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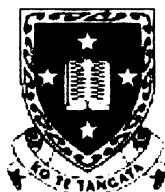
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**Evaluating the Potential
of Constructed Groundwater Systems to
Improve the Water Supply Operations of
Auckland City, New Zealand**

A Thesis submitted in partial fulfilment
of the requirements for the degree of
Doctor of Philosophy
at The University of Waikato
Department of Earth Sciences

By

Edmund John Brown



**The
University
of Waikato**

*Te Whare Wānanga
o Waikato*

2002

Abstract

A range of constructed groundwater systems are evaluated for augmenting the potable water supply of Auckland City, New Zealand. The term ‘constructed groundwater systems’ encompasses any constructed means of recharging or extracting water from an aquifer. This may for example include infiltration wells and basins, injection wells, aquifer storage and recovery wells, and horizontal wells. The study motivation is to increase the sustainable yield from Auckland’s existing groundwater resources to help meet Auckland’s increasing water demand. Presently, 97% of the water supply comes from 10 storage reservoirs in the Hunua and Waitakere Ranges, and up to three percent from the Onehunga – Mt Wellington aquifer.

The Onehunga – Mt Wellington aquifer is a highly fractured basalt aquifer and has the potential to store water in its unsaturated zone. Various methods of artificially increasing the recharge of the Onehunga – Mt Wellington aquifer to increase its sustainable summer yield were investigated using a FEMWATER numerical model. Three sources of recharge water were identified in the Auckland Region: winter spillage from Watercare’s 10 reservoirs, groundwater from the dewatering of the Three Kings quarry, and wastewater from the Mangere Wastewater Treatment Plant. Artificial storage and recovery (ASR) wells, and soakage wells were simulated as a means of increasing the aquifer’s yield. The greatest simulated increase in yield was from winter injection of reservoir water via ASR wells, in conjunction with the summer extraction of the stored reservoir water. This simulation resulted in the average summer aquifer yield increasing from 9,100 to 44,100 cubic metres per day.

A hydrogeological review of the both the Hunua and Waitakere Ranges showed that there is little opportunity for utilising constructed groundwater systems below the regolith zone in those two locations. However, the storage capacity of the unconfined aquifers in the regolith layer of the Waitakere and Hunua Ranges could potentially be used to increase the efficiency of the existing water supply reservoirs presently losing water to seasonal spillage.

To increase the efficiency of the existing water supply reservoirs it is proposed that horizontal wells can be used to control the water table elevation in the reservoir catchments, permitting a degree of control over the discharge of the streams flowing to the reservoirs. The use of horizontal wells provides a range of control options, including increasing stream discharge during summer and reducing and delaying peak discharges from rainfall events

during winter. The use of horizontal wells in this manner is most suited to catchments where the magnitude of stream quickflow discharge is dependent on the depth of the surrounding water table. That is, for similar-sized rainfall events, a water table near the ground surface results in more quickflow stream discharge compared to a deeper water table.

The Upper Nihotupu water supply reservoir catchment in the Waitakere Ranges was selected as a study site to gather hydrological data to simulate use of horizontal wells. In the Upper Nihotupu catchment, there is a fourfold increase in quickflow discharge for similar sized rainfall events for a water table 0.5 m below the ground surface, compared to a water table 2.7 m below the surface. Based on this, a MODFLOW numerical groundwater model was utilised to simulate the operation of hypothetical horizontal wells within the Upper Nihotupu catchment, and to evaluate if the manipulation of stream discharge will increase the reservoir's storage capacity. Simulation results suggest there is potential for a significant increase in summer stream flow by draining the surrounding aquifer with the horizontal wells. During the 30 day period that the wells are draining the aquifer, the corresponding average reservoir delivery increased from 10,000 to 25,000 cubic metres per day. During the winter, the draining of the aquifer using horizontal wells results in reduced and delayed quickflow stream discharge. This in turn resulted in less spillage from the Upper Nihotupu reservoir. The increased control of the stream hydrograph by the simulated horizontal wells increased the average annual delivery of water from the Upper Nihotupu reservoir by seven percent.

Acknowledgements

I would like to thank my supervisor Dr Earl Bardsley for his support and guidance. Your comments, advice and humor have been invaluable.

I am grateful for the financial assistance given by Technology New Zealand in the form of a TIF scholarship and extra funding for conferences and equipment from Watercare Services Limited.

Thank you to Cameron King, Peter Thomas and Sohail Choudhry who made my time at Watercare very productive by providing data and answering all of my questions.

To all of the Staff in the Earth Sciences Department, thank you for your friendship and help. Particular thanks to Ian Blair, Dave Campbell, Roger Briggs and Sydney Wright.

Thank you to Ian Blair, Blair Lynch-Blosse and Roger Blume for providing support during fieldwork and hand augering many wells in atrocious weather. Also thanks to Roger and Karen Blume and Nick and Jan Brown for your friendly accommodation while I was in Auckland.

Thank you to my wife Catherine and beautiful daughter Abbey who gave me much love and support to see me through to the end. Thank you to my parents who have supported me through my university degrees and the rest of the family who have provided much needed sanity. Thanks to Joanna for the many chats and ideas that kept my motivation up.

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Symbols

Measurement	Units
millimetres	mm
metres	m
metres above mean sea level	m amsl
metres below mean sea level	m bmsl
metres per day	md ⁻¹
millimetres per day	mmd ⁻¹
metres per cubic metre per day	m/m ³ d ⁻¹
square metre	m ²
square metre per day	m ² d ⁻¹
kilometres	km
square kilometre	km ²
cubic centimeters	cm ³
cubic metres	m ³
cubic metres per second	m ³ s ⁻¹
cubic metres per day	m ³ d ⁻¹
litres per second	L/s
litres per hour per square metre	L/hr/ m ²
milligrams per litre	mg/L
grams per cubic metre	g/m ³
kilograms per cubic metre	kg/m ³

Quantity	Symbol
current-electrode half spacing (m)	<i>a</i>
potential electrode spacing (m)	<i>b</i>
weir crest breadth (m)	<i>b</i>
reservoir spillway crest level (m)	<i>B</i>
reservoir spillway discharge coefficient	<i>C</i>
average Penman evaporation for each rainfall event (mm)	<i>E</i>
gravitational acceleration (ms ⁻²)	<i>g</i>
Water level height above reservoir spillway crest (m)	<i>H</i>
gauged head relative to weir crest (m)	<i>h₁</i>
hydraulic conductivity (md ⁻¹)	<i>K</i>
well water column depth (m)	<i>L_w</i>

porosity	n
rainfall depth for each event (mm)	P
crest height of weir above mean bed level (m)	P_1
discharge (m^3d^{-1} or L/s)	Q
resistivity (ohm)	R
well radius including packing (m)	r_w
depth between impermeable layer and bottom of well (m)	s_i
storativity	S
specific yield	S_y
transmissivity (m^2d^{-1})	T
apparent resistivity (ohm m)	ρ_a
soil dry bulk density (kg/m^3)	ρ_b
particle density (kg/m^3)	ρ_p
Nihotupu Stream quickflow discharge volume per rainfall event (m^3)	V
depth to water table below ground surface prior to each rainfall event (m)	W_L

Chapter 1

Introduction

This chapter introduces the objectives of this thesis, Auckland City's water supply system, constructed groundwater systems for recharging and extracting groundwater, and how these methods could be utilised in the Auckland Region to meet the objectives of the thesis.

1.1 Objectives

This thesis presents a number of methods for augmenting the potable water supply of Auckland City using groundwater from local aquifers using constructed groundwater systems. An increase in yield will help meet the rising demand on the present water supply system, and improve security of supply. The three objectives of this thesis are:

- to evaluate the possible increase in groundwater yield from the Onehunga – Mt Wellington aquifer using constructed groundwater systems,
- to provide an overview of the hydrogeology of the Hunua and Waitakere Ranges to identify areas where constructed groundwater systems might be utilised,
- to evaluate if constructed groundwater systems can increase the storage capacity of Watercare's existing catchment reservoirs through improved stream-aquifer management.

1.2 Auckland City Water Supply System

The Water Business Unit of Watercare Services Limited is responsible for the collection, treatment and supply of high quality drinking water to six local network operators in the Auckland region. These operators are Manukau, North Shore and Waitakere City Councils, the Rodney District Council, MetroWater Ltd (owned by the Auckland City Council) and United Water Ltd (the water franchisee of the Papakura District Council). From these six operators the water is supplied to about one million people residing between Orewa in the north and Papakura in the south (Figure 1.1).

Watercare's only groundwater extraction is from the Onehunga – Mt Wellington aquifer, which contributes 3% of Watercare's total water supply (Figure 1.1). The Onehunga groundwater source is from the fractured basalt lava flows on the southern side of Auckland City and is Auckland's oldest existing supply, first developed circa 1880. Watercare's surface water sources are from five reservoirs in the Waitakere Ranges to the west of Auckland, providing 33% of Watercare's total supply, and five reservoirs in the Hunua Ranges to the south of Auckland providing 64% of the water (Figure 1.2). The five reservoirs in the Waitakere Ranges are the Upper and Lower Nihotupu reservoirs, which are in series in the Nihotupu catchment, the Upper and Lower Huia reservoirs which are in series in the Huia catchment, and the Waitakere reservoir in the Waitakere catchment (Figure 1.2). The development of these reservoirs took place between 1901 and 1971. The five reservoirs in the Hunua Ranges are the Wairoa and Cosseys reservoirs in the Wairoa catchment, and the Hays Creek, Upper Mangatawhiri and Mangatangi reservoirs are each at the head waters of their respective catchments (Figure 1.2). Development of the reservoirs in the Hunua Ranges was between 1951 and 1977. All of the reservoirs except the Lower Huia and Lower Nihotupu supply water to the filter stations by gravity flow. The available yield from the Onehunga – Mt Wellington aquifer and the 10 storage reservoirs is 327,000 m³d⁻¹.

During 2000, Auckland's water demand exceeded 320,000 m³d⁻¹ and was very close to the available yield of 327,000 m³d⁻¹. Watercare predicts that by the end of 2002 the water demand will exceed the capacity of the current system, and by 2050 there will be approximately two million water users. To meet the expected water demand Watercare needs to increase the yield from their existing water sources and develop new water sources. In recent years, water conservation methods, including leak management and public education have resulted in Aucklanders on average using only 0.195 m³d⁻¹ of water, compared to 0.23 m³d⁻¹ per person in both Sydney and Melbourne, Australia (Watercare Services Limited, 2001).

Due to the relatively small size of the reservoirs, spillage often occurs during winter due to high inflows. Conversely, during the following summer, the reservoirs' storage only just meets the present demand. During the winter and spring months (July to December) between 1997 to 2000 the average daily spillage from Watercare's 10 reservoirs was 157,884 m³d⁻¹ (Calculation of the reservoir spillage is in Appendix A). Nearly all of the catchments in the Auckland region suitable for reservoir development have already been

utilised. Two remaining catchments identified for their potential to be dammed are; the Lower Mangatawhiri catchment in the Hunua Ranges, and a catchment near Riverhead, 15 km west of Orewa. However, these two reservoir options are not viable due to public concern over the destruction of productive and environmentally important land. Also the new reservoirs are dependent on Auckland's rainfall and will therefore be adversely affected along with the rest of Auckland's reservoirs during future drought conditions. One viable water source is from the importation of water from outside the Auckland Region. In mid 2002, a NZ\$155 million pipeline and filter station will be completed to deliver high quality, treated water from the Waikato River 38 km south of Auckland in the Waikato region (Figure 1.1). The Waikato pipeline will initially provide on average 50,000 m³d⁻¹. The pipeline and filter station can be upgraded to provide 150,000 m³d⁻¹.

The Waikato pipeline is not a guaranteed solution to continually meet Auckland's water demand. There is the potential for contamination of the Waikato River from 33 point discharge sites upstream of the intake site. These include wastewater from Hamilton City and four other smaller communities, discharges from Kinleith timber mill, Wairakei geothermal power station, and an abattoir. There is also a 30 million cubic metre landfill proposed at a site 500 m from the Waikato River bank 18 km upstream from the intake.

The Auckland region is also seismically and volcanically active, which poses a potential risk to the infrastructure of the supply network. The Wairoa North Fault has been identified as having the greatest potential of any fault in the Auckland Region to generate a large earthquake in the future (Figure 1.2) (Woodward-Clyde, 2000). This has major implications for the Hunua Ranges storage reservoirs with the fault being approximately 1 km from the dam structures of the Cosseys and Wairoa reservoirs. Also, the raw water supply pipes from all the reservoirs in the Hunua Ranges, except Hays Creek, cross the Wairoa North Fault (Figure 1.2).

It is important to have a variety of water sources to provide security of supply. This was not the case in the Calleguas Municipal Water District (CMWD) in Southern California, where half a million people were left with little water after the main water supply pipeline was damaged by the 1994 Northridge earthquake. To prevent the reoccurrence of this water shortage, the CMWD setup an aquifer storage and recovery scheme using gravity driven injection and pumped extraction wells (Wolcott, 1999).

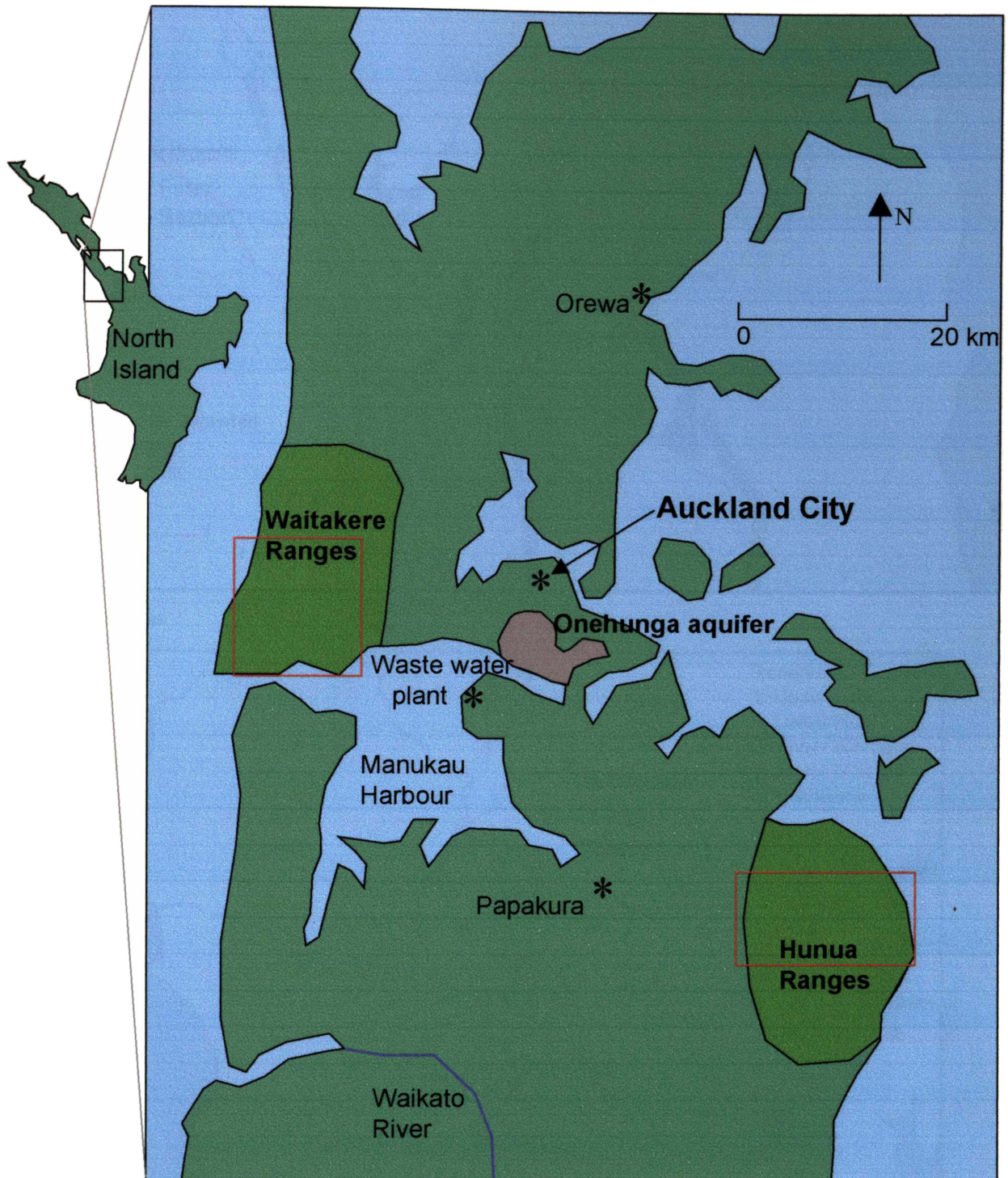
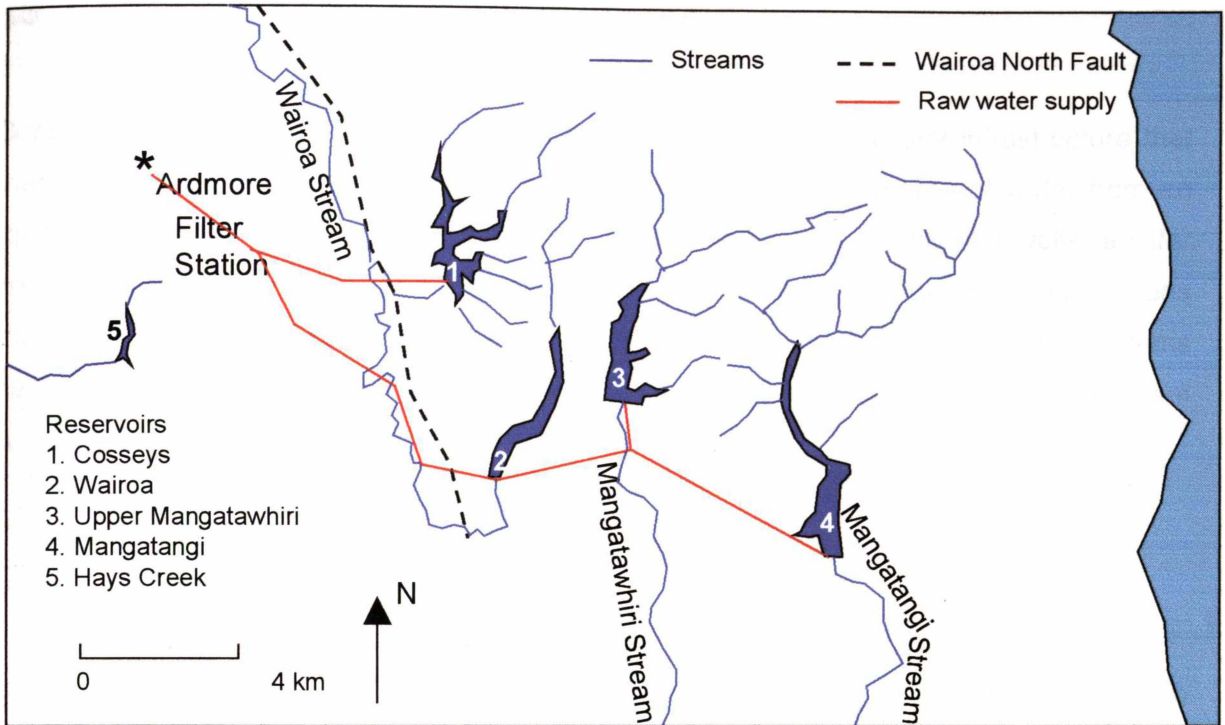
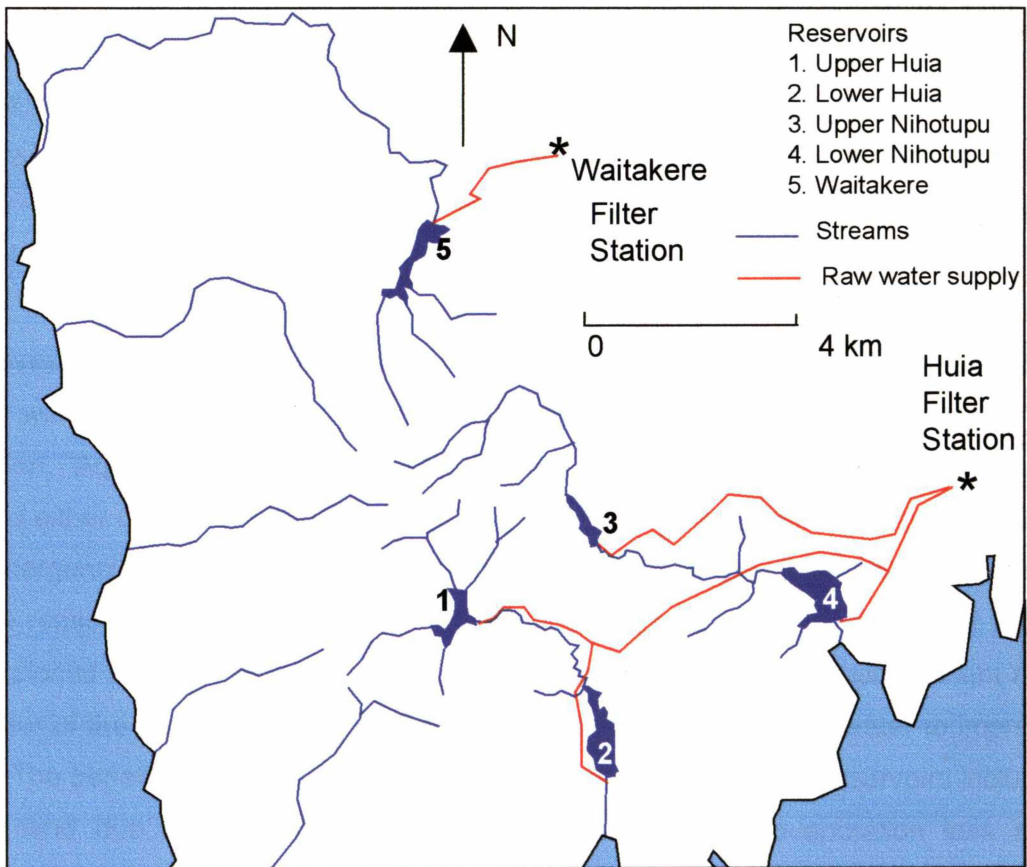


Figure 1.1. Auckland Region. Red boxes outline areas shown in detail in Figure 1.2.



a) Hunua Ranges



b) Waitakere Ranges

Figure 1.2. Watercare's reservoirs in the Hunua and Waitakere Ranges. Locations shown on Figure 1.1.

1.3 Constructed Groundwater Systems

'Constructed groundwater systems' in this thesis are defined to be any infrastructure that enables the artificial recharge of water into an aquifer, and/or discharge of water from an aquifer for potable water supply. This may include; vertical and horizontal wells, aquifer storage and recovery wells, and infiltration basins and trenches. The following sections provide an overview of some of the water sources and constructed groundwater systems that can be used for artificial aquifer recharge, and may possibly be used to augment Auckland City's water supply.

1.3.1 Artificial Recharge

Indirect artificial recharge has been used in Europe since the early 1800's where infiltration galleries were constructed along riverbanks. The water level of these galleries was lowered, inducing the river water to enter the aquifer and be naturally filtered as it flowed through the ground. Examples of these were; the River Clyde in Glasgow, Scotland, and the Garonne in Toulouse, France (Huisman and Olsthoorn, 1983). The first direct-method of artificial recharge was built in 1897 for the water supply of Gothenburg, Sweden. River water was pumped to infiltration basins to recharge the aquifer for water supply (Huisman and Olsthoorn, 1983)

The objectives for artificial recharge broadly fall into two categories, either utilising the aquifer for water storage, or remediation of the aquifer due to unsustainable use or contamination. Aquifer storage is either short term to meet the difference between supply and demand either daily or seasonally, or for long-term storage as a buffer against droughts or other emergencies (Huisman and Olsthoorn, 1983; Ma and Spalding, 1997; Mahesha, 2001). The reasons for aquifer remediation include; recharge to aid the recovery of water levels in exploited aquifers, development of a hydraulic buffer between a contaminant and the remainder of the aquifer, and to either flush or dilute the native groundwater to improve its quality. The benefits of storing water in aquifers compared to surface reservoirs include; no loss of land due to flooding, relatively inexpensive, reduced evaporation loss and possible improved water quality due to filtration through the aquifer (Pyne, 1994; Getchell and Wiley, 1995; Bouwer, 1996b). Disadvantages include: possible contamination due to activities at the ground surface above the aquifer; and clogging of the aquifer by the recharged water reducing the efficiency of the aquifer for the recharge and recovery cycle;

and in an unconfined aquifer the rate and duration of recharge may be limited by the occurrence of water table recharge mounds intersecting the ground surface (Pyne, 1994; Getchell and Wiley, 1995; Bouwer, 1996b).

The choice of whether to use artificial recharge, and the type of artificial recharge method, must be balanced between the following considerations identified by Pyne (1994):

- the required use and demand of the recharge water,
- availability of recharge water, for example daily, or seasonally,
- quality of the recharge water,
- size of available land at a proposed recharge site,
- aquifer geology,
- infiltration capacity,
- and economic considerations.

1.3.1.1 Water Source and Quality

There are three main water sources available for aquifer recharge in most urban environments. These are wastewater, stormwater, and surface water from streams and reservoirs. The quality of most water used for recharge needs to be improved to limit aquifer clogging and contamination. Organic and metallic toxicants, nitrogen compounds, and pathogens have the greatest adverse effects when the reuse of the water is for potable use (National Research Council, 1994). Most treatment processes of wastewater and potable water include chlorination. This has been considered a concern due to the potential formation of disinfection by-products (DBPs) like trihalomethanes (THMs) and halo-acetic acids (HAAs) which are thought to be carcinogenic (Asano, 1985; Driscoll, 1986; National Research Council, 1994). However, more recent studies have shown that THMs and HAAs are removed from the chlorinated water during aquifer storage over a period of several weeks so that the pretreatment may not be such a concern (Singer, *et al*, 1993; Pyne, 1998).

a) Wastewater

Wastewater may contain contributions from domestic and industrial sources. Combined sewer systems will also contain urban stormwater runoff. Wastewater is usually the most

readily available and plentiful water source. However, the use of wastewater to augment potable groundwater supplies is generally viewed as being unacceptable by the public, and does not meet the cultural requirements of New Zealand's indigenous Maori population.

Although some impacts of artificial recharge of groundwater with wastewater are not fully understood, experience with recharge projects have shown (within the limitations of toxicological testing) that water recovered from the aquifer poses no greater health risks than currently acceptable potable water supplies (National Research Council, 1994; Bouwer, 1996b). However, epidemiological studies are weakened by the recognition that the minimum observed period for human cancers that have been linked to chemical agents found in wastewater is about 15 years. Because of the relatively short period that groundwater containing wastewater had been consumed, it is unlikely that examination of cancer mortality rates would have detected an effect, if present (National Research Council, 1994). The viability of artificial recharge for potable use, and the risks associated with using impaired water for recharge will vary from site to site, and thus the appropriateness of all recharge is site-specific (Bouwer, 1996b). Due to the possible health risks associated with wastewater, extracted groundwater which is mixed with recharge wastewater is often used for non potable purposes, such as industry and irrigation of vegetables for raw consumption and livestock watering (Idelovitch and Michail, 1984; Kanarek, Aharoni and Michail, 1993; Kanarek and Michail, 1996). Utilising the groundwater mixed with wastewater for non-potable purposes in turn reduces the demand on the existing potable supply.

b) Stormwater

Urbanisation has resulted in many aquifers receiving less recharge via natural infiltration, this is especially the situation if the stormwater is diverted out of the catchment. Diverting this runoff from paved surfaces into an aquifer can represent a significant source of recharge, as well as being an inexpensive disposal option. Recharge of stormwater is usually via soakage wells. Artificial recharge with stormwater at Mount Gambia, Australia has occurred since the late 1800's. However, the recharging of the aquifer was an incidental side effect of stormwater disposal to reduce surface flooding (Dillon, *et al*, 1999).

The quality of recharged stormwater is the main concern as stormwater can contain fuels and oils from vehicles, pesticides from roadside spraying, and heavy metals such as lead, zinc, copper, cadmium and nickel (Price, 1994; Pitt, *et al*, 1996). However, there does not seem to be any evidence in the United Kingdom that any groundwater supply in contact with

stormwater recharge has been seriously affected (Price, 1994). The risk of contamination can be reduced if the soakage wells are equipped with sediment traps. The sediment traps need to be cleaned regularly to remove the accumulation of polluted water before the development of bacteria and high concentrations of pollutants accumulate. If not maintained the contents of the sediment traps can be flushed into the aquifer (Price, 1994).

c) Surface Water (Reservoirs and Rivers)

Diverted surface water is generally the highest quality recharge water available. However, even river or reservoir water may contain sediments that can block aquifer pore spaces. The settling of turbid water can be increased using coagulating or flocculating chemicals (Huisman and Olsthoorn, 1983). Chlorination may be necessary to stop algae or bacteria contamination and clogging of the aquifer (Digney and Gillies, 1995).

1.3.1.2 Injection Wells

Injection wells can be used to artificially recharge water into both unconfined and confined aquifers. Injection wells have been used in Kentucky, USA since the 1940's for the sole purpose of recharging exploited aquifers (Todd, 1980). A more recent example is in El Paso, Texas, where the injection of wastewater via wells has been used to recharge the Hueco Bolson aquifer to slow down the rate of the declining water levels. The Hueco Bolson aquifer provides 90% of the potable water supply for the city of El Paso (National Research Council, 1994). The wastewater recharge each year is equivalent to 10% of the total extracted volume of groundwater, this means that every 10 years the aquifer's resource lifetime is extended by 1 year. The tertiary treated wastewater is recharged via 10 injection wells, and then travels 1.2 km through the aquifer to the production wells for the municipal supply. The minimum residence time for the wastewater in the aquifer is 7 years. The reclaimed water is chlorinated before potable reuse, with no other treatment. No significant operating problems have been encountered in the 7 years since 1986 (National Research Council, 1994).

There are some physical aspects of injection well construction that separate them from normal extraction wells. If the injection wells are used intermittently or seasonally they tend to have increased rusting of the casing due to frequent dry and wet cycles. This rust can reduce water quality and be another cause of the well clogging. To stop rusting PVC casing

can be used for wells less than 150 m deep and 900 mm in diameter (Pyne, 1994). Cementing around the casing must be of high quality to stop the injected water flowing up the outside of the casing to the surface (Pyne, 1994).

The main limitation of injection wells is from clogging. Clogging of the well is typically caused by suspended solids, entrained air, and biological growth (Huisman and Olsthoorn, 1983; Pyne, 1994). Figure 1.3 shows the generalised relationship between time and resistance to flow for the three typical clogging processes.

a) *Suspended Sediments*

In unconsolidated formations, the aquifer behaves as a filter removing suspended sediments from the recharged water. This can result in particles physically blocking the pore spaces (Bowen, 1986). However, no treatment at all may be required for water entering a fractured rock aquifer with openings varying from a few centimetres to some metres wide. In this situation, the natural purification accompanying underground flow is also small, so pre-treatment may be necessary to prevent contamination of the aquifer by the recharged water (Huisman and Olsthoorn, 1983; Bowen, 1986).

b) *Air Entrapment*

Air entrapment can be prevented through; proper well design and operation, maintaining positive pressures, keeping the well screen below the water level in the well, and using recharge water with a similar temperature to the groundwater. A number of injection well experiments during the 1960's failed due to wells not being kept primed and under positive pressure (Pyne, 1994; Wolcott, 1999).

c) *Biological Growth*

Biological growth is directly related to the amount of carbon and nutrients present. These may be low in the recharge water but may increase in concentration due to their accumulation in the vicinity of the well. A residual chlorine level of 1 to 5 mg/L in the recharge water will control most biological growth (Crook, Asano and Nellor, 1990; Tompson, *et al*, 1999).

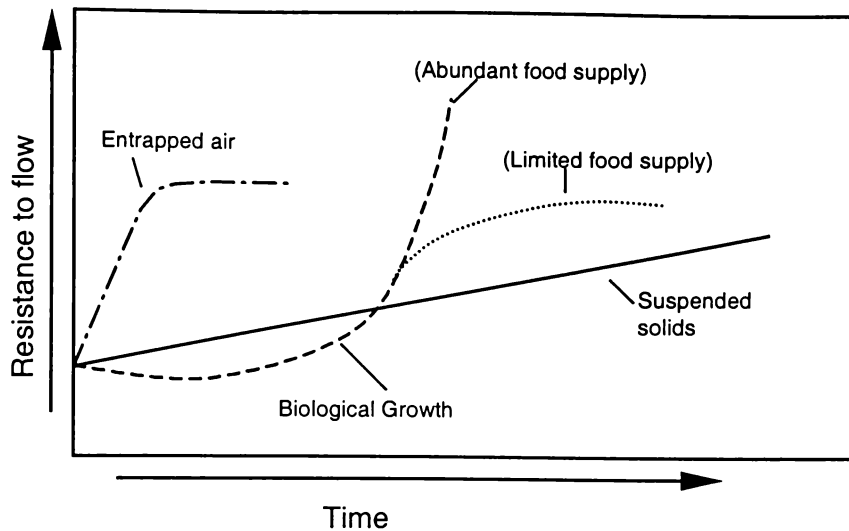


Figure 1.3. Time variation of typical injection well clogging processes. Adapted from (Pyne, 1994).

1.3.1.3 Aquifer Storage and Recovery Wells

Aquifer storage and recovery (ASR) wells are relatively new and are increasingly used for artificially recharging aquifers (Bouwer, 1996b). Compared to injection wells, ASR wells inject water into an aquifer and are then reversed to extract water out of the aquifer. This method has enabled the increased use of aquifers for temporary storage of water and also the use of aquifers with low water quality for storing high quality water (Bowen, 1986). In an ideal situation, the injected high quality water remains in a plume surrounding the well displacing the low quality groundwater. During extraction the well extracts this plume of high quality water leaving the low quality groundwater in the aquifer (Bowen, 1986). The recharge enables the storage of water during times of excess ready for extraction during times of high demand and shortfall, essentially utilising the aquifer as a storage reservoir. If the aquifer is recharged with potable water, the water extracted often needs minimal treatment before reuse for potable purposes. The use of ASR schemes can also be inexpensive compared to surface reservoirs. The cost of an ASR scheme in California was equivalent to one tenth of the cost of a new dam constructed at the same time east of Los Angeles for storing the same volume of water (Wolcott, 1999).

ASR in an unconfined aquifer is feasible, but there may be reduced efficiency in extracting all the injected water. The groundwater velocity is usually higher for unconfined aquifers, and as a result the recharged plume can often flow away from the well (Pyne, 1994). The greater the storage period before recovery of the recharged water, the greater the mixing

with the native groundwater. Therefore it may be impossible to recover nearly all the recharged water. This is of particular importance where the native groundwater is of lower quality than the recharged water (Pyne, 1994).

1.3.1.4 Soakage Wells (Vadose-Zone Wells)

Soakage wells are simply holes drilled or dug into permeable geology allowing the infiltration of water into an aquifer. They are typically used for the disposal of stormwater from roads and properties, and are much cheaper than recharge wells. Clogging can be minimised by using treated water and constructing a sediment trap. It is also important not to drill through clay layers, as this can produce muddy water when the well is filled with recharge water.

Soakage wells are used in Auckland City where the underlying geology is fractured basalt, providing a cost-effective means of stormwater disposal from roadways and residential properties. The wells are generally 100 to 150 mm in diameter, with an average depth of 13 m. Soakage wells intersecting fractured basalt or cavities have infiltration rates between 780 and 2,160 m³d⁻¹, with an average rate of 1,500 m³d⁻¹. Soakage wells intercepting solid basalt have infiltration rates up to 260 m³d⁻¹.

In the confined cavernous limestone aquifers of Orlando Florida, stormwater soakage wells can dispose of between 8 and 52,000 m³d⁻¹ per well. The wells are typically 300 mm in diameter, 120 m deep and have been in use since 1904 (National Research Council, 1994). The potentiometric surface of the receiving aquifer is well below the ground surface, enabling the wells to recharge the confined aquifer with large volumes of water under gravity feed. Widespread contamination of the aquifer has not occurred (National Research Council, 1994).

1.3.1.5 Infiltration Basins (Soil-Aquifer Treatment Systems)

Infiltration basins are large basins which are regularly flooded with water, allowing the infiltration of water through the permeable soils to recharge an aquifer. The recharged water can be extracted from the aquifer using wells or drains. Infiltration basins do not always require large-scale construction, as there may be existing river channels or gravel pits which can be flooded. Infiltration basins are generally flooded with high quality water

when the final required use is potable and at least primary treated wastewater if the final use is non potable, such as irrigation and agriculture. The volume of recharge via infiltration basins can be quite substantial. In Orange County California, using infiltration basins the water authority artificially recharges an average of 693,000 m³d⁻¹ of river water, wastewater, and water imported from outside the region. This is equivalent to 76% of the total water extracted from the aquifer system (Tompson, *et al*, 1999).

In unconsolidated and unfractured aquifers, the recharging of impaired waters via infiltration basins has the ability to greatly increase the quality of the recharged water as it filters through the underlying aquifer. The improved quality of the wastewater recharged through infiltration basins can be better than from conventional treatment methods (Rice and Bouwer, 1984). In Los Angeles, the first wide scale recharge of secondary wastewater via infiltration basins began in 1962, and there is no distinguishable difference in aquifer water quality due to the recharge (Crook, Asano and Nellor, 1990). A similar recharge-recovery system has been operating since 1977 in the Dan Region, Israel. This consists of intermittent flooding of infiltration basins with 40,000 m³d⁻¹ of secondary treated wastewater. This process resulted in a relatively high removal of suspended solids (mainly organic), biological oxygen demand, and a lowering in the total and dissolved oxygen demand. The total bacteria count was considerably reduced; coliform bacteria, fecal coliforms and enteroviruses were not detected in the reclaimed water. The extracted water was suitable for non-potable uses such as industry and irrigation of vegetables for raw consumption and livestock watering (Idelovitch and Michail, 1984; Kanarek, Aharoni and Michail, 1993; Kanarek and Michail, 1996). Similar removal of wastewater constituents have occurred at many sites throughout the world when using wastewater for aquifer recharge and reuse (Lance, Rice and Bouwer, 1978; Carlson, *et al*, 1982; Bouwer, 1991; Gerba, *et al*, 1991; Wilson, *et al*, 1995; Kopchynski, *et al*, 1996; Ouazzani, Bousselhaj and Abbas, 1996; Quanrud, *et al*, 1996).

To minimise clogging, infiltration basins are best operated in wet/dry cycles. This simply involves flooding the basin and then allowing it to drain and dry out. Once the basin is dry it is refilled with water. The drying out process helps to control the nutrient and sediment load and reduce the development of clogging layers in the basins. Also, a shallower water depth in the basin can result in less clogging due to the reduced compaction of the sediments at the bottom of the basin (Lee, Williams and Wang, 1992; Mushtaq, Mays and Lansey, 1994; Ma and Spalding, 1997).

1.3.2 Groundwater Extraction

The following sections overview constructed groundwater systems as used for groundwater extraction, some of which could be of relevance for augmenting Auckland City's water supply.

1.3.2.1 Vertical Wells

Vertical wells in this thesis are defined as vertical holes excavated to allow groundwater extraction. The simplest and oldest wells were hand-dug, and tend to be shallower than 15 m. Excavating larger diameter wells helps to increase the volume of stored water in the well. Deeper drilled wells are able to extract water from depths of several hundred meters. Pumps are required to lift the water to the surface if the static groundwater level is a substantial distance below the ground surface. Pumps which can extract water from a substantial depth, and fit down small-diameter wells have only been available since the 1880's (Todd, 1980).

1.3.2.2 Horizontal Wells

Qanats are the oldest form of horizontal wells, having provided groundwater from unconfined aquifers for domestic supply and irrigation in the Middle East and central Asia since 550 B.C. (Beaumont, 1971; Abu-Rizaiza and Mohorjy, 1994; Kahlowan and Hamilton, 1994; Lightfoot, 1995). Qanats are a form of subterranean aqueduct (Lightfoot, 1995), designed to collect and transport groundwater under natural gravity flow through gently sloping tunnels to surface canals for distribution (Figure 1.4). The tunnels were hand dug by skilled workers and are typically 1 to 5 km long, but have been as much 50 km in length (Kahlowan and Hamilton, 1994). A tunnel cross section is typically elliptical with a height of around 1.2 m and a width of 0.8 m (Beaumont, 1971). A vast number of qanats still remain in operation. In Iran about 22,000 qanats supply three quarters of all water used. The discharge of individual qanats varies with seasonal water table fluctuations, but rarely exceeds $1,400 \text{ m}^3 \text{d}^{-1}$ (Beaumont, 1971). The discharge of qanats cannot be regulated, there may be leakage loss through the base of the tunnel, and during winter water can often go to waste when there is more supply than demand.

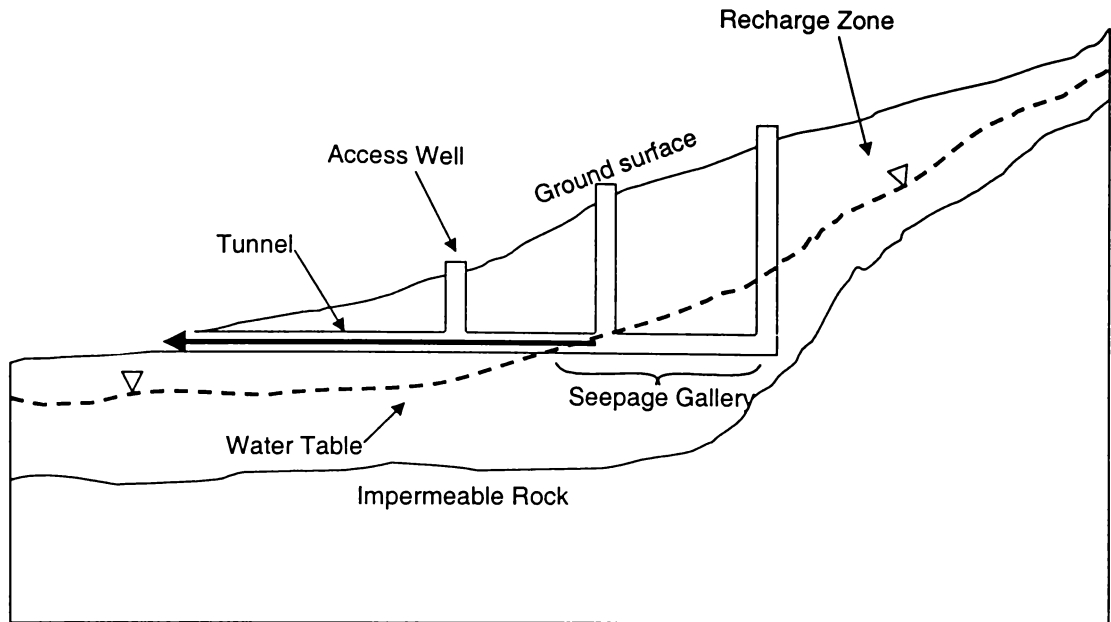


Figure 1.4. Schematic of a qanat system, adapted from Kahlow and Hamilton (1994).

Similar to the Middle Eastern and central Asian qanats are the horizontal tunnels used in Hawaii, called 'Maui tunnels'. These tunnels are either horizontal or have an incline from the ground surface to the water table, and are then horizontal at a level just above sea level (Figure 1.5). The latter of the wells are termed 'skimming wells' and are necessary for extracting groundwater from shallow fresh water bodies on top of saline water. The horizontal tunnels are used to collect groundwater that has been held behind impermeable rocks. One Maui tunnel is 300 m long and produces $50,000 \text{ m}^3\text{d}^{-1}$ (Peterson, 1973). However, due to economic considerations and the ability to extract water from greater depths with vertical wells no new Maui tunnels have been constructed since the 1950's.

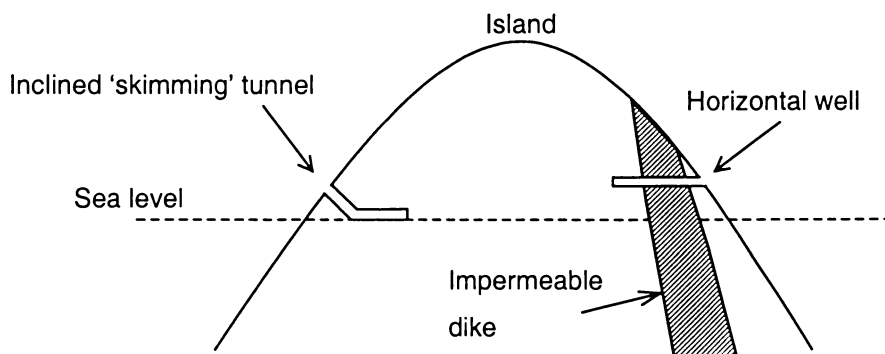


Figure 1.5. Schematic of Maui tunnels used on Hawaiian islands for collecting groundwater from just above sea level and from behind impermeable dikes. Adapted from Peterson (1973).

The use of horizontal wells for groundwater extraction is becoming popular again due to recent developments in horizontal well drilling technology. Horizontal well technology has been developed and used by the oil industry for many years, but it has only been in recent years that drilled horizontal wells have been utilised for groundwater extraction (Sawyer and Lieuallen-Dulam, 1998). Most applications are for drainage to aid slope stability (Craig and Gray, 1985), or groundwater remediation (Cleveland, 1994; Sawyer and Lieuallen-Dulam, 1998; Zhan, 1999). To date, there does not appear to be any utilisation of horizontal wells for potable water supply applications.

Comparisons between horizontal and vertical wells have been based on the horizontal wells' efficiency at remediating contaminated aquifers (Cleveland, 1994; Sawyer and Lieuallen-Dulam, 1998; O'Neil, *et al*, 1999; Steward, 1999; Steward and Jin, 2001). Sawyer and Lieuallen-Dulam (1998) concluded that a horizontal well aligned parallel to the direction of uniform flow is the most effective type of well for aquifer remediation. Horizontal wells are typically more expensive on a per-metre drilled basis than vertical wells. Horizontal wells installed in depths shallower than 10 metres depth are typically up to US\$700 per metre, compared to US\$125 - \$300 per metre for vertical wells (O'Neil, *et al*, 1999). However, lower operation and managing costs can often make horizontal wells the better option in the long term (O'Neil, *et al*, 1999). Horizontal wells can be drilled with little damage to the ground surface, compared to the excavation of drainage trenches or access roads for numerous vertical wells. With a large portable rig, the current maximum possible length of a horizontal well is 2,100 m for the installation of pipe up to 150 mm in diameter (Swanson, 1997). Care is required when installing long horizontal wells so that the complete length of the well screen is below the water table level. Screened sections of the well above the water table will lose water back to the aquifer (Steward and Jin, 2001).

Horizontal wells can provide one of the most effective methods to control the water table where the excavation of drains is not possible (Treadway, 1997; Steward and Jin, 2001). This in turn offers a means to control runoff. It has been well documented that groundwater levels exert a major control on infiltration, evaporation, and runoff (Neal, *et al*, 1992; Durand, Neal and Neal, 1993; Salvucci and Entekhabi, 1995b; Salvucci and Entekhabi, 1995a; Ribolzi, *et al*, 2000; Scanlon, Raffensperger and Hornberger, 2000; Beldring, 2002). A water table close to the ground surface tends to produce more runoff and evaporation than a deeper water table. Controlled drainage can significantly reduce the extent of runoff-generating partial areas, in turn reducing the peak outflow from storm events and allowing

the extra water to be retained within the catchment (Amatya, Gregory and Skaggs, 2000; Querner, 2000; Querner and van Lanen, 2001).

1.4 Augmenting the Auckland City Water Supply with Constructed Groundwater Systems

In this thesis, two applications of constructed groundwater systems are investigated for their ability to improve Watercare's existing water supply system. The first application is to increase the sustainable yield from the Onehunga - Mt Wellington aquifer. The second application is to utilise the aquifers in the Hunua and Waitakere Ranges to reduce winter spillage from the 10 reservoirs and to increase the effective summer reservoir storage without increasing the physical size of the reservoirs.

1.4.1 Existing Onehunga – Mt Wellington Groundwater Source

The Onehunga – Mt Wellington unconfined aquifer is located in the fractured basalt flows on the southern side of the Auckland isthmus (Figure 1.1). Watercare Services Limited are the main extractor of groundwater and presently utilise four production wells in the Onehunga area near the Manukau Harbour. However, there is often an imbalance between the public water demand and the available water supply from the Watercare wells. During winter the available aquifer supply exceeds demand, but during summer the converse occurs, where the demand exceeds the available aquifer supply. To reduce this seasonal imbalance between supply and demand, five constructed groundwater systems are evaluated in this thesis for their potential to increase Watercare's summer yield. The five constructed groundwater options presented in this thesis are; (1) aquifer storage and recovery (ASR) using the existing Watercare wells and also lowering of Watercare wells' intake screens, (2) seasonal aquifer recharge with reservoir water via ASR wells, (3) seasonal aquifer recharge with reservoir water via soakage wells, (4) recharging groundwater from the Three Kings quarry via soakage wells; and (5) recharging tertiary treated wastewater via soakage wells. In the Auckland Region there are three sources of readily available water for this recharge; (1) treated water from Watercare's 10 storage reservoirs during winter; (2) groundwater from the Three Kings quarry dewatering operation; and (3) highly treated wastewater from Watercare's Mangere treatment plant.

1.4.2 Groundwater and Surface Water Interaction in the Storage Reservoir Catchments of the Hunua and Waitakere Ranges

Groundwater in the Hunua and Waitakere Ranges is not currently utilised except as a source of recharge for the 10 reservoirs. Below the regolith zone in both ranges there is little groundwater, other than in the weathered crush zones of fault lines. Thus, future groundwater utilisation is mainly limited to the regolith layer, and possibly the fault zones.

Watercare's reservoir catchments are vegetated in protected forests, which limits the accessibility for developing some constructed groundwater systems. An alternative approach may be to utilise horizontal wells to give an element of control of sub-forest water table elevations in the reservoir catchments. The horizontal wells could be constructed with minimal disturbance of the ground surface and would not require pumping because the water drains under gravity flow. One option is to use horizontal wells to drain the regolith zone aquifers in Watercare's reservoir catchments. During winter this may reduce the partial-areas of the catchments contributing to runoff during winter storm events, which result in reservoir spillage. During summer the draining of the aquifers may augment the discharge of the streams flowing into the reservoirs.

The operation of horizontal wells for controlled water supply is envisaged as shown in Figure 1.6. The wells are drilled on a slight incline into a hillside, with the screened length below the water table. Water entering the well flows under gravity to the lower end, where an outlet valve controls the release of water. After opening the valve, the water table lowers as the groundwater drains away. Conversely, closure of the valve allows the water table elevations to recover as recharge percolates down to the water table. This in turn, allows some control over the discharge of the streams supplying the reservoirs. In a sense, the storage capacity of the unconfined aquifers becomes an extension of the operational storage of the reservoirs. This concept is most applicable for catchments where high water tables enlarge the partial-areas contributing to increased quickflow discharges and reservoir spillage.

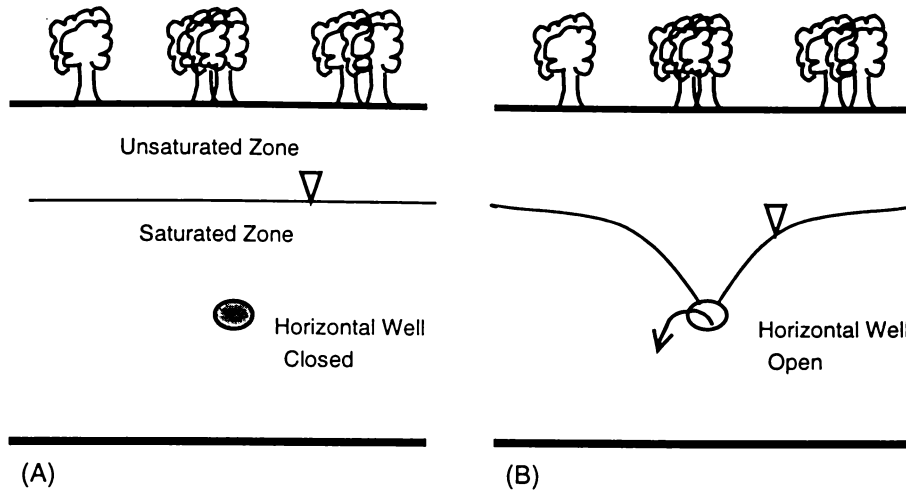


Figure 1.6. Envisaged operation of horizontal well manipulating the water table elevation within the regolith. (A) Initial conditions with drains closed and water table at its normal level, (B) drains opened and water table lowering with discharge from the lowest end of the well. When the well is closed the water table will recover to the level in (A).

1.5 Thesis Structure

This first chapter has introduced Watercare's water supply system for Auckland City and provided a brief overview of the constructed groundwater systems which could be used to increase Auckland's potable water supply from existing and currently underutilised groundwater sources in the region.

Chapter 2 covers the hydrogeological information collected on the Onehunga – Mt Wellington aquifer which is then used in Chapter 3 for the model-based evaluation of five scenarios for increasing the aquifer's recharge and yield. The evaluation of the five scenarios also incorporates economic considerations.

Chapter 4 provides an overview of the hydrogeology of the Hunua and Waitakere Ranges to evaluate the potential use of constructed groundwater systems for reducing reservoir spillage and augmenting Watercare's water supply. Chapter 5 discusses the collection of hydrogeological information on the regolith zone in the Upper Nihotupu catchment in the Waitakere Ranges. This catchment has been selected as a trial site to evaluate the use of horizontal wells for reducing the spillage from the Upper Nihotupu reservoir during winter, and increasing the reservoir yield during summer through using the surrounding unconfined aquifer as a storage extension of the surface reservoir. The hydrogeological information

discussed in Chapter 5 is used in Chapter 6 for a model-based evaluation of horizontal wells for augmenting Watercare's supply. The evaluation also incorporates economic considerations.

Conclusions for the thesis are presented in Chapter 7.

Chapter 2

Evaluation of Constructed Groundwater Systems in the Onehunga – Mt Wellington Aquifer: Site Investigations

2.1 Introduction

The Onehunga – Mt Wellington unconfined aquifer is located in the fractured basalt flows on the southern side of the Auckland isthmus (Figure 2.1). The aquifer¹ covers an area of 40 km² and is one of Auckland's oldest water sources dating back over 100 years. Watercare Services Limited is the main user of groundwater in the Onehunga – Mt Wellington aquifer, and presently operate four production wells in the Onehunga area near the Manukau Harbour. There is often an imbalance between water demand and available water supply from the Watercare wells. During winter the available aquifer supply exceeds demand, but during summer the converse occurs where the demand exceeds the available aquifer supply. To reduce this imbalance between supply and demand, five constructed groundwater systems are evaluated for their potential to increase Watercare's summer yield. These include development of the existing Watercare production wells, and both seasonal and annual artificial storage and recovery schemes using a variety of different constructed groundwater systems and recharge water sources. The Onehunga – Mt Wellington aquifer has the potential to be recharged with more water than is available from natural recharge. Over much of the aquifer there are large thicknesses of unsaturated highly permeable rock suitable for storing large volumes of water.

This chapter collates the available information relating to the Onehunga – Mt Wellington aquifer, giving an overview of the geology and dynamic behaviour of the aquifer system and its suitability for the constructed groundwater systems. This information is used to develop a conceptual and numerical groundwater model of the aquifer to enable model-based evaluation of the various constructed groundwater systems for increasing the recharge and sustainable yield (Chapter 3).

¹ References in Chapters 2 and 3 to the 'aquifer' refer to the unconfined volcanic Onehunga – Mt Wellington aquifer, unless otherwise stated.

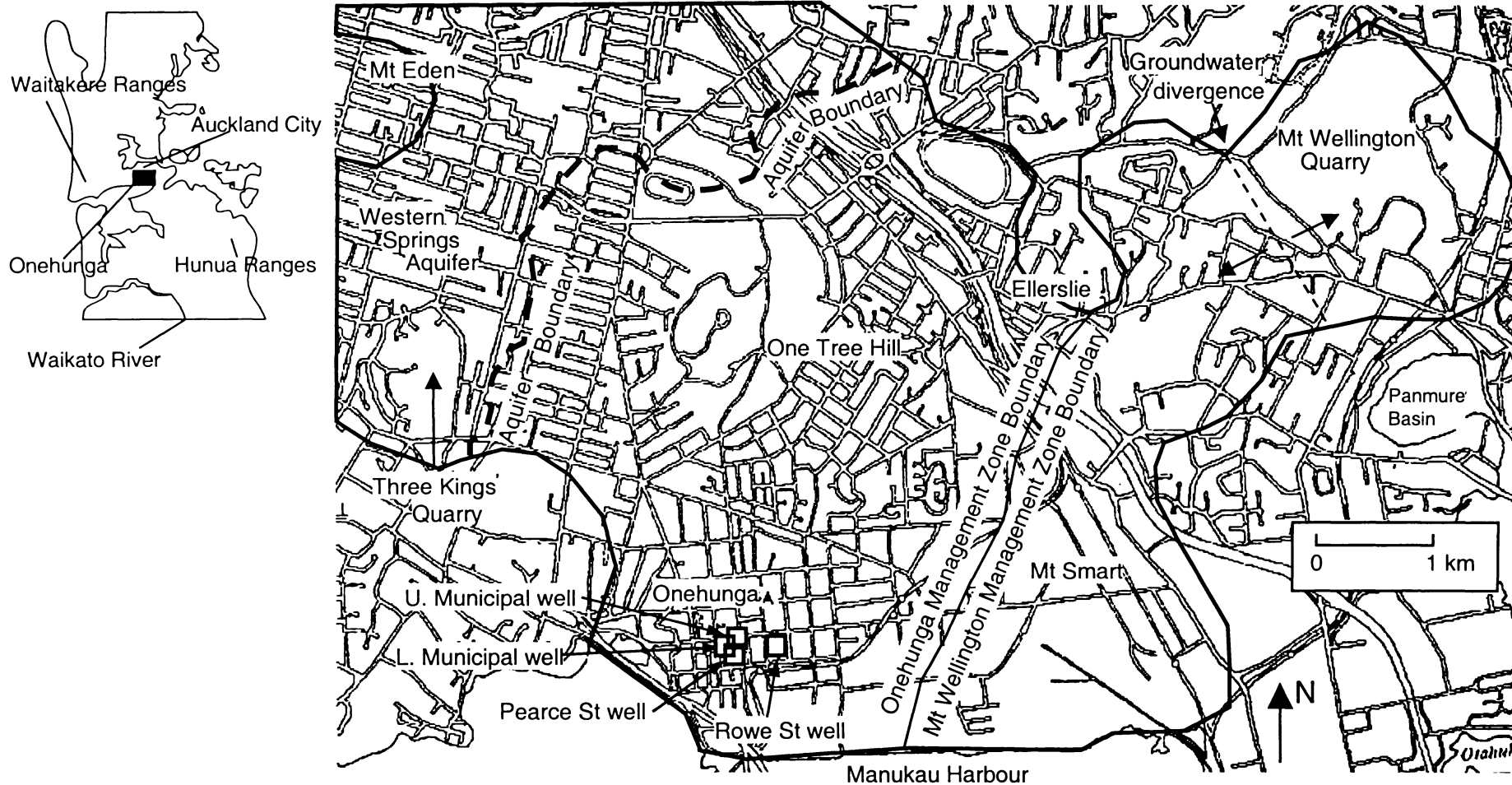


Figure 2.1. Location and management zones of the Onehunga – Mt Wellington aquifer on the southern side of the Auckland isthmus. Also included is the head of the adjacent Western Springs aquifer.

2.2 Previous Investigations

A brief overview of the available publications on the Onehunga – Mt Wellington aquifer is given below. There are numerous other private consultancy reports and accompanying resource consents that contain information on pumping tests, geology, and water use. These have not been summarised in this section but are referred to in appropriate places in the thesis.

a) Geology

Kermode (1992a) and Searle (1981) both provide detailed explanations of Auckland's geological history.

Five geological studies of the Onehunga – Mt Wellington aquifer have been completed using geophysical methods. The first two studies were by Lawton and Hochstein (1974) and Hochstein and Lawton, (1975). Both utilised gravity and magnetic surveys, and both were measured along four parallel transects across basalt flows in the Onehunga area. Cassidy (1985), used gravity and resistivity measurements recorded on a grid in the Onehunga area to find the depth of the volcanic aquifer. Roberts (1980) used airborne magnetics, gravity surveys and DC resistivity soundings to measure the aquifer thickness in the Mt Wellington area. The most recent geophysical survey was by Affleck (1999), covering an area northwest of One Tree Hill. This survey utilised both ground-penetrating radar and gravity surveys to measure the extent of the tuff and lava flows.

b) Aquifer Use and Management

Pattle Delamore Partners Limited (1991) reviewed all of Auckland's aquifers, noting their resource availability, quality, security of supply, and potential users. Smaill (1993a; 1993b) produced two management plans for the Onehunga – Mt Wellington aquifer, dividing the aquifer into the One Tree Hill – Onehunga Management Zone and the Mt Smart – Mt Wellington Management Zone (Figure 2.1). Both plans discussed water allocation, aquifer recharge, storm water disposal, and the monitoring of groundwater levels and quality. The management plans were developed to ensure the long-term sustainable use of the groundwater resource, and to protect and improve the groundwater quality. Riley

Consultants Limited (1995) reviewed Mt Wellington aquifer's hydrological parameters, groundwater availability, the effects of extraction, and the potential pollution of the aquifer.

Recent interest in the possible dewatering of the Three Kings quarry from its present approximate level of 55 m down to sea level has prompted three studies by Carryer & Associates Limited (1994), Groundsearch EES Limited (1994; 1996). These studies provide a hydrogeological overview of the quarry area including geophysical methods and groundwater flow modelling using the MODFLOW numerical modelling software.

c) Groundwater Theses

Roberts (1980) researched the groundwater surface flooding in the Mt Wellington area. Roberts' investigation extensively used geophysical methods (airborne magnetics, gravity surveys and DC resistivity soundings) to determine the thickness and extent of the volcanic aquifer. The average thickness of the aquifer in the area of the Mt Wellington quarry was estimated by Roberts to be 40 m.

Watson (1995) developed a three-dimensional finite difference steady state model using MODFLOW for the Onehunga area of the aquifer. Due to the inability of Watson's model to converge on a solution, the model domain was reduced to the vicinity of the Watercare supply wells. The model was then used to evaluate effects of possible hazardous spills in the aquifer.

Namjou (1996) evaluated the use of the Mt Wellington Quarry as a landfill. This study investigated both the basalt and Waitemata aquifers to determine the preferential pathways which contaminants might follow. Namjou attempted to utilise ground penetrating radar but found its depth of penetration to be too shallow. A three-dimensional finite difference numerical model was developed using MODFLOW. The model domain covered an area north-east of the Mt Wellington groundwater divide (Figure 2.1), presently caused by the quarry dewatering operation.

Viljevac (1998) researched the hydrogeology of the Western Springs aquifer, and developed a numerical groundwater model using MODFLOW. Difficulties were encountered using MODFLOW due to the complex nature of the geology. These were overcome at the expense of simplifying the geology and using a single-layer model.

2.3 Geology

2.3.1 Regional Setting

The geological development that resulted in the Auckland isthmus and the Onehunga – Mt Wellington aquifer began 25 – 30 million years ago during the Miocene period. The greywacke formation that underlies most of New Zealand was in a state of submergence. During this period the Waitemata Group was laid down in a 2 km thick sequence, consisting of alternating sandstone and mudstone (Kermode, 1992a). In the late Miocene - early Pliocene deep tectonics produced block faulting, resulting in the Manukau Harbour and Waitakere Ranges dropping in relation to the Auckland block. Rivers and coastal processes eroded the faulted outlines of these blocks. During the subsequent Pleistocene period there were four ice ages resulting in extreme fluctuations of sea level. When the sea level was reduced the existing rivers eroded deeper and wider into the Waitemata sediments. When sea level rose there was a development of terraces and deposition of marine sediments (Searle, 1981; Kermode, 1992a).

After this period of sea level fluctuation, the Waitemata and Manukau Rivers dominated the Waitemata Group. The Manukau River is the smaller of the two and its tributaries underlie the area of the Onehunga – Mt Wellington aquifer (Figure 2.2). Between 10,000 and 7,000 years ago, sea levels rose by approximately 30 m to the present level. During the sea level fluctuations between 60,000 and 600 years ago, more than 50 volcanoes erupted in the Auckland region over a 500 km² area (Searle, 1981). Mt Smart, Mt Wellington, One Tree Hill and Three Kings volcanoes produced lava flows which infilled the valleys and tributaries of the Manukau River, producing the fractured basalt aquifers on the southern side of the Auckland isthmus. Table 2.1 summarises the geological events that formed the Auckland landscape, relating particularly to the Onehunga – Mt Wellington aquifer.

Table 2.1. Time chart of the geological events that produced the present Onehunga – Mt Wellington Aquifer. Adapted from Searle (1962, p106).

Era	Period	Years since period began	Dominant conditions	Events of local importance
Quaternary	Recent	10,000	Rise in sea level to maximum and fall to present level	Modern harbour formed silts and muds deposited. Eruptions continue spasmodically.
	Pleistocene	1 million	Fluctuations in sea level during four ice ages	Auckland's volcanoes begin erupting. Deep gulying of Waitemata and Manukau Rivers and their tributaries. Waitemata terrain excavated into flights of terraces. Silts and peats deposited in valleys during times of high sea level in the Manukau lowlands.
Tertiary	Pliocene	12 million	Faulting creates major earth blocks Erosion Orogeny uplifts area	Auckland city a land area. Waitemata terrain eroded leaving Waitakere Ranges as high hills. Manukau lowlands a basin of sedimentation, silts, sands, and shell beds deposited.
	Miocene	26 million	Waitemata sedimentation	Eruption of Manukau Breccia. Sandstones, mudstones, volcanic grit beds and conglomerates of Waitemata Group deposited.
	Oligocene	38 million	Fluctuating conditions	Erosion occurs. Auckland probably a land area.
Mesozoic	Jurassic	150 million	Uplift brings New Zealand geosyncline to an end	'Greywacke' terrain uplifted and exposed to erosion.
	Triassic	180 million	New Zealand geosyncline	New Zealand area a deep trough in which sediments now constituting the foundation of the North Island are deposited. These include compact greywacke, sandstone, argillite, conglomerates.
Palaeozoic	Permian	200 million		
	Carboniferous	250 million		

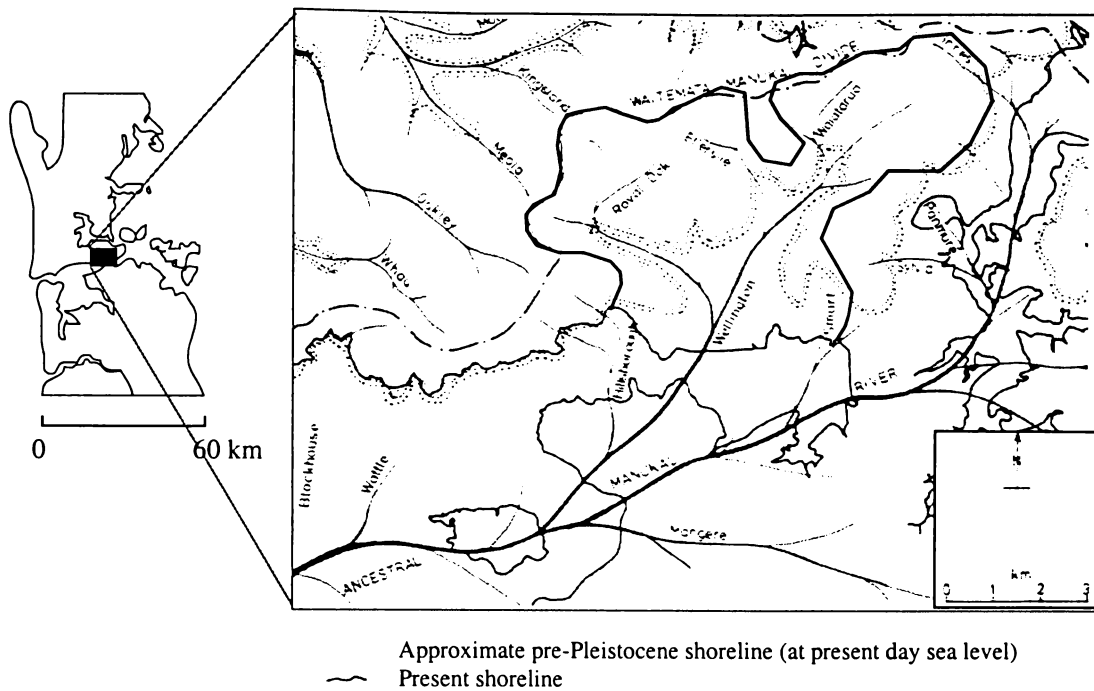


Figure 2.2. Possible drainage pattern of the Auckland urban area during the ice ages. The outlined area in the middle is the Onehunga – Mt Wellington catchment, adapted from Searle (1981, p. 40).

2.3.2 Onehunga – Mt Wellington Area

The Onehunga – Mt Wellington aquifer is made up of volcanic material erupted by the One Tree Hill, Mt Wellington, Mt Smart and Three Kings volcanoes (Figure 2.3). The volcanic centre of Three Kings is just outside the aquifer boundary but is included in this study because the tuff ejected during its eruptive phase covers a substantial section of the aquifer. Also groundwater dewatering at the volcanic centre of Three Kings may have an important impact of the neighbouring Onehunga – Mt Wellington aquifer. Of the four volcanoes, only Mt Wellington has been radiometrically dated with any confidence to 9160 ± 320 years before present (Kermode, 1992a). It is generally thought that Mt Smart erupted first, then One Tree Hill (~20,000 years ago), closely followed by Three Kings, and most recently Mt Wellington (Searle, 1981; Cox, 1989; Homer, Moore and Kermode, 2000).

The volcanic basalt field of the aquifer includes scoria cones at the volcanic centres, massive lava flows along the ancestral valleys of the Waitemata Group, and scoriaceous tuff and ash air-fall deposits that blanket the topography (Figure 2.3). The lava flows were often multiple events flowing beside or on top of previous flows. In some cases, ash, tuff or peat deposits were layered between successive lava flows. The outer edges of the lava flows are often scoriaceous, rubbly and locally cavernous, compared to the centre of the flows

where there are few vesicles, and the basalt is dominated by vertical to sub-vertical cooling joints (Searle, 1981).

Mt Smart erupted at the seaward end of the Mt Wellington valley (Figure 2.3). A 50 m high scoria cone and associated lava blocked the Mt Wellington drainage valley, resulting in the formation of peat areas. The extreme heat produced by subsequent lava flowing over the peat produced an impermeable, porcelain-like material up to 10 m thick. This is evident in drill-hole logs ARC456 and ARC463 (Appendix B) as an impermeable layer between the Waitemata sediments and the volcanic lava flows in the Mt Wellington Valley. During the mid 1900's, Mt Smart was extensively quarried and is now only 20 m above sea level (Searle, 1981).

