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Groundwater study of the Quairading townsite

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Disclaimer

The contents of this report were based on the best available information at the time of publication. It is based in part on various assumptions and predictions. Conditions may change over time and conclusions should be interpreted in the light of the latest information available.

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Summary

A hydrogeological investigation and a flood risk analysis were carried out for the town of Quairading with the aim of accelerating the implementation of effective salinity management options.

Quairading is not currently affected by salinity, although surface water problems occur. However, the community is concerned that salinity could develop in the future.

Fourteen piezometers, 12 observation bores and one production bore were installed at 15 sites within and around the townsite. The depth at which bedrock was found ranged from 18 m at a site on the north-western edge of the town to about 52 m at a site in the southern part of Quairading. Groundwater level depth was less than 4 m below ground at monitoring sites on the north-western and western town perimeter. At all other sites drilled, groundwater levels were about 5 to 20 m deep. The shallow groundwater in the west and north-west was considered to be caused by barriers to groundwater flow, probably one or more mafic dykes.

A groundwater pumping test showed that the production bore site could only sustain low pumping rates (about 1.5 L/second). There appeared to be only limited connection between the groundwater bodies monitored in shallow observation bores and those monitored in the deeper piezometers.

Recharge below cleared areas to the north-west of the townsite was considered to supply the shallow groundwater system in the north-west and west of the townsite. Recharge within the townsite was thought to be the source of most of the groundwater found underneath the main area of the town. There is not yet enough groundwater data to test these views. It is also not known whether groundwater levels are rising or are stable. The groundwater modelling indicated that Quairading was not at risk from shallow groundwater, but it should be noted that the model was based on limited data, the rates of recharge and groundwater rise used in it were assumptions, and it did not attempt to incorporate possible groundwater barriers.

The study concluded that the town has a high risk of flooding, inundation and waterlogging.

It was recommended that measurement of groundwater levels in the established monitoring network be continued at monthly intervals and that the results are analysed and reviewed annually, and that this continue for at least 10 years to determine whether groundwater problems are worsening and where and when most recharge occurs.

Meanwhile, efforts should be made to reduce the amount of recharge occurring within the townsite and the catchment to the north-west, giving particular regard to surface water management and revegetation.
1. Introduction

The Rural Towns Program commissioned a hydrological investigation of the Quairading镇site. 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 hydrological assessment for each town consisted of a hydrogeological investigation and a flood risk analysis. For Quairading, the investigation included a drilling program, the establishment of a piezometer network, a pumping test and groundwater flow modelling. This report documents background information for the town and its catchment (Section 2) and the hydrogeological and flood risk investigations (Sections 3 to 5) and then recommends steps for managing the salinity issues of the town effectively (Section 6).
2. Background information

Author: Travis Cattlin, Catchment Hydrology Group, Agriculture Western Australia

Quairading is about 160 km east-south-east of Perth (Figure 2-1). The town has approximately 600 permanent residents and services the local agricultural industry. Quairading is a wheatbelt town and sporting activities dominate social life. It is located on the valley floor to accommodate the railway system. There is concern in the community that salinity could soon affect town infrastructure.

Figure 2-1. Regional setting of Quairading townsite
2.1 Description of the town catchment

Gently sloping valley floors, valley sides, granite outcrops, gravel ridges and deep yellow sandplain characterise the town catchment. The point of discharge is directly into the Yenyenning Lakes system that drains into the Avon River system (Figure 2-1).

![Figure 2-2. Quairading townsite and its catchments](image)

2.2 Geology

Field inspection of rocks around the Quairading catchment indicated that the bedrock is characterised by recrystallised Archaean basement of quartz monzonites, grading into granodiorite composition in some locations, and granite in others. Deformation stages have introduced faulting, at both regional and local scales, and intense folding of the recrystallised plutons. Post-dating the deformations was the intrusion of Proterozoic mafic dykes, either fine- or medium-grained (Chin 1986). Weathering of the granite plutons, and subsequent deposition along the fault lines and ancient drainage lines, created palaeodrainage systems that still control the location of drainage lines. Intense weathering of the lateritised duricrust during the Cainozoic era formed deep yellow sandplain upslope and alluvium towards the valley floor. Silcrete developed above the kaolinised zone and beneath the duricrust and the profile was overlain by alluvial and colluvial deposits (Chin 1986).
2.3 Climate

Quairading has hot, dry summers and cool, wet winters. Most rainfall occurs during the winter months (Figure 2-3). Mean annual rainfall is 375 mm (Bureau of Meteorology 2000, pers. comm.) and the annual class A pan evaporation rate is about 2,200 mm (Luke et al. 1987).

![Figure 2-3. Mean monthly rainfall at Quairading (Bureau of Meteorology 2000)](image)

2.4 Drainage

The following description is based on observations made during the drilling program and on conversations with residents and shire representatives.

Surface water flow is similar to other central wheatbelt drainage systems: flow is seasonal and unsustaining.

There are three defined subcatchments within the town catchment (Figure 2-2). The subcatchment east of the town that drains southwards is characterised by an upper valley sandplain. Downslope, shire production bores capture fresh to brackish groundwater, although they rarely extract large amounts. Surface water drainage in this subcatchment has no impact on town infrastructure due to the discharge point being downslope from the town.

The second subcatchment drains from the north-west through the centre of the town (Figure 2-2) and is considered to contribute to groundwater recharge. The natural drainage line flows just south of the main street, directly through the sporting complex and eventually into the Yenyenning Salt Lakes system. Town dams have been constructed on it. Surface water drainage is impeded by structures such as the railway line and the grain depot (Figure 2-4). Residents have noted surface run-off ponding around the grain depot and upslope of the railway lines and believe that the water recharges a shallow aquifer system. This was not evident at the time of investigation (May and June 2000).
The third subcatchment, located to the west of the town (Figure 2-2), directs some run-off through the town, but most drains southwards into the Yenyennying Salt Lakes system. The water that does flow through the town is directed along a natural drainage line which bypasses the grain depot and the railway line, so does not add to the ponding problem noted above.

