Geohydrology of Catchment

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by

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WATER RESEARCH COMMISSION

EFFECTS OF URBANIZATION ON CATCHMENT WATER BALANCE

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ABSTRACT

The proportion of rainwater infiltrating the ground is generally of the order of 90%, so groundwater plays a significant role in a catchment water balance. The bulk of the rainwater penetrating the ground is held in the upper layers to subsequently evaporate by transpiration. The balance seeps out of the catchment through aquifers and fissures. It is very difficult to estimate both evapotranspiration and groundwater release rates, and this report describes a number of attempts to assess the groundwater flow and retention. Whatever number of tests are conducted they only give a sample of results and budget limits must be set in view of the costly nature of geohydrological tests.

The base rock is granite up to 30m deep, overlain by decomposed granite, and granitic soils. Dykes and fissures cross the catchments.

In situ investigations were conducted in an attempt to establish groundwater flows and volumes for two research catchments.

6 boreholes were drilled on the Sunninghill catchment and 11 boreholes on the Waterval catchment (all percussion drilled).

Boreholes were drilled to observe subsurface geological profiles, to monitor groundwater levels and to conduct various test in. Water levels in boreholes were observed to vary over time but some of the reasons for these fluctuations are unclear. For instance levels rose to above ground level in some holes to the west of Sunninghill after drilling and this is probably due to artesian conditions prevailing. Water levels also rose temporarily after rainstorms, and this could be due to surface water leaking past the collar of the borehole.

Pump tests were conducted to establish aquifer permeabilities and storativities. Transmissivites ranged from 2 to 40 x 10^{-5} m^2/s and storativity 1-5 x 10^{-3}.

Environmental isotope analysis indicated water age of up to 50 years.

Tracer tests were conducted to estimate groundwater velocities with limited success.

Seismic transverses on Waterval identified the transition from surface soils to weathered granite, and anomalies like dykes.

It appears that most subsurface water leaving Sunninghill flows to the west and surfaces at the dyke, to be measured in the weir constructed in a stormwater channel.

Flow from Waterval appears along deep decomposed dykes. Estimates of total aquifer outflow vary from 56 000 to 280 000 Mm^3/an (omitting an erroneous tracer result).

No trend or drop in groundwater levels was noticed over the four years of observation. A daily fluctuation of 30mm was detected.

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1 INTRODUCTION

In this report an attempt is made to establish an understanding of the groundwater regime in the Waterval and Sunninghill catchment areas.

The groundwater component plays an important role in a water balance system and in spite of, or perhaps because of, its elusive character deserves considerable attention. By intensive study of the subsurface conditions a substantial degree of uncertainty concerning the groundwater runoff can be removed. In this way the groundwater component becomes an active element of the water balance, instead of an accumulation of quantities which have not been accounted for in some other way.

Unfortunately, the collection of data related to subsurface conditions is often difficult, cumbersome, and expensive, particularly so when the underground is inhomogeneous or irregular. The aquifers in the present study consist of fractured rock covered by decomposed rock and soil and intersected by dykes. Even if unlimited financial resources were available it is not realistically possible to map all the water bearing layers and all the minor flow obstructing features. Therefore, a certain degree of generalisation is required, which will negatively influence the accuracy of the groundwater flow evaluation.

For the acquisition of subsurface data use was made of the expertise from multiple disciplines. The composing materials and their distribution are of crucial importance to the description of the aquifers, which makes input from earth sciences experts essential. Regional geological information provides the general context, while site-related features are obtained by geophysical surveys and borehole logs. The presence and movement of groundwater is studied by means of a variety of tests involving contributions from geohydrologists and physicists.

2 GEOLOGICAL RÉSUMÉ

The Sunninghill and Waterval catchments are situated just north of Johannesburg and form part of the independent suburb of Sandton (see figure 2.1). The base rock in this area consist primarely of ancient granite dating back some 3200 million years. This base rock is geologically known as the Johannesburg-Pretoria granite dome, which occupies an ovoid area of approximately 700 square kilometres between those two towns.

Igneous intrusions or dykes are prolifically developed in all the areas underlain by granite rock. The dykes vary considerably in orientation, age, and in chemical and mineralogical composition. However, all dykes have a high concentration of dark minerals containing magnesium and iron. Therefore, these dykes are classified as ferromagnesian or mafic. Some dykes appear on the surface, but the majority is decomposed and does not outcrop, their presence being noted mainly by changes in colour of the soil.

A comprehensive geological description of the Johannesburg-Pretoria dome is given by Anhaeusser (1973) and is included in this report as Appendix A. Figure 2.2 is derived from this paper and shows that a major diabase/dolorite dyke is present at the western part of the Sunninghill catchment, while a Pilanesberg dyke (consisting of diabase and porphyritic quartz) passes through the north eastern tip of the Waterval catchment.

A site-specific investigation was conducted by Barker and Associates in June 1986 (see Appendix B). Their report contains information on the geomorphology of both catchments.

Based on a literature survey and experience they expect the presence of shear zones and an abundance of unsheared faults and joints. Weathering is likely to occur in these features. Friable, granular rock with a high quartz content can form an indication of a shear zone, fault or joint. During subsequent drilling of the boreholes it was observed that at least the first major water bearing layer of each hole contained considerable amounts of coarse quartz crystals.

A distinction is made between old and young dykes. Particularly the youger dykes apparently tend to break down rapidly under humid conditions. Erosion of these dykes leads to deposition of clayey soils in the lower lying areas.

The thickness of the soil cover in the two catchments varies from 1 metre at the top of the hill to unknown depth at the bottom. The convex sections halfway down the hill are covered by sandy hillwash, while the lower concave slopes are covered by a broad range of grain sizes (alluvial clayey sands up to fine gravel).

<u>Reference</u>

Anhaeusser, C.R. (1973), *The Geology and Geochemistry of the Archaean Granites and Gneisses of the Johannesburg-Pretoria Dome* in *Symposium on Granites, Gneisses and related Rocks*, Edited by L.A. Lister, Geological Soc. of South Africa, Special Publication No. 3, pp 361-385.

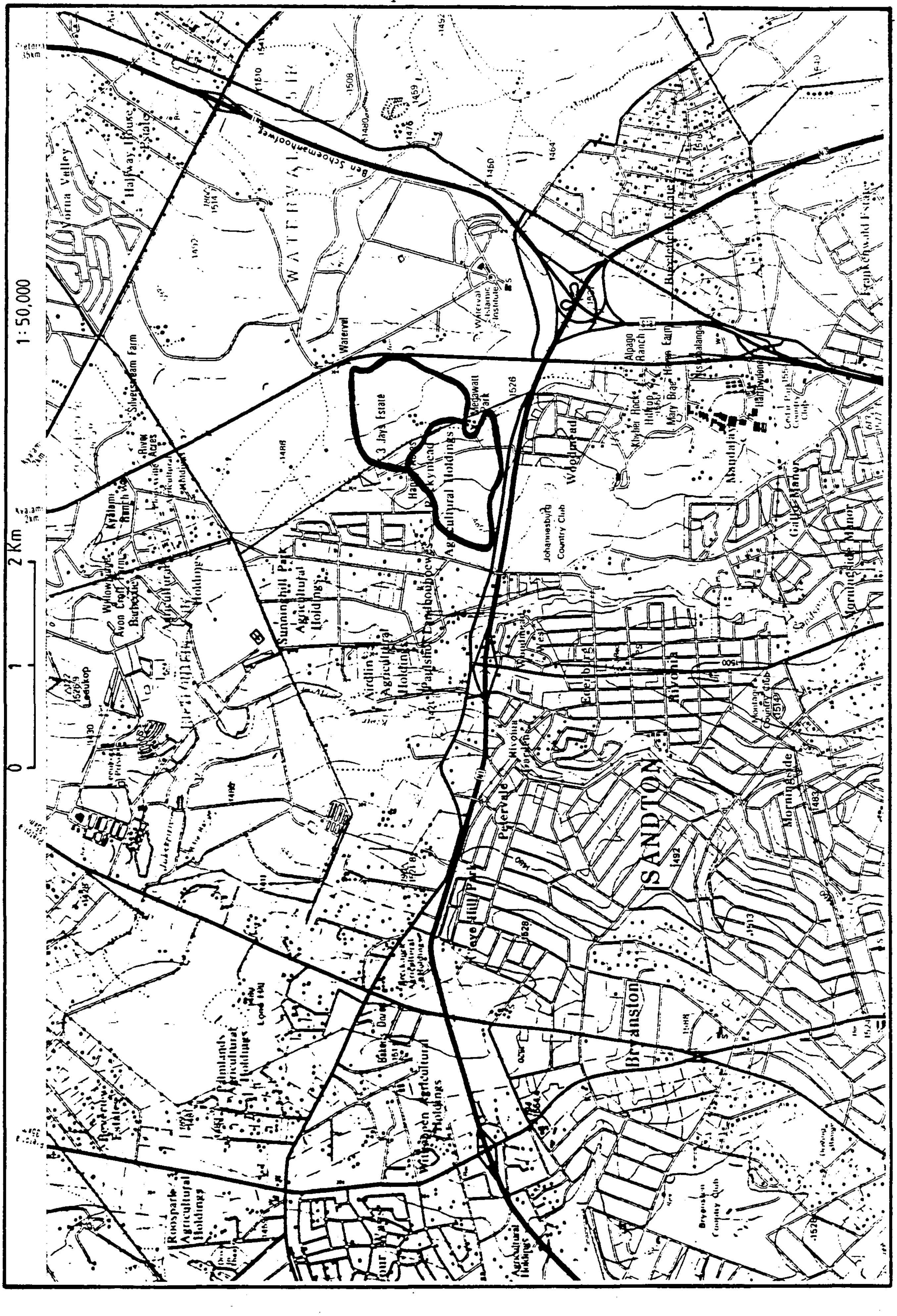


Fig.2.1 Locality map of catchments. Scale 1/50 000

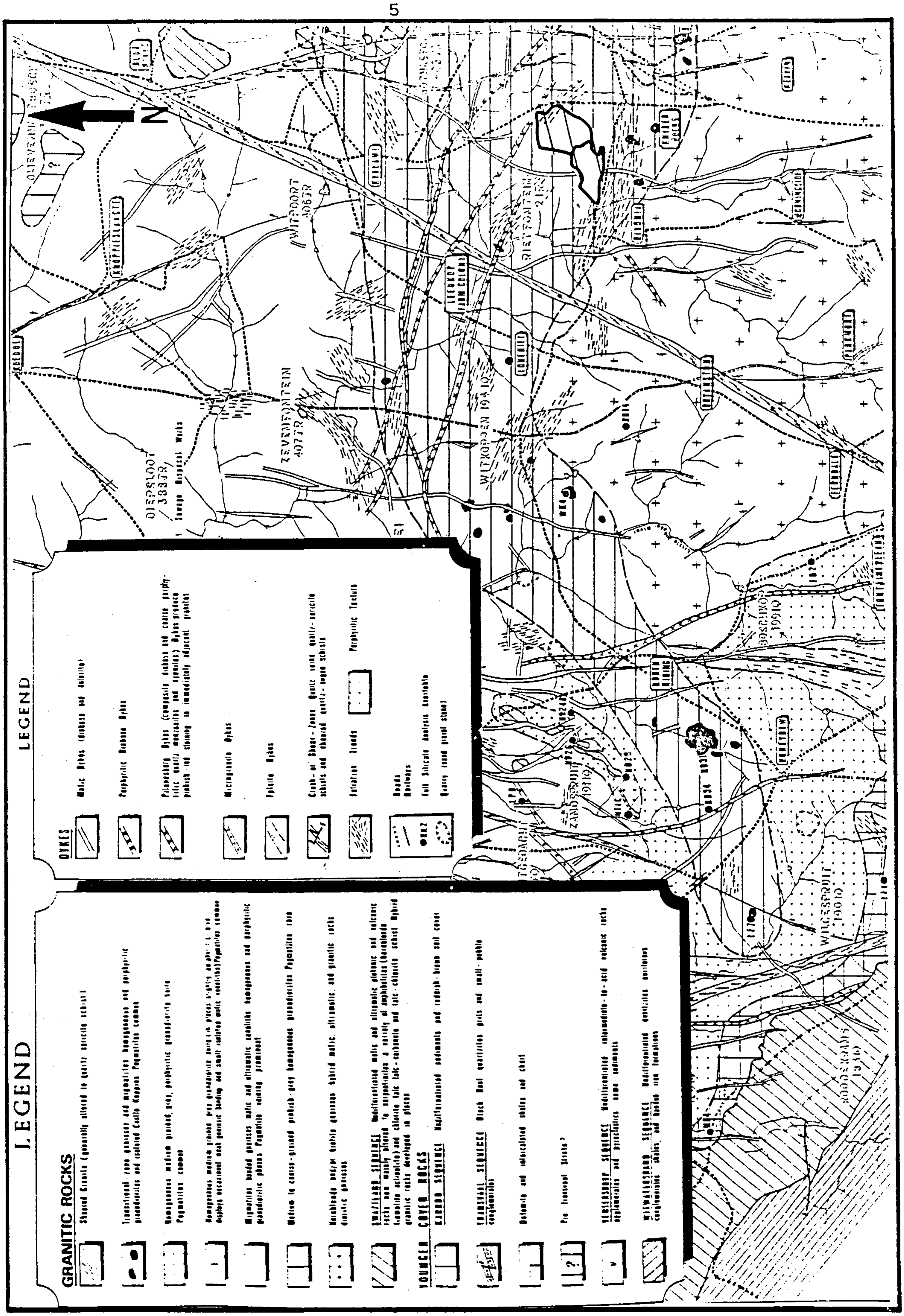


Figure 2.2 GEOLOGICAL REPRESENTATION OF THE JOHANNESBURG-PRETORIA GRANITE DOME

3 RÉSUMÉ OF GEOPHYSICAL STUDIES

3.1 Introduction

This chapter contains a brief overview of the results obtained from geophysical surveys and aerial photograph analyses. The review is based on a report by Mony and two reports by Antoine, listed in the Appendix.

The points of interest, which gave rise to the commissioning of the geophysical studies, were the depth to base rock, the depth to a possible phreatic water surface, and the possible presence of groundwater barriers and conduits.

It should be stressed that not only aerial photograph analysis, but also geophysical methods (such as magnetic, resistivity, and electro-magnetic surveys) are unable to detect water. What they can do is to reveal subsurface anomalies. The influence of these anomalies is, however, ambiguous. Geological dykes are a case in point. While they originally would have formed a groundwater barrier (hence the name), they may develop into outstanding aquifers within a favourable geological environment and after having been subjected to some degree of fracturing or folding. So, additional information is required to come to conclusive statements. One obvious case is the "strong linear" on the western boundary of the Sunninghill Park catchment. Groundwater is surfacing and forming a swamplike feature in the landscape. In this case it is not difficult to conclude that a dyke of low permeability is obstructing the groundwater flow and forcing it to the surface.

Individual geophysicists have strong, but widely diverse opinions about different survey techniques. Antoine (Appendix D) rejects the seismic refraction method as of little value for the purpose of a geohydrological assessment, and favours electro-magnetic surveys. However, in an editorial article in the Borehole Water Journal (April 1986) serious doubt is expressed on the effectiveness of electro-magnetic methods in groundwater exploration. They advocate the use of seismic refraction surveys. The time input requirements of the different techniques may possibly have influenced the opinions.

Having mentioned some limitations of geophysical surveys, here follows the findings of Mony and Antoine.

3.2 Sunninghill Park

Only Mony was requested to report on the Sunninghill catchment. Based on aerial photograph analysis and 1 magnetometer survey he detects a number of anomalies, one of which is mentioned above (see figure 3.1). He suggests that the groundwater leaves the catchment as artesian water across the barrier, but also along fractures trending in a NW/SE and NE/SW direction.

3.3 Waterval

In order to confirm observations from an aerial photograph study Mony conducted 5 magnetometer traverses and 2 resistivity tests (see figure 3.2). Each one of these appears to show a geological anomaly. On the basis of these anomalies Mony suggests to divide this highly complex area into three geohydrological units. The triangular, northernmost unit would have a fairly deep decomposition zone. The second unit is formed by a tapering segment following the slope in the middle of the catchment. It has a shallower decomposition zone. The third unit is enclosed on the NW by the boundary of the second unit and follows conveniently the southeastern surface boundary of the catchment. This last unit is considered to have a much more clay type residual soil cover. Mony seems to imply that irrespective of the nature of the anomaly (barrier or conduit), it can be used as a delineation in the groundwater regime.

Antoine conducted one seismic refraction traverse, five large scale and four small scale electro-magnetic surveys and 15 vertical electrical soundings in the Waterval catchment (see figure 3.3). The result of the seismic refraction test is presented in figure 3.4. The rising and falling line in this graph represents the transition from weathered granite to loose, uncompacted soils. For some reason Antoine has been unable to delineate the transition from fresh to weathered bedrock, which even in a granite environment normally doesn't pose a problem.

The results of the electro-magnetic survey and vertical soundings were submitted to an interpolating routine, which resulted in two contour maps (see figures 3.5 and 3.6). The contours represent the "overburden conductivity thickness product". This parameter is apparently a relative expression, depending on the setting of the survey equipment, as is shown by the values of the trough just above the centre in the drawings of figures 3.5 and 3.6, which differ by a factor of approximately 30. In order to correlate the "overburden conductivity thickness product" to the actual depth to bedrock, additional information is required e.g. one or more borehole logs. In Antoine's second report the trough is studied in more detail and a drawing with the actual depth to bedrock is provided (see figure 3.7). It is worthwhile to note that the fault indicated in this figure seems to correspond with line 8 of figure 3.2.

Caution should be exercised in the use of figure 3.7. Although there doubtlessly is a baserock trough in the vicinity of borehole BW5, the actual depth to baserock depends on the use of that borehole log. As will be described in chapter 4 these logs are to a certain extent subjective. Therefore, the actual trough as well as the other contour lines may be out by some factor. Similarly, the comments by Mony on the depth of weathering have to be treated cautiously. While seemingly provided by an expert in his field, they appear to be based on the borehole logs we provided him.

<u>References</u>

Parasnis, D.S. (1986), *Principles of Applied Geophysics*, Chapman & Hall, 4th ed.

Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A. (1988), *Applied Geophysics*, Cambridge Univ. Press, Cambridge.

Borehole Water Location: Adapt or Die, Borehole Water Journal, April 1986, pp.4-6.

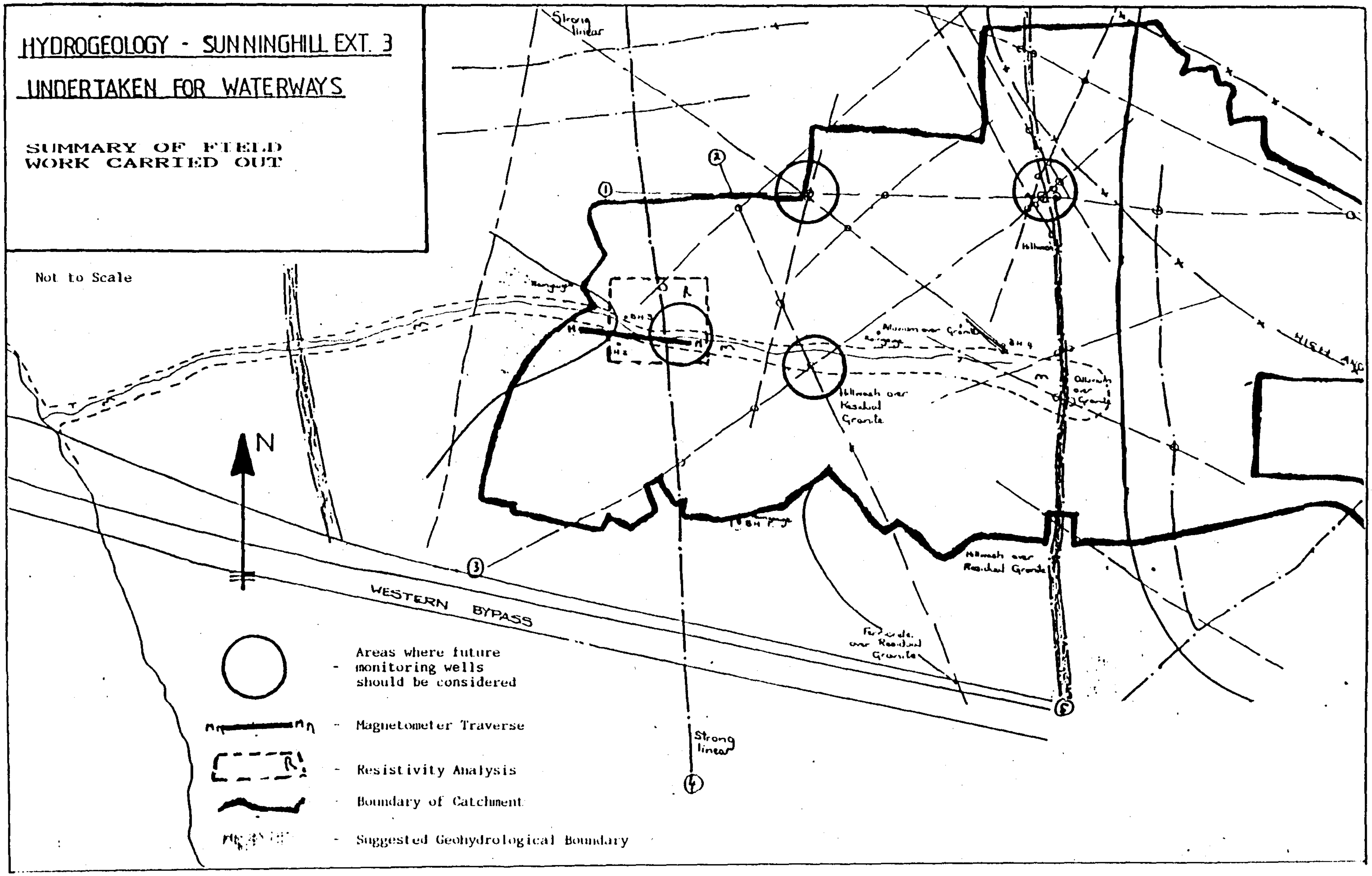


Figure 3.1 Geological features in the Sunninghill catchment (according to Mony)

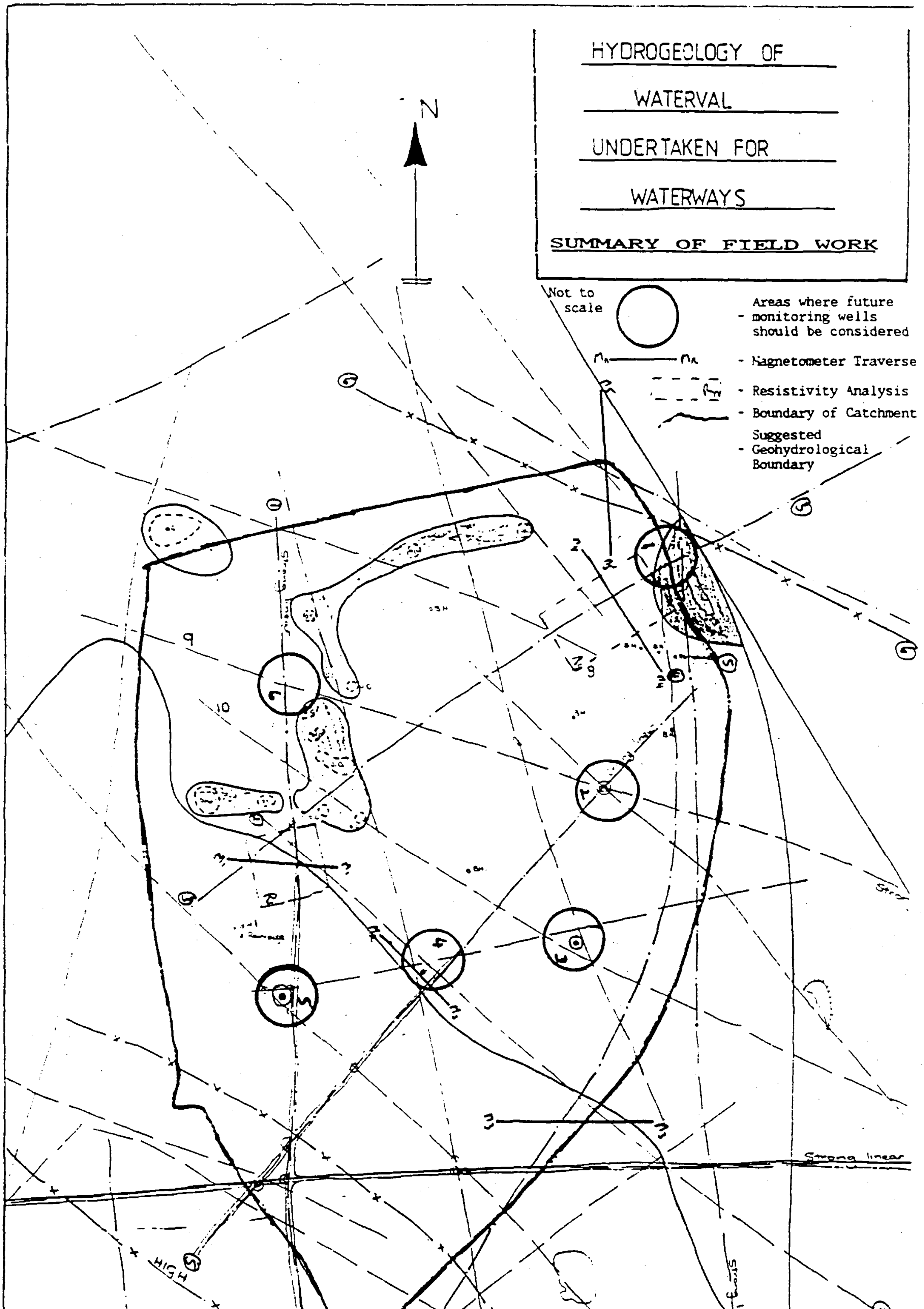


Figure 3.2 Geological features in the Waterval catchment (according to Mony)

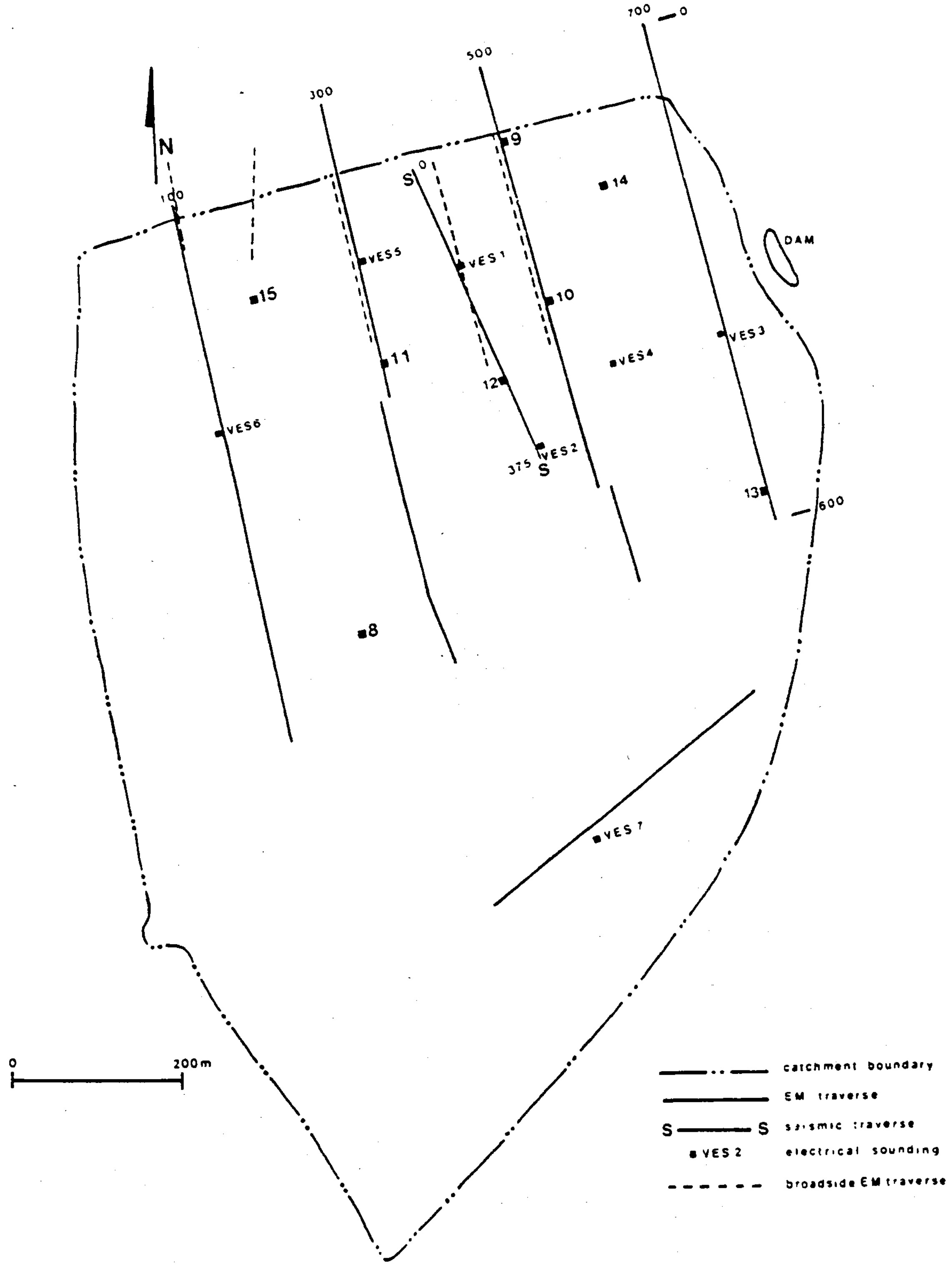


Figure 3.3 Location of geophysical surveys conducted by Antoine in the Waterval catchment

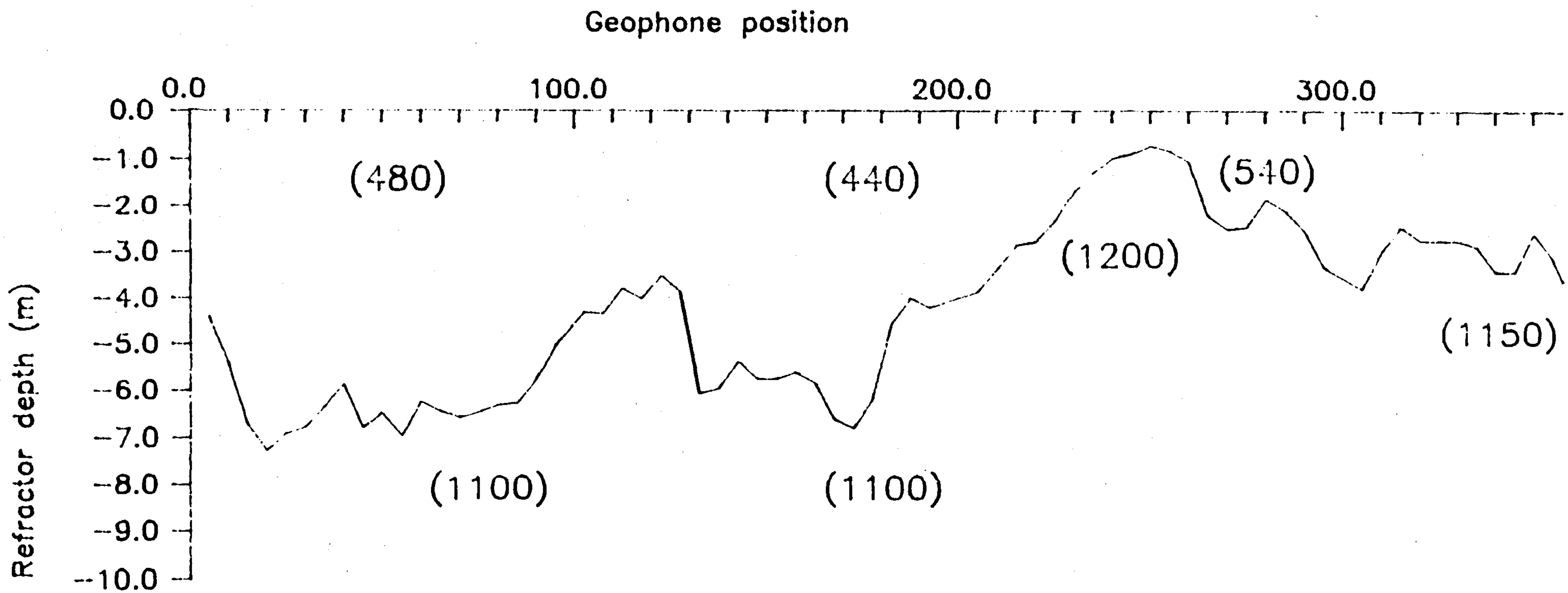


Figure 3.4 Interpreted seismic refraction section (according to Antoine)

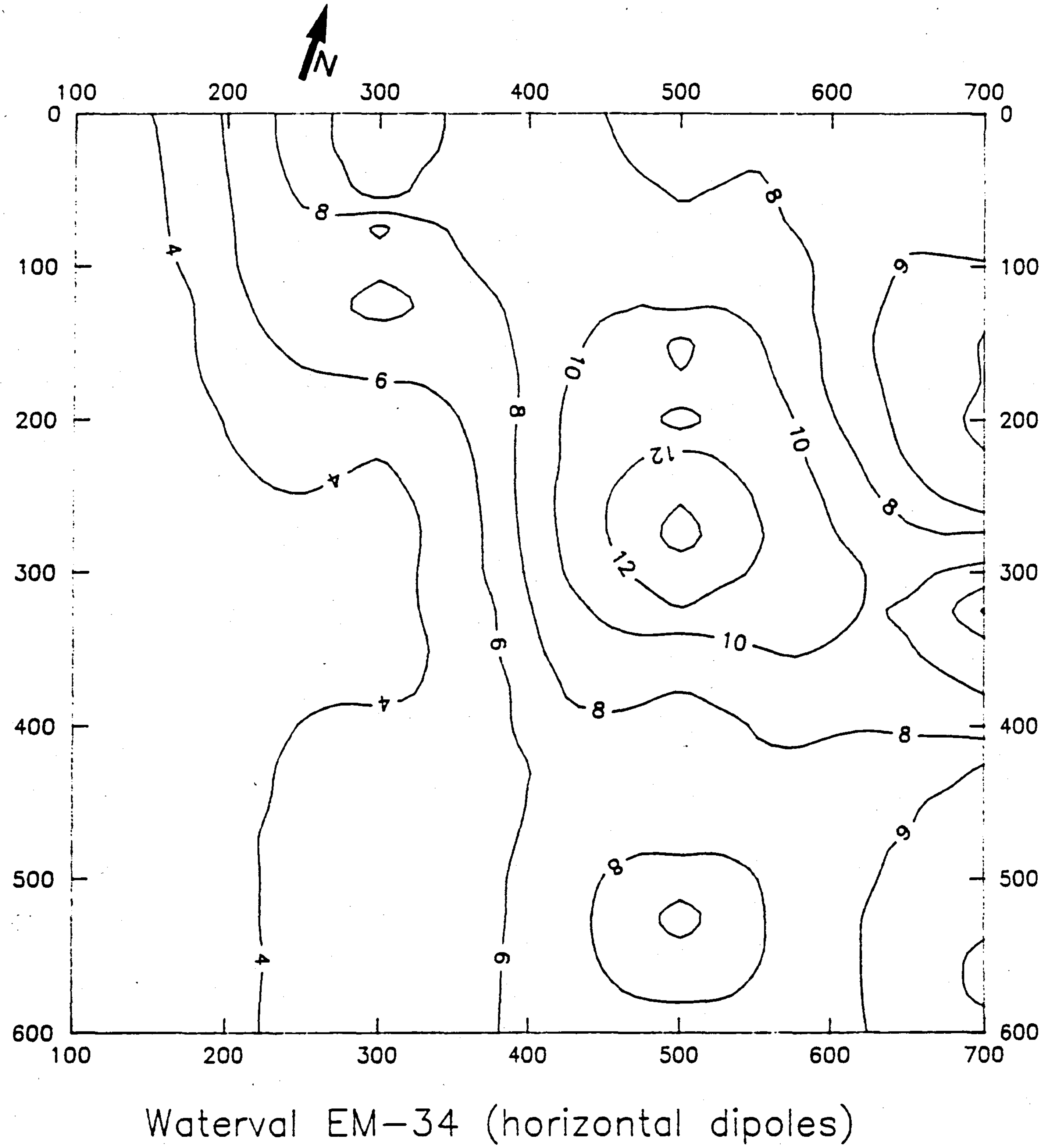


Figure 3.5 Contour map of "overburden conductivity thickness product" interpolated from the EM-34 traverses surveyed in the horizontal dipole mode with a contour interval of 2mS (according to Antoine)

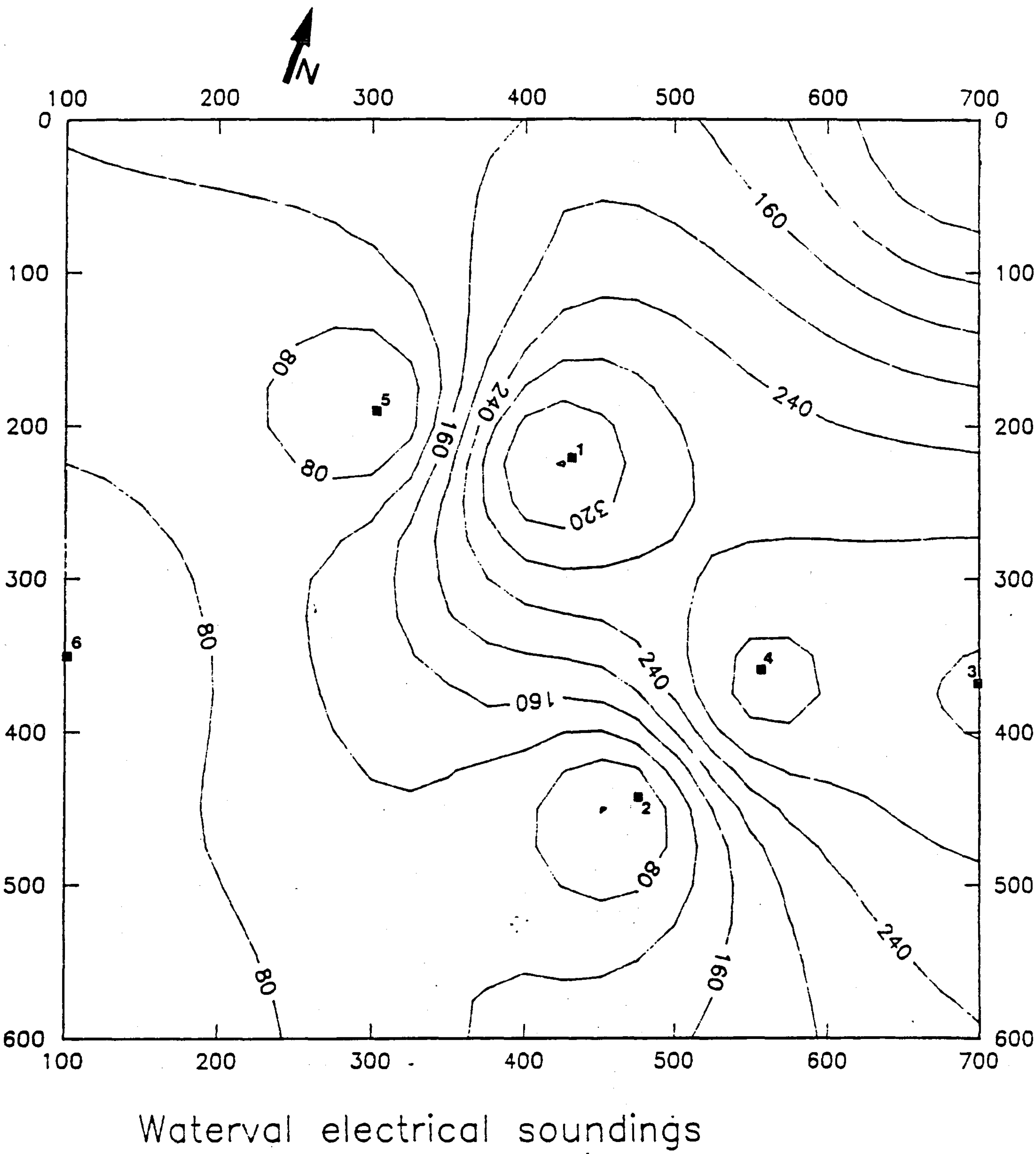


Figure 3.6 Contour map of "overburden conductivity thickness product" interpolated from the Schlumberger vertical soundings with a contour interval of 40 mS (according to Antoine)

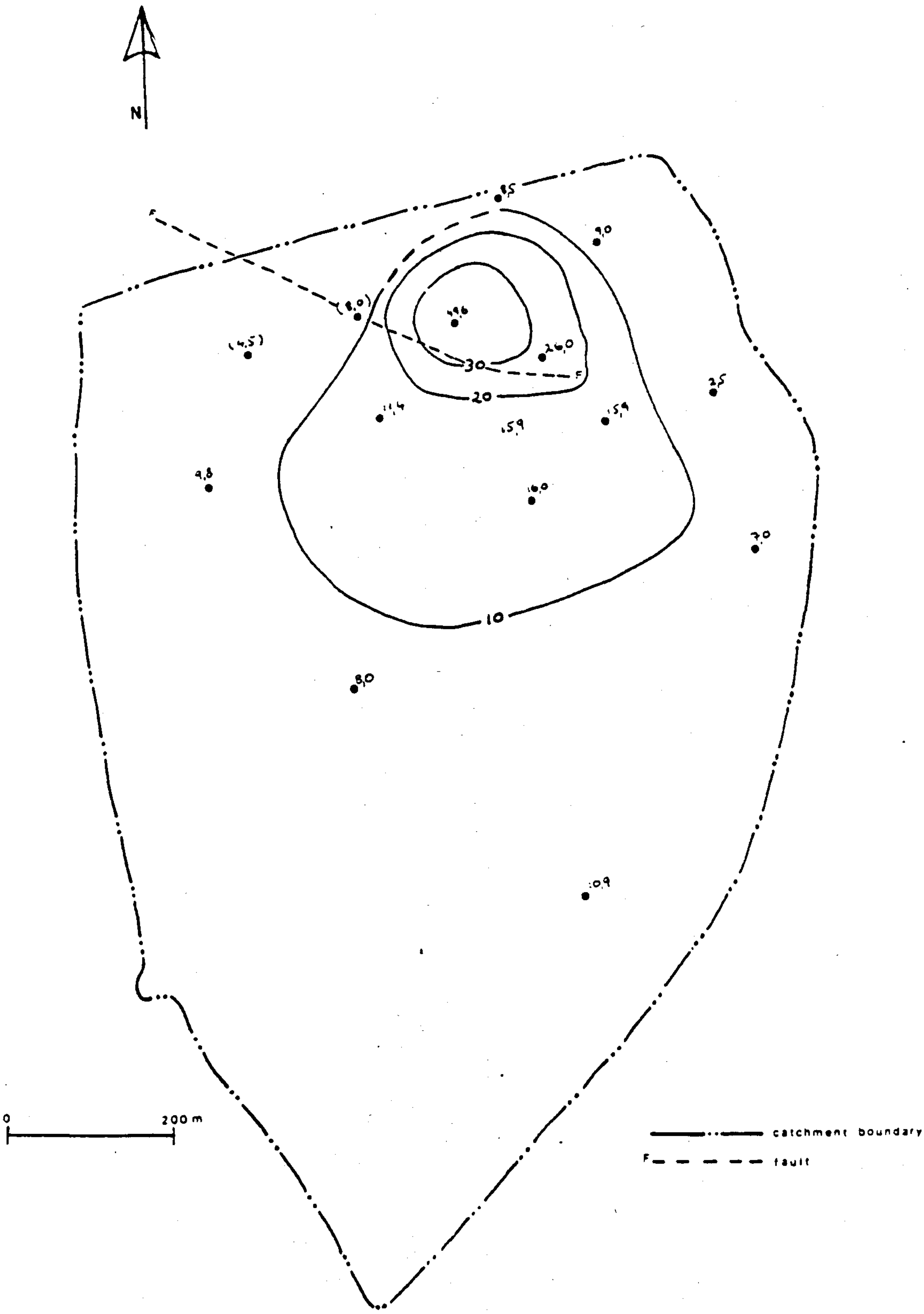


Figure 3.7 Contour plan of interpreted depth to basement (according to Antoine)

4 BOREHOLES: LOGS AND WATER LEVEL OBSERVATIONS

In total 17 boreholes have been drilled under the present contract, of which 6 in Sunninghill Park and 11 in the Waterval catchment. A comprehensive land survey to determine the exact location of the boreholes was only conducted in 1989. As a result some of the older drawings may indicate a slightly different position of the boreholes. Figures 4.1 and 4.2 provide the proper positions.

Borehole BW4 is in fact not more than a first attempt to a borehole, as the shaft of the drill got stuck in the mud, and further drilling was considered not feasible. At present the hole has essentially collapsed and the length is only some 4.5 metres below surface. For some unknown reason casing has been put in at a later date and waterlevel observations have been continued.

There has been an exchange of coding between boreholes BW6 and BW7 at some stage during the contract. The log of the present borehole BW7 is given as BW6, and BW6 as BW7. In all the other reporting the new coding has been followed. The casing of the present BW7 was vandalised in September 1987 and the borehole messed up. Water level observation in this hole have since then been discontinued.

All drilling was done with a percussion drilling rig by Mr J. Goodspeed. The borehole logs presented in this chapter are based on an interpretation of chips of rock blown out of the hole during drilling. Especially after encountering the first water bearing layer this interpretation can become difficult. The fact that the groundwater aspect of the contract was alternatingly supervised by three different people may further have contributed to some variation in the interpretations, particularly in respect of the degree of weathering.

Tables 4.1 and 4.2 provide a record of the observed water levels in the boreholes as measured from the top of the casings. Measurements were essentially taken on a weekly basis, with incidently some longer intervals.

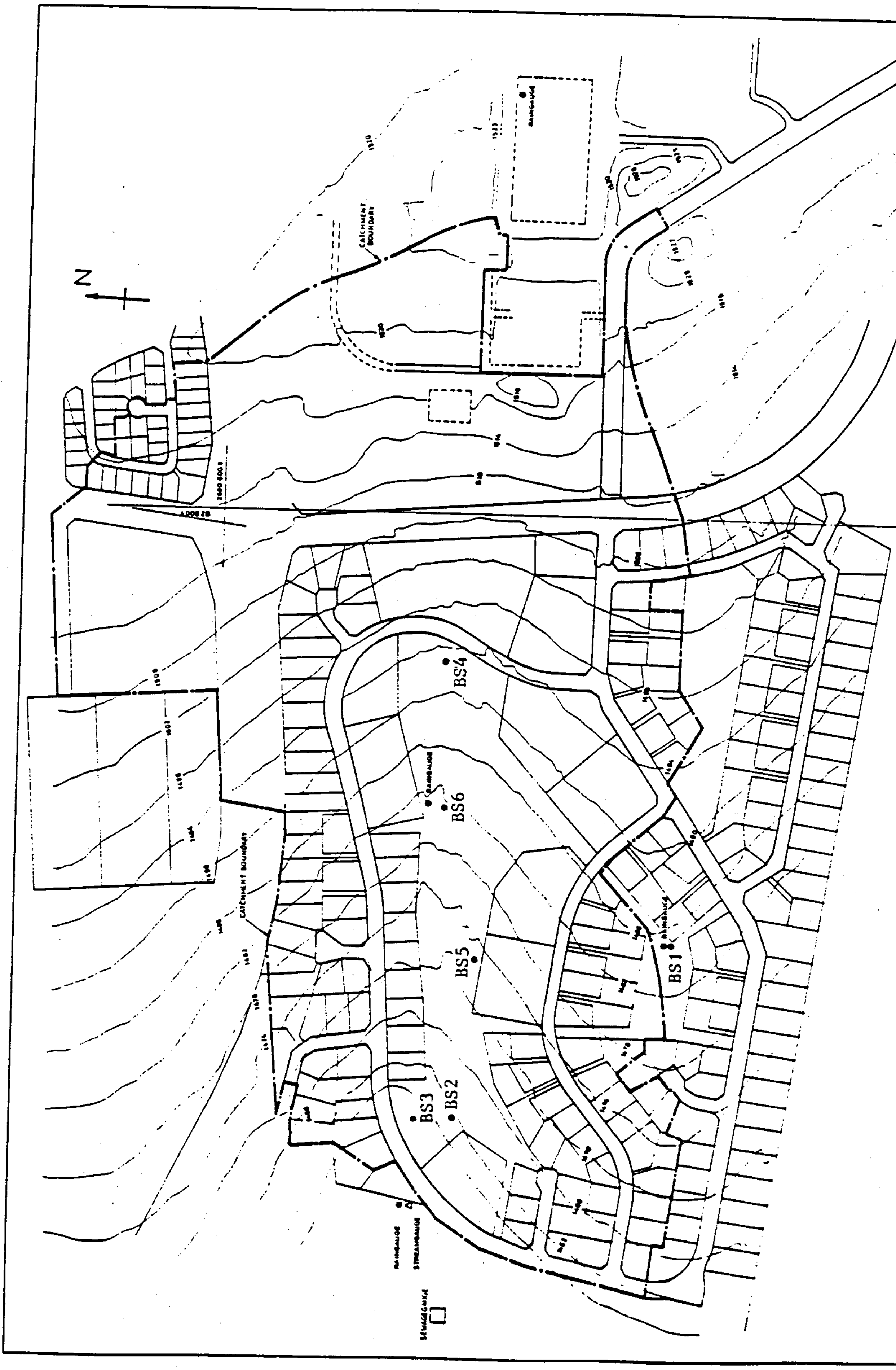


Figure 4.1 BOREHOLES AT SUNNINGHILL

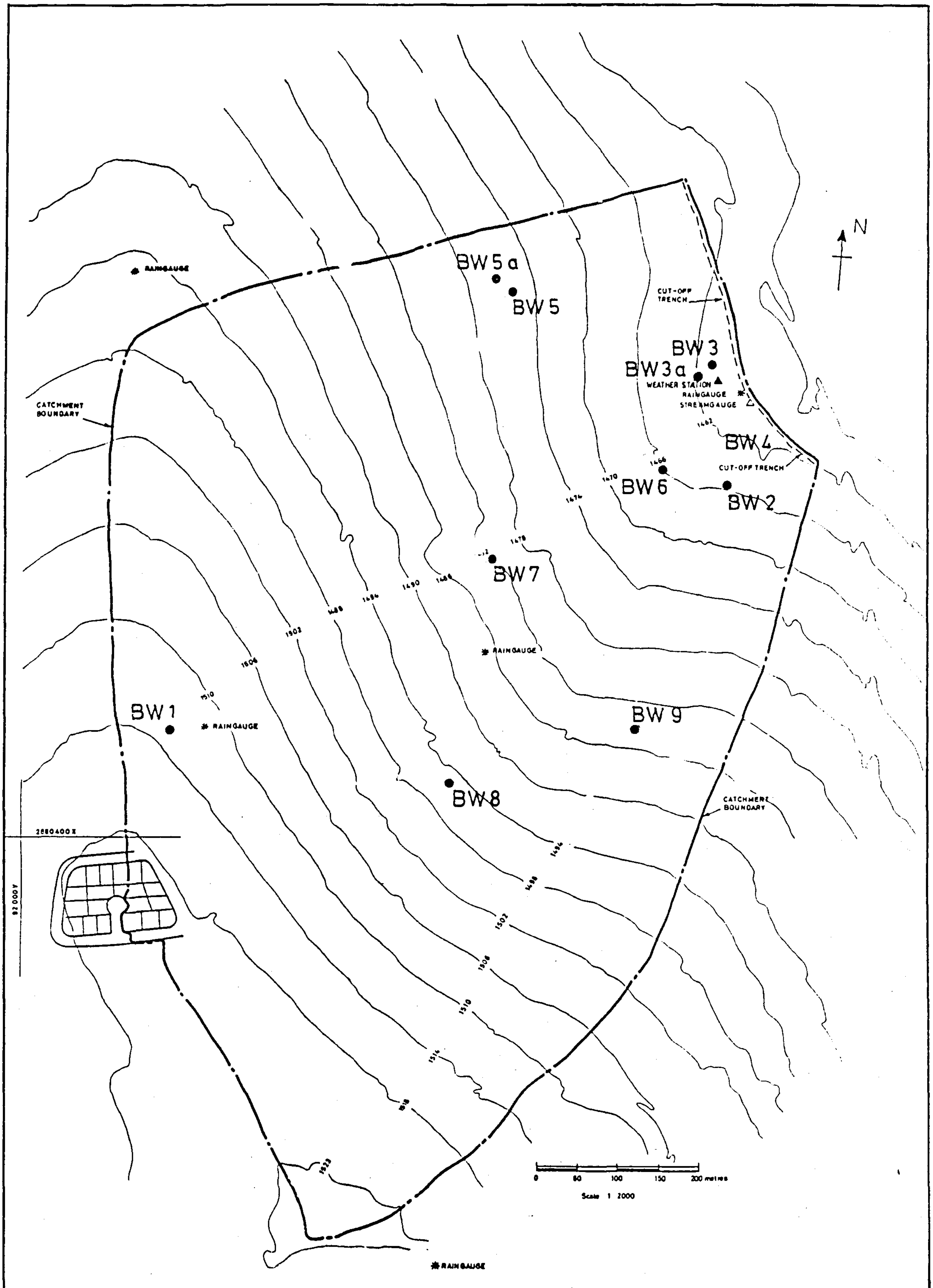


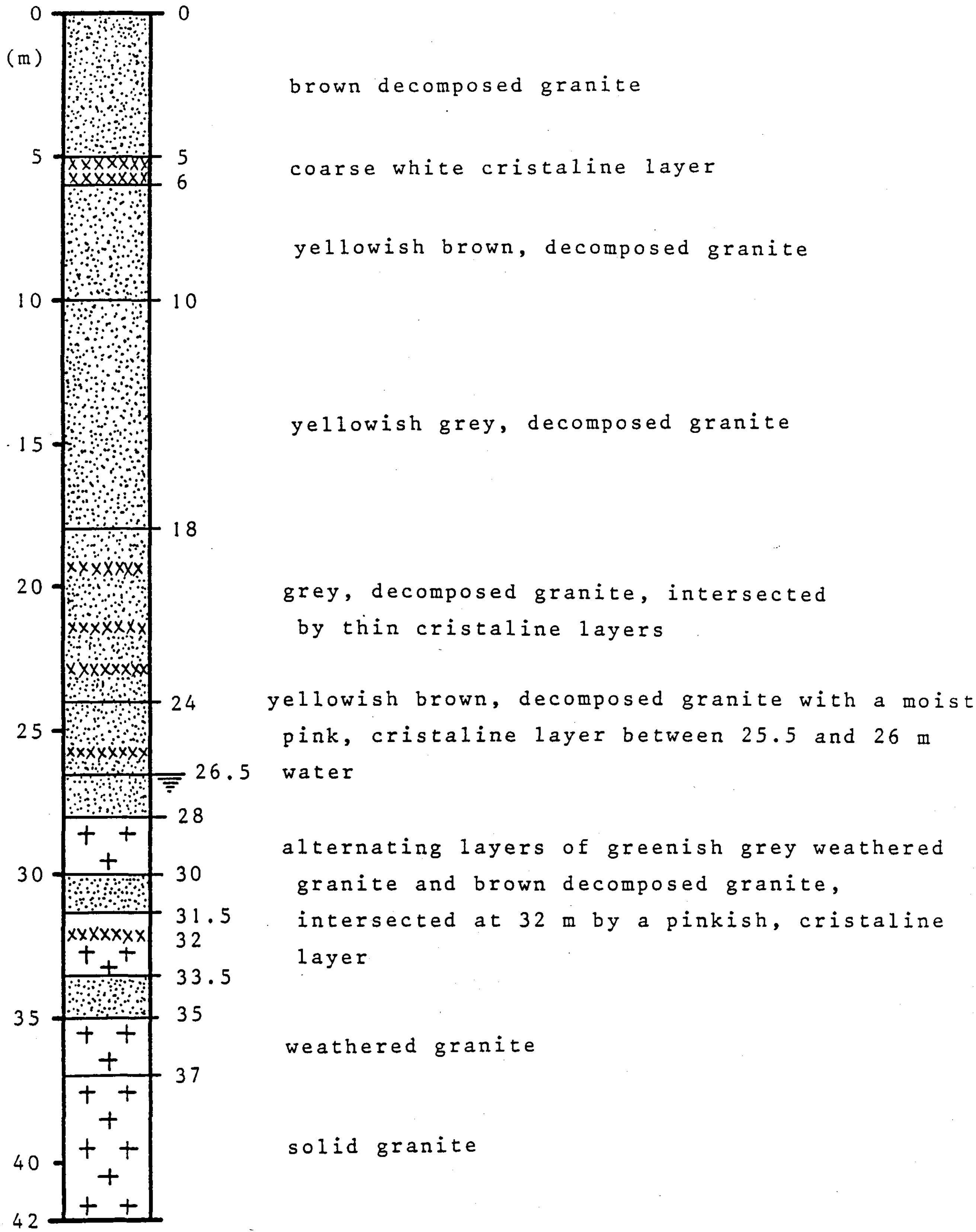
Fig. 4.2 BOREHOLES AT WATERVAL

In order to visualise the water level observations the measurements were plotted for each calender year. Due to the low levels in some of the holes in the Waterval catchment the vertical scale here had to be chosen fairly large. As a result the fluctuations in this catchment may at a first glance appear to be less significant. However, in both catchments a gradual rise of water levels is discernable during the wet summer season (October through March).

