Soil-Aquifer Treatment Project
Groundwater Modelling

A report prepared by DIPE for the Power & Water Corporation

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Executive Summary

Two key components to the successful implementation of a soil-aquifer treatment scheme at the AZRI site are:

- adequate physical capacity of the aquifer to store the required volume of reclaimed water, whilst maintaining groundwater levels at a sufficient depth beneath the basin and surrounding topographic lows, and
- recoverability of infiltrated water from production wells

Numerical modelling provides a means of assessing the significance of these issues as well as a useful tool for predicting the hydraulic response of future operation strategies of soil-aquifer treatment.

The objectives of the modelling are to:

- predict the hydraulic impact of a 600 ML/yr SAT scheme with respect to storage, mounding and recoverability of the infiltrated water;
- identify the key parameters that may constrain the success of SAT and where possible, to identify the range of values which may be problematic; and
- recommend the future investigations needed to ensure the viability of a 600 ML/yr soil-aquifer treatment scheme.

The conceptual model development was based upon the work presented by Knapton et al., (2004a). The following conclusions were drawn from the modeling to date:

- Numerical modelling indicate that the infiltration of 600 ML/yr at the Pilot Soil-Aquifer Treatment scheme is feasible in terms of available storage, the mound response beneath the infiltration basins and the impact of the mounding on topographic lows to the north east of the site, namely the Todd River.
- The resultant mound height of 6 metres can be expected beneath “Kidney” shaped infiltration basins with surficial infiltration rates of 0.3 m/d, and an increase of 3.5 metres can be expected at the north western boundary. This equates to approximately 10 metres below both the basins and the surface elevation at the north western boundary.
- Analytical and numerical modelling indicate that hydraulic conductivities of between 20 and 50 m/d are required to simulate the mounding observed during an 18 ML field trial at the AZRI site. This is comparable to the values determined in previous studies in similar sediments.
• Lateral migration perpendicular to the flow direction would be relatively small and contained largely to the palaeochannel feature, which is the target for SAT operations. However, the migration of the plume down gradient could be quite considerable if no water is extracted for reuse with an advective velocity of approximately 300 metres per year.

• Under a regime where extraction volumes are comparable or greater than the infiltrated volume, little reclaimed water is likely to move beyond the extraction bores, which can be placed to maximise capture. The extraction bores will also reduce the mounding due to the infiltration.

The model describes a simplified concept of the shallow Quaternary aquifer, however, has highlighted some shortfalls in the present understanding of the area. The next steps are to:

• Modify the localised model to fit the infiltration data as it becomes available.

• Develop the unsaturated zone of the model to better represent the conditions observed on site.

• Develop a regional model that incorporates conservative solute transport to determine the extent of plume migration and degree of interception by pumping bores over the long term.

• As part of the regional model determine the relative contributions to recharge from the various sources identified.

• Develop and maintain a monitoring program of the AZRI site and infiltration sites to provide further validation of the model beyond the initial investigation program.
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1. Introduction

1.1 Background

The Alice Springs Water Reuse Scheme (ASWRS) is a joint initiative of Power Water Corporation, the Department of Business, Industry and Resource Development and the Department of Infrastructure, Planning and Environment (DIPE), with technical support from CSIRO Land and Water (CLW). The project aims to develop a water recycling scheme for the township of Alice Springs by making productive use of 1200-1800 ML (million litres) of wastewater per year.

Stage 1 of ASWRS proposes to:

- provide additional treatment of the effluent from the Alice Springs Waste Stabilisation Ponds to a standard suitable for irrigation use
- pipe this recycled water approximately 8 kilometres to the Arid Zone Research Institute (AZRI) site
- utilise soil-aquifer treatment (SAT) to recharge and polish approximately 600 ML/yr of the reclaimed water into the underlying shallow aquifer initially for 3 years
- extract the polished water from nearby bores to supplement irrigation supplies

Key issues which need to be addressed regarding the feasibility of soil-aquifer treatment are the location of:

- Soil profiles which prevent clogging beneath the surficial layer and that allow for practicable infiltration rates.
- Suitable aquifer(s) below the infiltration sites, with the following attributes;
  - unconfined aquifer to allow for the direct connection between the unsaturated and saturated zones
  - sufficient groundwater storage to meet project needs (initially 600 ML/yr for 3 years)
  - the top of the recharge mound(s) do not intersect the base of the infiltration basins
  - recovery site(s) in close proximity to the basins such that they are, sufficiently removed for minimum residence time and that maximum recovery of input water can be achieved
- Site(s) which can meet the cultural, social, regulatory requirements and irrigation water demands

Since July 2003 DIPE and CLW have jointly investigated the technical feasibility of SAT at the Arid Zone Research Institute (AZRI), located in the western portion of the “Outer Farm Area”.


Works entailed a series of laboratory studies on clogging, site soil and hydrogeological characterisation, field infiltration trials, groundwater modelling of a conceptual scheme and review of the literature on processes affecting quality of infiltrated water in the subsurface, with the intention of informing the design of a SAT scheme operating at the 600 ML/yr scale.

If deemed suitable these technologies will then be used in the treatment and storage of reclaimed water from the Alice Springs Waste Water Treatment Plant.

1.2 Objectives
As stated previously two key components to the successful implementation of a soil-aquifer treatment scheme at the AZRI site are:

- adequate storage capacity of the aquifer to recharge the required volume whilst maintaining groundwater levels at a sufficient depth beneath the basin and surrounding topographic lows, and
- recoverability of infiltrated water from production wells due to extended travel times or excessive mixing with more saline ambient groundwater.

Numerical modelling provides a means of assessing the significance of these issues as well as a useful tool for predicting the hydraulic response of future operation strategies of SAT.

The objectives of the modelling are to:

- predict the hydraulic impact of a 600 ML/yr SAT scheme;
- identify the key parameters that may constrain the success of SAT and where possible, to identify the range of values which may be problematic; and
- recommend the future investigations needed to ensure the viability of a 600 ML/yr soil-aquifer treatment scheme.

1.3 Model Steps
1) Model development
2) Calibrate the Steady State Model to observation data at AZRI.
3) Calibrate the model to the transient data from the infiltration trials described by Knapton et al., (2004a) and Knapton et al., (2004b).
4) Sensitivity analysis, ie determine what are the key assumptions which have a significant impact on the predictions.
5) Use the calibrated model to predict response in the long term to different infiltration scenarios.

a) Steady state simulation of the infiltration at 600 ML/yr under two hydraulic loading regimes:

- Square basins where the infiltration rate is the same as that observed during the infiltration trials in basin A (3 m/d)

- “Kidney” basins where the infiltration rate has been reduced to approximately 0.3 m/d due to clogging processes.

b) Steady state particle tracking of infiltration under different extraction regimes for the two basin geometries
2. Hydrogeological Setting and Conceptual Model

2.1 Hydrogeology
The geology of the area has been summarised by Knapton et al., (2004), Stevens, (1986) and Quinlan and Woolley, (1969). The surficial aquifer of interest to the soil-aquifer treatment scheme is the relatively thin layer (30-33 metres maximum thickness) of Quaternary aged sediments. The Quaternary sediments are underlain by a thick sequence (up to 300 metres) of Tertiary aged clays with several thin sandy aquifers. The thin aquifers within the Tertiary clays are confined and separated from the Quaternary sediments by the intervening clay aquiclude. The palaeochannel feature identified by Berry, (1991) and Quinlan and Woolley, (1969) has been located at AZRI and is considered the major aquifer of the Quaternary sediments.