2.5 Hydrogeology

No documented information on the hydrogeology of the catchment was found. Piezometer records from agricultural catchments in the region from about 1990 to 1995 show groundwater levels rising in many parts of the landscape (Nulsen 1998). At those sites with rises, the water levels rose about 0.5 to 3 m over the period.

2.6 Town water disposal

During winter 2000, septic systems in the northern part of the town were being converted to a mains sewerage system. Prior to this, residents considered that the septic systems discharged water during summers but that groundwater flowed into them during winters.

The sewage treatment ponds are downslope of the townsite and are not considered to have an impact on the groundwater system below the town.
Figure 2-4. Locations of piezometers and bores, their groundwater levels and electrical conductivity values on 20 November 2000, and locations of cross-sections in Figure 3-1 and 3-2
3. Hydrogeology investigation

Authors: Travis Cattlin, Catchment Hydrology Group, Agriculture Western Australia and Fay Lewis (Fay Lewis Consulting)

The hydrogeology investigation aimed to determine which salinity management options would be most effective in Quairading. It included a drilling program coupled with the installation of a piezometer monitoring network, a pumping test and groundwater flow modelling. Most of the investigation is described in Sections 3.2 to 3.4, and management options are discussed in Section 3.5. The effects of some of these options were then tested using a groundwater flow model (Section 4).

3.1 Available information

The Quairading Shire Council had drilled several observation bores around the town as a Land Conservation District and Community Landcare Coordinator initiative. No records of drill depth, sample logs, groundwater influence or depth are available.

3.2 Method

Fifteen sites (one production bore and 14 monitoring sites) were established for the Community Bores Project during May and June 2000. Piezometers were installed at each monitoring site and observation bores were installed at 12 (Figure 2-4).

3.2.1 Drill site selection

Drill site selection was based on availability of council land, suitable spacing of monitoring sites (200 to 400 m), interpreted geological structures (e.g. mafic dykes), landscape position and predicted locations of palaeodrainage lines.

The Quairading Shire Council had proposed the drilling program and preferred that monitoring sites be established on council-vested land.

Since dykes can inhibit groundwater flow, it was important to investigate their role.

Landscape position influences groundwater flow, and so it was important to monitor a range of positions. This was also valuable for setting up a computer groundwater flow model.

Locating the position of any palaeodrainage system was considered to be important as a palaeodrainage channel could contain thick aquifers with high hydraulic conductivity values, and such aquifers could be suitable targets for dewatering systems.

3.2.2 Drilling methods

LA Boyle Pty Ltd were contracted to drill the chosen sites and install piezometers, observation bores and one production bore. Most sites were drilled using reverse circulation methods. A 5"-diameter bit and hammer were used at the first three sites.
Because 'puggy' clays blocked the inner tube and sample hose, a 4"-diameter bit and hammer were used at the remaining holes.

The production bore (named 00QUPB1) and two shallow bores were drilled using rotary air blast methods. The production bore was drilled using an 8"-diameter bit and accompanying hammer.

Piezometers were generally drilled to bedrock ('deep' bores, with 'D' at the end of their name) and the observation bores ('shallow' bores, with 'S' at the end of their name) were installed either just below the watertable or within a perched or shallow groundwater system. Details are listed in Table 3-1.

### 3.2.3 Piezometer, observation bore and production bore construction

In both piezometers and observation bores, 50 mm-diameter class 9 PVC casing was installed. Two-metre lengths of slotted casing with 1 mm-aperture slots were positioned at the bottom of each hole and the remainder of the hole was 'plain cased'. The slotted section was gravel packed with 1.6 mm- to 3.2 mm-diameter washed graded gravel, and the piezometers were sealed using bentonite plugs. The remaining annulus of each hole, including the observation bores, was packed with drill chips. Bentonite seals were not installed in the annuluses of observation bores as the aim was to measure the levels of the shallow watertables. The production bore was constructed with 125 mm-diameter class 12 PVC casing. Slotted casing was installed over the lower 36 m of the 42 m-deep hole.

### 3.2.4 Drill sample analyses

One bulk sample and one chip tray sample were taken per metre from each bore. Descriptive logs were recorded and are available at [http://www.agric.wa.gov.au/environment/links/RMtechreports/](http://www.agric.wa.gov.au/environment/links/RMtechreports/). Chip tray samples are stored for geological reference at the Agriculture Western Australia office, Merredin (Dryland Research Institute).

### 3.2.5 Groundwater monitoring and sample analyses

Groundwater levels were measured and samples were taken each month. Agriculture Western Australia plans to continue monitoring for a total of 18 months. The salinities of the samples were analysed by measuring electrical conductivity (EC) at Agriculture Western Australia laboratories in South Perth. Results are stored on the Agriculture Western Australia AgBores database.