At the end of 1988 an experiment was started to determine the short term water level variations in the boreholes. To this end a pressure transducer was placed in BS1. Automatic recordings were made every half hour. By correlating the readings of the pressure transducer to the weekly manual readings it was determined that the position of the pressure transducer was 10.40 metre below the top of the casing and that each reading unit represented 0.00134 metre. The experiment ran for 4 separate periods, ranging from 5 to 17 days. During the last period the pressure transducer started to play up and the recordings became meaningless.

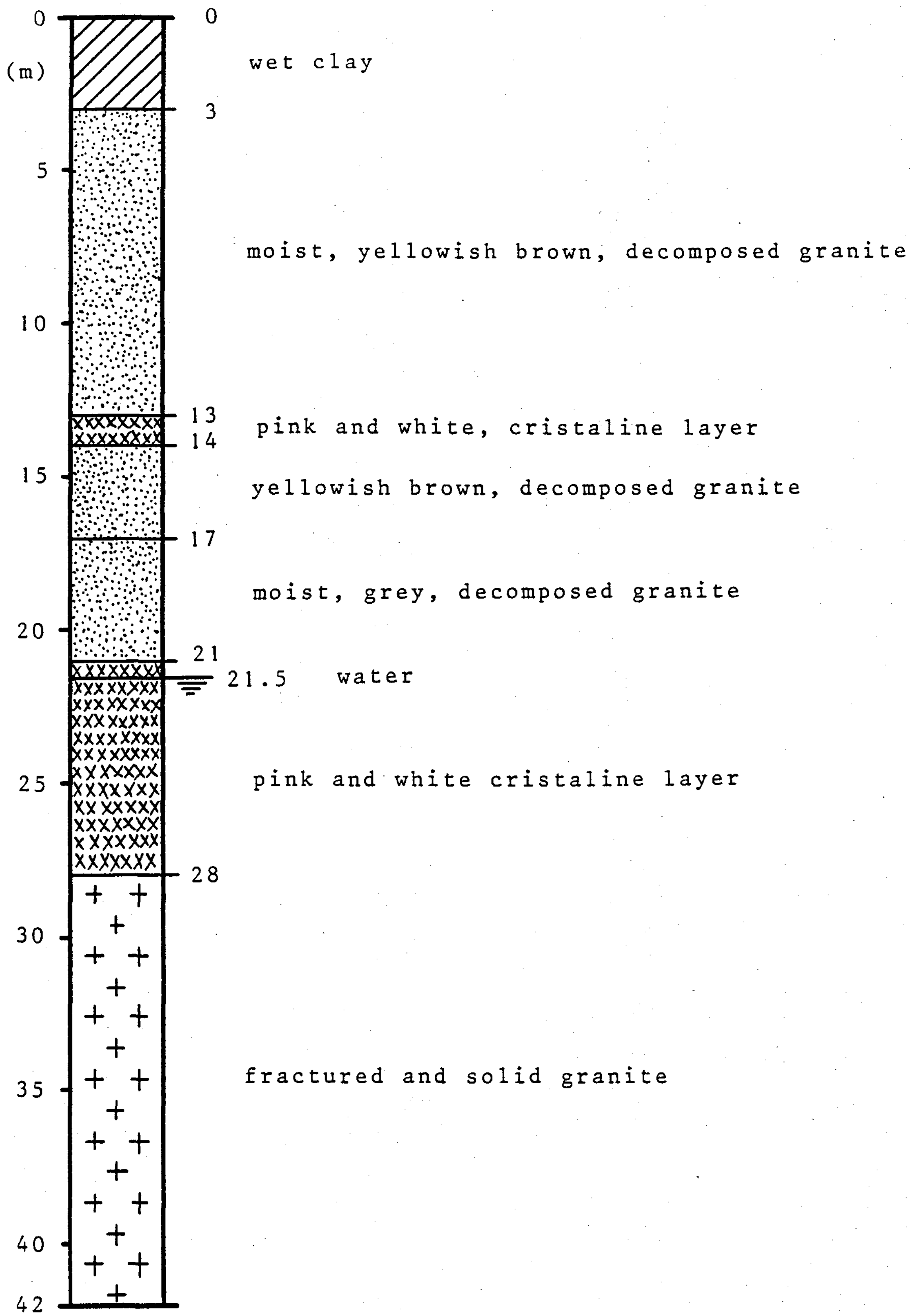
Apart from the long term rising water level an interesting daily fluctuation with an amplitude of ± 0.03 metre is detectable. During the night and morning the waterlevel drops, while it rises during the afternoon (see figures 4.30 to 4.33). This phenomenon has not yet been conclusively explained. On the one hand it is possible that it represents a technical flaw, such as the heating up of the recorder box. On the other hand it may indeed reflect the actual groundwater fluctuations, in which case a multitude of reasons could be suggested. To name a few: planetary interaction (but not the moon), varying pressure in leaking water supply pipes, etc.

Another 7 day test was initiated on 21th March 1990, recording the borehole water levels at BS1 at 5 minute intervals. Unfortunately, no manual recordings were available for this period and it turned out to be impossible to correlate the readings of the pressure transducer to the actual water levels.



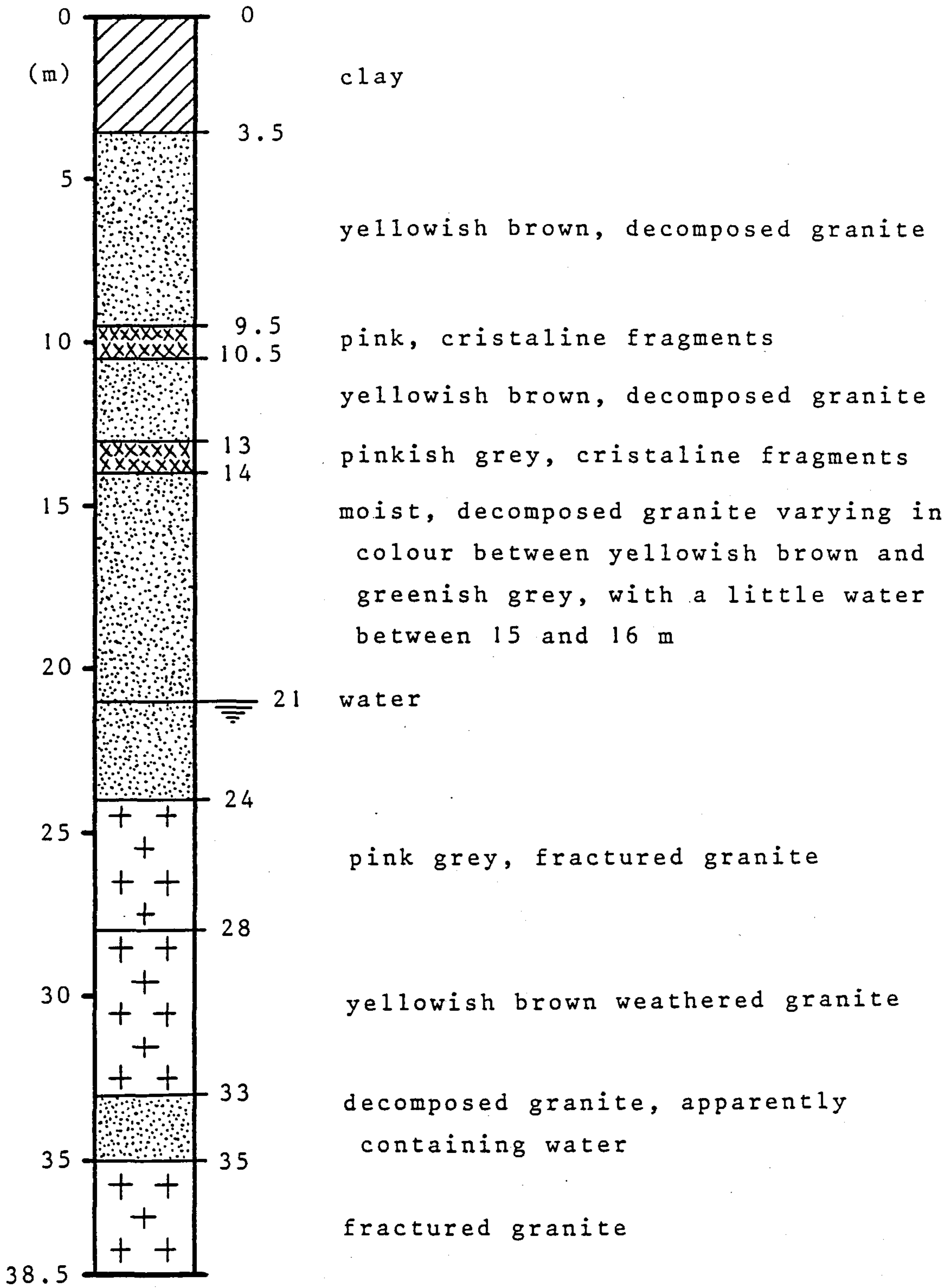
Borehole BS1 - Drilled 7-8-1986
- Top Sunninghill Catchment
- 6m Casing

Fig. 4.3 Borehole Log



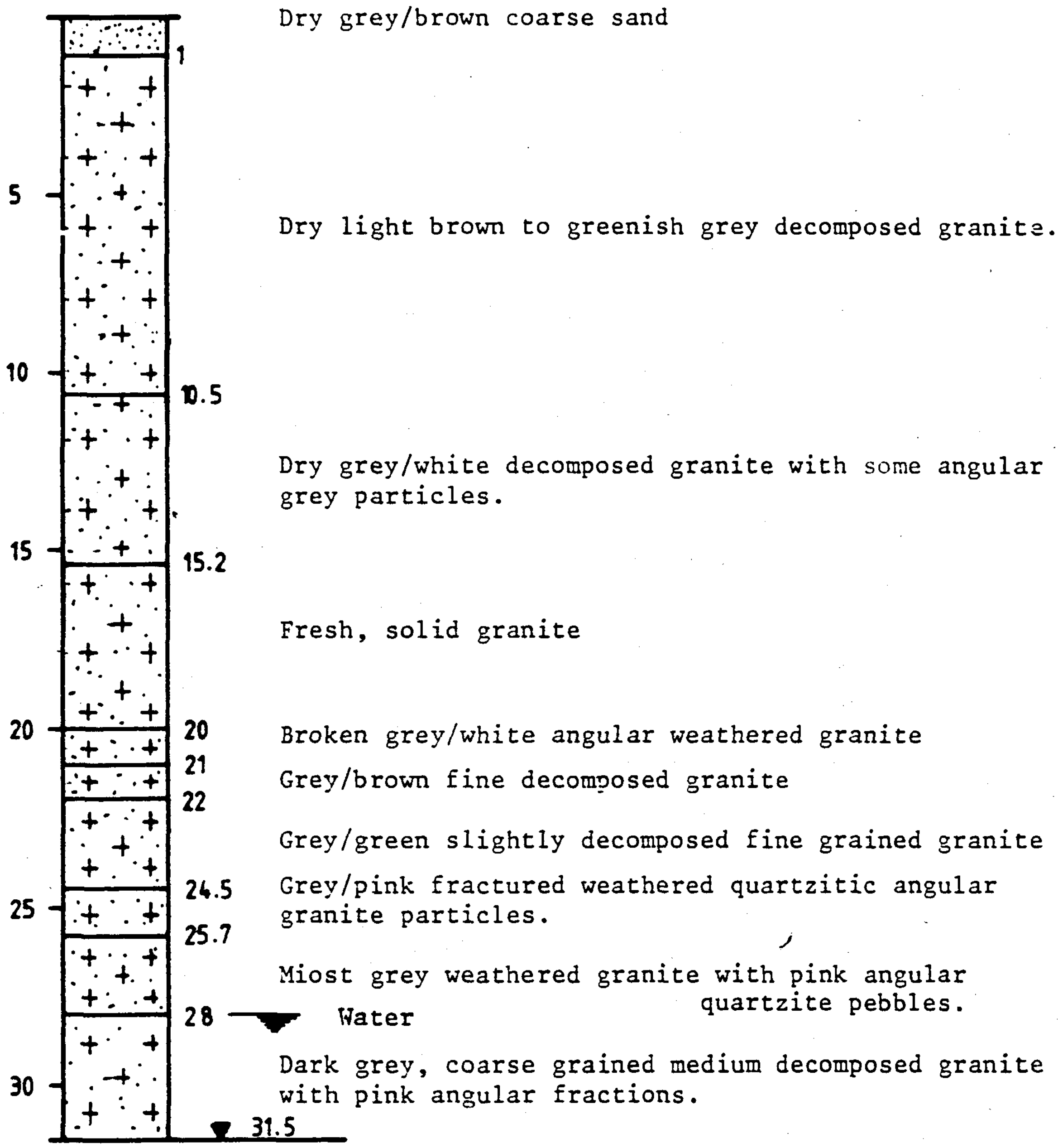
Borehole BS2 - Drilled 8-8-1986
- Bottom Sunninghill Catchment
- 6m Casing

Fig. 4.4 Borehole Log



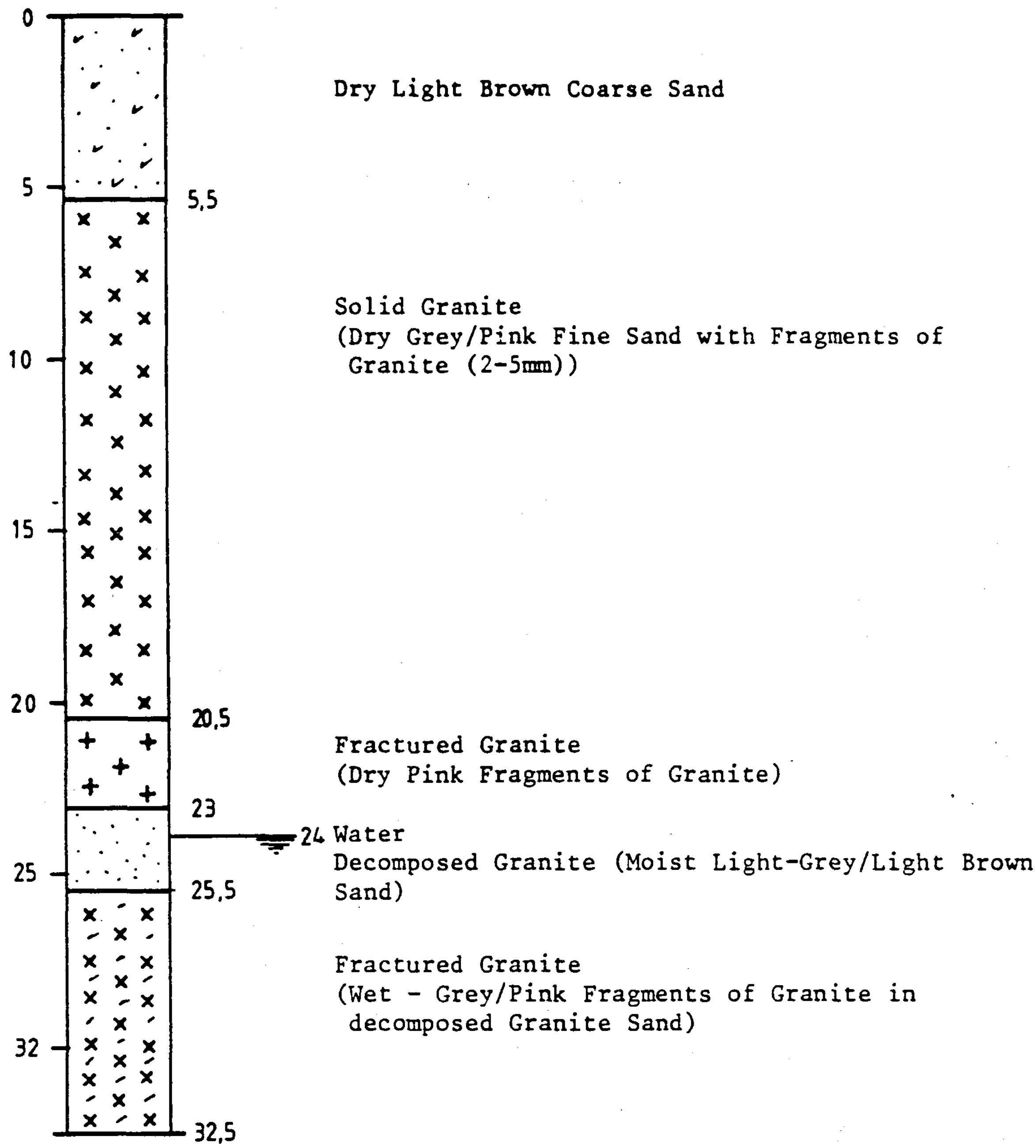
Borehole BS3 - Drilled 8-8-1986
- Bottom Sunninghill Catchment, near road
- 8m Casing

Fig. 4.5 Borehole Log



Borehole BS4 - Drilled 12 September, 1986.
- Seepage at 24.5
- Water at 28m
- 3m Casing

Fig. 4.6 Borehole Log



Borehole BS5 - Drilled 29-3-1989

- 4m Casing

Fig. 4.7 Borehole Log

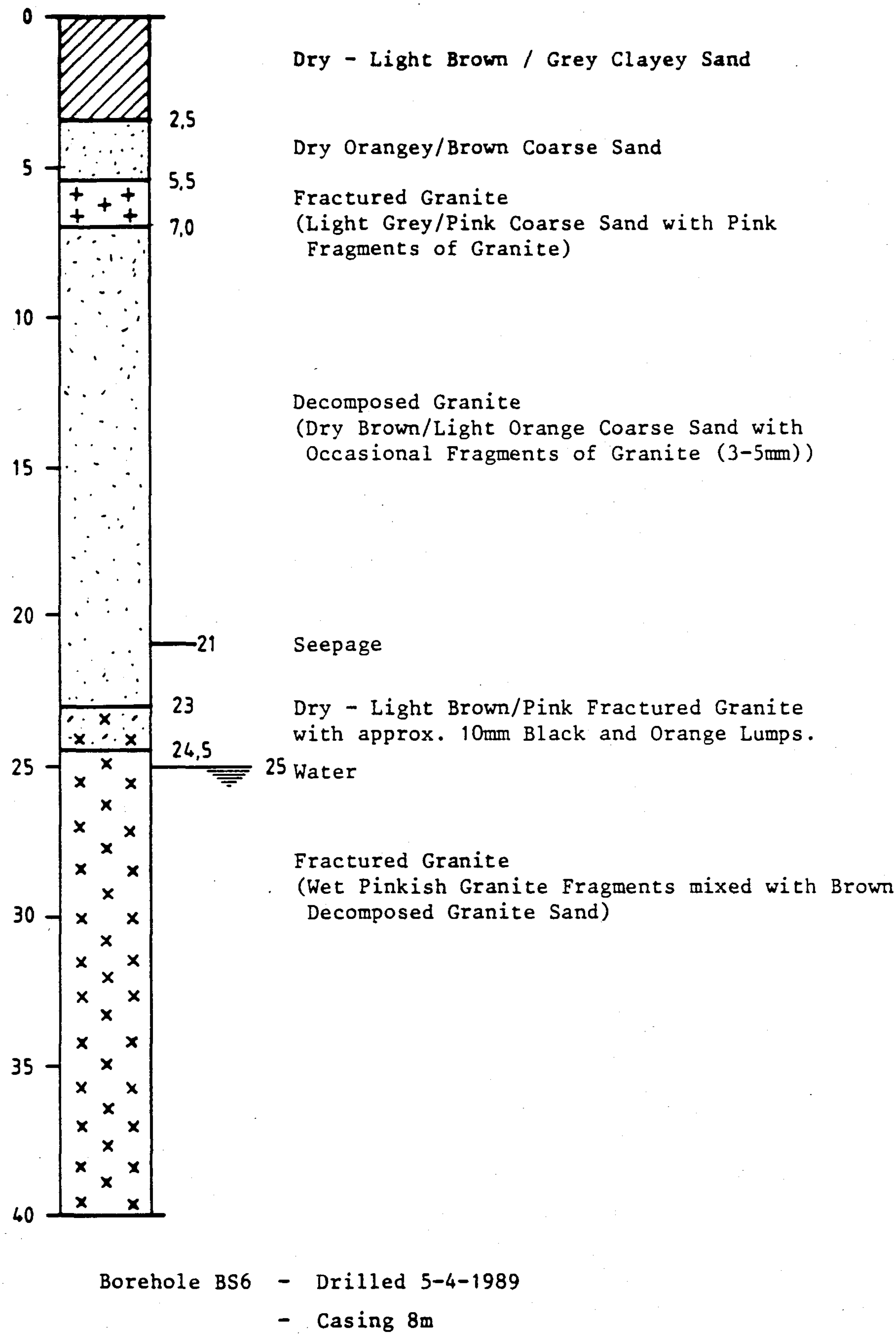
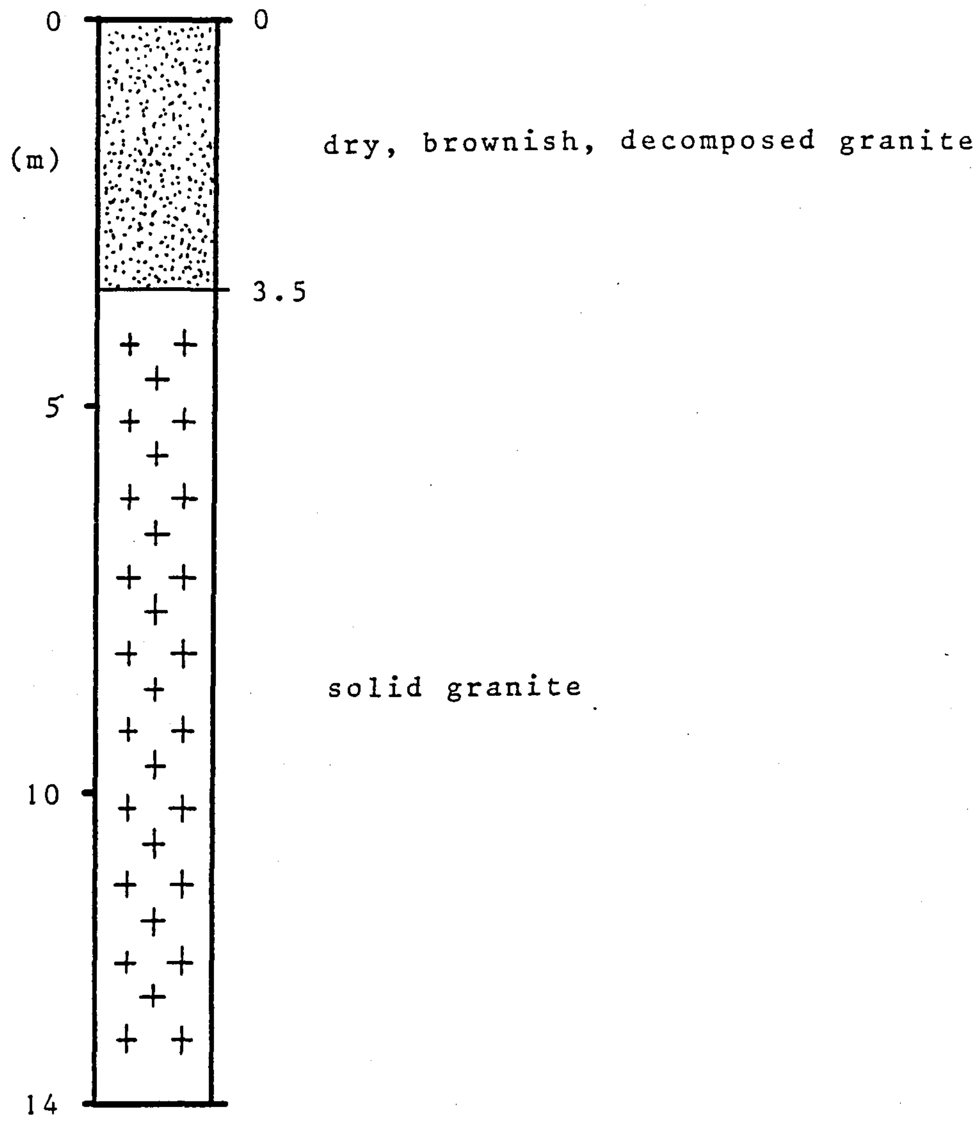
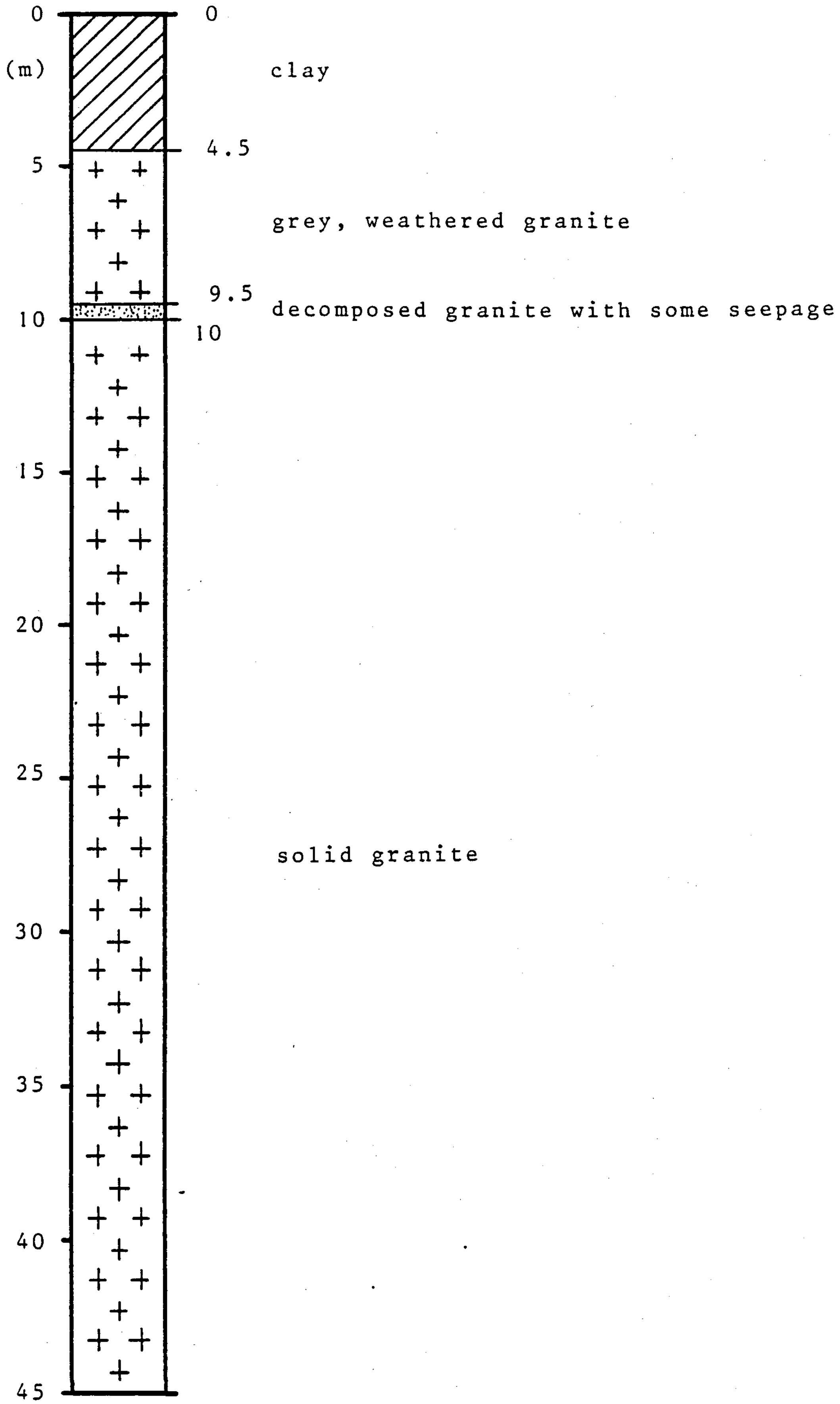


Fig. 4.8 Borehole Log



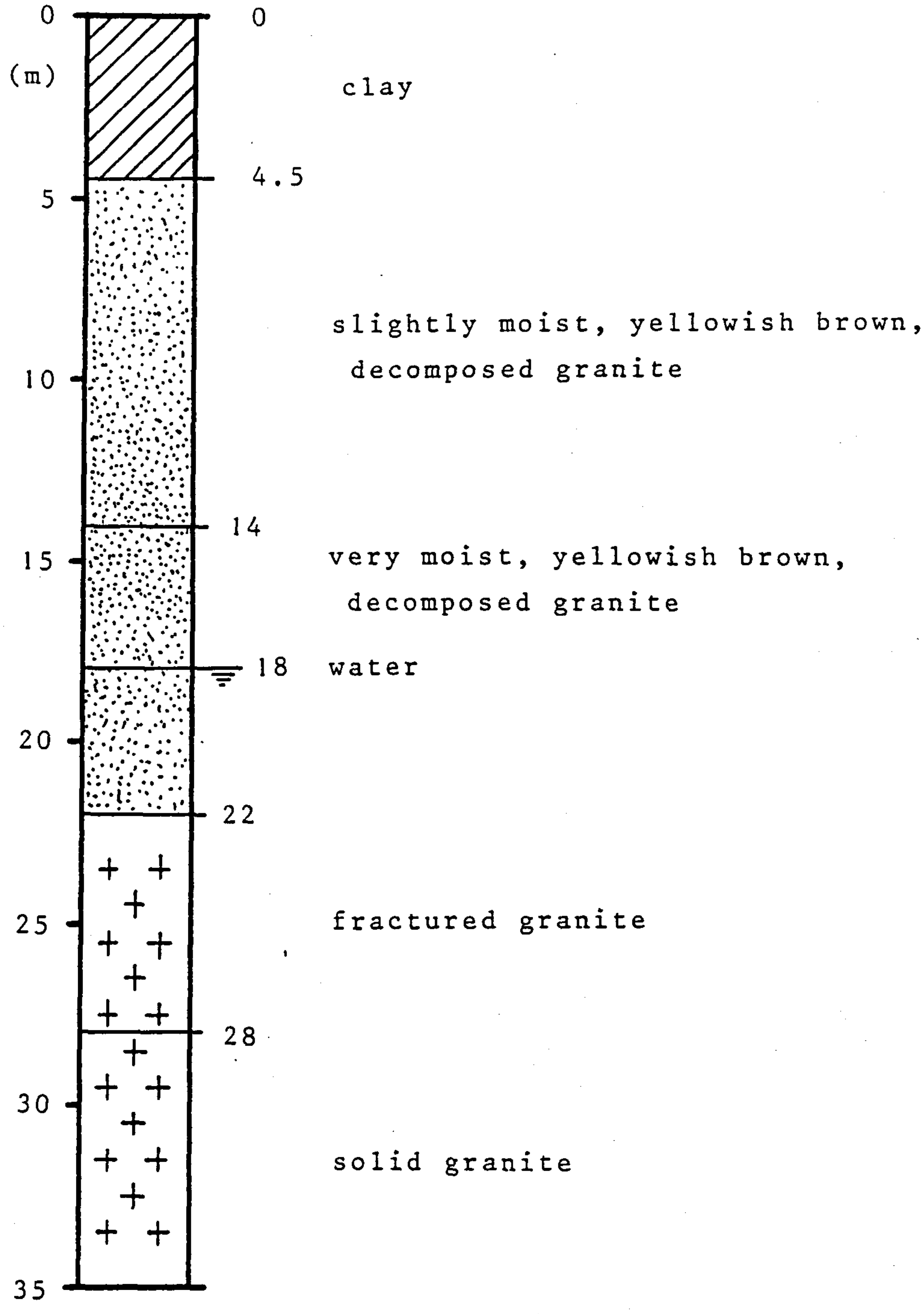
Borehole BW1 - Drilled 6-8-1986
- Top Waterval Catchment
- 2m Casing

Fig. 4.9 Borehole Log



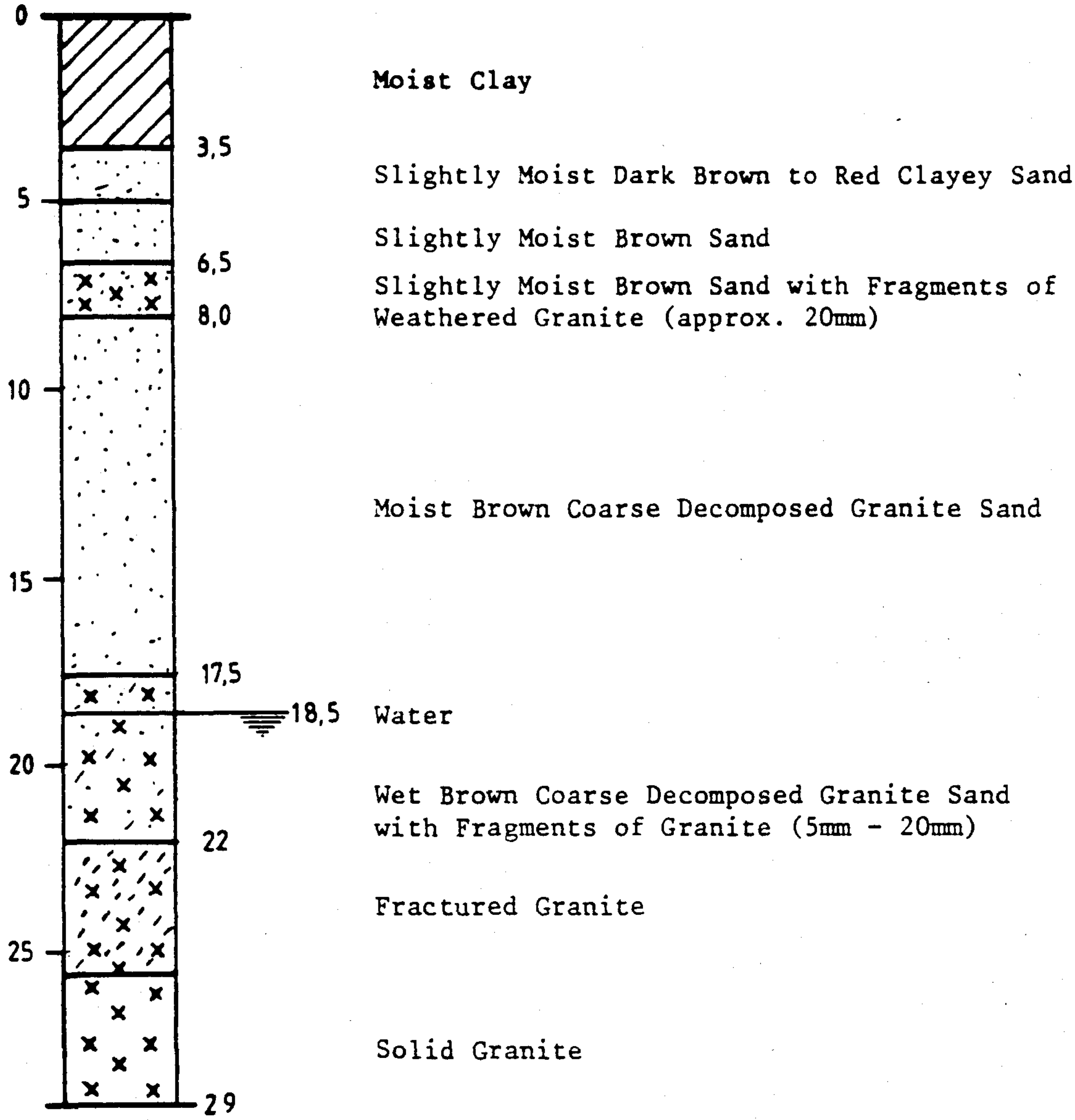
Borehole BW2 - Drilled 5-8-1986
- Bottom Waterval Catchment
- 1m Casing

Fig. 4.10 Borehole Log



Borehole BW3 - Drilled 12-8-1986
- Bottom Waterval Catchment, near windmill
- 3m Casing

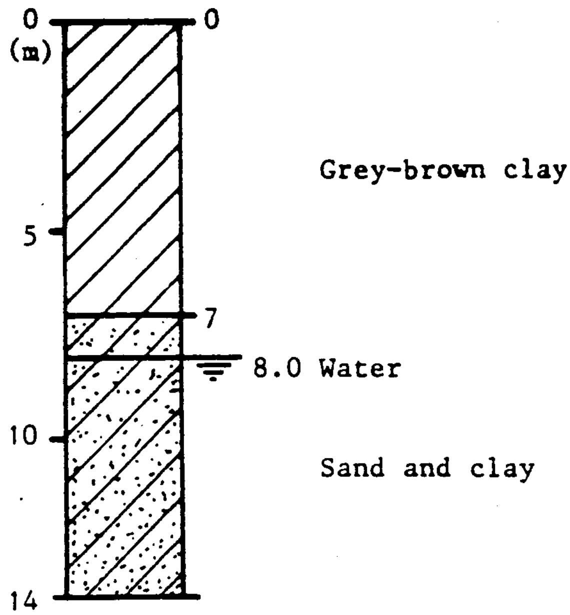
Fig. 4.11 Borehole Log



Borehole BW3a - Drilled 30-3-1989

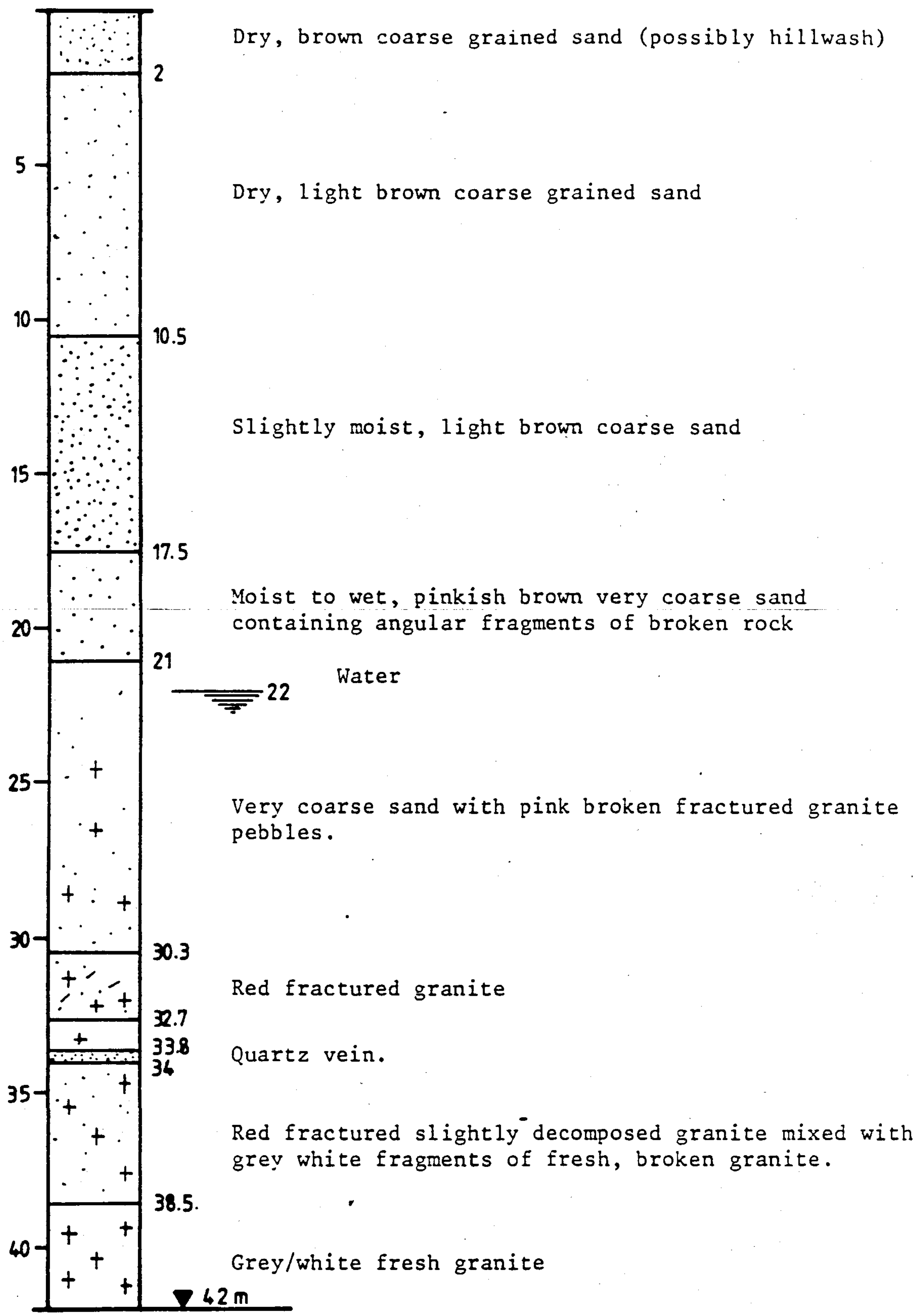
- Casing 14m
- Seepage Water 12m

Fig. 4.12 Borehole Log



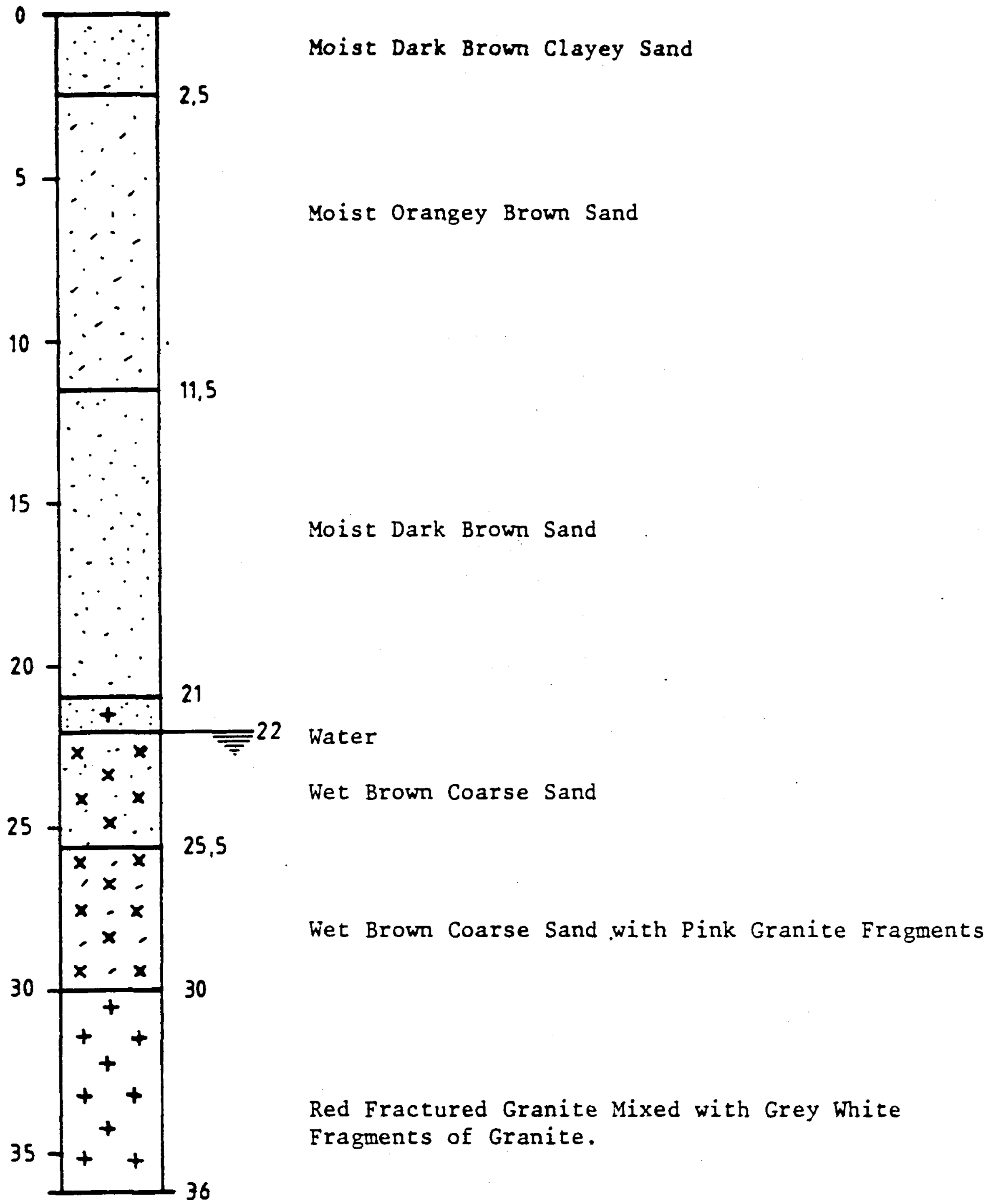
Borehole BW4 - Drilled 11-8-1986
- Bottom Waterval Catchment
- Casing added later

Fig. 4.13 Borehole Log



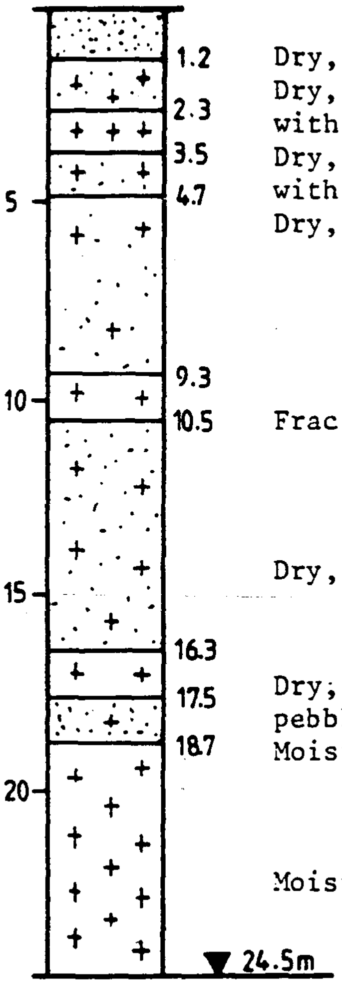
Borehole BW5 - Drilled 11 September, 1986
- Water encountered at 22m
- 7m casing

Fig. 4.14 Borehole Log



Borehole BW5a - Drilled 30-3-1989
- 8m Casing

Fig. 4.15 Borehole Log



Dry, brown sand
Dry, grey/red fairly weathered coarse grained granite with large (10mm) pebbles
Dry, grey/red fairly weathered coarse grained granite with large (20 - 30mm) angular pebbles.
Dry, grey/red fine grained granite with angular pebbles.

Fractured quartz pebbles.

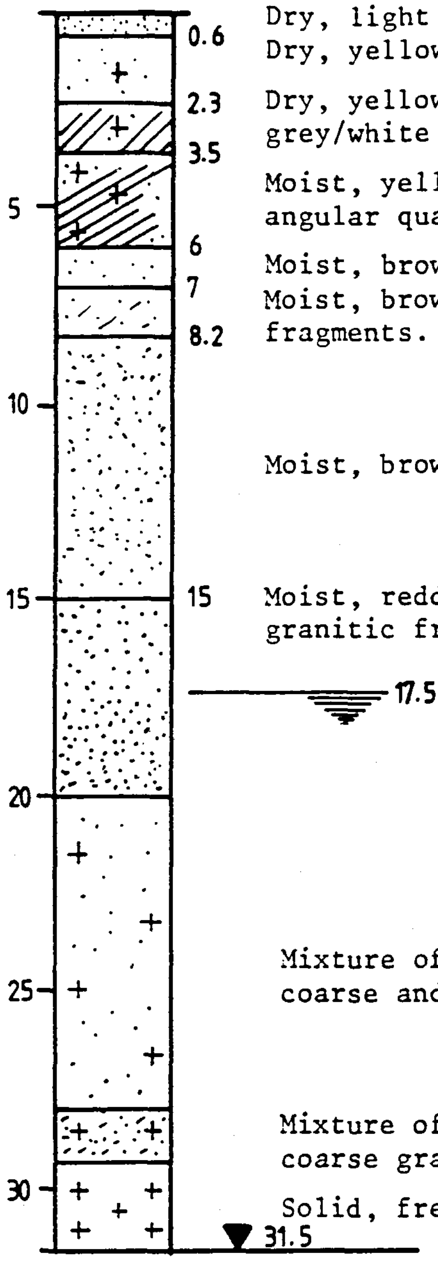
Dry, light brown medium grained decomposed granite.

Dry, grey/pink weathered granite with angular quartzitic pebbles.
Moist, greenish/grey hard layer with rounded pebbles

Moist, grey slightly weathered solid granite

Borehole BW6
- Drilled 11 September, 1986.
- NO WATER encountered, possibly due to close proximity of lots of bluegum trees.
- 1m Casing.
- Presently coded as BW7

Fig. 4.16 Borehole Log



Dry, light brown to grey clayey sand.
Dry, yellow, lightly weathered granite.

