries to indicate the direction of and to allow estimation of sub-surface flows across boundaries. Large scale, closely contoured topographic maps of the alluvium are available (Ref. 20) which, together with some observations of water levels in wells these should allow isolines of the water table to be drawn up for various typical conditions. This operation has already been carried out for the area on two previous occasions (Ref. 5 and 6).

Infiltration Characteristics

No attempt has been made as yet to investigate infiltration of moisture into the soil in the study area. No work has been done in this area to the knowledge of the authors and it is expected that the best initial approach will be by the exponential relationships commonly used.

Soil Moisture Changes

It is expected that an observation network will be set up early next year to collect data on soil moisture movements under various climatic and hydrologic conditions using a neutron moisture meter. Further research is required to determine the best technique of using data from this network.

Aquifer Recharge from Percolation

There is no data presently available on this input and it is expected that an allowance for this recharge will be made in trial and error adjustments to the model parameters.

Evapotranspiration

Climatic data in the form of temperature, humidity and wind observations are available at Scone Research Station, just to the east of the catchment for 15 years. These records will be used to determine estimates of evapotranspiration. Frecker (Ref. 21) has determined an empirical relationship for evaporation at any point in the Hunter Valley, based on a study of all evaporation records in the valley.

Water Extraction

Water is pumped from the ground mainly for irrigation and for the municipal water supply to the town of Scone. Most properties also have domestic and stock water supplies but only a small volume of water is used for this purpose.

The predominant method of extracting water is by electric pumps located in wells, although, in some areas, diesel pumps are used. The power supply to the whole area is provided by the Upper Hunter County Council which has made records of quarterly meter readings available. Thus the relative amounts of power consumed in each season are available, and it is planned to relate these power consumptions to the volumes of water used. Although general details of all pumps in the area have been collected, the wide variations in head, pipe lengths, sprinkler types, nozzle sizes, pump brands and sizes, impeller types and varying suction heads make generalisations difficult. However this type of approach has been successfully used by Domenico, Schulke and Maxey (Ref. 16). Williamson (Ref. 6) has also derived a relationship between pump diameter and rate of delivery to irrigation sprays for the study area.

Irrigation Requirements

Estimation of the water required for various crops in differing months and climatic conditions, together with the efficiency of the irrigators plant and technique will be made from standard agricultural works.

6. THE CONJUNCTIVE USE MODEL

A conjunctive use study aims at determining the optimum allocation of water from various sources and has the objective function stated usually in terms of economic benefit such as minimum cost or maximum profit. Various constraints are imposed in the form of maximum volumes of storage and maximum rates of flow. Specific or variable costs and benefits are associated with each of these constraints. Thus the conjunctive use model can take the form of a mathematical programming problem, whose solution is a fairly routine task. It is the determination of the various inputs that presents difficulties.

The formulation of inputs and constraints for this model will depend largely on the results obtained from the operation of the water system model. The conjunctive use model will probably be similar to that used by Dracup (Ref. 15) in that it will be a linear programming allocation model, with constraints in terms of the relationships between various sources of water as determined by the water system model but with the objective function probably stated in terms of benefits to various individuals or various groups.

Physical inputs to the system or physical restrictions will be evaluated in the water system model. The benefit of each unit of water from each source will have to be determined in terms of profitability of that unit of water. This work will allow the benefit function to be set up for maximisation.

Constraints on the function will be maximum quantities of water available from various sources, and maximum rates of water transfer or removal from various sources, as determined for different circumstances by the water system model.

Such a conjunctive use model will allow the optimum use of the existing system to be identified, and it will allow the investigation of several possible methods of increased development of the water resources of the study area, such as:

* head storages for controlled releases in the creeks,
* low weirs to aid recharge from the creek beds,
* low weirs to allow pumping from the creeks,
* small storages on tributary creeks adjacent to the alluvium to supplement underground supplies,
* some form of underground barrier to reduce sub-surface flow leaving the area as a loss,
* providing charges for water to reduce excessive use.

Each such proposal could be evaluated physically in the water system model and then the optimum benefit calculated in the conjunctive use model for comparison with other proposals. Alternatively, several or all of such proposals could be considered together to obtain an overall optimum.

Many facets of this part of the project have yet to be considered, as it is still very much in the formative stages.

