GEOLOGICAL SURVEY
OF
WESTERN AUSTRALIA

REPORT 2

A REAPPRAISAL OF THE YULE RIVER AREA: PORT HEDLAND TOWN WATER SUPPLY, AND AN APPRAISAL OF THE EFFECTS OF LONG-TERM PUMPING IN THE LAKE ALLANOOKA AREA

by J. R. Forth

1972

FOREWORD

Both of these reports concern water supplies for important regional centres, namely Port Hedland and Geraldton.

Earlier work by the Geological Survey, in co-operation with the Public Works Department, had located and delimited underground water supplies of suitable quality and quantity for the purpose by geological considerations and exploratory bores.

The areas were then developed and subsequent observation of the effects over limited periods of withdrawal of water has enabled the author to calculate permissible yields over longer periods of time.

An early appreciation of the effects of long-term pumping is most important. Withdrawal of water at an excessive rate would not only deplete the reserves (a relatively short-term problem) but also result in contamination of the aquifers by non-potable water from adjoining water provinces, resulting in semi-permanent spoiling of the supply.

As further groundwater development progresses, it may be possible to reassess the likely effects, but in the meantime the importance of operating within the limits set out in these reports cannot be over emphasized.

J. H. Lord,
DIRECTOR

November, 1972

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WESTERN AUSTRALIA

REPORT 2

A REAPPRAISAL OF THE YULE RIVER AREA: PORT HEDLAND TOWN WATER SUPPLY, PAGES 3 TO 34 AND AN APPRAISAL OF THE EFFECTS OF LONG-TERM PUMPING IN THE LAKE ALLANOOKA AREA, PAGES 35 TO 62

by J. R. Forth

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A REAPPRAISAL OF THE YULE RIVER AREA: PORT HEDLAND TOWN WATER SUPPLY

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A REAPPRAISAL OF THE YULE RIVER AREA: PORT HEDLAND TOWN WATER SUPPLY

SUMMARY

This report is a reappraisal of the effects of pumping in the Yule area. Estimates were primarily based upon geological considerations and a flow-net analysis, because insufficient hydraulic data were available. A pumping rate of 4 x 10^6 gpd (18 x 10^3 m^3/d) could be safely maintained for 12 months in 1972/73, and a long-term rate of 1.9 x 10^6 gpd (8.6 x 10^3 m^3/d) could be maintained, without undue mining of reserves, if the pumping were spread over a sufficiently large area. No positive conclusions were reached as to the effects of a long-term rate of 2.5 x 10^6 gpd (11.3 x 10^3 m^3/d), although it may be possible to maintain this rate.

DISCUSSION OF METHOD

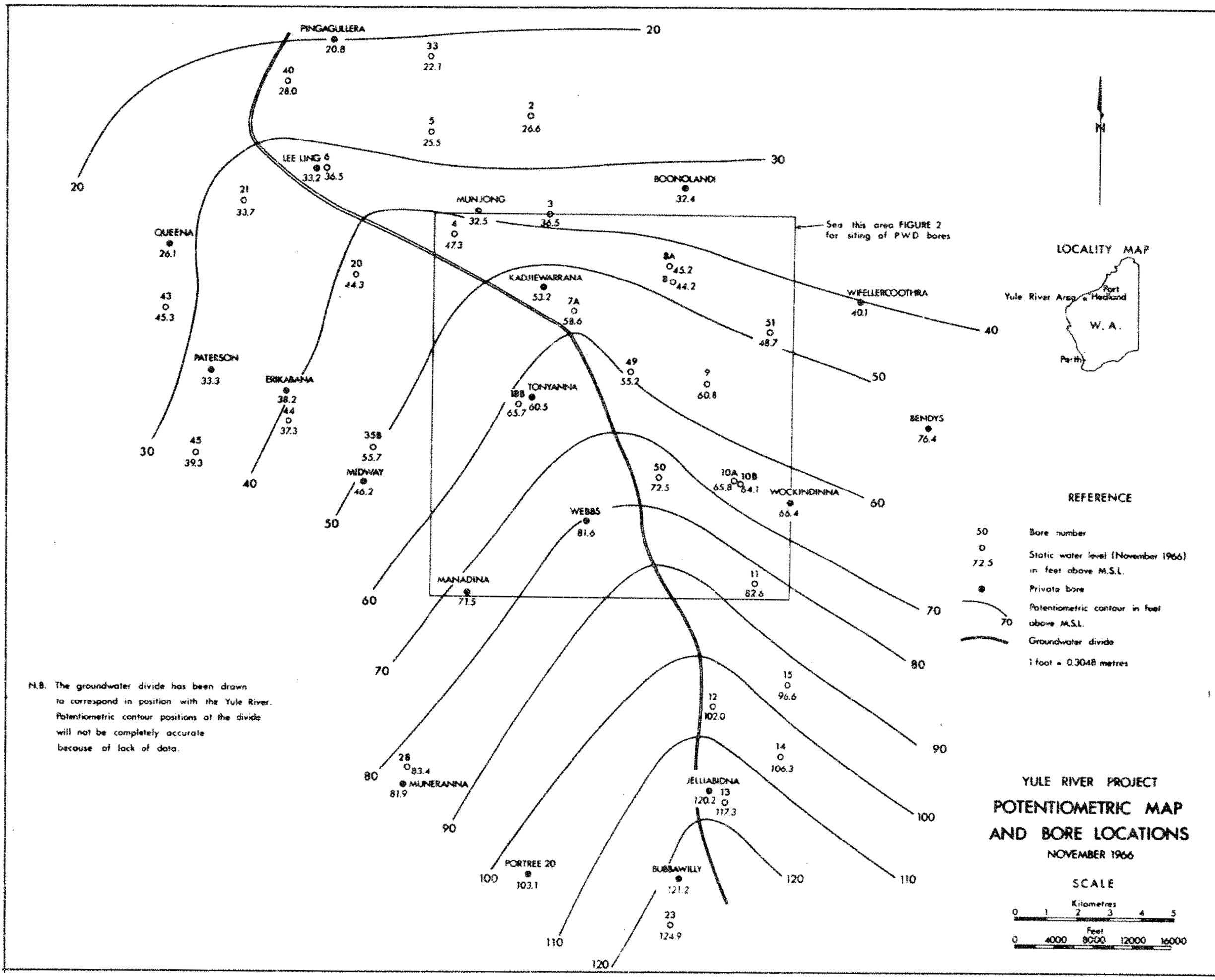
At the Yule River, information is required on the amount of water that can be pumped and where it can be pumped from, and on the behaviour of the individual bores. For these purposes the whole aquifer system must be understood and details of its hydraulic properties must be known. If the aquifer system were uniform, it would only be necessary to establish its boundaries by geological techniques and then to make a single pump test to determine the hydraulic properties of the system. It is clear, from the data collected, that the aquifer system has very widely varying properties, and it will not be possible to reach a simple solution to any of the questions asked.

The most common aquifer test (pumping test), upon analysis, gives a set of hydraulic coefficients which approximately represent the characteristics of the aquifer in the vicinity of the pumped bore. The analysis methods used to derive these properties assume ideal conditions e.g. isotropy and homogeneity. When the derived coefficients are applied to the complete flow system, the same assumptions must necessarily be made. In a valley of river-deposited sediments as at the Yule River, homogeneity can be assumed to apply to the whole system, but total isotropy cannot be expected to apply. The common analysis techniques available for the general solution of flow problems do not allow for solution for anisotropy and we must assume that over the whole system flow will, in the regional sense, be isotropic.

For the Yule River aquifer system, transmissivity is the fundamental unknown variable in terms of the regional flow system. Its variation has been studied by making certain assumptions, such as isotropy and homogeneity. A close examination of the flow net will show that the assumption of perfect isotropy has not been rigidly adhered to. Flow lines and isopotentials have been made to cross as nearly as possible at right angles, but rigid adherence to this principle would have led to unreasonable distortion in some of the flow lines where drawn between adjacent isopotentials. In some cases the problem is not one of anisotropy but is due to slight inaccuracies in the potentiometric map which cannot be defined to perfection. This does not mean that the flow net is inherently incorrect. In the regional sense it is thought to reasonably represent variations in transmissivity. Comparison of the flow net to the isopachs of saturated aquifer thickness, to the pump test results, and to the basement contours, shows that the data all fit together and that the net, although generalized, is probably a good regional representation of the variation in aquifer transmissivity.

INTRODUCTION

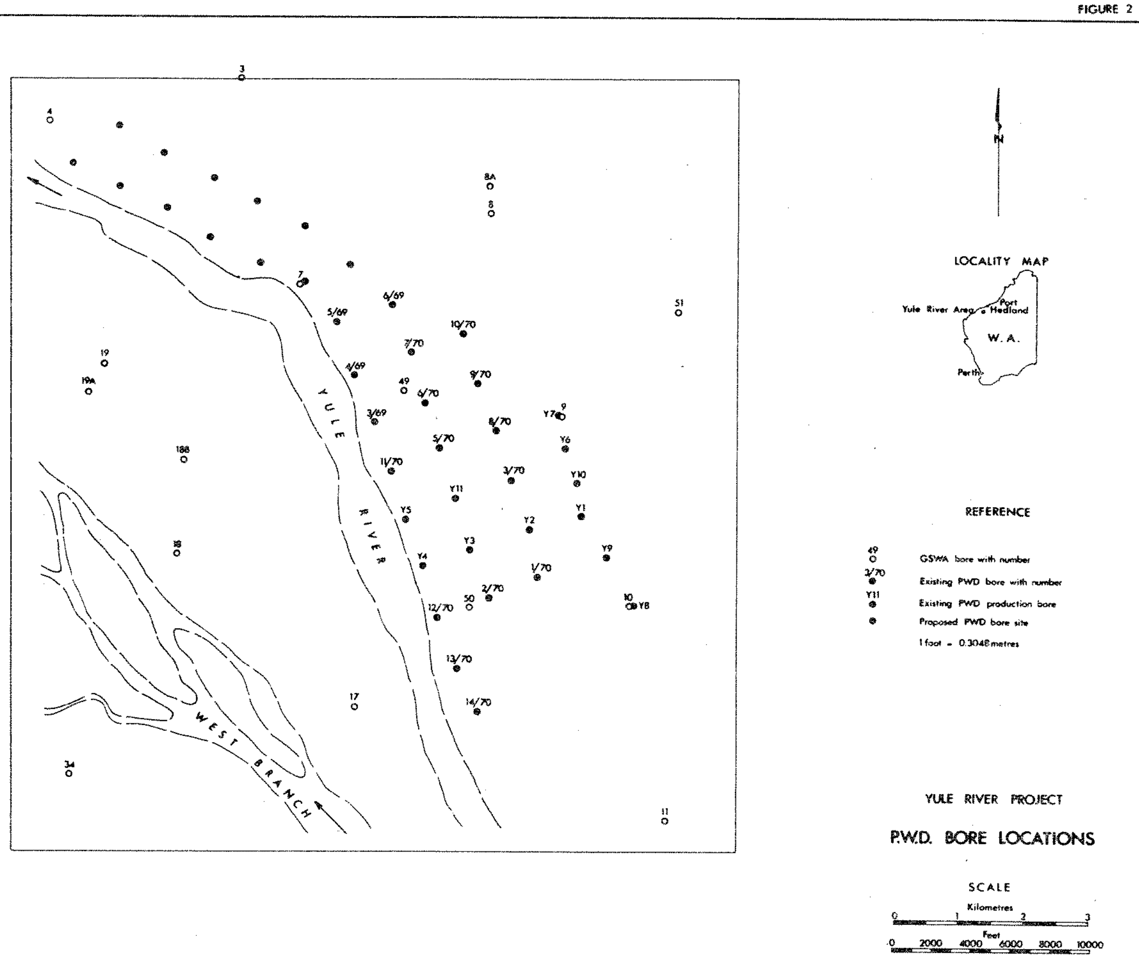
In 1965/66 the Geological Survey of Western Australia made a hydrogeological investigation of the Yule River alluvial valley, 42 miles (67.6 km) west of Port Hedland. The results of the investigation were presented by Whincup (1967). The Public Works Department (P.W.D.) has since installed production bores. Bore locations are shown in Figures 1 and 2.



In 1971 the P.W.D. requested the Geological Survey to re-assess the area in terms of the following production criteria:

1. Continuation of the present production rate of 1.9×10^6 gpd (8.6×10^3 m^3/d).
2. An anticipated future production rate of 2.5×10^6 gpd (11.3×10^3 m^3/d).
3. Production at a rate of 4×10^6 gpd (18×10^3 m^3/d) for 12 months in the years 1972/73.

FIGURE 2



YULE RIVER PROJECT

P.W.D. BORE LOCATIONS

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DATA

Sources of information for this re-assessment were:

1. Data collected during the G.S.W.A. 1965/66 investigation.
2. Water level observations made since.
3. Results of P.W.D. production bore drilling and testing.
4. Routine drawdown and recovery observations in all production bores (Figs. 3 and 4).
5. P.W.D. records of water pumped for supply to Port Hedland.

The Yule River area has not been visited by the writer, but Dr. A. D. Allen who participated in the original field investigation has verbally given much information and helpful comment for this report.

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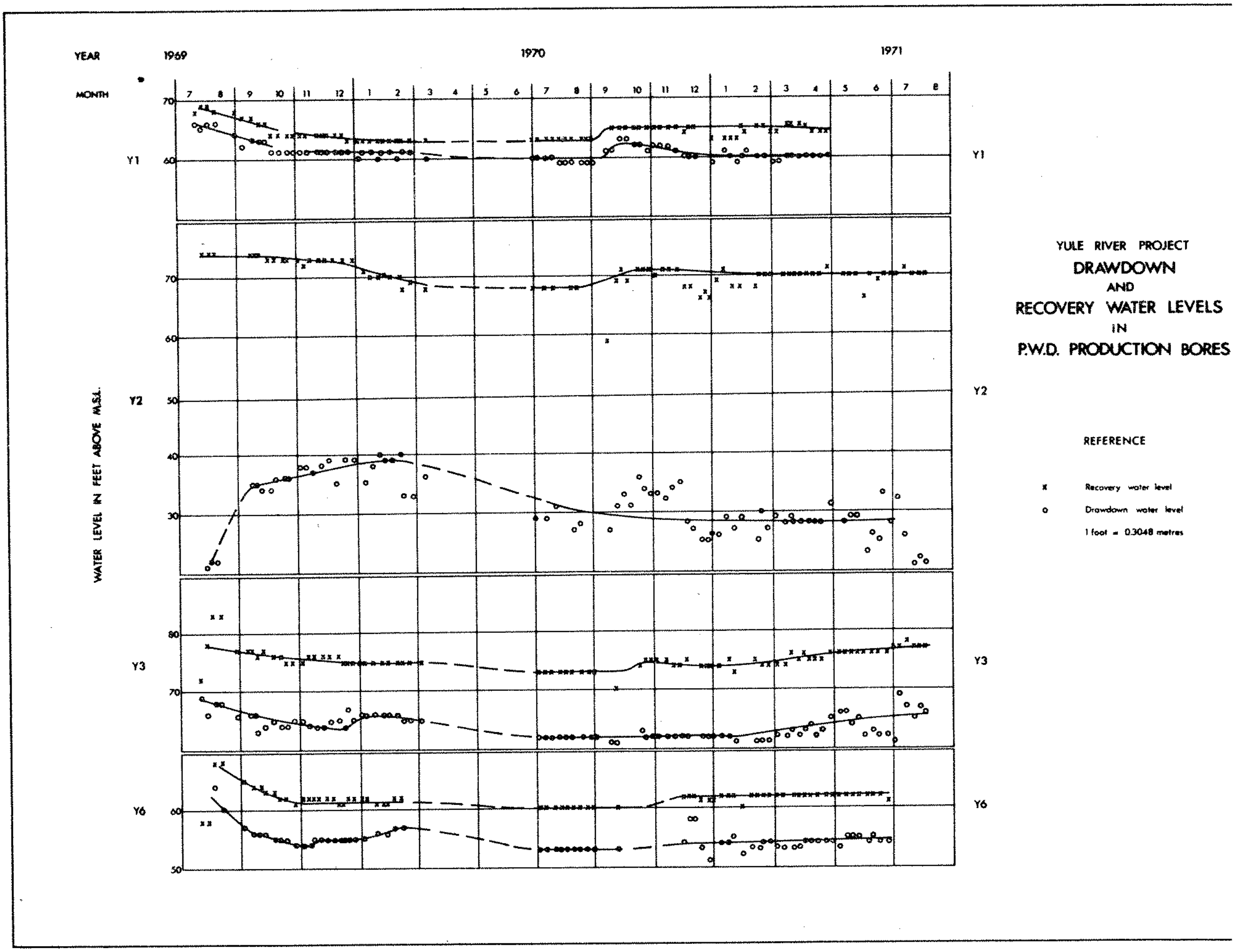
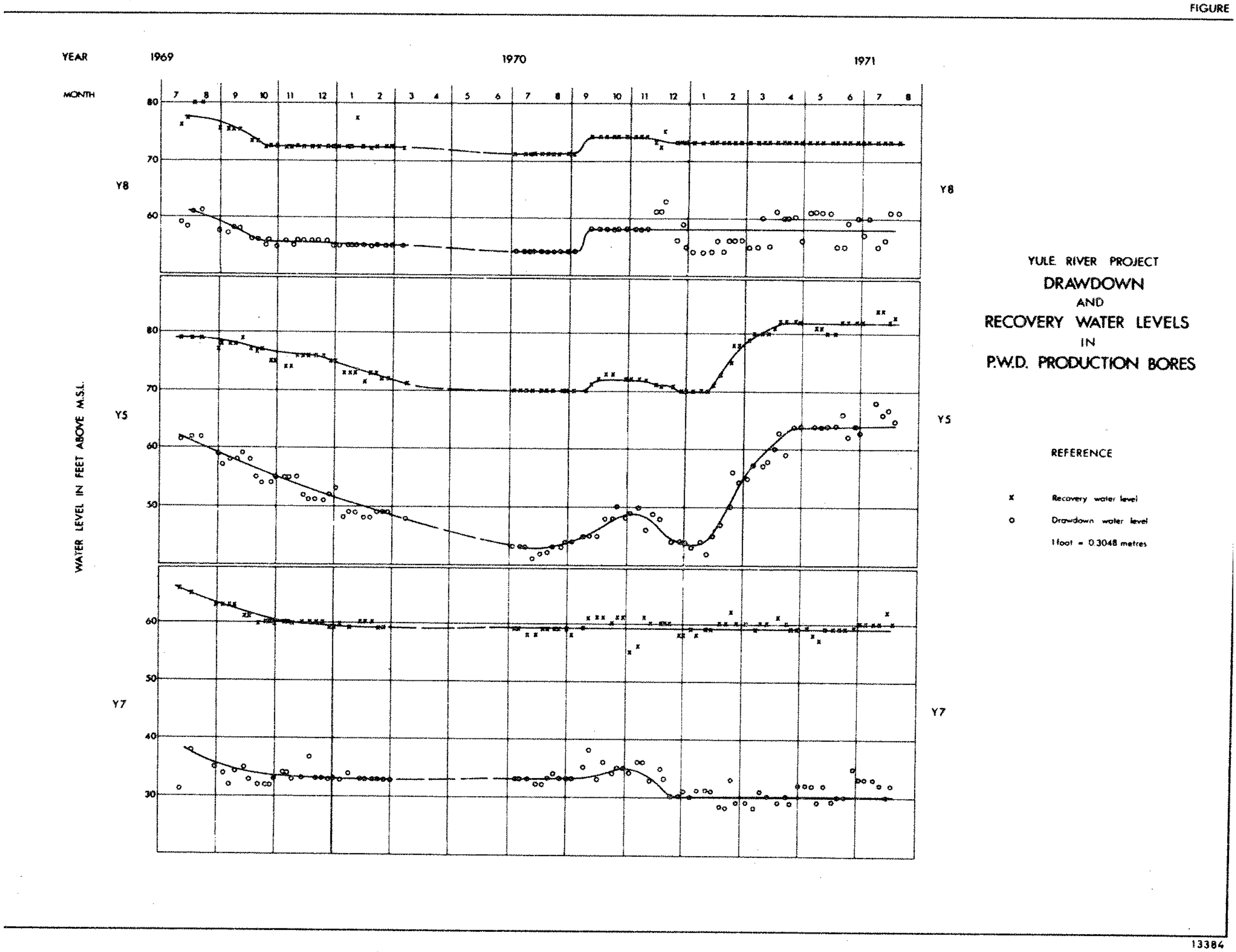




PREVIOUS WORK

Whincup (1967) described the geology and structure of the area. From pump tests on some exploration bores he derived hydraulic properties for the aquifers and estimated that a production rate of 0.8 x 10^6 gpd (3.64 x 10^3 m^3/d) could be maintained. He assumed that aquifer recharge was from river flooding with a 3.5-year return period.