2.2 Observation Data
Historical groundwater hydrograph data is sparse within the Outer Farm Basin, at least within the Quaternary sediments. Observation bores within the Quaternary sediments and located in and around AZRI are tabulated below (Table 2.1):

<table>
<thead>
<tr>
<th>Registration No.</th>
<th>Model Layer</th>
<th>Easting [m]</th>
<th>Northing [m]</th>
<th>MP RL [mAHD]</th>
<th>May 2004 SWL [mAHD]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RN17849</td>
<td>2</td>
<td>385999.0</td>
<td>7371765.0</td>
<td>544.4</td>
<td></td>
</tr>
<tr>
<td>RN17936</td>
<td>2</td>
<td>387655.8</td>
<td>7370333.4</td>
<td>549.4</td>
<td>532.4</td>
</tr>
<tr>
<td>RN17937</td>
<td>2</td>
<td>386990.4</td>
<td>7371152.9</td>
<td>552.5</td>
<td>537.2</td>
</tr>
<tr>
<td>RN17938</td>
<td>2</td>
<td>386824.7</td>
<td>7371501.9</td>
<td>553.7</td>
<td>540.3</td>
</tr>
<tr>
<td>RN17939</td>
<td>2</td>
<td>386473.5</td>
<td>7371996.8</td>
<td>555.4</td>
<td>545.0</td>
</tr>
<tr>
<td>RN17942</td>
<td>2</td>
<td>387244.0*</td>
<td>7370890.0*</td>
<td>552.9</td>
<td>536.2</td>
</tr>
<tr>
<td>RN17943</td>
<td>2</td>
<td>387317.0*</td>
<td>7370722.0*</td>
<td>552.2</td>
<td>535.6</td>
</tr>
<tr>
<td>RN17989</td>
<td>2</td>
<td>387880.2</td>
<td>7371773.1</td>
<td>551.7</td>
<td>537.9</td>
</tr>
<tr>
<td>RN17952-18</td>
<td>1</td>
<td>387238.0*</td>
<td>7371096.0*</td>
<td>553.2</td>
<td></td>
</tr>
<tr>
<td>RN17953</td>
<td>2</td>
<td>387241.0*</td>
<td>7371098.0*</td>
<td>553.1</td>
<td></td>
</tr>
</tbody>
</table>

Note: All bore positions collected using differential GPS except (*)
MP = measuring point

2.3 Groundwater Levels

2.3.1 Quaternary aquifer groundwater hydrographs
The groundwater levels across the site have been recorded over the past 12 months since October 2003. The hydrographs from the north western portion of the site (RN17939 and RN17938) show the greatest variation (1.76 metres) compared to the south eastern area where RN17936 shows the least variation, with an overall decrease of 0.48 metres. The period from October 2003 to May 2004 shows the greatest decline in heads. The heads for May 2004, were taken a week or so prior to the recharge event due to the flow in the Todd River, hence the relatively small apparent decline
in heads over the similar period of time from May to November 2004 (note the head rise at RN17940 due to the point recharge at the infiltration trials).

![Graph of groundwater level decline](image)

**Figure 2.1** Decline in groundwater level along the axis of the palaeochannel with time.

2.3.2 Groundwater head distribution
The head distribution across the site has been interpolated from the monitoring bores within the AZRI property and historic monitoring bores to the north west of the AZRI site. The groundwater levels decrease to the southeast from Blatherskite Gap and exhibit a gradient of approximately 0.006. Groundwater contours for May 2004 are presented in **Figure 2.2**.
2.4 Conceptual Model

2.4.1 Hydrostratigraphic Units

The hydrostratigraphy employed in the model is based on the lithological logs from the at the AZRI site investigations described by Knapton and Lennartz, (2005). The investigations have determined that the target aquifer for the soil-aquifer treatment system is a palaeochannel of coarse grained sediments, overlain by finer grained sediments of clayey silts, clays and sands. A summary of the site stratigraphy is listed below:
- Basal sequence of relatively impervious clay from between 18 to 33 mBGL where the palaeochannel is absent and 33 mBGL where the channel is present.

- Incised channel feature of inter-layered coarse and fine grained sediments 18-33 mBGL

- Relatively consistent thickness of fine grained sediments from 12-18 mBGL.

- Medium to very coarse grained sediments from 0-12 mBGL.

Groundwater flow is to the southeast with the water table 9 metres below ground level in the north west and 17 metres below ground level in the south eastern portion of the site and approaches the top of the palaeochannel aquifer. The water table is above the contact between the finer textured sediments and the palaeochannel sediments, therefore, the palaeochannel aquifer should be considered as unconfined to semi-unconfined.

Conceptually, therefore, the saturated component of the site can be considered as a two layer system, with the palaeochannel represented as a fully saturated, unconfined to semi-unconfined high permeability aquifer of semi-infinite extent (infinitely long and approximately 400 metres wide), overlain by a less permeable layer of infinite extent with a variable saturated thickness.

The diagrammatic plan and cross-section of the conceptual model are presented in Figure 2.3 and Figure 2.4 respectively.
Figure 2.3 Plan view of the AZRI site depicting the location of the palaeochannel feature. Section A-A' is depicted in Figure 2.3.
Figure 2.4  Cross-section A-A' of the conceptual model.

### 2.4.2 Water balance calculations

The water balance estimate is to be used to set bounds on the permissible range of hydraulic parameters. Knapton, (2005) determined from the input components recharging the Inner Farm Basin an estimate of the throughflow entering the Outer Farm basin for the year 2002. The throughflow is between 500 and 800 ML/yr. Jueken, (2004) determined that from the geochemical analysis that the contribution to the water balance due to diffuse recharge from rainfall was considered minimal. Recharge from the Todd River, which for the flow of 4 days duration was estimated at 90 ML. The component of recharge from the portion of the Todd in the Outer Farm basin where flow occurred is also estimated at 90 ML for 2002. Overflows into Ilparpa Swamp and the subsequent infiltration along St Mary’s Creek were not factored as the flows from Ilparpa to St Mary’s Creek were not recorded, but may have contributed up to 100 ML. Discharge of groundwater at the surface through evaporation or transpiration over the model area is considered to be minimal and the groundwater output is essentially the same as the inputs. Therefore a value of between 600-700 ML/yr has been adopted for the groundwater inflow at the north western boundary (Table 2.2).

**Table 2.2  Inputs to the Inner Farm Basin and Outer Farm Basin for 2002 from (Knapton, 2005).**

<table>
<thead>
<tr>
<th>Inflow from Town Basin (ML)</th>
<th>Irrigation (ML)</th>
<th>Pond Leakage (ML)</th>
<th>Flow to Ilparpa/St Mary’s Creek (ML)</th>
<th>Todd IF (ML)</th>
<th>River OF (ML)</th>
<th>Todd River OF (ML)</th>
<th>Total (ML)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Input</td>
<td>80*</td>
<td>369</td>
<td>(?)</td>
<td>90*</td>
<td>90*</td>
<td></td>
<td>629</td>
</tr>
</tbody>
</table>

* estimate from Read, 2004
* estimated for a flow of 4 days in 2002
2.4.3 Infiltration Rates

Initial Infiltration Rates

Trial basin “A”, used in the infiltration tests (Knapton et al., 2004), was constructed over the fine textured loam soil and showed an average infiltration rate of 3 m/d over a 2 month period. The distribution of the surficial soils in the area indicate that this infiltration rate can be expected over the area employed by the pilot scheme (Knapton and Lennartz, 2005).

Long Term Infiltration Rates

Infiltration rates of operating soil-aquifer treatment systems invariably decline with time due to the formation of the clogging layer on the bed of the infiltration basin (Bouwer, 2002). Estimates of the operational infiltration rate for the Alice Springs pilot soil-aquifer treatment system determined from column studies indicate that an order of magnitude decrease in the infiltration rate from 3 m/d to 0.3 m/d can be expected (Knapton et al., 2004). From this work and in the absence of other site specific data 0.3 m/d will be used to simulate the pilot soil-aquifer treatment system over the long term.

Operational infiltration rates of sites worldwide indicate that despite the variability of underlying lithologies, the management of clogging is the key to maximising infiltration (Bouwer, 2002). A summary of infiltration rates and soil types of operational soil-aquifer treatment systems are tabulated below (Table 2.3). This summary indicated that generally infiltration rates of 0.3 m/d can be expected, so the value employed is likely to be an over estimate.

Table 2.3 Summary of site characteristics of operational soil-aquifer treatment sites worldwide derived from the National Research Council (1994).