7. CONCLUSION

The research project outlined in this paper has been operating for just a few months and few results are available. As the project unfolds there will undoubtedly be changes in the method outlined here,but the objective of optimising the conjunctive use of water resources will remain the same.

The immediate problem is the simulation of the water system of the study basin to provide the necessary input data for the investigation of various plans of operation and the effects of any physical changes in the basin. It will then be possible to derive a suitable economic model, which will probably take the form of a linear programming allocation model. This will be used to identify an optimum management plan for the area under study.

While the proposals are hampered by data deficiencies, the authors recognise that this is a typical situation for much of Australia. It is expected, however, that the research programme will provide sound guidelines for conjunctive use studies in other parts of the Hunter Valley and Australia.

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PAPER No.12

NUCLEAR TECHNIQUES IN GROUNDWATER HYDROLOGY

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1. INTRODUCTION

The tracing of movement in various systems is an obvious and much applied use for radioisotopes; consequently their application to problems in hydrology received attention as soon as artificial radioisotopes became freely available.

However, the existence in Nature of radioactive forms of hydrogen and carbon and two stable isotopes of hydrogen and three of oxygen, increases the number of possible techniques which are available to the hydrologist.

2. STABLE ISOTOPES

The stable isotopes, H (99.985%), D (0.015%), ^{16}O (99.76%) ^{17}O (0.04%) and ^{18}O (0.20%) can form 6 varieties of water but only 3 are in useful concentration $H_2{}^{16}O$, $HD^{16}O$, $H_2{}^{18}O$ with molecular weights of 18, 19 and 20 respectively.

The physical properties of these three forms of water vary slightly and, in particular, their vapour pressures are different, the vapour pressure decreasing with increasing molecular weight. This results in slight fractionation when phase changes occur in water i.e. vapour to liquid or vice versa. This fractionation effect becomes more marked with decreasing temperature and is independent of the relative composition. Thus the ratio (α_D) for D/H in the phase change water to vapour

$$\alpha_D = \frac{HDO/H_2O \text{ (liquid)}}{HDO/H_2O \text{ (vapour)}}$$

is 1.120 at $0^\circ C$ and 1.066 at $30^\circ C$ while the same ratio (α_{18}) for oxygen 18

$$\alpha_{18} = \frac{H_2{}^{18}O/H_2{}^{16}O \text{ liquid}}{H_2{}^{18}O/H_2{}^{16}O \text{ vapour}}$$

is 1.0114 at $0^\circ C$ and 1.0087 at $30^\circ C$. The absolute values of deuterium as HDO and oxygen 18 in oceanic waters are quoted as 158 $\pm$ 2 ppm and 1988.5 $\pm$ 2.5 ppm respectively.

As most water vapour in the atmosphere arises from evaporation over the ocean surfaces, oceanic water can be considered a standard material. Craig (1961) proposed "standard mean ocean water" (SMOW) as a convenient reference level for reporting stable isotope ratios. Deviations from this standard are known as δ values expressed as

$$\delta_D = \frac{D/H\ (\text{sample})}{D/H\ (\text{SMOW})} - 1 \times 10^3\ ^o/oo$$

$$\delta_{18} = \frac{18/16\ (\text{sample})}{18/16\ (\text{SMOW})} - 1 \times 10^3\ ^o/oo$$

Positive δ values denote enrichment and negative values depletion within any one phase.

From this it can be seen that

1) Ocean water is enriched in both isotopes compared to moisture evaporated from it,

2) Precipitation will enrich the water phase relative to the vapour phase.

3) Precipitation over a large land mass will cause increasing depletion of the vapour phase as rainfall drifts inland.

4) Because fractionation increases with decreasing temperature, a latitude effect is observed.

5) Owing to cooling during passage of the moisture-laden atmosphere over mountain ranges a relative enrichment in precipitation will occur with further depletion in the vapour phase.

6) Although the effects are small the large quantities of water in the hydrological cycle are thus characterised by the alteration in their content of stable isotopes.

Analysis of precipitation and waters which have not been subjected to evaporation show a good linear relationship of the general type

$$\delta_D = 8\delta_{18} + y$$

where the slope, 8, of the curve is an important characteristic of natural fractionation processes. Factors which would influence this slope would be a) slow evaporation to give almost equilibrium conditions or b) very fast evaporation where diffusion effects become apparent. During condensation as rain, there are various disturbing factors such as exchange with the vapour phase or evaporation from droplets during precipitation.