GEOLOGY

The area is a silty flood plain of Quaternary alluvial sediments overlying Archaean granite, schist, gneiss, diorite, quartzite and siltstone.

The Quaternary sediments comprise a sequence of clay, silt, sand and gravel with minor calcareous, siliceous and ferruginous beds, having a thickness of 55 feet (17 m) in the south and more than 230 feet (70 m) in the north, occurring in the following general sequence.

A thin layer of coarse surface sands is underlain by sandy silt and clay to a depth of 30 to 50 feet (9 to 15 m). Poorly sorted silty sand and gravel from 50 to 60 feet (15 to 18 m) are invariably at or below water table. The gravels are lenticular and vary widely. From 60 to 110 feet (18 to 33 m) there are silts and clays which may be slightly sandy. These overlie a second sandy and gravelly sequence from 110 to 130 feet (33 to 40 m). Sandy clay and silt underlie this second gravelly section to a depth of approximately 180 feet (55 m). Below this, bores in the northeast intersected hard kaolinitic claystone overlying laterite which may be the remnant of a Tertiary lateritized surface.

The above sequence lies unconformably upon the Archaean bed-rock, which showed wide variation in degree of weathering.

Aquifers in the alluvium may be approximately related to the two sandy gravelly layers. Water was also encountered in the bed-rock in some bores.

WATER PRODUCTION

Table 1 shows monthly production from the P.W.D. bores (Fig. 2).

TABLE 1. Monthly Pumpage In 10^6 Gallons

MONTH	VOLUME			
	1968	1969	1970	1971
January		19.430	31.989	51.102
February		16.777	34.989	38.003
March		24.543	27.090	46.095
April		31.604	33.447	47.410
May		27.996	33.268	36.984
June		25.498	28.855	35.520
July		19.940	30.246	
August		19.884	30.458	
September	7.187*	28.860	38.150	
October	18.269	26.449	50.232	
November	19.938	32.084	45.991	
December	16.717	30.714	50.944	
	62.111	303.779	435.659	255.114
Rate (mgd)	0.598	0.831	1.194	1.410

* September 1968 excluded from rate calculations because pumpage was for part of the month only

1 million gallons = 4,546 m^3

The figures show a steadily increasing rate of pumping with an average rate for the first 12 full months of 0.72×10^6 gpd (3.27×10^3 m^3/d) and for the last 12 months of 1.38×10^6 gpd (6.27×10^3 m^3/d). Total production to date is $1{,}057 \times 10^6$ gallons (4.8×10^6 m^3).

WATER LEVEL RECORDS

At the conclusion of its exploration programme the Geological Survey requested th Public Works Department to make monthly water level readings at specified observation bores in the area. Readings were made in July, September, and October 1967 and were then discontinued. At the request of the writer the Public Works Department made one further set of observations in August 1971 (Table 2). Whincup (1967) has tabulated all earlier readings.

TABLE 2. Water Level Observations

BORE	July 1967	September 1967	October 1967	August 1971
2	27.04	26.93	26.89	26.52
3	37.66	37.68	37.72	
4	48.03	47.04	46.50	
6	37.66	37.08	36.79	37.27
7A	59.72	57.99	57.30	62.31
8				44.19
8A	43.38	35.34	48.46	
8B	43.80	43.73	43.84	
9	60.41	60.20	60.41	55.21
10A	66.22	66.11	66.13	62.38
10B	64.90	64.82	64.85	60.10
11	82.66	82.02	81.90	
12	103.52	103.48	103.38	104.82
13	112.62	116.13	115.86	118.22
14	106.28	106.07	106.12	
15	96.71	96.32	96.30	
18B	66.04			
20	45.92			44.92
21	33.80			
23				124.10
28				82.02
33	22.39	22.31	22.27	
35B	54.82		54.42	
40	28.99	28.38	28.12	
43	43.69		43.40	
44	38.69		38.27	
45	39.30		38.95	
49	55.70	55.48	55.33	
50	74.10	73.84	73.71	
51	50.60	50.40	50.34	

All readings in feet above M.S.L.
1 foot = 0.3048 metre

This discontinuation in the requested observation programme has made it very difficult to assess the effects of past pumpage.

The Public Works Department make weekly drawdown and recovery observations in all production bores (Figs. 3 and 4)

SURVEY DATUM LEVELS

Three datum levels have been used for surveying in the area:

1. Admiralty Chart Datum (A.C.D.);
2. Low Water Mark Fremantle (L.W.M.F.);
3. State Mean Sea Level (M.S.L.)

All records in this report are quoted in terms of the State Mean Sea Level datum.

The relationship between the three is:

1. Admiralty Chart Datum is 10.36 feet (3.16 m) below State Mean Sea Level;
2. Low Water Mark Fremantle is 2.48 feet (0.76 m) below State Mean Sea Level.

Table 3 lists certain pertinent bore information in terms of State Mean Seal Level.

TABLE 3. Reduced Levels in Feet above M.S.L. for Tops of Casing

BORE	LEVEL	BORE	LEVEL
1	46.65	23	152.89
2	60.85	24	131.90
3	73.43		
4	70.98	25	138.72
5	57.61	26	127.16
6	54.66	27	121.39
7	89.75	28	110.86
7A	88.49*	29	101.21
8	78.82	30	111.32
8A	78.98	31	109.28
8B	77.50*	32	47.78
9	92.89	33	47.39
10	98.83	34	97.52
10A	98.88*		
10B	98.30*	35B	83.07
11	114.58	36	90.81
12	130.40	37	84.06
13	143.48	38	74.40
14	135.28	39	57.05
15	129.00	40	43.84
16	116.34	41	49.15
17	104.68	42	42.47
18	90.91	43	67.77
18B	86.125	44	71.27
19	80.69	45	72.65
19A	80.55	46	92.74
20	66.80	47	102.38
21	53.51	48	132.38
22	142.23	49	85.10*
		50	98.50*
		51	85.50*

* Not surveyed accurately

P.W.D. Production Bores

BORE	LEVEL	R.L. BOTTOM OF AIR LINE
Y 1=A=1/67	100.0	37.7
Y 2=B=2/67	101.8	5.6
Y 3=C=3/67	102.4	61.2
Y 4=D=2/68	107.4	–
Y 5=E=5/67	103.9	22.5
Y 6= 9/67	95.9	19.5
Y 7= 7/67	94.4	12.9
Y 8=T=1/68	107.2	45.8
Y 9=S=1/69	104.4	33.0
Y10=Z=2/69	102.2	25.8
Y11=N=4/70	99.9	28.5

1 foot = 0.3048 metre

REASSESSMENT

A reassessment was required in terms of three specified pump rates:

1. 1.9×10^6 gpd, (8.6×10^3 m^3/d);
2. 2.5×10^6 gpd, (11.3×10^3 m^3/d);
3. 4.0×10^6 gpd, (18×10^3 m^3/d).

Attempts were therefore made to derive a mathematical model equivalent to the aquifers. For this purpose the structure, geology and hydraulic properties of the area were re-examined in detail. Because of the wide variation in some properties, and also because of insufficient information on some aspects, it was not possible to derive a simple mathematical model which could be used for forecasting. By graphical flow-net analysis however, it has been possible to make some estimate of the possible effects of pumping.

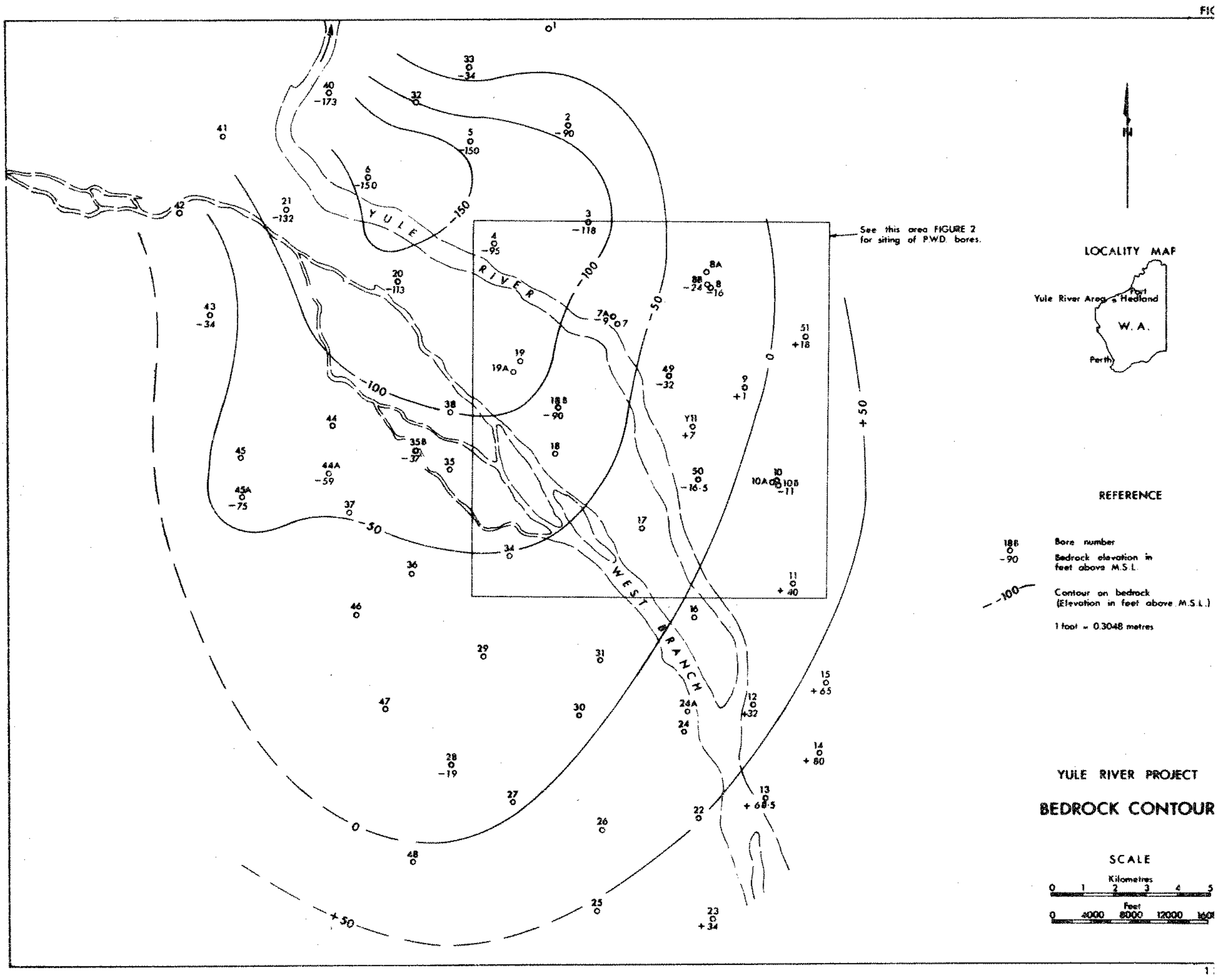
STRUCTURE

Depths to bed-rock were taken from bore logs, and a contour map (Fig. 5) of the Archaean surface was drawn. The contours indicate a rectangular valley with a northwesterly axis. The present course of the Yule River is very close to the original valley axis.

In such an infilled valley environment, the best prospects for obtaining water supplies will be in the coarse sands and gravels of old river channels. Silts and fine-grained sediments will predominate near the valley sides (where the sedimentary sequences will also be thinner) and prospects for good bores will be poor.

The valley floor is of fairly regular shape, with two areas of slight deepening where prospects for water production may be enhanced. The first passes from bores 49 and 8 through bores 5 and 6 to bore 40. The second extends northeasterly from bore 36 to bore 20 and on to meet the first zone near bore 6.

These two zones lie in the deepest parts of the valley, where the coarsest sediments are most likely to be found.



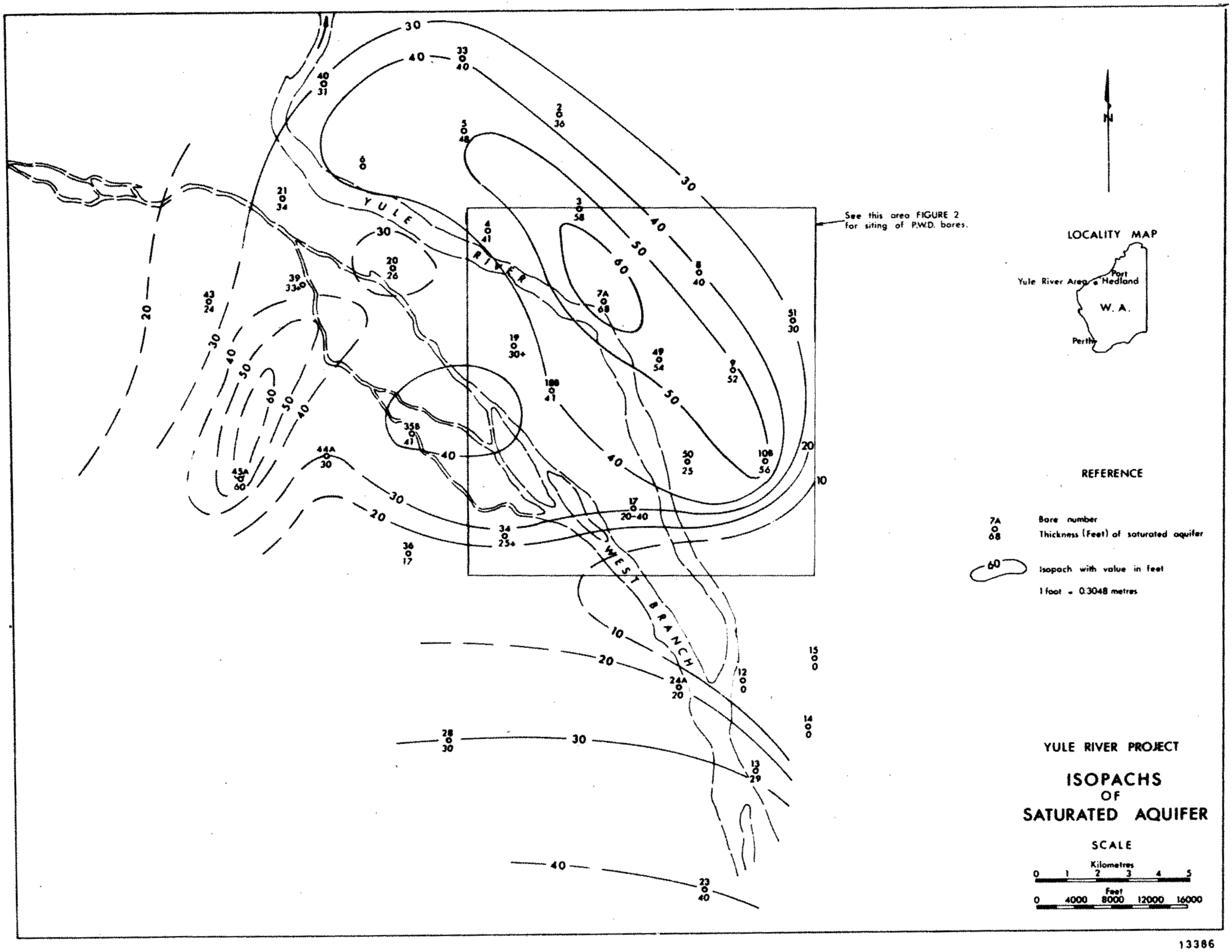
GROUNDWATER OCCURRENCE

Bore logs show no general relationship between lithology and groundwater occurrence.

Total saturated aquifer thicknesses (where multiple aquifers were noted, the thicknesses were summed) were determined, and the isopach pattern (Fig. 6) was found to fit well with the contours of the Archaean basement.

Aquifer thicknesses are especially marked in three areas. Two correspond to the two zones of valley deepening indicated by the basement contours and the third area of thickening is between bores 39 and 45A.

The information is sparse but there are indications that thicker aquifers may be found near bore 23.



POTENTIOMETRIC MAP

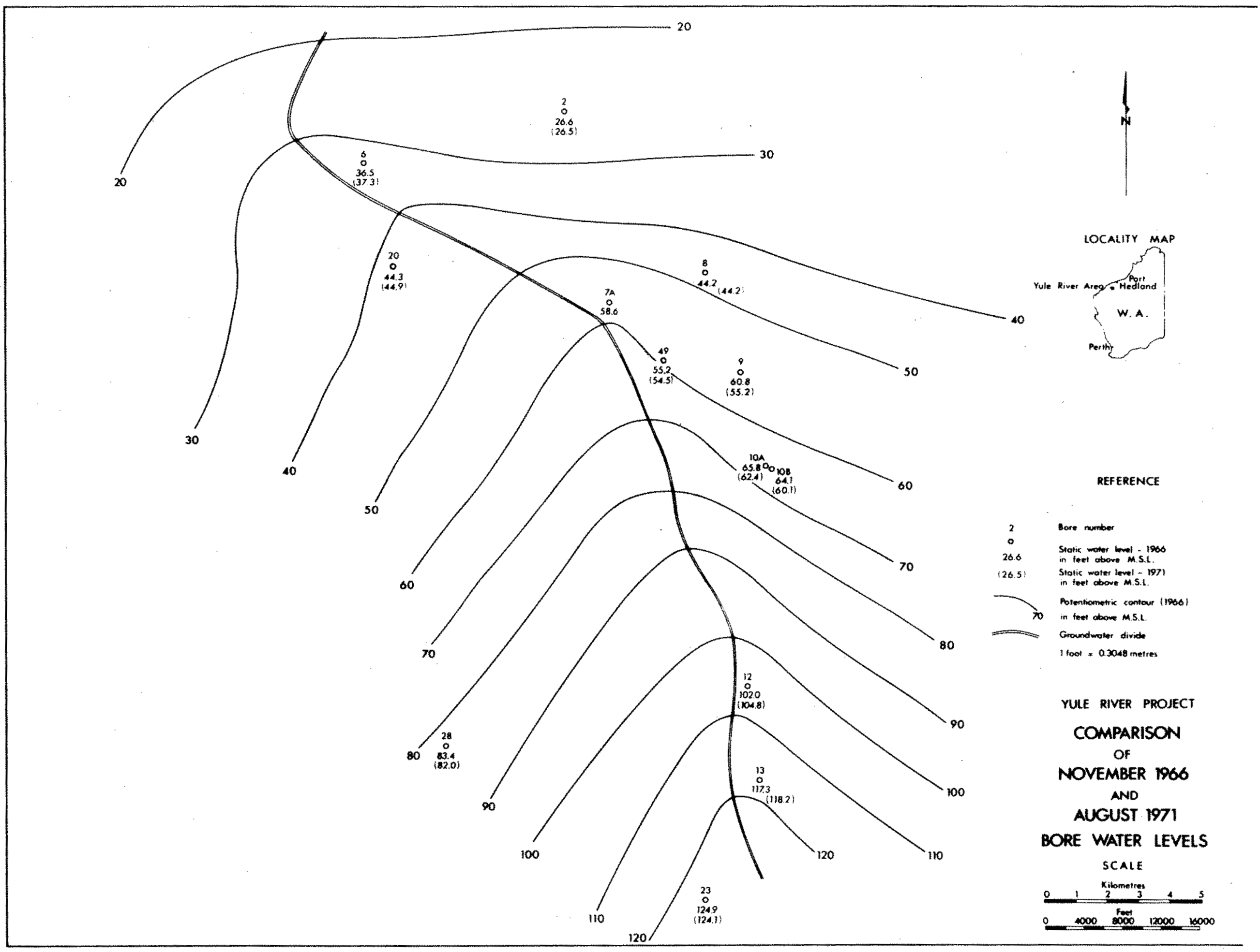
From the various sets of bore water-level readings, those of November 1966 were chosen for the preparation of a potentiometric map (Fig. 1). This particular set of readings included most of the G.S.W.A. and many of the private bores.

The map shows a clear pattern of aquifer recharge from the river, the groundwater divide corresponding to the course of the river bed. There is no indication of recharge from the West Branch (shown on Fig. 5) except near bore 21, where the mound of recharge appears to be much broader than elswhere.

The potentiometric contours indicate river recharge to the aquifers throughout the length of the investigated area. Bores have been screened at widely varying depths and there is apparently good hydraulic continuity throughout the aquifer system.

Recharge is uniform to the east and west of the river. Underflow caused by recharge is to the north and to the northwest beyond the area investigated. There is no evidence of any substantial underflow into the area from the south.

FIG



On Figure 7, bore water levels taken in August 1971 are compared with readings made in November 1966. The 1966 equipotentials are also shown. River flows in June 1971 will have recharged the aquifers before the readings were taken in August, and bores 6, 7A, 12, and 13, all close to the river, show higher water levels than the November 1966 readings. Farther from the river the 1971 levels are similar to or lower than the 1966 readings. Bores 49, 9, 10A, and 10B are lower by up to 5.6 feet (1.8 m). Readings from these bores were probably made while pumping was in progress in the production bores (also shown). Recovery water levels in the production bores are given on Figures 3 and 4 and indicate that the observed levels from the exploration bores are too low.