<table>
<thead>
<tr>
<th>Location</th>
<th>Project Name</th>
<th>Operation Duration (years)</th>
<th>Area (Ha)</th>
<th>Volume Processed per Annum (ML)</th>
<th>Soil Type</th>
<th>Infiltration Rate (m/day)</th>
<th>Depth to Water Table (mBGL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arizona, USA</td>
<td>Flushing Meadows 23rd Avenue project</td>
<td>37 years</td>
<td>0.78</td>
<td>155.4</td>
<td>Loamy sand</td>
<td>0.23</td>
<td>3</td>
</tr>
<tr>
<td>California, USA</td>
<td>Montebello Forebay</td>
<td>42 years</td>
<td>16</td>
<td>8,000</td>
<td>Sand and gravel</td>
<td>0.14</td>
<td>15</td>
</tr>
<tr>
<td>Israel</td>
<td>Dan Region Project</td>
<td>27 yrs</td>
<td>41.7</td>
<td>98,550</td>
<td>Fine sand</td>
<td>0.064</td>
<td>Variable</td>
</tr>
<tr>
<td>Alice Springs</td>
<td>AZRI</td>
<td>N/A</td>
<td>2</td>
<td>600</td>
<td>Loam-sand (prior to clogging)</td>
<td>5.1 – 44.5</td>
<td>16</td>
</tr>
</tbody>
</table>
3. Model Design

3.1 Model Development

3.1.1 Initial model
The initial modelling documented in Knapton et al., (2004) covered an area of 9.2 km$^2$ or 920 Ha. The model was simplified to a single, homogeneous and isotropic unconfined layer and conducted as a saturated system (ie no unsaturated modelling of the vadose zone). The initial model used a 33 metre layer with the upper surface/slice defined by the local digital elevation model (50 metre resolution).

At the time that the model was developed, limited groundwater data were available, and the topographic contours were used as a proxy to determine the geometry of the boundaries. The northwest and southeast constant head boundaries followed the topographic contour values 557 mAHD and 547 mAHD.

To simulate the effects of the palaeochannel a strip of higher conductivity material was incorporated into the model.

3.1.2 Modified model
This model was found to be lacking with respect to the conceptualisation of the aquifer system and the downstream extent of the model (Knapton et al., 2004). The model was extended to incorporate the following aspects:

- a two layer saturated system, where layer 2 has two zones corresponding to the palaeochannel etc.
- the boundary geometry was modified/improved to correspond to the groundwater contours determined from the monitoring bores both inside and outside the modelling domain

The increase in the range of the model to the south east was not incorporated as the level of control/detail on the boundary conditions to the south east has not been improved since those recommendations were made. The changes to the boundaries of the model, however, have resulted in the model domain covering an area of 9.5 km$^2$ or 950 Ha.

3.2 Model Specifications
The model domain is 9.5 km$^2$ in area and covers most of the AZRI site, spanning from approximately the 557 mAHD contour at the upstream to the 547 mAHD contour at the downstream, based on the 1 metre topographic data. The upstream and downstream boundaries coincide with the groundwater contours 544.5 and 528 mAHD. The model boundary extents are seen in Figure 3.1.
3.3 Model Implementation

3.3.1 Numerical Model Code
The finite element package FEFLOW® v5.1 from WASY was used to simulate flow processes (Diersch, 2004). The finite elements allow for more flexibility in the grid design. FEFLOW is a fully three dimensional finite-element package capable of simulating unsaturated and saturated flow and contaminant transport. FEFLOW also has a built-in mesh-design, problem editing and graphical post processing display modules that allow rapid model development, execution and analysis. A 32-bit PC workstation under Windows xp was used as the hardware platform for the numerical simulations.
3.3.2 Numerical Model Spatial Discretisation
The superelement mesh and model were developed with the FEFLOW package using the automatic TMesh (Delaunay) option (Diersch, 2004). This feature offers the ability to define the local variation of mesh density by allowing for the refinement of the mesh around specified features.

The mesh was generated using the following parameters:

- refinement around lines = 4.0
- speed and quality = 4.0
- total number of elements = 200

The resultant mesh used in the steady state calibration process is presented in Figure 3.2 and comprises 4001 elements and 2038 nodes.

![Figure 3.2 Spatial discretisation of the model domain.](image)

3.3.3 Temporal Discretisation
The model has been developed as a steady state model, which is time independent. However, the transient simulation used to calibrate against the infiltration trial data uses the automatic time step control in FEFLOW, which employs the forward Adams Bashforth/backward trapezoid time integration scheme (Diersch, 2004).

3.3.4 Assigning Hydraulic and Material Parameters to the Model Mesh
The material parameters were assigned to the mesh using the model editing features of the FEFLOW package. Parameters not assigned to an entire layer or slice, such as the areas where...
infiltration occurs, were assigned using the “JOIN” method in FEFLOW, which allows for regions
defined as polygons to be directly assigned material properties from the tables associated with
each polygon. Voigt, (1998) describes methods for assigning regional time constant and time
dependant data to the model mesh.

The default boundary condition of a border node in FEFLOW is no-flow, so only the constant head
nodes required defining.

3.4  Model layers
The total thickness of the model is 33 metres. 3 slices were defined for the 2 layer model
corresponding to the surface, bottom of the upper layer and the bottom of the palaeochannel
feature. The depth to each slice was determined from the stratigraphy which is summarised in
section 2.4.1. The local digital elevation model (DEM) for the Alice Springs area was used to
define the upper slice of the model. The bottom of the first layer is 18 metres below the surface
slice. The second layer is 15 metres thick with slice 3 at the base of the second layer, being 33
metres below the surface slice. Both were calculated from the DEM using the math function in the
Surfer program (Golden Software, 1999).

The elevations of each slice were assigned to each node in the mesh using the Akima interpolation
algorithm incorporated in the FEFLOW software (Diersch, 2004).

Slice 1 was set as a “free surface”, slice 2 was set as a “mixed” type and slice 3 was set to “fixed”.

3.5  Boundary Conditions

3.5.1  Constant Head (Dirichlet type) Boundaries
The direction of groundwater flow in the area is to the south east from Blatherskite Gap. The north
west and south east constant head boundaries have been located along the groundwater contours
544.5 and 528 mAHD respectively. The constant head conditions were applied to slice 1 and slice
2 of the model.

3.5.2  Constant Flux (Neumann type) Boundaries
The south west and north east boundaries are located along flow lines and run perpendicular to the
groundwater contours. These boundaries are considered no-flow boundaries and assume that no
recharge from flow in the section of the Todd River adjacent to and downstream of AZRI occurred
during the study period. This is considered valid since flow across the north eastern no flow
boundary would only occur during flood events where the flow in the Todd extends beyond AZRI
and recharge causes flow perpendicular to the axis of the Todd River and across the boundary.
The groundwater contours determined for the site over the study period indicate that the
contribution from the Todd River is predominantly from the north east and that the no flow conditions are valid for the year 2003-2004 considered here.

The boundary attributes of the model are defined below in Table 3.1, the constant head values provide a groundwater gradient of 0.0058 under steady state conditions.

### Table 3.1  Model boundary conditions for steady state and transient models

<table>
<thead>
<tr>
<th>Boundary</th>
<th>Boundary Condition</th>
<th>RL (mAHD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NW</td>
<td>Constant Head</td>
<td>544.5</td>
</tr>
<tr>
<td>SW</td>
<td>No Flow</td>
<td>N/A</td>
</tr>
<tr>
<td>NE</td>
<td>No Flow (no flow in Todd River)</td>
<td>N/A</td>
</tr>
<tr>
<td>SE</td>
<td>Constant Head</td>
<td>528</td>
</tr>
</tbody>
</table>

3.5.3 Neumann (flux) Boundary at the north western Inflow Boundary

The increase in head at the centre of the model due to the infiltration of 600 ML/yr changed the hydraulic gradient at the north western boundary, resulting in a reduction of the flow into the model. This is inconsistent with the likely response of the system, so a constant flux condition was employed to simulate the continuous flow into the area from upstream. The groundwater hydrograph response observed in section 2.3.1 indicated that the north western boundary would respond more than the south eastern boundary, which was kept at a constant head of 528 mAHD.