Fig. I (Dansgaard, 1964) shows some typical δ_D and δ_{18} values which have been recorded for a few latitudes in Australia.

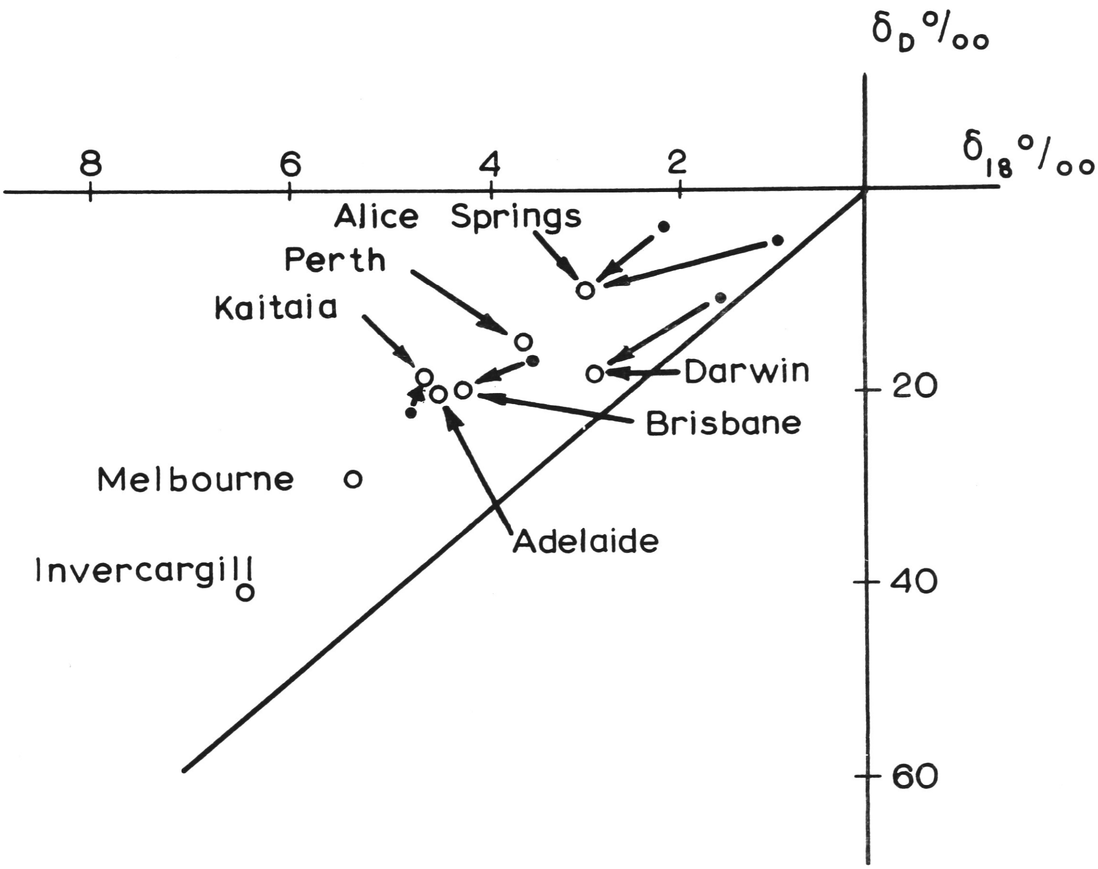

Fig. 1: Typical δ_D and δ_{18} values recorded for several latitudes. Weighted (open circles) and unweighted (dots) means are shown for Australian and New Zealand stations.

It will be apparent from Fig.I that it should be possible to provide some information on the origin of underground waters if only to rule out some possible sources. Considerable information on recharge can therefore be obtained from such observations.

3. TRITIUM AND CARBON 14

Interest in tritium and carbon 14 is mainly centred on the fact that both radioisotopes act as built-in clocks, of half lives 12.26 years and 5568 years respectively. In pre-thermonuclear times, both of these isotopes were produced at a presumably constant rate in the stratosphere and slowly transferred to the troposphere, finally appearing as water molecules and dissolved CO_2 in rain. The concentration of each would therefore be constant. However, if the H_2O and CO_2 were removed from the hydrological and biological cycles then concentrations would decrease due to natural decay and give a measure of their age.