If the recharge process follows the pattern described in a later section, direct rainfall recharge has not yet reached the aquifers and further rises in bore water levels should have occurred in 1971 with a peak in September or October. At the conclusion of the 1971 recharge, water levels over the whole area should probably have been higher than in November 1966, but the increase in water levels would not be large and it is thought that if sufficient readings were available for a potentiometric map to be drawn for the 1971 recharge event, it would not differ substantially from Figure 1.

AQUIFER HYDRAULICS

Many of the bores have been pump-tested by the drawdown and recovery methods, and two controlled tests have been made using bores Y8, 10A and 10B.

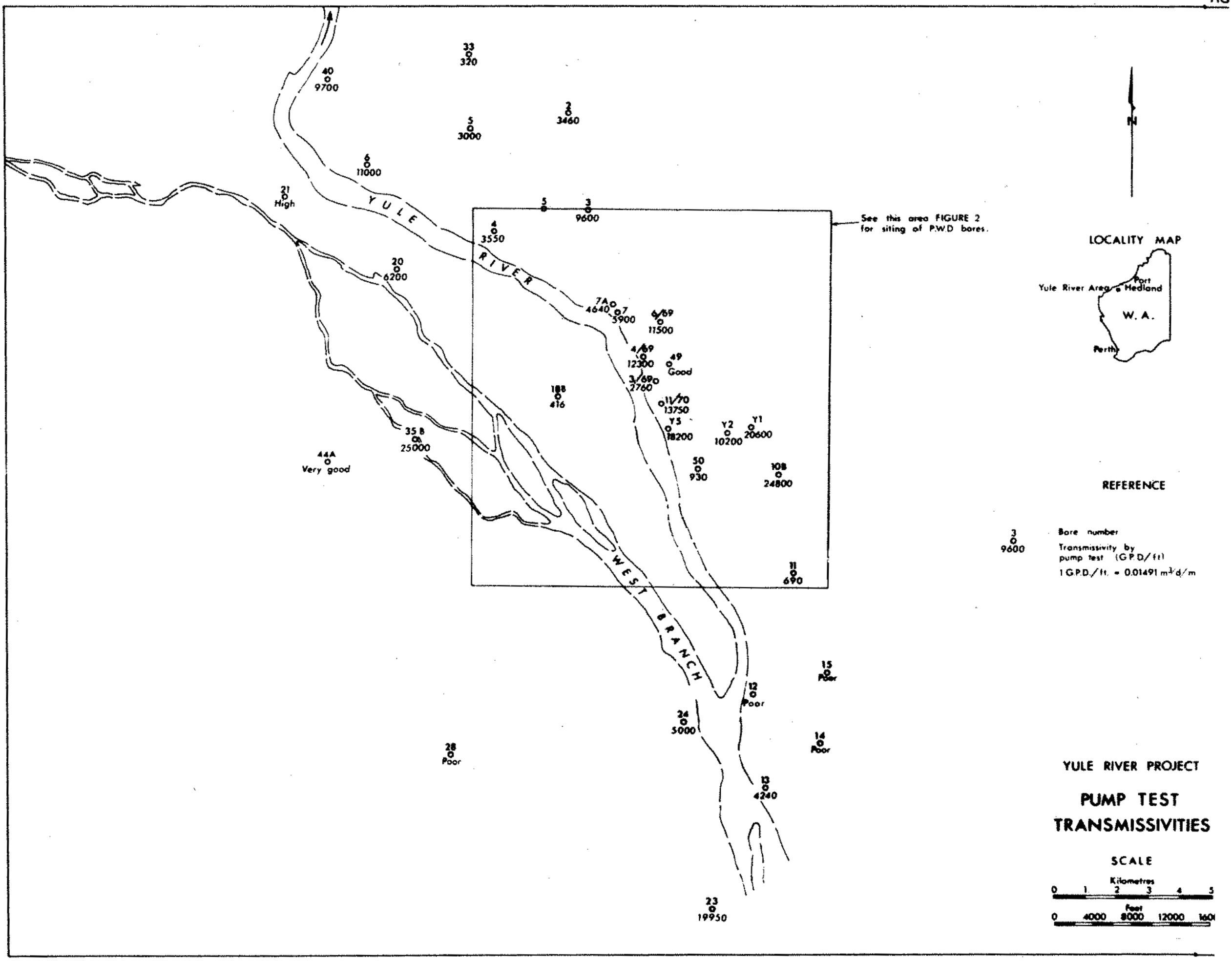
All tests have been re-computed and the results are shown in Table 4 and Figure 8.

TABLE 4. Pump Test Results

BORE	AQUIFERS	SCREEN POSITION feet below top of casing	TRANS-MISSIVITY RESULT gpd/ft^2	COMMENTS
2	36-68, 112-116	35- 55	3,460	Lower aquifer not tested
3	35-49, 97-99, 112-120, 125-126	105-135	9,600	Casing inserted incorrectly. Upper aquifers may be good
4	56-80, 113-130	50- 70 110-130	3,550	Pump test not supervised. Full length screen would probably give higher result
5	32-70, 100-110	31- 51	3,000	Lower aquifer not tested
6	37-78, 118-122 146-150, 196-205	58-208	11,000	Probably reliable
7	38-55	38- 58	5,900	Upper aquifer only
7A	37-60, 75-90 110-130, 250-270	110-130 250-270	Poor 4,640	Lower and bed-rock
8B	55-73	60- 81	Poor	
9	35-52, 52-58 59-92	52- 98 50- 90	Drawdowns not collected Very high, leaky artesian	
10B	35-75, 89-108	70-121	24,800	Leaky artesian
11	57-76	57- 76	600	Poor bed-rock aquifer
12	No significant aquifers			
13	29-37, 54-75	29- 37, 54- 75	4,240 Variable pump rate	Result probably low
14	No significant aquifers			
15	No significant aquifers			
18B	36-60, 137-154	130-156	416	Upper aquifer not tested
20	38-56, 160-168	38- 58 152-172	1,645 4,550	
21	17-33, 75-85 112-130	12- 33	Variable pump rate	
23	80-120	76-116	19,950	Probably reliable
24	15-35	Not tested		Probably good
28	40-70	40- 72		Very poor
33	25-65, 65-80, 80-86	35- 58	320	Upper aquifer only
35B	40-44, 47-50, 52-80	40- 80	25,000	Probably reliable
40	33-39, 40-45, 165-185	165-181	9,700	Lower aquifer only
49	34-66, 86-108			Not tested but probably good
50	28-40, 97-110	28- 40, 95-110	930	
51	36-68			Not tested but probably good
Y1	60-81	69- 81	20,600	
Y2	49-102+	49-102+	10,200	
Y3	36-53, 53-70	43- 56		Leaky artesian. High T
Y5	35-109+	35-109+	18,200	
Y8	36-103+	40-102+	16,500	Close to 10B
4/69	36-112	36-112	12,300	
6/69	44-115	44-115	11,500	
11/70	51-123	51-123	13,750	

1 gallon per day per foot = 0.014915 $m^3/d/m$

FIG



Results show a wide variation of approximately 300-25,000 gpd/ft (4.5-376 $m^3/d/m$) in transmissivity. This wide variation has two main causes:

1. Variation in lithology has given a real variation in transmissivity.

2. The pump tests have been done under variable conditions (e.g. different screen lengths and positions) and this has given an apparent variation in transmissivity.

In silty, sandy alluvial sequences of sediments, it is difficult to define aquifers and aquicludes separately. There is probably a complete gradation between the two, and where only the most favourable portions of the sedimentary sequence were screened for testing, leakage from the non-screened sequences will probably have contributed to the pumpage. In some cases the length of screening was very short by comparison to the total thickness of sediments, and significant aquifers may not have been tapped. This effect can be illustrated by an example.

P.W.D. bore Y8 was drilled 165 feet (50 m) from G.S.W.A. bore 10B. In 10B, good aquifers were reported from 35 to 75 feet (10.7 to 22.9 m) and from 99 to 105 feet (30.2 to 32.0 m). Slotted casing was installed in 10B from 70 to 121 feet (21.3 to 36.9 m) and pump tests made. The results showed leaky artesian conditions and the transmissivity was computed at 24,800 gpd/ft (375 $m^3/d/m$). Bore Y8 was drilled only to 88 feet (26.8 m) with a good aquifer reported from 43 to 78 feet (13.1 to 23.8 m) and tested. The calculated transmissivity was 16,500 gpd/ft (248 $m^3/d/m$). The lower result is almost certainly due to only one of the two aquifers being tapped.

The results of the testing do not generally represent total transmissivity of the multi-aquifer system. They do provide some indication of lateral variation and show a zone of good transmissivity running from bore 10B to bores 40 and 6. This is in accordance with the interpretation of the isopach and bed-rock contour maps. Bores 35B and 23 gave high transmissivities which are in accordance with Figures 5 and 6.

The problems associated with interpreting pump tests which are not really comparable makes it impossible to make any reliable estimation of aquifer hydraulic properties from the calculated results.

As a general conclusion it can be stated that to obtain the best results from pump tests, bores should be drilled through all possible aquifers and be screened for the full thickness of saturated sediments. The same conclusion applies to yields from production bores at the Yule River.

FLOW-NET ANALYSIS

Areal variation in transmissivity was further studied by making a flow-net analysis of the area.

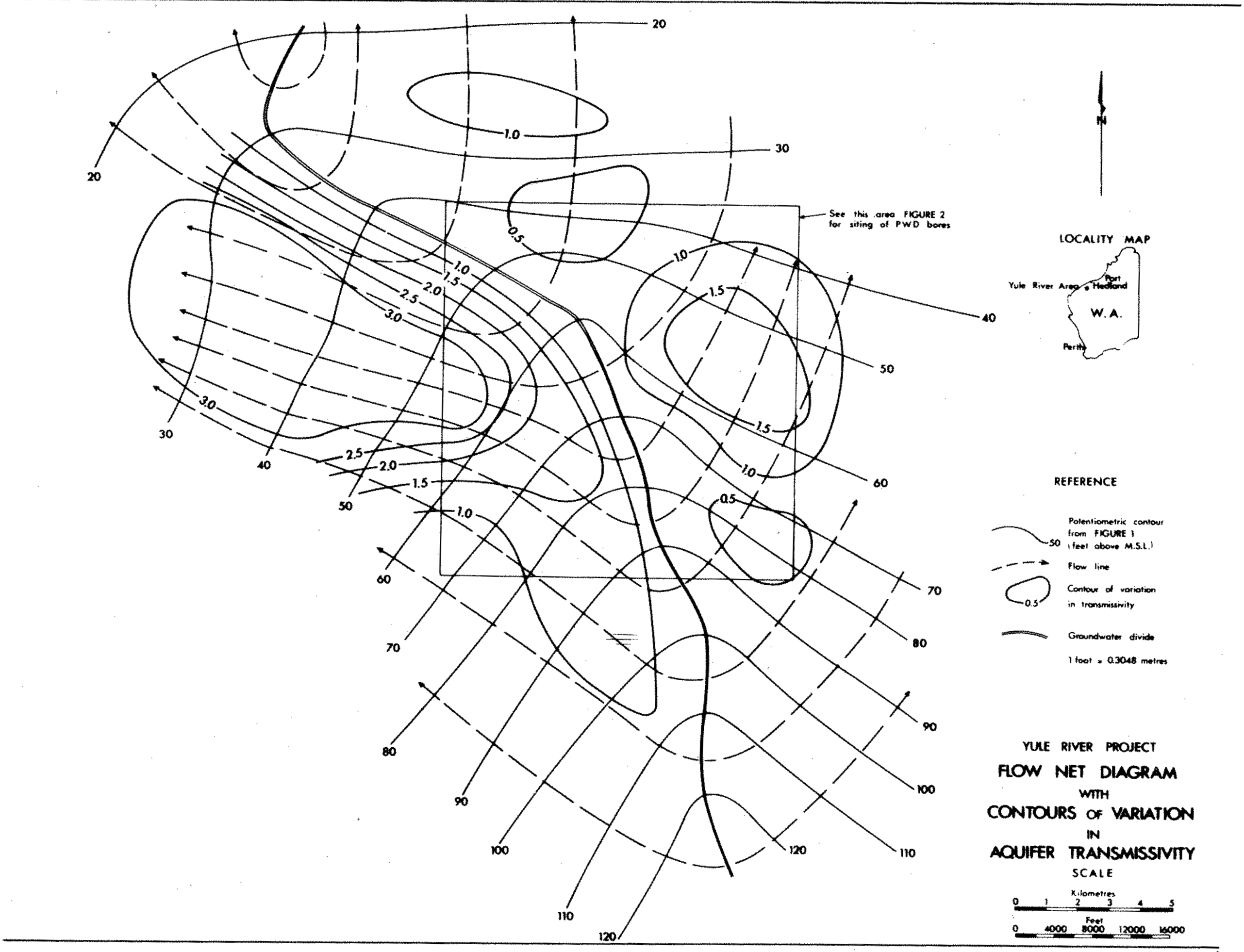
If the sediments in the area were homogeneous, isotropic and of constant thickness, then the potentiometric contours would be symmetrical and uniformly spaced. For the purposes of this analysis the aquifers must be assumed to be isotropic, i.e. the flow will always be at right angles to the potentiometric contours. Following this assumption, variation in spacing of the contours will be caused only by variation in homogeneity and thickness. The sum effects of thickness and permeability variation are expressed then as changes in aquifer transmissivity.

The spacing of equipotential contours where they cross the groundwater divide is a reflection of the vertical permeability governing recharge. During a recharge event varying amounts of water are percolating from the river to the aquifer, along the divide. If percolation were uniform, the divide would have a constant gradient. Variations in the divide gradient are considered to have resulted from variation in percolation rates, with those sections of the divide showing a steepening of gradient being related to areas with high percolation rates. This provides a method of subdividing areally the amount of recharge to the aquifers. The quantity of total recharge is not known, but along the divide, between each pair of adjacent equipotentials, it is assumed that there is the same amount of water recharging the aquifers. This has provided the basis for the construction of the flow net.

The net was constructed by making the number of flow channels equal to the number of potentiometric drops. Each flow line was started near the divide at a point midway between two equipotentials. At the Yule River, flow from the recharge mound will quickly assume a direction nearly at right angles to the groundwater divide.

Between the 120 and 20-foot (36.6 and 6.1 m) equipotentials there are 10 drops, so there will be 10 flow channels to the east and 10 to the west of the recharge divide. From its defined point of origin, each flow-line path was drawn so it crossed successive equipotentials are nearly as possible at right angles. The drawn system of flow lines and equipotential contours (Fig. 9) forms the flow net and divides the area into a system of rectangles.

FIGUR



The flows into each channel have been assumed to be equal, and as isotropy has been assumed to apply, flow is constant along any channel. The flow equation, for any single rectangle on the set, will be according to Darcy's Law:

$$Q = Tiw \qquad \text{... ... ...} \qquad 1$$

where Q = flow rate

T = transmissivity

i = hydraulic gradient

w = width of the rectangle = distance between the bounding flow lines.

Now $i = \frac{dh}{l}$ 2

where dh = change in head along the rectangle

l = length of the rectangle

= distance between the bounding equipotentials.

From equation 1,

$$T = \frac{Q}{iw} \qquad \ldots \qquad 3$$

Substituting for i from equation 2,

$$T = \frac{Ql}{dh.w} \qquad \ldots \qquad 4$$

In all rectangles, dh has been made constant by construction, and equal to 10 feet (3m). Through all rectangles, the flow, Q, is assumed to be identical and constant:

$$\therefore T = C\frac{l}{w} \qquad \ldots \qquad 5$$

where C is a constant $= \frac{Q}{dh.}$

The transmissivity of any rectangle is therefore equal to a constant, times the ratio of length to width of the rectangle.

The ratio of length to width was calculated for every rectangle of the flow net. The ratio value was assumed to be applicable to the centre point of each rectangle and the contours of variation of transmissivity on Figure 9 were drawn.

If the constant C can be determined for any single rectangle in the flow net, then the absolute transmissivity can be calculated for any other rectangle in the net. Visual comparison of the flow-net transmissivity (Fig. 9) with the pump-test values (Fig. 8) suggests that C may have a value of approximately 10,000. Very few of the pump tests have given results which can be taken with certainty as being typical of the whole aquifer system, and it is impossible to accurately derive a value for C.

In two areas the flow-net transmissivities appear to be substantially incorrect. In the south-east corner, transmissivities are very low although the net indicates moderate values. To the east of the divide between the 20 and 30-foot (6.1 and 9.1 m) equipotentials, the net does not adequately reflect the reasonably high transmissivities indicated by pump tests.

In the first area the fault probably lies with the construction of the potentiometric map and the flow net. In the area between the 20 and 30-foot (6.1 and 9.1 m) equipotentials the discrepancy is apparent rather than real. If additional equipotentials were drawn, at 2-foot (60 cm) intervals for example, then the higher transmissivities indicated by some pump tests would show. The flow net as drawn in Figure 9 has smoothed local variations and averaged transmissivities over large areas.

A comparison of the maps of aquifer thickness, basement contours and the flow net, indicates the areas best suited to production. The present production field centred about bore 49 is apparently well placed. Production bores 13/70 and 14/70 (Fig. 2) were sited where the aquifers were thinning rapidly and their failure is not surprising. The area south of bore 11 on the eastern bank of the river is apparently completely unsuited for production. Most of the present production bores take advantage of good aquifer conditions between bores 10 and 9.

Two lines of bores (Fig. 2) have been proposed by the P.W.D. as extensions to the present production field. Available data suggest that instead of siting these bores close to and parallel to the Yule River, it would be better to run lines of bores from 6/69 towards bore 3, from there to bore 6 and on to bore 40.

On the western side of the river, bores sited between 18B and 35B are likely to be good. Favourable conditions are indicated to the west of these, but increasing salinity (see Water Quality section) may pose a problem and care would be necessary in planning production.

Aquifers thicken near bore 23 and water quality is good. Though contours of relative transmissivity have not been drawn, pump-test data from this one bore indicate that production in this area may be feasible.

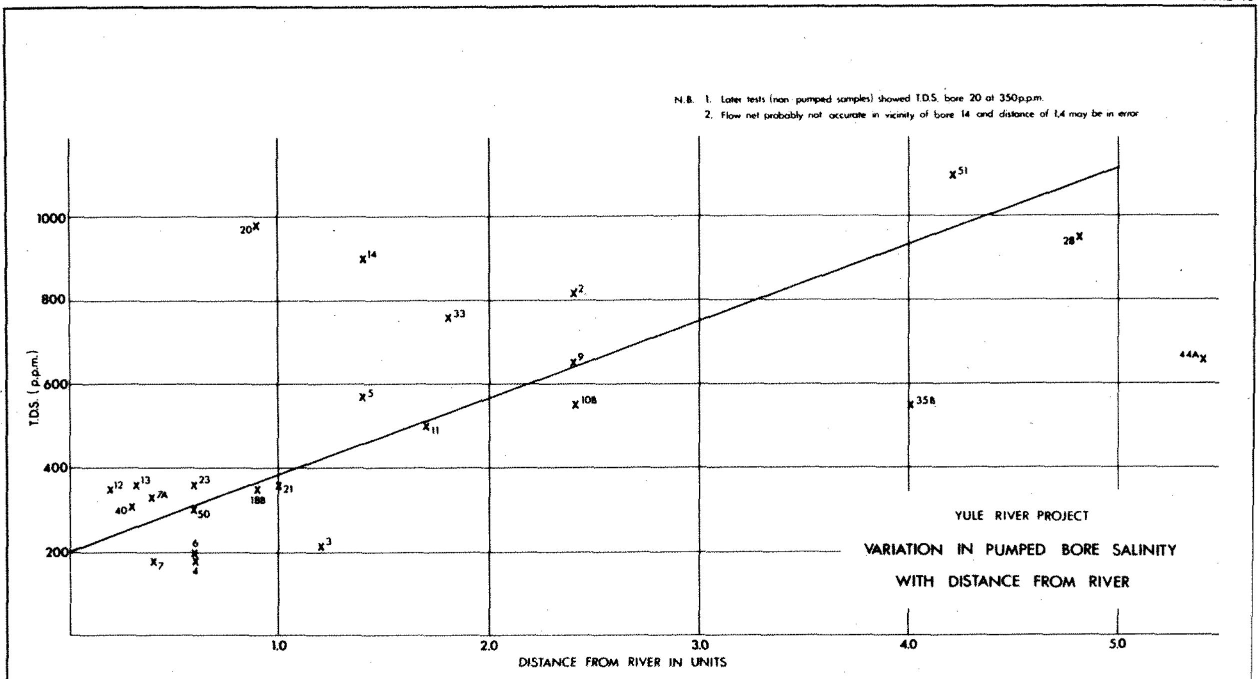
WATER QUALITY

Two features dominate the pattern of salinity variation:

1. A well marked trend of increase in salinity with distance from the river.
2. The presence of a high salinity layer in the upper few feet of water in many bores.