The Neumann (flux) boundary in FEFLOW allows for a “constant flux area” or “depth integrated flux” to be employed so that the discharge through the boundary is treated as either area dependent or area-independent. The flux flowing through a boundary would normally be dependant on the saturated cross-sectional area and the hydraulic gradient, which in an unconfined system varies due to variations in the location of the upper free surface (water table). The effect of setting the flux through the boundary ($Q_{in}$) to an area-independent condition is to allow the head to be determined from the resultant groundwater gradient and aquifer parameters (hydraulic conductivity and saturated aquifer thickness).

The flux for each of the boundary nodes was determined from the calibrated unstressed model using the FEFLOW Fluid Flux Analyser and Budget Analyser tools.
3.6 Hydraulic Parameters

Previous studies by Berry, (1991) and Quinlan and Woolley, (1965) in the vicinity of AZRI have determined the hydraulic characteristics of the Quaternary alluvium a summary of these values is presented in Table 3.2.

Table 3.2 Hydraulic parameters of the study area.

<table>
<thead>
<tr>
<th>Report</th>
<th>Hydraulic Conductivity K (m/d)</th>
<th>Specific Yield (Sy)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Range</td>
<td>Average</td>
</tr>
<tr>
<td>Quinlan and Woolley, 1965</td>
<td>28 - 85</td>
<td>46</td>
</tr>
<tr>
<td>Berry, 1992</td>
<td>1-117</td>
<td>45</td>
</tr>
</tbody>
</table>

3.6.1 Layer 1 Hydraulic Conductivity

The saturated hydraulic conductivity values determined for the two dominant soil types in the area provide an estimate of the upper layer. The loamy soil was measured at 0.09 m/d and the gravels were measured at 20 m/d (Knapton et al., 2004). In-situ soil permeameter measurements of the saturated hydraulic conductivity provide a range of 0.5 to 18.7 m/d for the ‘loam’ soil with an average of 5 m/day (Knapton and Lennartz, 2005).

Dual ring infiltration rates from both soil types indicate that the hydraulic conductivity of both is greater than determined from the laboratory measurements. Estimates of the ‘loam’ range from 2 to 11 m/d averaging 4.5 m/d compared to the ‘sand’, which ranges from 20 to 90 m/d and average 45 m/d.

The upper layer is a combination of the two soil types and may be assumed to exhibit a bulk hydraulic conductivity somewhere between the two extremes, and probably lower than their arithmetic mean. Hydraulic conductivities of 5, 10 and 15 m/d were utilised in this study for the entire upper layer.

3.6.2 Layer 2 Hydraulic Conductivity Distribution

The zonation of the hydraulic conductivity depicted in Figure 3.3 is based on the location of the palaeochannel feature (Knapton and Lennartz, 2005). Zone 1 corresponds to the predominantly clay material flanking the channel and has been assigned a value of 0.86 m/d (presently data is not available to provide adequate ranges). Zone 2 represents the palaeochannel sediments. Further analysis of the infiltration trials using the Hantush solution (Knapton et al., 2005 and Appendix A)
has estimated the hydraulic conductivity for Zone 2 as having a lower bound of 20 m/d and an upper bound of 50 m/d.

![Figure 3.3 Distribution of hydraulic conductivity zones for Layer 2.](image)

3.6.3 Storage Coefficient
Steady state simulations are independent of porosity (Anderson and Woessner, 2002); however, to determine the particle track of the plume migration using the Darcy velocity a porosity is required.

The fillable porosity (both above the water table and the water replaced by the infiltrated water) should be taken as the difference between existing and saturated water content of the material outside the wetted zone below the infiltration system (Bouwer, 2002). Therefore, using this rationale, an initial water content of between 0 and 0.10 and saturated water contents of between
0.3 and 0.35 as observed at the infiltration trial sites (Knapton et al., 2004), a porosity estimate of 0.25 has been used.

3.7 Steady State Model Calibration

The dependent variables that will be considered in the calibration process are the hydraulic heads and the groundwater throughflow.

The measure of the "goodness" of fit used is the root mean square error (RMSE) where:

\[
RMSE = \sqrt{\frac{\sum_{i=1}^{n} (h_{\text{obs}(i)} - h_{\text{model}(i)})^2}{n}}
\]

and

RMSE is the root mean square error (metres)

\(h_{\text{obs}(i)}\) is the \(i^{th}\) observed water level (metres)

\(h_{\text{model}(i)}\) is the \(i^{th}\) modelled water level (metres)

\(n\) is the number of observations

The target for calibration will be the minimisation of the overall RMSE and provide a throughflow in line with the estimate of between 600 and 700 ML/yr determined from water balance analysis (section 2.4.2).

The steady state model has been calibrated against the observed heads on site for May 2004. The results using hydraulic conductivities for Layer 1 = 10 m/d and Layer 2, zone 2 = 40 m/d are presented in Figure 3.4 and shows a RMSE value of 0.48 metres.
Figure 3.4  Comparison of modelled heads with observed heads for Layer 1 K = 10 m/d and Layer 2, Zone 2 K = 40 m/d.

Simulations using values of hydraulic conductivity for Layer 1 of 5, 10 and 15 m/d were run with different values of Layer 2, zone 2 of 10, 20, 30, 40 and 50 m/d to determine the sensitivity of the model. RMSE values were determined for each scenario and a “matrix” of the results produced (Figure 3.5). The “matrix” shows a RMSE minima running diagonally from Layer 1 = 5 m/d and Layer 2 – Zone 2 = 30 m/d through to Layer 1 = 10-15 m/d and Layer 2 - Zone 2 = 50 m/d, however, the higher hydraulic conductivity combinations to the top right of the “matrix” have throughflow of greater than 800 ML/yr. The lower conductivity combination to the lower left produces a throughflow of less than 500 ML/yr. The ranges of hydraulic conductivity considered in the transient calibration are presented in Table 3.3
3.8 Transient Model Calibration

The transient simulation uses a 6x6 metre basin and assumes an infiltration rate of 11.1 m/d over 36 days to infiltrate 400 m$^3$/d or a total volume of 14.4 ML. The modelled response was taken from observation points 5 metres to the south east of the centre of the infiltration area to simulate the monitoring bores at the basin B infiltration site. Transient simulations of the system were performed using the range of parameters assessed during the steady state calibration (section 3.7). Table 3.3 presents the results of the modelled head rise at approximately 5 metres down gradient from the centre of the basin to provide a comparison with the piezometer response from RN17952-18 and RN17953 (Knapton et al., 2005).

Table 3.3  Summarised results of the transient model simulations.

<table>
<thead>
<tr>
<th>Hydraulic Cond.</th>
<th>Modelled Response</th>
<th>Observed Response</th>
<th>Throughflow</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[m/d]</td>
<td>[m]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>K$_1$</td>
<td>K$_2$</td>
<td>Obs 1</td>
</tr>
<tr>
<td>10</td>
<td>30</td>
<td>1.14</td>
<td>0.68</td>
</tr>
<tr>
<td>15</td>
<td>30</td>
<td>0.88</td>
<td>0.65</td>
</tr>
<tr>
<td>10</td>
<td>40</td>
<td>0.98</td>
<td>0.56</td>
</tr>
<tr>
<td>15</td>
<td>40</td>
<td>0.75</td>
<td>0.54</td>
</tr>
<tr>
<td>10</td>
<td>50</td>
<td>0.85</td>
<td>0.458</td>
</tr>
<tr>
<td>15</td>
<td>50</td>
<td>0.64</td>
<td>0.445</td>
</tr>
</tbody>
</table>

K$_1$ is the hydraulic conductivity of Layer 1.
K$_2$ is the hydraulic conductivity of the palaeochannel.
3.9 Sensitivity

3.9.1 Analytical Modelling
Analytical modelling was used to determine the sensitivity of the system to variations in the hydraulic parameters with respect to the response observed in the infiltration trials (Knapton et al., 2005). The analytical model was used as its implementation was considerably quicker compared to the setup of the finite element groundwater model.