In practice, this age determination is not as simple as would appear from the above. Thus transfer of tritium from stratosphere to troposphere shows seasonal effects so that there is always a Spring and early Summer flush. Estimations in areas of recent recharge might show variations from month to month, a fact that has been used to determine flow velocities. Intermingling of old and newer waters would produce odd results, while very old waters might be subject to small recharge. Carbon 14, in entering the soil, is subject to considerable biological change and, having penetrated below the depth of biological action is then subject to exchange with carbonates in the rock. Finally, the difficulty of measuring what is, in fact one tritium atom in 10^{18} hydrogen atoms, approximately 30 atoms per ml H_2 or 30,000 atoms per ml H_2O, makes the technique tedious and time consuming.

Of particular interest is the large increase in tritium content in rain since the advent of the thermonuclear era. This has resulted in seasonal increases from about 10 - 20 Tritium Units to 3 - 4,000 T.U. being recorded in the northern hemisphere by about 1962, decreasing each year. These increases have provided a unique marker for water precipitated since about 1958. This effect has spread to the southern hemisphere to a small extent, resulting in rainfall figures of up to 150 T.U. approx. instead of the pre-1954 maximum of 10 - 20 T.U. There has been some indication of a slight rise since the French nuclear explosions in the South Pacific but further results are required to confirm this.

4. REACTOR-PRODUCED RADIOISOTOPES

Other available techniques employ reactor produced radioisotopes. In general, ions such as halides move freely in underground water, showing little absorption in soil. Various organic complexes of many metals may also be used, thus providing a wide choice of half-lives and radiation energies. Further work is also being carried out on the use of inactive tracer elements, the estimations being carried out by neutron activation analysis.

5. GROUNDWATER STUDIES USING NATURAL ISOTOPES

Several excellent examples of the use of naturally occurring isotopes are quoted in the literature. In Iceland, for example, ground water provides water supply, a source of hydroelectric power and thermal energy for heating. Owing to the volcanic origin of the area, interconnections exist between ground waters, streams, lakes, etc., which complicate accounting and planning. By regular sampling of precipitation and streams, and reconnaissance sampling of springs and ground waters, contour maps have been made showing average deuterium input in precipitation. Comparison of ground water samples with the maps indicates immediately whether input is local or from distant sources and where recharge has taken place. Tritium estimations in conjunction with the deuterium levels indicate travel time and flow patterns.

Some findings are of interest. Lake Thinquallavatn, the principle source of hydropower, receives substantial recharge via sub-surface flow from a distant glacier - indicated by the 7% depletion in deuterium in the lake water compared to local precipitation. In a typical thermal area, hot springs in uplands agreed with local D determinations whereas hot springs in nearby lowlands were markedly depleted. The tritium levels for lowland springs indicated thermo-nuclear period recharge down to 800 metres. At 1200 m no tritium was detected, thus defining the depth of a superficial circulation system.

In a detailed investigation of the Vienna basin a principal aquifer is about 35 km long by 4 km wide, filled with gravel to 100 - 150 metres. It acts like a pipe, receiving recharge from an upper alluvial fan and discharging water to streams, springs, etc., at its lower end. Analysis of O^{18} content showed the ground waters to be mainly snowmelt from the Alps while T and C^{14} contents were used to confirm other results. It was found that large springs in the Eastern Alps represent a varying blend of recent recharge with an older base flow. The proportions of the two could be calculated by periodic sampling. Thermal springs issuing along border faults showed a three component blend of current year recharge, recharge from post 1965, and old water whose C^{14} content was only 20% of that of modern water.

Comparison of travel time permitted a flow velocity to be calculated which agreed well with estimates from conventional methods.

A similar detailed isotopic investigation of an area on the southwest coast of Jamaica was undertaken to explore possible development of irrigation supplies from an existing aquifer. Local rainfall was low and it was considered that recharge was likely from mountainous inland areas. Isotopic analysis established that the water contained no detectable tritium, and C^{14} measurements indicated that it was at least several thousand years old. A marked depletion in D and O^{18} indicated that the waters dated from a generally colder climatic period. The water was unlike any from modern precipitation and suggested that recharge and circulation were minimal. The proposal to use it for irrigation was therefore abandoned.

6. APPLICATION OF REACTOR PRODUCED RADIOISOTOPES

Many interesting applications of artificial radioisotopes have been described. Among surface water problems, run off, stream flow gauging, use of γ-density gauges in silt burden studies and siltation studies have been widely described. Intake and pattern of flow in lakes, etc. all come under the heading of surface water problems which can be studied by addition of radioactive tracers.