Increasing salinity with distance from recharge source is a normal pattern. Where aquifers are homogeneous, distance can be taken as a simple linear measurement from the recharge source. A correction for non-homogeneity was made in this case by measuring distance along flow lines and using, as a measure of distance, the number of flow-net rectangles between the sample point and the recharge source (the Yule River).

FIGURE 10



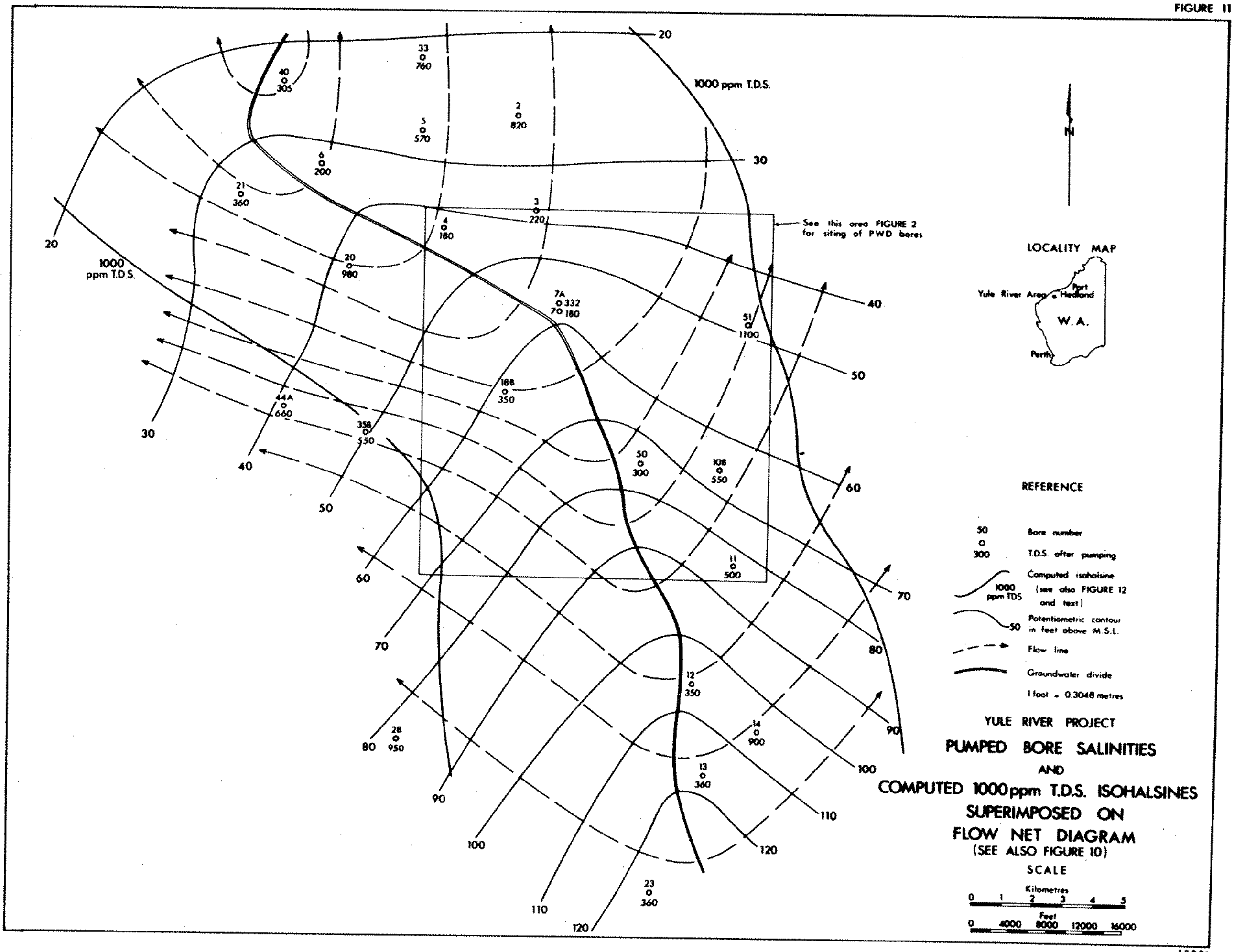
One distance unit on abscissa is equivalent to one flow net rectangle on FIGURE 9 .

13390

The only salinities accepted for the correlation were those taken at the conclusion of a pumping test. Figure 10 shows measured salinity plotted against distance from river. Despite some scattering of the points it was possible to draw an approximate line of correlation.

Salinities increase above 1,000 ppm TDS at distances of slightly more than four flow rectangles from the river. This relationship has been used on Figure 11 to define the area which can be expected to have water with salinity lower than 1,000 ppm TDS.

FIGURE 11



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Where the computed isohalsine of this value is in poor agreement with the salinities actually measured, for example at bores 20, 35B and 44A, this is due to inevitable errors in the flow-net plot resulting from data inadequacy.

AQUIFER RECHARGE

There is evidence to support two sources of aquifer recharge:

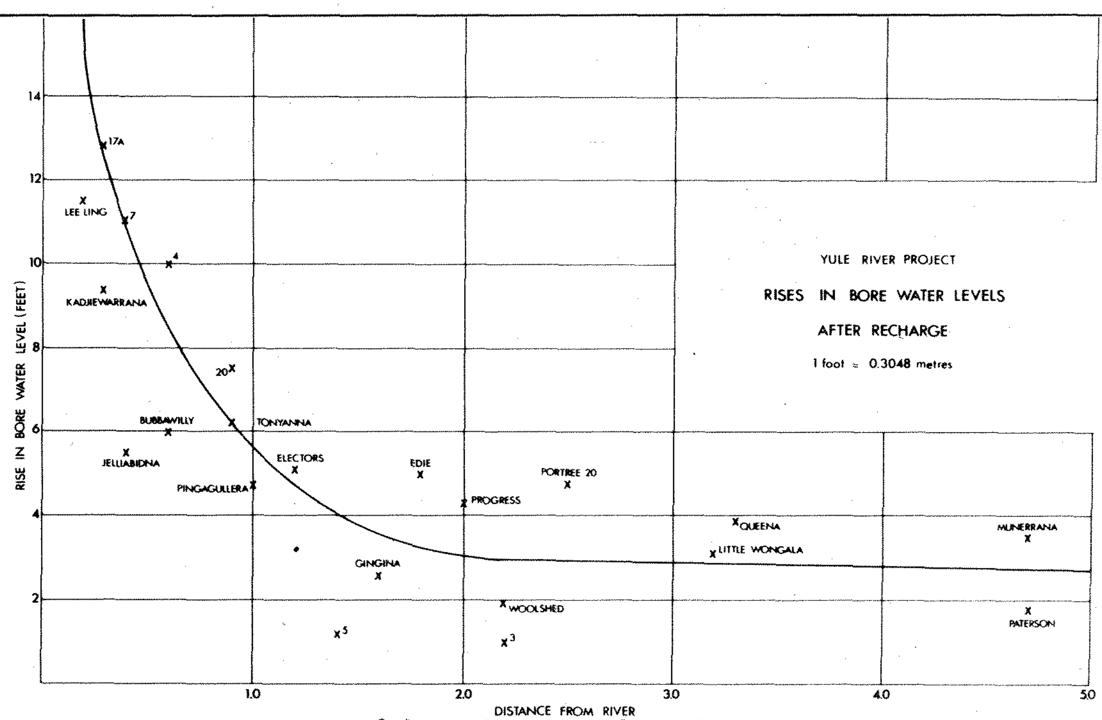
1. Recharge from river.
2. Direct recharge from rainfall.

The potentiometric contours quite clearly indicate that there is recharge from the river, but the evidence for rainfall recharge is not so clear.

Water levels (recovery and drawdown) from the production bores (Figs. 3 and 4) show an apparent recharge event in August to September 1970. There is no record of river flow in 1970 but in May of that year two storms brought a total of 5.41 inches (137.5 mm) of rain to Port Hedland. The recorded rises in bore water levels are uniform and occurred practically simultaneously throughout the aquifer. They cannot be related to a reduction in pumping as they occurred at a time when pumpage was being increased.

The 100-day delay between the storm and rise in bore water levels can be matched by events from the 1966 storms; Whincup's Plate 16 shows a plot of recorded water levels for bores 3, 4, 5, 7 and 20. Following cyclonic rainfall in April of that year there was a sharp rise in bore water levels when the river flooded, but bore water levels did not peak until approximately 100 days after the storm event. The almost immediate response of bore water levels to flooding, followed by a much later peak, could be interpreted as the result of two recharge processes.

FIGUR



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From all available well and bore records, total rises in water levels, following the 1966 flooding, were plotted (Fig. 12) against distance from the river. As on Figure 10, distances were measured along the direction of flow lines, the measure of distance being the number of flow-net rectangles between the observation point and the river. At about three distance units from the river, the rises in water level become asymptotic to distance. This pattern is also interpreted as being the result of two recharge processes. There is apparently a broad mound of water resulting from river recharge, and also a general more sheet-like recharge of water which is the result of rainfall infiltration.

In many bores Whincup (1967) has reported the presence of a high-salinity layer in the uppermost groundwater levels. This he interpreted as being the result of fluctuating water levels, and the high specific retention of the silt and clay which overlies the upper sand aquifers. During flooding, water rises into the silt and clay and after levels have fallen the water that is retained in the clay becomes highly saline. When water levels again rise, the rising fresher water is contaminated by the adsorbed salts and saline water.

The higher salinity layer can be more simply explained by assuming aquifer recharge by rainfall from major storms. Light rainfalls wet only the surface sands where by evaporative processes salts will tend to be concentrated. When rainfalls of sufficient magnitude to give aquifer recharge occur, the percolating rainfall will dissolve the residue salts in the upper soil and carry them down as a wave to the aquifers. Water levels indicate that it takes approximately 100 days for this rainfall recharge to reach the aquifers. This would account for the peak in salinities recorded in May and June of 1966. The water in the clay and silt above the aquifers will always tend to be more saline than the water below, where the aquifers are being diluted by the fresher water from river recharge. The changing position of the isohalsine, as shown by Whincup (1967), can equally be satisfactorily explained by a theory of river recharge coupled with rainfall infiltration.

In 1966 the average increase in salinity recorded at station wells and bores was 21 per cent. This apparently suggests a very large influx of saline water. Sampling of these bores is confined to near the surface and the increases only apply to the upper few feet of water. River recharge is probably the most significant replenishment source.

By assuming that the process of rainfall recharge occurs as described, it is possible to calculate a value of vertical permeability for the silts and clays overlying the upper sand aquifers.

It is approximately 30 feet (9.1 m) from ground surface to water level. Assuming that the downward percolation will take place under a hydraulic gradient approximating to unity, then if the process takes 100 days, vertical permeability = $\frac{30}{100} \times 6.23 = 1.87$ gpd/ft^2 (0.092 m^3/dm^2). Davis and De Wiest (1966) show clays, sands and fine sands (poor aquifers) as having permeabilities in the range 10^{-2} to 10 gpd/ft^2 (5×10^{-4} to 0.5 m^3/d/m^2). The upper 30 feet (9.1 m) of sediments in the Yule River area have been described by Whincup (1967) as sandy silt and clay, indicating that the computed vertical permeability is therefore feasible.

When a rainstorm is of sufficient magnitude to give flooding in the river, and heavy rain falls in the investigation area, the recharge process is as follows:

1. Following river flooding there is rapid infiltration into the river-bed gravels and a recharge mound forms under the river.

2. Water levels in bores will respond quickly to the mound formation as pressure changes are transmitted through the semi-confined and confined aquifers.

3. Approximately 100 days after the storm, rainfall recharge may reach the aquifers, giving a peak in bore water levels and an increase in salinity in the upper few feet of water.

4. Water levels for some distance from the river will probably fall very slowly after flooding, as the recharge mound disseminates and replaces water flowing out from the investigation area.

There is insufficient information available to accurately determine the total volume of aquifer recharge in 1966. From Figures 11 and 12 it is possible to calculate the change in aquifer volume for the area bounded by the 1,000 ppm TDS isohalsines and the 120 and 20-foot (36.6 and 6.1 m) potentiometric contours.

For the 1966 recharge event the volume was 23,770 x 10^6 ft^3 (672 x 10^6 m^3). Whincup (1967) did a similar calculation and then applied a specific yield of 0.02 to derive a recharge volume of 3,200 x 10^6 gallons (14.5 x 10^6 m^3). The above aquifer volume change, with the same specific yield, would be equivalent to 2,960 x 10^6 gallons (13.4 x 10^6 m^3).

A specific yield of 0.02 is a value that would normally be associated with clays. The Yule River aquifers are silty sands and gravels for which a more appropriate value would perhaps be 0.15. Whincup chose a value of 0.02, by adopting results obtained by Farbridge (1967) in the neighbouring Turner River. There, by comparing the volume of pumped water with the volume of the developed cone of depression, a value of 0.02 was obtained. Whincup considered that aquifer conditions were similar at the Yule River.

The Turner River aquifers are clayey sands with minor gravel and kankar. This suggests that a specific yield as high as 0.10 may well be appropriate. Farbridge (1967) described the Turner River aquifers as "apparently confined by the top sand-clay-silt layer". At the Turner pumping area, a cone of depression would first develop in the piezometric surface as elastic storage is lost. Eventually, drawdowns may be sufficiently great for all elastic storage to have been lost, and further water would be obtained by dewatering of the aquifer sediments. In such a situation the storage coefficient will have passed from artesian range (0.00005 to 0.005) to water table range (0.02 to 0.2) (Todd, 1963). This may account for the derived storage coefficient which is too large for artesian range and yet appears to be too low for the described sedimentary conditions.

At the Yule River area the upper aquifers range from unconfined to confined and Whincup's choice of 0.02 storage coefficient may well be appropriate for the recharge process. The described lithologies suggest that the dewatering storage coefficient may well quickly increase to a value much greater than 0.02.

An approximate check on the volume of recharge can be made by making the following assumptions:

1. Most of the recharge is from the river.

2. Between the 120 and 20-foot (36.6 and 6.1 m) potentimetric contours there is a 21-mile (33.8 km) length of river bed which for 75 days in 1966 was occupied by a river having an average width of 300 feet (91 m).

3. Though infiltration was initially at a much higher rate, an average permeability for the silty river bed could be assumed at 1 gpd/ft^2 (4.9 x 10^{-2} $m^3/d/m^2$).

Then, assuming that the hydraulic gradient of percolation is approximately unity, the volume of recharge is: 21 x 5,280 x 75 x 300 x 1 = 2,500 x 10^6 gallons (11.3 x 10^6 m^3) which is in accordance with the previously computed value.

Recharge from the river can also be estimated by computing the outflow from the mound during the dissemination process. The mound has a length of 21 miles (33.8 km) and flow is to both sides. An average gradient was found from measurement on Figure 9 to be 1.3 x 10^{-3}. A study of Figures 8 and 9 indicates that an appropriate transmissivity value for the sediments in the vicinity of the divide would be 10,000 gpd/ft (150 $m^3/d/m$). Assuming then that the time of dissemination is 3.5 years (the estimated average recharge interval) total flow from recharge is 21 x 2 x 5,280 x 0.0013 x 10,000 x 3.5 x 365 = 3,670 x 10^6 gallons (16.7 x 10^6 m^3). The accuracy of this calculation is mainly dependent upon the adopted values of transmissivity and time. Whincup (1967) estimated from a study of pump-test results that the average hydraulic conductivity in the Yule River area is 300 gpd/ft^2 (14.7 $m^3/d/m^2$) and that the average total thickness of aquifer is 33 feet (10 m). This corresponds to an average transmissivity of 9,900 gpd/ft (149 $m^3/d/m$) which is essentially identical with the value used here The true time of dissemination is not known and any error in its estimation will directly affect the accuracy of these results.

Available data suggests, therefore, that river recharge in 1966 was approximately 3,000 x 10^6 gallons (13.6 x 10 m^3).

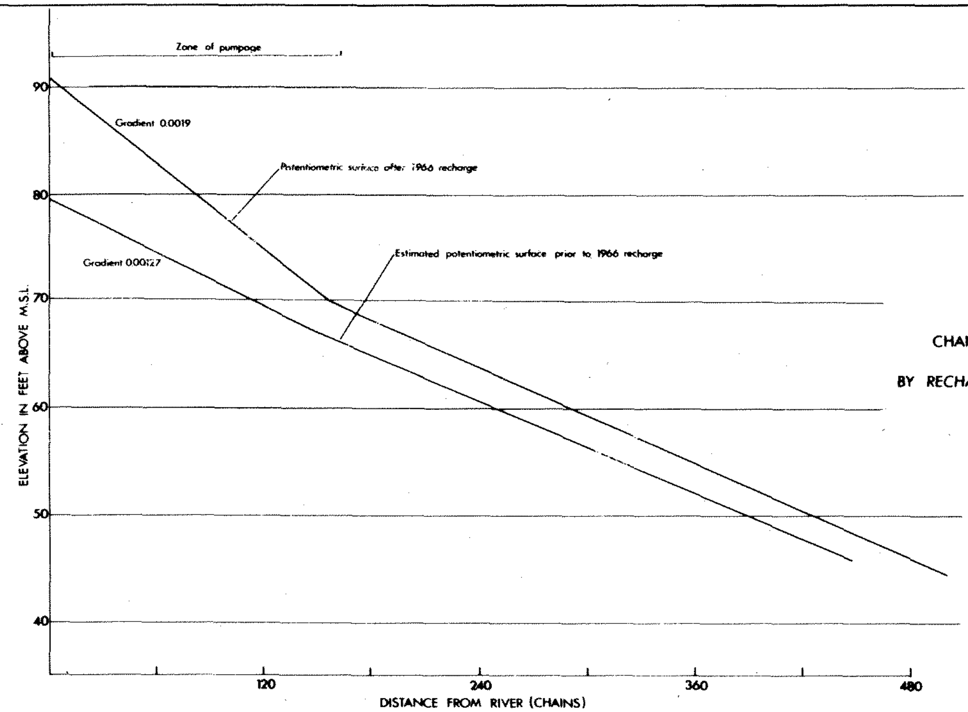
EFFECTS OF PUMPAGE

The Yule River area is complex and the aquifers have very variable hydraulic properties. Even if the hydraulic properties were precisely known it would be extremely difficult to calculate the effect of pumping.

Some simple calculations have been made for a single flow-net channel to illustrate possible effects of pumping. The flow-net channel used is that whose origin has as its centre point the contact between the 80-foot (24.3 m) equipotential and the groundwater divide. The flow-net channel runs northeastwards from the Yule River through the present production area.

On Figure 13 the potentiometric surface is shown in section prior to and after recharge in April 1966.

After recharge the gradient of the steeper part of the surface was 1.90 x 10^{-3} and before recharge 1.27 x 10^{-3}. The width of the flow channel is 6,450 feet (1,930m). The flow-net transmissivities indicate that an average value for the channel could be 10,000 gpd/ft (150 $m^3/d/m$). With a mean gradient of $\frac{0.0019 + 0.00127}{2} = 1.60 \times 10^{-3}$, daily flow from the pumped area would be 103,000 gpd (470 m^3/d), if there were no pumpage.



If only the zone of pumpage is considered, then for this one flow channel, recharge gave an aquifer storage change of 470 x 10^6 ft^3 (13.3 x 10^6 m^3). Again assuming that the dissemination process takes 3.5 years, a daily outflow of 103,000 gpd (470 m^3/d is equivalent to a recharge of 132 x 10^6 gallons (0.60 x 10^6 m^3). The corresponding specific yield will be $\frac{132 \times 10^6}{470 \times 10^6 \times 6.232} = 0.045$. If the specific yield is derived partly from elastic storage and partly from porosity storage the value could be reasonable.

If successive recharge events are of similar magnitude and occur at an average interval of 3.5 years as suggested by Whincup, then this value, 103,000 gpd (470 m^3/d) per flow channel represents the maximum amount of water available for pumpage without mining water.

The P.W.D. production field covers four flow channels. For the first 12 months of operation the average production rate was 180,000 gpd (818 m^3/d) per flow channel. Assuming all the water came from the recharge mound, this, using the same factors as above, would have given a general lowering of water levels of approximately 3 feet (0.9 m) in the production zone (as indicated on Figure 13). By comparison with this estimate, actual drawdowns in the production wells after 12 months were:

Y 1	4.0 feet	(1.2 m)
Y 2	5.5 feet	(1.7 m)
Y 3	3.5 feet	(1.1 m)
Y 6	5.0 feet	(1.5 m)
Y 7	4.0 feet	(1.2 m)
Y 8	4.0 feet	(1.2 m)
Y 5	7.5 feet	(2.3 m)

The difference between the above figures and the estimate of 3 feet (0.9 m) might appear to be large, but it must be borne in mind that the above figures apply to the production bores. Away from the bores the drawdowns will be somewhat less and the overall estimate of 3 feet (0.9 m) is likely to be reasonable.