The analytical modelling was conducted using the method of Finnemore, (1993), based on Hantush, (1967), which enables the maximum height of the water table beneath the centre a rectangular recharge area to be calculated. The solution assumes that the aquifer is unconfined, of infinite extent, homogeneous and isotropic, there is vertical recharge at a uniform rate, the top of the mound does not contact the bed of the spreading basin and the height of the mound is small in relation to the initial aquifer saturated thickness. The analytical modelling results are presented in Appendix A.

3.9.2 Hydraulic Conductivity
The transient simulation of the infiltration basins using both the analytical and numerical models is seen to be highly dependant on the hydraulic conductivity of the model. The analytical model indicates that an order of magnitude decrease in the hydraulic conductivity will result in an order of magnitude increase in the resultant head at the centre of the basin, this is almost irrespective of the of the storage coefficient used.

3.9.3 Storage coefficient
The effects of storage are relevant to the transient simulations and the particle tracking results. Sensitivity of the mounding height to storage was investigated using the Hantush solution (Appendix A). The results indicate that the storage coefficient has minimal effect on the mound height over the short time frames investigated.

One impact that altering the storage coefficient has on the model relates to the particle tracking results. Particle tracking utilises the Darcian groundwater velocity and therefore has an inverse relationship to the effective porosity employed. Therefore, increasing the effective porosity from say 0.1 to 0.3 results in a reduction in the groundwater velocity by a third and subsequently a decrease in the distance travelled over a given time by a factor of 3.

3.9.4 Boundary Conditions
The steady state model is strongly dependant on the values of the constant head boundaries. The north west and south east constant head boundary conditions were varied by ±0.5 metres of the
final boundary conditions used in the steady state model. It was found that the variation resulted in an increase in the overall RMSE of between 0.1-0.2 metres.

3.10 Calibrated Model Results
The head distribution using a Layer 1 hydraulic conductivity of 10 m/d and a hydraulic conductivity for the palaeochannel feature in Layer 2 of 40 m/d is presented in Figure 3.6.

Figure 3.6  Head distribution using Layer 1 hydraulic conductivity = 10 m/d and Layer2, Zone 1 = 0.86 m/d, Zone 2 hydraulic conductivity = 40 m/d.
4. Mounding Response to Infiltration using Two Basin Geometries

4.1 Introduction
The groundwater mounding response to the infiltration is a key aspect for the successful implementation of soil-aquifer treatment. The aquifers should be unconfined and sufficiently transmissive to accommodate lateral flow of the infiltrated water away from the recharge area without forming high groundwater mounds that interfere with the infiltration process (Bouwer, 2002).

The capillary fringe is commonly about 0.3 m for medium sands, more for fine sands/loams and less for coarse sands, a conservative estimate of the maximum capillary fringe is approximately 1 metre (Bouwer, 2002). If the water table rises such that the capillary fringe reaches the bottom of the basin, then the infiltration rates will start to decrease (Bouwer, 2002). Therefore, if the water is kept to at least 1 metre below the bottom of the infiltration basin, then infiltration rates will be maintained. Similarly other negative effects, such as water logging and associated salinisation will also be avoided if the water table is kept to below at least 1-2 metre of the surface.

Another specific criterion of the project is that the effects of infiltration on the water table should not result in the discharge of groundwater at surface features such as the Todd River or affect services such as private septic systems in the rural residential area.

The “ultimate” or quasi-equilibrium mound heights due to artificial recharge can be obtained from steady state analysis, where the mound is considered to be in equilibrium with the rate of recharge due to infiltration and the lateral flow away from the mound (Bouwer, 2002). Steady state modelling has been utilised in this way to determine the effects of infiltration on the groundwater level.

4.2 Simulations using Rectangular Basins at 3 m/d Infiltration Rate

4.2.1 Recharge Basin Geometry and Infiltration Rates
Bouwer (2002) indicates that the resultant mound height at the centre of the recharge area can be reduced using long thin basins, where the width of the basin is equivalent to the depth of the water table. Therefore, in order to define the maximum mound height that would result from the infiltration of 600 ML/yr square basins have been utilised in the simulations.

The pilot scheme is estimated to infiltrate 600 ML/yr (1644 m³/d). Infiltration trials using town water conducted at AZRI indicate that the initial rate of infiltration for the fine textured soil is around 3 m/d (Knapton, et al., 2005). The area required to infiltrate the 600,000 m³/yr at 3 m/d is:

\[
600,000 / 365 / 3 = 550 \text{ m}^2
\]
However, assuming that the system utilises a minimum of two basins with 7 days wet and 7 days dry cycles, the area required is 1100 m$^2$ to provide continuous infiltration and accommodate the wetting and drying cycles. The effective infiltration rate over the total area is therefore 1.5 m/d. The simulation using a rectangular infiltration system has been conducted using an area of 1100 m$^2$ and an infiltration rate of 1.5 m/d.

4.2.2 Mesh refinement to incorporate the infiltration area

The spatial discretisation of the model mesh was refined around the infiltration area, where the greatest variation in heads occur. The initial mesh was generated using the parameters described in section 3.3.2 and refined manually in the FEFLOW mesh editor.

4.2.3 Initial Modelling

A rectangular recharge area of 1100 m$^2$ with an infiltration rate of 1.5 m/d was applied to the calibrated steady state model in the area identified as being the most likely position of the pilot scheme (Knapton et al., 2004a). The mounding response can be seen in Figure 4.1. The mound directly beneath the basins is 5.1 metres above the steady state groundwater level prior to infiltration or approximately 10.5 metres below the basin floor.

4.2.4 Verification of Response Using Throughflow Estimates

The response seen in Figure 4.1 would seem a reasonable result, however, when the water budget for the system is examined, it indicates that the inflow from the north western boundary ($Q_{in}$) has been reduced from 1936 m$^3$/d to 1163 m$^3$/d (Table 4.1) and the total outflow flux ($Q_{out}$) of 2807 m$^3$/d. The input from the north western boundary is reduced due to the change in the hydraulic gradient from 0.0058 to 0.0034. As a direct consequence of Darcy’s law the flow through the NW boundary can be expected to reduce to 0.0034/0.0058 = 58.6% of the initial inflow. To demonstrate, a plot along the axis of the palaeochannel indicates that the hydraulic gradient decreases from 0.0058 to 0.0034 near the north western boundary (Figure 4.1).

$$1936 \text{ m}^3/\text{d} \times 58.6\% = 1134 \text{ m}^3/\text{d} \ (\text{cf } 1163 \text{ m}^3/\text{d} \text{ from the model})$$
4.2.5 Model Modification to Account for Constant Inflow at the North Western Boundary

Although the model is internally consistent, in practice the actual volume entering from the northwest will be constant. Therefore, in accordance with Darcian theory, the system will respond by increasing the head at the north western boundary until the groundwater gradient is such that the required volume can flow. In order to account for the discrepancy between the inflow from the unstressed model (no infiltration) and the inflow determined for the stressed model (with infiltration of 1644 m$^3$/d) the constant head boundary was adjusted, using trial and error, from 544.5 mAHD to 547.9 mAHD to produce an inflow through the north western boundary of 1936 m$^3$/d. The total outflow from the model at the south eastern boundary is then 3584 m$^3$/d (Table 4.1).

4.2.6 Results of the Modified Model Incorporating an Elevated North Eastern Boundary

The effect of the change in the north western boundary can be seen in Figure 4.2 and when compared to the mounding response determined in section 4.2.4. The contours of the mound are presented in Figure 4.3, the maximum increase in head, which is beneath the centre of the basins, is 7.8 m above the initial groundwater surface or 8 metres below the floor of the basin. It should be noted the contours indicate that the location of the north eastern boundary exerts some influence on the mound response.
4.2.7 Neumann (flux) boundary at the north western inflow boundary
The Neumann (flux) boundary in FEFLOW allows for a “constant flux area” or “depth integrated flux” to be employed so that the discharge through the boundary is treated as area-independent.