The lava flows derived from One Tree Hill cover the greatest portion of the Onehunga – Mt Wellington aquifer, extending over approximately 18 km² (Figure 2.3). There is evidence that the volcanic activity at One Tree Hill extended over a long period of time. This is shown in drill-hole logs ARC745 and ARC405 near the Manukau Harbour where peat and tuff deposits are found between successive lava sheets (Carrier & Associates Limited, 1992). These drill-hole logs can be found in Appendix B. The basalt aquifer under the peat and tuff deposits is semi-confined. The One Tree Hill eruptive sequence resulted in lava flows in all directions, but the dominant direction was south due to the existing alignment of the Waitemata valleys (Figure 2.4). It is from these valley-confined lava flows that most of the groundwater users extract their water.

The Mt Wellington lava flowed down the Mt Wellington drainage valley towards the Manukau Harbour (Figure 2.4). Before reaching the harbour, the flow was blocked by the Mt Smart and One Tree Hill lava fields. The Mount Wellington lava dammed up behind these until it escaped down a narrow gap to the west of the Mt Smart field to the Manukau harbour (Figure 2.3).

Lava from Three Kings volcanic centre did not enter the Manukau valleys, but flowed in a westerly direction down the Waitemata valley towards Western Springs. Three Kings was a very explosive eruption and produced a large amount of tuff which mantles the western side of the One Tree Hill lava flows (Figure 2.3). Between the Three Kings quarry and One Tree Hill, the tuff is up to 30 m in thickness and can extend from the present land surface down to the pre-volcanic Waitemata surface (Figure 2.5).

Since the 1960's, the Manukau Harbour foreshore has been reclaimed, forcing the shoreline seawards by 700 m. This has also produced some semi confining of the aquifer and restricts groundwater flow from the Onehunga – Mt Wellington aquifer to the harbour.

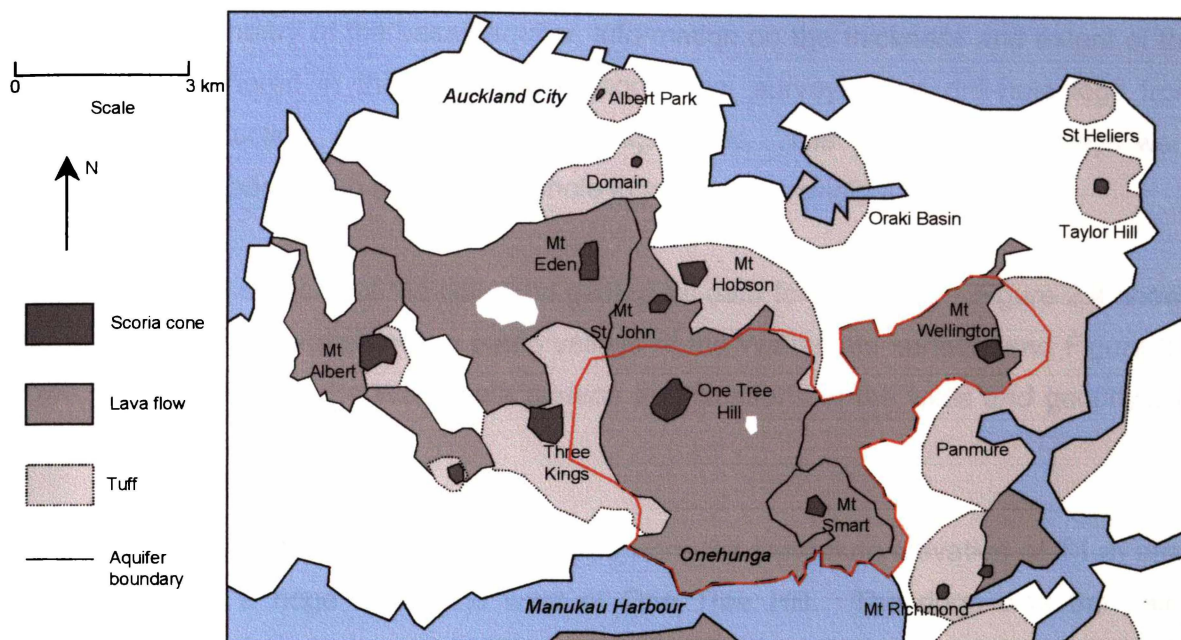


Figure 2.3. Map of the Auckland Isthmus volcanic field and Onehunga – Mt Wellington aquifer boundary. Adapted from Homer, Moore and Kermode (2000).

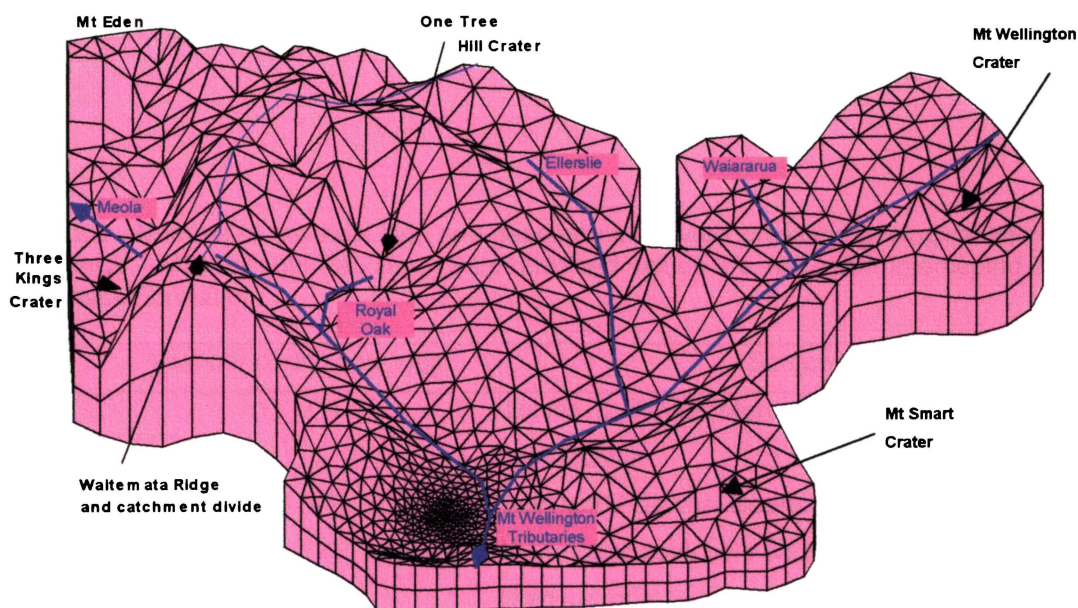


Figure 2.4. Inferred Waitemata topographic form and the aquifer boundary, based on geological information summarised in Table 2.2. Blue lines show tributaries of the Manukau River prior to the volcanic eruptions.

2.3.2.1 Aquifer Geometry

The basalt aquifer lies within the paleo Waitemata valleys. These valleys have a major role in controlling the direction of groundwater flow and define the aquifer's boundaries. To ascertain the geometry of the basalt aquifer, information on the thickness and extent of the geology was gathered in this study from geophysical surveys, and drill-hole logs from observation, production, and stormwater soakage wells. The geophysical surveys were checked and compared with any new drill-hole logs since the original survey.

Table 2.2 lists the sources of the collected geological data for this study. Figure 2.4 shows a three-dimensional diagram of the paleo valleys of the Waitemata surface, and Figure 2.6 shows the elevations of the Waitemata surface and locations of the wells and geophysical surveys listed in Table 2.2.

The underlying Waitemata Group rocks extend from the maximum elevation of 74 m amsl where they form a ridge north and west of One Tree Hill. The ridge was the paleo catchment boundary between the Waitemata and Manukau Rivers (Figure 2.2), and now divides the present Onehunga – Mt Wellington aquifer from the Western Springs aquifer. The manner in which the Waitemata valleys are infilled by the basalt lava flows is shown in Figure 2.5. The basalt in the vicinity of One Tree Hill is approximately 85 m thick, and in the Onehunga area 20 to 30 m thick. At the north end of Mt Wellington valley near the quarry the basalt is up to 40 m in thickness (Figure 2.7).

Table 2.2. Number of drill-hole logs and geophysical surveys providing geological information on the Onehunga – Mt Wellington aquifer. Parenthesised values are wells that intercepted the underlying Waitemata Formation.

Number of drill-hole logs	Source
11 (11)	Tonkin & Taylor Mercury Energy CBD power cable tunnel
156 (41)	Auckland Regional Council drill-hole logs
52 (12)	Metrowater soak hole data
19 (0)	Niederer private drilling company
4 geophysical surveys	M.P. Hochstein & D.C Lawton, J Cassidy, G.W. Roberts, and D. Affleck

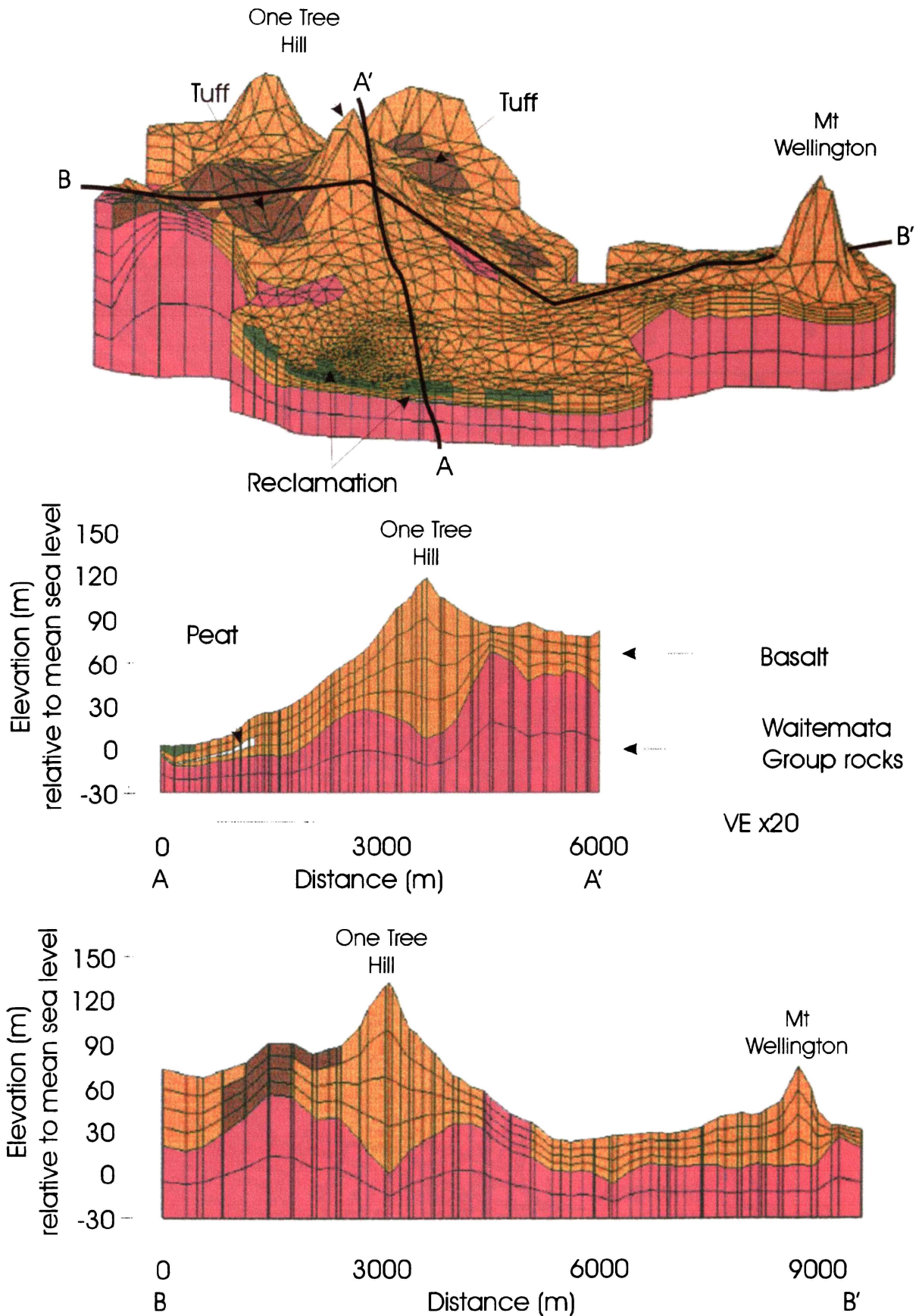


Figure 2.5. Three-dimensional view, and two cross-sections through the Onehunga – Mt Wellington aquifer showing the main geological groups.

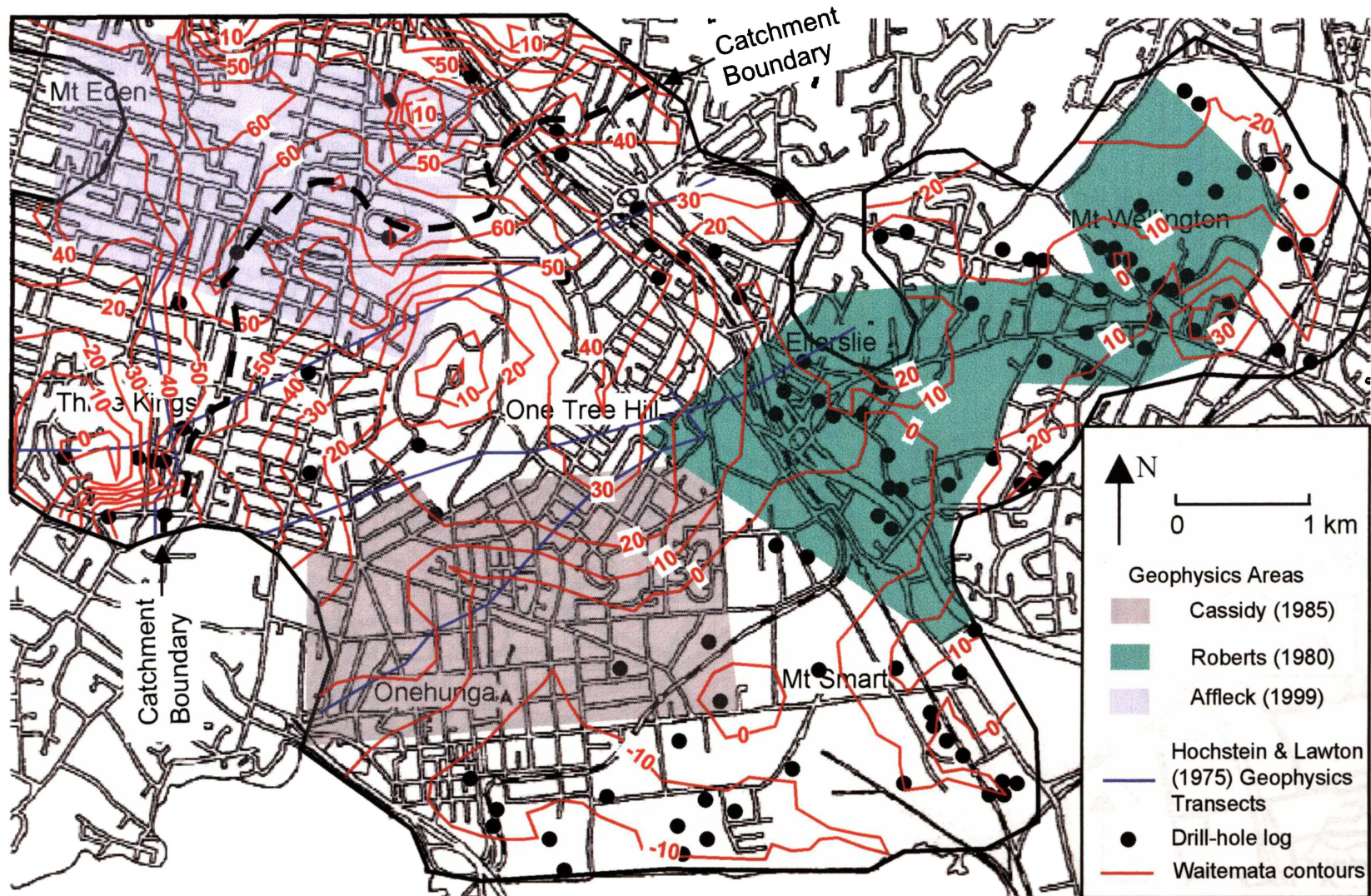


Figure 2.6. Contours of the Waitemata upper surface, and the location of the drill-hole logs and geophysical surveys used to define the contours (m).

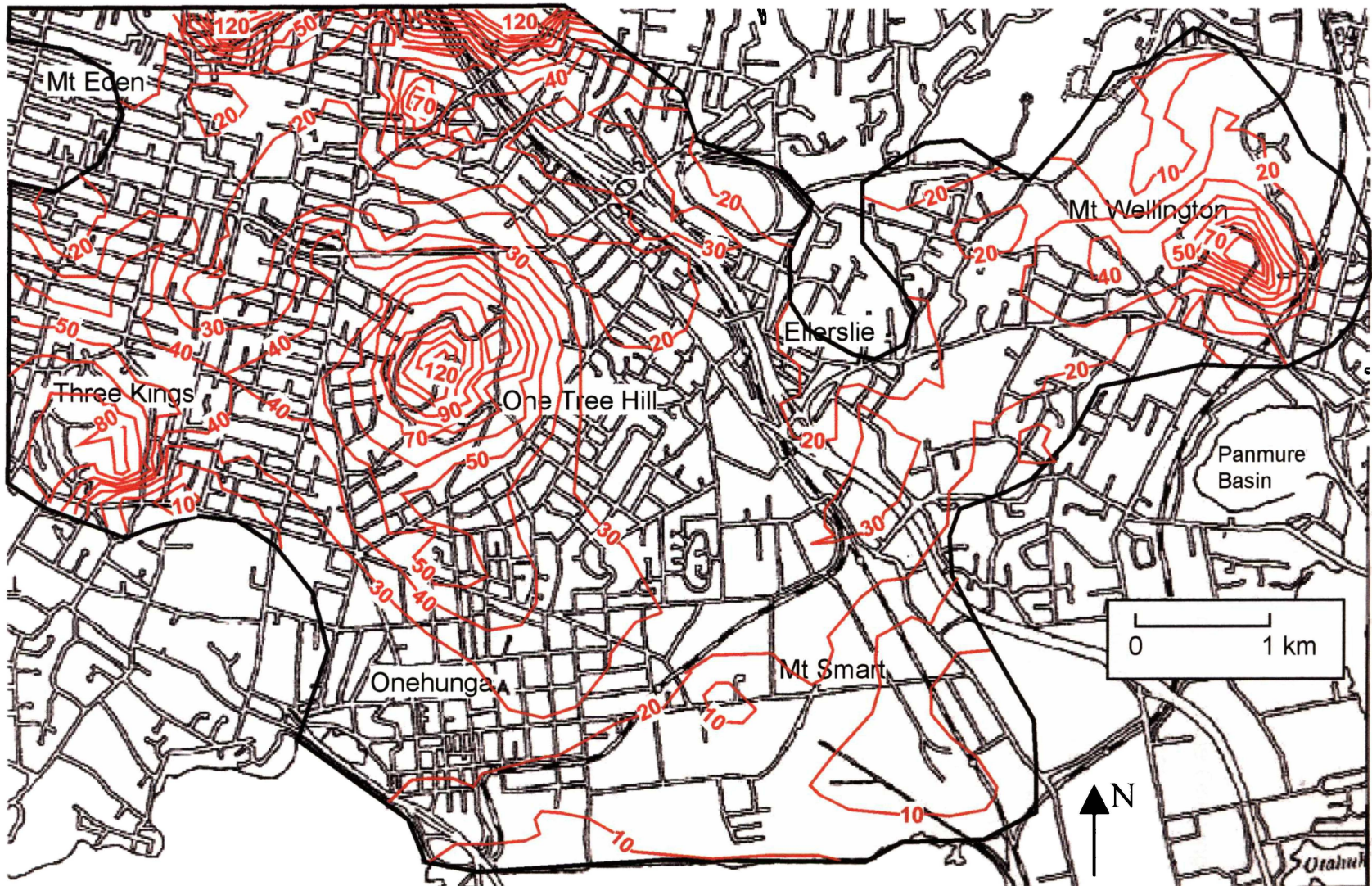


Figure 2.7. Volcanic aquifer thickness (10 meter contour interval).

2.4 Hydrogeology of the Onehunga – Mt Wellington Basalt Aquifer

2.4.1 Historical Municipal Groundwater Supply

Ground water was first extracted for potable reticulated supply from the Onehunga aquifer circa 1880, by the then Onehunga Borough Council. The supply was not connected to the Auckland City water supply as reservoirs in the Waitakere Ranges were the preferred choice. In the early 1900's arrangements were made for the adjoining borough councils to use the Onehunga supply. At one stage the Onehunga Borough supplied water to Mangere, Mount Roskill, Hillsborough, and Ellerslie (Auckland Regional Council, 1990). During the Second World War the Onehunga and Auckland City supplies were interconnected. In 1948 the Upper Municipal and Pearce Street wells were constructed, both are still in use today by Watercare Services Limited (Figure 2.8).

In 1950 the old Rowe Street well became polluted from seawater intrusion and all the boroughs, except Onehunga and Mangere were transferred to the Auckland supply. To overcome the salinity problem a new well was dug which is the present Rowe Street well. The old well is now operated by the industry Bumper Replacements (Figure 2.8). The risk posed by saline intrusion for the wells has been greatly reduced by the reclamation of the Manukau Harbour foreshore during the 1970's. This reclamation work produced a localised semi-confinement of the aquifer, in effect damming the aquifer exit and pushing the sea 700 m further away from the wells (Figure 2.8). Also, all Watercare's wells except the Lower Municipal well (drilled in 1975), have their extraction intakes above mean sea level (Auckland Regional Council, 1990; Smaill, 1993b).

The Rowe Street pumping station equipment was upgraded in 1975 after extensive fire damage. In 1981 a new filtration plant was opened with a maximum design output of 18,000 m³d⁻¹. In the early 1990's the Rowe Street site was again upgraded with a theoretical maximum output of 43,000 m³d⁻¹, and a consented maximum output of 30,000 m³d⁻¹ (Auckland Regional Council, 1990; Smaill, 1993b).

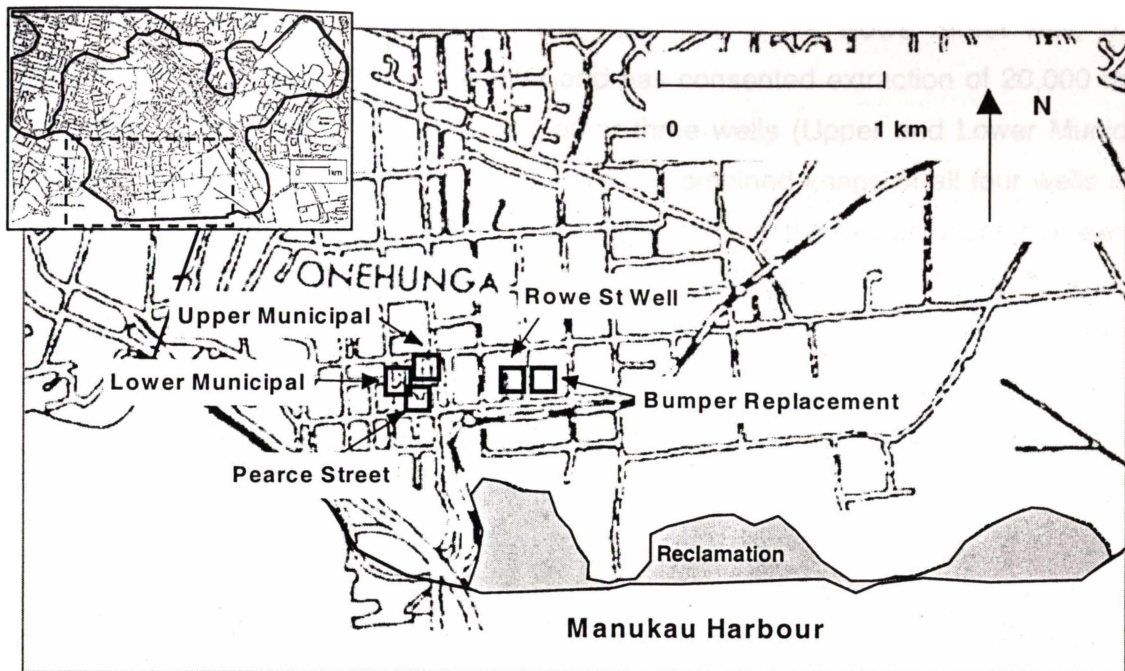


Figure 2.8. Production wells in the Onehunga area included in this study, and the foreshore area reclaimed during the 1970's (shaded area).

2.4.2 Present Groundwater Allocation and Use

The current aquifer management plan operated by the regulatory authority (Auckland Regional Council) divides the Onehunga – Mt Wellington aquifer into the eastern and western management zones, shown in Figure 2.1. The eastern zone is the Mt Smart – Mt Wellington management zone, with an estimated recharge of 20,300 m³d⁻¹ and sustainable extraction of 17,255 m³d⁻¹ (Smaill, 1993a). The area to the west is the Onehunga – One Tree Hill management zone, with an estimated recharge of 27,300 m³d⁻¹ and sustainable extraction of 23,200 m³d⁻¹ (Smaill, 1993b). Fifteen percent of the estimated recharge is not allocated as a protective measure to prevent saline intrusion of the lower aquifer area from the Manukau Harbour. The Auckland Regional Council uses these estimates of aquifer recharge when allocating groundwater to the various aquifer users.

a) Onehunga – One Tree Hill Management Zone

There are at least nine major groundwater users in the Onehunga – One Tree Hill management zone (Table 2.3). The single main user is Watercare Services Limited with consented use of 70% of the average annual recharge to the Onehunga – One Tree Hill management zone. This water is extracted from four wells in the Onehunga area supplying

2.5% of Auckland City's water supply. The main well is the Rowe Street well, which penetrates a cavernous zone of the aquifer and has consented extraction of 20,000 m³d⁻¹. The combined consented extraction of the other three wells (Upper and Lower Municipal and Pearce Street) must not exceed 16,700 m³d⁻¹. Combined usage of all four wells must not exceed 30,000 m³d⁻¹, and for 12 consecutive months the usage must not exceed 19,041 m³d⁻¹. Pumping records for the Onehunga Borough water supply have been recorded since 1927. However, until 1993 the records were for the total extraction from all the wells, and not the individual well use. From mid 1993 to the end of the year 2000 the average pumping rate of the Rowe Street well was 6,600 m³d⁻¹ and the Pearce Street well 2,250 m³d⁻¹. The Upper and Lower Municipal wells were used intermittently and had an average combined discharge of 750 m³d⁻¹.

Combined usage of the four wells only equates to 35% of the aquifer's recharge in the One Tree Hill – Onehunga management zone, or half of the consented allocation. This is due to a combination of low water levels during the summer and autumn months (January – June) limiting the pumping rate, and reduced water demand during winter and spring (July – December). The seasonal use of the wells is shown in Figure 2.9, where the Pearce Street well was only used during the summer months, and the Rowe Street well yield was more constant. However the Rowe Street well still had a slight decline in extraction during the winter months.

Many springs have been destroyed by urban development over the last 100 years. There are now only two springs remaining in the Onehunga area (Figure 2.10). Groundwater is extracted from the Captain Springs by Lichensteins for industrial use, and spring flow at the Rowe Street well provides water for the Bycroft wetland. The industries Southern Cross Leathers and Sutherlands also used groundwater from the Rowe Street spring until 2001 for industrial use. When Watercare's Rowe Street well's extraction rate exceeds 9,000 m³d⁻¹ the natural spring flow to the Bycroft wetland can be reduced. When the natural spring flow is reduced Watercare is required to divert groundwater from the Rowe Street well to the wetland to maintain a minimum residual flow of 0.35 L/s.

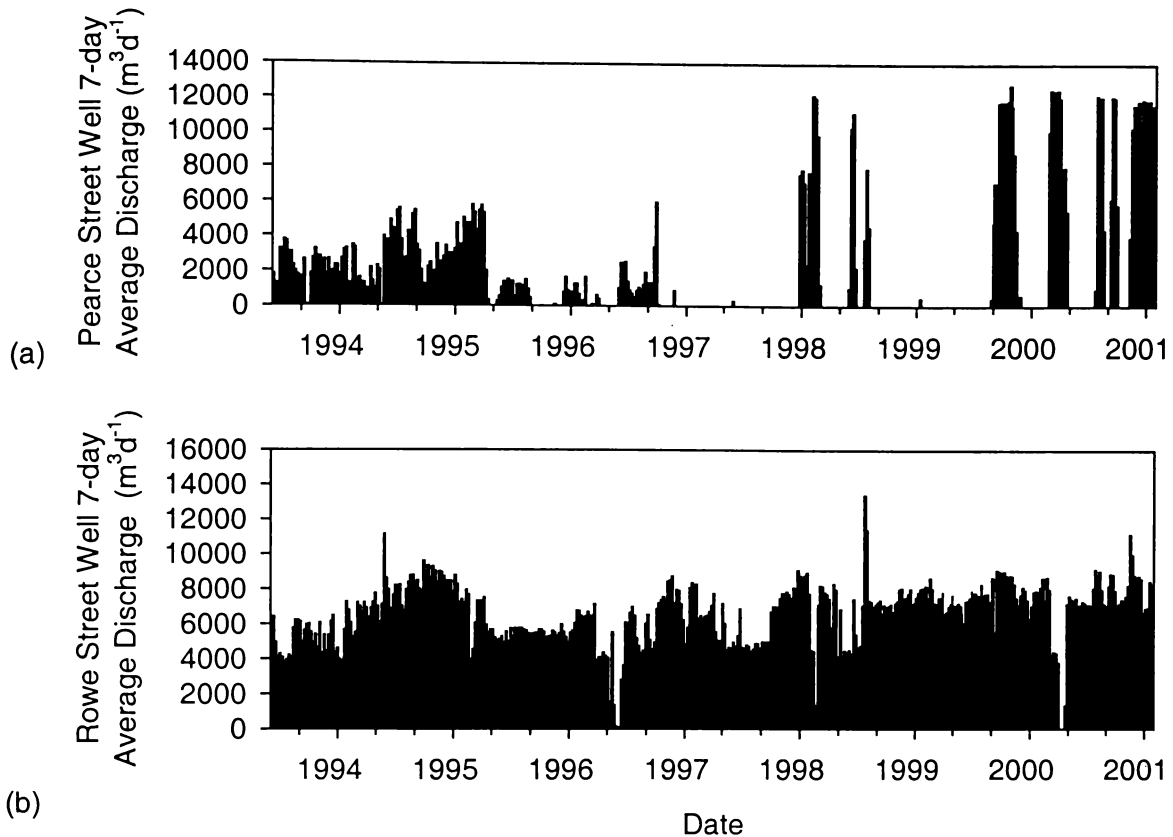


Figure 2.9. 7-day average discharges from Watercare's Pearce Street and Rowe Street wells.

b) Mt Smart – Mt Wellington Management Zone

The main groundwater user in the Mt Smart – Mt Wellington management zone is the Mt Wellington quarry, located near the head of the aquifer. This is one of New Zealand's largest quarries and the aquifer has been dewatered since 1946 to enable quarrying below the normal water table depth. The dewatering has resulted in the localised reversal of groundwater flow and produced a zone of groundwater divergence across the aquifer (Figure 2.10). South of the zone of divergence there are 12 users of the aquifer with consented extraction of 7,882 m³d⁻¹, 70% of the total recharge.

Spring discharge near the Mt Wellington quarry flows into the Panmure Basin groundwater system (Figure 2.10). The flow from this spring is channeled through underground drains. Spring discharge also occurs along the boundary of the basalt flows of the Mt Wellington valley and the low conductivity Waitemata Group rocks near the Remuera Golf Club (Figure 2.10).

Table 2.3. Average daily groundwater allocation by the ARC.

Name	Sub catchment	Consent usage m ³ d ⁻¹	Grid reference ¹	Status ²
Watercare	Lower Onehunga	43,000	2669800 6473700	issued
Horizon Yarns	Lower Onehunga	160	2671100 6473500	issued
Bumper Replacements	Lower Onehunga	50	2670000 6473800	issued
Auckland Anodisers ³	Lower Onehunga	210	2670900 6473900	issued
Screencraft	Lower Onehunga	400	2669600 6473400	issued
Lichensteins	Lower Onehunga	1,120	2670400 6473500	expired
Southern Cross Leathers	Lower Onehunga	450	2669800 6473550	expired
Sutherland & Co.	Lower Onehunga	550	2669800 6473550	expired
Greenlane Hospital	One Tree Hill	600	2669100 6476900	issued
Sanitarium	One Tree Hill	100	2668200 6475000	issued
Mt Smart Stadium	Mt Smart	90	2671900 6474500	issued
Winstone Wallboards	Mt Smart	400	2671100 6474600	issued
Tasman Insulation	Mt Smart	120	2672700 6475000	issued
Auckland Abattoir	Mt Smart	1,450	2674100 6472700	issued
Carter Holt Harvey	Mt Smart	1,650	2672400 6473700	issued
South Park	Mt Smart	1,652	2673100 6473600	issued
Fletcher Wood Panels	Mt Smart	1440	2671900 6474700	issued
ACI Glass	Mt Wellington	500	2672210 6475540	issued
Marist Brothers Football Club	Mt Wellington	70	2672400 6477600	issued
Wilson & Horton	Mt Wellington	100	2672200 6476200	issued
Alexandra Park	Mt Wellington	210	2669100 6477300	issued
Fulton Hogan	Mt Wellington	200	2672900 6476000	issued
Winstone Aggregates Limited	Mt Wellington	4,200	2673900 6477600	issued
	N/E of divide			

¹ Grid reference for the NZMS 260 map series sheet R11.

² Status of consent for groundwater extraction. Issued status allows user to extract groundwater up to the consent amount. Expired users no longer extract groundwater.

³ Auckland Anodisers also known as Yuassa Batteries

2.4.3 Groundwater Flow Pathways

The dominant groundwater flow direction is from north to south along the paleo drainage valleys of the Waitemata group (Figure 2.10). However, there are two exceptions to this: firstly, near the Manukau Harbour where the aquifer base is below sea level, the preferential flow path is through zones of higher conductivity in the basalt aquifer. This is apparent in the vicinity of Mt Smart where the preferred groundwater flow path is around, rather than through, the lower hydraulic conductivity lava from the Mt Smart eruptions (Figure 2.10). Secondly, a reversal of groundwater flow occurs near the Mt Wellington quarry due to aquifer dewatering operations (Figure 2.10).

2.4.4 Groundwater Levels

Groundwater levels in the Onehunga area of the basalt aquifer have been recorded spasmodically since 1972. A detailed monitoring program by the Onehunga Borough Council occurred from 1972 to 1975. After this there was no monitoring in the Onehunga area until the late 1980's. The most complete set of data is from 1993 to 2000. Monitoring in the Mt Wellington area has occurred intermittently since 1977.

There are 30 water level monitoring sites operated by the Auckland Regional Council in the Onehunga – Mt Wellington aquifer and headwaters of the Western Springs aquifer, their locations are shown in Figure 2.11. Of the 30 sites, five are automatically recorded, the four Watercare wells' water levels are recorded at least once a week, and the remaining 21 manual sites are measured at least once a month. Seasonal fluctuation patterns of the water levels are similar over the whole aquifer with a summer low and winter high. At the lower end of the aquifer near the Manukau Harbour, fluctuations are typically less than 1 m, compared to further up the aquifer catchment towards One Tree Hill where the fluctuations are approximately 5 m (Figure 2.12)

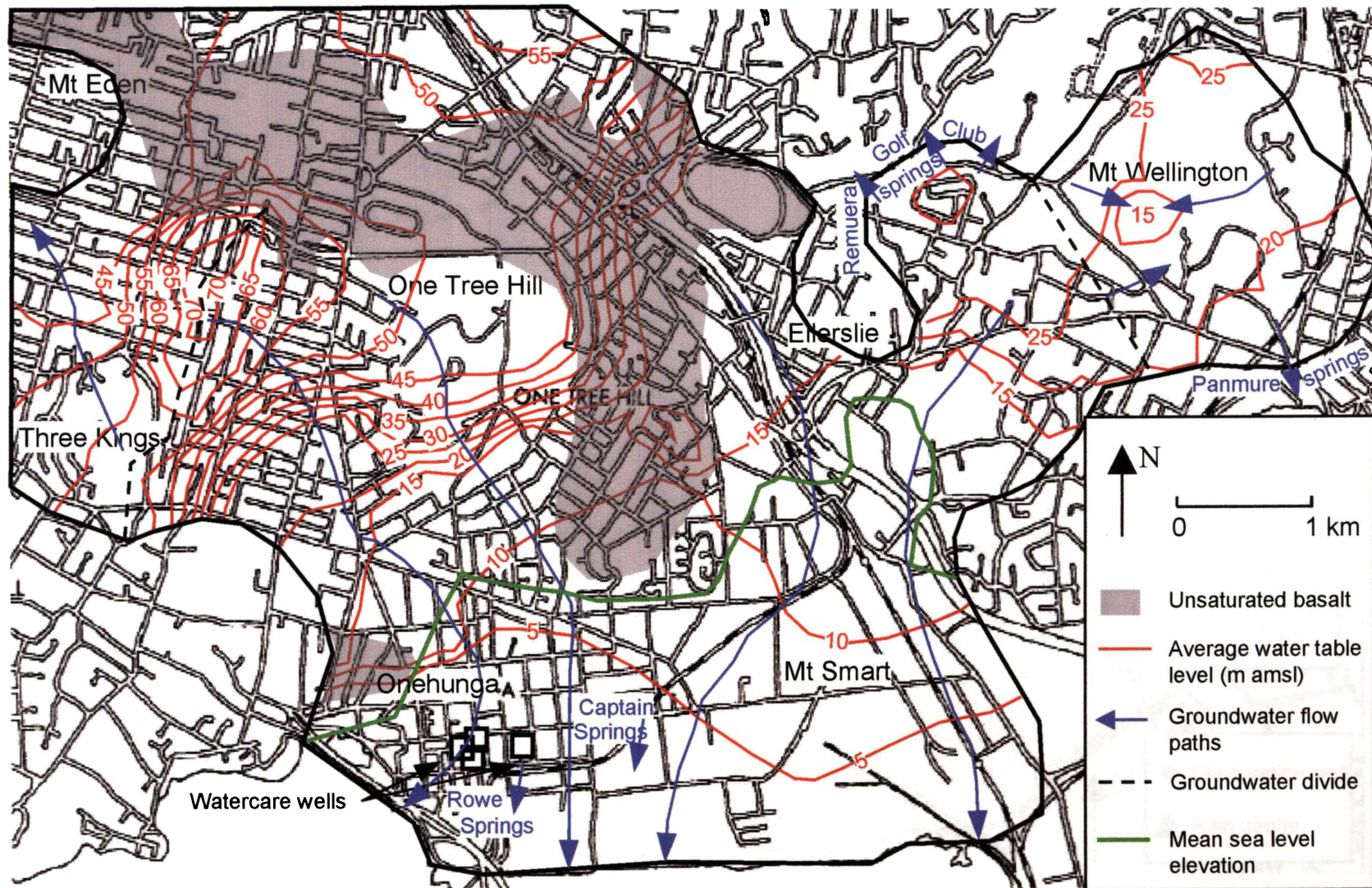


Figure 2.10. General flow directions and zones of unsaturated basalt for the Onehunga – Mt Wellington aquifer.

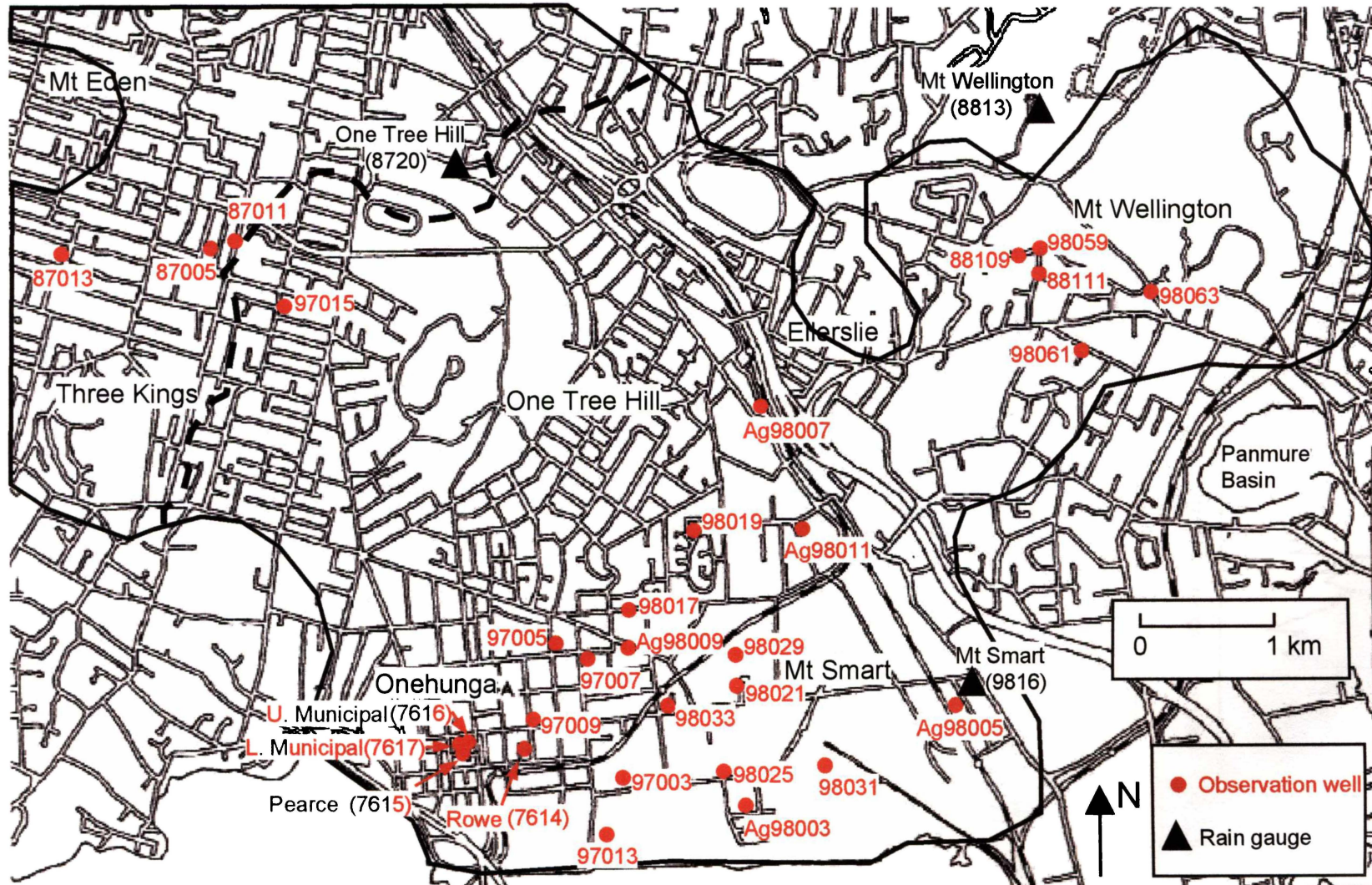


Figure 2.11. Location of monitoring points for groundwater levels and rainfall. Groundwater levels monitoring wells with the prefix 'Ag' are monitored automatically by the Auckland Regional Council (ARC). Identification numbers are those utilised by the ARC.

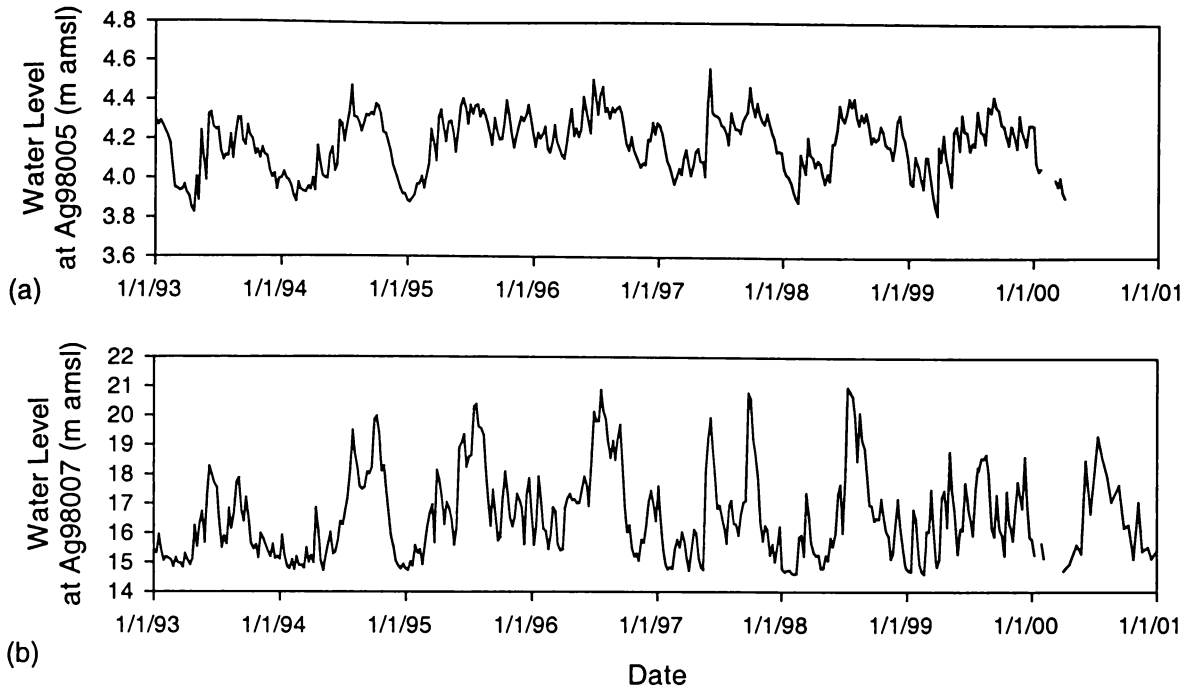
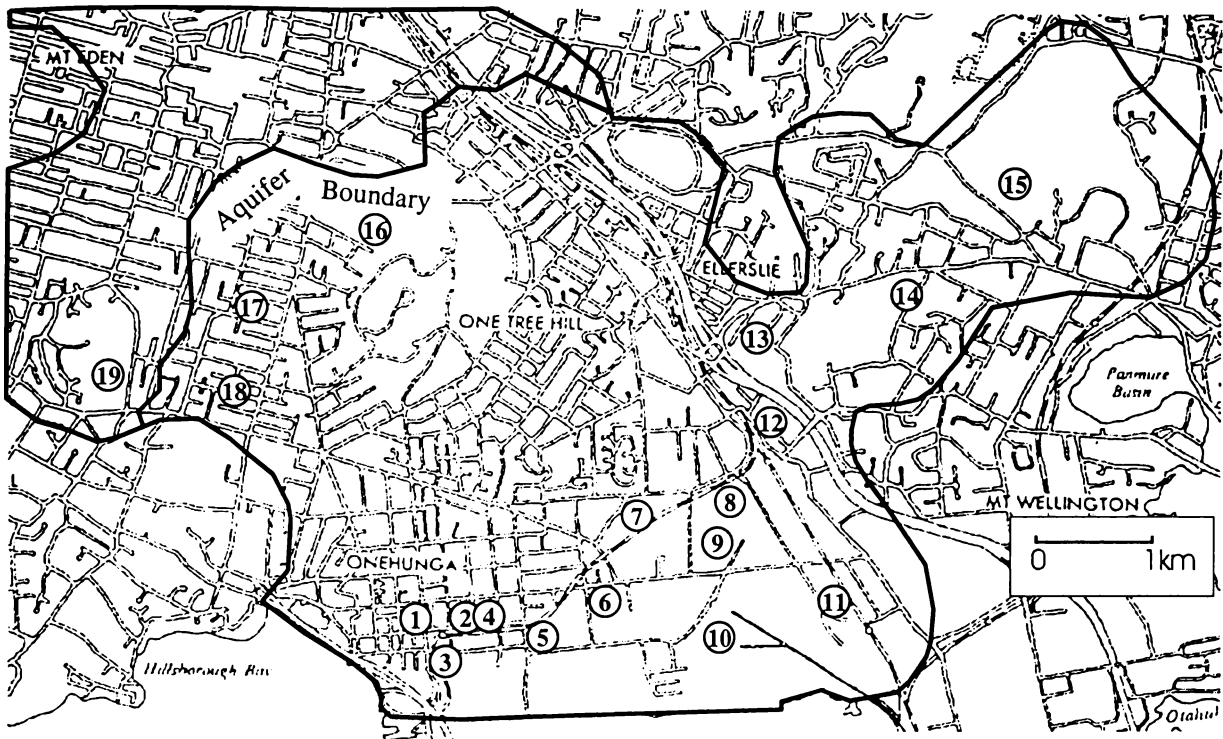


Figure 2.12. Observation well water levels recorded near the Manukau Harbour (Ag98005) and further up the catchment towards One Tree Hill (Ag98007).

2.4.5 Hydraulic Parameters

Hydraulic properties vary greatly over very short distances due to the anisotropic and heterogeneous nature of the fractured basalt aquifer. The estimated average hydraulic conductivity for the basalt aquifer between Onehunga and Mt Smart is 320 md^{-1} , Mt Smart 36 md^{-1} and Mt Wellington 80 md^{-1} , compared to the Waitemata Group rocks 0.07 md^{-1} (Smaill, 1993b; Namjou, 1996). No information is available on the vertical conductivity for the basalt aquifer or the underlying Waitemata Group rocks. The ratio of horizontal to vertical hydraulic conductivity will increase where increased stratification of the basalt occurs due to layers of low conductivity tuff and peat. For a fractured aquifer with both a low and high degree of stratification, the ratio of horizontal to vertical conductivity can range between 1/2 and 1/100, respectively (Walton, 1991). Namjou (1996) achieved calibration of a numerical model of the Mt Wellington quarry area with a ratio of 1/10 for the basalt aquifer, and between 1/10 to 1/100 for the Waitemata Group rocks. Figure 2.13 shows the locations and values of the measured hydraulic parameters for the Onehunga – Mt Wellington aquifer.



- | | |
|--------------------------------|---|
| 1. Lower Municipal (Watercare) | T = 6176 m ² d ⁻¹ (Pattle Delamore Partners Limited, 1994) |
| 2. Rowe Street (Watercare) | T = 33,000 to 58,000 m ² d ⁻¹ (Pattle Delamore Partners Limited, 1994) |
| 3. Screencraft Ltd. | T 108 m ² d ⁻¹ , S = 0.00001 (Carrier & Associates Limited, 1991) |
| 4. Bumper Replacements | T = 5745 m ² d ⁻¹ (Pattle Delamore Partners Limited, 1994) |
| 5. Lichenstein | T = 260 m ² d ⁻¹ , Sy = 0.2 (Carrier & Associates Limited, 1992) |
| 6. Auckland Anodisers | T = 18.2 m ² d ⁻¹ , S = 0.0029 (Carrier & Associates Limited, 1992) |
| 7. Winstone Wallboards | T = 665 m ² d ⁻¹ , K = 37 md ⁻¹ , Sy = 0.11 (Carrier & Associates Limited, 1992) |
| 8. Fletcher Wood Panels | T = 301 - 723 m ² d ⁻¹ , Sy = 0.04 - 0.16 (Babbage Consultants, 1995) |
| 9. Mt Smart Stadium | T = 1637 m ² d ⁻¹ (Auckland Regional Council, 1996) |
| 10. Rapa casings | T = 15 m ² d ⁻¹ (Carrier & Associates Limited, 1992) |
| 11. Industrial Park Holdings | T = 46 - 748 m ² d ⁻¹ (Pattle Delamore Partners Limited, 1988) |
| 12. ACI Glass | T = 75 m ² d ⁻¹ , K = 4.5 md ⁻¹ (Auckland Regional Council, 1995) |
| 13. Wilson & Horton | T = 600 m ² d ⁻¹ , K = 37 md ⁻¹ (Auckland Regional Council, 1994a) |
| 14. Mt Wellington | T = 1340 m ² d ⁻¹ , K = 86 md ⁻¹ (Roberts, 1980) |
| 15. Mt Wellington Quarry | K = 0.2 to 86 md ⁻¹ (Namjou, 1996) |
| 16. Greenlane Hospital | T = 410 m ² d ⁻¹ , K = 12 md ⁻¹ (OPUS, 1998) |
| 17. Orakau Ave (ARC97015) | K = 9.7 md ⁻¹ (Viljevac, 1998) |
| 18. Epworth Ave (OBC18) | K = 1 md ⁻¹ (OBC 18 drill-hole log – Appendix B) |
| 19. Three Kings Quarry | T = 4200 m ² d ⁻¹ , Sy = 0.15 (Groundsearch EES Limited, 1994) |

T = Transmissivity S = storativity Sy = Specific yield K = Conductivity

Figure 2.13. Measured hydraulic parameters and site locations.

The area of the aquifer utilised by Watercare Services Limited for their wellfield is within a fractured and cavernous zone at the lower end of the aquifer. The conductivities in this narrow zone are between 5 and 40 times greater than the surrounding area. Conductivity values in the cavernous zone have been estimated at 475 md^{-1} for the Lower Municipal well and 3,500 md^{-1} for the Rowe Street well (Pattle Delamore Partners Limited, 1994).

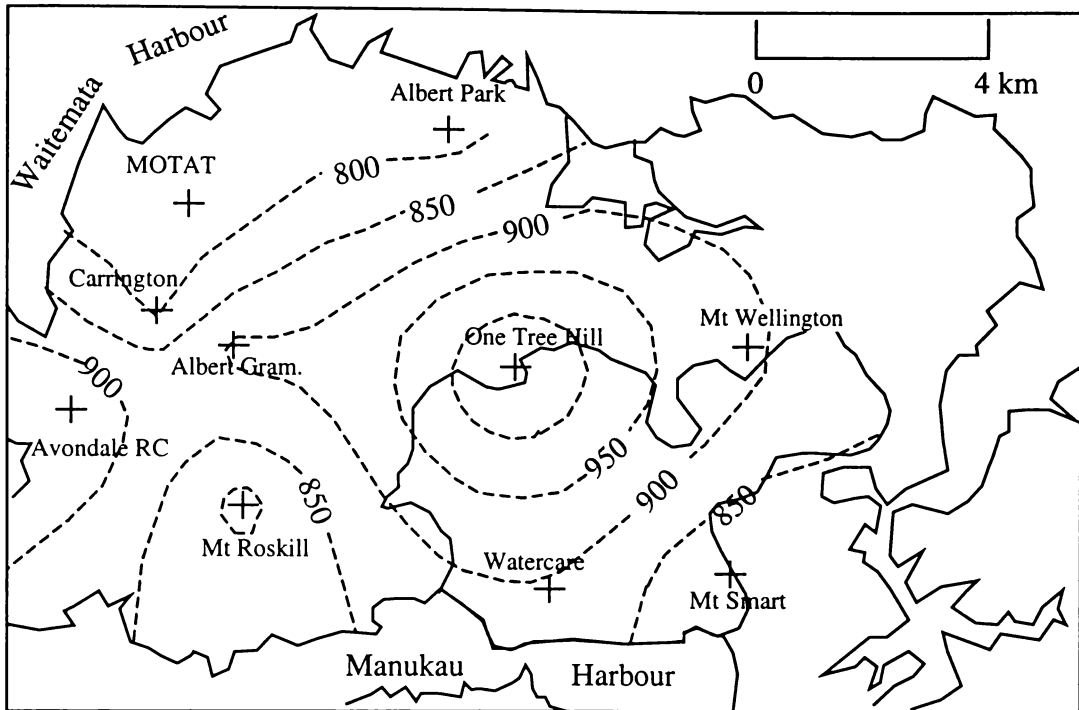
Specific yield values for the unconfined areas of the basalt aquifer are in the typical range of 0.04 to 0.2, and the storativity for the semi-confined areas range from 0.00001 to 0.0029 (Figure 2.13).

Porosity values in the Mt Wellington basalt flow range from 0.16 for granular basalt, to 0.02 for dense basalt with an average value of 0.07 (Roberts 1980). These values are towards the lower end of the porosity range (0.05 to 0.5) for fractured basalt (Freeze and Cherry, 1979 p.37). The porosity in the cavernous and scoriaceous zones will be considerably higher.

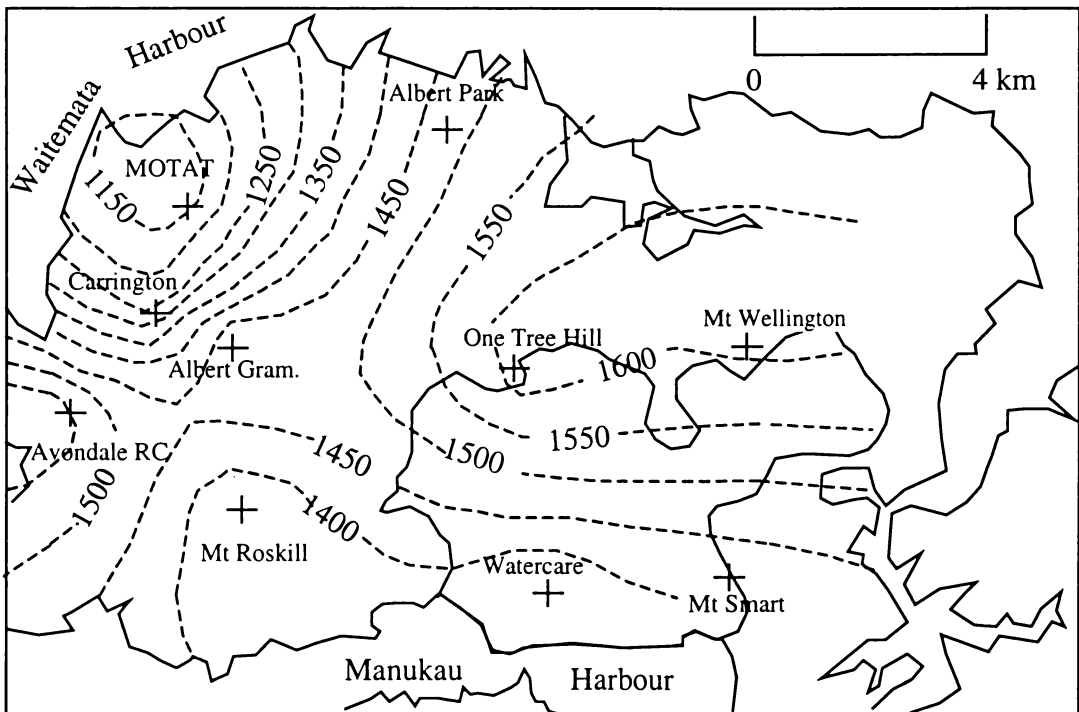
2.4.6 Rainfall and Evaporation

a) Rainfall

The rainfall spatial distribution for the Auckland isthmus is influenced by topography, with the greatest annual rainfall depth during both dry and wet years occurring at the higher topographic elevations near the middle of the isthmus (Figure 2.14). Typically the annual rainfall at the head of the aquifer near One Tree Hill is 1362 mm, 176 mm per year more than near the lower end of the aquifer at Mt Smart, 1186 mm (Table 2.4). The average annual rainfall for the Onehunga – Mt Wellington aquifer recorded by rain gauges at One Tree Hill, Mt Smart and Mt Wellington was 1294 mm for the years 1993 to 2000. Figure 2.15 shows that rain has fallen on the Onehunga – Mt Wellington aquifer during most weeks between 1993 and 2000. The seasonal pattern of the average rainfall for the three rain gauges shows the greatest rainfall during winter with a reduction during summer. However, large rainfall events do occur semi-regularly throughout the year.



a) 1993 annual rainfall isohyets in mm (dry year).



b) 1995 annual rainfall isohyets in mm (wet year).

Figure 2.14. Isohyets for the Auckland isthmus.

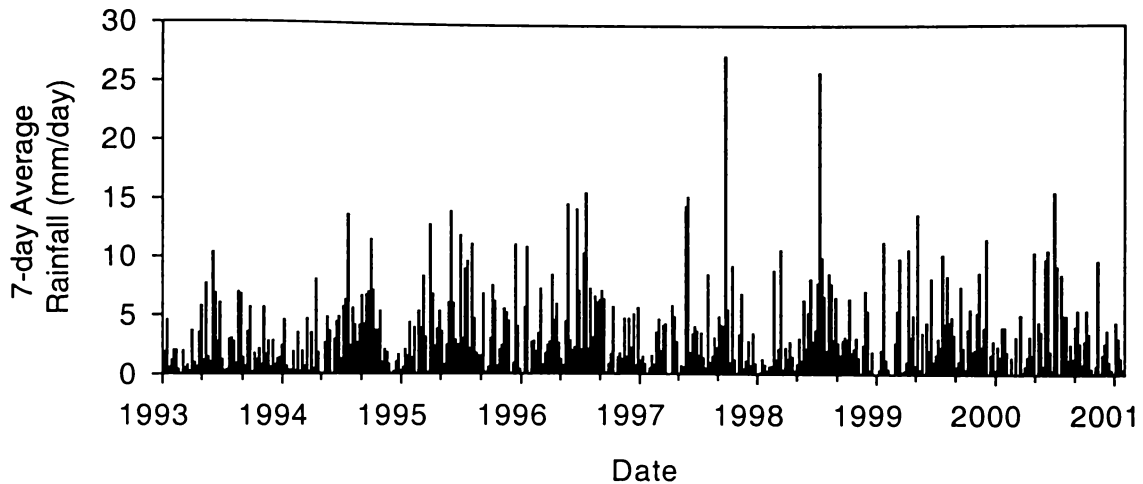


Figure 2.15. Seven-day average rainfall for the Mt Smart, Mt Wellington and One Tree Hill rain gauges.

Table 2.4. Annual rainfall totals from the Mt Smart, Mt Wellington and One Tree Hill rain gauges.

Elevation	Mt Smart 15 m	Mt Wellington 40 m	One Tree Hill 80 m
Year	Rainfall (mm)		
1993	797	909	1064
1994	1069	1228	1223
1995	1408	1614	1630
1996	1523	1448	1726
1997	1110	1428	1301
1998	1248	1344	1438
1999	1223	1486	1338
2000	1106	1228	1174
Average 1993-2000	1186	1336	1362

b) Evaporation

The potential evapotranspiration rate for the Auckland isthmus is estimated by the New Zealand Meteorological Service as 1150 mm/year. The actual evaporation for the basalt aquifer will be less due to the thin layer of well-drained soils limiting the availability of moisture for the plants, coupled with rapid downward recharge. The actual evapotranspiration of the vegetated areas has often been estimated to be approximately 75% of the potential evaporation (Pattle Delamore Partners Limited, 1991; Smaill, 1993a; Smaill, 1993b; Watson, 1995; Namjou, 1996; Viljevac, 1998). For the potential evaporation of 1150 mm/year the average actual evapotranspiration is estimated to be 865 mm/year for the Auckland area.

2.4.7 Recharge Mechanisms

Recharge to the Onehunga – Mt Wellington aquifer is via natural infiltration at the ground surface, and induced recharge where street and building runoff is diverted to soakage wells. An average infiltration rate of 15 L/s (or 1300 m³d⁻¹) was measured by the Auckland City Council for 43 existing stormwater soakage wells drilled in fractured basalt, cavities, and scoria in the Onehunga – Mt Wellington aquifer. Soakage wells intercepting solid basalt have infiltration rates up to 260 m³d⁻¹. The location of each soakage well test is detailed in Appendix C. The infiltration tests were typically 30 minutes in duration and the wells are generally 100 to 150 mm in diameter with an average depth of 13 m.

The response of groundwater levels to recharge events in the Onehunga – Mt Wellington aquifer is typically within 24 hours due to the high infiltration capacity of the fractured basalt. This quick response of the groundwater levels to rainfall events is shown in Figure 2.16 for observation well Ag98009. At this location in the aquifer, there is 15 m of unsaturated rock between the ground surface and the mean water table elevation.

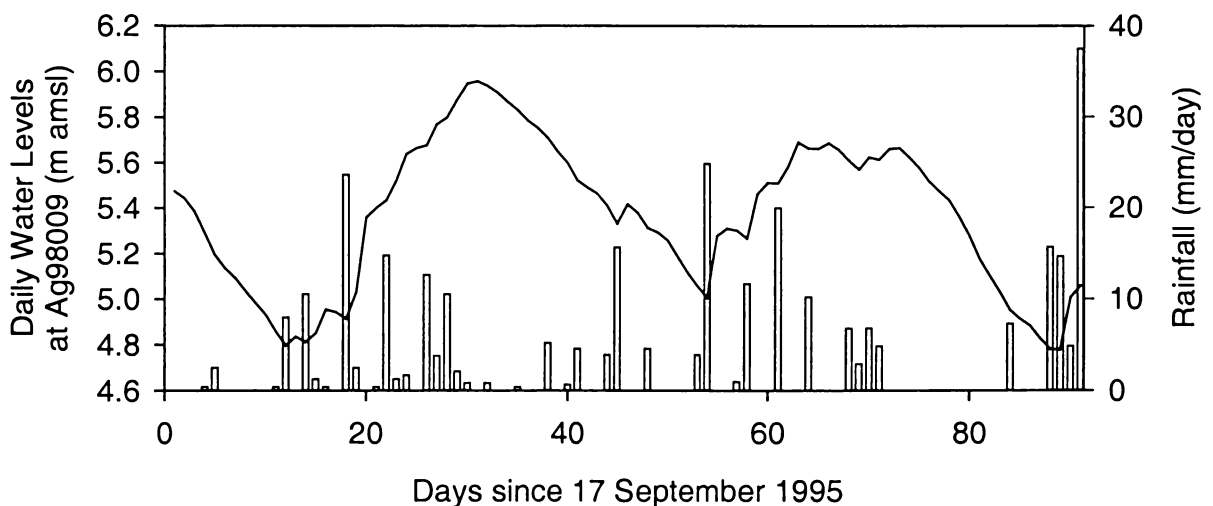


Figure 2.16. Groundwater level response to recharge events.

2.4.7.1 Recharge Estimation Methodology

For the present study, the recharge for Auckland's basalt aquifers was estimated as the fraction of rainfall that was not lost to runoff and evapotranspiration for the various drainage types and land uses of the aquifers. This method was initially used on Auckland's volcanic aquifers by Pattle Delamore Partners Limited (1991) and Viljevac (1998).

Pattle Delamore Partners Limited (1991) identified three categories of drainage type for Auckland’s basalt aquifers. These were; (1) soakage, (2) partial-soakage, and (3) sewered areas. Soakage areas relate to those where permeable basalts are near the ground surface and all runoff enters the groundwater system via soakage wells. Partial soakage is due to impermeable near-surface rocks. In these areas the only recharge is generally from residential property soakage holes. In sewered areas nearly zero recharge occurs, with a large portion of the runoff being piped from the catchment.

Pattle Delamore Partners Limited (1991) and Viljevac (1998) identified four categories of land use for Auckland’s basalt aquifers. These were parks, quarries, residential, and industrial areas. Based on measurements of runoff from rainfall events by the Auckland City Council, runoff factors for the different land uses and drainage types are listed in Table 2.5.

Table 2.5. Runoff as a percentage of the rainfall for the Auckland basalt aquifers based on runoff measurements by the Auckland City Council

Land Use	Drainage Type		
	Soakage	Partial soakage	Sewered
Parks	0	20	20
Residential	0	30	50
Industrial	0	90	90
Quarry	0	-	-

The actual evapotranspiration of the vegetated areas was estimated to be 865 mm/year for the Auckland area, as discussed in section 2.4.6. The area of vegetation in the different land types was estimated by Pattle Delamore Partners Limited (1991) to be; 95% for parks, 50% from residential, and 10% industrial/commercial. Parks were estimated to produce 66% of the rainfall to evapotranspiration, quarries 57%, residential areas 35%, and industrial areas 7%.

2.4.7.2 Estimate of Onehunga – Mt Wellington Recharge

In this thesis, both soakage and sewered drainage types were identified in the Onehunga – Mt Wellington aquifer. Four categories of land use were also identified in the Onehunga – Mt Wellington aquifer. These were; parks, quarries, residential, and commercial areas. Overlaying the spatial distribution of the three categories of drainage type and the four

categories of land use for the Onehunga – Mt Wellington aquifer produced five recharge zones. The five recharge zones were; (1) parks with soakage, (2) residential properties with soakage, (3) quarries with soakage, (4) sewerred residential properties, and (5) sewerred commercial areas.

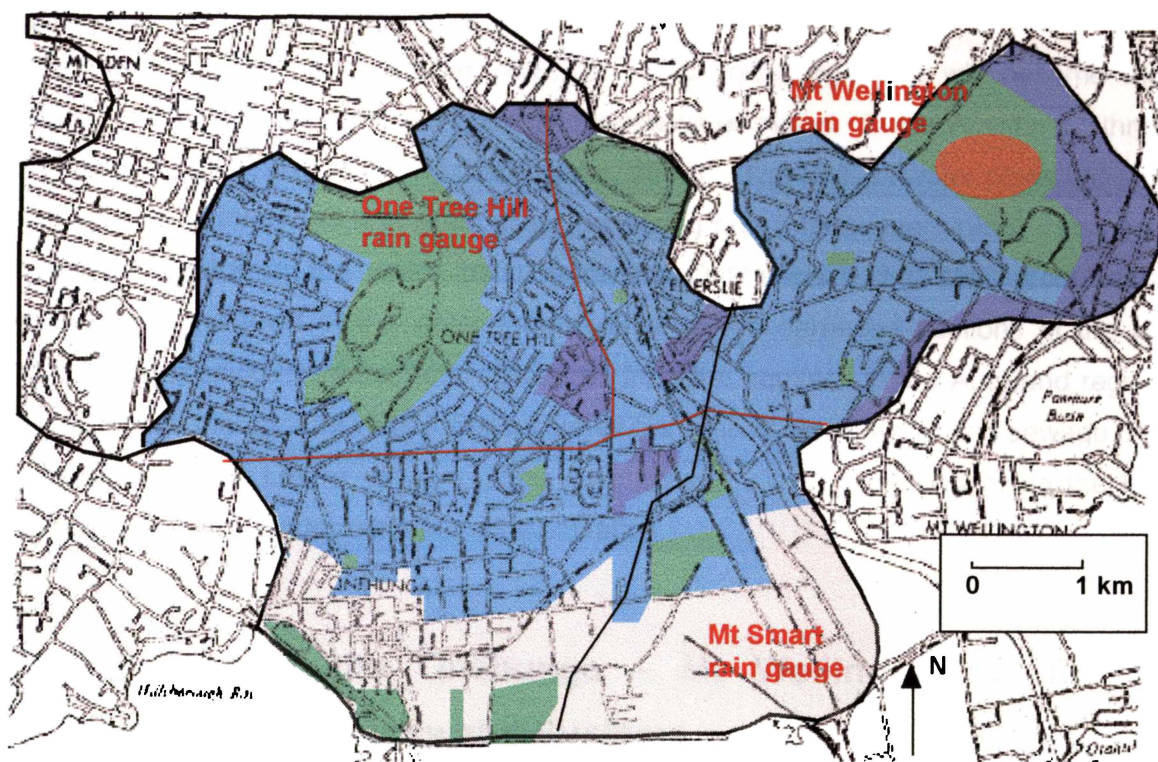
The actual evapotranspiration was 865 mm/year as discussed in section 2.4.6.

