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Damien Addison

Resource Management Technical Report 203

Rural Towns Program

## Disclaimer

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# Summary

A groundwater study was carried out in the townsite of Bakers Hill. It aimed to accelerate the implementation of effective salinity management options. The study consisted of a drilling investigation and installation of a piezometer network, a pumping test, groundwater flow modelling and a flood risk analysis.

Piezometers were installed at 12 sites and one production bore was installed. At the sites drilled, bedrock was struck at depths between 10 and 25 m below ground level. A typical regolith profile consisted of a poorly-developed sandy laterite over white kaolin clay over yellow limonitic clay over fresh granite.

The groundwater levels in piezometers varied from around ground level (groundwater was discharging on the lower slope) to approximately 9 m deep below an upper slope site. Groundwater electrical conductivity values varied from 110 to 2,740 mS/m.

The constant rate pumping test was run for 24 hours and the water level in the production bore was still falling steadily at the end of the test. Drawdown effects were only localised: no effect was noted at a site 65 m from the production bore.

It is not known whether groundwater levels below the townsite are rising. It is essential that the monitoring network be measured frequently, regularly and over a long time period so that the salinity risk can be determined.

Recharge within the townsite zone (including the golf course and sports grounds upslope of the built-up area) was considered to be the principal source of groundwater below the town. There are opportunities to reduce townsite recharge immediately, and some of these would have additional benefits. Groundwater abstraction is not likely to be effective because the geological characteristics of the townsite would limit the zone of impact to small areas.

# Contents

# List of figures

# List of tables

# 1. Introduction and background

*Authors: Damien Addison and Muhammad J. Siddiqi (Agriculture Western Australia)*

The Rural Towns Program commissioned a groundwater study of the Bakers Hill townsite. It was part of a larger investigation (called the Community Bores Project) which covered 23 towns and aimed to accelerate the implementation of effective salinity management options.

The groundwater study for Bakers Hill consisted of a drilling program, piezometer installation and monitoring, a pumping test, groundwater flow modelling and a flood risk analysis. This report documents the background information for the town and its catchment (Sections 1.1 to 1.5) and the hydrogeological and flood risk investigations (Sections 2 to 4) and then recommends steps for managing the salinity issues of the town effectively (Section 5).

## *1.1 The town of Bakers Hill*

Bakers Hill townsite is about 70 km east of Perth (Figure 1-1) on the Great Eastern Highway. The district which includes the town has a population of approximately 270, according to the Australian Bureau of Statistics. The town is a service centre for people living in the surrounding rural areas.

The town boundary extends about 6 km from the town centre in all directions. Small rural landholdings and hobby farmers occupy most of this area. For the purpose of this investigation, only the town centre on the highway (Figures 1-2 and 1-3), was considered.

Observed symptoms of land degradation in the Bakers Hill area include increased salinity in Clackline Creek, rising watertables, waterlogging along drainage lines, hillside seeps, pavement failures on the Great Eastern Highway, occasional localised flooding and the emergence of salt tolerant vegetation. Several ponds constructed for aquaculture north-west of the town have become saline and a former vineyard was abandoned when the soil and water became too saline.

## *1.2 Description of the catchment*

Bakers Hill is in the Chitty catchment, drained by the Clackline Brook (Figure 1-2). This catchment occupies 3500 ha and extends from Bakers Hill to Clackline. The subcatchment area that contains the town centre occupies only 45 ha.

The town is built on a gentle slope (Figure 1-3). The slope dips to the north-west at about 5 per cent. There is a golf course above the town, on the upper slope, and a small paddock below the town, on the lower slope and valley floor. There is approximately 40 m difference in elevation between the valley floor and drainage divide. The golf course is well vegetated and occupies approximately 25 ha.

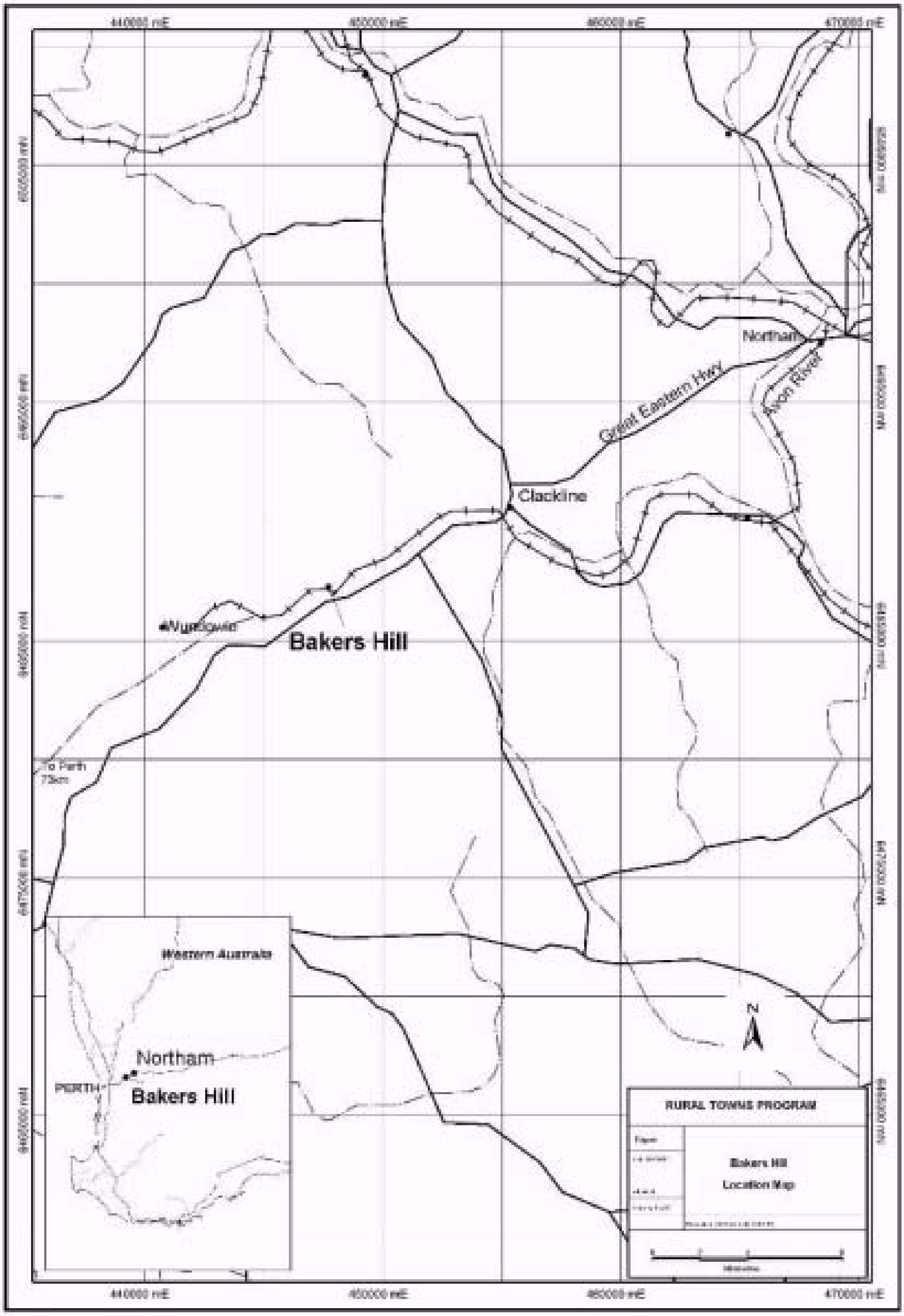

Figure 1-1. *Regional setting of Bakers Hill townsite*

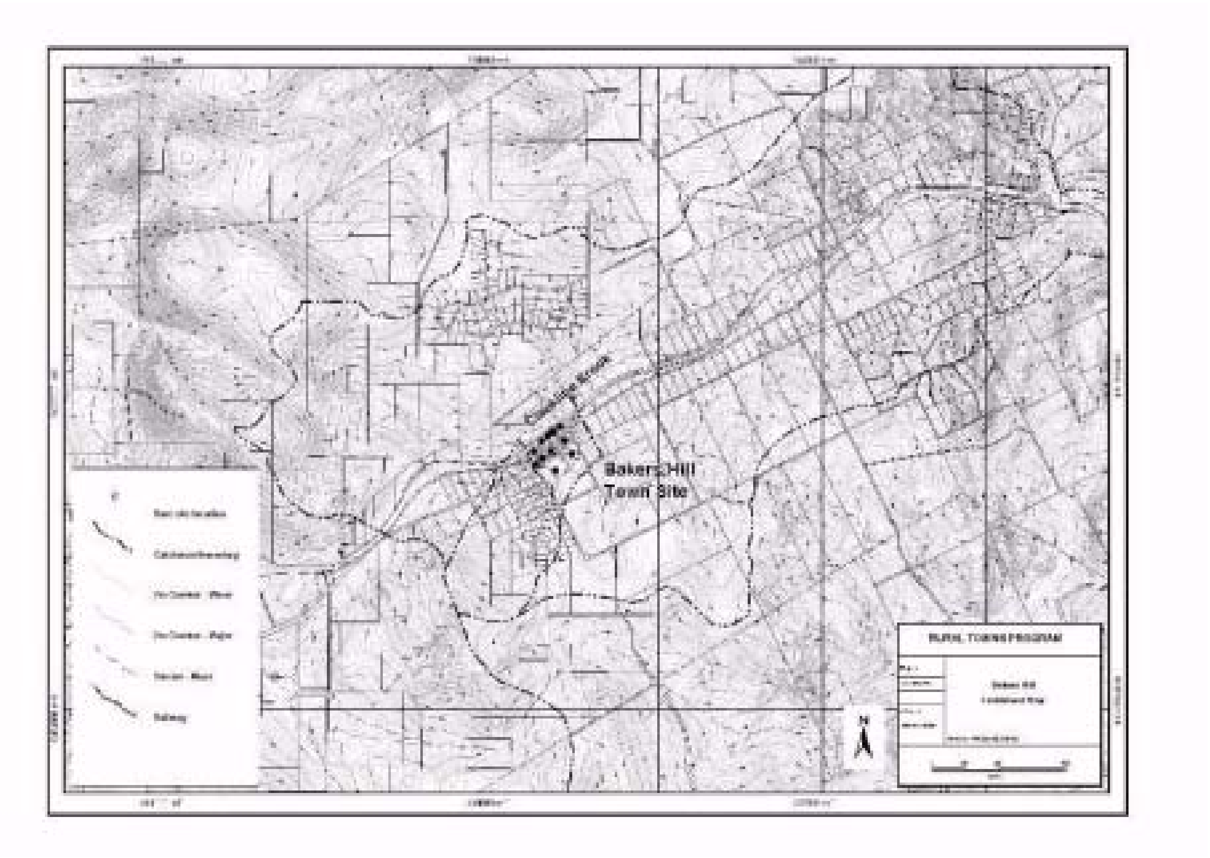

*Figure 1-2. Location of the Bakers Hill townsite in the Clackline Brook catchment*