The P.W.D. has requested that an estimate be made of the effects of three pumping rates: 1.9 x 10^6 gpd, 2.5 x 10^6 gpd and 4.0 x 10^6 gpd (8.6 x 10^3 m^3/d, 11.3 x $10^3 m^3$/d and 18 x 10^3 m^3/d) for 1 year only.

If the recharge to a flow channel (see above) is equivalent to 0.013 x 10^6 gpd (470 m^3/d) for 3.5 years then we would need, respectively, 18, 24, and 11 flow channels, the last for 1 year only.

As a first conclusion, it appears that a rate of 4.0 x 10^6 gpd (18 x 10^3 m^3/d) for 1 year can be satisfied without seriously depleting the aquifers, if the production is spread over 11 flow channels.

There are, in total, 20 flow channels according to Figure 9, (10 on each side of the divide), though some doubt must be expressed as to the accuracy of portions of the map. A tentative conclusion might be that by spreading pumping over the whole area the lower rate can be safely met, and even the higher rate might possibly be met without undue depletion of the available storage.

From Figures 6 and 11 it has been calculated that there are 135 x 10^9 ft^3 (3.82 x 10^9 m^3) of aquifer containing water with less than 1,000 ppm TDS. As stated earlier a reasonable storage coefficient for these aquifers might be 0.15, in which case, they hold 126 x 10^9 gallons (574 x 10^6 m^3) of potable water. Only a small proportion of this water can be considered as available for withdrawal if water quality standards are to be maintained over a long period of time. To prevent migration of the 1,000 ppm isohalsine toward the production area, a hydraulic gradient away from that area must be maintained. Therefore the water that is held in storage at elevations greater than water with more than 1,000 ppm TDS is that which can be withdrawn from storage safely. This volume was calculated from Figures 6 and 11 and found to be appromixately 2,000 x 10^6 gallons (9.09 x 10^6 m^3). Pumping would have to be spread over nearly all the investigated area if this volume were to be utilized. As pumping will presumably be restricted to those areas where the aquifers are reasonably thick and the transmissivities are satisfactory, probably not more than half of this volume can be regarded as usable storage.

The total volume of water in storage is so large that mining of reserves could be contemplated. Under such a situation a close watch must be kept for saline contamination If mining of water is carried out, the 1,000 ppm isohalsine will migrate inwards as the hydraulic gradient is reduced. Rainfall recharge as suggested will continue to add more saline water to the aquifers and at the same time, utilization of good quality flood recharge water will reduce the dilution that is at present taking place. Mining of stored water will effectively accelerate the rate of increase in salinity in the aquifers.

FUTURE EXPLOITATION

On Figure 14 the investigated area has been subdivided into three classifications:

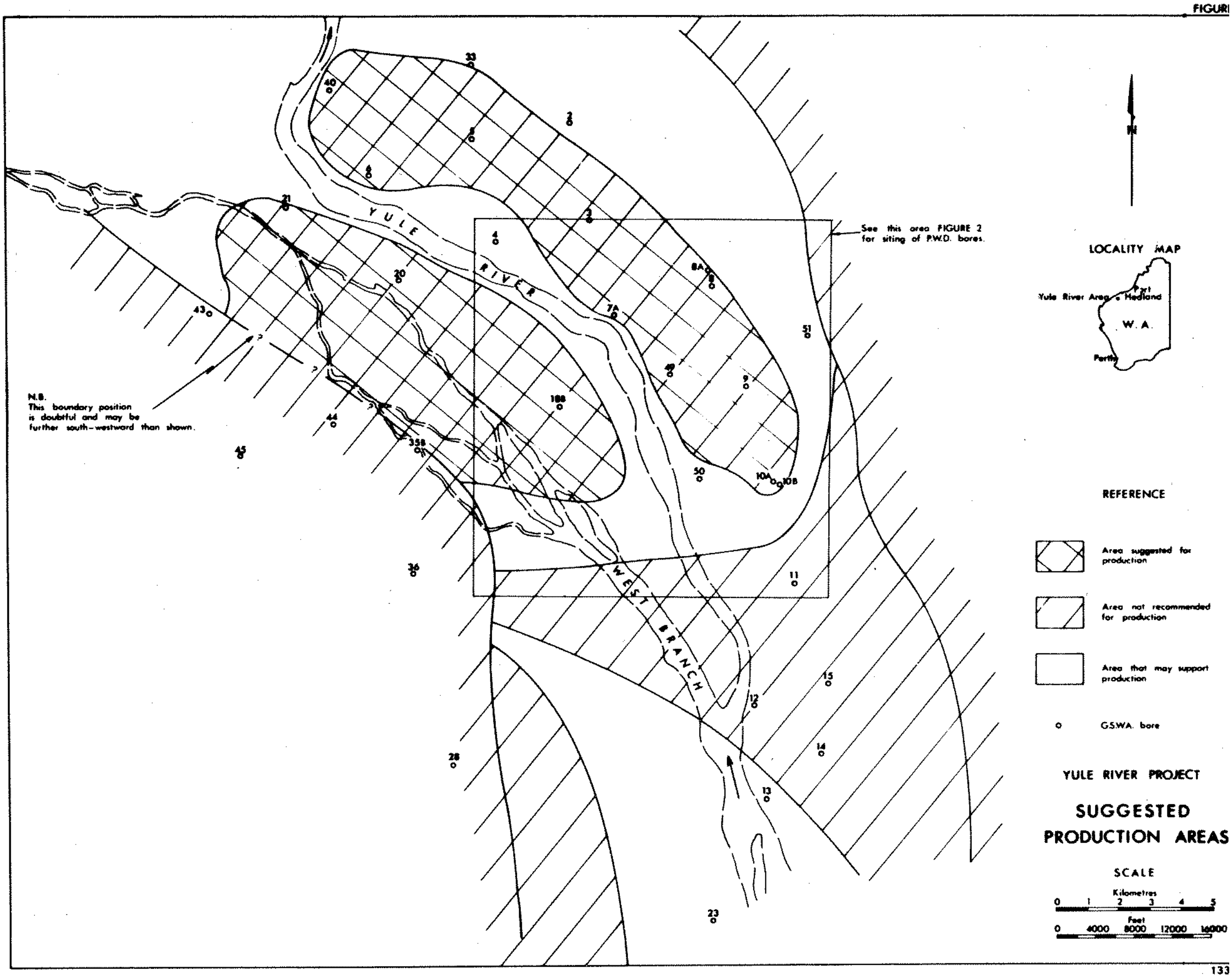
1. Areas suggested for production.

2. Areas not recommended for production.

3. Areas that may support production.

The subdivision has been based on available data on transmissivity, aquifer thickness and water quality.

FIGURI



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Two areas are proposed as the most suitable for production. The area east of the Yule River includes the present P.W.D. bore field. The area to the west of the river is believed to be good. Only limited data were available from this area but all information ties into a general pattern indicating good prospects. The western margin of this recommended area is the estimated 1,000 ppm isohalsine. Bore data on Figure 11 indicate that the true 1,000 ppm isohalsine may be even further westward and that production from the indicated area may well be quite safe from saline contamination.

There are two areas very close to, and underlying, the river which have specifically not been recommended for future exploitation because available data suggests that permeabilities may locally be rather low. These areas are, firstly, in the vicinity of bore 4, and secondly a narrow zone east of bore 18B.

Pumping near to, or even in the river bed, may be desirable in the vicinity of bores 6 and 40 where a lowering of water levels under the river bed could conceivably increase future recharge during flood events. Water level fluctuations will be very large in such a situation and pumped drawdowns, because of the closeness to the groundwater divide, will be slightly greater than experienced elsewhere. Water quality will be very good and if pumpage can increase future recharge beyond that which would have occurred naturally, then the large fluctuations on bore water levels will not be disadvantageous.

The spacing of bores in the present production field appears to be satisfactory. The only record of bore interference while pumping is in progress has been from Y8, where on some occasions there has been a drop in production because of pumping from Y9.

An indication of bore spacing in the future can be obtained from the flow-net diagram. As each flow rectangle is carrying the same quantity of water, a constant number of bores per flow channel and rectangle should be maintained.

CONCLUSIONS AND RECOMMENDATIONS

1. Flow analyses in this report have been based on an assumption that percolation from the river has governed the shape of the potentiometric surface at the groundwater divide. From this it was concluded that flow channels of equal discharge could be defined. Disregarding possible inaccuracies in the potentiometric map caused by inadequacies in the available data, the accuracy of the quantitative assessments is dependent upon the validity of the assumption used as a basis for flow-net construction. It is considered that the observations that have been made on basement configuration, aquifer thicknesses and aquifer hydraulic properties show such overall agreement with the flow-net analysis that the concept, used for the derivation of flow channels of equal discharge, is sufficiently correct as to be usable.

2. If the present production field is increased it is estimated that a production rate of 1.9×10^6 gpd (8.6×10^3 m^3/d) can be maintained.

3. It may be possible to maintain a production rate of 2.5×10^6 gpd (11.3×10^3 m^3/d) in the future but this cannot be estimated with any certainty.

4. It should be possible to pump at a rate of 4.0×10^6 gpd (18×10^3 m^3/d) for 12 months in 1972-73. The aquifers were recharged in 1971 and by spreading production over a wider area it should be quite safe to pump at this high rate.

5. The above conclusions are all based on extremely limited data. It is essential that in the future observations of bore water levels be made in all available observation wells.

6. Because of the danger of saline contamination, vertical salinity logging should be carried out periodically in observation wells of suitable design. This will be particularly important for bores near the 1,000 ppm isohalsine.

7. There are little pump-test data available which adequately represent the hydraulic systems of the area. Controlled testing of new production bores should be done in the future.

8. Because of the paucity of reliable data and because of the complex nature of the alluvial aquifers of the Yule River area, much of this report has been based on estimated factors. Accordingly it must be borne in mind that any conclusions reached may not be accurate.

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AN APPRAISAL OF THE EFFECTS OF LONG-TERM PUMPING IN THE LAKE ALLANOOKA AREA

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APPENDICES

FIGURES

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AN APPRAISAL OF THE EFFECTS OF LONG-TERM PUMPING IN THE LAKE ALLANOOKA AREA

SUMMARY

From a study of water level observations, pump tests, and abstraction records, hydraulic properties are derived for the Allanooka area which can be hydrologically subdivided into three parts;

1. Eastern Sector;
2. Northern Sector;
3. Southern Sector.

The Eastern Sector has poor groundwater potential and is not recommended for exploitation. The Northern Sector is the present production area. Calculations show that the present production rate of 2.19 x 10^6 gpd (9,960 m^3/d) can be maintained for a further 10 years. Future exploitation is recommended for the Southern Sector, where it is concluded that large supplies can be obtained from existing storage. The effect of an abstraction rate of 5 x 10^6 gpd (22,700 m^3/d) is shown and it is concluded that this rate could be maintained without saline contamination becoming a problem.

INTRODUCTION

The domestic water supply potential of the Allanooka area, 35 miles (48 km) southeast of Geraldton, was investigated and reported on by Allen (1965). Production started in August 1967 and has since continued at an increasing rate. Water level records have been kept for 3½ years and some water levels have declined. In addition Ngarlingue Spring has ceased to flow.

The water supply potential has been reassessed using water level observations, pumping records and pump tests.

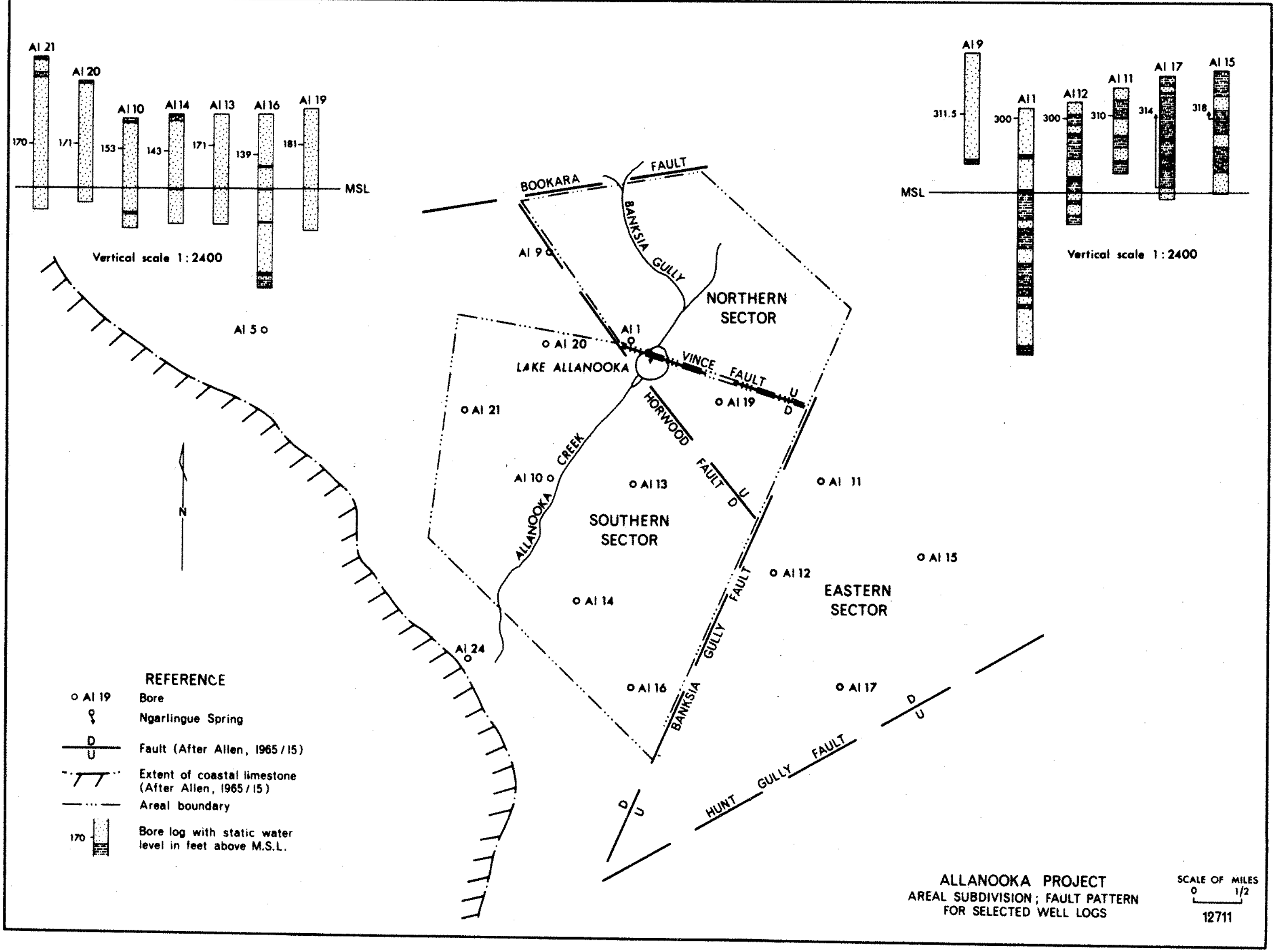
GEOLOGY, STRATIGRAPHY AND STRUCTURE

Information obtained from wells drilled since the completion of Allen's investigation is in accord with his geological interpretation except for a minor adjustment to the position of the Vince Fault. Water level records and well logs (Figs. 1 and 2) indicate that it should lie slightly to the north and be skewed to a more northwesterly trend. This would not change the original geological interpretation of the area.

Allen showed the western end of the Vince Fault as truncated by the Horwood Fault, and this has been accepted. An alternative would be to delete the Horwood Fault, and extend the Vince Fault farther westward, passing just northward of bore 20.

On Figure 2 the water level contours within the eastern sector show a marked steepening suggestive of a fault passing northwestward along a line south of bores 12, 17, and Horwood 2. This could be an extension southeastward of the Horwood Fault and has been so illustrated on Figure 2. Some support for the existence of the Horwood-Vince Fault intersection is the nearby occurrence of Ngarlingue Spring and the lake.

FIGURE 1

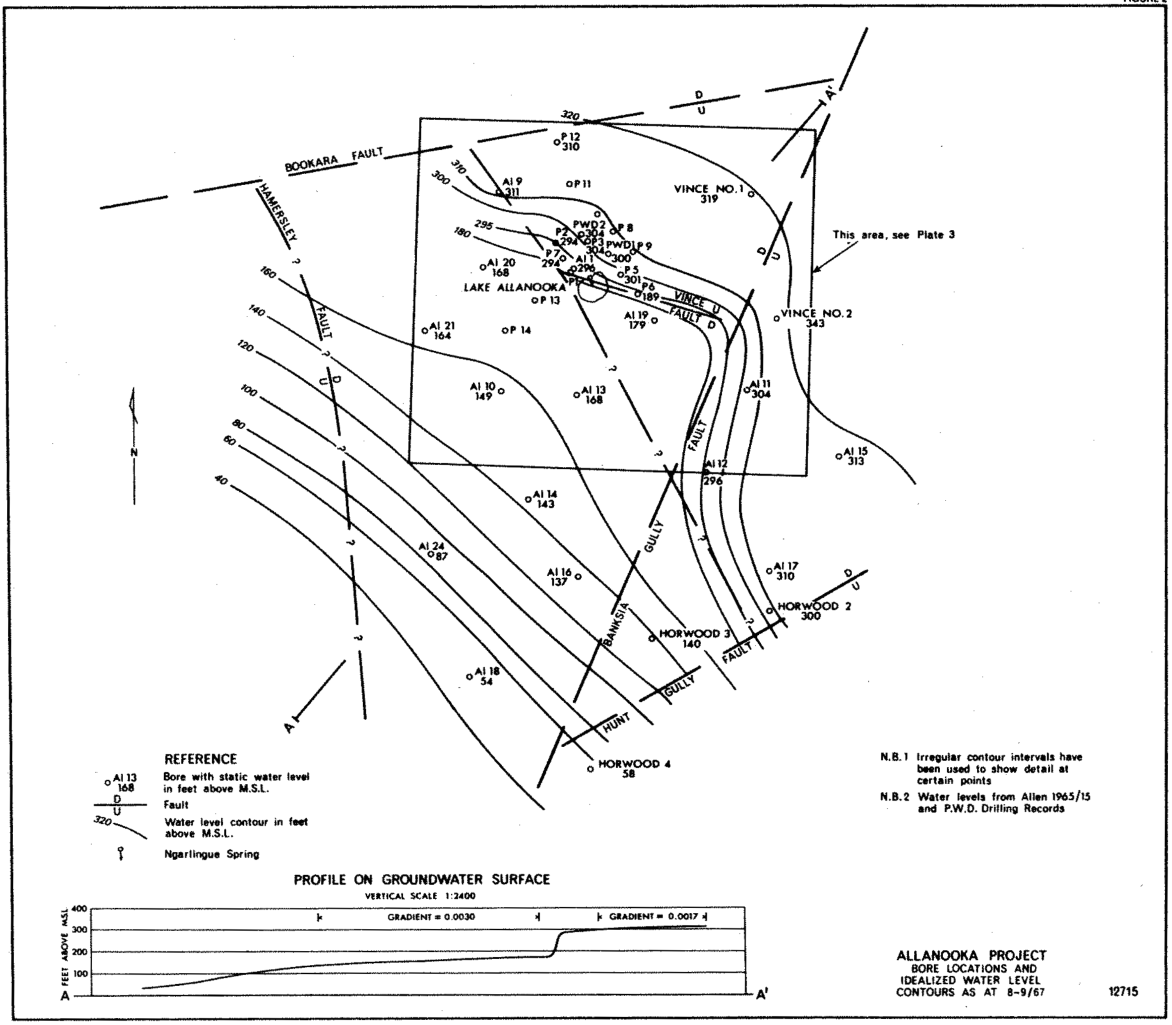


ALLANOOKA PROJECT
AREAL SUBDIVISION; FAULT PATTERN
FOR SELECTED WELL LOGS

SCALE OF MILES
0 1/2

12711

FIGURE 2



The matter is rather of academic interest, not greatly affecting the conclusions reached; except that if the Horwood Fault does not exist northwest of Lake Allanooka, rather more water would be available from the Northern Sector.

Potable water can be obtained from aquifers 100 to 300+ feet (30 to 90+ m) thick in the non-marine Yarragadee Formation (Upper Jurassic) which directly underlies the area. The formation is a series of variable, medium to coarse-grained clayey lenticular sandstones, thick sandy siltstones and minor shales which can be divided into an upper sandstone and a lower alternating siltstone-sandstone section.