To refine the recharge estimation, the Onehunga – Mt Wellington aquifer was divided into three zones using the rainfall of the nearest rain gauge. The three rain gauges were at One Tree Hill, Mt Wellington, and Mt Smart. Combining the three rainfall zones and the five recharge zones the aquifer was divided into a further 12 zones (Figure 2.17).

A summary of the recharge for the 12 zones used in this study is given in Table 2.6 for both a dry and wet year, 1993 and 1995 respectively, and also for the average rainfall from 1993 to 2000. The estimated recharge for 1993 was 34,479 m³d⁻¹, 1995 was 57,836 m³d⁻¹, and 1993-2000 average was 48,257 m³d⁻¹. The calculations of recharge, evaporation, and runoff for the 12 zones used in this study are in Appendix D.

The Onehunga – Mt Wellington aquifer recharge estimated in this study is similar to previous estimates using the same methodology (Pattle Delamore Partners Limited, 1991; Smail, 1993b). Pattle Delamore, with uniform rainfall of 1.2 m/year, calculated an aquifer recharge of 45,673 m³d⁻¹; and the Auckland Regional Council applying uniform rainfall of 1.29 m/year calculated 47,936 m³d⁻¹.

The recharge analysis highlights the importance of the various drainage types on the infiltration rate of rainfall into the aquifer. The recharge to the Onehunga – Mt Wellington aquifer could be significantly reduced if further areas of the aquifer were connected to the sewer pipe network. Using the same recharge methodology as above, but converting all soakage areas to sewerred, except for the parks and quarry, the recharge in 1993 is estimated to be 12,342 m³d⁻¹, 1995 is estimated to be 20,694 m³d⁻¹, and 1993-2000 average is estimated to be 17,372 m³d⁻¹, a reduction in recharge of 64%.



Onehunga - One Tree Hill Management Zone ← | → Mt Smart - Mt Wellington Management Zone






	Quarry - evaporation 57%, recharge 43%,
	Commercial area - rainfall removed via sewer pipes 90%, evaporation 7%, recharge 3%
	Residential area - rainfall removed via sewer pipes 50%, evaporation 35%, recharge 15%
	Residential area - evaporation 35%, recharge 65%
	Parks - evaporation 34%, recharge 66%

Figure 2.17. Aquifer drainage and land use types for estimating aquifer recharge. Red lines divide catchment by nearest rain gauge. Black line divides aquifer by the two management zones.

Table 2.6. Summary of the aquifer recharge based on rainfall from three sites and the aquifer's drainage and land use types shown in Figure 2.17.

Rainfall	1993	1995	Average 1993 to 2000
Mt Smart	797	1408	1060
One Tree Hill	1064	1630	1362
Mt Wellington	909	1614	1336
Recharge	m³d⁻¹ (mmd⁻¹)	m³d⁻¹ (mmd⁻¹)	m³d⁻¹ (mmd⁻¹)
Onehunga - One Tree Hill Zone	23,356 (1.266)	38,102 (2.067)	31,874 (1.728)
Mt Smart - Mt Wellington Zone	11,123 (0.912)	19,733 (1.619)	16,382 (1.343)
Total recharge (m³d⁻¹)	34,479 (1.126)	57,836 (1.888)	48,257 (1.575)

Another potential source of aquifer recharge and/or discharge that will become more important in time is from damaged water and wastewater mains. In the Auckland isthmus, 53% of all the pipes need replacing (Metrowater, 1998). Over 25% of water pipes are made of inferior fibre cement and are failing at an increasing rate (Metrowater, 1998). This could add another factor to the water balance of the Onehunga – Mt Wellington aquifer. Depending on the level of the water table relative to the damaged sections of the pipe networks, there could be recharge or discharge from the aquifer. In the Auckland region it has been found that groundwater infiltration rates per area of storm water and sewage pipes can be up to 0.686 L/hr/m² (Brown, 2000 *pers comm*). This is dependent on the head gradient between the water table and pipe elevation.

2.5 Suitability of the Onehunga – Mt Wellington Aquifer for Constructed Groundwater Systems

The Onehunga – One tree Hill management zone is well suited for artificial aquifer recharge due to the high infiltration capacity of the fractured basalt aquifer and the large thickness of basalt in the vicinity of One Tree Hill (Figure 2.5 and Figure 2.7). In the vicinity of One Tree Hill there is approximately 40 m of unsaturated basalt rock above the average water table level. The average infiltration rate of soakage wells in the fractures and scoriaceous rock of the Onehunga – Mt Wellington aquifer is 1,300 m³d⁻¹.

The Mt Smart – Mt Wellington management zone is not as suitable for artificial aquifer recharge. Any water recharged in the northern end of the Mt Wellington area will most likely be extracted at the Mt Wellington quarry. The recharge may also cause flooding at the quarry, and also there is limited seasonal aquifer storage capability due to the expected short residence time of the groundwater in the quarry area. Any water recharged into the Mt Wellington zone south of the groundwater divide will not flow naturally to the existing Watercare wells and would require new extraction wells. Also, the aquifer is only 20 to 30 m thick with 5 to 15 m of unsaturated aquifer above the water table. The Mt Smart – Mt Wellington zone also has a lower hydraulic conductivity (average of 85 md⁻¹) than for the Onehunga – One Tree Hill zone (average of 320 md⁻¹).

The urban land use of the Onehunga – Mt Wellington aquifer restricts the use of infiltration basins for recharging the aquifer due to the flooding of large areas of the land surface.

Otherwise the high infiltration rate of the aquifer and the thin layers of surface soils would have been suitable. A similar recharge option to the infiltration basins is soakage wells. Soakage wells have the added benefit of not requiring large areas of flooded land. Recharge by injection wells would also be achievable, except that it may be unnecessary if soakage wells could recharge the same volume.

Injection wells and soakage wells are capable of increasing the aquifers' recharge, but do not directly increase Watercare's groundwater yield. For this to happen the recharged water needs to be extracted by the existing Watercare wells in Onehunga, or new extraction wells need to be constructed to extract the recharged water. Aquifer storage and recovery (ASR) wells could be used for the seasonal storage and extraction of water to meet the seasonal difference between supply and demand. The limitation of the ASR wells in the unconfined Onehunga – Mt Wellington aquifer is that the injected water may flow away from the ASR wells and not be able to be extracted when required. However, if the ASR wells are near One Tree Hill, any water that may flow away from the wells is most likely to move towards the existing Watercare wells where some of it may be extracted (Figure 2.10).

2.6 Summary

The Onehunga – Mt Wellington aquifer is one of Auckland's oldest potable water sources dating back to the 1880's. Watercare currently utilise four wells in the Onehunga area near the Manukau Harbour. From the information presented in this chapter, there is potential for the aquifer's natural recharge to be increased from artificial recharge for the purpose of increasing the groundwater yield from Watercare's wells. This may be of importance in the future if the land uses and drainage types of the aquifer change, and the natural recharge is reduced. The basalt aquifer's groundwater flow field incorporates two zones of flow divergence. Groundwater from the Mt Wellington and Mt Smart area flows into the eastern end of the Manukau Harbour and groundwater from the One Tree Hill and Onehunga area flows to the western end of the Manukau Harbour. The second zone of groundwater flow divergence is at the southern end of the Mt Wellington area. Groundwater dewatering of the Mt Wellington quarry has produced a localised reversal of the natural groundwater flow field.

The One Tree Hill area is the most likely location for artificially recharging the aquifer to increase the yield from the existing Watercare wells. There is approximately 40 m of

unsaturated aquifer suitable for receiving artificial recharge in the vicinity of the One Tree Hill scoria cone. The Onehunga – One Tree Hill area of the aquifer also has the highest average hydraulic conductivity of 320 md^{-1} , compared to 36 md^{-1} for the Mt Smart area, and 85 md^{-1} for the Mt Wellington Valley. The high infiltration capacity of the aquifer is demonstrated by the quick response of the water table levels to rainfall events and the high recharge rate of the existing stormwater soakage wells in the Onehunga – Mt Wellington aquifer.

Chapter 3 presents a numerical model of the Onehunga – Mt Wellington aquifer based on the information presented in this chapter. This will be used for model-based evaluation of the various constructed groundwater systems for increasing the recharge of the Onehunga – Mt Wellington aquifer and Watercare's groundwater yield.

Chapter 3

Evaluation of Constructed Groundwater Systems in the Onehunga – Mt Wellington Aquifer: Model Development and Simulation

3.1 Introduction

This chapter presents the development of a transient numerical model of the Onehunga – Mt Wellington aquifer using the hydrogeological information collated in Chapter 2. The model was used to evaluate five options of constructed groundwater systems for increasing the recharge, and thus Watercare's yield from the Onehunga – Mt Wellington aquifer. The five constructed groundwater options presented in this chapter are; (1) aquifer storage and recovery (ASR) using the existing Watercare wells, and lowering of Watercare wells' intake screens (2) seasonal aquifer recharge with reservoir water via ASR wells, (3) seasonal aquifer recharge with reservoir water via soakage wells, (4) recharging groundwater from the Three Kings quarry via soakage wells; and (5) recharging tertiary treated wastewater via soakage wells. There are three sources of readily available recharge water in the Auckland Region. These are; treated water from Watercare's 10 storage reservoirs during winter, groundwater from the Three Kings quarry dewatering operation, and highly treated wastewater from Watercare's Mangere treatment plant.

3.2 Model Development

3.2.1 Conceptual Model

A conceptual model of the Onehunga – Mt Wellington aquifer was developed using the information presented in Chapter 2. The conceptual model indicates which data and numerical boundary conditions are required to numerically model the aquifer¹ (Anderson and Woessner, 1992a; 1992b; Konikow and Bredehoeft, 1992a). The components of the Onehunga – Mt Wellington aquifer conceptual model are discussed below and shown in Figure 3.1.

¹ References in this chapter to the 'aquifer' refer to the unconfined volcanic Onehunga – Mt Wellington aquifer, unless otherwise stated.

a) Model Domain Boundary

The model domain boundary follows the Manukau Harbour on the southern edge, and has been extended to include the upper reaches of the neighbouring Western Springs and Mt Eden aquifers to the north and west (Figure 3.1). The remaining model domain boundaries of the Onehunga – Mt Wellington basalt aquifer are defined by the interface between the basalt aquifer and the adjacent low permeable Waitemata Group rocks (Figure 3.1).

In this study, a limited vertical portion of the Waitemata Group rocks beneath the Onehunga – Mt Wellington basalt aquifer has also been included. This is to enable evaluation of the Waitemata aquifer's response to the five constructed groundwater options (Figure 3.1).

b) Hydraulic Parameters

The average horizontal hydraulic conductivity of the basalt aquifer in the Mt Wellington area is 80 md^{-1} , the Mt Smart area 36 md^{-1} , and in the Onehunga area 320 md^{-1} . The conductivities can range between $3,500 \text{ md}^{-1}$ for the cavernous zone of the aquifer at the Rowe Street well, to 0.5 md^{-1} for areas of solid unfractured basalt (Pattle Delamore Partners Limited, 1994). The vertical conductivity is estimated to range between 1/2 and 1/100 of the horizontal conductivity (Walton, 1991; Namjou, 1996). The porosity for the Onehunga – Mt Wellington aquifer ranges between 0.02 and 0.3, and the specific yield ranges between 0.04 and 0.2 (Roberts, 1980; Smaill, 1993a; Smaill, 1993b). The lower conductivity of the peat and reclamations near the foreshore has produced semi-confinement of the basalt aquifer. The storativity for the semi-confined zones is between 0.00001 and 0.0029 (Carrier & Associates Limited, 1991; 1992). The basalt aquifer is highly heterogeneous and large variations in all these parameters can occur over very short distances.

The average horizontal conductivity of the Waitemata Group rocks is 0.07 md^{-1} and the typical vertical conductivity is around 1/30 of the horizontal conductivity. The specific yield is 0.38, storativity 1×10^{-6} , and porosity is 0.45 (Namjou, 1996).

c) Groundwater Flow System

Above mean sea level, the groundwater flow direction in the basalt aquifer is controlled by the paleo valleys of the Waitemata Group rocks. Below mean sea level (bmsl), the paleo valleys merge into one large valley and the groundwater flow direction is mainly controlled by the hydraulic conductivity of the basalt aquifer. In the One Tree Hill area, the groundwater

flow is in a southerly direction towards the Manukau Harbour (Figure 3.1). In the Mt Wellington area, the dominant flow direction is down the Mt Wellington Valley entering the Manukau Harbour on the eastern side of the Mt Smart volcanic centre. At the head of the Mt Wellington Valley, north of the zone of groundwater divergence the flow direction is to the north converging on the Mt Wellington quarry (Figure 3.1).

d) Aquifer Recharge and Discharge

Recharge to the aquifer is via direct natural infiltration of rainfall at the ground surface and from induced recharge where street and building runoff is diverted into soakage wells. The average annual aquifer recharge (1993 – 2000) estimated during this thesis for the Onehunga – One Tree Hill management zone was $31,874 \text{ m}^3\text{d}^{-1}$, and for the Mt Smart – Mt Wellington management zone $16,380 \text{ m}^3\text{d}^{-1}$. The recharge for the Mt Smart – Mt Wellington aquifer was further divided into two zones. Southwest of the Mt Wellington quarry groundwater divide the recharge was $11,590 \text{ m}^3\text{d}^{-1}$, and northeast of the divide the recharge was $4,790 \text{ m}^3\text{d}^{-1}$. The estimation of aquifer recharge for this thesis is discussed in detail in section 2.4.7.

Aquifer discharge from the One Tree Hill – Onehunga management zone is from nine production wells, two springs and natural flow into the Manukau Harbour. Between 1993 and 2000 Watercare's four Onehunga wells extracted on average $9,600 \text{ m}^3\text{d}^{-1}$, or 31% of the Onehunga – One Tree Hill management zone recharge. The maximum groundwater extraction allowance set by the Auckland Regional Council for the four Watercare wells is a maximum of $19,040 \text{ m}^3\text{d}^{-1}$ over 12 consecutive months, or 62% of the recharge. The eight other groundwater users are allocated $3010 \text{ m}^3\text{d}^{-1}$ over a 1 year period, or 10% of the aquifer recharge. As a requirement of the Auckland Regional Council, at least 15% of the aquifer recharge must continue to flow into the Manukau Harbour to minimise contamination by saline intrusion (Figure 3.1). This leaves 13% or $4,130 \text{ m}^3\text{d}^{-1}$ unallocated in the One Tree Hill – Onehunga management zone.

Aquifer discharge from the Mt Smart – Mt Wellington management zone is from dewatering of the Mt Wellington quarry, twelve production wells, natural spring flow and natural flow to the Manukau Harbour. North of the groundwater divide, the Mt Wellington quarry and Panmure springs discharge $4,200 \text{ m}^3\text{d}^{-1}$ and $500 \text{ m}^3\text{d}^{-1}$ respectively. Together, these are equivalent to the total recharge north of the groundwater divide (Smaill, 1993a; Namjou, 1996). South of the groundwater divide, twelve users are allocated up to $7,882 \text{ m}^3\text{d}^{-1}$ over a 1 year period, or 70% of the recharge. A further $1700 \text{ m}^3\text{d}^{-1}$ (or 15% of the aquifer recharge)

is reserved for natural through flow to the Manukau harbour. This leaves 1700 m³d⁻¹ (or 15% of the aquifer recharge) unallocated in the Mt Smart – Mt Wellington management zone.

3.2.2 Model Software Selection

Initial model development was performed using MODFLOW, a three-dimensional finite difference groundwater flow code developed by the United States Geological Survey. However, due to the steep topography and complicated geology of the Onehunga – Mt Wellington aquifer, a model could not be developed where adjacent model cells overlapped by the 75 percent value needed for numerical stability (Guiguer, 1998). Reducing the grid size to 50 by 50 m cells and simplifying the geometry of the geology failed to substantially improve the overlap and stability problem.

To overcome the numerical instability of MODFLOW, the three-dimensional finite element model FEMWATER was selected as alternative software to model the aquifer. The FEMWATER model has the capacity to produce a deformed mesh, which accurately represented the aquifer geology and maintained numerical stability. FEMWATER was developed in the early 1990's through a cooperative research agreement between the U.S. Environmental Protection Agency and the U.S. Army Engineer Waterways Experimental Station. FEMWATER is a modern implementation of two older models 3DFEMWATER (flow) and 3DLEWASTE (transport). The FEMWATER program has been integrated into the Department of Defense Groundwater Modeling System (GMS). GMS contains a graphical user environment that allows efficient model set up and visualisation (Yeh, *et al*, 1997). A limitation of FEMWATER is errors in the reporting of the water balance. This is particularly the case for transient simulations where FEMWATER correctly models the pressure head and fluxes, but reports erroneous values in the water balance output (Herrmann, 2000 *pers comm*; Reeves, 2000 *pers comm*). On the other hand, FEMWATER does have the advantage of being relatively inexpensive and has good numerical stability.

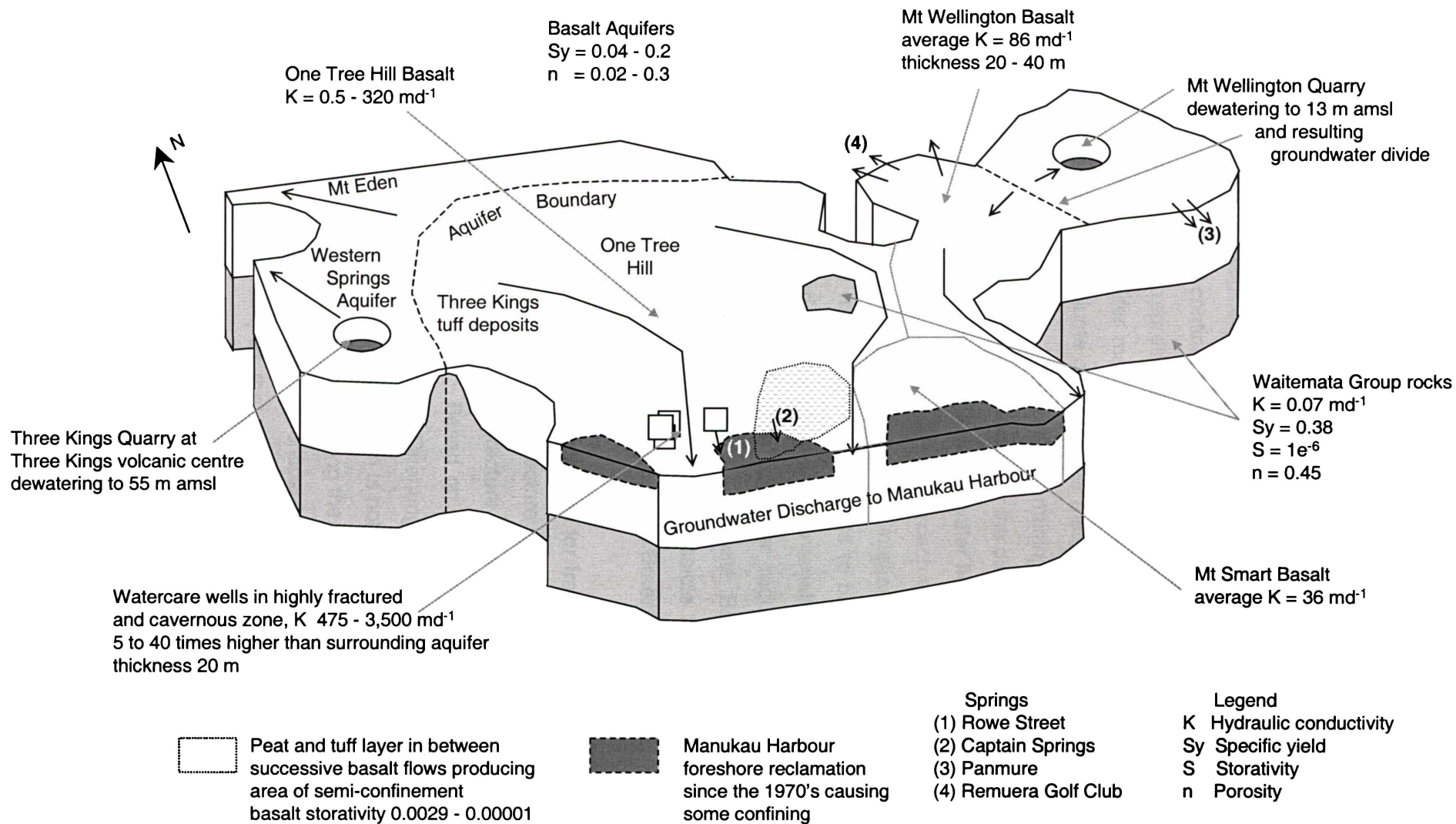


Figure 3.1. Conceptual model of the Onehunga – Mt Wellington aquifer.

3.2.3 Equivalent Porous Medium

A major assumption when numerically modelling fractured aquifers is treating the aquifer system as an equivalent porous medium (EPM). The EPM approach assumes that the fractured aquifer is hydrologically equivalent to a porous medium. That is, the fractured material can be treated as a continuum by replacing the primary and secondary porosity, and hydraulic conductivity with a representative elementary volume (REV). The REV describes the minimum volume of aquifer material at which point the hydraulic parameters are not sensitive to changes in the volume of the aquifer material (Anderson and Woessner, 1992a; Domenico and Schwartz, 1998). However, determining the size of a REV is very difficult. The obvious advantage of the EPM approach over the discrete approach, where all individual fractures need to be accounted, is that the groundwater flow can be calculated using Darcy's Law. Due to the spatial averaging of the EPM approach when modelling fractured flow, it is not possible to adequately interpret small-scale measurements (Cacas, *et al*, 1990). Therefore, the scope of the model application needs to be defined before using the EPM approach. Discrete fractured models are limited to small-size problems due to the vast amount of information needed and large computer requirements to handle a system containing hundreds or thousands of fractures (Berkowitz, Bear and Braester, 1988; Cacas, *et al*, 1990). Furthermore, it remains virtually impossible to determine the location and characteristics of all the fractures (Berkowitz, Bear and Braester, 1988).

The EPM approach was found to be most effective for fractured aquifers with the following parameters; distributed rather than constant fracture alignment, high fracture density, uniform small fracture aperture, and a low ratio of fracture length to aquifer area (Long *et al*, 1982; Berkowitz, Bear and Braester, 1988). Based on extensive work by Namjou (1996) in the vicinity of the Mt Wellington quarry in the Onehunga – Mt Wellington aquifer the following information is available: the fracture orientation is not controlled by a dominant joint direction; the occurrence of a drawdown cone surrounding the quarry indicates that the fractured basalt is interconnected and therefore above the critical fracture density; the flow through the joint set should be relatively uniform, as the maximum variation in fracture aperture was three millimetres, with an average aperture width of six millimetres. There is also a low ratio of fracture length to aquifer area, with an average measured fracture length of two metres, which is considerably smaller than the aquifer region. The only difference in aquifer properties in the Onehunga area of the aquifer, is a network of caverns ranging from a few centimeters to many metres in diameter (Figure 3.2).

The extent of the Onehunga – Mt Wellington aquifer model is large enough that the spatial averaging of the aquifer properties is not unreasonable when simulating the general groundwater flow patterns.



Figure 3.2. Large lava cave near Three Kings quarry in the Western Springs aquifer (Photo: L. Homer).

3.2.4 Model Mesh

It is important to construct a mesh of the model domain that is computationally efficient while still capturing the general aquifer spatial variations. A small number of elements will not always result in a fast simulation. A recommended balance is to have a larger number of elements and less iterations to achieve an accurate answer, rather than a small number of elements and many iterations for an inaccurate answer (Yeh, *et al*, 1997). The model mesh for the Onehunga – Mt Wellington aquifer presented in this chapter was constructed using wedge shaped elements with planar faces. Planar element faces are required if the model is used for transport analysis. Otherwise the particle tracking algorithm used by FEMWATER may break down (Yeh, *et al*, 1997).

The model mesh for the Onehunga – Mt Wellington aquifer was first constructed in two dimensions in the horizontal plane. This was then converted into a three-dimensional mesh using the thickness of the aquifer geology.

a) Development of Two-dimensional Mesh

To aid in defining the geometry of the model mesh for the Onehunga – Mt Wellington aquifer, the aquifer boundary, water level observation points and pumping wells were represented with model nodes (Figure 3.3). In the vicinity of the four Watercare pumping wells, where steep head gradients were expected, the mesh size was set at 25 m. For the remaining wells, where the pump rates were considerably lower, the mesh size was set at 40 m. The average mesh size of the external boundary elements was set at 335 m. From the location of the model nodes representing the observation and pumping wells, the model mesh was generated automatically with the elements increasing in size towards the most distant boundaries by a maximum of 40%. The mesh was refined further to remove any highly skewed or irregular shaped elements that may have resulted in numerical instability. The final two-dimensional mesh is shown in Figure 3.3.

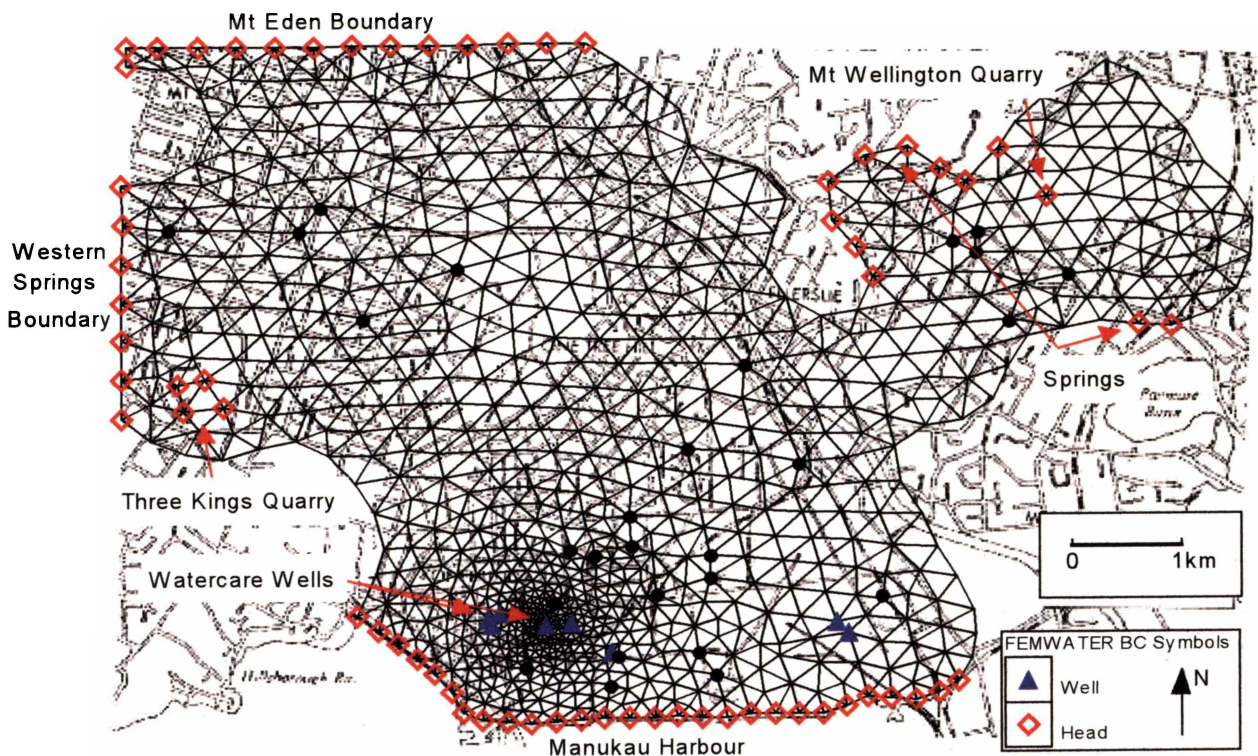


Figure 3.3. Two-dimensional model mesh and location of boundary conditions. Black circles show location of observation wells used to help define the finite element mesh geometry.

b) Development of Three-dimensional Mesh

The two-dimensional mesh was converted into a three-dimensional mesh based on the collected geological information described in section 2.3. The geology was divided into five types: the Waitemata Group rocks, tuff, landfill material, peat, and the basalt aquifer.

Initially one layer was constructed. The top surface of this layer matches the topography of the land surface, and the bottom surface of the layer matches the paleo topography of the underlying Waitemata Group. This initial layer was then subdivided into four inner layers of equal thickness. In areas of the aquifer with tuff, peat, and landfill materials, the thickness of these four inner layers were adjusted to match the thickness of these three materials. The remaining portions of the four inner layers that were not adjusted represent the basalt aquifer. The Waitemata Group is represented by a fifth layer under the existing four layers. This fifth layer was given a flat base (30 m bmsl), and has a maximum thickness of 104 m and a minimum thickness of 20 m. Figure 2.5 shows the geometry of the model layers matching the aquifer's geology.

The aquifer topography was based on digitised elevations from an Auckland City Council engineering services 5 m contour map, scale 1:5000 (Cooper, 1993). Refinement of the elevations was made using surveyed ground level of the wells in the aquifer. The summits of the One Tree Hill and Mt Wellington volcanic cones were lowered in the model to aid numerical stability. One Tree Hill was lowered from 196 m to 139 m above mean sea level (amsl) and Mt Wellington from 120 m to 100 m amsl. This does not change the volume of saturated aquifer.

3.2.5 Boundary Conditions

In the GMS interface software for FEMWATER five boundary condition types are supported: (1) point source or sinks, typically used to represent wells; (2) specified head and or concentration, often applied to aquifer interfaces with streams, lakes, and coast lines; (3) specified flux boundary is applied to an element face often to represent recharge; (4) gradient flux boundary; and (5) variable conditions. The variable condition option is typically applied to the upper most model element faces to represent evaporation, seepage due to evaporation and seepage exit faces. They are termed 'variable' as they correspond to either the specified head or flux boundaries (Yeh, *et al*, 1997).

Based on the physical boundaries of the aquifer, the numerical boundary conditions listed below were applied to the model. Boundary locations are shown in Figure 3.3.

a) Point Source/Sink Boundaries

Eight groundwater wells with known pumping rates were represented in the model with point source boundaries applied to nodes corresponding to the level of the intake screens. The flux is in units of m^3d^{-1} .

b) Specified Head Boundaries

The coastal boundary of the Manukau Harbour was represented with specified head boundaries fixed at zero metres elevation.

Dewatering of the Three Kings and Mt Wellington quarries has resulted in the quarries' water table elevations remaining relatively constant at approximately 53 m and 13 m respectively. The volume of groundwater abstracted from the quarries has not been accurately recorded, limiting the use of point source/sink boundaries to represent the groundwater discharge. Thus, the dewatering wells for the Mt Wellington and Three Kings quarries were simulated with specified head boundary conditions with elevations of 53 m and 13 m to represent the respective water tables.

The springs in the Mt Wellington area were simulated with specified head boundary conditions corresponding to the spring elevation.

The Western Springs and Mt Eden model boundaries have been positioned so that they are distant from the main area of interest, the Onehunga – Mt Wellington aquifer. These two model boundaries are not aligned with any actual aquifer boundaries such as a groundwater divide or geological feature. The Western Springs and Mt Eden boundaries (Figure 3.3) are represented with specified head boundaries to mimic the estimated average water levels of these aquifers. In reality, the water levels of these aquifers would fluctuate seasonally and for individual rainfall events. Transient specified head boundaries were not applied, due to the lack of observation wells to provide the appropriate head values. The sensitivity of the model domain to the applied fixed head values is described as part of the model sensitivity analysis, Appendix E.

c) Specified Flux Boundaries

Specified flux boundaries were used to apply 7-day averaged aquifer recharge to the upper face of the ground surface elements. The model recharge input units are md^{-1} .

d) Zero Flux Boundaries

A zero flux boundary was applied in locations where the basalt aquifer is adjacent to the Waitemata Group rocks. It is assumed that there will be insignificant flow between the two formations due to the relatively low permeability of the Waitemata Group rocks compared to the basalt aquifer.

The Captain Springs and the Rowe Street well springs in the Onehunga area were not incorporated into any boundary conditions due to their close proximity to the Watercare wells. Applying a boundary condition in this location could limit the model's ability to simulate the change in water level due to pumping because of proximity effects.

3.2.6 Model Input Data

The transient model was developed using aquifer recharge, discharge and water level data from 1993 to 2000.

3.2.6.1 Recharge

The completed model consisted of thirteen recharge zones, with recharge applied to the model as 7-day averages. Twelve of these recharge zones are based on the aquifer's land uses, drainage types and rainfall depth from the nearest rain gauge (Figure 3.4). The distribution and allocation of recharge for these twelve zones is discussed in section 2.4.7.2. A thirteenth recharge zone was required for a small residential area north west of One Tree Hill to stop model-generated surface flooding. This thirteenth zone is mostly in the Western Springs aquifer near the groundwater divide produced by the paleo Waitemata drainage valleys. A simple black-box recharge model was devised to calculate the recharge rate for this zone. This recharge model was adjusted during the groundwater model calibration phase. This new recharge zone is shown in Figure 3.4. The recharge model is described in more detail in Appendix D.

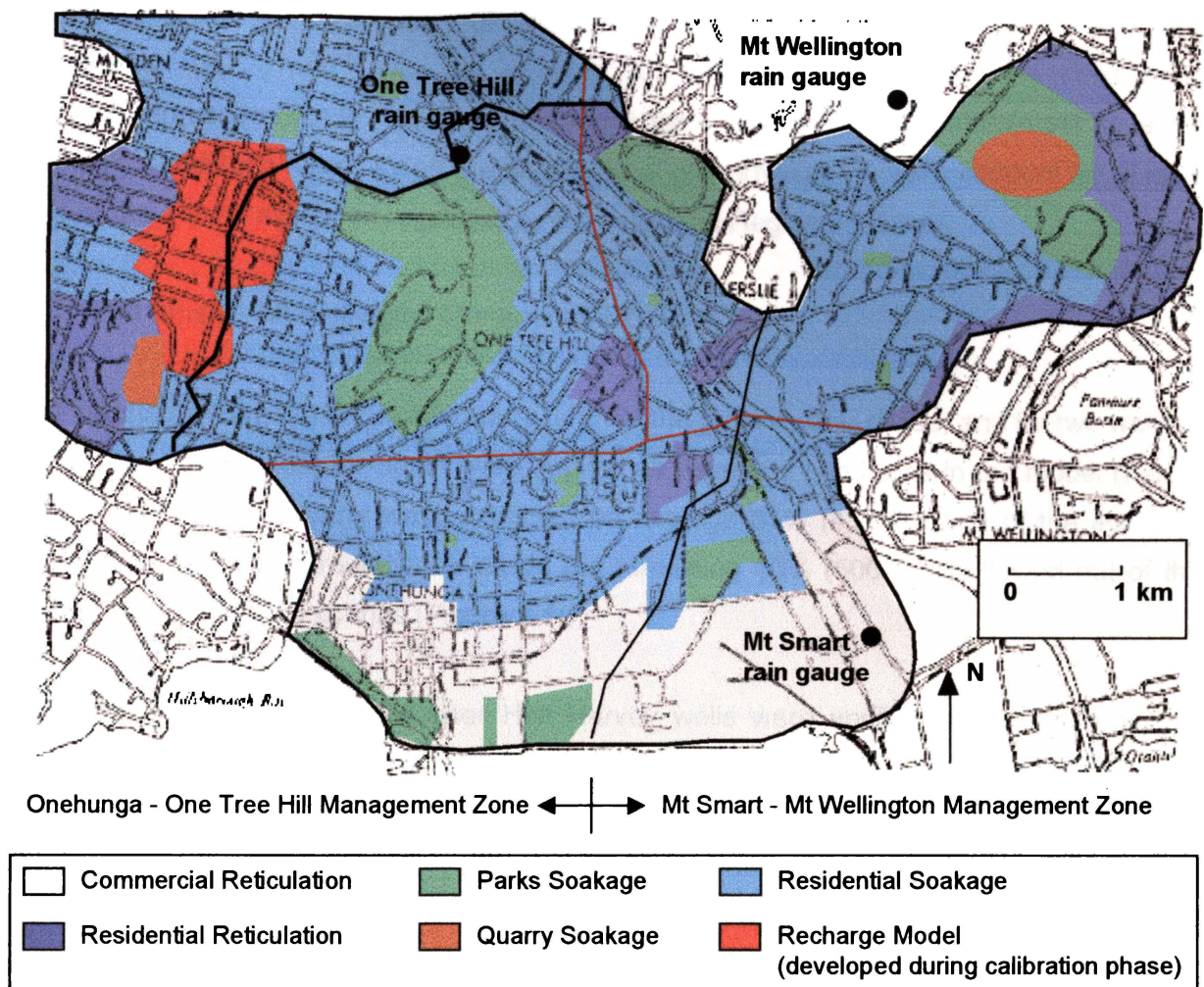


Figure 3.4. Model recharge zones. Red lines divide catchment by the nearest rain gauge. Black line defines boundary of management zones.

3.2.6.2 Well Discharge

Discharges from wells were applied to the model as 7-day averages.

a) One Tree Hill - Onehunga Management Zone

Well discharges from Watercare’s four production wells, the old Rowe Street well at Bumper Replacements and the Lichenstein well were applied as boundary conditions in the model (Figure 3.5). These wells were used as boundary conditions in the model as their combined maximum yield allocated by the Auckland Regional Council is 66% (average of 20,210 m³d⁻¹) of the aquifer’s annual recharge in the One Tree Hill – Onehunga area. The Watercare wells are allocated 62% (average of 19,040 m³d⁻¹) of the aquifer’s annual recharge in the One Tree Hill – Onehunga area. The combined allocation for both Bumper Replacements and

Lichensteins is 4% (1170 m³d⁻¹) of the annual recharge. Six percent of the allocated groundwater (2,220 m³d⁻¹) was not included due to a lack of data from the other users discussed in section 2.4.2. Thus, in the Onehunga – One Tree Hill management zone the boundary conditions representing the discharge of groundwater via wells account for at least 90% of the annual allocation regulated by the Auckland Regional Council.

b) Mt Smart – Mt Wellington Management Zone

Well discharge from the Winstone quarry north of the groundwater divide, and for two wells at Carter Holt Harvey near Mt Smart were applied as boundary conditions in the model (Figure 3.5). The Winstone quarry wells extracted on average 87% (4,200 m³d⁻¹) of the available recharge north of the groundwater divide the remaining 13% (500 m³d⁻¹) flowed out of the aquifer at the Panmure springs (Figure 3.1).

South of the divide, the two Carter Holt Harvey wells were applied in the model with a combined average extraction of 11% (1200 m³d⁻¹) of the recharge, 15% (1700 m³d⁻¹) was unallocated, and a further 15% (1700 m³d⁻¹) was reserved for natural discharge into the Manukau harbour. The remaining 59% (6,660 m³d⁻¹) was not included due to a lack of data from the other users, discussed in section 2.4.2. However, much of this remaining 59% may not have been extracted as many of the small users have been allocated groundwater, but only use it infrequently, if at all (Small, 2000 *pers comm*).

In the Mt Smart – Mt Wellington management zone the boundary conditions representing the discharge of groundwater via wells account for at least 47% of the annual allocation regulated by the Auckland Regional Council.

3.2.6.3 Groundwater Levels

Groundwater levels were used during the evaluation of the model's calibration and validation. Thirty observation wells in the aquifer provided time series of point water level observations; five wells provided daily levels, four at least weekly, and the remaining 21 approximately monthly levels. A further 13 wells provided groundwater levels observed at the time of well construction (Figure 3.5). These 13 sites were used to aid model calibration in areas without the available time series of water level observations. The recorded groundwater levels are discussed in section 2.4.4.

The spatial distribution of the initial water levels for the model calibration were estimated from measured water levels on the 1 January 1998, using kriging interpolation.

3.2.6.4 *Hydraulic Parameters*

The hydraulic parameters identified in section 2.4.5 provided an estimate of the plausible range for the parameters applied in the model. Initial estimates were then adjusted as part of the calibration process.

The initial hydraulic conductivities for the basalt aquifer were, 320 md^{-1} in the One Tree Hill and Onehunga area, 35 md^{-1} in the Mt Smart area, and 86 md^{-1} in the Mt Wellington area. The initial vertical conductivity was set at 1/10 of the horizontal. The specific yield was set at 0.15, the porosity at 0.2, and the storativity of the confined zones of the aquifer near the Manukau Harbour were set at 0.0001.

The initial hydraulic conductivities of the tuff, peat and reclamations were set at 2 md^{-1} , the specific yield 0.3, and the porosity 0.4.

The hydraulic conductivity of the Waitemata Group rocks was set at 0.07 md^{-1} , specific yield 0.38, storativity 1×10^{-6} , and porosity 0.45. The vertical conductivity was set at 0.002 md^{-1} , 1/30 of the horizontal value.

3.3 Overview of Evaluating Model Goodness-of-Fit

The process of checking whether any model will have good predictive capabilities generally requires model calibration (or history matching), validation (independent history matching), and postaudit. During calibration, model output variables such as water level and concentration are compared to observed data sets. The model is considered calibrated after parameter values have been adjusted so that the model reproduces historical data within some subjectively defined acceptable level. There are no rules other than one's judgement (Anderson and Woessner, 1992a; Konikow and Bredehoeft, 1992a; 1992b).

Typically, when calibrating a model there are many unknown parameters, making it impossible to obtain a unique solution (Konikow and Bredehoeft, 1992a). Numerous identical models can be calibrated to the same level of acceptance, and all can have a

different solution (Freyburg, 1988). Generally the longer the matched history period the more reliable the model will be for predictions (Konikow and Bredehoeft, 1992a; 1992b).

After the model is calibrated, if further observation data are available, a model validation can be performed. The validation (or in some texts 'verification') tests the model's ability to independently reproduce another part of the historical record not used during the calibration. If possible, the calibration and validation data sets should apply different stresses on the model, rather than being a replication. The validation process is important because a model with an acceptable calibration may not have good predictive capabilities (Freyburg, 1988). Once a model has been used to make predictions it is useful to perform a postaudit to compare the goodness-of-fit between the prediction and what actually occurred. This can provide important information if the model needs to be revised (Konikow, 1986; Anderson and Woessner, 1992b).

Konikow and Bredehoeft, (1992a) questioned the general view of most groundwater modellers by stating "groundwater models cannot be validated". Due to the uncertainty in conceptualisation and parameter estimation that is inherent with groundwater models, the issue is raised as to whether a model can ever be demonstrated as "correct". According to Konikow and Bredehoeft, (1992a), the only remaining option is to invalidate the model, and this will in turn increase the users' understanding of the model's limitations. However, De Marsily, Combes and Goblet, (1992) disagreed with these comments by Konikow and Bredehoeft. De Marsily, Combes and Goblet, (1992) stated that as long as the model reproduces the observed behaviour of the groundwater system, the model can then be used for making predictions, and the longer the validation data set the more confident the user can be in the model's validity.

In this thesis, the goodness-of-fit between the Onehunga – Mt Wellington aquifer's modelled water levels, and the observed data was evaluated both quantitatively and qualitatively. Model calibration was performed using 1998 water level data, and the validation using 1999 water level data. After the calibration and validation was completed, further water level data became available. This new data for the years 1993 to 2000, excluding 1998 and 1999, was used for another model validation. Contour maps of the modelled and observed water levels were used to provide a qualitative measure of the spatial distribution of model error. Time series of the modelled and observed water levels for individual observation wells were also compared to ensure both data sets followed the same temporal trend and were of similar magnitude. Quantitative evaluation of the goodness-of-fit between the two sets of water level data was performed using the root mean squared error (RMSE) and the mean absolute error

(MAE). The RMSE and MAE provided an evaluation of the error in units of the water levels (m).

Because of the sensitivity of the RMSE to extreme values, Legates and McCabe Jr (1999) recommend using MAE. The Pearson's correlation coefficient (r) and the coefficient of determination (R^2) were not used during the evaluation because they can be misleading. It is possible for models to have a poor fit between observed and modelled data, while still achieving a high correlation using r and R^2 (Legates and McCabe Jr, 1999).

3.4 Calibration and Validation of the Transient Model

A trial and error process was used during the model calibration to match the modelled and observed water levels. The trial and error process required dividing the model domain into 32 zones with differing hydraulic properties, adjusting the magnitude of the hydraulic properties for each zone, adjusting the size of the zones, and developing a recharge model which is described in section 3.2.6.1. The locations of the water level observation sites used during the model calibration are shown in Figure 3.5. Details of the numerical model's solver and iteration parameters used to achieve calibration are in Appendix F.

The model calibration was terminated when an RMSE of 1.20 m and MAE of 0.81 m was achieved between the observed and modelled water levels for the 30 observation wells. The model validation produced a slightly better fit with an average RMSE of 1.03 m and MAE of 0.77 m.

The final magnitudes of the 32 zones required to achieve the given calibration are shown in Figure 3.6 and Table 3.1. Areas of clay, peat, tuff and solid basalt required a lower relative hydraulic conductivity than areas of fractured basalt and scoria. The lowest hydraulic conductivity (0.0032 md^{-1}) was for the Waitemata rocks, and the greatest conductivity value ($1,000 \text{ md}^{-1}$) was for the cavernous basalt in the vicinity of the Rowe Street well. A conductivity of $1,000 \text{ md}^{-1}$ was required for the area near the Rowe Street well. This is lower than the actual conductivity of $3,500 \text{ md}^{-1}$ (Pattle Delamore Partners Limited, 1994). The average vertical conductivity for the basalt aquifer was 1/7 of the horizontal values and 1/12 for the Waitemata Group rocks.

The model's cumulative water balance for the period 1993–2001 shows a total water input of $1.75 \times 10^8 \text{ m}^3$, output of $1.975 \times 10^8 \text{ m}^3$, and the difference is 11%. An error of approximately 1% is usually acceptable (Anderson and Woessner, 1992a). The error in the above water balance is thought to be related to errors in the output of the water balance information rather than the model, as discussed in section 3.2.2. One obvious water balance reporting error is for the first simulation time step where the water balance report recorded the recharge flux as a discharge.

Table 3.1. Hydraulic parameters excluding conductivity used to calibrate the model domain

Lithology	Specific yield	Storativity	Porosity
Basalt	0.15	0.000010	0.20
Waitemata (mudstone/sandstone)	0.38	0.000001	0.45
Reclamation (landfill)	0.31		0.38
Peat	0.36		0.43
Tuff	0.31		0.38

3.4.1 Evaluation of Model Calibration and Validation

There is little temporal variation in the MAE and RMSE for the point values of observed and modelled water levels calculated approximately every seven days between 1993 and 2000 (Figure 3.7a). The exception to this, is an increase in error during 1998 and 1999. This increase was due to the inclusion of water level observation data measured between September 1998 and December 1999 from observation wells, 87005, 87011 and 87013. This data was not available during the calibration process. But it was applied to the model after the calibration process to evaluate the model's goodness-of-fit. The model generally under-estimated the water levels of the three wells by up to 8 m, and over accentuated the water table fluctuations in response to recharge events (Figure 3.8). Excluding these three wells from the calculation of MAE and RMSE decreases the errors during 1998 and 1999 (Figure 3.7b).

The observed and modelled water levels for both the calibration and validation periods are shown in Figure 3.8. Wells with a good match between observed and predicted water levels during the calibration also have a good match during the validation period. Examples of this are for observation wells 7614, 7615, Ag 98007, and 98033. The opposite also occurs, where a poor match during calibration has resulted in a poor match during the validation period. Examples of this are for observation wells Ag98005, 88111, 98021, and 98031. The

spatial distribution of the observation wells which were deemed to have an acceptable match between the observed and modelled water levels during the calibration analysis are shown in Figure 3.5. Figure 3.5 identifies areas of the aquifer which are most likely to produce good or poor predictions of the aquifer's water levels based on the observed goodness-of-fit during the calibration and validation. The Mt Smart and Western Springs areas have limited predictive capabilities (Figure 3.5).

The model calibration in the One Tree Hill area was limited by the sparse number of observation wells (Figure 3.5). The predictive capability of this area of the model is relatively unknown. The most useful data set was for well 97015, with monthly records of water levels from mid 1998 to present.

The modelled water levels generally match the observed water levels for the four Watercare wells. Exceptions to this have occurred when the model has under predicted the drawdown by up to 1 m at the Rowe Street (7614) and Pearce Street (7615) wells during times of reduced recharge and increased pumping stress. This occurred for short periods during 1994, 1995, 1999, and 2000. Watercare's Upper Municipal well (7616) has the worst fit of the four Watercare wells. This well is up-gradient from the three other wells, was used intermittently and had the lowest pump rate. However, as the water levels are consistently lower than the other wells, this raises the possibility of a datum error. It is recommended that when new data becomes available, the model should be recalibrated with the existing data to provide an improved fit for the observation wells near the Western Springs and Onehunga – Mt Wellington aquifer boundary, the Mt Smart area, and the four Watercare supply wells. The new data can then be used for an independent validation.

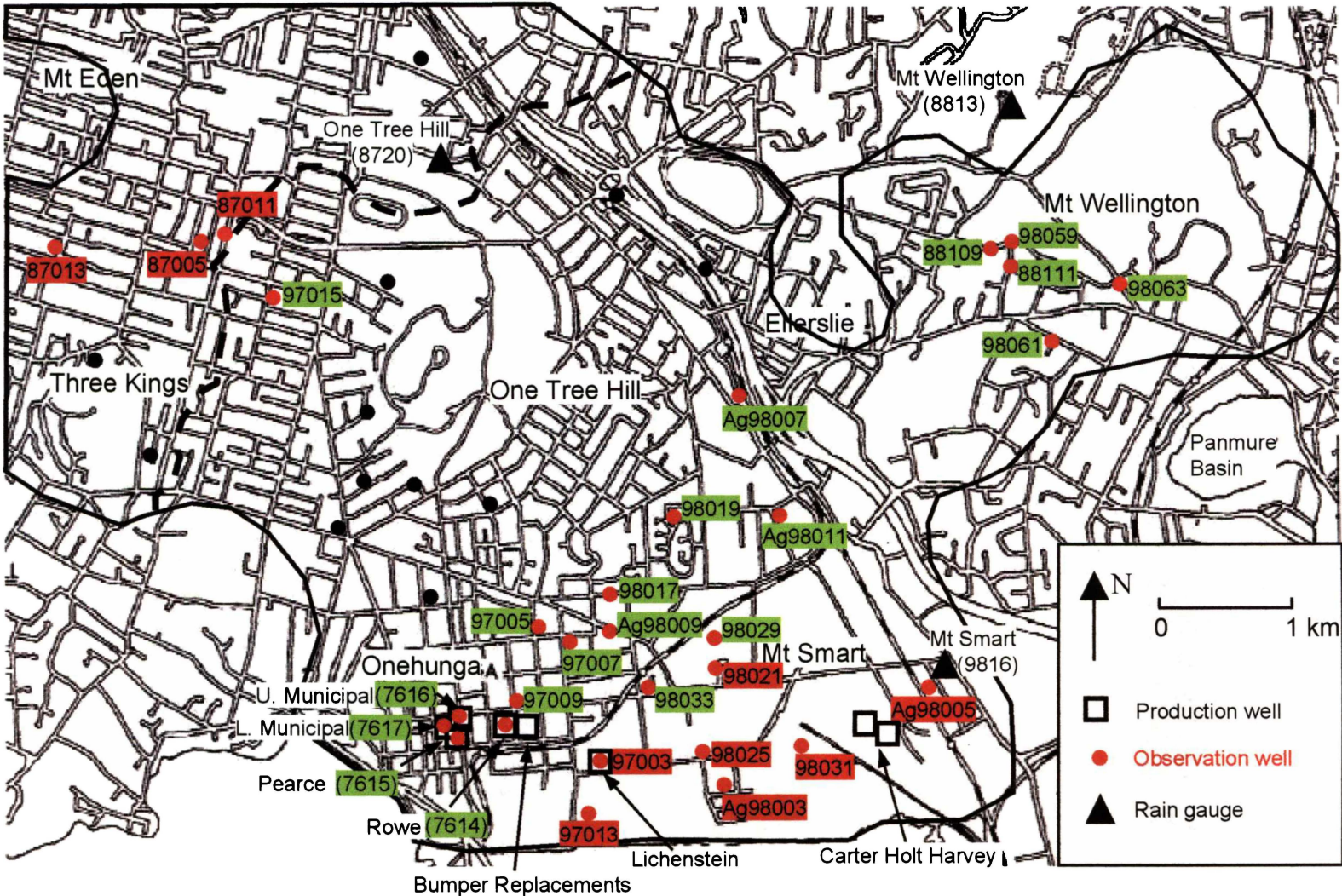


Figure 3.5. Spatial distribution of observation wells for model calibration and validation. Well labels shaded green indicates a good match between observed and modelled data during the calibration and validation. Well labels shaded red indicates a poor fit. Black circles mark locations of water levels observed only during well construction. Prefix 'Ag' identifies automated water level recorder wells.

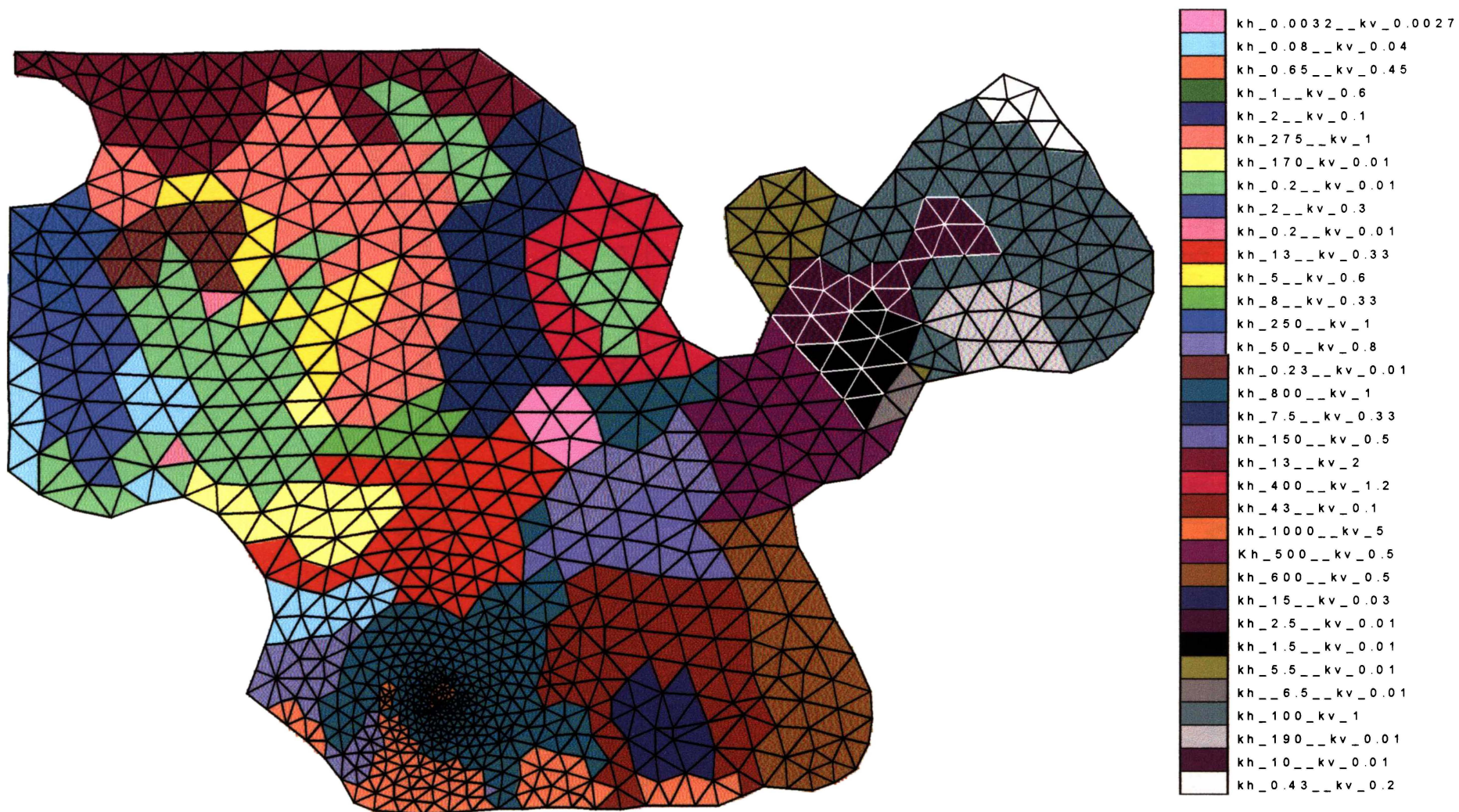
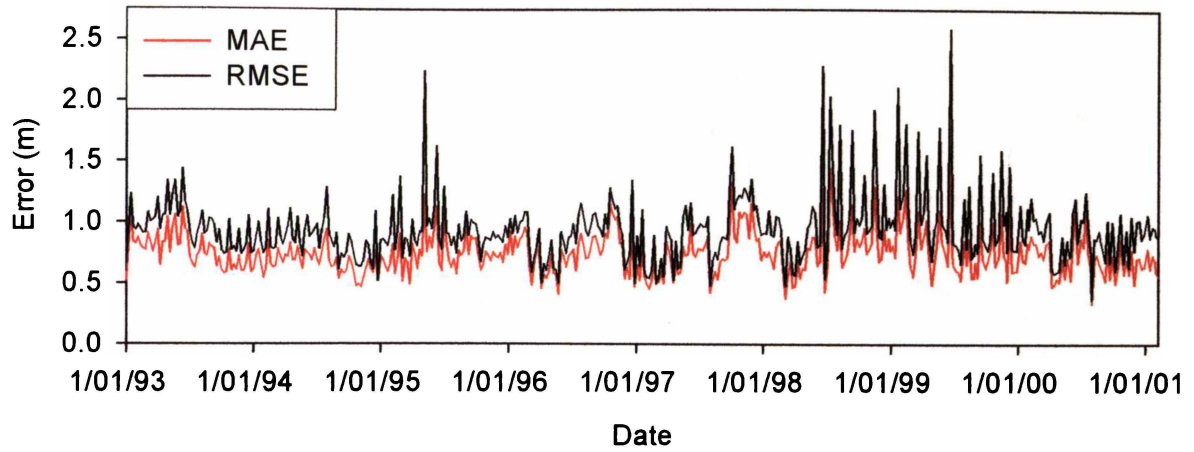
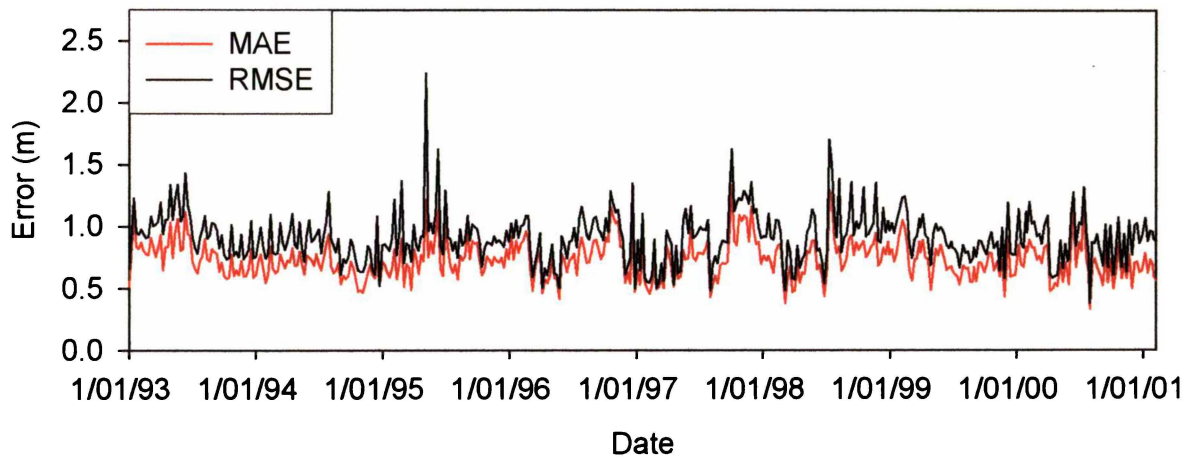


Figure 3.6. Hydraulic conductivity zones which gave calibration of the model. Horizontal (kh) and vertical (kv) hydraulic conductivity values are in units of md^{-1} .



a)



b)

Figure 3.7. a) Calibration and validation RMSE and MAE for the observed and modelled water levels from the 30 observation wells calculated approximately every seven days, and b) the same data except excluding observation wells, 87005, 87011, 87013. The 1998 data was used for the model calibration.

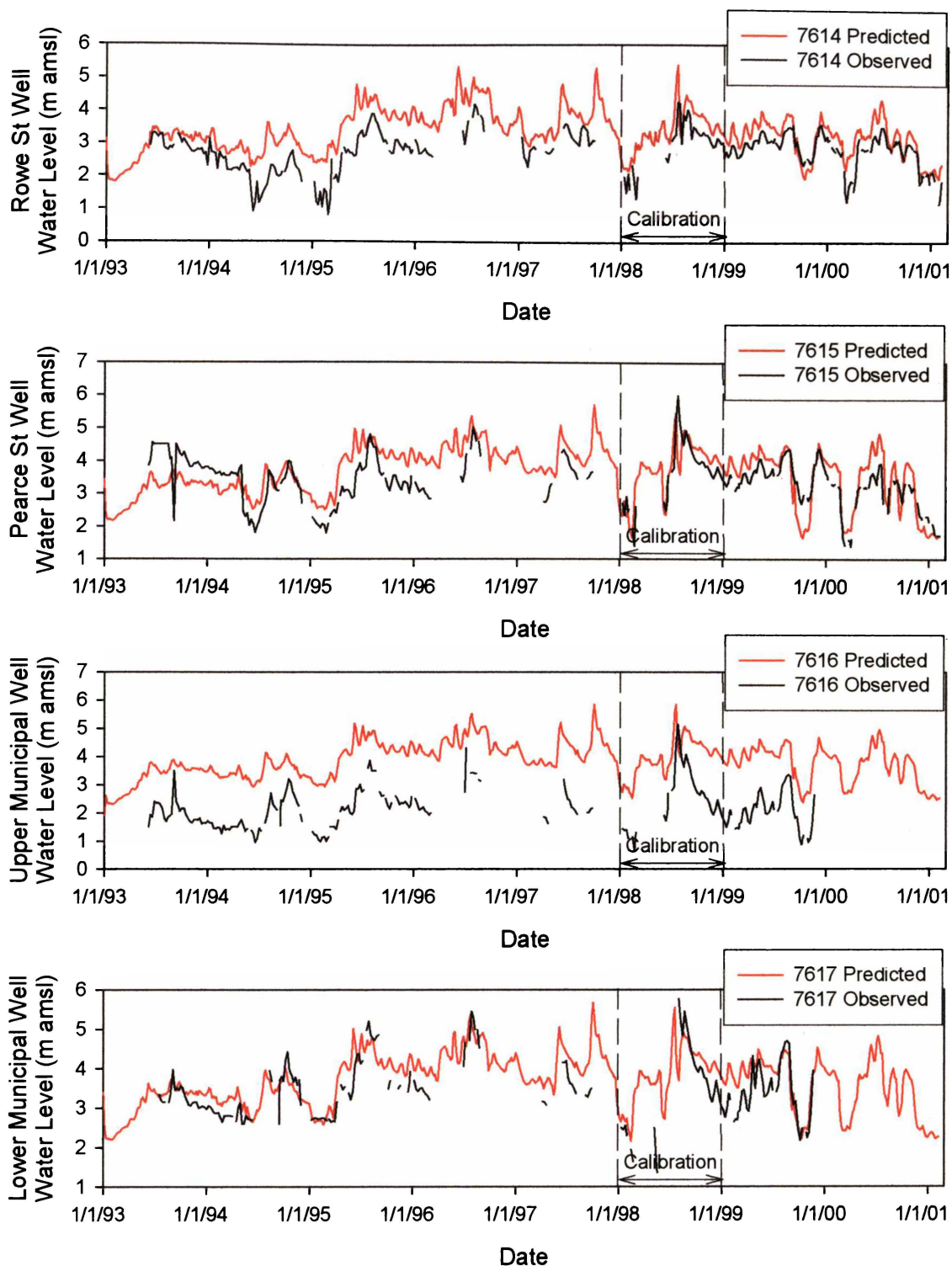


Figure 3.8. Calibration and validation time series of observed and simulated water levels from 1993 to 2000. The observation well identification number used by the Auckland Regional Council is shown in the legend of each plot.

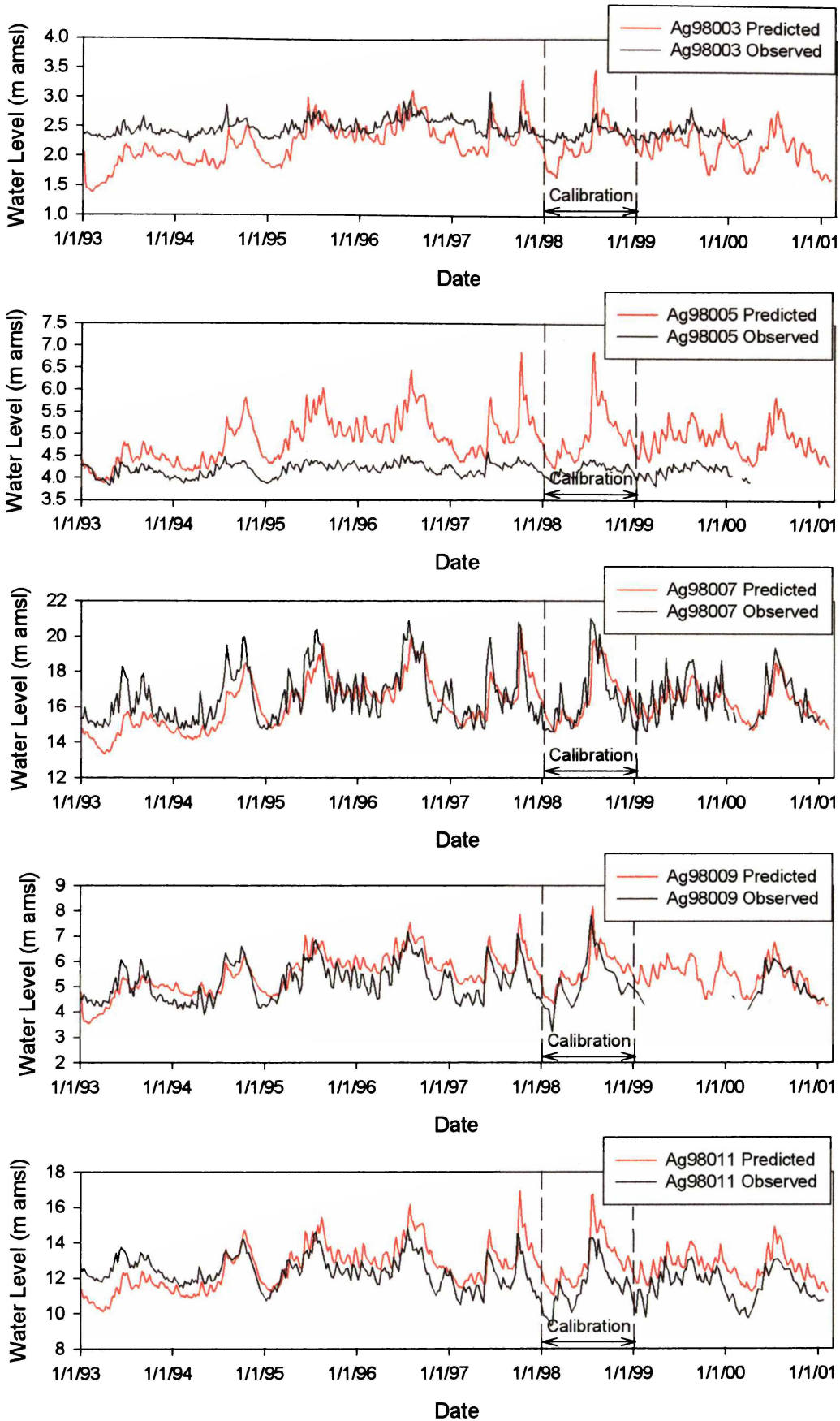


Figure 3.8. (continued).

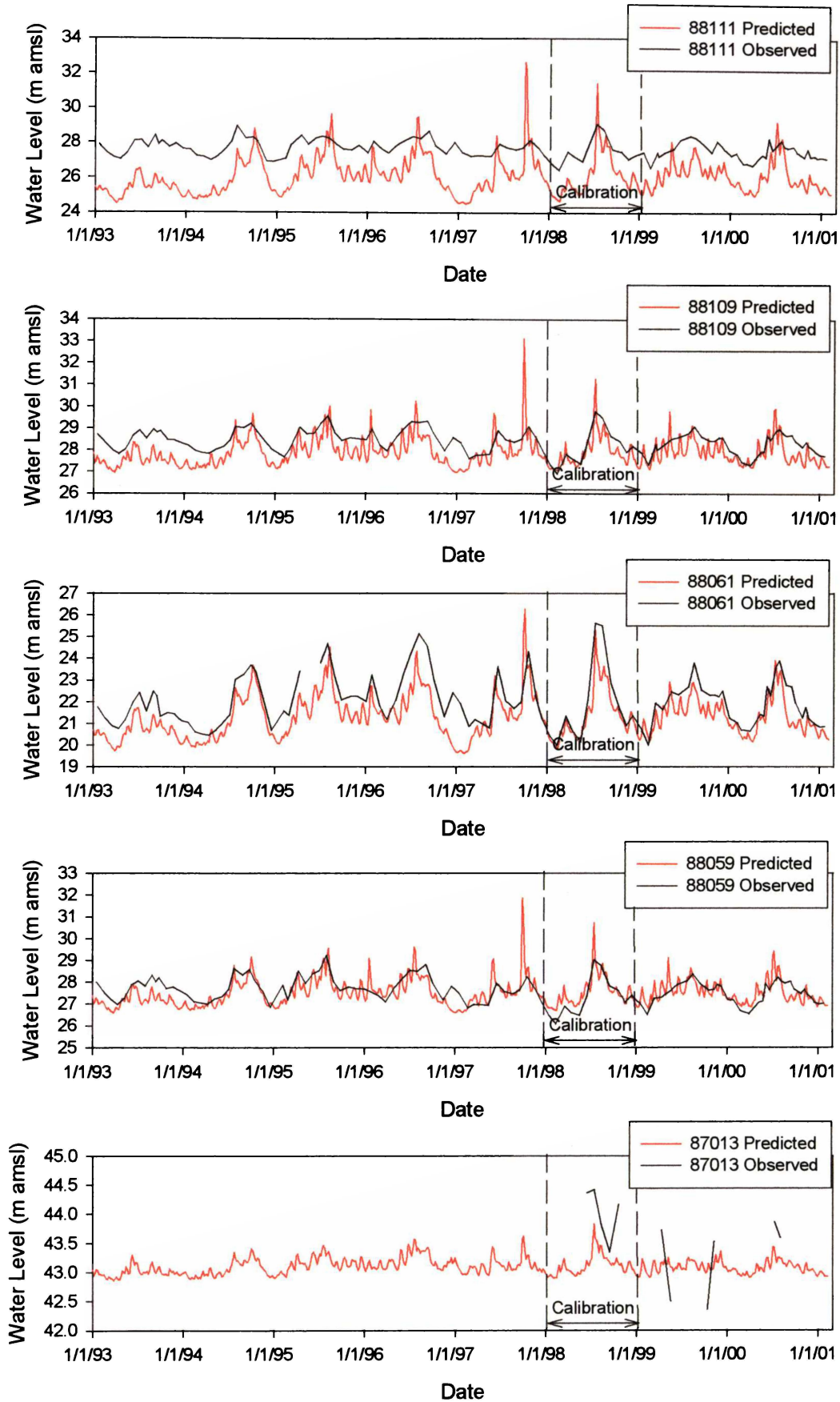


Figure 3.8. (continued).