Dry, yellow, lightly weathered granite mixed with grey/white silty sand.

Moist, yellow, clayey silt matrix containing small angular quartzitic pebbles.

Moist, brown, silty sand with coarse fractions

Moist, brown coarse grained sand with pinkish quartzitic fragments.

Moist, brown, coarse grained sand (decomposed granite)

Moist, reddish, dark brown coarse sand with angular granitic fragments (approx. 1mm)

Water

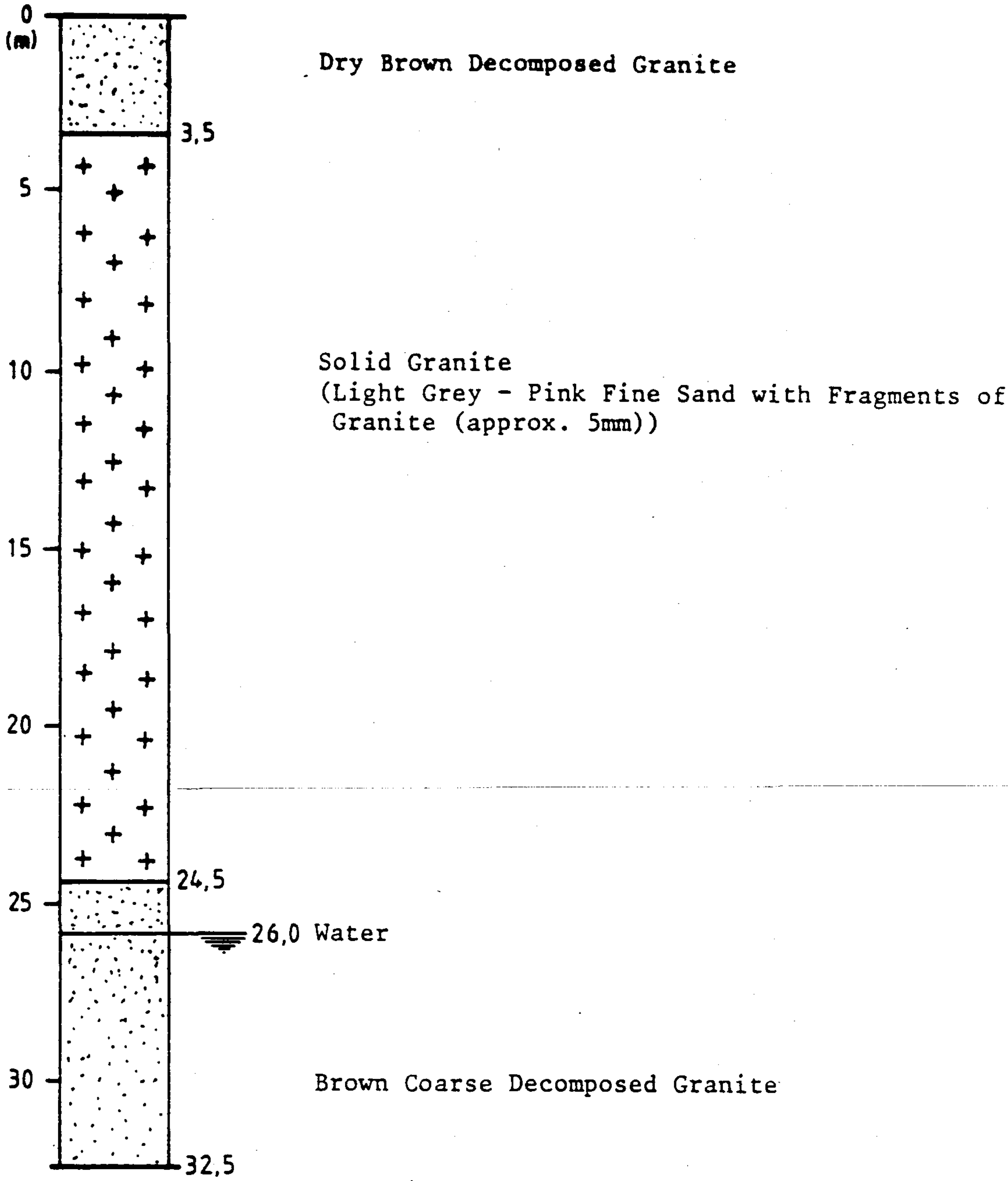
Mixture of pinkish and grey/black granite fragments, coarse and fractured.

Mixture of pinkish and grey/black granite fragments, coarse grained.

Solid, fresh, hard, pink granite.

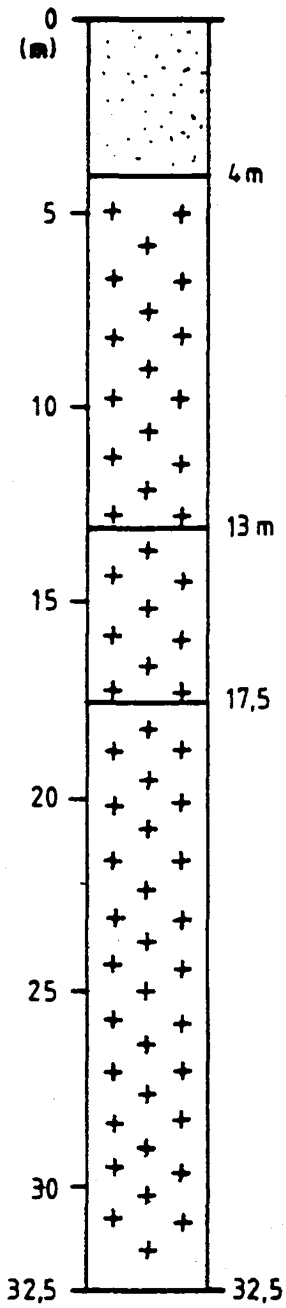
Borehole BW7	-	Drilled 11 September, 1986.
	-	Seepage at 10,5m
	-	Water at 17,5m
	-	2m Casing.
	-	Presently coded as BW6

Fig. 4.17 Borehole Log



Borehole BW8 - Drilled 29-3-1989
- 4m Casing

Fig. 4.18 Borehole Log



Dry Brown-Grey Sand

Solid Granite
(Dry Light Grey-Pink Fine Sand with Fragments of Granite (approx 5mm))

Fractured Granite
(Pink Granite Fragments, Coarse and Fractured)

Solid Granite
(Dry Light Grey-Pink Fine Sand with Fragments of Granite (approx. 10mm))

Borehole BW9 - Drilled 30-3-1989
- 4m Casing
- No Water

Fig. 4.19 Borehole Log

Table 4.1 Borehole water levels in the Sunninghill catchment (metres below top casing)

Date			BS1	BS2	BS3	BS4	BS5	BS6
27	8	1986	12.16	9.52	1.65	-9.00	-9.00	-9.00
4	9	1986	12.29	9.68	1.15	-9.00	-9.00	-9.00
12	9	1986	12.26	9.67	1.41	-9.00	-9.00	-9.00
19	9	1986	12.40	9.89	2.05	-9.00	-9.00	-9.00
28	9	1986	12.33	9.86	2.01	-9.00	-9.00	-9.00
2	10	1986	12.46	9.91	1.86	-9.00	-9.00	-9.00
9	10	1986	12.38	9.86	2.10	1.88	-9.00	-9.00
20	10	1986	12.52	10.04	0.84	1.97	-9.00	-9.00
7	11	1986	12.42	9.50	0.40	1.90	-9.00	-9.00
18	11	1986	12.15	9.59	0.38	1.85	-9.00	-9.00
28	11	1986	11.92	9.45	0.45	1.80	-9.00	-9.00
5	12	1986	11.40	9.30	0.30	1.60	-9.00	-9.00
12	12	1986	11.30	9.35	0.40	1.56	-9.00	-9.00
19	12	1986	11.37	9.48	0.32	1.24	-9.00	-9.00
6	3	1987	8.19	6.93	0.32	0.96	-9.00	-9.00
13	3	1987	8.23	6.84	0.35	0.94	-9.00	-9.00
20	3	1987	8.14	6.64	0.33	0.78	-9.00	-9.00
27	3	1987	7.80	6.36	0.27	0.82	-9.00	-9.00
3	4	1987	7.43	5.99	0.33	0.76	-9.00	-9.00
10	4	1987	7.37	5.84	0.22	0.67	-9.00	-9.00
21	4	1987	7.45	5.39	0.22	0.52	-9.00	-9.00
27	4	1987	7.57	5.52	0.22	0.55	-9.00	-9.00
4	5	1987	7.64	5.51	0.23	0.41	-9.00	-9.00
11	5	1987	7.45	5.44	0.25	0.33	-9.00	-9.00
18	5	1987	7.62	5.40	0.26	0.27	-9.00	-9.00
25	5	1987	7.90	5.49	0.27	0.30	-9.00	-9.00
29	5	1987	8.09	5.51	0.29	0.31	-9.00	-9.00
8	6	1987	8.31	5.52	0.30	0.27	-9.00	-9.00
15	6	1987	8.30	5.52	0.29	0.25	-9.00	-9.00
22	6	1987	8.68	5.61	0.30	0.32	-9.00	-9.00
29	6	1987	8.61	5.58	0.31	0.26	-9.00	-9.00
3	7	1987	8.60	5.49	0.29	0.21	-9.00	-9.00
13	7	1987	8.78	5.61	0.30	0.24	-9.00	-9.00
15	7	1987	8.87	5.64	0.32	0.28	-9.00	-9.00
6	8	1987	9.17	5.73	0.37	0.28	-9.00	-9.00
13	8	1987	9.23	5.73	0.34	0.25	-9.00	-9.00
20	8	1987	9.41	5.87	0.26	0.33	-9.00	-9.00
27	8	1987	9.46	5.90	0.28	0.31	-9.00	-9.00
3	9	1987	9.50	5.90	0.36	0.26	-9.00	-9.00
10	9	1987	9.62	5.95	0.30	0.29	-9.00	-9.00
17	9	1987	9.69	5.98	0.34	0.28	-9.00	-9.00
24	9	1987	9.71	5.96	0.38	0.24	-9.00	-9.00
9	10	1987	9.64	5.92	0.34	0.24	-9.00	-9.00
15	10	1987	9.58	5.88	0.32	0.23	-9.00	-9.00
30	10	1987	9.42	5.82	0.40	0.05	-9.00	-9.00
6	11	1987	9.38	5.81	0.42	0.01	-9.00	-9.00
14	11	1987	9.38	5.81	0.45	0.00	-9.00	-9.00
23	11	1987	9.36	5.80	0.47	0.00	-9.00	-9.00
13	12	1987	9.23	5.77	0.46	0.00	-9.00	-9.00
21	12	1987	8.98	5.48	0.44	0.00	-9.00	-9.00
31	12	1987	8.82	5.40	0.40	0.00	-9.00	-9.00
10	1	1988	8.70	5.37	0.40	0.00	-9.00	-9.00
17	1	1988	8.63	5.32	0.40	0.00	-9.00	-9.00
26	1	1988	8.66	5.37	0.41	0.00	-9.00	-9.00
3	2	1988	8.68	5.39	0.41	0.00	-9.00	-9.00
12	2	1988	8.80	5.48	0.46	0.00	-9.00	-9.00
8	3	1988	3.87	5.43	0.41	0.00	-9.00	-9.00
15	3	1988	8.45	5.20	0.20	0.00	-9.00	-9.00
22	3	1988	8.78	5.05	0.38	0.00	-9.00	-9.00
30	3	1988	8.12	5.01	0.40	0.00	-9.00	-9.00

Table 4.1 (continued) Borehole water levels in the Sunninghill catchment (metres below top casing)

Date			BS1	BS2	BS3	BS4	BS5	BS6
7	4	1988	7.89	4.92	0.39	0.00	-9.00	-9.00
12	4	1988	8.08	4.97	0.38	0.00	-9.00	-9.00
19	4	1988	7.89	4.89	0.34	0.00	-9.00	-9.00
2	5	1988	8.25	4.93	0.39	0.00	-9.00	-9.00
10	5	1988	8.31	4.97	0.42	0.00	-9.00	-9.00
17	5	1988	8.36	4.96	0.42	0.00	-9.00	-9.00
23	5	1988	8.43	4.97	0.42	0.00	-9.00	-9.00
1	6	1988	8.79	5.06	0.44	0.00	-9.00	-9.00
7	6	1988	8.64	5.03	0.41	0.00	-9.00	-9.00
14	6	1988	8.70	5.04	0.44	0.00	-9.00	-9.00
28	6	1988	8.76	5.12	0.44	0.00	-9.00	-9.00
4	7	1988	9.11	5.20	0.42	0.00	-9.00	-9.00
19	7	1988	9.19	5.25	0.25	0.00	-9.00	-9.00
26	7	1988	9.26	5.32	0.25	0.00	-9.00	-9.00
15	8	1988	9.50	5.49	0.26	0.00	-9.00	-9.00
23	8	1988	9.52	5.54	0.30	0.00	-9.00	-9.00
30	8	1988	9.54	5.52	0.30	0.00	-9.00	-9.00
5	9	1988	9.64	5.57	0.32	0.00	-9.00	-9.00
13	9	1988	9.69	5.65	0.35	0.00	-9.00	-9.00
19	9	1988	9.70	5.62	0.33	0.00	-9.00	-9.00
26	9	1988	9.74	5.64	0.46	0.00	-9.00	-9.00
3	10	1988	9.80	5.66	0.56	0.00	-9.00	-9.00
11	10	1988	9.82	5.71	0.76	0.00	-9.00	-9.00
18	10	1988	9.84	5.67	0.55	0.00	-9.00	-9.00
25	10	1988	9.86	5.67	0.53	0.00	-9.00	-9.00
31	10	1988	9.85	5.65	0.55	0.00	-9.00	-9.00
7	11	1988	9.84	5.65	0.54	0.00	-9.00	-9.00
14	11	1988	9.81	5.68	0.53	0.00	-9.00	-9.00
21	11	1988	9.75	5.69	0.55	0.00	-9.00	-9.00
29	11	1988	9.76	5.69	0.55	0.00	-9.00	-9.00
5	12	1988	9.66	5.50	0.49	0.00	-9.00	-9.00
12	12	1988	9.58	5.46	0.43	0.00	-9.00	-9.00
19	12	1988	9.48	5.32	0.43	0.00	-9.00	-9.00
4	1	1989	9.31	5.27	0.47	0.00	-9.00	-9.00
9	1	1989	9.25	5.18	0.42	0.00	-9.00	-9.00
16	1	1989	9.20	5.14	0.47	0.00	-9.00	-9.00
24	1	1989	9.14	5.14	0.47	0.00	-9.00	-9.00
7	2	1989	9.09	5.12	0.45	0.00	-9.00	-9.00
14	2	1989	8.89	4.97	0.41	0.00	-9.00	-9.00
21	2	1989	8.67	4.82	0.39	0.00	-9.00	-9.00
28	2	1989	8.23	4.50	0.32	0.00	-9.00	-9.00
7	3	1989	7.97	4.35	0.38	0.00	-9.00	-9.00
14	3	1989	7.76	4.25	0.37	0.00	-9.00	-9.00
21	3	1989	7.71	4.29	0.38	0.00	-9.00	-9.00
29	3	1989	7.72	4.33	0.39	0.00	-9.00	-9.00
4	4	1989	7.78	4.34	0.40	0.00	-9.00	-9.00
15	4	1989	8.20	4.41	0.39	0.00	0.00	8.68
2	5	1989	8.06	4.40	0.38	0.00	0.00	2.68
8	5	1989	8.12	4.38	0.39	0.00	0.00	2.73
15	5	1989	8.21	4.32	0.39	0.00	0.00	2.88
23	5	1989	8.27	4.35	0.38	0.00	0.00	2.78
28	5	1989	8.86	4.40	0.39	0.00	0.00	2.77
18	6	1989	8.39	4.36	0.29	0.00	0.00	2.76
30	6	1989	8.41	4.35	0.39	0.00	0.00	2.67
27	6	1989	8.43	4.30	0.38	0.00	0.00	2.61

Table 4.1 (continued) Borehole water levels in the Sunninghill catchment (metres below top casing)

Date			BS1	BS2	BS3	BS4	BS5	BS6
3	7	1989	8.44	4.30	0.39	0.00	0.00	2.61
10	7	1989	8.52	4.35	0.38	0.00	0.00	2.68
17	7	1989	8.58	4.37	0.37	0.00	0.00	2.70
24	7	1989	8.69	4.47	0.37	0.00	0.00	2.79
31	7	1989	8.70	4.49	0.36	0.00	0.00	2.74
7	8	1989	8.79	4.49	0.34	0.00	0.00	2.76
14	8	1989	8.83	4.56	0.35	0.00	0.00	2.79
21	8	1989	8.95	4.62	0.35	0.00	0.00	2.79
28	8	1989	9.02	4.70	0.36	0.00	0.00	2.85
4	9	1989	9.08	4.35	0.36	0.00	0.00	2.77
11	9	1989	9.17	5.03	0.40	0.00	0.00	2.88
18	9	1989	9.17	5.05	0.39	0.00	0.00	2.83
25	9	1989	9.31	5.14	0.44	0.00	0.00	2.94
2	10	1989	9.37	5.19	0.50	0.00	0.00	2.89
9	10	1989	9.42	5.23	0.48	0.00	0.00	2.91
16	10	1989	9.47	5.30	0.68	0.00	0.00	3.00
23	10	1989	9.55	5.33	0.56	0.00	0.00	3.00
30	10	1989	9.61	5.45	0.77	0.00	0.00	3.03
8	11	1989	9.62	5.39	0.56	0.00	0.00	3.02
13	11	1989	9.61	5.35	0.53	0.00	0.00	3.02
20	11	1989	9.58	5.34	0.65	0.00	0.00	3.01
27	11	1989	9.42	5.26	0.52	0.00	0.00	2.92
4	12	1989	9.28	5.14	0.48	0.00	0.00	2.69
11	12	1989	9.21	5.10	0.56	0.00	0.00	2.81
18	12	1989	9.10	5.07	0.53	0.00	0.00	2.70
23	12	1989	9.16	5.26	0.85	0.00	0.00	2.71
10	1	1990	9.10	5.25	0.88	0.00	0.00	2.68
16	1	1990	9.11	5.30	0.80	0.00	0.00	2.72
22	1	1990	9.30	5.40	0.55	0.00	0.00	2.73
31	1	1990	9.19	5.48	1.10	0.00	0.00	2.73
7	2	1990	9.22	5.51	0.88	0.00	0.00	2.76
3	4	1990	9.10	5.15	0.60	0.00	0.00	2.65
10	7	1990	9.28	5.80	0.55	0.00	0.00	2.60
17	4	1990	9.25	4.85	0.52	0.00	0.00	2.65
24	4	1990	9.23	4.83	0.50	0.00	0.00	2.65
3	5	1990	9.25	4.95	0.46	0.00	0.00	2.50
14	5	1990	9.08	5.10	0.47	0.00	0.00	2.90
17	5	1990	9.00	55.10	0.47	0.00	0.00	2.30
23	5	1990	9.20	5.35	0.48	0.00	0.00	2.40
16	10	1990	10.00	5.75	0.60	0.00	0.00	2.80
23	10	1990	10.05	5.73	0.64	0.00	0.00	2.85
30	10	1990	10.25	5.75	0.85	0.00	0.00	2.90
7	11	1990	10.30	5.60	0.97	0.00	0.00	2.93
13	11	1990	10.28	5.60	0.99	0.00	0.00	2.94
20	11	1990	10.10	5.70	1.07	0.00	0.00	2.80

Table 4.2 Borehole water levels in the Waterval catchment (metres below top casing)

Date			BW1	BW2	BW3	BW3a	BW4	BW5	BW5a	BW7	BW6	BW8	BW9
27	8	1986	7.44	7.81	6.07	-9.00	2.30	-9.00	-9.00	-9.00	-9.00	-9.00	-9.00
4	8	1986	7.45	7.39	6.11	-9.00	2.38	-9.00	-9.00	-9.00	-9.00	-9.00	-9.00
12	8	1986	7.34	7.98	6.20	-9.00	2.50	-9.00	-9.00	-9.00	-9.00	-9.00	-9.00
18	9	1986	7.44	8.07	6.28	-9.00	2.75	-9.00	-9.00	-9.00	-9.00	-9.00	-9.00
25	9	1986	7.36	8.10	6.28	-9.00	2.75	-9.00	-9.00	-9.00	-9.00	-9.00	-9.00
2	10	1986	7.38	8.00	6.19	-9.00	2.75	-9.00	-9.00	-9.00	-9.00	-9.00	-9.00
9	10	1986	7.33	8.03	6.20	-9.00	2.73	13.04	-9.00	12.39	8.36	-9.00	-9.00
20	10	1986	7.33	8.03	6.22	-9.00	2.73	13.10	-9.00	12.41	8.34	-9.00	-9.00
7	11	1986	7.20	7.63	5.70	-9.00	2.60	12.40	-9.00	10.20	8.35	-9.00	-9.00
19	11	1986	7.02	7.19	5.24	-9.00	1.81	13.14	-9.00	12.32	8.18	-9.00	-9.00
28	11	1986	6.80	7.10	5.07	-9.00	2.00	13.14	-9.00	12.41	8.13	-9.00	-9.00
5	12	1986	7.00	6.80	4.60	-9.00	1.90	13.20	-9.00	12.20	8.23	-9.00	-9.00
10	12	1986	6.80	7.00	4.63	-9.00	1.84	13.15	-9.00	12.10	8.20	-9.00	-9.00
19	12	1986	6.49	9.82	5.00	-9.00	2.26	13.19	-9.00	12.19	8.00	-9.00	-9.00
6	3	1987	3.08	6.13	4.44	-9.00	2.36	12.83	-9.00	11.60	7.60	-9.00	-9.00
13	3	1987	3.13	6.23	4.52	-9.00	2.40	12.86	-9.00	11.61	7.37	-9.00	-9.00
20	3	1987	2.90	8.99	4.25	-9.00	1.62	12.81	-9.00	11.61	7.25	-9.00	-9.00
27	3	1987	2.61	4.49	3.66	-9.00	0.24	12.78	-9.00	11.57	6.94	-9.00	-9.00
3	4	1987	2.27	5.21	3.58	-9.00	0.24	12.61	-9.00	11.35	6.79	-9.00	-9.00
10	4	1987	2.27	5.12	3.85	-9.00	0.26	12.63	-9.00	11.30	6.81	-9.00	-9.00
21	4	1987	2.14	5.37	4.02	-9.00	0.27	12.53	-9.00	11.21	7.05	-9.00	-9.00
27	4	1987	2.17	4.38	4.19	-9.00	0.27	12.77	-9.00	11.23	7.20	-9.00	-9.00
4	5	1987	2.16	5.42	4.06	-9.00	0.29	12.39	-9.00	11.11	6.81	-9.00	-9.00
11	5	1987	2.26	5.63	4.26	-9.00	0.30	12.32	-9.00	11.00	6.86	-9.00	-9.00
13	5	1987	2.36	5.73	4.32	-9.00	0.31	12.33	-9.00	10.98	6.88	-9.00	-9.00
25	5	1987	2.36	5.90	4.38	-9.00	0.32	12.35	-9.00	10.97	6.92	-9.00	-9.00
29	5	1987	2.40	5.96	4.41	-9.00	0.33	12.38	-9.00	11.11	6.96	-9.00	-9.00
8	6	1987	2.49	6.09	4.52	-9.00	0.35	12.38	-9.00	11.07	6.98	-9.00	-9.00
15	6	1987	2.42	6.08	4.42	-9.00	0.36	12.34	-9.00	11.02	6.99	-9.00	-9.00
22	6	1987	2.56	6.17	4.60	-9.00	0.38	12.38	-9.00	11.23	7.12	-9.00	-9.00
29	6	1987	2.60	6.21	4.61	-9.00	0.39	12.35	-9.00	11.05	7.11	-9.00	-9.00
3	7	1987	2.58	6.21	4.62	-9.00	0.39	12.27	-9.00	11.01	7.01	-9.00	-9.00
13	7	1987	3.70	6.30	4.68	-9.00	0.42	12.28	-9.00	11.00	7.13	-9.00	-9.00
15	7	1987	2.69	6.36	4.75	-9.00	0.44	12.30	-9.00	11.05	7.15	-9.00	-9.00
3	7	1987	2.91	6.50	4.89	-9.00	0.46	12.16	-9.00	10.96	7.17	-9.00	-9.00
13	8	1987	2.84	6.52	4.80	-9.00	0.48	12.11	-9.00	10.94	7.16	-9.00	-9.00
20	8	1987	2.38	6.62	5.00	-9.00	0.49	12.16	-9.00	11.00	7.23	-9.00	-9.00
27	8	1987	2.89	6.60	4.92	-9.00	0.49	12.11	-9.00	10.95	7.21	-9.00	-9.00
3	9	1987	2.91	6.53	4.27	-9.00	0.50	12.06	-9.00	10.90	7.18	-9.00	-9.00
10	9	1987	2.93	6.48	4.88	-9.00	0.50	12.09	-9.00	10.92	7.20	-9.00	-9.00
17	9	1987	2.96	6.50	4.86	-9.00	0.51	12.08	-9.00	10.70	7.20	-9.00	-9.00
24	9	1987	2.96	6.42	4.79	-9.00	0.52	12.02	-9.00	10.50	7.13	-9.00	-9.00
9	10	1987	2.81	5.93	4.29	-9.00	0.38	11.96	-9.00	10.22	6.86	-9.00	-9.00
15	10	1987	6.69	5.70	4.09	-9.00	0.35	11.94	-9.00	10.19	6.77	-9.00	-9.00
30	10	1987	2.61	5.74	4.22	-9.00	0.36	11.84	-9.00	10.59	6.70	-9.00	-9.00
6	11	1987	2.60	5.80	4.35	-9.00	0.36	11.83	-9.00	10.42	6.66	-9.00	-9.00
12	11	1987	2.65	5.85	4.48	-9.00	0.38	11.81	-9.00	10.36	6.71	-9.00	-9.00
13	11	1987	2.72	5.97	4.55	-9.00	0.35	11.80	-9.00	10.30	6.79	-9.00	-9.00
13	12	1987	2.69	5.91	4.48	-9.00	0.30	11.30	-9.00	9.30	6.73	-9.00	-9.00
21	12	1987	2.65	5.85	4.27	-9.00	0.23	11.30	-9.00	-9.00	6.72	-9.00	-9.00
31	12	1987	2.62	5.94	4.35	-9.00	0.26	11.75	-9.00	-9.00	6.72	-9.00	-9.00
10	1	1988	2.65	6.18	4.59	-9.00	0.28	11.71	-9.00	-9.00	6.76	-9.00	-9.00
17	1	1988	2.63	6.21	4.59	-9.00	0.28	11.65	-9.00	-9.00	6.76	-9.00	-9.00
26	1	1988	1.77	2.35	4.76	-9.00	0.30	11.68	-9.00	-9.00	6.84	-9.00	-9.00
2	2	1988	2.81	2.43	4.70	-9.00	0.32	11.75	-9.00	-9.00	6.80	-9.00	-9.00
12	2	1988	2.93	6.40	4.72	-9.00	0.32	11.86	-9.00	-9.00	6.85	-9.00	-9.00
8	3	1988	2.86	6.27	4.30	-9.00	0.24	11.63	-9.00	-9.00	6.74	-9.00	-9.00
15	3	1988	2.35	5.75	3.91	-9.00	0.16	11.50	-9.00	-9.00	6.05	-9.00	-9.00
22	3	1988	2.68	5.55	3.96	-9.00	0.20	11.55	-9.00	-9.00	6.40	-9.00	-9.00
26	3	1988	2.51	5.65	4.06	-9.00	0.25	11.45	-9.00	-9.00	6.46	-9.00	-9.00

Table 4.2 (continued) Borehole water levels in the Waterval catchment (metres below top casing)

Date			BW1	BW2	BW3	BW3a	BW4	BW5	BW5a	BW7	BW6	BW8	BW9
7	4	1988	2.55	5.69	4.18	-9.00	0.27	11.39	-9.00	-9.00	6.44	-9.00	-9.00
12	4	1988	2.58	5.76	4.23	-9.00	0.27	11.43	-9.00	-9.00	6.50	-9.00	-9.00
19	4	1988	2.58	5.67	4.08	-9.00	0.28	11.12	-9.00	-9.00	6.44	-9.00	-9.00
2	5	1988	2.75	5.91	4.46	-9.00	0.29	11.42	-9.00	-9.00	6.52	-9.00	-9.00
10	5	1988	2.77	5.96	4.48	-9.00	0.30	11.36	-9.00	-9.00	6.57	-9.00	-9.00
17	5	1988	2.72	6.06	4.53	-9.00	0.32	11.29	-9.00	-9.00	6.60	-9.00	-9.00
25	5	1988	2.72	6.14	4.62	-9.00	0.32	11.29	-9.00	-9.00	6.62	-9.00	-9.00
1	6	1988	2.81	6.27	4.81	-9.00	0.33	11.40	-9.00	-9.00	6.75	-9.00	-9.00
7	6	1988	2.76	6.21	4.70	-9.00	0.35	11.22	-9.00	-9.00	6.70	-9.00	-9.00
14	6	1988	2.92	6.31	4.71	-9.00	0.37	11.21	-9.00	-9.00	6.69	-9.00	-9.00
28	6	1988	2.95	6.32	4.80	-9.00	0.39	11.30	-9.00	-9.00	6.71	-9.00	-9.00
4	7	1988	3.06	6.84	4.71	-9.00	0.40	11.16	-9.00	-9.00	6.34	-9.00	-9.00
19	7	1988	3.11	6.42	4.82	-9.00	0.42	11.24	-9.00	-9.00	6.79	-9.00	-9.00
26	7	1988	3.11	6.48	4.85	-9.00	0.43	11.14	-9.00	-9.00	6.81	-9.00	-9.00
15	8	1988	3.18	6.65	4.89	-9.00	0.47	11.14	-9.00	-9.00	6.87	-9.00	-9.00
23	8	1988	3.19	6.75	5.20	-9.00	0.49	11.09	-9.00	-9.00	6.90	-9.00	-9.00
30	8	1988	3.22	6.72	5.20	-9.00	0.50	11.07	-9.00	-9.00	6.88	-9.00	-9.00
5	9	1988	3.24	6.69	4.97	-9.00	0.51	11.09	-9.00	-9.00	6.90	-9.00	-9.00
13	9	1988	3.26	6.71	5.00	-9.00	0.53	11.12	-9.00	-9.00	6.39	-9.00	-9.00
19	9	1988	3.24	6.70	4.98	-9.00	0.53	11.11	-9.00	-9.00	6.90	-9.00	-9.00
26	9	1988	3.28	6.67	4.96	-9.00	0.54	11.04	-9.00	-9.00	6.85	-9.00	-9.00
3	10	1988	3.32	6.64	5.10	-9.00	0.56	11.03	-9.00	-9.00	6.91	-9.00	-9.00
11	10	1988	3.34	7.00	5.22	-9.00	0.57	11.03	-9.00	-9.00	6.94	-9.00	-9.00
16	10	1988	3.35	6.95	5.16	-9.00	0.58	11.03	-9.00	-9.00	6.90	-9.00	-9.00
25	10	1988	3.36	6.78	5.08	-9.00	0.59	11.03	-9.00	-9.00	6.92	-9.00	-9.00
31	10	1988	3.35	6.77	5.09	-9.00	0.59	11.03	-9.00	-9.00	6.91	-9.00	-9.00
9	11	1988	3.36	6.78	5.10	-9.00	0.60	11.04	-9.00	-9.00	6.91	-9.00	-9.00
14	11	1988	3.45	6.92	5.16	-9.00	0.61	11 04	-9.00	-9.00	6.90	-9.00	-9.00
21	11	1988	3.46	6.92	5.15	-9.00	0.62	11.02	-9.00	-9.00	6.80	-9.00	-9.00
29	11	1988	3.46	6.91	5.15	-9.00	0.61	11.03	-9.00	-9.00	6.81	-9.00	-9.00
5	12	1988	3.52	6.96	3.27	-9.00	0.64	11.05	-9.00	-9.00	7.09	-9.00	-9.00
12	12	1988	3.53	6.86	5.23	-9.00	0.64	11.03	-9.00	-9.00	6.94	-9.00	-9.00
13	12	1988	3.54	6.83	5.21	-9.00	0.65	11.04	-9.00	-9.00	6.91	-9.00	-9.00
4	1	1989	3.60	7.12	5.47	-9.00	0.67	11.06	-9.00	-9.00	7.07	-9.00	-9.00
6	1	1989	3.61	7.14	5.46	-9.00	0.67	11.06	-9.00	-9.00	7.04	-9.00	-9.00
16	1	1989	3.56	7.23	5.46	-9.00	0.68	11.01	-9.00	-9.00	6.99	-9.00	-9.00
24	1	1989	3.59	7.33	5.60	-9.00	0.69	11.05	-9.00	-9.00	7.01	-9.00	-9.00
9	2	1989	3.65	7.28	5.58	-9.00	0.70	11.06	-9.00	-9.00	7.05	-9.00	-9.00
14	2	1989	3.63	7.10	5.30	-9.00	0.70	11.08	-9.00	-9.00	6.95	-9.00	-9.00
21	2	1989	3.31	6.53	4.58	-9.00	0.71	11.04	-9.00	-9.00	6.58	-9.00	-9.00
28	2	1989	3.17	6.28	4.42	-9.00	0.71	11.00	-9.00	-9.00	6.51	-9.00	-9.00
11	3	1989	3.07	6.12	4.37	-9.00	0.67	10.94	-9.00	-9.00	6.43	-9.00	-9.00
14	3	1989	2.99	6.07	4.38	-9.00	0.68	10.85	-9.00	-9.00	6.41	-9.00	-9.00
21	3	1989	2.97	6.14	4.46	-9.00	0.68	10.88	-9.00	-9.00	6.44	-9.00	-9.00
25	3	1989	2.94	6.18	4.56	-9.00	0.69	10.36	-9.00	-9.00	6.45	-9.00	-9.00
4	4	1989	2.96	6.29	4.66	-9.00	0.68	10.90	-9.00	-9.00	6.50	-9.00	-9.00
25	4	1989	3.03	6.29	4.60	4.79	0.73	10.91	11.15	-9.00	6.63	20.36	15.76
2	5	1989	3.04	5.24	4.34	4.67	0.73	10.85	11.09	-9.00	6.54	20.35	15.76
8	5	1989	3.06	5.96	4.39	4.67	0.74	10.87	11.10	-9.00	6.65	20.40	15.73
15	5	1989	3.10	6.90	4.44	4.76	0.75	10.89	11.13	-9.00	6.92	20.44	15.82
23	5	1989	3.17	6.15	4.49	4.83	0.75	10.91	11.15	-9.00	6.99	20.38	15.82
29	5	1989	3.12	6.38	4.64	5.00	0.76	10.91	11.15	-9.00	7.10	20.39	15.87
13	6	1989	3.06	6.09	4.50	4.86	0.68	10.90	11.14	-9.00	6.70	20.35	15.83
20	6	1989	3.03	6.10	4.47	4.80	0.68	10.81	11.04	-9.00	6.66	20.32	15.77
27	6	1989	3.02	6.13	4.48	4.73	0.69	10.81	11.04	-9.00	6.69	20.30	15.71

Table 4.2 (continued) Borehole water levels in the Waterval catchment (metres below top casing)

Cate			BW1	BW2	BW3	BW3a	BW4	BW5	BW5a	BW7	BW6	BW8	BW9
3	7	1989	3.03	6.32	4.53	4.83	0.70	10.80	11.02	-9.00	6.61	20.30	15.73
10	7	1989	3.04	6.18	4.60	4.93	0.72	10.79	11.03	-9.00	6.70	20.29	13.73
17	7	1989	3.04	6.23	4.61	4.98	0.72	10.78	11.03	-9.00	6.69	20.29	15.71
24	7	1989	3.04	6.36	4.66	5.04	0.73	10.76	11.09	-9.00	6.74	20.30	15.70
31	7	1989	3.07	6.35	4.76	5.08	0.75	10.87	11.05	-9.00	6.73	20.35	15.74
7	8	1989	3.08	6.42	4.32	5.14	0.75	10.85	11.06	-9.00	6.80	20.37	15.75
14	8	1989	3.09	6.50	4.87	5.17	0.75	10.85	11.03	-9.00	6.75	20.76	15.74
21	8	1989	3.09	6.55	4.90	5.23	0.76	10.70	11.03	-9.00	6.78	20.32	15.78
28	8	1989	3.12	6.65	4.98	5.29	0.77	10.79	11.06	-9.00	6.82	20.37	15.79
4	9	1989	3.09	6.70	5.00	5.33	0.77	10.76	11.05	-9.00	6.87	20.09	15.78
11	9	1989	3.11	6.77	5.06	5.39	0.79	10.77	11.06	-9.00	6.87	20.40	15.84
18	9	1989	3.14	6.83	5.12	5.45	0.80	10.76	11.08	-9.00	6.85	20.38	15.76
5	9	1989	3.20	6.97	5.26	5.60	0.82	10.84	11.08	-9.00	6.94	20.42	15.89
2	10	1989	6.22	7.02	5.27	5.61	0.82	10.75	11.08	-9.00	6.89	20.42	15.80
3	10	1989	3.25	7.84	5.36	5.64	0.83	10.74	11.02	-9.00	6.91	20.43	15.83
16	10	1989	3.38	7.11	5.26	5.71	0.85	10.82	11.05	-9.00	6.97	20.51	15.85
22	10	1989	3.32	7.16	5.38	5.74	0.86	10.81	11.06	-9.00	6.93	20.54	15.36
22	10	1989	3.32	7.16	5.38	5.74	0.26	10.81	11.06	-9.00	6.93	20.54	15.36
25	10	1989	3.77	7.17	5.44	5.80	0.37	10.83	11.08	-9.00	7.00	20.58	15.22
5	11	1989	3.55	7.10	5.23	5.70	0.37	10.83	11.06	-9.00	6.89	20.60	16.00
9	11	1989	3.34	6.80	5.63	5.50	0.37	10.65	11.09	-9.00	6.96	20.61	16.01
30	11	1989	3.63	6.09	5.14	5.90	0.78	10.67	11.18	-9.00	6.73	20.67	16.05
1	12	1989	3.46	6.60	4.87	5.22	0.60	10.68	11.05	-9.00	6.85	20.56	15.96
3	12	1989	3.46	6.62	5.31	5.31	0.64	10.95	11.11	-9.00	6.90	20.56	15.28
23	12	1989	5.54	5.10	5.19	5.58	0.67	10.93	11.27	-9.00	7.00	20.63	18.08
31	1	1990	3.61	7.12	5.35	5.68	10.85	10.85	11.10	-9.00	6.70	20.65	16.85
7	2	1990	3.45	7.15	5.41	5.70	0.68	11.06	11.18	-9.00	6.69	20.66	16.05
6	3	1990	3.80	6.30	5.85	5.40	0.80	11.10	11.05	-9.00	6.85	20.75	16.89
18	4	1990	3.80	6.30	5.20	5.40	0.75	10.85	11.00	-9.00	6.35	20.30	16.87
17	4	1990	3.35	6.33	5.20	5.40	0.77	10.65	10.95	-9.00	6.90	20.85	16.20
24	4	1990	3.63	6.82	5.13	5.25	0.77	10.90	10.95	-9.00	6.80	20.80	16.15
3	5	1990	3.86	6.84	5.20	5.38	0.74	10.80	11.00	-9.00	6.75	20.90	16.10
10	5	1990	3.85	6.65	4.90	5.30	0.72	10.75	11.15	-9.00	6.80	20.80	16.15
17	5	1990	3.90	6.70	4.85	5.10	0.73	10.65	11.10	-9.00	6.90	20.65	16.30
22	5	1990	3.95	6.78	4.90	5.30	0.74	10.90	11.25	-9.00	6.23	20.70	16.35
16	10	1990	4.55	7.55	5.85	6.32	0.96	11.00	11.40	-9.00	7.20	21.10	16.91
23	10	1990	4.60	7.50	5.90	6.30	1.00	11.10	11.30	-9.00	7.20	21.20	16.90
30	10	1990	3.90	7.60	5.90	6.38	1.00	11.70	11.60	-9.00	6.90	22.40	16.95
7	11	1990	4.70	8.10	6.20	6.30	0.98	11.60	11.65	-9.00	7.52	22.60	17.85
18	11	1990	4.71	8.15	6.10	6.35	1.00	11.65	11.60	-9.00	7.50	22.50	17.90
29	11	1990	4.65	7.95	6.20	6.60	1.10	11.20	11.35	-9.00	7.30	21.20	16.90