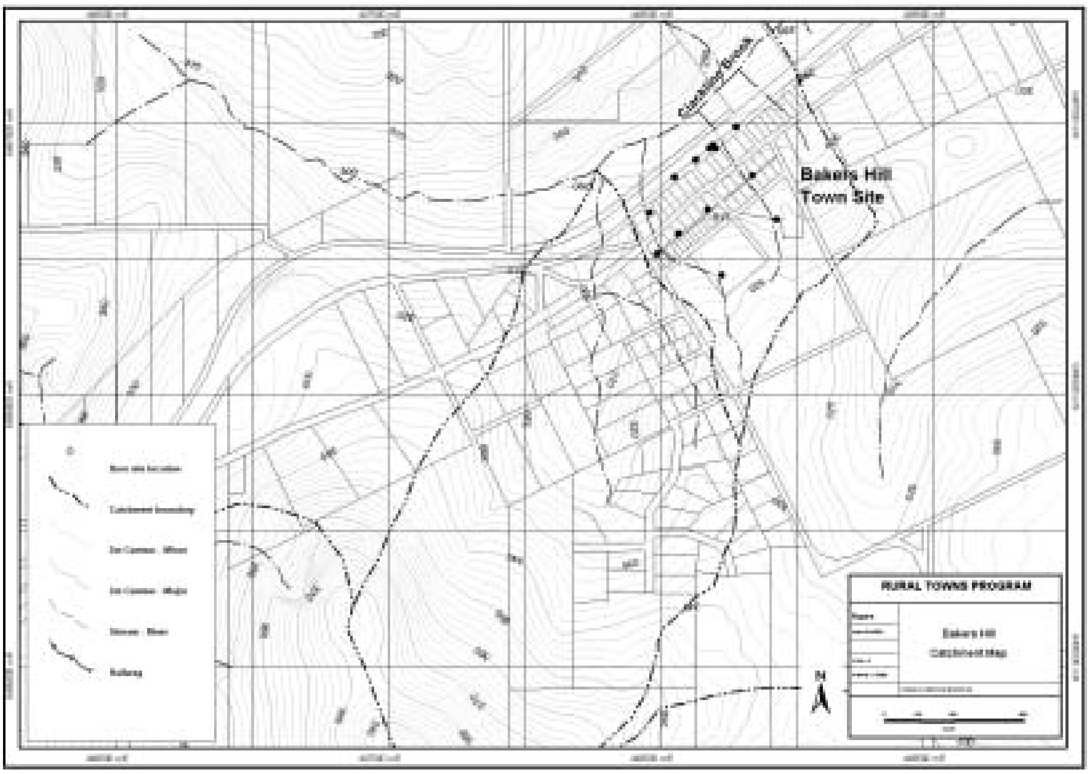

*Figure 1-3. Location of the Bakers Hill townsite (detail)*

## 1.3 Geology

Bakers Hill townsite overlies granitoid bedrock (Wilde and Low 1978). Exposures around the townsite show that the regolith consists of residual clay and some poorly-developed laterite.

## 1.4 Climate

Bakers Hill has a temperate climate as defined by the Köppen classification system (Bureau of Meteorology 2000a). It experiences distinctly dry (and hot) summers. Most of the rain is received during the winter months but summer storms occasionally contribute significant amounts of rain. The average annual rainfall is 605 mm (Bureau of Meteorology 2000b).

Mean daily maximum temperatures range from 31.9°C in January to 15.1°C in July. Mean daily minimum temperatures range from 16.1°C in February to 6.3°C in August.

## 1.5 Hydrogeology

No hydrogeology studies have previously been carried out in the town. Groundwater investigations carried out following clearing for agriculture on a CSIRO research station about 1.5 km south of Bakers Hill documented sharp rises in groundwater levels below sites high in the landscape, as well as the spread of salinity along valley floors (Peck *et al*. 1979). This implies that groundwater systems may be localised and not well-connected to systems lower in the landscape.

## 1.6 Drainage

The Bakers Hill town subcatchment drains to the north-west into the Clackline Brook, an ephemeral tributary of the Avon River (Figure 1-1). Surface water is managed with a combination of open and piped drains that discharge into natural waterways. There is no harvesting of surface water.

There are two subtle natural drainage lines that pass through the main part of the town (Figure 1-3). They both drain from the golf course to the north-west.

## 1.7 Town water supply and disposal

The Water Corporation pipes water supplies into the town. All properties use septic systems.

# 2. Hydrogeology investigation

*Authors: Damien Addison (Agriculture Western Australia) and Fay Lewis (Fay Lewis Consulting)*

The hydrogeology investigation aimed to determine which salinity management options would be most effective in Bakers Hill. This section of the report describes the methods and results of the drilling program, the installation of a groundwater monitoring network and the pumping test, and then presents an interpretation of the groundwater systems affecting the townsite and a discussion of options for managing shallow groundwater. Some of these options were then explored further using a groundwater flow model (Section 3).

## *2.1 Pre-existing information*

Residents mentioned that several boreholes had been drilled for the purpose of supplying garden water, but in every case, the water produced was saline and very small in volume. There is no profile information available for these holes.

The local primary school has measured the salinity of surface water at certain locations around town. The data collected so far is insufficient to establish any trends.

## *2.2 Method*

Twenty piezometers and one production bore (00BH13I) were installed at 13 sites (Figure 2-1 and Table 2-1).

### 2.2.1 Drill site selection

Preliminary drill hole locations were planned with the use of aerial photographs and site inspections. Access to and availability of land restricted the number of potential drill sites. All final drill sites are on some form of public land.

Three approximate transects were planned at various levels across the slope (Figure 2-1). The lowest transect is north-west of the highway on a gravel road that runs behind the commercial blocks of Bakers Hill. The next transect up the slope is south-east of the highway in an alley. The uppermost 'transect' contains only two holes and is located above the town on the golf course. This represents the uppermost reaches of the town catchment.

The holes were also planned to identify the primary lithologies represented in the area. Inspection of rock exposures around the town indicated that these included granite, a quartz-rich, ferromagnesian-poor phase of the granite, and a mafic unit.

A 'public awareness piezometer' was placed in the information bay and a 'school piezometer' was placed on the golf course directly behind the school.

Choosing a location for the production bore was made difficult by the overall low-yielding nature of the clay soils. The final site chosen was selected for the relatively high localised piezometric pressure (the nearby deep piezometers had high water levels), relatively deep weathered profile and suitability for drilling. Some areas within the Bakers Hill townsite were almost impossible to drill with the available equipment due to soft puggy clays. Any attempt to construct a bore in these areas was likely to fail.

*Figure 2-1. Groundwater level depths (metres below ground) and groundwater electrical conductivity (EC, in milliSiemens per metre) for piezometers on 3 August 2000, and locations of the cross-sections in Figures 2-2 to 2-5*

### 2.2.2 Drilling methods

LA Boyle Drilling Pty Ltd carried out the drilling with a small truck-mounted multipurpose rig. Reverse-circulation-percussion methods were used to drill 100 mm-diameter holes for the construction of piezometers, and a combination of rotary-air-blast and rotary-mud methods was used to drill a 200 mm-diameter hole for a production bore.

Due to the poor ground conditions, most holes collapsed before the casing could be run back down the hole. When this happened, rods were run back down the hole to hold it up while the casing was run down the inside of the rods. This made the installation of the gravel pack and the bentonite seal very difficult.

### 2.2.3 Piezometer and production bore construction

All piezometers were constructed with 50 mm-diameter PVC casing. Each piezometer has an end-capped 2 m-length of slotted (0.5 mm-wide slots) PVC at the bottom of the hole. The annulus around this intake section was packed with 16 x 32 gravel pack (about 0.6 to 1.2 mm-diameter grains) to 2 m above the intake section and then the annulus around the pipe above was sealed with bentonite (depths of intake sections are listed in Table 2-1). Each hole was then back-filled to the surface with drill cuttings.

The production bore was constructed with 127 mm-diameter class 6 slotted PVC casing. It was gravel-packed with 16 x 32 gravel to the surface and sealed with cement at the collar.

A galvanized iron standpipe was installed over every piezometer and the production bore. The standpipes were set in concrete and have lockable caps.

### 2.2.4 Drill sample analyses

The drilling method produced bulk samples from a cyclone for every metre drilled. These samples were temporarily stored in large plastic bags until the hole was completed.

Bulk samples for the deepest hole drilled at each site were subsampled. Chip trays with a small sample from each metre drilled were prepared. These samples have been stored in the South Perth compound of Agriculture Western Australia.

Detailed logs of each site drilled are available at <http://www.agric.wa.gov.au/environment/links/RMtechreports/>.

### 2.2.5 Groundwater sample analyses

Groundwater levels were measured and water samples collected on a monthly basis. Electrical conductivity (EC) values of samples were measured in Perth laboratories of Agriculture Western Australia. Results are stored on the Agriculture Western Australia AgBores database.

### 2.2.6 Surveying

A global positioning system was used to determine locations and elevations of piezometers and the production bore (Table 2-1). The accuracy was assumed to be similar to the ±30 mm horizontal and ±40 mm vertical accuracies achieved in surveys for other towns in the Community Bores Project.

### 2.2.7 Pumping tests

Multi-rate and constant-rate pumping tests were carried out by Test Pumping Australia to establish aquifer parameters. The test methods are described in Appendix 1.

## 2.3 *Results*

### 2.3.1 Profile descriptions

Detailed drill logs are available at <http://www.agric.wa.gov.au/environment/links/RMtechreports/> and the cross-sections in Figures 2-2 to 2-5 illustrate the profiles.

At most sites drilled in the town, the regolith material was between 15 and 25 m thick.  A typical profile contained a poorly-developed sandy laterite over white kaolin clay over yellow limonitic clay over fresh granite.  The clays were very puggy (i.e. sticky and plastic) and contained very little residual quartz.  The transition from limonitic clay to fresh granite was usually very sharp.  In some locations, there was a mottled zone present.