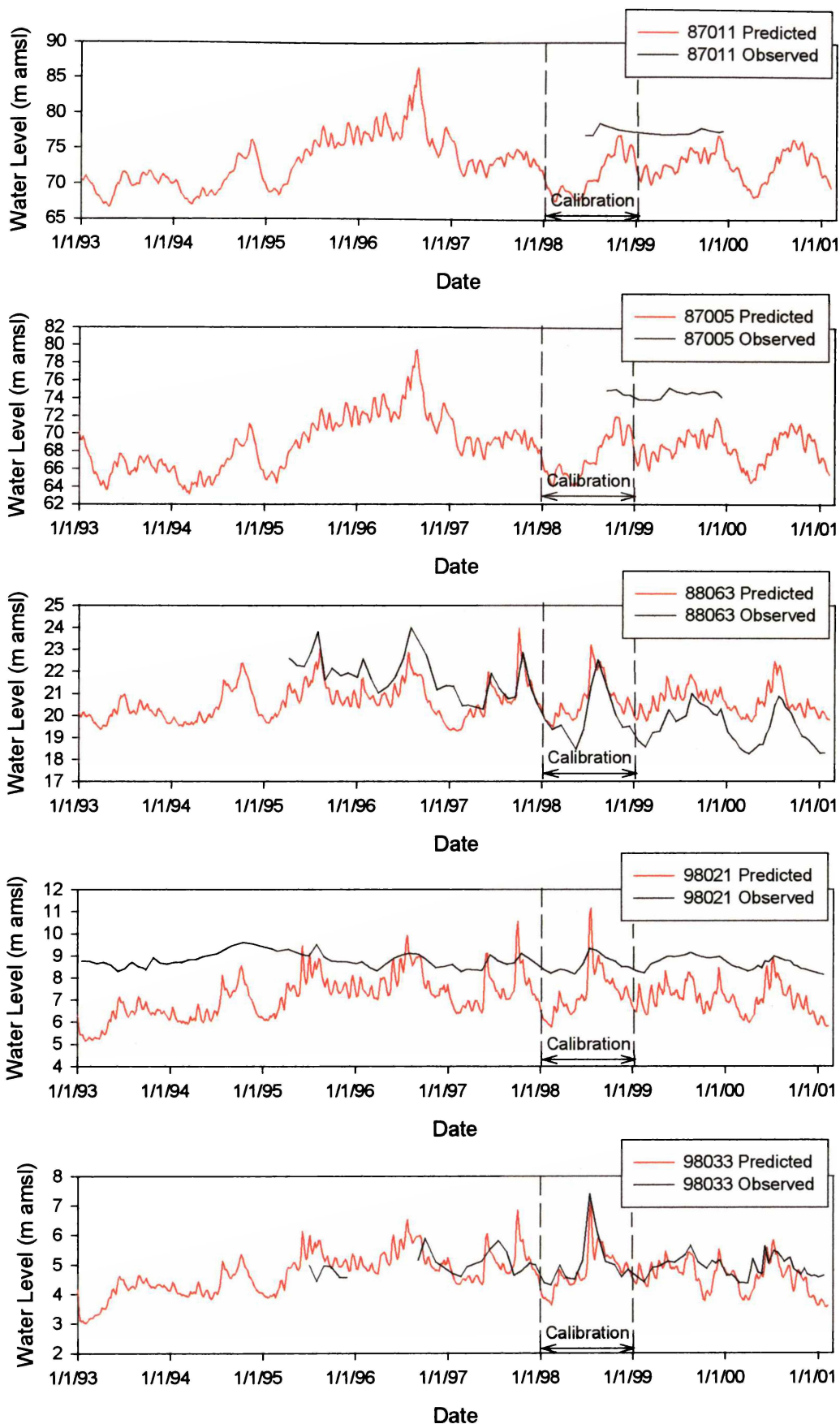


Figure 3.8. (continued).

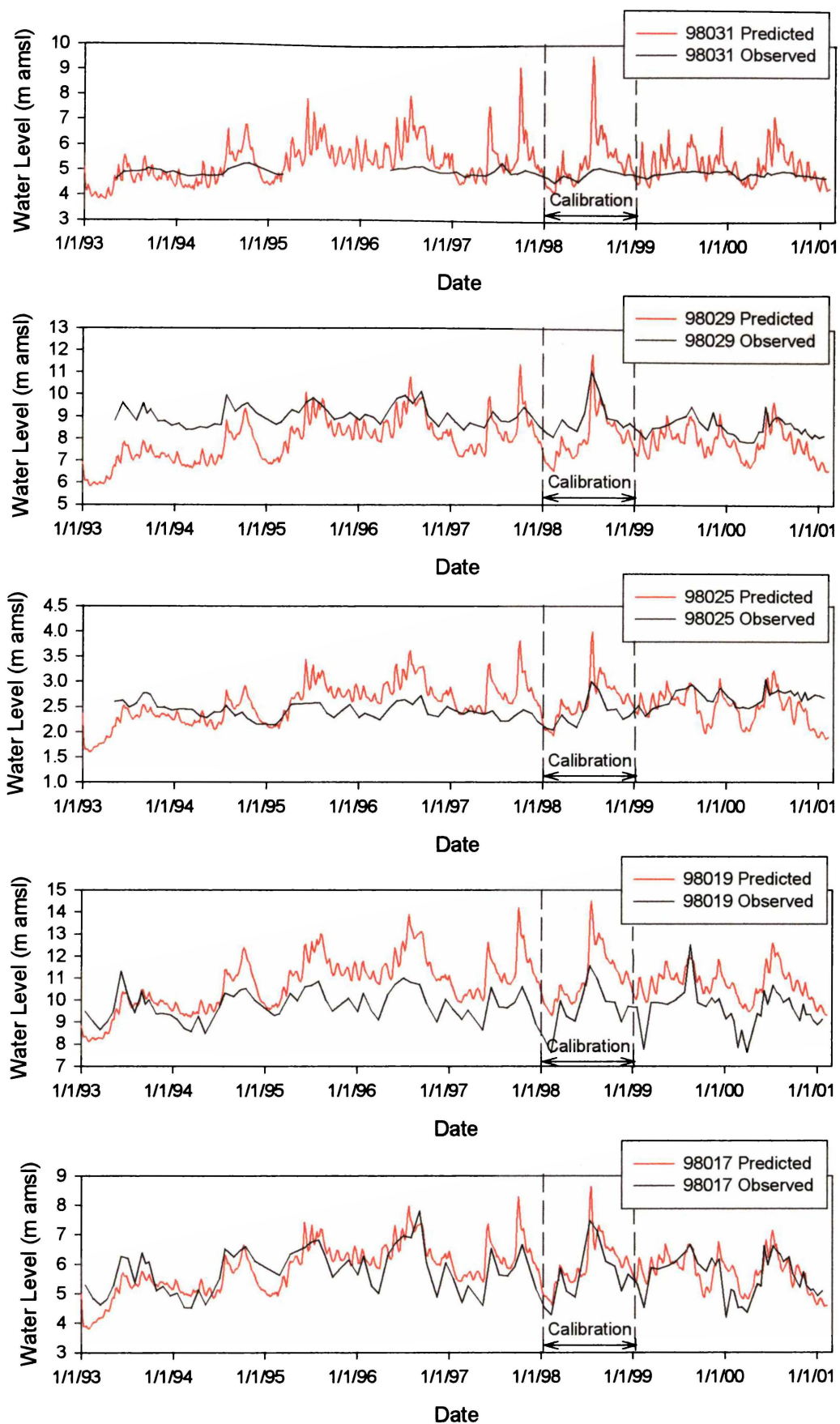


Figure 3.8. (continued).

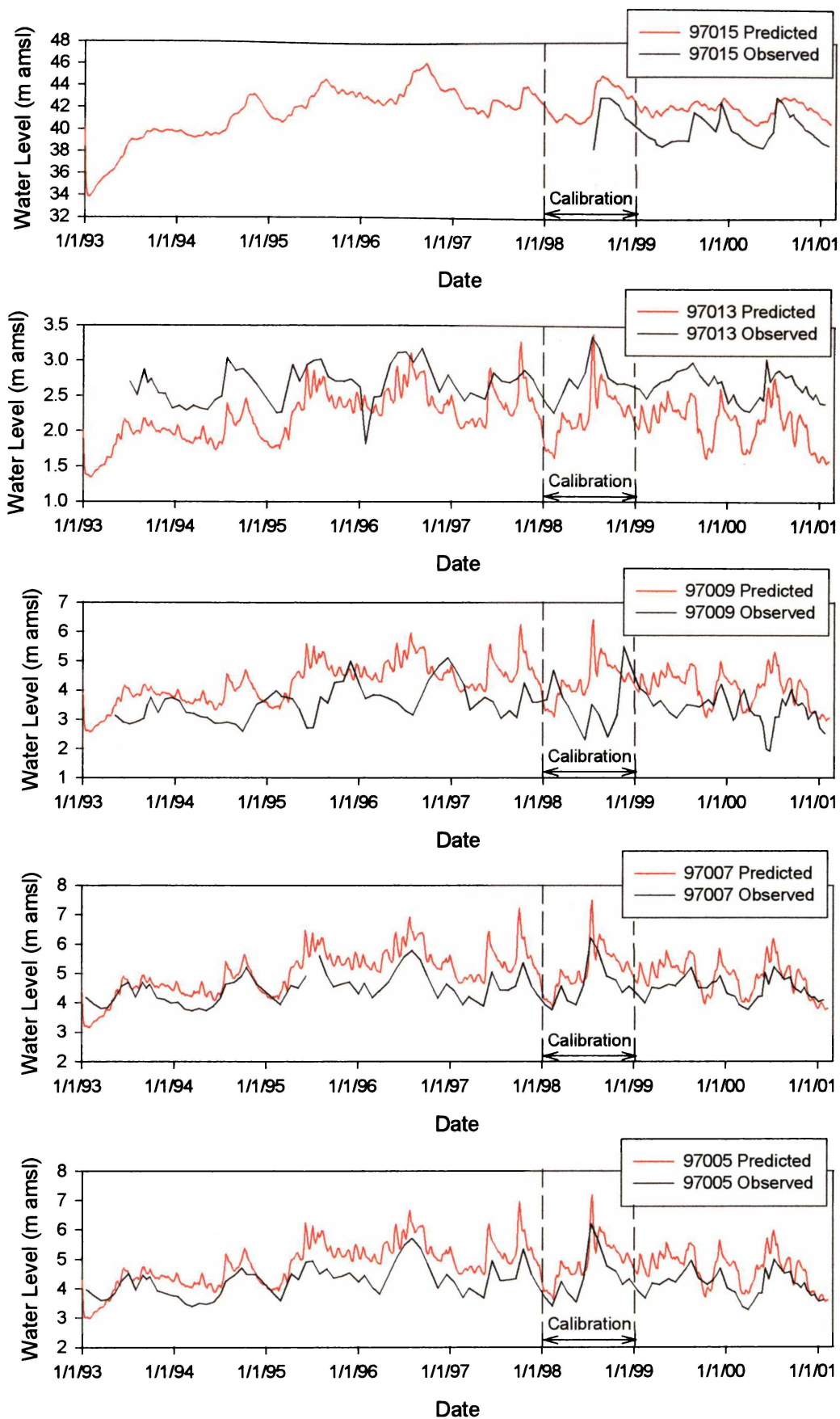


Figure 3.8. (continued).

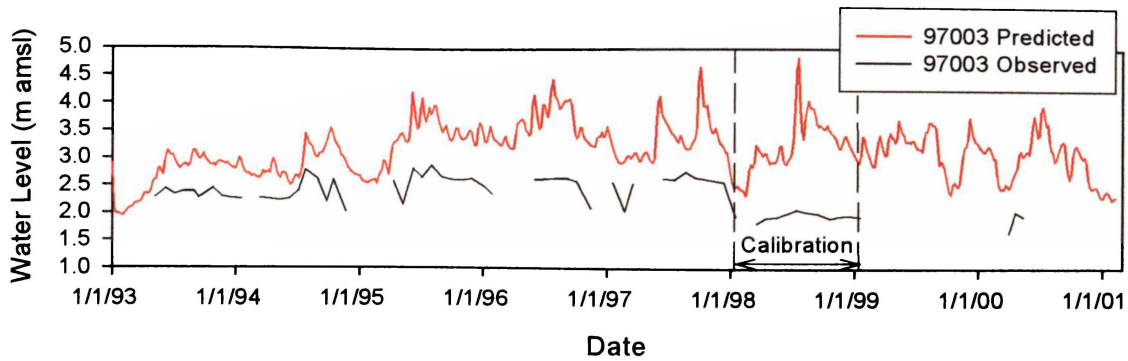


Figure 3.8. (continued).

3.5 Simulation of constructed groundwater systems for increasing the Onehunga – Mt Wellington Aquifer yield

Five scenarios for increasing the yield from the Onehunga – Mt Wellington aquifer are evaluated using the calibrated model. These are; (1) Aquifer Storage and Recovery (ASR) using the existing Watercare wells, and lowering of Watercare wells' intake screens (2) seasonal injection and extraction of reservoir water via ASR wells, (3) seasonal recharging of reservoir water via soakage wells, (4) recharging groundwater from the Three Kings quarry via soakage wells, and (5) recharging tertiary treated wastewater in the Mt Smart area via soakage wells. Figure 3.9 shows the location of the constructed groundwater systems proposed for increasing the Onehunga – Mt Wellington aquifer's yield. The Onehunga – Mt Wellington aquifer is suitable for receiving artificial recharge due to the relatively high infiltration capacity, specific yield, and transmissivity of the geology. Also, large areas of the aquifer are either unsaturated for the full thickness of the basalt, or have a significant thickness of unsaturated rock above the water table. The suitability of the Onehunga – Mt Wellington aquifer for artificial recharge is discussed in more detail in Chapter 2.

The economic analysis of the five constructed groundwater systems is discussed in section 3.6.

3.5.1 Recharge Water Sources

In the Auckland region there are three main sources of water readily available for artificially recharging the Onehunga – Mt Wellington aquifer. Presently these sources are not utilised and are going to waste.

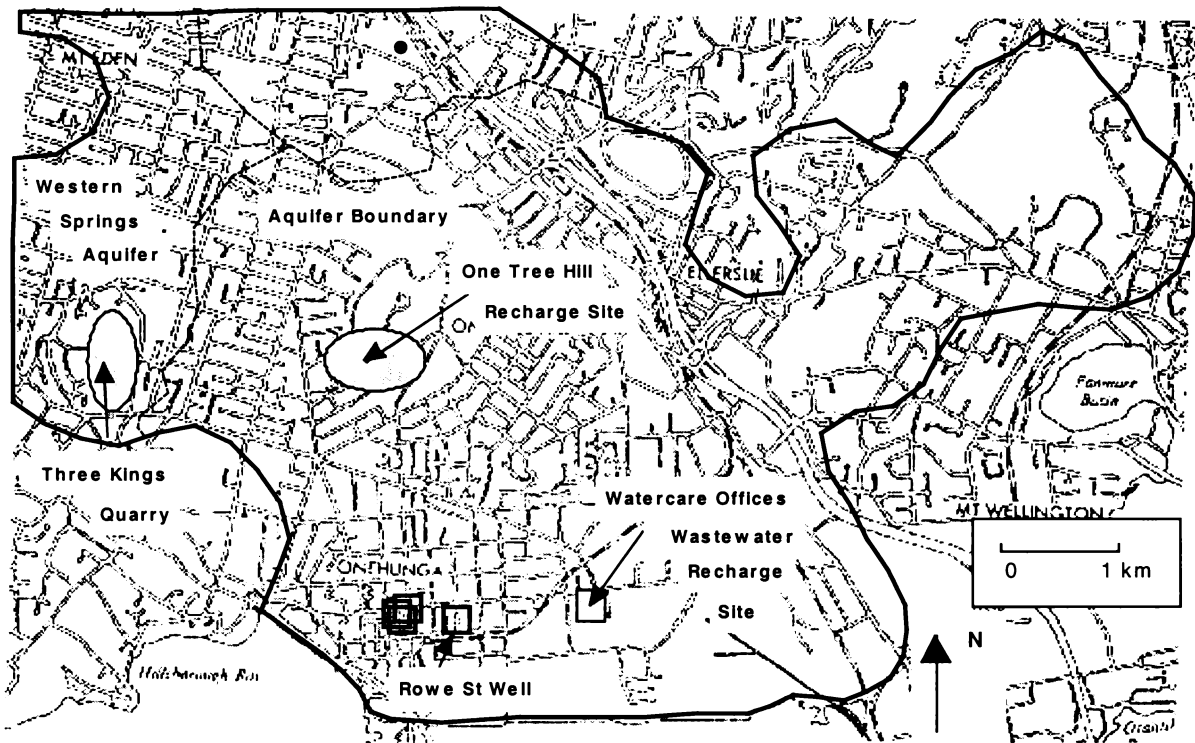


Figure 3.9. Aquifer area and locations of simulated constructed groundwater systems.

a) Reservoir Spillage

One source of recharge water is from Watercare's 10 storage reservoirs in the Hunua and Waitakere Ranges. The average total spillage during winter and spring (July to December) for the years 1997 to 2000 was 28,892,856 m³, or an average daily discharge of 15,788 m³d⁻¹ per reservoir, this is detailed in Appendix A. The minimum volume of spillage during winter and spring between 1997 and 2000 was recorded in 1999. The average daily spillage during winter and spring in 1999 was 6,048 m³d⁻¹ per reservoir. Figure 3.10 shows the combined daily reservoir spillage of Watercare's 10 reservoirs.

To reduce this spillage during winter, reservoir water could be recharged into the Onehunga – Mt Wellington aquifer for storage until summer. The water could be accessed directly from the supply pipes or the Three Kings and One Tree Hill storage tanks. Water sourced from

the supply pipelines and storage tanks may also have enough head pressure to force the water into the aquifer via the soakage wells and ASR wells. Reservoir water accessed from the supply pipes and storage tanks will have been treated to potable quality, which involves adding fluoride to the water. This may limit the use of the treated water for recharging the aquifer, as the residents currently using the Onehunga groundwater supply require through personal choice water with no added fluoride.

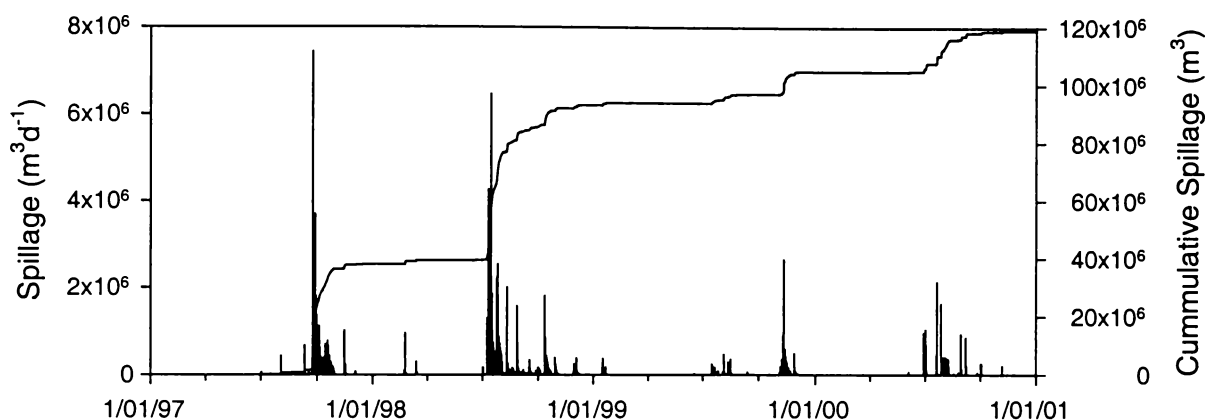


Figure 3.10. Daily spillage from Watercare's 10 storage reservoirs

b) Wastewater

Tertiary treated wastewater will be available from the Mangere Treatment plant in October 2003 after the completion of an upgrade. The wastewater discharge will typically exceed $290,000 \text{ m}^3\text{d}^{-1}$ (Watercare Services Limited, 2001), and the expected quality standards after the upgrade are listed in Table 3.2. The majority of the wastewater will be discharged directly from the treatment plant into the Manukau Harbour, with a small volume being used by nearby industries. A 5.5 km supply pipeline would be required from the plant to the Onehunga – Mt Wellington aquifer. The treated wastewater would be available 24 hours a day throughout the year.

The main difficulties in implementing artificial recharge with wastewater will be appeasing the public perception towards drinking recycled wastewater, and concerns about contaminating the aquifer. Treated wastewater also has the potential of clogging the aquifer by remnant suspended sediments and through bacterial activity. This is less likely however, in fractured rocks due to the larger pore spaces (see section 1.3.1.1).

Table 3.2. Future discharge standards for the Mangere Wastewater Treatment Plant from Watercare Services Limited (2001).

Parameter (Monthly mean g/m ³)	Future Standard (Effective 1 October 2003)
BOD	15
Suspended Solids	15
Total Nitrogen	9.5 Summer 35 Winter
Total Ammonia	3 Summer 5 Winter
Total Phosphorous	9
Faecal Coliforms (monthly median)	80/100 ml

c) Extracted Groundwater

Groundwater is available from the dewatering of the Three Kings Quarry in the neighbouring Western Springs aquifer (Figure 3.9). The quarry is situated within a basalt formation extending approximately 20 m bmsl to the relatively impermeable Waitemata Group (Carrier & Associates Limited, 1994). A large volume of water possibly exceeding $10 \times 10^6 \text{ m}^3$ is stored beneath the quarry (Carrier & Associates Limited, 1994; Groundsearch EES Limited, 1994). Quarrying will lower the water table from 55 m amsl down to sea level. At present the extracted water is being diverted into the stormwater system and thus lost from the aquifer system. One option, which had been considered, is for Metrowater (another Auckland water supply authority) to set up a filter station near the Three Kings quarry and supply potable water to the surrounding area. Even though the groundwater will meet the New Zealand drinking standards after treatment (Auckland Regional Council, 1994b), public pressure due to concerns over the perceived water quality have resulted in this plan being deferred.

To save on the expense of developing a new filter station at the Three Kings quarry as required by Metrowater, the extracted groundwater could be recharged into the Onehunga – Mt Wellington aquifer. A 2 km supply pipeline of 0.3 m diameter would be required between the quarry and One Tree Hill in the Onehunga – Mt Wellington aquifer. The ground surface elevation at the quarry and One Tree Hill are both 80 m amsl. The groundwater extraction pumps for the quarry dewatering may be capable of pumping the water through the pipeline. Otherwise a booster pump may be required. The Three Kings groundwater would be diluted by the existing groundwater, and the existing filter station at Rowe Street could be used for

water treatment. The average daily supply of groundwater from the quarry will be 7,500 m³ for approximately the first 3.5 years until the water table reaches sea level, after this the extraction will be adjusted to maintain this level (Carryer & Associates Limited, 1994). An average pump rate of 1500 m³d⁻¹ will be required to maintain the lowered water level based on the estimated natural recharge to the quarry.

3.5.2 Preparation for Numerically Modelling Onehunga – Mt Wellington Aquifer's Constructed Groundwater Systems

To simulate 8 years of operating the constructed groundwater systems in options 1, 2a, 2b, and 2c discussed in the following sections, the model was run using 1998 calibration data for the first year, and 1999 validation data cycled for the following 7 years. For option 3 the model was run for 20 years using average 1998 and 1999 data. For all the options the model was initially run without the simulated constructed groundwater system to provide a control data set of the 'normal' water levels. This was to enable comparisons of the change in the aquifer flow field due to the simulated constructed groundwater systems. For options 2b, 2c, and 3, the increase in yield from the Watercare wells due to the simulated constructed groundwater systems was calculated as the increase in extraction rate required to return the wells' water levels to the normal level of the control data set.

For option 1 the model was run using density dependent flow, with the saltwater of the Manukau Harbour having a density of 1025 kg/m³ and the groundwater 1000 kg/m³. Based on work in the Onehunga – Mt Wellington aquifer by Watson (1995) and Namjou (1996) the longitudinal mechanical dispersion was set at 15 m and transverse dispersion was set at 0.75 m. Molecular dispersion was not modelled and was assumed to be minimal compared to mechanical dispersion and the advective transport process (Domenico and Schwartz, 1998). Only the advection transport mechanism was simulated for options 2a, 2b, 2c, and 3, where the recharged reservoir water and groundwater were assumed to have similar densities and chemical composition to the existing groundwater.

A 25 kg/m³ concentration tracer was applied to the Manukau Harbour specified head boundary conditions in option 1, and to the artificially recharged water in options 2a, 2b, 2c, and 3. The tracer in each option enabled monitoring of the mixing between the 'foreign' water and the 'native' groundwater of the aquifer.

3.5.3 Option 1: Redevelopment of Watercare’s Rowe and Pearce Street Wells

Watercare’s Rowe and Pearce Street wells are located at the southern end of the aquifer. The annual yield from both of these wells is less than half of the maximum allowable extraction rate. This is due to low water levels during summer restricting the available yield, while in winter the water demand reduces thus requiring a reduced yield. Two methods of increasing the yield from the Rowe and Pearce Street wells are evaluated in option 1. These are; using of the Rowe and Pearce Street wells for aquifer storage and recovery (ASR), and the effect of lowering the wells’ intake screens to increase their yield. The Upper and Lower Municipal wells are not included in this option due to their relatively small physical size and pumping capacity.

The Rowe and Pearce Street wells are unsuitable for aquifer storage and recovery (ASR) as a method of increasing the yield from the aquifer. There is only 2.8 m of unsaturated rock between the average water table level and the ground surface, and both these wells extract water from large caverns, which are open to the ground surface. Thus injection of water will most likely result in surface flooding. Surface flooding due to a rise in aquifer water table levels above the ground surface has already been recognised as a problem in the Onehunga Township. The most recent flooding to date was after heavy rainfall during June and July 1998 (Figure 3.11). Artesian groundwater flow continued for many days after the rainfall events had finished. A major contribution to the flooding was the high infiltration rate of the aquifer allowing large volumes of the rainfall to be recharged, combined with the reclamation of the Manukau Harbour foreshore restricting the groundwater discharge into the harbour.



Figure 3.11. Groundwater flooding in the Onehunga township after extensive rainfall during June and July 1998 (Photograph from ARC Regionwide newspaper August 1998).

A more feasible option to increase the yield from the Rowe and Pearce Street wells is to lower their intake screens. Due to the configuration of the intake screens the two wells cannot extract water when the water table is below 0.7 m amsl, even though the basalt aquifer extends approximately 10 m bmsl. The required redevelopment of the wells should be minimal. The drilled wells currently extend below the bottom of the intake screens. Thus only the intake screens need to be lowered.

There are two main limitations to the effectiveness of lowering the wells' intake screens. The first is from saline contamination of the aquifer from the Manukau Harbour. As discussed in section 2.4.1, the old municipal supply well near the existing Rowe Street well was contaminated by salt water in the 1950's before the reclamation of the foreshore. The second limitation is the linear reduction in well yield with aquifer depth (Pattle Delamore Partners Limited, 1994). The change in water level per unit discharge is five times greater for a water table depth of 0.7 m amsl compared to 2.6 m amsl. This is discussed in more detail in Appendix G.

A simulation was performed over an eight-year period to evaluate the possible saline intrusion from the Manukau Harbour if the Rowe and Pearce Street wells' water tables were maintained at mean sea level. The water table level at the wells was simulated using specified head boundary conditions, and the aquifer recharge was reduced by 50% to simulate a severe drought.

The simulation showed that after eight years of maintaining the Rowe and Pearce Street wells at sea level there would be no movement of the saline water from the foreshore into the aquifer.

The increase in well yield was estimated using a specific discharge of $4.0 \times 10^{-4} \text{ m}^3\text{d}^{-1}$. This value is based on the estimated linear reduction in specific discharge for the Rowe Street well, discussed further in Appendix G. Lowering the intake screens from 0.7 m to sea level equates to an increase in yield of $1,750 \text{ m}^3\text{d}^{-1}$ for the Rowe Street well, which is an increase of approximately 25% on the average summer yield of $7,000 \text{ m}^3\text{d}^{-1}$ for 1998 and 1999. The specific drawdown is not known for the Pearce Street well, using the same value as for the Rowe Street well the increase in discharge would again be $1,750 \text{ m}^3\text{d}^{-1}$, which is an 85% increase on the average summer yield of $2,100 \text{ m}^3\text{d}^{-1}$ for 1998 and 1999. For both wells this equates to a 40% increase in summer yield. However, this is highly dependent on the specific discharge of the aquifer between 0.7 m amsl and sea level.

3.5.4 Option 2a: Seasonal Recharge and Extraction of Reservoir Water Via ASR Wells at One Tree Hill

Options 2a, 2b and 2c all utilise the One Tree Hill scoria cone as a hypothetical artificial recharge site (Figure 3.9 and Figure 3.12). The benefits of One Tree Hill are: high infiltration rates; large depth of unsaturated rock; two existing underground water storage tanks for water supply; and open terrain allowing easy access for drilling equipment. The nearest available geological drill-hole logs are for ARC5683 (600 m to the west), and OBC3 and 4 (600 m to the south). Drill-hole logs are in Appendix B. These logs show highly fractured vesicular basalt and scoria for most of the aquifer depth with some 2 m thick layers of solid basalt. There is at least 35 m of unsaturated rock above the water table.

The closest pump test near One Tree Hill was at Greenlane hospital, 1,000 m to the north. The estimated transmissivity obtained was $480 \text{ m}^2\text{d}^{-1}$, with a saturated thickness of 40 m. Water table observations during the well construction showed a quick response to rainfall events (OPUS, 1998). This implies that recharge to the aquifer was transmitted quickly through the 45 m of unsaturated basalt and scoria at the hospital.



Figure 3.12. One Tree Hill scoria cone, and potential site for artificial aquifer recharge. View is to the north. (Photo: L. Homer).

Option 2a involves the seasonal aquifer storage and recovery (ASR) of reservoir water using wells. The reservoir water is injected during winter and extracted during summer.

In the numerical model simulation of option 2a the reservoir water is recharged and extracted using aquifer storage and recovery (ASR) wells located at 20 m amsl, at the interface of the basalt and Waitemata geology. A trial and error process was used to identify the optimum volume of water to inject and extract. It was found that injecting more than 35,000 m³d⁻¹ resulted in the migration of the recharge water plume into the neighbouring Western Springs aquifer, and an extraction rate greater than 35,000 m³d⁻¹ resulted in the wells becoming dry. Based on this the results discussed below are for the injection and extraction of 35,000 m³d⁻¹. The water was injected and extracted on a 6 monthly cycle

Simulated water table levels at the ASR site during the eight-year simulation show the peaks and troughs of the ASR cycles remaining relatively constant (Figure 3.13). The water table increased by approximately 8 m during injection and declined to 8 m below normal during extraction. The average water level increase at the Rowe Street well was 0.4 m during the injection period and declined to 0.4 m below normal during the extraction period. There was a 30-day delay in water level response between the recharge site and the Rowe Street well. The spatial extent of the change in water table levels covers a large area of the model domain. However, there was minimal change in the water levels in the Western Springs aquifer due to the relatively low permeability of the Waitemata Group rocks dividing the two aquifers (Figure 3.14).

The recovery capability of the aquifer was evaluated by stopping the ASR at the end of an extraction period after four years of operation. Once stopped the water table at the recharge site returned to normal within 1.5 years (Figure 3.13). This indicates that the ASR operation was sustainable when operating the seasonal injection and extraction of 35,000 m³d⁻¹.

The simulation results in Figure 3.14 show that the main concentration of the recharge plume remained centered on the ASR site during the simulation. Some spreading of the plume occurred in a southerly direction along the paleo valley towards the Watercare wells and Manukau Harbour. The travel time for the plume to reach the Watercare wells was 180 days. The radius of the injection plume (designated by tracer concentrations greater than 0.2 kg/m³) at the end of the first injection period was 850 m, and increased to 1300 m after the eighth injection period. After each extraction period the radius of the plume remained the same as for the end of the previous injection period. The difference was caused by the concentrations at the centre of the plume being considerably lower following the extraction

period (Figure 3.14). This implies that not all the injected water was being extracted by the wells, resulting in the increased mixing of the injected water. As there is no significant difference in the quality of the groundwater and the injected reservoir water, the degree of mixing is not a concern (Pyne, 1994).

During the summer months, the ASR scheme increased the average 1998 and 1999 aquifer yield from 9,100 m³d⁻¹ to 44,100 m³d⁻¹. The artificial aquifer recharge during winter in turn also reduces reservoir spillage.

After extraction, the water could be piped under gravity flow to the existing Rowe Street filter station, which has a maximum capacity of 43,000 m³d⁻¹. The required pipeline length is 2.6 km and at least 0.36 m in diameter (Chadwick and Morfett, 1991). The number of ASR wells required is highly dependent on the pumps' capacity, diameter of the wells, and the transmissivity and specific yield of the aquifer. The estimated maximum extraction rate of each ASR well is 7500 m³d⁻¹. This is based on the pumping capacity of a 250 mm diameter production well at Three Kings Quarry required to lift water from 80 m below ground level (Groundsearch EES Limited, 1994; Brown, 2001 *pers comm*). This rate does not take into account possible well clogging. However, this should be minimal due to the highly fractured and scoriaceous geology, combined with the highly treated recharge water.

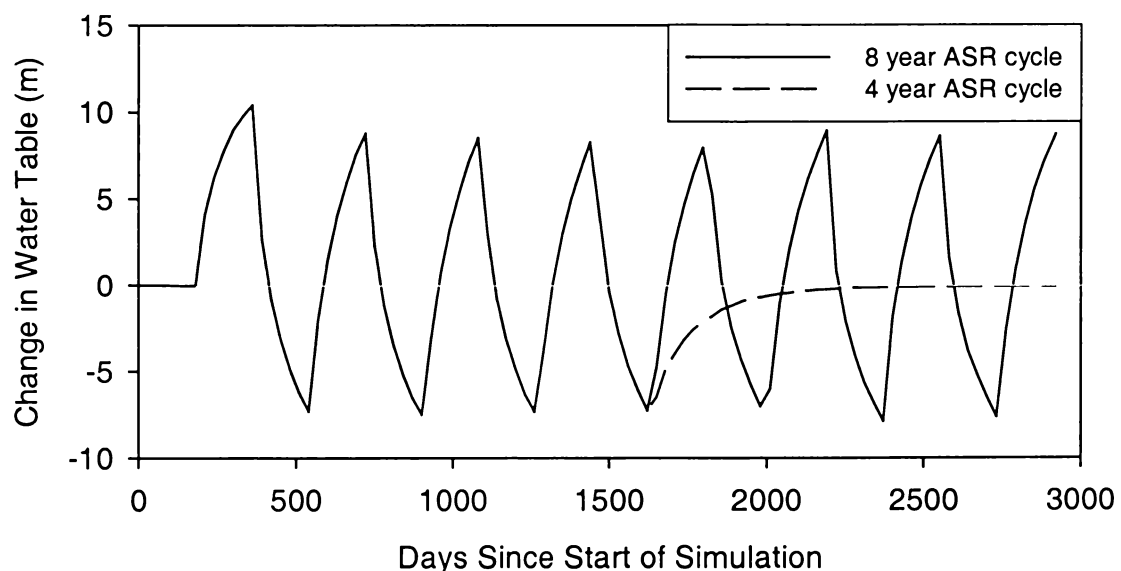
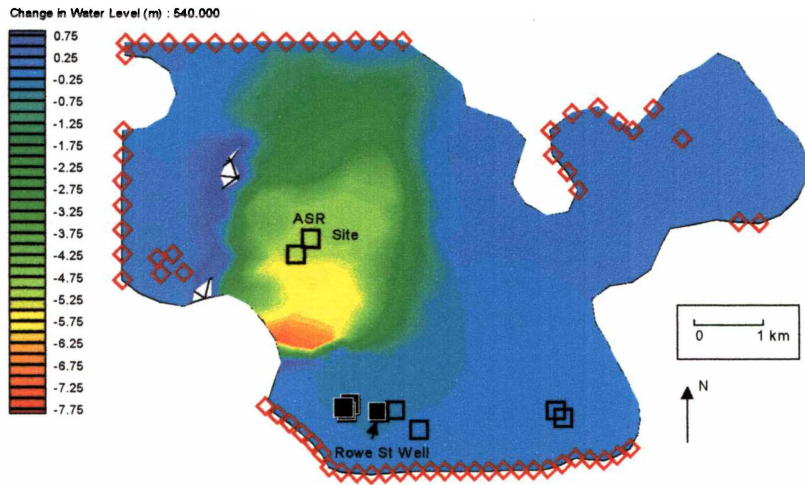
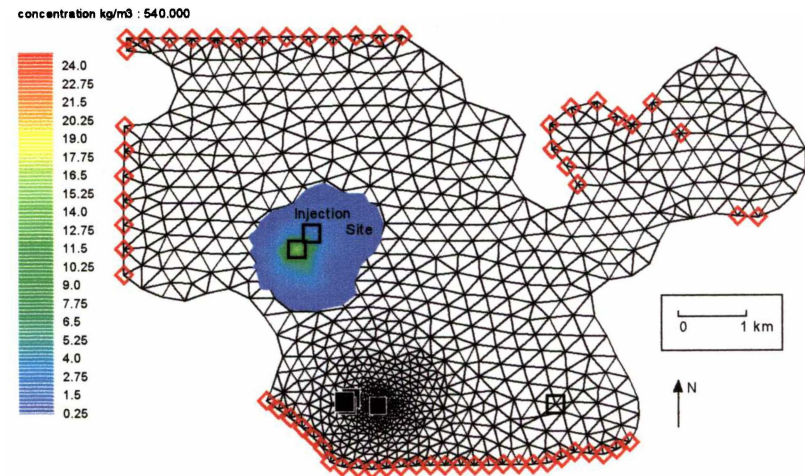


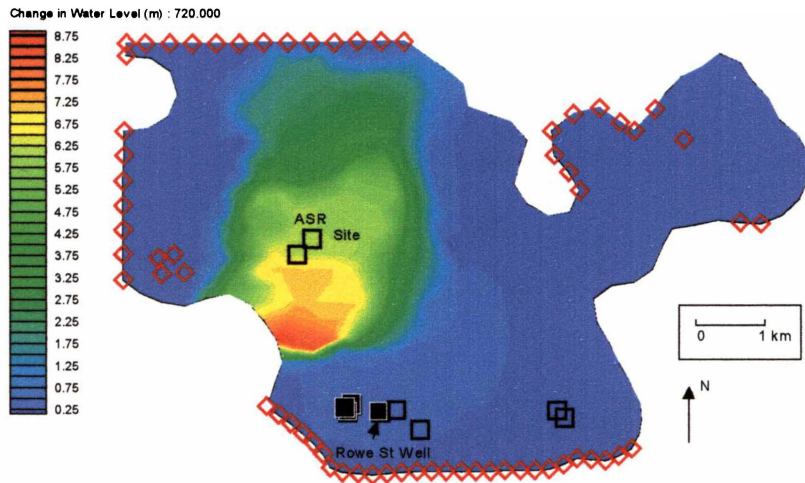
Figure 3.13. Time series of simulated change in water table levels at the ASR site. Eight-year ASR cycle – continuous cycle of six monthly injection and extraction. Four-year ASR cycle - cycle of six monthly injection and extraction for the first four years only.



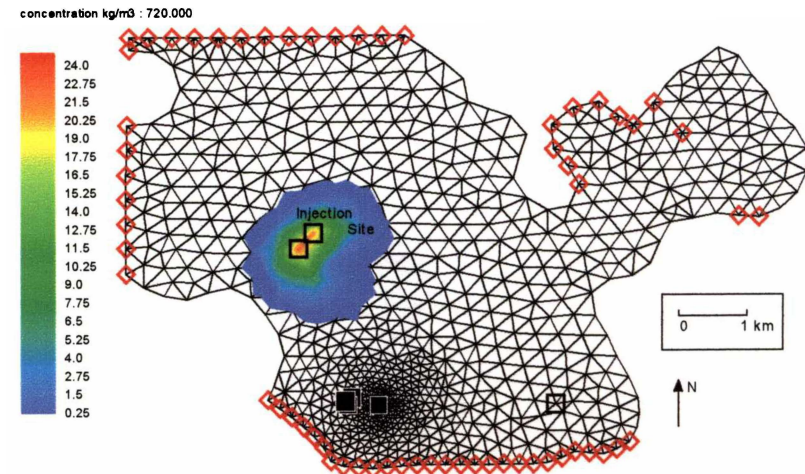
a) Day 540, change in water levels prior to second recharge period.



b) Day 540, tracer concentrations prior to second recharge period.

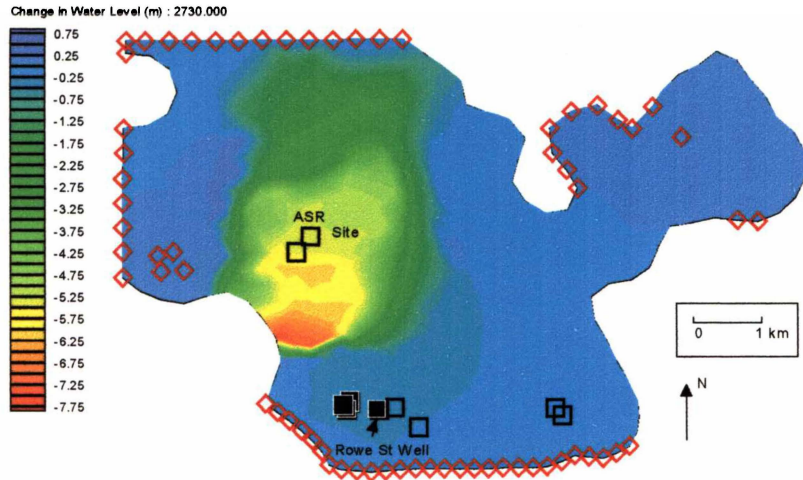


c) Day 720, change in water levels at end of second recharge period.

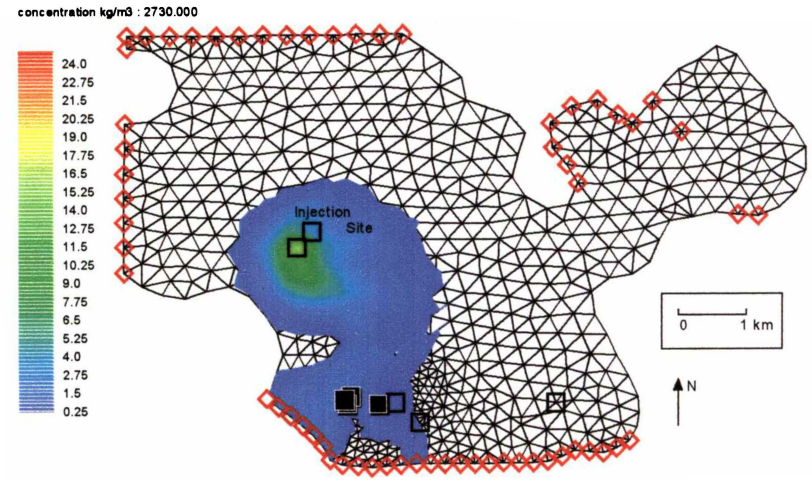


d) Day 720, tracer concentrations at end of second recharge period.

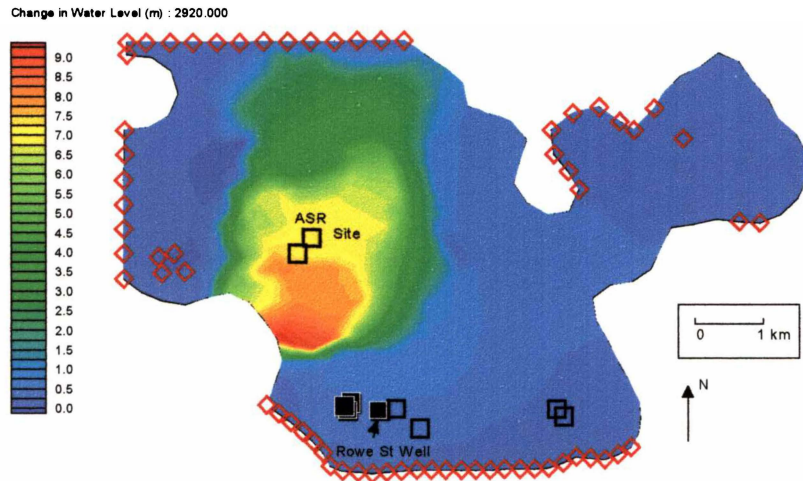
Figure 3.14. Simulated spatial extent of the change in water levels and the degree of mixing of the recharge water and the groundwater at specified time steps for option 2a. The lower limit of concentration contours is 0.05 kg/m^3 , initial concentration was 25 kg/m^3 . The black squares represent Watercare wells, unfilled squares represent other users.



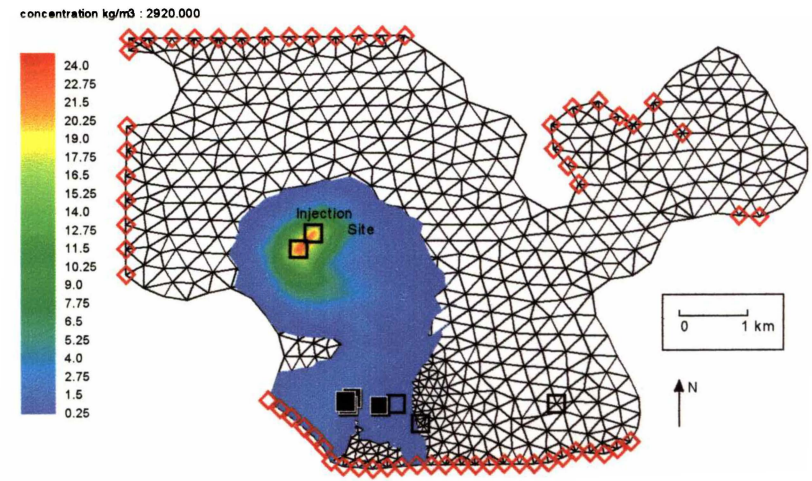
e) Day 2,730, change in water levels prior to eighth recharge period.



f) Day 2,730, tracer concentrations prior to eighth recharge period.



g) Day 2,920, change in water levels at end of eighth recharge period.



h) Day 2,920, tracer concentrations at end of eighth recharge period.

Figure 3.14. (continued).

3.5.5 Option 2b: Seasonal recharge of Reservoir Water Via Soakage Wells at One Tree Hill

An alternative seasonal recharge method to the ASR wells in option 2a (discussed in section 3.5.4) is the use of soakage wells for recharging the aquifer with reservoir water. The recharge location is again at the base of the One Tree Hill scoria cone (Figure 3.12), and the artificial recharge of 35,000 m³d⁻¹ of treated reservoir water is applied during the winter months (July to December). The recharged reservoir water can flow with the groundwater as it travels along the paleo Waitemata valley towards the existing Watercare wells where it is extracted.

An average infiltration rate of 15 L/s (1300 m³d⁻¹) was measured by the Auckland City Council for 43 existing stormwater soakage wells drilled in fractured basalt, cavities, and scoria in the Onehunga – Mt Wellington aquifer. Details of the soakage well tests are in Appendix C. The infiltration tests were typically 30 minutes in duration and the wells were generally 100 to 150 mm in diameter with an average depth of 13 m. Based on the infiltration tests, at least 27 equivalent soakage wells would be required to recharge 35,000 m³d⁻¹ of reservoir water.

Another estimate of the soakage well's recharge rate Q can be calculated using Equation 3.1 developed by Zanger for the reverse auger-hole pump test when the aquifer conductivity K is known (Bouwer, 1978). The water depth in the well L_w should not be less than 10 times the hole radius r_w and where the depth s_i is greater than $2L_w$ (Figure 3.15). If the depth s_i is less than $2L_w$ then Equation 3.2 can be used.

$$Q = \frac{2\pi K L_w^2}{\ln(2L_w/r_w) - 1}$$

Equation 3.1

$$Q = \frac{K\pi L_w(3L_w + 2s_i)}{3\ln(L_w/r_w)}$$

Equation 3.2

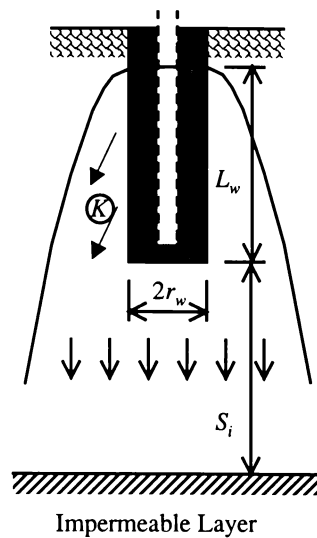


Figure 3.15. Infiltration well with sand or gravel fill and perforated supply pipe. Adapted from Bouwer (1996b).

Applying Equation 3.1 for a 125 mm diameter well, with a 10 m water column, and an aquifer conductivity of 12 md^{-1} (measured at Greenlane Hospital) a single soakage well's recharge capacity is $1,580 \text{ m}^3\text{d}^{-1}$, and would require 22 soakage wells to recharge $35,000 \text{ m}^3\text{d}^{-1}$. To account for the likely situation of the installed soakage wells having lower infiltration rates more soakage wells may be required. Larger diameter soakage wells have the potential to intercept more fractures in the aquifer geology and increase the soakage well's recharge capacity. However, the smaller diameter soakage wells were evaluated to provide a conservative estimate of the potential recharge capacity of soakage wells for artificially recharging the Onehunga – Mt Wellington aquifer.

In the simulation, the infiltration wells were represented with specified flux boundaries applied to the surface of two elements at the base of One Tree Hill. The area of the two elements was $71,604 \text{ m}^2$, which equated to a flux of 0.49 md^{-1} to recharge a total of $35,000 \text{ m}^3\text{d}^{-1}$.

Figure 3.16 shows the simulated change in water table levels at the soakage wells and Rowe Street well due to the artificial recharge from the soakage wells. As expected, the water table level increases during the recharge periods and declines during the non-recharge periods. There is a 75-day delay in the water level response at the Rowe Street well from the recharge at the soakage wells, which is more than twice as long as for the ASR wells in option 2a. The water table below the soakage wells increased on average at the end of each 6 month recharge period by 10.3 m and declined to 2.2 m above normal at the end of each 6 month period of no recharge. The water table level at the recharge site after each 6 month period is approximately 2 m greater than for the ASR wells in option 2a. The water table

level at the Rowe Street well increased on average at the end of each 6 month recharge period by 0.65 m and declined to 0.2 m above normal at the end of each 6 month period of no recharge.

Figure 3.17 shows the spatial distribution of the tracer concentration of the recharged water and change in groundwater levels, at the beginning and end of the second recharge period, and also the eighth recharge period. The change in water levels covers a much larger area of the aquifer than the extent of the recharge water plume. The extent of water level increase on an annual basis remains relatively constant, with little difference between Figure 3.17 a & e, and Figure 3.17 c & g. In contrast to the mixing extent of the recharge water, which increased constantly throughout the simulation due to increased mixing with the groundwater (Figure 3.17 b, d, f, & h). The arrival time of the recharge water plume to the Rowe Street well was 388 days after the start of recharging. However, an initial rise of 0.1 m in the Rowe Street water level occurred after only 75 days. The concentration plume in Figure 3.17 shows the flow path of the recharged water along the paleo valley towards the Watercare wells. Mixing of the reservoir water and the groundwater resulted in a maximum concentration at the Rowe Street well of 4.5 kg/m^3 , or 18% of the initial concentration, 25 kg/m^3 . The recharge plume in this option takes twice as long to reach the Rowe Street well, and results in more mixing between the recharged water and the groundwater than for option 2a using the ASR wells.

The increased water levels in the Onehunga area of the aquifer may result in surface flooding, especially during a wet winter where the increase in natural and artificial recharge may result in a very high water table. However, due to the 75-day delay in response of the water levels in the vicinity of the Rowe Street wells, the peak occurs at the end of February when the aquifer water levels are declining, reducing the risk of flooding. There is also an increase in water levels in the neighbouring Western Springs aquifer. The specified head boundary conditions at the Three Kings quarry limits the model's ability to accurately show the increase in water levels in the vicinity of the quarry. Based on the change in water levels in the remainder of the Western Springs aquifer, the change in water levels at the Three Kings quarry was estimated to be less than 0.5 m.

The Waitemata ridge dividing the two aquifers forms a relatively impermeable boundary between the Western Springs aquifer and the recharge site. This is shown by the steep hydraulic gradient aligned north to south along the Waitemata ridgeline in Figure 3.17 a, c, e, & g.

Recharge via soakage wells increased the water levels in the vicinity of the Rowe Street well by only 0.2 m at the beginning of summer (January) and gradually increased to 0.6 m above normal at the end of autumn (June). The increase in water levels during summer equated to an increase in Watercare’s summer yield of only 10% or 700 m³d⁻¹ from the Rowe Street well, and approximately 20% or 300 m³d⁻¹ from the Pearce Street well. The method of recharge via soakage wells was very inefficient, with the combined increase in yield for the two Watercare wells equating to only three percent of the artificially recharged volume.

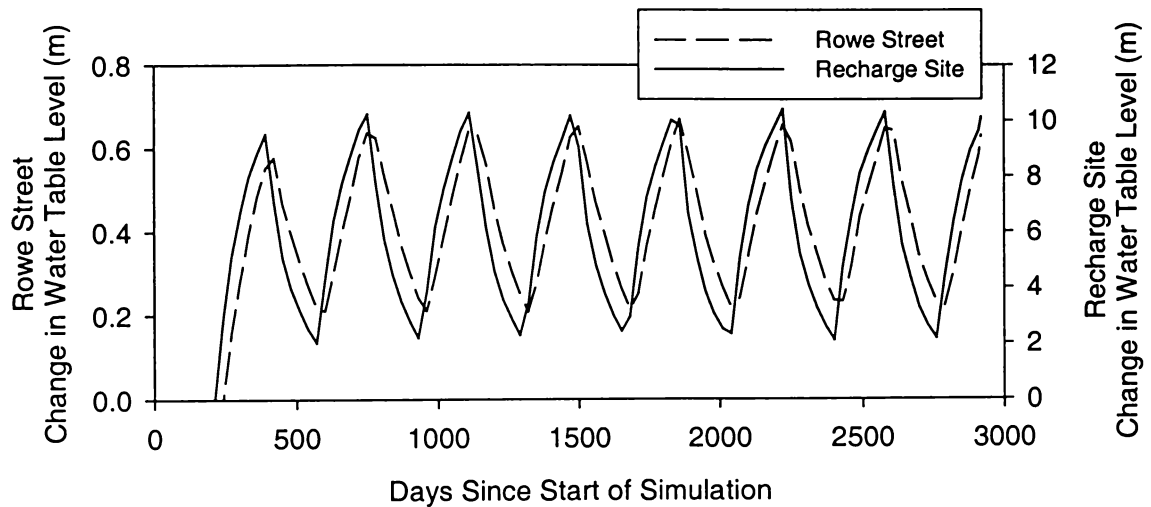
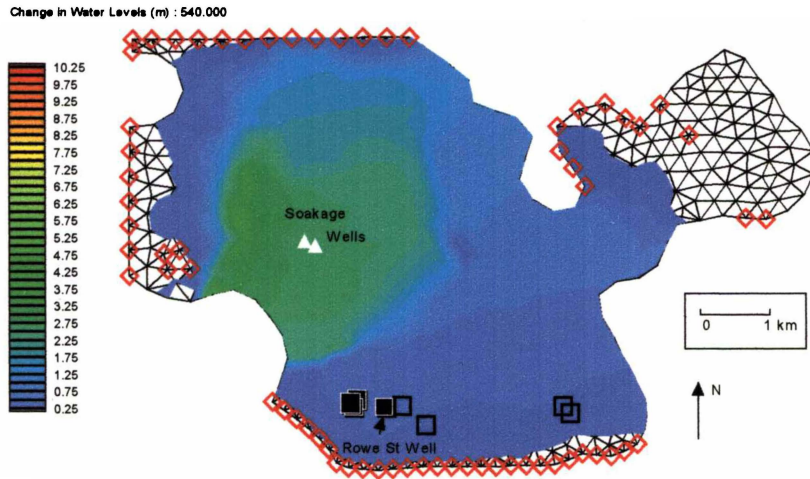
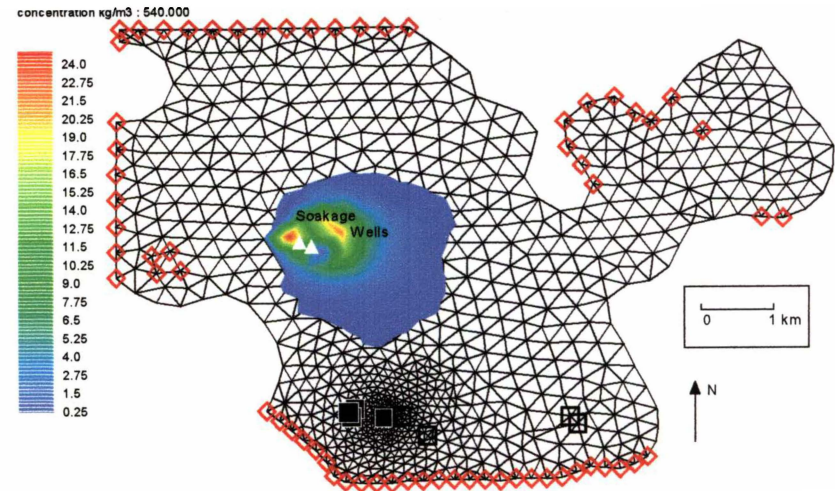


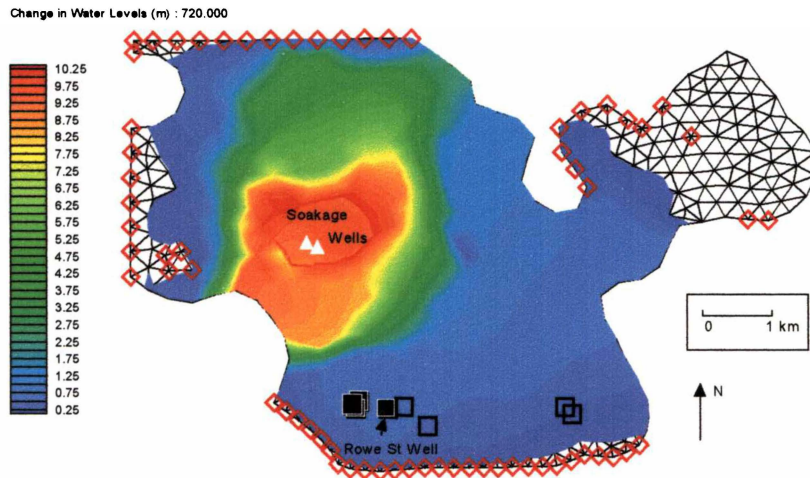
Figure 3.16. Eight-year time series of water table levels at the recharge site and Rowe Street well.



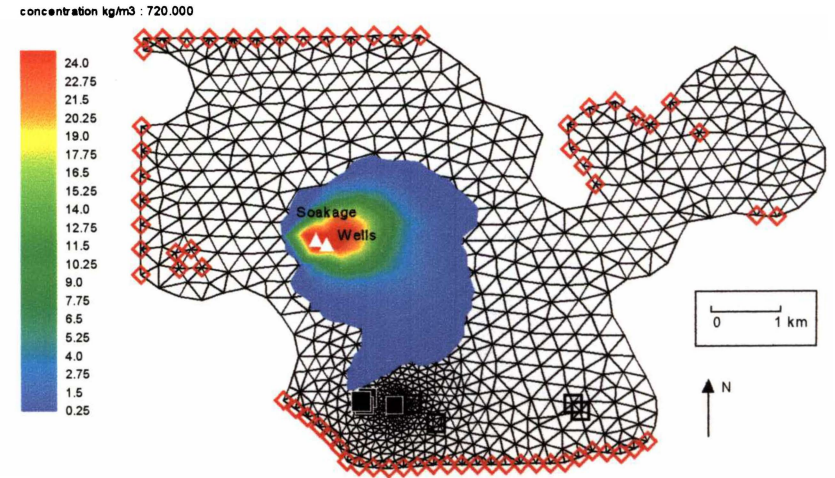
a) Day 540, change in water levels prior to second recharge period.



b) Day 540, tracer concentrations prior to second recharge period.

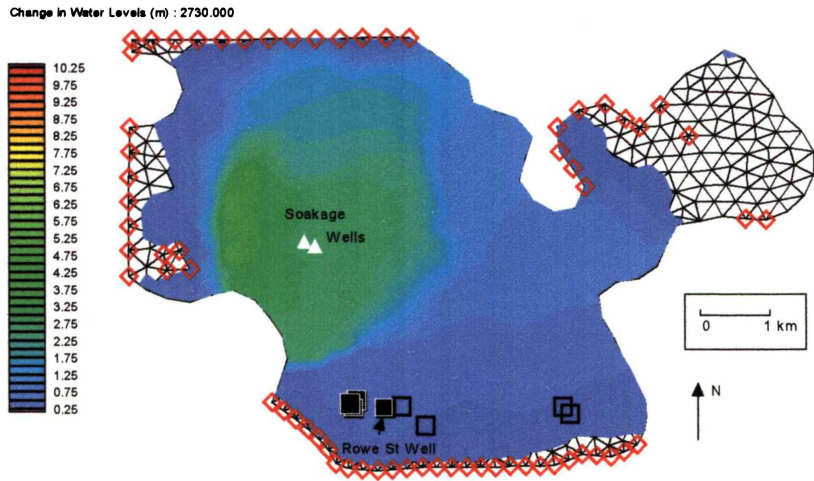


c) Day 720, change in water levels at end of second recharge period.

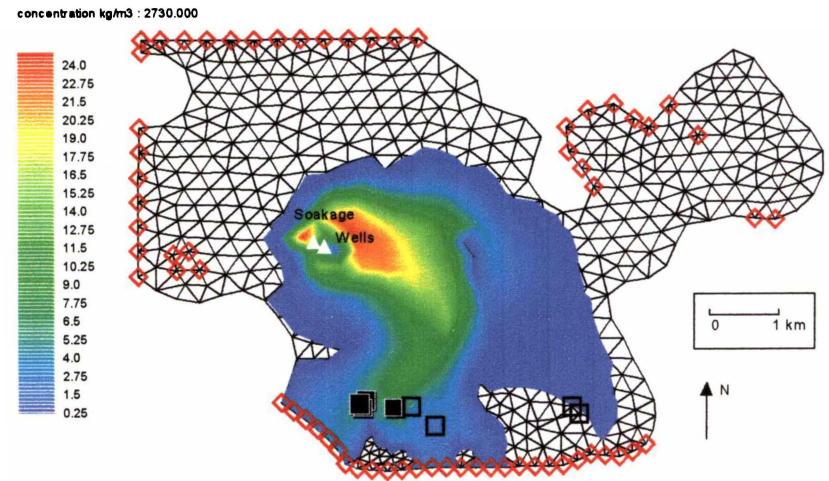


d) Day 720, tracer concentrations at end of second recharge period.

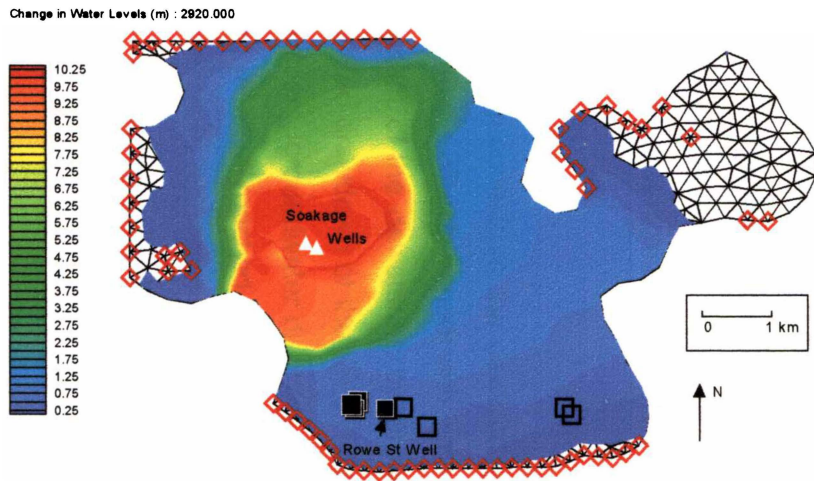
Figure 3.17. Simulated spatial extent of the change in water levels and the degree of mixing of the recharge water and the groundwater at specified time steps for option 2b. The lower limit of concentration contours is 0.05 kg/m^3 , initial concentration was 25 kg/m^3 . The black squares represent Watercare wells, unfilled squares represent other users.



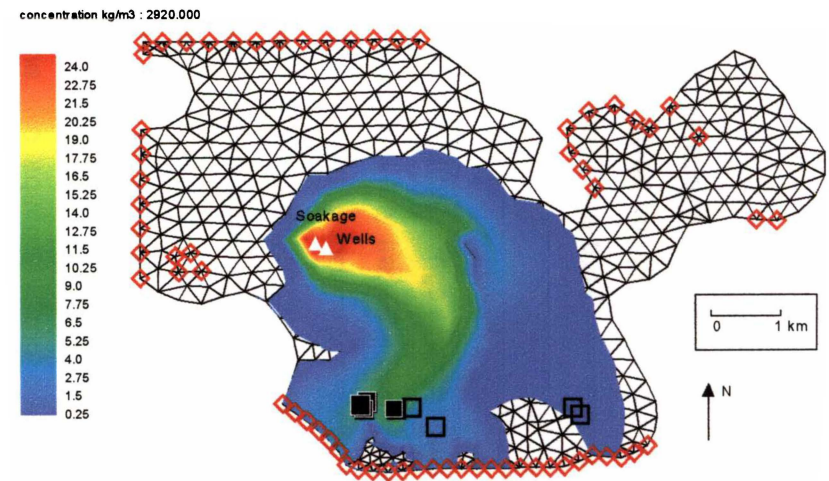
e) Day 2,730, change in water levels prior to eighth recharge period.



f) Day 2,730, tracer concentrations prior to eighth recharge period.



g) Day 2,920, change in water levels at end of eighth recharge period.



h) Day 2,920, tracer concentrations at end of eighth recharge period.

Figure 3.17 (continued).

3.5.6 Option 2c: Recharge of Three Kings Groundwater Via Soakage Wells at One Tree Hill

This option evaluates the benefits of artificially recharging water extracted from the Three Kings Quarry in the neighbouring Western Springs aquifer into the Onehunga – Mt Wellington Aquifer (Figure 3.9). The recharge is applied at the One Tree Hill site, which was also used for option 2a (section 3.5.4).

Winstone Aggregates Limited plan to excavate the Three Kings quarry down to sea level to remove the scoria. This will require dewatering the quarry from the current water table level of 57 m amsl. The quarry dewatering has been modelled independently for Winstone using the MODFLOW groundwater software (Carrier & Associates Limited, 1994; Groundsearch EES Limited, 1994). It was projected that four wells pumping at a combined rate of 7,500 m³d⁻¹ over a minimum of 3.5 years will reduce the water table to sea level (Groundsearch EES Limited, 1994; 1996). This modelling also indicated that there would be minimal changes in the water levels of the Onehunga – Mt Wellington aquifer due to the ridge of Waitemata Group rocks dividing the two aquifers (Groundsearch EES Limited, 1994).

Based on the average infiltration rate of 1,300 m³d⁻¹ for stormwater soakage wells identified in section 3.5.5 at least 6 soakage wells would be required to recharge the quarry water at a rate of 7,500 m³d⁻¹. Applying Equation 3.1 with the same well parameters identified in section 3.5.5, at least 5 soakage wells would be required to recharge 7,500 m³d⁻¹. To account for the likely situation of the installed wells having lower infiltration rates more wells may be required.

The simulated water table levels of the drawdown cone from the dewatering at the quarry were controlled by 4 specified head boundaries. These varied with time to match the groundwater levels simulated in the assessment of the dewatering by Groundsearch EES Limited (1996). Figure 3.18 shows the simulated drawdown of the water table at the Three Kings quarry. Up to day 1245 the predicted quarry groundwater discharge was 7500 m³d⁻¹, after this the quarry water level is predicted to have reached sea level and the pumping would be reduced to 1,500 m³d⁻¹ from day 1246 to day 2920 to maintain this level. The artificial recharge of the Three Kings water at One Tree Hill was simulated using specified flux boundary conditions applied to the top face of two model elements. The two elements

cover an area of 71,604 m², which relates to a flux of 0.105 md⁻¹ to recharge a total of 7,500 m³d⁻¹, and 0.21 md⁻¹ to recharge 1,500 m³d⁻¹.

Figure 3.19 shows the approximate change in water table levels at the soakage wells and Rowe Street well due to the artificial recharge. An initial water level rise of 0.1 m at the Rowe Street well occurred 90 days after the commencing of the recharge. This is 15 days longer than for the seasonal recharge via soakage wells of 35,000 m³d⁻¹, in option 2b. The arrival time of the plume of recharge water to the Rowe Street well was after 730 days, compared to 342 days in option 2b. The water table levels below the soakage wells increased by 5.5 m up to day 1245 when the recharge rate was 7,500 m³d⁻¹, and declined to 1.4 m above normal during the lower recharge rate of 1,500 m³d⁻¹. The water table at the Rowe Street well stayed constantly 0.4 m above normal for the duration of the higher recharge rate (up to day 1245). This is similar to the increase in water levels in option 2b (0.2 to 0.6 m) recharging 35,000 m³d⁻¹. After day 1245 the increase in water levels reduces to 0.1 m above normal for the lower recharge rate.

Figure 3.20 shows the spatial distribution of the increase in water levels and the extent of the recharge water plume after 1245 days recharging 7,500 m³d⁻¹, and on day 2920 with the lower recharge of 1,500 m³d⁻¹. The increase in water levels covers a larger area than the plume of artificially recharged water, however the main area of water level increase remained centred on the recharge site. The increase in water levels is not completely contained within the Onehunga- Mt Wellington aquifer, with a small increase in the water levels in the Western Springs aquifer. The specified head boundary conditions at the Three Kings quarry in the Western Springs aquifer limits the model's ability to accurately estimate the increase in water levels in the vicinity of the quarry. Based on the change in water levels in the remainder of the Western Springs aquifer, the change in water levels at the Three Kings quarry were estimated to be less than 0.20 m. Figure 3.20 also shows there was no direct recycling of the Three Kings groundwater, caused by it flowing from the recharge site back to the Three Kings quarry. The plume of recharged water flowed along the paleo drainage valleys towards the existing Onehunga wells, in a similar way as for option 2b.

