REPORT

WAIMEA WATER AUGMENTATION COMMITTEE

Waimea Water Augmentation Phase 2 – Water Resource Investigations

Report prepared for:

WAIMEA WATER AUGMENTATION COMMITTEE

Report prepared by:

TONKIN & TAYLOR LTD

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Executive Summary

This report presents the results of **water resource investigations** completed as part of Phase 2 of the Waimea Water Augmentation Feasibility Study. It is based on a potential dam on the Lee River, at a site approximately 300 metres upstream of the confluence of Anslow Creek and the Lee River in Tasman District.

This report addresses the following:

- confirmation of the potential future water demand for irrigation use, as well as long-term community and industrial demands, plus in-stream flow requirements;
- groundwater-surface water interaction modelling of the Wairoa River and Waimea aquifers and development of a flow augmentation regime for meeting future demand and a residual flow in the Waimea River of 1100 ℓ/s at Appleby Bridge;
- confirmation of flow characteristic and water availability of the Lee River;
- assessment of flood hydrology and development of design flood hydrographs for feasibility design of the dam spillway and construction diversion;
- dam storage modelling to confirm the required storage capacity at the selected dam site on the Lee River to service the identified needs;
- description of a preliminary operating regime at the proposed dam including the flow changes anticipated in the Lee River immediately below the dam and at the Wairoa Gorge; and
- description of an operating regime incorporating a hydropower add-on.

A peer review process was integrated into the water resources investigations in which external peer reviewers were engaged to review critical aspects of the study.

Compared with Phase 1, there are a number of enhancements in the analytical methods and databases applied, and several changes to the demand parameters. The more notable differences are that:

- in terms of demand, the irrigation service area has been expanded to include 250 ha of Rabbit Island, and an increased urban and industrial take based on projected demand in 100 years' time has been modelled;
- the dam site is 1.4 km upstream of the previously identified potential site (moved for geotechnical reasons), and has an 8% smaller catchment area, i.e. now 77.5 km^2 versus 83.8 km^2 in Phase 1.

The main points and key findings from this study are as follows:

- TDC installed a new flow monitoring station on the Lee River (Lee at Waterfall Creek) in April 2007, and the data from the station have been used to confirm the surface water resources of the Lee River and at the dam site, as well as the flood response of the upper Lee catchment to storm rainfall.
- The estimated long-term mean flow of the Lee River at the proposed dam site is 3.60 m^3/s, which represents 22.2% of the flow recorded at Wairoa Gorge; this flow is marginally higher than the Phase 1 estimate despite a smaller catchment.
- Detailed flood assessments have been completed for dam engineering feasibility design. The 10 year, 100 year, 1000 year and 10,000 year return period peak

inflows at the dam site are estimated to be 255 m^3/s, 375 m^3/s, 496 m^3/s and 616 m^3/s respectively; the Probable Maximum Flood is estimated to be 1094 m^3/s.

- More extensive groundwater–surface water interaction modelling has been completed, which includes the severe 2000/2001 drought and a more "normal" year (2004/2005) in addition to the 1982/1983 drought; the modelling results have been used to develop more robust empirical functions (based on river flow and demand) to determine the flow augmentation required at Wairoa Gorge on a daily basis.
- Utilising the full Wairoa River flow record from 1958 to 2008, a daily simulation of the proposed Lee River reservoir behaviour has been completed, in which releases from the dam are made to meet downstream needs, including in-stream minimum flows of 510 ℓ/s at the dam toe and 1100 ℓ/s at Appleby Bridge; the upgraded dam storage modelling uses an accurate (variable) irrigation demand pattern based on historical rainfall and other climatic data and the refined flow augmentation functions.
- Based on the updated water demands and upgraded storage modelling, a dam with a gross storage capacity of 13 million m^3 (net 12.0 million m^3 plus 1.0 million m^3 for dead storage) will provide security against a 66 year return period drought; on the same basis, the 1982/1983 drought is ranked a 33 year return period event while the 2000/2001 drought is ranked a 65 year return period event.
- At the proposed dam site, the full supply level corresponding with a gross storage capacity of 13 million m^3 is RL 196.4 m; this has been rounded up to RL 197 m for the dam feasibility design. At this level (RL 197 m), the reservoir will extend approximately 4 km upstream from the dam, cover an area of approximately 65 hectares, and have a gross storage of about 13.4 million m^3.
- A preliminary operating regime has been identified from the storage simulation; it is anticipated that, when operating to the full future demand, the reservoir will be full for 83% of the time and within 1 m of full for about 90% of the time on long-term average; the maximum release required from the dam to meet downstream water demands (apart from the environmental flushing flow requirement of 5 m^3/s) will be 2230 ℓ/s.
- After construction of the dam and implementation of the flow augmentation regime, the downstream flow regime will be very similar to the pre-existing regime for the majority of the time; the departures are during summer low flow periods when river flows will be boosted by flow releases from the dam, and during refilling of the reservoir typically over summer (short periods) and autumn when river flows will be slightly lower.

The proposed dam on the Lee River offers an opportunity for a cost-effective hydropower scheme to be added. A preliminary optimisation to determine the preferred arrangements, particularly the size of the power plant and the additional operational storage is described in the accompanying "Engineering Feasibility Report".

With the addition of the preferred hydropower arrangements at the dam (comprising a 1.0 MW turbine and generator with design generation flow capacity of 2.5 m^3/s plus 250,000 m^3 of hydro operational storage) the reservoir level will fluctuate for much of the time within a 0.39 m range between RL 196.61 m and RL 197.0 m rather than remain steady at the full supply level of RL 197.0 m (for the "without hydro" case).

1 Introduction

1.1 Background

In 2007 Tonkin & Taylor Ltd and its sub-consultants completed a Phase 1 pre-feasibility evaluation of a number of options to provide water storage for long-term irrigation and community supplies in the Waimea Basin, Tasman District. The evaluation was undertaken on behalf of the Waimea Water Augmentation Committee (WWAC). The overall principle of the study was to identify and develop a water augmentation scheme to capture excess water for storage and release that water back into the Waimea River system during periods of high water demand and/or low natural water flows to augment those supplies, either directly or via recharging of the groundwater system.

The outcome of that Phase 1 study was to focus feasibility investigations on a water storage dam and reservoir site located in the upper Lee River catchment, a tributary of the Waimea River.

In 2007 WWAC initiated Phase 2 of the study, to take the Lee investigation programme to feasibility level.

This report presents the results of **water resource investigations** completed as part of the Phase 2 feasibility study. It is based on a potential dam on the Lee River in Tasman District, at a site approximately 300 metres upstream of the confluence of Anslow Creek and the Lee River. The required storage capacity of the reservoir has been determined to be approximately 13 million m^3, with a normal top water level to RL 197 m. The reservoir would extend approximately 4 km upstream from the dam, and cover an area of approximately 65 hectares (based on normal top water level).

Figure 1 shows the location of the proposed dam, and the indicative reservoir extent.

1.2 Scope

This report on water resource investigations addresses the following aspects:

- confirmation of the potential future water demand for irrigation use, as well as long-term community and industrial demands, plus in-stream flow requirements
- groundwater-surface water interaction modelling of the Wairoa River and Waimea aquifers and development of a flow augmentation regime for meeting future demand and a residual flow of 1100 ℓ/s at Appleby Bridge
- confirmation of flow characteristic and water availability of the Lee River
- flood hydrology and development of design flood hydrographs for feasibility design of the dam spillway, and for construction diversion
- dam storage modelling to confirm the required storage capacity at the selected dam site on the Lee River to service the identified needs
- description of a possible operating regime at the proposed dam including the flow changes anticipated at the dam site and at the Wairoa Gorge
- description of an operating regime based on a possible hydro-electric power add-on.

The work reported here incorporates input from the following entities:

- Tasman District Council (hydrometric data; urban and industrial demand)

- Tonkin & Taylor (overall integration; hydrological modelling and storage demand assessment; flood hydrology)
- GNS Science (groundwater modelling)
- AgFirst Consultants (consumptive water demand)
- Landcare Research (irrigation demand scheduling)
- Lincoln Ventures (external peer review)
- Engineering Geology (external peer review).

1.3 Peer Review

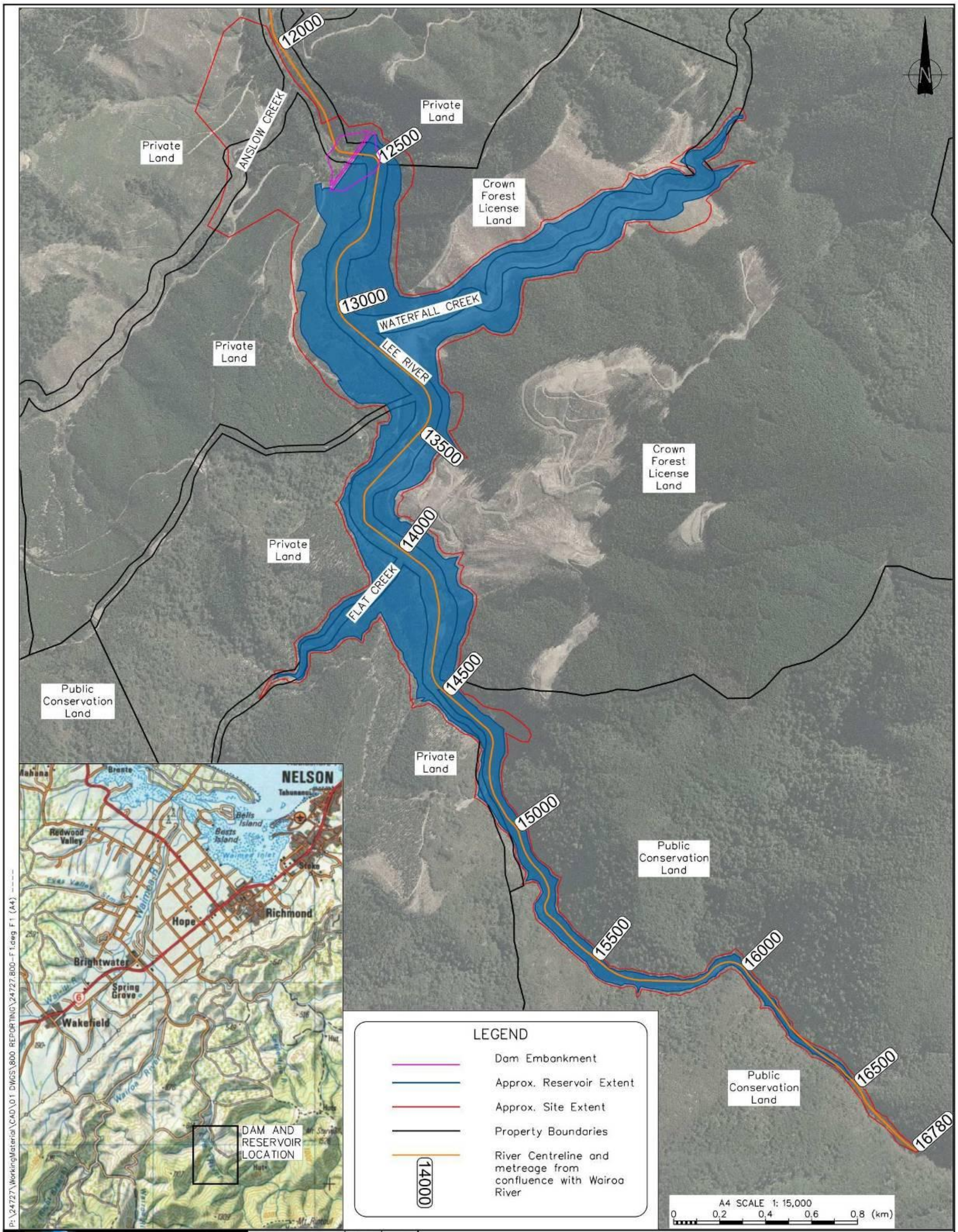
A peer review process was integrated into the water resources investigations in which a external peer reviewers, Dr Vince Bidwell of Lincoln Ventures Ltd and Dr Trevor Matuschka of Engineering Geology Ltd, were engaged to review critical aspects of the study.

Dr Bidwell was involved as follows:

- initial briefing and attendance at the technical workshop on 10th October 2007 (see Section 2.2)
- broad review of water demand parameters
- overview of groundwater modelling approach
- overview of surface water modelling approach and coupling with groundwater modelling
- review of hydrology, water augmentation concept/operation and assessment of storage requirement for the target drought security.

Three review notes were issued by Dr Bidwell addressing various aspects of the investigations. His second review note closed out some of the initial reservations/ uncertainties expressed in the first review note, while his third and final review note addressed remaining reservations. Dr Bidwell's peer review notes are included in Appendix A.

Dr Matuschka's primary involvement in this project was as external peer reviewer for geotechnical and dam engineering aspects, as described in the accompanying Dam Engineering Feasibility Study report. In this Water Resources Investigations report, Dr Matuschka reviewed the Catchment Hydrology and Flood Hydrology sections. His review note is also included in Appendix A.



Tonkin & Taylor Environmental & Engineering Consultants ☐ Auckland ■ Nelson ☐ Christchurch ☐ Hamilton ☐ Wellington ☐ Whangarei	DRAWN: DNV Sep.09 DRAFTING CHECKED APPROVED CADFILE : \\24727.800–F 1.dwg SCALES (AT A4 SIZE) 1: 15,000 PROJECT No. 24727.800	WAIMEA WATER AUGMENTATION COMMITTEE WAIMEA WATER AUGMENTATION – PHASE 2 SITE LOCATION PLAN FIG. No. FIGURE 1 REV. 0

2 Water Demand

2.1 Previous Phase 1 Study

Irrigation water demand in the previous Phase 1 study was based on an equivalent net irrigable area in the Waimea Basin of 5306 hectares. This figure was derived from an assessed gross area of 6,582 hectares before urban areas, road reserves, river reserves and other such areas were excluded. By including part of the lower Wai-iti Valley that is not serviced by the existing Wai-iti Community Dam (Kainui), i.e. between Wakefield and Aldourie Rd, the total net irrigable area increased to about 5600 hectares.

Besides irrigation water, a provision was made for urban and industrial water demand, for existing and additional future expected demand. Based on a 50 year planning horizon, Tasman District Council (TDC) provided a projection of the total urban and industrial demand of 820 hectare-equivalents, comprising an existing allocation of 540 hectare-equivalents and a future demand of 280 hectare-equivalents.

Table 2.1 summarises the approximate areas/zones and corresponding demand adopted for Phase 1.

Subsequently, and in addition to the demands listed in Table 1, an allowance for "future regional need" identified by TDC of 22,000 m^3/day was incorporated. This was modelled as a constant year round surface water take of 254 ℓ/s (=22,000 m^3/day).

Table 2.1 Water demand projection: Phase 1

Zone	Net Irrigable Area (Ha)	Urban Allocation (Area Equivalents - ha)	Total Hectare Equivalents	Approx. Demand (Peak Daily Flow) ℓ/s
Upper Catchments	Minimal		Minimal	5
Reservoir	580	56 (Brightwater & Wakefield)	636	316
Waimea West	385	23 (Redwood)	408	202
Hope & Eastern Hills (includes Upper and Lower Confined)	2170	154 (Richmond)	2324	1345
Golden Hills	300		300	149
Delta	1246	307 (Waimea)	1553	693
Redwood*	625		625	258
Wai-iti: Aldourie Rd to Wakefield	300		300	174
Future urban demand		280	280	162
TOTALS	**5,606 ha**	**820 ha**	**6426 ha**	**3304 ℓ/s**

2.2 Phase 2 Demand - Workshop 10 October 2007

A technical workshop on water demand was held on 10 October 2007 in Tasman District Council's (TDC) offices in Richmond. The purpose of this workshop was to update and agree the demand assumptions on which to base the live storage requirement for the proposed dam and the input to GNS' groundwater model.

Aspects that were discussed, and a summary of the decisions made at the workshop by the WWAC Technical Group, are as follows:

- Extent of irrigable area
  - provide for irrigation of 250 ha (or about 25%) of Rabbit Island
  - allow for irrigation of 300 ha in lower Wai-iti Valley
  - irrigation application rate to be based on 80% of irrigable area
  - in total have 5856 ha irrigable area (Phase 1 had 5606 ha)

- Crop mix in the irrigation service area
  - irrigation rate to be based on pasture throughout (conservative), with a peak application rate of 30 mm/week

- Design drought and the drought security standard
  - provisionally target 60 year return period drought standard (per Phase 1)
  - rationing not likely an issues given high drought security standard
  - review once drought return period versus storage volume relationship defined (and compare against 12 million m^3 storage for 60 year return period drought determined from Phase 1)

- In-stream residual flow
  - retain Phase 1 minimum flow of 1100 ℓ/s at Appleby Bridge

- Tasman District urban/industrial demand and planning horizon
  - adopt 100 year planning horizon
  - peak daily demand and annual demand pattern was subsequently confirmed by TDC (see Section 2.3.2)

- Future wider regional demand
  - retain Phase 1 allowance of 22,000 m^3/day (surface flow take at Wairoa Gorge), noting this level of demand not projected to be reached until 2060

- Climate change effects
  - mean precipitation may decrease and extremes may worsen
  - no specific measure other than adopting a high drought security standard (with the expectation that the standard may be lower with future climate change)

- Hydro-electric generation
  - explore conjunctive use of dam for power generation (expected to be modest)
  - warrants some optimisation but agreed not to increase dam size significantly to enhance hydro-generation

- Distribution to service areas beyond aquifer zone of supply
  - to determine approximate need for and cost of distribution (of irrigation water) to areas beyond the zone of effect of the aquifers

A summary covering both consumptive and in-stream water demand is provided in the following sub-sections in this chapter.

2.3 Phase 2 Consumptive Water Demand

2.3.1 Irrigation demand

Irrigation usage depends heavily on the actual rainfall pattern over the irrigation season, and to a lesser extent on other climatic variables (wind, temperature, etc.). Clearly, the total volume of water required will be greater when in a drought situation. It is reasonable to assume that high irrigation usage will often coincide with lower than average river flows over an irrigation season. This is because both variables are driven by rainfall patterns to a large extent – low rainfall generally corresponds with low river flows and high irrigation usage. In terms of reservoir storage utilisation, low inflows to the reservoir and high irrigation usage are compounding factors, and so must be captured appropriately in the reservoir simulation. Otherwise the result would be unconservative.

In Phase 1, whilst a time-series record of historical river flows from 1958 to 2005 was used to represent river flow availability (and reservoir inflows), an annually repeating irrigation usage pattern which corresponded with a drought year was adopted to represent consumptive water use. The 1982/83 year was selected as this design drought year. In terms of irrigation usage, the 1982/83 year plots at about the 90th percentile mark and as such may be interpreted as being the 1 in 10 highest usage year. Note that there were only two other years that could have been readily used instead of 1982/83. These years, for which complete datasets of soil moisture and aquifer recharge for groundwater modelling were available, were the 2000/01 year, which was more severe and likely to be too conservative, and the 1990/91 year, which was not a high usage year.

For the current Phase 2 assessment, Landcare Research was commissioned to compute a time-series of irrigation usage corresponding with the period of available river flow data (1958 to 2008). Landcare Research's irrigation scheduling model was used to compute unit area irrigation demands based on the following parameters:

- Soil type, specifically soil moisture holding capacity (38 mm, 78 mm or 130 mm)
- Daily rainfall (viz. based on Nelson Airport data from 1941) and other climate parameters
- Crop type (viz. all pasture, as noted earlier)
- Maximum weekly application rate (viz. 30 mm/week)

A measure of irrigator behaviour viz. "aggressive" versus "modest" (efficient) irrigator. A relatively aggressive behaviour (i.e. a regime where the soil is kept very moist all the time) was assumed in order for the computed demands to be comparable with those computed independently by AgFirst Ltd (Nelson).

A corresponding time-series record of the soil drainage for each of the modelled soil types was also generated from irrigation scheduling, which was used as input to GNS' groundwater model (see Section 3.2) for particular years (1982/1983, 2000/2001, and 2004/2005). Table 2.2 provides a brief summary of the irrigation requirement, rainfall and recharge for the three soil types for the two selected drought years (1982/1983 and 2000/2001) and a recent "normal" year (2004/2005).

Figure 2.1 shows the daily total irrigation demand (inclusive of Waimea East Irrigation Scheme) for the 1982/1983 drought year based on the full service area viz. 5856 ha inclusive of Rabbit Island.

Figure 2.2 shows the year to year variation in the predicted irrigation demand as computed by Landcare Research based on historical climate data . High demand years, in excess of 25 million m^3 per annum, are indicated by red bars i.e. 1973, 1983 and 2001 (year ending 30 June). By contrast, in low usage years such as 1996 and 2002, the irrigation demand can be as low as 11 million m^3 or less than half the requirement in a drought year.

Table 2.2 Summary of irrigation requirement for 3 soil types for selected years

Soil moisture holding capacity (mm)	Irrigation Requirement for Year			
	1982/1983 Rainfall = 664 mm		2000/2001 Rainfall = 681 mm	2004/2005 Rainfall = 1004 mm
	AgFirst's estimate (mm)	Landcare Research (mm)	Landcare Research (mm)	Landcare Research (mm)
38	515	522	539	345
78	499	511	522	288
130	475	502	515	231
	Seasonal Recharge (% of rainfall)			
38	-	35	38	38
78	-	30	34	29
130	-	25	30	24

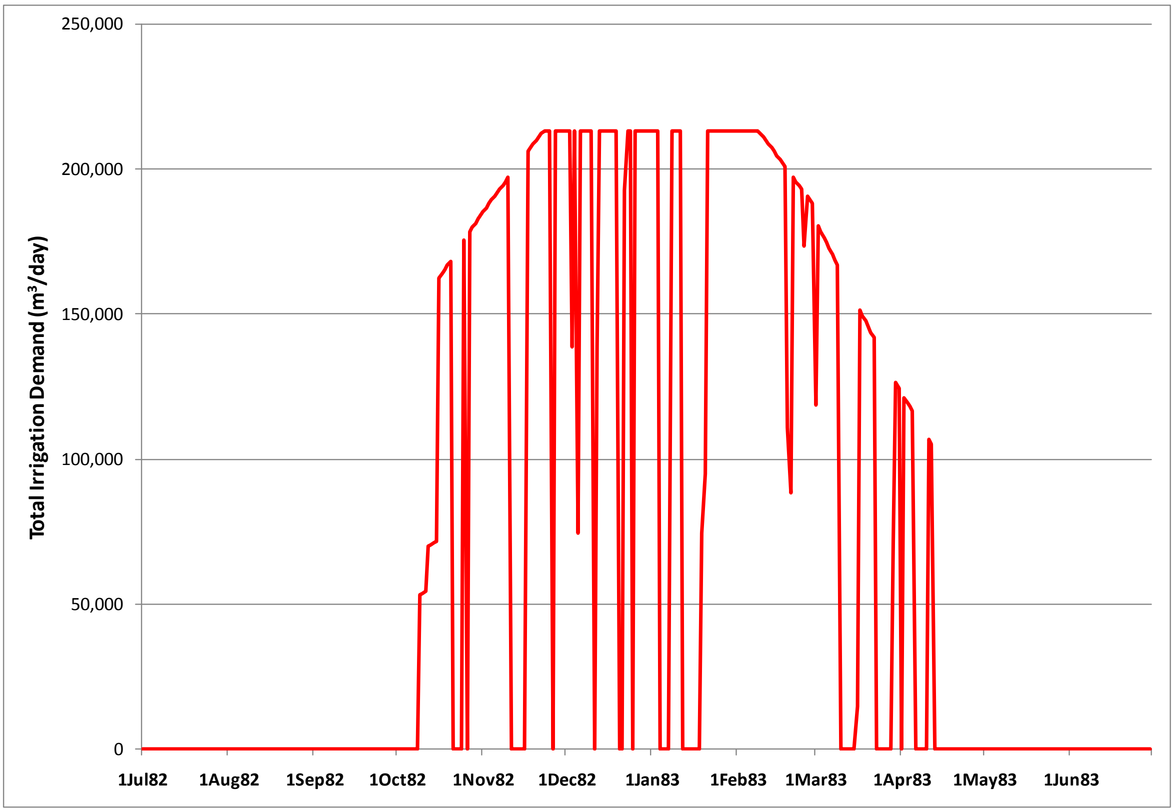


Figure 2.1 Design Irrigation Demand for the 1982/1983 Drought Year

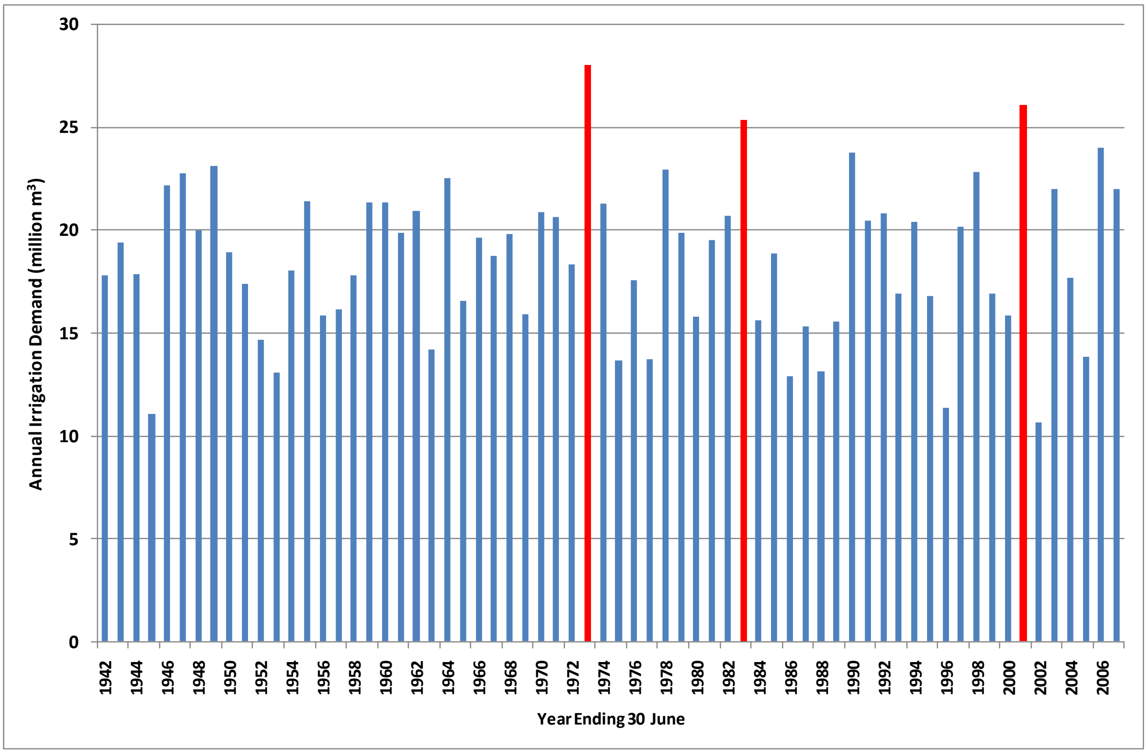


Figure 2.2 Annual Irrigation Demand Volumes from 1942 to 2007(year ending 30 June)

2.3.2 Urban and industrial demand

Based on a 100 year planning horizon, an urban and industrial demand with a nominal peak daily demand of 60,000 m^3/day has been projected by TDC (pers. comm. J. Cuthbertson / J. Thomas, and supplied in spreadsheet under e-mail dated 27th February 2008). This demand is to be met from the aquifers of the Waimea Plains. [NB demand in the spreadsheet cells add to 63,795 m^3/day. However, this includes the volumes assigned to Wakefield (1300 and 2495 m^3/day). As these are supplied by the existing Wai-iti/Kainui Dam, they do not impose an additional demand on the proposed Lee River Dam. They are included in the spreadsheet and added into the GNS groundwater model purely for modelling purposes (these 'cells' are in-built to the model).]