### 3.2.6 Surveying

Locations (eastings and northings) and elevations of piezometers and bores were surveyed using a differential global positioning system which was accurate to about ±10 mm both vertically and horizontally.
3.2.7 Pumping tests

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

Table 3-1. Depth details for monitoring sites and production bore

<table>
<thead>
<tr>
<th>Bore name</th>
<th>Drilled depth bgf (m)</th>
<th>Cased depth bgf (m)</th>
<th>Ground surface elevation above AHD## (m)</th>
<th>Intake interval depth bgf (m)</th>
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<tbody>
<tr>
<td>00QU01D</td>
<td>52 (49)***</td>
<td>52</td>
<td>246.20</td>
<td>50–52</td>
</tr>
<tr>
<td>00QU01S</td>
<td>10</td>
<td>10</td>
<td>246.21</td>
<td>8–10</td>
</tr>
<tr>
<td>00QU02D</td>
<td>23</td>
<td>23</td>
<td>250.98</td>
<td>21–23</td>
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<td>00QU02S</td>
<td>18</td>
<td>18</td>
<td>250.95</td>
<td>16–18</td>
</tr>
<tr>
<td>00QU03D</td>
<td>34 (31)***</td>
<td>34</td>
<td>255.26</td>
<td>32–34</td>
</tr>
<tr>
<td>00QU03S</td>
<td>9</td>
<td>9</td>
<td>255.25</td>
<td>7–9</td>
</tr>
<tr>
<td>00QU04D</td>
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<td>50 (46)***</td>
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<td>258.18</td>
<td>48–50</td>
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<td>6</td>
<td>258.25</td>
<td>4–6</td>
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<tr>
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<td>254.11</td>
<td>22–24</td>
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<td>254.11</td>
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</tr>
<tr>
<td>00QU07D</td>
<td>48 (40)***</td>
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<td>250.29</td>
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<td>249.49</td>
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<tr>
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<td>7–9</td>
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<tr>
<td>00QU10D</td>
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</tr>
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<td>00QU12D</td>
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<td>36</td>
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<td>9–11</td>
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<td>250.60</td>
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<td>31</td>
<td>246.27</td>
<td>29–31</td>
</tr>
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<td>10</td>
<td>246.26</td>
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</tr>
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<td>42 (36)***</td>
<td>42</td>
<td>246.11</td>
<td>12–42</td>
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</tbody>
</table>

Notes: #: bgf: below ground level; ##: AHD: Australian Height Datum; ###: number in brackets is depth at which fresh bedrock was struck
3.3 Results


3.3.1 Profile descriptions

A generalised profile is approximately 10 m of alluvium and colluvium over 10 to 15 m of kaolinitic clay or highly-weathered granite over 5 to 15 m of 'saprolite grits' or deep basal aquifer with fresh granite below. The thicknesses of the profiles can be seen in the cross-sections in Figure 3-1 and 3-2. At the sites where drilling reached bedrock (noted in Table 3-1), it was found at depths between 18 m (at site 00QU06) and about 52 m (at site 00QU10).

Figure 3-1. West to east cross-section through Quairading (see Figure 2-4 for location); groundwater levels, represented by d (deep piezometer) and s (shallow observation bore), measured on 20 November 2000
3.3.2 Groundwater data

Groundwater level measurements and EC values are listed in Table 3-2, and the changes in groundwater levels and salinities across the townsite are illustrated in Figure 2-4.

The groundwater levels in the piezometers and bores were related to topography and location relative to the postulated mafic dykes (Figure 2-4, 3-1and 3-2). Monitoring sites drilled on the north mid slopes, upslope of the dyke, had water levels 2.5 to 5 m below ground level (bgl) and bores on the mid to upper slopes had water levels from 7 to 29 m bgl. Bores on the valley floor, downslope of the postulated dykes, had water levels of 9 to 10 m bgl. The southern mid to upper slope bores had water levels ranging from 19 to 27 m bgl (bore 00QU11D was dry at 27 m bgl).

EC values throughout the townsite ranged from 140 to 1,330 mS/m in the deep bores and 363 to 3,240 mS/m in the shallow bores (Table 3-2).

3.3.3 Pumping test drawdowns

Details of the pumping tests are given in Appendix 1. The multi-rate test was planned as a four-steps, but the pumping rates used were too high for the aquifer conditions (between 0.2 and 0.4 L/s) and the test was stopped during the third step because the bore ran out of water (Figure 3-3).
Table 3-2. Groundwater levels and salinities for dates in winter and late spring (bgl: below ground level)

<table>
<thead>
<tr>
<th>Bore</th>
<th>27/06/00</th>
<th>20/11/00</th>
<th>27/06/00</th>
<th>20/11/00</th>
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</thead>
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<td>157</td>
<td>159</td>
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<td>6.90</td>
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<td>1810</td>
<td>2140</td>
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<td>13.68</td>
<td>575</td>
<td>604</td>
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</tr>
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<td>DRY</td>
<td>DRY</td>
<td>DRY</td>
</tr>
<tr>
<td>00QU11D</td>
<td>DRY</td>
<td>DRY</td>
<td>DRY</td>
<td>DRY</td>
</tr>
<tr>
<td>00QU12D</td>
<td>8.87</td>
<td>8.79</td>
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</tr>
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<td>7.80</td>
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<td>5.55</td>
<td>2700</td>
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</tr>
<tr>
<td>00QUPB1</td>
<td>9.37</td>
<td>9.32</td>
<td>1690</td>
<td>1770</td>
</tr>
</tbody>
</table>
Figure 3-3: Drawdown versus time for multi-rate test

The pumping rate for the constant rate test was also set too high (0.2 L/s) and was decreased after 40 minutes (to 0.15 L/s) because of large drawdowns (Figures 3-4 and 3-5). It appears from the plot in Figure 3-4 that the pumping rate was decreased on two or three later occasions. The drawdown curve also steepened several times (Figure 3-4). The early change (after about four minutes) may have been due to well storage effects, and any of the changes may have resulted from increases in discharge rates or from aquifer barrier effects. Flow meter readings supplied by Test Pumping Australia indicated that there was a slight increase in discharge rate between 210 and 360 minutes, which could explain the minor steepening of the drawdown about that time. However, the clearer steepening from about 540 minutes coincided with a decrease in discharge rate, indicating an aquifer barrier effect was the cause. No flow meter readings were made between one and 12 minutes, so the cause of the steepening curve during that period is not known.