At the top of the town catchment on the golf course, the regolith was much thinner than elsewhere below the town (approximately 10 to 15 m).  In this area, clays were less well-developed and saprolitic textures were preserved.

A few profiles over granite contained relatively high amounts of residual quartz. These sites coincided with subtle topographic and bedrock highs.

### 2.3.2 Groundwater data

Groundwater level depths for 3 August 2000 are listed in Table 2-1 and the variation across the townsite is illustrated in Figure 2-1.  The watertable level varied from around ground level (groundwater was discharging on the lower slope) to approximately 9 m deep on the upper slope (Figure 2-1).  Electrical conductivity values varied from 110 to 2,740 mS/m.  Most were between 1,000 and 2,000 mS/m (Figure 2-1).

Site visits and drilling were undertaken during winter, and it was noted that waterlogging was associated with a thin layer of sandy and gravelly lateritic soil at sites both upslope and downslope of the Great Eastern Highway.  The water involved was considered to be perched and unrelated to the main groundwater system at sites where groundwater in piezometers was several metres below the surface.

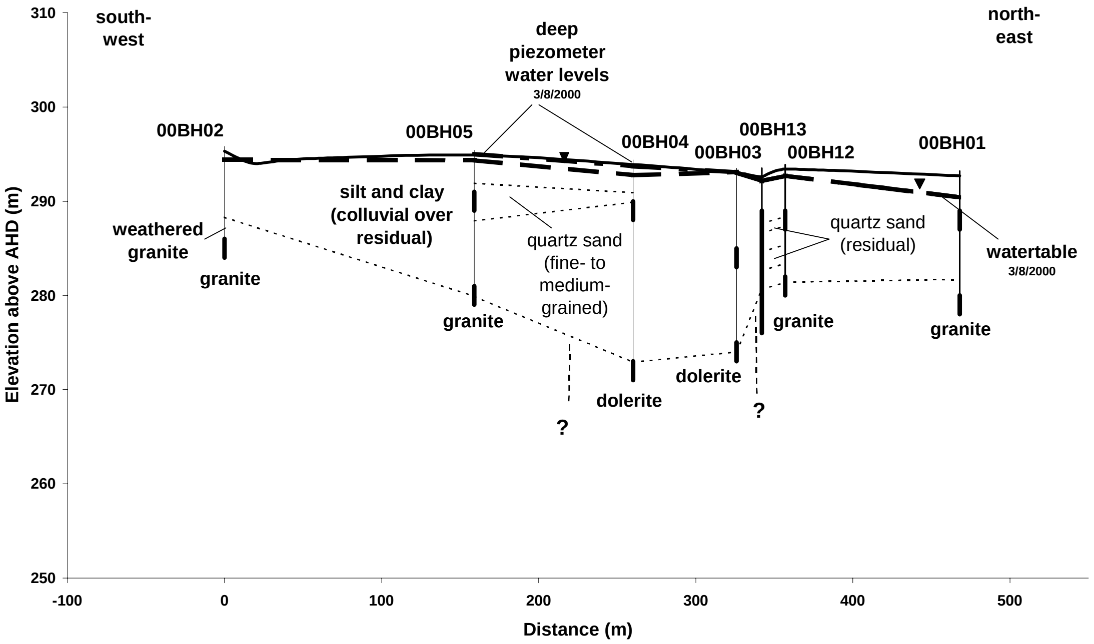

*Figure 2-2. South-west to north-east cross-section from bores 00BH02 to 00BH01 (see Figure 2-1 for location)*

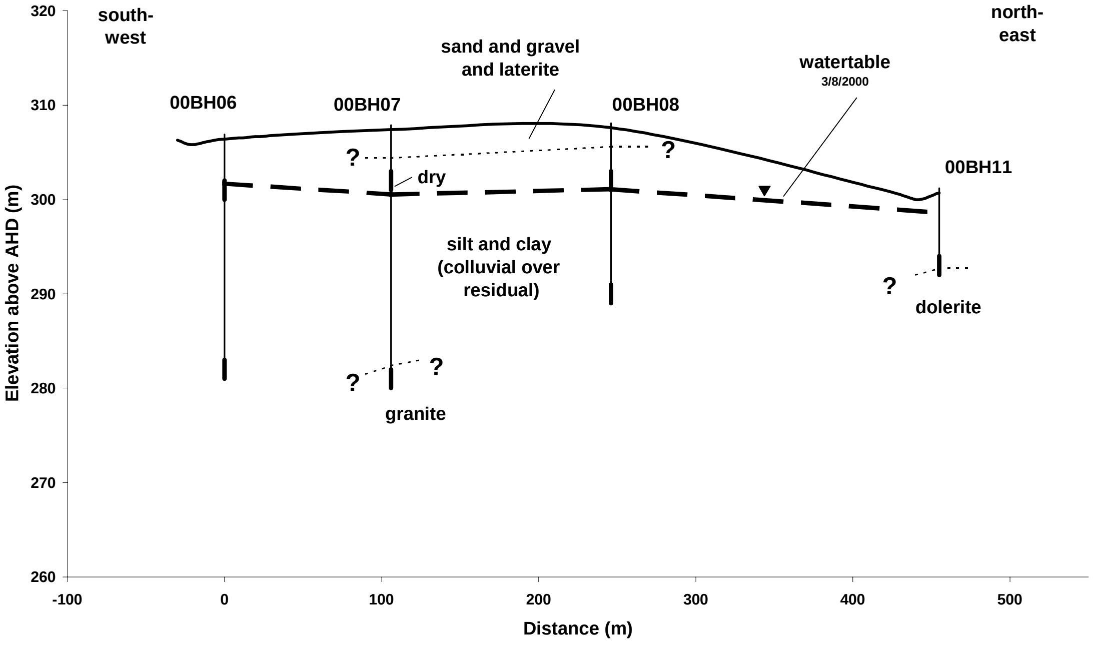

*Figure 2-3. South-west to north-east cross-section from bores 00BH06 to 00BH11 (see Figure 2-1 for location)*

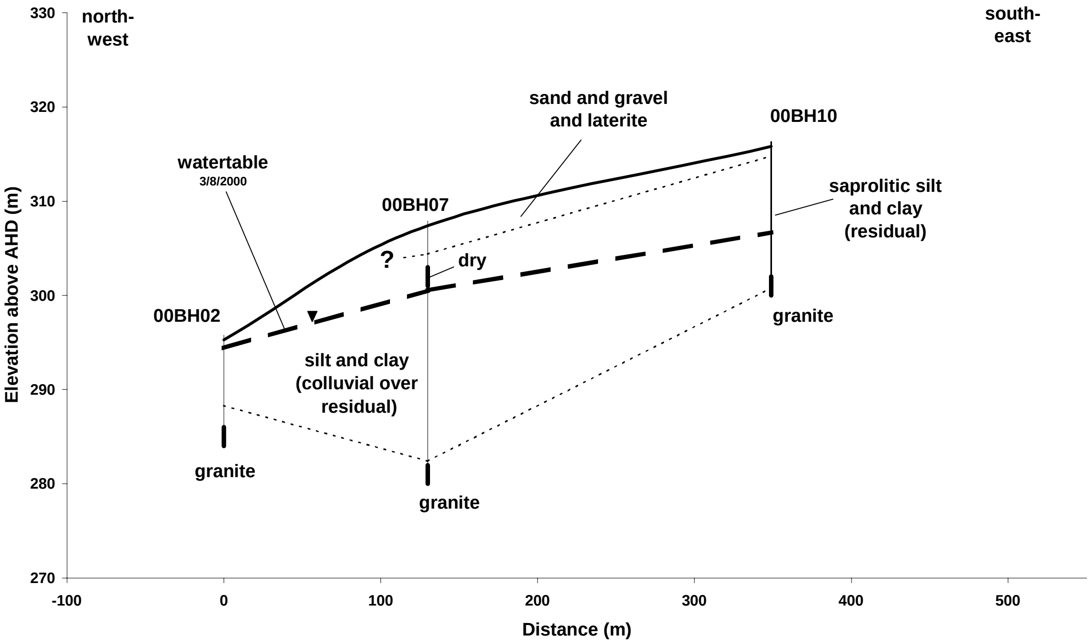

*Figure 2-4. North-west to south-east cross-section from bores 00BH02 to 00BH10 (see Figure 2-1 for location)*

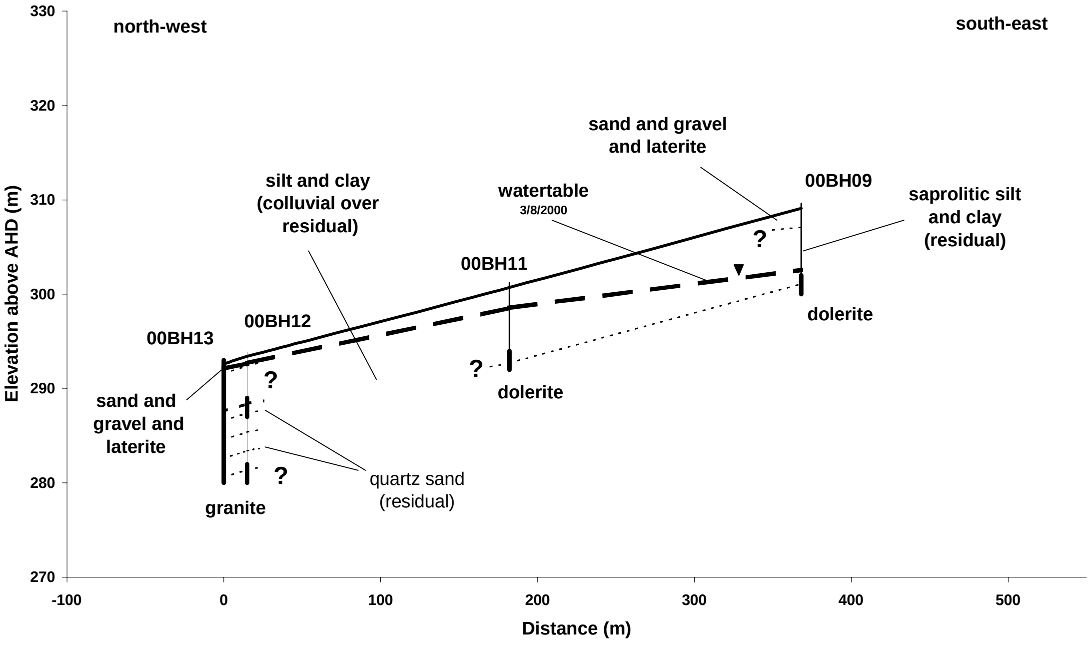

*Figure 2-5. North-west to south-east cross-section from bores 00BH13 to 00BH09 (see Figure 2-1 for location)*