A seasonally varying demand pattern (varying by day) was provided by TDC. Demand was strongly seasonal with the peak demand occurring in January (average demand for the month about 47,000 m^3/day including Wakefield) while the lowest demand was for the month of June (average demand of about 5000 m^3/day including Wakefield).

Figure 2.3 below shows the modelled annual urban and industrial demand pattern including Wakefield demand (6% of total) for all years.

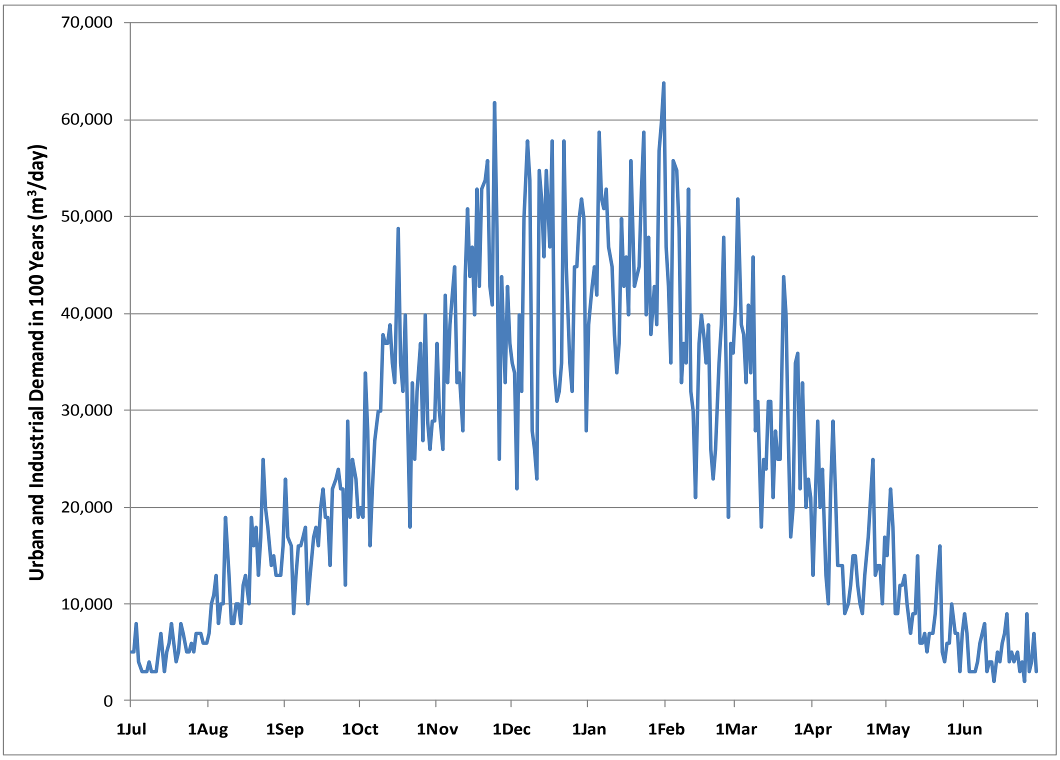


Figure 2.3 Projected Urban and Industrial Demand Pattern in 100 Years' Time

2.3.3 Future regional demand

An allowance for the wider future regional demand was confirmed by WWAC at the technical workshop on 10 October 2007. Similar to Phase 1, a constant year-round surface water take of 22,000 m³/day (254 ℓ/s) at Wairoa Gorge was assumed for this demand component.

2.3.4 Total consumptive demand

Constituents of the total consumptive demand are: the irrigation demand (from both the Waimea aquifers and the Wairoa River) peaking at 213,400 m^3/day; the urban and industrial demand peaking at 63,800 m^3/day; and a constant surface water take of 22,000 m^3/day for future regional need.

As noted earlier, the pattern of demand, in terms of both timing and location, has been re-assessed and refined compared with Phase 1. In Phase 1, a simplistic and conservative approach was adopted whereby a repeating demand pattern based on the 1982/83 drought year was used for the entire available record from 1958 to 2005.

In the current analysis, for the long-term simulation of storage behaviour for the period 1958 to 2008, the modelled irrigation demand follows the irrigation schedule developed by Landcare, which is based on historical climate data and service area assumptions outlined in Section 2.3.1. However, a repeating annual pattern has been retained for the urban and industrial demand component.

Table 2.3 compares the total water demand for Phase 1 and Phase 2 for the year ending 30 June 1983.

Table 2.3 Total water demand for 1982/1983 water year

	Irrigation Take		Urban and Industrial Take (Groundwater) (million m^3)	Future Regional Need (Surface Water) (million m^3)	Total Demand (million m^3)
	WEIS (million m^3)	Groundwater (million m^3)			
Phase 1 (previous)	**5.49**	**22.26**	**4.06**	**8.03**	**39.8**
Phase 2 (current)	**4.96**	**20.45**	**[1] 8.94**	**8.03**	**42.4**
Change	**- 0.53**	**-1.81**	**4.88**	**0.00**	**2.6**

Note 1 Urban and industrial take for Phase 2 includes Wakefield allowance (6% of total) which is supplied by the Wai-iti Dam.

Figure 2.4 compares the Phase 1 and Phase 2 water demand profiles for the 1982/1983 water year, with groundwater and surface water takes shown separately.

For this Phase 2 study, details of the modelled spatial distribution of water takes and its basis, including the particular aquifers from which demand is met, is provided in the groundwater report by GNS Science (attached as Appendix B).

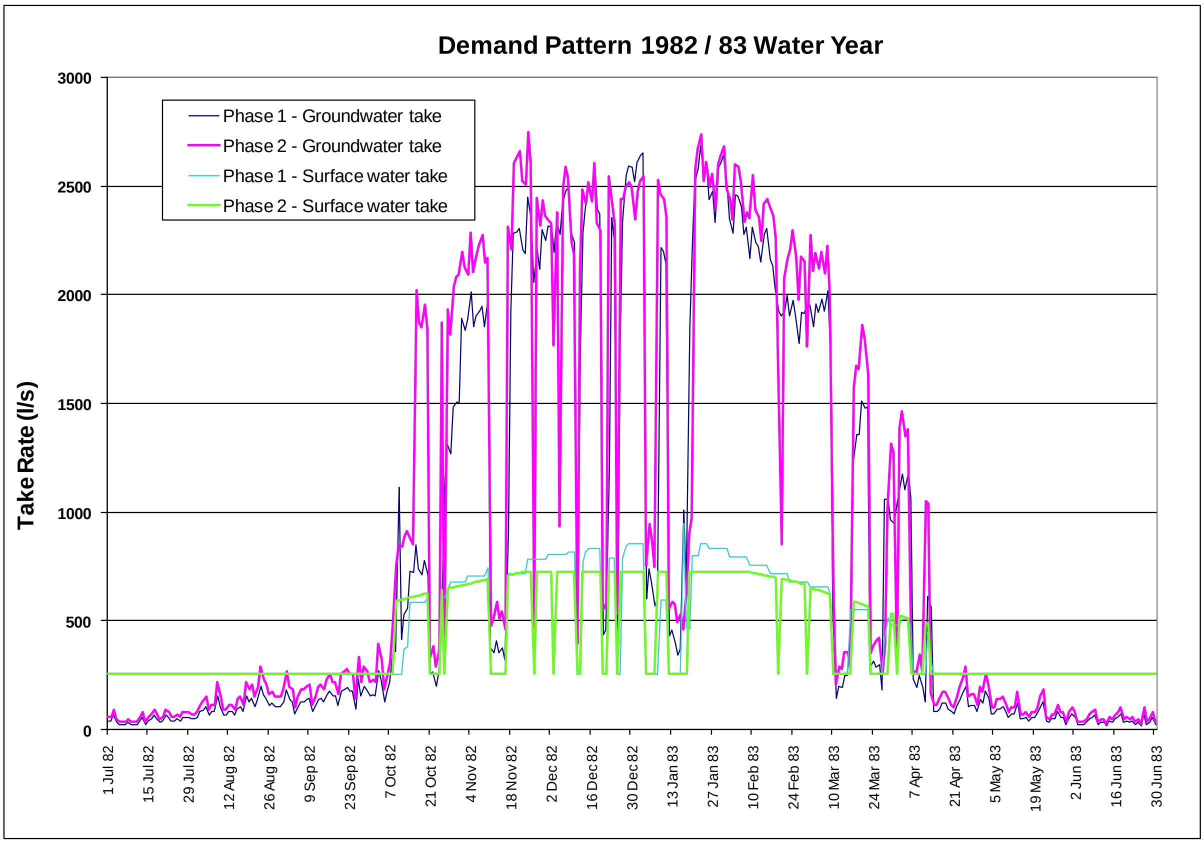


Figure 2.4 Comparison of Phase 1 and Phase 2 Water Demand Profiles for the 1982/1983 Water Year Based on Future Assumed Allocations

2.4 In-Stream Requirements

In Phase 1, Cawthron undertook an assessment of the minimum flows required to provide instream habitat in the lower Wairoa/Waimea River and immediately below the potential dam site. Different minimum flows were identified to span a range from an "environmental benchmark" minimum flow that would be conservative in terms of environmental protection, to a minimum flow that would be weighted towards out-of-stream values.

Their findings from Phase 1 were:

1. instream residual flow requirements at Appleby
   - 1300 ℓ/s (environmental benchmark)
   - 800 ℓ/s (minimum flow retaining 80% of the adult brown trout habitat)
   - 500 ℓ/s (minimum flow retaining 70% of the adult brown trout habitat).
2. instream residual flow requirements immediately below the potential dam site:
   - existing MALF (environmental benchmark)
   - 1 in 5 year low flow
   - 1 in 10 year low flow

WWAC subsequently took a decision to assess the live storage requirements necessary to maintain flows covering much of this range, specifically requesting Tonkin & Taylor to assess two minimum flow scenarios at Appleby Bridge: 600 ℓ/sec and 1100 ℓ/s.

For the reach immediately below the proposed dam, a residual flow equal to the 7-day mean annual low flow (7-day MALF) was conservatively assumed.

For the current Phase 2 study, and as outlined in Section 2.2, WWAC has taken a decision to provide for an in-stream minimum flow of 1100 ℓ/s at Appleby Bridge.

As part of Phase 2, Cawthron has undertaken additional investigations and habitat modelling for the Lee River in order to assess an appropriate minimum flow below the proposed dam, and to provide an indication of the flushing flows required to flush sediment and algae from this reach of the river. Details of Cawthron's study are provided in the accompanying report "Aquatic Ecology" in the series of reports completed for Phase 2.

As a result of these investigations, the environmental benchmark of MALF (viz. 510 ℓ/s at the proposed dam site on the Lee River) has been confirmed as an appropriate minimum flow below the dam. In addition, Cawthron has recommended that a 5 m^3/s flushing flow capability be provided at the dam as part of an adaptive management approach to potential algal proliferation.

3 Groundwater – Surface Water Interaction Modelling

3.1 Overview

In order to estimate the required storage capacity at the proposed dam on the Lee River to meet the future water demands as described earlier, an approach similar to Phase 1 has been adopted. This approach relies on a reservoir storage simulation model driven by historical Wairoa River flows from 1958 to 2008.

Groundwater modelling was carried out by GNS for a selected number of drought years (1982/1983 and 2000/2001) and an average year (2004/2005) to determine the flow augmentation required to preserve a flow of at least 1100 ℓ/s at Appleby Bridge. The flow augmentation patterns were back-analysed to develop appropriate functions to represent the groundwater – surface water interaction component within the reservoir storage simulation model.

Finally, the cycle of annual storage draw downs predicted by the reservoir simulation model was analysed using a standard extreme value analysis to develop a relationship between drought severity (return period) and the maximum drawdown; viz. the required live storage. See Section 6 for the dam storage modelling and drawdown analysis.

In terms of detail, a significant difference compared with Phase 1 was the nature of the flow augmentation functions developed from the more extensive series of groundwater modelling runs in Phase 2.

3.2 Groundwater Modelling

Groundwater modelling was undertaken by GNS. GNS' full report is included in Appendix B to this report. The modelling was undertaken to determine the augmented flows required at Wairoa Gorge (Irvines) to maintain a pre-determined residual flow at Appleby Bridge while meeting unrestricted abstractive demands from the Wairoa River and Waimea aquifers.

This entailed multiple runs of the existing Waimea Plains groundwater model, which has been developed and calibrated by GNS in collaboration with TDC over the past few years. The modelling adopted the future water demands as described in Section 2.

The groundwater model was developed to simulate recharge to and abstractions from the confined and unconfined aquifer systems which underlie the Lower Wairoa/Waimea River plains. Particular care was exercised in the representation of the groundwater-river interaction. Using this model, the sequence of required augmented flows at Wairoa Gorge was determined for a number of scenarios from repeated (trial-and-error) runs of the model. The work was structured into the following four stages.

Stage 1	Update and recalibrate the Waimea Plains groundwater-river interaction model by using new surveyed river cross-section data for the Wai-iti, Wairoa and Waimea Rivers, a new rainfall-recharge model, and a new groundwater abstraction model.
Stage 2	Evaluate the effect of water abstraction on river flows at the Nursery-Appleby Bridge location on the downstream end of the Waimea River and undertake scenario simulations to evaluate the effects of streambed changes in the Wairoa/Waimea River on groundwater resources.

Stage 3 Input future water demand corresponding with the assessed irrigable area in the Waimea Plains and other water uses (see Section 2).

Compute the daily minimum river flow augmentation at Irvine-Wairoa Gorge required to maintain a minimum of surface flow rate of 1100 ℓ/s at Nursery-Appleby Bridge (while meeting unrestricted abstractive future demand in the period 1 July through 30 June of the following year) for the 1982/1983 and 2000/2001 drought years and the 2004/2005 average year with the future water demands. Model additional scenarios for the 2000/2001 year only for an alternative minimum flow of 600 ℓ/s at Nursery-Appleby Bridge, and partial abstractive demands of 75% and 50% of the full future water demand (the latter two scenarios for a minimum flow of 1100 ℓ/s at Nursery-Appleby Bridge).

Following dam storage modelling, undertake a series of forward simulations to confirm that the proposed augmented flow regime at Irvine-Wairoa Gorge (based on empirical flow functions developed by Tonkin & Taylor Ltd – see Section 3.3 later) can meet full downstream requirements while maintaining a minimum residual flow of 1100 ℓ/s at Nursery-Appleby Bridge.

Stage 4 Determine the "zone of effect" by analysing the predicted groundwater level changes in response to changes in river flow. Doing so allows mapping of the zone of river recharge to the aquifer, thus confirming the areas where water supply abstraction can best be made.

In Stage 3, a distributed model of the future water demand was developed, covering the 5,906 ha of expected irrigable area in the Waimea Plains, urban and industrial water demand (for a 100 year horizon), 250 ha of irrigable area on Rabbit Island, plus irrigation demand in Brightwater/Wakefield and Redwood Valley. A pasture crop was assumed over a range of soil types having 38, 78, or 130 mm soil moisture holding capacity. The future water demand model was based on daily climate data for the period of 1 July through 30 June of the following year for the 1982/1983 and 2000/2001 drought years (2000/2001 being somewhat more severe) and the 2004/2005 average year. The future daily peak water demand was estimated to be 3,351 ℓ/s, including all direct surface water abstraction (viz. the Waimea East Irrigation Scheme (WEIS) and 255 ℓ/s for Future Regional Need), and covered a total of 5,923 ha of irrigable area in the model domain (the actual area modelled marginally exceeded the expected area involved due to the irregular shape of the area and the fit of rectangular model cells).

Groundwater modelling showed that, to meet the future water demand and maintain a minimum flow of 1100 ℓ/s at Nursery-Appleby Bridge, the average augmented river flow required at Wairoa Gorge over the driest part of the 2000/2001 drought (1 February through 31 March 2001) would be 2,822 ℓ/s (upstream of the WEIS take). Similarly, modelling of the scenario for the 1982/1983 drought (with the driest part of it also being the 1 February through 31 March time frame) showed that an average augmented flow at the Wairoa Gorge of 2,744 ℓ/s would have been required to meet the target minimum flow of 1,100 ℓ/s at Nursery-Appleby Bridge. With abstractions at the future demand level, and without any augmentation, natural river flow is predicted to be able to maintain a river flow above 1,100 ℓ/s at Nursery-Appleby Bridge at all times except for 3 days in the 2004/2005 irrigation season.

To maintain a minimum flow of at least 600 ℓ/s at Nursery-Appleby Bridge, the model predicted that the minimum augmented river flow at Irvine-Wairoa Gorge would need to be 2,474 ℓ/s, compared with 2,822 ℓ/s to maintain a minimum flow of 1,100 ℓ/sec at Nursery -Appleby Bridge, for the 2000/2001 drought year.

The model predicted that when 50% of the full unrestricted future water demand is applied, river flow at Nursery-Appleby Bridge would increase by about 622 ℓ/s compared with the average flow with full future water demand in the period 1 February to 31 March 2001 period (driest part of the 2000/2001 drought year).

Forward simulation was undertaken to assess whether the augmented river flow regime at Irvine-Wairoa Gorge developed by Tonkin & Taylor Ltd based on empirical relationships between natural river flow and abstractive demand could maintain a minimum 1,100 ℓ/s flow at Nursery-Appleby Bridge for the 1982/1983 and 2000/2001 drought years. Forward simulation results indicated that the proposed augmented river flows would maintain river flow above 1,100 ℓ/s at Nursery-Appleby Bridge on most days, but not all days, in the critical periods of late March 1983 and April 2001.

For April 2001, the flow at Nursery-Appleby Bridge would be below 1100 ℓ/s for 5 days dropping to a minimum of 952 ℓ/s, while in March 1983, the flow would be below 1100 ℓ/s for 6 days dropping to a minimum of 873 ℓ/s. Subsequently, some further adjustments to the proposed flow augmentation regime were made to reduce these occurrences and compensate for the shortfalls (40,000 m^3 for the 1982/1983 year and 84,000 m^3 for the 2000/2001 year).

It is noted that when the flow augmentation scheme is put into operation, it will be possible (and also preferable) to make use of real-time flow monitoring data at Nursery-Appleby Bridge to refine dam flow releases so as to achieve a flow above 1,100 ℓ/s at Nursery-Appleby Bridge at all times.

3.3 Coupling of Groundwater Modelling With Dam Storage Simulation

Results from groundwater modelling in the first part of Stage 3 included predicted daily time-series flows for Appleby Bridge and the augmented flow required at Wairoa Gorge for the three nominated water years 1982/1983, 2000/2001 and 2004/2005. The pattern (timing and quantity) of surface flow loss in the river reach between Wairoa Gorge and Appleby Bridge predicted by the groundwater model was analysed to determine its dependence on a range of variables, including:

- natural river flow at Wairoa Gorge
- rate of groundwater abstraction
- combinations of the above with various lags or lead times and aggregations
- targeted minimum flow at Appleby Bridge (600 or 1100 ℓ/s)
- time-of-year (within the irrigation season).

From this assessment relationships were developed between the flow augmentation required and the natural flow at Wairoa Gorge (Irvines) for varying levels of groundwater demand. These relationships allow the groundwater system behaviour to be coupled (although imprecisely – see below) with the catchment surface water system and the simulated operation of the storage dam on the Lee River. Effectively, these relationships specify the required flow release on a day-to-day basis from the proposed storage dam in the upper catchment to supplement tributary inflows between the dam site and Wairoa Gorge.

The "black box" approach adopted here by necessity has its limitations as it simplifies the physical processes involved in the groundwater-surface water interaction and aquifer storage dynamics. The empirical flow relationships have also been based specifically on the 1982/1983 and 2000/2001 hydrological/hydro-geological/water demand conditions, and may thus be less representative for other years in the long-term storage simulation.

Limitations inherent in the adopted approach have been addressed through the series of forward simulations in the Stage 3 groundwater modelling, which was to confirm and if necessary (as was the case) to progressively fine-tune the flow augmentation functions.

Figure 3.1 shows a plot of the final version of the fitted flow augmentation function, which is dependent on groundwater demand, against flow augmentation data obtained from groundwater modelling. Unlike Phase 1, it was found that by including the groundwater take in the flow augmentation function the correlation was significantly improved. This improvement has resulted mainly from the larger pool of groundwater modelling results available for function fitting. The single equation (independent of demand) from Phase 1 is also shown for comparison.

In addition, a supplementary augmentation function was incorporated which operated on the sequence of groundwater demands for the preceding 4 days. A 4 day time frame was selected after trials with different time frames and variations/combinations. The larger of the forecast augmentation from this 4-day function and the former function (based on 1-day demand) determined the release requirement for that day. Figure 3.2 shows this supplementary (4-day) flow augmentation function together with the primary (1-day) function.