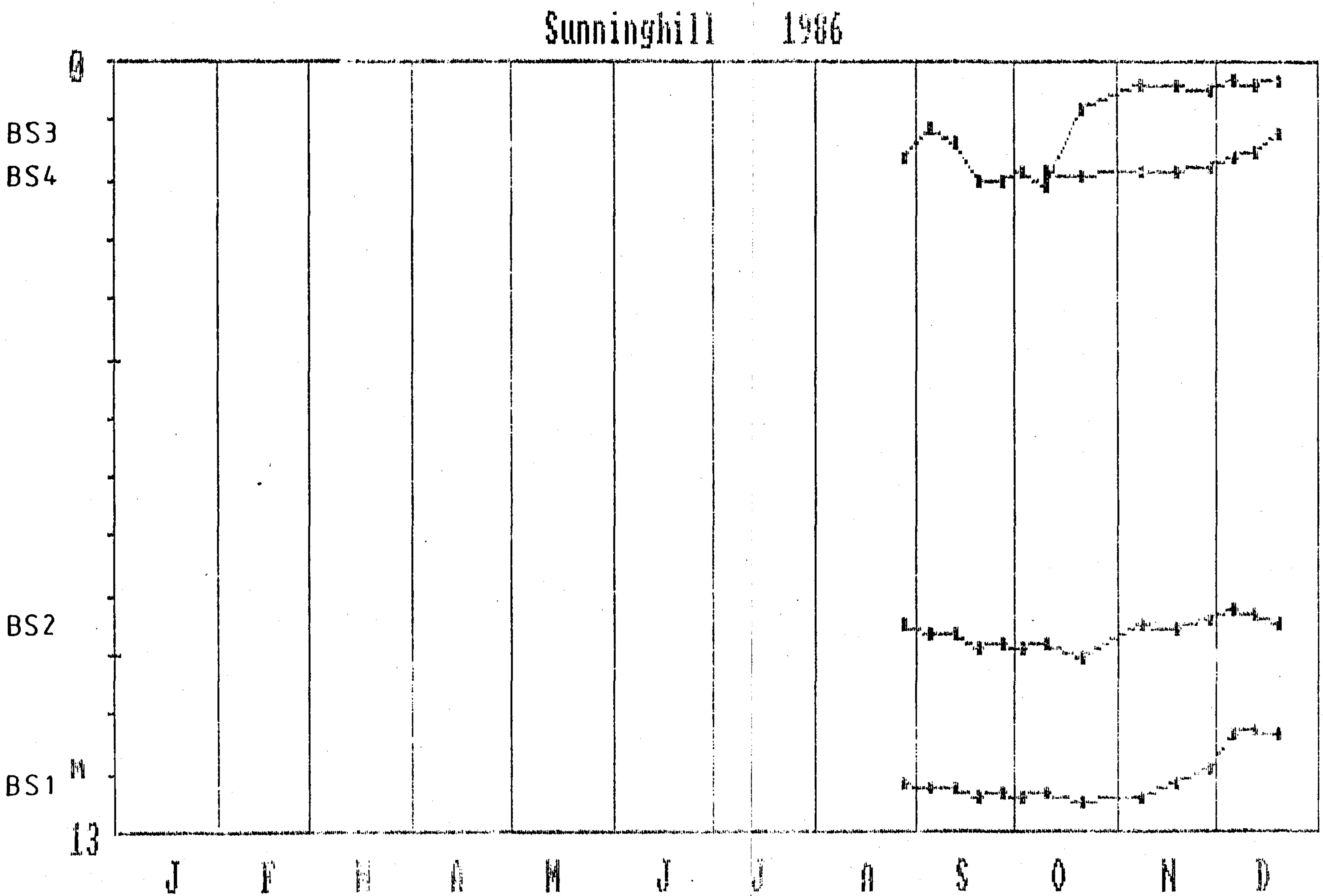


Fig. 4.20 Water Level Variations

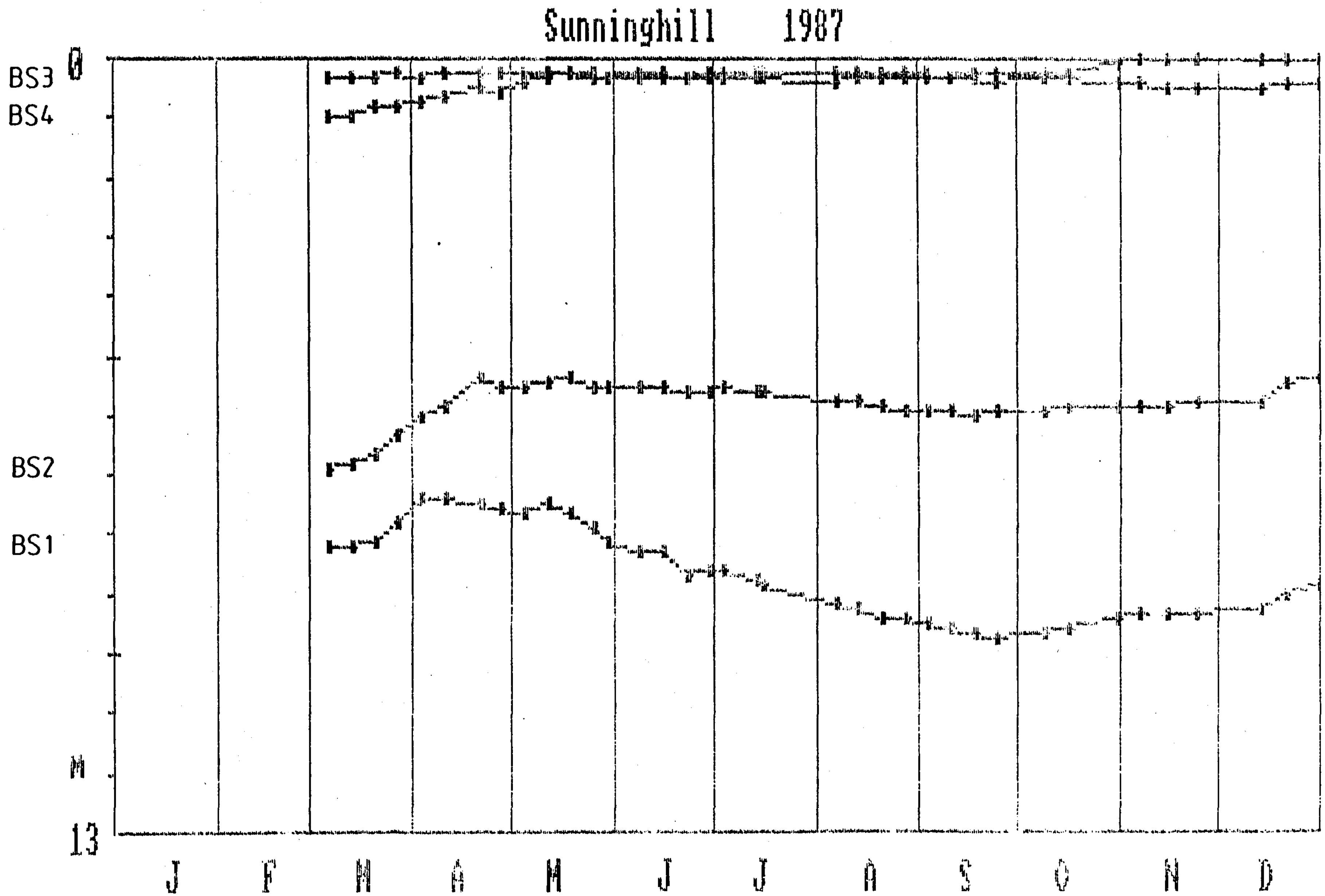


Fig. 4.21 Water Level Variations

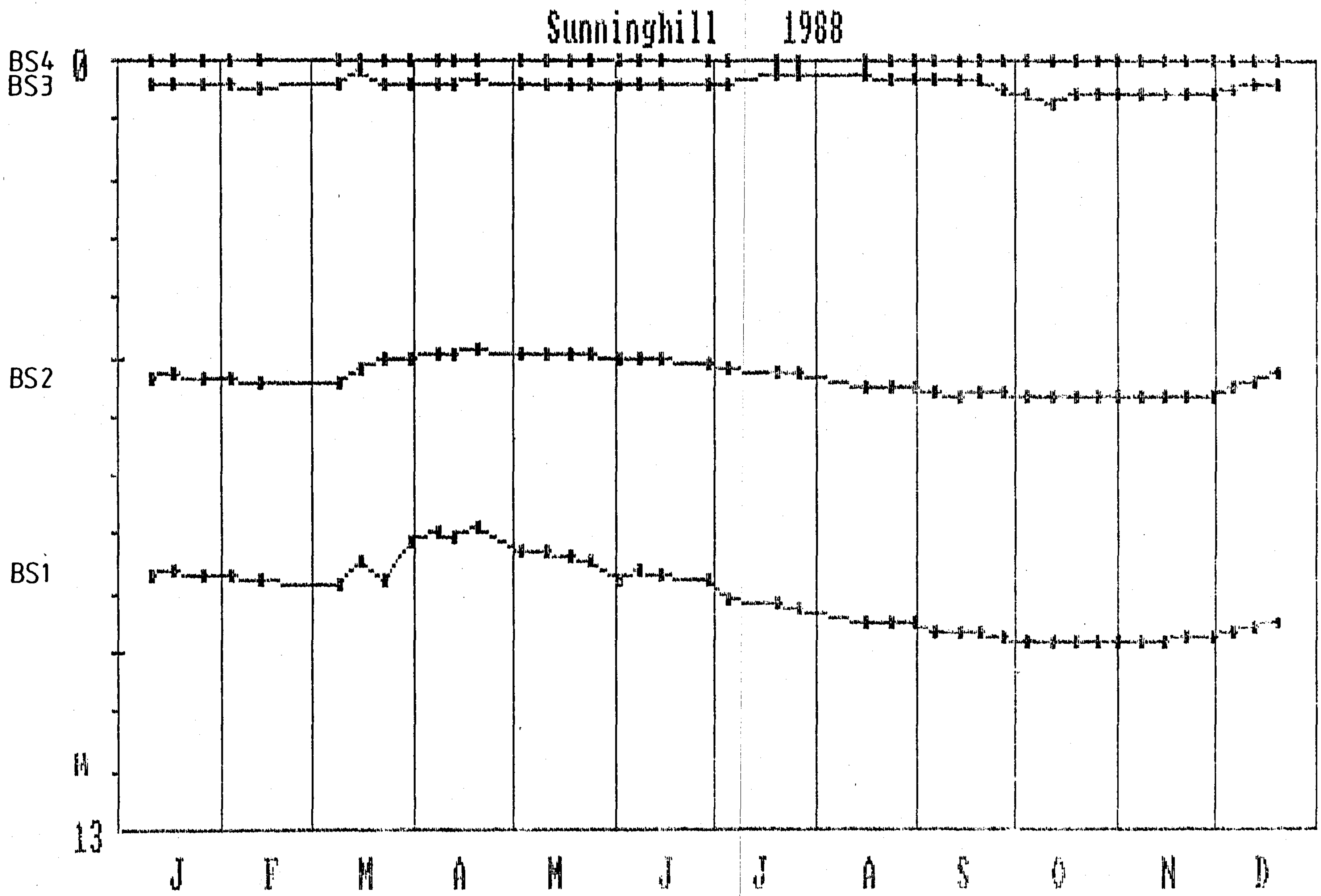


Fig. 4.22 Water Level Variations

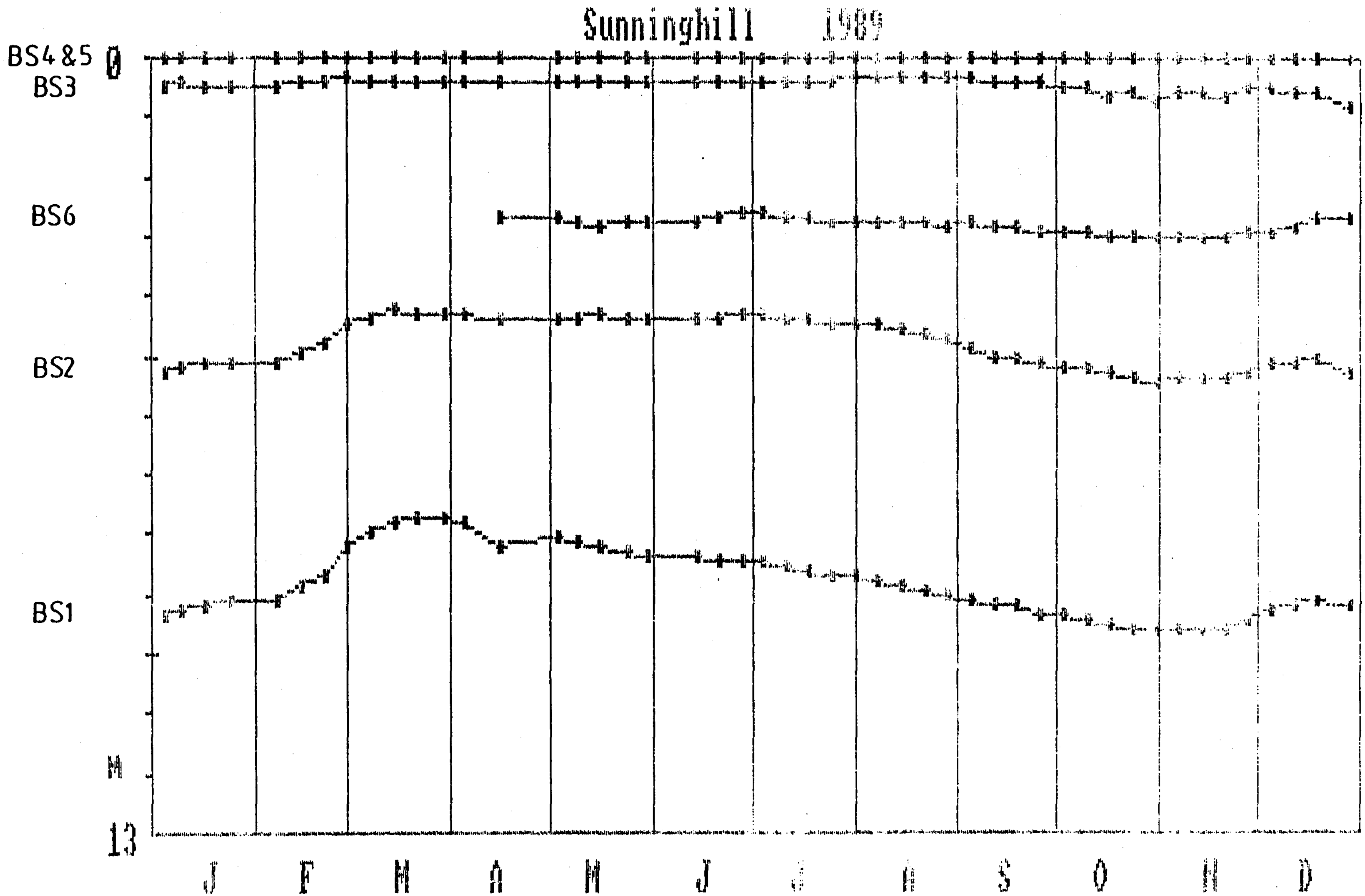


Fig. 4.23 Water Level Variations

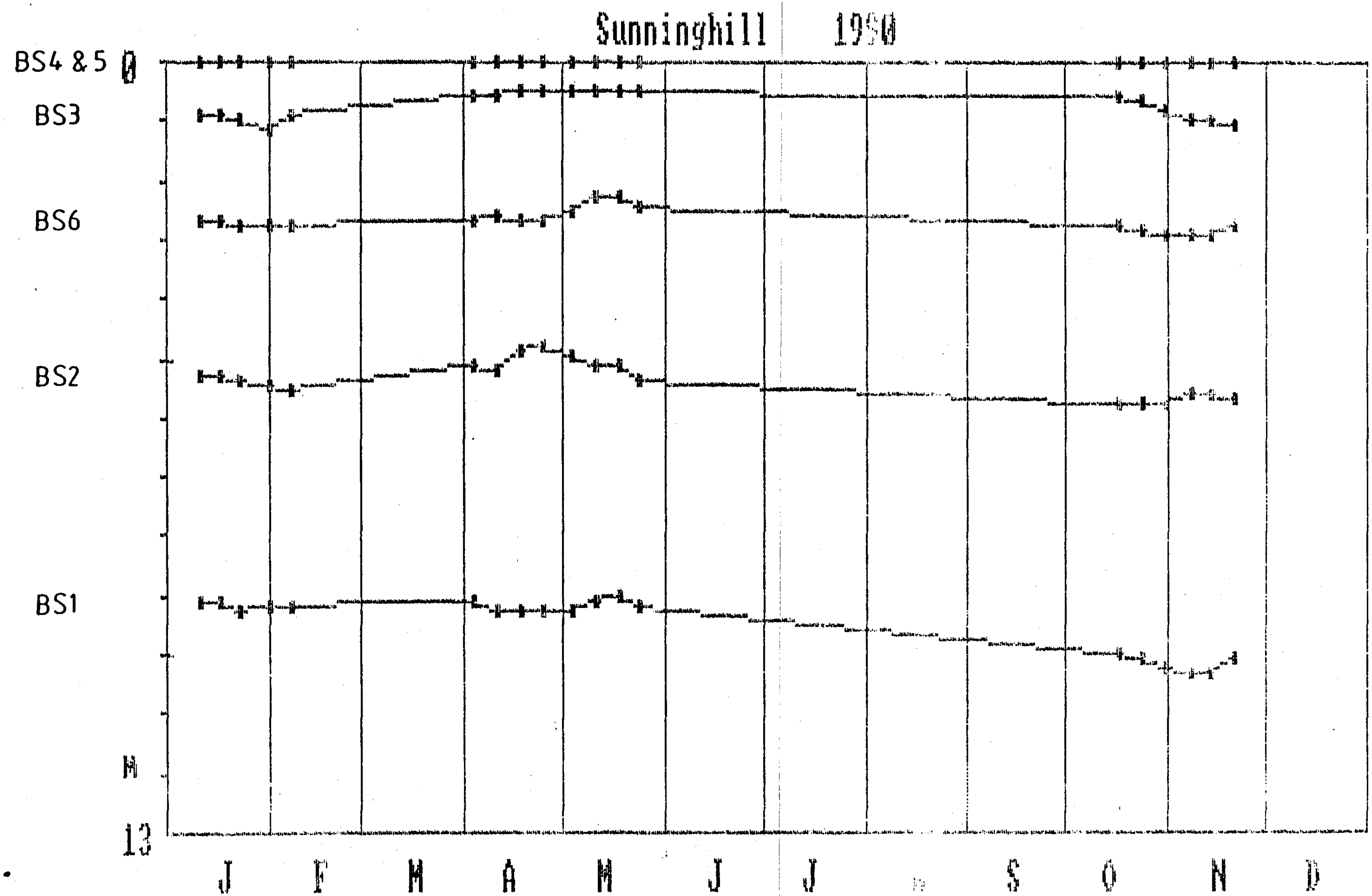


Fig. 4.24 Water Level Variations

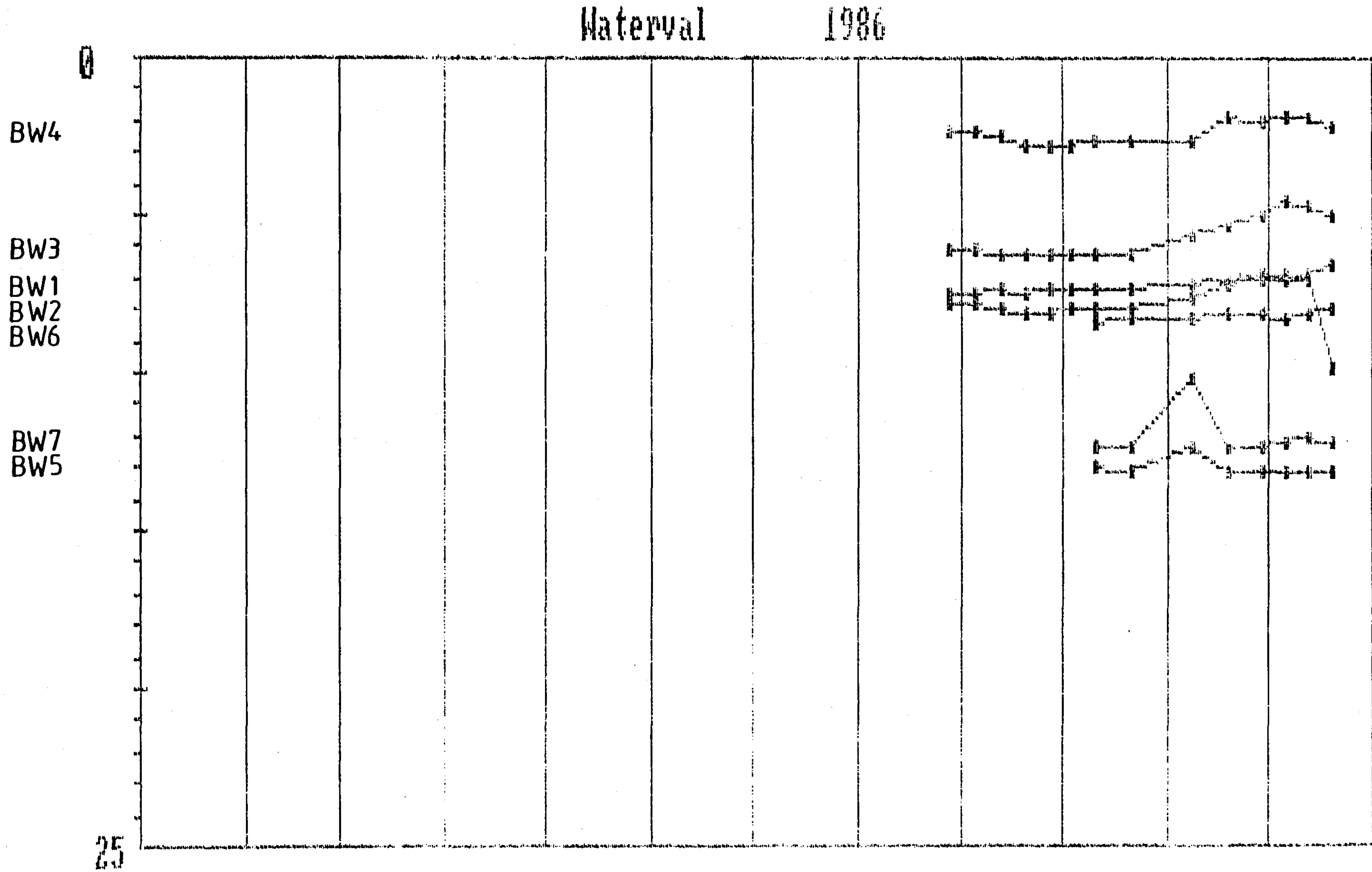


Fig. 4.25 Water Level Variations

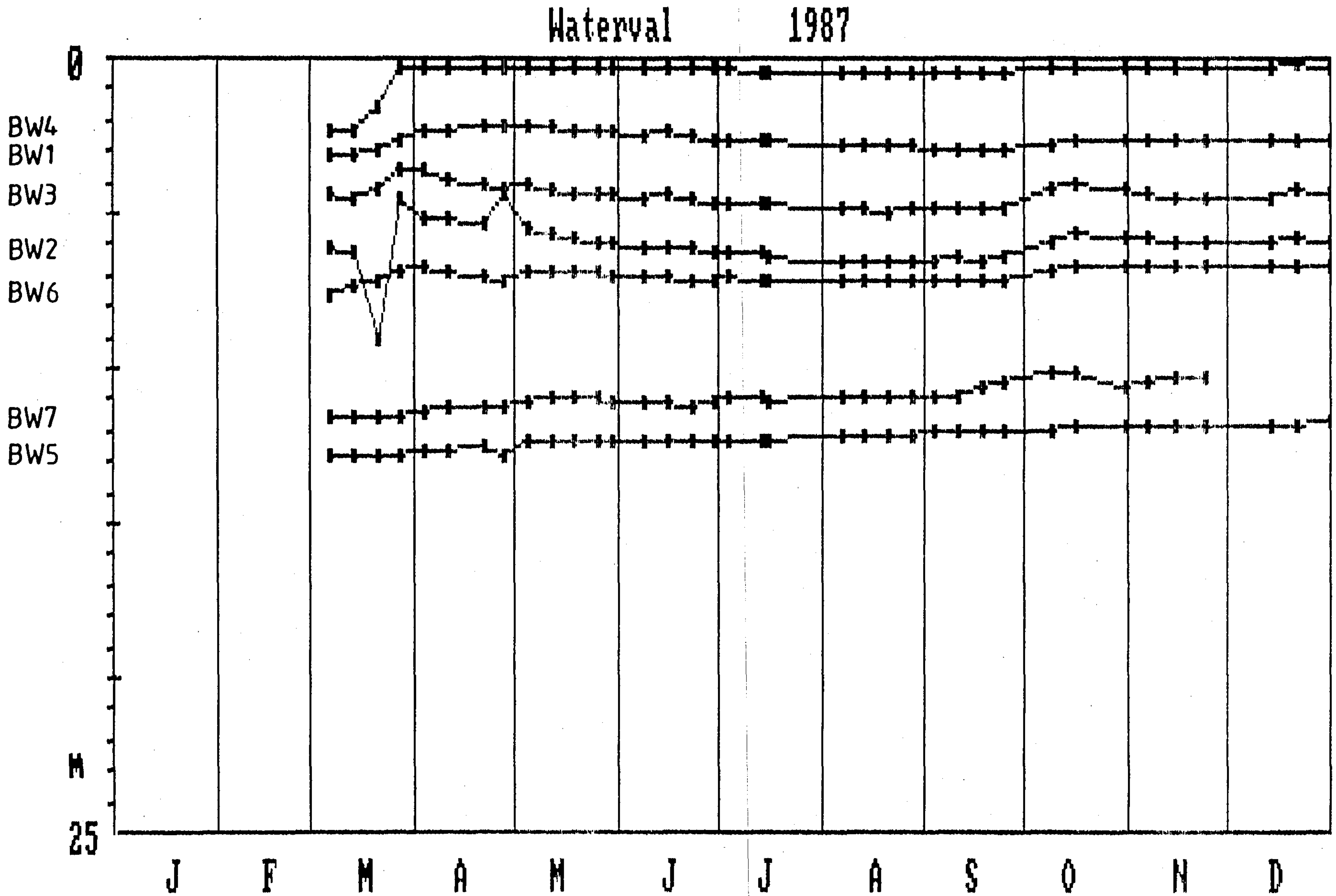


Fig. 4.26 Water Level Variations

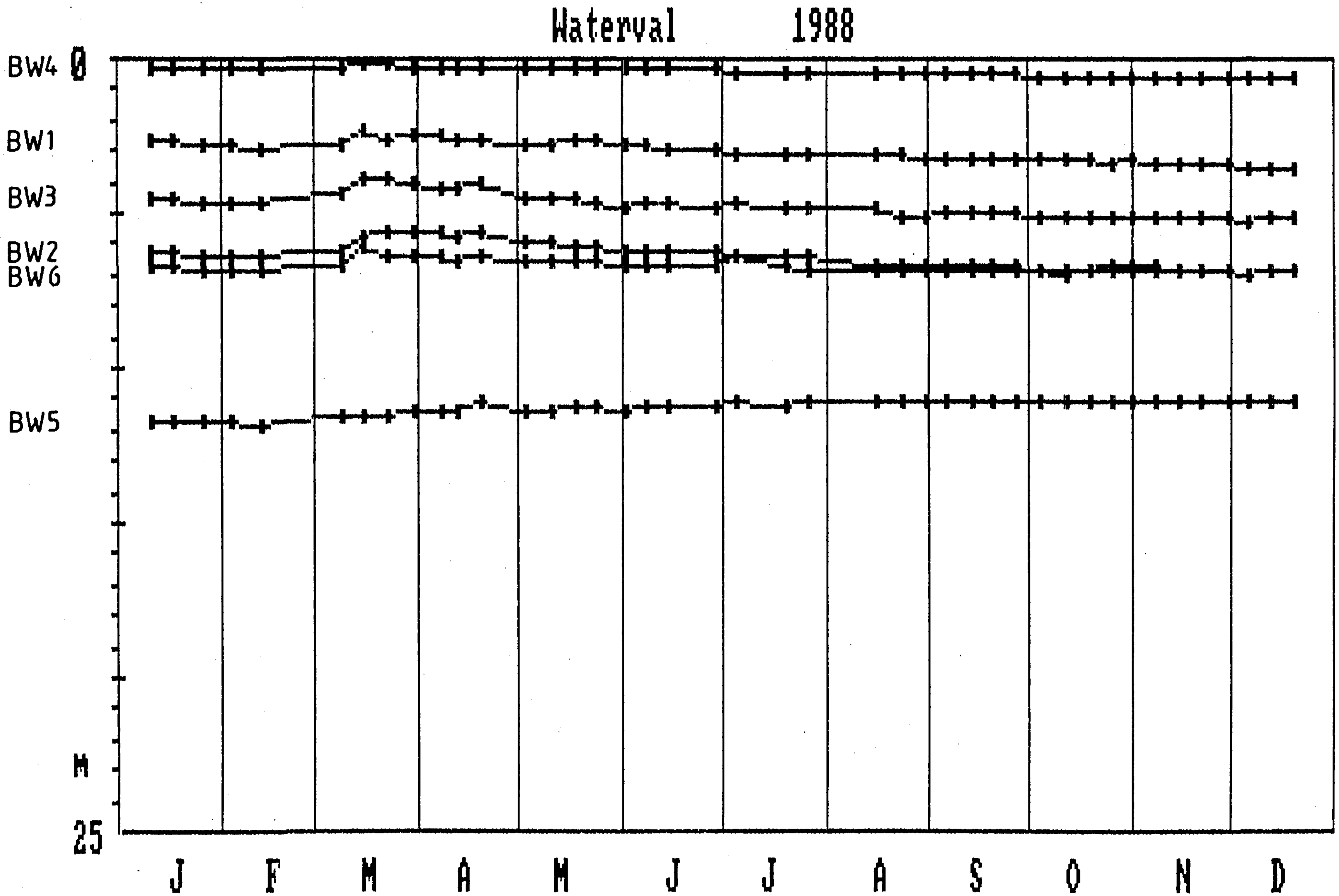


Fig. 4.27 Water Level Variations

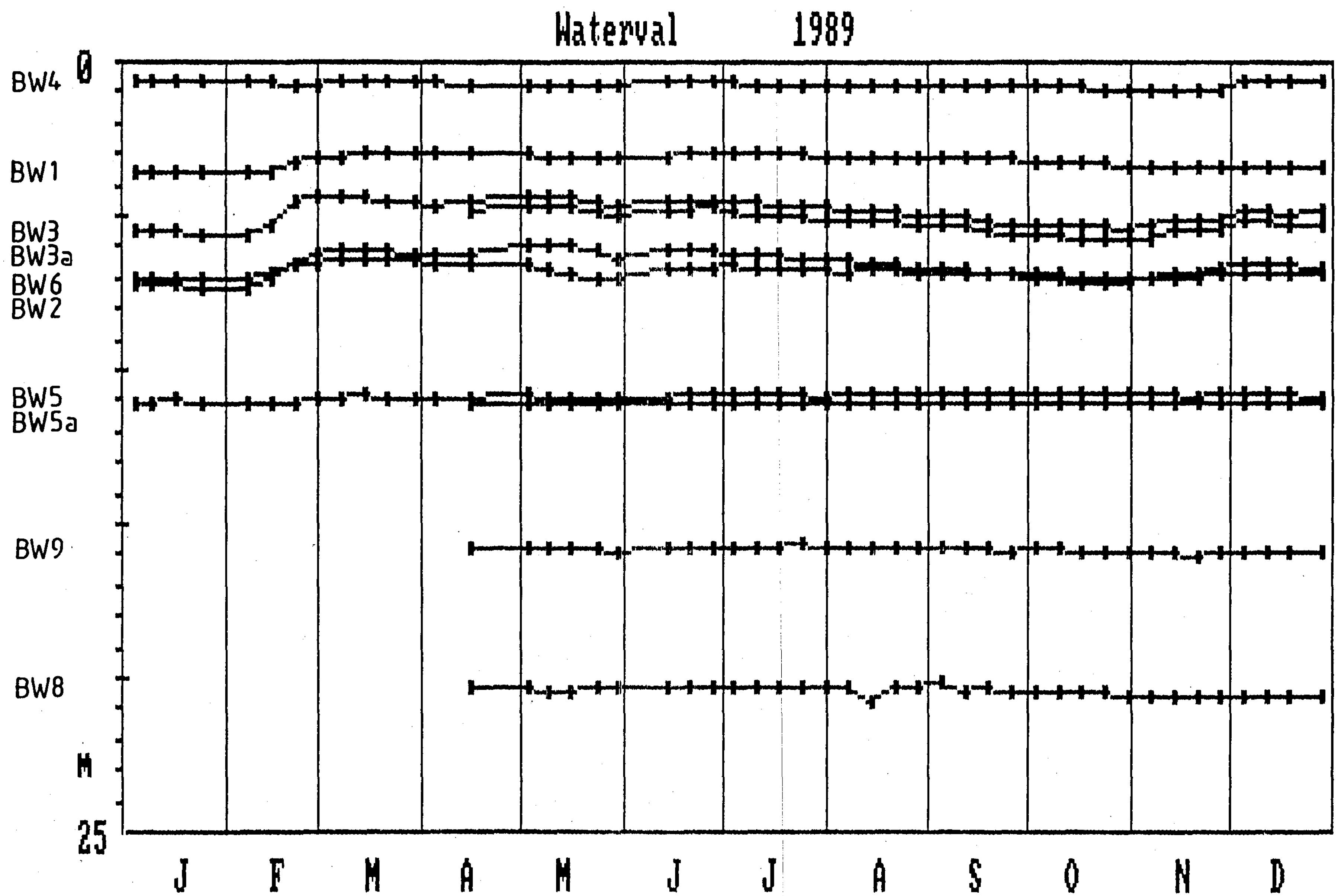


Fig. 4.28 Water Level Variations

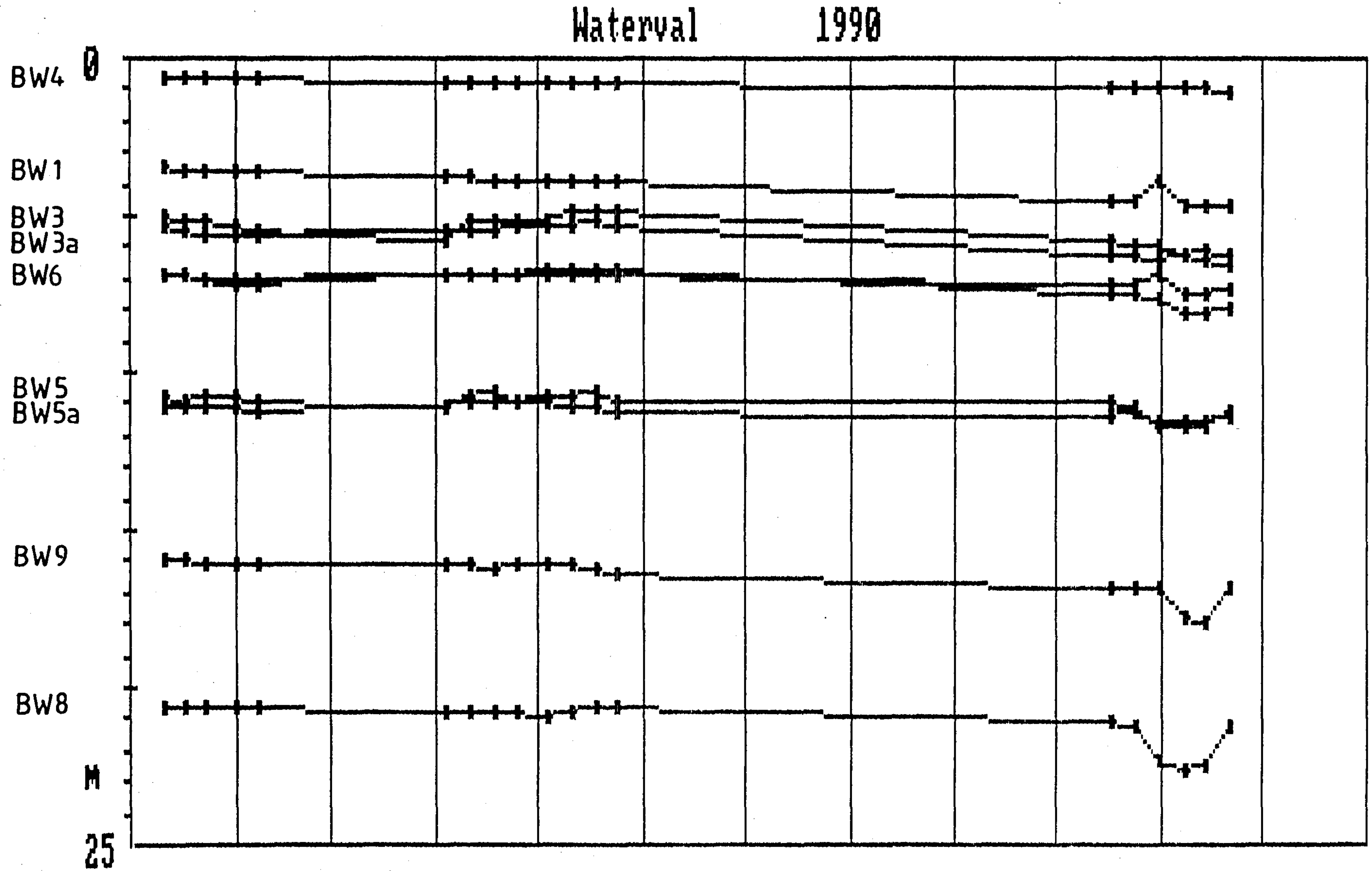


Fig. 4.29 Water Level Variations

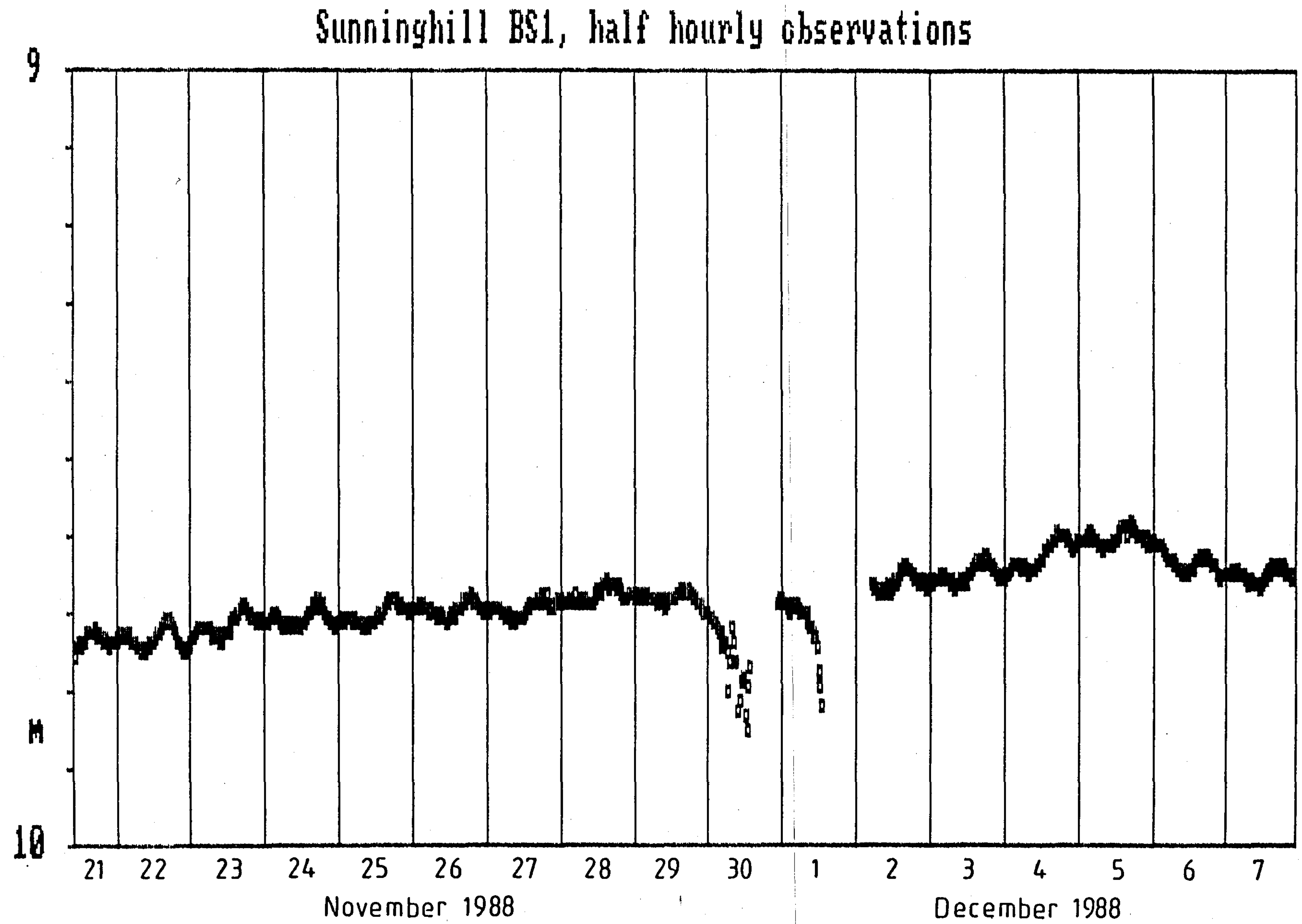


Fig. 4.30 Water Level Variations

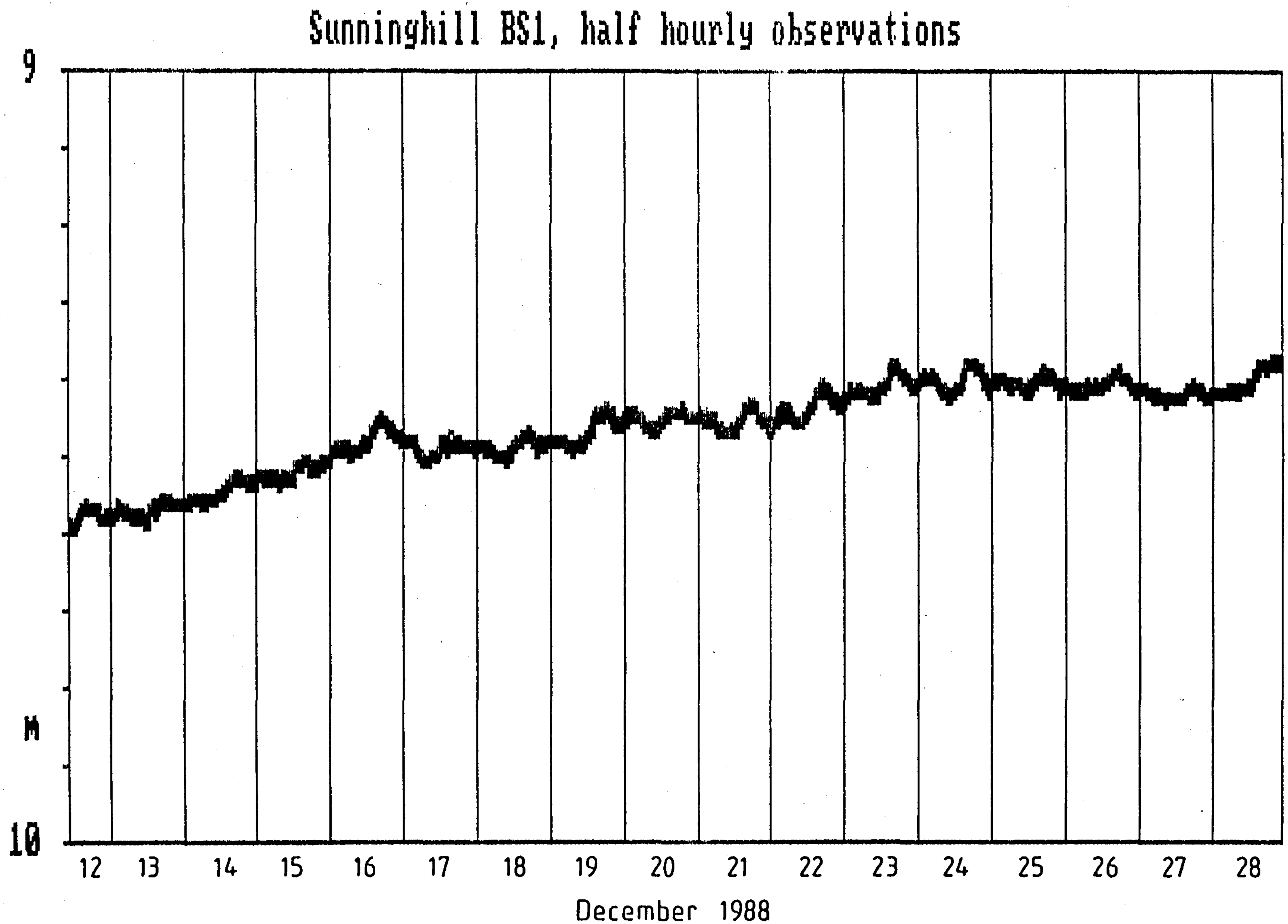


Fig. 4.31 Water Level Variations

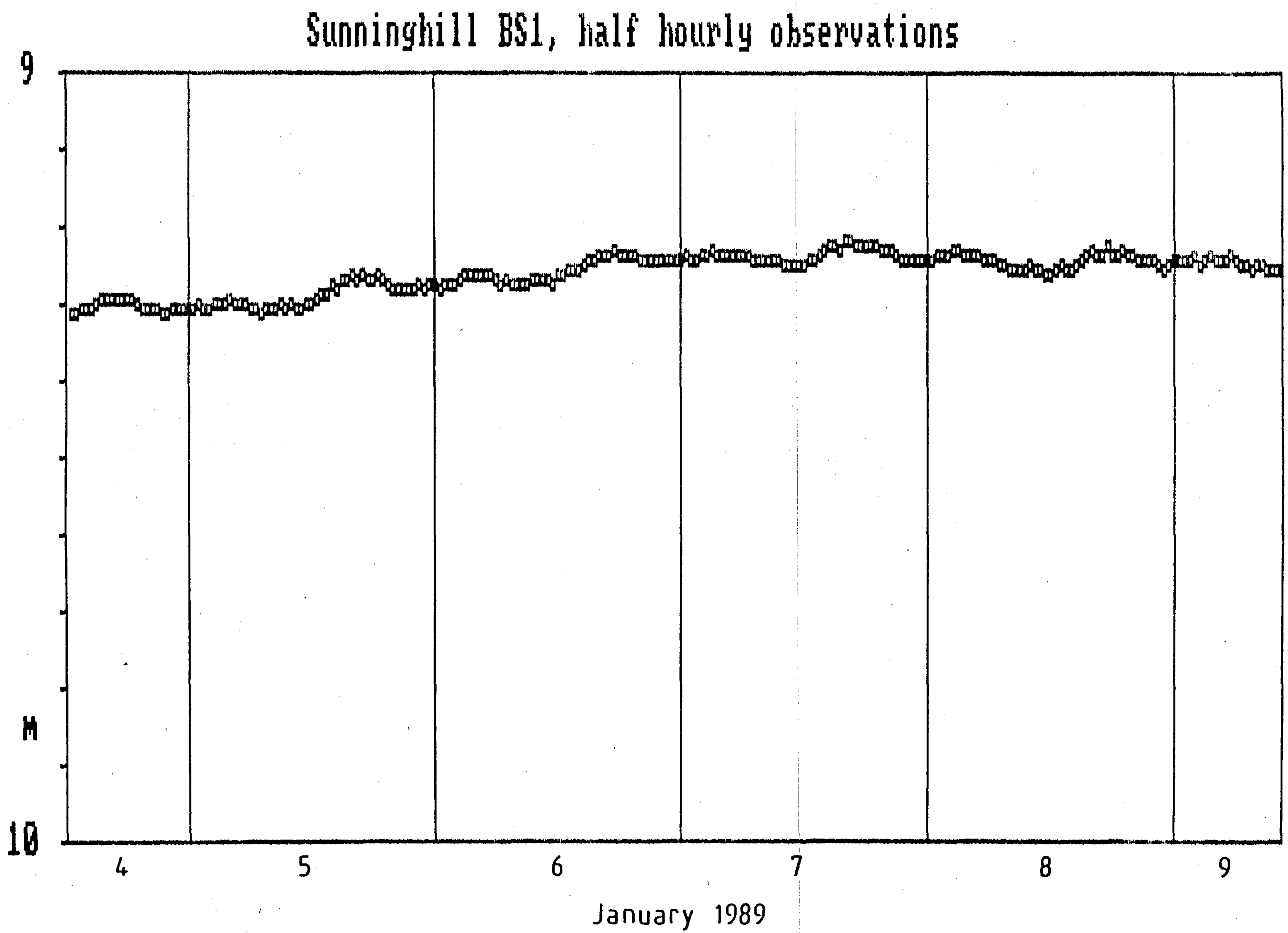


Fig. 4.32 Water Level Variations

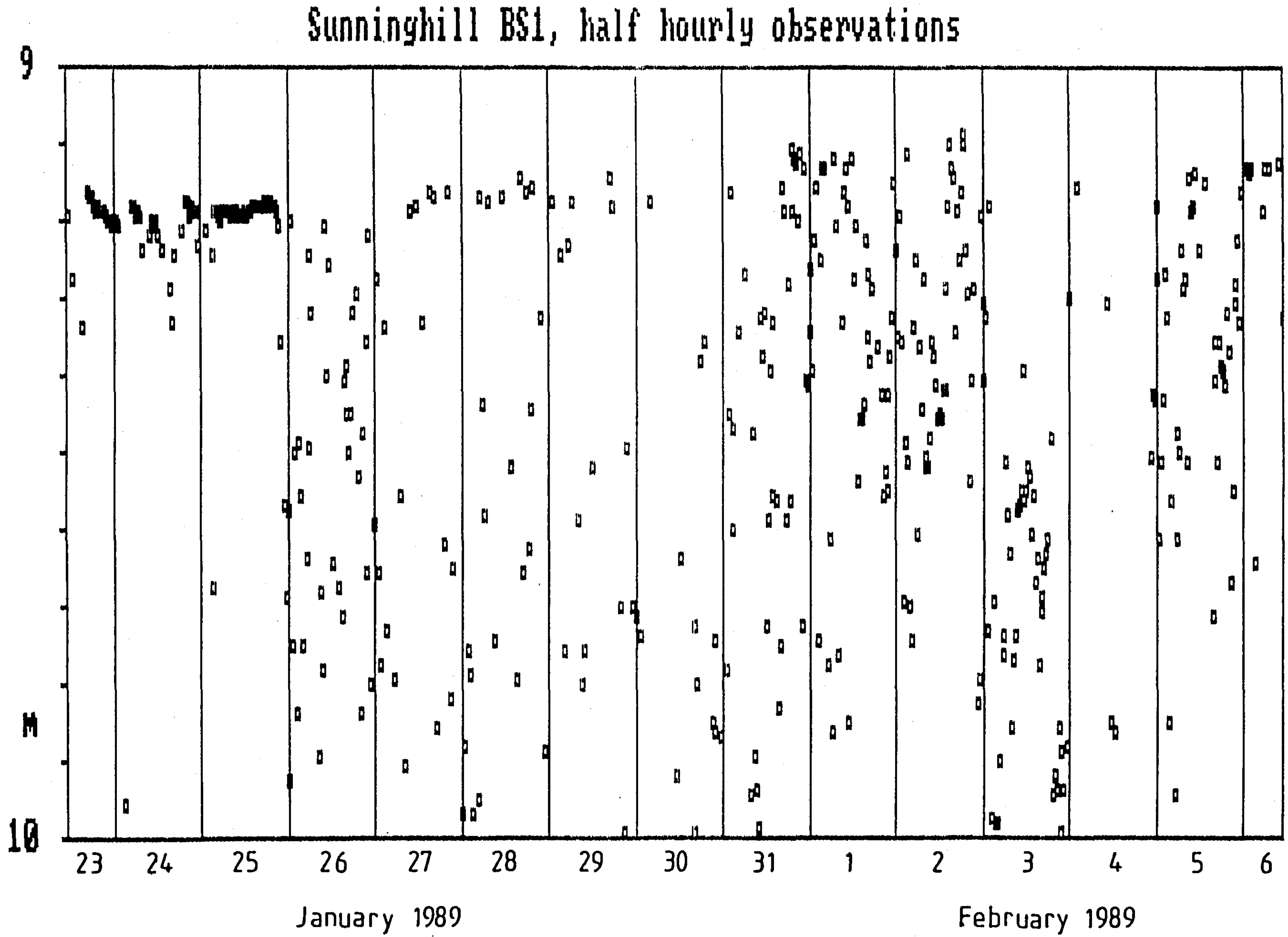


Fig. 4.33 Water Level Variations

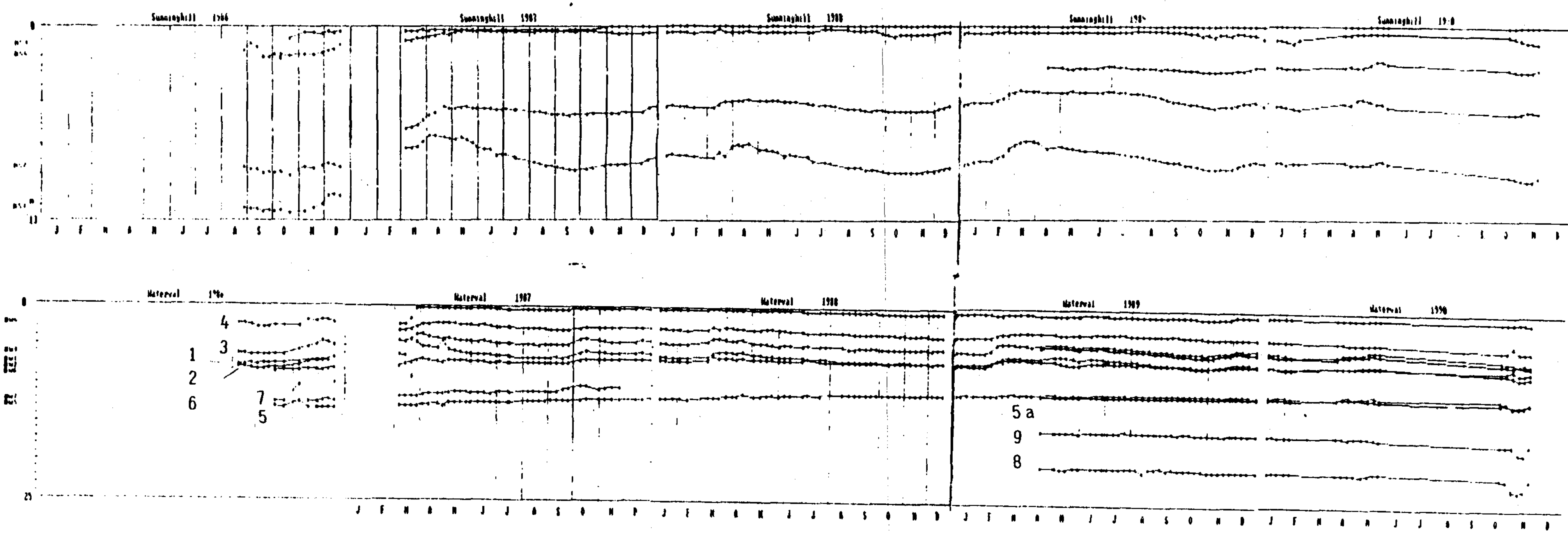


Fig. 4.34 Long term trends in water levels

A composite picture of the water level variations (Fig. 4.34) shows little variation in trend in groundwater level. The undeveloped catchment (boreholes BW 1-7) if anything show a slow falling in water table over the 5 years of observation. This will indicate a dry period in history. The most marked drop is in boreholes BW 9 and 8, where the water level fell 2m (-20 to -22m) in 2 years.

Sunninghill boreholes show a lesser rate of fall in water level. However, the water levels plotted are for boreholes in the water-course which are often artesian and therefore do not drop in level. Borehole BS1, after the initial recovery following drilling and pump testing shows the most fall (3m over 5 years) and this is the only borehole on a high point near the catchment watershed. The fall from 7m to 10m below surface could be serious in drying out upper soil layers, but insufficient data is available to substantiate the trend ie. the boreholes must be drilled at higher positions and dry and wet sequences need to be covered.

5 PUMPING TESTS

5.1 Introduction

A range of methods has been developed for the investigation of aquifer characteristics by means of boreholes. In this chapter we look at methods which are based on the temporary abstraction or injection of water.

Since analysis of the test data often involves the use of the Darcy equation the methods are strictly speaking only applicable to confined aquifers. However, it has been found that under certain circumstances unconfined aquifers may be analysed in a similar fashion.

The use of Darcy's equation further implies that we are dealing with homogeneous, isotropic aquifers. This condition is seldom met, as even unconsolidated, granular aquifers are often intersected by less permeable layers, such as clay lenses. Since it is virtually impossible to map all the irregularities of a fractured rock aquifer the same analytical methods are often used for the study of secondary aquifers.

Minor inaccuracies may result from the violation of the assumption of infinite boundaries. In an aquifer of limited permeability the area affected by water injection or abstraction is relatively small, thus making the requirement of infinite boundaries less stringent.

The assumption of laminar flow conditions is generally correct for an aquifer, except in the direct vicinity of a borehole while a pumping test is in progress. This may influence the imposed water level and may have to be taken into account when analysing a single hole pumping test.

The most common borehole tests in groundwater investigations are listed below. In addition a list of references is given.

- field pumping test - non-steady state (1, 4, 5, 6),
  - steady state (1, 3, 4, 5),
- recovery test (1, 6),
- maximum yield test (9),
- packer test (2, 3, 7, 8),
- constant head test (2, 3, 7, 10),
- rising or falling head test (2, 3),
- bailer or slug test (1).

1) Mandel & Shiftan (1981), 2) Clayton, Simons & Matthew (1982), 3) Weltman & Head (1983), 4) Heath & Trainer (1968, US units), 5) Todd (1959, US units), 6) Boswinkel (1983), 7) US Dept. of the Interior, Small Dams (1965, US units), 8) BS 5930 (1981), 9) SABS 045-1974 (1974), 10) Brink, Partidge & Williams (1982).

The aim of borehole tests is to achieve an understanding of the subsurface conditions, at least in the vicinity of the borehole. The main indicators in this respect are the transmissivity and storage coefficient for confined aquifers, and the permeability and specific yield (or effective porosity) for phreatic aquifers. Occasionally, additional information may be derived from irregularities in the plotted results (see e.g. Mandel and Shiftan (1981), Weltman and Head (1983), Houlden (1984)), provided of course that the irregularities are not due to faulty field procedures. Irregularities may *inter alia* result from the presence of leaky aquifers, unpredicted boundaries, and boreholes which are insufficiently deep to penetrate the aquifer fully. Rushton and Redshaw (1979) provide a numerical method of analysing pumping tests. They recommend it as complementary to the conventional analytical methods, and as particularly useful for explaining irregularities.

5.2 Pumping test analysis

When water is withdrawn from a borehole at a constant rate, the water level in the borehole will initially drop rapidly. Due to the radial flow towards the borehole the piezometric level in the vicinity of the hole will form a cone of depression (non-steady state, non-steady shape). After a certain amount of time the water level in the borehole will stabilize, while the cone of depression continues to widen (non-steady state, partly steady shape). In the end an equilibrium situation may be reached, whereby the recharge of the aquifer equals the rate of abstraction. The piezometric level surface at this stage remains constant (steady state, steady shape). See figure 5.1.

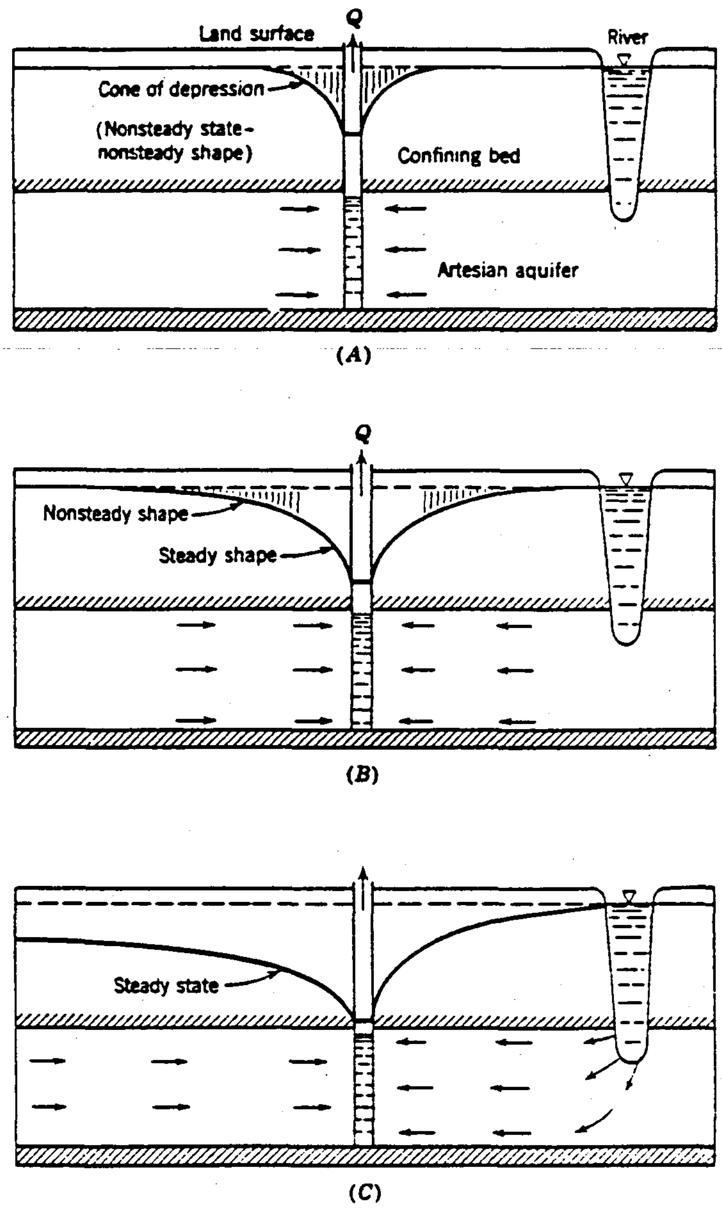


Figure 5.1 The transition from non-steady state to steady state conditions in the vicinity of a pumping well; (A) early stage, (B) intermediate stage, (C) final stage (after Heath and Trainer, 1968)

Taking into account the costs of pumping tests, it is only in exceptional cases that the actual steady state will be reached. In practice the intermediate stage is often considered to represent the steady state. This is acceptable when possible observation holes are situated in the steady shape area, in which case all borehole water levels remain constant.

Based on a combination of Darcy's equation and the law of continuity the general equation for two-dimensional radial flow to a pumped well is given by:

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

where s = drawdown in relation to pre-test water level (m), r = distance to pumping well (m), S = storage coefficient (-), and T = transmissivity (m^2/s).

For (quasi-) steady state conditions the right hand side reduces to zero. The remaining left hand side can then be converted into a first-order, ordinary differential equation. This leads to the formula of Dupuit for phreatic aquifers and the formula of Thiem for confined aquifers.

If the drawdown in a phreatic aquifer is less than 10% of its saturated thickness the behaviour during pumping is similar to that in a confined aquifer. Under those circumstances the Thiem method may therefore be employed to determine reliable values of permeability for the saturated layer.

Most often the duration of a pumping test ranges from several hours to a day, and a (quasi-) steady state is not reached. In those cases, and (quasi-) confined conditions the test results may be analysed using the Theis equation:

$$s=\frac{Q\,W(u)}{4\,\pi\,T}$$

where

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

and

$$W(u) = \int_u^\infty \frac{e^{-u}}{u} du$$

and s = drawdown (m), r = distance to pumping well (m), S = storage coefficient (-), T = transmissivity (m^2/s), t = time after start of test (s), and Q = constant abstraction rate (m^3/s).

The well function W(u) can be approximated by the infinite series:

$$W(u) = -0.577216 - \ln(u) + u - \frac{u^2}{2*2!} + \frac{u^3}{3*3!} - \frac{u^4}{4*4!} + \ldots$$

Alternatively, W(u) can be derived from tables for discrete values of u. See e.g. table 5.1.

Table 5.1 Values of W(u) for various values of u (After Freeze and Cherry, 1977).

u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
× 1	0.219	0.049	0.013	0.0038	0.0011	0.00036	0.00012	0.000038	0.000012
× 10^{-1}	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
× 10^{-2}	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
× 10^{-3}	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
× 10^{-4}	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
× 10^{-5}	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
× 10^{-6}	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
× 10^{-7}	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
× 10^{-8}	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
× 10^{-9}	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
× 10^{-10}	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
× 10^{-11}	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
× 10^{-12}	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.97	24.86
× 10^{-13}	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
× 10^{-14}	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
× 10^{-15}	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

SOURCE: Wenzel, 1942.