### Table 2-1. Piezometer and bore construction details, and groundwater depths and electrical conductivity values on 3 August 2000

| Bore name | Easting AGD84# (m) | Northing AGD84# (m) | Ground elevation AHD## (m) | Drilled depth (m) | Screened interval depth bgl### (m) | Groundwater level depth bgl### (m) | Groundwater EC (mS/m) |
|---|---|---|---|---|---|---|---|
| **00BH01I** | 448777.4 | 6487491.7 | 292.7 | 15 | 13–15 | 2.27 | 120 |
| **00BH01S** | 448775.3 | 6487490.6 | 292.6 | 6 | 4–6 | 2.19 | 140 |
| **00BH02I** | 448461.1 | 6487175.4 | 295.3 | 11 | 9–11 | 0.93 | 2,120 |
| **00BH03D** | 448680.4 | 6487412.8 | 293.0 | 20.3 | 18.3–20.3 | -0.04 | 1,170 |
| **00BH03I** | 448678.7 | 6487411.4 | 293.0 | 10 | 8–10 | -0.06 | 1,060 |
| **00BH04D** | 448629.9 | 6487370.8 | 293.9 | 22.5 | 20.5–22.5 | 0.19 | 1,450 |
| **00BH04S** | 448627.9 | 6487369.3 | 293.9 | 6 | 4–6 | 1.15 | 1,600 |
| **00BH05I** | 448552.1 | 6487305.5 | 294.9 | 16 | 14–16 | -0.12 | 2,480 |
| **00BH05S** | 448549.9 | 6487304.1 | 294.9 | 6 | 4–6 | 0.52 | 2,730 |
| **00BH06D** | 448487.1 | 6487022.3 | 306.4 | 25.5 | 23.5–25.5 | 4.73 | 1,820 |
| **00BH06S** | 448489.3 | 6487024.0 | 306.3 | 6 | 4–6 | 4.59 | 1,480 |
| **00BH07D** | 448563.6 | 6487095.0 | 307.4 | 27 | 25–27 | 6.88 | 1,940 |
| **00BH07S** | 448565.5 | 6487096.6 | 307.4 | 6 | 4–6 | DRY |  |
| **00BH08I** | 448673.2 | 6487183.5 | 307.6 | 19 | 17–19 | 6.50 | 670 |
| **00BH08S** | 448671.9 | 6487185.8 | 307.5 | 6 | 4–6 | 6.43 | 2,460 |
| **00BH09S** | 448926.8 | 6487150.3 | 309.1 | 9 | 7–9 | 6.51 | 2,740 |
| **00BH10I** | 448723.6 | 6486945.3 | 315.8 | 16 | 14–16 | 9.09 | 1,860 |
| **00BH11S** | 448837.6 | 6487312.3 | 300.7 | 9 | 7–9 | 2.17 | 110 |
| **00BH12I** | 448702.0 | 6487410.6 | 293.4 | 13 | 11–13 | 0.67 | 1,150 |
| **00BH12S** | 448700.5 | 6487412.3 | 293.3 | 6 | 4–6 | 0.57 | 1,020 |
| **00BH13I** | 448692.8 | 6487422.8 | 292.6 | ~16.5 | ~3.5–16.5 | 0.50 | 1,100 |

Notes: #: AGD84 – Australian Geodetic Datum 1984; ##: AHD – Australian Height Datum; ###: bgl – below ground level

### 2.3.3 Pumping test drawdowns and aquifer parameters

Details of the pumping tests are given in Appendix 1. The constant rate pumping test used a rate of 0.2 L/s but was only run for 24 hours and the drawdown was still increasing steadily at the end of the test (Figure 2-6). Early drawdown increments (up to about 12 minutes) in the constant rate test were relatively high and may have been due to well storage effects.

Drawdowns at the end of the test in the nearest shallow piezometers, 00BH03I and 00BH12S (about 15 m away from the production bore) were between 1.8 and 2.7 m. However, in piezometers 00BH04S, 00BH05S and 00BH01S (about 65, 100 and

110 m away from the production bore, respectively) no drawdowns were recorded. (Groundwater levels actually rose during the pumping test, presumably as a result of rain showers noted in the Test Pumping Australia log or barometric pressure fluctuations). The shape of the 'cone of depression' could not be defined from the available monitoring sites.

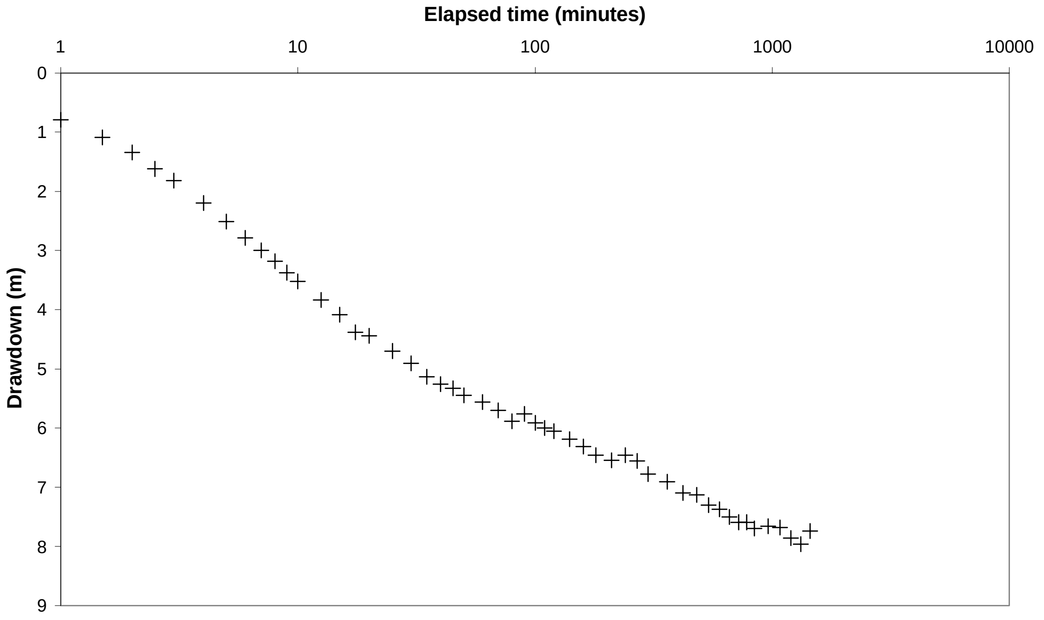

*Figure 2-6. Production bore drawdown versus time for constant rate test*

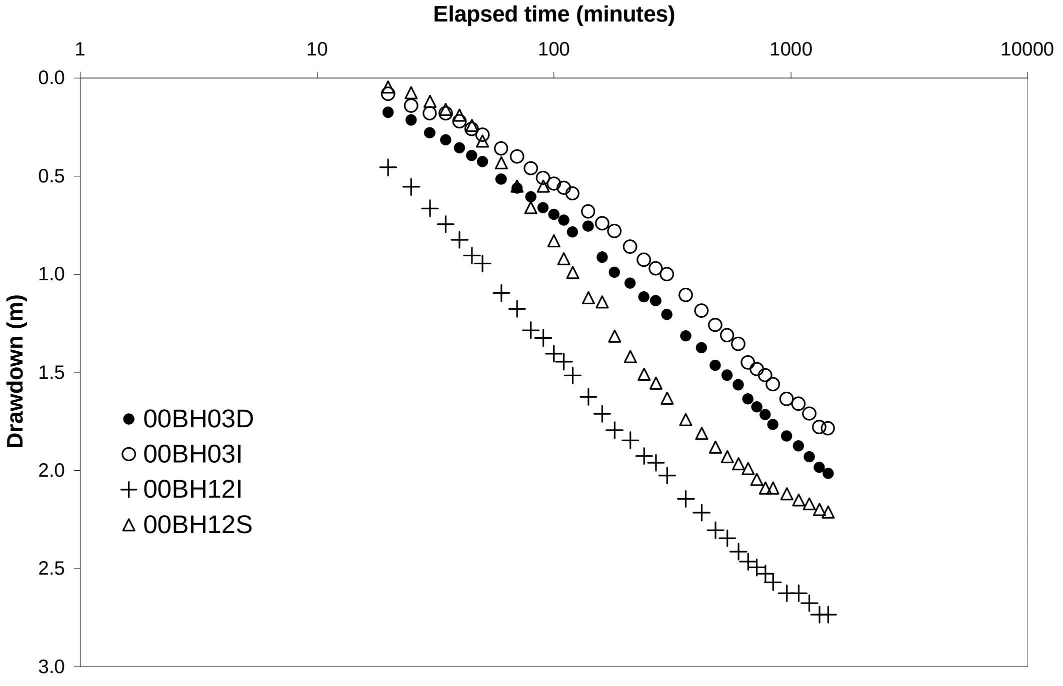

*Figure 2-7. Monitoring piezometer drawdowns versus time for constant rate test (piezometers were about 15 m away from the production bore)*

The transmissivity of the pumped aquifer was estimated to be about 2 $m^2$/day (Appendix 1), giving a hydraulic conductivity of the order of 0.2 m/day, for the pumped aquifer.

## *2.4 Interpretation and discussion*

### 2.4.1 Geology

Bakers Hill is on a residual clay profile produced from weathering of granite and doleritic bedrock. Observations indicated that a quartz-rich phase of granite was more resistant to weathering than surrounding granite, and as a result it produced a subtle topographic and bedrock high. The residual profiles that contained relatively high proportions of quartz were considered to have formed from this type of granite. The dolerite implied that dykes intruded the granitoid bedrock, but their widths and trends were not identified. The topography and groundwater discharge between the townsite and the Clackline Brook were typical of those associated with bedrock structures such as dykes and fractures.

### 2.4.2 Groundwater

This section presents an interpretation of the recharge and groundwater flow processes affecting Bakers Hill, based on the available information. It then discusses the risk of further salinity.

A simple zoning system for considering the sources of groundwater recharge affecting a townsite was applied to towns in the Community Bores Project. It is described and was applied to Bakers Hill.