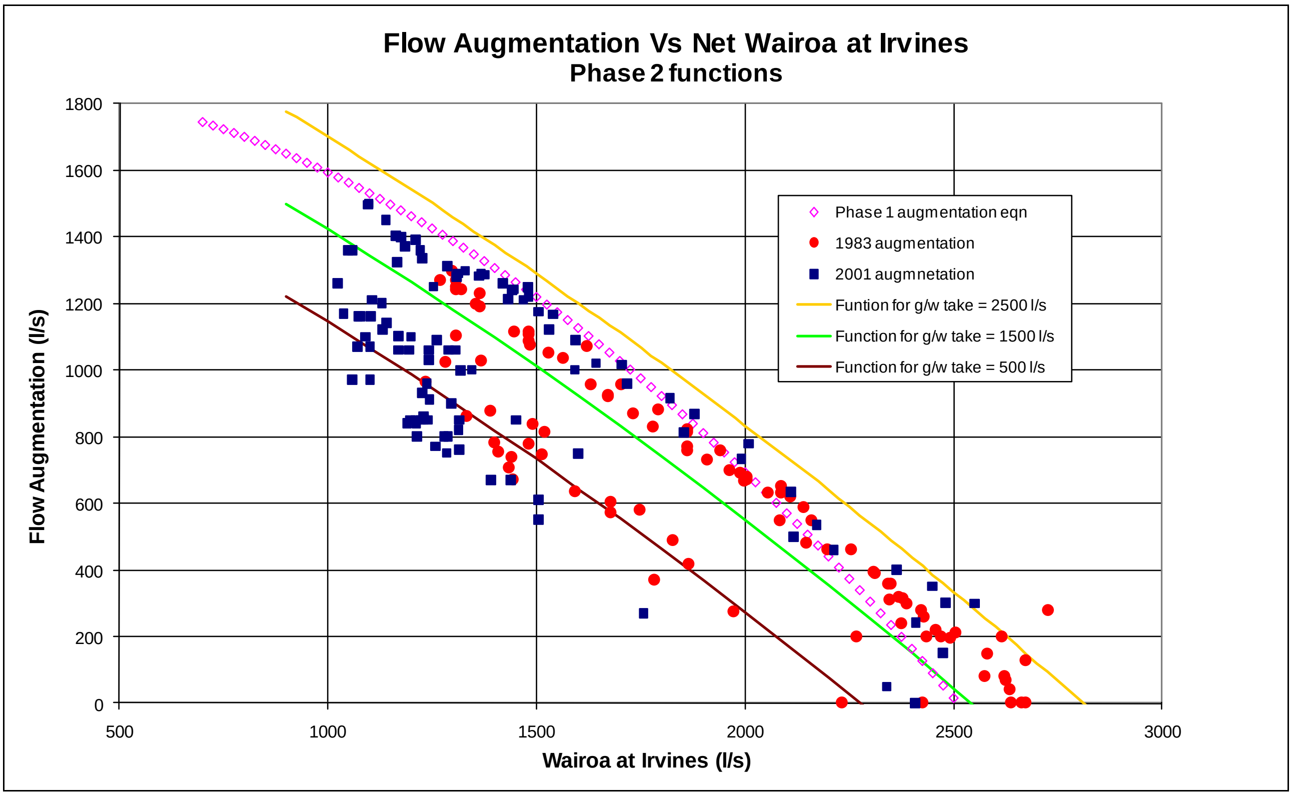


Figure 3.1 Flow augmentation functions for maintaining a minimum Appleby Bridge flow of 1100 ℓ/s - based on Wairoa at Irvines flows less all surface water abstractions

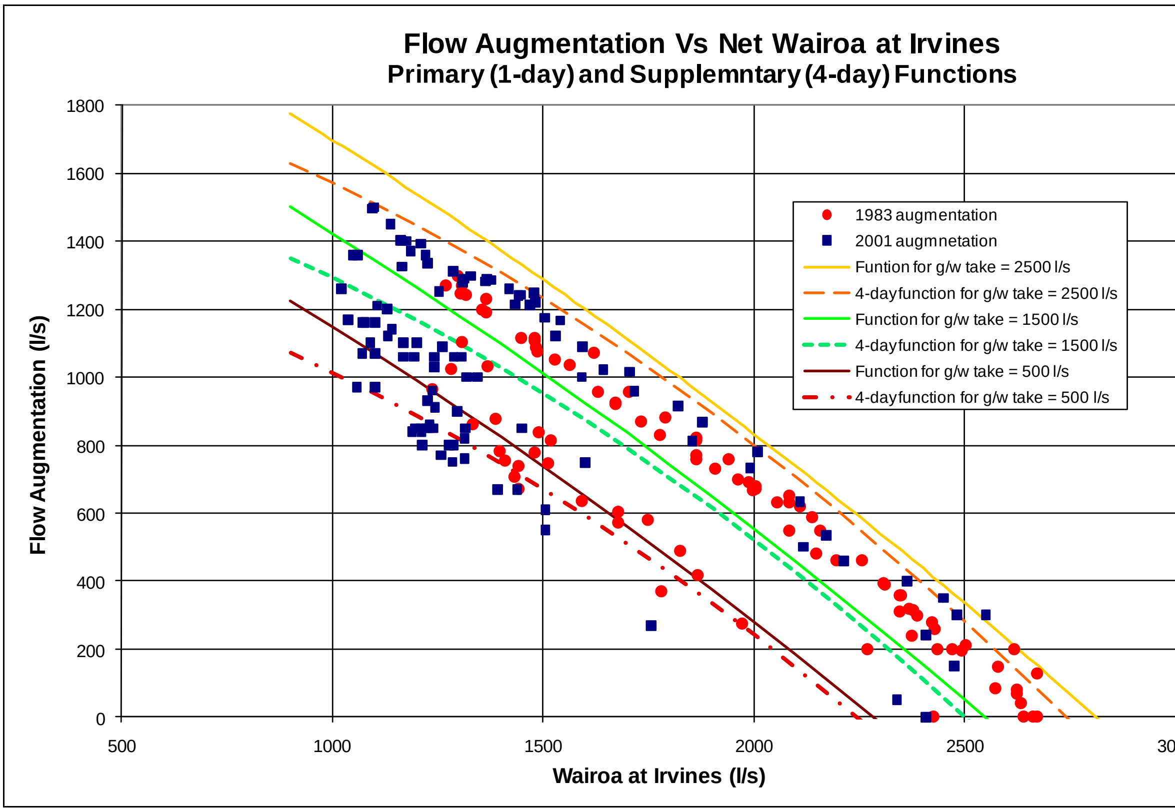


Figure 3.2 *Phase 2 flow augmentation functions: primary function based on 1-day demand and supplementary function based on 4-day demand*

4 Catchment Hydrology

4.1 Introduction

Hydrological aspects considered in this section of the report include:

- catchment water balance and assessment of mean flow at the dam site
- low flow frequency assessment at dam site.

Flood frequency analysis and design flood estimation for the proposed dam is considered in Section 5.

Assessments of catchment hydrology from Phase 1 are updated in this section. At the commencement of Phase 2, TDC installed an automatic flow recording station on the Lee River upstream of the dam site. There were no continuous flow records available for the Lee River previously. Data gathered from this station has been utilised in the current Phase 2 hydrological assessments.

The other notable difference compared with Phase 1 is that the preferred dam site has been shifted upstream some 1.4 km (from Chainage 11010 to Chainage 12430), with an attendant reduction in catchment area of about 8% (77.5 km^2 versus 83.8 km^2 originally). Whereas the previous dam site was located downstream of Anslow Creek, the currently preferred site is upstream of Anslow Creek.

4.2 Update of catchment water balance and mean flows

TDC installed a recording station on the Lee River upstream of the Waterfall Creek confluence (Site 57536 Lee at Waterfall Creek)[a] and commenced monitoring of flows on 20 April 2007. The catchment area above the recording station, which is located at about Chainage 13500, is approximately 65.3 km^2, while the catchment area above the dam site (Chainage 12430) is about 77.5 km^2. One full year of flow data from this station has been used to update the catchment water balance and mean flow estimates viz. April 2007 to April 2008.

The nearest and most representative flow recorder with continuous long term flow records is the Wairoa at Irvines[b] (Site 57521) which commands a catchment area of 463 km^2 which includes the Lee River catchment.

Figure 4.1 plots the contemporaneous flows from the Lee at Waterfall Creek and the long term Wairoa River flow recorder at Irvines. Note that the Wairoa at Irvines flow has been factored by 0.2 to enable a visual check on the coherence of the two datasets. As expected, this plot clearly shows that flows in the upper Lee behave in the same way as flows in the Wairoa River, and that both flow regimes are in complete synchronisation. Therefore, it is acceptable to generate a synthetic record of the Lee River flows (back to November 1957) by simply scaling the Wairoa River historical flows.

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[a] This recording station is also referred to as Lee River above Waterfall Creek elsewhere in this report.

[b] The Wairoa at Irvines flow recording station is located just upstream of the Wairoa Gorge and is sometimes referred to as the Wairoa Gorge recorder, although a discontinued recording station called Wairoa at Gorge (Site 57502, located about 900 m downstream of Wairoa at Irvines) operated between November 1957 and December 1992 before being replaced by the current Wairoa at Irvines recorder. The records from both stations have been combined for the current study and the combined record is referred to as Wairoa at Gorge/Irvines or simply as Wairoa at Irvines.

It is interesting to note that over this period of flow monitoring (April 2007 to April 2008), the mean flow recorded at Irvines was only 11.0 m^3/s, which is about two thirds of the long term mean flow (16.2 m^3/s). In fact, since records began in 1957, this was the 3rd driest 12 month May-to-April period. (The driest May-to-April occurred in 2005/2006). The recorded mean flow in the Lee River above Waterfall Creek (the flow recording site) over this period was 2.31 m^3/s, which equates to an annual volume of 73 million m^3.

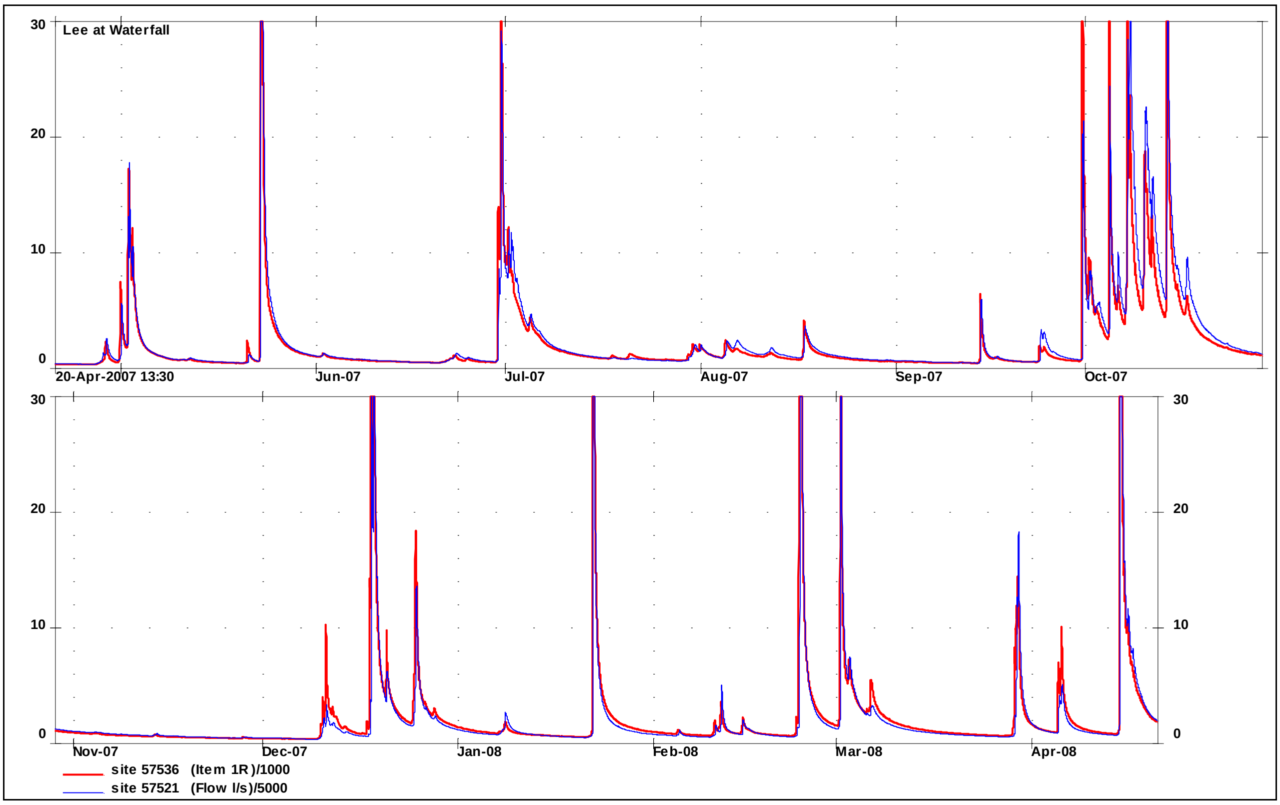


Figure 4.1 Lee at Waterfall Creek flows(red) plotted against Wairoa at Irvines flows factored downwards by 0.2 (blue) from April 2007 to April 2008. All flows in m^3/s.

TDC has carried out a series of spot flow gaugings in the Waterfall and Anslow Creeks to allow correlations to be developed with the contemporaneous flow recorded for the Lee River (Lee at Waterfall Creek). Based on these correlations, the estimated mean flow at the original dam site (Chainage 11010) is 2.68 m^3/s over the period selected for analysis (April 2007 to April 2008). This represents 24.4% of the flow at Irvines. At the currently preferred dam site (Chainage 12430), the estimated mean flow is 2.55 m^3/s over the same period.

However, catchment water balance calculations from Phase 1 suggested that mean flow at the original dam site should be 21.6% of the mean flow at Irvines (compared with 24.4% estimated currently). The end result is that the currently inferred long term mean flow at the original dam site is some 6% higher than anticipated based on Phase 1 results. It is likely the underestimation at the dam site (and upper reaches of the Lee) during Phase 1 was a result of the underestimation in the orographic rainfall at the headwaters of the catchment. Phase 1 studies relied on the New Zealand Meteorological Service 1:500,000 scale rainfall normal map for 1941 – 1970 for this information.

For Phase 2, adjustments were made to the mean annual rainfall distribution and the rainfall gradients across the catchment so that flow estimates from the catchment water balance analysis matched the recorded mean flow. Based on this approach, the long-term mean flow at the original dam site is now estimated to be 3.80 m^3/s. At the currently preferred dam site the long- term mean flow is estimated to be 3.60 m^3/s. At the flow recording site on the Lee, the estimated long-term mean flow is 3.20 m^3/s, or about 40% greater than recorded.

Table 4.1 summarises the mean flow estimates. For subsequent analyses, a synthetic flow record at the dam site was generated by factoring the full Wairoa flow record (1958 to 2008) by the ratio of the estimated long-term at-site mean flow to the long-term Wairoa mean flow shown in this table. The applicable flow ratio for the original dam site (Chainage 11010) was 0.235 while the ratio for the currently preferred dam site (Chainage 12430) was 0.222.

Table 4.1 Lee River mean flow estimates

Location	Catchment area (km^2)	Observed or inferred mean flow over monitoring period ($Q_{07/08}$) (m^3/s)	Long-term mean flow (Q_{LTM}) (m^3/s)	Ratio $Q_{07/08}$ / Q_{LTM}
[1] Wairoa River at Gorge/Irvines	463	11.0	16.2	67.8%
[2] Lee River below Anslow Creek (Chainage 11010) (original dam site)	83.8	2.68	3.80	70.5%
[3] Lee River above Anslow Creek (Chainage 12430) (current dam site)	77.5	2.55	3.60	70.8%
[4] Lee River above Waterfall Creek (flow recorder)	65.3	2.31	3.20	72.2%

Figure 4.2 provides a comparison of the flow duration curves from the Lee at Waterfall Creek with the Wairoa at Gorge/Irvines. The latter (Wairoa at Gorge/Irvines) has been simply scaled based on the proportions determined above to represent the flow at the dam site at Chainage 12430. This is shown by the blue and green curves which correspond with the estimated flow at the dam site for the 12 month period April 2007 to April 2008 and the longer term situation 1958 to 2008 respectively. The red curve shows the actual flow recorded in the Lee River above Waterfall Creek over April 2007 to April 2008, which is known to be a very dry period compared with longer term average.

When additional data from the Lee at Waterfall Creek recorder became available in February 2009 (record from 20 April 2007 to 26 February 2009), a further mean flow check was performed to confirm that the scaling factors remained appropriate.

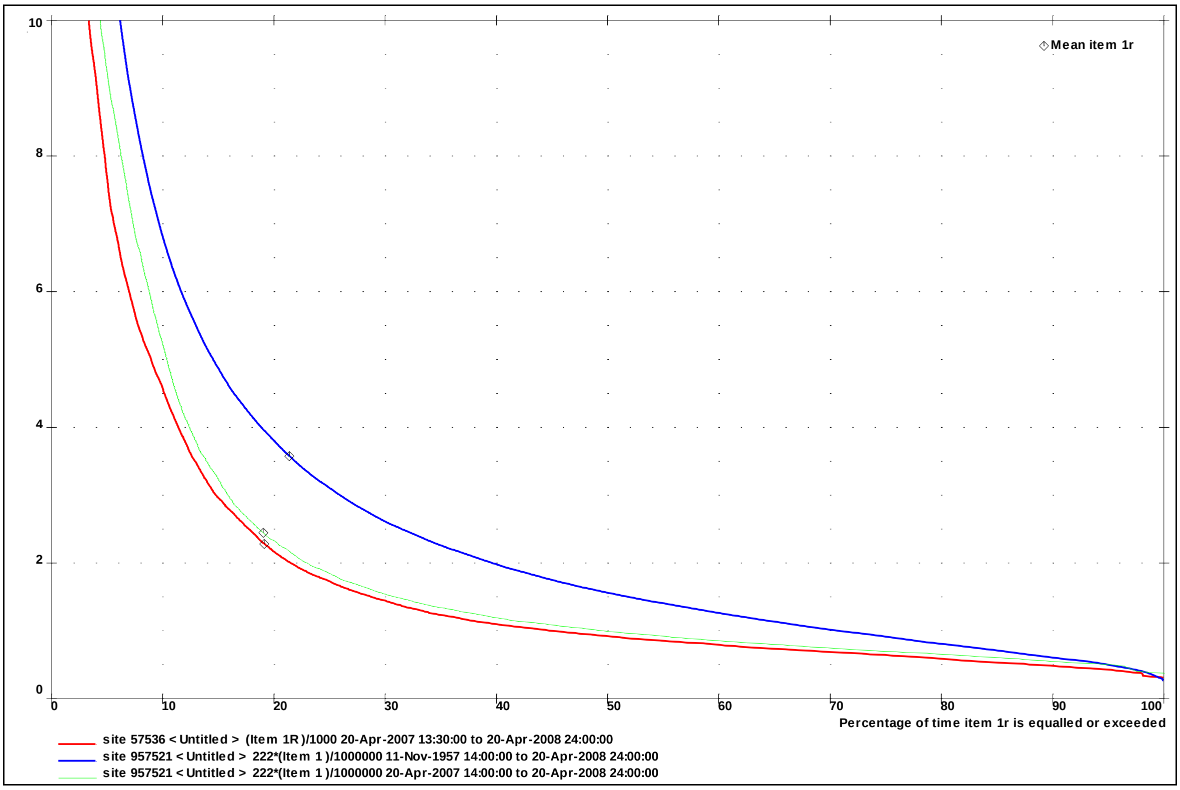


Figure 4.2 Flow duration curves representing Lee River flows: Red – Lee at Waterfall Creek April 2007 to April 2008; Green – Lee at dam site April 2007 to April 2008 scaled from Wairoa at Irvines; Blue – Lee at dam site 1958 to 2008 scaled from Wairoa at Irvines. All flows in m^3/s.

4.3 Low Flow Analysis

By itself, the flow record available for the Lee River (Lee at Waterfall Creek) is too short for an assessment of low flow frequency. In the wider catchment, long-term flow records for Wairoa Gorge dating back to November 1957 provide the most appropriate basis for estimating low flow parameters at the dam site.

On a unit area basis, low flows are related to land use, average rainfall and geology.

Figure 4.3 below shows the low flow frequency distribution fitted to the annual minimum flows (7-day averaged flows) recorded at Wairoa Gorge between 1958 and 2008. Of the three trial distributions (General Extreme Value (GEV), Normal and Lognormal), the GEV distribution was selected as the best-fit curve. Table 4.2 summarises the low flow frequency analysis results for the Wairoa Gorge.

It should be noted that the low flow episodes in the Wairoa River are affected by Nelson City Council's water supply take from the Roding River. The Roding River is a tributary which enters the lower Lee River about 3.5 km upstream of Wairoa Gorge. The catchment area above the water supply intake (Roding Weir) is approximately 39 km^2, or just over 8% of the full catchment at Wairoa Gorge. However, minimum residual flows of 50 ℓ/s and 100 ℓ/s were required to be maintained below the water supply intake from 2003 and 2008 respectively. Prior to 2003, no residual flow below the intake weir was required.

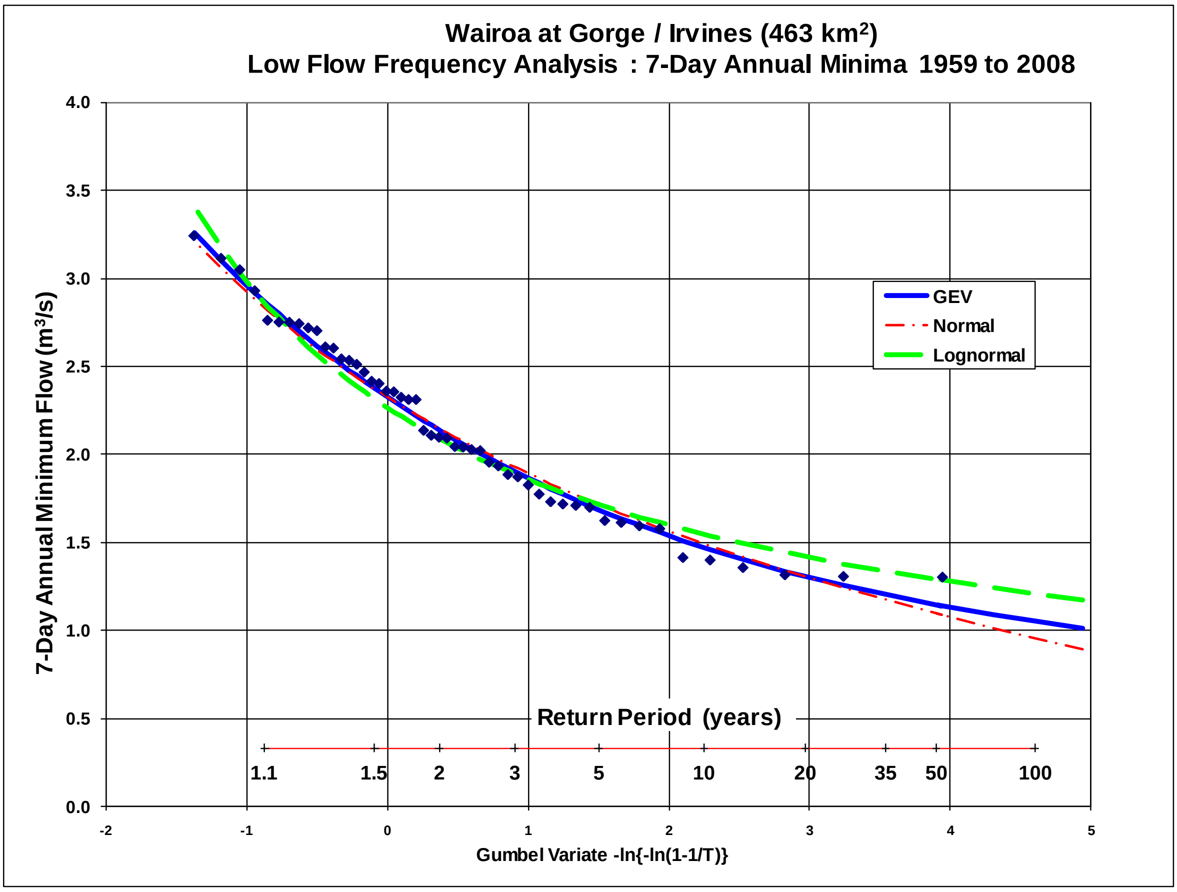


Figure 4.3 Wairoa at Irvines/Gorge Low Flow Frequency Analysis

Table 4.2 Wairoa River and environs low flow frequency analysis results (based on 7-day averaged flows)

Flow Gauging Site	Data Period	Catch-ment (km²)	[1] Mean Annual Rainfall (mm p.a.)	Mean Annual Low Flow		5-Year Low Flow		10-Year Low Flow	
				(l/s)	(l/s/km²)	(l/s)	(l/s/km²)	(l/s)	(l/s/km²)
Wairoa River at Irvines /Gorge	1958 - 2008	463	[2] 1609	2150	4.64	1690	3.65	1470	3.17

Notes:
1. Mean annual rainfall estimated from the 1:500,000 scale map of 1941- 1970 rainfall normals. Values have not been adjusted to match the period of flow data.
2. The mean annual rainfall for Wairoa at Gorge is likely to be an underestimate.

In transposing low flow parameters from Wairoa Gorge to the dam site, low flows were assumed to be primarily proportional with catchment area. An adjustment was made on the basis of the correlation between the short-term Lee at Waterfall Creek flow record and the Wairoa Gorge flow record. This adjustment reflects the higher catchment-averaged rainfall in the Upper Lee compared with the overall catchment above Wairoa Gorge, and results in a higher low flow on a unit area basis for the dam site.

In Phase 1, a 7-day MALF of 0.47 m³/s was estimated for the original dam site (Chainage 11010). The current estimate of the 7-day MALF is 0.51 m³/s for the same site.

For the currently preferred dam site (Chainage 12430), the 7-day MALF is estimated to be 0.49 m^3/s. The 7-day 5 year and 10 year low flows are estimated to be 0.38 m^3/s and 0.33 m^3/s respectively.

5 Flood Hydrology

5.1 Introduction

For Phase 2, flood hydrology has been focussed on design flood estimation at the proposed dam site (at Chainage 12430) on the Lee River. Three methods have been used to compute design floods for a range of return periods, comprising the following:

- Method 1: flood frequency analysis based on long-term flow records at Wairoa Gorge
- Method 2: flood hydrograph derived from repeated frequency analysis of flood volumes for a range of durations
- Method 3: flood hydrograph simulation based on a design rainstorm using a rainfall-runoff model calibrated to recorded storm rainfall and flood events

The first and third methods are standard accepted approaches to design flood estimation. The first method provides estimates of the peak flow only for a range of return periods, while the third method produces a full hydrograph which can be used for reservoir flood routing and thus design of a spillway at a dam.

The second method is a new flow-based method which produces a full design hydrograph without requiring storm rainfall data. For a particular return period event, the synthetic hydrograph essentially comprises nested pairs of volume-duration data covering the full flood duration. While this method arguably produces the most accurate and consistent estimates of both flood volume and peak flow in a single hydrograph, it is nevertheless an innovative method. Thus, the results have been checked and confirmed against the standard accepted approaches described earlier.

The first and second methods rely heavily on the long-term flow record at Wairoa Gorge/Irvines which spans from 1958 to 2008. Correlations between the short-term record for the Lee River (Lee at Waterfall Creek), which commenced recording in April 2007, and this record have been used to translate the results to the dam site.

The Probable Maximum Flood (PMF) has also been computed based on an assessment of the Probable Maximum Precipitation (PMP) using the calibrated rainfall-runoff model.

5.2 Method 1: Flood Frequency Analysis

Figure 5.1 below shows the flood frequency distribution fitted to the annual maximum flows recorded at Wairoa Gorge/Irvines. The one standard error envelope is also shown (68.3% confidence interval). The estimated 100 year return flood peak is 1560 m^3/s (± 120 m^3/s).

Peak flow estimates for the proposed dam site on the Lee River have been computed from flood parameters for the Wairoa Gorge/Irvines site using the transposition equation recommended by McKerchar and Pearson (Flood Frequency in New Zealand, Publication No. 20 Hydrology Centre, DSIR, 1989), which assumes flood peaks are related by catchment area ratio raised to the power of 0.8, viz.:

$$Q_{\text{Lee dam site}} = Q_{\text{Wairoa Gorge}} \times (\text{Catchment Area}_{\text{Lee dam site}} / \text{Catchment Area}_{\text{Wairoa Gorge}})^{0.8}$$

When the appropriate catchment areas are substituted (463 km^2 for Wairoa and 77.5 km^2 for the dam site), this equation simplifies to:

$$Q_{\text{Lee dam site}} = 0.239\, Q_{\text{Wairoa Gorge}}$$

Interestingly, this flood flow transformation factor above of 0.239 is only marginally greater than the ratio of the assessed long-term mean flow at the dam site (3.60 m³/s) to the long-term mean at Wairoa Gorge (16.2 m³/s) of 0.222 (see Section 4.2).