The increase in water levels at the Watercare wells during summer due to the artificial recharge rate of 7,500 m³d⁻¹ equates to an increase in summer yield of only 5% or 350 m³d⁻¹ from the Rowe Street well, and approximately 10% or 150 m³d⁻¹ from the Pearce Street well. The combined increase in yield for the two Watercare wells equates to six percent of the artificially recharged volume. This increase was greater than for option 2b (3% of the recharged water), and indicates that a lower continuous recharge rate may be more effective

for increasing the discharge from the Watercare wells. The increase in water levels due to the lower artificial recharge rate of $1,500 \text{ m}^3\text{d}^{-1}$ was only 0.1 m for the Watercare wells and was not evaluated to estimate the possible increase in yield.

The use of ASR wells for recharging the Three Kings groundwater into the aquifer have not been simulated due to the public's expressed concerns over the quality of the water, discussed in section 3.5.1. Compared to ASR wells, soakage wells enable increased mixing of the recharged water with the native groundwater. This is important, as the public perception is that standard water treatment methods will not be able to treat the Three Kings groundwater to an acceptable level. Mixing of the Three Kings groundwater with the Onehunga groundwater will dilute the Three Kings groundwater prior to standard treatment and might alleviate the public's expressed concerns about the water quality.

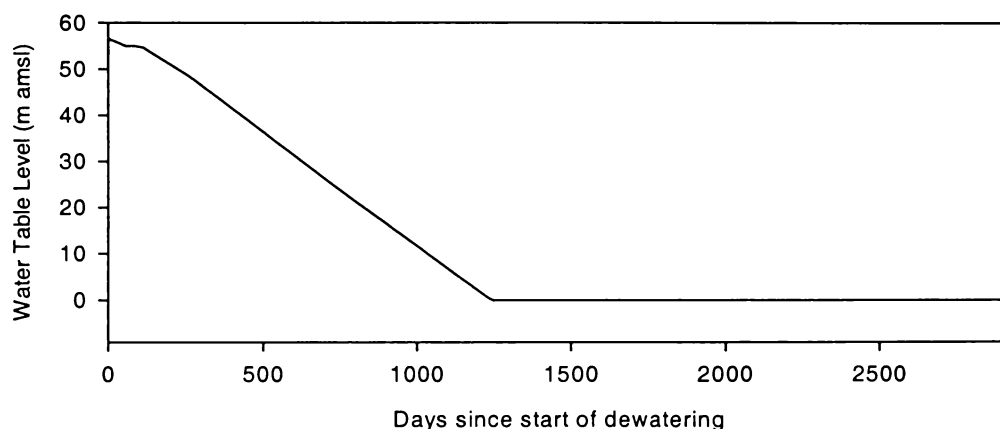


Figure 3.18. Time series of simulated draw down of the water table at the Three Kings quarry (Groundsearch EES Limited, 1996).

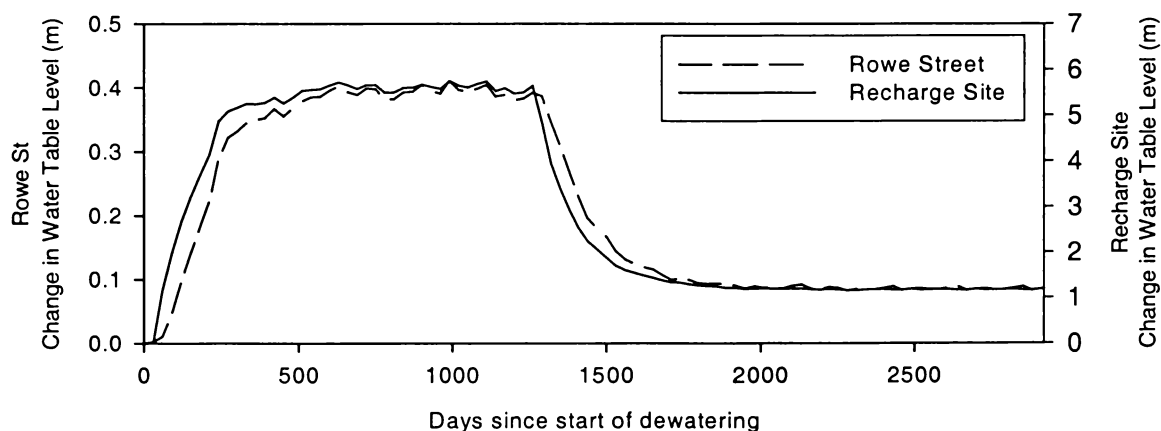
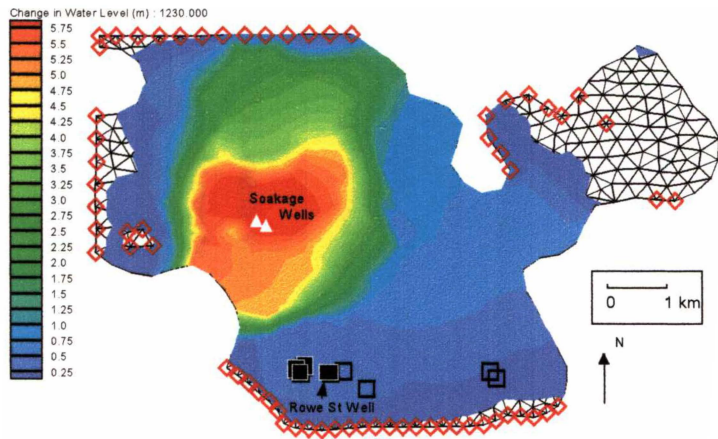
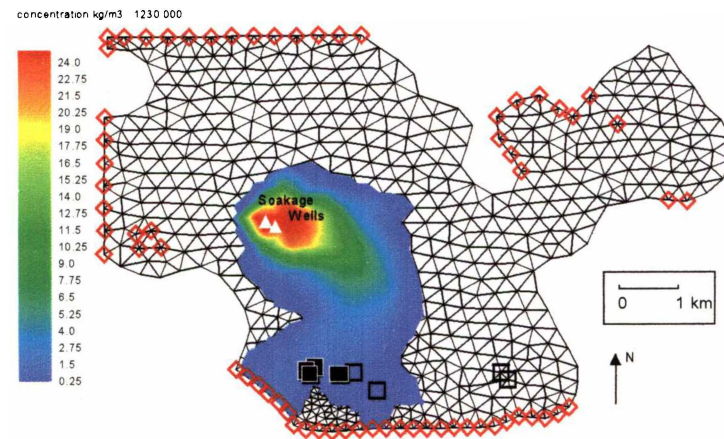


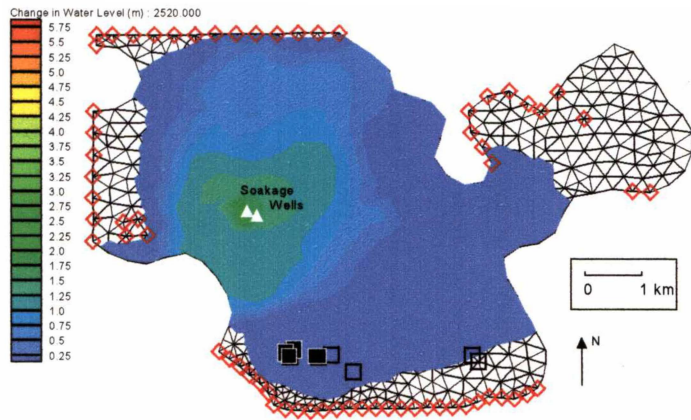
Figure 3.19. Time series of the simulated change in water table levels at the injection site and at the Rowe Street well for the simulation.



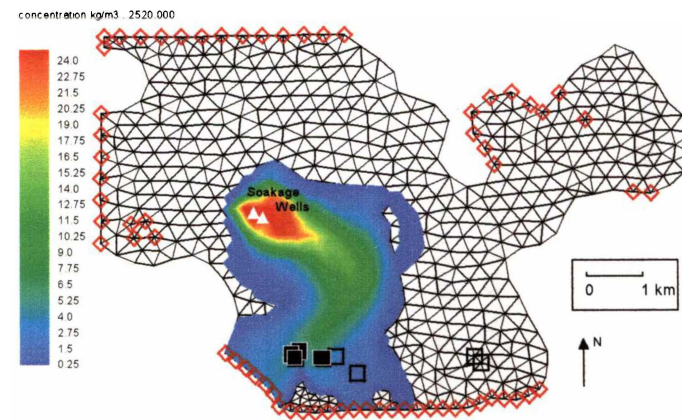
a) Day 1,230, change in water level with recharge rate $7,500 \text{ m}^3\text{d}^{-1}$.



b) Day 1,230, tracer concentrations with recharge rate $7,500 \text{ m}^3\text{d}^{-1}$



c) Day 2,520, change in water level with recharge rate $1,500 \text{ m}^3\text{d}^{-1}$ since day 1245.



d) Day 2,520, tracer concentrations with recharge rate $1,500 \text{ m}^3\text{d}^{-1}$ since day 1245.

Figure 3.20. Simulated spatial extent of the change in water levels and degree of mixing of the recharge water and groundwater for option 2c. Edge of plume is defined by a concentration of 0.05 kg/m^3 , initial concentration was 25 kg/m^3 . Black squares represent Watercare wells, unfilled squares represent other users.

3.5.7 Option 3: Recharge of Highly Treated Wastewater via Soakage Wells Near Mt Smart

This scenario evaluates the possible increase in yield from the Rowe and Pearce Street wells due to artificial recharge via soakage wells of highly treated wastewater from the Mangere Wastewater Treatment Plant.

The hypothetical recharge site for the soakage wells is adjacent to the Watercare wells at the Church Street Watercare Depot (Figure 3.9). A 5.5 km pipe would be required between the Wastewater plant and the recharge site. The geology of the basalt aquifer at the Watercare Depot is expected to be vesicular basalt from 1 to 10 m depth, peat from 10 to 11 m and then a further 4 m of basalt until intersecting the Waitemata Group rocks at 15 m depth. The recharge site geology is based on nearby drill-hole logs ARC 745, 50 m west of the proposed recharge site, and ARC 737, 400 m to the south. Drill-hole logs are in Appendix B.

The peat layer produces a semi-confining of the lower basalt. This may restrict the movement of the wastewater plume to the unconfined section of the aquifer where the soakage wells are located. This semi-confining is evident in the storage parameters of the Auckland Anodisers well in the confined section which has a storativity of 0.0029, compared to the nearby Lichenstein and Winstone Wallboard wells in the unconfined section with specific yield values of 0.2 and 0.11 respectively. The locations of these wells are shown in Figure 2.13. The transmissivity of the Lichenstein well is $260 \text{ m}^3\text{d}^{-1}$, with a saturated thickness of 6 m equating to a conductivity of 43 md^{-1} , and the transmissivity of the Winstone Wallboard well is $665 \text{ m}^2\text{d}^{-1}$ with a saturated depth of 20 m equating to a conductivity of 33 md^{-1} (Carryer & Associates Limited, 1992).

In the fractured and scoriaceous aquifer the degree of clogging should be minimal compared to other porous mediums. However, care needs to be taken to isolate the soakage wells from the peat layer at the recharge site. This peat layer could produce muddy water in the wells and clog the more permeable geology (Bouwer, 1996b).

The infiltration wells were simulated with specified flux boundaries applied to the upper surface of two elements (of the finite element model) at the Watercare Depot near Mt Smart. The area of the two elements was $19,570 \text{ m}^2$, which equated to a flux of 0.38 md^{-1} to inject $7,500 \text{ m}^3\text{d}^{-1}$. No decay or attenuation rates for the wastewater have been applied in the model. A 20-year simulation was run using average 1998 and 1999 data.

Simulated movement of the wastewater recharge plume is shown in Figure 3.21. There was limited lateral movement of the wastewater plume within the aquifer, with the dominant flow direction towards the Manukau Harbour. The increase in the lateral movement of the plume over the 20 year simulation was limited to concentrations less than 0.25 kg/m^3 (initial concentration of 25 kg/m^3). The width of the main plume at the recharge site (defined by the 0.25 kg/m^3 contour) after 10 years was 1000 m, and after 20 years had only increased to 1020 m. After 10 years the concentration at the Rowe Street well had increased from the background of zero to 0.0002 kg/m^3 and after 20 years had increased to 0.0005 kg/m^3 , or 0.002% of the initial wastewater injection concentration.

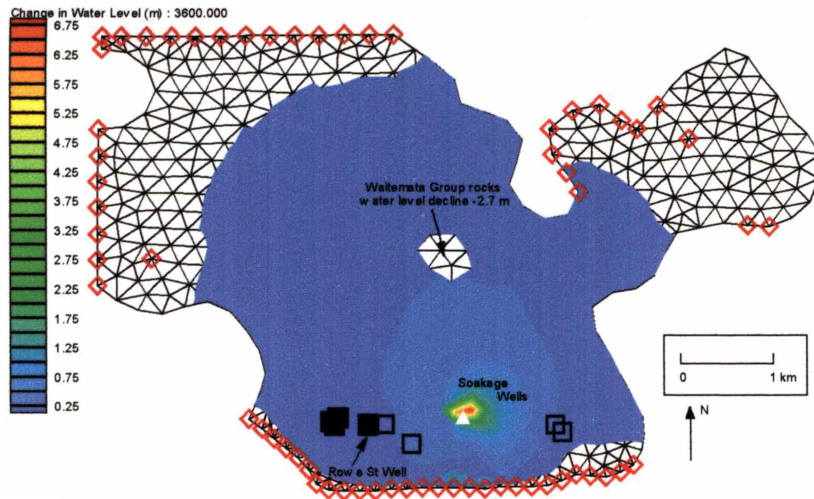
Figure 3.21 also shows the change in water levels due to the recharge. An increase in the water levels greater than 1 m was restricted to the proximity of the recharge site. Water level increases less than 1 m covered a much larger area of the aquifer and were restricted within its boundaries. There was no change in the water level at the head of the Mt Wellington valley or in the Western Springs aquifer. An anomaly in the change in water levels occurred above the recharge site where the lower hydraulic conductivity Waitemata geology protrudes through the basalt aquifer. Here the water levels declined by 2.7 m below normal after 10 years and increased slightly to 1.7 m below normal after 20 years. The water levels at the recharge site increased from 4.35 m to 8.35 m amsl, within 1 m of the ground surface. In reality, the infiltration rate of the soakage wells will decrease as the water level rises closer to the ground surface. This may be improved by decreasing the recharge volume or increasing the number of soakage wells and the area they cover. Also, infiltration rates will tend to decrease over time. The artificial recharge caused the Rowe and Pearce Street water levels to increase by 0.6 and 0.4 m respectively.

The increase in water levels at the Watercare wells equates to an increase in annual yield of 30% or $2,084 \text{ m}^3\text{d}^{-1}$ from the Rowe Street well and approximately 20% or $420 \text{ m}^3\text{d}^{-1}$ from the Pearce Street well. The combined increase in yield for the two Watercare wells equates to 33% of the artificially recharged volume.

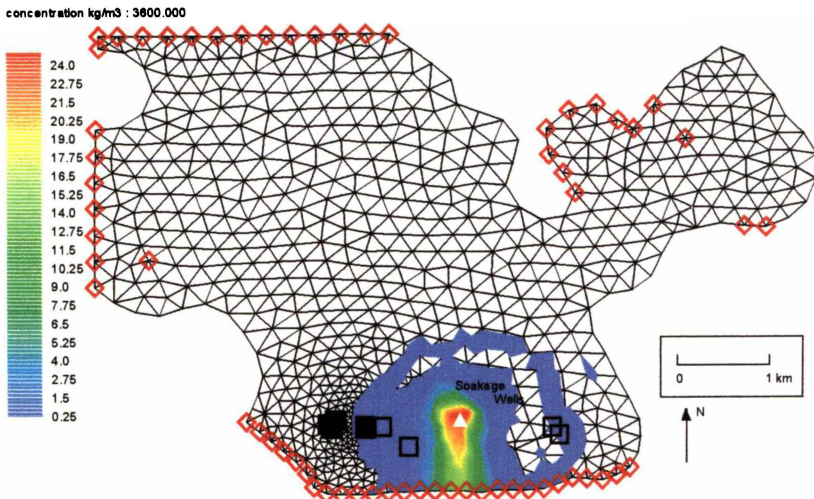
The wastewater recharge was limited to $7,500 \text{ m}^3\text{d}^{-1}$ to limit the spread of the wastewater plume towards the Watercare supply wells and also to reduce the risk of surface flooding. Surface flooding could easily occur as there is only a 5 m thickness of unsaturated rock between the water table level and the ground surface in the recharge area. Based on the average infiltration rate of $1,300 \text{ m}^3\text{d}^{-1}$ for stormwater soakage wells identified in section 3.5.5 at least 6 soakage wells would be required to recharge $7,500 \text{ m}^3\text{d}^{-1}$. This estimate does not

take into account the likely reduction in the effective length of the soakage wells due to an increase in the aquifer's water levels during the artificial recharge.

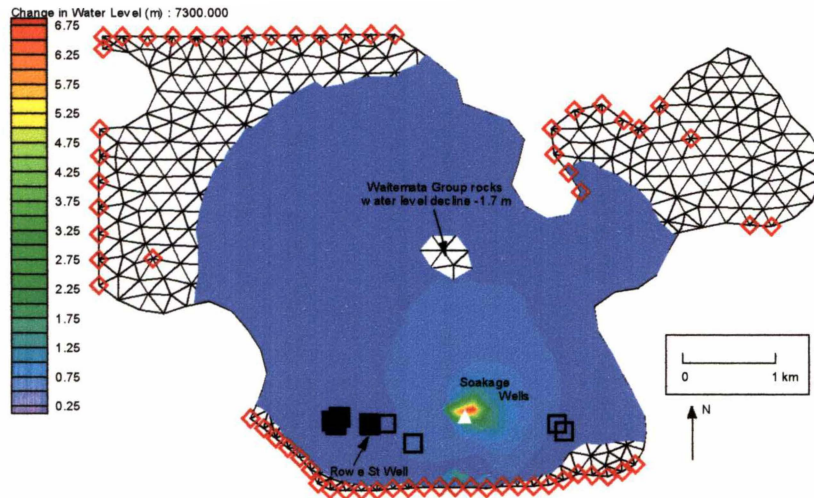
The soakage well's recharge capacity can be estimated using Equation 3.2. This equation is similar to Equation 3.1 used in option 2b and 2c except it calculates the soakage well's recharge capacity when the water table is at a similar level to the water column in the soakage well. Using Equation 3.2, the soakage well's recharge capacity at the beginning of the simulation would have been $460 \text{ m}^3\text{d}^{-1}$, and would require 17 soakage wells to recharge $7,500 \text{ m}^3\text{d}^{-1}$. This was for a 125 mm diameter well with a 4 m water column, the water table and water column bottom are at the same level, and the aquifer conductivity was 38 md^{-1} (average of Winstone Wallboards and Lichenstein conductivity). At the end of the artificial recharge simulation, the aquifer's water table had risen to within 1 m of the ground surface. The recharge capacity of each soakage well would be reduced to $43 \text{ m}^3\text{d}^{-1}$, and would require 175 soakage wells to recharge $7,500 \text{ m}^3\text{d}^{-1}$.



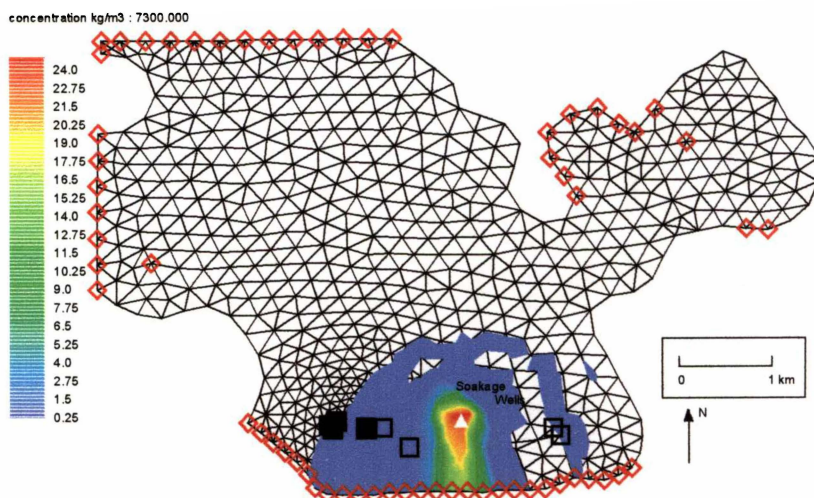
a) Day 3,600, change in water levels.



b) Day 3,600, tracer concentrations.



c) Day 7,300, change in water levels.



d) Day 7,300, tracer concentrations.

Figure 3.21. Simulated spatial extent of change in water levels and the degree of mixing of the groundwater and recharged wastewater for option 3. Lower water level contour limit is 0.05 m. Lower tracer concentration contour limit is 0.000025 kg/m³.

3.6 Summary and Economic Overview

Groundwater extraction from the Onehunga – Mt Wellington aquifer is unable to meet the local water demand during the summer months of the year. Various methods of redistributing the seasonal difference between available aquifer yield and demand were evaluated with a numerical groundwater model of the Onehunga – Mt Wellington aquifer. A three-dimensional finite element numerical groundwater model was developed using the FEMWATER code. The model was calibrated (history matching) using observed water level data from 1998. An independent validation (independent history matching) was performed using observed water levels from 1993 to 2000. The completed model was then used to evaluate five constructed groundwater systems for increasing the yield from the Onehunga – Mt Wellington aquifer. The five constructed groundwater systems were; (1) aquifer storage and recovery (ASR) using the existing Watercare wells, and lowering of Watercare wells' intake screens (2) seasonal injection and extraction of reservoir water via ASR wells, (3) seasonal recharging of reservoir water via wells soakage wells, (4) recharging groundwater from the Three Kings quarry via soakage wells, and (5) recharging tertiary treated wastewater in the Mt Smart area via soakage wells.

In the Auckland region there are three main sources of water readily available for artificially recharging the Onehunga – Mt Wellington aquifer. These are; water from Watercare's 10 storage reservoirs during the winter months, groundwater from the nearby Three Kings quarry dewatering operation, and tertiary treated wastewater from Watercare's Mangere Wastewater Treatment Plant.

Option 1: Evaluating the increase in yield due to redeveloping the Rowe and Pearce street wells.

There does not appear to be any potential for operating the Watercare wells as aquifer storage and recovery (ASR) wells due to the high risk of surface flooding.

Another option is to lower the intake screens of the Rowe and Pearce Street wells. Lowering the Rowe Street and Pearce Street wells' intake screens from their current level of 0.7 m amsl to sea level should not induce saline contamination of the aquifer from the Manukau Harbour, even during periods of extremely low rainfall. Lowering the intake screens could increase the combined average summer yield of the two wells by 40% or 3,500 m³d⁻¹.

However, this is highly dependent on the specific discharge of the aquifer between 0.7 m amsl and sea level.

Option 2a: Seasonal reservoir water injection and extracting of 35,000 m³d⁻¹ with ASR wells at One Tree Hill.

The One Tree Hill scoria cone provides high infiltration rates, and a 35 m thickness of unsaturated rock between the water table and ground surface. Based on the numerical model simulation, this option has the potential to increase the summer yield from the Onehunga – Mt Wellington aquifer from 9,100 m³d⁻¹ to 44,100 m³d⁻¹. Not all the injected water was extracted on the next extraction cycle resulting in mixing with the native groundwater. This should not be a concern as both the recharge water and groundwater are of similar quality. The recharge of reservoir water during the winter also reduces spillage from the Watercare reservoirs. Existing pipe networks and storage tanks could supply the water from the reservoir to the aquifer. The pressure head of the water supplied from the pipe networks and storage tanks may be substantial enough to force the reservoir water into the aquifer without pumping.

Option 2b: Seasonal reservoir water recharge of 35,000 m³d⁻¹ with soakage wells at One Tree Hill.

This scenario was similar to option 2a with the exception of replacing the ASR wells with soakage wells. The recharged water flowed along the paleo valleys of the basalt aquifer towards the existing Watercare wells where it was extracted. The estimated recharge capacity of each soakage well was 1300 m³d⁻¹, which equates to 22 soakage wells to recharge 35,000 m³d⁻¹. The recharge via soakage wells increased the water levels in the vicinity of the Rowe Street well by 0.2 to 0.6 m during the summer and autumn months (January to June). The increase in water levels during summer equated to an increase in Watercare's summer yield of only 10% or 700 m³d⁻¹ from the Rowe Street well and approximately 20% or 300 m³d⁻¹ from the Pearce Street well. This option of recharge was very inefficient, with the combined increase in yield for the two Watercare wells equating to only three percent of the artificially recharged volume.

Option 2c: Recharge of Three Kings quarry groundwater with soakage wells at One Tree Hill.

This scenario is similar to option 2b, except groundwater is recharged into the aquifer and at a much lower rate. Groundwater from the Three Kings quarry is perceived by the public as being unsuitable for use even after treatment. One solution is to mix the Three Kings groundwater with Onehunga – Mt Wellington aquifer groundwater to improve its water quality through dilution, prior to treatment. Option 2c simulated the recharging and mixing of the recharged water as it flowed along the paleo valleys of the basalt aquifer towards the existing Watercare wells where it was extracted. For the first 3.5 years of quarry dewatering $7,500 \text{ m}^3\text{d}^{-1}$ would be available, after this the rate would be decreased to $1,500 \text{ m}^3\text{d}^{-1}$ to maintain the quarry water table level. Compared to option 2a and 2b this source is available throughout the year, but only for 3.5 years from the start of the quarry dewatering. The model water levels increased by 0.4 m at the Watercare wells due to the recharged water.

The increase in yield for the Watercare wells was only evaluated for the summer months, as this is the time of year when the available supply does not meet demand. The water level increase of 0.4 m equates to an increase in summer yield for the Rowe Street well of only 5% or $350 \text{ m}^3\text{d}^{-1}$, and approximately 10% or $150 \text{ m}^3\text{d}^{-1}$ from the Pearce Street well. The combined increase in yield for the two Watercare wells equates to 6% of the artificially recharged volume. This increase was greater than for option 2c, and indicates that a lower continuous recharge rate may be more effective for increasing the discharge from the Watercare wells. The increase in water levels due to the lower artificial recharge rate of $1,500 \text{ m}^3\text{d}^{-1}$ was only 0.1 m for the Watercare wells and was not evaluated to estimate the possible increase in yield.

Option 3: Recharge of treated wastewater with soakage wells in the Mt Smart area.

Wastewater was simulated to be recharged adjacent to the Watercare wells at the Watercare Church Street depot. The wastewater was used as an indirect method of increasing the yield from the Watercare wells. The wastewater was recharged via soakage wells and flowed towards the Manukau Harbour avoiding the Watercare wells. However, due to the water mounding at the recharge site the water levels at the Watercare wells also rose enabling their yield to increase. The recharge rate was limited to $7,500 \text{ m}^3\text{d}^{-1}$ to reduce the lateral movement of the wastewater plume towards the Watercare wells and to not cause surface flooding at the recharge site. The soakage wells' recharge rate was dependent on the water table level. The estimated recharge rate per soakage well with a low water table was

460 m³d⁻¹, which equates to 17 soakage wells to recharge 7,500 m³d⁻¹. For a high water table, the rate was 43 m³d⁻¹ and equates to 175 soakage wells.

After 20 years the model showed that the concentration of wastewater at the Rowe Street well was only 0.002% of the initial wastewater concentration (25 kg/m³). The artificial recharge caused the Rowe and Pearce Street water levels to increase by 0.6 and 0.4 m respectively. The artificial recharge increased the Rowe Street well water levels by 0.6 m, which equates to an increase in the summer yield of 30% or 2084 m³d⁻¹. The rise in water levels at Pearce Street well water was 0.4 m, which equates to an increase in the summer yield of 20% or 420 m³d⁻¹. The combined increase in yield for the two Watercare wells equates to 33% of the artificially recharged volume.

Economic Overview

Table 3.3 summarises the cost per cubic metre in New Zealand dollars for the increase in yield from the five options. Option 1 is the most cost effective with low capital and operating costs, producing an average increase in yield of 3,000 m³d⁻¹. The capital and operation costs for option 2a are approximately twice that for option 1. However, the increase in average daily yield for option 2a is nearly 12 times greater than for option 1. Option 2b, 2c, and 3 all have expensive capital costs. All the options except option 2b have similar operating costs. Details of the cost analysis are in Appendix H.

Table 3.3. Cost in New Zealand dollars per cubic metre of water for the five options

	Increase in yield m ³ d ⁻¹	First year* \$/m ³	Remaining Years \$/m ³
Option 1	3,000	2.18	2.00
Option 2a	35,000	4.60	3.76
Option 2b	1,000	28.67	23.21
Option 2c	500	30.98	3.58
Option 3	2,500	18.14	2.30

*Assumes that the capital cost are paid in the first year.

Chapter 4

Overview of Groundwater Resources in the Hunua and Waitakere Ranges

4.1 Introduction

The Hunua and Waitakere Ranges cover a large area of the Auckland region. There is the potential to extract groundwater from within these two ranges to augment Auckland City's water supply. Groundwater is not currently extracted in the vicinity of Watercare's surface reservoirs in either of the ranges. This chapter provides an overview on the hydrogeology of the Waitakere and Hunua Ranges based on previous geological surveys, and investigations during the construction and maintenance of the reservoirs. This overview is used to identify possible locations and constructed groundwater methods for extracting groundwater from the ranges to augment Auckland City's water supply.

4.2 Waitakere Ranges

4.2.1 Hydrogeology

The majority of the rocks in the Waitakere Ranges belong to the Waitemata and Waitakere Groups and are of early Miocene age (Hayward, 1983; Kermode, 1992a). The oldest rocks are those of the Waitemata Group, which consist of interbedded mudstone and graded sandstone. The Waitemata Group rocks are overlain by the volcanic and volcanic-derived rocks of the Waitakere Group. The Waitakere Group rocks dominate approximately the upper 800 m of the geology in the central and southern Waitakere Ranges, and can be divided into three main formations (Figure 4.1). These are the Lone Kauri, Piha, and Nihotupu Formations. The Piha and Nihotupu Formations are laterally equivalent.

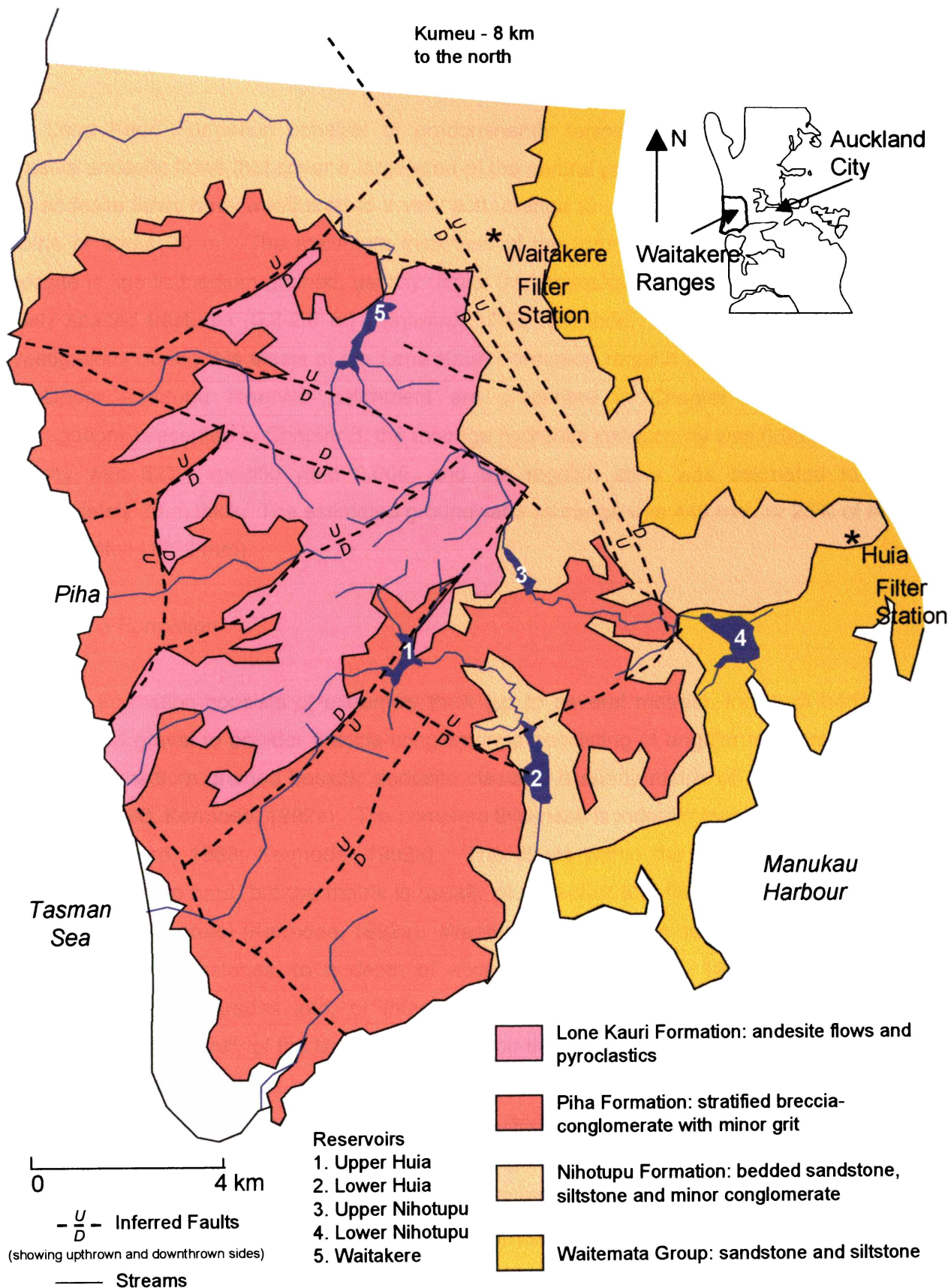


Figure 4.1. Simplified geology of the southern and central Waitakere Ranges. Adapted from Hayward (1983) and Kermodé (1992b).

a) Lone Kauri Formation

The Lone Kauri Formation consists of predominantly terrestrial extrusive and shallow intrusive andesite flows that cover a large area of the central part of the Waitakere Ranges. The andesite flows have weathered to a very soft, orange to red brown sandy clay loam to depths of about 20 m. The maximum thickness of the formation is about 150 m. The andesite is fine to medium grained, usually dense (rarely vesicular), hard to very hard, with widely spaced fractures (0.2-2.0 m) (Hayward, 1983; Kermode, 1992a). Hydrogeological investigations during this thesis of the Lone Kauri Formation regolith zone in the vicinity of the Upper Nihotupu reservoir catchment are presented in Chapter 5. From the investigations presented in Chapter 5, the average hydraulic conductivity was 0.03 md^{-1} , the porosity was 62%, specific yield 0.066, and the regolith zone was estimated to be approximately 20 m thick. The estimated groundwater recharge was 440 mm, or 26% of the annual rainfall (1700 mm).

b) Piha Formation

The Piha Formation consists of extremely thick (up to several metres), indistinct beds of poorly sorted gravel to boulder breccia-conglomerate consisting of angular to subrounded, very fine to medium grained basaltic andesite clasts in a sandy matrix of similar material (Hayward, 1983; Kermode, 1992a). The complete thickness is indeterminable but is at least 800 m (Hayward, 1983; Kermode, 1992a). The clasts within the Piha Formation are moderately hard to hard, but the matrix is usually much softer and the fractures are widely spaced (several metres) (Kermode, 1992a). Weathering produces a soft to moderately soft clay with some corestones, to a depth of about 20 m (Kermode, 1992a). The Piha Formation generally grades into, or interfingers with the laterally equivalent Nihotupu Formation. In the vicinity of the Upper Huia reservoir the Piha Formation has weathering to a depth of 18 m, and joints mapped in nearby outcrops are widely spaced and usually infilled with secondary mineralisation (Riley Consultants Ltd, 1990). The porosity of the unweathered Piha Formation measured by George (1993) was 27%, and increased to 70% in the regolith zone. Two drill-holes by Riley Consultants Limited (1988; 1989) into the Piha Formation near the base of the Waitakere Reservoir gave a hydraulic conductivity of 0.86 md^{-1} and a porosity of 15%.

c) Nihotupu Formation

The Nihotupu Formation is dominantly massive or well-bedded volcanic grit, sandstone, and minor siltstone with a minimum thickness of 450 m (Hayward, 1983; Kermodé, 1992a). The formation is moderately hard to hard with widely spaced fractures (0.2-2.0 m), consisting of massive beds of poorly sorted sandstone containing angular to subrounded grains of basaltic andesite. Weathering produces very soft to soft silty clays to depths of about 20 m. The Nihotupu Formation generally grades into, or interfingers with the laterally equivalent Piha Formation. Examination of 31 drill-hole logs at the Huia Filter station by Mansergh (1988), and preliminary investigations prior to the construction of the Lower Huia reservoir by the Auckland City Council (1966), identified a 10 to 18 m thick mantle of clay over unweathered Nihotupu Formation consisting of gritty sandstone, sandstone and mudstone. From 112 rock samples the average porosity measured by Salter (1992) for the Nihotupu Formation was 23%. These recordings are summarised in Table 4.1.

The inflow of groundwater from the Nihotupu Formation was observed in all the tunnels for the raw water pipe between the Upper Nihotupu reservoir and the Huia filter station, and the Waitakere reservoir and the Waitakere filter station. However, the volume of groundwater inflow was very small and isolated to areas with closely spaced joint sets (Villars and Salter, 1990a; 1990b; Salter, 1992). Investigations by Tonkin & Taylor Ltd (1991a) in the stream and cliff face near the Upper Nihotupu reservoir showed that the Nihotupu Formation joints are planar, continuous and have an aperture up to 2 mm. However jointing was less frequent below the regolith zone. Two drill-holes at the base of the Upper Nihotupu reservoir by Tonkin & Taylor Ltd (1991a) identified a highly fractured zone between 17 and 32 m depth, which was assumed to be the crush zone of an east-west fault at 20.5 m. The hydraulic conductivity of the two drill-holes was between 0.035 md⁻¹ and 0.007 md⁻¹.

Table 4.1. Porosity values for the Nihotupu Formation from Salter (1992).

Lithotype	Number of Samples	Porosity (%)		
		Minimum	Maximum	Average
Very coarse Sandstone	16	18.5	33.1	22.9
Coarse Sandstone	15	18.7	29.6	22.6
Medium Sandstone	37	11.7	31.8	20.4
Fine Sandstone	21	9.5	30.6	24.3
Siltstone	23	16.1	32.8	27.7

4.2.1.1 Flow-Duration Curves

A flow-duration curve for each of Watercare's ten reservoirs was calculated to give an indication of the relative portion of baseflow from the discharge of the streams flowing into each of the reservoirs. The flow-duration curves for the inflow of the five reservoirs in the Waitakere Ranges are shown in Figure 4.2. The flow-duration curves were estimated from the water balance of each reservoir between November 1999 to June 2001. The flow-duration curves' data are listed in the Appendix C. The streams flowing into the Upper Nihotupu and Waitakere reservoirs have similar flow-duration curves, with a sustaining baseflow. The Waitakere reservoir catchment is in the Lone Kauri Formation, and the Upper Nihotupu reservoir catchment is predominantly in the Lone Kauri Formation with a small portion of the catchment in the Nihotupu Formation. Both the Lower Huia and Lower Nihotupu reservoirs have similar flow-duration curves. There is less baseflow for these two catchments compared to the Waitakere and Upper Nihotupu reservoirs. The Lower Huia catchment is mostly in the Nihotupu Formation with some Piha Formation, and the Lower Nihotupu catchment is an even distribution of both the Nihotupu and Piha Formations. The Upper Huia reservoir inflow flow-duration curve shows the least baseflow of the five reservoirs. The Upper Huia reservoir catchment predominantly consists of the Piha Formation and some of the Lone Kauri Formation.

From the combinations of flow-duration curves and geology, the Lone Kauri Formation has the greatest groundwater discharge to streams during drier periods, and the Piha Formation has the least discharge. Thus, as the portion of the reservoir catchments covered in the Piha Formation increases, the groundwater baseflow contribution decreases.

4.2.2 Feasibility of Constructed Groundwater systems

Groundwater yields from the deeper unweathered rocks in the Waitakere Ranges will most likely be small. This has been observed in Kumeu, northeast of the mapped area in Figure 4.1, where maximum yields of $120 \text{ m}^3\text{d}^{-1}$ of groundwater were obtained at depths of up to 300 m from fractured zones and crevices in the Waitemata Group sandstone (Hayward, 1983). The fault and regolith zones of the Lone Kauri and Nihotupu Formations may yielded groundwater in large enough quantities to make a substantial contribution to the Watercare water supply (Figure 4.1).

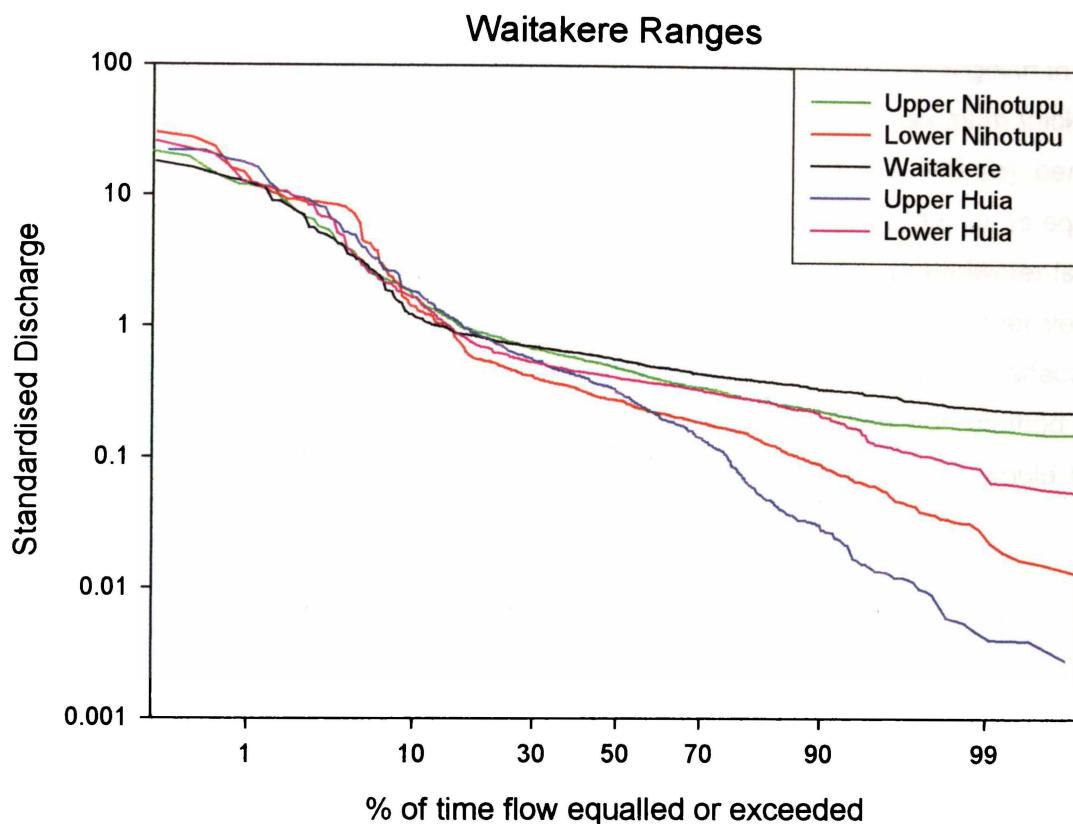


Figure 4.2. Flow-duration curves for the stream inflow of the five reservoirs in the Waitakere Ranges between November 1999 and June 2001. Standardised discharge is achieved by dividing daily discharge by the average discharge.

a) Fault Zone Groundwater

There are at least six normal faults in the Lone Kauri and Nihotupu Formations (Figure 4.1), and their total length covered by the two formations is approximately 36 km. The annual recharge to the fault zones from rainfall infiltration is estimated in this study to be 237,600 m³, or a daily average of 650 m³. This is assuming that the annual recharge is 440 mm (estimated for the Lone Kauri Formation regolith in Section 4.2.1), and the width of the faults crush zone is 15 m (based on the drill-hole observation by Tonkin & Taylor Ltd, 1991a). The annual recharge is a very small volume of water and would not make a substantial contribution to Watercare's current water supply. If all the recharge was successfully extracted it would be equivalent to a 0.20% increase in the annual potable water delivery.

If groundwater in the fault zones was only extracted during severe droughts, the groundwater could be completely extracted over a short period of time, and then left for many years to recover. For every one metre of saturated aquifer depth in the fault zone the

volume of groundwater is 35,640 m³. This assumes the fault zone has a specific yield of 0.066, based on the estimated specific yield for the Lone Kauri Formation regolith in Section 4.2.1. If the saturated aquifer was 150 m thick (this is the assumed maximum thickness of the Lone Kauri Formation), and the groundwater was extracted over a 180 day period, the average daily yield would be 29,700 m³. The average daily yield of 29,700 m³ is equivalent to a 9% increase in the annual potable water delivery. However, the groundwater is spread over a number of long narrow faults and would require a large number of either vertical or horizontal wells to extract the groundwater. Vertical wells will most likely be ineffective due to the narrow width of the fault zone aquifer. The most effective extraction method may be horizontal wells using gravity flow to drain the groundwater. The aquifer would take 23 years to recover using the estimate annual recharge of 237,600 m³.

The unsaturated portion of the aquifer in the fault zones could be used for aquifer storage and recovery. During winter, reservoir water could be recharged and stored until summer to meet seasonal differences between water demand and yield. Applying the same aquifer parameters as discussed above, if the average unsaturated aquifer depth in the fault zone is 20 m, the volume of storage could be 712,800 m³. This is equivalent to only 2 days of average daily water delivery for Auckland City. Once again this would require a large number of wells and would not be cost effective.

b) Regolith groundwater

Utilisation of groundwater in the regolith zones of the Waitakere Ranges may be a more feasible groundwater source for augmenting the Auckland City water supply. One limiting factor is the relatively shallow thickness of the saturated aquifer in the regolith zones, typically less than 20 m. Vertical wells will be relatively ineffective due to the small effective screen length of the wells, and this effective length will only decrease with continued pumping. Thus, many vertical wells pumping at a low extraction rate would be required to minimise the localised water table drawdown at each well. A large number of vertical wells would require access roads for drilling equipment, which would destroy much of the vegetation in the Waitakere Ranges. An alternative option is to drill horizontal wells into the regolith zone parallel with the water table. The incline of the wells will enable gravity drainage of the groundwater from the regolith zone and could be discharged directly into the streams flowing to Watercare's existing reservoirs. This option is evaluated in Chapters 5 and 6 and will not be dealt with further in this chapter.

4.3 Hunua Ranges

4.3.1 Hydrogeology

The oldest and most common rocks in the central and northern Hunua Ranges covering the area of Watercare's five reservoirs are those of the Waiheke Group (Figure 4.3). The Waiheke Group consists of 8,500 m of mainly Upper Jurassic interbedded indurated mudstone (argillite) and graded sandstone (greywacke). These sedimentary rocks were tightly folded and faulted during the early Cretaceous (Schofield, 1976). Their partial erosion during the Cretaceous period was followed by the deposition of the Tauranga, Waitemata, and Te Kuiti Groups. The Tauranga, Waitemata, and Te Kuiti Groups consist of coal measures, siltstone, sandstone, limestone, clays, peats, and gravels, derived from local sources and are typically less than 1,300 m in depth (Schofield, 1976). The Waiheke Group dominates the central and northern Hunua Ranges, while south of the Mangatangi Fault, the Manaia Hill Group dominates (Figure 4.3).

The Wairoa Fault displaces a basement of greywacke that encompasses very weak to moderately strong sandstone, siltstone, mudstone, limestone, and conglomerate and coal measures. Two differing topographies can be roughly defined to the east and west of the Wairoa North Fault. The eastern block is characterised by simple drainage and deeply eroded stream valleys. The western block valleys are shallow but the drains are complicated due to frequent captures, blockages, and reversals (Healy, 1935; Cocks, 1991).

The hydrogeology of the greywacke (sandstone) and argillite (mudstone) is discussed in more detail below.

a) Greywacke

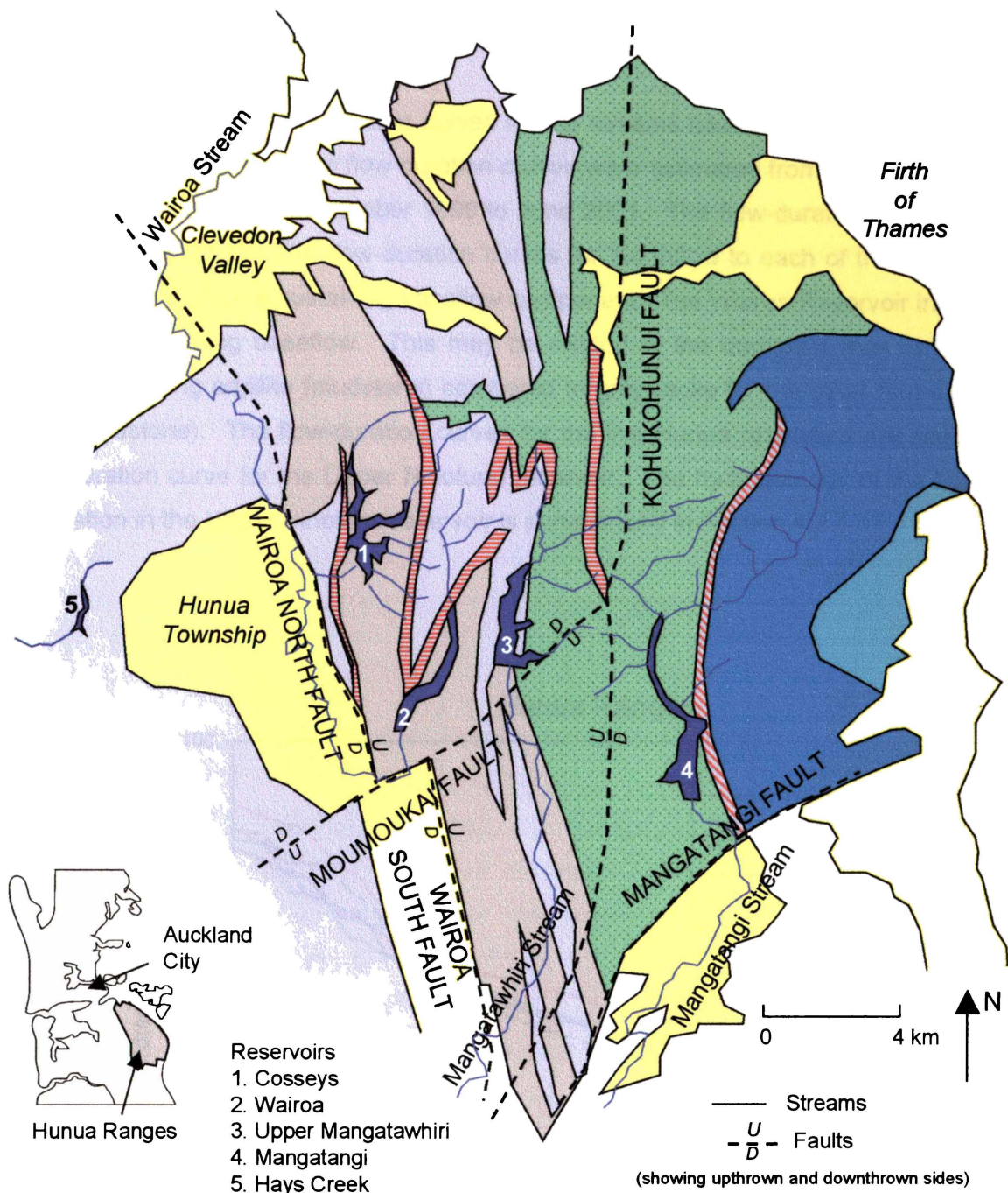
Greywacke (Duders Formation) near the Upper Mangatawhiri dam is moderately to highly weathered to a depth of 13 m, with some manganese staining (Tonkin & Taylor Ltd, 1991b). Weathering of the Duders Formation near Hays Creek reservoir is to only three metres depth. There is a transition between three and nine metre depth to a dense unweathered greywacke (Villars and Salter, 1990c; Works Consultancy Services, 1990). Evidence from 85 drill-holes in the Orere Sandstone show weathering up to 40 m in depth within a distance of 330 m from the Mangatangi Stream (Firth, 1970; Whittome, 1974). The porosity of the

weathered Orere Sandstone ranges from 16 to 30%. The degree of weathering and porosity decreases with depth. The average porosity of the unweathered Orere Sandstone was 8%. Joints in the unweathered greywacke rock are usually sealed with secondary mineralisation.

The only groundwater use near the Hunua Ranges is for the Clevedon Valley, north of the five reservoirs (Figure 4.3) (Smith, 1985). The greywacke rocks in the Clevedon Valley, with exception of the fault zones and well-fractured areas, are poor producers of groundwater. The yield from these rocks is derived from secondary porosity and is dependent on the degree of rock fracturing. The groundwater yield from 100 mm diameter bores in the fractured zones is usually less than $30 \text{ m}^3\text{d}^{-1}$ (Smith, 1985). The groundwater sulphate concentrations are typically high, 13 to 24 g/m^3 , and boron levels were up to 18 g/m^3 (Smith, 1985). These concentrations are typical for groundwater from hydrothermally altered greywacke with the water moving upwards along major fault zones. This theory is supported in the Clevedon Valley by the fact that water samples taken from near the major fault zones have higher boron concentrations (Smith, 1985). Manganese concentrations in the greywacke were up to 0.33 g/m^3 (Smith, 1985).

b) Argillite

Argillite (Clevedon Mudstone Formation) dominates the lower half of the Wairoa reservoir catchment. The depth of weathering at higher elevations ranges between 20 and 45 m, and reduces to less than three metres adjacent to the major streams (Auckland Regional Authority, 1972; Works Consultancy Services, 1991). The geology of the Cosseys reservoir catchment is predominantly argillite also from the Clevedon Mudstone Formation. The argillite is highly weathered to nine metres depth with manganese staining. The degree of weathering gradually reduces between 9 and 23 m depth. The argillite porosity reduces from 35% in the regolith zone to ten percent 28 m below the surface in the unweathered rock.



Tauranga Group: terrestrial deposits, clays, peat and gravel from local sources

Clevedon Mudstone: mudstone with fine roundstone conglomerate at base (Waiheke Group)

Kiripaka Formation: lensoid chert, red mudstone manganese ore (Waiheke Group)

Duders Formation: graded sandstone and mudstone, rare thin chert (Waiheke Group)

Ore Sandstone: predominantly thick sandstone (Waiheke Group)

Mangatangi Formation: graded sandstone and mudstone (Waiheke Group)

Waikorariki Formation: chert, dove grey and red mudstone, manganese ore (Waiheke Group)

Chamberlins Formation: graded sandstone and mudstone (Waiheke Group)

Figure 4.3. Simplified geology of the northern and central Hunua Ranges. Adapted from Schofield (1976).

4.3.1.1 Flow-duration curves

Figure 4.4 shows the flow-duration curves for the streams flowing into the five reservoirs in the Waitakere Ranges. The flow-duration curves were estimated from the water balance of each reservoir between November 1999 to June 2001. The flow-duration curves' data are listed in Appendix C. The flow-duration curves for the inflow to each of the five reservoirs are similar and show a sustaining baseflow component. The Wairoa Reservoir inflow shows the least sustaining baseflow. This may be related to the dominant rock in the Wairoa catchment being argillite (mudstone) compared to greywacke for the other four catchments (fine sandstone). The flow-duration curves for the five Hunua reservoirs are similar to the flow-duration curve for the Upper Nihotupu reservoir. The hydrogeology of the Lone Kauri Formation in the Upper Nihotupu reservoir is summarised in Section 4.2.1.

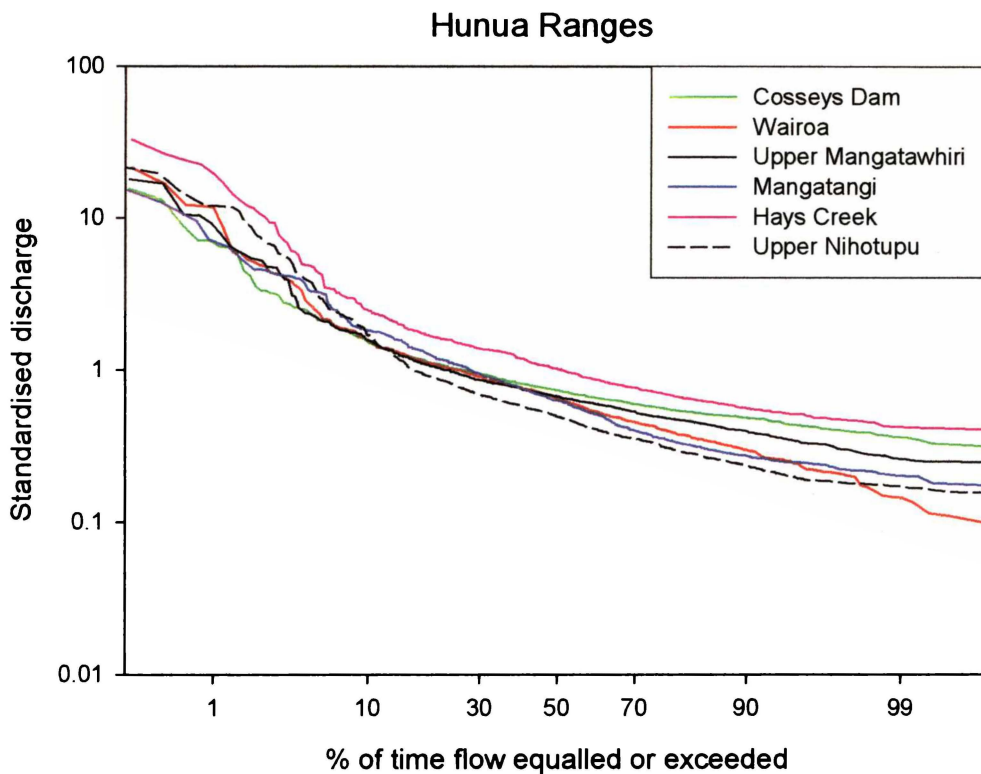


Figure 4.4. Flow-duration curves for the five reservoir catchments in the Hunua Ranges and the Upper Nihotupu Reservoir in the Waitakere Ranges between November 1999 and June 2001. Standardised discharge is achieved by dividing daily discharge by the average discharge.

4.3.2 Feasibility of Constructed Groundwater systems

The deeper unweathered indurated greywacke and argillite of the Hunua Ranges hold little groundwater, with porosity values typically less than 10%. Small yields from the unweathered greywacke have been observed in the Clevedon Valley, north of the Hunua Ranges. The greywacke in the Clevedon Valley, with exception of the fault zones and well-fractured areas, are poor producers of groundwater. The yield from these rocks is derived from secondary porosity and is dependent on the degree of rock fracturing. The groundwater yield from 100 mm diameter bores in the fractured zones is usually less than 30 m³d⁻¹ (Smith, 1985).

There are three possible groundwater sources in the Hunua Ranges. These are: the fault zones of the Orere Sandstone and Duders Formation; regolith layer of the Orere Sandstone and Duders Formation; and the Tauranga Group sediments near the Hunua Township (Figure 4.3).

a) Orere Sandstone and Duders Formation Fault Zone Groundwater

Greater groundwater yields are more likely from the greywacke (sandstone) rather than the argillite (mudstone) due to the coarser sediment size of the greywacke. The Orere Sandstone and Duders Formation are the largest expanses of predominantly thick sandstone (Figure 4.3). The Orere Sandstone is 2,200 m thick and forms the central and generally highest parts of the Hunua Ranges (Figure 4.3) (Schofield, 1976). The Duders Formation is greater than 3,000 m in thickness (Schofield, 1976). The Kohukohunui Fault is aligned along the length of the Orere Sandstone and is between the Mangatawhiri and Mangatangi reservoirs. The degree of weathering in the vicinity of the fault is unknown. The fault dip is towards the east producing a large area over which wells can be drilled to intercept the fault. The fault is approximately 12 km in length in the vicinity of the central Hunua Ranges, and the exposed section of the fault is typically at an elevation of 380 m amsl.

The estimated annual groundwater recharge is 79,200 m³, or a daily average of 220 m³. This is based on the following assumptions: annual recharge is 440 mm (estimated for the Lone Kauri Formation regolith in Section 4.2.1), and the width of the faults' crush zone is 15 m (based on the drill-hole observation for the Lone Kauri Formation by Tonkin & Taylor Ltd, 1991a). The annual recharge is only equivalent to 0.07% of the annual water supply.

The groundwater could be utilised for emergency supply to meet the difference between supply and demand during droughts. For every one metre of saturated aquifer, the volume of groundwater is 11,900 m³. This is applying a specific yield of 0.066 (from Upper Nihotupu catchment). The difficulty is to efficiently remove this groundwater. Horizontal wells could be used in some locations to drain the groundwater and discharge it directly into streams flowing to the Watercare reservoirs. These locations are isolated to areas where the horizontal wells can be drilled into the hillside and intersect the fault crush zone.

The fault zone aquifer may also be utilised for aquifer storage and recovery (ASR). Water could be recharged from the nearby Mangatangi and Mangatawhiri reservoirs during the winter when there is excess and spillage occurs. The available storage depends on the depth of unsaturated rock below the ground surface, and its specific yield. If the water table is 20 m below the surface and the weathered zone is 12 km by 15 m, with a specific yield of 0.066, the rechargeable volume is 237,600 m³. To recharge this volume over 180 days (winter) would require an average recharge rate of 1,320 m³d⁻¹. The average daily spillage between July and December from the Upper Mangatawhiri reservoir for the year 1997 to 2000 was 20,278 m³ and the Mangatangi reservoir was 62,260 m³. Thus, this ASR would do very little to alleviate the volume of reservoir spillage.

There may also be some water quality concerns with the groundwater extracted in the vicinity of the weathered fault zones. Manganese concentrations in the Clevedon Valley were elevated in the vicinity of fractured greywacke. This may be particularly important in the vicinity of the Waikorariki and Kiripaka Formations, which contain manganese ore (Figure 4.3).

b) Alluvium Groundwater

A second option is to utilise the Tauranga Group rocks in the Hunua township area. These consist of minor pumice and locally derived alluvium (Figure 4.3). All aspects relating to the hydrogeology are unknown. But they are more likely to have hydraulic properties more suitable for groundwater injection or extraction than for the greywacke and argillite rocks of the Hunua Ranges. The limitations are that the lateral extent and thickness of the deposits may be relatively small.

c) Orere Sandstone and Duders Formation Regolith Groundwater

As identified for the Waitakere Ranges in section 4.2.2, the utilisation of groundwater from the regolith may be a more viable source for augmenting Auckland City's water supply. Utilisation of the groundwater in the regolith using horizontal wells is evaluated in Chapters 5 and 6 and will not be dealt with any further in this chapter.

4.4 Summary

There is little scope for large-scale groundwater use in both the Waitakere and Hunua Ranges. The unweathered rocks of both the Hunua and Waitakere Ranges have porosity values less than 10%, and the joints are infilled with secondary mineralisation. Groundwater discharges in the Clevedon Valley, north of the Hunua Ranges, and in Kumeu, north of the Waitakere Ranges yielded less than $120 \text{ m}^3\text{d}^{-1}$ from fractured unweathered rock.

The volume of groundwater available from the fault zones will be relatively small due to the narrow width of the crush zone. To extract this groundwater would also require a large number of wells making the option inefficient.

The more feasible source of groundwater is from the regolith zones. The regolith zones for both ranges are typically 20 m thick, but can vary between 2 m and 40 m in thickness. The average porosity is approximately 30%, and can be as high as 70%. Based on the flow duration curves for the catchments of Watercare's 10 reservoirs, the Lone Kauri and Nihotupu Formation in the Waitakere Ranges yield more groundwater than the Piha Formation. In the Hunua Ranges the greywacke rocks of the Orere Sandstone and Duders Formation yield more groundwater than the argillite rocks.

Groundwater from the regolith zone is the most feasible option and is evaluated in Chapters 5 and 6.

Chapter 5

Evaluation of Horizontal Wells: Field Investigation

5.1 Introduction

Currently the Hunua and Waitakere Ranges are only utilised by Watercare for the surface storage of water in 10 reservoirs. These reservoirs have limited storage capacity and lose water through spillage during significant wet periods, while having inadequate storage during dry periods. The obvious solution of increasing the reservoir size, or constructing new reservoirs, is becoming increasingly unpopular due to the undesirability of flooding productive land or existing ecosystems.

As discussed in Chapter 4, groundwater is largely limited to the regolith zones in both of the ranges. The regolith zones are approximately 20 m in thickness. The geology of the Hunua Ranges mainly consists of unfractured, indurated sandstone and mudstone, and the Waitakere Ranges' geology mainly consists of sandstone, breccia-conglomerates, and andesite flows. Below the regolith zone in both the ranges there is little available groundwater, except possibly in the crush zone of the faults.

One approach of augmenting Watercare's effective reservoir storage capacity without increasing the size of the existing reservoirs, or constructing new reservoirs, is to utilise the groundwater in the regolith zone. Horizontal wells may give an element of control over the water table elevations in the regolith zones in the reservoir supply catchments. This may in turn allow some control over the discharge of the streams supplying Watercare's 10 reservoirs. In a sense, the storage capacity of the unconfined aquifers becomes an extension of the operational storage of the reservoirs. This concept is most applicable for catchments where high water tables enhance the stream discharge producing the reservoir spillage.

The Upper Nihotupu catchment in the Waitakere Ranges has been selected as a trial site for evaluating the possibility of augmenting the Auckland water supply with groundwater from the regolith zone aquifer¹ using horizontal wells. This chapter presents the data and information collected from field investigations in the Upper Nihotupu reservoir catchment.

¹ In Chapters 5 and 6 the term 'aquifer' refers to the unconfined aquifer in the regolith zone of the study site in the Upper Nihotupu reservoir catchment.

The data and information is used in Chapter 6 for a model-based analysis of a hypothetical horizontal well system in the Upper Nihotupu catchment. Some of the concepts presented in Chapters 5 and 6 are discussed in Brown and Bardsley (2002).

5.2 Hydrogeology of the Hunua and Waitakere Ranges

5.3 Study Site

The Nihotupu catchment is located 20 kilometres west of Auckland City at the southern end of the Waitakere Ranges. The Nihotupu Stream is dammed in two locations by the Upper and Lower Nihotupu Reservoirs, which are part of the Auckland water supply system. The study site is located on a plateau covering an area of 0.77 km² on the eastern side of the Nihotupu Stream, above the Upper Nihotupu Reservoir (Figure 5.1).

5.3.1 Geology

The regional geology of the Waitakere Ranges is discussed in Chapter 4. The geology of the upper 150 m of the field site consists of the Lone Kauri Formation, part of the Waitakere Group (Figure 5.2). The Lone Kauri Formation is predominantly andesite, formed during the last eruptive phase of the Waitakere Ranges (17 million years ago) (Hayward, 1976; Kermode, 1992a). The andesite is hard to very hard, with widely spaced fractures (0.2-2.0 m). It weathers to a very soft, orange to red-brown sandy clay loam to a depth of about 20 m.

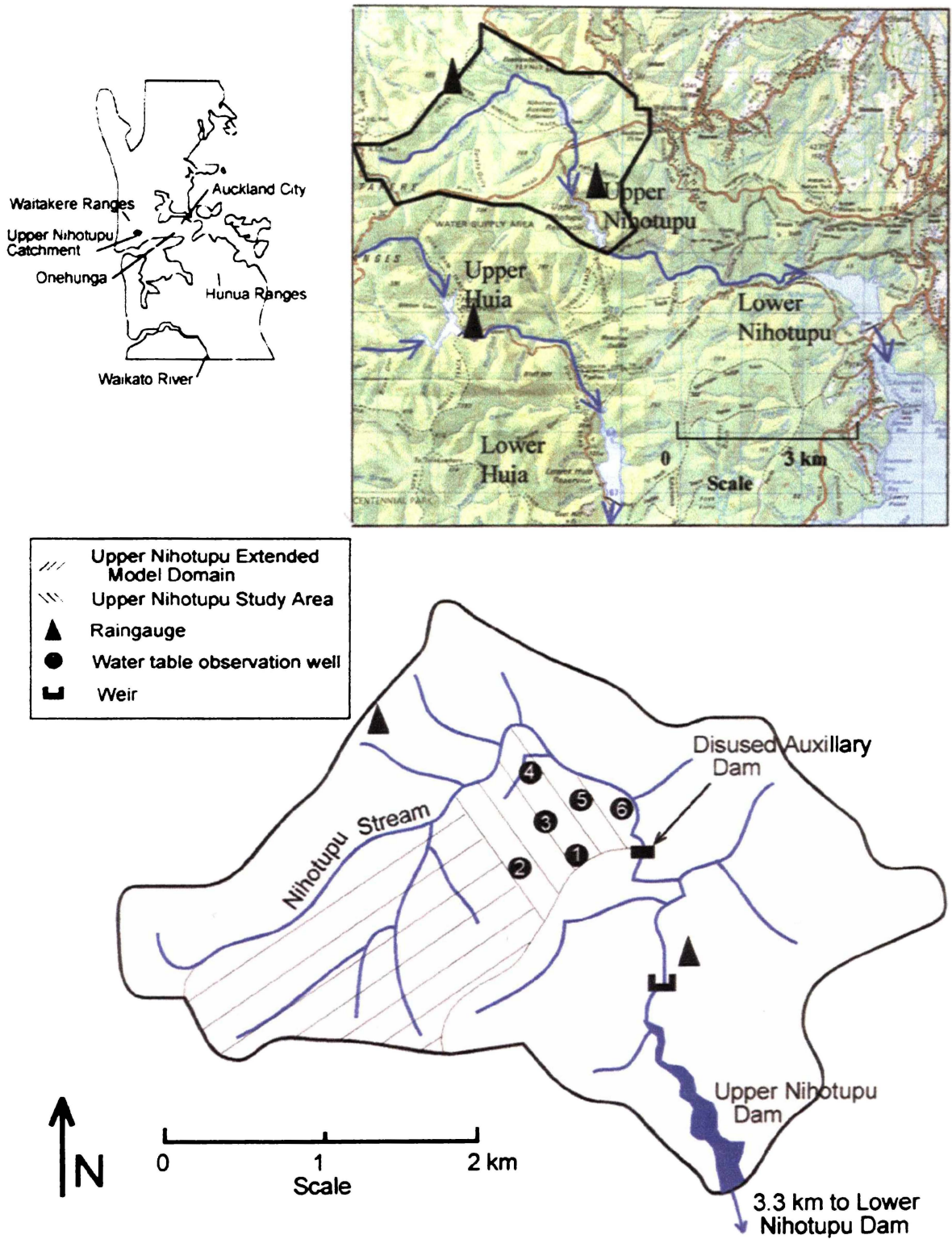


Figure 5.1 The Upper Nihotupu catchment and study area in the Waitakere Ranges, Auckland.