Figure 3-4. Production bore drawdown versus time for constant rate test
Drawdowns were clear up to 100 m away from the production bore in the piezometers (Figure 3-5) but the shape of the ‘cone of depression’ could not be determined from the available monitoring sites. There were no substantial drawdowns in observation bores at the monitored sites.

![Figure 3-5. Drawdowns in monitoring piezometers and observation bores versus time for constant rate test (site 00QU15 was about 12 m from the production bore; site 00QU12 about 100 m from the production bore)](image)

The transmissivity of the pumped aquifer was estimated to be about 2 m$^2$/day (giving an average hydraulic conductivity of about 0.1 m/day) for the whole profile (see Appendix 1).

### 3.4 Interpretation and discussion

This section presents an interpretation of the recharge, groundwater flow and discharge processes affecting Quairading, based on the available information. It then discusses the risk of salinity and the options for managing the risk.

#### 3.4.1 Recharge

A simple zoning system for considering the sources of groundwater recharge affecting a townsite was applied to the towns in the Community Bores Project. It is described and then applied to Quairading.
3.4.1.1 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. the slopes directly 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.

Again, the degree of connection between the groundwater bodies below the two zones will influence the magnitude of the effect of the downslope zone on the townsite groundwater levels. Groundwater levels in the downslope zone may be influenced by rain falling on the zone, 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 only the available data. Also, the importance of different recharge processes will vary from year to year and from season to season. However, one generalisation can be made. If a townsite (or part of a townsite) clearly has negligible groundwater input from either slopes above or a valley floor below, but still has problems caused by high groundwater levels, then it can be concluded that the water causing the problems is recharged solely within the townsite (or that part of the townsite). This is the case in several of the 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 make contributions.
3.4.1.2 Quairading recharge zones

In Quairading, it seems reasonable to infer that the recharge within the *townsite zone* is substantial, but the installation of a sewer system in the northern part of the town in 2000 is expected to reduce the recharge in that area.

For Quairading, the topography and land uses in the catchment above and below the townsite can be used to infer whether the slopes above and the valley floor below the townsite are also likely to be important. Parts of the *slopes above the town* to north-west and north of the townsite are cleared of native vegetation so it is assumed that there is groundwater recharge below them. However, the groundwater levels indicate that there are groundwater barriers below the north-western part of the townsite, so groundwater water from the slopes zone may be building up below the north-western part of the townsite and may not be contributing to the groundwater bodies underneath the south and eastern parts of the town. The groundwater monitoring between June and November 2000 also shows that groundwater levels at many sites (e.g. 00QU3D, 00QU4D, 00QD09D, 00QU10D, 00QU12s, see Table 3-2) continued to rise steadily during spring, rather than falling when the winter rains ended. This could mean that groundwater recharged in other areas during winter was later flowing to these sites, or that shallow perched water recharged in winter was slowly percolating down, or that the source of the recharge was imported water.

The topography and available drilling data imply that groundwater systems in the *downslope zone* do not have a substantial effect on the groundwater levels below the town.

Long-term frequent and regular monitoring of groundwater levels in different parts of the catchment 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, and whether recharge occurring in the catchments to the north and north-west contributes to groundwater below the town. Therefore, the network is a valuable asset.

3.4.2 Groundwater flow systems and discharge

There are two groundwater systems within the Quairading town catchment:

1. There is a deep basal aquifer. It is characterised as the permeable zone of weathering above the fresh granite basement and is semi-confined by a kaolinitic clay layer above. It is typically limonitic, the feldspar weathering is less advanced than in the regolith above and iron deposits are evident on quartz and feldspar crystal surfaces. The iron staining suggests the material is quite permeable - possibly within the range of $10^{-2}$ to $10^{-1}$ m/day.

2. There is a silcrete layer of variable thickness and where groundwater collects above it, the layer forms the lower boundary of a shallow unconfined aquifer.

The main groundwater flow is considered to be in the deep aquifer.
Shallow groundwater in the north-western part of the town is thought to result from mafic dykes (possibly several subparallel ones trending between north-north-east and east-north-east) inhibiting groundwater flow.

The available groundwater levels show downward gradients at sites in the east (00QU12, 00QU15, 00QU01) but upward ones in the west (00QU07, 00QU09, 00QU13). In the eastern sites, the measurements from the shallower observation bores are considered to represent a perched aquifer. In the western sites, the higher water levels in the deeper piezometers show that there is limited hydraulic connection with the groundwater at shallower depths. Regular long-term monitoring is required to determine whether the pressures in the deeper aquifer are rising and pose a threat in locations where the confinement is weakest.

The shallow groundwaters were brackish to saline so were markedly more saline than the samples from the deep basal aquifer (which were fresh to brackish). This indicates a lack of vertical mixing and it is possible that salt leaches into the shallow groundwater systems from the clay-rich topsoil ('salmon gum/gimlet' soils).

The relative freshness of the deeper groundwater and the differences in groundwater levels in the bores and piezometers would be explained if groundwater in the basal aquifer was recharged through the deep sandplain profiles on the slopes above the town (sandplain stores relatively little salt), and if the basal aquifer was semi-confined downslope from the sandplain by thick clay residuum.

Groundwater discharge does not currently occur within the townsite.

### 3.4.3 Assessment of salinity risk

The shallowest groundwater level at a monitored site was about 3 m below ground level (00QU07D). The relatively high level is assumed to result from a mafic dyke impeding groundwater flow. If this is the case, and the groundwater levels are rising, then there is a risk of salinity developing in the north-western part of the town.

Below other parts of the town, groundwater levels are currently too deep to cause salinity damage, but it is not known whether they are rising, and if so, at what rate.

Regular and frequent groundwater level measurements are required to determine the risk more accurately.

### 3.5 Management options

Currently, Quairading does not appear to have a salinity problem, although there are areas at risk. The Quairading community has the opportunity to take steps to manage the risk of salinity before it becomes a problem.