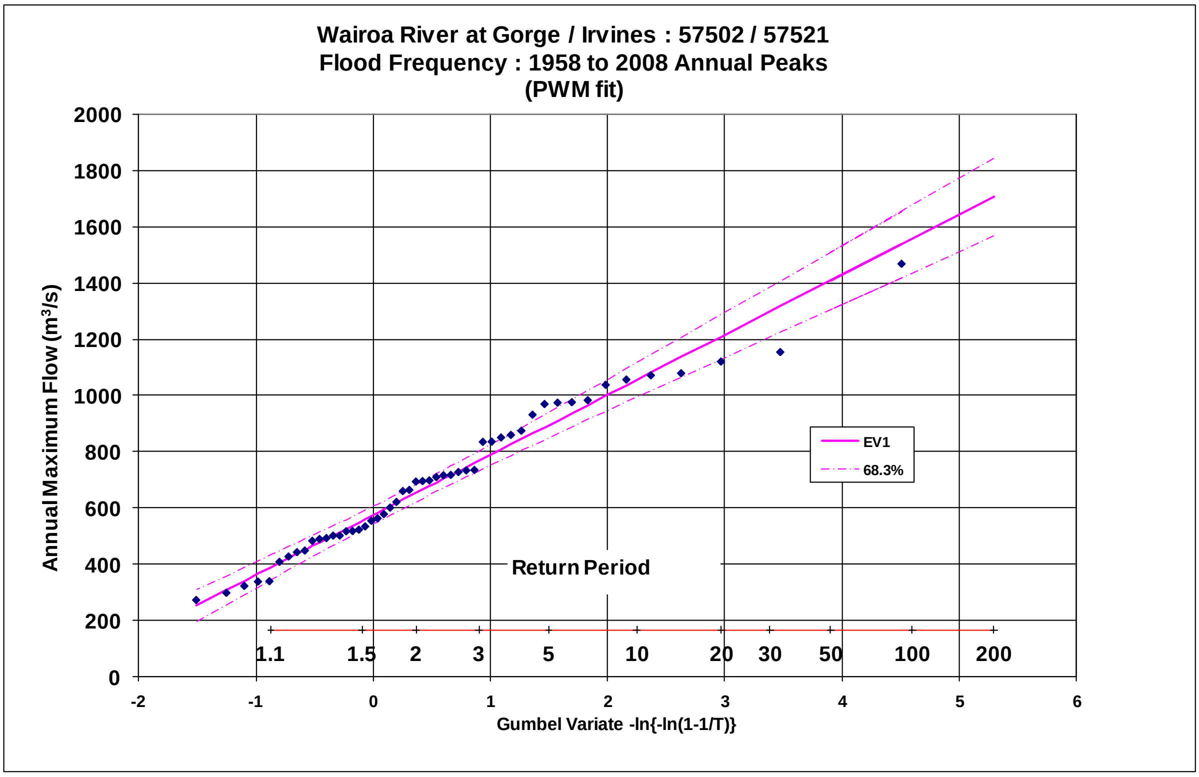


Figure 5.1 Wairoa at Gorge/Irvines flood frequency analysis – peak instantaneous flow

Table 5.1 summarises estimated flood peaks for a range of return periods for the Wairoa River at Gorge/Irvines and at the proposed dam site (rounded).

Table 5.1 Flood peak estimates for the Wairoa Gorge/Irvines and Lee dam site (instantaneous peak flows)

Parameter	Wairoa at Gorge/Irvines	Lee dam site at Chainage 12430
Catchment Area (km²)	463	77.5
Mean Flow (m³/s)	16.2	~ 3.60
Mean Annual Flood (m³/s)	698 ± 32	~ 168
10 Year Return Period Flood (m³/s)	1055 ± 60	~ 255
50 Year Return Period Flood (m³/s)	1410 ± 100	~ 340
100 Year Return Period Flood (m³/s)	1560 ±120	~ 375
200 Year Return Period Flood (m³/s)	1710 ±140	~ 410
1000 Year Return Period Flood (m³/s)	~ 2050 ±180	~ 490
10,000 Year Return Period Flood (m³/s)	~ 2550 ±250	~ 610

5.3 Method 2: Design Flood Hydrographs from Flood Volume Frequency Analysis

5.3.1 Overview

This method generates a full design hydrograph while eliminating the rainfall to flow transformation process and all its inherent assumptions and uncertainties. It is analogous to the "Chicago Method" used to construct a design rain storm from intensity-frequency-duration data. The synthetic design flood hydrograph is derived from volume-duration-frequency data determined from repeated flood frequency analysis applied to a range of averaging intervals. For a particular return period event, it is essentially comprised of nested pairs of volume-duration data covering the full flood duration. While this method requires a long and good quality flow record, it provides accurate and consistent estimates of both volume and peak flow, and does not require consideration of multiple storm durations to determine the critical event in the case of flood routing simulations.

Given appropriate flow data, this approach is potentially the most reliable and definitive of any design flood estimation method available. The long-term flow record at Wairoa Gorge/Irvines is considered appropriate for this method to be applicable.

5.3.2 Flood volume frequency analysis

The maximum accumulated flood volume for a particular duration can be found using a moving average calculation applied to the full instantaneous flow record, which, in New Zealand practice, typically has a "native" time resolution of 15 minutes. The flood volume is simply the average flow multiplied by the duration. In order to determine the volume frequency distribution for that particular duration for that record, a frequency analysis is carried out on the series of annual maximum flood volumes for that duration in the same manner as for peak flows (e.g. see Figure 5.1).

Such volume frequency analyses have been completed for durations of 1 hour, 3 hours, 6 hours, 12 hours, 18 hours, 24 hours , 36 hours, 48 hours, 72 hours, 96 hours and 120 hours on the Wairoa Gorge /Irvines record (1958 to 2008). Figures 5.2, 5.3 and 5.4 provide samples of the fitted volume frequency distributions for durations of 3 hours, 12 hours and 48 hours respectively.

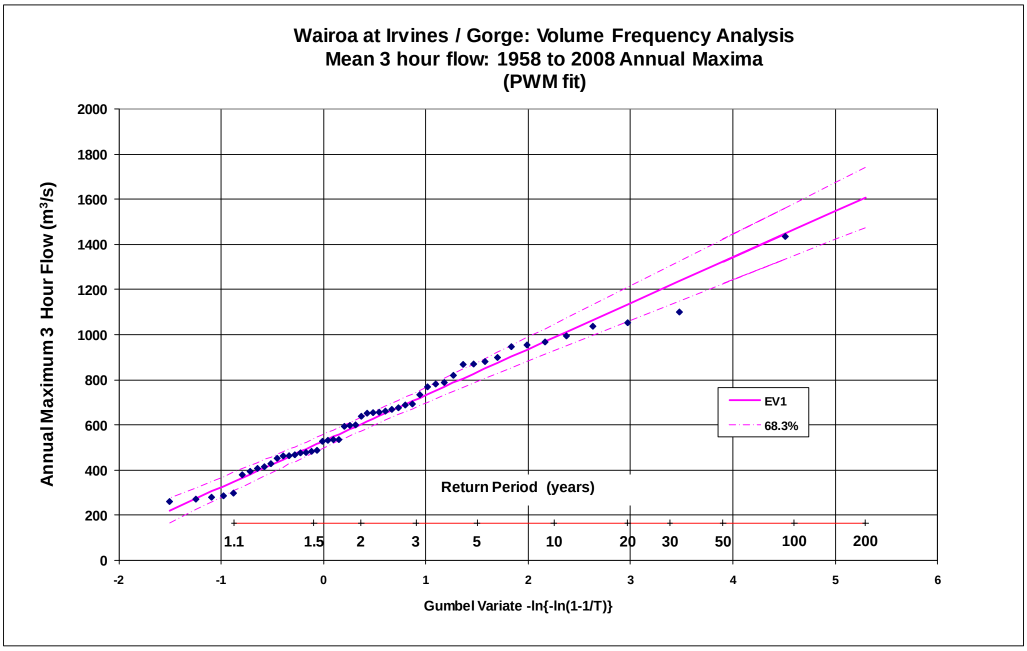


Figure 5.2 Wairoa at Gorge/Irvines volume frequency analysis for 3 hour duration

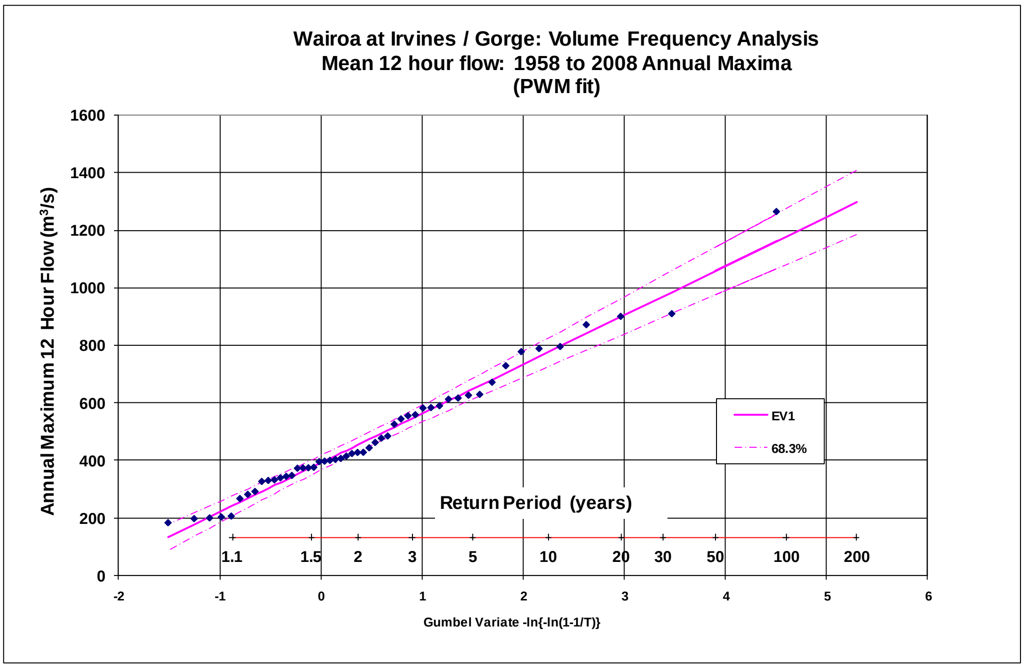


Figure 5.3 Wairoa at Gorge/Irvines volume frequency analysis for 12 hour duration

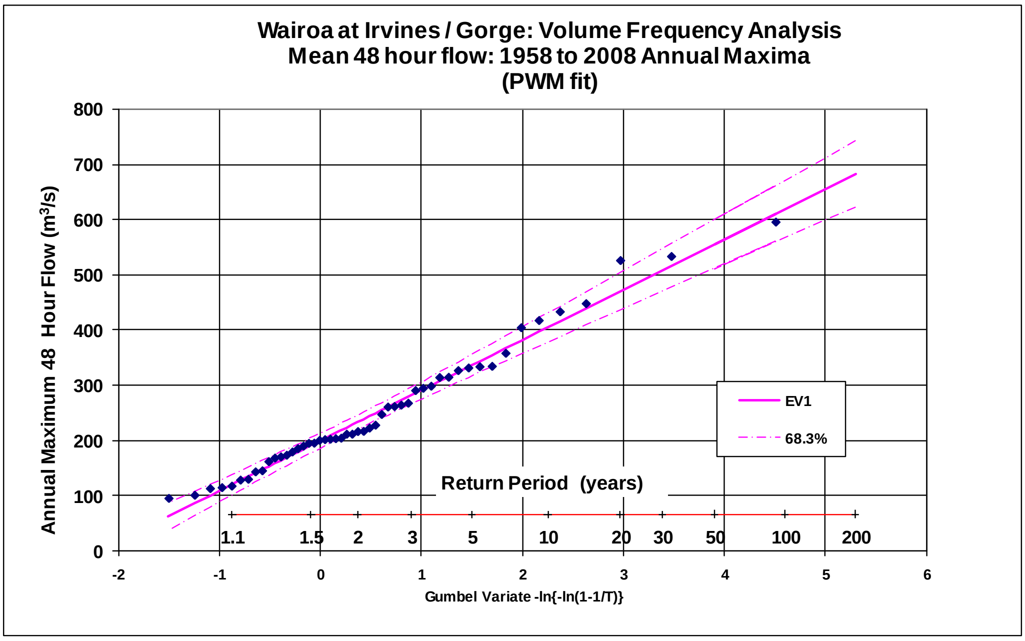


Figure 5.4 Wairoa at Gorge/Irvines volume frequency analysis for 48 hour duration

5.3.3 Volume–duration–frequency relationships

From the array of flood volume frequency relationships described above, a complementary relationship between flood volume and duration for a particular return period each can be derived. See for example Figure 5.5 for the 100 year return period case, noting that flood volume (in m³) is equal to the average flood flow (in m³/s) multiplied by duration (in seconds). An approximating curve has been fitted to the data points, in this case a quartic equation.

Figure 5.6 shows the family of similar curves of volume versus duration for return periods of 2.3, 5, 10, 20, 50, 100, 200, 1000 and 10000 years.

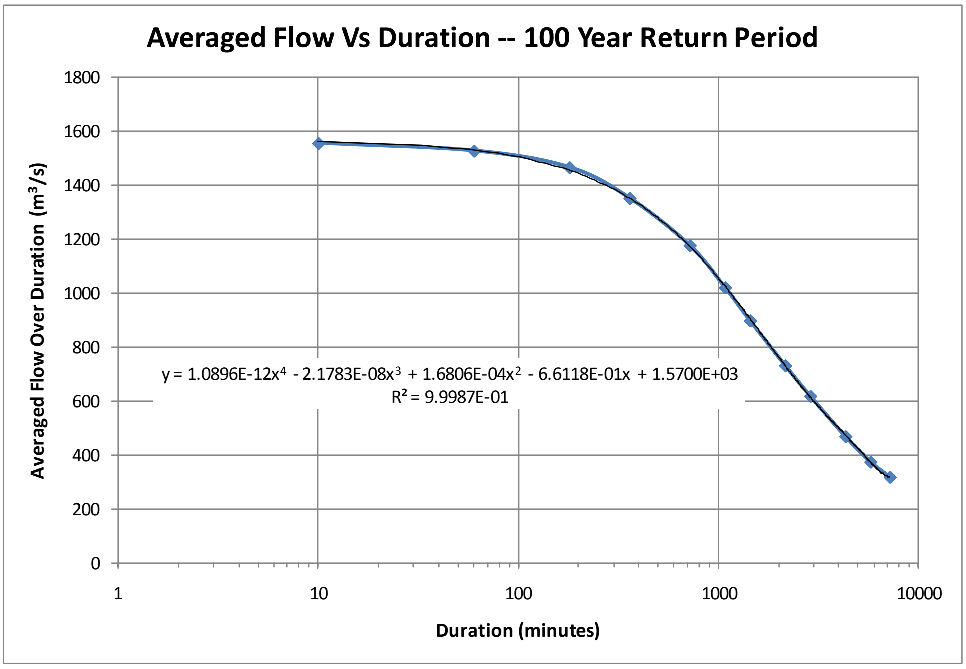


Figure 5.5 Wairoa at Gorge/Irvines flood volume versus duration relationship for 100 year return period

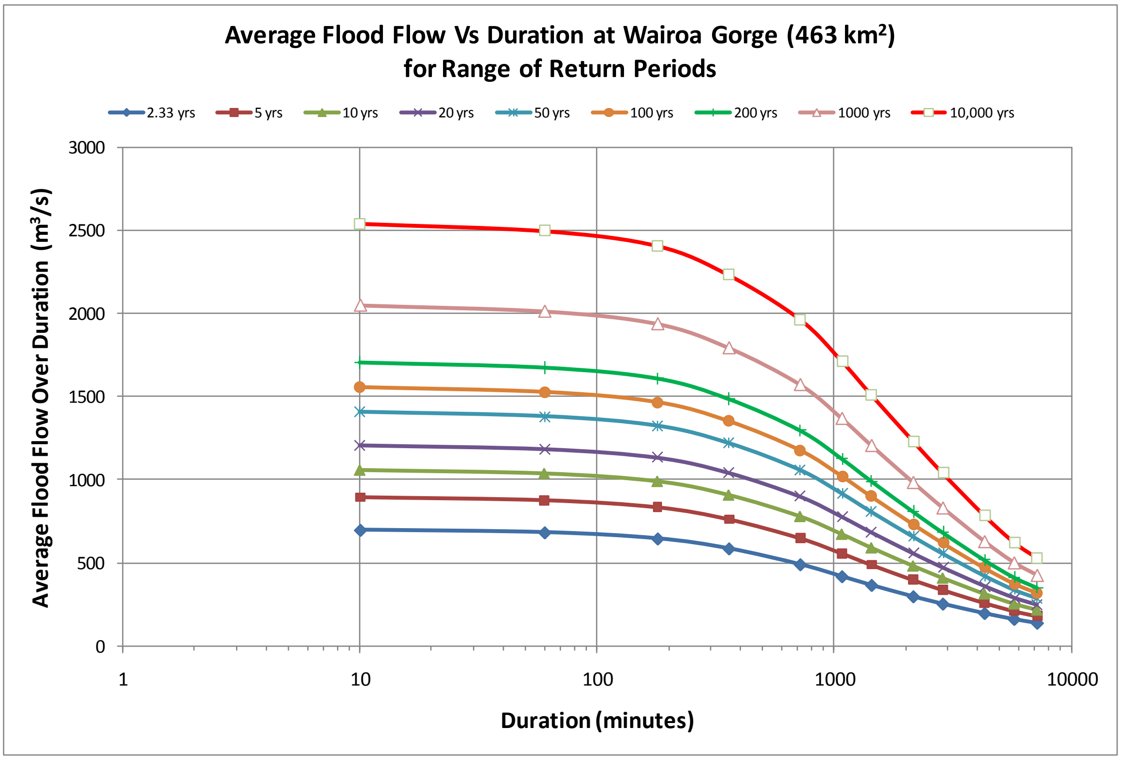


Figure 5.6 Wairoa at Gorge/Irvines flood volume versus duration relationship for a range of return periods

5.3.4 Design hydrographs from volume duration data

For each return period, a design hydrograph has been constructed from nested pairs of volume-duration data determined from the fitted curves shown in Figure 5.6. The approach used ensures that, within the hydrograph, the maximum flood volume over any particular duration matches the volume from the volume–duration curve for that return period.

In each case, the hydrograph has been truncated at 48 hours as the marginal flow for longer duration events was minor (at less than 1.5 mm/hr runoff in the 100 year return period event). Two other parameters are required to be selected to complete the hydrograph construction, namely the flow at the start of the flood and the time that the peak flow occurs. The flow at the start of the flood is invariably much lower than the flow at the tail end of the flood. While this was arbitrarily set equal to the long-term mean flow, the precise value (within reasonable bounds) has minimal effect on the overall shape of the hydrograph.

An assessment of historical flood hydrographs in the Wairoa flow record broadly indicates that the peak flow typically occurs at between 25% and 50% of the hydrograph duration. In several large flood events, the peak flow occurred around 30% to 40% into the event hydrograph. Therefore, a time-to-peak of 35% was selected for the design hydrographs.

Figure 5.7 shows the resulting design flood hydrographs for the Wairoa River based on the record at the Gorge/Irvines.

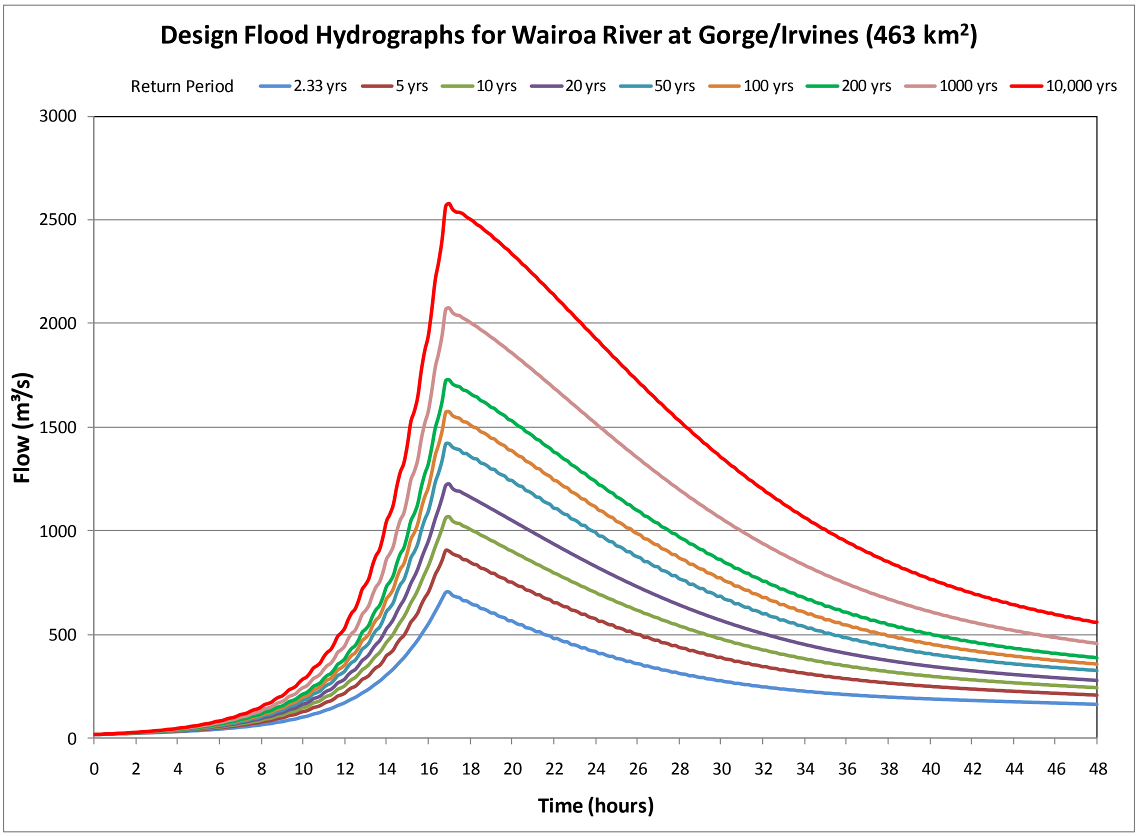


Figure 5.7 Wairoa at Gorge/Irvines design flood hydrographs for a range of return periods

5.3.5 Translation of Wairoa design flood hydrographs to the Lee River dam site

In translating the design flood hydrographs from the Wairoa River to the dam site on the Lee River, careful consideration was given to preserve the following hydrograph characteristics in relative terms:

- flood peak ratio
- flood volume ratio
- time base ratio (i.e. hydrograph base length)

As noted in Section 4.1 earlier, a flow recording station (Lee at Waterfall Creek, catchment area 65.3 km^2) was recently installed upstream of the proposed dam site, and continuous flow data has been available from 20 April 2007. For assessment of flood hydrology, data from this recorder from 20 April 2007 to 26 February 2009 has been acquired. Within this period, there have been 6 high flow/flood events which have been recorded concurrently at both the Lee and Wairoa (at Irvines) flow stations. Of these, the largest event, and by a considerable margin, is the flood which occurred on 24th November 2008. This flood peaked at 236 m^3/s in the Lee and 1078 m^3/s in the Wairoa, and is estimated to have been about a 14 year return period flood. See Section 5.4.3 later for details of this event, which was also used to calibrate the catchment rainfall-runoff model.

Assessment of all these flood events (while excluding two outliers – minor events in the Wairoa) indicated a remarkably consistent flood volume ratio between the Lee and the Wairoa, ranging from 0.203 to 0.211 (average 0.207). The flood peak ratio demonstrated a larger spread (as expected) but was still reasonably consistent between 0.21 and 28 when the same two outliers were omitted. Note that the McKerchar and Pearson approach

(Flood Frequency in New Zealand, Publication No. 20 Hydrology Centre, DSIR, 1989) predicts a flood peak ratio of 0.209 (i.e. equal to $(65.3/463)^{0.8}$).

The ratio of hydrograph base lengths was also found to be very consistent; excluding one outlier, the ratio of the Lee to Wairoa hydrograph base length ranged between 0.91 to 0.99 (average 0.96), which is close to unity, implying little difference. However, assessing the base length of a flood hydrograph is rather more subjective than assessment of the flood volume or the flood peak. This is because the point at which direct runoff in a flood hydrograph ends and the stream flow comprises only baseflow recession is not distinct.

Based on the findings above, and to simplify computation, it was decided to simply scale the ordinates of the Wairoa at Gorge/Irvines design hydrographs by the ratio given by the McKerchar and Pearson approach, without adjustment to the time base. Thus, the scaling factor used was $(77.5/463)^{0.8} = 0.239$ for the proposed dam site at Chainage 12430 on the Lee River. (For the Lee at Waterfall Creek site, to preserve the flood volume ratio and retain the flood peak ratio given by McKerchar and Pearson at the same time would have required the time base ratio to be 0.207/0.209 = 0.99, which is close to unity as assumed anyway).

Design flood hydrographs for the proposed dam site, which has a catchment area of 77.5 km², for return periods of 2.33, 5, 10, 20, 50, 100, 200, 1000 and 10,000 years have been computed on this basis, and are shown in Figure 5.8. The peak flow and 48 hour flood volume for each return period flood is noted in Table 5.2.

In the current study, the flood hydrographs shown in Figure 5.8 and summarised in Table 5.2 are the definitive (final) flood estimates for return periods up to 10,000 years.