### 2.4.3 The three recharge zones

The following comments assume that the recharge that causes groundwater to rise below townsites can occur in three 'zones':

1. the townsite itself;
2. slopes above the townsite; and
3. the valley floor downslope of the townsite.

Within the ***townsite zone***, the contribution of water can come from:

- direct recharge from rain infiltrating the ground where it falls;
- recharge from imported water supplies (e.g. leakages from pipes and storage facilities, overwatering, septic systems);
- indirect recharge below ponding areas which collect surface run-off generated on the slopes above the town and on the hard surfaces within the town; and
- indirect recharge below flowing surface water (creek flows, overland flow and unusual floods).

Recharge occurring on ***slopes above a townsite*** can affect groundwater levels below the town if the groundwater systems below the zones are connected. In most cases, the source of the recharge will be rain falling on the slopes and may be direct or indirect.

The groundwater system below a ***valley floor downslope of a townsite*** can affect the groundwater levels below the townsite in two ways. Rising valley groundwater levels may:

- cause the valley floor system to 'encroach' under the town; and
- inhibit the outflow of groundwater from below the town.

The degree of connection between groundwater bodies below the two zones will influence the magnitude of the effect of the downslope zone on townsite groundwater levels. Groundwater in the downslope zone may be influenced by rainfall, surface water flowing into the zone from the town and the slopes above the town, and surface water and groundwater flowing in from other areas.

The relative importance of these three zones differs from town to town but cannot be quantified with available data. Also, the importance of different recharge processes will vary from year to year and season to season. However, one generalisation can be made. If a townsite (or part of a townsite) clearly has negligible groundwater input from slopes above and valley floor below, but still has problems caused by high groundwater levels, it can be concluded that the water causing the problems is recharged solely within the townsite (or part of the townsite). This occurs in several towns in the Community Bores Project. A further implication that can then be drawn is that townsite recharge is also likely to be an important cause of groundwater rises in other towns, even if groundwater systems from slopes above and valley floors below also contribute.

### 2.4.4 Bakers Hill recharge zones and groundwater flow systems

The investigation focused on the main townsite area rather than the catchment to the west where larger landholdings are located. However, some comments on this western area are made at the end of this section.

In Bakers Hill, it seems reasonable to infer that the recharge causing groundwater problems occurs within the ***townsite zone*** (if the golf course and sports grounds are included). This is because the main part of the townsite has no agricultural land upslope from it (the catchment divide runs through the golf course), and therefore has no contributing ***slopes zone*** above the townsite. It is also assumed that the ***valley floor zone*** downslope of the townsite does not contribute groundwater to the townsite zone because its elevation is several metres lower and it is therefore likely that its groundwater elevation is also lower than that below the townsite. In addition, groundwater discharge occurs in the paddock between the townsite and the valley floor, and observations indicate that the discharge is caused by groundwater barriers impeding the flow of groundwater from the townsite to the valley floor.

Therefore, the groundwater system below the townsite of Bakers Hill was considered to be 'localised', being recharged locally and discharging on the lower slopes just above the valley floor of the Clackline Brook. The implication is that the water that becomes recharge within the townsite (including the golf course and sports grounds) is substantial and can alone cause groundwater problems.

It is not clear from the available groundwater records where most recharge in the townsite occurs (e.g. from septic systems, leaking pipes, drains or culverts; below overwatered gardens, 'bare ground', grassed areas or vacant blocks; or below land where water ponds) or what time of year it occurs (e.g. after winter rains, summer rains, or summer watering). Long-term frequent and regular monitoring of groundwater levels in different parts of the townsite can show where the important recharge areas are and when they are active. This will help to establish whether the rain is a more important factor than imported water supplies within the townsite. Therefore, the network is a valuable asset.

#### *2.4.4.1 The western catchment*

Although the investigation did not cover the catchment to the west of the main townsite, it can be assumed that both the cleared agricultural areas and the small holdings within the catchment contribute the recharge which causes the groundwater problems along the creekline. There is not enough information to determine which is the most significant recharge zone.

### 2.4.5 Assessment of salinity risk

In Bakers Hill, it appeared that the groundwater was shallow where the soil profiles become thinner downslope and where high pressures built up along permeable zones such as fracture zones in the bedrock and behind barriers such as dykes resistant to weathering.

The watertable below some sites along the north-western boundary of the townsite was already very shallow (less than 1 m deep), and the pressure of deep groundwater was high enough to raise the water levels in piezometers above ground level in some locations. Upslope from this zone, groundwater levels were deeper (mostly greater than 4 m below ground level at monitoring sites upslope of the Great Eastern Highway, although at one site downslope from the school, the water level was only about 2 m deep in August 2000). It is not known whether groundwater levels are stable or rising below the town. If rising, then damage is likely to spread upslope from the north-western edge, especially along the shallow gullies running through the town. Long-term groundwater monitoring is required.

## *2.5 Management options*

There are two main approaches to dealing with the risk of high groundwater levels and discharge: treat the cause by reducing groundwater recharge; treat the problem by abstracting groundwater.

### 2.5.1 Recharge reduction

It is likely that recharge within the townsite zone (including the golf course and sports grounds) is the dominant influence on groundwater levels below the town. There is scope to reduce townsite recharge, and some of the options have beneficial side-effects (e.g. reduced water supply costs and dependence, less waste of good quality rainfall water, less infrastructure damage from floods and surface run-on).

Monitoring where and when groundwater levels rise and fall will provide information on whether recharge only follows rainfall or occurs during dry periods too (in which case, imported water supplies will be implicated). However, it would be wise to take measures to reduce townsite recharge now and then use information gained from groundwater level records in the future to refine the recharge reduction measures.

Some recharge reduction measures for townsites thar may have other benefits are:

- checking for and mending leaks in water pipes, drains, culverts, dams and pools;
- replacing septic systems with a sewer system;
- monitoring the amount of water required by gardens, parks and sports grounds and eliminating overwatering;
- replacing grass and weeds and bare ground with perennial local plants in as many locations as possible;
- encouraging residents to replace some of their imported water supplies with water harvested from their own hard surfaces (roofs, drives).

The Water Corporation has an interest in reducing wastage of the water it supplies, and could be approached for assistance with some steps.

Groundwater level monitoring should be carried out monthly and records should be analysed by a hydrogeologist at least once a year. Monitoring should continue after any recharge reduction measures are taken so that the impacts can be assessed.

### 2.5.2 Groundwater abstraction

Groundwater abstraction by pumping from bores may be an effective option in some towns. However, groundwater drainage is unlikely to be effective as it only lowers groundwater levels along narrow zones either side of the drain. The drilling investigation showed that bedrock depth was variable and although the pumping test was carried out in a zone with deeper regolith profiles, its effect on watertables was limited to a small area around the pumped bore. Therefore, groundwater pumping does not appear to be a viable management option in Bakers Hill.

### 2.5.3 The western catchment

Most of the discussion on the main townsite zone will also apply to the western catchment. Groundwater abstraction is not likely to be a viable option because of the steepness of the landscape and the likely compartmentalisation of the groundwater systems by geological structures. Therefore, management should concentrate on recharge reduction. If piezometers were installed at several sites along the slopes of the catchment (not in the discharge areas) and monitored frequently for several years, the records could show whether groundwater levels are rising or are stable, and may also indicate where most recharge is occurring.

# 3. Groundwater flow modelling

*Authors: Anthony Barr and Daniel Pollock (CSIRO)*

Section 2 discussed a combination of management approaches which could be effective in Bakers Hill. This section describes a computer groundwater modelling study that aimed to assess the impacts of a selection of possible strategies.

***Note that the modelling was based on limited data and many assumptions and the results should be used with great caution (see warnings in Section 3.4).***

First, a suitable conceptual model was constructed based on the information gained from the drilling investigation and the pumping test, together with topographic and climatic data. This conceptualisation was adapted to the groundwater simulation program MODFLOW (McDonald and Harbaugh 1988) coupled with the pre- and post-processor Visual MODFLOW Version 2.8 (Waterloo Hydrogeologic 2000) and was then calibrated in steady-state against observed groundwater levels. The calibrated model was then used to simulate the effects of four different strategies: 'do nothing differently', groundwater pumping, groundwater drainage and tree planting.

Sections 3.1 and 3.2 describe the construction of the conceptual and computer models and the calibration of the computer model. The strategy simulations and their results are presented in Section 3.3 and their limitations are discussed in Section 3.4.

## 3.1 Model construction and conceptualisation

Conceptually, the groundwater model consisted of two layers: a combination of laterite, kaolin, limonitic clay and saprolite at the surface, over weathered saprolite, constituting the aquifer.

Inflow to the model domain, illustrated in Figure 3-1, was assumed to be through lateral flow from the south-east boundary of the townsite. Discharge from the area was assumed to be through the Clackline Brook parallel to the north-west boundary. The model domain extended 1.80 km from east to west between 447793 mE and 449593 mE Australian Geodetic Datum 1984 (AGD84) and 1.12 km north to south between 6486390 mN and 6488040 mN Australian Geodetic Datum 1984 (AGD84). This area extends beyond the town, and thus a number of cells were deactivated within the simulation to focus on the townsite. Each cell in the domain was 20 by 20 m, resulting in 90 columns and 56 rows for a total of 5,040 cells.

The top of the uppermost layer was taken as the land surface, which was extracted from 2 m-contour digital elevation models (DEMs) for the catchment (map sheets 21341SE and 21342NE, produced by the Spatial Resource Information Group, Agriculture Western Australia). Depths to the bases of each layer were interpreted from borehole logs and interpolated using inverse distance weighting to a 25 by 25 m grid covering the model domain. These depths were subtracted from the surface levels to create the upper and lower boundaries for the various layers. These data were imported into Visual MODFLOW and interpolated onto the model grid. The domain region was reduced considerably by excluding most cells. The model domain was constricted to cover most of the town residential area to the local river (Clackline Brook). The number of active cells was less than 780.