The transmissivity and the storage coefficient cannot be determined directly from the above equations because T occurs both in the argument of the function and as a divisor of the exponential integral. However, Theis devised a graphical method that employs the use of two graphs. One is a "type curve", which is a plot of W(u) versus u on double logarithmic paper. The second is a "data plot" displaying the drawdown measurements versus r^2/t also on double logarithmic paper. After superimposing the two graphs and visual inspection of the optimal agreement, values of u, W(u), s, and r^2/t are selected at any convenient point. The final result is obtained by substituting the values found:

$$T=\frac{Q\,W(u)}{4\,\pi\,s} \quad \text{and} \quad S=\frac{4\,u\,T}{r^2\,/\,t}$$

In case of a pumping test involving a step drawdown the Theis method may be applied to the individual sections. Due to the compression of the data on the r^2/t axis, separate graphs would be recommended for the second and further steps with an adjusted time scale. In the present report only the first step is analysed.

For small values of u the third and further terms on the right hand side of the infinite series approximation contribute little to the values of W(u). The omission of these terms facilitates the use of a more rapid solution method. The error that is introduced by this simplification is shown in table 5.2.

Table 5.2 Error introduced by simplification of the infinite series approximation of W(u) for various values of u.

u	RHS	reduced RHS	error	error (%)
.01	4.04	4.03	.00998	.25
.02	3.35	3.33	.01990	.6
.05	2.47	2.42	.04938	2.0
.10	1.82	1.73	.09755	5.4

Although an error of 5% might be acceptable in some geohydrological investigations, it would in general be preferable to use the Theis method whenever u is larger than 0.01.

For those cases where u is smaller than 0.01, Jacob (1946) introduced an efficient method to determine the aquifer characteristics. The equation

$$s=\frac{Q\,W(u)}{4\,\pi\,T}=\frac{Q}{4\,\pi\,T}\;(-.577216-\ln(u))$$

can be rewritten as

$$s=\frac{Q}{4\,\pi\,T}\left(2.30*{}^{10}\log\left(2.25*\frac{T\,t}{r^2\,S}\right)\right)$$

where s = drawdown (m), r = distance to pumping well (m), S = storage coefficient (-), T = transmissivity (m^2/s), t = time after start of test (s), and Q = constant abstraction rate (m^3/s).

Two important observations can be made:
1) If the argument of the logarithmic function is equal to one, the drawdown equals zero,
2) For values of time that are a factor ten apart the drawdown increases by a value equal to the coefficient preceding the logarithmic function or $\Delta s = 2.3 * Q / (4\,\pi\,T)$.

The most convenient way of solving the equation is graphically, this time using semi-logarithmic paper. Values of drawdown are plotted on the arithmetic scale and time on the logarithmic scale. The resulting graph (which should be a straight line) is referred to as the "time-drawdown graph". The drawdown over one log-cycle provides the transmissivity according to:

$$T=\frac{2.3*Q}{4\,\pi\,\Delta s}$$

The intersection of the time-drawdown graph with the zero drawdown line yields a value t_0, and the storage coefficient can then be determined with:

$$S=\frac{2.25\;T\;t_0}{r^2}$$

If several observation boreholes are available T and S can also be established with the Jacob method using the "distance-drawdown graphs" (See Heath and Trainer, 1968).

After the evaluation of T and S a check should be made to determine whether $u = (r^2 S) / (4 T t)$ is indeed less than 0.01.

5.3 Recovery test

The previously described evaluation methods of a drawdown test are sometimes also used when only a single pumping well is available. The distance to the pumping well is now represented by the effective well radius (r_w). The effective well radius refers to a region of turbulent flow, where Darcy's equation is not valid. The extent of this region is generally unknown. Therefore, the Theis method cannot be used since r^2/t cannot be evaluated. If the Jacob method is used, a value for the transmissivity may be found, but since the storage coefficient remains unknown, it can not be verified whether this method is applicable, i.e. $u < 0.01$. Occasionally these problems are circumvented by assuming that the effective well radius may be substituted by the actual well radius (r_0).

A more appropriate method for the analysis of a single well pumping test is provided by an analysis of the water level recovery in the borehole. To apply this method water level measurements should be taken after pumping has been discontinued. Stoppage of pumping is represented mathematically by the assumption that the well continues to be pumped at a constant rate and that, from the time of stoppage onward, injection of water is carried out into the same well at an equal rate, so that the imagined injection cancels the imagined continued pumping.

Introducing t' as the time passed since the pump has been stopped, the residual drawdown s' can be expressed as
$s' = s(t) - s(t')$. See figure 5.2.

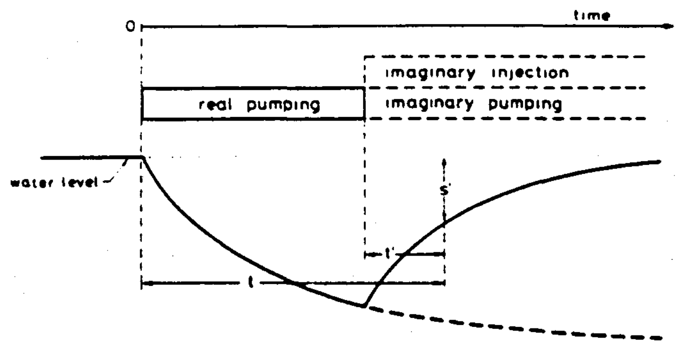


Figure 5.2 Single well recovery test (After Mandel and Shiftan, 1981)

According to Theis the above equation can be written as:

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

where u' = $(r^2\ S)\ /\ (4\ T\ t')$, assuming that the transmissivity and storage coefficient do not change. If W(u) and W(u') are substituted by their respective infinite series approximation, the residual drawdown yields:

$$s' = \frac{Q}{4\ \pi\ T}\left(-\ln(u) + \ln(u') + (u-u') - \frac{(u^2-u'^2)}{4} + \frac{(u^3-u'^3)}{18} - \ldots\right)$$

Since the terms beyond the logarithmic expressions rapidly approximate zero, irrespective of the absolute values of u, the residual drawdown can be written as:

$$s' = \frac{Q}{4\ \pi\ T}\ln\frac{u'}{u} = \frac{Q}{4\ \pi\ T}\ln\frac{t}{t'} = \frac{2.30\ Q}{4\ \pi\ T} * {}^{10}\log\frac{t}{t'}$$

Consequently, the decrease of s' equals 2.30 * Q / (4 π T) for values of t/t' that are a factor ten apart. The results of this test are again most readily obtained by a using a graphical approach. Using semi-logarithmic paper the residual drawdown is plotted on the arithmetic scale and values of t/t', or alternatively t'/t, on the logarithmic scale. Finally, the transmissivity is again evaluated by:

$$T = \frac{2.3\ Q}{4\ \pi\ \Delta s}$$

As in any single well test no information can be obtained about the storage coefficient, unless assumptions would be made about the effective well radius.

5.4 Step drawdown test

The drawdown in a pumping well reflects turbulent flow in the well and its immediate vicinity and laminar flow in the aquifer (see figure 5.3).

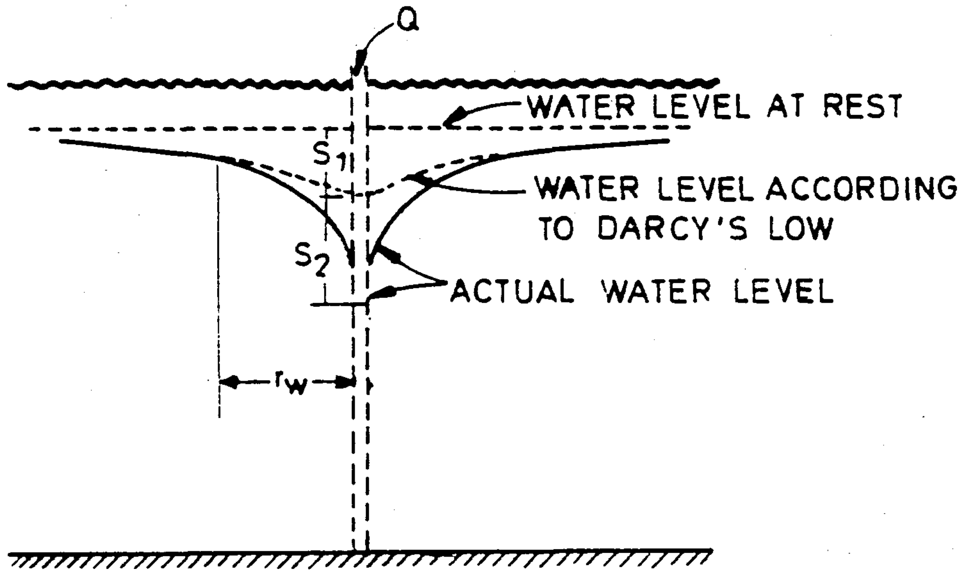


Figure 5.3 Drawdown in and near a pumping well (After Mandel and Shiftan, 1981).

The turbulent component of the drawdown can be estimated by conducting a step drawdown test. In this type of test a well is pumped at different discharge rates, for equal time intervals of approximately one hour, at each rate. If it is assumed that the turbulent drawdown is proportional to the discharge squared the total drawdown can be written as:

$$s_{tot} = s_{lam} + s_{turb} = \left(\frac{2.30}{4\ \pi\ T} * {}^{10}\log \frac{2.25\ T\ t}{S\ r_w^2}\right) * Q + C * Q^2$$

where r_w = the effective well radius (m), and C a constant (s^2/m^5). It can be shown (Mandel and Shiftan, 1981) that for equal time intervals the coefficient of the first term on the right hand side is constant, or $s_{tot} = D*Q + C*Q^2$. The constants D and C are found using again a graphical method. Division by Q reduces the right hand side into a linear relationship: $s_{tot} / Q = D + C*Q$. Hence the values of the total drawdown at the end of each time step divided by the discharge rate during that time interval are plotted on the vertical axis, while the corresponding discharge rate is plotted on the horizontal axis. The line approximating the observed points provides the constants D and C, from which the laminar and turbulent component of the drawdown can be deduced.

5.5 Maximum yield test

The maximum yield test, as described in SABS-045 (1974), and loosely followed by some drilling contractors, is often unsatisfactory. The suggested procedure consists of an initial high discharge rate until the water level in the borehole has dropped to just above the borehole pump. Thereafter the water level is kept constant by gradual reduction of the discharge rate. The test is continued until the change in discharge rate is less than 5% over a time interval of one hour. The "safe yield" is arbitrarily set at 60% of the latest observed discharge rate. A minimum test duration of 6 hours is recommended for private boreholes and of 72 hours for those used by public bodies for domestic supply.

It is obvious that this test only reflects on the steady shape condition, as the steady-state is seldom reached in such a short period of time.

5.6 Other tests

Similar and additional information about an aquifer can be obtained by a variety of other tests.

In a water injection test (packer or Lugeon test) sections of a borehole can be studied in detail. They are very useful for an understanding of the subsurface conditions and should be considered in future work in the Waterval / Sunninghill project.

A constant head test is unsuitable for the present project as the permeability of the aquifers is too high. From a preliminary test of this type it was concluded that several tankers would be required to provide the necessary water in order to maintain a constant head.

Rising and falling head tests may be applied when only a small section of a borehole is left uncased. In the present project there seem to be several water conducting layers. This would make the use of several, separate borehole set-ups necessary. Partly due to the costs this would involve and partly due to the lack of knowledge about the location of the underground conduits these type of tests were not considered.

Bailer and slug tests may be considered in future studies of the Waterval / Sunninghill project. They are simple, cheap and quick. However, the obtained information only gives a rough indication of the aquifer characteristics and additional tests have to be conducted.

5.7 Pumping tests in the Waterval catchment

In total four pumping tests have been conducted in the Waterval catchment. Two tests took place as single well tests in September 1986 in the holes BW3 and BW5. BW3 is situated at the bottom of the catchment, while BW5 is located halfway up the hill. The second two tests were performed in April 1989 after additional holes were drilled. Two of the four new holes were sunk near BW3 and BW5. All four tests will be discussed in detail.

1) BW3

duration of pumping test: 180 minutes.

observation frequency:	000-010 minutes:	every	½ minute,
	10- 60 "	"	minute,
	60-120 "	"	5 minutes,
	120-180 "	"	10 minutes,
	180-190 "	"	½ minute,
	190-197 "	"	minute.

pumping rate: 951 - 1244 Imp. gal/hr = 1.20 - 1.57 l/s.
most of the time 1.36 l/s, with a drop to 1.2 l/s after 7 minutes, and an increase to 1.57 l/s between 45 and 60 minutes and to 1.46 l/s between 140 and 160 minutes.

The resulting water levels (see figure 5.4) show a rather irregular pattern due to the varying pumping rates. The sudden kink during the recovery is probably due to the removal of the pump from the hole. During future groundwater tests drilling contractors should be discouraged to remove the pump while significant recovery is still taking place.

In figure 5.5 time has been plotted on a logarithmic scale. Most observations now lie between 0.01 and 0.1 days (= 14.4 and 144 minutes). If it were assumed that Jacob's method may be applied one would find that the transmissivity equals:

$$T=\frac{2.3\ Q}{4\ \pi\ \Delta s}=\frac{2.3*1.45*10^{-3}}{4*\pi*11.8}=2.3*10^{-5}\quad (m^2/s)$$

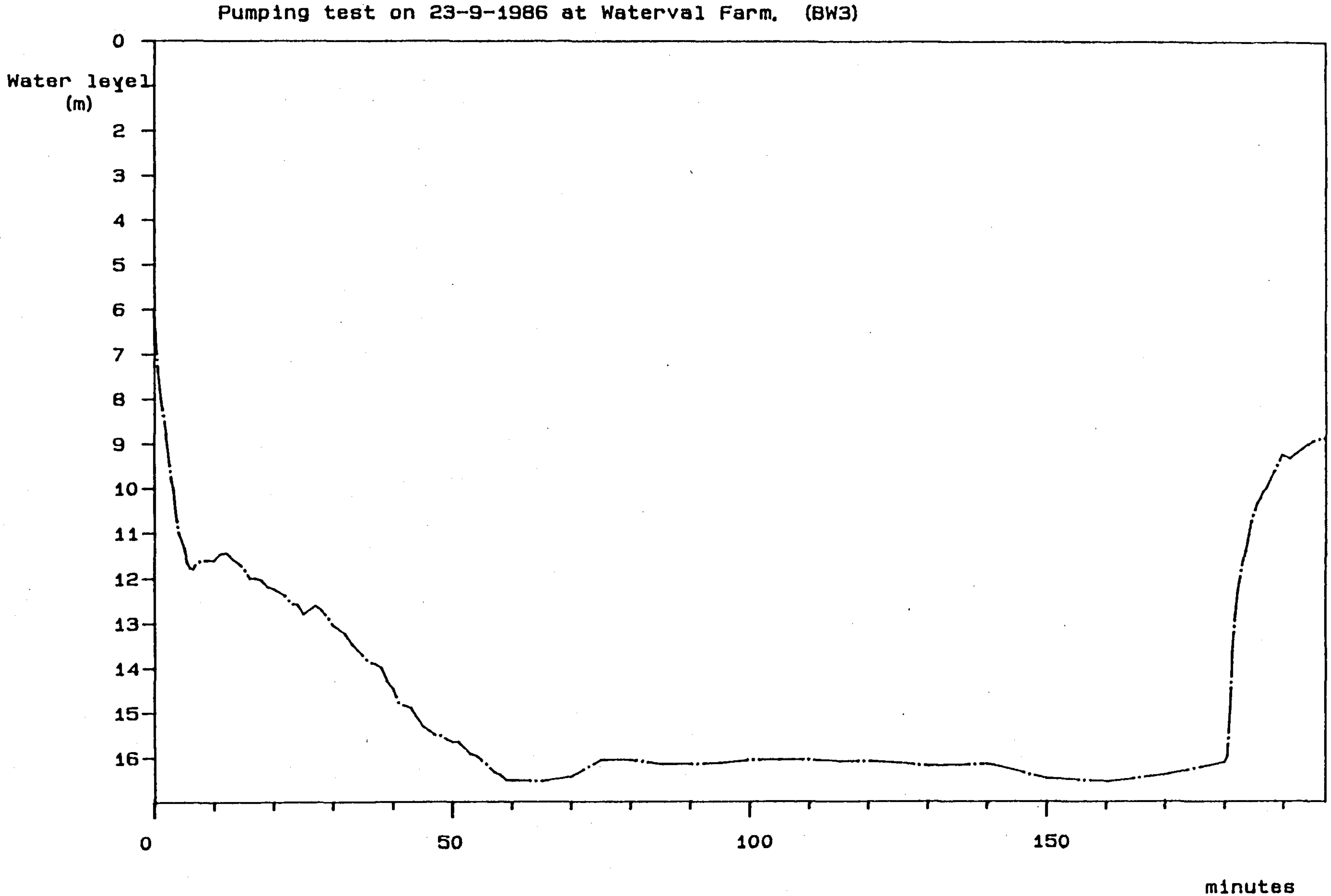


Figure 5.4 Water level observations during pumping test of BW3

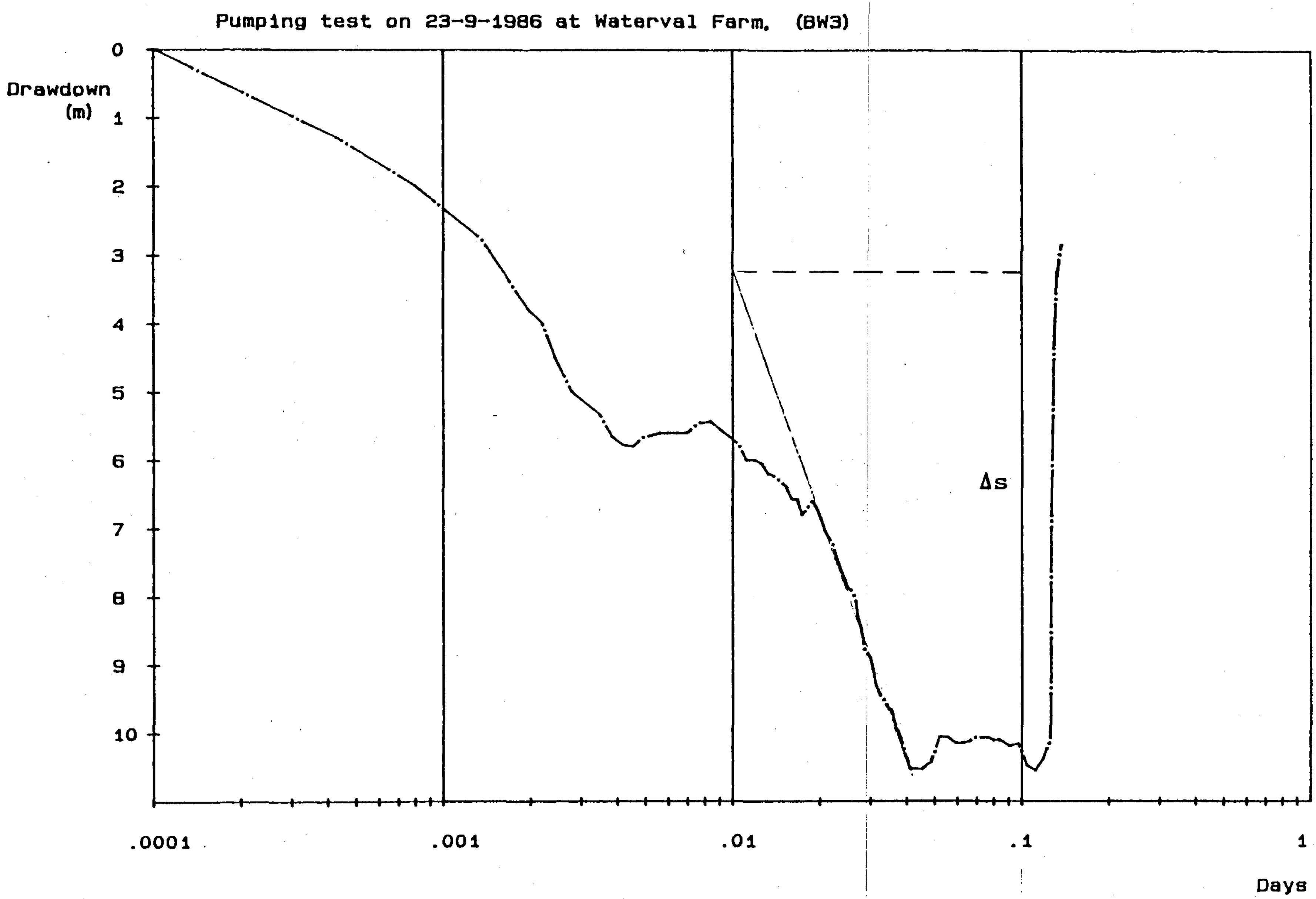


Figure 5.5 Pumping test analysis

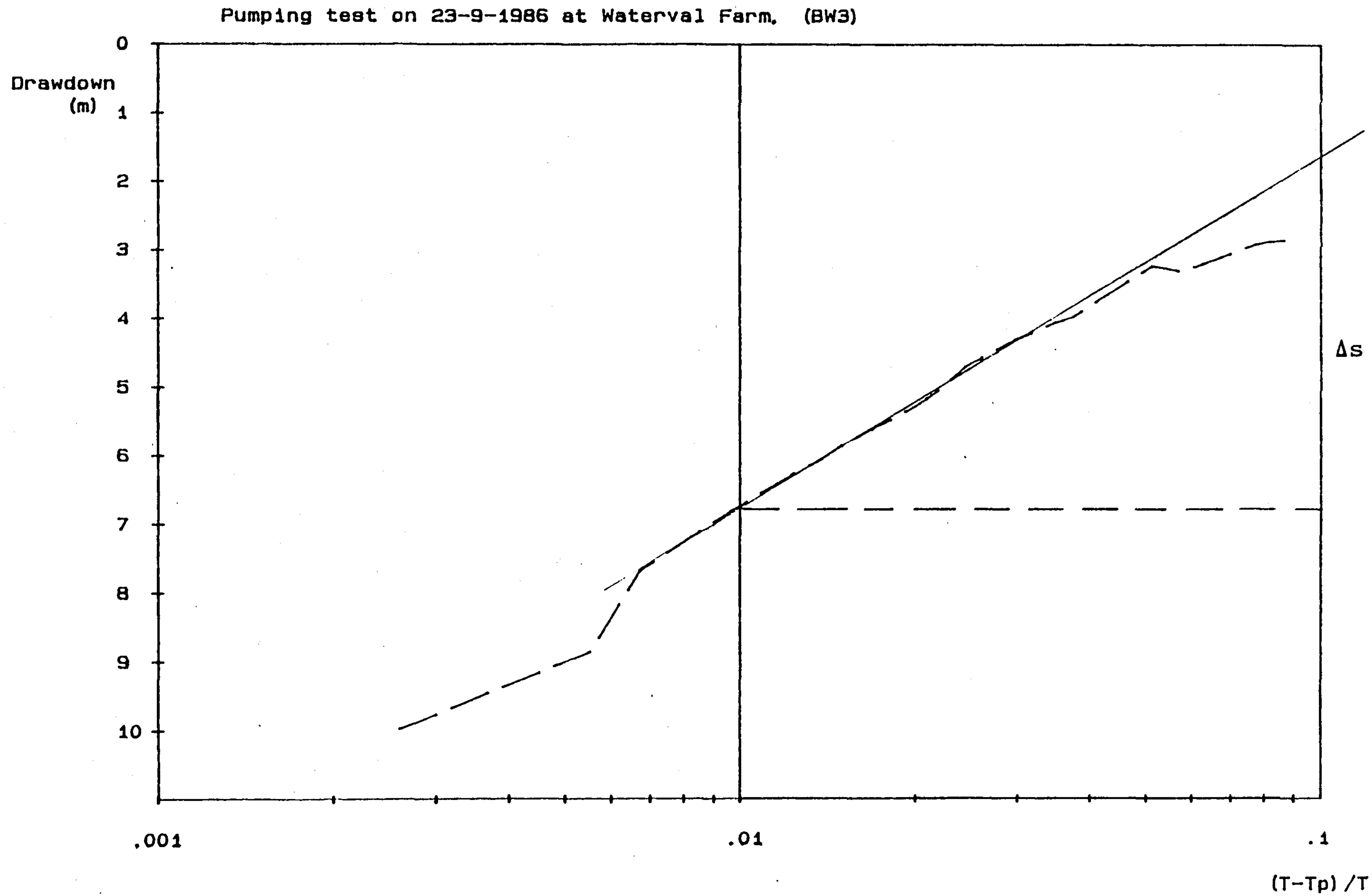


Figure 5.6 Recovery test analysis

More reliable information is provided by the recovery test (see figure 5.6). If irregularities at the beginning and end of the graph are neglected, and assuming an average discharge rate during pumping of Q = 1.45 m^3/s the transmissivity equation yields:

$$T=\frac{2.3\ Q}{4\ \pi\ \Delta s}=\frac{2.3*1.45*10^{-3}}{4*\pi*5.2}=5.1*10^{-5}\quad (m^2/s)$$

2) BW3 and BW3A

The second pumping test at BW3 was conducted after an observation hole was drilled at a distance of 5.60 m away from BW3 and topographically downstream.

duration of pumping test: 110 minutes.
observation frequency: 0-110 minutes: every minute,
110-120 " " ½ minute,
120-130 " " minute,
130-180 " " 5 minutes,
430 " final observation.
pumping rate: 0- 25 minutes: 0.96 - 1.00 l/s,
25- 42 " : 1.19 - 1.25 l/s,
42- 60 " : 1.32 l/s,
60- 80 " : 1.56 - 1.67 l/s,
80-110 " : 1.79 - 1.92 l/s.

As can be seen from the pumping rate and figure 5.7, the pumping test was conducted with a step drawdown. Unfortunately the time intervals were unequal and fairly short in duration. Using the Jacob method (see figure 5.8) for each of the five steps the following results are obtained:

1) Δs = 1.27 m, t_0 = $1.1*10^{-3}$ days = 95 seconds, Q = 1.0 l/s

$$T=\frac{2.3\ Q}{4\ \pi\ \Delta s}=\frac{2.3*1.0*10^{-3}}{4*\pi*1.27}=14.4*10^{-5}\quad (m^2/s)$$

$$S=\frac{2.25\ T\ t_0}{r^2}=\frac{2.25*14.4*10^{-5}*95}{5.6^2}=9.8*10^{-4}$$

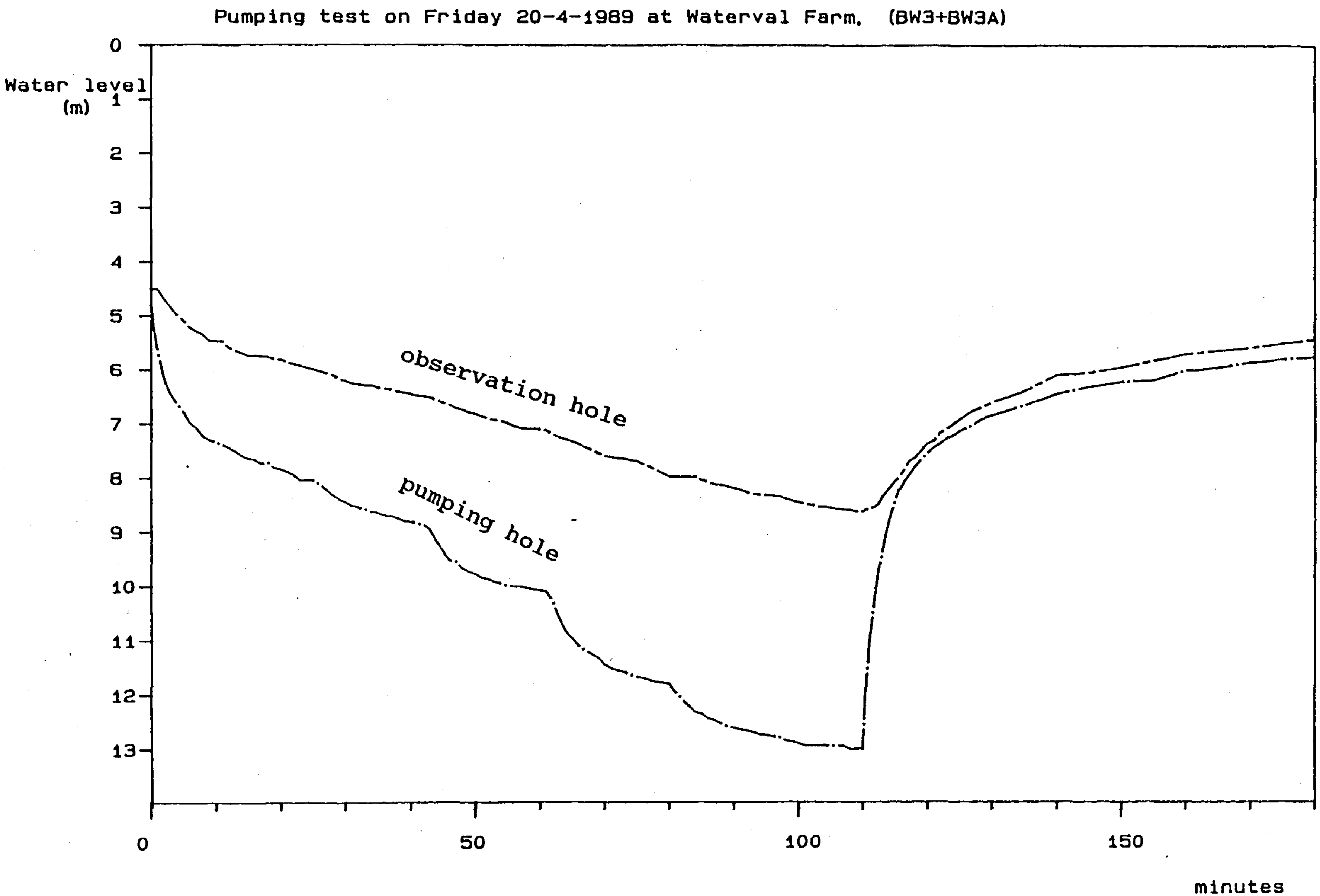


Figure 5.7 Water level observations

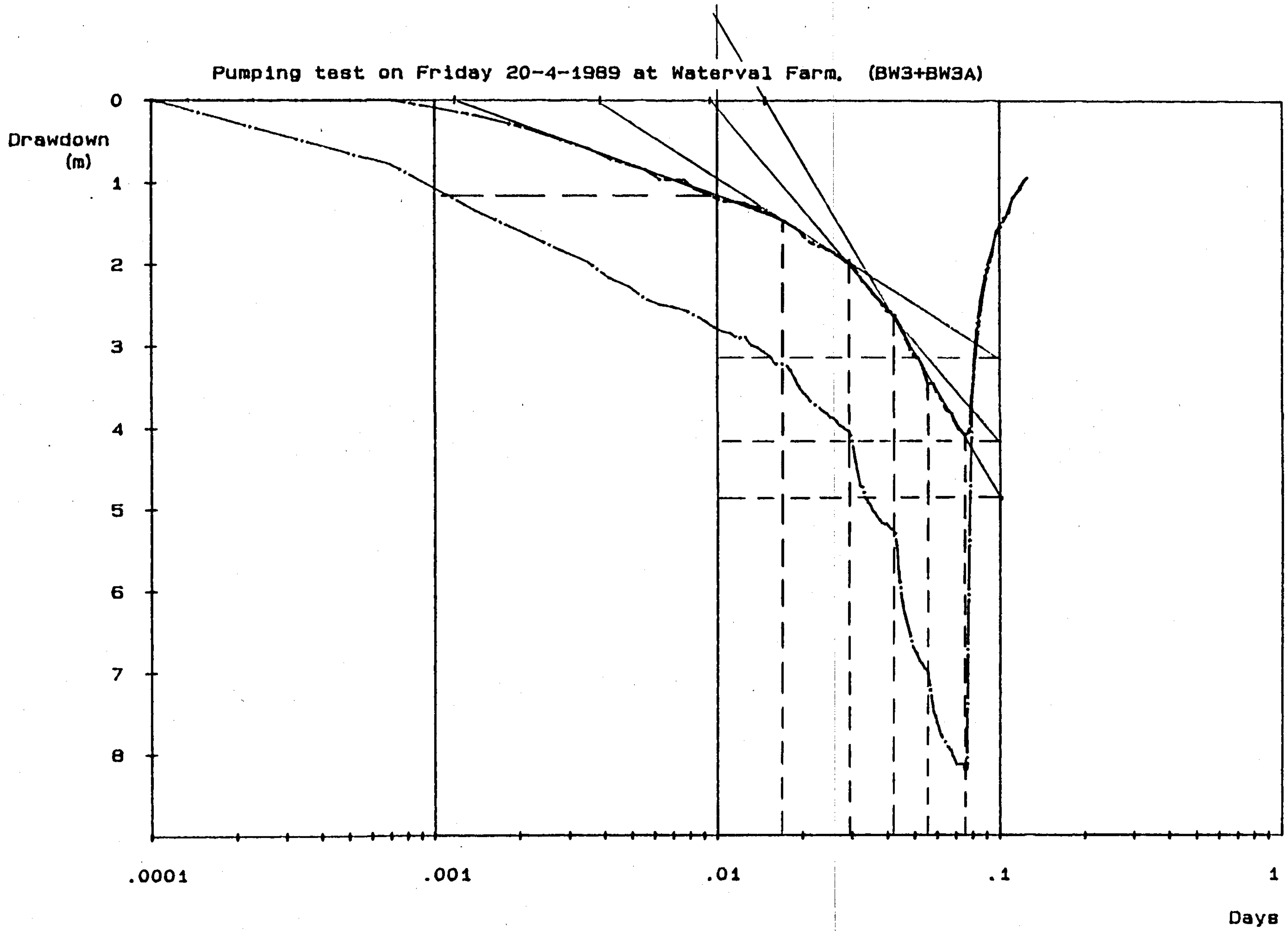


Figure 5.8 Pumping test analysis with Jacob method

$$u = \frac{r^2 S}{4 T t} = \frac{(5.6)^2 * 9.8 * 10^{-4}}{4 * 14.4 * 10^{-5} * 95} = 0.56$$

2) Δs = 2.20 m, $t_0 = 3.8*10^{-3}$ days = 328 seconds, Q = 1.2 l/s
$T = (2.3*1.2*10^{-3})/(4*\pi*2.20) = 10.0*10^{-5}\ m^2/s$
$S = (2.25*10.0*10^{-5}*328)/(5.6)^2 = 2.4*10^{-3}$
$u = ((5.6)^2*2.4*10^{-3})/(4*10.0*10^{-5}*328) = 0.60$

3) Δs = 4.00 m, $t_0 = 9.1*10^{-3}$ days = 786 seconds, Q = 1.3 l/s
$T = (2.3*1.3*10^{-3})/(4*\pi*4.00) = 6.0*10^{-5}\ m^2/s$
$S = (2.25*6.0*10^{-5}*786)/(5.6)^2 = 3.4*10^{-3}$
$u = ((5.6)^2*3.4*10^{-3})/(4*6.0*10^{-5}*786) = 0.57$

4) Δs = 5.72 m, $t_0 = 1.3*10^{-2}$ days = 1123 seconds, Q = 1.6 l/s
$T = (2.3*1.6*10^{-3})/(4*\pi*5.72) = 5.1*10^{-5}\ m^2/s$
$S = (2.25*5.1*10^{-5}*1123)/(5.6)^2 = 4.1*10^{-3}$
$u = ((5.6)^2*4.1*10^{-3})/(4*5.1*10^{-5}*1123) = 0.56$

5) Δs = 5.72 m, $t_0 = 1.3*10^{-2}$ days = 1123 seconds, Q = 1.85 l/s
$T = (2.3*1.85*10^{-3})/(4*\pi*5.72) = 5.9*10^{-5}\ m^2/s$
$S = (2.25*5.9*10^{-5}*1123)/(5.6)^2 = 4.8*10^{-3}$
$u = ((5.6)^2*4.8*10^{-3})/(4*5.9*10^{-5}*1123) = 0.57$

The values for u indicate that the use of the Jacob method is in fact inappropriate. Therefore, the observations are analysed using the Theis method. Since the discharge rate increased several times, sections of the graph have to be selected which correspond with constant rates. The longest section is found for the period from the start of the test to t = 25 minutes, or r^2/t = 0.02 to r^2/t = 0.26 (m^2/s). Optimal agreement between the type curve (W(u) versus u) and the data plot (s versus r^2/t) for this section was attained (see figure 5.9). A convenient point is selected, e.g. u = 0.1 and W(u) = 1, which reads on the axes of the data plot: s = 0.75 and r^2/t = 0.03. The transmissivity and storage coefficient are evaluated as follows:

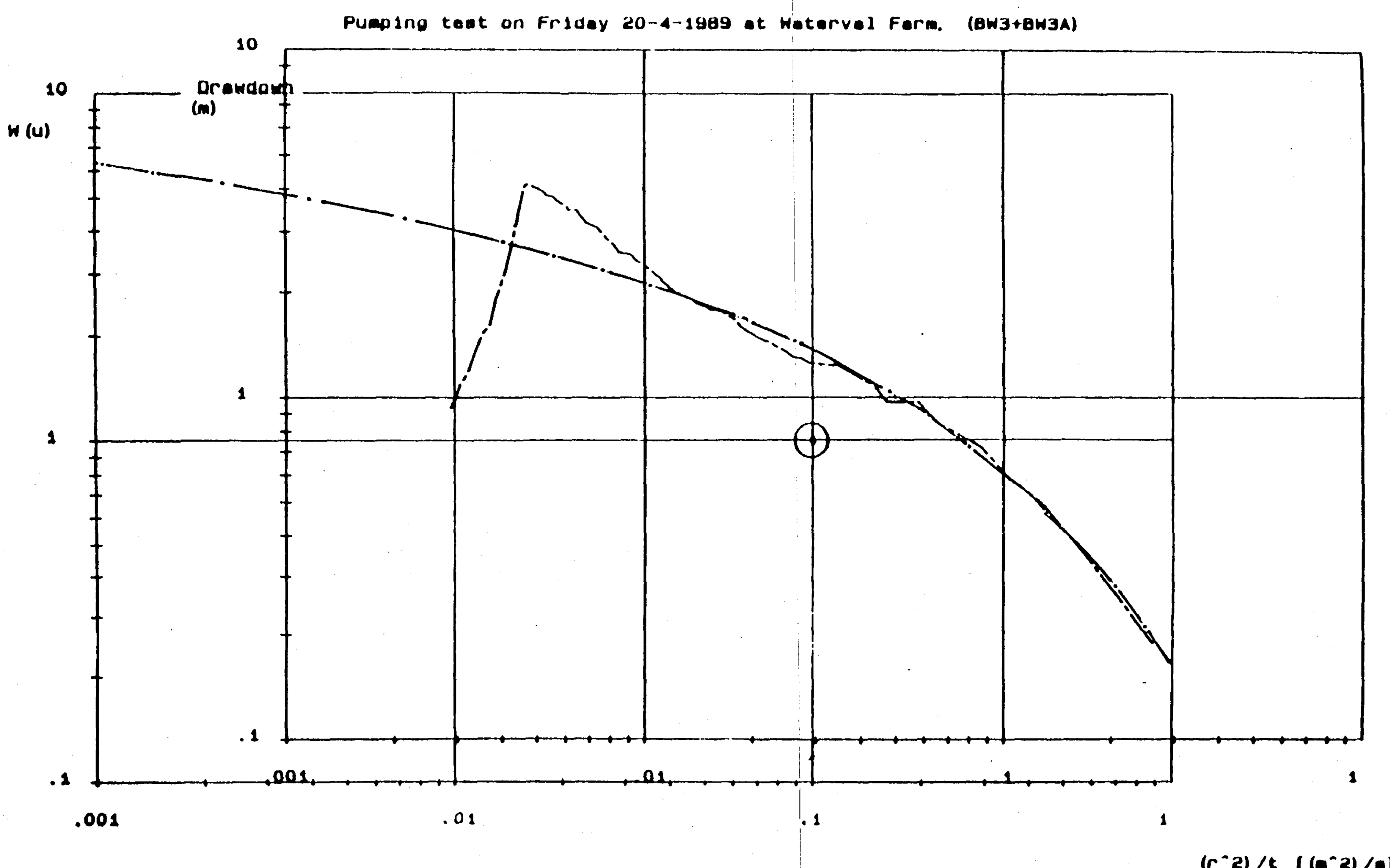


Figure 5.9 Pumping test analysis with Theis method

$$T=\frac{Q\ W(u)}{4\ \pi\ s}=\frac{1*10^{-3}*1}{4*\pi*0.75}=10.6*10^{-5}\quad (m^2/s)$$

and

$$S=\frac{4\ u\ T}{r^2/t}=\frac{4*0.1*10.6*10^{-5}}{0.03}=1.4*10^{-3}$$

Analysis of the water level recovery observations results in an observed Δs of 2.77 m per log-cycle for the linear part of the recovery (see figure 5.10). If the average pumping rate is set at 1.4 l/s, the transmissivity is:

$$T=\frac{2.3\ Q}{4\ \pi\ \Delta s}=\frac{2.3*1.4*10^{-3}}{4*\pi*2.77}=9.3*10^{-5}\quad (m^2/s)$$

While a step drawdown complicates the evaluation of a pumping test slightly it has the advantage to enable the separation of the turbulent form the laminar component of the total drawdown. For the present test the rules of the method were not strictly adhered to. Yet a tentative analysis does provide some useful information. Table 5.3 gives relevant data for this analysis.

Table 5.3 Data for the step drawdown analysis.

period (minutes)	Δt (minutes)	Q l/s	Q m^3/hr	s_{net} (m)	s_{net}/Q
0- 25	25	1.0	3.6	3.23	0.90
26- 42	17	1.2	4.3	4.05	0.94
43- 60	18	1.3	4.7	5.26	1.12
61- 80	20	1.6	5.8	6.99	1.21
81-110	30	1.85	6.7	8.19	1.22

A least squares approximation of s_{net}/Q versus Q results in the linear equation: $s_{net}/Q = 0.515 + 0.112*Q$ (see figure 5.11), or $s_{net} = s_{laminar} + s_{turbulent} = 0.515*Q + 0.112*Q^2$. For each discharge rate the laminar and turbulent component is worked out in table 5.4.

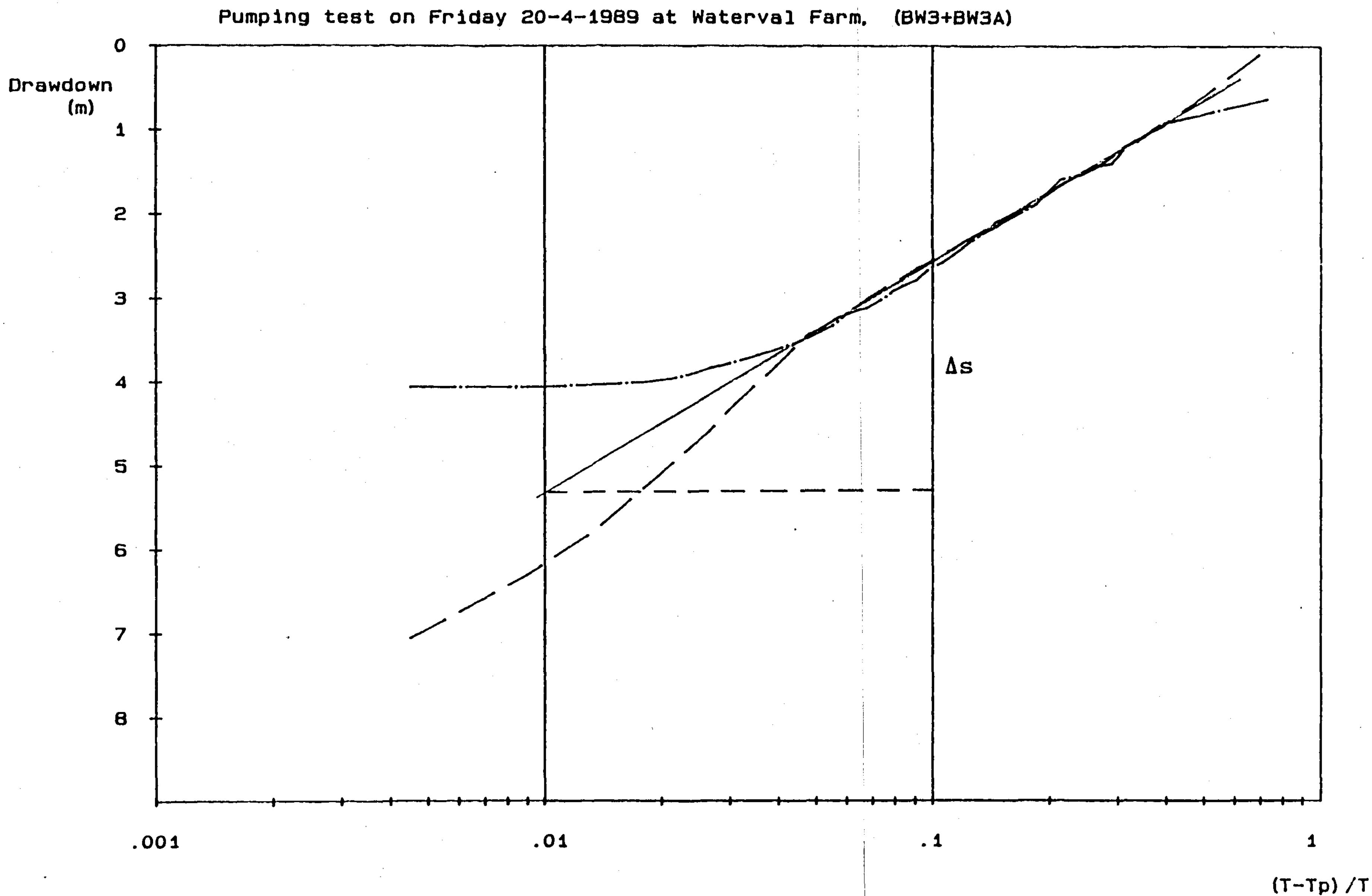


Figure 5.10 Recovery test

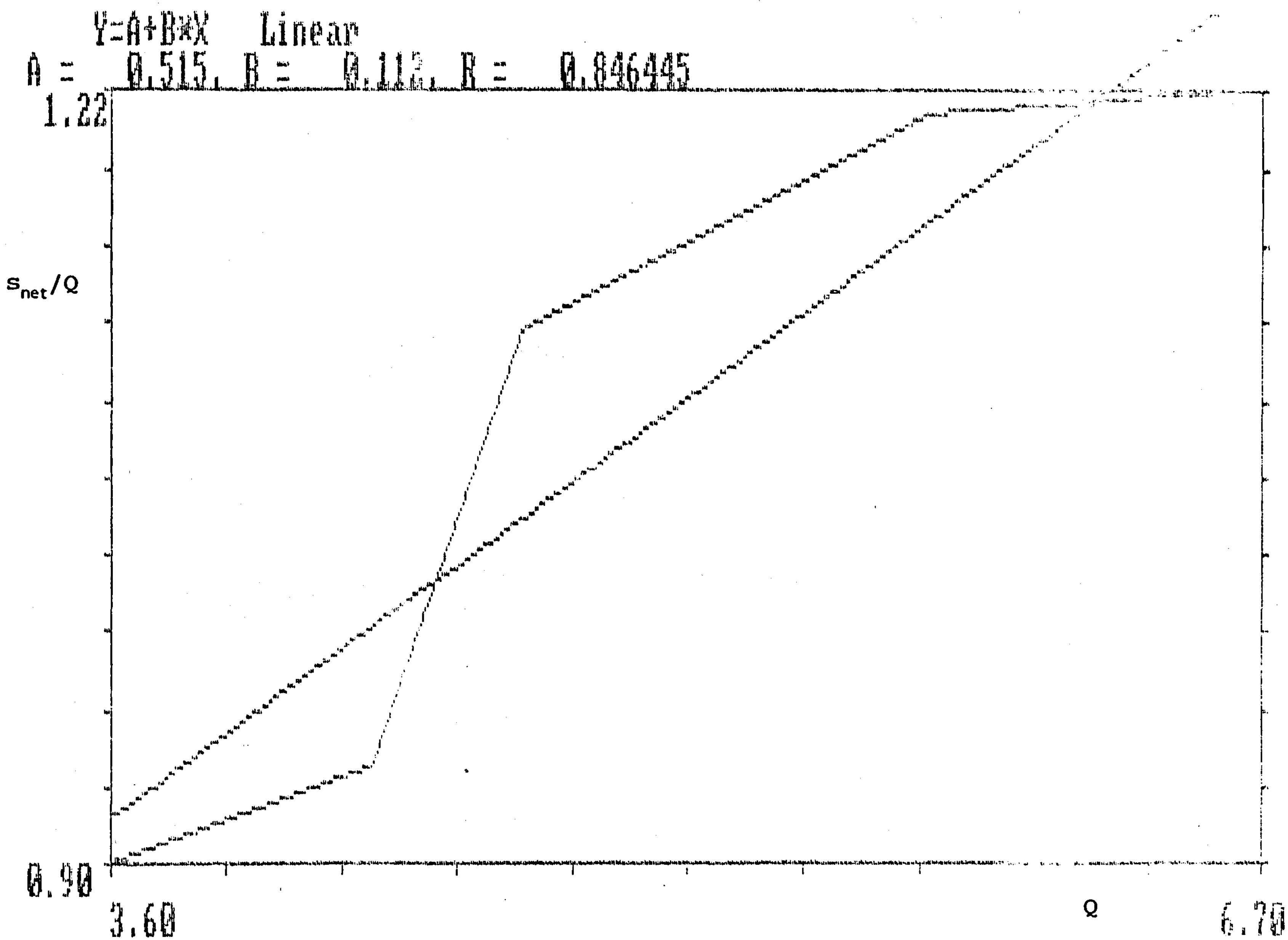


Figure 5.11 Least squares approximation of s_{net}/Q versus Q

Table 5.4 Laminar and turbulent component of the drawdown.

Q (m^3/hr)	$s_{laminar}$ (m)	$s_{turbulent}$ (m)	s_{total} (m)
3.6	1.85	1.45	3.30
4.3	2.21	2.07	4.28
4.7	2.42	2.47	4.89
5.8	2.99	3.77	6.76
6.7	3.45	5.03	8.48

From table 5.4 it follows that approximately half of the drawdown in the pumping hole is caused by turbulent flow in the direct vicinity of the hole. Looking back on the single hole test of the previous section it will be noted that the transmissivity is inversely proportional to the drawdown. Subsequently, if only the laminar component of the drawdown is taken into account the transmissivity would increase by approximately a factor two.

3) BW5

duration of pumping test: 110 minutes.

observation frequency:	0- 10 minutes:	every	½ minute,
	10- 30 "	"	minute,
	30-105 "	"	5 minutes,
	105-113 "	"	½ minute.

pumping rate: 0-110 minutes: 1620 Imp. gal/hr = 2.00 l/s.