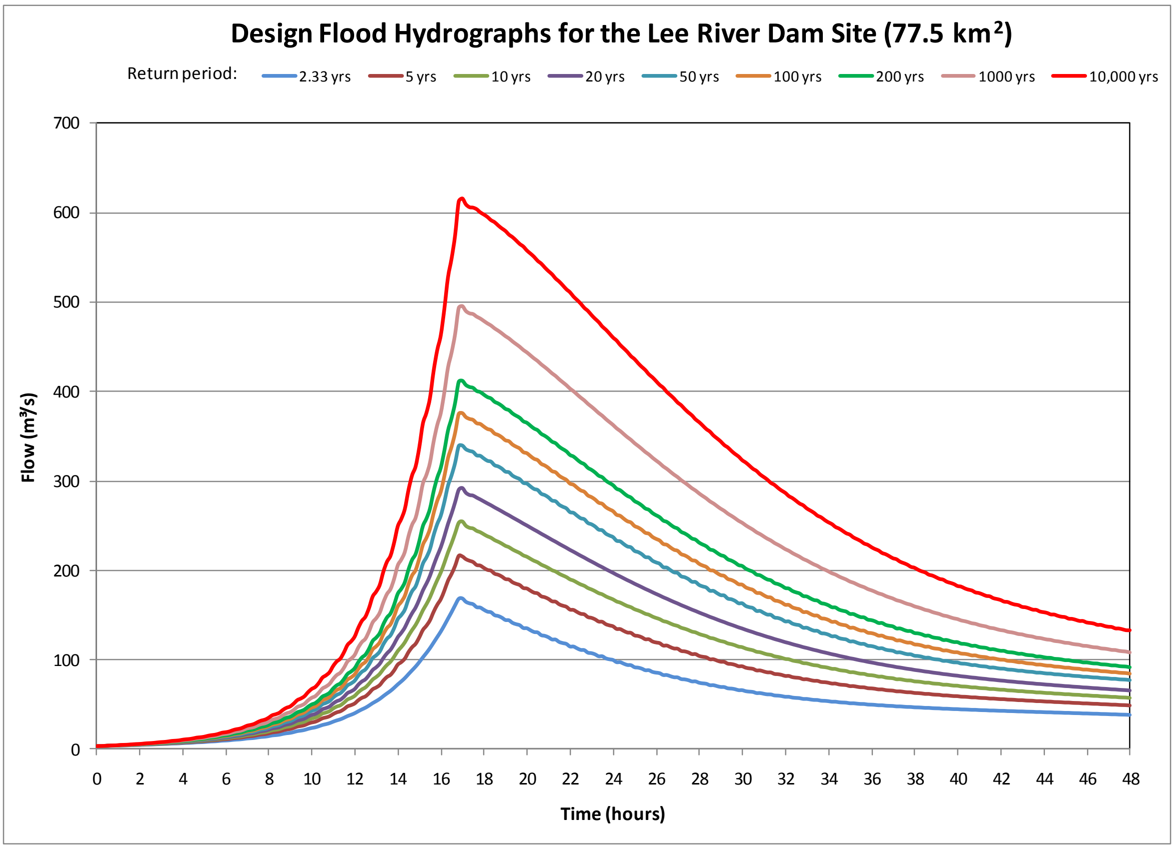


Figure 5.8 Design flood hydrographs for the proposed Lee River dam site for a range of return periods

Table 5.2 Peak inflow and flood volume at the proposed dam site on the Lee River

Flood Return Period	Peak Inflow (m^3/s)	48 Hour Flow Volume (million m^3)
2.33 years (mean annual flood)	168	10.3
5 years	216	13.8
10 years	255	16.6
20 years	292	19.2
50 years	339	22.7
100 years	375	25.0
200 years	412	27.9
1000 years	496	33.9
10,000 years	616	42.4

5.4 Method 3: Rainfall to Runoff Modelling

5.4.1 General

A conventional catchment rainfall-runoff modelling approach has been applied to the Lee River dam site to compare and validate the design flood hydrographs derived using the flood frequency method described in the previous section. The catchment model, which has been calibrated using a series of recorded storm rainfall and flood hydrograph data for the Lee River, has also been used to generate a PMF from the assessed PMP.

HEC-HMS (Hydrologic Modelling System, developed by the US Army Corp of Engineers) has been used to model the catchment response to storm rainfall and hence produce the appropriate design flood hydrographs. The key parts to this process were:

1. analysing historical storm rainfall and flood events in the catchment;
2. calibrating the model using the Lee River (above Waterfall Creek) flow record and rainfall data available for the Lee River and wider catchment;
3. adjusting the calibrated parameters to represent the smaller, but similar adjacent catchment (Waterfall Creek) which also contributes to the flow at the dam site;
4. determining the design rainfall depths and storm temporal profiles;
5. simulating the flow hydrographs based on these design rainstorms; and
6. combining the hydrographs to represent the design flood hydrograph at the dam site.

5.4.2 Storm rainfall analysis

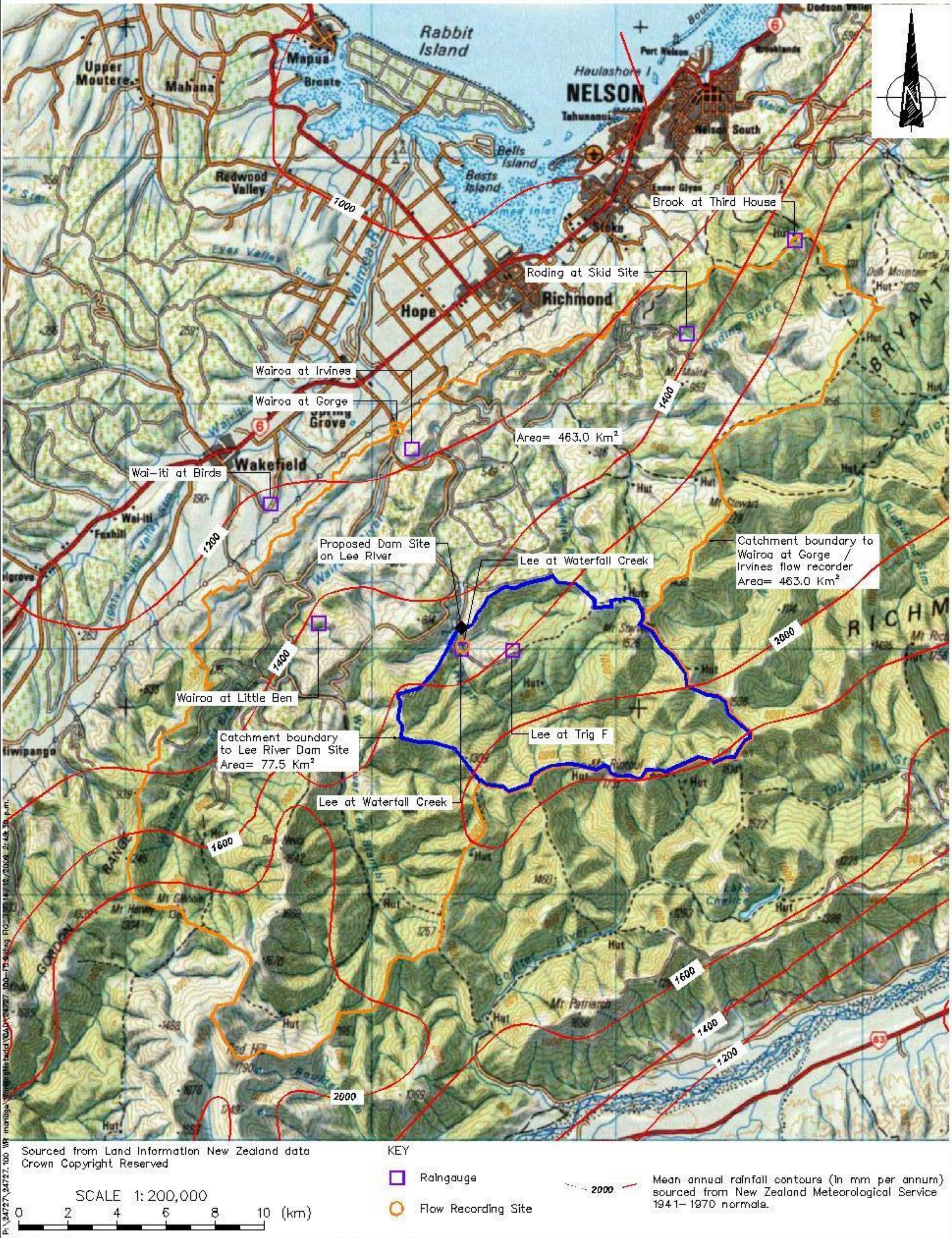
Rainfall data for a number of automatic gauges in the vicinity of the Lee River catchment and, owing to the sparseness of gauges, the wider area has been obtained from TDC in order to identify historical storm rainfall events (for calibration) as well as to analyse its spatial variability across the catchment. The latter is significant because of the known relatively steep rainfall gradient in the general area from the Wairoa Gorge up to the headwaters of the Lee and Wairoa River in the Richmond Range. That is, on average, storm rainfall intensity is expected to increase generally with elevation towards the headwaters of the Lee River.

Table 5.3 lists the data acquired and pertinent statistics for each gauge. Figure 5.9 shows the location of the gauges in relation to the catchment above the proposed dam site on the Lee River. Mean annual rainfall contours from the New Zealand Meteorological Service's map of rainfall normals (1941 to 1970) are also plotted on this figure and serve to illustrate the rainfall gradient noted above.

Table 5.3 Rainfall data summary

Site Name	Site No.	Elevation	Period of data	Mean annual rainfall for record period
Brook at Third House	133336	RL 670 m	1991 – 2009	1813 mm p.a.
Wairoa at Little Ben	134001	RL 427 m	1982 – 2009	1242 mm p.a.
Lee at Trig F	134236	RL 817 m	1989 – 2009	1613 mm p.a.
Wai-iti at Birds	134036	RL 153 m	1982 – 2009	1062 mm p.a.
Wairoa at Irvines	157511	RL 35 m	1993 – 2009	1036 mm p.a.
Roding at Skid Site	157522	RL 205 m	2001 – 2009	1325 mm p.a.
Lee at Waterfall Creek	157536	RL 180 m	2007 – 2009	[1] 1841 mm p.a.
Note: 1 The mean annual rainfall for Lee at Waterfall Creek is only for the 2008 year, and comparison against other adjacent records indicate that the longer term mean for this gauge may be some 14% lower at 1587 mm p.a.				

For the rain gauges with more than 15 years of continuous records (viz. all sites except Roding at Skid Site and Lee at Waterfall Creek), intensity-frequency relationships were developed for rainfall durations 10 minutes to 48 hours based on a series of frequency analyses of the raw data.



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WAIMEA WATER AUGMENTATION - PHASE 2

Catchment Area and Location of Gauges

FIG. No. Figure 5.9 REV. 0

Figure 5.10 and 5.11 are sample rainfall frequency distributions fitted to the 1 hour and 24 hour rainfall depth data respectively for the Lee at Trig F rain gauge.

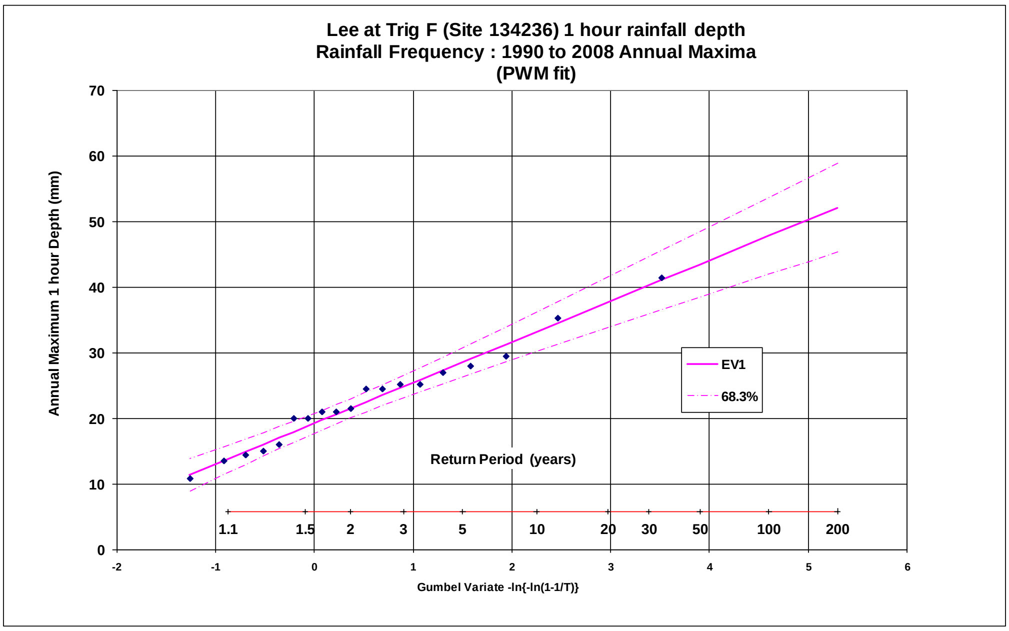


Figure 5.10   Lee at Trig F rainfall depth frequency analysis for 1 hour duration

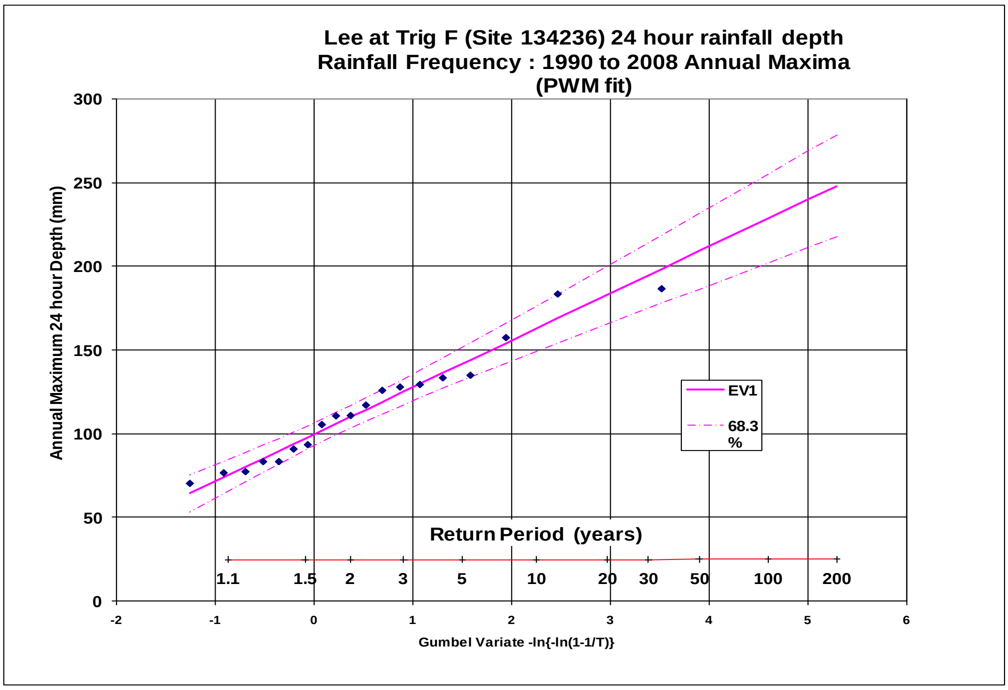


Figure 5.11   Lee at Trig F rainfall depth frequency analysis for 48 hour duration

Based on a series of such distributions, a rainfall depth-duration-frequency (DDF) chart has been developed for each rainfall record, an example of which is shown in Figure 5.12 (for the Lee at Trig F rain gauge).

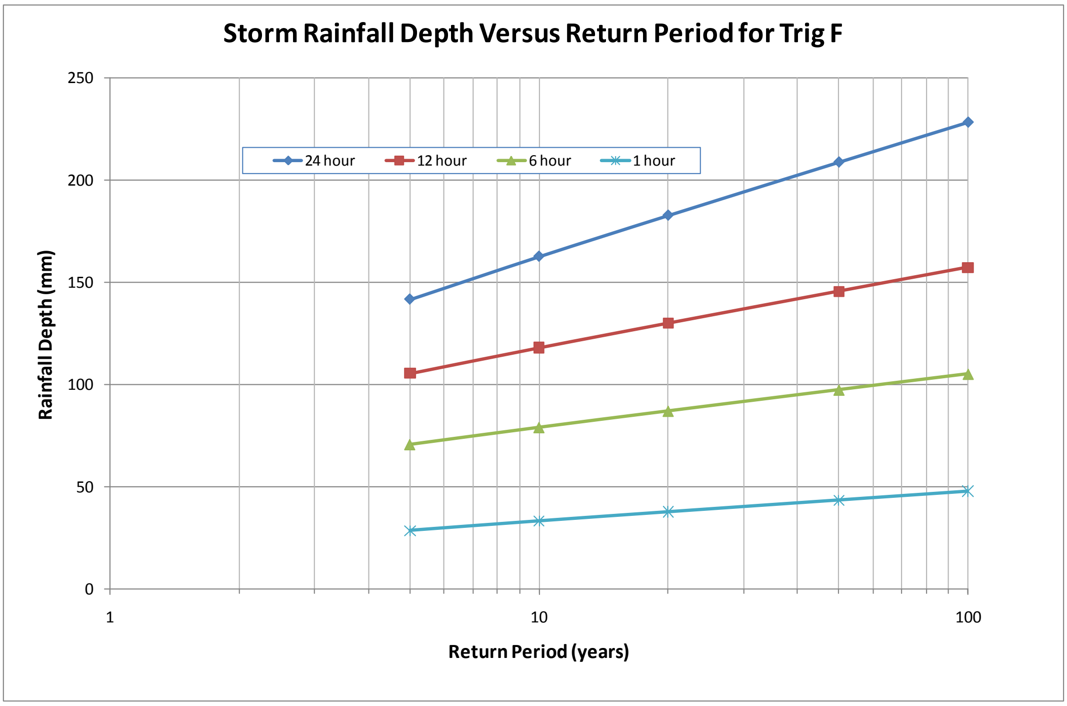


Figure 5.12 Lee at Trig F rainfall depth-duration-frequency relationship

Alternative estimates of the DDF characteristics for any site is available from the software High Intensity Rainfall Design System (HIRDS) developed by NIWA (and for which two versions exist i.e. Version 1.5 and Version 2.0), and from the 1980 National Water and Soil Conservation Organisation (NWASCO) publication "The frequency of high intensity rainfalls in New Zealand Part I". Comparison of estimates from at-site data with estimates from these alternative methods indicates that:

- HIRDS Version 2 consistently under-predicts the rainfall depths for longer durations, say 12 hours and longer
- HIRDS Version 1.5 tends to over-predict rainfall depth in general, especially for shorter durations, say 12 hours and shorter
- NWASCO 1980 over-predicts rainfall depths for the shorter durations but to a lesser degree than HIRDS Version 1.5.

Correlations between DDF and location parameters such as ground elevation and mean annual rainfall were investigated. Based on the 5 rain gauges with more than 15 years of data, the correlation between DDF and elevation of the rain gauge was inconclusive. However, there is a reasonably positive correlation between DDF and mean annual rainfall, such as shown in Figure 5.13, which shows the correlation for the 100 year return period rainfall depth. Similar correlations were found for other return period events.

An interesting feature of these correlations is that, for a particular return period, the rainfall depth is reasonably invariant with respect to mean annual rainfall for short durations up to 2 to 3 hours. With increasing storm duration, there is increasing dependence of rainfall depth on mean annual rainfall.

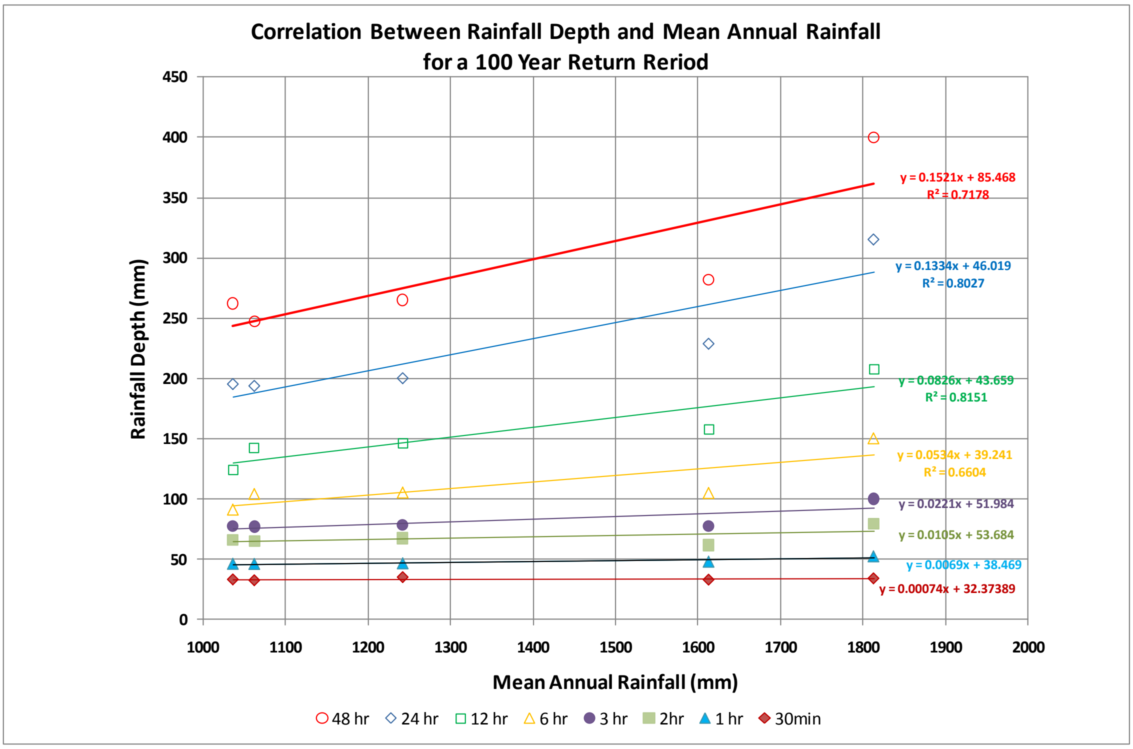


Figure 5.13 Correlation between rainfall depth and mean annual rainfall for 100 year return period

From the catchment water balance assessment (as noted in Section 4.2 earlier), the effective mean annual rainfall for the catchment above the flow recording site (Lee at Waterfall Creek) is estimated to be about 2200 mm p.a. and about 2120 mm p.a. for the catchment above the proposed dam site. These values of the mean annual rainfall have been used to extrapolate the rainfall depth from the relationships such as shown in Figure 5.13 above.

A comparative assessment of the rainfall temporal profile recorded at the available rain gauges for the large 24th November 2008 flood in the Lee River was carried out. From this assessment, it was determined (not surprisingly) that the Lee at Trig F was the most representative of the available rain gauges with regard to replicating the observed runoff pattern in the Lee River (Lee at Waterfall Creek). Thus, this rainfall record (Lee at Trig F) was selected for calibration of the catchment rainfall-runoff model. It is interesting to note that a substantial multiplication factor of between 1.4 and 1.5 was required to elevate the longer duration (≥ 6 hours) rainfall depths from the Lee at Trig F to represent the catchment-wide mean rainfall above the dam site and flow recording site on the Lee River.

5.4.3 Calibration of the rainfall-runoff model

Three storms were used to calibrate the model viz. 23rd May 2007, 22nd January 2008 and 24th November 2008. The latter calibration event (24th November 2008) was the largest flood event of the three and had an estimated return period of 14 years. Sample calibration results are shown in Figures 5.14and 5.15. The comparison demonstrates a reasonably good fit between the actual and predicted hydrographs for the Lee at Waterfall Creek flow recording site. Thus, the calibration has been successful and, accordingly, the calibrated model may be used to reliably compute the design flood hydrograph from a design rainstorm.