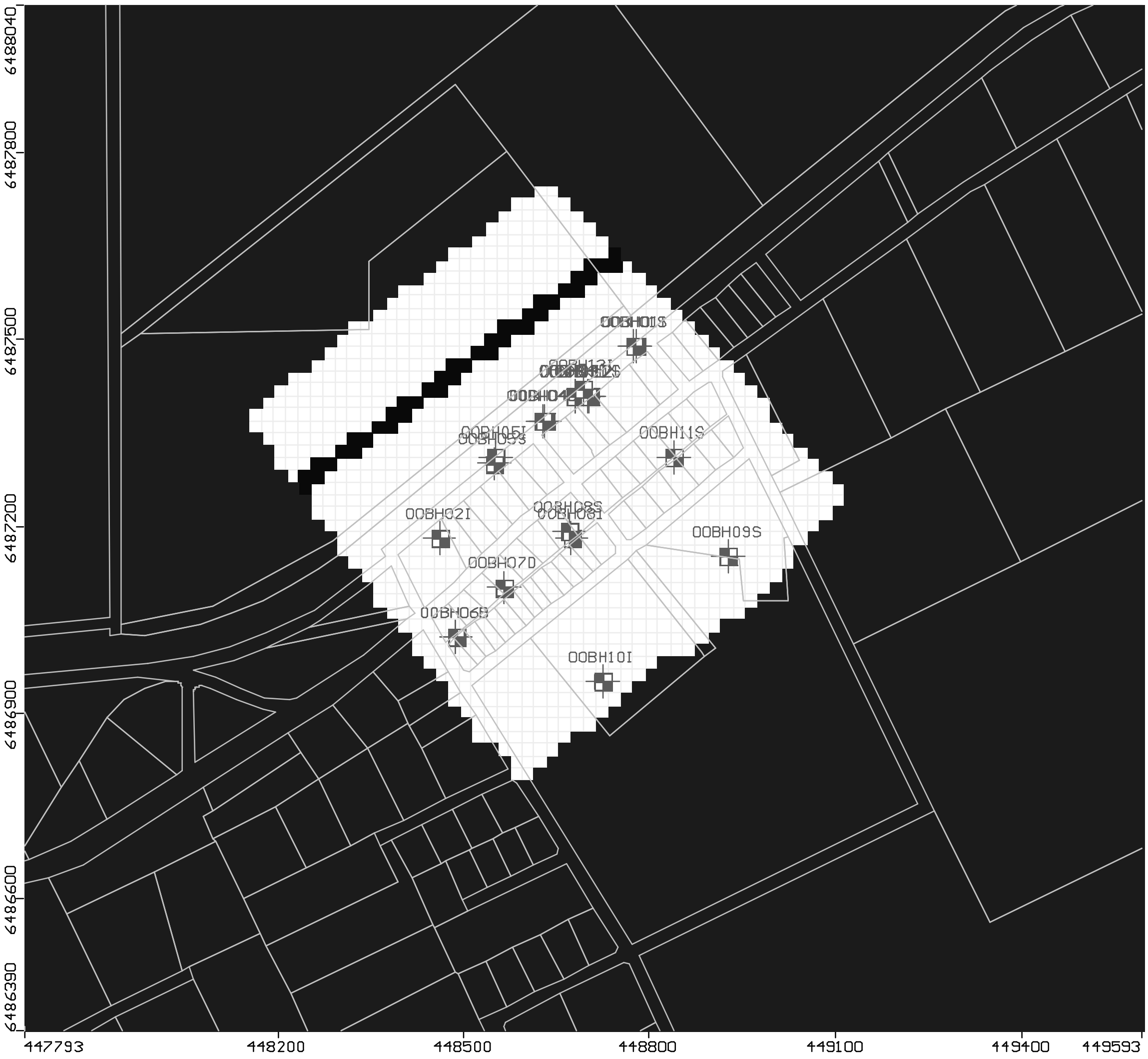

*Figure 3-1. Modelled region with boundary conditions (thick dark frame around the area represents the excluded cells; dark cells in the lower right of the centre show the specified head boundary cells in the south-east of the simulated region; dark cells in the upper left of the centre show the position of the Clackline Brook in the north-west of the simulated region), bore locations and grid (boundary scales are in metres, top of map is north)*

## 3.2 Steady-state model calibration

Groundwater records from other parts of the agricultural region of Western Australia show that watertables in many parts of the landscape are rising (Nulsen 1998) and it was inferred that the groundwater below Bakers Hill is also is in a state of flux. However, the lack of long-term water level records within the town mean that some assumptions had to be made. It was assumed in this groundwater modelling that the heads measured on 3 August 2000 are indicative of the steady-state groundwater system under the current climatic and land-use conditions. This assumption may be appropriate for Bakers Hill as the catchment for the town is relatively small. To confirm this, records of piezometric heads over a number of years are needed. The two quantities considered for calibrating the system were the hydraulic conductivity of the layers, in both the horizontal and vertical directions, and the net annual recharge to the groundwater of the system. Indicative values of hydraulic conductivity were estimated from the pumping test results, and the recharge was estimated from average annual increase in the watertable. A value of 0.45 m/day was used for the hydraulic conductivity, which is consistent with the results of George (1992) for saprolite grits. The rate of rise of the watertable in the region was conservatively estimated to be 0.10 m/year.

The effect of calibrating against the heads of a non-equilibrium system are, where the elevation of the watertable is increasing, that the parameterisation of the system will be a trade-off between underestimating the recharge and overestimating the hydraulic conductivity. Thus, the response times of the aquifer in this modelling will be quicker than the response time of the real aquifer. However, without longer datasets or starting the modelling from when the system was last in a steady state, prior to clearing, for the whole catchment, this method will at least provide an indication of the processes that are occurring within the town.

The inflow boundary in the south-eastern part of the townsite was simulated with a specified head 306 m above Australian Height Datum (AHD) in the southern part of the boundary and 303 m above AHD in the eastern part of the boundary (based on levels on nearby piezometers). These boundary conditions were applied in both layers, except for the middle part of the boundary in the upper layer, where the head was below the bottom of the layer. The Clackline Brook in the north-west was simulated using a river boundary condition. This boundary consisted of a river level and a basement level of 287 m and 285 m above AHD respectively in the western, upstream cell of the brook, and 282.8 m and 280.2 m above AHD respectively in the eastern downstream end. Grid and boundary conditions are shown in Figure 3-1). The conductance between the river and the aquifer was set to 100 $m^2$/d. The remaining boundary cells were treated as no-flow boundaries. Recharge was applied uniformly over the modelled region and based on 10 per cent of the annual average rainfall of the townsite (605.4 mm, Bureau of Meteorology 2000b).

Calibration of the steady-state model was accepted with a standard error of the estimate of 0.24 m from both layers for measurements made on 3 August 2000. The parameters used to achieve this are listed in Table 3-1. The hydraulic conductivity for both layers was taken as spatially uniform over the layer. The resulting depths to the groundwater for the calibrated model are shown in a map in Figure 3-2 and along a cross-section in Figure 3-3. Travel times below the townsite were about 20 years in the lower aquifer and 70 to 100 years in the upper layer, which, when compared to the travel times of the order of thousands of years for Merredin (Matta 2000), indicated a more dynamic system. This can be explained by the high conductivity of the bottom layer.

*Figure 3-2. Depth to groundwater (in metres, contour intervals are 1 m) for steady-state simulation*

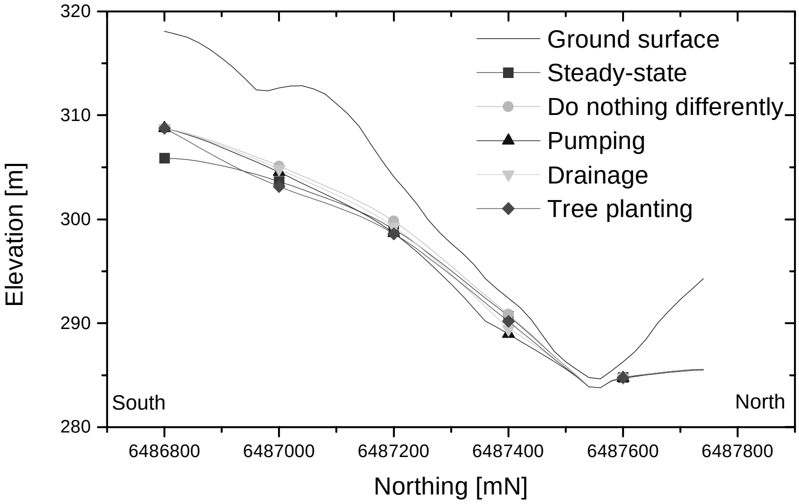

*Figure 3-3. South to north cross-section along the 448623 mE gridline for all simulations*

The model is quite sensitive to the selection of hydraulic conductivity and only slightly sensitive to the recharge. However, as mentioned above, calibration of this system is a trade-off between higher hydraulic conductivities and lower recharge rates. Therefore, although this is considered to be a good estimate of the parameters of the system, it is not a unique fitting of the data, and other parameterisations with increased recharge and hydraulic conductivities, or decreased recharge and hydraulic conductivities will also fit the measured levels.

**Table 3-1. Parameters used for the Bakers Hill model**

| | Layer | |
|---|---|---|
| **Parameter** | **1** | **2** |
| Horizontal hydraulic conductivity (m/day) | 0.05 | 0.3 |
| Vertical hydraulic conductivity (m/day) | 0.005 | 0.03 |
| Storativity ($m^{-1}$) | 0.0005 | 0.0005 |
| Effective porosity | 0.1 | 0.1 |
| Recharge (mm/year) | 60.0 | |
| Groundwater evaporation (mm/year) | 365 | |
| Groundwater evaporation extinction depth (m) | 1.0 | |

## *3.3 Dynamic simulations of strategies*

The dynamic simulations extended over 30-year periods. In this period, the head in the upper part of the region, the south-east boundary, were increased at a rate of 0.1 m/yr. This entailed some modification of the boundary conditions. The existing boundary conditions were increased at yearly intervals by 0.1 m. For the steady state simulation where the constant head was below the bottom of the top layer, a general head boundary was introduced, with the external heads equivalent to the constant head and the conductance set to 10 $m^2$/d. To enable these and other cells to become part of the simulation, the cell rewetting option was activated, using the options for wetting from both below and the sides with a rewetting factor of 1.0. The specific yield and confined storage coefficients from Table 3-1 were applied appropriately.

### 3.3.1 'Do nothing differently' strategy

The ‘do nothing differently’ strategy assumed that no changes in groundwater management would occur. The resulting depth to the watertable after 30 years is shown in Figure 3-4. The elevation of the watertable along a cross-section through the town is shown in Figure 3-3. This model predicted that under current management practices eastern parts of the commercial district, on the northern side of the highway, could eventually have a shallow watertable of 1.0 m or less from the ground surface. Also, residential blocks on the eastern side of the town could have the watertable within 2.0 m of the surface.