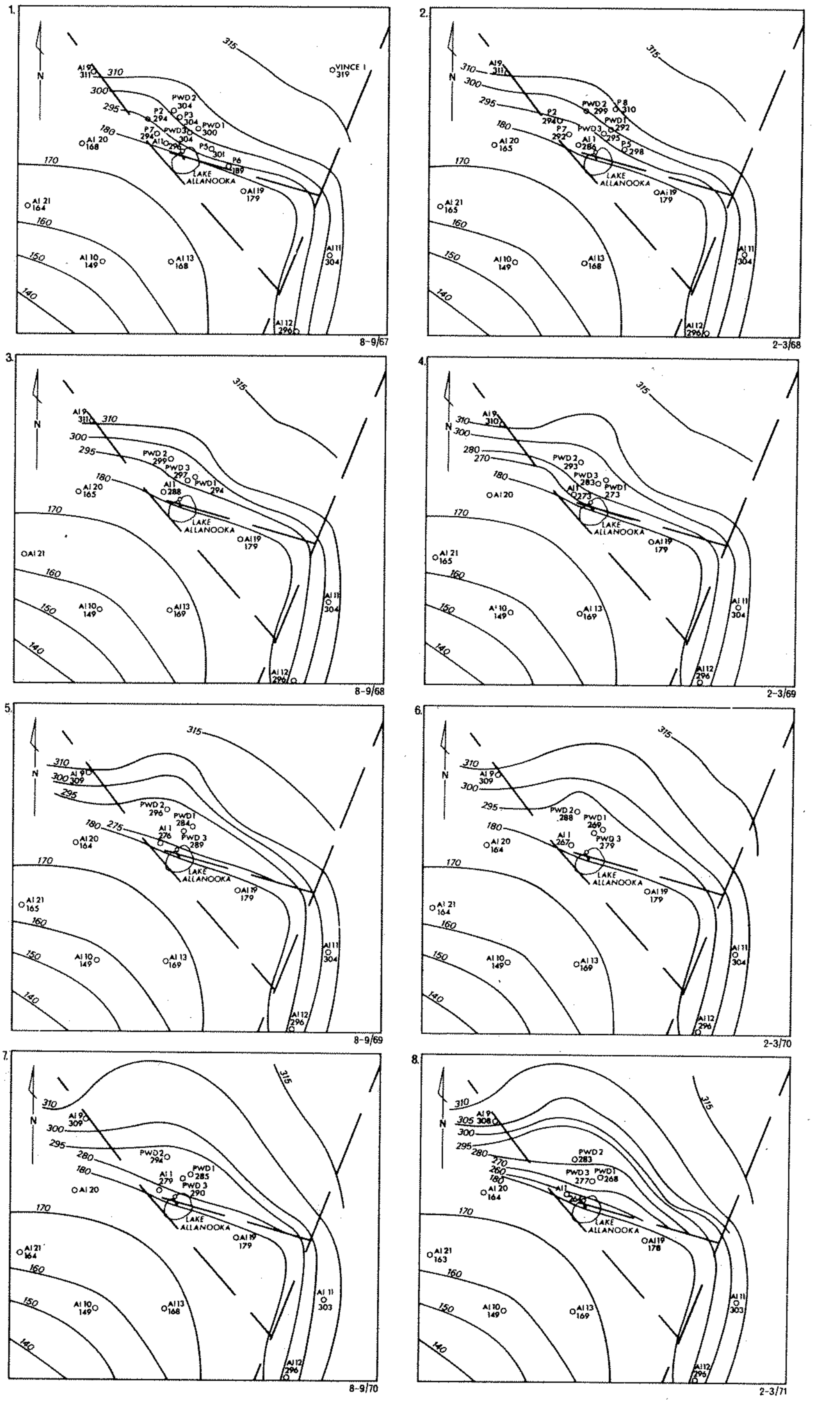
The area is covered by Quaternary coastal limestone in the southwest, and elsewhere by sandplains and laterite.

Geophysical investigations and drilling logs suggest a complex pattern of faulting (Fig. 1).

DATA

Water level observations have been made by Public Works Department (P.W.D.) staff at weekly, monthly or 3-monthly intervals at a series of observation wells. Results of the observations are shown in Figures 3 and 4, and Table 1.

FIGURE 3



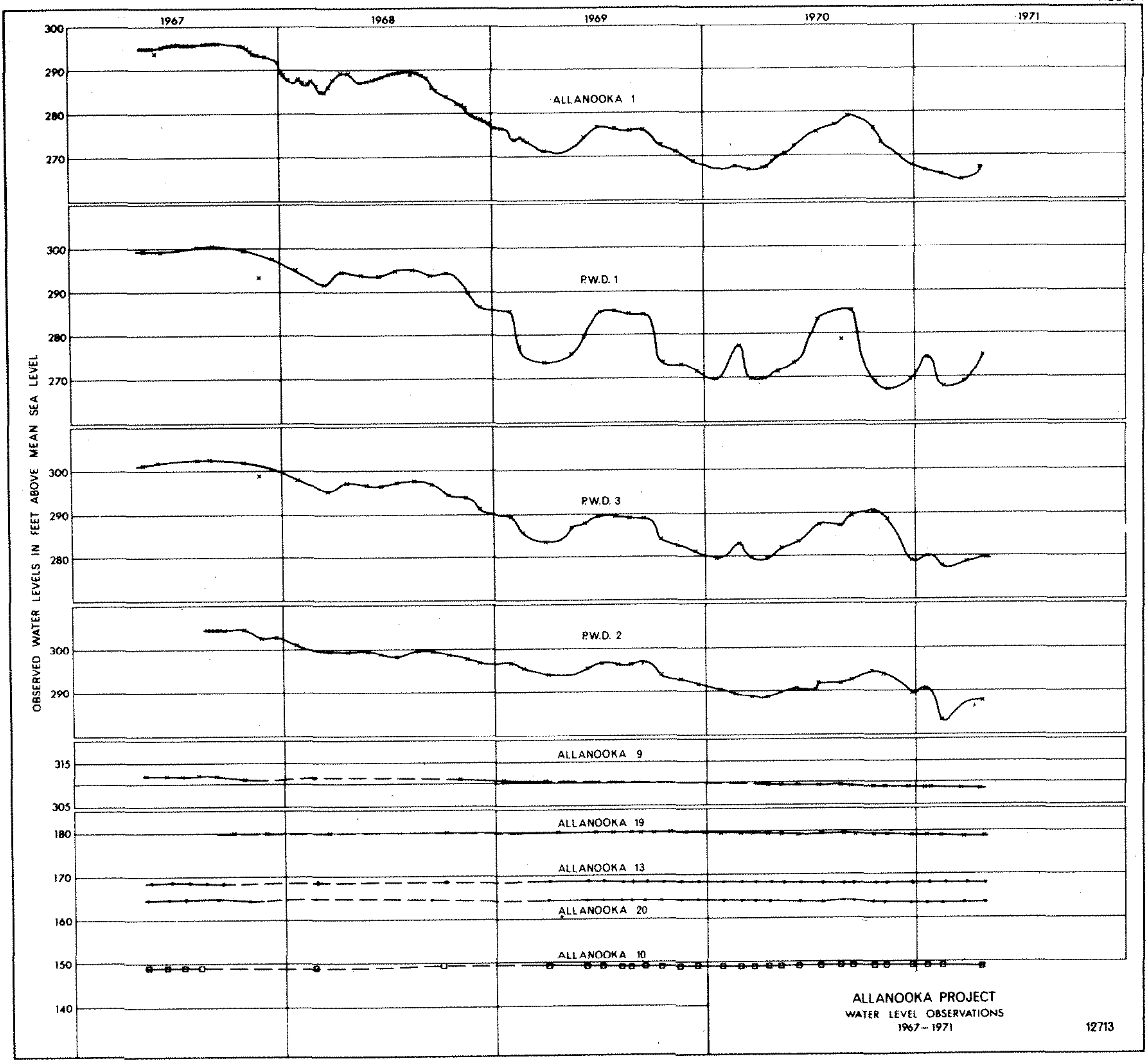
N.B. Irregular contour intervals have been used.

ALLANOOKA PROJECT
VARIATION IN WATER REST LEVELS
AT 6 MONTHLY INTERVALS FOR
PERIOD 8-9/67 TO 2-3/71

SCALE OF MILES
0 1/2

12716

FIGURE 4



ALLANOOKA PROJECT
WATER LEVEL OBSERVATIONS
1967 – 1971

TABLE 1. Water Level Observations

BORE	28-4-67	21-4-67	5-5-67	19-5-67	12-5-67	2-6-67	9-6-67	16-6-67
Production								
1	295.7	294.2	294.2	294.5	293.8	294.7		294.9
2	293.4	293.3	293.5	243.5	293.4	293.7	293.7	293.6
3	303.2	303.1	303.3	303.2	303.3	303.4	303.4	303.5
4						302.7	302.2	304.3
5	300.2	299.7	300.1	300.1	300.2	303.3	300.3	300.4
6								
7								
8								
11								
12								

	23-6-67	30-6-67	7-7-67	14-7-67	28-7-67	21-7-67	4-8-67	18-8-67
Production								
1	295.0	295.1	295.3	295.0	295.5	295.4	295.6	295.6
2	293.3	292.9	292.8	293.8	294.0	294.2	294.2	294.4
3	303.5	303.5	303.6	303.6	303.6	303.7	303.7	303.8
4			301.7	301.7	301.9	301.9	301.9	301.9
5	300.5	300.5	300.6	300.7	300.7	300.7	300.9	301.0
6		188.1	188.2	188.7	188.8	188.8	188.8	188.8
7								294.1
8								
11								
12								

	25-8-67	1-9-67	11-9-67	15-9-67	23-10-67	27-10-67	3-11-67	10-11-67
Production								
1	295.7	295.7	295.9	295.8				
2	294.4	294.4	294.7	294.7	294.4	294.3	294.3	292.0
3	303.8	303.9	304.0	303.9	303.8	303.8		
4	301.4	301.9	302.4	301.9				
5	301.0	301.0	301.1	301.1	300.7	300.6	299.9	297.0
6	188.9	189.0	189.0	189.1				
7	294.2	294.1	294.1	294.4				
8								
11								
12								

	17-11-67	19-12-67	26-12-67	2-1-68	9-1-68	16-1-68	4-2-68	9-2-68
Production								
1								
2	293.7							
3								
4								
5	297.9							
6								
7	292.3							
8		310.1	310.1	310.0	310.0	309.9	309.9	309.7
11								
12								

TABLE 1 (continued)

BORE	20-2-68	23-2-68	1-3-68	11-3-68				
8	309.7	309.7	309.5	309.4				
	20-10-70	10-11-70	22-12-70	19-1-71	16-2-71	22-3-71	22-4-71	
11	305.6	305.7	305.4	305.1	305.1	304.8	304.7	
12	309.7	309.8	309.6	309.4	309.5	309.5	309.4	
	19-5-67	3-7-67	20-2-68	4-6-69	1-7-69	28-8-69	16-9-69	18-12-69
Allanooka								
5		155.6		157.6	156.5		156.8	
11	303.8	303.9	303.9		303.6	303.7	303.6	303.6
12		296.0	296.0		296.1	296.1	296.1	296.1
14		142.6	142.8		143.3	143.3	143.3	143.3
15		312.9	313.1		312.8	313.0	312.9	312.8
16		137.4	137.5		137.6	137.6	137.7	137.7
17		310.0	310.2		310.1	310.1	310.2	310.2
18		53.7	54.1		54.9	54.8	54.9	54.9
21		164.3	164.8		164.6	164.7	164.6	164.5
	20-3-70	8-6-70	10-9-70	22-12-70	22-3-71			
Allanooka								
5								
11	303.6	303.6	303.5	303.5	303.5			
12	296.2	296.1	296.1	296.1				
14	143.3		143.3	143.0	143.2			
15	312.8	312.9	313.0	312.8	312.8			
16	137.7	137.7	137.8	137.7	137.8			
17	310.2	310.0	310.2	310.2	310.1			
18	54.8	54.8	54.8	54.8	54.8			
21	164.3	164.1	164.2	164.1	162.9			

Total monthly pumpage has been provided by the P.W.D. (Table 2).

TABLE 2. Monthly Pumpage in Millions of Gallons

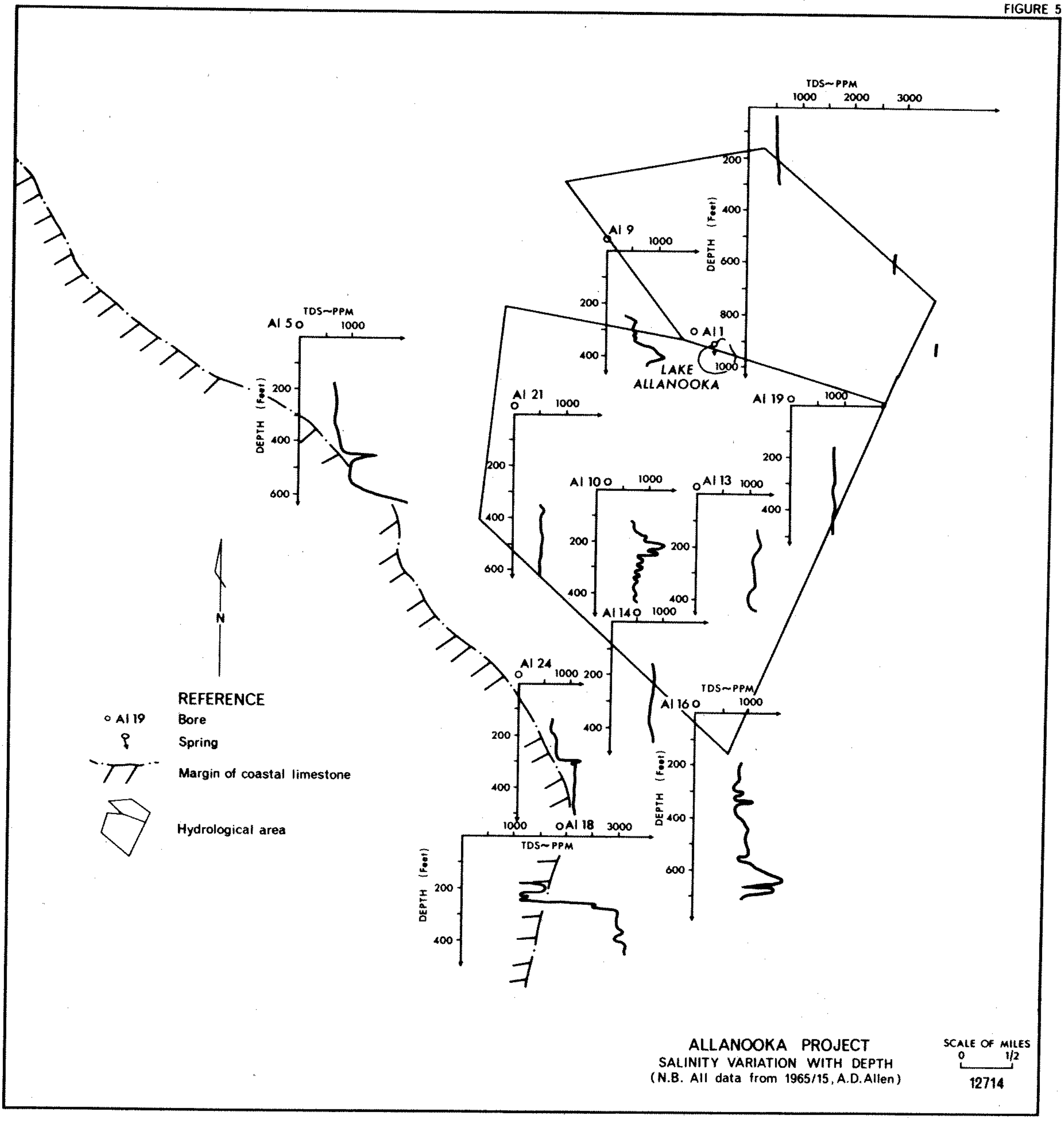
MONTH	1967	1968	1969	1970	1971
Jan.		38.1	81.9	101.8	109.6
Feb.		111.2	66.6	61.5	87.2
Mar.		43.9	64.0	81.5	83.2
Apr.		3.3	47.0	60.9	70.7
May		35.8	35.5	41.2	
June		18.6	33.8	72.6	
July		25.0	36.2	31.5	
Aug.		21.0	32.7	30.7	
Sept.		30.0	45.9	35.3	
Oct.		39.1	69.3	75.5	
Nov.	18.1	69.0	88.7	78.6	
Dec.	28.4	58.2	79.8	84.5	
Total	46.5	493.2	681.4	755.6	350.7

Specific capacity tests were made by the Geological Survey on some exploratory bores, and the P.W.D. has made drawdown and recovery tests on most production bores, and one controlled test on Production 4.

HYDROGEOLOGY

Figure 2 is a contour map of the groundwater surface, prepared from bore rest levels measured just prior to the start of pumping. From the data available it is difficult to derive such a map with any certainty and this is the writer's own interpretation.

FIGURE 5



ALLANOOKA PROJECT
SALINITY VARIATION WITH DEPTH
(N.B. All data from 1965/15, A.D.Allen)

SCALE OF MILES 0 1/2

12714

Allen has shown that seasonal fluctuation in the groundwater surface is likely to be small. Figure 2 may therefore be taken as representative of the stable flow conditions prevailing before production began.

The contour pattern indicates hydraulic continuity through the area with flow from northeast to southwest. The main intake is beyond the area being considered in this report. Recharge was reported by Allen as being from rainfall upon Yarragadee Formation outcrops or from concentration of runoff along drainage lines. A flexure in groundwater level contours at Banksia Gully Creek may indicate local groundwater recharge, confirming Allen's observations during a wet season.

Groundwater flow from the area is to the southwest, where coastal limestone overlies the Yarragadee Formation. Salinities increase in the direction of flow (Fig. 5) and exploratory wells 5, 18 and 24 show that near the coastal limestone the potable water appears to be wedging out over water of much higher salinity.

To the north of Lake Allanooka the groundwater surface is higher than 300 feet (90 m) above mean sea level, and to the south and west it is below 180 feet (55 m). The flow pattern and drilling records indicate hydraulic continuity, and the abrupt change in water levels is attributed to a zone of very low permeability related to fault activity on the Horwood, Vince and Banksia Gully Faults. Allen has reported *in situ* weathering of the feldspar grains, which when pulverized readily turn to clay. Fault activity could have produced zones of low permeability wherever post-weathering fault movement has occurred.

Well logs indicate thick (300+ feet, 90+ m) unconfined aquifers to the south and west of Lake Allanooka and a moderately thick (110 feet, 34 m) partly confined aquifer to the north. East of Banksia Gully Fault the aquifers are thin and are fully confined.

Water level, well logs and saturated aquifer thicknesses suggest the existence of three hydrological areas (Fig. 1):

1. Eastern Sector;
2. Northern Sector;
3. Southern Sector.

The boundaries are assumed to be fault controlled but in some cases have been chosen from other considerations.

EASTERN SECTOR

East of Banksia Gully Fault, aquifers are thin and fully confined, and will yield only small quantities of water. Because of the lenticular nature of the sediments, prediction in this area is difficult and has an element of uncertainty. Water level contours show that flow is to the southwest and will recharge the Southern Sector.

Flow gradients are steeper than elsewhere, suggesting that permeabilities may be low. The area is not recommended for further exploration or for production, especially since the flow is to the Southern Sector from which it could be pumped.

NORTHERN SECTOR

This is the present production area and is bounded by Vince Fault to the south, Bookara Fault to the north, Horwood Fault to the west, Banksia Gully Fault to the east, and, for this discussion, is terminated to the northeast by an arbitrarily chosen line which approximately corresponds to the 700-foot (210 m) ground level contour. It is unlikely that there will be production beyond this line, as the natural surface is more than 400 feet (120 m) above the groundwater level. In later calculations in this report lateral flow through this boundary is treated as recharge to the production area.

Outflow passes into the Southern Sector through the zone of low permeability at Vince and Horwood Faults.

Water level contours indicate that there may be vertical recharge from Banksia Gully Creek.

SOUTHERN SECTOR

As yet there has been little production from this area which is bounded by Vince Fault to the north, Banksia Gully Fault to the east, a groundwater divide to the west, and is limited on its southern side by the 120-foot (37 m) water level contour (simplified to a straight line). This limitation has been arbitrarily imposed to exclude the more saline water encountered to the south and west.

Lateral inflow is from the Northern and Eastern Sectors and Allen has reported vertical recharge from Allanooka Creek, which can flow only when Lake Allanooka is overfull. Groundwater flow from the area is to the more saline waters of the coastal limestone.

HYDROLOGY

EASTERN SECTOR

No quantitative estimates have been made for this area as it has very limited potential.

NORTHERN SECTOR

COEFFICIENT OF STORAGE

Figure 3 shows the water level contours at 6-monthly intervals, and from these it has been possible to calculate the volume of aquifer dewatered since production began. The progressive dewatering and cumulative pumpage are shown in Figure 6.

From the dewatering curve it is possible to calculate a coefficient of storage for the aquifers.