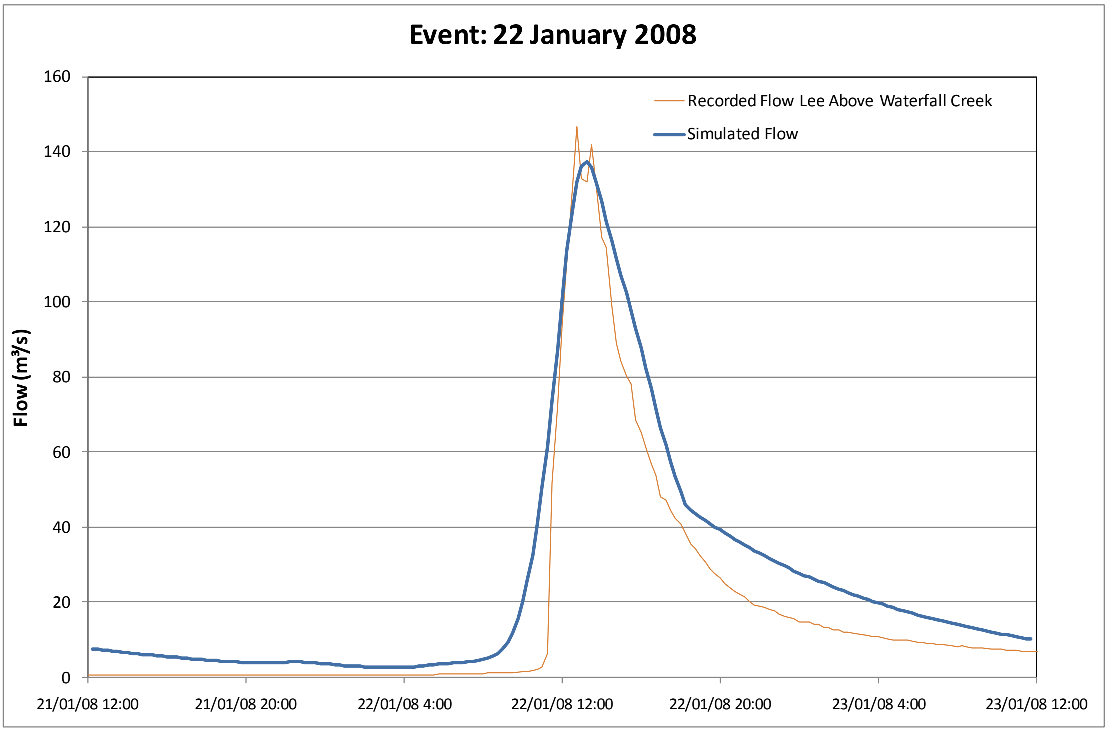


Figure 5.14 Calibration results for rainfall event on 22 January 2008 in the Lee River above Waterfall Creek

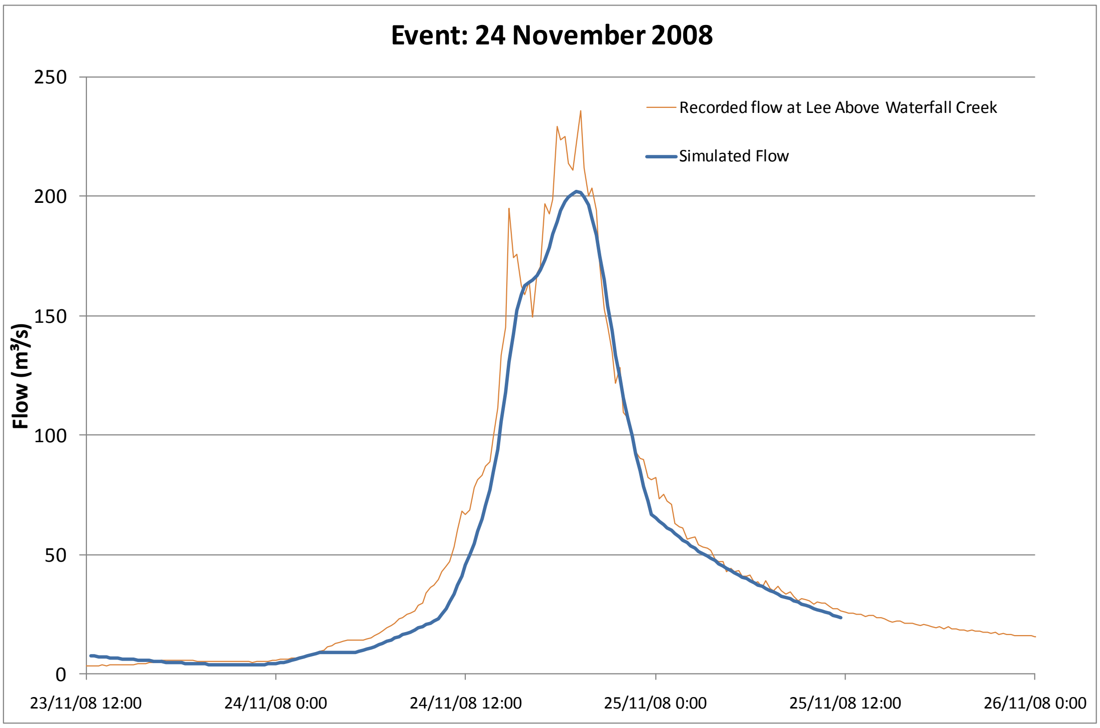


Figure 5.15 Calibration results for rainfall event on 24 November 2008 in the Lee River above Waterfall Creek

5.4.4 Validation of the design flood hydrograph derived from flood volume frequency analysis

The calibrated catchment model was also used to provide an independent check against the synthetic hydrographs derived from flood volume frequency analysis described in Section 5.3 earlier. Figure 5.16 shows a comparison between the model hydrograph generated using the HEC-HMS catchment model for the 100 year return period storm event (48 hour duration storm) and the synthetic flood hydrograph from frequency analysis. The flood hydrograph computed using the catchment rainfall-runoff model (which is considered the conventional approach) is comparable with the synthetic flow method in terms of both peak flow and overall flow volume. Note that the time-to-peak of the synthetic flood hydrograph has been adjusted (to occur at 50% of the storm duration) to provide a closer match to the shape of the HEC-HMS model hydrograph while retaining the same flood volume-duration characteristics.

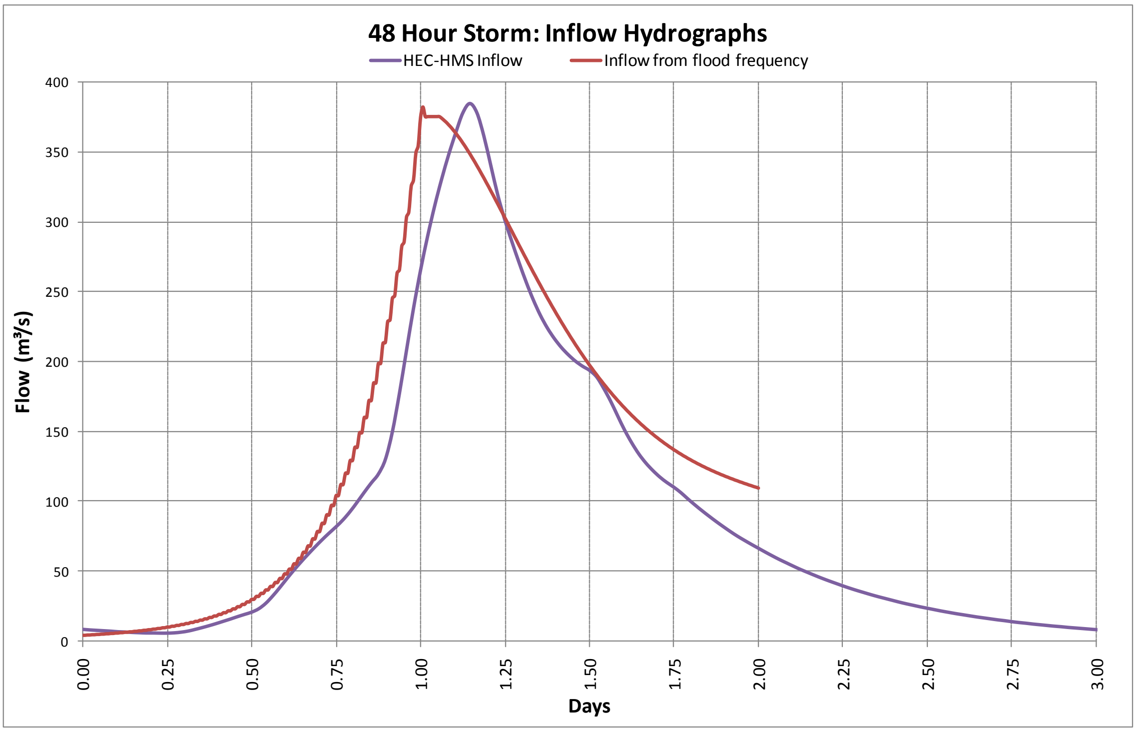


Figure 5.16 Comparison of the HEC-HMS model hydrograph for the 48 hour duration storm and the synthetic hydrograph from volume frequency analysis

Figure 5.17 shows the computed flood hydrographs using the calibrated HEC-HMS model for the 100 year return period rainstorm for durations ranging from 6 hours to 72 hours. This indicates that the 24 hour and 48 hour duration storms have comparable and the highest flood peaks compared with the other storm durations. However, owing to its larger overall volume the 48 hour duration storm is expected to be critical in terms of reservoir routing.

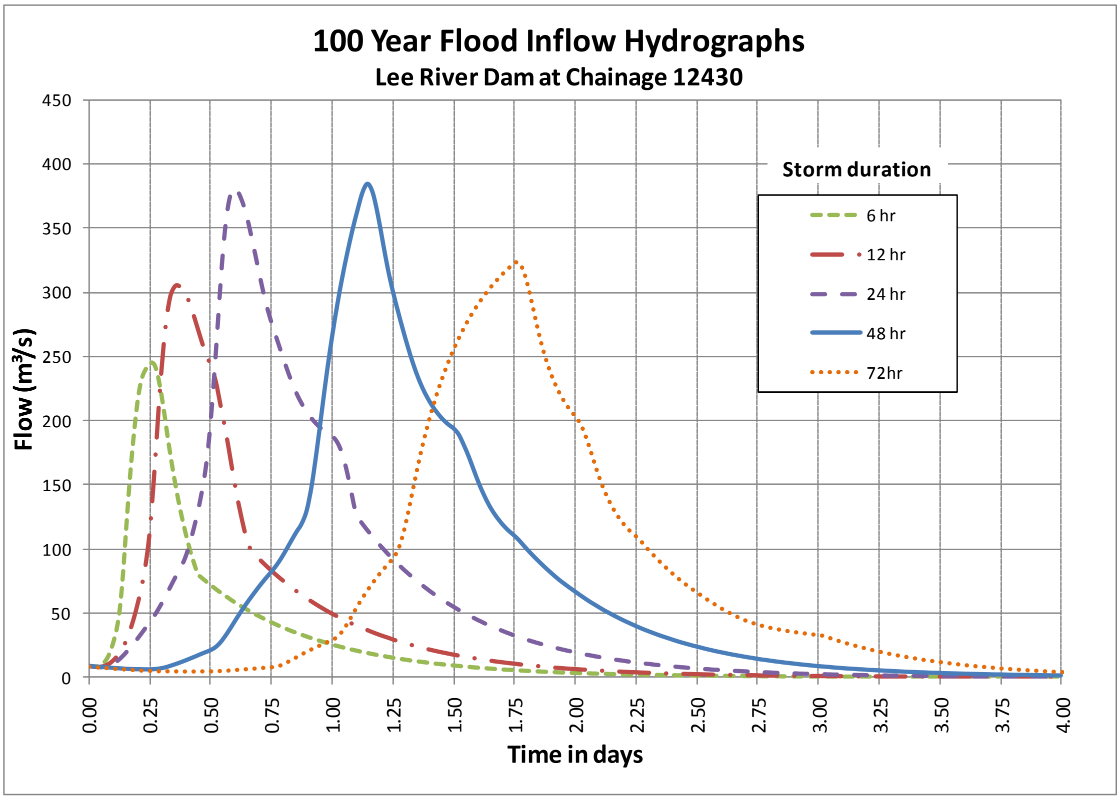


Figure 5.17 HEC-HMS model hydrographs for the 100 year return period storm event of different durations

5.5 Probable Maximum Flood

A Probable Maximum Flood (PMF) for the dam site (Figure 5.18) has been developed. This PMF has been computed from the Probable Maximum Precipitation (PMP) assessed for the catchment at the dam site using the 1995 NIWA approach, "A Guide to Maximum Precipitation in New Zealand" (Thompson and Tomlinson). The calibrated catchment rainfall-runoff model described in the previous section was used to generate the PMF from the 24 hour duration PMP.

Details of the PMP computation are provided in Appendix C.

The peak PMF inflow at the dam site is estimated to be 1094 m^3/s and the 24 hour flood volume 48 million m^3. Thus, the PMF is almost 3 times the 100 year return period flood both in peak flow and flood volume terms, i.e. the 100 year return period flood peak at the proposed dam site is 375 m^3/s and the 24 hour flood volume 18.6 million m^3.

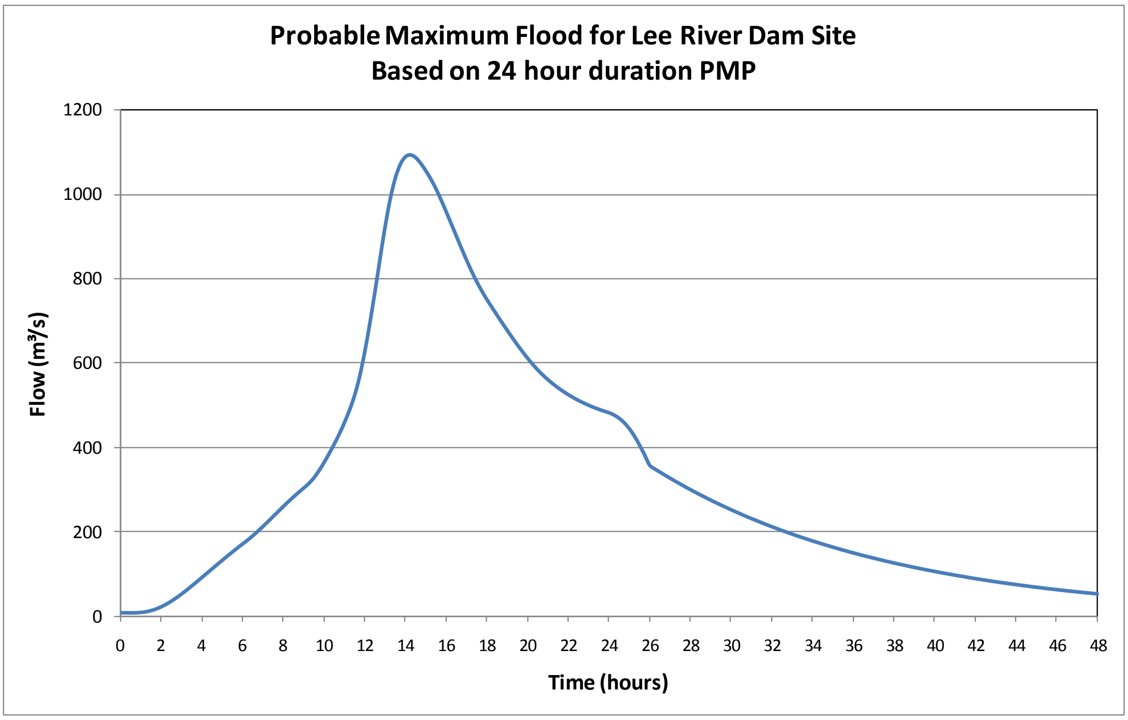


Figure 5.18 Probable Maximum Flood hydrograph for the Lee River dam site at Chainage 12430

6 Dam Storage Modelling

6.1 Overview

The live storage required at a dam is dependent on the following factors:

- consumptive water demand – this has been considered in Section 2
- environmental or residual flows for protection of in-stream values – this has been considered in Section 2
- inter-annual flow variability and the level of drought security desired – flow variability has been represented by the long-term Wairoa River flow record at the Gorge/Irvines; drought security is discussed in Section 6.2 below
- system characteristics and response – these revolve around the catchment characteristics, its drainage pattern and rainfall-runoff response, the river-aquifer interaction and other processes.

A simulation method which takes into the account the parameters and characteristics above has been used to model the dam storage behaviour at the Lee River dam site over the period of the Wairoa River flow record (1958 to 2008). The key to this spreadsheet-based model, which operates on a daily timestep, is the maintenance of an assessed threshold minimum flow (see Section 3.3) at the Gorge/Irvines, which varies according to level of demand and natural river flow. This threshold minimum flow ensures that a residual in-stream flow of at least 1100 ℓ/s is maintained at Appleby Bridge.

Predicted shortfalls in the natural river flow (less the inflow to the dam) must be met by controlled dam releases. It is noteworthy that the flow contributed by tributaries between the dam site and Wairoa Gorge represents about 78% of the total natural river flow at Wairoa Gorge. That is, of the 16.2 m^3/s mean flow (or 511 million m^3 per annum on average) at the Wairoa Gorge, some 12.6 m^3/s (or 398 million m^3 per annum on average) is derived from the intervening catchment between the dam site (at Chainage 12430) and the Wairoa Gorge.

The consumptive demand comprises both groundwater and surface water take from the Wairoa River and aquifers of the Waimea plains as described in Section 2.3, and consists of a future regional need of 22,000 m^3/day in addition to irrigation and urban and industrial demands.

Apart from the consumptive demand and the minimum flow requirement at Appleby Bridge, other aspects taken into consideration in modelling of the dam storage are: maintenance of a minimum residual flow at the toe of the dam equal to the 7-day MALF (0.51 m^3/s); and net evaporation from the reservoir surface. Net evaporation is a relatively minor component of the dam storage water balance. In the model, a repeating annual pattern has been assumed for net evaporation, which is based on open water evaporation less direct rainfall on the reservoir surface for a drought year (1982/1983).

6.2 Drought Definition and Security of Supply

For a given amount of live storage, the level of drought security provided by the dam and reservoir can be expressed as ability to meet the unfettered water demand over the entire duration of a drought with a particular return period, viz. the "design drought return period".

By using a standard approach similar to that applied to estimating floods or low flows, an analysis of the magnitude of the storage fluctuations over time (specifically the minimum level attained in each year of record) produces a relationship between the minimum storage and the expected recurrence frequency or return period. The live storage required is equal to the full storage less this minimum storage – this is called the storage drawdown.

Figure 6.1 (on page 50) provides an example of the simulated storage behaviour from 1958 to 2008, from which the magnitude of the drawdown in each year of the simulation period is evident (these appear as inverted spikes in the plot). This shows that the greatest "need" would have been in the 2000/2001 water year where a live storage in the order of 12 million m^3 would have been required. Figure 6.2 (on page 46) is the corresponding frequency plot of the storage drawdown from which the live storage required versus drought return period is determined. Using the 2000/2001 season again as an example, the frequency plot indicates that the 2000/2001 drought had a return period in the order of 65 years.

Note that there is an important and fundamental difference in the way the severity of a drought is defined for a river system with regulated storage and for one without (i.e. a run-of-river system).

To elaborate:

When required, storage is released from the reservoir to supplement natural river flows according to downstream requirements, typically under low flow conditions. In general, the highest flow releases occur when periods of high demand coincide with very low natural flows.

While the maximum rate of release is related to the magnitude of this shortfall on an instantaneous (or daily) basis, the level to which storage in the reservoir is drawn down depends on the sum of all the preceding releases made. That is, the storage drawdown is a reflection of the accumulated shortfall over time. Thus, for a storage reservoir, the critical situation is one in which the total volume of shortfall over an entire season (or longer if the dam were not full at the start of the season) is a maximum. The magnitude of any single short-lived shortfall episode rarely governs the storage requirement.

For a run-of-river system, the return period of a drought event is typically determined from an analysis of short-term low flow events, such as the instantaneous low flow, the mean daily low flow, or the mean 7-day low flow. So, what may be a significant drought event in a run-of-river system may not necessarily have the same level of significance when there is a storage reservoir because of the different timeframes being considered.

6.3 Confirmation of Storage Requirement

Figure 6.1 (page 50) is a time-series plot of the simulated storage behaviour for a dam on Lee River at Chainage 12430 with a full storage of 13 million m^3. Figure 6.2 (on page 46) shows the frequency distribution fitted to the annual series of maximum storage drawdown from which the quantity of live storage required versus drought return period has been ascertained. Table 6.1 summarises the storage requirements from the frequency distribution.

At the Workshop with the WWAC Technical Group on 10 October 2007, it was agreed that the 60 year return period drought standard from Phase 1 be retained for Phase 2 initially. Phase 1 studies showed that a 13.0 million m^3 capacity storage dam (inclusive of 1.0 million m^3 dead and sediment storage) would provide security in a 60 year return period drought.

The current analysis indicates that for the same 60 year return period drought standard, a marginally lower (3% lower) live storage of 11.6 million m^3 would be sufficient. A supplementary analysis has shown that there is negligible difference in storage requirements whether the dam was located at the original (Phase 1) dam site below Anslow Creek or at the currently preferred dam site above Anslow Creek. That is, at the lower (original) dam site a storage capacity about 1% less would be sufficient.

Adding a 1.0 million m^3 allowance for dead and sediment storage (as for Phase 1) gives a total required storage for the proposed dam site at Chainage 12430 of 12.6 million m^3. For feasibility design of the dam, this has been rounded up to 13 million m^3, which effectively provides a 66 year return period drought security.

Table 6.1 Required storage capacity versus drought return period

Drought Return Period (years)	Required Capacity for Lee River Dam at Chainage 12430	
	Live Storage (million m^3)	[1] Total Storage (million m^3)
10	5.53	6.5
20	7.57	8.6
35	9.48	10.5
50	10.85	11.9
60	11.60	12.6
100	13.90	14.9

Note 1 Total storage includes a nominal 1.0 million m^3 allowance for dead storage and long term sediment infill.

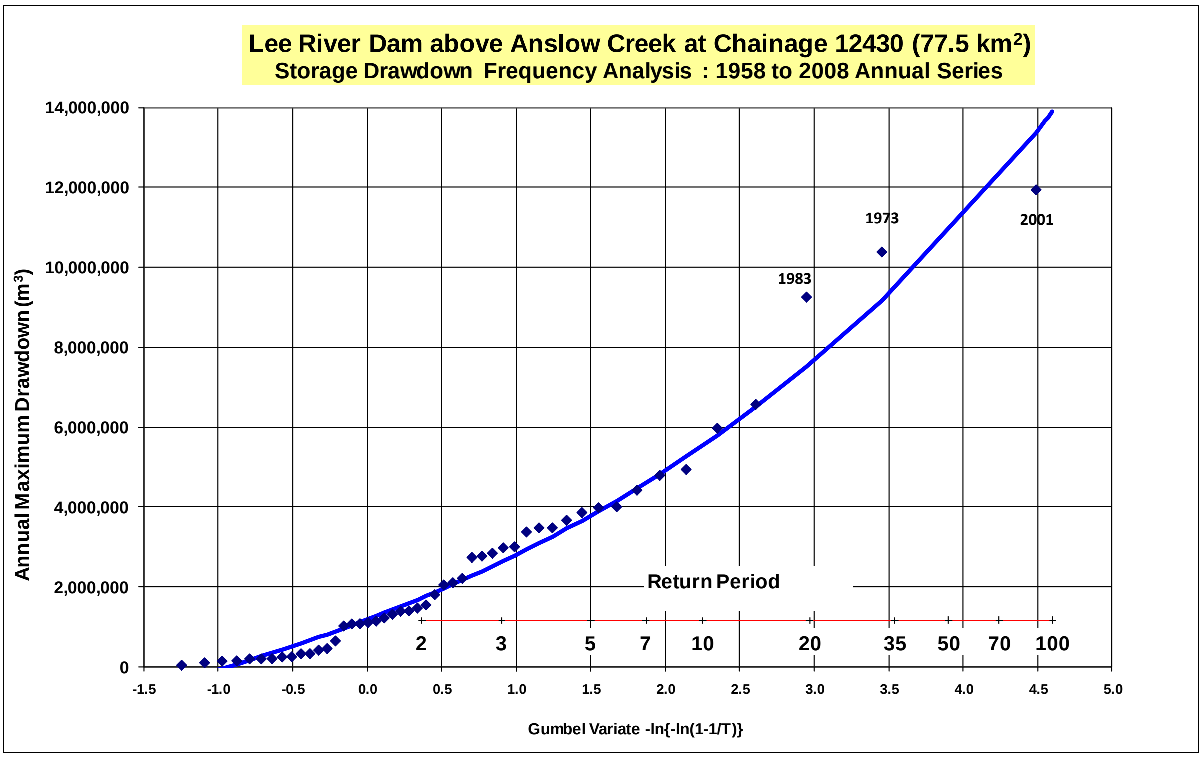


Figure 6.2 Storage drawdown frequency analysis for the Lee River dam site at Chainage 12430

6.4 Preliminary Operating Regime

For a gross storage capacity of 13 million m^3, the storage – elevation curve developed for the dam site, shown in Figure 6.3 indicates a full supply level (normal top water level, NTWL) of RL 196.4 m. This has been rounded up to RL 197 m for the feasibility design. The gross storage at this level (RL 197 m) is 13.4 million m^3.

Figure 6.4 (on page 51) is a time-series plot of the simulated reservoir storage behaviour (volume) from 1958 to 2008 for a dam at Chainage 12430 with a full supply level of RL 197 m and gross storage capacity of 13.4 million m^3. Figure 6.5 (page 52) shows the corresponding plot of reservoir level behaviour. Note that in both plots (and also in Figure 6.1), the water level rise above full supply level in flood events is not shown (i.e. storage is capped at full supply level).