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.

From other towns, it is assumed that water management within the townsite zone is important. 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 may have other benefits, such as reduced water supply costs and dependence, less waste of good quality rain water, and less infrastructure damage from floods and surface run-on. Some measures to consider are:

- continuing the replacement of septic systems with a sewer system;
- preventing surface water from ponding in areas where it may become recharge (see Section 5);
- monitoring the amount of water required by gardens, parks and sports grounds and eliminating overwatering;
- checking for and mending leaks in water pipes, drains, culverts, dams and pools;
- replacing grass and weeds 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.

There is also the opportunity to reduce recharge on the agricultural land to the north-west of the townsite by using surface water control structures to eliminate waterlogging (which should have the added benefit of increasing production) and by increasing the area planted to perennials.

As the groundwater is currently deep below most of the town, its abstraction is only relevant upslope of the groundwater barriers to the west and north-west of the town. The groundwater pumping test was performed near the centre of the town, and had only a small impact on the watertable (a fall of about 0.1 m only 14 m away after three days of pumping). Groundwater drainage is unlikely to be effective as it only lowers groundwater levels along narrow zones either side of the drain, and thewatertables are currently greater than 3 m deep. As groundwater abstraction using pumping or drainage is expensive, may cause settlement damage to infrastructure, and the removed water has to be carefully used or evaporated to avoid causing groundwater problems elsewhere, it does not appear to be a viable option for the west and north-west of the town. However, if sited appropriately, revegetation with perennial plants may play an abstraction role as well as the recharge reduction role mentioned above.
4. Groundwater flow modelling

Authors: Anthony Barr and Daniel Pollock, CSIRO Land and Water, Perth

Section 3.5 discussed a combination of management approaches which could be effective in Quairading. This section describes a computer groundwater modelling study that aimed to assess the impacts of a selection of possible strategies.

Firstly, 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 used to simulate the effects of three different strategies: ‘do nothing differently’, groundwater abstraction and tree planting. Groundwater drainage was not tested as a potential management option in this town as the depth to watertable in the model exceeded 5 m for the whole region.

Sections 4.1 and 4.2 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 4.3.

4.1 Model construction and conceptualisation

Conceptually, the groundwater model consisted of two layers: colluvium and clays (‘Layer 1’) over saprolitic grits (‘Layer 2’). Inflow to the model domain, illustrated in Figure 4-1, was assumed to be through lateral flow from the north-west boundary of the region modelled. Discharge from the area was assumed to be through the southern and eastern boundaries. The model domain extended 1.40 km from east to west (537 037.7 mE to 538 437.7 mE Australian Geodetic Datum 1984 (AGD84)) and 1.68 km north to south from 6 457 480 mN to 6 459 160 mN (AGD84). This incorporated the majority of monitoring sites in the town. Each cell in the domain was 20 m by 20 m, resulting in 70 columns and 84 rows for a total of 5,880 cells.

The top of the uppermost layer was taken as the land surface, which was extracted from 2 m-contour digital elevation models for the catchment (map sheet 23331NE, Spatial Resource Information Group, Agriculture Western Australia). The depth of each model layer was interpolated using inverse distance weighting to a 25 m 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.
4.2 Steady-state model calibration

Because watertables in many locations in the region around Quairading are known to be rising (Section 2.5), it was assumed that groundwater underneath the town was also in a state of flux. However, the absence of long-term water level records within the town meant that some assumptions had to be made about the system. It was assumed in this groundwater modelling that the heads measured on 31 July 2000 were indicative of the steady-state groundwater system under the current climatic conditions.
and land-use conditions. 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 these quantities can be estimated from the pumping test results for the hydraulic conductivity (see Appendix 1) and from the average annual increase in the watertable for the recharge. The pumping test calculated the hydraulic conductivity of the system to be approximately 0.5 m/day, which is consistent with the results of George (1992) for the saprolitic grits. However, for the clays, George estimated that the conductivity was an order of magnitude less. 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 predicted in this modelling will be quicker than the actual response time of the aquifer. However, without longer data sets 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.

Conceptualising the groundwater flow pattern for this town was quite difficult. The drilling program provided the regolith structure and depths to basement at the bore locations. Interpolation of these results suggested that the town is located over bedrock highs in the north and south with a bedrock low from the south-west through the centre of the town and heading east. This may indicate the location of a palaeochannel. Using this information, it was thought that the groundwater flow was from west to east in the town. However, examining the head distribution indicated that the flow is mainly from the north-west to the south and the east. Therefore, the groundwater model constructed for Quairading used a general head boundary in the north-west, with an external head of 255 m above AHD, based on the head in a nearby bore, and a conductance of 25.0 m²/d for the top layer and 7.0 m²/d for the bottom layer. The southern boundary was also simulated through a general head boundary consisting of an external head of 230 m above AHD and a conductance of 25.0 m²/d for the top layer and 7.0 m²/d for the bottom layer. The possible presence of palaeochannel sands was simulated through a general head boundary on the eastern side of the domain with an external head of 230 m above AHD and a conductance of 50.0 m²/d for the bottom layer only. The remaining boundaries were taken as no-flow boundaries.

Recharge was applied uniformly over the modelled region and based on 5 per cent of the annual average rainfall of the nearby townsite of Corrigin of 378.3 mm (Bureau of Meteorology 2000).

Calibration of the steady-state model was accepted with a standard error of the estimate of 0.71 m for all layers compared with the heads measured on 31 July 2000. The parameters used to achieve this are listed in Table 4-1. The high standard error was due to the presence of dykes in the region, which have a major effect on groundwater flow. The locations of these dykes are poorly-defined and were not included in the groundwater model. 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 4-2 and along a north to south
cross-section through the centre of the region in Figure 4-3. Currently, there is no groundwater within 8 m of the ground surface. This indicates that the observed surface ponding behind the railway embankment is caused by a local perched aquifer, slow infiltration, or the presence of dykes in the area. Travel times below the townsite started at 40 years, 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 basal layer.