As seen in figure 5.12 the drawdown is small in spite of the fairly high discharge rate. Again the recovery observations seem to have been influenced by the removal of the pump. Analysis with the Jacob method (see figure 5.13) results in a Δs of 0.90 m for the early part of the test (0-14.4 minutes) and Δs = 0.68 m afterwards. With Q = 2 l/s this results in a transmissivity of:

$$T=\frac{2.3\ Q}{4\ \pi\ \Delta s}=\frac{2.3*2.0*10^{-3}}{4*\pi*(0.68\ @\ 0.90)}=(4.1\ @\ 5.4)*10^{-4}\quad (m^2/s)$$

For the recovery test only a limited number of observations is available. The disturbance, probably caused by the removal of the pump, results in a slight shift of the graph (see figure 5.14). With Δs = 0.87 m per log-cycle the calculated transmissivity is:

$$T=\frac{2.3\ Q}{4\ \pi\ \Delta s}=\frac{2.3*2.0*10^{-3}}{4*\pi*0.87}=4.2*10^{-4}\quad (m^2/s)$$

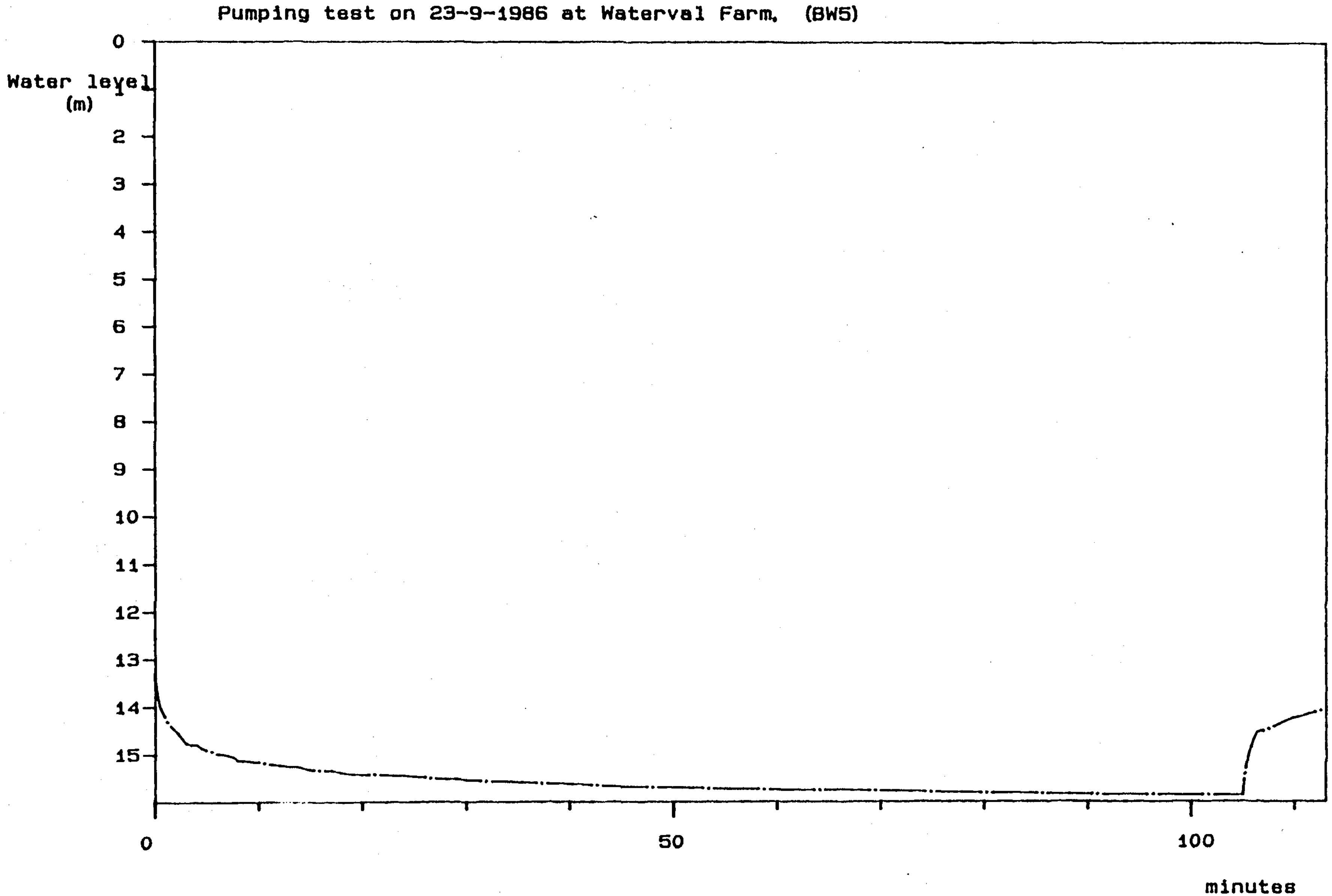


Figure 5.12 Water level observations during pumping test of BW5

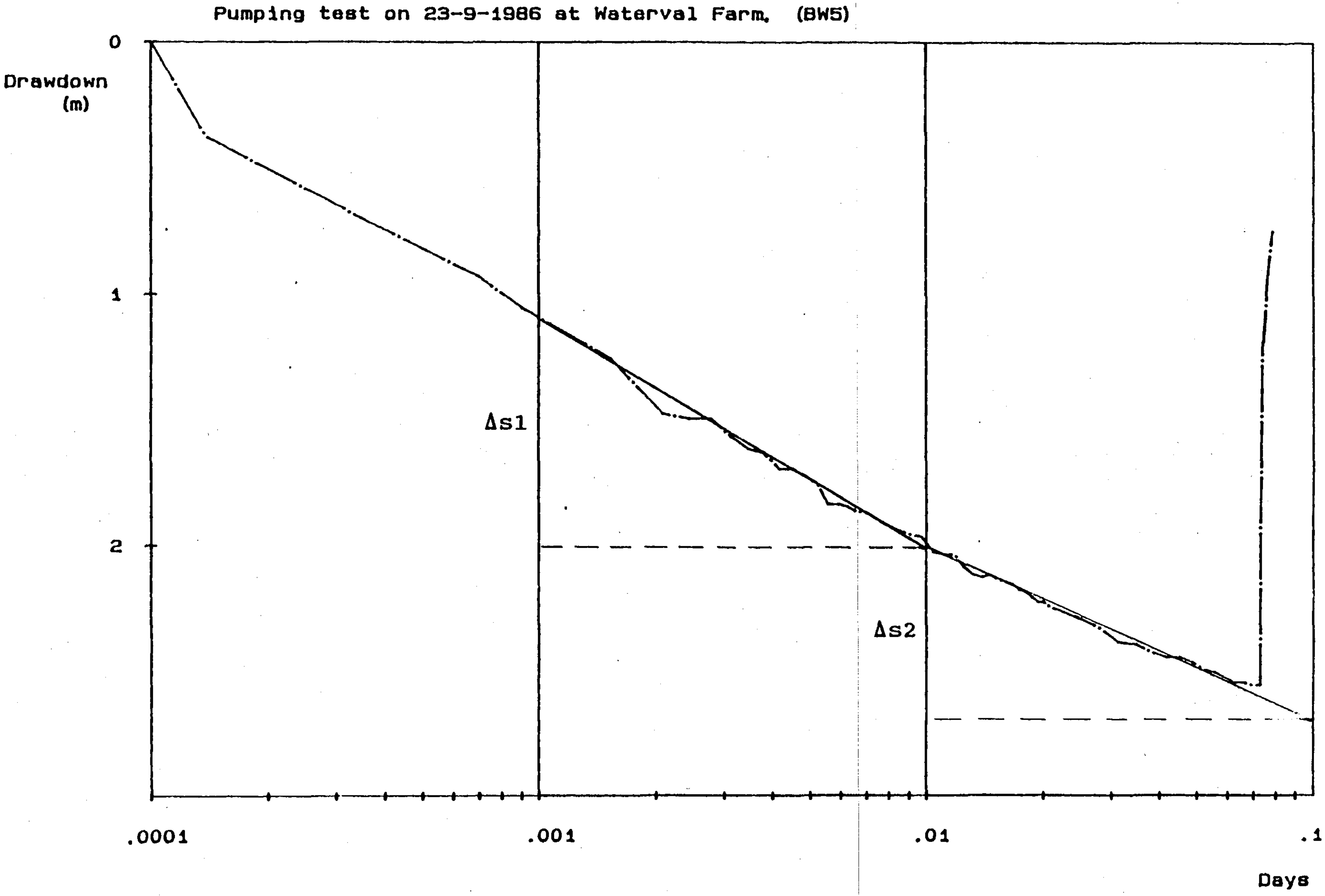


Figure 5.13 Pumping test analysis

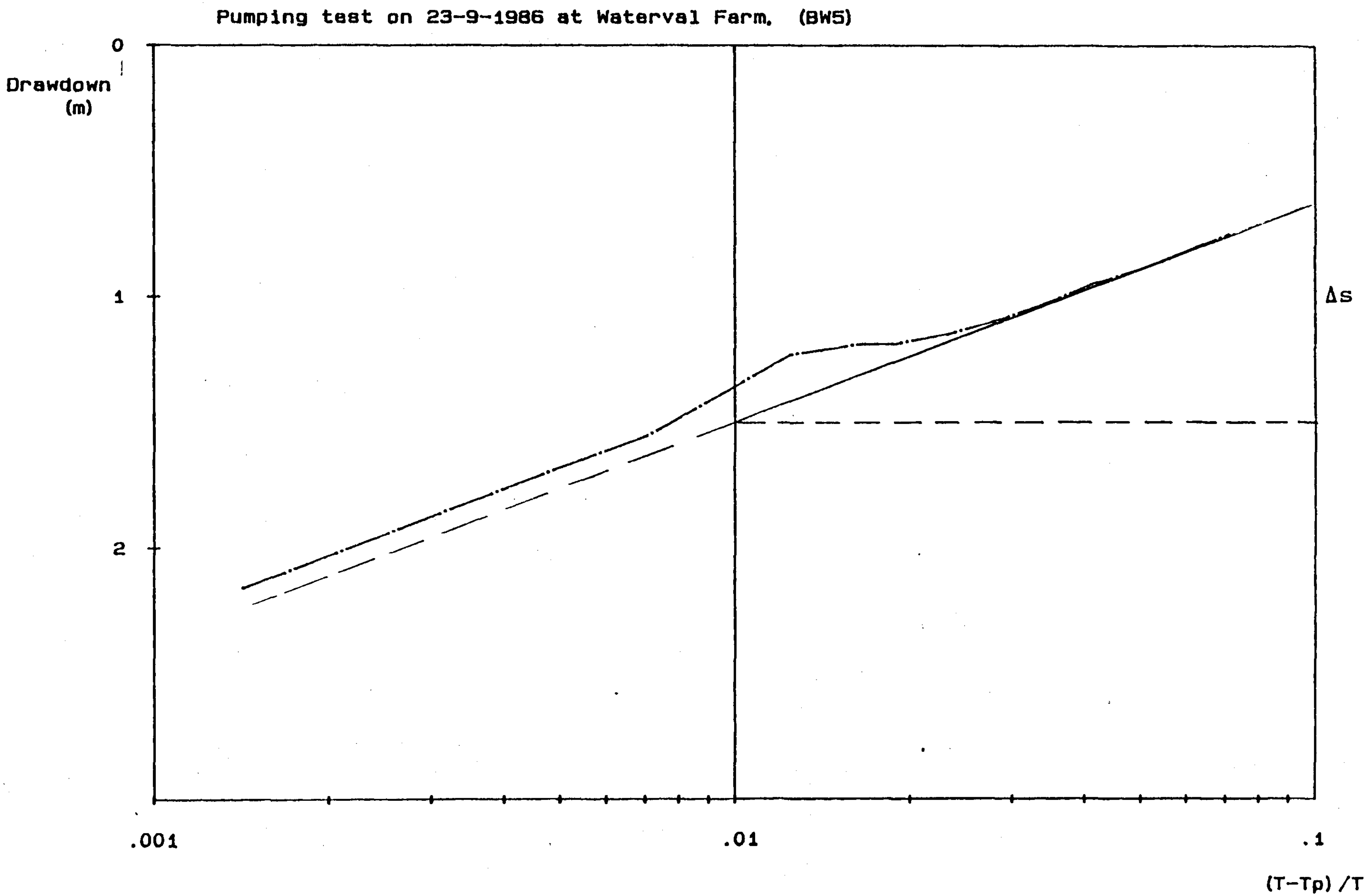


Figure 5.14 Recovery test analysis

4) BW5 + BW5A

The observation hole which lies 4.10 m away from the pumping hole, also lies somewhat lower than the pumping hole. Due to this fact and the apparently high permeability of the aquifer in this part of the catchment the initial waterlevel in the pumping hole is higher than in the observation hole. The high permeability also results in almost equal drawdown values, which explains the unusual situation that the water levels in the pumping hole do not drop below those in the observation hole (see figure 5.15).

duration of pumping test: 130 minutes.

observation frequency:	0- 20 minutes:	every	minute,
	20- 45 "	"	5 minutes,
	45- 60 "	"	minute,
	60- 80 "	"	5 minutes,
	80-130 "	"	10 minutes,
	130-135 "	"	½ minute,
	135-146 "	"	minute,
	150-170 "	"	5 minutes,
	170-210 "	"	10 minutes.

pumping rate: 0- 45 minutes: 1.25 l/s,
45-130 " : 1.8 l/s.

Analysis of the drawdown in the observation hole, using the Jacob method, gives a $\Delta s_1 = 0.56$ m and a $t_0 = 1.7*10^{-3}$ days (= 14.7 seconds) for the first part of the pumping test and a $\Delta s_2 = 1.37$ m and a $t_0 = 1.8*10^{-3}$ days (= 155.5 seconds) for the second part (see figure 5.16). The transmissivity and storage coefficient for the two periods is now:

$$T=\frac{2.3\ Q}{4\ \pi\ \Delta s}=\frac{2.3*1.25*10^{-3}}{4*\pi*0.56}=4.1*10^{-4}\quad (m^2/s)$$

$$S=\frac{2.25\ T\ t_0}{r^2}=\frac{2.25*4.1*10^{-4}*14.7}{4.1^2}=8.0*10^{-4}$$

$$u=\frac{r^2\ S}{4\ T\ t}=\frac{(4.1)^2*8.0*10^{-4}}{4*4.1*10^{-4}*14.7}=0.56$$

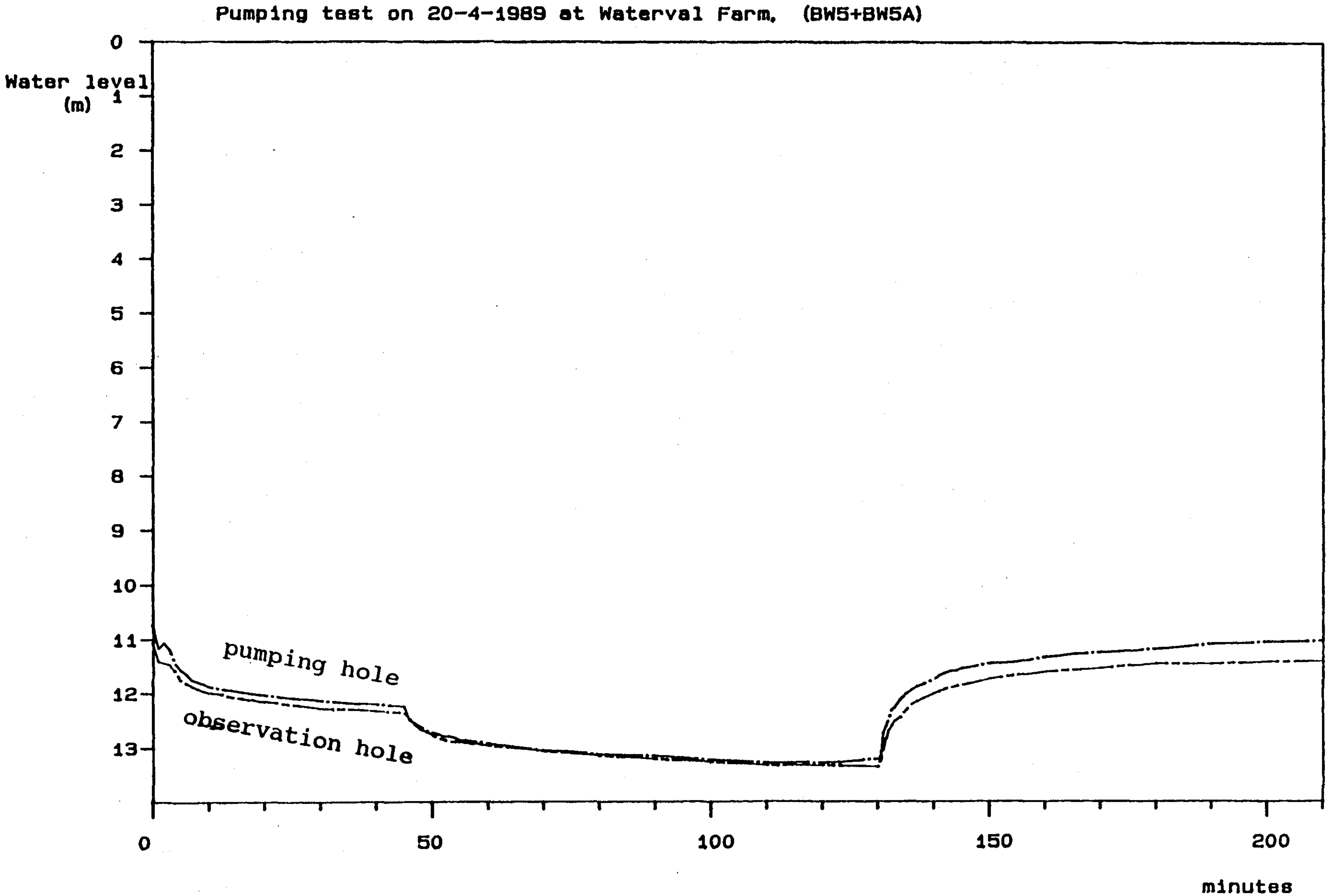


Figure 5.15 Water level observations

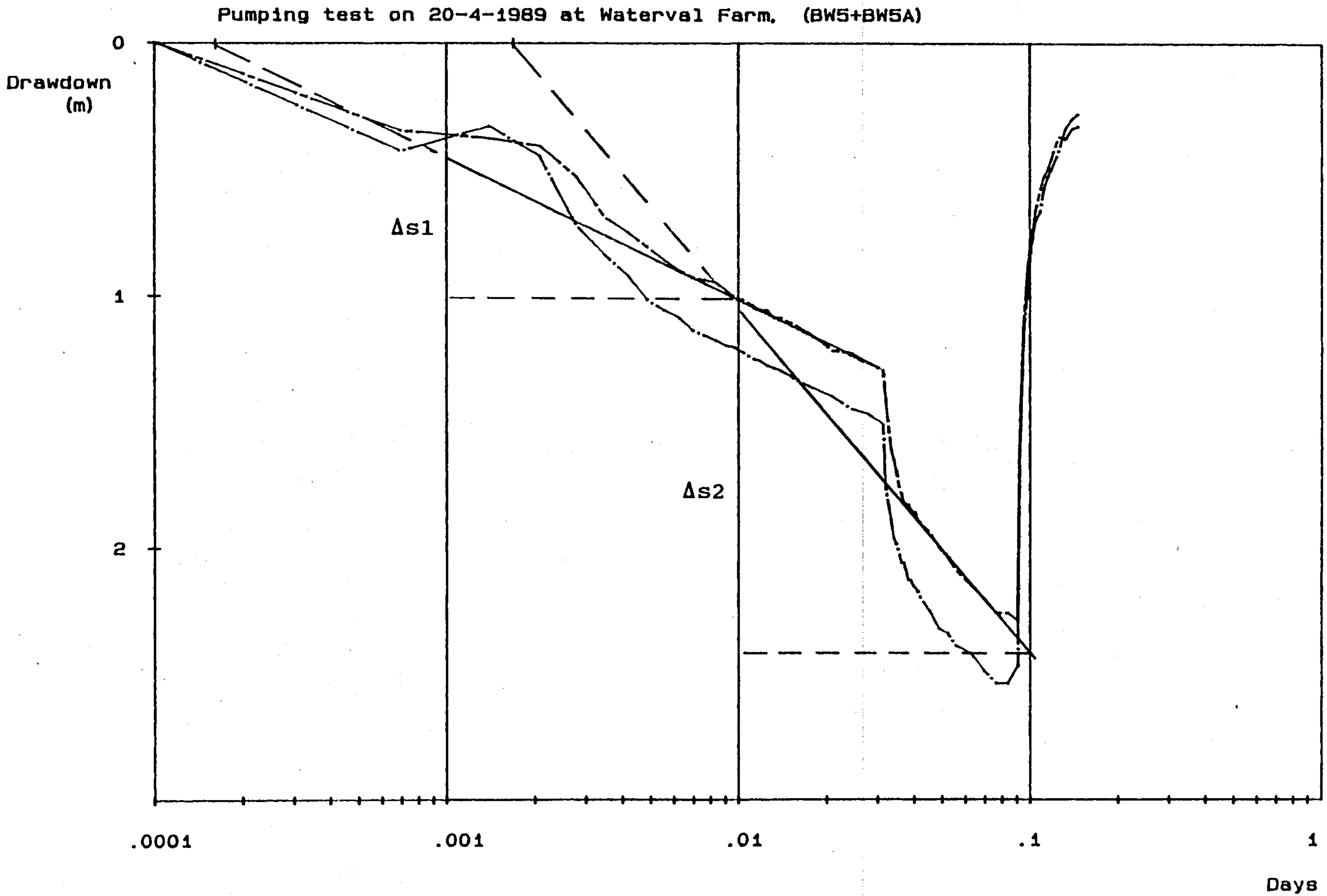


Figure 5.16 Pumping test analysis with Jacob method

and

$$T=\frac{2.3\ Q}{4\ \pi\ \Delta s}=\frac{2.3*1.8*10^{-3}}{4*\pi*1.37}=2.4*10^{-4}\quad (m^2/s)$$

$$S=\frac{2.25\ T\ t_0}{r^2}=\frac{2.25*2.4*10^{-4}*155.5}{4.1^2}=5.0*10^{-3}$$

$$u=\frac{r^2\ S}{4\ T\ t}=\frac{(4.1)^2*5.0*10^{-3}}{4*2.4*10^{-4}*155.5}=0.56$$

Obviously these results are unreliable as indicated by the value of u (>>0.01), and the Theis method should be applied. During the first five minutes of the pumping test the water levels in the observation hole show a slow response. Therefore agreement between the type curve and the data plot is sought for the period 5 to 45 minutes after the start of the test (r^2/t = 0.0062 to 0.056 m^2/s). With figure 5.17 we find e.g. u = 0.1, r^2/t = 0.122, s = 1, and W(u) = 3.4. These results yield:

$$T=\frac{Q\ W(u)}{4\ \pi\ s}=\frac{1.25*10^{-3}*3.4}{4*\pi*1}=3.4*10^{-4}\quad (m^2/s)$$

$$S=\frac{4\ u\ T}{r^2/t}=\frac{4*0.1*3.4*10^{-4}}{0.122}=1.1*10^{-3}$$

The analysis of the recovery data (see figure 5.18) provides a Δs = 1.02 m per log-cycle. The discharge rate during the second part of the test which lasted for 85 minutes will in this case have the most impact. We find:

$$T=\frac{2.3\ Q}{4\ \pi\ \Delta s}=\frac{2.3*1.8*10^{-3}}{4*\pi*1.02}=3.2*10^{-4}\quad (m^2/s)$$

Since only two different discharge rates were applied of considerably different duration, no attempt is made to separate the laminar from the turbulent component of the drawdown.

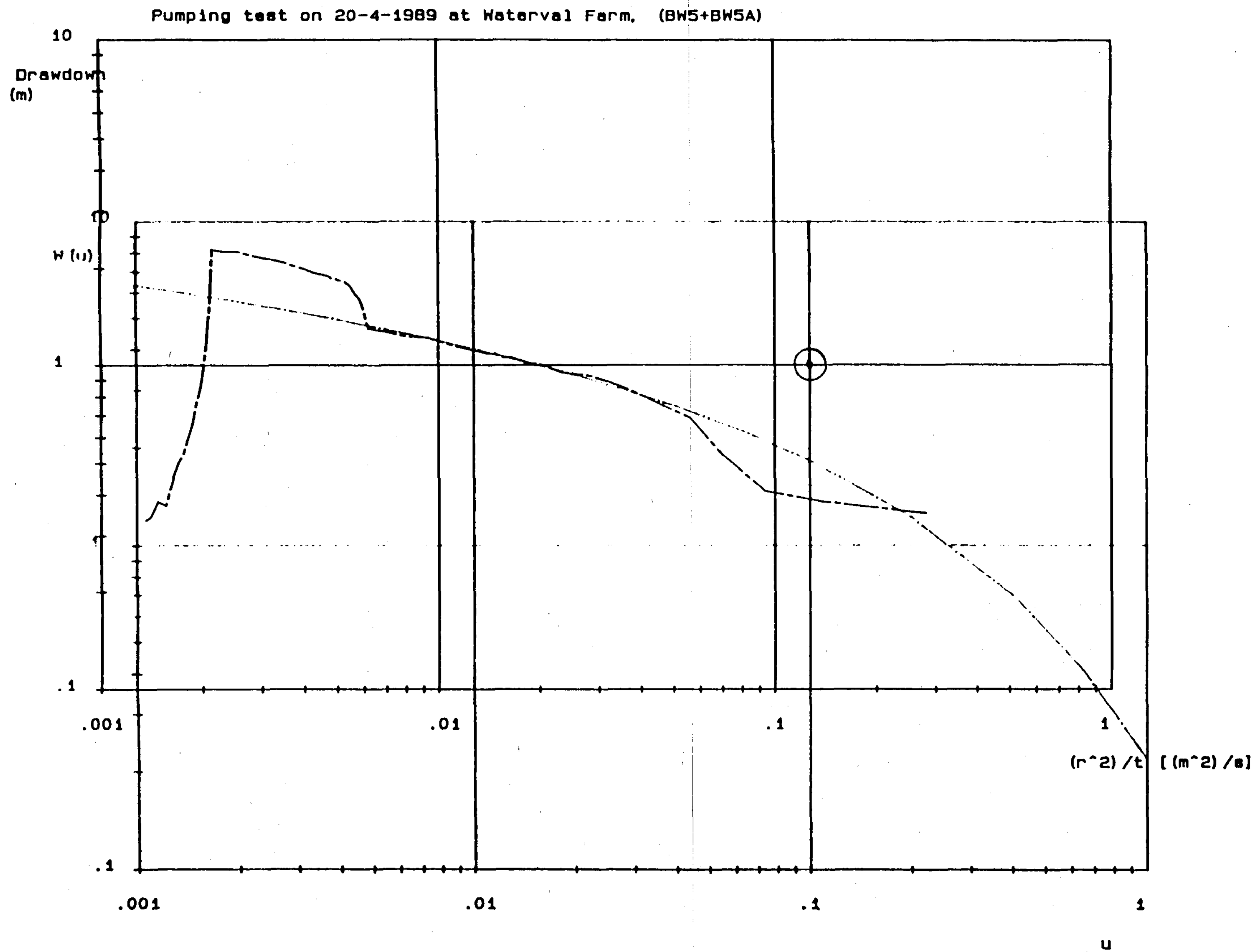


Figure 5.17 Pumping test analysis with Theis method

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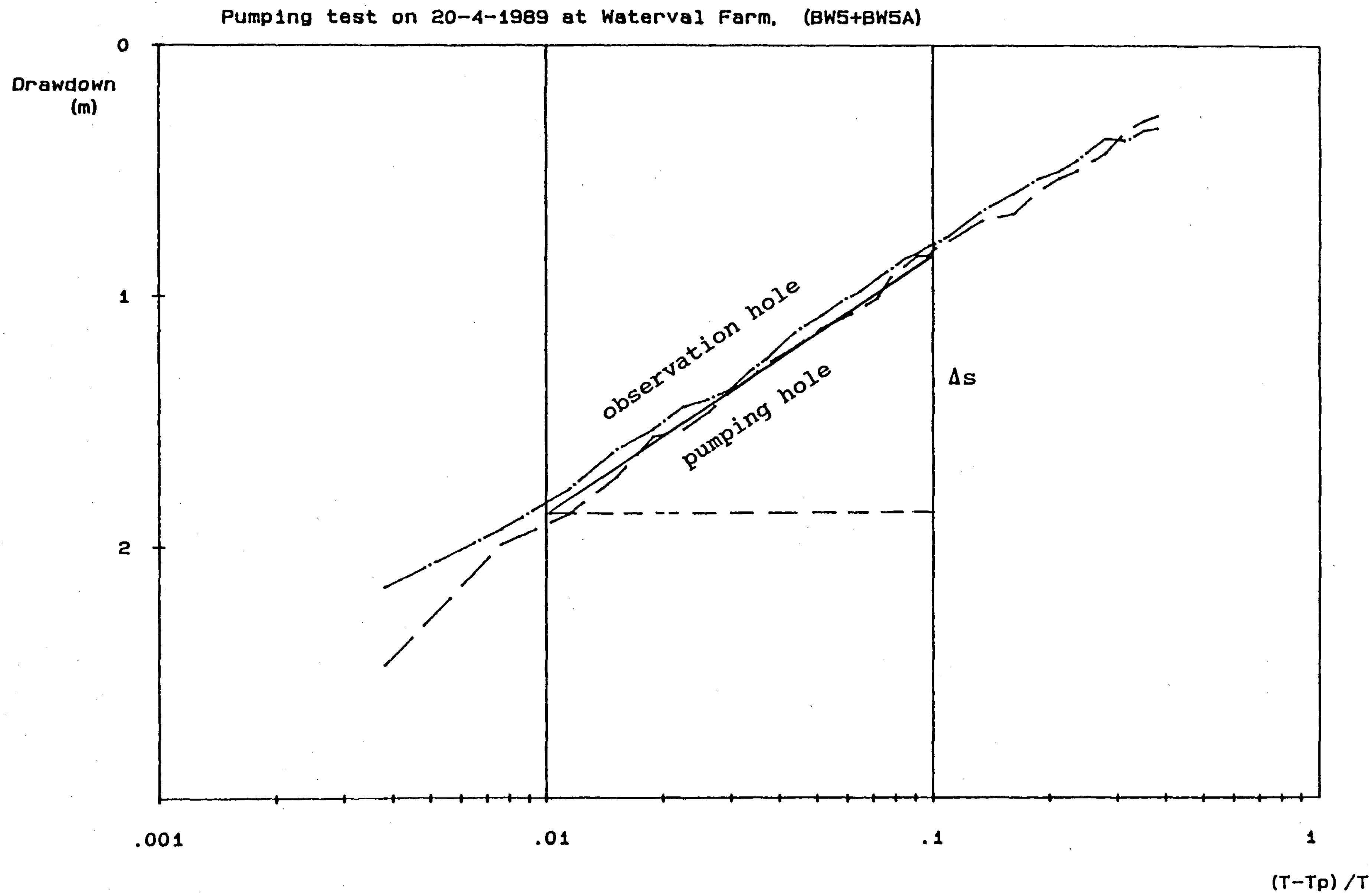

Figure 5.18 Recovery test

5.8 Résumé

Table 5.5 Résumé of test results at BW3

method	Transmissivity (10^{-5} m^2/s)	Storage coefficient (10^{-3})
single well Jacob	2.3	x
single well recovery	5.1	x
pumping test Jacob	14.4	0.98
	10.0	2.4
	6.0	3.4
	5.1	4.1
	5.9	4.8
pumping test Theis	10.6	1.4
recovery test	9.3	x

Table 5.6 Résumé of test results at BW5

method	Transmissivity (10^{-4} m^2/s)	Storage coefficient (10^{-3})
single well Jacob	4.1 - 5.4	x
single well recovery	4.2	x
pumping test Jacob	4.1	0.8
	2.4	5.0
pumping test Theis	3.4	1.1
recovery test	3.2	x

Taking into account the degree of applicability of the various methods used, and the comments made in the previous section, it can be confidently concluded that the transmissivity and storage coefficient lie in the ranges of $(6\text{-}10)*10^{-5}$ m^2/s and $(1.5\text{-}3)*10^{-3}$ respectively in the vicinity of BW3, and in the ranges of $(3\text{-}4)*10^{-4}$ m^2/s and $(1\text{-}2)*10^{-3}$ respectively in the vicinity of BW5.

5.9 Tentative calculation of the groundwater runoff

In the previous sections it was aimed for to determine the aquifer characteristics of the Waterval catchment to the highest degree of accuracy. However, these aquifer characteristics only form a means to determine the groundwater runoff from the catchment. At present there are only two tested sites on ± 70 hectares, and precise aquifer boundaries are unknown. The following estimate of the maximum groundwater runoff rate should therefore be used with care.

The two values of transmissivity, that were found in the previous sections, are likely to be representative for the contour they are in. Since the groundwater runoff is determined by the lowest permeability, the transmissivity of BW3 should be used in this instance. If it were assumed that the groundwater boundaries correspond with the catchment boundaries, then the width of the cross-section at the outlet of the catchment is ± 400 m. The actual thickness of the aquifer system is unknown, but this poses no problem, since the transmissivity incorporates this parameter. The average slope of the catchment is approximately 1:18, and the groundwater slope is approximately the same. Combination of the law of continuity ($Q = v*A = v*b*h$) with Darcy's law ($v = k*i$) yields $Q = k*i*b*h$. With the transmissivity defined as $T = k*h$ we find $Q = b*T*i$, or $Q = 400*8*10^{-5}*(1/18) = 1.78*10^{-3}$ m^3/s = 154 m^3/day = 56 Ml/yr.

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6 ENVIRONMENTAL ISOTOPES IN GROUNDWATER STUDIES

6.1 Introduction

Atoms are made up of neutrons, protons, and electrons. For any one element, the number of protons (the atomic number) is invariant, but the number of neutrons may vary, resulting in different isotopes of the same element. Hydrogen, for example, exists in the form of three isotopes; it always has 1 proton, but may have zero, one, or two neutrons, giving atomic mass numbers (the total number of protons and neutrons) of 1, 2, and 3, designated ^{1}H (protium), ^{2}H (deuterium), and ^{3}H (tritium). Generally each element has one or more stable isotopes which account for the bulk of its occurrence on earth. For example, in the case of hydrogen, ^{1}H and ^{2}H are the stable isotopes, of which ^{1}H is by far the most abundant form.

Unstable isotopes undergo spontaneous radioactive decay by the loss of nuclear particles (α or β particles) and, as a result, they may transmute into a new element. Furthermore, the decay rate of a particular isotope is invariable so that a given quantity of the radioactive isotope will decay to its daughter product in a known interval of time; this is the basis of radio-isotopic dating methods. The measurement of the isotope concentration today will indicate the amount of time which has elapsed since the sample was emplaced. The amount of time which it takes for a radioactive material to decay to half its original amount is termed its half-life. For example, the half-life of tritium is 12.43 years; (some seemingly authoritative sources give an alternative value for the half-live of tritium:
$t_{1/2}$ = 12.26 years). The decay constant lambda is given by:
$\lambda = {}^{e}\log(2)$ / half-life.

For a radioactive isotope to be directly useful for dating it must possess several attributes: a) the isotope itself, or its daughter products, must occur in measurable quantities and be capable of being distinguished from other isotopes, or its rate of decay must be measurable; b) its half-live must be of a length appropriate to the period being dated; c) the initial concentration level of the isotope must be known; d) there must be some connection between the event being dated and the start of the radioactive decay process.

In general terms, radio-isotopic dating methods can be considered in three groups, those which measure a) the quantity of a radio-isotope as a fraction of a presumed initial level (e.g. ^{14}C dating) or the reciprocal build-up of a stable daughter product; b) the degree to which members of a chain of radioactive decay are restored to equilibrium following some initial external perturbation (uranium-series dating); c) the integrated effect of some local radioactive process on the sample materials, compared to the value of the local (environmental) flux (fission-track and thermo-luminescence dating).

In the present report attention is focussed on the environmental isotopes ^{3}H, ^{2}H, and ^{18}O and to some extent on ^{14}C. The adjective "environmental" refers to the atmospheric source of the isotopes as opposed to the artificially injected isotopes in groundwater studies.

6.2 Tritium

Tritium is produced naturally by cosmic ray interactions with atmospheric nuclei, principally nitrogen, in the higher stratosphere. Oxidation enables the tritium atoms to become incorporated into the water vapour of the lower stratosphere, whence they diffuse through the tropopause into the region of weather processes. The natural steady state concentration of tritium (<15 tritium units) in atmospheric moisture has been disturbed ever since 1952 by the enormous amounts of anthropogenic tritium produced by thermonuclear tests. The tritium concentration in northern hemisphere precipitation had increased by about 3 orders of magnitude, at its maximum level of 1963 and has been decreasing slowly ever since. In the southern hemisphere the influence has been considerably less strong (see table 6.1).

Table 6.1 Tritium concentrations in precipitation (in T.U.)

year	Ottawa Canada 1)	Europe (mean) 1)	Negev Israel 2)	Adelaide Australia 1)	Harare Zimbabwe 3)	Pretoria South Afr. 4)	Pretoria South Afr. 1)
1951	15	15	<7	5	x	x	x
1952	20	20	<7	5	x	x	x
1953	30	25	<7	5	x	x	x
1954	302	300	16	8.3	x	x	x
1955	45	35	18	8.0	x	x	x
1956	146	100	61	x	x	x	x
1957	126	125	48	22.1	x	15	13.7
1958	515	300	220	44.6	6.3	20	25.6
1959	540	450	130	27.7	6.7	20	15.5
1960	145	145	37	22.1	6.9	20	15.5
1961	219	110	71	18.1	11.3	20	15.5
1962	988	700	629	25.5	22.6	40.3	39.0
1963	3032	2500	880	45.2	48.7	61.8	59.7
1964	1565	1300	219	54.5	42.8	51.5	48.5
1965	865	580	318	50.5	45.6	51.5	43.3
1966	590	240	165	44.0	25.2	41.6	40.5
1967	315	160	112	36.3	25.6	51.5	51.2
1968	214	150	81	32.0	21.3	46.3	38.7
1969	x	x	75	32.6	19.8	x	43.3
1970	x	x	70	31.7	24.3	x	37.2
1971	x	x	60	25.8	21.7	x	29.9
1972	x	x	52	19.4	25.6	x	x
1973	x	x	36	13.2	x	x	x
1974	x	x	35	14.8	x	x	x
1975	x	x	39	x	x	x	x
1976	x	x	24	x	x	x	x
1977	x	x	28	x	x	x	x
1978	x	x	27	x	x	x	x
1979	x	x	26	x	x	x	x
1980	x	x	19	x	x	x	x
1981	x	x	20	x	x	x	x
1982	x	x	15	x	x	x	x
1983	x	x	15	x	x	x	x

Sources: 1) Marshall and Holmes (1979), 2) Gvirtzman and Magaritz (1986), 3) Wurzel (1983), 4) Bredenkamp, Schutte and du Toit (1974).

Tritium reaches the earth in minute concentrations in rain. On infiltration into the ground, the rainwater becomes to a greater or lesser extent isolated from the atmospheric source and the concentration drops according to the characteristic half-life of tritium. The tritium concentration therefore becomes a measure of the residence time of groundwater since the time of recharge or of the recharge-storage ratio.

The concentration of tritium is commonly expressed in terms of tritium units (TU), which is defined as 1 ^3H-atom per 10^{18} ^1H-atoms. Since the concentration in natural waters is very small, it is usual to enrich the water sample, prior to analysis, by electric reduction to about 1/500th of its initial volume. This large reduction is normally achieved in three stages by electrolysis, by batch processing. The electrolyte is usually NaOH obtained by the addition of 1 per cent anhydrous Na_2O_2 to each stage. At the conclusion of an electrolysis run the remaining solution has the NaOH converted to Na_2CO_3 by bubbling with CO_2 gas. The water enriched in ^2H and ^3H is then distilled over completely, the various steps in the analysis being designed to prevent fractionation. The enrichment by electrolysis is measured either by spiking one cell in the batch with a known addition of tritium, or by monitoring the enrichment of deuterium with a suitable mass spectrometer.

Tritium is detected by its emission of ß-particles, which have an energy spectrum distributed from a peak in counting rate at about $3*10^3$ eV to a maximum energy of 18 keV. Its energy spectrum enables tritium to be readily distinguished from other radioactive isotopes that could be present in the water, particularly ^{14}C, ^{40}K and the uranium and thorium series elements. For the radioactive measurement of tritium either proportional gas counters or liquid scintillation techniques are used. In the former method, tritium is converted into a gas (ethane) which is then put into a "proportional counter" capable of detecting ß particles (variations in output voltage pulses being proportional to the rate of ß-particle emission). Conversion of water to ethane is done by the following steps:

$H_2O + Mg \text{ ---> } MgO + H_2$ (at 600^0 C), and

$H_2 + C_2H_4 \text{ ---> } C_2H_6$ (with a palladium catalist).

In liquid scintillation procedures, the enriched water sample containing the tritium is placed in an instrument which detects scintillations (flashes of light) in the liquid, produced by ß-particle emissions. Laboratories in the southern hemisphere have to pay greater attention to sensitivity and freedom from contamination because of the lower tritium concentrations in the rain and thus also in the groundwater. It is obviously important to eliminate partial evaporation of the water samples before testing.

Clay soils may store up tritium inside microscopic pores and pockets with relatively immobile water. In addition, there is an isotopic exchange between the hydroxyl groups of the clay minerals and the water absorbed in them (Gvirtzman and Magaritz, 1986).

Due to the relative short half-life of tritium the effects of the hydrogen bomb tests are rapidly decreasing, especially in the southern hemisphere. The annual rest factor of radioactive isotopes can be expressed as arf = 1 - λ. For tritium arf equals 0.9442. This means that the peak values of 1963 at present (1990) have been reduced to $(0.9442)^{27}$ = 21% of their original concentration. Based on the last column of table 6.1 the present day values of the original tritium concentrations for South Africa are given in table 6.2.

Table 6.2 Rest values of South African tritium concentrations

year	years past	orig. conc.	present(1990) conc. (in TU)
1957	33	13.7	2.1
1958	32	25.6	4.1
1959	31	15.5	2.6
1960	30	15.5	2.8
1961	29	15.5	2.9
1962	28	39.0	7.8
1963	27	59.7	12.7
1964	26	48.5	10.9
1965	25	43.3	10.3
1966	24	40.5	10.2
1967	23	51.2	13.7
1968	22	38.7	11.0
1969	21	43.3	13.0
1970	20	37.2	11.8
1971	19	29.9	10.1

As can be seen from table 6.2 the present day concentrations of non-decayed tritium are approaching the pre-test input levels. For this reason the use of environmental tritium as a tracer in groundwater studies will soon be a thing of the past.

6.3 Deuterium and Oxygen-18

The stable isotopes deuterium and ^{18}O are fractionated at the stages of the hydrological cycle where a change of state occurs. In particular, because the vapour pressure of water molecules containing the heavy isotopic individuals, whether they be deuterium or ^{18}O, is less than the vapour pressure of the light molecules, water vapour tends to be depleted in the heavy isotopes. The liquid water body from which the water vapour evaporated tends to be enriched with them. There is a variation in the relative abundances of isotopic species present in precipitated water that depends upon the characteristic temperatures at which evaporation and condensation took place, as illustrated in figure 6.1. The large negative values are from the higher latitudes and the values nearer to zero are from lower latitudes or from surface waters.

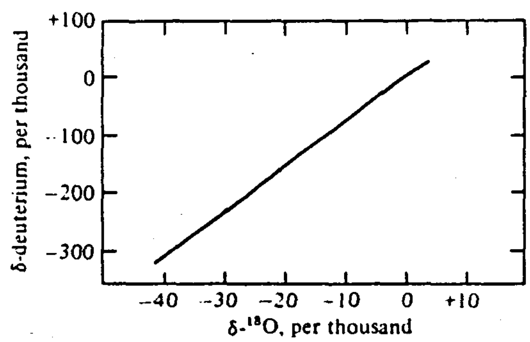

Figure 6.1 Deuterium and Oxygen-18 in precipitation and surface waters (after Marshall and Holmes, 1979).

As can be seen there is a very strong correlation of depletions of meteoric water in deuterium and Oxygen-18. The correlation is known as the Meteoric Water Line and can be approximated by the relation: $\delta D = d + 8 * \delta^{18}O$. The intercept d stems from fractionation at the ocean-atmosphere interface and varies from region to region. It is about 10 in Europe and South Africa, and about 24 in the eastern Mediterranean (Mandel and Shiftan, 1981). The concentrations of deuterium and Oxygen-18 are given as relative deviations δ from an international standard water (Standard Mean Ocean Water or SMOW) and are expressed in terms of δ^2H per mille (or δD per mille), and $\delta^{18}O$ per mille respectively. They are calculated using the following equation:

δD resp. $\delta^{18}O = [(R_{sample} / R_{standard}) - 1] * 1000$ (‰)
where $R = {}^{2}H / {}^{1}H$, and $R = {}^{18}O / {}^{16}O$ respectively.

The study of stable isotopes as tracers is more appropriate at the higher latitudes due to the seasonal input variations, which can be approximated by sine functions with the highest concentrations during summer. In tropical and coastal regions stable isotopes only have a modest tracer aptitude.

Water remaining in soil at any time is probably rarely more than 10 per cent of the total rainfall. It could conceivably be enriched in the heavy molecules in a variety of ways, determined by the nature of evaporation of the other 90 per cent of the rain, either directly by transpiration or by a combination of that and evaporation from the soil surface, and influenced by the temperature at which the change of state took place.

6.4 Radiocarbon

Radiocarbon (or Carbon-14) is produced in the upper atmosphere by neutron bombardment of atmospheric nitrogen atoms. The neutrons have a maximum concentration at around 15 km and are produced by cosmic radiation entering the upper atmosphere. Although cosmic rays are influenced by the Earth's magnetic field and tend to become concentrated near the geomagnetic poles (thus causing a similar distribution of neutrons and hence ^{14}C), rapid diffusion of ^{14}C atoms in the lower atmosphere obliterates any influence of this geographical variation in production. ^{14}C atoms are rapidly oxidized to $^{14}CO_2$, which diffuses downward and mixes with the rest of atmospheric carbon dioxide and hence enters into all pathways of the biosphere.

In the early 1960's the half-life of radiocarbon was established to be 5730 years. However, in 1955 Libby had proposed a half-life of 5568 years. To avoid confusion, it was decided to continue using Libby half-life (rounded to 5570 years) and this practice has continued.

Several aspects make the interpretation of radiocarbon concentrations in water samples very difficult. Firstly, the ^{14}C input concentration has to be corrected for the dilution of the organic carbon by inorganic carbon. Secondly, the concentration of carbon compounds (bicarbonate, carbonate, and CO_2) in mixing waters may differ appreciably. Usually a possibility of mixing is either tacitly omitted or the radiocarbon concentration is assumed to be proportional to the mixing components of water. However, it is self-evident that the radiocarbon concentration is weighted both by the volumetric flow rates and by the total dissolved carbon contents. As the interpretation of radiocarbon tests would contain a considerable degree of guess work it was decided not to test the samples for radiocarbon.

6.5 Environmental Isotopes and Groundwater Modelling

Environmental isotopes, and in particular tritium, have been widely used for the study of infiltration, vertical seepage in the unsaturated zone, and aquifer recharge (e.g. Bredenkamp *et al.* (1974), Gvirtzman and Magaritz (1986)). Analysis of soil moisture samples from different depths can provide an estimatation of the vertical water movement for a particular site.

On a broader scale, environmental isotope tracers can provide useful estimates of the water resources in small catchment areas. Like all tracers, they can provide the input and output data against which a (numerical) simulation model can be calibrated. The main contrast with artificially injected tracers is the fact that environmental isotopes are evenly recharged over the permeable sections of a catchment, and at a rate proportional to the rainfall. Due to this characteristic the ratio of storage to annual recharge of an aquifer can be established. The turnover time or mean transit time (T) reflects the mean age of water leaving the system and is defined as $T = V / Q$, where V is the volume of mobile water in the system and Q the volumetric flow rate.

Much effort has been directed towards the development of representative flow models. The possible applicability of a model depends on whether the aquifer is (partially) confined or unconfined, whether the aquifer increases in thickness or not, and whether the sampling wells are fully penetrating or have extended casing (see figure 6.2).

The top drawing in figure 6.2 represents the Piston Flow Model (PFM). Water in an aquifer enters at the outcrop of the stratum only, then flows within the confining beds of the aquifer and eventually is withdrawn by tube wells, or discharges as spring flow. The PFM assumes that the concentration of a tracer changes only due to radioactive decay, disregarding dispersion. Thus, it applies strictly speaking only to cases where the water has been separated and stagnant since the recharge time. Then, the age of the water is defined by: $C(t) = C(0) \exp(-\lambda t)$, where t here is the age of the water and C(0) is the initial concentration of a radiotracer. If both the dispersion is low and the input concentration is constant in time, or slowly variable, the PFM may be applicable to dynamic systems, and the age of the water is approximately equal to the turnover time T. An application of the PFM can be found in Wurzel (1983).

For the exponential (EM) it is assumed that the exponential distribution of transit times corresponds to a probable situation of exponentially decreasing permeability (and in some cases porosity as well) with the aquifer depth. The hydraulic gradient is assumed to be proportional to the distance from the water divide.

The Exponential-Piston Flow Model (EPM) is a combination of the two previous models. The EPM is described in more detail by Marshall and Holmes (1979), while an application of the EPM is given by Allison and Holmes (1973). An application in a multilayered geologic medium is given by Gureghian and Jansen (1985).

The linear model (LM) describes an aquifer with linearly increasing thickness and a constant hydraulic gradient. No

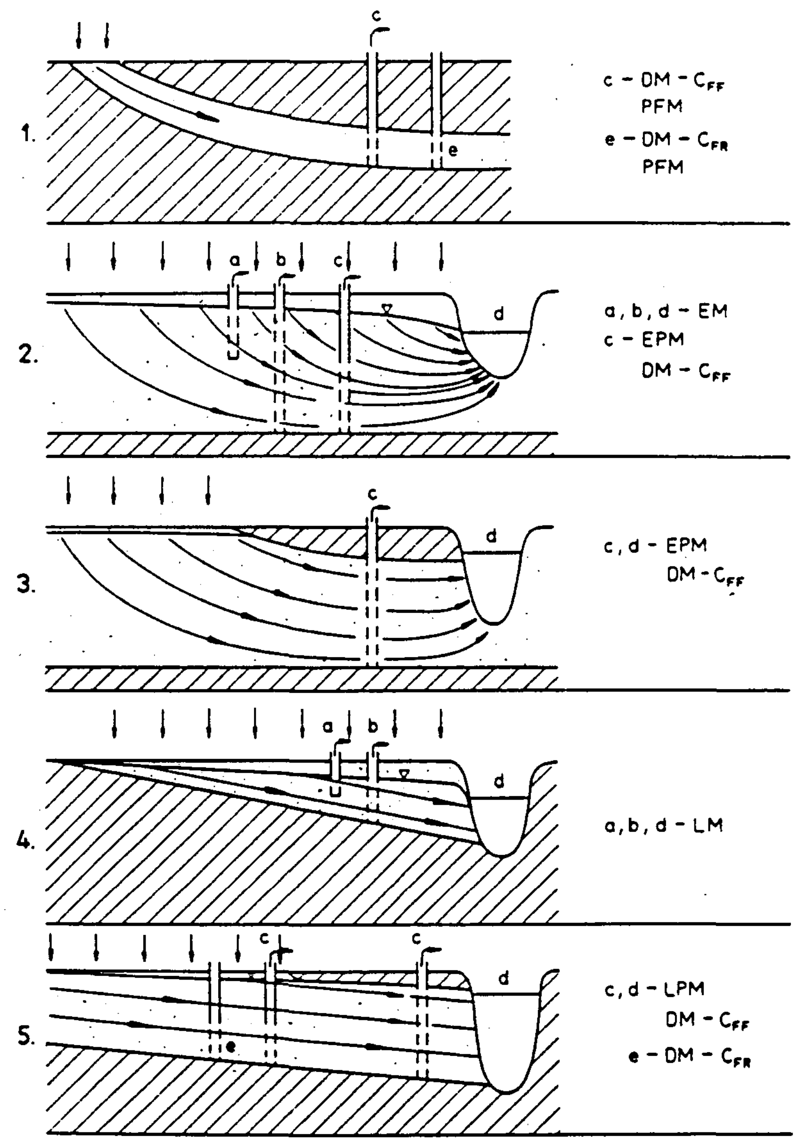

Figure 6.2 Schematic situations showing examples of possible applicability of particular models (After Malozewski and Zuber ,1983).

DM = Dispersion Model

PFM = Piston Flow Model

EM = Exponential Model

EPM = Exponential and Piston Flow Model

LM = Linear Model

LPM = Linear and Piston Flow Model

example of a practical applicability of this model has been known so far. When combined with the PFM it gives the Linear-Piston Flow Model (LPM).

The dispersion of tracer material during subsurface travel can be studied using the Dispersion Model (DM). An application of the DM is given by Maloszewski *et al.* (1983). Finally the Binomial Model (BM) was introduced as an approximation of the Dispersion Model. Wurzel (1983) shows an application of the BM.

6.6 Field tests on the Waterval catchment

On 20 April 1989 pumping tests were conducted on two borehole sets, each consisting of a pumping hole and an observation hole. The holes BW5 and BW5A are situated half-way up the catchment, while the holes BW3 and BW3A are situated at the bottom of the catchment. Five water samples were collected at different stages of the pumping tests (see figures 6.3 and 6.4). A further sample (nr 1) was bailed from an existing borehole (BW1) at the top of the catchment. All samples were tested at the Schonland Research Centre (University of the Witwatersrand) for tritium, deuterium and Oxygen-18. In addition, the samples were tested at McLachlan & Lazar for alkalinity and a range of conventional impurities.

The total cumulative abstraction during the pumping tests was 9.5 m^3 from BW3 and 12.5 m^3 from BW5. If an average porosity of 1% were assumed for the full submerged length of the borehole (31.4 m and 31.1 m resp.) the radius of the affected area could be calculated with $R = [V / (p\pi L)]^{\frac{1}{2}}$. This would yield $R_{BW3} = 3.1$ m and $R_{BW5} = 3.6$ m. In reality the lower sections of the boreholes are effectively impermeable except for intersecting layers of decomposed granite, while nearer to the surface the baserock is overlain by layers of increasing permeability. Therefore the pumped water must have been derived from a considerably larger area.