The flux flowing through a boundary would normally be dependant on the saturated cross-sectional area, which in an unconfined system varies due to variations in the location of the upper free surface (water table). The effect of setting the flux through the boundary \((Q_n)\) to an area-
independent condition is to allow the head to be determined from the resultant groundwater gradient and aquifer parameters (hydraulic conductivity and saturated aquifer thickness).

The flux for each of the boundary nodes was determined from the calibrated unstressed model using the FEFLOW Fluid Flux Analyser and Budget Analyser.

The resultant head determined at the north western boundary using constant flux conditions are consistent with both the unstressed and stressed simulations performed in section 3.10 and section 4.2.5 using constant head boundaries of 544.5 mAHHD and 547.9 mAHHD respectively. This result is fairly obvious as the trial and error determination of the constant head boundary in section 4.2.5 was based on the same criteria, ie \( Q_{in} = 1936 \text{ m}^3/\text{d} \) through the north western boundary.

### 4.3 Simulations using “Kidney” Shaped Infiltration Basins

#### 4.3.1 Introduction

The mounding response and plume migration using the “Kidney” shaped infiltration basins presented by Knapton et al., (2004a) were investigated. It is expected that the major difference to the response produced from the previous scenario is the mound height beneath the basins, as the area of infiltration and the infiltration rate have been altered.

#### 4.3.2 Recharge Basin Geometry and Infiltration Rates

The response to infiltration was simulated using the “Kidney” soil-aquifer treatment basin design (Knapton et al., 2004). The basin to the north east covers an area of 5876 m\(^2\) and the basin to the south east covers an area of 5841 m\(^2\)/d. It was assumed that the infiltration rate at each basin declined from 3 m/d to approximately 0.3 m/d, based on the clogging data presented in Knapton et al., (2004a) and Pavelic et al., (2004). Therefore, the average infiltration rate, assuming the basins have equal periods of wetting and drying, is 0.15 m/d and the total volume infiltrated per year is:

\[
(5876 \text{ m}^2 + 5841 \text{ m}^2) \times 0.15 \text{ m/d} = 1758 \text{ m}^3/\text{d} \text{ or } 641.5 \text{ ML/yr}
\]

#### 4.3.3 Steady State Groundwater Mounding Response to 600 ML/yr SAT Operation

The constant head boundary was adjusted as in section 4.3.3 to produce a flux of 1936 m\(^3\)/d through the north west boundary of the model.

Regionally the results depicted in Figure 4.4 and Figure 4.5 are similar to the results from the infiltration through a rectangular basin (section 4.2.6), however, the height of the mound directly below the basins is considerably lower, compare 6 metres with 7.8 metres or 10 and 8 metres below the basin floor. This is to be expected as the infiltration area has increased by an order of magnitude (11,717 m\(^2\) compared to 1100 m\(^2\)).
Figure 4.4  Mounding response to 600 ML/yr infiltration using the “Kidney” basin design, compared to initial water table.

Figure 4.5  Groundwater contours of steady state infiltration of 600 ML/yr (see Figure 4.3)
5. Steady State Simulations and Particle Tracking

5.1 Introduction
The particle tracking computation methods are based on the Darcian velocity distributions determined from the steady state head distribution (Anderson and Woessner, 2002).

\[ V_d = \frac{K}{\varepsilon} \cdot \frac{\delta h}{\delta l} \]

where

- \( V_d \) = darcy velocity
- \( K \) = hydraulic conductivity
- \( \varepsilon \) = specific yield
- \( \frac{\delta h}{\delta l} \) = groundwater gradient

This technique can provide point related information about groundwater age in the form of isochrones, which is often used to delineate well capture zones. It should be noted that particle tracking simulates advective transport and neglects to include dispersion processes.

The hydraulic head determined from the steady state simulation is independent of porosity, however, as noted above, to determine the particle track of the plume migration using the Darcy velocity a porosity is required. Following the discussion in section 3.6.3 a porosity of 0.25 was employed.

The simulations have been presented in isochrones (the distance covered by a particle for a given time) to show the migration of the plume. The results from layer 2 are presented as seen in

the distance travelled is greater due to the more transmissive material of the palaeochannel.

5.2 Steady State Particle Tracking to Simulate Infiltrate Migration for a Rectangular Basin
The first isochrone (Figure 5.1) calculated after 90 days has migrated downstream approximately 150 metres, with an average velocity of 1.67 m/d. The next isochrone calculated at 1 year (365 days), shows that the distance covered by the plume is 460 m, the average velocity has reduced to 1.26 m/d. Three years of infiltration has resulted in the infiltration plume front migrating approximately 1,380 m (average velocity = 1.26 m/d). The results of the particle tracking for both the upper and lower layers for 90, 365 and 1095 day time periods are presented in Table 5.1.
Table 5.1 Migration of Infiltrate with Time in Layer 1 and Layer 2

<table>
<thead>
<tr>
<th>Time [d]</th>
<th>Layer 1 [m]</th>
<th>Layer 2 [m]</th>
<th>Average Velocity [m/d]</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>130</td>
<td>150</td>
<td>Layer 1: 1.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Layer 2: 1.67</td>
</tr>
<tr>
<td>365</td>
<td>375</td>
<td>460</td>
<td>Layer 1: 1.03</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Layer 2: 1.26</td>
</tr>
<tr>
<td>1095</td>
<td>1,100</td>
<td>1,375</td>
<td>Layer 1: 1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Layer 2: 1.26</td>
</tr>
</tbody>
</table>

Figure 5.1  Particle tracking results calculated for layer 2 from the steady state infiltration response to 640 ML/yr into rectangular basins with an average infiltration rate of 1.5 m/d.
5.3 Steady State Simulations and Particle Tracking for a ‘Kidney’ Basin Geometry

Similar to section 5.2 the migration of the infiltrated water down gradient of the basins was simulated using particle tracking techniques (Figure 5.2). A summary of the distance that the plume front migrated from the south eastern edge of the basins is presented in Table 5.2.

### Table 5.2 Migration of Infiltrate with Time in Layer 1 and Layer 2

<table>
<thead>
<tr>
<th>Time [d]</th>
<th>Layer 1</th>
<th>Layer 2</th>
<th>Layer 1</th>
<th>Layer 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>51</td>
<td>95</td>
<td>0.57</td>
<td>1.05</td>
</tr>
<tr>
<td>365</td>
<td>177</td>
<td>350</td>
<td>0.48</td>
<td>0.96</td>
</tr>
<tr>
<td>1095</td>
<td>490</td>
<td>1,085</td>
<td>0.45</td>
<td>0.99</td>
</tr>
</tbody>
</table>

The resultant groundwater velocity in the palaeochannel is approximately 25% lower than the velocity produced from the square basin geometry in section 5.2. This is due to the lower mound height beneath the basins and consequent lower hydraulic gradient close to the basins compared to the previous case in section 5.2.
Figure 5.2  Particle tracking results calculated for layer 2 from the steady state infiltration response to 640 ML/yr into the “Kidney” basin design.
6. Steady State Simulations and Particle Tracking for Infiltration/Extraction Scenarios

6.1 Introduction

6.1.1 Recommended Retention Time of Infiltrated Water
The water quality of the treated effluent from the waste water treatment plant is expected to be of Class B with respect to the South Australian Reuse Guidelines and suitable for controlled direct reuse. Experience from other soil-aquifer treatment systems world wide indicate that soil-aquifer treatment is extremely efficient at removing suspended solids, microbial pathogens and to a lesser degree viruses (National Research Council, 1994). It can be expected then that once the infiltrated water has passed through the vadose zone it should be of at least Class A in quality. It also follows that the infiltrated water should be suitable for reuse at any point along its flow path from the infiltration basins and that the notion of retention times is not applicable in this case from a water quality point of view or with respect to reduction of risk to humans.

The uses for the water will be considered as restricted in line with the South Australian Guidelines for waste water reuse of Class B.

6.1.2 Extraction Rates and Bore Numbers
Airlift results from bores constructed in the AZRI area show yields of up to 10 l/s (RN17951) and yields of 2-4 l/s for most of the 100 mm monitoring bores constructed along the length of the palaeochannel feature. The works by Berry, (1991) indicate that bore yields from pumping tests in the same palaeochannel feature can be as low as 1 l/s and as high as 20 l/s. The bores have been assigned extraction rates of 5 l/s (430 m³/d).