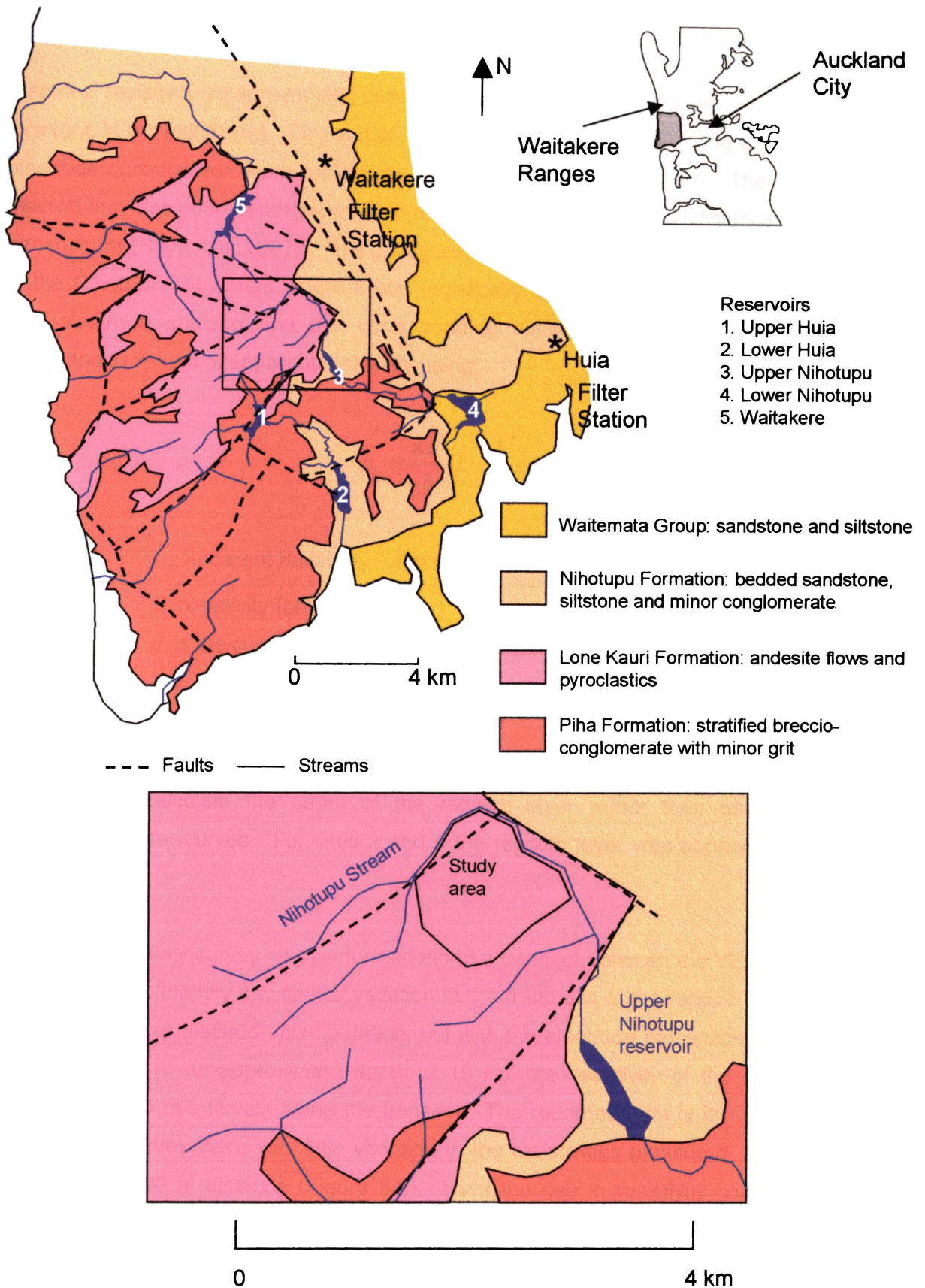


Figure 5.2. Geological map of the Upper Nihotupu catchment, the four formations presented are from the Waitakere Group. Adapted from Hayward (1983) and Kermodé (1992b).

5.3.1.1 Resistivity Survey

Electrical resistivity equipment was used to confirm the depth of the regolith layer at various locations in the catchment. Two vertical electrical soundings (VES) using the Schlumberger electrode configuration were performed adjacent to dip wells 2 and 6. The Schlumberger method provides good depth penetration and requires relatively short distances between the transmitters (Reynolds, 1997). This configuration was necessary in the scrubby vegetation of the Nihotupu catchment where it was logistically difficult to use long lengths of cables. The electrode spacing and the corresponding resistance values (Appendix C) were converted to values of apparent resistivity using;

$$\rho_a = \frac{\pi a^2}{b} \left[1 - \frac{b^2}{4a^2} \right] R;$$

Equation 5.1

where ρ_a apparent resistivity (ohm m)
 R resistivity (ohm)
 a is the current-electrode half-spacing (m)
 b is the potential electrode spacing (m), and $a \geq 5b$

A software routine developed by Shevnin, Bobachev and Berezina (1990), was used to automatically calculate the depth of the regolith layer rather than using theoretically calculated master curves. For sites 2 and 6 the regolith layer was approximately 20 m in thickness.

A second resistivity survey was performed along a transect between the VES surveys at dip wells 2 and 6 to identify any spatial variation in the thickness of the regolith geology. Using the Schlumberger electrode configuration, but with the electrode half spacing fixed at 40 m (corresponding to an approximate depth of 15 m), the resistivity of the subsurface was measured at 40 m intervals along the transect. The recorded data is in Appendix C. For most of this survey there was little variation in the resistivities measured, except between the 400 and 700 m distance (Figure 5.3). Here the rise in resistivity coincides with the survey passing over a small hill. The relatively constant resistivity measurements along the transect indicate that the regolith layer depth does not decrease to less than 15 m thickness.

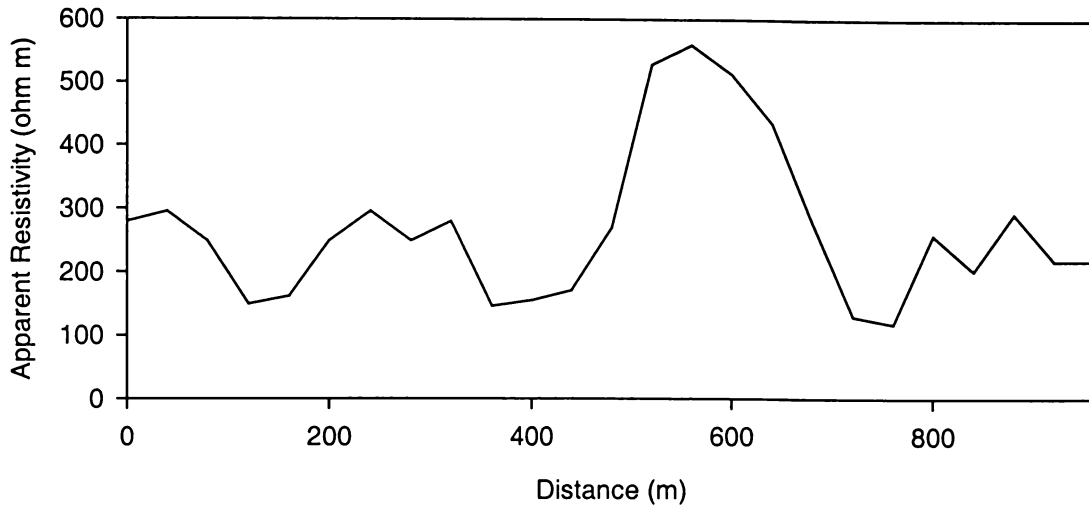


Figure 5.3. Resistivity survey using the Schlumberger electrode configuration between dip-wells 2 and 6. Sample depth ~ 15 m.

5.4 Subsurface Hydrology Measurements

5.4.1 Water table level monitoring

Six water table monitoring dip wells were hand-augured to a maximum depth of six metres, a perforated PVC pipe was placed in each hole and back filled with gravel (Figure 5.1). The top of the hole was lined with plastic and a 0.3 m deep concrete cap to stop the well operating as a soakage pit (Figure 5.4). Monthly water table levels were manually monitored between July 1999 and June 2001. The data is in Appendix I. There was also continuous monitoring of well 4 between November 1999 and October 2000 (Figure 5.5), where the water table was measured every 15-minutes using a capacitance probe. There is very good correlation when comparing all the monthly-recorded water levels for all the wells (Table 5.1). Dividing the monthly measurements into those recorded when there was no rainfall in the seven days prior, and those where there was rainfall resulted in very little change in the correlation coefficients (Table 5.2).

The average R^2 value for all the wells' water level measurements was 0.85. For events with no rainfall the seven days prior, R^2 was 0.83, and with rainfall the R^2 was 0.86. This implies that the water table is behaving as a spatially coherent unit, with the response to recharge and discharge occurring at the same time throughout the aquifer. Between July 1999 and June 2001 the water table fluctuated between 0 and 6 m below the ground surface, with a

summer minimum and winter maximum. The spatial variation of the water table elevation generated with kriging interpolation for a low summer water table (7 March 2000) and high winter water table (29 August 1999) are shown in Figure 5.6. There is little difference in the form of the contour patterns for the two seasons. This also implies that the water table is behaving a singularly coherent unit. As might be expected, the water table contours generally follow the topography of the catchment.

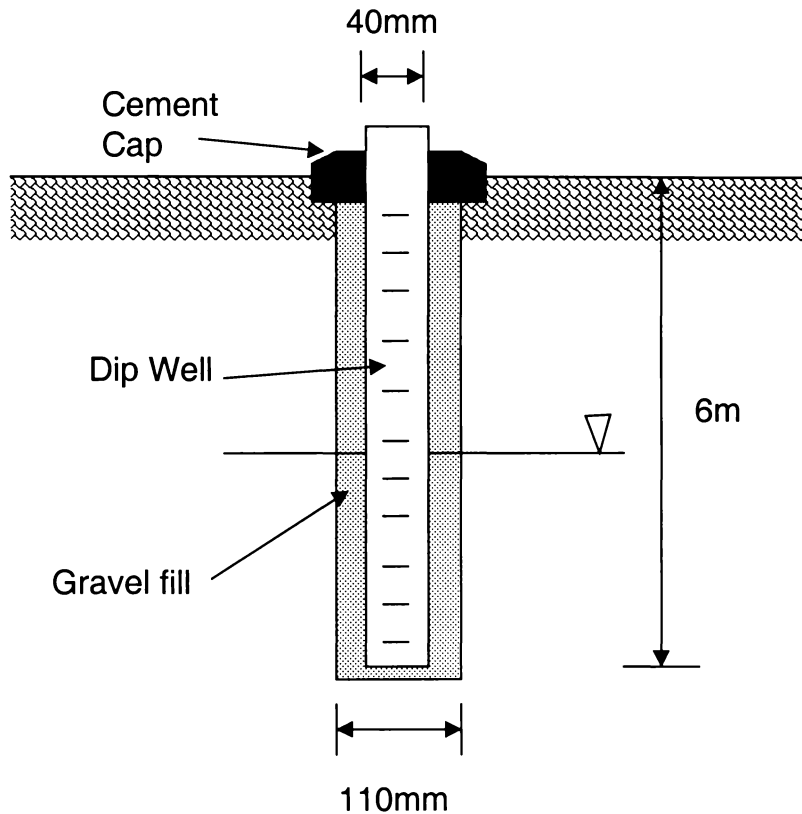


Figure 5.4. Construction details of the dip wells installed at the study site

Table 5.1. Correlation matrix of R^2 values from the linear regression of all the monthly water table levels between July 1999 and June 2001.

	Well 1	Well 2	Well 3	Well 4	Well 5	Well 6
Well1						
Well 2	0.93					
Well 3	0.72	0.84				
Well 4	0.86	0.93	0.78			
Well 5	0.87	0.92	0.87	0.88		
Well 6	0.82	0.79	0.89	0.75	0.87	

Table 5.2. Correlation matrix of R^2 values from the linear regression of monthly water table levels with no rainfall for the seven prior days, and also for water levels with rainfall in the prior seven days (shaded), between July 1999 and June 2001.

	Well 1	Well 2	Well 3	Well 4	Well 5	Well 6
Well 1		0.94	0.75	0.92	0.94	0.82
Well 2	0.91		0.81	0.94	0.95	0.83
Well 3	0.66	0.87		0.74	0.79	0.92
Well 4	0.8	0.92	0.82		0.94	0.77
Well 5	0.78	0.86	0.92	0.9		0.83
Well 6	0.84	0.76	0.77	0.77	0.88	

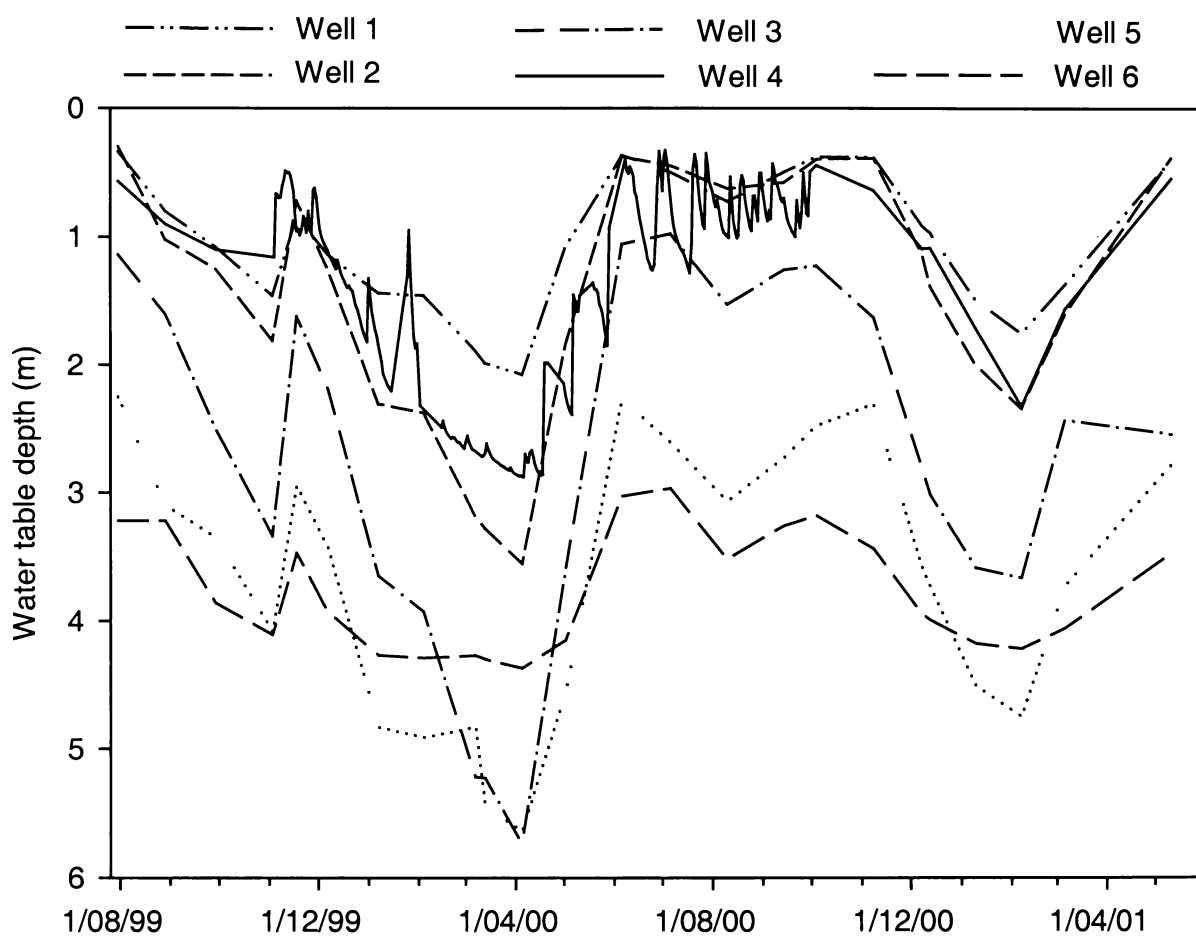


Figure 5.5. Water table depth below ground surface for the six monitoring wells between July 1999 and June 2001.

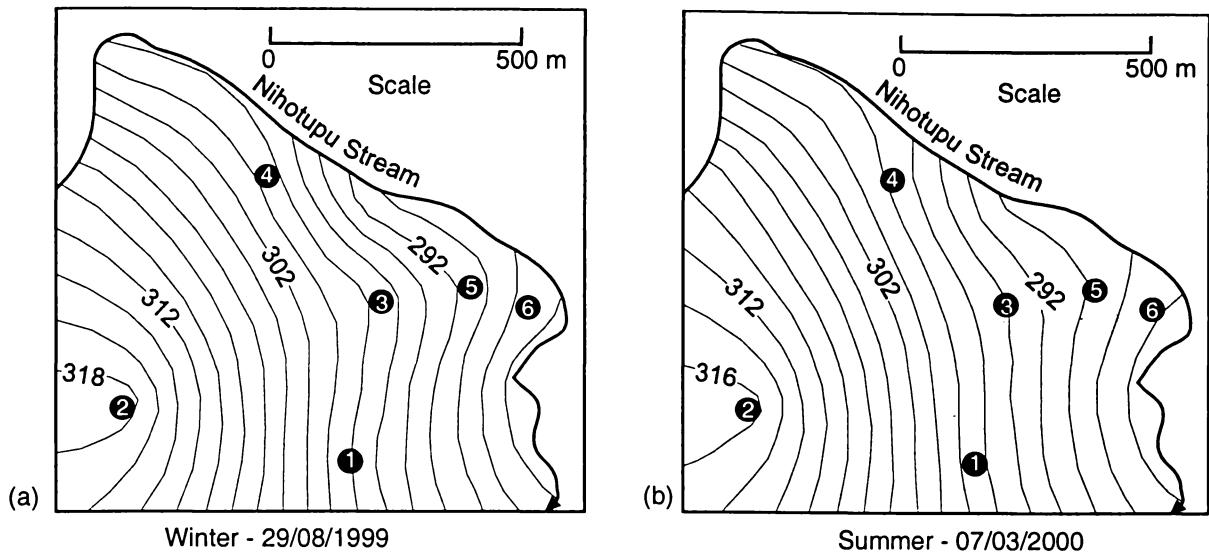


Figure 5.6. Contoured water table depth elevation for (a) winter and (b) summer.

5.4.2 Estimation of Groundwater Recharge

Aquifer recharge was assumed equivalent to the Nihotupu Stream's baseflow discharge. The average annual recharge for the two-year period between April 1999 and March 2001 was 440 mm, or 26% of the average rainfall (1700 mm). The estimation of the recharge is discussed in more detail in section 5.5.2.1.

5.4.3 Hydraulic Conductivity Measurement

The Upper Nihotupu aquifer's hydraulic conductivity was calculated using the Bouwer and Rice (1976) bail test. This test is based on the Theim equation of steady state flow to a well, and is designed for fully or partially penetrating wells in unconfined aquifers with packing material between the well casing and the aquifer. During the development of the bail test, Bouwer and Rice only tested their method on examples where the wells were screened below the water table (Bouwer and Rice, 1976). The only reason the fully screen casing situation was not simulated was due to the limitation of Bower and Rice's electrical resistivity analog model used to develop their method. To simulate this situation resulted in a short circuit of their electrical resistivity model (Bouwer and Rice, 1976).

The six dip wells constructed in the Upper Nihotupu study site were screened across the water table. The gravel packer between the casing and aquifer is larger than the casing diameter (Figure 5.4). Therefore to check if the Bouwer and Rice method was appropriate

for this well design a second hole with no packing or casing was constructed adjacent to well 6. The conductivity was estimated using the auger hole bail-test method as described by Boast and Kirkham (1971). The bail test methods are described in Appendix J.

Using the Bouwer and Rice bail test, the hydraulic conductivity for well 6 was 0.025 md^{-1} . The auger hole bail test conductivity on the adjacent augured hole was 0.021 md^{-1} . Based on this, the Bouwer and Rice method was found to give appropriate results for the dip wells when compared to the adjacent auger hole test. Bouwer and Rice bail tests were then performed on wells 2 and 4 giving conductivity values of 0.025 md^{-1} and 0.045 md^{-1} , respectively. All the test results are very similar, but are possibly greater than expected for clay loams, which are typically less than 0.0005 md^{-1} (Todd, 1980; Domenico and Schwartz, 1998).

Often bail tests under-estimate the hydraulic conductivity. There has been much discussion into the accuracy of bail test conductivity values. In particular why they are generally less than those of pump tests in the same geological formation (Brown, Narasimhan and Demir, 1995; Rovey II and Cherkauer, 1995; Bouwer, 1996a; Butler Jr and Healey, 1998a; Butler Jr and Healy, 1998b; Rovey II, 1998). Some of the reasons suggested by Butler Jr and Healy (1998a) and Bouwer (1996a) are: scale effects when projecting a point value to regional values; soil smearing of other disturbances around the well; reduced effective screen length and/or lower-conductivity skin surrounding the well than typically of surrounding formation; and failure to account for vertical anisotropy in the analysis of slug tests.

5.4.4 Porosity

One undisturbed 97.19 cm^3 soil sample was taken from well 6 at a depth of 3 m. The sample had a saturated weight of 0.1662 kg, and dry weight 0.0973 kg. The soil dry bulk density (ρ_b) was 1000 kg/m^3 , and had a particle density (ρ_p) of 2650 kg/m^3 , which is typical for most mineral soils (McLaren and Cameron, 1993). Using Equation 5.2 the porosity (n) of the sample is 62%. This fits in well with the typically porosity range of 57 to 68% for New Zealand silt loam, clay loam or sandy loam (McLaren and Cameron, 1993).

$$n = 1 - \frac{\rho_b}{\rho_p}$$

Equation 5.2

5.4.5 Specific Yield

The specific yield of the regolith layer was calculated from the change in groundwater volume discharge into the Nihotupu stream and the corresponding reduction in water table level during three rainless periods (Table 5.3). The water table recession curves for the three rainless periods measured at well 4 were linearly interpolated to the other five wells to provide a spatially averaged decline in the water table. The linear interpolation was based on the monthly water levels where no rainfall occurred in the seven days prior. The correlation coefficients of the wells' monthly water levels are discussed in section 5.4.1. The groundwater specific discharge was estimated from the Nihotupu Stream flow hydrograph scaled to the study area. Scaling of the hydrograph to the study area is discussed in more detail in section 5.5.2. Using the data in Table 5.3 the average specific yield for the three recession periods was 0.066. This compares well with the typically cited value of 0.03 for clay (Todd, 1980; Domenico and Schwartz, 1998).

Table 5.3. Specific yield estimates based on the average decline in water table and the Nihotupu Stream discharge for three separate water table recessions.

Recession periods	Average aquifer water table decline (m)	Nihotupu Stream total specific discharge (m)	Specific yield
19/02 – 01/03/2000	0.125	0.007	0.056
08/03 – 12/03/2000	0.046	0.003	0.065
18/03 - 06/04/2000	0.148	0.012	0.078

5.5 Surface Hydrology Measurements

5.5.1 Rainfall

Rainfall was recorded at an hourly interval by two rain gauges below the study catchment (Figure 5.1), these are the Upper Huia and Upper Nihotupu rain gauges. A third rain gauge located on Cutty Grass Track (CGT) above the field site was not used due to the gauge under-recording rainfall since November 2000. This is shown in Figure 5.7 by comparing the cumulative daily average rainfall for just the Upper Huia and Upper Nihotupu rain gauges minus the cumulative daily average rainfall for all three sites. Prior to September 2000 there is little difference between the two averages, but after November the under recording of rainfall at CGT results in an increased difference between the two averages. The daily average rainfall of the study site can be calculated using just the Upper Huia and Upper Nihotupu rain gauges. The daily average spatial rainfall for the duration of the study (July 1999 to June 2001) using the average of these two rain gauges is shown in Figure 5.8. The annual average rainfall is 1700 mm, with the average winter rainfall (June to November) being 1088 mm or 64% of the total annual rainfall, and the summer average rainfall (December to May) is 612 mm or 36%.

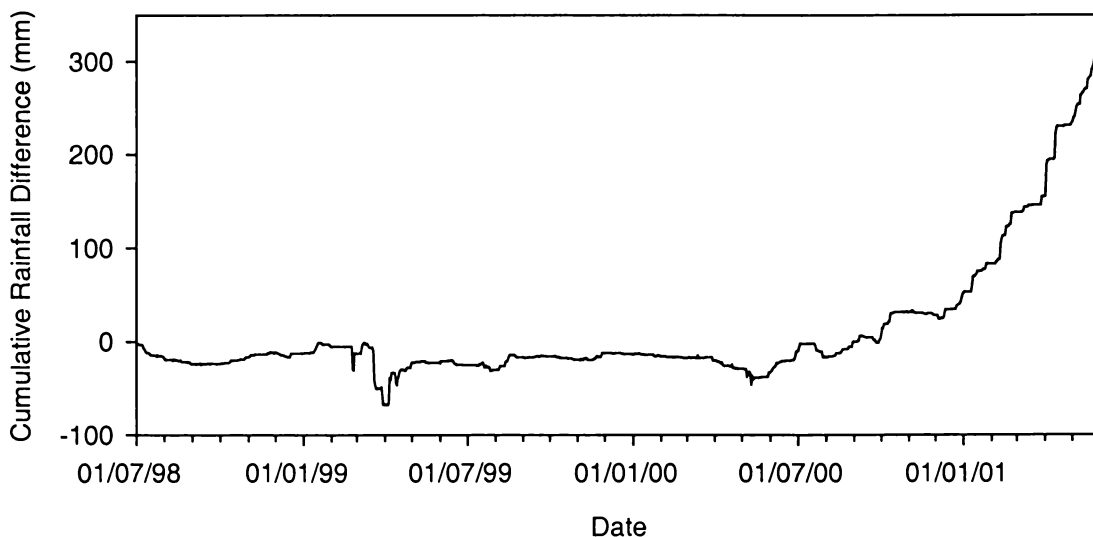


Figure 5.7. Average daily cumulative rainfall difference. Daily average cumulative rainfall for the Upper Nihotupu and Huia gauges minus the daily average cumulative rainfall for the Upper Nihotupu, Huia, and Cutty Grass Track gauge.

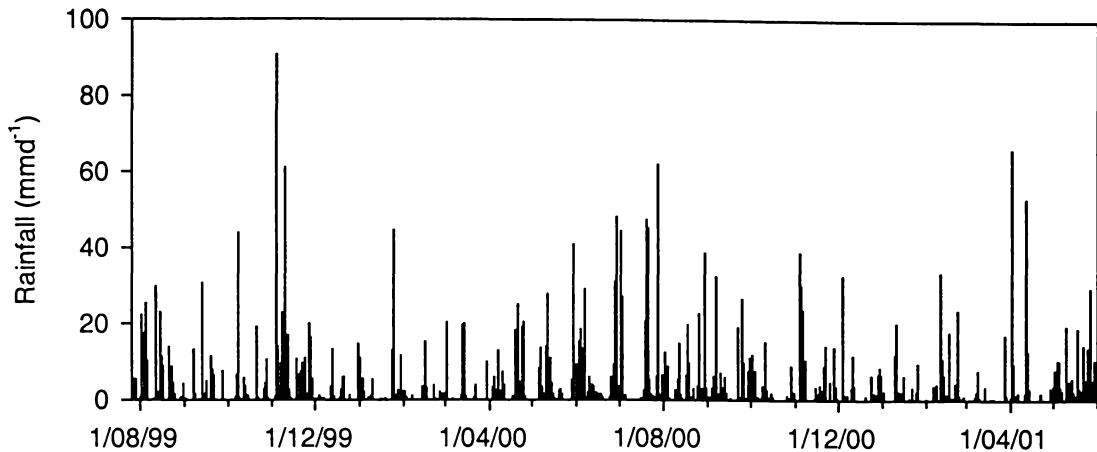


Figure 5.8. Daily average rainfall from the Upper Huia and Upper Nihotupu rain gauges between July 1999 and June 2001.

5.5.2 Nihotupu Stream Discharge

The Nihotupu Stream discharge is recorded continuously at a rectangular-profile weir 750 m downstream from the field area (Figure 5.1). The basic theoretical discharge equation for this full width weir was developed by Kindsvater and Carter (Ackers, *et al*, 1978) and takes the form:

$$Q = 0.567 \left(1 + 0.125 \frac{h_1}{P_1} \right) (b - 0.001) \sqrt{g} (h_1 + 0.001)^{\frac{3}{2}} \quad \text{Equation 5.3}$$

Where

Q = discharge (m^3s^{-1})

h_1 = gauged head, related to weir crest (m)

g = gravitational acceleration (9.81 ms^{-2})

b = crest breadth (3.67 m)

P_1 = crest height of weir above mean bed level (0.24 m)

Manual stream gaugings were carried out at the weir. These confirmed that the theoretical discharge provides an accurate calculation of the weir's discharge (In Appendix C). Manual gaugings were also carried out at the same time at the disused Nihotupu Auxiliary dam at the bottom of the field site (Figure 5.1). These manual gaugings enabled scaling of the weir

discharge to the appropriate discharge passing through the disused dam. The discharge at the disused dam was 66% of the discharge measured through the weir. The reduction in discharge was also equivalent to the reduction in catchment area above the weir (8.5 km²), to that of the area above the disused dam (5.5 km²). Next, the discharge was scaled to the area of the proposed model shown in Figure 5.1. Estimates of the model area discharge are based on a further reduction in catchment area. The model area is 2.9 km², which is 34% of the weir's catchment size. The weir discharge is then reduced by 34% to provide an estimate of the model area discharge, which will be used to calibrate the numerical groundwater model in Chapter 6. Figure 5.9 shows the daily specific discharge of the catchment area covered by the model. Due to the direct scaling of the discharge to the various catchment areas, the average annual specific discharge for any part of the catchment above the weir was 1040 mm, or 61% of the average annual rainfall.

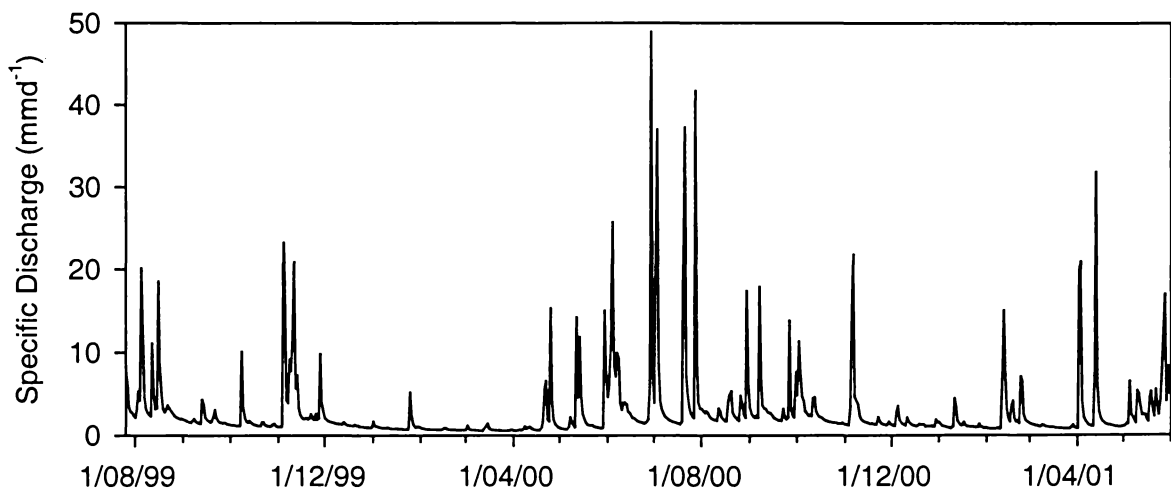


Figure 5.9. Daily total specific discharge for the catchment above the Upper Nihotupu weir (Figure 5.1).

5.5.2.1 Baseflow estimation

The baseflow of the Nihotupu Stream was estimated to provide a data set to calibrate the MODFLOW model against, and to give an estimate of the aquifer's recharge. Due to the subjective nature of estimating the baseflow discharge, it is recommended that a variety of methods are used as the estimates cannot be checked against a "true" value (Mau and Winter, 1997; Halford and Mayer, 2000). The evaluation of a correct value is based on how well the various methods compare (Mau and Winter, 1997; Halford and Mayer, 2000). Both

the graphical partitioning and recession-curve-displacement methods were applied to the Nihotupu stream hydrograph to provide estimates of the baseflow discharge. For both methods the total baseflow for the 26 months between March 1999 and May 2001 was assumed to be equivalent to the aquifer's recharge. Assuming zero change in storage is justifiable as the catchment is in its natural state with no groundwater or surface water pumping taking place, and the water table elevations were at the same levels at the beginning and end of the period analysed. Both methods provided similar results with the annual baseflow/recharge for the 26 months between March 1999 and May 2001 being approximately 440 mm, which is 26% the rainfall (1700 mm), or 42% of the total discharge (1040 mm). The strong baseflow component of the Nihotupu Stream is evident in Figure 5.10 by the gently sloping lower segment of the flow–duration curve. The strong baseflow component makes the graphical partitioning the more efficient of the two manual methods to apply, with minimal interpolation required when separating the hydrograph peaks from the clearly defined baseflow prior to, and after the peak. The two methods are discussed in more detail in the following sections.

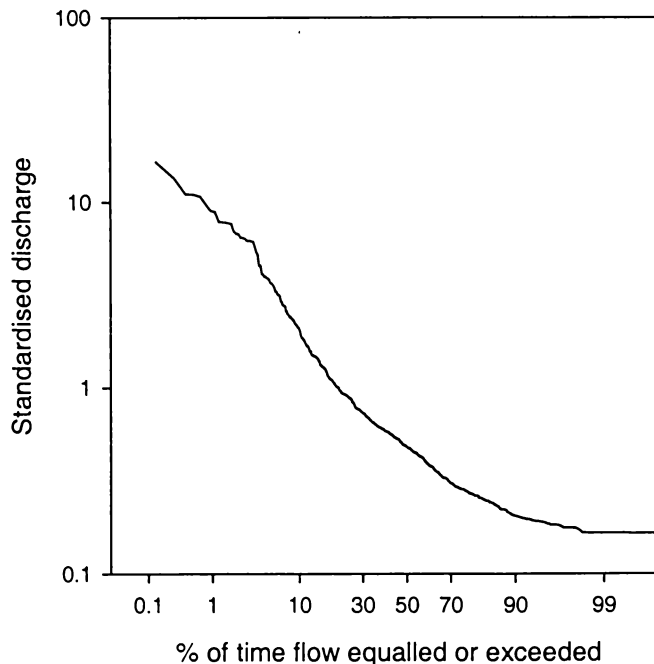


Figure 5.10. Flow-duration curve for the study area. Discharge standardisation was achieved by dividing daily discharges by mean discharge.

a) Graphical Partitioning

The process of manual graphical separation was performed in a similar way as developed by the Institute of Hydrology (England) and described in Mau and Winter (1997). Two assumptions are applied: (1) The discharge of the stream is assumed to be all baseflow during low flow periods. This is evident when the hydrograph of discharge verses time on a semilogarithmic plot reaches a constant recession gradient (exponential discharge decline); and (2) during periods of peak flow the baseflow component is found by interpolating from adjacent constant recession gradients. Using this method for the 26 months between March 1999 and May 2001, the baseflow/recharge equaled 40% of the total discharge, or 24% of the rainfall.

b) Recession-Curve-Displacement Method

This method developed by Rorabaugh (1964), estimates the groundwater baseflow component for each stream quickflow event from the stream hydrograph recession curves. The difference between the extrapolated recession curves, prior to, and following a rise in the hydrograph due to a quickflow event (after a calculated 'critical time') are used to calculate the total baseflow component of the hydrograph peak using a set of equations developed by Rorabaugh. There are several assumptions with this method which are rarely meet when dealing with field problems, these are: (1) the aquifer is homogeneous and isotropic; (2) of uniform thickness underlain by impermeable material; (3) equal distance between the groundwater divides and the stream for all places in the catchment; (4) prior to recharge the water level is at stream level and horizontal; (5) the stream fully penetrates the aquifer; (6) the full length of the stream is a sink for groundwater; and (7) regulation and diversion of flow are negligible (Rorabaugh, 1964; Rutledge and Daniel III, 1993; Mau and Winter, 1997; Halford and Mayer, 2000). When the major assumptions of the recession-curve-displacement method are grossly violated the method becomes a poor tool for estimating groundwater discharge/baseflow (Halford and Mayer, 2000).

For the Nihotupu field site, assumption 1, 3, 4, and 5 are not fully meet. However, manually executing this method as explained by Rutledge and Daniel III (1993) for each of the stream flow peaks between March 1999 and May 2001, the baseflow/recharge of the Nihotupu Stream equaled 43% of the total discharge, or 27% of the rainfall. The average regression slope for the quickflow events was 68 days per log cycle and the critical time was 15 days.

5.5.3 Evaporation

Daily Penman potential evaporation data was supplied from the Auckland International Airport 20 km southeast from the site. The calculated Penman potential evaporation is a measure of the expected rate from an extensive short green cover completely shading the ground and adequately supplied with water (Penman, 1963). The total Penman potential evaporation for the Auckland International Airport on an annual basis was 1190 mm or 70% of the total rainfall.

An estimation of the actual evaporation from the Nihotupu study area was made using a catchment water balance approach for the two-year period from April 1999 to March 2001, where the average annual rainfall was 1700 mm, and the Nihotupu Stream's specific discharge was 1040 mm. The storage change over this period was assumed to be zero. Thus, the remaining 660 mm or 39% of the rainfall was taken to be actual evaporation loss. The actual evaporation is 56% of the Penman potential evaporation. Assuming zero change in storage is justifiable as the catchment is in its natural state with no groundwater or surface water pumping taking place, and the water table elevations were at the same level at the beginning and end of period analysed.

Estimates of actual evaporation on time scales less than one year were achieved using the daily Penman evaporation values. The daily Penman estimates were scaled by 56% so that the annual Penman evaporation total equal the actual evaporation estimated from the water balance of the Upper Nihotupu catchment between June 1999 and May 2001 (Figure 5.11).

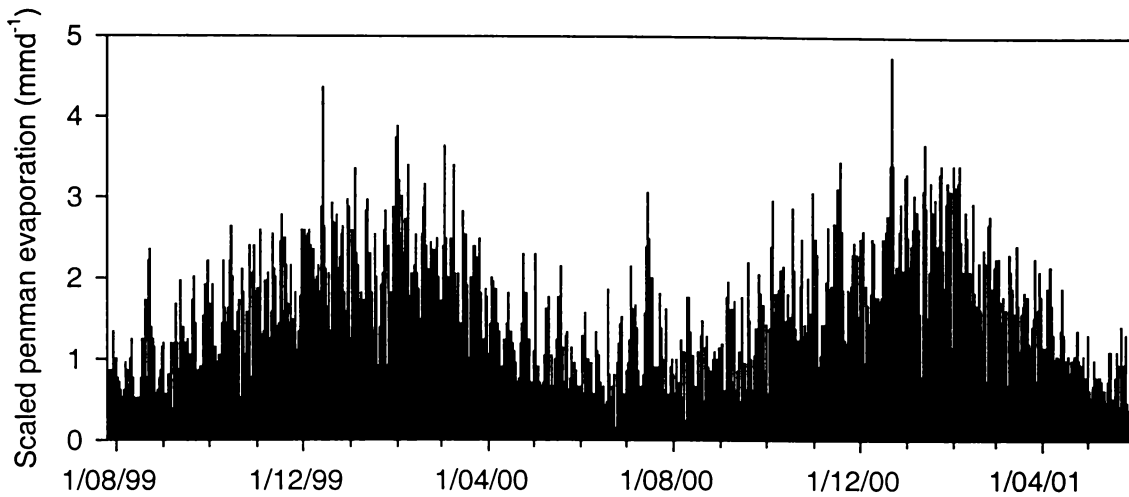


Figure 5.11. Daily Penman potential evaporation scaled to equal the actual evaporation estimated for the Upper Nihotupu catchment between June 1999 and May 2001.

5.5.4 Upper Nihotupu Reservoir Spillage

The Upper Nihotupu Reservoir spillway crest level is 217.28 m amsl, or the reservoir water level of 33.833 m. Spillage is calculated using a modified spillway-rating curve developed by (Ward and Webby, 1996) for the Nihotupu Upper Reservoir with an accuracy of +/- 5%.

$$Q = \left(CBH^{3/2} \right) 86400$$

Equation 5.4

Where; Q = discharge (m^3d^{-1})

C = discharge coefficient (approximate by $0.0835H^2 + 0.1209H + 1.6408$)

B = spillway crest length (34.14 m)

H = height above spillway crest (m)

Table 5.4 summarises the annual spillage from the Upper Nihotupu reservoir for the 42 months between July 1997 to December 2000. For this period, the average annual spillage was 1,780,703 m^3/year (specific discharge of 0.175 m^3/year), or 10% of the total rainfall. This average annual spillage is also equivalent to 74 days of average delivery from the Upper Nihotupu reservoir to the filter station (24,000 m^3d^{-1}). Figure 5.12 shows how spillage

from the reservoir generally occurred when the water table was relatively near the ground surface. Figure 5.13 shows for four isolated spillage events that the specific spillage is greater when the water table is closer to the ground surface and is not dependent on the amount of rainfall. Only four spillage events were analysed as these occurred when the capacitance probe at well 4 was automatically recording the water table level. There are no water table level measurements for the remaining spillage events. Spillage events were defined to be finished after seven days of no spillage.

Table 5.4. Upper Nihotupu reservoir spillage calculated with Equation 5.4. Average daily spillage is for the number of days that spillage occurred during each year.

Year	Days of spilling	Total Spillage (m ³)	Ave daily Spillage (m ³)
1997	31	1,959,460	63,208
1998	39	2,565,860	65,791
1999	18	829,310	46,072
2000	16	1,768,183	110,511

5.5.5 Relationship of Nihotupu Stream Discharge to Water Table Depth

Figure 5.14 shows that for the daily rainfall totals exceeding 10 mm per day (measured between November 1999 and October 2000), the Nihotupu Stream's peak discharge is greatest when the water table, measured at well 4, is near the ground surface. The quickflow discharge is assumed to be equivalent to the slope runoff. The relationship shown in Figure 5.14 was analysed further by isolating periods of stream quickflow discharge produced by isolated rainfall events. Fourteen events were isolated between November 1999 and October 2000 (Table 5.5). The quickflow discharge is the difference between the total discharge and the baseflow discharge calculated previously. The 14 rainfall events ranged between 17 and 125 mm, the corresponding water table depth ranged between 0.5 and 2.7 m, and the Penman evaporation between 1.5 and 7.5 mm.

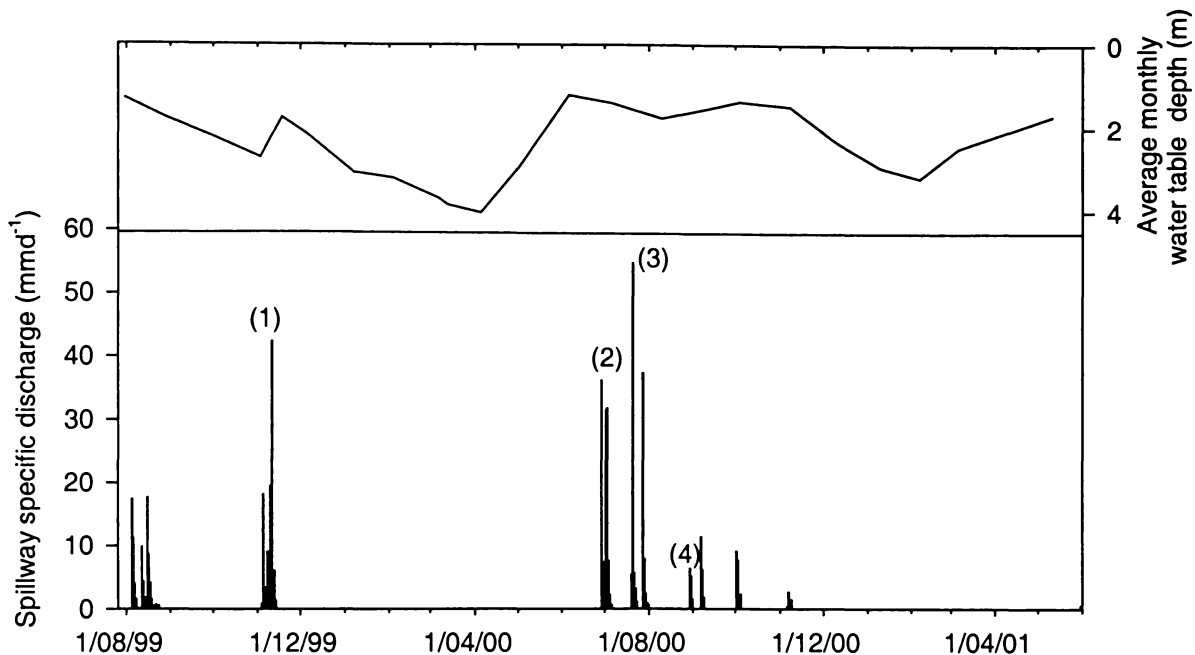


Figure 5.12. Daily total specific discharge due to reservoir spillage and the average monthly water table depth of the six observation wells. Parenthesised values identify spillage events analysed further in Figure 5.13.

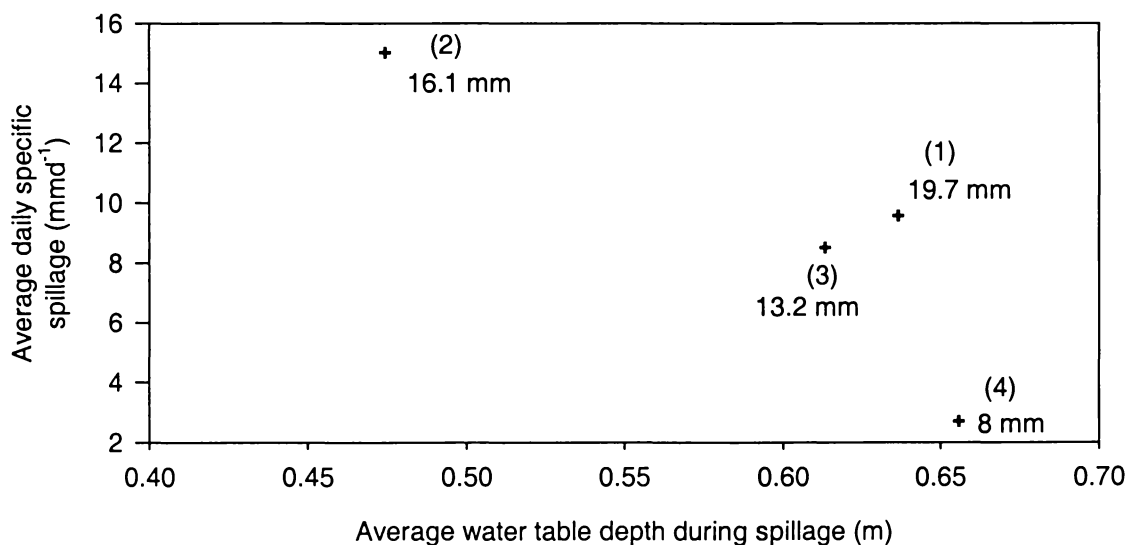


Figure 5.13. Four-isolated spillage events versus the average water table depth for the duration of the spillage recorded automatically at well 4. Average rainfall for the duration of the spillage is given for each data point. The parenthesised values refer to the spillage event shown in Figure 5.12.

The 14 rainfall events provided a means of testing the concept whereby the horizontal wells could possibly reduce the Nihotupu Stream's discharge by lowering the water table. The 14 events indicate that a lowered water table results in increased recharge from rainfall infiltration and a reduction in quickflow discharge volume (or runoff) (Figure 5.15). However, these observations are limited by the lack of data for large rainfall events when the water table is deep.

An empirical relationship (Equation 5.5), obtained from a weighted multiple linear regression fit, was derived to express the quickflow volume as estimated by hydrograph graphical partitioning. The independent variables were the total event rainfall, water table depth at well 4 prior to each event, and the average daily Penman evaporation for each of the 14 rainfall events. The multiple regression was weighted by the duration of each rainfall event. A good fit was achieved with a multiple coefficient of determination value of 0.93. Figure 5.16 shows the estimated quickflow discharge volume from the hydrograph graphical partitioning, verses that produced using Equation 5.5.

$$V = \exp(12.13 + 0.0775P - 0.0003P^2 - 2.4953W_L + 0.5854W_L^2 - 0.7338E + 0.0558E^2)$$

Equation 5.5

Where; V is quickflow discharge volume (m^3)

W_L is depth to water table below ground surface prior to rainfall event (m)

P is rainfall depth of rainfall event (mm)

E is average Penman evaporation for each rainfall event (mm)

An application of Equation 5.5 is presented to estimate the change in quickflow discharge due to a deeper water table, as may be achieved using horizontal wells. The same data used to derive Equation 5.5 are used, with the exception that the initial water table depth is set at 0.5 below the surface for all the events, and is then lowered to 2.7 m below the surface. Equation 5.5 indicates for the 14 rainfall events that there is a fourfold decrease in quickflow discharge by lowering the water table from 0.5 m to 2 m below the ground surface (Table 5.5). To maintain the catchment's water balance the aquifer's recharge would increase by a corresponding amount.

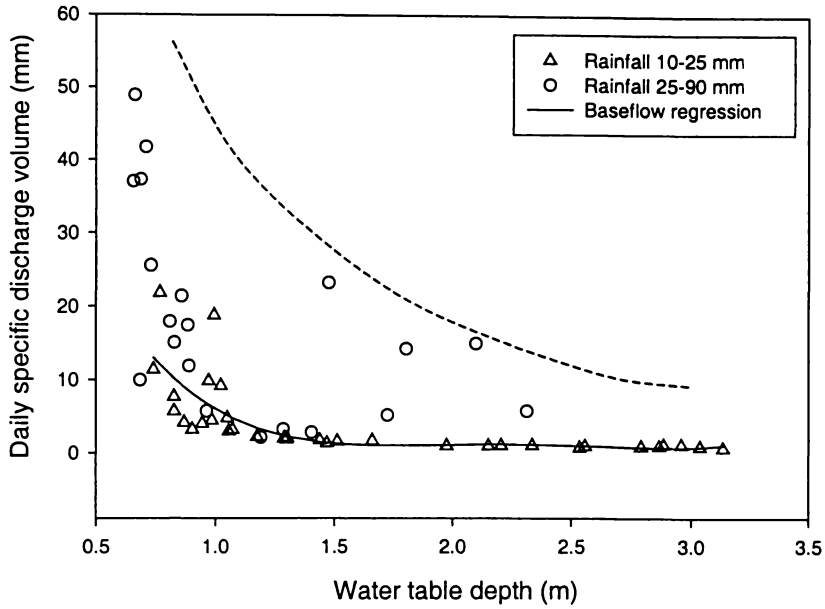


Figure 5.14. Daily specific discharge of the Nihotupu Stream versus the water table depth at well 4 for daily rainfall totals exceeding 10 mm between November 1999 and October 2000. A regression relationship was applied to the days when there was zero rainfall to provide an estimate of the stream's baseflow specific discharge. The dashed line is a subjective estimate of the specific discharge upper bound as a function of daily rainfall depth.

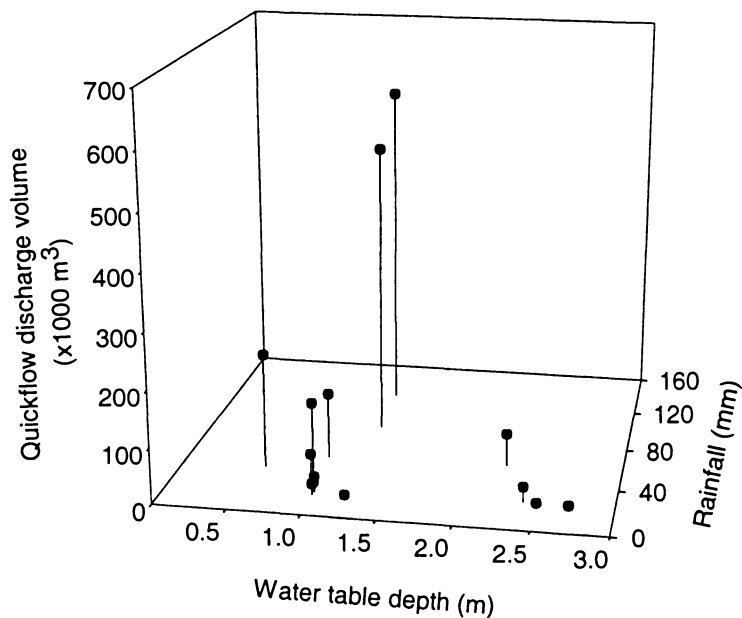


Figure 5.15. Individual rainfall event Data from Table 5.5 for developing Equation 5.5

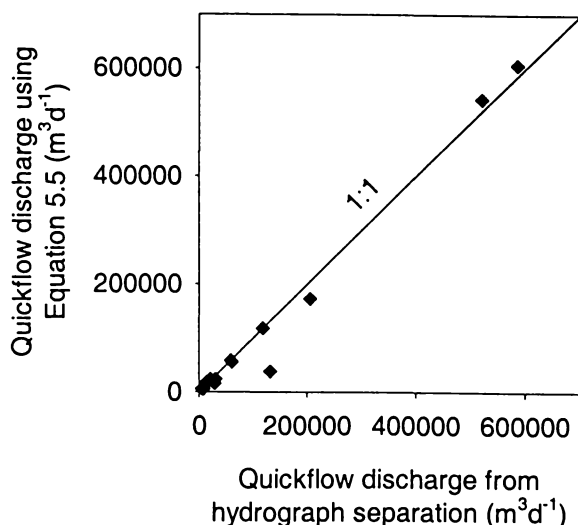


Figure 5.16. Comparison of the estimated quickflow using hydrograph graphical partitioning and estimated quickflow discharge using Equation 5.5 for the 14 rainfall events.

Table 5.5. Data used in deriving Equation 5.5 and the comparison of the measured and estimated quickflow discharge using Equation 5.5. The last two columns show the predicted quickflow discharge if the water table is at its highest and lowest level for the 14 rainfall events analysed.

Duration of Rainfall Event	Event Rainfall (mm)	Water table depth measured at well 4 (m)	Average evaporation (mm)	Measured Quickflow Discharge (m ³)	Predicted Quickflow Discharge (m ³)	0.5 m Water Table Depth Quickflow Discharge (m ³)	2.7 m Water Table Depth Quickflow Discharge (m ³)
21-23/11/99	24	0.99	3.67	13920	18219	40356	10271
25-30/11/99	55	0.97	3.17	118186	118328	255135	64933
13-14/12/99	17	1.23	7.55	8030	5697	16815	4280
15-18/2/99	23	2.49	5.80	5092	5548	24430	6218
25-26/1/00	58	2.21	3.45	60167	55431	262249	66743
2-3/3/00	22	2.70	6.30	6595	5664	22254	5664
7-9/5/00	25	2.40	2.37	28100	15821	72012	18327
28-30/6/00	89	1.22	1.73	521142	542515	1584260	403198
18-23/7/00	125	1.20	2.52	586174	606404	1733036	441062
12-13/8/00	21	1.01	2.45	30065	23606	53693	13665
26-27/8/00	27	0.96	1.55	58544	58366	124141	31594
31/8-1/9/00	42	0.57	2.30	205286	173012	197194	50186
23/09/00	19	1.00	2.20	20575	23624	53030	13496
26-27/9/00	37	0.93	4.05	131553	38019	77565	19740

5.6 Vegetation

A vegetation survey of the study site was performed to identify any plants that might be impacted as a result to a lowering of the water table.

Tree harvesting affected most of the Waitakere Ranges in the 100 years prior to the 1930's. As a result of this destruction there are few mature trees remaining in the field site area. The current vegetation consists of shrubs and regenerating native trees. The vegetation found in the field area is listed in Table 5.6, and their relative locations are shown in Figure 5.17. From this survey it has become apparent that the 'scrubby' vegetation such as Cutty grass (*Gahnia xanthocarpa*), and Kanuka (*Leptospermum ericoides*) dominate the swampy lowland areas, while Rimu (*Dacrydium cupressinum*) and Kauri (*Agathis australis*) are found on the well-drained ridges (Figure 5.18).

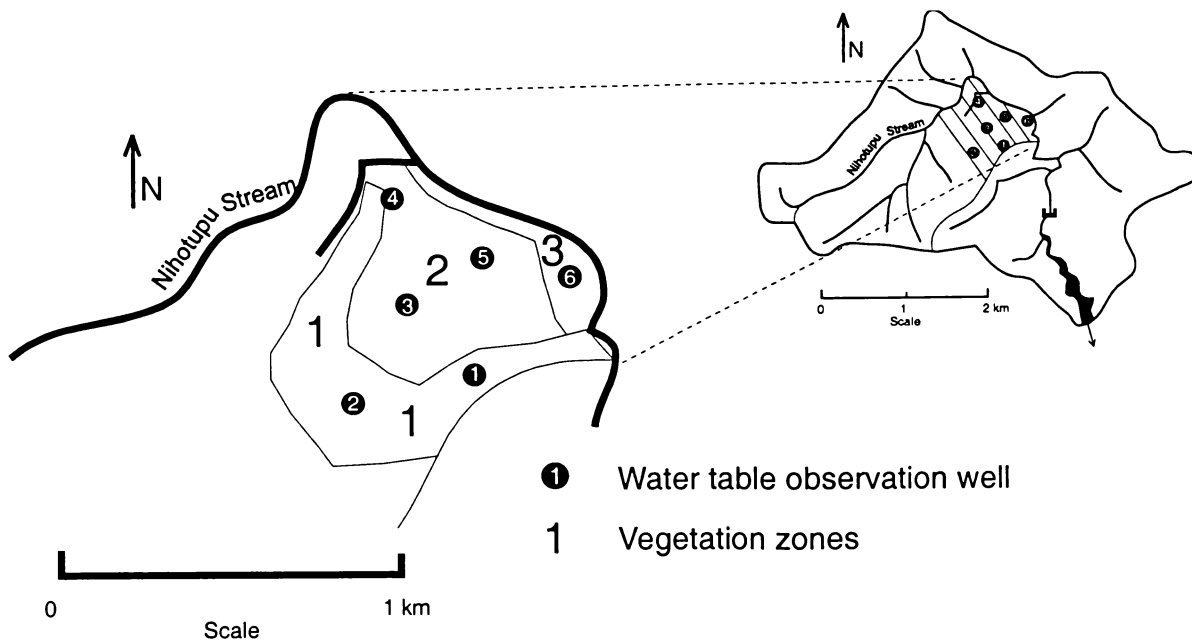


Figure 5.17. Location of the vegetation groups listed in Table 5.6. Zone 1: ridges, well drained. Zone 2: Swampy throughout the year. Zone 3: Well drained low elevation near the Nihotupu Stream.



Figure 5.18. Foreground shows Cutty grass *Gahnia xanthocarpa*, and Kanuka *Leptospermum ericoides* which dominate the swampy lowland areas, Rimu *Dacrydium cupressinum* and Kauri *Agathis australis* are found on the far ridge.

Table 5.6. List of vegetation found in the field site.

Section 1 Ridges, well drained	Section 2 Swampy throughout the year
Rimu <i>Dacrydium cupressinum</i>	Kanuka <i>Leptospermum ericoides</i> 75%
Akepiro <i>Olearia furfuracea</i>	Rimu <i>Dacrydium cupressinum</i> 25%
Lancewood <i>Pseudopanax crassifolius</i>	Flax <i>Phormium tenax</i>
Hangehange <i>Geniostoma ligustrifolium</i>	Braken <i>Pteridium esculentum</i>
Mingimingi <i>Cyathodes fasciculata</i>	Cutty grass <i>Gahnia xanthocarpa</i>
Kanuka <i>Leptospermum ericoides</i>	Hinau <i>Elaeocarpus dentatus</i>
<i>Coprosma australis</i>	Five finger <i>Pseudopanax arboreus</i>
Kauri <i>Agathis australis</i>	Mapou <i>Myrsine australis</i>
Tawari <i>Ixerba brexioides</i>	Mingimingi <i>Cyathodes fasciculata</i>
<i>Pittosporium tenuifolium</i>	
Ponga <i>Cyathea dealbata</i>	Section 3 Near the river, more forest like and better drainage, canopy at 3 m.
Lancewood <i>Pseudopanax crassifolius</i>	Ponga <i>Cyathea dealbata</i>
Ponga <i>Cyathea dealbata</i>	Rimu <i>Dacrydium cupressinum</i>
Cabbage tree <i>Cordyline australis</i>	<i>Pittosporium tenuifolium</i>
Mapou <i>Myrsine australis</i>	Kahikatea <i>Podocarpus dacrydioides</i>
Karaka <i>Corynocarpus Laevigatus</i>	Manuka <i>Leptospermum scoparium</i>
<i>Melicytus macrophyllus</i>	Toro <i>Myrsine salicina</i>
Akepiro <i>Olearia furfuracea</i>	<i>Corokia buddleioides</i>
<i>Coprosma lucida</i>	Heketara <i>Olearia rani</i>
	Rewarewa <i>Knightia excelsa</i>
	<i>Alseuosmia macrophylla</i>
	<i>Coprosma lucida</i>

5.7 Summary

The Upper Nihotupu catchment was selected as a trial site to simulate the use of horizontal wells to control the water table level in the Upper Nihotupu reservoir catchment, and in turn, have increased control of the stream inflow into the reservoir. This control may increase the current storage capacity of the reservoir without increasing its physical size.

For similar sized rainfall events in the Upper Nihotupu catchment there will be more quickflow discharge and reservoir spillage when the surrounding water table is nearer the ground surface, compared to a deeper water table. This relationship between water table depth and quickflow discharge indicates that if horizontal wells are able to lower the water table during winter there may be a fourfold reduction in the Nihotupu stream's quickflow discharge, and in turn, reduced reservoir spillage.

The hydrogeological information presented in this chapter was used to develop a numerical groundwater model of the study to enable the model-based evaluation of the operation of horizontal gravity wells for improving the functionality of the Upper Nihotupu reservoir (presented in Chapter 6).

Chapter 6

Model-Based Evaluation of Horizontal Wells

6.1 Introduction

Using the information in Chapter 5, a model-based evaluation of the operation of hypothetical horizontal wells in the Upper Nihotupu catchment is presented. A transient MODFLOW model of the Upper Nihotupu catchment aquifer is linked with a recharge model and reservoir operation model. The models are used to evaluate the effectiveness of the horizontal wells for enhancing the efficiency of the Upper Nihotupu reservoir.

The order of the MODFLOW model development presented in this chapter is: conceptual model creation, model domain development, aquifer¹ recharge model development, calibration, validation, and horizontal well simulation. A reservoir operation model of the Upper Nihotupu reservoir was also developed to evaluate the changes in reservoir spillage and water delivery² based on the MODFLOW output. The MODFLOW groundwater model, the aquifer recharge model and the Upper Nihotupu reservoir operation model were all incorporated and operated in a trial and error process with the aim of reducing the simulated spillage, and increase the water delivery from the Upper Nihotupu reservoir. Some of the concepts presented in this chapter are discussed in Brown and Bardsley (2002).

6.2 Conceptual Model

A conceptual model of the aquifer in the Upper Nihotupu study area was developed using the information presented in Chapter 5. The conceptual model provided a means of checking that all the appropriate data for the MODFLOW model had been collected, and provided a qualitative description of the various aquifer properties. The components of the Nihotupu catchment conceptual model are discussed below and shown in Figure 6.1.

¹ In this chapter the term 'aquifer' refers to the unconfined aquifer in the regolith zone of the study site in the Upper Nihotupu reservoir catchment.

² The term 'water delivery' refers to the discharge of reservoir water to the filter stations for potable consumption.

a) Model Extent and Aquifer Characteristics

The model domain exterior boundary follows the Nihotupu stream along the western and northern edges, and the catchment ridgeline along the southern and eastern edges (Figure 6.1). In the vertical extent only the regolith layer was modelled, consisting predominantly of sandy clay loams to an estimated depth of 20 m. The average horizontal hydraulic conductivity was 0.025 md^{-1} , specific yield 0.066, and porosity 0.62.

b) Flow System

The observed water table generally mimics the topography with the flow paths converging on the Nihotupu stream. Groundwater divides are assumed under the ridges between the neighbouring catchments. All aquifer recharge was from rainfall via soil infiltration. The estimated annual recharge was 440 mm. All groundwater discharge from the regolith layer is assumed to be via the Nihotupu stream and its tributaries. The annual specific discharge of the Nihotupu Stream measured at the Upper Nihotupu weir was 1040 mm. The aquifer recharge and catchment discharge are discussed in more detail in Chapter 5.

6.3 Model Development

The MODFLOW software (McDonald and Harbaugh, 1988), was selected as it enabled the simulation of the horizontal wells with the MODFLOW 'drainage' boundary condition, and produced water budgets for the various boundary conditions aiding the calibration and predictive evaluation of the Nihotupu groundwater model.

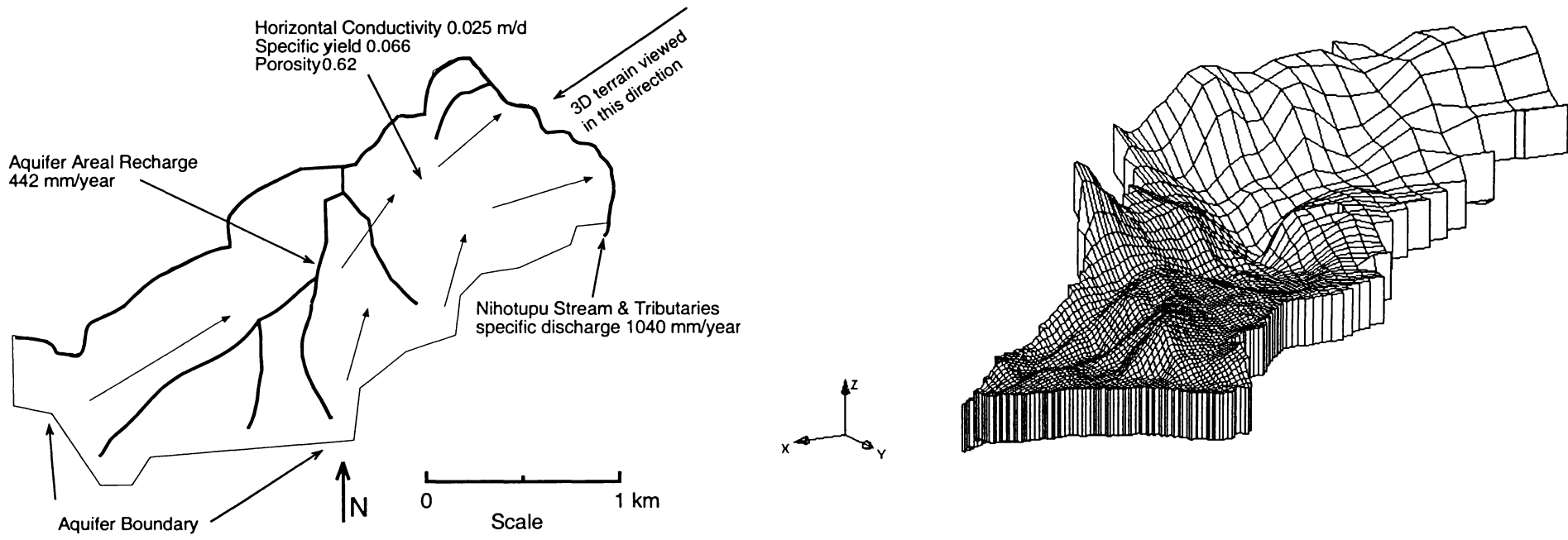


Figure 6.1. Conceptual model showing generalised groundwater flow direction, and three-dimensional model of the terrain (15 times vertical exaggeration).

6.3.1 Grid Discretization

The MODFLOW code utilises a finite difference grid that is block-centred. In the block-centred approach, the flux boundaries are located at the edge of the model. It was important to orient the grid to minimise the number of nodes that fell outside the model domain. These cells may not be active but they can still use up storage space in the arrays of the MODFLOW code. The size of the grid spacing was positioned as a function of the expected water table curvature, with smaller sized cells where the curvature of the water table was anticipated to be steeper. In the Nihotupu catchment, this was most likely to occur in the vicinity of relatively low hydraulic conductivity zones, the Nihotupu Stream and its tributaries, and the hypothetical horizontal wells. A compromise between grid size and practicality was required. A model of very small grid spacing is more likely to be able to accurately represent the groundwater flow dynamics of an aquifer system, but with slow computational speed, and vice versa for larger grid spacing.

The cell dimensions in the southwest corner of the model domain are 200 by 150 m, reducing to the northeast until a minimum cell size of 20 by 20 m was reached (Figure 6.2). The finer grid size was required in this area of the aquifer to accurately simulate the drawdown of the water table produced by the draining of the aquifer by hypothetical horizontal wells. It was found that a cell size of 20 by 20 m accurately reproduced the water table and discharge of the wells, further details are given in section 6.5.1.

6.3.2 Digital Elevation Model

The catchment topography for the MODFLOW model was based on a 25 m grid digital elevation model using the 20 meter contours of the NZMS 260 series map Q11, scale 1:50,000 (Stirling, 1979). Refinement of the elevation of dip-wells and stream water level monitoring points was achieved using a geographic positioning system (GPS), with a maximum error of ± 0.1 m in the horizontal and ± 2 m in the vertical. Multiple readings were taken at each site to reduce the degree of error. The dense vegetation canopy required the GPS receiver to be attached to a survey staff and raised five metres above the ground to enable contact with the satellites.