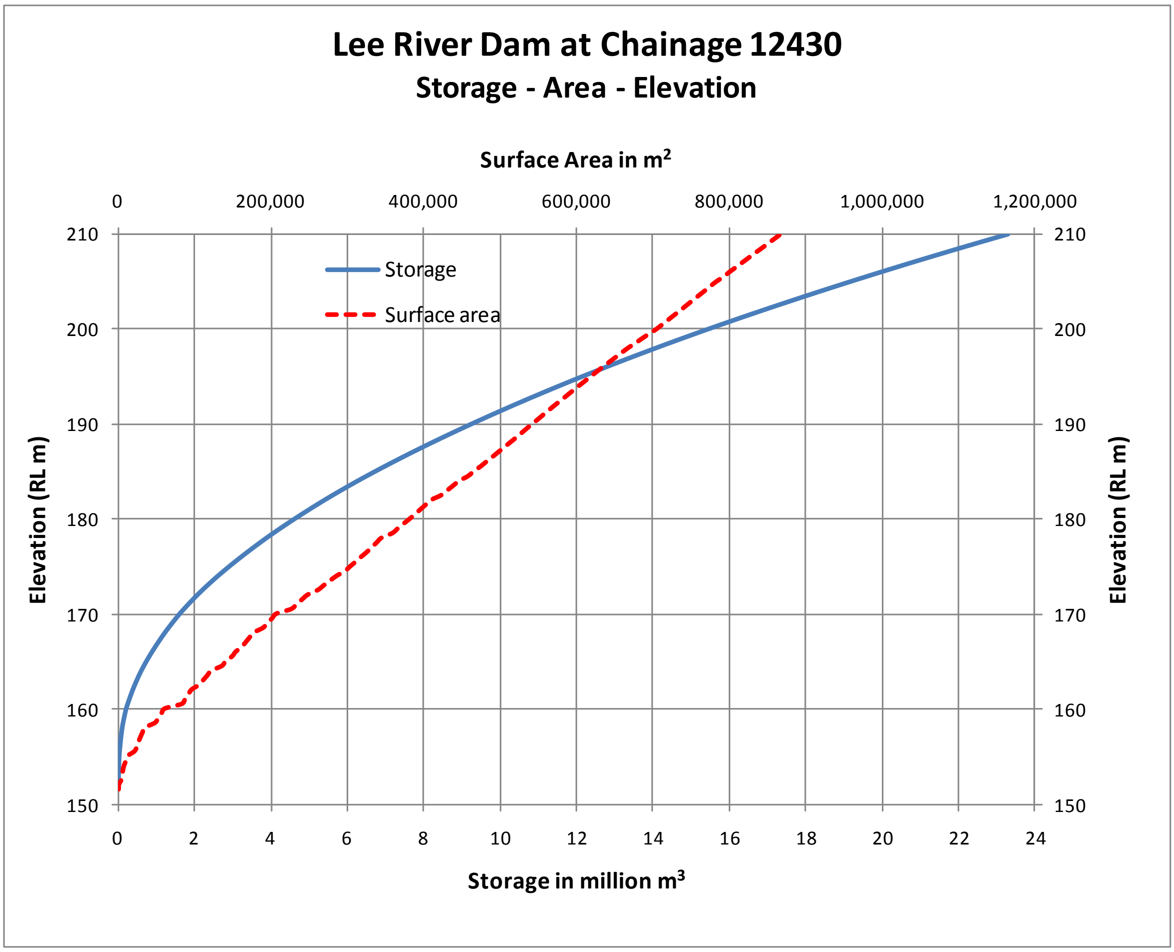


Figure 6.3 Storage – area – elevation relationships for the Lee River dam site at Chainage 12430

Figure 6.6, which is a drawdown duration curve, provides an indication of the proportion of time the reservoir would be full and the proportion of time for which the reservoir is above or below a particular level. In effect, this plot is a condensed form of the time-series data contained in Figure 6.5 (page 52). Figure 6.6 shows that the reservoir would be virtually full about 83% of the time, within 1 m of full about 90% of the time and within 5 m of full for about 96.5% of the time on long-term average assuming fully allocated supply.

Figure 6.7 (on page 53) compares the simulated river flows immediately below the Lee River dam before and after dam implementation for a sample period (1 July 1981 to 30 June 1983). The 1982/1983 water year is a drought year with a return period of about 33 years, whereas the 1981/1982 water year is a more typical year in terms of flows. Note that the pre-dam flows are represented by the reservoir inflows. Figure 6.8 (on page 54) compares the flows in the Wairoa River at Irvines before and after implementation of the storage dam on the Lee River.

As can be seen from Figure 6.7 (on page 53), the reservoir inflows or natural flows (blue) match the dam outflows (pink) for the majority of the time (i.e. the pink line plots over the blue line). Periods of flow augmentation provided by the dam are indicated by the dam outflow plotting higher than the reservoir inflow. In 1982, this occurs between late January and early April, while in the 1983 drought year, flow augmentation was provided from early November (1982) to mid April (1983). Reservoir refilling is indicated by periods where the reservoir inflow plots higher than the dam outflow. A clear example of this is seen in mid January 1983 where a fresh, peaking at about 10,000 ℓ/s, is captured entirely to reservoir storage.

A similar interpretation can be drawn from Figure 6.8 on the effect of flow augmentation. That is, Wairoa River flows at the Gorge/Irvines before and after Lee dam construction are almost identical most of the time, except over summer low flow periods during which the flow augmentation can be clearly seen (pink line plotting higher than the blue line between late January and early April 1982, and from November 1982 to April 1983). However, there is a notable difference between Irvines and the dam site in terms of flow regime changes. That is, the impact of the reservoir refilling is far less obvious at Irvines. For example, the fresh that occurred in mid January 1983 and the series of smaller freshes that preceded it are mostly preserved at Irvines albeit with a slight reduction in the peak flows (15% or so less). This is not unexpected and is attributed to the natural inflows from the tributaries below the dam continuing to contribute to the overall river flow. At the dam site, these freshes were absorbed entirely into the reservoir.

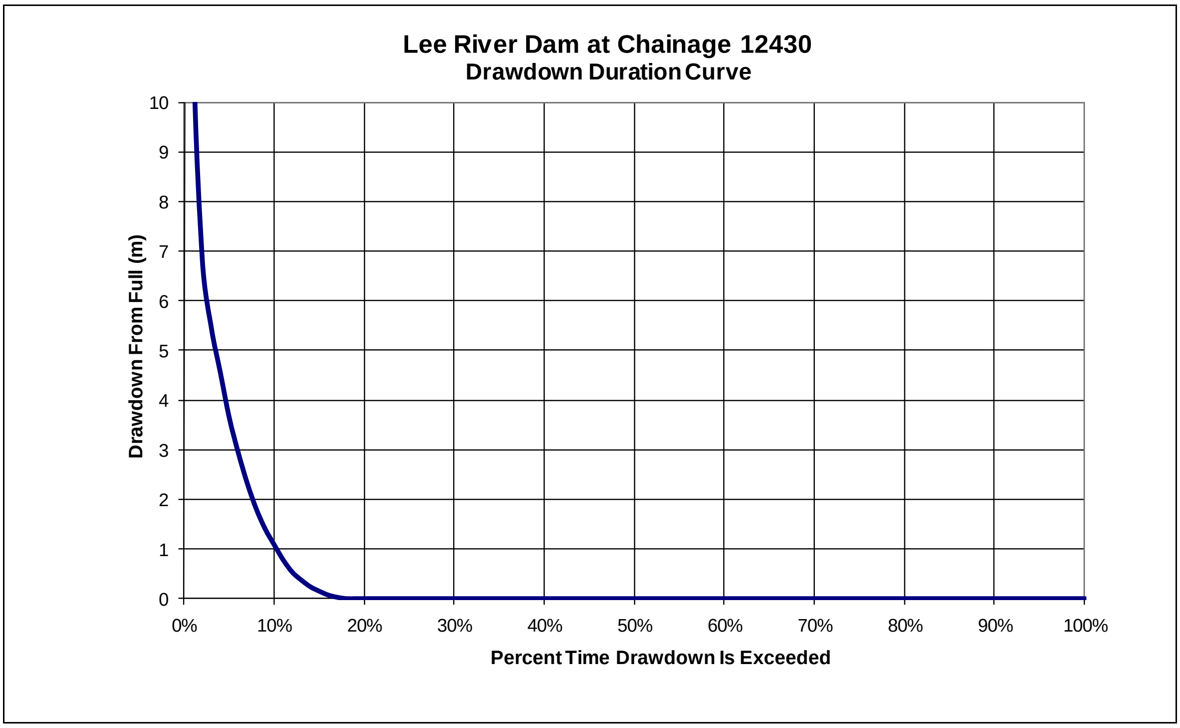


Figure 6.6 Lee River Dam at Chainage 12430 - Storage Drawdown Versus Duration

Table 6.2 shows the predicted changes in the monthly mean flows at the dam site and at Wairoa at Irvines resulting from operation of the proposed Lee River dam based on supplying the full water demand (per Section 2.3). The tabulated flows are monthly flows averaged over the full simulation period from 1958 to 2008.

Table 6.2 Predicted change in monthly mean flows resulting from operation of the Lee River dam (1958 to 2008 average)

Month	Lee River Dam Site			Wairoa at Irvines		
	Dam inflow (ℓ/s)	Dam outflow (ℓ/s)	Change (ℓ/s)	No dam (ℓ/s)	With dam (ℓ/s)	Change (ℓ/s)
January	2713	2745	32	12,210	12,240	32
February	1954	2122	168	8790	8960	168
March	2534	2517	-17	11,400	11,390	-17
April	3451	3313	-138	15,530	15,390	-138
May	3403	3299	-104	15,320	15,210	-104
June	4116	4081	-35	18,520	18,490	-35
July	4311	4280	-31	19,400	19,370	-31
August	4153	4150	-3	18,690	18,690	-3
September	4680	4699	19	21,060	21,080	19
October	4624	4614	-10	20,810	20,800	-10
November	3782	3810	28	17,020	17,050	28
December	3306	3358	52	14,880	14,930	52

Figure 6.1 Lee River dam site – simulated storage behaviour 1958 to 2008 for a dam with full supply storage of 13 million m³

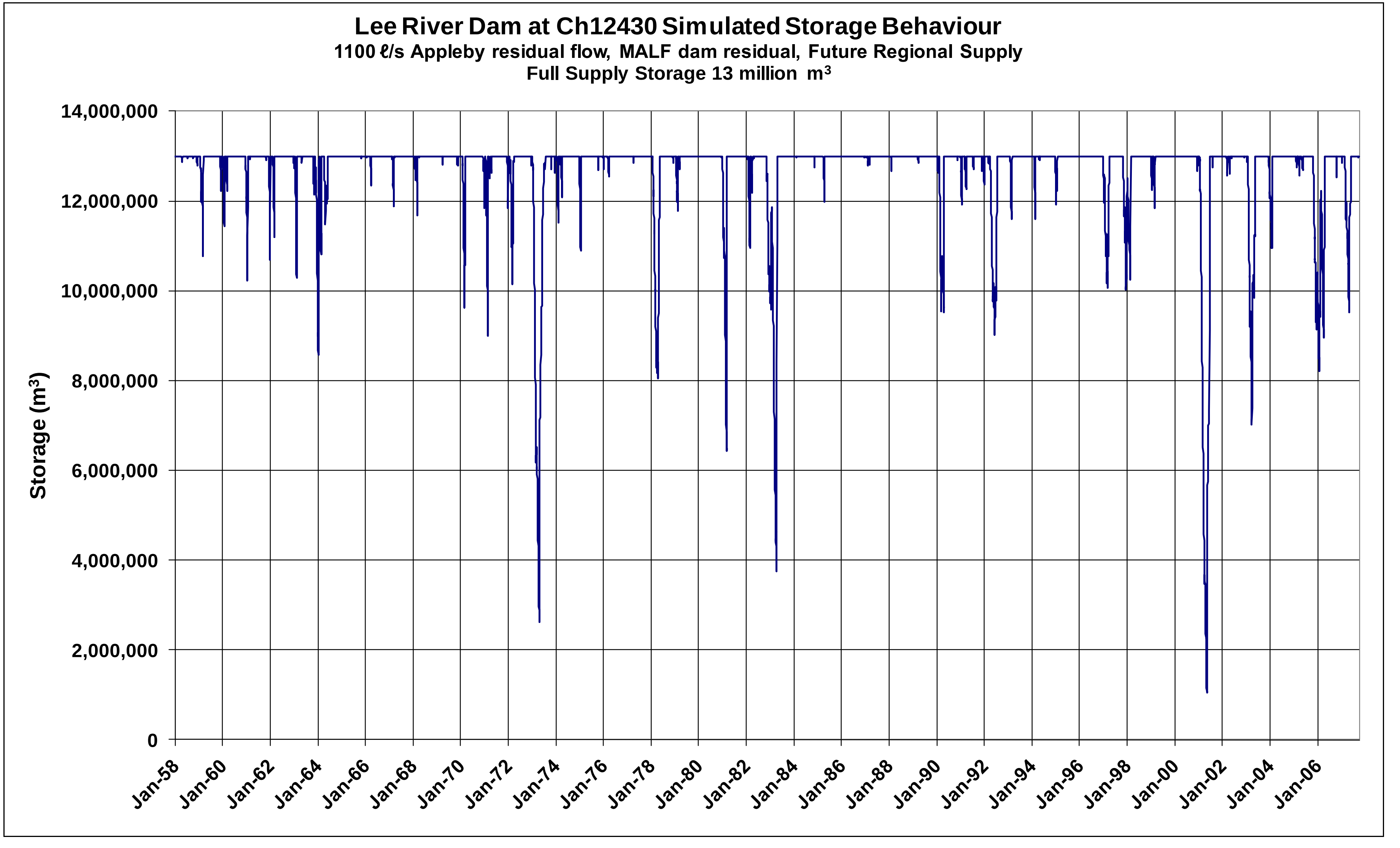


Figure 6.4 Lee River dam site – simulated storage behaviour 1958 to 2008 for a dam with full supply level of RL 197 m (13.4 million m³)

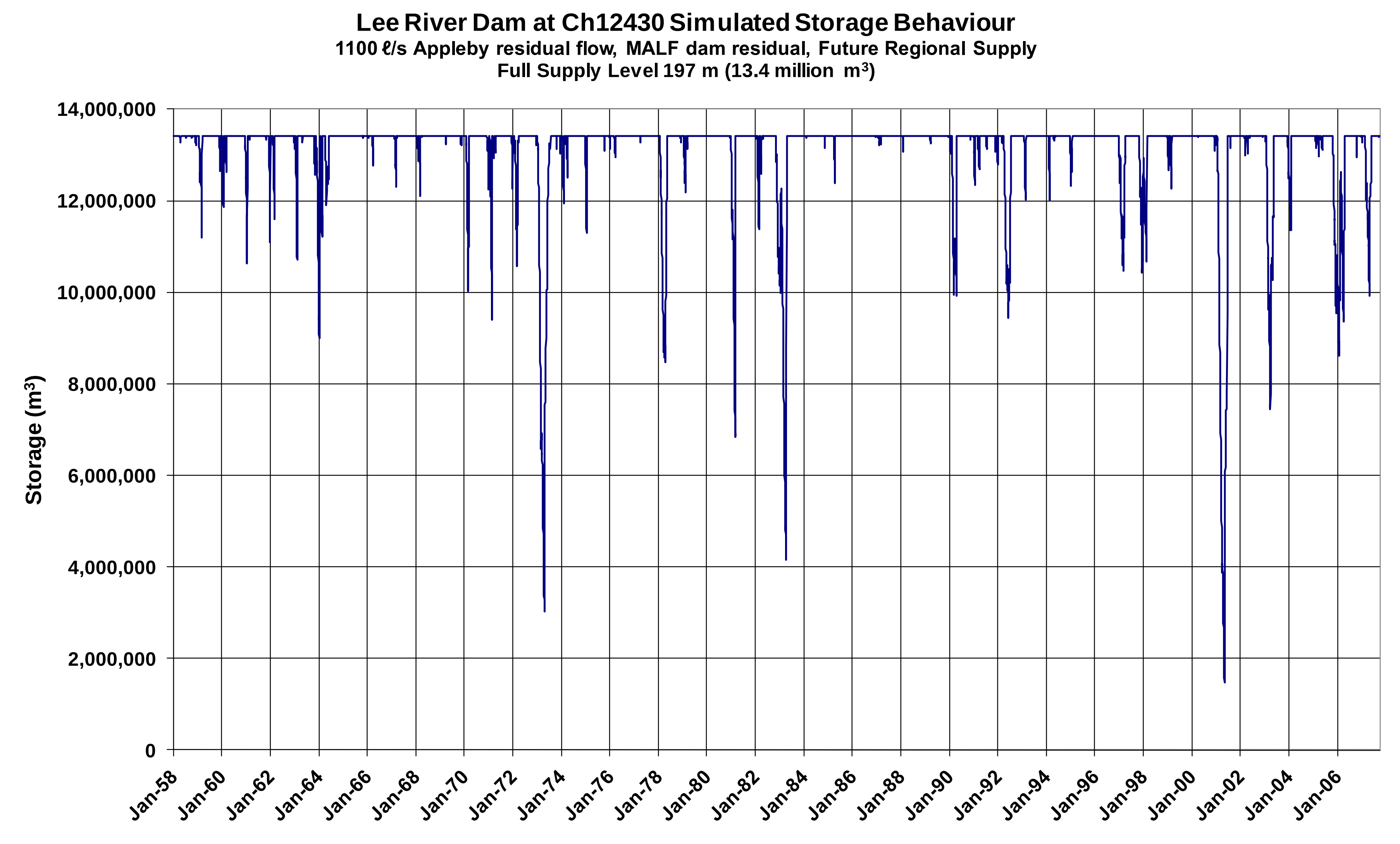


Figure 6.5 Lee River dam site – simulated reservoir levels 1958 to 2008 for a dam with full supply level of RL 197 m (13.4 million m³)

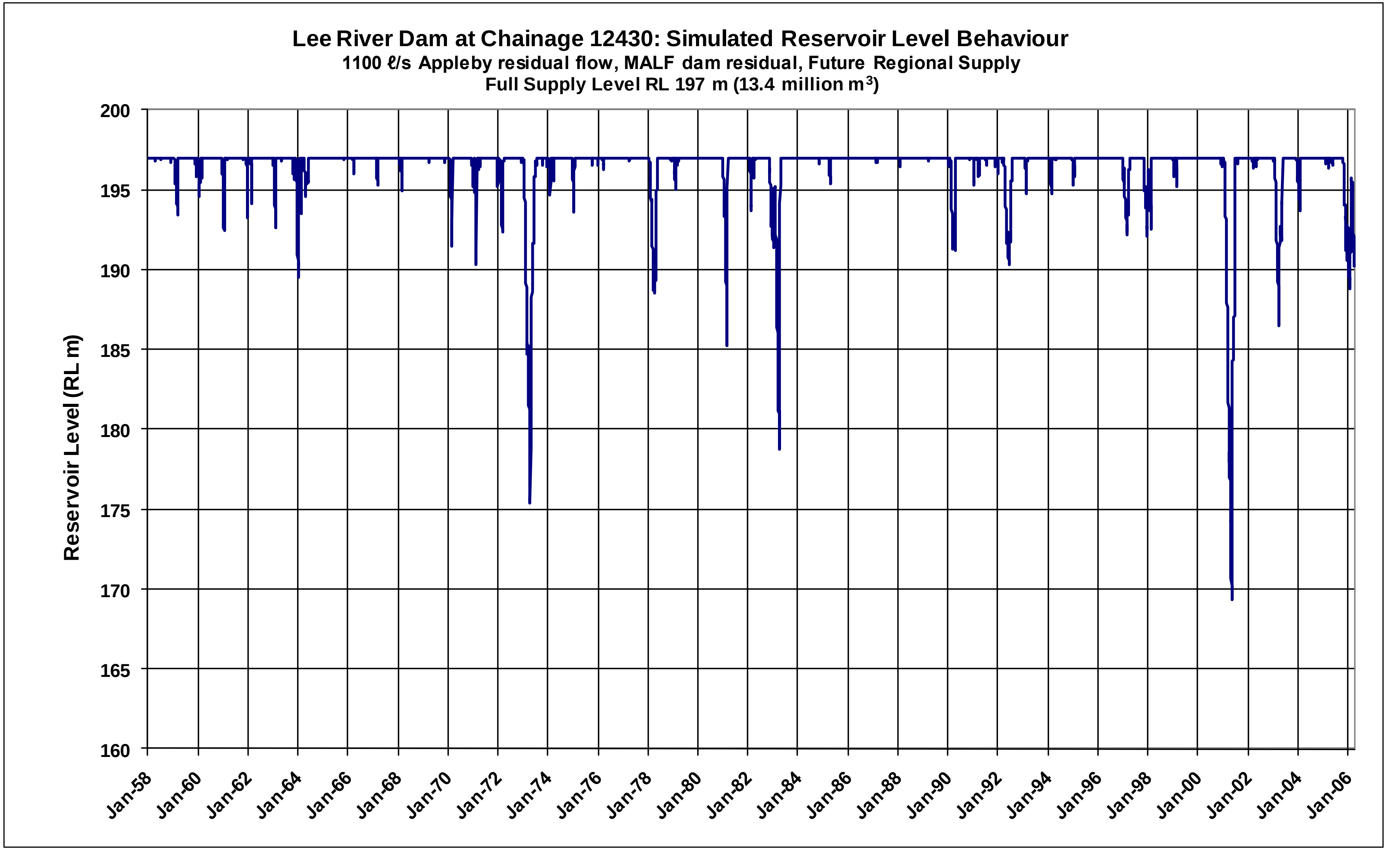


Figure 6.7 Comparison of flow hydrographs at the Lee dam site before and after (simulated) storage dam implementation 1981 to 1983

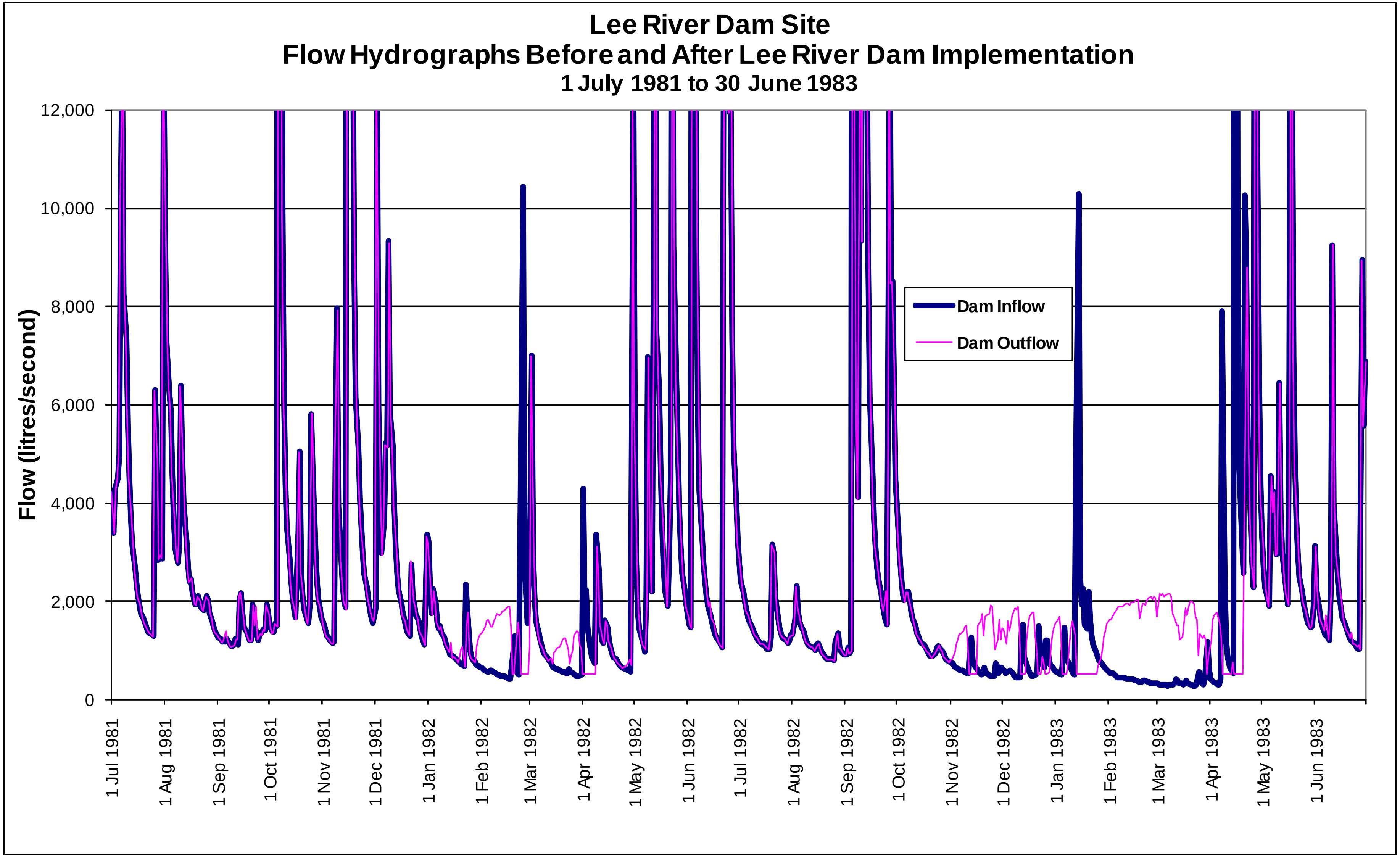
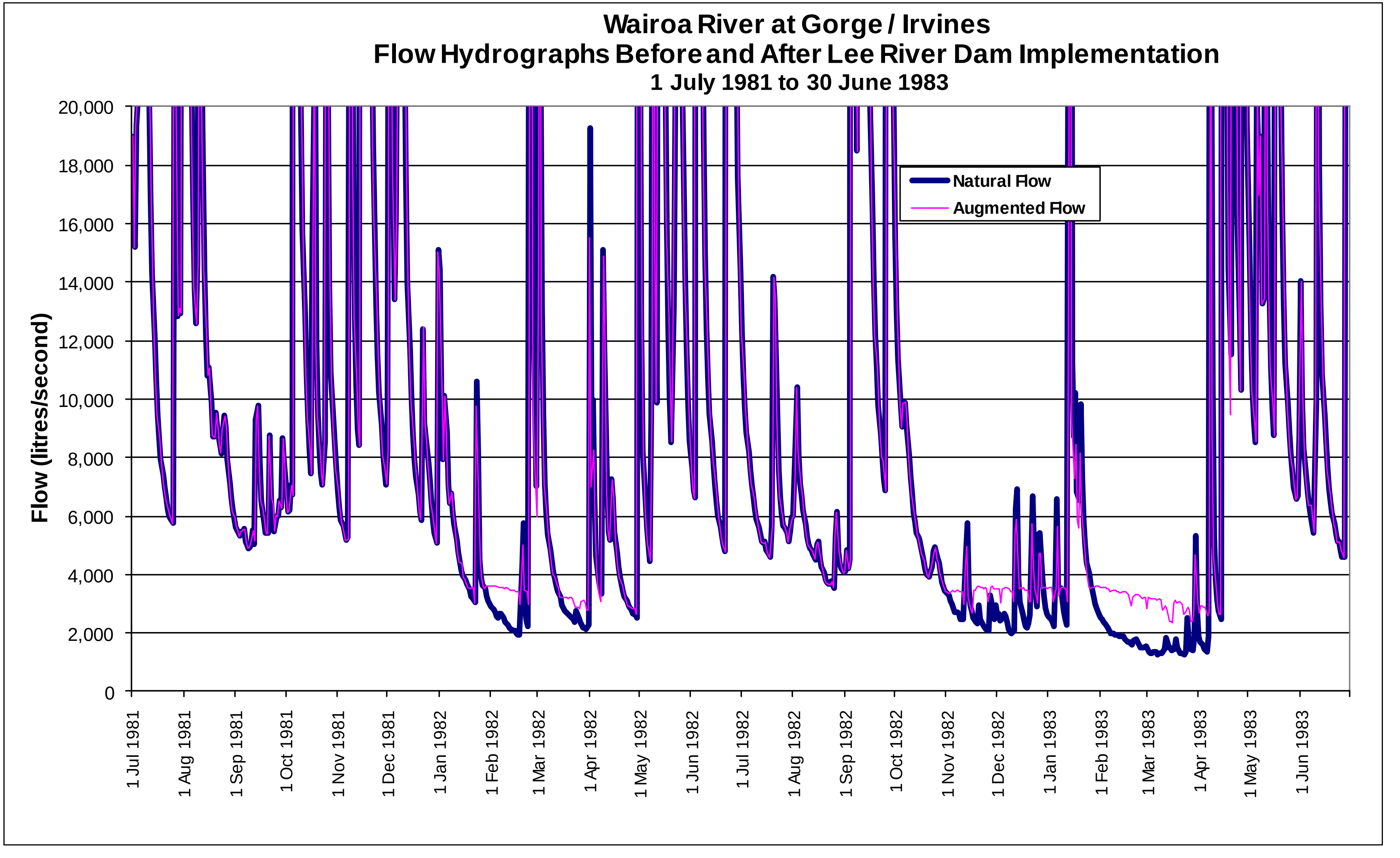


Figure 6.8 Comparison of flow hydrographs at Wairoa Gorge before and after (simulated) storage dam implementation 1981 to 1983



6.5 Operating Regime with Hydropower Add-on

A key component of any hydro-electric power scheme is the headworks which provides the flow and head (pressure) for hydro-generation to be realised via a turbine and generator at a power station. The proposed dam on the Lee River offers an opportunity for a cost-effective hydro-electric scheme to be added since the dam and reservoir would essentially already constitute the headworks for such a scheme. Additionally, a power station and other ancillary works (e.g. switchyard and transmission) would be required, plus modifications to the flow release arrangements.