The results of the tritium analysis of the water samples is given in table 6.3

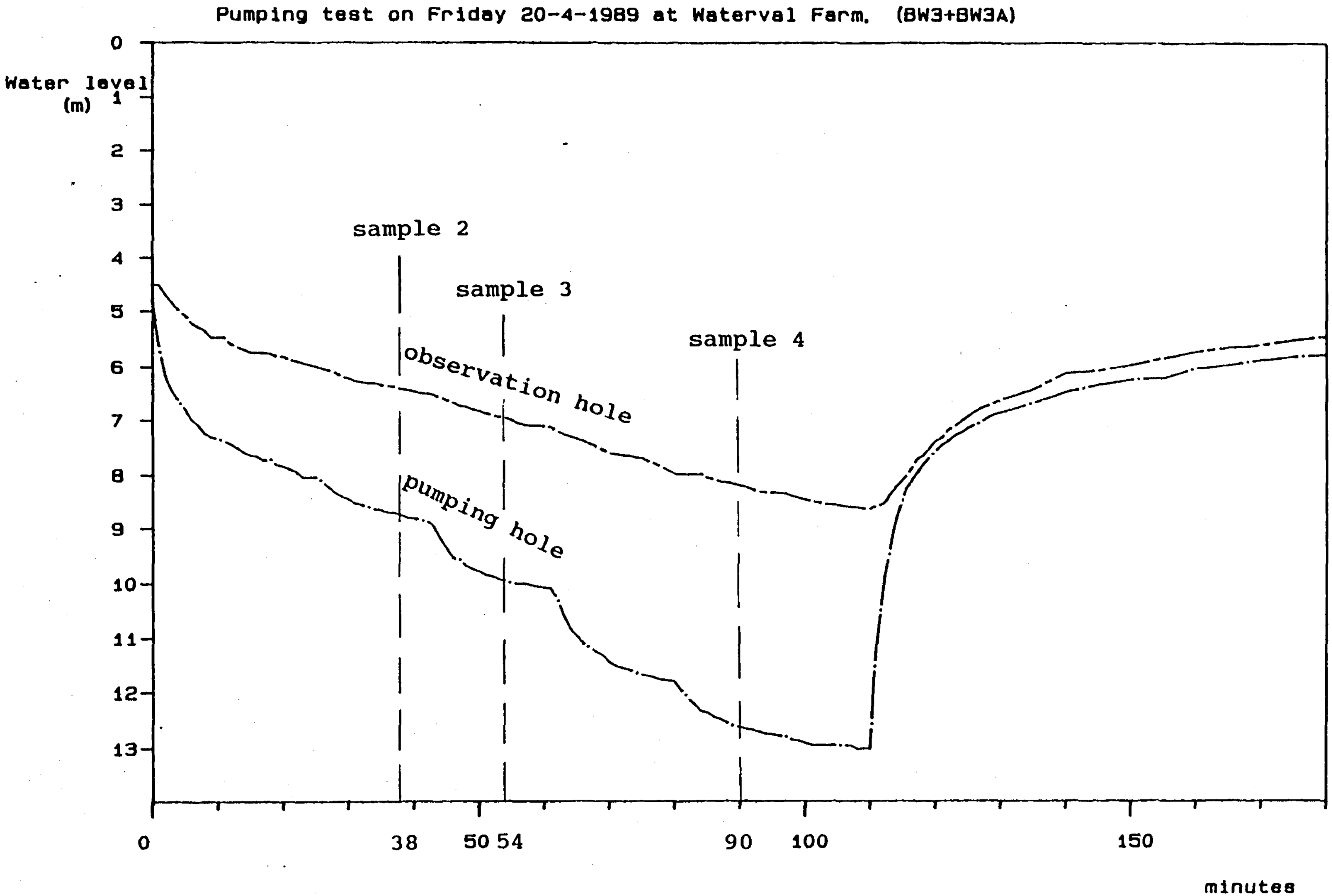

Figure 6.3 Water levels and sampling time during pumping test in borehole BW3

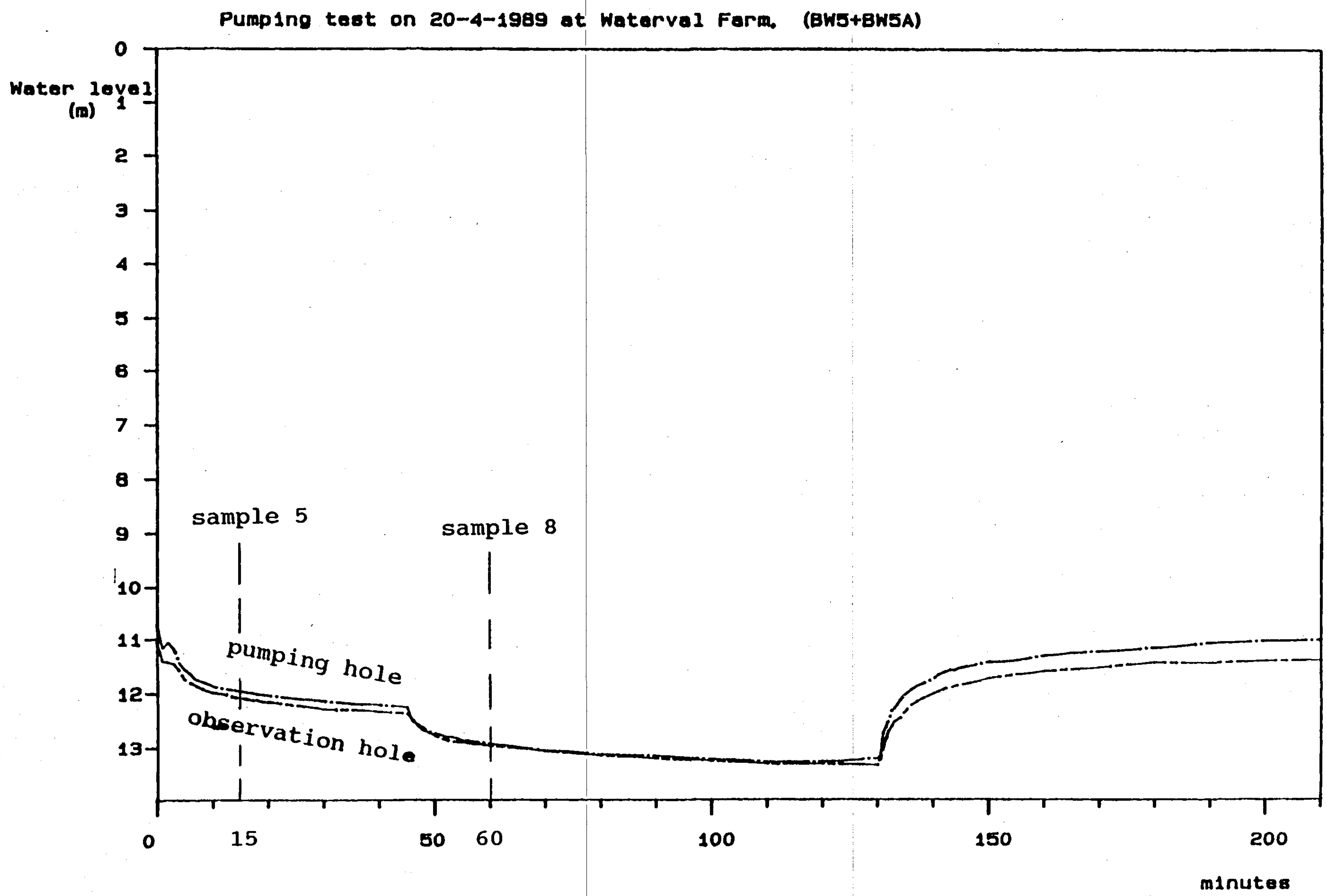

Figure 6.4 Water levels and sampling time during pumping test in borehole BW5

Table 6.3 Tritium analysis of borehole water samples

sample	borehole	time (minutes)	T.U.
1	BW1	x	5.8 ± 0.5
2	BW3	38	0.1 ± 0.2
3	BW3	54	0.2 ± 0.2
4	BW3	90	0.4 ± 0.2
5	BW5	15	0.5 ± 0.2
8	BW5	60	0.3 ± 0.2

Sample 1 shows infiltration of contemporaneous rainfall and implies a turnover time of no more than about a decade. The significance of this sample as far as the *in situ* groundwater at the top of the catchment is concerned is however doubtful. The sample was bailed out of the borehole. Unless there is considerable turnover or throughflow of groundwater in the borehole, the possibility exists that the standing water column represents no more than surface runoff or shallow seepage water. The tritium values of sample 2 to 8 all lie at or below the limit of detectability. Sample 4 and 5 may contain just measurable tritium ($>2\sigma$). In any case, the turnover time of the groundwater lies in the range of 30 to >50 years.

The results of the stable isotope analysis are given in figure 6.5. The data points are labelled with the sample numbers and error bars represent routine standard deviations. The Meteoric Water Line is added by way of reference.

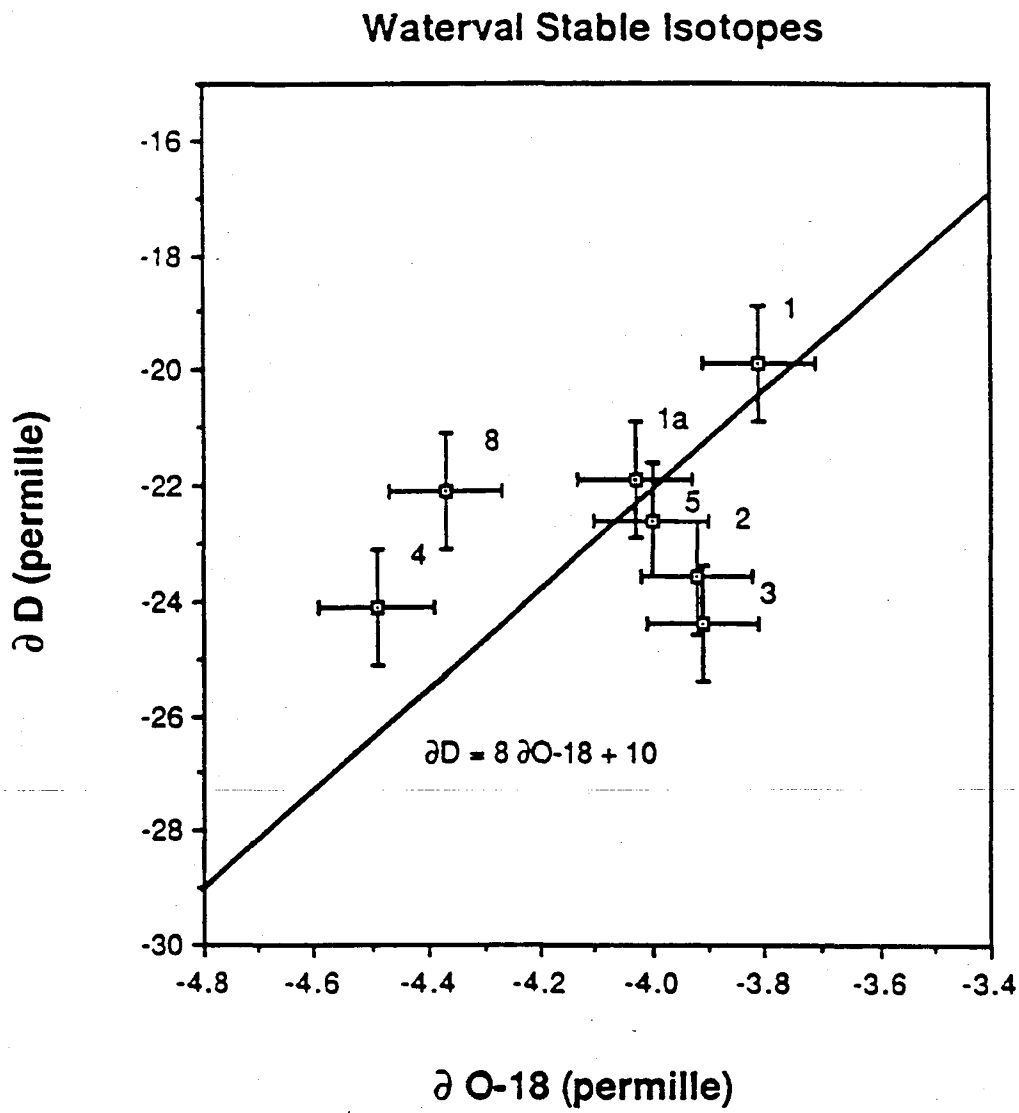

Figure 6.5 Stable isotope analysis

The following comments were supplied together with the analysis:

a) The points all lie on or near the Meteoric Water Line. There is no evidence of significant surface evaporation before recharge.

b) When compared to the range of isotopic values which can occur in rain water, the Waterval results all lie close together, indicating very similar recharge conditions for all the groundwater sampled. This is especially so for the samples from early in the pump tests and from borehole BW1.

c) Unintentionally, separate measurements were performed on the two bottles constituting sample 1. Both isotopes show a small but significant, concordant, shift between the two aliquots. If the two samples are simply splits of the same field sample, a small amount of evaporation seems to have occurred from at least one of the two bottles. Similar shifts might have affected some of the other samples as well; (there was a hiatus of several months between the sampling and the actual isotope measurements).

d) A significant and similar small shift in mainly oxygen-18 seems to have occurred in both borehole BW3 and borehole BW5. This suggest that water of a somewhat different composition (and residence time?) was gradually drawn in during both pumping tests. Small chemical changes might likewise have taken place.

Finally, the results of the chemical analysis of the borehole water samples are given in table 6.4. Water sample nr 1 was not submitted for chemical analysis.

Table 6.4 Chemical analysis of borehole water samples

	BW3			BW5	
sample	2	3	4	5	8
pH value	7.65	7.55	7.45	7.35	7.25
Conductivity mS/m	21.9	21.9	21.9	14.8	13.8
Total Dissolved Solids	176	188	190	104	158
Calcium, Ca	17.6	17.5	17.4	10.8	11.6
Magnesium, Mg	6.6	6.2	6.6	4.0	4.0
Sodium, Na	23	22	21	15	15
Potassium, K	3.0	0.9	0.7	18	1.8
Bicarbonate, HCO_3	115	117	115	76	81
Carbonate, CO_3	nil	nil	nil	nil	nil
Chloride, Cl	4.0	4.0	5.0	5.0	5.0
Sulphate, SO_4	3	3	2	2	2
Nitrate, NO_3	1.26	1.45	1.03	4.0	1.45
Phosphate, PO_4	0.1	0.1	0.2	0.3	0.3

Results are expressed in mg/l where applicable

Although small differences in the chemical composition can be observed between water from boreholes BW3 and BW5, the observations for each hole individually are in general very similar. From this it can be concluded that the groundwater, which was affected by the pumping activity, is well mixed.

6.7 Conclusions

The analysis of the groundwater samples from the Waterval catchment seems to indicate that there are two distinct groundwater regimes present. The one consisting of an essentially stagnant water mass stored in fissures and the pores in the decomposed granite bordering these fissures. The second is formed by an unconfined aquifer on top of the base rock. A balance seems to have been established, whereby little to no recharge of the first aquifer takes place. However, prolonged pumping will eventually deplete the deeper aquifer, after which recharge may take place.

All samples were apparently derived from the lower confined aquifer with the exception of sample 1, which was simply bailed out of the borehole and was of considerably younger age.

Particularly the results of the tritium test were disappointing in that they do not enable a groundwater model to be calibrated. Similarly, the deviations of the stable isotopes from the Meteoric Water Line are too small to allow model calibration.

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7 TESTS WITH ARTIFICIAL TRACERS

7.1 Introduction

Many types of non-radioactive and radioactive tracers are available to determine the velocity of ground water, such as salt (NaCl or $CaCl_2$), fluorescent dyes, and the radio-isotopes ^{3}H, ^{131}I, ^{29}Br, and ^{51}Cr-EDTA, each group having its own advantages and disadvantages.

For the present study use has been made of regular kitchen salt for its ease of access and detection. Obvious disadvantages are the increase in unit weight of the salt-water mixture and the potential chemical interaction between the tracer and the aquifer matrix. Density effects caused by the use of electrolytes, such as salt, are unavoidable due to the relatively high concentration of tracer needed. The maximum concentration that can be applied at a water temperature of 20°C is 264 gr/l of NaCl when the salt-water solution becomes saturated (Mandel & Shiftan, 1981). The solubility of minerals decreases with dropping temperatures but is practically unaffected by pressure. In the case of sodium chloride the temperature dependency is very small (Petrucci, 1985).

Since considerable stirring is required to dissolve the salt crystals, especially at high concentrations, the tracer material should be dissolved before being injected into the aquifer. The salt can be dissolved on site using the borehole water for this purpose, as illustrated in figure 7.1. However, the process of completely dissolving all the salt can take several hours. This would create a problem when analyzing the test results with the theory of the point dilution method, which requires an almost instantaneous injection. Since furthermore a borehole pump was not available an alternative procedure was followed. A highly concentrated brine was prepared in the laboratory and then injected evenly over the submerged length of the borehole. For an even distribution of the tracer material a 35m long hose was slowly lowered into the borehole, starting at the water surface,

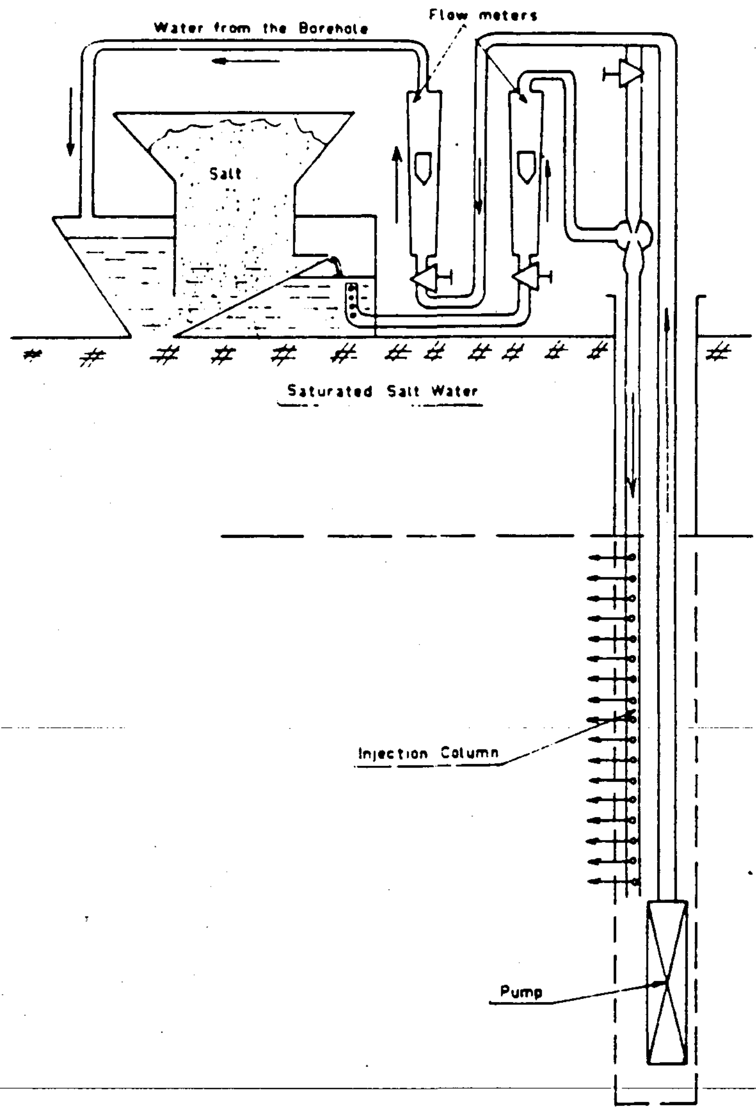


Fig. 7.1 Single-well geoelectrical method : injection device
(After Fried, 1975)

while the brine was poured in at the other end. The disadvantage of this method is the creation of a temporary head which forces the water and brine mixture into the aquifer, and so disturbs the natural groundwater flow. For this reason the amount of injected solution should be kept to a minimum, and thus the brine as concentrated as possible, the saturation level being the upper limit.

If the concentration of salt in the groundwater before injection is approximately zero, then the concentration in the borehole immediately after injection can be calculated with

$$C_0 = \frac{V_2}{V_1+V_2} * C_{brine}$$

where C_0 = concentration of salt immediately after injection, V_1 = initial volume of water in borehole, and V_2 = amount of brine with a salt concentration of C_{brine}.

When working with high concentrations of salt in an aqueous solution it is important to be aware of the volumetric influence of the salt. The density of kitchen salt is 2170 kg/m^3. This means that e.g. 200 gr dissolved in 1 liter pure water equals 200 gr/1.092 liter solution = 183 gr/l solution.

7.2 Conductivity

Conductivity is a numerical expression of the ability of an aqueous solution to carry an electric current. This ability depends on the presence of ions, their total concentration, mobility, valence, and relative concentrations, and on the temperature on measurement.

In the International System of Units (SI) conductivity is reported as millisiemens per meter (mS/m). The still frequently encountered micro mhos per centimeter (μmhos/cm) is related to the SI units by 1 mS/m = 10 μmhos/cm (Greenberg *et al.*, 1985). Approximate ranges of electrical conductivity values are given in table 7.1.

Table 7.1 Approximate ranges of electrical conductivity values

distilled water	0.05	-	0.5	mS/m
rainwater	0.5	-	3	mS/m
fresh groundwater	3	-	200	mS/m
sea water	4500	-	5500	mS/m
brines			>10000	mS/m

The relation between conductivity, concentration and temperature is given for sodium chloride solutions in the comprehensive graph 7.2.

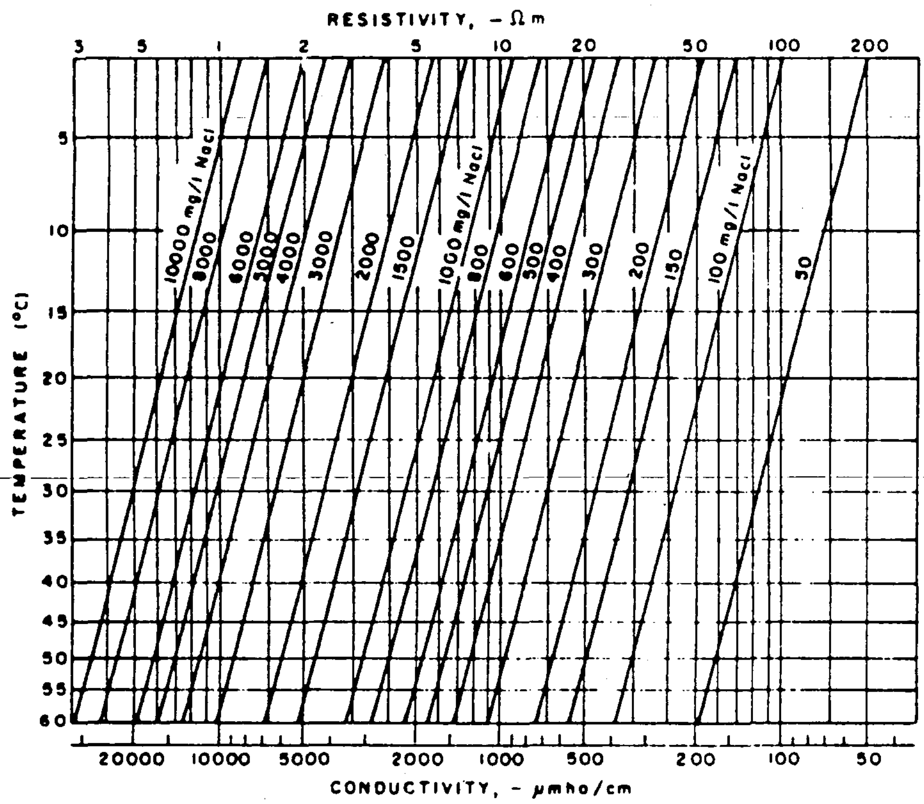


Figure 7.2 Resistivity and conductivity of sodium chloride solutions at different temperatures (from Mandel and Shiftan, 1981 and after Keys and MacCary, 1971).

From figure 7.2 a graph was derived (which relates conductivity to concentration at a fixed temperature of 20°C. This graph was extended by laboratory experiments to also cover higher concentrations (see figure 7.3).

Borehole measurements were conducted using a Hanna Instruments Conductivity Meter (HI 8333). For optimal accuracy this meter provides 4 ranges :

0.0 - 199.9	μS/cm	i.e. 0 - 20	mS/m
0 - 1999	μS/cm	i.e. 0 - 200	mS/m
0.00 - 19.99	mS/cm	i.e. 0 - 2000	mS/m
00.0 - 199.9	mS/cm	i.e. 0 - 20000	mS/m

Due to a background conductivity of 7 to 22 mS/m in the groundwater, fairly high concentrations were used for the injected solution. In order to avoid large numbers and unfounded accuracies the observations during the tests were recorded in mS/cm.

The conductivity of the water in the boreholes was tested at discrete depth intervals. Water samples from these points were collected with a cylindrical perspex sampling device. The samples were processed on site. Although the groundwater temperature was very constant at approximately 20°C the temperature of each sample was tested. When necessary the temperature correcting dial on the conductivity meter was adjusted. Next the conductivity was measured. Finally the sampling device, sample container, thermometer and conductivity probe were rinsed with distilled water before the next sample was taken.

7.3 Single borehole tests

Although the tests with artificial tracers were conducted in those holes, which have a nearby observation hole, an attempt was made to derive additional information from the reduction of concentration over time in the injection hole. The analysis of this phenomenon is known as the borehole dilution technique or point dilution method. The point dilution method aims to relate the observed dilution of a tracer introduced in a well to the rate of the undisturbed groundwater flow in the aquifer.

The method, which attracted considerable attention during the 1960's in continental Europe, is most often combined with the

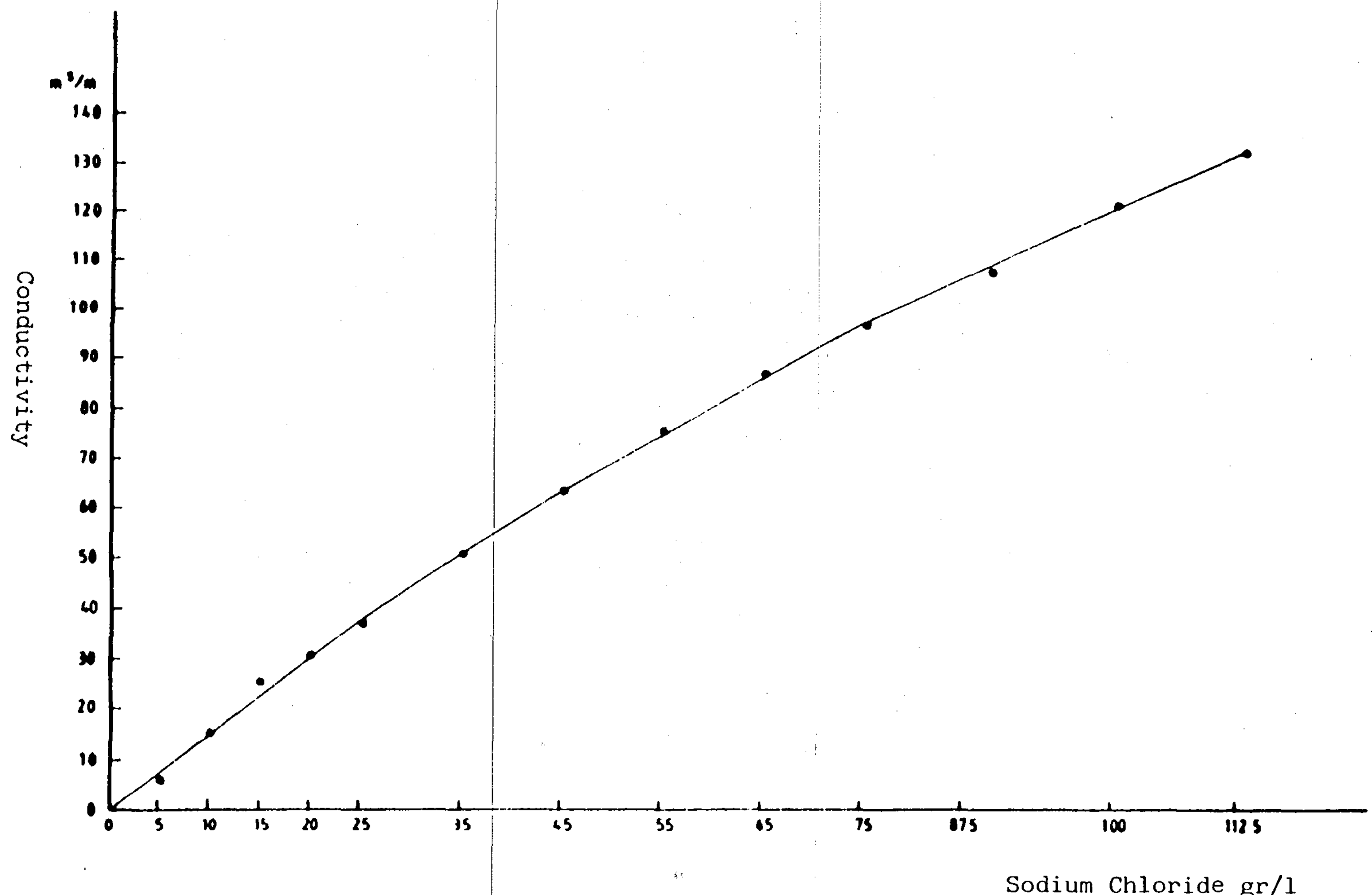


Fig. 7.3 Concentration of Conductivity and T.D.S. (Cl.)

application of radioactive tracers. An interesting field application is illustrated by Raymond and Bierschenk (1957). Freeze and Cherry (1977) describe the method briefly, while two state of the art reviews are available: Halevy *et al.* (1966) and Drost *et al.* (1968). Gaspar and Oncescu (1972) spend one chapter on this subject.

Since the theory is essentially developed for application to a homogeneous aquifer consisting of unconsolidated granular elements the well design is assumed to comprise a well screen surrounded by a gravel filter. The presence of a borehole with or without a gravel filter, will have an influence on the lateral flow pattern (see figure 7.4).

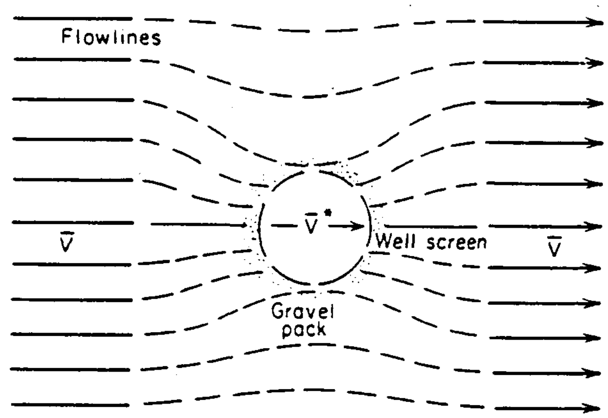


Figure 7.4 Distortion of flow pattern caused by the presence of a borehole (After Freeze and Cherry, 1977).

The average linear velocity of the groundwater in the formation beyond the zone of disturbance is v. The average bulk velocity across the centre of the well bore is denoted by v^*. It will be assumed that the tracer is non-reactive and that it is introduced instantaneously at concentration C_0 into the borehole. The vertical cross-sectional area through the centre of the submerged segment of the hole and perpendicular to the flow is denoted as A. The volume of this well segment is W. At time $t > 0$, the concentration C in the well decreases at a rate:

$$\frac{dC}{dt} = -\frac{A\,v^{*}\,C}{W}$$

which, upon rearrangement, yields:

$$\frac{dC}{C} = -\frac{A\,v^{*}\,dt}{W}$$

Integration and use of the initial condition, $C = C_0$ at $t = 0$, leads to:

$$v^{*} = -\frac{W}{A\,t}\ln\left(\frac{C}{C_0}\right) = \frac{W}{A\,t}\ln\left(\frac{C_0}{C}\right)$$

The actual groundwater velocity is related to the apparent velocity by:

$$v = \frac{v^{*}}{n\,\alpha}$$

where n is the porosity and α is an adjustment factor that depends on the radii and hydraulic conductivities of the well screen and gravel pack. The usual range of α for tests in sand or gravel aquifers is from 0.5 to 4. In the case of a borehole of infinite permeability or, more exactly, of a well without filter tube and filtering envelope the α-coefficient has a value of 2, provided that the walls are not clogged up with e.g. mud.

If r represents the well radius and L the submerged section of the borehole the cross-sectional area A equals $2r*L$. The volume in which dilution takes place (W) then equals πr^2*L. The actual groundwater velocity in the vicinity of an uncased borehole can thus be expressed as:

$$v = \frac{1}{2n}\,\frac{\pi r^2 L}{2rLt}\ln\left(\frac{C_0}{C}\right) = \frac{\pi r}{4nt}\ln\left(\frac{C_0}{C}\right)$$

7.4 Application of the point dilution method.

In total three tests were conducted in the Waterval Catchment using electrolytes as a tracer. During the first trial test (started on 5 June 1990) in borehole BW5 halfway down the slope of the catchment, useful experience was gained.

During the injection a fixed tube was used hanging down to the middle of the submerged borehole section, under the assumption that the injected brine would dilute itself evenly and spontaneously. This turned out to be a wrong assumption. A subsequent attempt to mix brine and borehole water by moving an object up and down the hole proved ineffective (see figure 7.5). It was also shown that due to the irregularity of the aquifer formation the tracer material was washed out unevenly. This made it clear that in subsequent tests the observations should be made at more regular depth intervals.

Although not shown in figure 7.5 the tracer concentration close to the bottom of the borehole showed a highly irregular pattern. Either due to vertical density currents or an accumulation of undissolved salt particles a layer of very high tracer concentration persisted on the bottom of the borehole for the entire duration of the tests. Depending on the degree of perturbance during sampling of the lower end of the hole, water samples came up with widely varying concentrations. For this reason the observations for the bottom of the borehole were not included in the calculations.

For an evaluation of the test results the observations for a particular time step are averaged. The weight given to each observation is taken proportionally to half the distance to the observation points above and below, where applicable. By averaging over the relevant submerged section of the hole, possible vertical density currents are accounted for.

Combination of the observations between 11.5 m and 30 m below surface yield the average concentrations as shown in figure 7.6 (top). Given a borehole diameter of $2r = 0.15$ metre and assuming

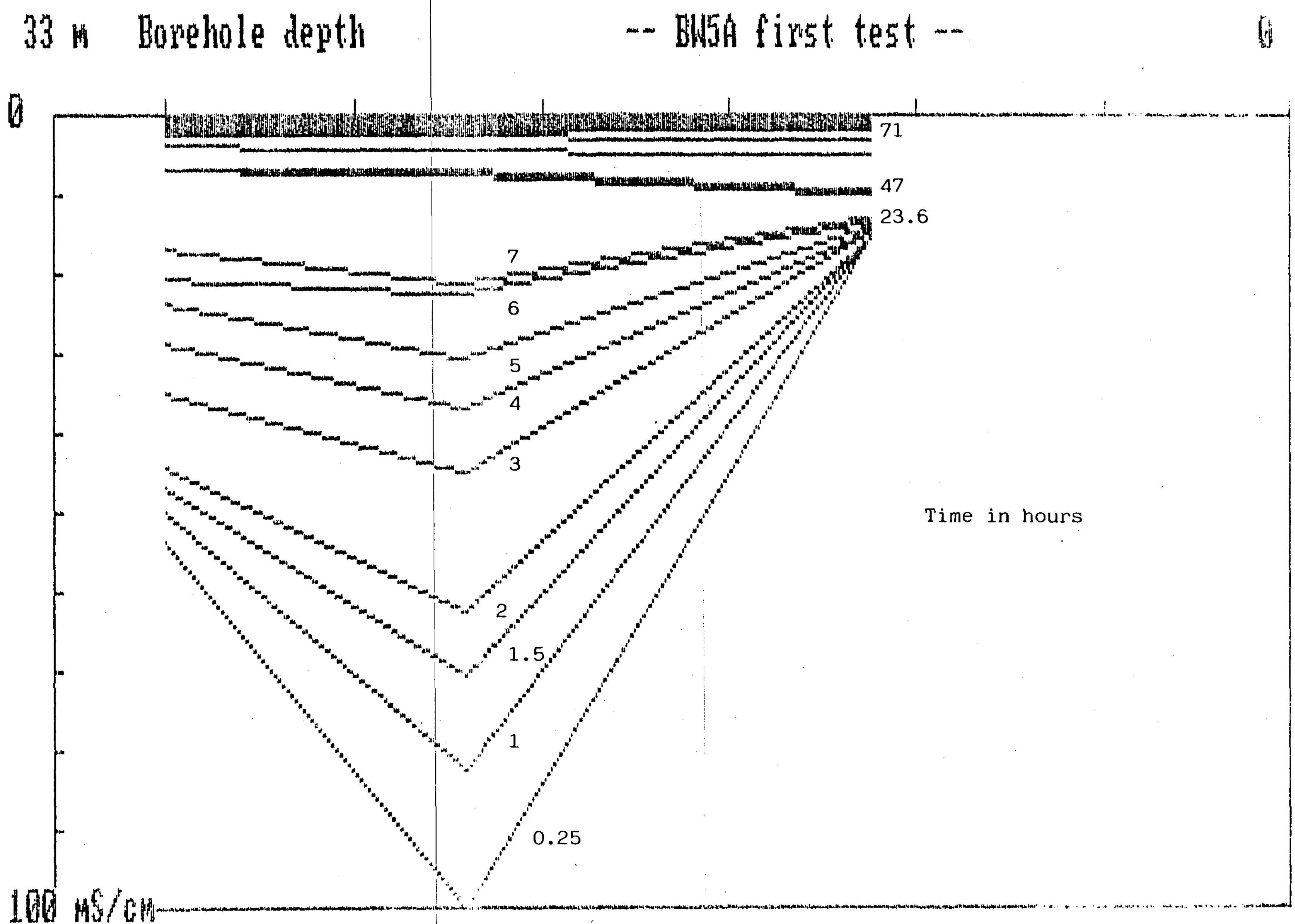
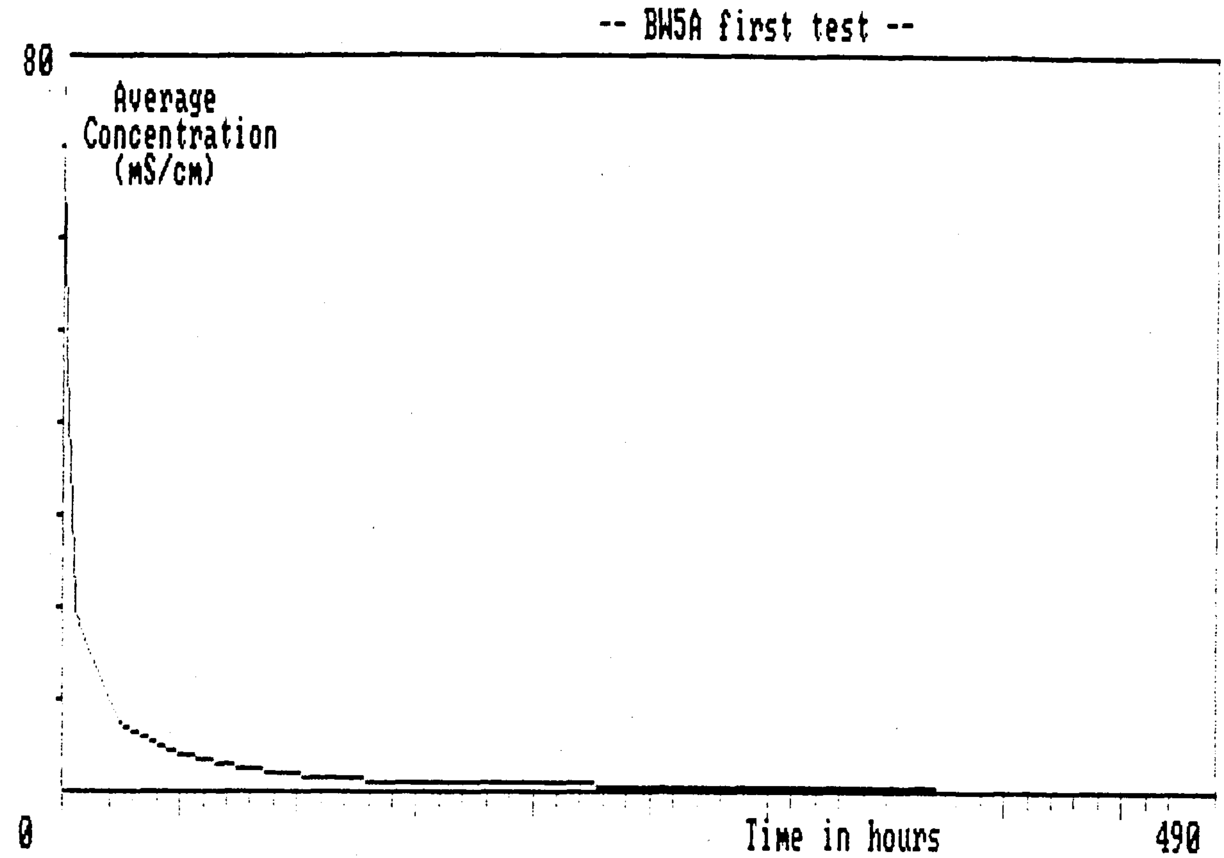


Fig. 7.5 Dilution of tracer material in borehole BW5A during the first test



Co = 71

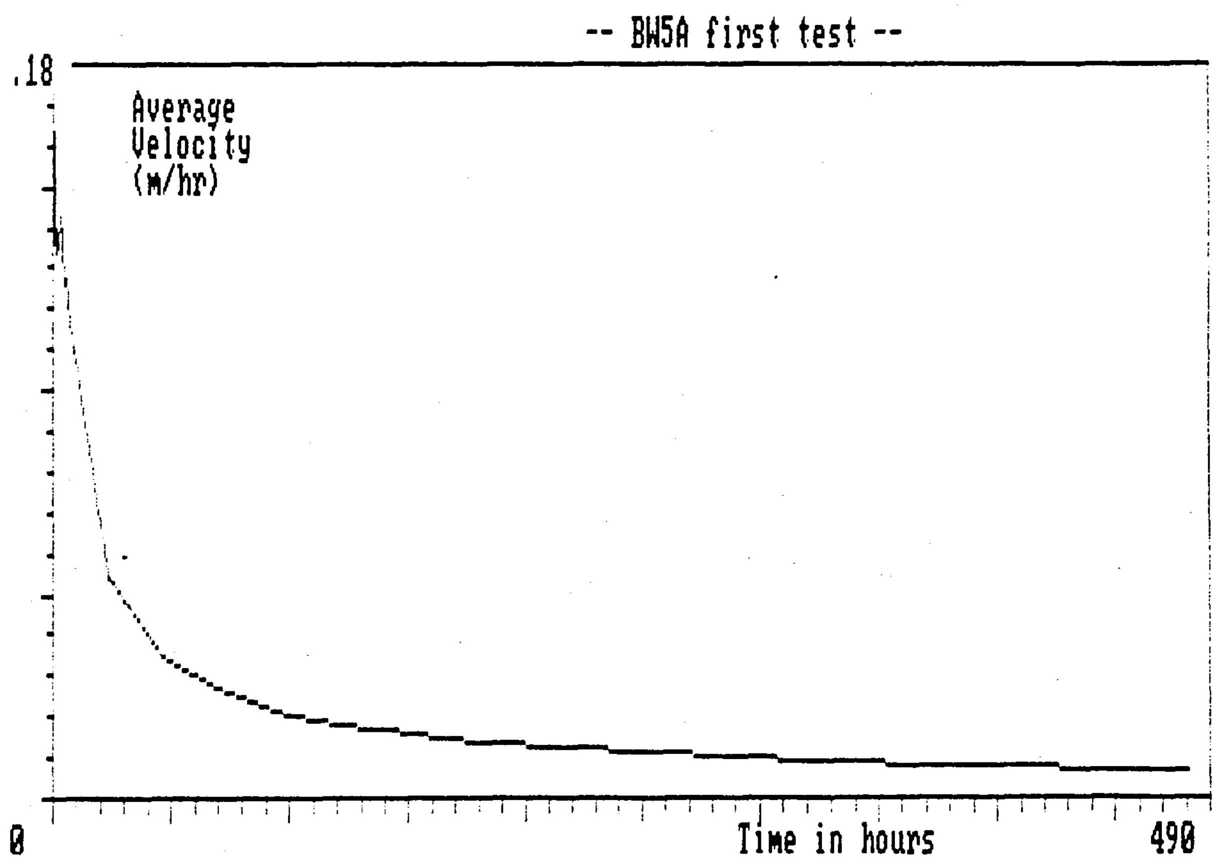


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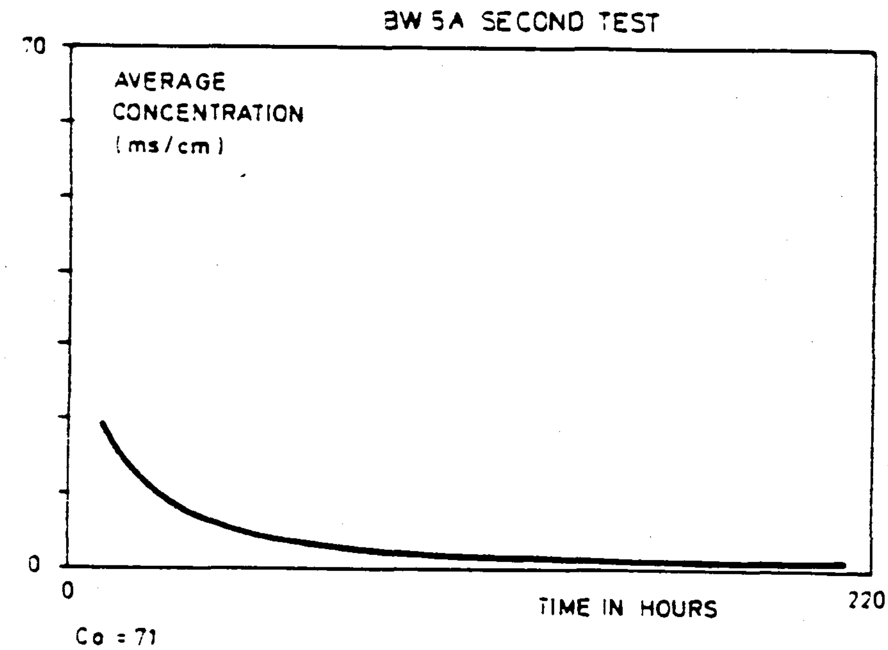
Fig. 7.6a Average concentration and groundwater velocity in borehole BW5A during the first test



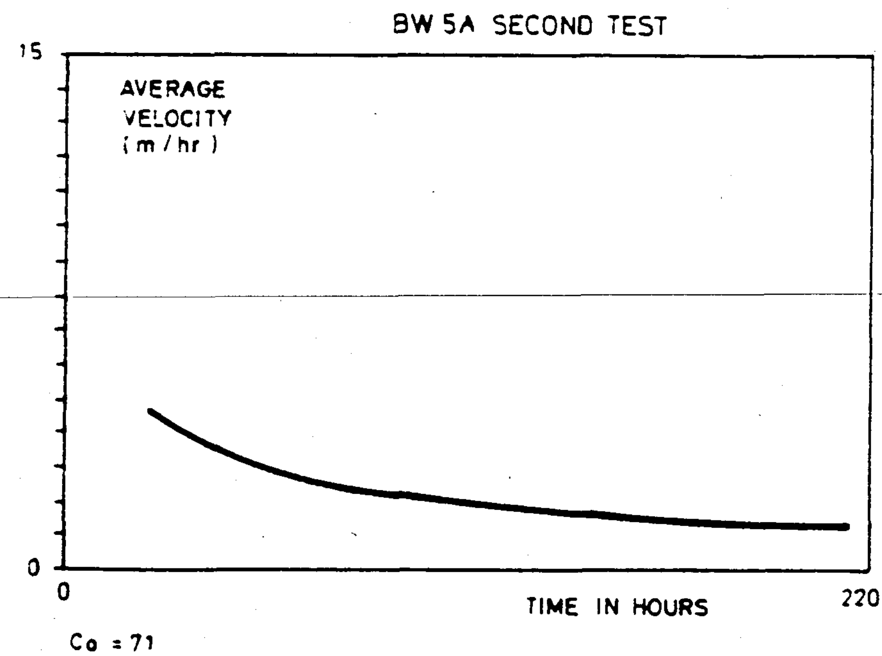


Fig. 7.6b Average concentration and groundwater velocity in borehole BW5A during the first test

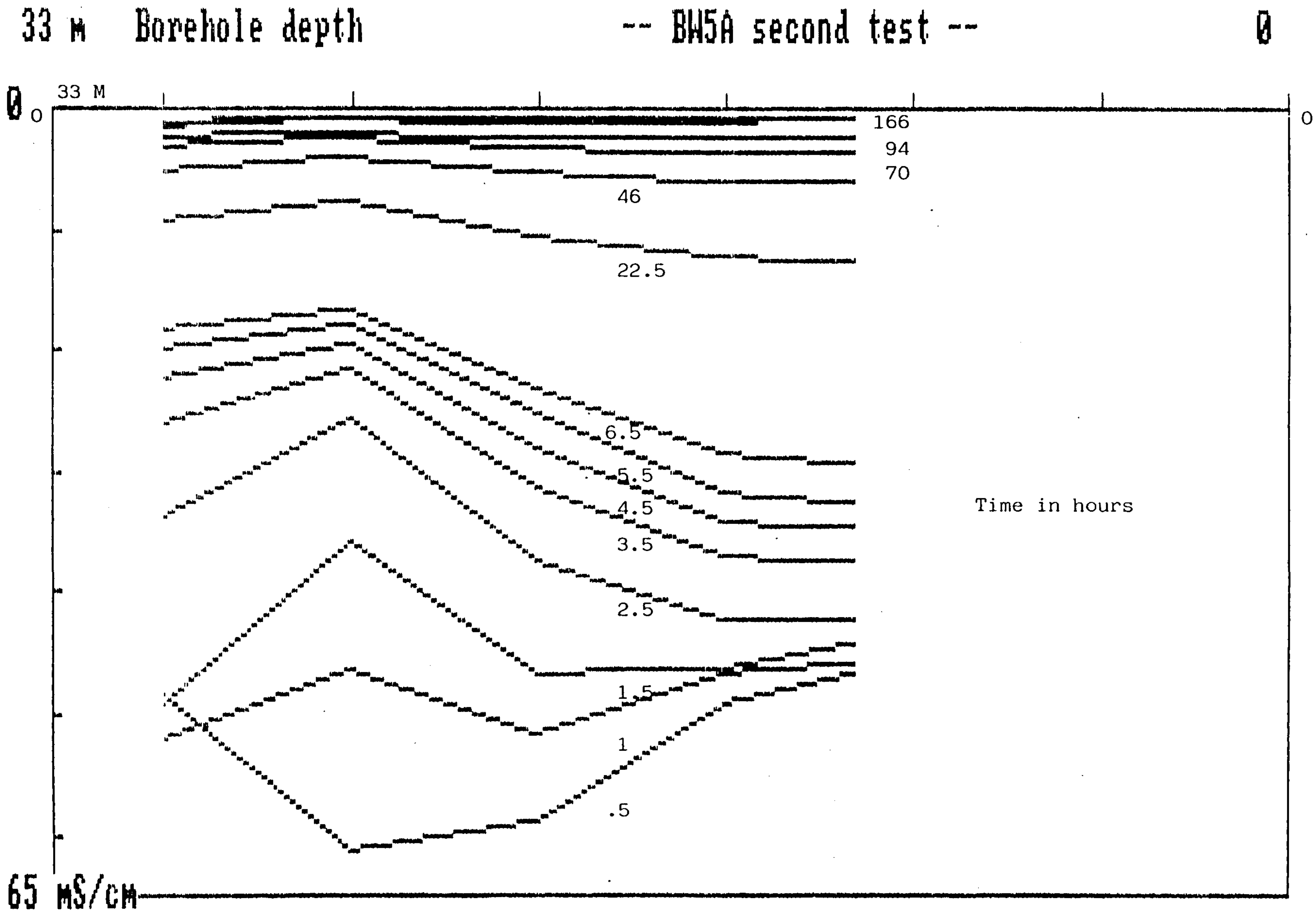


Fig. 7.7 Concentration versus distance

an initial concentration of C_0 = 71 mS/cm, and a porosity of n = 0.1, the groundwater velocities were calculated and are shown in figure 7.6 (bottom).