The general equation of continuity is:

$$I = O + \triangle S_t$$

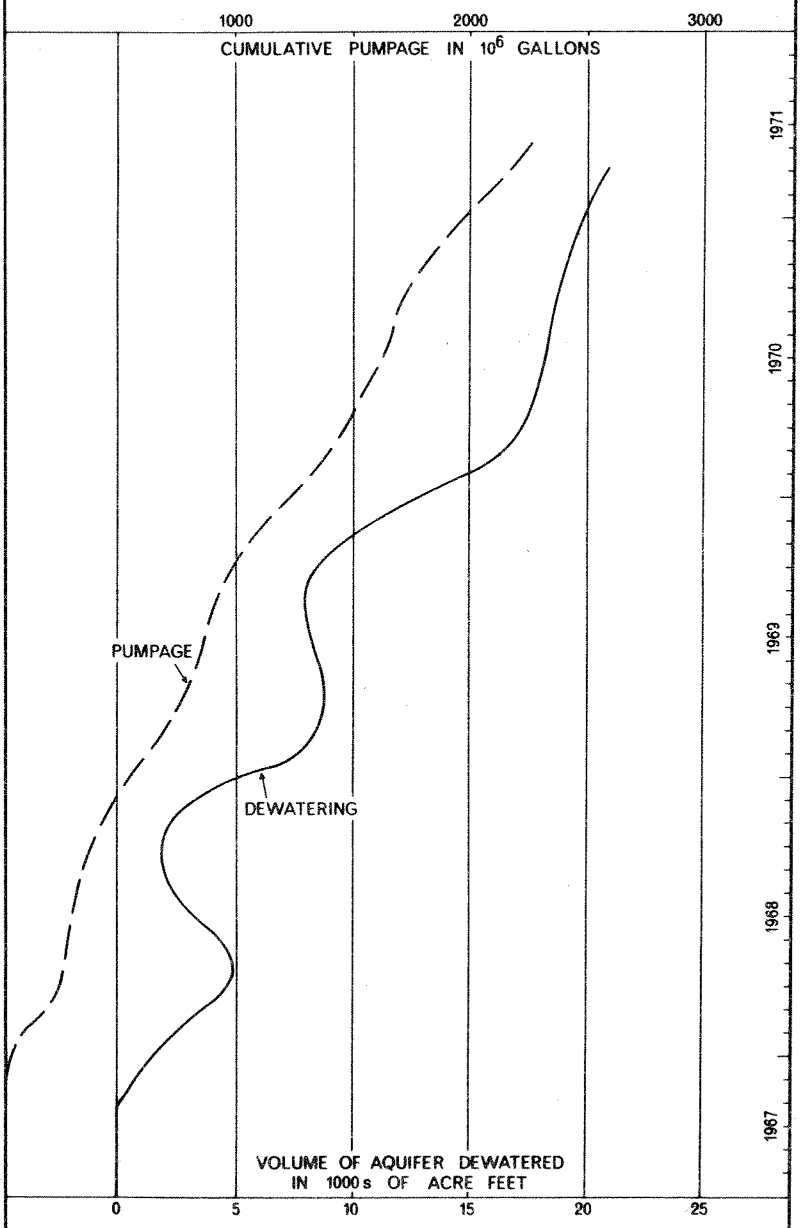
where I = inflow

O = outflow

$\triangle S_t$ = change in storage.

In this case the inflow has two components: lateral inflow and vertical recharge. The application of the equation is restricted to periods when vertical recharge is negligible, i.e. the dry season. Outflow also has two components; pumpage and lateral outflow of which the pumpage is known. If the aquifers are undisturbed, over a short period of time

FIGURE 6



ALLANOOKA PROJECT
AQUIFER DEWATERING AND
CUMULATIVE PUMPAGE

12708

$\triangle S_t$ would normally equal zero and lateral inflow would equal lateral outflow. If the aquifers were pumped, $\triangle S_t$ would be positive and lateral outflows would slowly reduce. The response of lateral outflow to changing aquifer storage is normally very slow, and during a short period of changing storage, lateral inflows and outflows will still be equal.

Solution of the equation for the Northern Sector was made for the period December 1967 to March 1968 when vertical recharge could be assumed to be negligible. During this short period, lateral outflow should still nearly equal lateral inflow, and the observed change in aquifer storage can be directly related to pumpage.

A volume of 3,150 acre feet (3.89×10^6 m^3) of aquifer was dewatered while 810 acre feet (1.00×10^6 m^3) of water were pumped. The coefficient of storage is therefore $\frac{810}{3{,}150} = 0.26$ which is reasonable for the Allanooka sandstones. The calculations were not repeated for the 69/70 and 70/71 dry seasons because lateral inflows and outflows were obviously out of balance.

VERTICAL RECHARGE

Using a continuity equation during the wet season will give a value for vertical recharge in an area which is bounded by suitably located groundwater divides or physical boundaries. A time period is chosen for which the aquifer storage and hydraulic gradients are the same at the beginning and the end (Fig. 6). Lateral inflows have then been equal to lateral outflows and variation in aquifer storage is due to vertical recharge and pumpage. As storage has returned at the end of the period to its original value, pumpage and vertical recharge must be equal. The method may underestimate vertical recharge but the errors involved need not necessarily be large.

Suitable wet season periods chosen for the Northern Sector were March to December 1968, and from April to mid-October 1969. The area has fault boundaries except to the northeast where an arbitrary line drawn beyond the influence of pumping was used as a boundary. It can be assumed that lateral inflow through this boundary is constant. All changes in storage were thus within a designated area.

During the chosen time period, $\triangle S_t = O$ and lateral inflow and outflow are assumed to be equal. Under these conditions, pumpage should equal vertical recharge.

In 1968, vertical recharge was calculated at 300×10^6 gallons (1.36×10^6 m^3) and in 1969 at 283×10^6 gallons (1.29×10^6 m^3).

TRANSMISSIVITY

Specific capacity tests in some of the exploration bores were reported on by Allen. They were carried out with varying degrees of penetration, varying screen lengths and with screens in different positions in the aquifer. Two bores, Allanooka 1 and 5, were tested with full penetration and full length perforation. Using Walton's curves for the conversion of specific capacity to transmissivity, the respective values are 33,000 gpd/ft and 6,000 gpd/ft (490 m^3/d/m and 89 m^3/d/m). Allanooka 5 is very close to Vince Fault and is likely to be affected by the low permeability zone found there. Allanooka 1 is in the present production area and its calculated transmissivity of 33,000 gpd/ft (490 m^3/d/m) should be representative.

The P.W.D. has conducted drawdown and recovery tests on its production bores, and on Production 4 a controlled test using Production 1 and Production 3 as observation wells.

The test results were strongly affected by delayed yield as gravity drainage from storage and also by the effects of partial penetration. It was not possible to analyze the tests satisfactorily except in the case of Production 2 which was tested on 14th March, 1966. The effects of delayed yield were overcome on this test. Corrections for partial penetration were made to the drawdown data by the method outlined by Walton, and using Cooper and Jacob's straight-line analysis method a transmissivity value of 39,000 gpd/ft (580 $m^3/d/m$) was calculated.

Neither of the computed transmissivity values can be considered reliable, but on lithological considerations are probably of reasonable magnitude. Some further discussion on this subject can be found in Appendix 1.

SOUTHERN SECTOR

As yet there has been little production from this area and it has not been possible to make the types of analyses described in the previous section. The area is lithologically similar to the north and the same hydrological properties have therefore been assumed to apply.

GROUNDWATER INVENTORY

EASTERN SECTOR

No inventory has been prepared for this sector, which has poor groundwater potential. Water passes from it to the Southern Sector from where it can be recovered by pumping.

NORTHERN SECTOR

The area is 4.6 $miles^2$ (12 km^2), and bore logs show saturated thicknesses of 110 to 120 feet (33 to 36 m). With a storage coefficient of 0.26, potable water storage is about 23,000 x 10^6 gallons (105 x 10^6 m^3) or 5,000 x 10^6 gallons/$mile^2$ (8.8 x 10^6 m^3/km^2).

The contours on Figure 2 show a hydraulic gradient of 1.7 x 10^{-3} for groundwater flowing to the production area. A cross-sectional length of 20,000 feet (6,100 m) was measured along a strip between the 310 and 320-foot (94 and 98 m) contours. If an upper value for transmissivity is chosen at 40,000 gpd/ft (600 $m^3/d/m$), annual lateral inflow is 496 x 10^6 gallons (2.26 x 10^6 m^4).

Vertical recharge in 1968 and 1969 has been estimated at 300 x 10^6 and 283 x 10^6 gallons (1.36 x 10^6 and 1.29 x 10^6 m^3). If assumed to be approximately constant then the average annual lateral outflow prior to production was 496 + 300 = 796 x 10^6 gallons (3.61 x 10^6 m^3). Most of this quantity became lateral inflow to the Southern Sector while a small portion appeared as Ngarlingue Spring.

Aquifer dewatering to date is 1,480 x 10^6 gallons (6.73 x 10^6 m^3) or 6.5 per cent of the water stored in the upper aquifers. Pumpage for the same period totals 2,300 x 10^6 gallons (10.46 x 10^6 m^3) which is equivalent to a continuous rate of 1.82 x 10^6 gpd (0.00827 x 10^6 m^3/d). The average rate over the last 12 months is 2.19 x 10^6 gpd (0.00996 x 10^6 m^3/d). The closeness of the two rates suggests that the present state of dewatering should be reasonably representative of the longer term continuous rate, and not be too heavily influenced by the constantly occurring variations in production.

The difference between total pumpage and aquifer dewatering is 820 x 10^6 gallons (3.73 x 10^6 m^3), accounted for by reduction in outflow from the area. The time period for these observations is 3½ years, but for inventory purposes this may be regarded as three seasons only. If it is assumed for simplicity that pumpage effects were identical each season, then lateral outflow has been reduced by 273 x 10^6 gallons (1.24 x 10^6 m^3) per year or by 35 per cent.

The reduction will in fact have been less in the first year and greater in the third as the cone of depression extended. The reduction has been achieved very quickly and indicates an efficient use of resources, and shows that the present production field has been very well sited.

There are two geological factors which account for this efficiency:

1. Vertical recharge at Banksia Gully Creek is immediately "upstream" from the present production field, giving maximum opportunity for this water to be pumped before it can escape through the bore field.

2. A zone of very low permeability exists immediately "downstream" from the production field which will cause increased drawdowns in the production area. The reduction in head immediately "upstream" from the zone will reduce flow through the zone. The trap-like effect can be seen in Figure 3 where the changing flow pattern in the production field is illustrated.

Lateral outflow will continue to be reduced as pumpage continues. It can be anticipated that only very small quantities of water will, in the future, flow from the Northern into the Southern Sector.

The effects of future pumping are discussed in Appendix 1.

SOUTHERN SECTOR

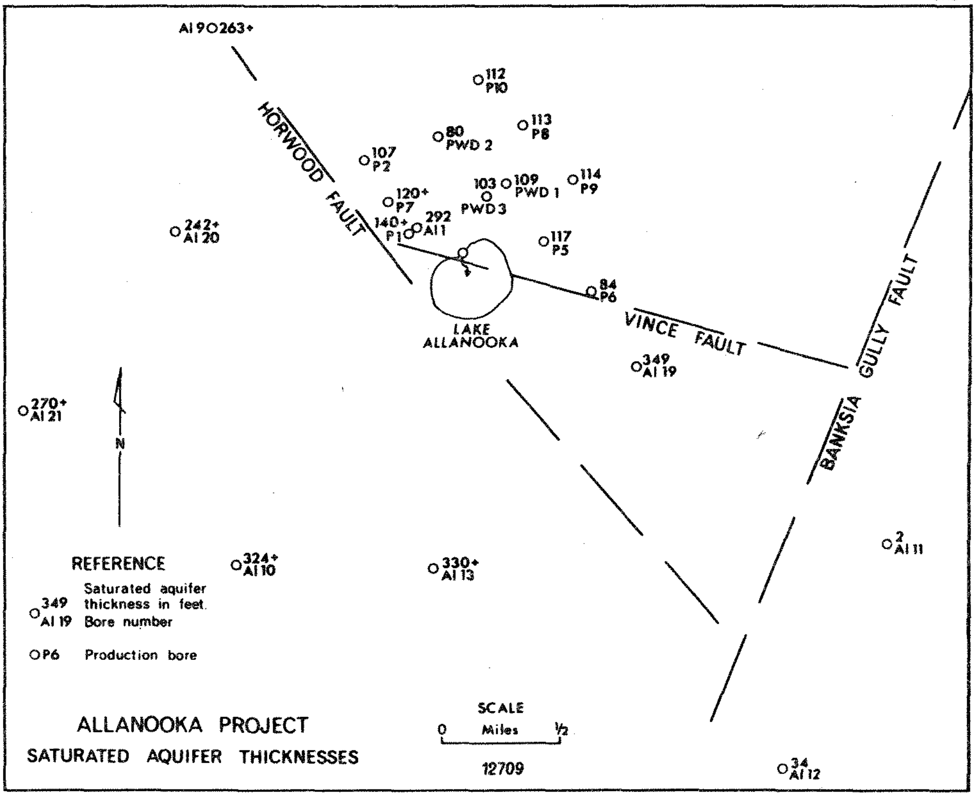
As yet the only producing wells in this area are P6 and P14. Production so far is low and the area is little changed from its original state. Reduction of flows from the Northern Sector may already have had some effect on aquifer storage (see water level records AL 9, 13, 19 and 20, Fig. 4).

The rate of vertical recharge is unknown, but overflow from Lake Allanooka has in the past contributed to groundwater replenishment. The pattern of water level contours indicates that most of the past lateral inflow was from the Northern Sector, and was estimated at 800 x 10^6 gallons/year (3.6 x 10^6 m^3/year). The hydraulic gradient on Figure 2 has been measured at 3.0 x 10^{-3}. A cross-sectional length of inflow of 21,500 feet (6,550 m) was measured at the 180-foot (54.9 m) water level contour. Assuming a similar transmissivity to the Northern Sector, then total lateral inflow to the Southern Sector was 942 x 10^6 gallons/year (4.28 x 10^6 m^3/year).

Lateral outflow from the Northern Sector is estimated to have been reduced by at least 273 x 10^6 gallons/year (1.214 x 10^6 m^3/year) since production began. Lateral inflows to the Southern Sector should now therefore be in the order of 1 x 10^6 gpd (4,550 m^3/d). Production in the Southern Sector must be based on present storage and until further data are available, replenishment should be assumed to be negligible.

The Southern Sector is just over 10 miles^2 (25.9 km^2) and the aquifers more than 300 feet (91 m) thick (Fig. 7). Adopting a storage coefficient of 0.26 as for the Northern Sector, then aquifer storage is approximately 14,000 x 10^6 gallons/mile^2 (24.6 x 10^6 m^3/km^2).

FIGURE 7



Appendix 2 includes an estimate of the effects of future production and suggestions for bore locations in the Southern Sector.

WATER QUALITY

Water with less than 1,000 ppm TDS can be expected within the upper aquifers (Fig. 5). The salinity will increase towards the southwest and with increasing depth.

There are no foreseeable salinity problems in the Northern Sector but in the Southern Sector there is a possibility of saline encroachment from the southwest. Calculations on the effects of abstraction in the Southern Sector indicate that considerable withdrawals may be safely made if production is kept to the area indicated (Appendix 2).

Investigation bores Al 10 and Al 13 showed water of slightly higher salinity than was encountered in neighbouring bores (Fig. 5). The variation appears to be localized, and because the higher salinity water is only 1,000-1,200 ppm TDS it could be diluted with the lower salinity water that is more generally available.

CONCLUSIONS AND RECOMMENDATIONS

1. The present production area is well sited to make use of vertical recharge and underflow. No expansion or increase in abstraction rates is recommended within the Northern Sector.

2. Future expansion of production is recommended for the Southern Sector where, on the basis of present hydrological knowledge, abstractions are assumed to come from present storage. Bores should be located on lines running southeast from the pipeline (not shown). Such a layout will take advantage of any vertical recharge and underflow that may occur. The suggested layout in Figure 9 is also designed to avoid contamination from more saline water to the southwest.

3. The present programme of water level observations should be continued.

4. Salinity observations should be made at 1 to 3-monthly intervals in Allanooka 18 and 5.

5. There is no apparent geological reason for Production 13 being dry. Redrilling nearby is recommended.

6. Very limited hydraulic data on the aquifers were available for this report. It is suggested that when new production wells are drilled, controlled pumping tests of sufficient duration be carried out to more accurately determine aquifer properties.

REFERENCES

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Walton, W. C., 1962, Selected analytical methods for well and aquifer evaluation: Illinois State Water Survey Bull. 49.

Cooper, H. H., Jr., and Jacob, C. E., 1946, A generalized graphical method for evaluating formation constants and summarizing well-field history: Am. Geophys. Union Trans. v.27 (4).

APPENDIX 1

NORTHERN SECTOR

EFFECTS OF LONG-TERM PUMPING

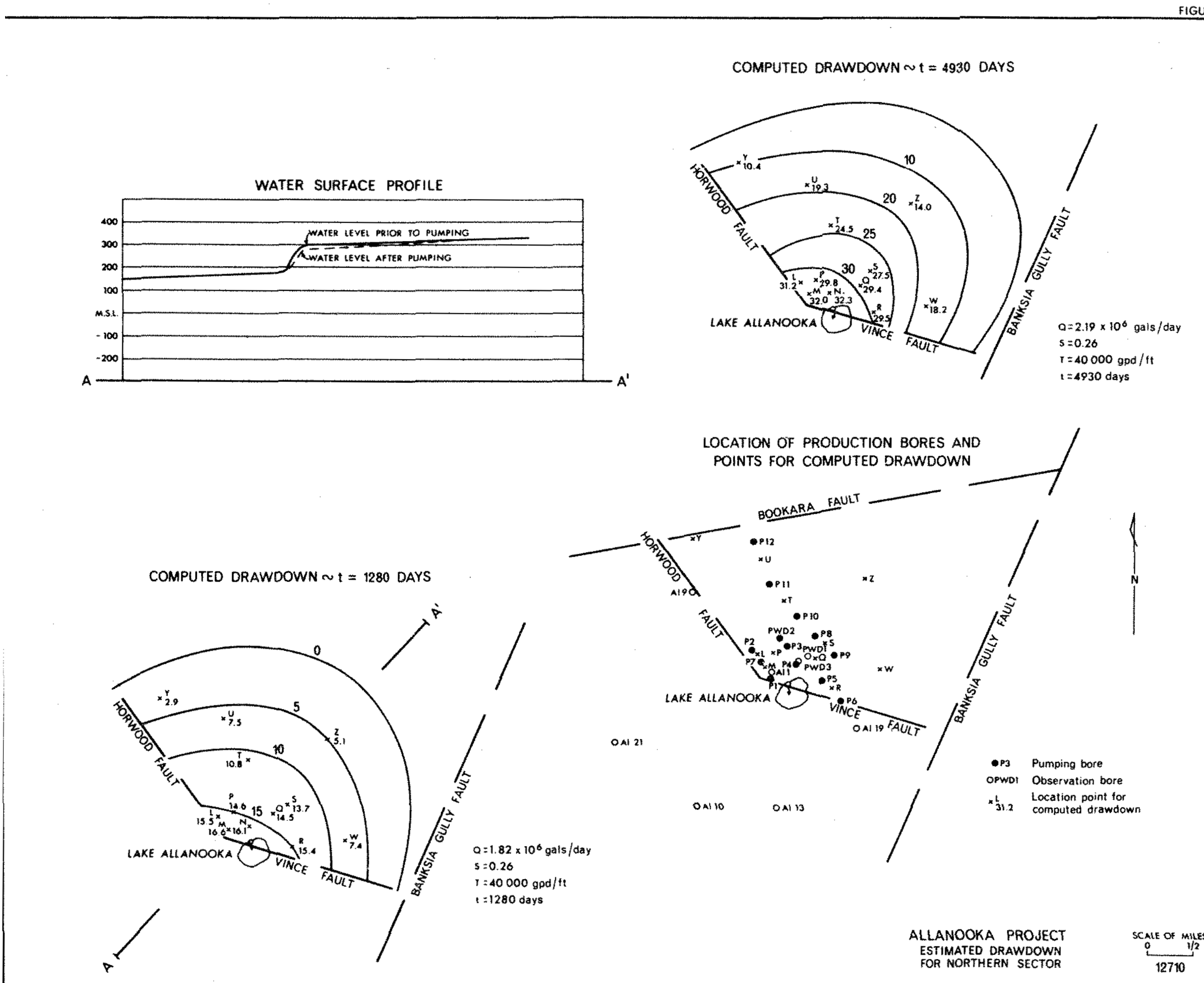
Hydraulic properties were assigned to the upper aquifer and the effects of an imposed pumping programme calculated by standard techniques.