The model is quite sensitive to the selection of hydraulic conductivity and 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 would also fit the measured levels.

Figure 4-2. Depth to groundwater (in metres, contour intervals are 1.0 m) for steady-state simulation
Figure 4-3. South to north cross-section through centre of the domain (Figure 4-1) for all simulations

Table 4-1. Parameters used for the Quairading model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal hydraulic conductivity (m/day)</td>
<td>1.0</td>
</tr>
<tr>
<td>Vertical hydraulic conductivity (m/day)</td>
<td>0.01</td>
</tr>
<tr>
<td>Storativity (m$^{-1}$)</td>
<td>0.001</td>
</tr>
<tr>
<td>Effective porosity</td>
<td>0.1</td>
</tr>
<tr>
<td>Recharge (mm/year)</td>
<td>18.9</td>
</tr>
<tr>
<td>Groundwater evaporation (mm/year)</td>
<td>365</td>
</tr>
<tr>
<td>Groundwater evaporation extinction depth (m)</td>
<td>1.0</td>
</tr>
</tbody>
</table>

4.3 Dynamic simulations of strategies

The dynamic simulations extended over 30-year periods. In this period, the general head boundary external heads were raised at a rate of 0.1 m/y.
4.3.1 ‘Do nothing differently’ strategy

The ‘do nothing differently’ strategy assumed that no changes in groundwater management would occur. The elevation of the watertable along a cross-section through the town is shown in Figure 4-3. The resulting depth to the watertable after 30 years is shown in Figure 4-4. This model predicted that the watertable would be 7 m or more below the ground surface over the whole region. This implies that the town is not at risk if the watertable is rising at the assumed rate.

![Figure 4-4. Depth to groundwater after 30 years (in metres, contour intervals are 1.0 m) for ‘do nothing differently’ simulation](image)

4.3.2 Groundwater abstraction strategy

Groundwater abstraction through three wells located within the town was tested in the model as a potential management option (Figure 4-5). The pumping rate used for the pumping test (see Appendix 1) was 0.15 L/s or 12.9 m³/d. Therefore, the well in the model was assigned a discharge rate of 10 m³/d.
The resulting modelled depth to groundwater after 30 years is shown in Figure 4-6 and the watertable elevation along a cross-section can be seen in Figure 4-3. For the conditions modelled, after 30 years the depth to the watertable was greater than 7 m for the whole town.

Figure 4-5. Modelled region with management options (the circular objects labelled P1 to P3 represent locations of simulated pumps, the dark area in the north-west represents area planted to trees, boundary scales are in metres, top of map is north)
Figure 4-6. Depth to groundwater after 30 years (in metres, contour intervals are 1.0 m) for pumping simulation

4.3.3 Tree planting strategy

Tree planting was also tested in the model as a potential management option. The modelled area of tree planting was in the north-west of the modelled region. 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 4-7 and the watertable elevation along a cross-section is shown in Figure 4-3. This option slightly reduced the area where the depth to groundwater was less than 7.0 m, compared to the ‘do nothing differently’ strategy.
Figure 4-7. Depth to groundwater after 30 years (in metres, contour intervals are 1.0 m) for tree planting simulation

4.3.4 Discussion of groundwater modelling

The groundwater modelling in Quairading was undertaken using limited data. Therefore, the results are indicative only and may not represent what is happening in the town. The model was calibrated in steady-state against the heads measured at the end of July 2000. The assumption of a steady-state groundwater system is inappropriate, but represents the best method for applying a groundwater model to the town. The measured depths to groundwater indicate that the western part of the town has a deep watertable. Even if the watertable rises at the regional rate of 10 cm/year for 30 years, the watertable will still be less than 6 m beneath the town. If the rate of watertable rise is substantially faster, then it may reach the surface within the 30 year period. The results showed that tree planting had very little effect.
5. Flood risk analysis

Author: Travis Cattlin, Catchment Hydrology Group, Agriculture Western Australia

5.1 Objective of this study and approach

The objective of this part of the Community Bores Project was to assess the flood risk (high, moderate or low) of the town. This was done by calculating the peak flood flow generated by the agricultural catchments of the town (at a point just downstream of the townsite) and the volume of run-off that could be generated both within the total catchment above the town and 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 agricultural catchment 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 agricultural catchment peak flows were calculated for 24-hour storms and the townsite run-off volumes were calculated for 1- and 6-hour storms for 2-, 5-, 10-, 20-, 50- and 100-year average recurrence intervals (ARIs) based on historical events.

5.2 Input data

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

5.2.1 Available information

The following information was collated for the Quairading catchments:

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

5.2.2 On-site observations

5.2.2.1 High run-off areas

A granite outcrop above slopes with clay-rich soils (which tend to shed water rapidly) at the head of the catchment drains through the middle of the town (Figure 2-2). Houses, roads, the grain depot, and industrial and retail buildings, which are considered to generate stormwater run-off, cover about 230,000 m² (about 30 per cent of the town). In the townsite run-off volume calculations, the run-off coefficient for these areas was assumed to be 0.9, with minimal interception from containing structures like rainwater tanks or dams.
5.2.2.2 Structures influencing surface water flow

The railway line and the adjacent water pipeline inhibit the flow of the main drainage line through the town. Adjoining land is commonly inundated during winter and after intense summer rainfall events.