From a water balance perspective the number of bores required to extract the infiltrated volume of 1757 m³/d is between 4 and 5 bores, (1720 m³/d and 2150 m³/d). The actual water demand for the end use for the water, however, will probably be seasonal, with greater demand during the summer months and lower during the winter months. The number of wells to cope with this kind of scenario has been estimated at 11 bores at 5 l/s. Having said this, the scenarios considered here will only incorporate the minimum number of bores to recover the infiltrated volume of 1757 m³/d.

6.1.3 North western boundary conditions
The constant flux boundary conditions described in section 3.5.3 were employed to adjust the inflow from the northwest as the hydraulic gradient varied with the stresses imposed by the infiltration and extraction regimes.
6.2 Steady State Particle Tracking to Simulate Infiltration into a Rectangular Basin and Interception using Extraction Bores

Two scenarios have been considered in terms of extraction/infiltrate interception simulations for the rectangular infiltration area from section 4. A time frame of <10 years has been used for the particle tracking to demonstrate the long term effectiveness of the extraction bores.

The first scenario presented in Figure 6.1 infiltrates 600 ML/yr into an 1100 m² basin at 1.5 m/d with 4 extraction bores situated approximately 150 metres down stream of the basin which equates to 90 days travel time with 50 metres separation.

![Figure 6.1](image-url)
The second scenario (Figure 6.2) has 4 bores which are orientated along the axis of the palaeochannel with the closest bore located 150 metres downstream of the basins with 150 metres separation so that the bores intercepted the water within 500 metres of the basins.

![Figure 6.2](image)

Figure 6.2  Scenario 2 infiltration through the rectangular basins at 600 ML/yr and combined extraction of 630 ML/yr from 4 bores located along the axis of the palaeochannel.

### 6.3 Steady State Particle Tracking to Simulate Infiltration into “Kidney” Basins and Interception using Extraction Bores

As discussed earlier in section 6.1.2 between 4 and 5 extraction bores yielding 5 l/s are required to balance the input from the infiltration basins. The effects of borefield geometry were investigated by orientating the wells both perpendicular to the flow direction with approximately 50 metres separating each bore and along the axis of the channel at approximately 100 metre separations.
The later was employed so that the bores intercepted the water within 500 metres of the basins (after a maximum retention time of approximately 1.5 years for the south eastern most bore). The simulations presented in Figure 6.3 and Figure 6.4 indicate that both geometries are capable of recovering the infiltrated water.

![Figure 6.3](image_url) Scenario 3 infiltration through the “Kidney” basins at 640 ML/yr and combined extraction of 790 ML/yr from 5 bores located perpendicular to the axis of the palaeochannel.
Figure 6.4 Scenario 4 infiltration through the “Kidney” basins at 640 ML/yr and combined extraction of 790 ML/yr from 5 bores located along the axis of the palaeochannel.
7. Discussion

7.1 Implications of the Modelled Response to 600 ML/yr Infiltration Scheme

The results of the modelling for both the high infiltration rate rectangular basin and the “Kidney” basins design indicate that the groundwater mound is unlikely to impact upon the surface. At the infiltration rate of 3 m/d the groundwater mound increased to a maximum of 7.5 metres above the ambient conditions which is approximately 8.5 metres below the base of the infiltration basin.

The response at the north east boundary at the models closest point to the Todd River the elevated groundwater in response to the infiltration is 3.5 metres above the ambient level which is 7-8 metres below the topographic low of the Todd River.

7.2 Plume Containment and Borefield Geometry

At present, using the Darcian velocity estimates of groundwater flow, it is expected that the reclaimed water plume will migrate along the palaeochannel feature to the SE at around 330 metres/yr, therefore, over a three year period (with no extraction) the plume is likely to have moved 1100 metres down stream from the basins. As there is no requirement to detain the water for a specified period of time (Knapton et al., 2004), the extraction bores can be located at the sites where the most efficient recovery of the water can be attained. Obviously the highest yields will be where the available drawdown is greatest, ie in the areas where the mounding is greatest.

Six options are available in terms of the containment of the infiltrated water if water is infiltrated for greater than three years with no extraction:

1. no extraction continues to takes place;
2. the palaeochannel feature is located and extraction bores constructed to intercept the reclaimed water on the Airport North Block;
3. extraction takes place at a lower rate than required to contain the reclaimed water and some water moves off site followed by scenario 1 or 2;
4. bores are located in areas where the overall extraction is equal to the infiltration, but is a mixture of the infiltrated and current groundwater and some water moves off site followed by scenario 1 or 2;
5. if the above scenarios are found to be unacceptable logistically or from a regulatory point of view then it is suggested that the water is intercepted within the confines of the AZRI site (preferably closer to the SAT basins) and used for fodder production or other beneficial uses such as experiments relating to reclaimed water use for horticulture at the Horticulture Block (these may also be suitable for the irrigation requirements of the Horticultural Developer as production bores located in areas where the water table will...
be elevated are expected to sustain higher yields and the water can be reticulated from a relatively central point).

6. extraction of all the reclaimed water for reuse from bores along the palaeochannel at sites suitable for the end user(s).

### 7.3 Model Limitations
The current model has the following limitations in that it:

- does not take into account perching effects and unsaturated flow conditions due to variations in the lateral distribution and hydraulic parameters of the strata observed in the unsaturated zone;

- does not account explicitly for recharge from the Todd River (necessary for long term performance, especially with respect to the northern and north western boundaries and

- the boundary conditions along the south eastern boundary are poorly defined, requiring further works to adequately resolve this issue.
8. Conclusions
The following conclusions have been determined from the modelling completed to date:

- Numerical modelling indicate that the infiltration of 600 ML/yr at the Pilot Soil-Aquifer Treatment scheme is feasible in terms of available storage, the mound response beneath the infiltration basins and the impact of the mounding on topographic lows of the Todd River to the north east of the site.

- The resultant mound height of 6 metres can be expected beneath “Kidney” shaped infiltration basins with surficial infiltration rates of 0.3 m/d, and an increase of 3.5 metres can be expected at the north western boundary. This equates to approximately 10 metres below both the basins and the surface elevation at the north western boundary.

- Analytical and numerical modelling indicate that hydraulic conductivities of between 20 and 50 m/d are required to simulate the mounding observed during an 18 ML field trial at the AZRI site. This is comparable to the values determined in previous studies in similar sediments.

- Lateral migration perpendicular to the flow direction would be relatively small and contained largely to the palaeochannel feature, which is the target for SAT operations. However, the migration of the plume down gradient could be quite considerable if no water is extracted for reuse with an advective velocity of approximately 300 metres per year.

- Under a regime where extraction volumes are comparable or greater than the infiltrated volume, little reclaimed water is likely to move beyond the extraction bores, which can be placed to maximise capture. The extraction bores will also reduce the mounding due to the infiltration.
9. Recommendations
The modelling has highlighted some shortfalls in the present understanding of the area. The next steps are to:

- Modify the localised model to fit the infiltration data as it becomes available.
- Develop the unsaturated zone of the model to better represent the conditions observed on site.
- Develop a regional model that incorporates conservative solute transport to determine the extent of plume migration and degree of interception by pumping bores over the long term.
- As part of the regional model determine the relative contributions to recharge from the various sources stated in section 2.4.2.
- Develop and maintain a monitoring program of the AZRI site and infiltration sites to provide further validation of the model beyond the initial investigation program.
10. References


Knapton, A., Pavelic, P., P., Dillon, and Low, B., (2005), Field Infiltration Tests with Potable Water to Assess Hydraulic Risks for an Intended Soil Aquifer Treatment Trial for Alice Springs, Northern Territory, Department of Infrastructure, Planning and Environment, Alice Springs, Report No. 28/2005


**Analytical Mounding Estimation from Rectangular Infiltration Basins**

**Introduction**

Using the method of Finnemore, (1993), based on Hantush, (1967) enables the maximum height of the water table beneath the centre a rectangular recharge area to be calculated. For convenience only Basin B was simulated. Hantush (1967) presented the following equations for:

\[
\begin{align*}
    h_m^2 - h_i^2 &= (2w/K)\tau S'[0.5A/(4\pi\tau)^{0.5},0.5B/(4\pi\tau)^{0.5}] \\
    \nu &= Kb/\varepsilon \\
    b &= 0.5 h_i(0) + h(t)
\end{align*}
\]

where

- \(h_m\) is the maximum height of the mound above the base of the aquifer
- \(h_i\) is the initial height of the water table above the base of the aquifer
- \(K\) the hydraulic conductivity
- \(\varepsilon\) is the storativity (specific yield) of the aquifer
- \(w\) is the constant rate of percolation from a rectangular recharge area of length \(A\) and width \(B\)
- \(b\) is a constant of linearization

and the function \(S'\) is an integral expression

Equation (1) is nonlinear owing to the definition of \(b\) in Equation (3); however, the solution is readily obtained using successive approximations.