The following conditions were applied:

1. Horwood Fault, Vince Fault and Banksia Gully Fault were assumed to be impermeable boundaries.

2. The storage coefficient used was 0.26 as derived earlier.

3. The 12 production bores were assumed to be pumping at a steady, uniform and continuous rate (Fig. 8).



4. Production 6 is very close to the Vince Fault and was assumed to be in the Northern Sector, though observed water levels indicate that it should be placed in the Southern Sector. Strata descriptions recorded in the bore log suggest it belongs to the Northern Sector. The bore in practice is a poor producer and any error in these calculations caused by placing it in the north will be small.

5. Calculations have been made with transmissivity values of 35,000, 40,000 and 45,000 gpd/ft (522, 600 and 671 $m^3/d/m$). As discussed below a value of 40,000 gpd/ft (600 $m^3/d/m$) was adopted for the final computations.

COMPUTATION PROCEDURE

Production started in November, 1967 and records were available to May 1971, a total period of 1,280 days. The average pumping rate for the period is 1.82×10^6 gpd (8,270 m^3/d). Computation techniques were tested by calculating aquifer dewatering for a 1,280-day period using the above pumping rate and then comparing with observed dewatering.

The 12 P.W.D. bores were each assumed to be pumping at a steady rate of 0.151 x 10^6 gpd (686 m^3). Using the Theis equation (and with appropriately placed image wells) drawdowns were calculated for location points L, M, N, P, Q, R, S, T, U, W, Y, and Z (Fig. 8). A contour map of aquifer drawdown was then prepared and the change in aquifer storage found by planimetering the map.

The computations were successively made for transmissivity values of 35,000, 40,000 and 45,000 gpd/ft (522, 600 and 671 m^3/d) with the following results:

Transmissivity gpd/ft	Calculated Dewatering: Volume aquifer dewatered acre feet	Volume of water 10^6 gallons
35,000	26,900	1,900
*40,000	23,700	1,670
45,000	21,200	1,500
Observed Dewatering (Fig. 7)	20,880	1,480

* See Table 3 and Figure 8

Vertical recharge to the area may be of the order of 300 x 10^6 gallons/year (1.36 x 10^6 m^3/year). The computation procedures do not contain any allowance for this and should therefore over-estimate aquifer dewatering. The Horwood and Vince Fault boundaries are not completely impermeable and will therefore tend to reduce the difference between computed and observed dewatering.

Since calculated dewatering should be greater than that observed, the 45,000 gpd/ft (671 m^3/d/m) transmissivity calculations probably give too low a value. The 40,000 gpd/ft (600 m^3/d/m) value was adopted for further work.

As a further check on the adopted hydraulic values, drawdowns were computed for observation wells as follows:

Observation bore	Observed drawdown (feet)	Computed drawdown (feet)
P.W.D. 1	25	20
P.W.D. 2	16	18
P.W.D. 3	22	22
Allanooka 1	28	21

The results are considered satisfactory because the calculated drawdowns based on uniform hydraulic conditions agree reasonably well with the observed drawdowns despite the lack of uniformity which actually exists.

TABLE 3. Northern Sector – Calculated Drawdown in Present Production Area

Q = 1.82 x 10^6 gpd ≡ 0.151 x 10^6gpd/bore
T = 40,000 gpd/ft S = 0.26 t = 1,280 days
R = Real Bore I = Image Bore

Location		P 1	P 2	P 3	P 4	P 5	P 6	P 7	P 8	P 9	P10	P11	P12	Total
L	R	1.1	1.7	1.0	0.7	0.5	0.3	1.7	0.6	0.5	0.7	0.6	0.2	15.5
	I	0.8	0.9	0.6	0.4	0.5	0.3	0.9	0.4	0.3	0.4	0.3	0.1	
M	R	1.7	1.2	1.2	0.9	0.7	0.4	1.7	0.7	0.6	0.7	0.4	0.3	16.6
	I	0.9	0.9	0.6	0.6	0.4	0.4	0.9	0.3	0.3	0.4	0.2	0.2	
N	R	1.3	0.8	1.2	1.3	1.0	0.7	1.0	0.9	0.9	0.7	0.4	0.1	16.1
	I	0.7	0.6	0.4	0.7	0.7	0.7	0.7	0.3	0.5	0.2	0.2	0.1	
P	R	1.3	1.2	0.2	1.0	0.7	0.5	1.5	0.8	0.3	0.9	0.6	0.2	14.6
	I	0.7	0.7	0.5	0.6	0.5	0.5	0.7	0.3	0.4	0.2	0.2	0.1	
Q	R	0.8	0.5	1.0	1.5	1.2	0.8	0.7	1.2	1.5	0.8	0.3	0.2	14.5
	I	0.4	0.3	0.2	0.3	0.7	0.8	0.4	0.1	0.5	0.1	0.1	0.1	
R	R	0.7	0.4	0.6	1.0	1.7	1.7	0.5	0.7	1.0	0.5	0.2	0.1	15.4
	I	0.4	0.2	0.2	0.8	1.2	1.7	0.3	0.4	0.8	0.1	0.1	0.1	
S	R	0.7	0.6	1.0	1.3	1.0	0.6	0.6	1.5	1.5	0.9	0.6	0.2	13.7
	I	0.4	0.3	0.2	0.2	0.6	0.6	0.2	0.1	0.4	0.1	0.1	0.1	
T	R	0.5	0.7	0.8	0.6	0.4	0.3	0.6	0.6	0.5	1.2	1.3	0.5	10.8
	I	0.2	0.5	0.2	0.3	0.2	0.3	0.4	0.1	0.2	0.1	0.2	0.1	
U	R	0.2	0.5	0.4	0.3	0.2	0.1	0.3	0.4	0.3	0.6	1.3	1.2	7.5
	I	0.1	0.2	0.2	0.1	0.1	0.1	0.2	0.2	0.1	0.1	0.2	0.1	
W	R	0.2	0.2	0.3	0.5	0.6	0.8	0.2	0.4	0.7	0.2	0.1	0.1	7.4
	I	0.1	0.1	0.2	0.3	0.5	0.8	0.1	0.3	0.4	0.1	0.1	0.1	
Y	R	0.1	0.2	0.1	0.1	0.1	–	0.1	0.1	0.1	0.1	0.4	0.6	2.9
	I	0.1	0.1	0.1	–	–	–	0.1	0.1	–	0.1	0.2	0.1	
Z	R	0.2	0.2	0.3	0.3	0.3	0.2	0.2	0.6	0.6	0.6	0.4	0.3	5.1
	I	0.1	0.1	0.1	0.1	0.2	0.2	0.1	–	–	–	–	–	

To test the effects of longer term pumping, the calculations were then extended for a further 1, 2, and 10 years (Table 4 and Figure 8) with an abstraction rate of 2.19 x 10^6 gpd (9,960 m^3/d), a transmissivity value of 40,000 gpd/ft (600 m^3/d/m) and a storage coefficient of 0.26 to give the following results:

Time (days)	Volume aquifer dewatered (acre feet)	Volume of water (10^6 gallons)
1,280 + 365 = 1,645	30,400	2,140
1,280 + 730 = 2,010	32,100	2,260
*1,280 + 3,650 = 4,390	59,200	4,170

* See Figure 8

TABLE 4. Calculated Drawdown in Present Production Area

S = 2.19 x 10^6 gpd ≡ 0.1825 x 10^6 gpd/bore
T = 40,000 gpd/ft S = 0.26 t = 4,930 days
R = Real Bore I = Image Bore

Location		P 1	P 2	P 3	P 4	P 5	P 6	P 7	P 8	P 9	P10	P11	P12	Total
L	R	2.0	2.5	2.0	1.5	1.1	0.8	2.5	1.3	1.1	1.4	1.1	0.6	31.20
	I	1.6	1.8	1.3	1.0	1.1	0.8	1.8	0.9	0.8	0.9	0.8	0.5	
M	R	2.5	2.2	2.1	1.7	1.4	1.0	2.5	1.4	1.2	1.3	1.0	0.6	32.0
	I	1.8	1.6	1.2	1.1	1.0	1.0	1.8	0.7	0.8	0.9	0.7	0.5	
N	R	2.3	1.6	2.2	2.3	1.8	1.4	2.0	1.7	1.6	1.4	0.9	0.5	32.3
	I	1.4	1.1	0.9	1.5	1.5	1.4	1.4	0.6	1.1	0.8	0.6	0.3	
P	R	2.3	2.1	0.6	1.9	1.4	1.0	2.4	1.6	0.9	1.7	1.2	0.6	29.8
	I	1.4	1.5	1.0	1.2	1.1	1.0	1.5	0.8	0.8	0.7	0.7	0.4	
Q	R	1.5	1.1	1.8	2.4	2.2	1.6	1.4	2.1	2.4	1.5	0.8	0.5	29.4
	I	0.9	0.8	0.6	1.2	1.5	1.6	0.9	0.5	1.0	0.5	0.4	0.2	
R	R	1.4	0.9	1.2	1.8	2.5	2.5	1.1	1.4	1.8	1.0	0.6	0.4	29.5
	I	0.9	0.7	0.7	1.6	2.1	2.5	0.8	1.0	1.6	0.5	0.3	0.1	
S	R	1.4	1.1	1.8	2.1	1.8	1.3	1.2	2.4	2.4	1.7	1.2	0.6	27.5
	I	0.9	0.7	0.6	0.6	1.2	1.3	0.7	0.5	0.9	0.4	0.4	0.3	
T	R	1.1	1.4	1.6	1.2	0.9	0.6	1.3	1.2	1.0	2.1	2.2	1.1	24.5
	I	0.8	1.0	0.6	0.7	0.7	0.6	0.9	0.5	1.6	0.5	0.6	0.4	
U	R	0.7	1.0	1.0	0.8	0.6	0.4	0.8	0.9	0.7	1.3	2.1	2.1	19.3
	I	0.5	0.8	0.5	0.4	0.4	0.4	0.6	0.5	0.3	0.5	1.6	0.4	
W	R	0.7	0.5	0.8	1.0	1.3	1.6	0.6	0.9	1.5	0.7	0.4	0.3	18.2
	I	0.5	0.4	0.6	0.8	1.0	1.6	0.5	0.8	1.0	0.3	0.3	0.1	
Y	R	0.4	0.5	0.5	0.3	0.3	0.2	0.5	0.4	0.3	0.5	0.9	1.3	10.4
	I	0.3	0.5	0.4	0.2	0.2	0.2	0.4	0.4	0.1	0.4	0.6	0.6	
Z	R	0.6	0.6	0.8	0.8	0.7	0.7	0.6	1.1	1.1	1.1	1.0	0.7	14.0
	I	0.4	0.4	0.3	0.4	0.6	0.7	0.4	0.2	0.2	0.2	0.3	0.1	

Calculations were also made to check on the effect of change in water levels in the present production bores over the 10-year period. The drawdown calculations assumed uniform pumping and aquifer conditions.

Bore	Time (days)	Calculated drawdown (feet)	Depth from TOS* to original water level (feet)
Production 1	4,930	43	110
2	4,930	41	77
3	4,930	42	59
4	4,930	43	80
5	4,930	41	90
6	4,930	49	65
7	4,930	43	92
8	4,930	38	93
9	4,930	38	91
10	4,930	37	83

* TOS – Top of screen

The results indicate that abstraction for a further 10 years at this rate is feasible. In practice there will be considerable variation in the actual drawdowns. Pumping rates will be variable and for short periods drawdowns can be expected to be much greater than indicated above.

APPENDIX 2

SOUTHERN SECTOR

EFFECTS OF LONG-TERM PUMPING

An assessment of the effects of long-term pumping was made by assigning hydraulic properties to the upper aquifer in the area and then imposing a pumping programme.

The following conditions were applied:

1. Horwood Fault, Vince Fault and Banksia Gully Fault were assumed to be impermeable boundaries.

2. Pumping was at a uniform continuous rate of 0.5×10^6 gpd (2,273 m^3/d) for each of 10 wells over a 10-year period.

3. Selected well locations are shown on Figure 9. It has been assumed that P13 can be successfully re-drilled, and this bore and P14 have been included in the bore field. Bore locations are based on lines trending southeast from the existing pipelines to take advantage of flow direction and possible recharge. Sites have been selected as far as possible to avoid future contamination from more saline water from the west and south. Of the suggested bore sites shown, P13, P14, Pp1, Pp3, Pp4, Pp5, Pp6, Pp7, Pp8 and Pp9 were used in the calculations. Sites Pp2, Pp10 and Pp11 could be used but it is preferable for production to be kept well to the north of the more saline water.

4. Aquifer properties used were:

   T = transmissivity = 40,000 gpd/ft (600 m^3/d/m)

   S = storage coefficient = 0.26.

COMPUTATION PROCEDURE

Using the Theis equation, drawdowns at various radii were calculated for a single well in an aquifer with the above properties, pumping for 3,650 days at a constant rate of 0.5 x 10^6 gallons per day (2,273 m^3/d). Using this curve of drawdown it is possible to calculate the drawdown at any point caused by all 10 wells pumping together. To allow for the effects of impermeable boundaries, image well positions for all 10 wells were located and boundary effects were calculated. Drawdowns were thus calculated for points A to K (Fig. 9) and a contour map of the drawdown over the whole area was drawn.

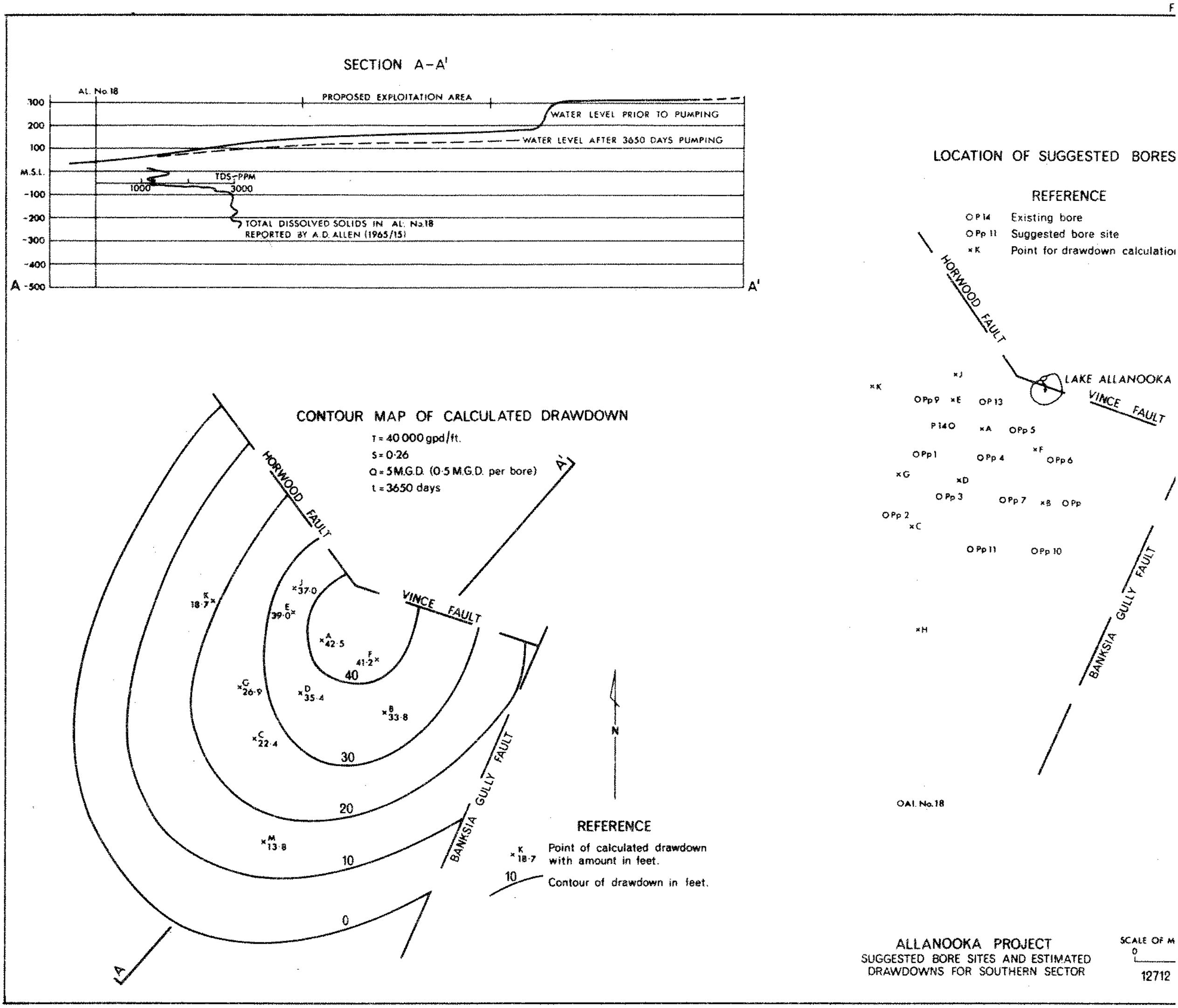


TABLE 5. Calculated Drawdowns on Proposed Exploitation Area

Q = 0.5×10^6 gpd
t = 3,650 days
R = Real Bore

T = 40,000 gpd/ft
S = 0.26
I = Image Bore

Location		Drawdown at point location according to pumping source										Drawdown Total
		P13	P14	Pp1	Pp3	Pp4	Pp5	Pp6	Pp7	Pp8	Pp9	
A	R	6.3	5.8	4.4	4.1	5.8	5.8	4.1	4.1	3.4	4.4	69.2
	I	3.4	2.6	1.8	1.2	1.8	2.8	2.0	1.4	1.7	2.3	
B	R	3.2	3.0	2.8	3.4	4.1	4.1	5.8	5.4	7.0	2.0	55.9
	I	2.2	1.7	1.2	0.9	1.4	2.3	1.6	2.0	1.4	1.4	
C	R	2.6	3.4	4.4	5.8	3.6	2.6	2.5	3.9	2.5	3.0	41.3
	I	1.2	0.9	0.5	0.4	0.5	1.0	0.7	0.5	0.5	0.8	
D	R	3.9	4.4	5.0	7.0	5.8	4.4	3.6	5.4	3.4	3.4	59.6
	I	2.2	1.7	1.0	1.0	1.2	1.8	1.3	0.8	1.0	1.3	
E	R	5.8	6.3	5.4	3.6	4.4	4.5	3.0	3.2	2.3	5.8	63.3
	I	3.6	2.6	1.4	1.0	1.0	2.6	1.7	1.2	1.4	2.2	
F	R	4.4	3.9	3.0	3.4	5.0	5.8	7.0	4.6	4.6	2.8	67.1
	I	3.2	2.6	1.8	1.2	2.0	3.2	2.5	1.7	2.2	2.2	
G	R	3.2	4.4	6.3	5.0	3.9	3.0	2.3	3.2	2.0	4.1	46.5
	I	1.5	1.2	0.8	0.4	0.6	1.3	1.4	0.5	0.5	0.9	
H	R	1.5	2.0	2.6	3.4	2.5	1.7	1.8	3.9	2.2	1.5	28.6
	I	1.0	0.5	0.4	0.2	0.9	0.8	0.5	0.3	0.4	0.5	
J	R	5.8	5.0	3.9	3.0	3.9	4.1	3.6	2.8	2.0	5.8	61.4
	I	3.4	2.8	2.0	1.3	1.7	3.0	1.8	1.4	1.5	2.6	
K	R	2.8	3.4	3.4	2.5	2.5	2.2	1.4	1.7	1.0	4.4	35.9
	I	1.7	1.4	1.0	0.6	0.9	1.4	0.8	0.6	0.8	1.4	

Results of the drawdown calculations are shown in Table 5.

The method assumes that total drawdown at a point is the sum of the drawdowns caused by a series of pumping bores. The localized drawdown at each pumping bore has been ignored and the error introduced by this in calculating changes in aquifer storage will be small. At any bore pumping under the conditions outlined above, drawdowns in the bore will be much greater than the contoured amounts shown on Figure 9. The increase in drawdown in a pumping bore has been estimated at approximately 65 feet (20 m) assuming a partial penetration of the aquifer of 50 per cent. This figure is only presented as an indication of magnitude and should not be taken as a prediction.

RESULTS

Figure 9 shows contours of the calculated drawdowns, together with a cross section, to illustrate the change in the water table. The change in the saturated volume of aquifer is calculated to be 216,000 acre feet (226×10^6 m^3) of aquifer.

The computations have assumed that there is no vertical recharge, and also that the northern and eastern boundaries are impermeable.

Some vertical recharge probably occurs but it is impossible to make any estimate. The proposed boundaries are not impermeable and there will be some flow through them in to the Southern Sector. To compensate for this, image wells were only provided for the northern Vince Fault boundary and not for the eastern Banksia Gully Fault boundary.

The analysis is crude but should nevertheless provide a usable picture of the effects of concentrated pumping over a long period. Wells of smaller capacity with slightly closer spacing would produce similar results.

If pumping is kept to the suggested area, saline contamination should not become a problem. Full-length perforation observation wells should be maintained to check on possible movement of the saline water from the south and west if heavy pumping is started.

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