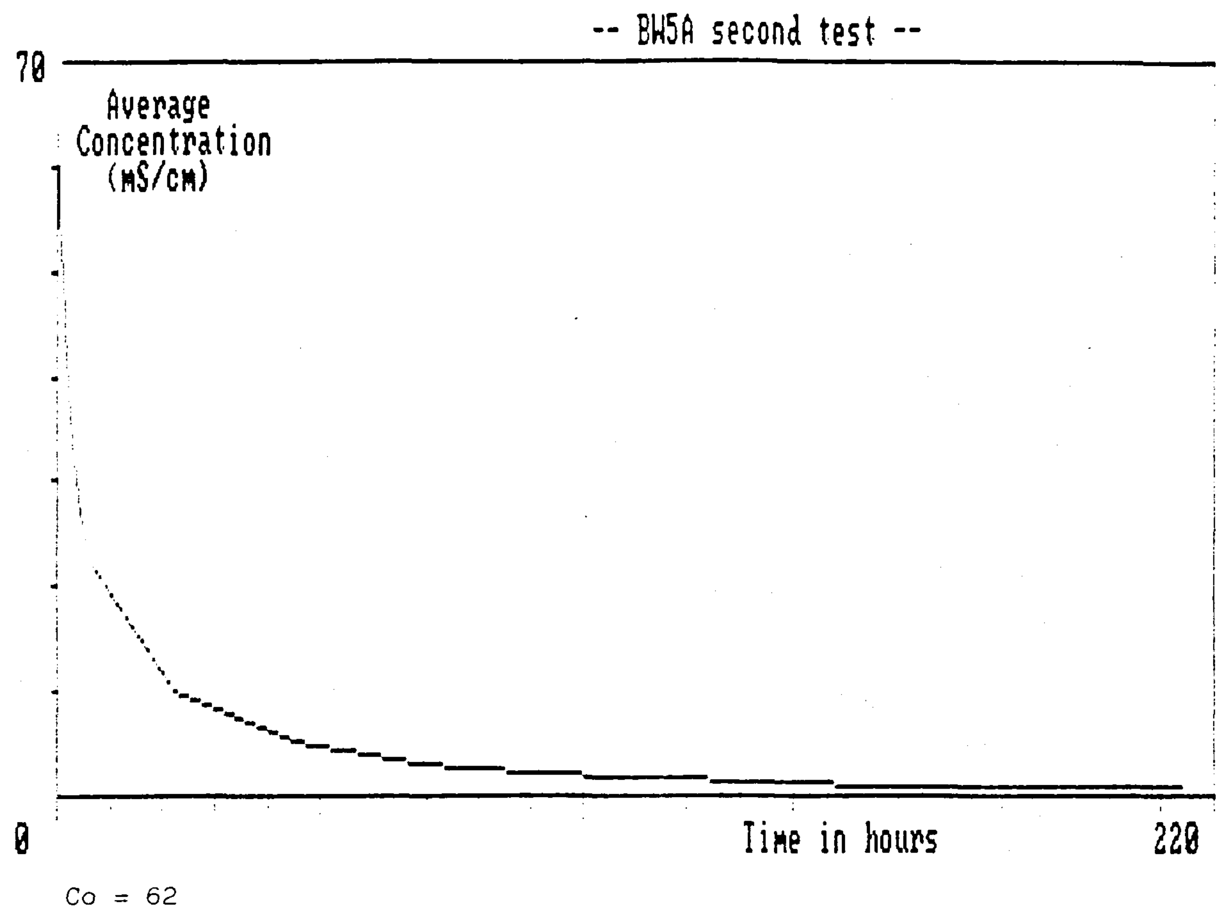
Although the initial concentration C_0 is essentially fixed by the amount of brine added to the borehole water, in practice there is a certain degree of uncertainty about its exact figure. This is caused by the injection procedure, which can take up to 15 minutes.

The choice of the initial concentration has a significant influence on the initial values of the calculated groundwater velocity. However, this influence tapers off rapidly. The choice of the porosity value is more difficult, as its influence has an immediate bearing on the velocity value due to its inverse proportionality.

The groundwater velocity calculated with the point dilution method reduces approximately exponentionally over time. This aspect will be discussed at the end of this chapter.

After the salt concentration had reduced to pre-test levels a second test was started in borehole BW5 on 25th June 1990. A more even distribution of the injected brine was obtained by gradually lowering a hose down the borehole while continually recharging the hose with brine on the other end. ± 150 l of brine (with a concentration of ± 210 gr of salt per liter pure water) was added to the approximately 370 liters of borehole water (± 0.14 mS/cm = ± 0.8 gr/l). Due to the permeability of the aquifer the water level in the borehole remained fairly constant. Therefore the brine-water mixture did spread out into the aquifer well beyond the confinements of the borehole.

The dilution process during this test is illustrated in table 7.2 and figure 7.7.



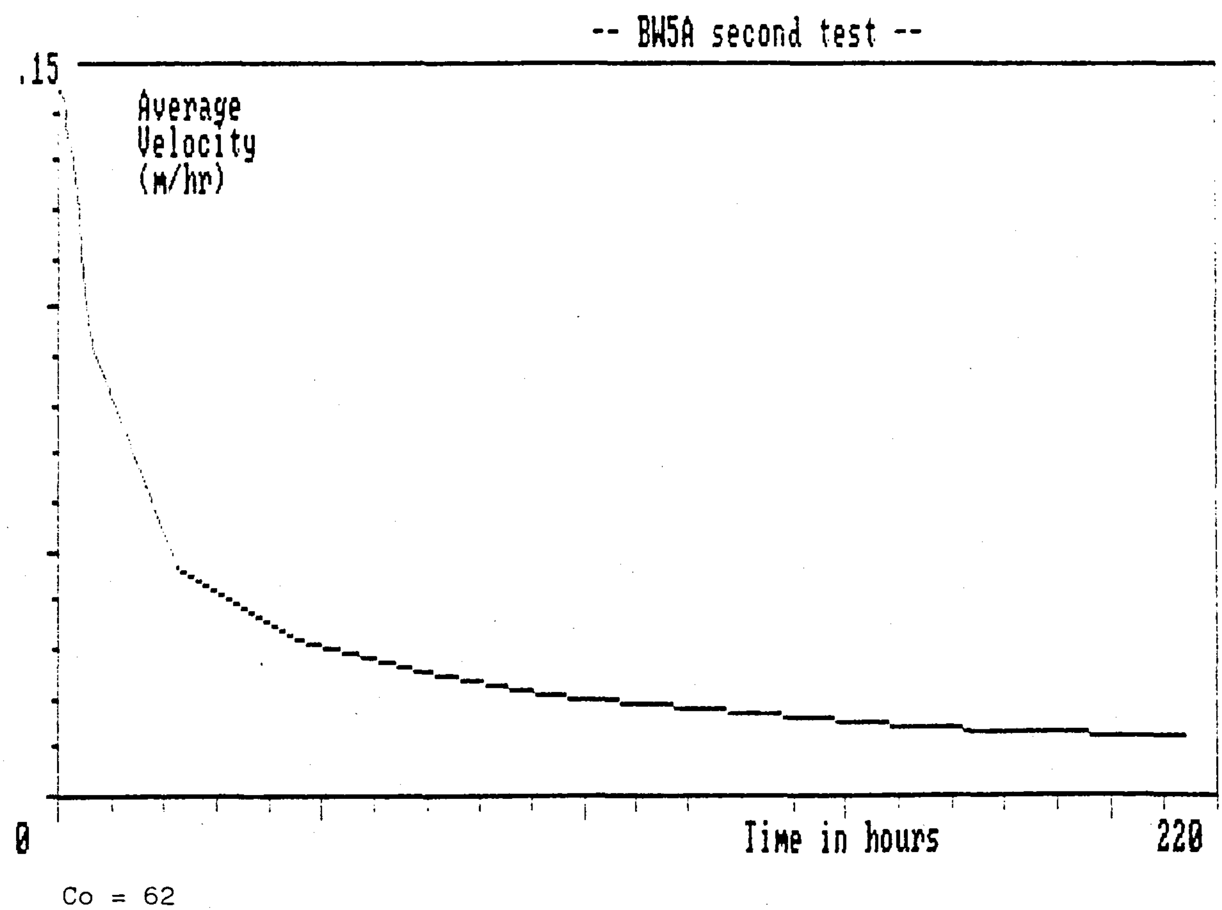


Fig. 7.8 Concentration versus time

Table 7.2 Observed concentrations during the second test in injection hole BW5A (mS/cm)

Time (hours)	Depth below surface (metres) 11.6	15	20	25	30	32.5	aver.	red aver.
t≤0	0.14	-	0.14	0.16	0.31	22.6	-	-
0.5	46.9	49.0	59.1	61.4	48.5	41.5	53.7	54.9
1.0	45.8	46.6	51.7	46.5	52.2	125.1	53.4	48.7
1.5	44.2	46.2	46.7	35.8	49.2	125.6	49.0	43.7
2.5	42.3	42.3	37.5	25.7	3.7	36.1	35.3	35.3
3.5	37.3	37.1	31.2	21.6	26.1	106.8	34.2	29.8
4.5	34.5	34.2	27.9	19.4	22.5	88.5	30.3	26.9
5.5	32.6	31.6	25.1	17.89	19.85	105.3	29.2	24.6
6.5	29.2	28.6	23.3	16.53	18.22	19.15	22.1	22.5
22.5	12.68	12.39	10.41	7.74	9.28	43.9	12.2	10.2
46	6.23	6.09	5.23	4.12	5.17	94.0	10.5	5.2
70	3.65	3.55	3.24	2.61	3.45	12.45	3.8	3.2
94	2.32	2.50	2.28	1.93	2.63	59.7	5.7	2.3
166	0.90	1.06	1.05	0.99	1.51	90.0	6.4	1.1
214	0.61	0.67	0.72	0.72	1.14	3.01	0.9	0.8

An hour after the start of the test the concentration became fairly evenly distributed, due to dilution around 20 metre below surface and an even stronger dilution around 25 m-. Around 30 m- the concentration has increased somewhat during the second half hour, presumably due to a downward movement of water with a higher density. After the first hour the dilution continues gradually and remains particularly strong around the 25 m- level. The reader is reminded that during drilling of BW5A the first major water carrying layer was encountered at ± 22 metres below surface.

The average concentration and the average groundwater velocity are given in figure 7.8. An initial concentration of C_0 = 62 mS/cm and a porosity of n = 0.1 was assumed. The results are very similar to those of the previous test as shown in figure 7.6.

The third tracer test was conducted in borehole BW3A and was initiated on 26 June 1990. This hole lies at the bottom of the catchment . Little dilution took place below the 20m- level (see table 7.3 and figure 7.9).

Table 7.3 Observed concentrations in injection hole BW3A (mS/cm)

Time (hours)	Depth below surface (metres) 5.5	10	15	20	25	30	aver.	red aver.
$t \leq 0$	0.18	0.34	0.24	0.21	0.53	0.63	-	-
0.25	28.8	33.9	43.8	36.1	65.3	79.1	46.9	40.4
1.0	22.8	31.1	32.8	29.4	62.8	134.4	47.4	34.2
2.0	22.0	27.7	27.2	23.2	64.0	78.6	38.8	30.4
3.0	23.1	27.8	25.0	21.0	65.2	117.6	42.2	30.0
5.0	21.6	22.7	19.6	17.5	63.8	79.2	35.1	25.7
23.5	10.07	9.68	8.23	7.70	63.7	84.1	27.6	15.8
47.5	4.88	4.78	3.95	3.81	62.0	88.1	24.6	11.7
72.0	2.74	2.76	2.30	2.36	60.4	81.0	22.3	9.9
144.0	0.89	0.89	0.76	0.79	57.5	91.6	21.7	8.1
192.0	0.49	0.56	0.47	0.50	56.3	94.7	21.5	7.7

Because the observations of the 25m level were used in the calculations the average concentration decreased more slowly over time (see figure 7.10, top). As a result the calculated groundwater velocity decreased more rapidly (see figure 7.10, bottom). For the calculation of the groundwater velocity an initial concentration Co = 43 mS/cm and a porosity n= 0.1 were assumed.

Ideally, the point dilution technique should provide us with a single value for the groundwater velocity. The reason that it doesn't in the present tests can most likely be attributed to 1) the type of tracer material used, 2) the type of aquifer studied, and 3) the length of borehole segment studied.

The calculated groundwater velocity is based on the values of two parameters, i.e. t and C(t). Rewriting:

$$V = \frac{W}{n\,\alpha\,A\,t}\ \ln\left(\frac{C_o}{C}\right)$$

and denominating $\beta = (V\ n\ \alpha\ A)\ /\ W$, yields $C(t) = C_0\ e^{(-\beta t)}$. Consequently, if the observed average concentrations deviate from a neat negative exponential function, the results will be erratic. In the present tests no combination of C_0 and β can be found that satisfies both the start and the tail of the observations.

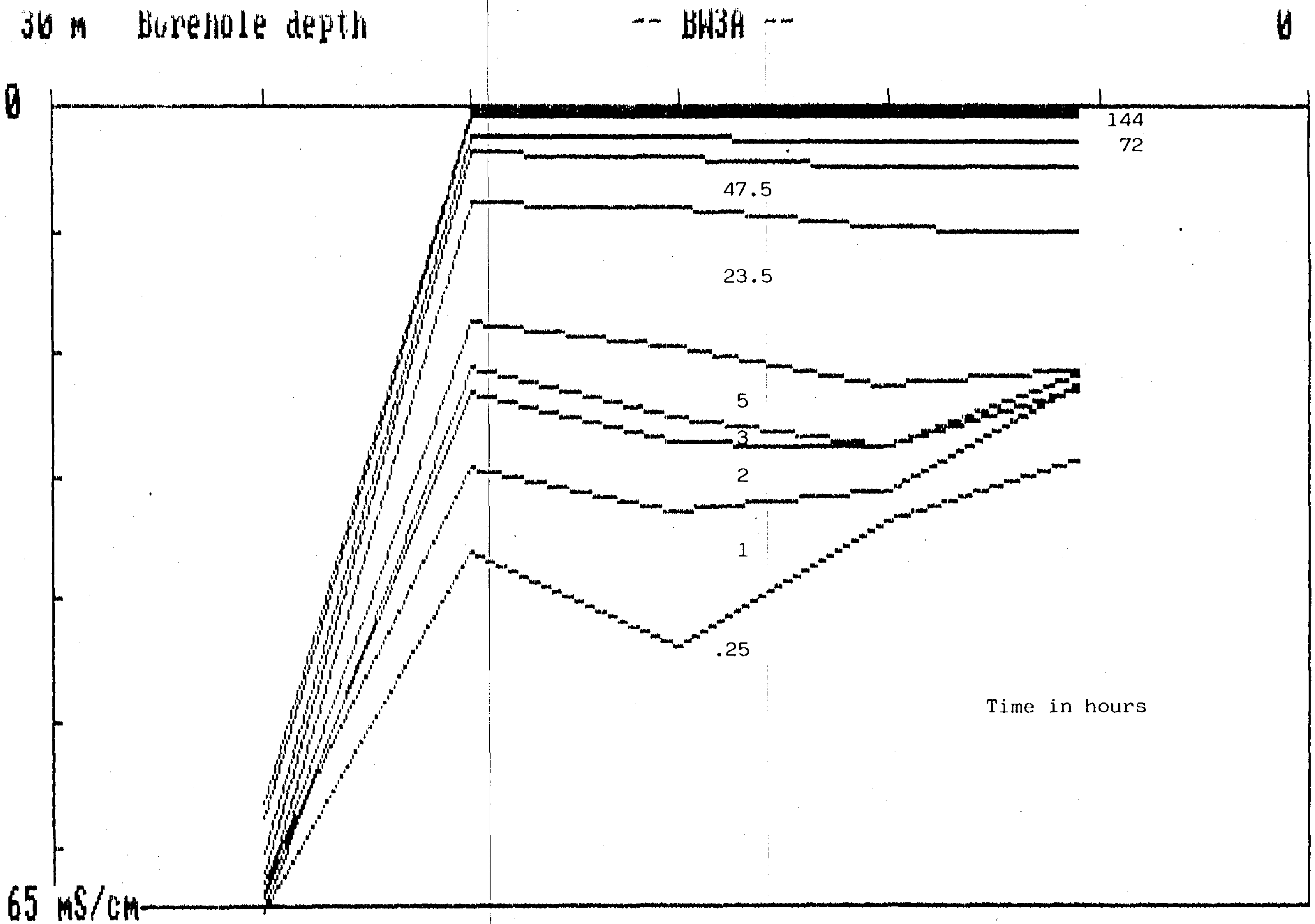
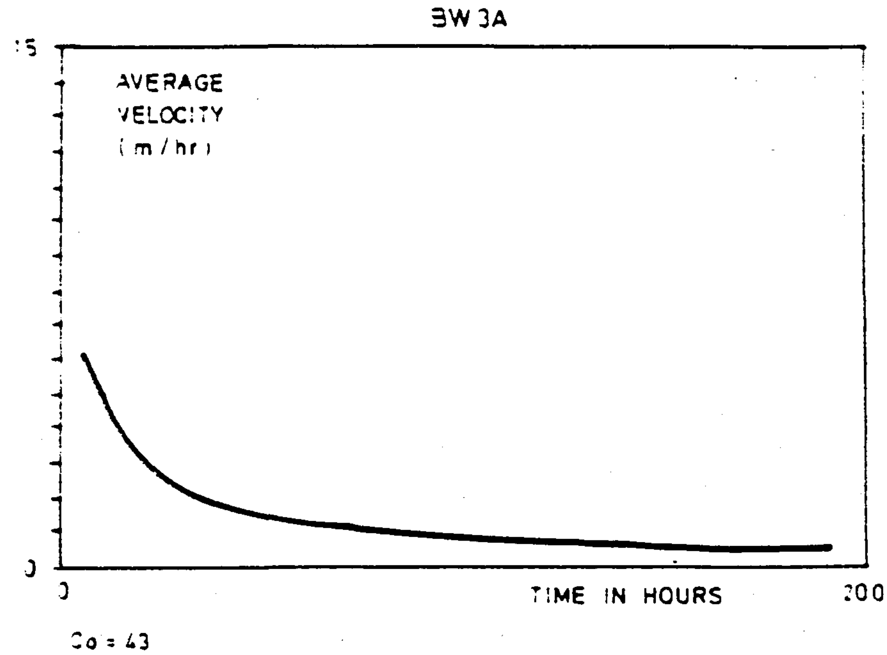


Fig. 7.9 Concentration versus distance



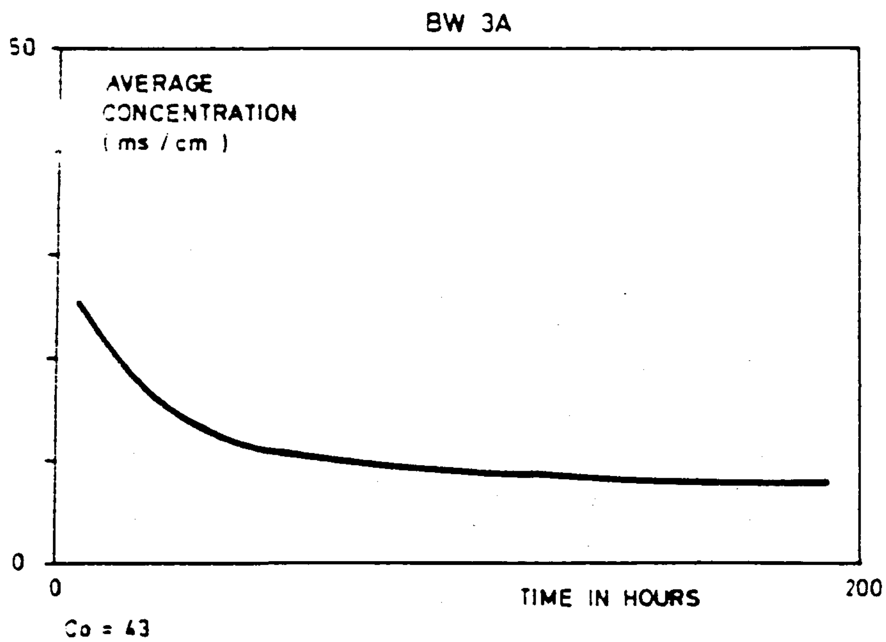


Fig. 7.10 Concentration versus time

It could be argued that the induced head is a major cause for the (initial) high velocity of the groundwater. In that case the lowest value would be most representative of the actual velocity. With this value (i.e. ± 0.005 m/hr) the groundwater runoff will be estimated.

The depths at which groundwater flow takes place ranges from between 12 m and 30 m below surface in BW5 and between 6 m and 20 m below surface in BW3A, giving an average aquifer thickness of approximately 16 metres.

For an aquifer width of approximately 400 m the groundwater runoff from the Waterval catchment can be estimated at $Q = W*h*v = 400*16*0.005*24 = 768\ m^3/day = 280$ Ml/yr

A lower limit for the groundwater runoff can be determined in the following simplistic way. In approximately 14 days the tracer concentration has dropped to its original level before the test. This means that the complete volume of brine (± 150 liters) has been washed away. Consequently, the average washout rate is 0.01 m^3/day for the width of the hole. If this value is extrapolated to the whole width of the aquifer one finds $(400/0.15)*0.01 = 30$ m^3/day or 10 Ml/yr.

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8 TRACER TESTS WITH INJECTION AND OBSERVATION HOLES

8.1 Introduction and theory

In this chapter the passage of a tracer cloud in the groundwater is studied which results from tracer injection some distance upstream. Advection and dispersion are both taken into account, while diffusion, which only has a significant influence at very low flow velocities, has been disregarded. In order to incorporate dispersion it was assumed that the fractured rock aquifer in the vicinity of the boreholes may be simplified and considered to be homogeneous and isotropic. The movement of tracer material is studied in a one-dimensional plane.

The one-dimensional form of the advection-dispersion equation is:

$$D \frac{\partial^2 C}{\partial x^2} - v \frac{\partial C}{\partial x} - \lambda RC = R \frac{\partial C}{\partial t}$$

where
- D = longitudinal dispersion coefficient (m^2/day),
- C = concentration (e.g. gr/m^3 or mS/m)
- v = pore velocity (m/day)
- x = distance from injection hole (m)
- t = time since start of injection (days)
- λ = rate of decay of tracer material (1/day)
- R = retardation factor (-), to account for dissolved or absorbed pollutant mass

For homogeneous, isotropic porous media the dispersion coefficient is given by $D = \alpha v$, where α = (geometrical) dispersivity (m), sometimes also referred to as the intrinsic longitudinal dispersion coefficient. Commonly α ranges from 1 to 50 metres.

The transport equation can either be solved analytically or numerically. The analytical solution requires a number of simplifications, such as homogeneity of the aquifer, parallel flow of constant velocity, constant retardation factor, reaction rate, and dispersivities. While a numerical solution method can handle more complex aquifer configurations, often the necessary information is not available to take advantage of its greater flexibility. In order to avoid the major disadvantage of numerical solution methods, namely numerical dispersion, an analytical solution was opted for.

Given a set of initial and boundary conditions the transport equation can be solved using the Laplace transform technique. The solutions often incorporate an expression with the integral of a negative exponential function, which cannot be solved analytically. This integral is known as the complementary error function, often abbreviated to "erfc". It is the complement of the error function or "erf". Thus :

$$erfc(x) = 1-erf(x) = 1-\frac{2}{\sqrt{\pi}} \int_0^x e^{(-t^2)} dt$$

For negative values of the argument the following definitions apply : erf(-x) = - erf(x) and erfc(-x) = 2 - erfc(x) = 1 + erf(x). Both functions are shown in figure 8.1.

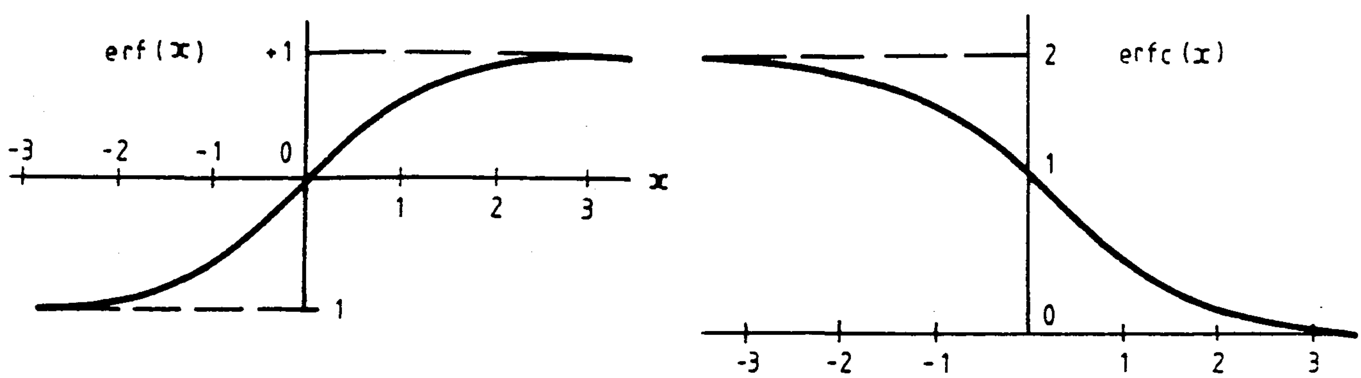

Figure 8.1 The error function and the complementary error function.

The values of the (complementary) error function for various arguments can either be read from a table or calculated numerically. An effective and accurate approximation is given in Abramowitz and Stegun (1972) :

$$erfc(x) = (a_1 y + a_2 y^2 + a_3 y^3 + a_4 y^4 + a_5 y_5) \exp(-x_2)$$

for $x \geq 0$ with $y = 1/(1 + px)$ and $p = .3275911$,
$a_1 = .254829592$, $a_2 = -.284496736$, $a_3 = 1.421413741$,
$a_4 = -1.453152027$ and $a_5 = 1.061405429$.
For $x < 0$ the identity $erfc(-x) = 2 - erfc(x)$ is applied.

A common combination of initial and boundary conditions describes the situation whereby the aquifer is initially free from tracer or pollutant material. Then, from t=0 a constant and permanent injection of material takes place. Thus:

$C(x,0) = 0$ for $x > 0$
$C(0,t) = 0$ for $t < 0$
$C(0,t) = C_0$ for $t \geq 0$
$C(\infty,t) = 0$ for all t

A solution to the transport equation under these conditions was given by Ogata and Banks (1961). Many authors have quoted this solution in its original version (e.g. Kinzelbach, 1986) and in simplified versions (e.g. Fried, 1975, and Bear and Verruijt, 1987).

In its simplest form with $\lambda = 0$ and $R = 1$ the solution is given by:

$$\frac{C(x,t)}{C_0} = \frac{1}{2}\, erfc\left(\frac{x-vt}{\sqrt{4Dt}}\right)$$

This version could be used when the pollutant has spread sufficiently far away from its source. This condition is satisfied when either the observation point (x) or the advective displacement (v*t) is large compared to the dispersivity (α), e.g. $x/\alpha > 10$ (Kinzelbach, 1986).

If this condition cannot be met a slightly more elaborate version of the Ogata and Banks solution could be used:

$$\frac{C(x,t)}{C_0} = \frac{1}{2}\, erfc\left(\frac{x-vt}{\sqrt{4Dt}}\right) + \frac{1}{2}\exp\left(\frac{xv}{D}\right) erfc\left(\frac{x+vt}{\sqrt{4Dt}}\right)$$

However, Javandel *et al.* (1984) report on a solution by Van Genuchten (1982), based on the same initial and boundary conditions, but with considerably different results:

$$\frac{C(x,t)}{C_0} = \frac{1}{2} erfc\left(\frac{x-vt}{\sqrt{4Dt}}\right) - \frac{1}{2}\left(1+\frac{vx}{D}+\frac{v^2 t}{D}\right)\exp\left(\frac{xv}{D}\right) erfc\left(\frac{x+vt}{\sqrt{4Dt}}\right)$$

$$+ \sqrt{\frac{v^2\ t}{\pi\ D}}\ \exp\left(-\frac{(x\ -\ vt)^2}{4\ D\ t}\right)$$

It was observed that the simulated progress of pollutant according to Van Genuchten is just slightly less than that according to Ogata and Bank's shortest version but considerably less than the more elaborate version of Ogata and Bank's solution.

If either $R \neq 1$ or $\lambda \neq 0$ the complete version of Ogata and Bank's solution has to be used:

$$\frac{C(x,t)}{C_0} = \frac{1}{2}\exp\left(\frac{x(v-U)}{2D}\right) erfc\left(\frac{Rx-Ut}{\sqrt{4DRt}}\right)$$

$$+ \frac{1}{2}\exp\left(\frac{x(v+U)}{2D}\right) erfc\left(\frac{Rx+Ut}{\sqrt{4DRt}}\right)$$

$$with \quad U = \sqrt{v^2 + 4DR\lambda}$$

Again, Van Genuchten's solution is somewhat more sophisticated due to more complex coefficients and the addition of a third term:

$$\frac{C(x,t)}{C_0} = \frac{v}{v+U}\exp\left(\frac{x(v-U)}{2D}\right) erfc\left(\frac{Rx-Ut}{\sqrt{4DRt}}\right)$$

$$+\frac{v}{v-U}\exp\left(\frac{x(v+U)}{2D}\right) erfc\left(\frac{Rx+Ut}{\sqrt{4DRt}}\right)$$

$$+\frac{v^2}{2DR\lambda}\exp\left(\frac{vx}{D}-\lambda t\right) erfc\left(\frac{Rx+vt}{\sqrt{4DRt}}\right)$$

A disadvantage of Van Genuchten's solution is the fact that separate equations have to be used to cover the situations where $\lambda=0$ and $\lambda\neq0$, as for zero decay the last equation will cause a division by zero.

The above equations all apply to the same boundary conditions, namely a constant and continuous source of some chemical, which starts discharging at t = 0. However, situations may arise that necessitate different boundary conditions. For some of these analytical solutions are available.

A Crenel-type injection refers to a constant discharge of chemical starting at t=0, as in the previous case, but being discontinued at $t = t_0$. Thus the boundary condition: "$C(0,t) = C_0$ for $t \geq 0$" is substituted by two new conditions i.e. "$C(0,t) = C_0$ for $0 \leq t \leq t_0$" and "$C(0,t) = 0$ for $t > t_0$".

Between $t = 0$ and $t = t_0$ the solution with the new conditions is identical to that for a continuous source. After $t = t_0$ an imaginary removal of the chemical takes place. Let any of the above solutions be represented by f(x,t). Then the solution for a Crenel-type input function is given by:

$$\frac{C(x,t)}{C_0} = f(x,t) \quad for \quad 0 \le t \le t_0 \quad and$$

$$\frac{C(x,t)}{C_0} = f(x,t) - f(x,t-t_0) \quad for\ t > t_0$$

If the duration of the chemical discharge is so short that it can be considered an instantaneous injection, the initial condition can be expressed by means of the Dirac-function. Kinzelbach (1986) gives as the solution to this case:

$$C(x,t) = \frac{\Delta M}{2wmnR\sqrt{\pi\alpha vt/R}} \exp\left(-\frac{(x-vt/R)^2}{4\alpha vt/R}\right) \exp(-\lambda t)$$

where ΔM = injected pollutant mass (mg),
w = width of one-dimensional aquifer (m),
m = depth of aquifer or thickness of saturated flow (m),
n = effective porosity (-)

With $\Delta M = M' * t$ (where M' is input rate of pollutant mass (mg/s), C_o = M'/ (wmnv), and D = αv, the above equation can be reformulated to :

$$\frac{C(x,t)}{C_o} = \frac{1}{2}\sqrt{\frac{v^2 t}{\pi DR}} \exp\left(-\frac{(Rx-vt)^2}{4DRt}\right) \exp(-\lambda t)$$

For R=1 and λ = 0 this expression resembles the third term in Van Genuchten's solution, but differs from it by a factor 1/2.

In the context of the random walk approach to solute transport by advection and dispersion both Kinzelbach (1986) and Bear and Verruijt (1987) give the following solution for a unit mass, injected at the point x = 0 at time t = 0:

$$\frac{C(x,t)}{C_o} = \frac{1}{2}\sqrt{\frac{1}{\pi Dt}} \exp\left(-\frac{(x-vt)^2}{4Dt}\right)$$

An extension of applicability of analytical solution methods is obtained by relaxing the constraint of constant input concentration. A field situation may occur whereby a buried source of pollution releases at an ever decreasing rate a particular chemical. This process could be translated to the initial condition $C(0,t) = C_0 \exp(-\beta t)$, where β is a constant. This case is also covered by the solution provided by Van Genuchten and illustrated in Javandel *et al.* (1984).

For the analysis of the present tests the analytical solution to the solute transport equation by Genuchten is used, as it is more generally applicable than the Ogata and Banks solution, and appears to be more accurate at short distances from the source as well as during the early stages of the tests. Retardation and decay of the solute are disregarded, while the injection is described by a Crenel-type function.

8.2 Application of an analytical solution to the tracer tests

Of the three tracer tests, which were started on 5, 25 and 26 June 1990, only one provided useful results. During the first test at BW5A and BW5 insufficient data were collected to describe the average concentration changes in the observation hole. The test at boreholes BW3A and BW3 posed the unfortunate problem that no concentration changes occurred in the observation hole. This is surprising since the pumping test showed the two boreholes to be clearly interlinked. This phenomenon can probably be explained by the presence of a highly irregular flow pattern in the vicinity of the holes due to a strong inclination of the water-bearing fractures.

The second test at BW5A and BW5 provided interesting and useful data (see table 8.1 and figure 8.1). Most of the tracer material enters the observation hole at a depth of between 25 and 35 metres below surface. Below 32.5 metres the tracer material is trapped as is evidenced by the initial concentrations at 32.5 m- and 35 m- which form the residue of the previous test at these holes. Probably due to the sampling technique some observations

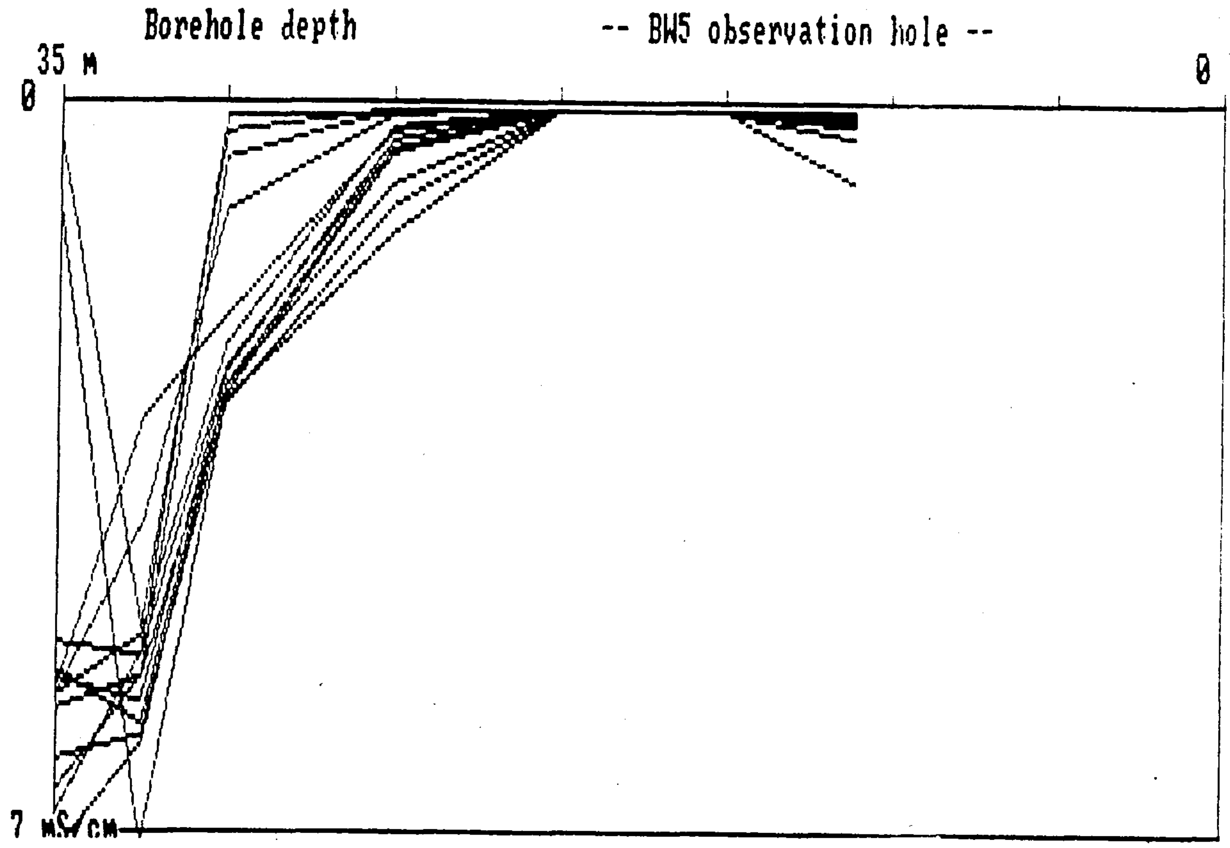

Fig. 8.1

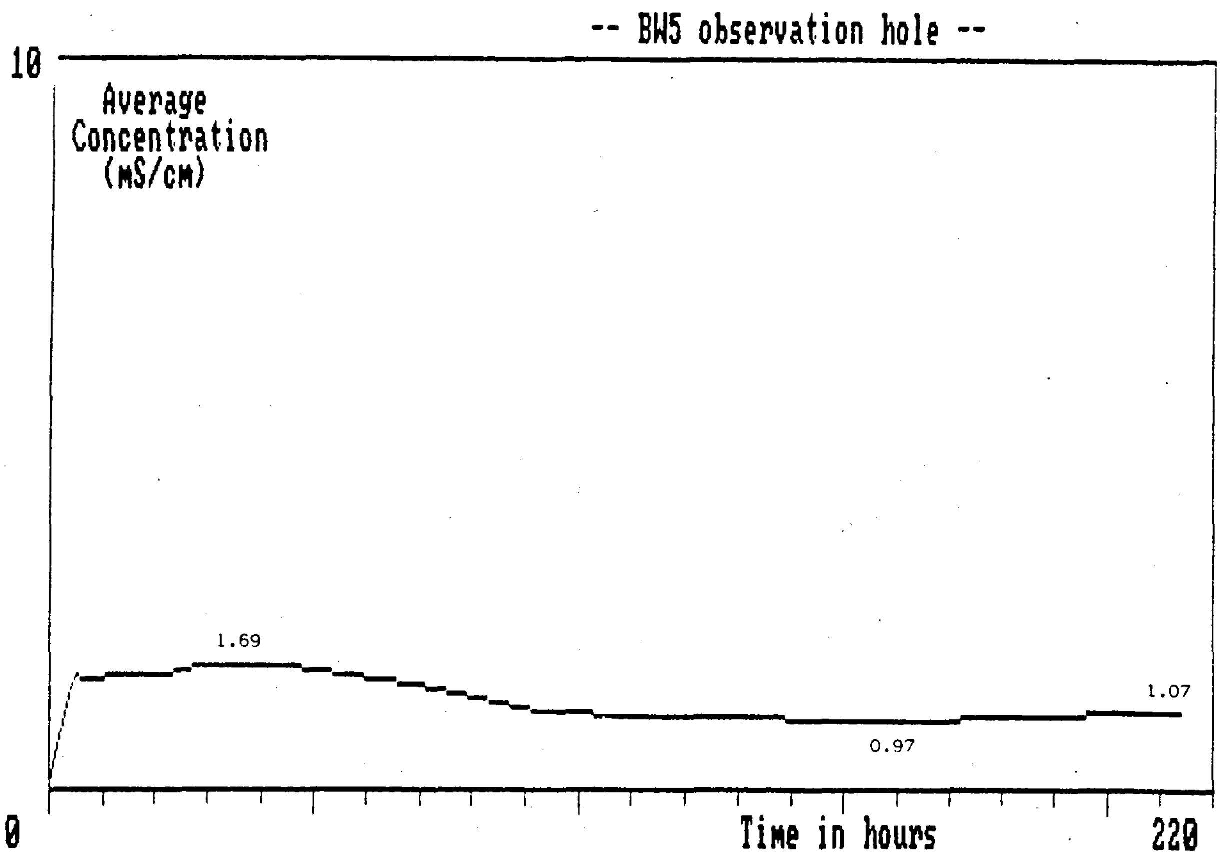

Fig. 8.2 Concentration observations at different depths over time (top) and the average concentration versus time (bottom) in observation hole BW5

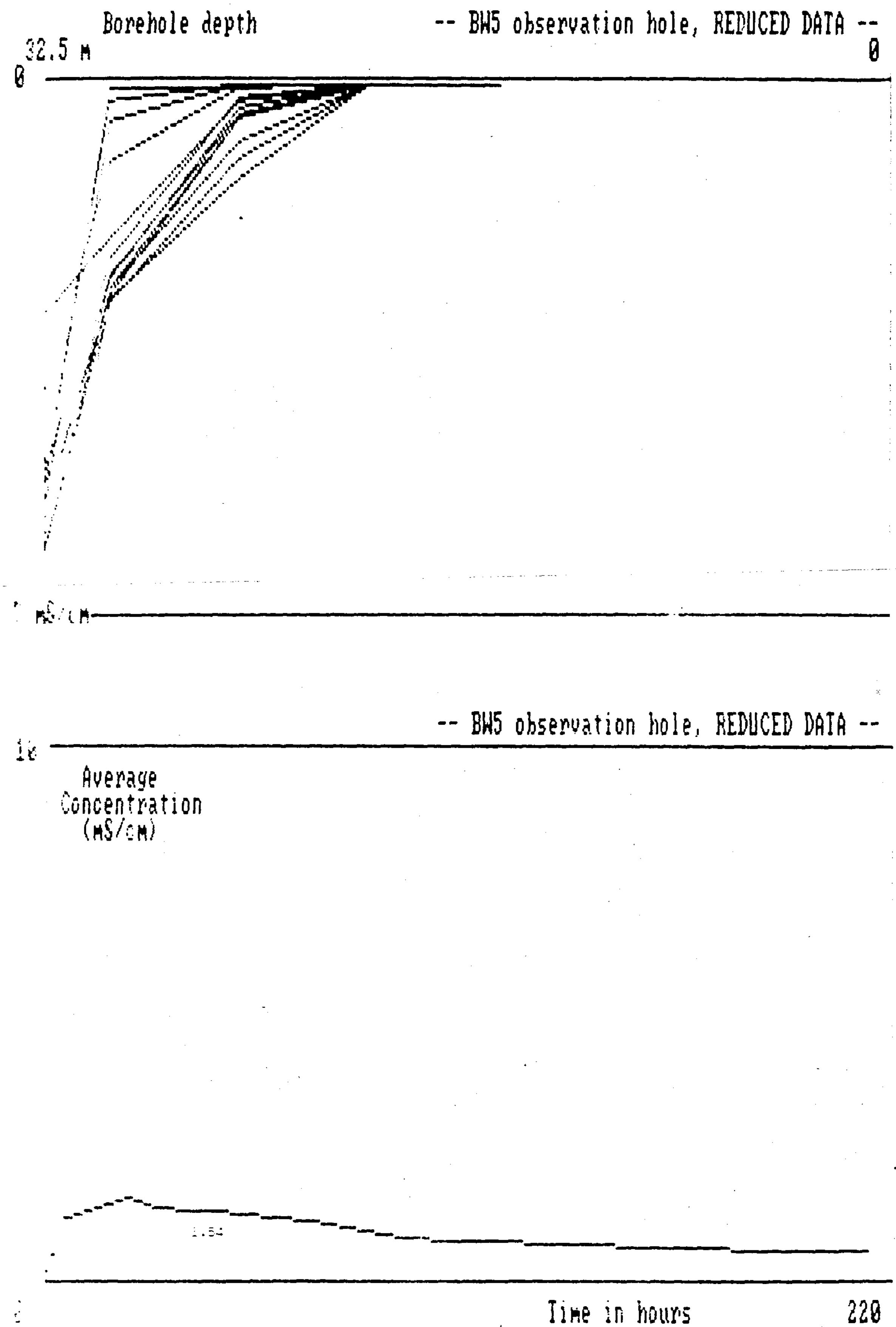

Fig. 8.3 Concentration versus distance (top) and concentration versus time (bottom)

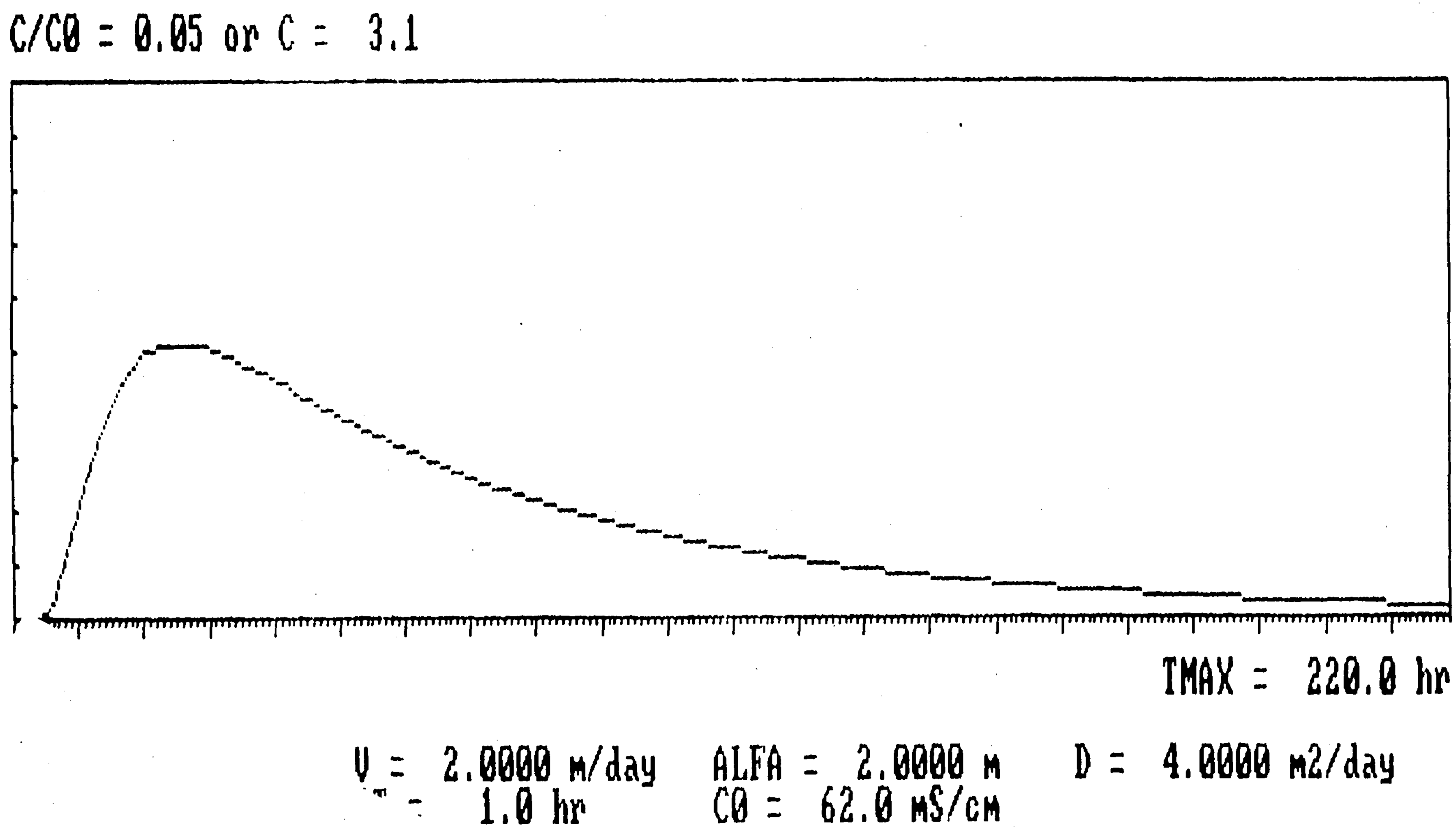

Fig. 8.4 Simulated concentrations in observation borehole BW5

It is interesting to observe that while the major deep level peak occurs approximately 23 hours after the start of the test, there is a smaller peak near the surface after ± 6 hours. This probably represents the tracer material which managed to find its way to the observation hole through the soil and decomposed granite on top of the solid granite. This possibility seems to be confirmed by the fact that at 15 m- and 20 m- the concentration remains at pre-test levels.

Figure 8.2 shows the average concentration versus time. The tail end as well as the overall average is distorted by the presence of the trapped tracer material at the bottom of the hole. Therefore a new average was evaluated excluding the information from the top level and bottom level (see figure 8.3).

Because the observation hole is close to the injection hole (4.1 m) and the concentration peak passes the observation hole after only ± 23 hours, the analytical solution as provided by Van Genuchten will be used to calibrate the simulation model. Due to the use of sodium chloride as a tracer, decay will be disregarded, while due to lack of information the retardation will be assumed to equal unity.

The criteria for calibration are the time and magnitude of the concentration peak as well as the behaviour of the "tail" or residual concentration after passage of the peak.

Both a higher pore velocity and a higher dispersivity will advance the peak, but while a higher pore velocity increases the magnitude of the peak, a higher dispersivity does the opposite. A longer duration of the injection period causes a higher peak. Finally, an increased dispersivity will lift the tail of the graph.

Since the duration of injection is essentially a fixed value there is a limitation to the degree the dispersivity can be increased, and thus the tail lifted. If it is realized that the observed tail is primarily made up by trapped tracer material a more rapid drop of concentration in the simulation is acceptable. A satisfactory degree of congruity is obtained with v = 2 m/day, α = 2 m and t_o = 2 hrs (see figure 8.4).

As with the point dilution method the total groundwater runoff from the Waterval catchment could be evaluated to be Q = w*h*v = 400*16x2 = 12 800 m^3/day = 4 672 Ml/year.

8.3 Results and conclusion

Considering that the total runoff from the Waterval catchment will average approximately 700 mm/yr * 70 ha = 490 Ml/yr, the artificial tracer tests give severely overestimated results. This is primarily due to the type of tracer material used, which requires a substantial surcharge resulting in a disturbance of the natural groundwater flow regime.

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Table 8.1 Observed concentration in observation hole BW5 (mS/cm)

Time (hours)	Depth below surface (metres) 11.1	15	20	25	30	32.5	35	aver.	red. aver.
t≤0	0.10	-	0.08	0.08	0.28	3.21	4.38	-	-
1.5	0.10	0.07	0.08	0.09	0.30	5.02	0.39	0.65	0.48
2.5	0.23	0.08	0.07	0.15	0.12	5.28	5.16	0.92	0.48
3.5	0.36	0.10	0.08	0.26	2.37	2.37	5.69	1.33	0.99
4.5	0.20	0.09	0.08	0.41	2.62	5.74	5.50	1.43	1.12
5.5	0.77	0.10	0.09	0.51	2.75	5.97	5.41	1.55	1.20
6.5	0.16	0.07	0.08	0.51	2.83	5.53	5.81	1.48	1.18
22.5	0.14	0.08	0.08	1.32	2.89	7.12	1.09	1.57	1.54
29.5	0.17	0.08	0.10	1.04	2.92	6.09	6.29	1.69	1.40
46.0	0.12	0.08	0.09	0.82	2.75	6.17	7.21	1.67	1.30
70.0	0.14	0.09	0.09	0.47	2.56	5.28	6.83	1.46	1.10
94.0	0.17	0.09	0.09	0.32	2.08	3.05	5.62	1.06	0.79
166.0	0.12	0.09	0.08	0.13	1.11	4.08	5.67	0.97	0.60
214.0	0.13	0.08	0.08	0.10	0.58	5.50	6.59	1.07	0.58

9 CONCLUSIONS AND RECOMMENDATIONS

This report contains both the background information and a description of a wide variety of tests conducted to obtain an understanding of the groundwater regimes in the Sunninghill and Waterval catchments.

The aquifers in these two catchments are underlain by old granite. The depth to base rock varies from zero at the top of the Waterval catchment to approximately 40 to 50 metres in some of the lower lying areas. The fresh base rock is covered by a layer of weathered granite. The degree of weathering increases towards the surface. In the lower sections of both catchments a top layer of fine hillwash covers the aquifer. The weathered granite is intersected by a highly irregular pattern of fissures, which seem to be the most important groundwater storage areas. The actual groundwater flow appears to take place closer to the surface, probably through the weathered granite. The influence of the many geological dykes depends on their degree of decomposition. Some form barriers, while others consist of stretches of increased permeability providing conduits to the groundwater.

It turned out to be virtually impossible to establish the groundwater runoff from the Sunninghill catchment due to a groundwater barrier at the lower end of the catchment. This barrier disrupts the groundwater flow in the already highly inhomogeneous aquifer. In addition to that it was found through geophysical research that faults and fissures are likely to intersect this barrier at topographically higher points.

The Waterval catchment, although equally inhomogeneous, appears to have a better defined outflow cross-section. Therefore, attention was focussed on evaluating the potential and actual groundwater velocity. Together, these parameters would provide the groundwater runoff. Interpretation of the pumping test results provided the most realistic groundwater runoff figure, i.e. ± 154 m^3/day or 54 Ml/year.

Table 9.1

Summary of estimated groundwater outflow from Waterval

Method	m^3/d	m^3/an	Remarks
Pump tests	154	56 000	
Tritium aging	274	100 000	
Salt injection	768	280 000	
Tracer -	12 800	4 672 000	Surcharge accelerated flow
Precipitation	1 370	500 000	For comparison only

Depletion of ground water :
Waterval maximum rate 1m/yr x 700 000m^2 x 0.25 = 180 000m^3/an (probably overestimated and nearer 100 000m^3/an). Therefore net input to Waterval aquifer = 300 000 - 100 000 = 200 000 m^3/an.

Estimated groundwater flow from Sunninghill :

(Groundwater levels constant over period)
Artesian flow over weir = 1 l/s = 100 000 m^3/an
Plus some flow along 2 fractures, say 10 000m^3/an
Total outflow from groundwater ≈ 200 000m^3/an

The tests involving artificial tracers proved less successful than anticipated, due to the type of tracer used. In order to obtain measurable results, relatively large quantities of brine were required. The resulting surcharge in the boreholes caused a forced spread of tracer material, which overshadowed the natural groundwater flow.

The study of environmental tracers, such as tritium, in groundwater projects will soon be a thing of the past. In the present study it proved useful, as it showed the presence of a stagnant water body, not affected by frequent recharge.

Time limitations prevented an analysis of the borehole water level records. If it were assumed that no seepage takes place along the casings an interesting correlation might be evaluated between rainfall and groundwater recharge. The drop in borehole water levels during the dry season, together with an averaged value for the aquifer porosity, will also provide an estimate of the groundwater runoff.

A P P E N D I X :

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