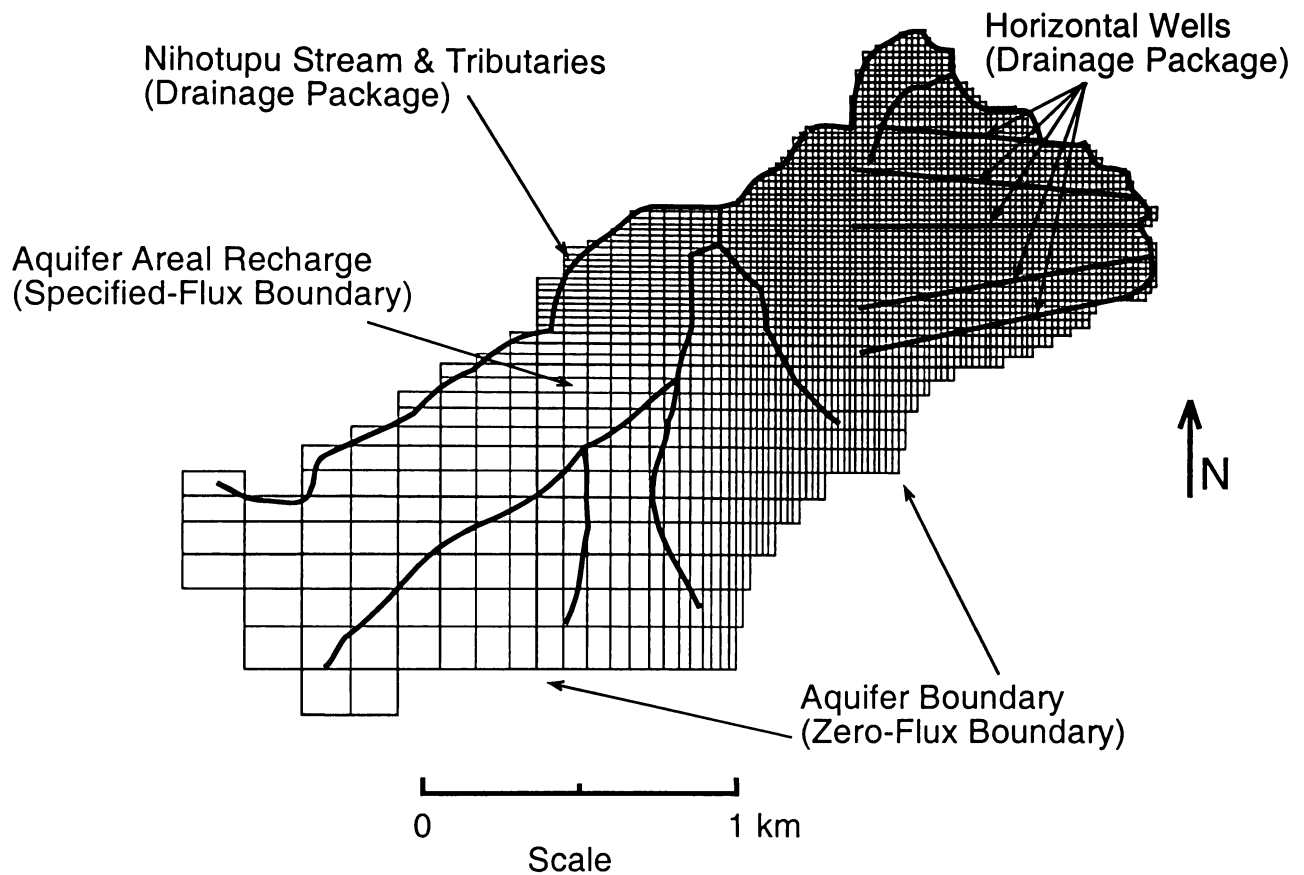


Figure 6.2. MODFLOW grid and boundary conditions

6.3.3 Boundary Conditions

The selection of the boundary conditions representing the physical and hydraulic characteristics of the aquifer was an important step in model construction. The flow pattern produced by the boundary conditions needs to make hydrological sense, especially during simulations where the hydrological stresses applied to the model may be different than those of the calibration and validation stage (Anderson and Woessner, 1992a).

a) Nihotupu stream

The MODFLOW drainage module was selected to represent the Nihotupu Stream in the model. Each model cell with a drainage boundary condition requires the elevation of the stream bed and a conductance value. The initial stream elevations were based on the digital elevation model of the catchment and were adjusted during the calibration process. The conductance is a lumped term describing all the head loss between the drain and the groundwater system. Head loss is caused by converging flow patterns near the drains,

presence of foreign material surrounding the drain, drain pipe material, and blocking of the drain from plant roots (McDonald and Harbaugh, 1988). In most situations, the detailed information required to calculate this conductance value are not available (Guiguer, 1998). The conductance value of the Nihotupu Stream was adjusted during calibration.

The drainage module limits the groundwater discharge from the aquifer into the Nihotupu Stream by the difference in elevation of the aquifer water table above the drainage boundary elevation. If the water table rises above the drainage module elevation then there will be an increase in baseflow discharge for the Nihotupu Stream. If the aquifer water table declines below the drainage boundary elevation there will be no discharge from the aquifer to the stream and a decrease in the baseflow discharge. This is particularly important in simulations involving the operation of the horizontal wells, where sections of the water table may be dewatered below the level of the stream. This situation cannot be easily modelled with a specified head boundary, or the MODFLOW river module. Both these boundary conditions require the stream's water level rather than stream bed elevation to be defined by the model user for each model time step. If incorrect stream water level elevations are selected, this may result in unrealistic recharge from the stream to the aquifer and an incorrect baseflow discharge from the simulated Nihotupu Stream.

b) Catchment no-flow boundary

The southern and eastern margins of the model are aligned with a topographic ridge. It is assumed that the ridge is a zone of groundwater divergence and is represented by a zero-flux flow boundary in the model.

6.3.4 Model Initial Conditions

The model's initial water table condition was based on the average levels recorded during March 2000 and 2001 at the six dip wells. In areas where there were no water table observations, additional estimates were based on observed springs, and the elevation of the Nihotupu Stream and its tributaries. The model simulated a four-month lead-in period (starting in March) prior to the calibration period between 26 July 1999 and 30 June 2000. Without a lead-in period, a good fit between observed and modelled water levels was achieved but not for the Nihotupu Stream's baseflow between July and September 1999. This was most likely due to inconsistencies between the initial conditions and the model

hydrological inputs and parameters (Franke, Reilly and Bennett, 1987). The error was isolated to the early model time steps due to the reduced influence of the initial model conditions with simulation progress (Anderson and Woessner, 1992a). This was rectified with the four-month lead-in period.

6.3.5 Aquifer Recharge Model

Recharge to the aquifer was defined as the portion of the rainfall that reached the water table. The MODFLOW model input of recharge was calculated as a 5-day average flux (md^{-1}) using the aquifer recharge model (ARM) developed during this study for the Upper Nihotupu catchment. The ARM is shown in Figure 6.3. The recharge was based on the catchment's water balance, with some additional upper and lower bounds, which are discussed below.

For the Nihotupu catchment, rainfall minus quickflow stream discharge, minus evaporation gives an estimate of the stream's baseflow, which over a long period of time is equivalent to the aquifer's recharge, discussed in section 5.5.2.1. However, over a 5-day period, the baseflow does not represent the corresponding aquifer recharge. Thus, upper and lower bounds have been applied to the recharge estimated by the ARM. These bounds were adjusted during the calibration of the MODFLOW model. The upper bounds reduce the recharge produced from large rainfall events. The lower bound maintains a minimum level of recharge during periods when the ARM water balance segregation of evaporation and quickflow discharge results in a very low recharge.

The total recharge from the ARM applied to the calibrated MODFLOW model between March 1999 and May 2001 was 460 mm, or 27 % of the rainfall. This matches the recharge estimate in section 5.5.2.1 for the two hydrograph separation methods.

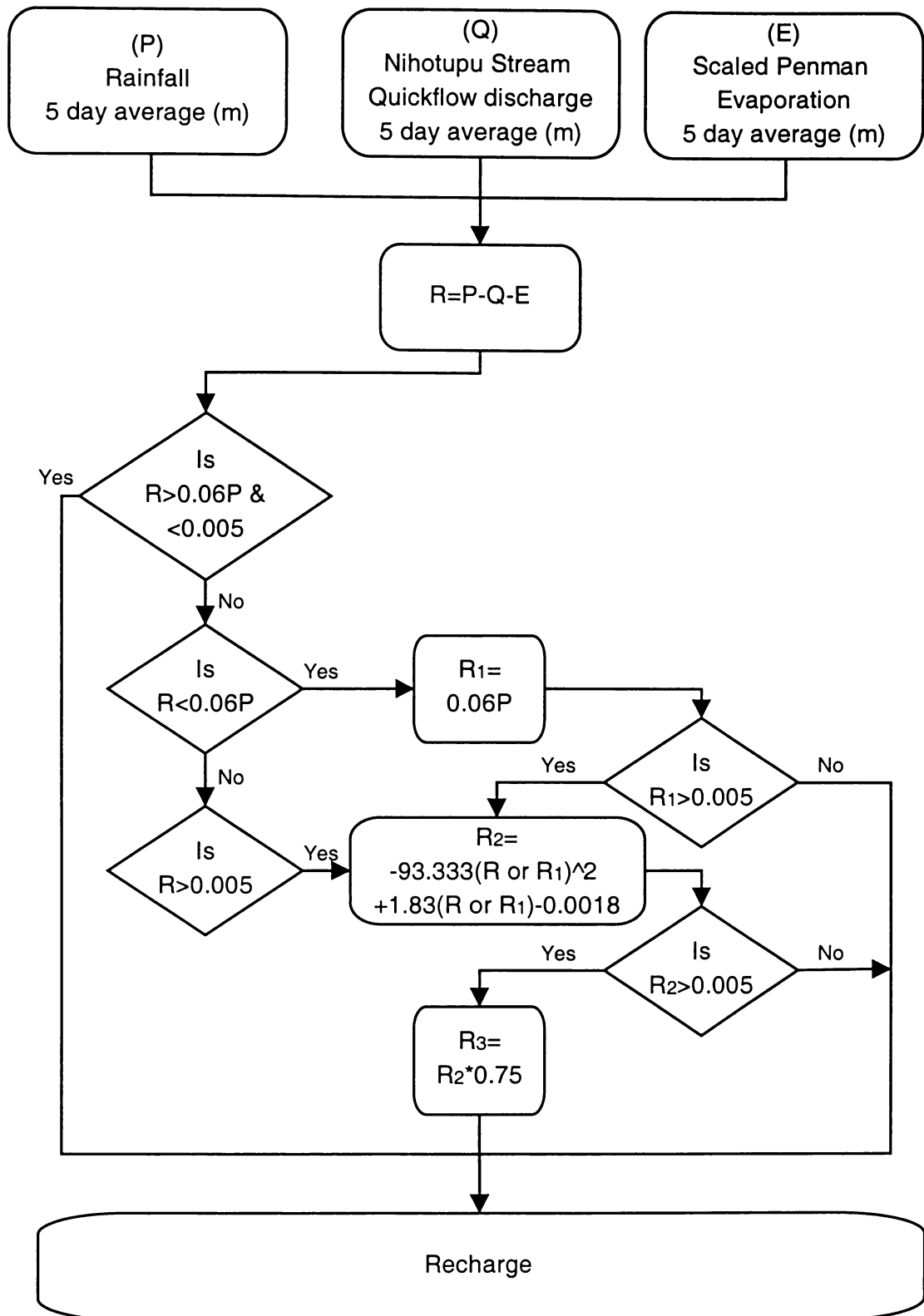


Figure 6.3. Flow chart of Upper Nihotupu aquifer recharge model.

6.4 Transient Model Calibration and Validation

A trial and error process was used to calibrate the transient MODFLOW model using observed and modelled water table levels, and Nihotupu Stream baseflow discharge between 26 July 1999 to 30 June 2000. The validation period was for data observed between 1 July 2000 and 31 May 2001.

Calibration was achieved by dividing the model domain into ten separate geological zones. The delineation of each zone was adjusted during the calibration process. The final zones and hydraulic parameters used to achieve model calibration are shown in Figure 6.4. All the parameters are within a reasonable range to those measured in the field and presented in Chapter 5. The conductance value required by the MODFLOW drainage boundary representing the Nihotupu Stream and its tributaries was also adjusted during calibration. A conductance value of 3 md^{-1} was selected for the Nihotupu Stream, and 1 md^{-1} for the tributaries.

Contour maps of the modelled and observed water levels were used to provide a qualitative measure of the spatial distribution of model error during the initial stages of model calibration. Time series of the modelled and observed Nihotupu stream baseflow discharge were compared to ensure they followed the same temporal trend, and were of similar magnitude (Figure 6.5a). This process was repeated for the time series of the modelled and observed water levels for the six observation wells to ensure the data sets followed the same temporal trend, and were of similar magnitude (Figure 6.5b–g). Quantitative evaluation of the goodness-of-fit between modelled and observed data was performed using the root mean squared error (RMSE) and the mean absolute error (MAE).

The model calibration was terminated when an RMSE of 0.61 m and MAE of 0.48 m was achieved between the observed and modelled water levels, and an RMSE of 0.63 mmd^{-1} and MAE of 0.51 mmd^{-1} was achieved between the observed and modelled specific stream baseflow discharge. For the calibration period, the observed and modelled total baseflow specific discharge volumes were 382 mm and 348 mm respectively.

The model validation produced a slightly better fit with an average RMSE of 0.44 m and MAE of 0.33 m between the observed and modelled water levels, and an RMSE of 0.61 mmd^{-1} and MAE of 0.53 mmd^{-1} was achieved between the observed and modelled

specific stream baseflow discharge. For the validation period, the observed and modelled total baseflow specific discharges were 426 mm and 380 mm respectively.

There was zero percent difference in the water inputs and outputs of the model's cumulative volumetric budget for the calibration and validation periods. The components of the budget are listed in Table 6.1.

One limitation of the model is due to its spatial extent, covering an area larger than the study area, with no water table observation data available to calibrate the southern end of the model. The model was extended beyond the study area to enable the model boundaries to align with the existing physical catchment boundaries. Errors in the water table level in the extended area of the model domain did not have a significant effect of the water levels in the study area section of the model. The sensitivity of the model to the parameters applied to achieve calibration are presented in Appendix K.

The model output data produced during the calibration and validation process provides a control data set for evaluating the changes in water table depth and stream discharge as a result of the simulated horizontal well operations.

Zone	Conductivity (m/d)		Specific yield	Porosity
	Horizontal	Vertical		
1	0.30	0.15	0.01	0.57
2	0.40	0.15	0.01	0.57
3	0.25	0.15	0.02	0.57
4	0.20	0.15	0.02	0.57
5	0.10	0.15	0.02	0.57
6	0.50	0.15	0.02	0.57
7	0.45	0.15	0.02	0.57
8	0.15	0.15	0.01	0.57
9	0.60	0.15	0.02	0.57
10	0.08	0.15	0.01	0.57

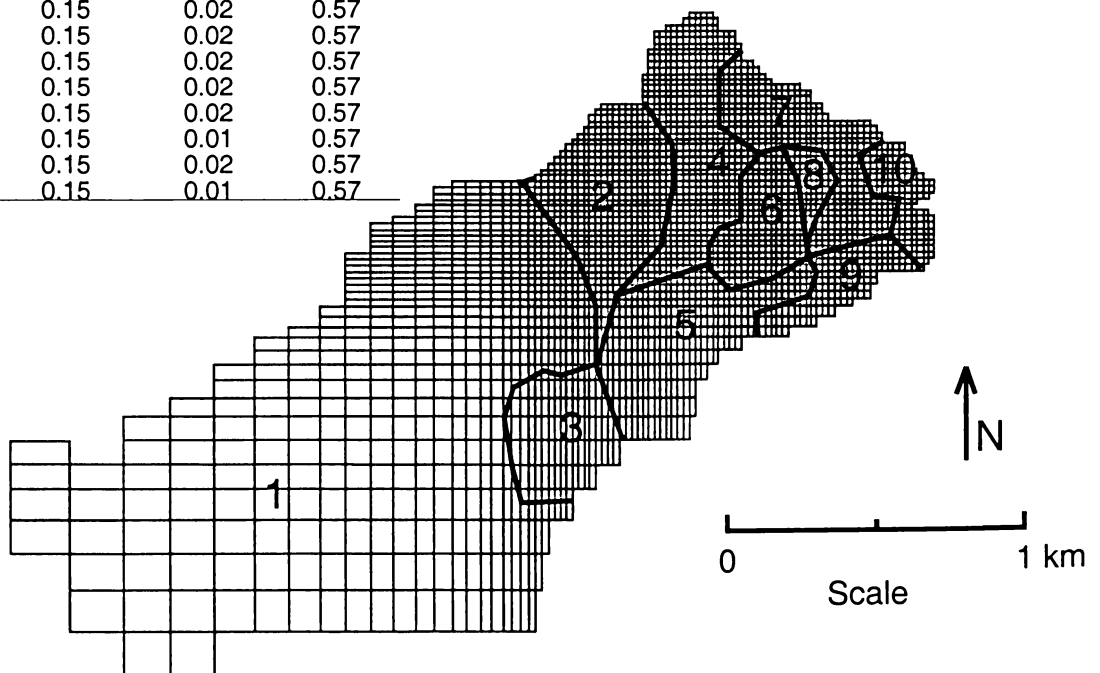


Figure 6.4. Hydraulic parameters and zones for the calibrated model.

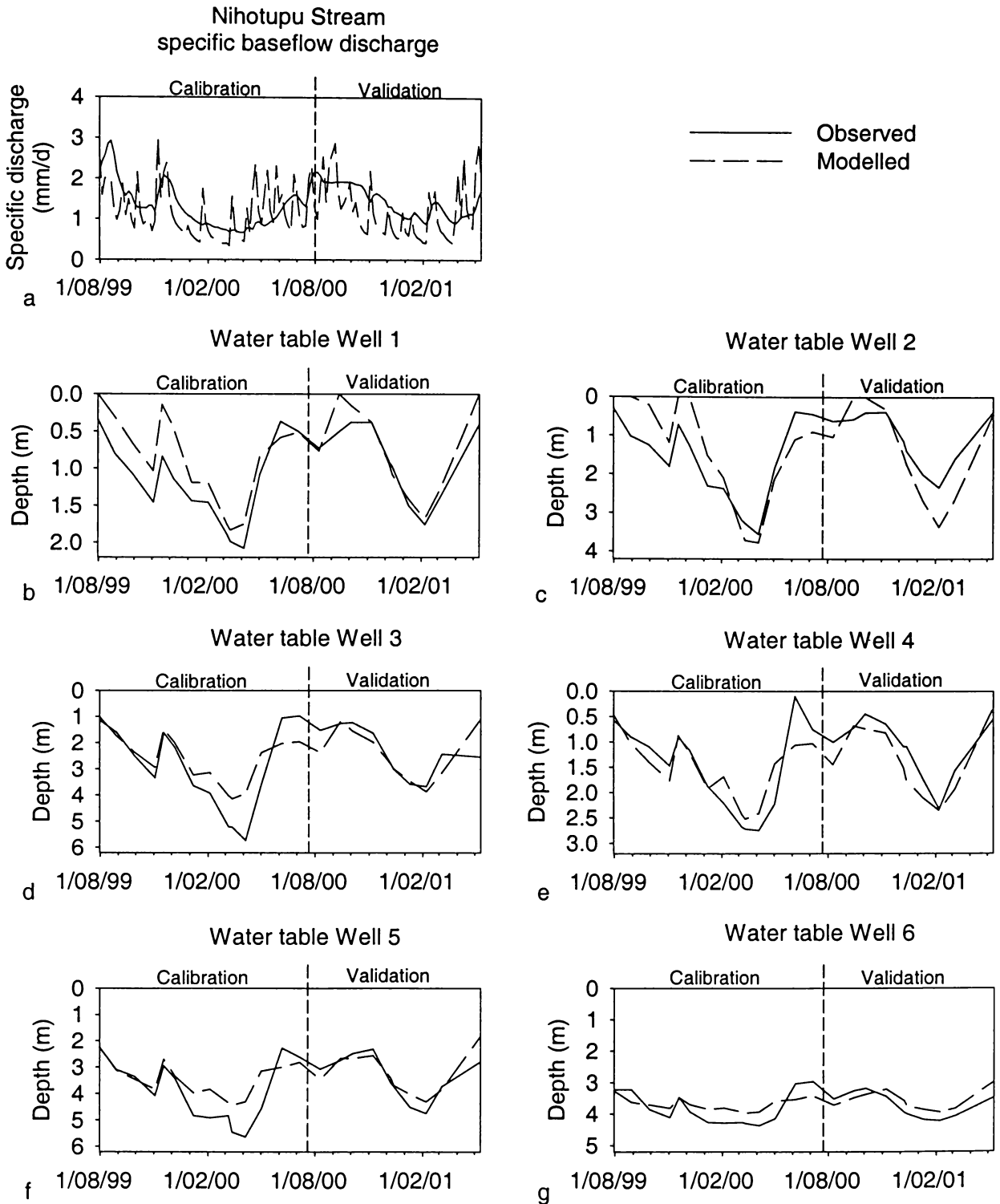


Figure 6.5. Calibrated results for (a) Nihotupu Stream baseflow discharge and (b-g) water table levels between the 26 July 1999 to 30 June 2000. Validation was between 1 July 2000 to 31 May 2001 respectively.

Table 6.1. Model cumulative volumetric budget for both the calibration and validation periods.

Cumulative Volumes (In)	m ³	Cumulative Volumes (Out)	m ³
Storage	730830	Storage	790342
Recharge	2158795	Drains	2099273
Total In	2889625	Total Out	2889615
In - Out	10	% Discrepancy	0

6.5 Preparation for Simulation of Horizontal Wells

The operation of the horizontal wells was simulated for the period 1 November 1999 to 31 May 2001, this period coincides within the model calibration and validation periods (26 July 1999 to 31 May 2001). The model output produced during the calibration and validation process provides a control data set of water table levels and Nihotupu Stream baseflow discharge without the simulated operation of the horizontal wells. This control data set was used for evaluating the changes in water table depth and stream discharge as a result of the simulated horizontal well operation. The actual Nihotupu reservoir delivery and spillage during this period also provided a control data set to evaluate the operation of the simulated horizontal wells.

6.5.1 Parameter Estimation for the Horizontal Well Boundary Condition

The MODFLOW drainage package was used to simulate the hypothetical horizontal wells. This boundary condition is suitable for the simulating underground drains (Pohll and Guitjens, 1994), or in this case horizontal gravity wells.

The conductance value required by the MODFLOW drainage boundary condition used to simulate the horizontal wells was estimated using the Kraijenhoff van de Leur-Maasland analytical model described by Wesseling and Kessler (1973). The Kraijenhoff van de Leur-Maasland model was derived to determine the discharge and water table elevation between two parallel drains (Wesseling and Kessler, 1973). A simple rectangular aquifer was modelled with four parallel equally spaced drains using both MODFLOW and the Kraijenhoff van de Leur-Maasland analytical model. The conductance of the drainage boundary in the

MODFLOW model was adjusted until the drain discharge and water table elevations between the parallel drains matched those of the analytical model. From this analysis a maximum drain conductance of 4.2 md^{-1} (for a model cell size of 20 by 20 m) was selected when the horizontal wells are open and draining the aquifer, and 0 md^{-1} when they are closed. A description of the analytical and numerical models is given in Appendix L.

Due to the irregular boundaries and homogeneous nature of the Nihotupu model, a trial and error process was used to estimate the appropriate number and location of the horizontal drains. The process indicated that five horizontal wells in a fan configuration produced good results (Figure 6.2). Using less than five wells resulted in some areas of the study area not being substantially drained. Using more than five wells did not significantly improve the drainage of the study area. The horizontal wells were positioned six metres below the lowest observed water table (April 2000). The lowest end of the wells was positioned at the same elevation as the Nihotupu stream.

The discharge capability of the wells was quantified using the Hydraulic Research Station Chart for a plastic pipe (Chadwick and Morfett, 1991). Based on the average well gradient used in the model (2.25/100), and the well diameter (0.15 m), a maximum discharge of $3,197 \text{ m}^3\text{d}^{-1}$ was assigned to each well. The MODFLOW drainage package assumes that the drain is only half full so the maximum model discharge for each horizontal well is $1,598 \text{ m}^3\text{d}^{-1}$.

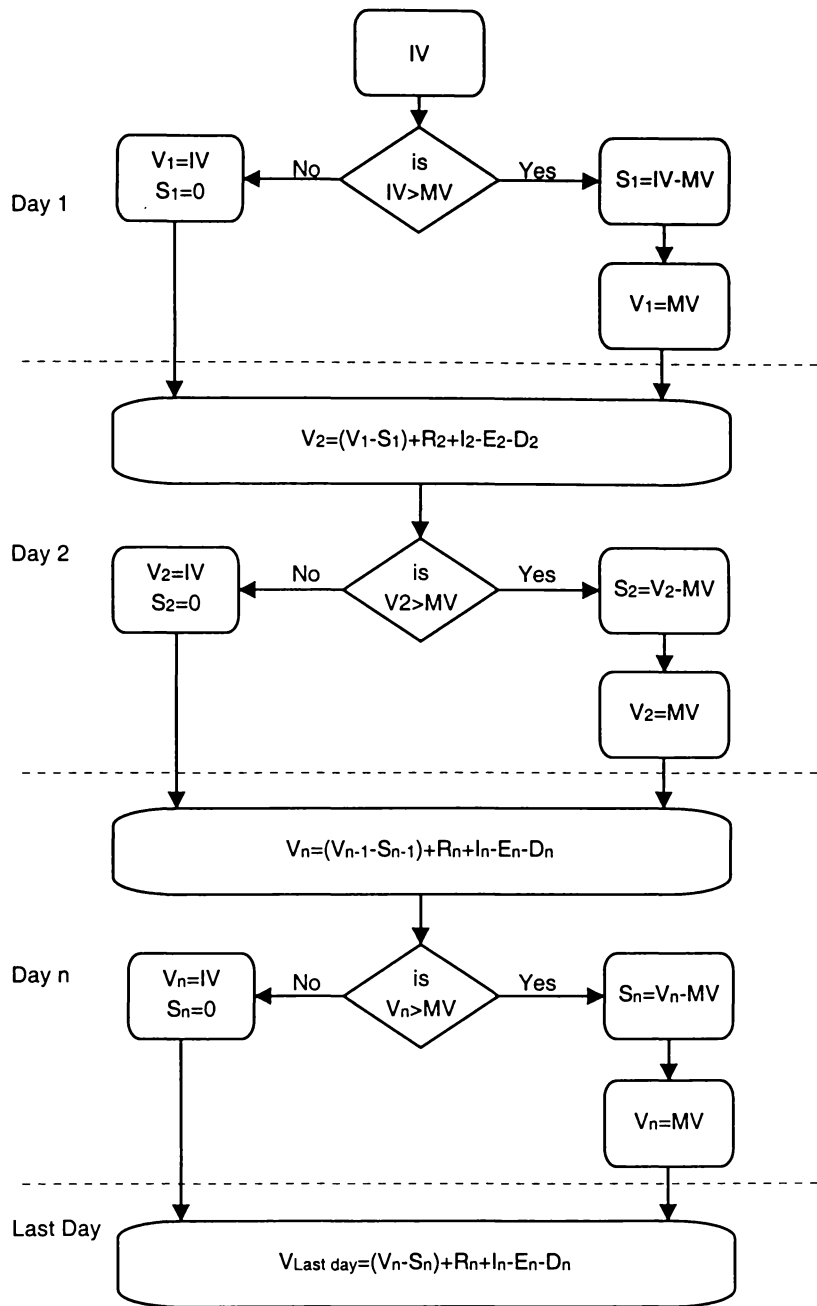
6.5.2 Upper Nihotupu Reservoir Operation Model

The Upper Nihotupu reservoir operation model (UNROM) was developed during the present thesis to evaluate the daily change in reservoir spillage and water delivery as a result of the MODFLOW simulation of the horizontal wells. UNROM is based on the reservoir's water balance, shown in Figure 6.6 and described below. UNROM is an isolated operation model and does not directly include aspects such as water demand and the state of the other nine reservoirs. However, these aspects are indirectly applied by not allowing the adjusted delivery from the reservoir to exceed the maximum yield of $31,000 \text{ m}^3\text{d}^{-1}$, or lower the lake level below the minimum summer volume of $1,156,000 \text{ m}^3$ that occurred for the control data set. The control data set is the result of Watercare's previous operation model WHODAT (Weekly Headworks Operating Data) and the current 'water source optimisation model' which were both designed to optimise the combined use of Watercare's 10 reservoirs and Onehunga groundwater source.

The two main inputs to UNROM are the Nihotupu stream discharge and rainfall directly on the reservoir water body (area of 120,000 m²). The main outputs are spillage, delivery for water supply, and evaporation from the water body. The Nihotupu inflow will be provided as an output of the MODFLOW model during the simulation of the Horizontal wells. Rainfall is applied as an average of the Upper Huia and Upper Nihotupu rain gauges. The evaporation from the reservoir surface is simply the Penman potential evaporation from the Auckland airport. Delivery to the filter station is adjusted by trial and error to minimise spillage while keeping within the limitation discussed above.

The UNROM shown in Figure 6.6 requires an initial measured reservoir water volume. If the initial reservoir volume is less than the reservoir maximum volume of 2,216,300 m³ (stage height 33.84 m) then there is no spillage. If the volume exceeds the maximum volume the difference is lost to spillage and the new reservoir volume equals the maximum volume. For each day the process is repeated, except all reservoir inputs are added to the previous days calculated reservoir volume, and all outputs are subtracted.

The accuracy of the UNROM was first tested to check if the Nihotupu Stream inflow measured at the weir between November 1999 and May 2001 could be accurately reproduced when the reservoir spillage was known (section 5.5.4). Figure 6.7 shows that the stream discharge estimated by the UNROM and MODFLOW is very similar to that measured by the Upper Nihotupu weir. The MODFLOW Nihotupu stream discharge is calculated as the MODFLOW baseflow output plus the quickflow component from the ARM (section 6.3.5). The slight difference between the two methods shown in Figure 6.7 could result in large errors when estimating the reservoir spillage. To account for this error during the simulation of the horizontal wells, the UNROM estimate, as calculated without the operation of the horizontal wells (control data set) was applied to UNROM. However, to account for the change in discharge due to the simulated operation of the wells, the UNROM Nihotupu stream discharge (control data set) was scaled by the ratio of the MODFLOW control data set to the MODFLOW discharge for the simulation of the horizontal wells.



IV = Initial reservoir volume (m^3)
 MV = Reservoir maximum volume ($2,216,300 m^3$)
 S = Daily Reservoir spillage (m^3d^{-1})
 V = Reservoir volume (m^3)
 R = rainfall volume directly on reservoir volume (m^3d^{-1})
 I = Nihotupu Stream inflow (m^3d^{-1})
 E = Evaporation from reservoir (m^3d^{-1})
 D = Water delivery from reservoir (m^3d^{-1})
Subscripts for each parameter indicate the relevant day.

Figure 6.6. Upper Nihotupu Reservoir operation model

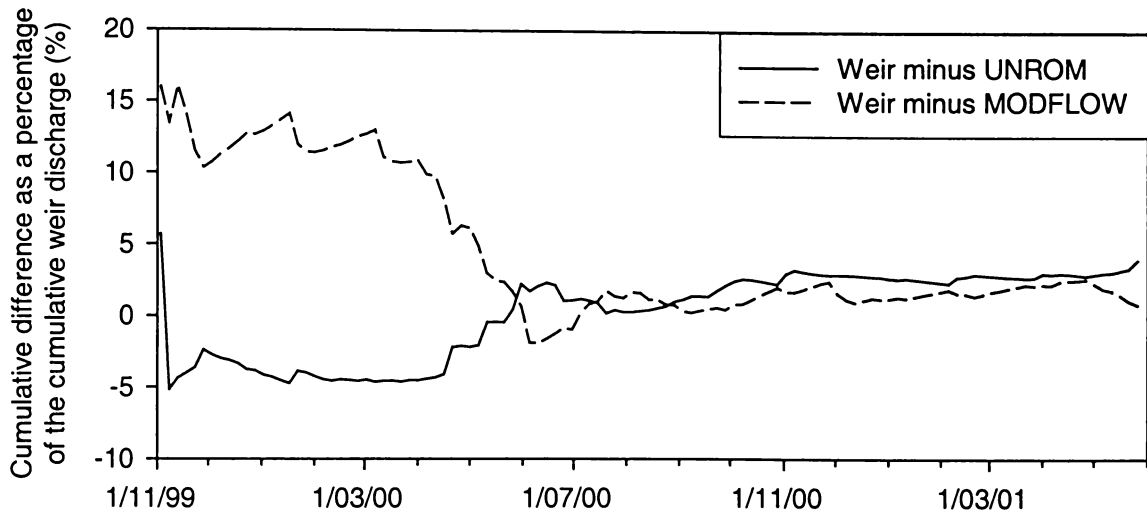


Figure 6.7. Cumulative difference between the Nihotupu Stream discharge measured at the Nihotupu weir and the discharge calculated by the UNROM and MODFLOW represented as a percentage of the cumulative weir discharge.

6.6 Horizontal Well Simulation for Enhancing Reservoir Efficiency

The operation of the horizontal wells was simulated for the period 1 November to 31 May 2001. However, the horizontal wells were only simulated as being open during the late summer and winter months (17 March 2000 to 30 November 2000), with the exception of their temporary closure for the duration of seven storm events during the winter. The operation of the wells over the summer and winter months demonstrates how the horizontal wells can result in increased stream discharge and reservoir delivery in the late summer months, and reduced peak discharge and reservoir spillage from storm events during the winter.

A trial and error process was used to apply a new recharge rate to the MODFLOW model in response to the draining of the aquifer by the hypothetical horizontal wells, while at the same time trying to reduce the Upper Nihotupu reservoir spillage and increase the water delivery. This process is shown in Figure 6.8, and the various components of the process are discussed below.

a) Recharge

The recharge applied to the calibrated MODFLOW model was adjusted through a trial and error process to account for the increased infiltration capacity of the aquifer due to lowering of the water table using horizontal wells.

To calculate the adjusted recharge rate for the MODFLOW simulation of the horizontal wells, the arbitrarily chosen initial estimate of the reduction in quickflow discharge is applied to the aquifer recharge model (ARM), presented in section 6.3.5. To maintain the catchment's water balance, the reduction in quickflow discharge would be balanced by a corresponding increase in aquifer recharge. The maximum recharge limit is set equivalent to the fourfold reduction in quickflow discharge, as discussed in section 5.5.5; or a recharge rate less than this if it results in the simulated water table in the MODFLOW model rising above the level had the wells not been simulated (control data set). The reduced quickflow discharge applied to the ARM is also added to the baseflow discharge simulated by the MODFLOW model to give the total discharge for the Nihotupu stream.

b) MODFLOW

In the MODFLOW aquifer model, the only adjustable condition, other than the recharge, was the conductance parameter of the drainage boundary representing the horizontal wells. For fully open wells this was set at 4.2 md^{-1} and for closed wells 0 md^{-1} . Due to the wells being simulated as fitted with an outlet valve, a variable discharge between these limits was possible.

Prior to opening the wells after a simulated storm event the water table elevations were checked. If the simulated water table level were below the elevation for the control data set the well outlet valve was opened fully with a conductance of 4.2 md^{-1} . This occurred for the first three times when the wells were simulated opened. However, if the water tables were at the same level during the simulation the wells' outlet valves were only partially opened using a conductance of 1.5 md^{-1} . This occurred for the remaining four times when the wells were simulated opened. If the wells were opened fully when the water tables were at the same level, the resulting Nihotupu Stream discharge would be greater than for the control data set, and produce more spillage rather than reducing it.

The total Nihotupu Stream discharge based on the MODFLOW baseflow output and the quickflow discharge from the ARM was scaled up by the ratio of model catchment area to the drained area (4.9:1). This scaling provided a discharge estimate of the whole catchment under the influence of horizontal wells. This scaling is justifiable based on stream flow gaugings during this study. The stream flow gaugings identified that the change in stream discharge as a result of taking measurements at different locations along the stream was equivalent to the change in catchment area from one measurement location to another (section 5.5.2). It is this scaled total discharge which was applied to the aquifer-reservoir operation model (Figure 6.8).

c) Aquifer-Reservoir Operation Model

The Aquifer-Reservoir Operation Model (AROM) (Figure 6.8), combines the Upper Nihotupu Reservoir Operation Model (UNROM) discussed in section 6.5.2, the MODFLOW model with the horizontal wells, and the Aquifer Recharge Model (ARM).

The ARM was adjusted to provide the maximum recharge to the MODFLOW model without reducing the quickflow discharge of the Nihotupu Stream by more than fourfold, or allowing the MODFLOW water table to rise above the level had the wells not been operating. The MODFLOW simulated horizontal wells are opened and closed by adjusting the drain conductance value, with the purpose of reducing reservoir spillage and increasing reservoir delivery. The UNROM evaluates the change in reservoir spillage and delivery due to the modified Nihotupu Stream hydrograph produced by the MODFLOW model and ARM. These models were operated in a trial and error process as shown in Figure 6.8 to achieve the results discussed in the following section.

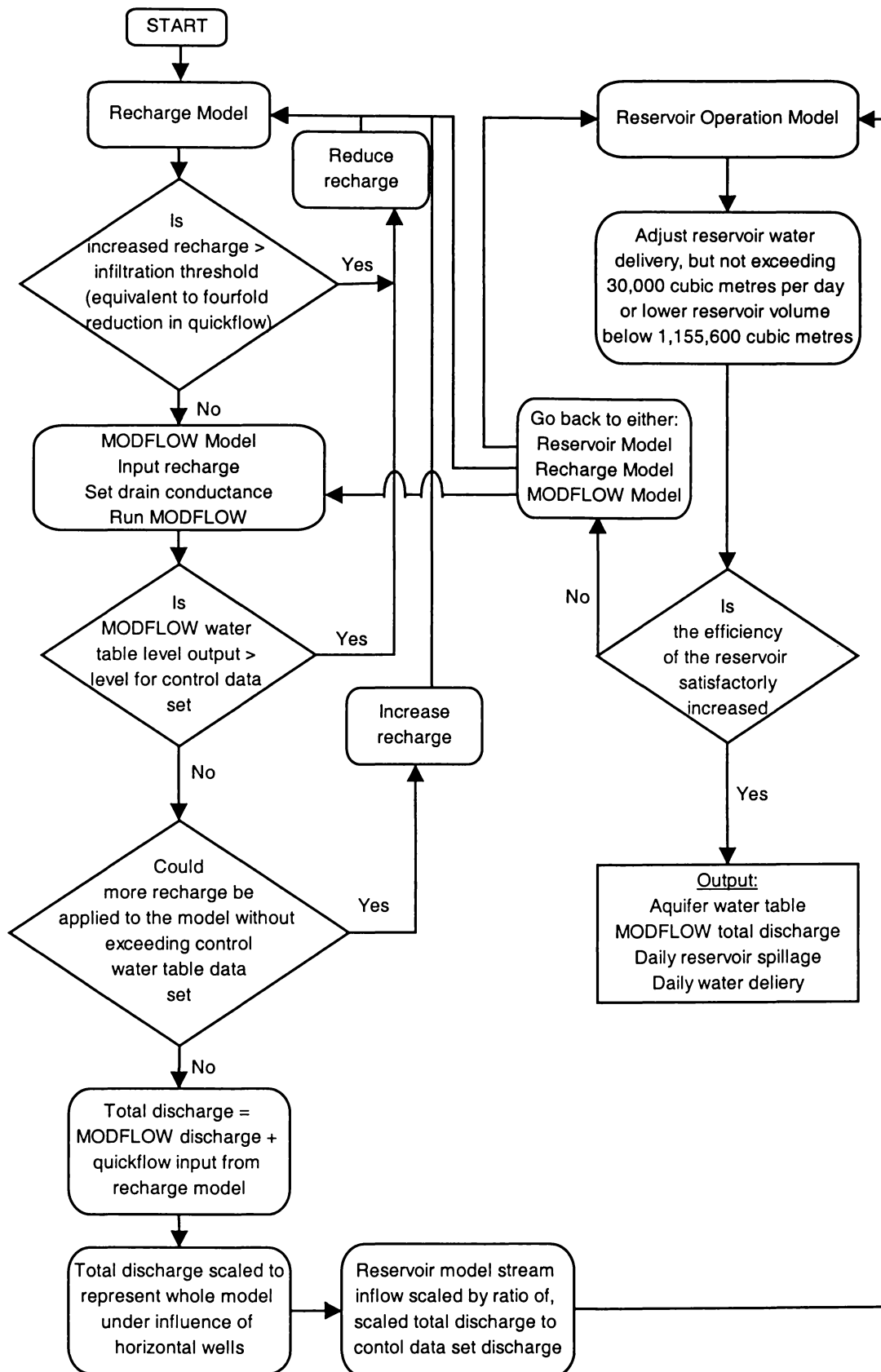


Figure 6.8. Trial and error operation of the aquifer-reservoir operation model (AROM) which encompasses the MODFLOW model, aquifer recharge model (ARM), and Upper Nihotupu reservoir operation model (UNROM) to determine the effect of the hypothetical operation of the horizontal wells.

6.6.1 Observations

The model data produced during the calibration and validation process provided a control data set for evaluating the simulated changes in water table depth and stream discharge as a result of the horizontal wells operation.

Figure 6.9a shows the spatially averaged water table depth for the control data and the operation of the wells. Once the wells are opened (17 March 2000), prior to the first rainfall event, drainage causes a steady decline in water table levels to a maximum depth of 3 m below the control data. After this, the water table rises during rainfall events when the wells are closed, and declines afterwards when the wells are reopened. The increased water storage in the aquifer produced by the operation of the wells is indicated in Figure 6.9a by the increased magnitude of the water table fluctuations compared to those of the control data. In the middle to late period of the simulation (June 2000) the storage capacity of the aquifer is reached, this is shown where the water table rises to the level of the control water table. The wells are finally closed on the 30 November 2000 and there is a gradual recovery of the water table.

The applied recharge for the one-year period from when the horizontal wells were first simulated as being open (17 March 2000 to 17 March 2001) was 719 mm, or 42% of the total rainfall. This is an increase of 58% on the original recharge of 456 mm applied to the calibrated model without the simulated horizontal wells.

Figure 6.9b shows the Nihotupu Stream's specific discharge for both the control data and for the operation of the horizontal wells (catchment area 8.5 km²). The simulated operation of the horizontal wells produced a hydrograph with reduced peak quickflow discharge and increased baseflow. For the first 30 days (17 March 2000 to 15 April 2000) after the wells were opened the total specific discharge volume for the 30 day period increased from 20.3 to 78.2 mm (or 285% increase). For the seven storm events during the winter months (16 April 2000 to 30 November 2000) when the horizontal wells were temporarily closed, there was a reduction in the average peak discharge for each event from 10.34 to 7.94 mmd⁻¹ (or 23% reduction). However, after each event the stored water was released by reopening the wells producing a delayed discharge peak, which exceeded the control discharge at that time. Between 16 April 2000 and 30 November 2000, the total specific discharge volume decreased from 926.6 to 887.8 mm. From the 30 November 2000 to 17 March 2001 the wells were simulated closed to allow the recovery of the water table to its

normal position for summer. During this recovery period there was a reduction in the stream's total specific discharge volume from 152.7 to 145.4 mm. For the 12-month period from when the wells first opened, 17 March 2000 through to 17 March 2001 there was an increase in specific discharge volume of approximately 1%.

Figure 6.9c shows the cumulative daily specific spillage for the Upper Nihotupu reservoir (catchment area 10.17 km²). The operation of the horizontal wells, combined with the increased delivery from the reservoir has reduced the spillage by 30 mm or 10 % of the original spillage, this is also equivalent to 12 days of average delivery (24,000 m³d⁻¹).

Figure 6.9d shows the cumulative daily specific delivery of water from the Upper Nihotupu reservoir (catchment area 10.17 km²). When the wells were open for the 30 days during summer, draining the aquifer, the increase in stream inflow enabled an average increase in delivery from 0.98 to 2.5 mmd⁻¹ (or 10,000 to 25,000 m³d⁻¹). This increase was also equivalent to 19 days of average annual delivery (24,000 m³d⁻¹). However, over the whole year the reduction was only equivalent to 13 days average delivery. After the wells were finally closed there was a reduction in groundwater inflow to the Nihotupu Stream as the water table recovered. This reduction in discharge was equivalent to six days of average delivery.

The maximum discharge of each well was 765 m³d⁻¹ with an average of 350 m³d⁻¹. This is within the limits of the wells capacity estimated to be 1600 m³d⁻¹ in Section 6.5.1.

6.6.1.1 *Application to Other Catchments*

The hydrogeology of Watercare's 10 reservoir catchments is discussed in Chapter 4.

Other than the Upper Huia reservoir catchment, the remaining catchments may be suitable for using horizontal wells to increase the storage capacity of the reservoirs. This statement is based on limited hydrogeological information which includes; a strong baseflow component shown by the flow-duration curves, indicating relatively high groundwater discharge into the streams, and the regolith layers are of similar thickness and porosity. Further investigations are required to evaluate if reservoir spillage in the catchments is related to aquifer water tables close to the ground surface, as identified in the Upper Nihotupu catchment.

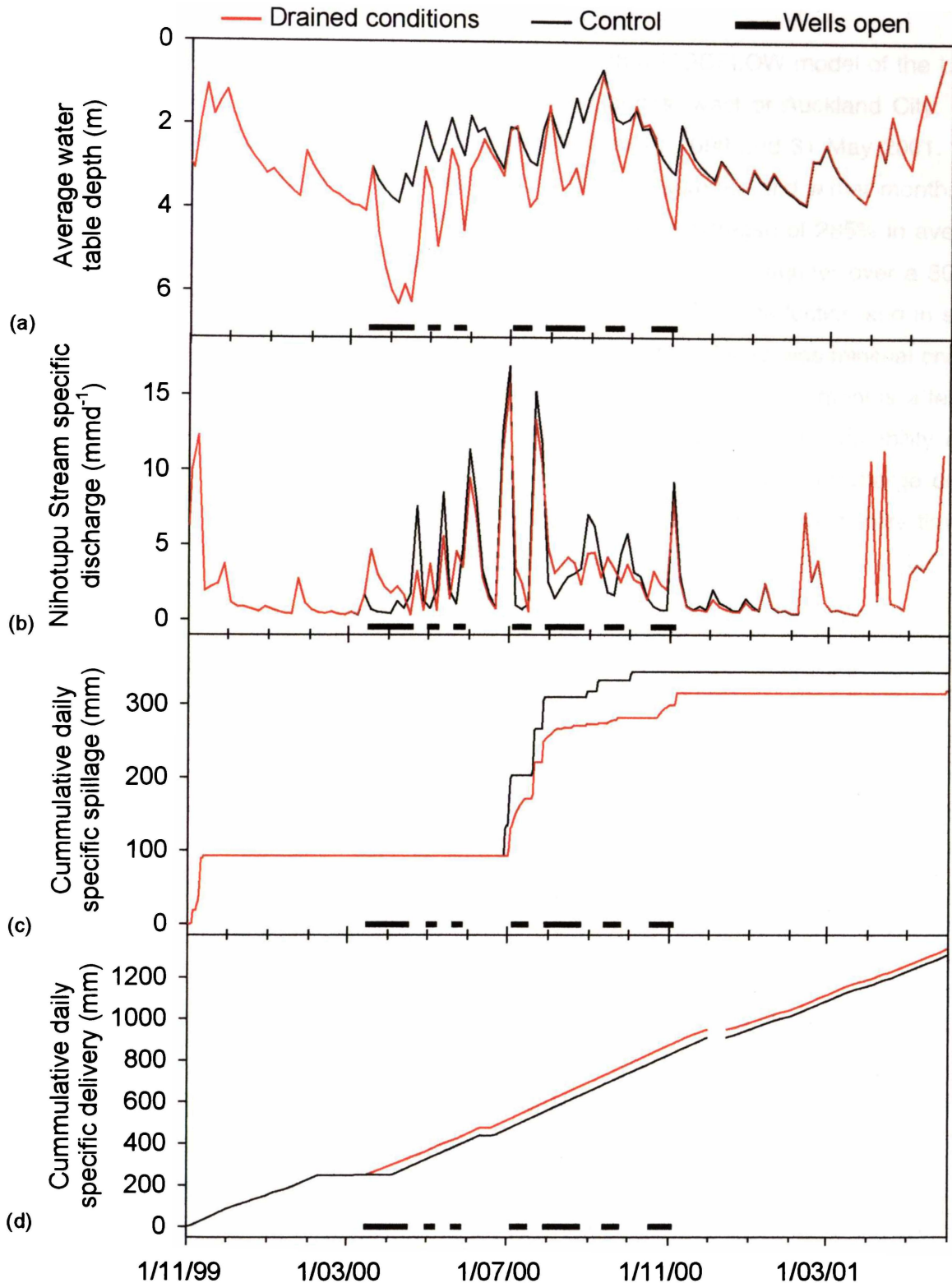


Figure 6.9. Simulated model results under natural and drained conditions a) average water table depth below ground surface for the six observation wells, b) specific discharge for the Nihotupu stream under natural and drained conditions, c) cumulative specific discharge for the reservoir spillage, and d) cumulative specific reservoir water delivery. The horizontal bars represent periods when the wells are simulated to be open. The first three open periods the wells were assigned a conductance of 4.2 md^{-1} , and the last four open periods were assigned a conductance of 1.5 md^{-1} .

6.7 Summary

The operation of horizontal wells was demonstrated with a MODFLOW model of the Upper Nihotupu water supply catchment in the Waitakere Ranges, west of Auckland City. The model simulated the operational period between November 1999 and 31 May 2001. The simulated operation of the horizontal wells was for the late summer and winter months (17 March 2000 to 30 September 2000). The model showed an increase of 285% in average stream discharge during the late summer due to the draining of the aquifer over a 30-day period. During the winter months the horizontal wells resulted in a reduction and in some cases a delay in the peak discharge from winter rainfall events. There was minimal change in the modelled total water output from the Nihotupu Stream for the 12 months after the wells were initially simulated open. However, there was an increased manageability of its regime by reducing stream quickflow peaks and increasing the stream's discharge during summer. This increased control of the Nihotupu Stream's regime can provide more time for reservoir operators to implement management options to reduce spillage and increase water delivery.

The Nihotupu Stream discharge simulated by the MODFLOW model for the horizontal wells was applied to a reservoir operation model to quantify the change in reservoir spillage and delivery. The reduction in spillage from the reservoir was equivalent to 12 days of average annual water delivery ($24,000 \text{ m}^3\text{d}^{-1}$), and the water delivery increase was equivalent to 13 days delivery. The increase in reservoir delivery, and the decrease in winter spillage for the modelled conditions, produces an increase in reservoir efficiency equivalent to an extra 25 days per year of average annual delivery.

In catchments where there is a strong relationship between quickflow stream discharge and the depth of the water table, horizontal wells could provide a means of enhancing the efficiency of existing limited storage reservoirs. This is achieved by manipulating catchment water tables to produce increased stream discharge during dry periods by draining the aquifer, and reduced discharge during wet periods due to the enhanced recharge produced by the pre-draining of the aquifer. This in turn increases the water delivery from the reservoir during the dry periods and reduces the spillage during the wet periods. This could be beneficial in catchments where water table storage is available, and the construction of larger or new reservoirs is not a viable option due to economic or environmental issues. To the author's knowledge, horizontal wells have not previously been used to manipulate water table levels for the purpose of controlling stream flow discharge as presented in this thesis.

In reality, the operation of horizontal wells may not be as effective as simulated due to difficulties in positioning the wells throughout a catchment. However for the simulation presented, horizontal wells appear to have potential to effectively increase existing limited storage capacity reservoirs, providing a minimal-impact alternative to more obvious solutions such as new or larger reservoirs. Horizontal wells can be installed with minimal disturbance to the environment, while providing increased utilisation of existing water resources. The operating costs are also low with gravity drainage being utilised to control the water table level. Given the currently available drilling technology, horizontal wells or similar gravity drainage systems could provide a useful extension to reservoir operation in any supply catchment where water table storage can be manipulated.

Economic Overview

Economic analysis of the horizontal wells in the Upper Nihotupu showed that if the construction costs are paid in the first year of operation the water would cost NZ\$44.94/m³, and in the following years the maintenance and operating costs would be NZ\$2.50/m³. In the first year the operation is quite expensive, however, one aspect of the cost analysis that has not been included is the monetary worth, and intrinsic worth to the community, of land that is not flooded if the horizontal wells were able to defer the construction of new reservoirs. Details of cost analysis are in Appendix H.

Chapter 7

Conclusion

Constructed groundwater systems provide a wide variety of options for increasing the water availability of municipal water supply systems. Constructed groundwater systems may alleviate the need for importing water from outside the region and may defer the construction of new reservoirs that may flood productive or environmentally important land. This thesis has evaluated the potential of constructed groundwater systems to augment the water supply for Auckland City, New Zealand.

In the Onehunga – Mt Wellington aquifer the increase in yield from the simulated hypothetical constructed groundwater systems were evaluated using a numerical model of the aquifer. The most suitable constructed groundwater systems identified in this study were; (1) seasonal injection and extraction of reservoir water via aquifer storage and recovery (ASR) wells, (2) recharging tertiary treated wastewater into the Mt Smart area of the aquifer via soakage wells, and (3) lowering of Watercare's Rowe and Pearce Street wells' intake screens.

- 1) The model-based evaluation showed that utilising the Onehunga – Mt Wellington aquifer for ASR, Watercare's summer aquifer yield could increase from 9,100 to 44,100 m³d⁻¹. In the simulation, water from Watercare's ten reservoirs was injected into the aquifer during winter when there was excess and reservoir spillage often occurred, and stored until summer when it was extracted to meet the increased water demand. The One Tree Hill scoria cone provided a suitable site for ASR wells, with a large depth of unsaturated highly permeable fractured basalt and scoria. Existing pipe networks and storage tanks could supply the water from the reservoir to the aquifer.
- 2) The average summer yield from the Watercare wells could be increased from 9,100 to 11,600 m³d⁻¹ due to the artificial aquifer recharge of highly treated wastewater via soakage wells. The simulations showed that the increased yield from the Watercare wells was achieved without reducing the quality of the groundwater extracted by the wells. The wastewater was recharged adjacent to the Watercare wells and flowed directly to the Manukau Harbour, avoiding direct contact with the Watercare wells. The wastewater recharge mounding of the water table caused an increase in groundwater

levels at the nearby Watercare wells enabling the increase in extraction. The wastewater recharge rate was limited to 7,500 m³d⁻¹ to limit the lateral movement of the wastewater plume towards the Watercare wells and to prevent surface flooding at the recharge site. This method of wastewater disposal may result in the improved quality of the wastewater as it filters through the aquifer to the Manukau Harbour. This may also be a more culturally acceptable wastewater disposal method for the Maori community, compared to the current disposal directly into the Manukau Harbour.

- 3) Lowering the intake screens to sea level may increase the combined average summer yield of Watercare's Rowe and Pearce Street wells by 40% or 3,500 m³d⁻¹. However, this increase is highly dependent on the specific discharge of the aquifer. Lowering the intake screens from their current level of 0.7 m amsl to sea level should not induce saline contamination of the aquifer from the Manukau Harbour.

Preliminary economic analysis of the three constructed groundwater systems for the Onehunga – Mt Wellington aquifer showed that lowering of the intake screens is the most cost effective option. If the construction costs of lowering the screens are paid in the first year of operation the water is estimated to be NZ\$2.18/m³, and in the following years the maintenance and operating costs are estimated to be NZ\$2.00/m³. The ASR of reservoir water is estimated to cost NZ\$4.60/m³ in the first year and NZ\$3.76/m³ in the following years. However, compared to the lowering of the intake screens of the Watercare wells, the ASR option provided the greatest increase in water supply, and reduced some of the winter reservoir spillage. The increase in groundwater yield from the recharging of highly treated wastewater is estimated to cost \$30.98/m³ in the first year and NZ\$3.58/m³ in the following years.

In the Hunua and Waitakere ranges there is little opportunity for the use of constructed groundwater systems. The geology of the Hunua Ranges is predominantly indurated greywacke (fine sandstone) and argillite (mudstone) of the Waiheke Group, and in the Waitakere Ranges the upper 800 m of geology is predominantly andesite lava and andesite derived rocks of the Waitakere Group. The low porosity (less than 10%) of both the Waiheke and Waitakere Groups' aquifers limits their use for augmenting the Auckland water supply. Constructed groundwater systems may be utilised in the crush zone of the faults or in the regolith layer where the porosity is greater. However, the fault zones are long and narrow, and often have secondary mineralisation in the jointing reducing the effective porosity. The possible increase in yield from the volume of groundwater that could be extracted, and/or the volume of water that could be temporarily stored in the fault zone

aquifers is small. The utilisation of the regolith layer aquifers using horizontal wells was evaluated in the Upper Nihotupu water supply catchment in the Waitakere Ranges, west of Auckland City.

In the Nihotupu Catchment, there is a strong relationship between stream quickflow discharge and the depth of the water table in the surrounding regolith aquifers. For similar-sized rainfall events there is a fourfold increase in quickflow discharge for a water table 0.5 m below the ground surface, compared to a water table 2.7 m below the surface. Based on this relationship of stream discharge and water table depth there is the potential to use horizontal wells to give some degree of control of the magnitude of stream discharge and in turn increase the storage capacity of the Upper Nihotupu reservoir. The operation of horizontal wells was demonstrated with a MODFLOW model of the Upper Nihotupu catchment. The model showed an increase of 285% in average stream discharge during the late summer months due to the draining of the aquifer over a 30-day period. During this 30 day period the increase in stream flow enabled the simulated average daily delivery from the reservoir to increase from 10,000 to 25,000 m³. During the winter months, the horizontal wells resulted in a reduction and in some cases a delay in the peak discharge from winter rainfall events. The increase in reservoir delivery during summer, combined with the decrease in winter spillage for the modelled conditions, was estimated to produce an increase in reservoir efficiency equivalent to an extra 25 days per year of average annual delivery.

Preliminary economic analysis of the horizontal wells in the Upper Nihotupu showed that if the construction costs are paid in the first year of operation the water would cost \$44.94/m³. In the following years the maintenance and operating costs would be \$2.50/m³. In the first year the operation is quite expensive, but one aspect of the cost analysis that has not been included is the monetary worth, and intrinsic worth to the community, of land that is not flooded if the horizontal wells were able to defer the construction of new reservoirs.

The operation of horizontal wells is not of course limited to increasing the storage capacity of municipal water reservoirs. Horizontal wells could also: enhance the efficiency of hydropower schemes, so that water is released at peak times when electricity prices are high; reducing catchment flooding; or increase stream flow to meet the fluctuating demand for irrigation and other purposes. Horizontal wells operated in this manner are best suited for catchments where there is a strong relationship between quickflow stream discharge and the depth of the water table, and the construction of larger or new reservoirs is not a viable option due to economic or environmental issues.

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Appendix A: Reservoir Spillage

Daily reservoir spillage was calculated for Watercare's 10 storage reservoirs using spillway rating curves documented by Ward and Webby (1996), the reservoirs' water levels were recorded at 15 minute intervals and were converted to spillage in units of m^3s^{-1} .

The spillway ratings have been determined using hydraulic methods applied to the geometry of the spillway structures. These ratings are theoretical, and rely on estimations of roughness factors, approach conditions, and other losses. The rating curves used should have an overall accuracy of $\pm 5\%$ (Ward and Webby, 1996). Table A.1 lists the spillway crest levels used in the calculation of reservoir spillage, and Table A.2 summarises the winter spillage from Watercare's 10 reservoirs. The daily spillage volumes (m^3) for each reservoir are in detailed in Appendix C, File: Daily reservoir spillage 1997_2001.xls.

Table A.1. Spillway crest level for Watercare's 10 reservoirs.

Reservoir	Full Level (m)
Waitakere	19.812
Upper Huia	28.956
Upper Nihotupu	33.833
Lower Huia	27.737
Lower Nihotupu	15.575
Cosseys	28.662
Upper Mangatawhiri	27.289
Wairoa	35.120
Mangatangi	54.511
Hays Creek	16.258

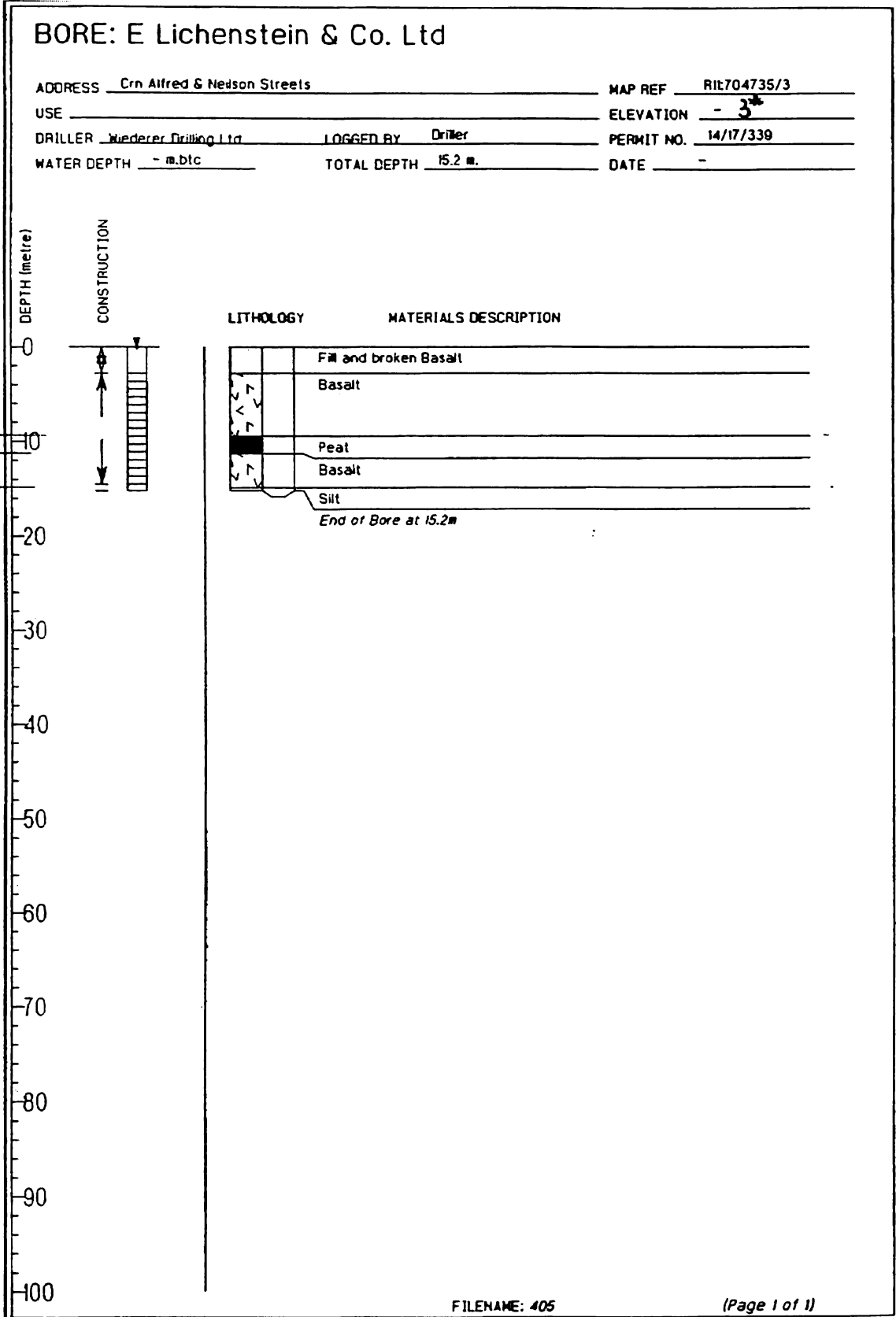
Table A.2. Summary of reservoir winter (July to December) spillage for the years 1997 to 2000.

Reservoir	Total spillage (m^3)	Daily average spillage (m^3) (732 days)	Daily average specific spillage (mm)
Waitakere	2,985,648	4,079	0.49
U. Huia	480,274	656	0.08
U. Nihotupu	6,446,860	8,807	0.88
L. Huia	15,696,797	21,444	1.90
L. Nihotupu	16,122,994	22,026	1.78
Cosseys	2,535,219	3,463	0.16
Wairoa	7,010,581	9,577	0.74
U. Mangatawhiri	14,843,897	20,279	0.79
Mangatangi	45,573,977	62,260	1.63
Hays Creek	3,875,179	5,294	0.83
Total	115,571,426	157,884	

Appendix B: Selected Drill-Hole Logs

Other drill-hole logs are in attached CD ROM, Folder: Drill-hole logs.

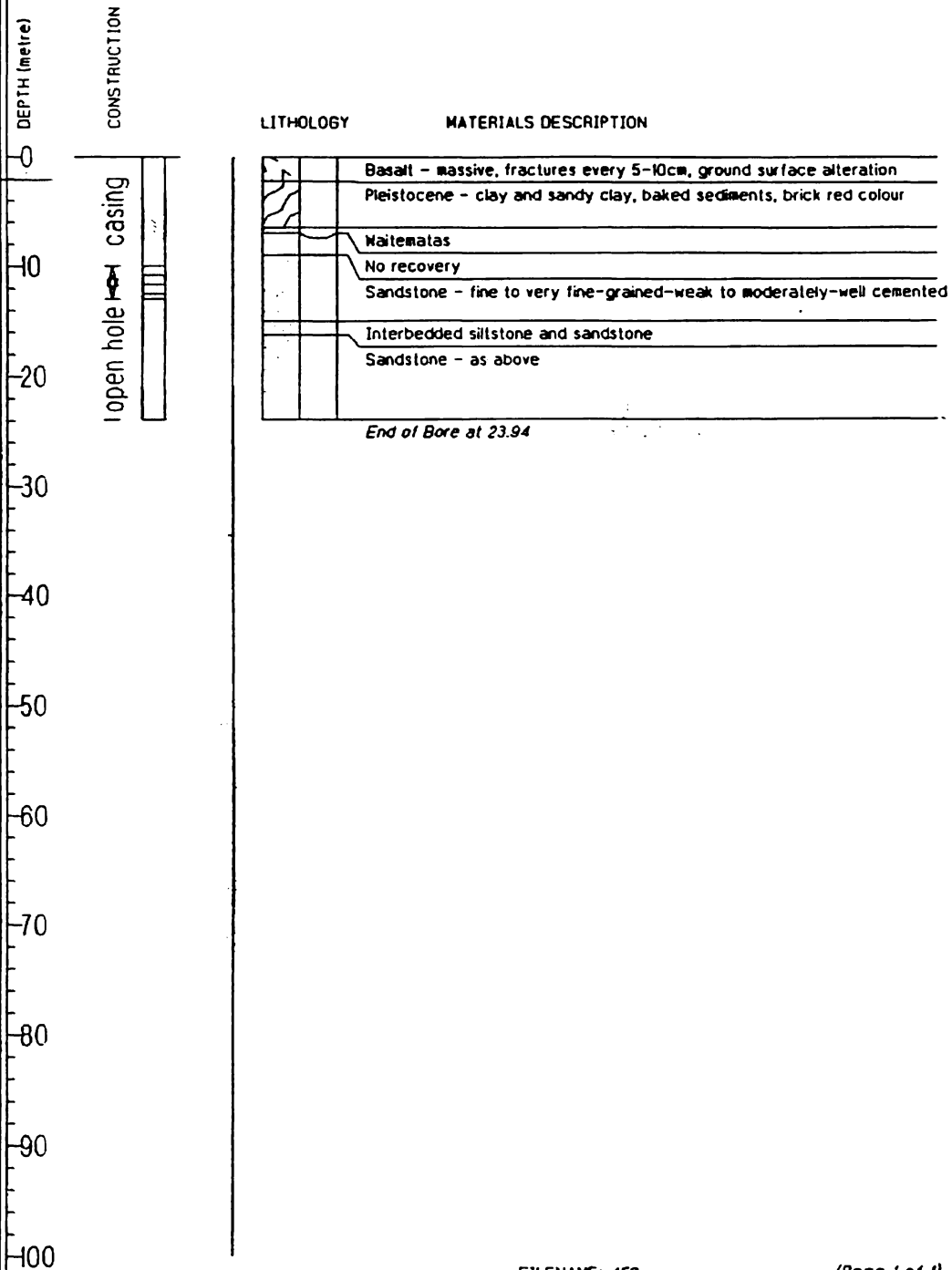
ARC 405



ARC 456

BORE: ARC Refuse

ADDRESS Winstones Quarry, Mt Wellington MAP REF R1E745770
 USE Water level and quality monitoring ELEVATION 2178
 DRILLER Keith Stevens LOGGED BY K. Stevens PERMIT NO. 14/17/390
 WATER DEPTH ground surface m.btc TOTAL DEPTH 23.94 m. DATE 2/2/90



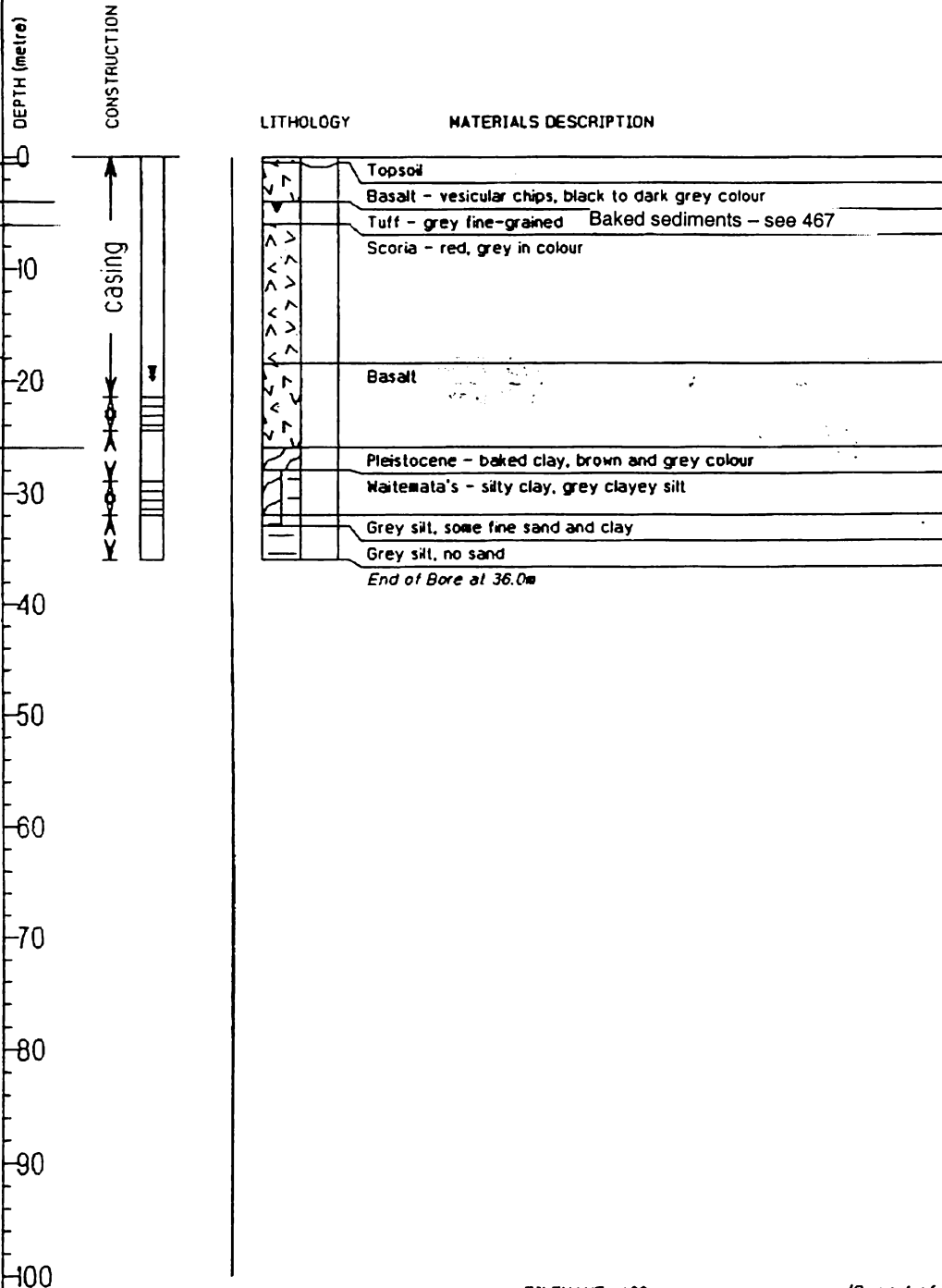
FILENAME: 456

(Page 1 of 1)

ARC 463

BORE: ARC Refuse

ADDRESS Winstones Quarry, Mt. Wellington MAP REF R11:747778
 USE Water level & quality monitoring ELEVATION 40.58
 DRILLER M.Patterson LOGGED BY GB PERMIT NO. 14/17/397
 WATER DEPTH 19.7 m.b.l.c TOTAL DEPTH 36.0 m. DATE 6/3/90



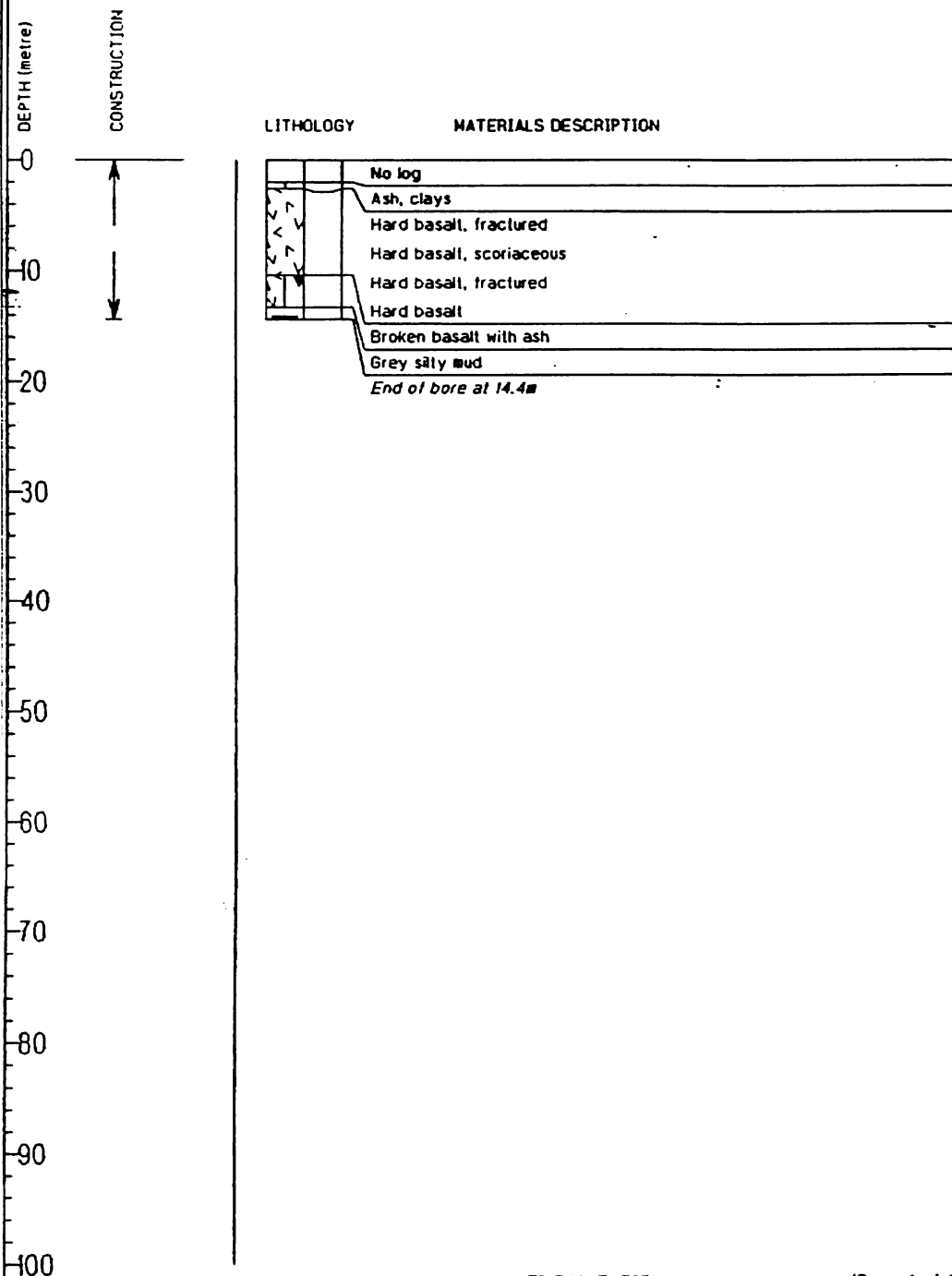
FILENAME: 463

(Page 1 of 1)

ARC 737

BORE: Horizon Yarns Ltd.