*Figure 3-4. Depth to groundwater after 30 years (in metres, contour intervals are 1 m) for 'do nothing differently' simulation*

### 3.3.2 Groundwater pumping strategy

Groundwater abstraction through five bores was tested in the model as a potential management option. The abstraction wells were placed along a line north-west of the commercial properties (Figure 3-5). Given that the sustainable yield from the pumping test (Appendix 1) was established as 0.2 L/s, or 17 m$^3$/d, and to avoid excessive drawdown in the simulated pumps, each well in the model was assigned a discharge rate of 5 m$^3$/d. The resulting modelled depth to groundwater after 30 years is shown in Figure 3-6 and the watertable elevation along a cross-section in Figure 3-3. This shows that, for the conditions modelled, the depth to the watertable under most of the commercial area was lowered to greater than 3 m. However, there was still an area of rising watertables under the residential areas.

### 3.3.3 Groundwater drainage strategy

The option of a drainage system to remove groundwater was tested using a drain located to the north-west of the commercial district (Figure 3-5). The drain was located approximately 2 m below the ground surface, from an elevation of 293 m in the west to 287 m in the east. The resulting depth to groundwater after 30 years is shown in Figure 3-7 and the watertable elevation along a cross-section can be seen in Figure 3-3. The model predicted that this strategy would also lower the watertable under the majority of the commercial district. The exception was in the east of the simulated area, with some residential areas being affected. The overall effect of the drainage after 30 years is not as great as that for the pumping.

*Figure 3-5. Modelled region with management options (simulation area in the centre; the circular objects in a line from south-west to north-east through the centre represent pump locations; adjacent light grey lines represent the simulated drain; dark areas south-east and north-west of pumps represent tree planting areas; boundary scales are in metres; top of map is north)*

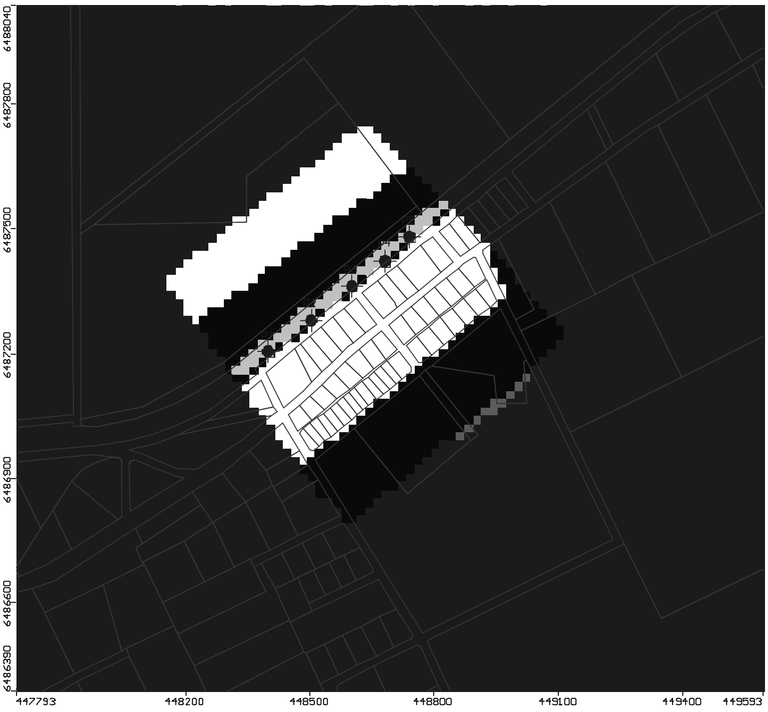

*Figure 3-6. Depth to groundwater after 30 years (in metres, contour intervals are 1 m) for pumping simulation*

*Figure 3-7. Depth to groundwater after 30 years (in metres, contour intervals are 1 m) for drainage simulation*

### 3.3.4 Tree planting strategy

Tree planting was considered for some of the areas surrounding the town (Figure 3-5). It was assumed that the trees reduced recharge under the planted areas to zero, but that they did not extract water from the watertable. The resulting depth to groundwater after 30 years is shown in Figure 3-8 and the watertable elevation along a cross-section is shown in Figure 3-3. There was some improvement for the commercial area compared with the 'do nothing differently' strategy, but it was the least effective of the three management options.

*Figure 3-8. Depth to groundwater after 30 years (in metres, contour intervals are 0.5 m) for tree planting simulation*

## 3.4 Discussion of groundwater modelling

The groundwater modelling in Bakers Hill was undertaken using limited data. Therefore, the results are indicative only and may not represent what is happening in the town.

Models should be calibrated for several dates to cover the range of groundwater levels that occur. Because of limited groundwater level data, the model was only calibrated in steady-state against the heads measured in August 2000. The assumption of a steady-state groundwater system is inappropriate, but represents the best method for applying a groundwater model to the town.

Models should also be validated using independent data sets. Since no independent data were available, the model was not validated.

The model results are sensitive to both the recharge rate and values of hydraulic conductivity used, but the values used were only estimated from limited information or assumed.

Assumptions were made about groundwater levels along the boundaries of the modelled area, although it is not known whether they are stable or rising over the long-term, nor how the rates vary along the boundaries. If the rate of watertable rise is quicker or slower than the rate assumed, then the effects will be correspondingly sooner or later.

The role played by geological structures (such as dykes and faults) is not known, and the model did not account for them.

The model results are very dependent on the DEM data (which represents the land surface elevation). It is possible that there are inaccuracies in the DEM data set.

Recharge was applied evenly across all of the modelled area, but in reality it will vary spatially. Recharge below unvegetated areas and areas of annual plants (weeds, grasses, etc.) is likely to be greater than below areas covered by buildings or impervious surfaces, or under perennial vegetation. Since it is these unvegetated and grassed areas which were 'planted' with trees in the model (Section 3.4) the reduction in recharge is likely to be greater in reality than that assumed in the model. Therefore, revegetation is likely to be a more effective option than the model indicated.

# 4. Flood risk analysis

*Authors: Muhammad J. Siddiqi and Ali Mahtab (Agriculture Western Australia)*

The town falls within the Chitty catchment which covers an area of approximately 3,500 ha drained by the Clackline Brook. The area of Clackline Brook catchment above the Bakers Hill townsite is approximately 900 ha. Two minor drainage lines run north-west through the town and join the Clackline Brook which runs north-east below the town.

Surface water is managed in the town by a combination of open and piped drains discharging to natural waterways. Isolated flooding and ponding affects a few properties in winter in the area bounded by Berry Brow Road, St George Street and Keane Street. The flooding is largely caused by inadequate provision for natural surface drainage and low capacity of underground drains.

At the western end of the town, near Jordi Road, a natural waterway drains through a culvert under the Great Eastern Highway. However, the waterway is overgrown with vegetation that restricts the flow, causing water to pond upstream of the culvert. There is no water harvesting in the town.

## *4.1 Objective of this study and approach*

The objective of this part of the Community Bores Project was to determine the flood risk (high, moderate or low) of the town to assess how frequently substantial volumes of flood water provided a potential source of groundwater recharge. This was done by calculating the peak flood flow generated by all of the catchment for the town and the run-off generated within the townsite, and comparing these with the flow accumulation characteristics of the catchment.

The Urban Drainage Design (UDD) model was used to calculate peak flows for the catchment for the town because it accounts for the spatial variation in flow rates across catchments, whereas some other methods (e.g. Rational and Time-Area approaches) assume flow is uniform across catchments. The UDD model also allows precipitation rate, catchment slope, surface roughness, interception, depression storage, infiltration and evaporation to be considered. The procedures used are discussed in detail in Ali *et al*. (2001).

The peak flood flows were calculated for 2-, 5-, 10-, 20-, 50- and 100-year average recurrence intervals (ARIs) based on historical events. The run-off volumes generated by pervious and impervious (i.e. generating high run-off) surfaces within the townsites were calculated for 20-, 50-, and 100-year ARIs.

## *4.2 Input data*

The information required to run the UDD model and calculate the run-off volumes was derived from available sources and from a site visit.

### 4.2.1 Available information

The following information was collated for the Bakers Hill catchment:

- rainfall intensities (estimated from Institution of Engineers 1987);
- 2-metre elevation contours derived from a digital elevation model (DEM) produced by the Department of Land Administration.

### 4.2.2 On-site observations – structures influencing surface water flow

The location, size and condition of the existing infrastructure (roads, railway line, grain depot, shire dams) influencing the natural flow of surface water were noted during the visit to the town. Impervious areas within the town consist of houses, roads and retail buildings.

More than half of the town roads are kerbed with inlet pits and a piped drainage system. Because of the heavy clay soils in the town, many residents also discharge roof run-off into the street drains. This is generally via small diameter PVC pipes cut into the kerb. Many of these pipes had been broken or were blocked and would not be working effectively.

### 4.2.3 Derived information used in the calculations

A grid of the study area was derived from the DEM and this was used to predict flow directions, flow accumulations, streamlines, watershed boundaries, and slope and length of the streams. Details of the procedures used to create the grid are given in Ali et al. (2001).

Observations made during the site visit and interpretations of aerial photographs and the elevation contours were used to derive the following:

- area of catchment (pervious and impervious);
- area generating high run-off;
- area generating high recharge;
- infiltration (maximum and minimum likely rates);
- roughness coefficient (Manning's n).

A report by Ali *et al.* (2001) contains descriptions of how the information was used in the UDD model and how run-off volumes for the town catchment were estimated.

Run-off volumes were calculated separately for the 'pervious' and 'impervious' parts of the town (i.e. high run-off generating) areas using run-off coefficients of 0.1 for the former and 0.9 for the latter.

### 4.2.4 Model calibration

To ensure that the best results are obtained using UDD modelling, the model should be calibrated using actual flow data. However, as there is no gauging station in the Bakers Hill town catchment, parameters used for a calibrated model derived for the Moora townsite (Ali *et al*. 2001) were substituted.

## 4.3 Results

Results are summarised in Tables 4-1 and 4-2.