The solution assumes that the aquifer is unconfined, of infinite extent, homogeneous and isotropic, there is vertical recharge at a uniform rate, the top of the mound does not contact the bed of the spreading basin and the height of the mound is small in relation to the initial aquifer saturated thickness. Knapton et al., (2004b) indicate that the mounding response has had no effect on water levels within 300 metres up or down gradient of the infiltration site. Although the actual palaeochannel aquifer is of limited extent, it is expected that influence from the boundaries has had little effect on the mound observed, and therefore the assumptions of the analytical solution hold for a trial of this duration.

Clearly the dimensions of the mound, and of particularly interest the height, are governed by the basin size and shape, infiltration rate and duration and the aquifer characteristics. The sensitivity of mounding to variations in the basin geometry and hydraulic parameters were investigated,
in conjunction with results of the infiltration trials inferences about the long term infiltration response were determined.

**Effects of Hydraulic Conductivity and Storage Coefficient on Mound Response**

Initially it was optimistically assumed all the water recharged contributed to the water table with negligible time delay.

**Figure A-1** presents the results with respect to the head build-up beneath the basin after 36 days of continuous infiltration using an infiltration area of 6x6m, saturated aquifer thickness of 16 m, infiltration rate of 11.1 m/day and hydraulic conductivity values of 5, 10, 20, 50 and 100 m/day. Mounds for specific yields of 0.01, 0.1 and 0.25 are included to demonstrate its secondary importance. This plot shows that a value of aquifer hydraulic conductivity in the order of 35-50 m/day is needed to produce the height of the mound observed. The equivalent transmissivity values for a saturated thickness of 16 metres would be 560-800 m²/day. However these figures probably represent an upper limit on K given that unsaturated zone effects have not been taken into account in this scenario.

![Figure A-1](image)

**Figure A-1** Mound response to 35 days of infiltration at 11.1 m/d and various hydraulic parameters.

**Effect of storage coefficient on Mound Height**
The effect of the storage coefficient on the mound height was investigated using storage coefficient values of 0.01, 0.1 and 0.25. The model used the values of hydraulic conductivity employed in the previous section (5, 10, 20, 50 and 100 m/day). It was found that the height of the resultant mound for a 36 day period was relatively insensitive to variations in the storage coefficient. Decreasing the storage coefficient by 25 times increases the resultant mound height by approximately 15%.

The fillable porosity should be taken as the difference between existing and saturated water content of the material outside the wetted zone below the infiltration system (Bouwer, 2002). Therefore, using this rationale an initial water content of between 0 and 0.10 and saturated water contents of between 0.3 and 0.35 as observed at the infiltration trial sites (Knapton and Lennartz, 2004 and Knapton et al., 2004a), a porosity estimate of 0.25 has been used.

**Effect of Recharge Area on Mound Height**

The analytical model has been compared with the results of basin B (Knapton et al., 2004), where the average infiltration rate was 11.3 m/day and approximately 75% of the total water infiltrated at the site was applied and the resultant mound height was 0.56 metres above the initial water level.

To ascertain the sensitivity of the model to the effective infiltration area, the variations in mounding response due to variations in the basin area were investigated. Using recharge areas of 36, 100, 225, 400 and 1440 m², the resultant mound height was calculated with a constant hydraulic loading of 400 m³/day i.e. the infiltration rate was adjusted with the basin dimensions to obtain the daily recharge volume of 400 m³/day. The variation in the mound height at the centre of the basin for a specific yield of 0.25 was less than 50%, and as expected increasing the basin dimensions resulted in the reduction of the mound height.

Table A-1: Resultant mound in response to variations in infiltration area.

<table>
<thead>
<tr>
<th>Basin Side (m)</th>
<th>Basin Area (m²)</th>
<th>Aquifer K (m/day)</th>
<th>Infiltration rate (m/day)</th>
<th>Volume (m³)</th>
<th>Mound Height (m)</th>
<th>% Increase in basin area</th>
<th>% change in mound height</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>36</td>
<td>30</td>
<td>11.1</td>
<td>400</td>
<td>0.86</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>100</td>
<td>30</td>
<td>4.0</td>
<td>400</td>
<td>0.77</td>
<td>178</td>
<td>1.4</td>
</tr>
<tr>
<td>15</td>
<td>225</td>
<td>30</td>
<td>1.8</td>
<td>400</td>
<td>0.71</td>
<td>525</td>
<td>4.2</td>
</tr>
<tr>
<td>20</td>
<td>400</td>
<td>30</td>
<td>1.0</td>
<td>400</td>
<td>0.68</td>
<td>1011</td>
<td>8.5</td>
</tr>
<tr>
<td>38</td>
<td>1440</td>
<td>30</td>
<td>0.3</td>
<td>400</td>
<td>0.59</td>
<td>3900</td>
<td>32.4</td>
</tr>
</tbody>
</table>

The characteristics of the strata at the site indicate that the hydraulic conductivity of the underlying silty sands from 12 metres below ground level are expected to be less permeable than the overlying coarser grained sediments. The response observed in the shallow piezometers also indicates that perching occurs between 10-12 metres below ground level (Knapton et al., 2004b).
Comparing these results with the observed increase in the mound of 0.56 m indicates that if the underlying aquifer has a hydraulic conductivity of 30 m/d the effective recharge area necessary is approximately 1440 m$^2$.

As a comparison a simplified estimate of the water stored in the unsaturated zone can be made and therefore the effective area of infiltration. Approximately 5 days of infiltration occurred prior to a response in the piezometers was observed (Knapton et al., 2004b). The volume infiltrated is therefore $5 \times 403 \text{ m}^3/\text{day} = 2015 \text{ m}^3$. For a porosity of 0.25 the volume of soil filled is therefore $4 \times 2015 \text{ m}^3 = 8060 \text{ m}^3$.

If it is assumed that the volume filled is cylindrical with a height of 15 metres (the depth of the water table below ground level) then the area is $7200 / 15 = 480 \text{ m}^2$ which equates to an area with a side of approximately 22 metres.

If the stepped response due to perching is assumed to fill a volume approximating a conical feature ($\text{Vol} = \frac{1}{3} \times h \times \pi \times r^2$) then the area at the water table affected is approximately 1,440 m$^2$ which equates to an area with a side of approximately 38 metres (6 times the side dimension of the infiltration basin).

Figure A-2 indicates that the range of hydraulic conductivities that could be expected depending on the degree of perching in the unsaturated zone is between 30 and 50 m/d.

The area estimated above (1440 m$^2$ or 38 m basin side) resulted in a mound height of 0.56 m for 36 days of infiltration, at 400 m$^3$/day with a hydraulic conductivity of 30 m/d (see Figure A-2).
Figure A-2 Variation of mound height in response to effective infiltration area.

Mound height with time – estimating steady state conditions

The mound response determined from the Hantush solution indicates that the system approaches steady state conditions after approximately 180 days. The increase in mound height below the centre of the basin increases by less than 10% over the following 180 day period and less than 16% after 2 years of infiltration.