The catchment to the north-west of the town has natural drainage characteristics. Another small tributary, which flows near and parallel to the Cunderdin Road, is channelled and has culverts before it enters the main drainage line. This limits inundation by rainfall events with intensities of less than about 15-years ARI (based on peak flow and waterway width analysis). The natural drainage line discharges along the railway line and the main street. This cause inundation, waterlogging and is assumed to result in recharge of the underlying shallow groundwater system.

The grain depot has waterlogging and inundation problems due to the main drainage line discharging run-off water around the bin. Roof run-off is discharged directly onto the ground, and is assumed to be another source of groundwater recharge.

Stormwater run-off from town buildings, roads and houses is thought to contribute to the inundation around the railway. Stormwater infrastructure was being improved at the time of investigation and the septic systems in the north-western part of the town were being replaced by a mains sewer system.

5.2.2.3 Waterways

A large gully dam constructed along the main drainage line by the shire mitigates the velocity of catchment run-off. This minimises erosion and silting. The dam’s purpose is to supply fresh water to irrigate the local recreation ground.

As the waterway proceeds through the town, it is narrow and this results in the sports ground flooding and being waterlogged in average winter rainfall periods.

Waterway definition in all tributaries is excellent above and below the sporting complex area. They are grassed and appear stable due to no visual erosion and minimal silting. The main drainage line requires widening and definition around the sporting complex to allow rainfall events with ARIs greater than 10 years (based on peak flow and waterway width analysis) to discharge without flooding.

5.2.3 Information derived for peak flow model

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 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.

5.2.4 Peak flow model calibration

The UDD model should be calibrated using measured flow data. However, there is no gauging station in the catchment that contains the town of Quairading. Moora is the closest town to Quairading for which reliable flow records are available. The calibration achieved for the UDD model for the town of Moora was assumed to be valid for Quairading too.

5.3 Results

5.3.1 Peak flood flow

Table 5-1 shows approximate peak flows for rainfall events of various ARIs. The results suggest that rainfall run-off attributed to the town catchment would cause minimal erosion and silting in ARIs of less than 10 years. This is due to the waterways being wide enough to contain 1:10 year flows at a velocity that will minimise erosion. They will not prevent erosion as velocity would exceed 1 m/s.

Table 5-1. Peak flood flow for storms with a range of ARIs for the catchment of the town of Quairading

<table>
<thead>
<tr>
<th>ARI (years)</th>
<th>Peak flood (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>6.5</td>
</tr>
<tr>
<td>5</td>
<td>13</td>
</tr>
<tr>
<td>10</td>
<td>28.5</td>
</tr>
<tr>
<td>20</td>
<td>44</td>
</tr>
<tr>
<td>50</td>
<td>52</td>
</tr>
<tr>
<td>100</td>
<td>72</td>
</tr>
</tbody>
</table>

5.4 Run-off volumes

The results of the run-off volume calculations for the catchment and the townsite are in Table 5-2. The run-off volumes do not take evapotranspiration into account.

5.5 Flood risk assessment

Quairading can be classified as a high-risk town for flooding, inundation and waterlogging. This assessment was based on the assumption that rainfall events of 10- or 15-year ARIs would cause some inundation problems because of poorly-defined drainage lines.
Table 5-2. Run-off volumes generated by rainfalls of various ARIs, durations and intensities for the catchment of Quairading (area of 1,275 ha) assuming a run-off coefficient of 0.1 and the high run-off surfaces (total area of 23 ha) in the townsite assuming a run-off coefficient of 0.9

<table>
<thead>
<tr>
<th>Average recurrence interval (years)</th>
<th>Rainfall duration (h)</th>
<th>Rainfall intensity (mm/h)</th>
<th>Rainfall intensity (mm)</th>
<th>Catchment run-off volume (m$^3$)</th>
<th>Townsite run-off volume (m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1</td>
<td>15.0</td>
<td>15.0</td>
<td>19,800</td>
<td>3,200</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>4.1</td>
<td>24.6</td>
<td>29,100</td>
<td>4,800</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>19.8</td>
<td>19.8</td>
<td>23,000</td>
<td>3,800</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>5.9</td>
<td>35.4</td>
<td>37,900</td>
<td>6,200</td>
</tr>
<tr>
<td>10</td>
<td>1</td>
<td>24.0</td>
<td>24.0</td>
<td>26,800</td>
<td>4,400</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>7.0</td>
<td>42.0</td>
<td>52,000</td>
<td>8,500</td>
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<tr>
<td>20</td>
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<td>29.0</td>
<td>29.0</td>
<td>31,900</td>
<td>5,200</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>8.6</td>
<td>51.6</td>
<td>60,400</td>
<td>9,900</td>
</tr>
<tr>
<td>50</td>
<td>1</td>
<td>37.0</td>
<td>37.0</td>
<td>38,300</td>
<td>6,300</td>
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<td></td>
<td>6</td>
<td>11.0</td>
<td>66.0</td>
<td>72,700</td>
<td>11,900</td>
</tr>
<tr>
<td>100</td>
<td>1</td>
<td>44.0</td>
<td>44.0</td>
<td>44,600</td>
<td>7,300</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>14.0</td>
<td>84.0</td>
<td>91,800</td>
<td>15,000</td>
</tr>
</tbody>
</table>

5.6 Recommendations for flood risk management

The town catchment requires surface water engineering to reduce the peak flow, reduce erosion of drainage lines and reduce the impact of flooding following rainfall events of greater than 10-year ARIs. This can be achieved by improving the surface flow through culverts (especially under the railway line), engineering grade banks, introducing grassed stable waterways, and installing levees or w-drains along the main drainage line to promote flow through the sporting complex and into the Yenyenning Salt Lakes system.

Waterway improvement designs should aim to control erosion in a 20-year ARI rainfall event. Designing for any greater ARI may be impractical and expensive.