**Table 4-1. Peak flood flow for the catchment for Bakers Hill town**

| ARI (years) | Peak flood ($m^3/s$) |
|---|---|
| 2 | 2.8 |
| 5 | 4.5 |
| 10 | 9.3 |
| 20 | 16.3 |
| 50 | 26.2 |
| 100 | 43.9 |

**Table 4-2. Run-off volumes for the pervious and impervious areas of the townsite generated by rainfalls of various ARIs, duration and intensities**

| Average recurrence interval (years) | Rainfall duration (hours) | Rainfall intensity (mm/h) | Rainfall (mm) | Run-off volume | |
|---|---|---|---|---|---|
| | | | | Pervious area ($m^3$) | Impervious area ($m^3$) |
| **20** | **1** | 29.5 | 29.5 | 1,770 | 10,620 |
| | **6** | 9.0 | 54.0 | 3,240 | 19,440 |
| | **24** | 3.5 | 84.0 | 5,040 | 30,240 |
| **50** | **1** | 35.8 | 35.8 | 2,150 | 12,890 |
| | **6** | 11.0 | 66.0 | 3,960 | 23,760 |
| | **24** | 4.4 | 105.6 | 6,340 | 38,020 |
| **100** | **1** | 39.1 | 39.1 | 2,350 | 14,080 |
| | **6** | 12.6 | 75.6 | 4,540 | 27,220 |
| | **24** | 5.1 | 122.4 | 7,340 | 44,060 |

## 4.4 Flood risk assessment

The criteria to classify a town's relative flood risk level were based on the calculated rates of flow and the *accumulation potential* of the townsite and catchment above the town. The accumulation potential depends on the relative magnitudes of the potential inflows and outflows. The peak flood flows for the catchment for 20-, 50- and 100-year ARIs for storms of 24 hours duration were compared to the catchment area, the accumulation potential of the catchment and the flow generated within the townsite. Table 4-3 shows the flood risk to the town of Bakers Hill for 20-, 50- and 100-year ARI storm events of 24 hours duration.

**Table 4-3. Flood risk to the Bakers Hill townsite for 20-, 50- and 100-year ARI storm events of 24 hours duration**

| ARI (years) | Peak flood flow for entire catchment ($m^3/s$) | Volume of flood generated by townsite ($m^3$) | Accumulation risk | Flood risk | Overall flood risk |
|---|---|---|---|---|---|
| 20 | 16.3 | 35,280 | Low | Low | Low |
| 50 | 26.2 | 44,350 | Low | Low | |
| 100 | 43.9 | 51,410 | Medium | Medium | |

## 4.5 Conclusion

Bakers Hill is at low risk from flooding from storm events with up to 50-year ARIs and at medium risk from storms with 100-year ARIs. Localised flooding may be associated with rainfall events with ARIs greater than 20 years, with low-lying areas mainly affected. For storm events with ARIs greater than 50, a considerable area of the town may have localised flooding.

## 4.6 Warning

The peak flood flow and run-off values estimated in this report should not be used as inputs for the design of any engineering structures such as drains, culverts or diversion banks as the input parameters used for this study would not be suitable for such uses. It is recommended that for any specific use the peak flood flow should be estimated again for the conditions existing in the catchment at that time. Detailed descriptions of the input parameters for this study and their limitations are in Ali *et al.* (2001).

# 5. Conclusions and recommendations

Groundwater levels below parts of the Bakers Hill townsite are already shallow, and it is not known whether they are rising. Most of the recharge causing the rise was assumed to occur within the townsite (including the golf course and sports grounds upslope of the built-up area) but there is not yet enough information to determine when and where most recharge occurs. However, some general recommendations for managing the risk can be made. There are opportunities for reducing recharge, but groundwater abstraction by pumping is not likely to be effective.

## *5.1 Recommendations*

1. **Reduce townsite recharge**, giving particular regard to surface water management and planting local perennial species on vacant areas where groundwater is not too saline. Consider taking the steps listed in Section 2.5.1.

2. **Measure groundwater levels** in the monitoring network monthly and analyse and review them annually, and continue to do so for at least ten years to determine whether groundwater problems are worsening and where and when most recharge occurs.

## 6. Acknowledgments

Jim Prince and Ed Solin (Agriculture Western Australia) helped collect the information for the hydrogeological investigation.

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# Appendix 1. Pumping test

*Author: Ron Colman (Test Pumping Australia)*

As part of the hydrological investigation of Bakers Hill, a pumping test was carried out in the production bore (00BH13I). It aimed to establish aquifer parameters for use in the groundwater modelling study (Section 3).

## *A1.1 Method*

Test Pumping Australia was contracted to carry out and analyse the pumping test.

There were two parts to the test, which were performed on 29 and 30 August 2000. The first part was a multi-rate test (that is, a series of step increases in the pump rate, with the discharge being maintained at a constant value within each step). The results of this part of the test were assessed before setting the pump rate for the second part, which was a constant rate test.

The static water level in the bore before the multi-rate test was 0.97 m below the reference point (which was 0.6 m above ground level). The test was conducted on 29 August 2000 with four 30-minute steps at discharge rates of 0.10, 0.15, 0.20 and 0.25 L/s.

The constant rate test started on 29 August 2000 and lasted 1440 minutes (24 hours) at a pumping rate of 0.2 L/s. The drawdowns in the production bore and four piezometers (at sites 00BH03 and 00BH12) were measured at intervals throughout. The rate of recovery of the water level in the bore was measured at the completion of the test.

During the tests, the flow rate was monitored using an orifice weir assembly and water levels were measured using an electric water level probe. Table A1-1 summarises relevant details.

**Table A1-1. Details of the pumping test**

| | |
|---|---|
| Pump inlet depth below ground level | 16 m |
| Available drawdown in production bore | 14 m |
| Pump | electric submersible |

Computerised calculations of aquifer parameters were made.

## *A1.2 Results*

### A1.2.1 Multi-rate test

The total drawdown in the production bore at the end of the multi-rate test was 7.65 m. The multi-rate test data is presented in Figure A1-1 as a plot of drawdown versus time for each of the four steps.

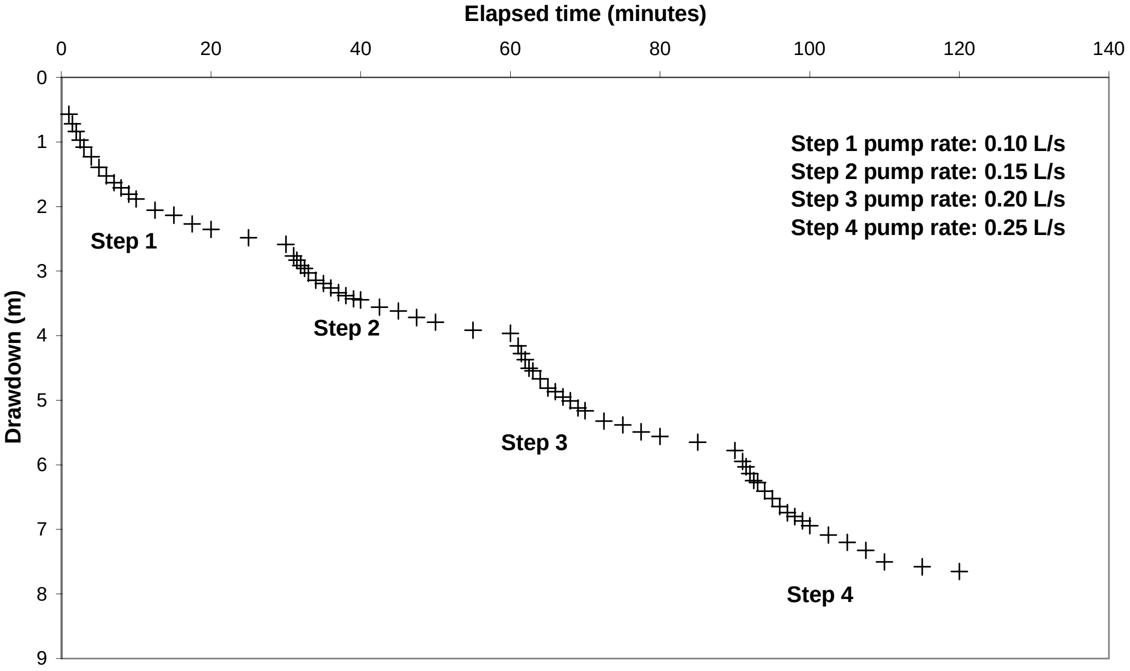

*Figure A1-1. Drawdown versus time for the multi-rate test*

### A1.2.2 Constant rate test

Total drawdown in the test bore at the end of the test was 7.96 m. The drawdown data are presented and discussed in Section 2.3.3 of the report.

### A1.2.3 Aquifer parameters

A summary of the calculated aquifer transmissivities is presented in Table A1-2 and of other parameters and measurements made during the test in Table A1-3.

**Warning: The drawdown data were only analysed using computerised methods designed for homogeneous, isotropic confined and unconfined aquifers of large areal extent. Since the aquifer which was pumped does not fit these criteria, the results should only be considered as indicative.**

## Table A1-2. Production bore and monitoring site details and transmissivity values (see 'Warning' above) calculated for the Bakers Hill production bore and nearby piezometers (AHD: Australian Height Datum; NR: analysis not relevant)

| Bore name | Intake interval above AHD (to nearest metre) | Lateral distance from pump (m) | Final drawdown (m) | Transmissivity ($m^2$/day) | | |
|---|---|---|---|---|---|---|
| | | | | Cooper and Jacob (time-drawdown) | Theis (curve fitting) | Theis & Jacob recovery |
| **00BH13I** | 276-289 | 0.1 | 7.96 | 1.7 | 1.4 | 1.8 |
| **00BH12S** | 287-289 | 12.7 | 2.21 | NR | NR | NR |
| **00BH12I** | 280-282 | 15.0 | 2.74 | 2.9 | 2.3 | 1.6 |
| **00BH03D** | 273-275 | 15.6 | 2.02 | 2.8 | 2.6 | 2.1 |
| **00BH03I** | 283-285 | 17.9 | 1.79 | 2.9 | 2.7 | 2.3 |

## Table A1-3. Summary of measurements and calculated parameters

| Parameter or measurement | Results |
|---|---|
| Aquifer thickness (m) | 15.3 |
| Well loss | Low – 10.2% |
| Electrical conductivity (mS/m) | 1,150 |
| Acidity (pH) | 5.3 |
| Safe yield (L/s) | 0.20 |

**Note:** Test Pumping Australia considered that the constant rate pumping test indicated that the bore might be capable of maintaining a long-term abstraction rate of 0.2 L/s. At this rate, it expected that there would be drawdown effects from pumping up to 50 m from the pumping bore.