Among ground water problems the rate and direction of flow can be observed by using either a single or multiple well technique. In the single well method a suitable radioisotope is injected into the well, mixed and the rate of disappearance used to determine the velocity of flow through the well. Sections of a bore hole could be blocked off by inflatable plugs to permit studies of laminar flow at various depths. Direction of flow can be determined by using a γ-emitting isotope which would show adsorption in the aquifer. The well is spiked with the isotope and allowed to remain undisturbed until all the isotope has moved into the aquifer, where it is adsorbed. By inserting a rotatable collimated detector into the well the angle of maximum count rate can be determined giving the direction of flow. Using the multiple well technique, a central well is spiked with the radioisotope and the surrounding wells monitored for the appearance of radioactivity.

The specific yield of an aquifer has been determined by injecting a radioisotope into a bore and pumping on an adjacent bore until radioactivity is detected. If the pumping rate and time to first appearance of radioactivity is noted then this volume of water (discharge x time) is equal to πr^2 hy where r = distance from injection to discharge, h = thickness of aquifer (small in relation to r) and y = specific yield.

7. BOREHOLE LOGGING

During the past few years, more interest has been shown in the use of instrumental methods of borehole logging. Most of the instruments employing nuclear techniques have been developed for the oil industry but can be of much use in hydrology. The first is the γ-log measuring the natural radioactivity of the borehole profile for stratigraphic correlation and identification of lithological units. Variations of density in the profile may be obtained using the γ-γ log in which a γ-emitting source in conjunction with a detector adequately shielded from the source are lowered down the borehole at constant rate. The backscattered radiation gives a measure of density, and, if the core constituents are known it can be used to measure porosity. Measurements can be complicated by casing, kind of fluid (air or water) in the hole and by variations in hole diameter. A gamma gamma log which can be pressed against the casing is of some assistance but is not recommended where hole diameter is known to vary. In one case, the effect due to varying hole diameter was used to identify aquifers, because zones with greatly increased hole diameters were commonly aquifers.

Neutron logging would seem to offer the greatest potential in ground water studies because the response is mostly from hydrogen. The probe may be used in cased or uncased holes, in liquid filled, and dry bores. The neutron log can therefore measure porosity in saturated rocks and moisture content in unsaturated rocks. Considerable development of this instrument is being undertaken to improve sensitivity to hydrogen.

A variation of the neutron log to indicate activation, in particular of chlorine, permits definition of zones of marked change in salinity. In this case change of filters over the detector and the use of pulse height analysis will be of considerable value. Within a few years much more intense neutron sources will become available and we can look forward to a new period of advance in neutron logging equipment.

8. PRESENT DEVELOPMENT STAGE

In Australia, facilities for using such techniques have been rather limited. For some of them, either the cost has been too great or experience in large scale applications of radioisotopes has been lacking. It was therefore decided to set up a specific project within the Research Establishment of the A.A.E.C. devoted to the development of nuclear techniques such as have been described and make a detailed assessment of their value in water resources problems. To date, effort has been devoted mainly to the setting up of facilities for the estimation of tritium, oxygen 18 and carbon 14 in natural waters, and it is intended to provide similar facilities for deuterium. While progress is being made, it should be emphasised that estimations at these very low levels of concentration are difficult and time consuming.

Some preliminary attempts are being made to study the exchange of carbon 14 with typical carbonates as found in rocks, in order to test the reliability of the dating method in water. Further development of field trials using artificial radioisotopes as tracers will remain an integral part of the programme.

The major contribution which the A.A.E.C. can make in this field is in the development and assistance in use of hydrological techniques which are basically nuclear or which make particular demands on the special resources of the A.A.E.C. The A.A.E.C. cannot undertake a comprehensive hydrological programme on its own initiative. This is the responsibility of other Federal or State Government Departments. Neither is it considered that nuclear techniques could supplant conventional methods. They may assist, however, in the further elucidation of problems by providing confirmatory evidence to test a hypothesis. In only a few cases will their application give unique information.

Little of value would be obtained from a programme such as we envisage without the support and advice of the appropriate University, State and Federal bodies whose specific interest is development and conservation of water resources. We would hope that such collaboration will be readily forthcoming.

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