5.7 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 described in Ali et al. (2001).
6. Summary of recommendations

1. Reduce townsite recharge where there are additional benefits in doing so, giving particular regard to:
   - surface water management (see Section 5.6 for details);
   - revegetation;
   - eliminating overwatering of gardens, parks and sports grounds;
   - continuing the replacement of septic systems with a sewer system; and
   - eliminating leaks in water pipes, drains, culverts, dams and pools.

2. Reduce recharge below the slopes above the townsite by
   - surface water management; and
   - revegetation.

3. Measure groundwater levels in the monitoring network monthly and analyse and review them annually, and continue to do so for at least 10 years to determine whether groundwater problems are worsening and where and when most recharge occurs.
7. Acknowledgments

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

8. References


9. Appendix 1: Pumping test

Author: Ron Colman, Test Pumping Australia

As part of the hydrological investigation of Quairading, a pumping test was carried out in the production bore (00QUPB1). It aimed to establish aquifer parameters for use in the groundwater modelling study (Section 4).

9.1 Method

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

Two parts to the test were performed between 16 and 20 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 were assessed before setting the pump rate for the second part, which was a constant rate test.

For the constant rate test, the bore was pumped at a constant discharge rate for 4,320 minutes (72 hours) and the drawdowns in the production bore and in two piezometers and two observation bores (at sites 00QU12 and 00QU15) 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 9-1 summarises relevant details.

Table 9-1. Details of the pumping test

| Pump inlet depth below ground level | 38 m |
| Available drawdown in production bore | 28 m |
| Pump | electric submersible |

9.2 Results

9.2.1 Multi-rate test

The static water level in the bore before the test began was 9.83 m below the reference point (which was 0.45 m above ground level). The multi-rate test was conducted on 16 August 2000 and four 30-minute steps were planned. However, the pump ran out of water after 72.5 minutes, during the third step. The discharge rates for the three steps were 0.2, 0.3 and 0.4 L/s. The total drawdown in the production bore at the end of the multi-rate test was recorded as 26.7 m.

9.2.2 Constant rate test

The constant rate test started on 17 August 2000 and lasted 4,320 minutes. The initial pumping rate was 0.2 L/s, but was reduced to 0.15 L/s after 40 minutes. The
drawdown in the production bore at the end of the test was 11.88 m. Drawdown and
distance details are listed in Table 9-2.

Table 9-2. Measurement distances from pump and drawdowns in the
production bore, monitoring piezometers and bores for constant rate test

<table>
<thead>
<tr>
<th>Bore name</th>
<th>Lateral distance from pump (m)</th>
<th>Bearing from pump (degrees)</th>
<th>Height of measuring datum agl* (m)</th>
<th>Initial groundwater level depth bmd## (m)</th>
<th>Final drawdown (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>00QUPB1</td>
<td>0.10</td>
<td>0.45</td>
<td>9.950</td>
<td>11.880</td>
<td></td>
</tr>
<tr>
<td>00QU15D</td>
<td>11.25</td>
<td>222</td>
<td>0.50</td>
<td>9.565</td>
<td>2.694</td>
</tr>
<tr>
<td>00QU15S</td>
<td>13.75</td>
<td>222</td>
<td>0.50</td>
<td>5.850</td>
<td>0.095###</td>
</tr>
<tr>
<td>00QU12D</td>
<td>99.00</td>
<td>197</td>
<td>0.50</td>
<td>9.390</td>
<td>0.355</td>
</tr>
<tr>
<td>00QU12S</td>
<td>100.00</td>
<td>197</td>
<td>0.50</td>
<td>7.945</td>
<td>0.055###</td>
</tr>
</tbody>
</table>

Notes: #: agl: above ground level; ##: bmd: below measuring datum; ###: appeared due to barometric fluctuations rather than pumping effects

The drawdown data were analysed using computerised methods designed for confined and unconfined aquifers only. A summary of results is presented in Tables 9-3 and 9-4.

Table 9-3. Transmissivity values calculated using different analyses for the Quairading production bore and nearby monitoring sites

<table>
<thead>
<tr>
<th>Bore name</th>
<th>Intake interval above AHD (to nearest metre)</th>
<th>Cooper and Jacob (time-drawdown)</th>
<th>Theis (curve fitting)</th>
<th>Theis &amp; Jacob recovery</th>
</tr>
</thead>
<tbody>
<tr>
<td>00QUPB1</td>
<td>204—240</td>
<td>1.12</td>
<td>0.86</td>
<td>1.94</td>
</tr>
<tr>
<td>00QU15D</td>
<td>215—217</td>
<td>2.16</td>
<td>1.66</td>
<td>1.16</td>
</tr>
<tr>
<td>00QU15S</td>
<td>236—238</td>
<td>NV#</td>
<td>NV#</td>
<td>NV#</td>
</tr>
<tr>
<td>00QU12D</td>
<td>210—212</td>
<td>7.43</td>
<td>6.08</td>
<td>NR##</td>
</tr>
<tr>
<td>00QU12S</td>
<td>235—237</td>
<td>NV#</td>
<td>NV#</td>
<td>NV#</td>
</tr>
</tbody>
</table>

Notes: #: NV: not valid as no clear pumping drawdown; ##: NR: insufficient recovery

Projection of the data obtained during the constant rate pumping test indicated that the bore is capable of maintaining a long-term abstraction rate of 0.15 L/s. At this rate, it is likely that the drawdown effects from pumping (e.g. the cone of depression) could be detected up to 200 m from the production bore.

Table 9-4. Summary of measurements and calculated parameters

<table>
<thead>
<tr>
<th>Parameter or measurement</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aquifer thickness (m)</td>
<td>32</td>
</tr>
<tr>
<td>Electrical conductivity (mS/m)</td>
<td>636</td>
</tr>
<tr>
<td>Acidity (pH)</td>
<td>6.3</td>
</tr>
</tbody>
</table>