ADDRESS 273 Nelson Street, Perouse MAP REF R1E711735
 USE _____ ELEVATION 6^m
 DRILLER Drillwell Exploration LOGGED BY Driller PERMIT NO. 14/17/871
 WATER DEPTH 4.80 m.btc TOTAL DEPTH 14.4 m. DATE 17/7/91



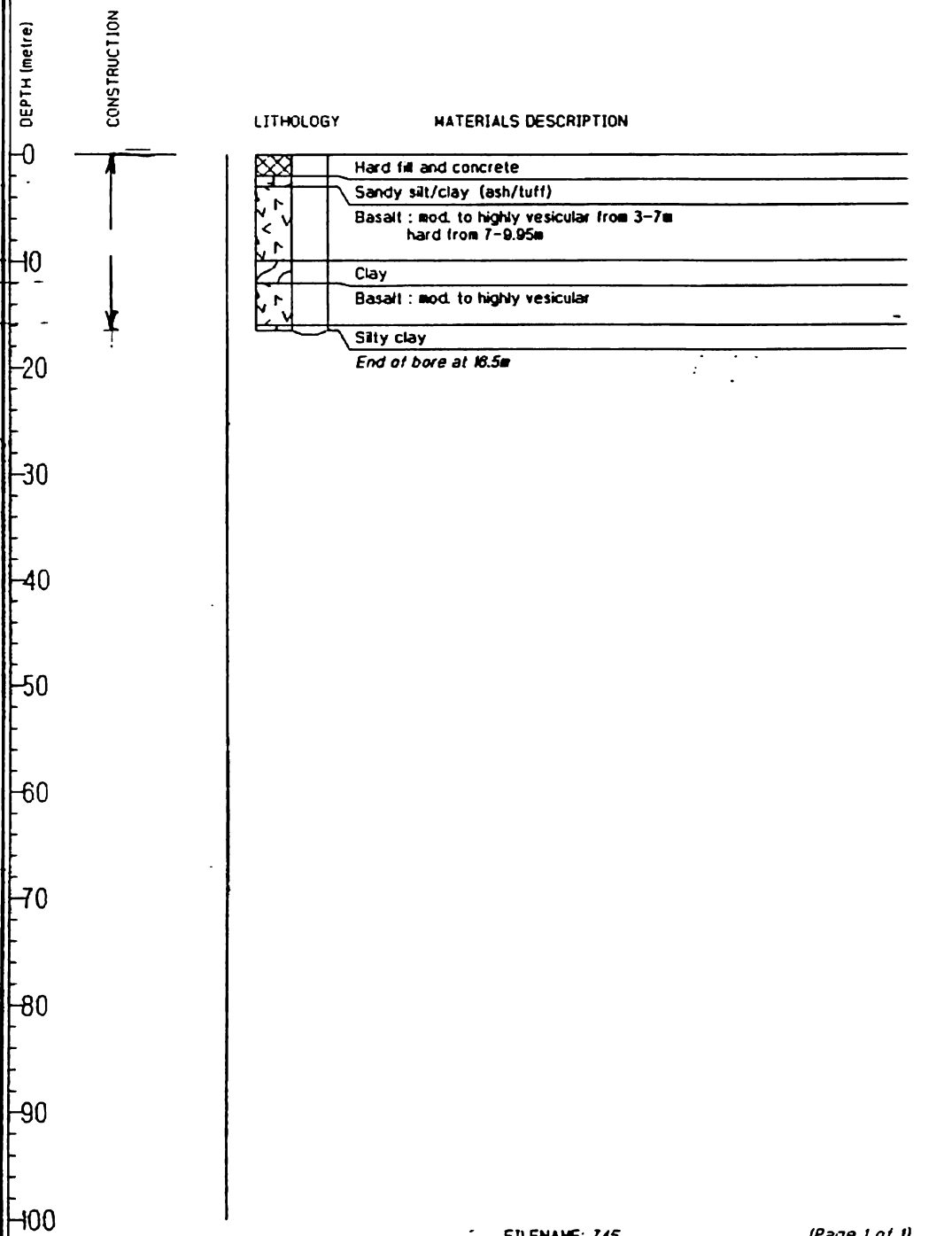
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ARC 745

BORE: Auckland Anodisers

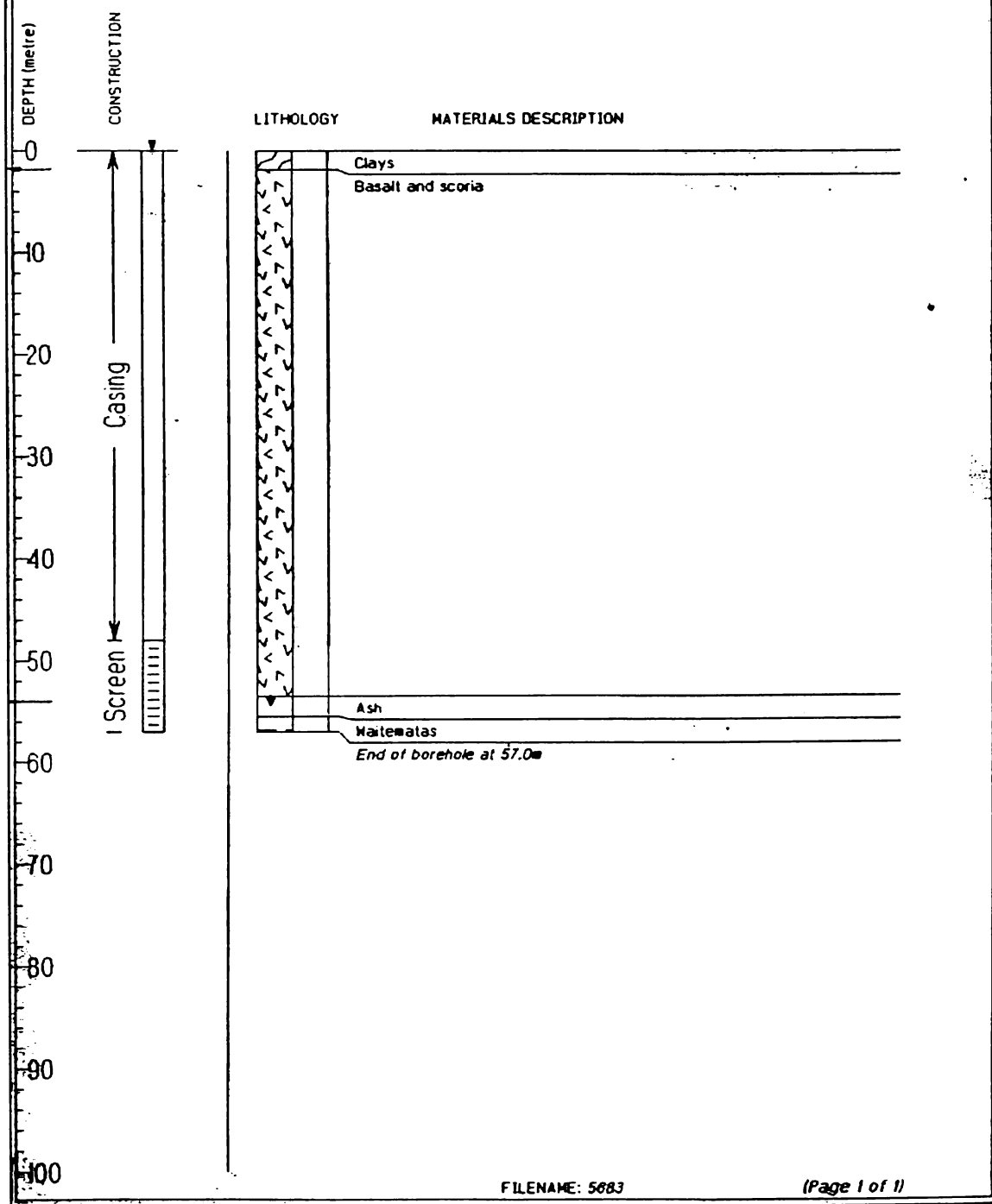
ADDRESS Captain Springs Road, Onehunga MAP REF R1:709739
 USE _____ ELEVATION 9⁺
 DRILLER Kim Well-drillers LOGGED BY Carrier & Assoc. PERMIT NO. 14/17/879
 WATER DEPTH m.btc TOTAL DEPTH 16.5 m DATE 22/8/91



ARC 5683

BORE: ARC Environment

ADDRESS Rd Reserve, Tiwai St, Epsom MAP REF R11892755
 USE Monitoring ELEVATION 70*
 DRILLER Drillwell Exploration NZ Ltd LOGGED BY Driller PERMIT NO. C512-12-1952
 WATER DEPTH m.btc TOTAL DEPTH 57 m DATE 1/4/97



OBC 3

RECORD OF BOREHOLE NO. 3 EXTENSION JOB NO. 2088

SITE: ONEHUNGA BOROUGH SHEET NO. 1 of 3

REDUCED LEVEL METRES.	DESCRIPTION OF SOIL	SOIL GRAPH	DEPTH (Metres)	DEPTH (FT)	SAMPLE TYPE	PENETRATION RESIST. (blows/ft)	CORRECTED PENETRATION RESISTANCE	NATURAL MOISTURE CONTENT AND ATTERBURG LIMITS (%)	BULK DENSITY p.c.f.	OTHER TESTS
							UNDRAINED SHEAR STRENGTH (P.S.I)			
74.60	Borehole reamed to 14'			5						
				10						
				15						
		Featured vesicular Basalt		20						
		Vesicular Basalt		25						
		Solid Basalt		30						
		Fractured 12" c		35						
		Scoriaceous Basalt 12" c		40						
		Vesicular Basalt 12" c		45						
		Fractured Scoriaceous Vesicular Basalt		50						
		Solid Basalt occasional fractures		55						
		Solid Basalt		60						
		Vesicular Basalt 18" c		65						
		Fractured vesicular Scoriaceous Basalt		70						
		Solid Basalt		75						
		Vesicular Basalt		80						
		Well Fractured		85						
		Vesicular Basalt 18" c		90						
		Well Fractured Vesicular Basalt		95						
		Well Fractured 8" c		100						
	Scoriaceous Vesicular Basalt 4" c									

zn barrel sample Double or triple tube core sample Thin walled tube sample 4" dia B.R.s sample Standard penetration test sample.

TONKIN & TAYLOR CONSULTING CIVIL & FOUNDATION ENGINEERS

OBC 3 (continued).

METRES 0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200		RECORD OF BOREHOLE NO. 3 Extension		JOB NO 2088					
		SITE: ONEHUNGA BOROUGH		SHEET NO 2 of 3					
DESCRIPTION OF SOIL	SOIL GRAPH	DEPTH (Metres)	DEPTH (FT)	SAMPLE TYPE	CORRECTED PENETRATION RESISTANCE	UNDRAINED SHEAR STRENGTH (P.S.I.)	NATURAL MOISTURE CONTENT AND ATTERBURG LIMITS (%)	BULK DENSITY p.c.f.	OTHER TESTS
						• Triaxial test • remoulded • Vane test • remoulded	Wp W W _L * * *		
Well Fractured, Scoriaceous vesicular Basalt	✓ ✓		105						
Solid Basalt	✓ ✓	33	110						
Solid Basalt, occasional fractures	✓ ✓		115						
Solid Basalt	✓ ✓	36	120						
Fractured Solid Basalt	✓ ✓		125						
Well Fractured vesicular Basalt	✓ ✓	39	130						
Fractured Solid Basalt	✓ ✓		135						
Well Fractured vesicular Basalt	✓ ✓	42	140						
Fractured Solid Basalt	✓ ✓		145						
Well Fractured Scoriaceous Vesicular Basalt	✓ ✓	45	150						
Well Fractured vesicular Basalt	✓ ✓		155						
Well Fractured Solid Basalt	✓ ✓	48	160						
Fractured Solid Basalt	✓ ✓		165						
Solid Basalt	✓ ✓	51	170						
Well Fractured Scoriaceous Basalt	✓ ✓		175						
Fractured Solid Basalt	✓ ✓	54	180						
Fractured Vesicular Basalt.	✓ ✓		185						
VOLCANIC GRIT, welded, porous, fractured	✓ ✓	57	190						
CLAY, silty stiff light grey brown	✓ ✓		195						
Core washed away	✓ ✓	60	200						

Open barrel core sample
 Double or triple tube core sample
 Thin walled tube sample
 4" dia BR's sample
 Standard penetration test sample.

TONKIN & TAYLOR CONSULTING CIVIL & FOUNDATION ENGINEERS

OBC 3 (continued).

<p style="text-align: center;">RECORD OF BOREHOLE NO. 3 Extension JOB NO. <u>2088</u></p> <p style="text-align: center;">SITE: <u>ONEHUNGA BOROUGH</u> SHEET NO. <u>3 of 3</u></p>											
REDUCED LEVEL METRES.	DESCRIPTION OF SOIL	SOIL GRAPH	DEPTH (Metres)	DEPTH (FT)	SAMPLE TYPE	PENETRATION RESIST. (blows/ft)	CORRECTED PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND ATTERBURG LIMITS (%)	BULK DENSITY p.c.f.	OTHER TESTS
							UNDRAINED SHEAR STRENGTH (P.S.I)				
							<input type="checkbox"/> Triaxial test <input type="checkbox"/> remoulded <input type="checkbox"/> Vane test <input type="checkbox"/> remoulded		w_p w w_L x — — —		
8.76	CLAY silty stiff light grey.		63	205							
	SILT, clayey hard lt. grey			210							
	FINE SAND silty weakly cemented 3" bands SILTSTONE light grey.		66	215							
<p>END OF BOREHOLE AT 216'</p> <p>NOTE Borehole cased to 216' with 1 1/8" internal diameter perforated p.v.c. casing, joined by external metal joiners. c = cavity</p>											
<p> <input type="checkbox"/> Open barrel core sample <input type="checkbox"/> Double or triple tube core sample <input type="checkbox"/> Thin walled tube sample <input type="checkbox"/> 4" dia BR's. sample <input type="checkbox"/> Standard penetration Test sample. </p>											
<p>TONKIN & TAYLOR CONSULTING CIVIL & FOUNDATION ENGINEERS</p>											

OBC 4

RECORD OF BOREHOLE NO. 4

SITE: ONEHUNGA BOROUGH

JOB NO. 2088
SHEET NO. 1 of 2

FEET REDUCED LEVEL METRES.	DESCRIPTION OF SOIL	SOIL GRAPH	DEPTH (Metres)	DEPTH (FT)	SAMPLE TYPE	PENETRATION RESIST. (blows/ft)	CORRECTED PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND ATTERBURG LIMITS (%)			BULK DENSITY p.c.f.	OTHER TESTS
							UNDRAINED SHEAR STRENGTH (P.S.I.)		W _p	W	W _L		
195.56	Clay			5									
59.61	DC 20365			10									
	Basalt			15									
				20									
				25									
	Scoria			30									
	Fractured scoriaceous basalt			35									
	60" C			40									
	Fractured vesicular basalt			45									
				50									
	Fractured scoriaceous basalt			55									
	Fractured vesicular basalt			60									
	24" C			65									
	12" C			70									
	Fractured vesicular basalt			75									
	48" C			80									
	Fractured scoriaceous basalt			85									
	50" C			90									
	Solid basalt			95									
	Vesicular basalt			100									

Open barrel core sample
 Double or triple tube core sample
 Thin walled tube sample
 4" dia. S.P. sample
 Standard penetration test sample.

TONKIN & TAYLOR CONSULTING CIVIL & FOUNDATION ENGINEERS

OBC 4 (continued)

		RECORD OF BOREHOLE NO. 4				JOB NO. 2088				
		SITE: ONEHUNGA BOROUGH				SHEET NO. 2 of 2				
FEET REDUCED LEVEL METRES.	DESCRIPTION OF SOIL	SOIL GRAPH	DEPTH (Metres)	DEPTH (FT)	SAMPLE TYPE	CORRECTED PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND ATTERBURG LIMITS (%)	BULK DENSITY p.c.f.	OTHER TESTS
						UNDRAINED SHEAR STRENGTH (P.S.)	PENETRATION RESIST. (blows/ft)			
						Trial test remoulded Vane test remoulded		Wp W _L * * *		
93.56	Solid basalt	✓								
29.13	Fractured scoriaceous basalt	4" C K K K K		105						
		24" C K K K K	3	110						
	Fractured vesicular basalt	✓		115						
		12" C K K K K	6	120						
	Fractured scoriaceous basalt	✓		120						
	Fractured solid basalt	✓		120						
	Fractured scoriaceous basalt	18" C K K K K	9	125						
		✓		130						
	Fractured vesicular basalt	✓		130						
	Fractured solid basalt	✓		135						
		30" C K K K K	12	140						
	Fractured vesicular basalt	✓		145						
		✓		150						
	Fractured scoriaceous basalt	✓		150						
	Fractured vesicular basalt	✓		155						
	Fractured solid basalt	✓		155						
	Fractured vesicular basalt	18" C K K K K	18	160						
		✓		160						
		✓		165						
		✓		165						
		21" C K K K K	21	170						
	Solid basalt	✓		170						
		✓		170						
		✓		175						
		24" C K K K K	24	175						
20.08	END OF BOREHOLE AT 175'-6"									
6.12	Note Borehole cased to 175'-6" with 1 1/8" internal diameter PVC perforated casing. C = Cavity									

open barrel core sample
 Double or triple tube core sample
 Thin walled tube sample
 4" dia B.R.S. sample
 Standard penetration Test sample.

TONKIN & TAYLOR CONSULTING CIVIL & FOUNDATION ENGINEERS

OBC 18

RECORD OF BOREHOLE NO. 18 & 18A		JOB NO. 2088											
SITE: ONEHUNGA BOROUGH.		SHEET NO. 1 of 2											
FEET REDUCED LEVEL METRES	DESCRIPTION OF SOIL	SOIL GRAPH	DEPTH (Metres)	DEPTH (FT)	SAMPLE TYPE	PENETRATION RESIST. (blows/ft)	CORRECTED PENETRATION RESISTANCE		NATURAL MOISTURE CONTENT AND ATTERBURG LIMITS (%)		BULK DENSITY p.c.f.	LABORATORY PERMEABILITY K	
							UNDRAINED SHEAR STRENGTH (P.S.I.)		W _p	W			W _L
181.89 55.44	SILT sandy brown, (Weathered volcanic tuff)		5	10	□	□							
			15	20									
			6	25									
			9	30									
			12	40									
	Fractured vesicular Basalt with clayey silt layers. Fractured vesicular Basalt. Solid Basalt Vesicular Basalt Fractured Vesicular Basalt Solid Basalt 3" c Well Fractured scoriaceous vesicular Basalt Fractured Vesicular Basalt. Fractured Scoriaceous Basalt Fractured Vesicular Basalt Well fractured Vesicular Basalt Fractured vesicular Basalt. SOLID BASALT Occasional fractures		25	30	□	□							
			35	45									
			12	50									
			15	55									
			18	60									
			9" c	65									
			70	75									
			6" c	80									
			9" c	85									
			24" c	90									
CLAY sandy hard light grey brown core washed away.		95	100	□	□								
		30	100										

open barrel core sample
 Double or triple tube core sample
 Thin walled tube sample
 4" dia B.S. sample
 Standard penetration test sample.

TONKIN & TAYLOR CONSULTING CIVIL & FOUNDATION ENGINEERS

Appendix C: CD ROM Contents

- Daily reservoir spillage data from Watercare's ten reservoirs. File: Daily reservoir spillage 1997_2001.xls.
- Drill-hole logs for the Onehunga – Mt Wellington aquifer. These are from three sources. The Auckland Regional Council (ARC logs), Mercury Energy power cable tunnel (Mercury), and the Onehunga Borough Council (OBC). Folder: Drill-hole logs.
- Flow duration data for Watercare's ten reservoirs between November 1999 to June 2001. Data includes standardised discharge (daily discharge divided by average discharge) and percentage of time that each event is exceeded. File: Reservoir Flow duration data.xls.
- Onehunga – Mt Wellington aquifer FEMWATER numerical model. Folder Onehunga model.
- Observation and modelled water levels for the 30 observation wells for the calibration (1998) and validation (1993-2000). File: Onehunga model calibration data.xls.
- Soakage wells infiltration rates and locations for the Onehunga – Mt Wellington aquifer. All data sourced from Metrowater. File: Metrowater Soakage wells.xls
- Gauging cards for the Upper Nihotupu weir and disused dam. File: Nihotupu stream gauging cards.doc.
- Hydrological data for Upper Nihotupu Catchment. This includes; average daily rainfall for the Upper Nihotupu and Huia rain gauges, Penman, evaporation for Auckland airport, Water levels from capacitance probe at well 4, and Upper Nihotupu weir stage. File: Upper Nihotupu hydrological data.xls.
- Resistivity data for Schlumberger electrode configuration surveys adjacent to wells 2 and 6, also a transect between these two wells. File: Resistivity data.xls.

- Water level measurement from the hydraulic conductivity tests of wells 2, 4, and 6. As well as a separate auger hole test on a hole adjacent to well 6. File: Conductivity tests data.xls.
- Nihotupu aquifer numerical model. Folder Nihotupu model.
- Upper Nihotupu model calibration (1999) and validation data (2001). This includes the baseflow discharge of the Nihotupu Stream and water table levels from the six observation wells. Water levels are recorded as level above mean sea level and depth below ground surface. File: Nihotupu model calibration data.xls.

Appendix D: Onehunga – Mt Wellington Aquifer Recharge

Table D.1. Onehunga – Mt Wellington aquifer recharge for 1993.

One Tree Hill - Onehunga Management Zone					
Mt Smart Rainfall 1993		797.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	1.0575	0.0	0.66	0.34	785.10
Residential	3.9798	0.0	0.35	0.65	5648.59
Reticulation Area					
Industry	2.7108	0.9	0.07	0.03	177.58
Residential	0.1593	0.5	0.35	0.15	52.18
One Tree Hill Rainfall 1993		1064.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	1.5030	0.0	0.66	0.34	1489.66
Residential	6.2559	0.0	0.35	0.65	11853.65
Reticulation Area					
Residential	0.5418	0.5	0.35	0.15	236.91
Mt Wellington Rainfall 1993		909.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	0.5020	0.0	0.66	0.34	425.06
Residential	1.6389	0.0	0.35	0.65	2653.00
Reticulation Area					
Residential	0.0927	0.5	0.35	0.15	34.63
Total	18.4417				23356.34

Mt Smart - Mt Wellington Management Zone					
Mt Smart Rainfall 1993		797.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	0.1980	0.0	0.66	0.34	147.00
Residential	1.0314	0.0	0.35	0.65	1463.88
Reticulation Area					
Industry	3.0942	0.9	0.07	0.03	202.69
Residential	0.1837	0.5	0.35	0.15	60.17
Mt Wellington Rainfall 1993		909.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	1.1070	0.0	0.66	0.34	937.34
Residential	4.4226	0.0	0.35	0.65	7159.16
Reticulation Area					
Residential	1.6533	0.5	0.35	0.15	617.61
Quarry	0.5000	0.0	0.57	0.43	535.44
Total	12.1902				11123.29
Total both zones	30.6319				34479.63

Table D.2. Onehunga – Mt Wellington aquifer recharge for 1995.

One Tree Hill - Onehunga Management Zone					
Mt Smart Rainfall 1995		1408.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	1.0575	0.0	0.66	0.34	1386.98
Residential	3.9798	0.0	0.35	0.65	9978.94
Reticulation Area					
Industry	2.7108	0.9	0.07	0.03	313.71
Residential	0.1593	0.5	0.35	0.15	92.18
One Tree Hill Rainfall 1995		1630.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	1.5030	0.0	0.66	0.34	2282.09
Residential	6.2559	0.0	0.35	0.65	18159.25
Reticulation Area					
Residential	0.5418	0.5	0.35	0.15	362.93
Mt Wellington Rainfall 1995		1614.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	0.5020	0.0	0.66	0.34	754.73
Residential	1.6389	0.0	0.35	0.65	4710.60
Reticulation Area					
Residential	0.0927	0.5	0.35	0.15	61.49
Total	18.4417				38102.89

Mt Smart - Mt Wellington Management Zone					
Mt Smart Rainfall 1995		1408.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	0.1980	0.0	0.66	0.34	259.69
Residential	1.0314	0.0	0.35	0.65	2586.13
Reticulation Area					
Industry	3.0942	0.9	0.07	0.03	358.08
Residential	0.1837	0.5	0.35	0.15	106.29
Mt Wellington Rainfall 1995		1614.0 mm			
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	1.1070	0.0	0.66	0.34	1664.32
Residential	4.4226	0.0	0.35	0.65	12711.64
Reticulation Area					
Residential	1.6533	0.5	0.35	0.15	1096.61
Quarry	0.5000	0.0	0.57	0.43	950.71
Total	12.1902				19733.48
Total both zones	30.6319				57836.38

Table D.3. Average Onehunga – Mt Wellington aquifer recharge between 1993 and 2000.

One Tree Hill - Onehunga Management Zone					
Mt Smart Rainfall Average 1993-2000					1186.0 mm
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	1.0575	0.0	0.66	0.34	1168.29
Residential	3.9798	0.0	0.35	0.65	8405.56
Reticulation Area					
Industry	2.7108	0.9	0.07	0.03	264.25
Residential	0.1593	0.5	0.35	0.15	77.64
One Tree Hill Rainfall Average 1993-2000					1362.0 mm
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	1.5030	0.0	0.66	0.34	1906.87
Residential	6.2559	0.0	0.35	0.65	15173.56
Reticulation Area					
Residential	0.5418	0.5	0.35	0.15	303.26
Mt Wellington Rainfall Average 1993-2000					1336.0 mm
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	0.5020	0.0	0.66	0.34	624.74
Residential	1.6389	0.0	0.35	0.65	3899.23
Reticulation Area					
Residential	0.0927	0.5	0.35	0.15	50.90
Total	18.4417				31874.29

Mt Smart - Mt Wellington Management Zone					
Mt Smart Rainfall Average 1993-2000					1186
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	0.1980	0.0	0.66	0.34	218.74
Residential	1.0314	0.0	0.35	0.65	2178.37
Reticulation Area					
Industry	3.0942	0.9	0.07	0.03	301.62
Residential	0.1837	0.5	0.35	0.15	89.53
Mt Wellington Rainfall Average 1993-2000					1336.0 mm
	Area (km ²)	Fraction to runoff	Fraction to evaporation	Fraction to recharge	Recharge (m ³ d ⁻¹)
Soakage Area					
Parks	1.1070	0.0	0.66	0.34	1377.65
Residential	4.4226	0.0	0.35	0.65	10522.15
Reticulation Area					
Residential	1.6533	0.5	0.35	0.15	907.73
Quarry	0.5000	0.0	0.57	0.43	786.96
Total	12.1902				16382.77
Total both zones	30.6319				48257.06

Recharge model for Onehunga – Mt Wellington aquifer

The black box rainfall recharge model developed during the calibration of the Onehunga – Mt Wellington aquifer for the area near Three Kings Quarry is described below. The purpose of the black box model is to simulate the reduction of aquifer recharge during large rainfall events, otherwise the aquifer water table level was above the ground surface during the groundwater model calibration.

Recharge Model Steps:

Recharge is applied as a seven-day average in units of md^{-1} .

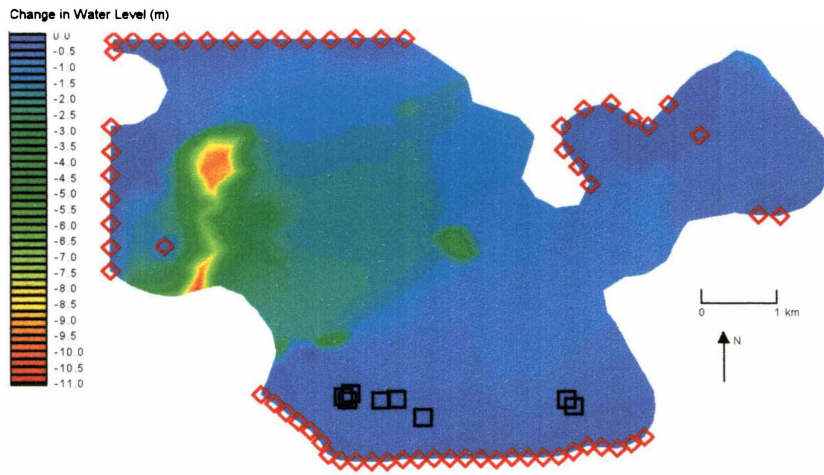
1. Recharge applied as residential recharge (RR) using rainfall from the One Tree Hill rain gauge.
2. Identify if the average RR for the week of interest (referred to as week (a)) and the three weeks prior (weeks b,c,d) is greater than or less than 0.004 md^{-1} .
 - if the average RR for the four weeks $< 0.004 \text{ md}^{-1}$; and if week (b) recharge is greater than 0.0026 md^{-1} , then week (a) recharge equals week (a) plus 70% of week (b) minus 0.0026 md^{-1} . If week (a) exceeds 0.0026 md^{-1} then it equals 0.0026 md^{-1} .
 - if the average RR for the four weeks $< 0.004 \text{ md}^{-1}$, week (a) equals RR value from step 1 for this week.
 - Step 2 is repeated for every week.
3. Identify any groups of weeks where the recharge calculated in step 2 for each of the weeks exceeds 0.0025 md^{-1} . A group is defined as one or more consecutive weeks exceeding 0.0025 md^{-1} . A separation is defined as one or more consecutive weeks not exceeding 0.0025 md^{-1} .
 - the first week in the group equals 85% of the RR recharge rate from step 1 for this week, with a maximum threshold of 0.004 md^{-1} .
 - the second week = 0.0026 md^{-1} .
 - the third and following weeks equal 65% of 0.0026 md^{-1} .

Appendix E: Onehunga – Mt Wellington Aquifer Model Sensitivity Analysis

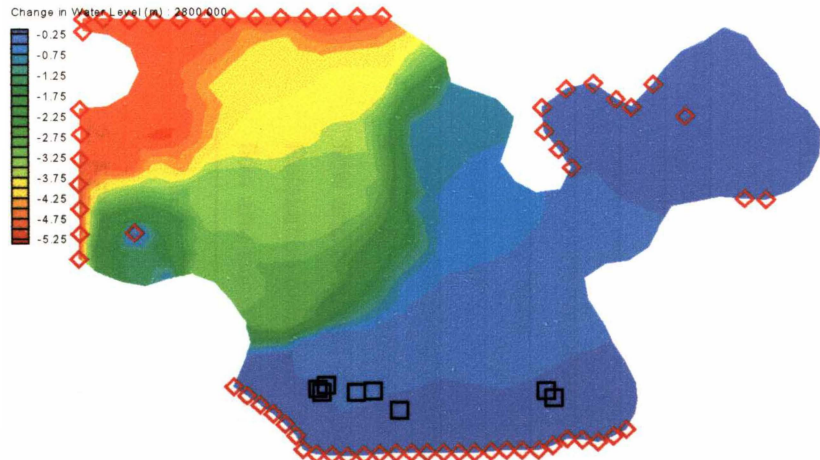
Sensitivity analysis helps to quantify the uncertainty in calibrated models (Anderson and Woessner, 1992). Having a validation period nine times longer than the calibration period reduced the uncertainty of the Onehunga – Mt Wellington groundwater model. However, when aspects of the model were changed to predict the aquifer's response to the constructed groundwater systems presented in section 3.5, uncertainty was reintroduced to the model. The main change to the model for the scenarios presented in section 3.5 was an increase in aquifer recharge. Thus it is useful to quantify the sensitivity of the model to large variations in recharge compared to the range applied during the calibration and validation process. The sensitivity of the model was evaluated as the average change in water levels between the calibrated and sensitivity model runs across the model domain. The results are shown in Figure E.1. For both a 25% increase and then decrease in recharge over the whole model domain, the greatest change in water levels was for the One Tree Hill area, and for the low conductivity Waitemata Group rocks and tuff deposits near Three Kings. There was little change in the water levels near Mt Wellington and Onehunga due to the specified head boundary conditions in these areas (Figure E.1a & b).

A second sensitivity analysis was used to evaluate the appropriateness of the specified head values along the Western Springs and Mt Eden boundaries. As discussed in Section 3.2.5, these heads were fixed at a constant level for the duration of the calibration simulation even though it was expected that the actual levels would fluctuate. During the sensitivity analysis, the model was run with specified head values set at five metres above and then below the values used in the calibration. The model results were evaluated to identify which areas of the aquifer were strongly influenced by these two boundaries. Five metres was the estimated typical range of fluctuations for the aquifer between winter and summer. Figure E.1c & d shows for both an increase and decrease in the specified head values, similar areas of the aquifer are affected. This was in the vicinity of the adjusted boundary conditions and the low conductivity tuff deposits and Waitemata group rocks between One Tree Hill and Three Kings. In the vicinity of Mt Wellington and Onehunga there was very little change.

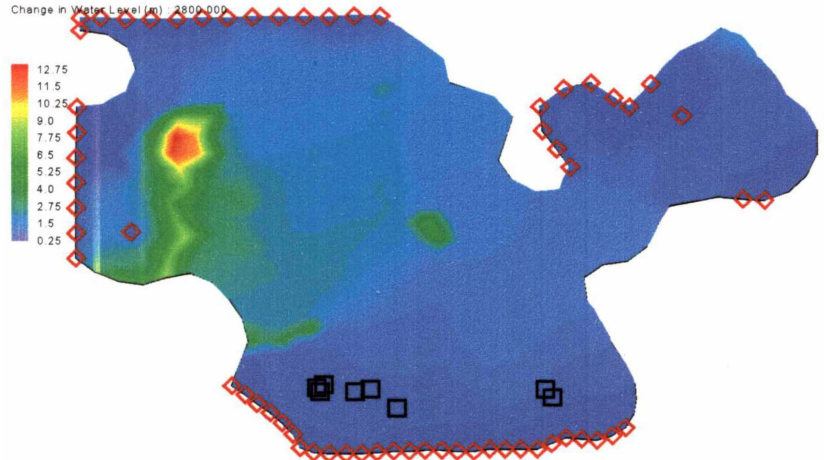
The sensitivity analysis confirms that the water levels calculated by the model are sensitive to changes in aquifer recharge, and that the selection of the specified head values along the Western Springs and Mt Eden boundaries do not significantly affect the modelled water levels in the Onehunga – Mt Wellington aquifer.



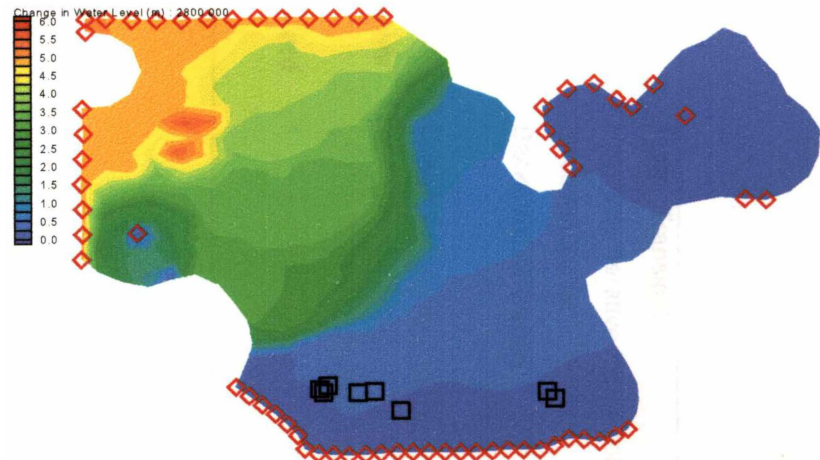
a) 25% decrease in aquifer recharge



c) 5 m decrease in Western Springs and Mt Eden specified head boundary values



b) 25% increase in aquifer recharge



d) 5 m increase in Western Springs and Mt Eden specified head boundary values

Figure E.1. Change in water levels (m) from calibrated model during model sensitivity analysis. Contour interval is 0.25 m.

Appendix F: Onehunga – Mt Wellington Aquifer Model Solver and Iterations

Table F.1. FEMWATER solver options.

Simulation Type	Flow - Transient
Solver selection	Pointwise iterative matrix solver
Quadrature selection	Nodel/gaussian
Weighting factor	Backward difference
Mass Lumping	
Sorption model control (isotherm)	Linear
Relaxation parameter for solving nonlinear flow and transport equations	1
Relaxation parameter for solving linearized flow and transport equations	1

Table F.2. FEMWATER iteration parameters.

Flow Simulation	
Maximum iteration for non-linear equation	400
Maximum cycle/time step for variable boundary condition	1
Maximum iteration for linear equation	600
Transient convergence criterion	0.001 m
Transport Simulation	
Maximum iteration for non-linear equation	40
Maximum iteration for linear equation	400
Convergence criterion	0.001 kg/m ³
Flow and Transport Simulation	
Maximum Iterations	20
Iteration parameter	0.5
Convergence criterion for head	0.01 m
Convergence criterion for concentration	0.05 kg/m ³

Appendix G: Rowe Street Well Specific Drawdown

Table G.1. Specific drawdown for a pump test on the Rowe Street well on the 24 May 1994 by Pattle Delamore Partners Limited (1994). The change in drawdown is measured at the end of each pumping step.

Step	Discharge rate (m ³ d ⁻¹)	Change in discharge rate (m ³ d ⁻¹)	Water Table level (m amsl)	Change in drawdown (m)	Specific discharge for each change in drawdown and discharge (m/ m ³ d ⁻¹)
	0	-	2.57	-	-
1	2764	2764	2.44	0.134	0.0000485
2	4060	1300	2.37	0.069	0.0000531
3	7430	3370	1.99	0.380	0.0001128
4	9504	2074	1.63	0.350	0.0001688
5	11232	2728	0.92	0.712	0.0002610

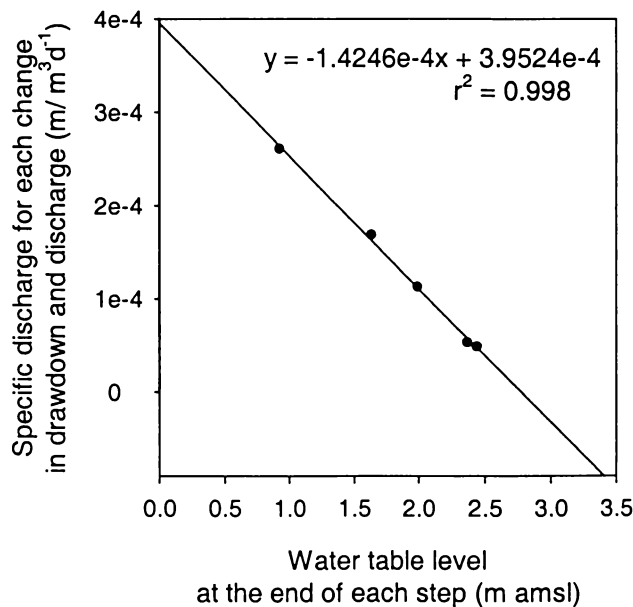


Figure G.1. Specific drawdown for the Rowe Street well at the end of each pumping step.

Appendix H: Economic Assessment of the Constructed Groundwater Systems

Table H.1 shows the costs for construction and operation estimated in this thesis for the various constructed groundwater systems presented in Chapters 3 and 6. The treatment and operation costs of Watercare's existing filter treatment plants are listed in Table H.2. The costing assumes that all the capital costs for each option analyses are covered in the first year of operation. For the remaining years the only costs are from maintenance and operation of each option.

Costing for the various constructed groundwater systems have been obtained from a number of sources including:

- previous work by Kingston Morrison (1995).
- drilling costs from Brown (2001).
- pump costs (Waikato Pumps, 2002)
- previous work by Watercare Services Limited

The actual cost of electricity used by the pumps is difficult to estimate as this will fluctuate because the electricity is purchased at a variable rate. A constant cost value of \$0.1/m³ of water pumped has been selected. This cost is based on previous work by Kingston Morrison (1995) and Watercare Services Limited.

None of the constructed groundwater systems should require the purchase of land. The land is owned by either the Auckland Regional Council or Watercare Services Limited.

Capital cost covers the expense of developing each constructed groundwater option and installing the required monitoring wells. It is assumed in this economic analysis that the capital costs are covered in the first year of operation.

The required pipe diameter was calculated using charts from Chadwick and Morfett (1991, p. 103), based on the pipeline hydraulic gradient and required volume per day. The purchase and installation cost of a 400-mm diameter pipe is approximately \$1000 per metre.

Table H.1. Cost analysis for the five constructed groundwater systems presented in Chapter 3 and the operation of horizontal wells presented in Chapter 6.

Option 1: Watercare Wells Redevelopment

Capital costs	units	cost per unit	cost
Land	Existing wells		
Well extension	1	\$50,000	\$50,000
Resource consents & reports			\$50,000
Contingency	15%		\$15,000

Sub total \$115,000

Operation and Maintenance Costs	units per 6 months	cost per unit	
Pump electricity 0.5 kWh/m ³	638750 m ³	\$0.10	\$31,938
Rowe Street water treatment	638750 m ³	\$1.95	\$1,245,563

Sub total \$1,277,500

Year	\$/m ³
First Year	2.18
Remaining Years	2.00

Option 2a: One Tree Hill ASR 35000 m³d⁻¹

Capital costs	units	cost per unit	cost
Land	One Tree Hill (ARC)		
ASR wells 250mm/80m + pumps	8	\$150,000	\$1,200,000
Water Storage tank	1	\$40,000	\$40,000
Deliver pipe 360 mm	500 m	\$1,000	\$500,000
Monitoring wells with coring 80m	6 m	\$12,000	\$72,000
Resource consents & reports			\$50,000
One Tree hill Site development			\$200,000
Pipe to Rowe Street	2600 m	\$1,000	\$2,600,000
Contingency	15%		\$699,300

Sub total \$5,361,300

Operation and Maintenance Costs	units per 6 months	cost per unit	
Pump electricity ^A 0.5 kWh/m ³	6387500 m ³	\$0.10	\$319,375
Maintenance			\$120,000
Rowe Street water treatment	6387500 m ³	\$1.95	\$12,455,625
Hunua Ranges water treatment ^B	3193751 m ³	\$1.19	\$3,800,563
Waitakere Ranges water treatment ^B	3193751 m ³	\$2.30	\$7,345,626

Sub total \$24,041,189

Year	\$/m ³
First Year	4.60
Remaining Years	3.76

^A Only pumping during extraction, gravity injection

^B Assuming equal volumes of recharge water from both ranges

Option 2b: One Tree Hill Soakage 35000 m³d⁻¹

Capital costs	units	cost per unit	cost
Land	One Tree Hill (ARC)		
soakage wells 150mm/15m	27	\$2,000	\$54,000
Water Storage tank	1	\$40,000	\$40,000
Deliver pipe 360 mm	500 m	\$1,000	\$500,000
Monitoring wells with coring 80m	6	\$12,000	\$72,000
Resource consents & reports			\$50,000
One Tree Hill Site development			\$150,000
Contingency	15%		\$129,900

Sub total \$995,900

Operation and Maintenance Costs	units per 6 months	cost per unit	
Pump electricity 0.5 kWh/m ³	0 m ³	\$0.10	\$0
Maintenance			\$80,000
Rowe Street water treatment	182500 m ³	\$1.95	\$355,875
Hunua Ranges water treatment ^B	3193750.5 m ³	\$1.19	\$3,800,563
Waitakere Ranges water treatment ^B	3193750.5 m ³	\$2.30	\$7,345,626

Sub total \$4,236,438

Year	\$/m ³
First Year	28.67
Remaining Years	23.21

^B Assuming equal volumes of recharge water from both ranges

Option 2c: Three Kings Groundwater 7500 m³d⁻¹

Capital costs	units	cost per unit	cost
Land	One Tree Hill (ARC)		
Delivery pump	Use existing pumping wells		
Soakage wells 150mm/15m	6	\$2,000	\$12,000
Water Storage tank	1	\$40,000	\$40,000
Deliver pipe 300 mm	2000 m	\$1,000	\$2,000,000
Monitoring wells with coring 80m	6 m	\$12,000	\$72,000
Resource consents & reports			\$50,000
Contingency	15%		\$326,100

Sub total \$2,500,100

Operation and Maintenance Costs	units per 6 months	cost per unit	
Pump electricity ^C 0.5 kWh/m ³	1368750 m ³	\$0.10	\$68,438
Maintenance			\$80,000
Rowe Street water treatment	91250 m ³	\$1.95	\$177,938

Sub total \$326,375

Year	\$/m ³
First Year	30.98
Remaining Years	3.58

^C cost of pumping may be shared between the quarry operation and Watercare

Appendix I: Upper Nihotupu Monthly Water Level Data

Monthly water table levels are listed in Table I.1. Automatic water table measurements at well 4 are listed on the attached CD-ROM, File: Upper Nihotupu hydrological data.xls.

Table I.1. Upper Nihotupu study area monthly water table levels.

	Well 1 water table (m amsl)	Well 2 water table (m amsl)	Well 3 water table (m amsl)	Well 4 water table (m amsl)	Well 5 water table (m amsl)	Well 6 water table (m amsl)
Ground Elevation	298.600	320.000	299.000	299.600	294.600	290.600
26-Jul-99	298.260	319.700	297.890	299.030	292.350	287.270*
28-Aug-99	297.800	318.980	297.400	298.700	291.500	286.960*
27-Sep-99	297.520	318.750	296.500	298.500	291.250	286.740
3-Nov-99	297.140	318.190	295.660	298.130	290.520	286.490
27-Nov-99	297.760	319.280	297.380	298.650	291.650	287.130
6-Dec-99	297.460	318.740	296.810	298.530	291.170	286.670
4-Jan-00	297.190	317.690	295.350	297.900	289.770	286.330
4-Feb-00	297.140	317.620	295.070	297.280	289.940*	286.310
6-Mar-00	296.705	316.805	294.060*	296.910	289.770	286.330
12-Mar-00	296.605	316.722	293.950*	296.870	288.980*	286.240
5-Apr-00	296.520	316.440	293.590*	296.730	288.690*	286.230
2-May-00	297.520	318.160	295.400	297.350	290.040	286.450
7-Jun-00	298.230	319.620	297.940	299.080	292.330	287.570
6-Jul-00	298.100	319.550	298.020	298.950	291.990	287.630
7-Aug-00	297.870	319.370	297.470	298.710	291.530	287.080
13-Sep-00	298.100	319.420	297.740	298.830	291.870	287.340
5-Oct-00	298.220	319.600	297.770	299.085	292.120	287.420
9-Nov-00	298.220	319.610	297.365	298.975	292.290	287.160
7-Dec-00	297.680	318.860	296.190	298.480	291.010	286.650
14-Dec-00	297.625	318.605	295.980	298.400	290.865	286.605
10-Jan-01	297.100	317.990	295.410	297.870	290.090	286.420
7-Feb-01	296.840	317.650	295.330	297.270	289.850	286.380
6-Mar-01	297.220	318.390	296.560	298.030	290.870	286.540
11-May-01	298.200	319.610	296.450	299.050	291.810	287.120
12-Jun-01	298.190	319.610	297.560	299.045	292.255	287.365

* no values measured, these are based on linear regression with well 2.

Appendix J: Upper Nihotupu Hydraulic Conductivity Tests

Auger Hole-Test Technique

The auger hole hydraulic conductivity technique was applied to a hole dug adjacent to well 6.

Equation J.1 (Boast and Kirkham 1971) allows estimation of the hydraulic conductivity (K) near the well from the recovery in the water table over time (dy/dt) after a slug of water has been removed.

$$K = C \frac{(-dy / dt)}{864}$$

Equation J.1

Where C is a dimensionless constant based on the geometry of the auger hole, water table elevation, and depth to the aquifer's underlying material. Numerous combinations of the above parameters have been tested by Boast and Kirkham (1971) to produce a table of C values.

In this equation K and $-dy / dt$ have the units of md^{-1} , C and 864 are dimensionless.

The auger hole adjacent to well 6 extended 1.8 m below the watertable, and was saturated for half of its depth. The auger hole diameter was 0.065 metres, and the depth to the underlying impermeable layer was approximately 20 metres below the watertable. A C value of 1.43 was selected for this auger hole based on the ratio of s/d and d/a . The calculated conductivity was 0.0212 md^{-1} using the following parameters as shown in Figure J.1.

$dy = 0.63 \text{ m}$	$dt = 0.04919 \text{ days}$	$a = 0.033 \text{ m}$	$s = 14.4 \text{ m}$
$d = 1.8 \text{ m}$	$d/a = 54$	$s/d = 8$	

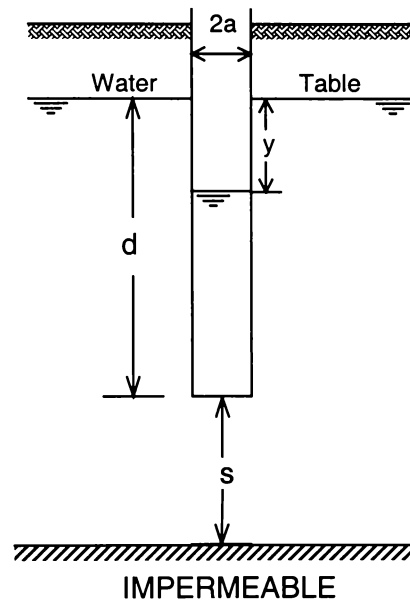


Figure J.1. Geometry and symbols of the auger hole method adapted from Boast and Kirkham (1971).

Bouwer and Rice Bail Test

Bail tests on wells 2, 4, and 6 were performed using the Bouwer and Rice (1976) method, the geometry of the well used in their method is shown in Figure J.2.

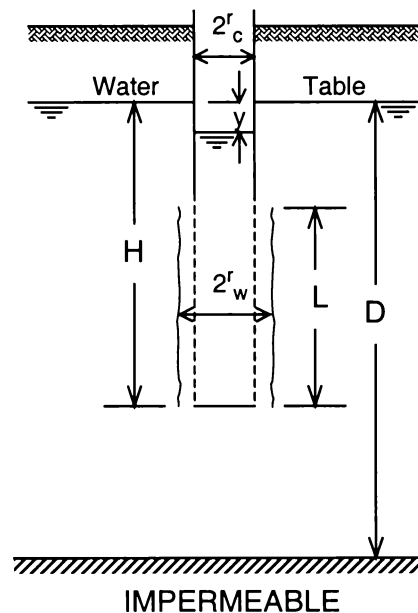


Figure J.2. Geometry and symbols of a partially penetrating, partially perforated well in an unconfined aquifer with a gravel pack or developed zone around the perforated section, from Bouwer and Rice (1976).

The conductivity (K) is calculated using Equation J.2 and the parameters listed in .Table J.1

$$K = -\frac{r_c^2 \ln(R_e / r_w)}{2L} \frac{1}{t} \ln \frac{y_0}{y_t}$$

Equation J.2

where the radius of the well and effective radius of the packing zone is;

$$r_c = [r_c^2 + n(r_w^2 - r_c^2)]^{0.5}$$

Equation J.3

and,

$$\ln \frac{r_e}{r_w} = \left[\frac{1.1}{\ln(H / r_w)} + \frac{A + B \ln(D - H) / r_w}{L / r_w} \right]^{-1}$$

Equation J.4

Table J.1. Parameters for calculating aquifer the conductivity using the Bouwer and Rice Bail Test.

Parameter	Well 2	Well 4	Well 6
D m	20	20	20
H m	3.52	4.38	2.11
L m	3.52	4.38	2.11
r_w m	0.055	0.0325	0.055
r_c m	0.0345	0.03	0.0345
L/r_w	63.9	146	38.4
A^*	3.4	5.5	2.6
B^*	0.56	1	0.45
$\ln[(D - H)/r_w]$	5.7	6	5.78
$\ln(r_e/r_w)$	2.72	3.299	2.27
t seconds	6000	1500	12500
y_0	0.95	1	1.35
y_t	0.1	0.1	0.0045
K md ⁻¹	0.025	0.045	0.025

* values from Figure 3 in Bouwer and Rice (1976).

Table J.2. Water level data for Upper Nihotupu aquifer bail tests.

Auger Hole Bail Test		Bouwer and Rice Bail Test					
Adjacent to well 6		Well 6 Bail Test		Well 4 Bail Test		Well 2 Bail Test	
Seconds	Rise in water level (m)	Seconds	Rise in water level (m)	Seconds	Rise in water level (m)	Seconds	Rise in water level (m)
20	0.635	0	1.350	0	0.9000	0	0.9500
60	0.630	100	1.300	100	0.7800	80	0.6600
90	0.620	200	1.240	200	0.7000	180	0.5200
120	0.610	300	1.190	300	0.6200	240	0.4500
170	0.600	400	1.140	500	0.5000	360	0.3700
210	0.590	500	1.100	700	0.4100	420	0.3200
260	0.580	700	1.000	1000	0.2400	540	0.2800
310	0.570	1000	0.900	1250	0.1750	650	0.2300
370	0.560	1250	0.800	1400	0.1400	750	0.2200
430	0.550	1600	0.700	1700	0.0280	800	0.2050
497	0.540	2000	0.600	2000	0.0035	900	0.1900
563	0.530	2500	0.500	2300	0.0002	1000	0.1800
640	0.520	2900	0.400			2000	0.1200
710	0.510	4250	0.220			4080	0.1050
802	0.50	4800	0.170			4300	0.1045
892	0.490	6000	0.075				
993	0.480	6600	0.045				
1085	0.470	7200	0.030				
1155	0.460	9600	0.030				
1272	0.450	11800	0.024				
1356	0.440	14400	0.023				
1465	0.430						
1615	0.420						
1750	0.410						
1846	0.40						
2057	0.390						
2230	0.380						
2399	0.370						
2551	0.360						
2703	0.350						
2837	0.340						
2975	0.330						
3116	0.320						
3300	0.310						
3499	0.300						
3673	0.290						
3870	0.280						
4080	0.270						
4305	0.260						
4581	0.250						
4755	0.240						
4904	0.230						
5117	0.220						
5400	0.210						
5610	0.200						
5865	0.190						
6110	0.180						
6397	0.170						

Appendix K: MODFLOW Sensitivity Analysis

Each of the parameters used to achieve the model calibration were individually adjusted and the resulting changes in the model water levels at the six existing observation wells and change in total stream baseflow discharge at the end of the simulation were measured (Table K.1). The model was most sensitive to changes in recharge. Changes in hydraulic conductivity or specific yield of the geological zones made little difference to the water levels or stream discharge. There was also little change in the stream discharge due to a change in the drain boundary conductance used to simulate the streams. This may be due to the small head difference between the aquifer and elevation of the drain boundary representing the stream.

Table K.1. Sensitivity analysis for the change in water levels and stream baseflow discharge from the calibrated and validation data.

Aquifer and hydrologic characteristic varied	Hydrologic value simulated	Percent change	Head changes		Discharge percent change	Aquifer and hydrologic characteristic varied	Hydrologic value simulated	Percent change	Head changes		Discharge percent change
			RMSE (m)	MAE (m)					RMSE (m)	MAE (m)	
Zone 1						Zone 7					
Horizontal hydraulic conductivity (m/d)	0.45	+ 50	0.04	0.02	0.5	Horizontal hydraulic conductivity (m/d)	0.675	+ 50	0.41	0.25	-0.11
	0.3	0	0	0	0		0.45	0	0	0	0
	0.15	- 50	0.07	0.03	-1.1		0.225	- 50	0.8	0.48	-0.4
Specific yield	0.015	+ 50	0	0	-2.1	Specific yield	0.03	+ 50	0.05	0.04	-0.32
	0.01	0	0	0	0		0.02	0	0	0	0
	0.005	- 50	0.01	0	2.3		0.01	- 50	0.13	0.1	-0.09
Zone 2						Zone 8					
Horizontal hydraulic conductivity (m/d)	0.6	+ 50	0.2	0.08	0.09	Horizontal hydraulic conductivity (m/d)	0.225	+ 50	0.11	0.06	-0.22
	0.4	0	0	0	0		0.15	0	0	0	0
	0.2	- 50	0.32	0.13	-0.17		0.075	- 50	0.27	0.12	-0.23
Specific yield	0.015	+ 50	0.02	0.01	-0.22	Specific yield	0.015	+ 50	0.06	0.03	-0.23
	0.01	0	0	0	0		0.01	0	0	0	0
	0.005	- 50	0	0	0.24		0.005	- 50	0.05	0.03	-0.21
Zone 3						Zone 9					
Horizontal hydraulic conductivity (m/d)	0.375	+ 50	0.02	0.01	-0.12	Horizontal hydraulic conductivity (m/d)	0.9	+ 50	0.09	0.05	-0.18
	0.25	0	0	0	0		0.6	0	0	0	0
	0.125	- 50	0.02	0.01	-0.44		0.3	- 50	0.12	0.06	-0.29
Specific yield	0.03	+ 50	0.02	0.01	-0.51	Specific yield	0.03	+ 50	0.04	0.02	-0.29
	0.02	0	0	0	0		0.02	0	0	0	0
	0.01	- 50	0.02	0.01	0.07		0.01	- 50	0.02	0.01	-0.14
Zone 4						Zone 10					
Horizontal hydraulic conductivity (m/d)	0.3	+ 50	0.43	0.19	-0.1	Horizontal hydraulic conductivity (m/d)	0.12	+ 50	0.12	0.08	-0.2
	0.2	0	0	0	0		0.08	0	0	0	0
	0.1	- 50	0.69	0.31	-0.5		0.04	- 50	0.28	0.18	-0.26
Specific yield	0.03	+ 50	0.16	0.09	-0.6	Specific yield	0.015	+ 50	0.05	0.03	-0.27
	0.02	0	0	0	0		0.01	0	0	0	0
	0.01	- 50	0.07	0.04	0.2		0.005	- 50	0.04	0.02	-0.18
Zone 5						All Zones					
Horizontal hydraulic conductivity (m/d)	0.15	+ 50	0.1	0.05	0	Recharge		+ 25	0.75	0.61	25.8
	0.1	0	0	0	0			0	0	0	0
	0.05	- 50	0.11	0.06	-0.7			- 25	0.84	0.67	-26.2
Specific yield	0.03	+ 50	0.16	0.07	-0.8	Nihotupu Stream Conductance (m/d)	4.5/1.5/0.15	+ 50	0.28	0.23	0.05
	0.02	0	0	0	0		3/1/0.1	0	0	0	0
	0.01	- 50	0.13	0.06	0.43		1.5/0.5/0.05	- 50	0.63	0.51	-1
Zone 6											
Horizontal hydraulic conductivity (m/d)	0.75	+ 50	0.11	0.06	-0.14						
	0.5	0	0	0	0						
	0.25	- 50	0.19	0.13	-0.42						
Specific yield	0.03	+ 50	0.04	0.03	-0.48						
	0.02	0	0	0	0						
	0.01	- 50	0.04	0.02	0.03						

Appendix L: MODFLOW Drain Conductance Estimation

Step 1: Calculation of drain spacing using an Glover-Dunn equation

Calculation of the drain spacing using the Glover-Dunn equation as shown in the example below were adapted from Wesseling and Kessler (1973) using parameters from the Nihotupu aquifer calibrated numerical model described in Chapter 6.

The estimation of the drain spacing is based on the following simulated example. Every 30 days the water table is instantaneously recharged (R_i) by 0.0344 m of rainfall. The soil has an effective porosity $\mu = 0.02$, and the recharge causes an instantaneous rise in the water table, $\Delta h = R_i/\mu = 0.0345/0.02 = 1.72$ m. The maximum permissible height of the water table is set at 4 m below the surface. The drain elevation is 6 m below the surface so that $h_0 = 6 - 4 = 2$ m. The water table must be lowered by $\Delta h = 1.72$ m in the next 30 days so that the water table is 4 m below the surface before the next recharge event. Therefore, we have $h_{30} = h_0 - \Delta h = 2 - 1.72 = 0.28$ m. If the depth to the impervious layer below the surface is 20 m, the average hydraulic conductivity $K = 0.27$ m, and the radius r_0 of the pipe drain is 0.15 m, the drain spacing, discharge, and water table level can be calculated. From the above information we have

$K = 0.27 \text{ md}^{-1}$	$t = 30 \text{ days}$	$D = 14 \text{ m}$	$r_0 = 0.15 \text{ m}$
$\mu = 0.02$	$h_0 = 2$	$h_{30} = 0.28 \text{ m}$	

Substituting the above data into the Glover-Dunn equation the drain spacing L is,

$$L = L_0 - C = 162.7 - 47.5 = 115.2 \text{ m}$$

Equation L.1

where,

$$L_0 = \pi \left[\frac{kDt}{\mu} \right]^{1/2} \left[\ln \frac{1.16h_0}{h_i} \right]^{-1/2} = \pi \left[\frac{1.9 * 14 * 30}{0.02} \right]^{1/2} * \left[\ln \frac{1.16 * 2}{0.28} \right]^{-1/2} = 162.7 \text{ m}$$

Equation L.2

and,

$$C = D \ln \frac{D}{u} \text{ where } u = \pi r_0 = 47.5$$

Equation L.3

Step 2: Analytical computation of water table height and drain discharge

The Kraijenhoff van de Leur-Maasland equation (Wesseling and Kessler, 1973, Eq. 50 & 52, p. 37) calculates the water table elevation midway between two parallel drains and the discharge rate in response to a constant and continuous recharge. The following example is adapted from Example 9 in Wesseling and Kessler (1973, p. 39), and utilises the same parameters as given in Step 1 above.

The recharge as applied in Step 1 is converted from an instantaneous event to a constant rainfall event over the 30 days, $R = 0.00115 \text{ md}^{-1}$. Replacement of D , by d -value in Kraijenhoff van de Leur-Maasland equation to account for convergence of flow in the vicinity of the drains, $d = 8.2 \text{ m}$.

From Table 3 in Wesseling and Kessler (1973) page 40, the water table elevation and drain discharge was calculated where j is 12 days, and $\frac{R}{\mu} j$ is 0.069 m. The water table level and drain discharge is shown in Table L.1.

Step 3: MODFLOW computation of water table height and drain discharge

A MODFLOW model was constructed with the same parameters as used in the above analytical model. The model domain consisted of one layer with 20 by 20 m cells. The drains were at a depth of six metres below the ground surface and 115 m apart. The conductance value of the drains was adjusted until the discharge of each drain and the water table in between two drains was similar to that achieved by the analytical model. A conductance of 4.2 md^{-1} produced a good match. The results are shown in Table L.1 and Figure L.1.

Table L.1. Comparison of water table elevation midway between two drains and the discharge for the Wesseling and Kessler (1973) analytical model and a MODFLOW model with a drain conductance of 4.2 md^{-1} .

Days	ANALYTICAL		MODFLOW	
	Discharge	height	Discharge	height
1	5.60	0.12	2.0	0.12
5	11.30	0.58	7.8	0.57
10	16.31	1.05	13.0	1.01
15	19.13	1.37	17.0	1.34
20	22.50	1.60	19.9	1.58
25	23.60	1.76	22.0	1.76
30	24.80	1.86	23.7	1.88

Discharge $\text{m}^3\text{d}^{-1}/100$ metre length of drain.

Height m above drain elevation

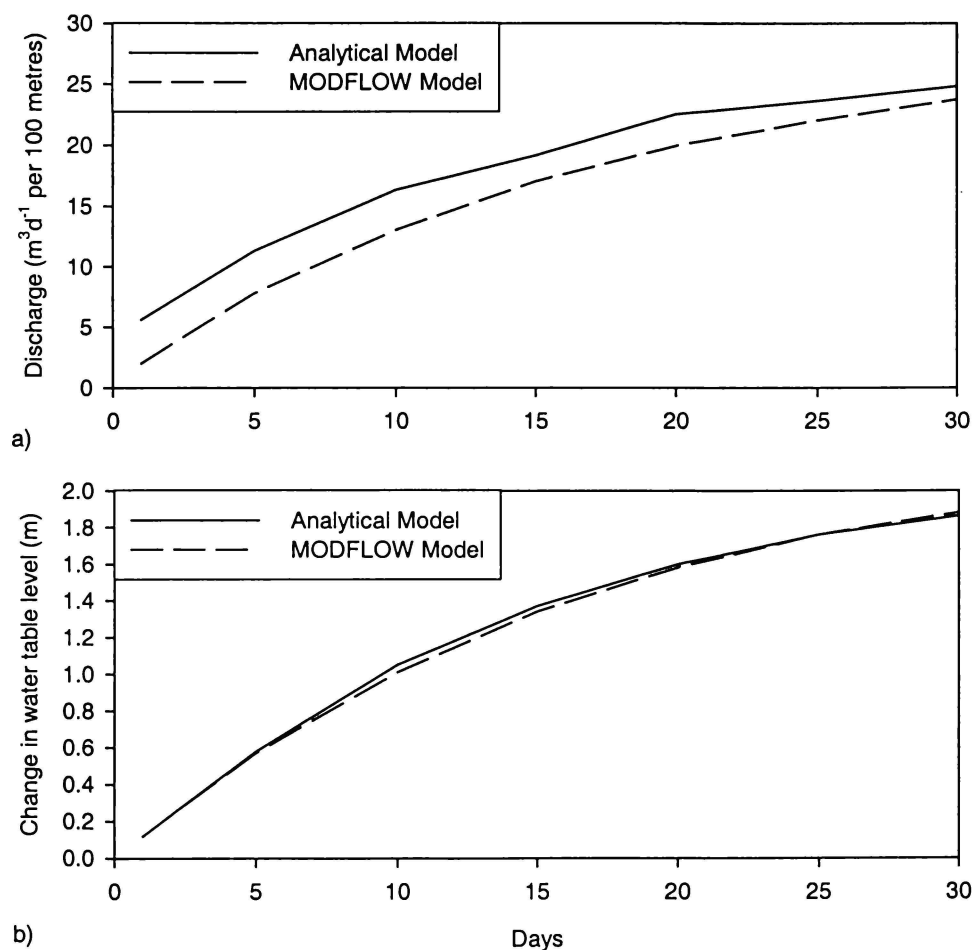


Figure L.1. Comparison of a) discharge and b) water table elevation midway between two drains (spacing 115 m) for the Wesseling and Kessler (1973) analytical model and a MODFLOW model with a drain conductance of 4.2 md^{-1} .