A preliminary optimisation to determine the preferred hydropower add-on arrangements, particularly the size of the power plant and the additional operational storage, has been completed. Details of this optimisation and the proposed layout are described in the accompanying Dam Engineering Feasibility Study report (Section 11.6).

The preferred hydropower add-on comprises the following:

- a residual flow unit (turbine and generator) with a flow capacity of 0.51 m^3/s matching the dam residual flow and power output of 0.20 MW; plus
- a main unit (turbine and generator) with a flow capacity of 2 m^3/s and power output 0.79 MW; and
- an operational storage volume of 250,000 m^3 for hydropower regulation to enhance capture of inflows (that would otherwise be spilled) to generation.

With regard to the last point, and as noted in Section 6.4 earlier, based on a full supply level of RL 197.0 m, the gross storage capacity available is 13.4 million m^3. This is 400,000 m^3 in excess of the 13 million m^3 targeted. The required hydropower regulation storage of 250,000 m^3 is less than the available “excess” storage. Therefore, the “with hydro” option is able to retain the same full supply level as the “without hydro” option, viz. RL 197.0 m. However, compared with the “without hydro” option, hydropower operation will result in the reservoir level fluctuating frequently within a tight band between RL 196.61 m and RL 197.0 m (a 0.39 m range) rather than remain at a relatively constant level (at RL 197.0 m).

Figure 6.9 (on page 58) provides a comparison of the simulated reservoir level behaviour from the “with hydro” versus “without hydro” options for a sample period (1 July 1981 to 30 June 1983). The selected simulation period includes both a drought year (1982/1983) and a more typical year in terms of river flow availability (1981/1982).

Figure 6.10 (on page 59) compares the simulated dam outflows for the “with hydro” and “without hydro” options over the same period. Flow duration curves of the dam outflow for both options for the full simulation period 1958 to 2008 are compared in Figure 6.11.

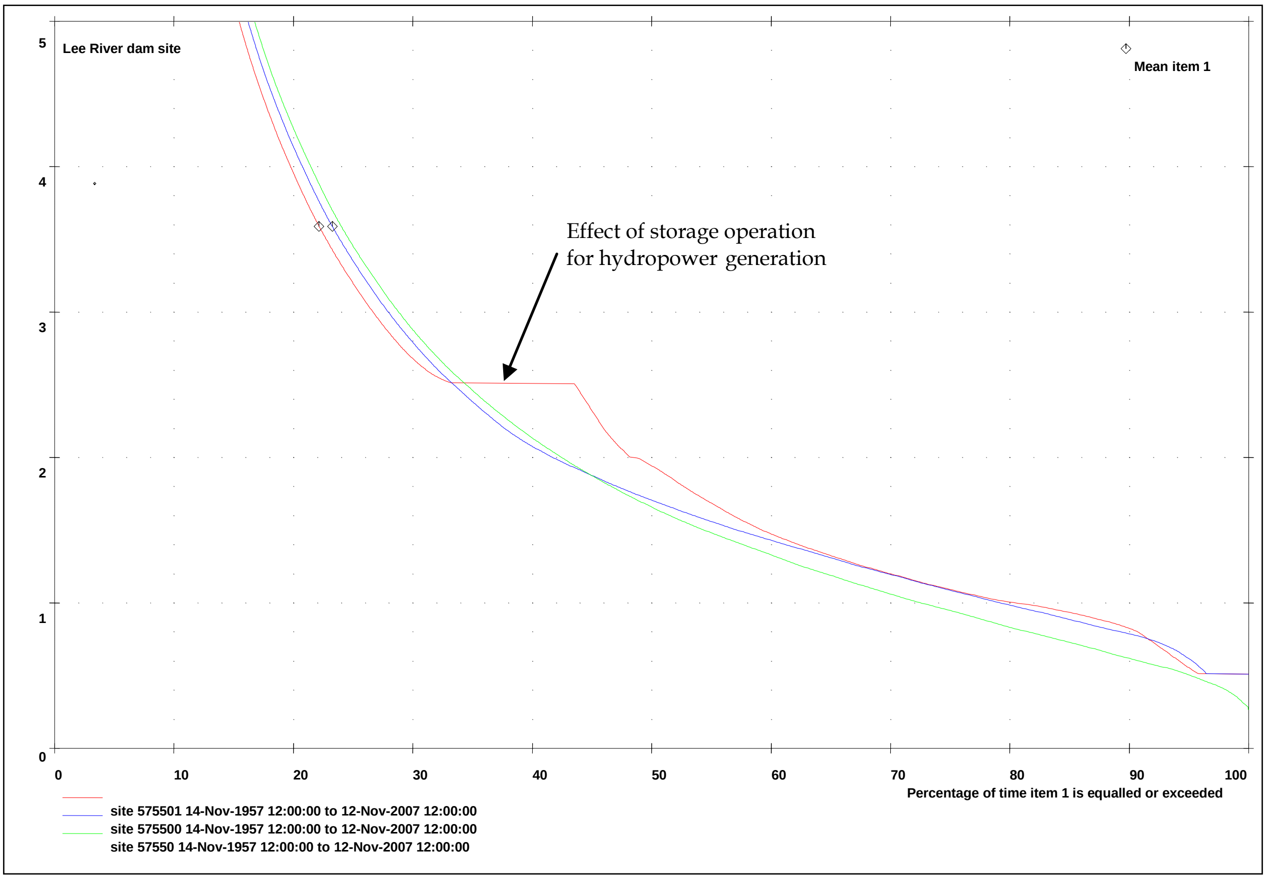


Figure 6.11 Flow duration curves representing the simulated flows at the Lee River dam site for the period 1958 to 2008: red – dam outflow from the "with hydro" option; blue – dam outflow from the "without hydro" option; green – dam inflow. All flows in m^3/s.

6.6 Sedimentation Potential

The Wairoa River, of which the Lee River is a tributary, appears to have a relatively low to moderate sediment load in comparison with many other rivers in New Zealand. The river transports the great majority of its sediment load during flood events. Flows below mean flow are virtually free of suspended sediment.

In Phase 1, it was estimated that over a 100 year period, the amount of sediment that would be trapped within the Lee dam reservoir would be of the order of 600,000 m^3. This estimate used a relationship between river flow and suspended sediment concentration measurements at Wairoa Gorge between 1976 and 1992 with the results then extrapolated to the Lee River.

A recent study by NIWA in 2009 (Analysis of Suspended Sediment Data from Upper Lee River, Nelson, November 2009) funded by an Envirolink Small Grant Fund has further refined this estimate. NIWA's sediment yield estimate at the flow recording site (Lee at Waterfall Creek) is 2900 tonnes per year, equivalent to 45 tonnes per year per km^2 of contributing catchment. This estimate was derived by calibrating the water turbidity record, available for Lee at Waterfall Creek from April 2007 to April 2009, with measured suspended sediment concentrations at the same site.

Over a 100 year period, this sedimentation rate would translate to a reservoir infill volume of approximately 300,000 m^3 at the currently proposed dam site. This is about half what was allowed for in the Phase 1 assessment. Thus, the dead storage allowance of 1 million m^3 for the dam, a large part of which was earmarked for long-term sediment infill, would appear to be generous.

Figure 6.9 Simulated reservoir levels at the Lee River dam site – comparison of "without hydro" and "with hydro" scenarios July 1981 to June 1983 based on common full supply level of RL 197.0 m

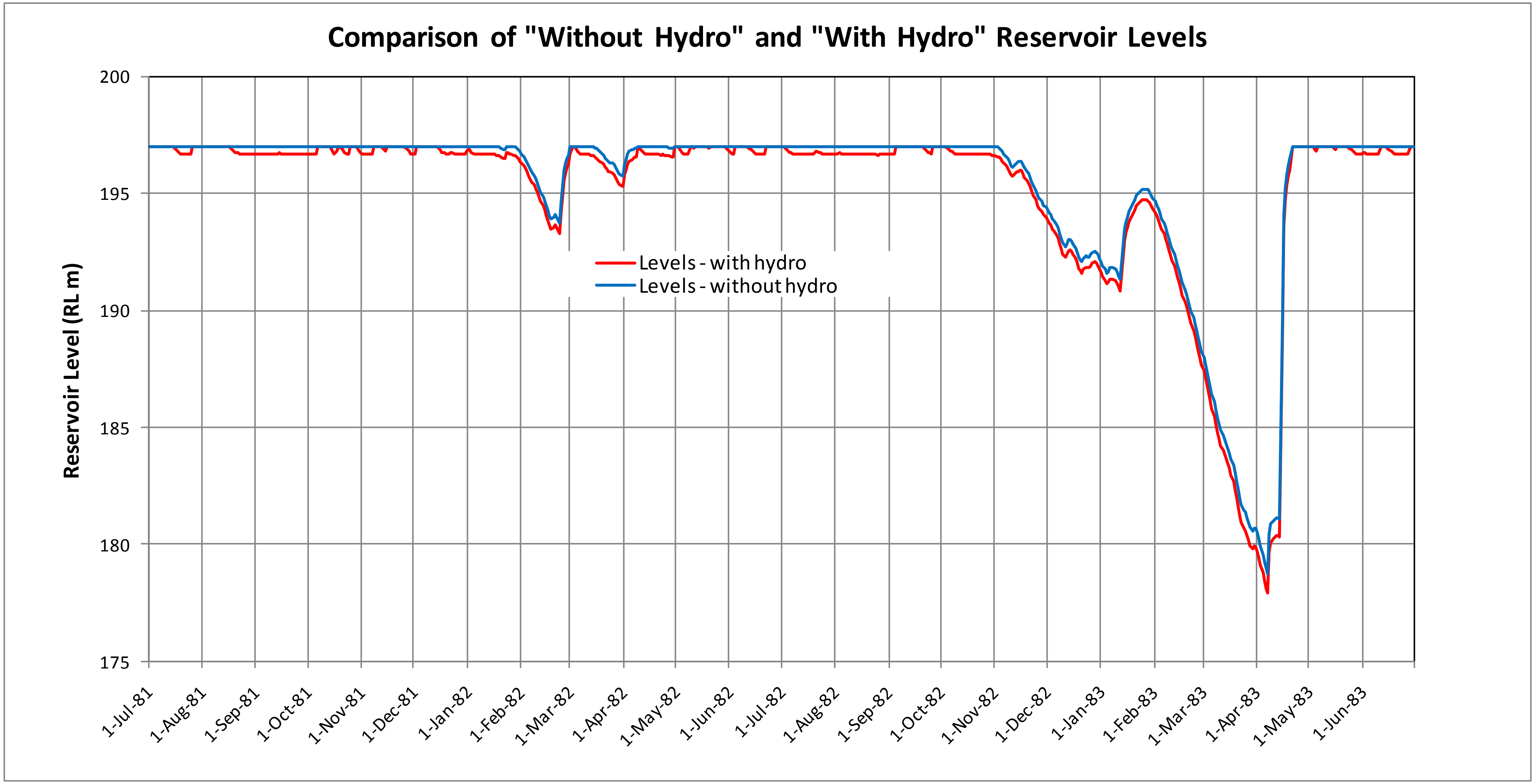
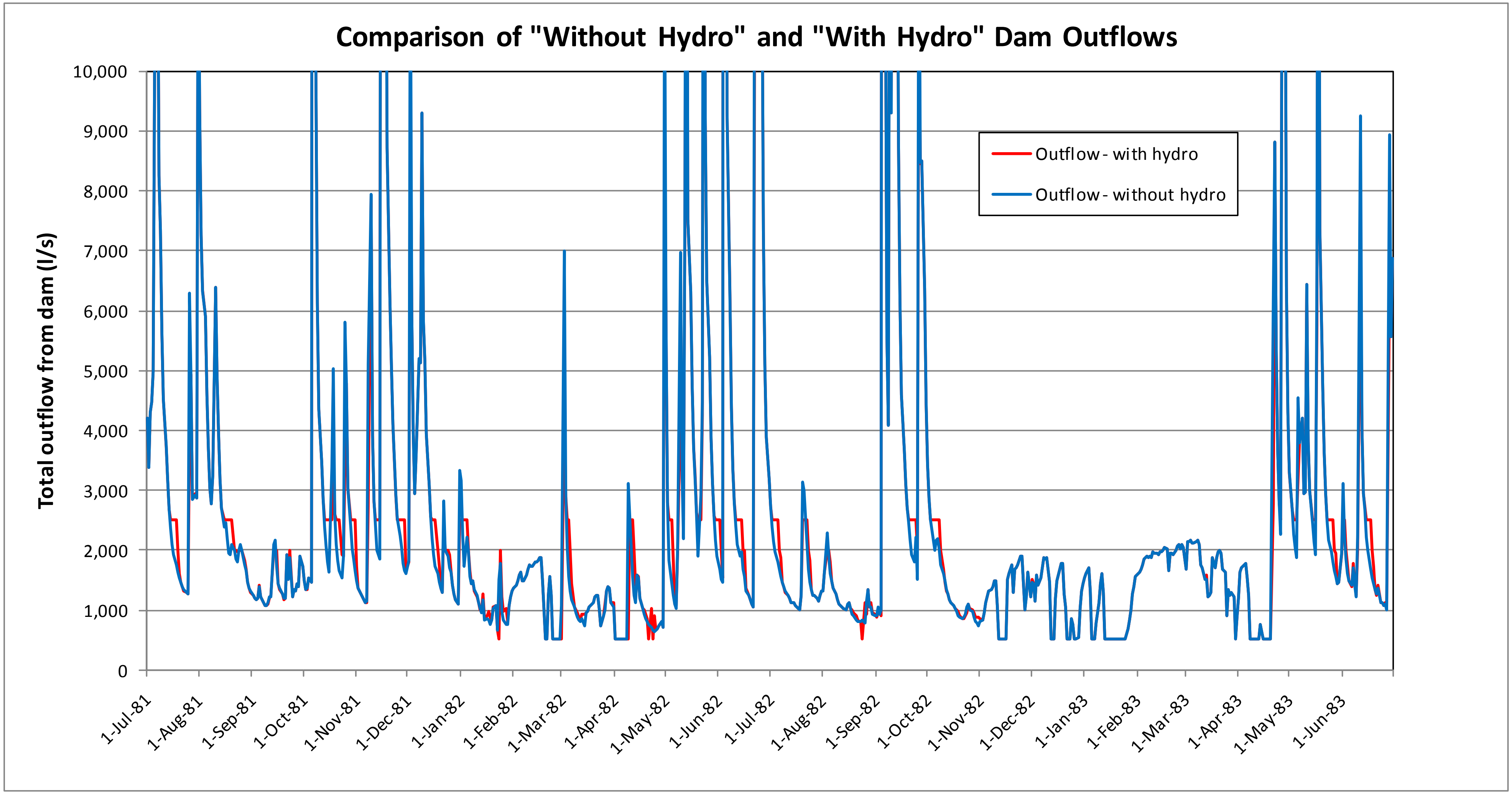


Figure 6.10 Simulated dam outflows at the Lee River dam site – comparison of "without hydro" and "with hydro" scenarios July 1981 to June 1983



7 Applicability

This report has been prepared for the benefit of the Waimea Water Augmentation Committee with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

TONKIN & TAYLOR LTD
Environmental and Engineering Consultants

Prepared by:

....
David Leong

Authorised for Tonkin & Taylor by:

....
Sally Marx
Project Director

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Appendix A: Peer Review Comments

Waimea Water Augmentation Project

Overview of groundwater modelling in regard to groundwater - surface water coupling

Review Note 1

Vince Bidwell, Lincoln Ventures Ltd
10 June 2008

Introduction

This document is a contribution to the peer review of water resource investigations for the Waimea Water Augmentation Project. Within the agreed scope of the peer review it addresses aspects of:

- Overview of groundwater modelling approach
- Overview of surface water modelling approach and coupling with the groundwater modelling.

The specific focus in this note is on the results presented in the spreadsheet received from David Leong on 4 June 2008: *Final result for T & T.xls*

This review analysis is to be read in association with the accompanying spreadsheet:
Final result for T & T_Bidwell review 1.xls

Other information considered

The following documents were also considered in this overview:

- Waimea Water Augmentation – Component 1 Water Demand and Availability. Tonkin & Taylor Ltd, May 2007.
- Waimea Plains groundwater flow modelling with the STREAM package. T. Hong, GNS Client Report 2000/34
- Effects of abstraction on groundwater levels and river flows in the Waimea Plains: modelling and management scenario simulations for droughts inclusive of various Waimea East Irrigation Scheme (WEIS) pumping scenarios. T. Hong, GNS Client Report 2003/69.
- Waimea Water Augmentation – review of catchment modelling and storage. Memo from David Leong to Andrew Fenemor (Landcare), 12 April 2006.
- A Three-dimensional Model for Management of the Waimea Plains Aquifers, Nelson. A.D. Fenemor, Publication No. 18 of the Hydrology Centre Christchurch, 1988.
- Waimea Water Augmentation Project: review of catchment modelling & storage requirement. Andrew Fenemor, May 2006.

Review comments

1. I have calculated river flow loss ("river loss") for 2000-2001 and 2004-2005 as:

   River loss = observed Irvines – observed WEIS – observed Nursery

   and plotted these as "River loss v Nursery"

2. These data suggest that the maximum river loss does not exceed about 1000 L/s, and may be closer to about 700 L/s for low flows at Nursery. The 2004-2005 data show much lower river losses associated with "average" flows, thus demonstrating the sensitivity of river losses to groundwater levels in the aquifers.

3. These general results are consistent with the river gauging results shown in Figure 9 of Fenemor (1988).

4. The Fenemor review (May 2006; para. 50) raises a question about transmission of increased demand across the Plains and the consequences of increased drawdown. I am also concerned about how an aquifer with river recharge input at a maximum of about 1000 L/s (requiring lowered groundwater levels) can store and transmit sufficient water to meet future demand of the order of 2500 L/s for a few weeks.

5. These concerns about groundwater transmission and storage are raised without knowledge of details of the groundwater model and piezometric responses to abstraction stress at critical times and locations in the aquifers.

6. If the maximum river recharge to the aquifer is 1000 L/s (at low groundwater levels), the desired minimum flow at Appleby (similar to Nursery?) is 1100 L/s, and future WEIS river take is about 400 L/s, then the minimum required Irvines flow would be about 2500 L/s (Irvines = WEIS + River Loss + Appleby minimum flow).

7. This value of 2500 L/s corresponds well to Irvines natural flow, for zero augmentation flow, on the operating characteristic shown in Figure 2.4 of the T&T Component 1 report.

8. I have a question about the remainder of the operating characteristic (Figure 2.4; T&T), for values of Irvines natural flow that are less than 2500 L/s. If WEIS take, river loss, and Appleby minimum flow are at the above values during a drought stress period, then Irvines flow minimum should be maintained at 2500 L/s (given unquantified concerns about the state of the aquifer levels). These assumptions imply that the operating characteristic would be a straight line with slope of unity, so that:

   Augmentation flow = 2500 L/s – Irvines natural flow

   for Irvines natural flow < 2500 L/s.

9. Since the WEIS take will vary with time, and the river loss is not precisely known (varies with state of the aquifer and possible long term river bed trends), then an operating characteristic based on Nursery/Appleby flows would seem to be a more stable system control.

10. I recognise that there are only short periods of flow record at Nursery and therefore the augmentation flow is characterised in terms of the long, continuous Irvines record. The departure of this characteristic from the idealised straight line appears to be because it incorporates the variations in WEIS take and river loss as generated by the model.

11. Without access to internal model details, I am unable to comment about the optimality of the augmentation characteristic (Figure 2.4; T&T), in terms of water management, in comparison with more direct Nursery/Appleby system control. However, any differences do have implications for total augmentation volumes from dam storage.

Waimea Water Augmentation Project

Review of water resource, water demand, and operational aspects of “Storage Volumes and Drought Security Standard”

Vince Bidwell, Lincoln Ventures Ltd
19 June 2008

Introduction

This document reviews the basis for results presented in a draft memorandum (“WWAC memo”) from David Leong, 13 June 2008. These results are directed to a WWAC workshop on 23 June 2008.

The critical components contributing to storage volume and drought security are:

- Hydrology of the Lee Catchment, which supplies the storage dam
- Water demand for irrigation, urban, and industrial use in Waimea Plains
- Water augmentation operation to maintain specified minimum flows in Waimea River at Appleby Bridge

Hydrology of the Lee Catchment

1. There is only one year of flow records (April 2007-April 2008) for the Lee River near the proposed dam site. This record is compared with the much longer, and more reliable, record for the Wairoa River at Irvines. The mean flow for the Wairoa for this 2007-2008 period was 68% of long term average, a drier than normal year.

2. Comparison of the one year of concurrent flow records shows that the Lee had a higher mean flow relative to the Wairoa (24.4% at Site CH11010) than estimated in Phase 1 (21.6%). This result leads to examination of the relevance of this drier year and a recheck of the rainfall distribution within the catchment.

3. I have examined the documents *Hydrology Update.pdf* and *MAR adjustment.xls* (received 13 June 2008) which describe the procedure used for reconciling the mean annual rainfall isohyets and rainfall loss with the new streamflow record for Lee River. The procedure involves adjusting rainfall magnitudes according to a fitted equation and fitted rainfall loss that allows a match to Wairoa flow and Lee flow during the dry 2007-2008 year, and Wairoa normal year.

4. The effect of this procedure is to increase mean annual rainfall, for the respective catchment sub-areas, by an increase that is graduated (not linear) from zero at 1350 mm/y up to a 19% increase at 2050 mm/y. The rainfall loss, which is still applied on a whole catchment basis, has increased from the Phase 1 value of 479 mm/y to 624 mm/y. This latter value of loss is likely to be more realistic.

5. As a result of this adjustment to mean annual rainfall, the Phase 1 estimate of long term mean flow for Lee River below Anslow Creek has been revised from 3.58 m^3/s to 3.80 m^3/s. The WWAC memo suggests that a synthetic record for the Lee River could

be obtained by scaling the Wairoa River historical flows. I agree with this statement but the choice of scale factor needs to be more explicit, somewhere in the range 0.21 – 0.25? The re-evaluation of mean annual rainfall has been done effectively, on the basis of best use of the new one-year Lee River record, and fortunately for a dry year. The remaining risk is that the orographic effect for this dry year may not be representative of most dry years. There is no easy answer to this risk but there should be caution about attaching too much precision to the change in estimate of long term yield at the Lee River dam.

Water demand

6. I note that the Phase 2 value of 29.39 million m^3 for “Groundwater Take (incl. urban & indust.)” in Table 2 of the WWAC memo corresponds to “GNS modelled G/W excl. WEIS” in the document *Demand-Phase2.xls (Landcare 1983).*

7. Section 2 of the WWAC memo states that revision of the modelled irrigation demand follows the schedule developed by Landcare. The document *Hydrology Update.pdf* contains Section F: Reconciling GNS and Landcare demands, and refers to the document *Demand-Phase2.xls*.

8. I have attempted to work through the reconciliation calculations in these documents but I am not convinced that I have complete closure. Given that the reduction in groundwater take from 31.81 (Phase 1) to 29.39 million m^3 (Phase 2) is an important feature of Table 2 in the WWAC memo, I would like to see a clear explanation.

9. The Phase 2 value of 4.96 million m^3 for WEIS (Table 2, WWAC memo) corresponds to the Landcare estimate for 48.5% of the irrigable area for 38 mm soils (1187 ha), which is equivalent to an irrigated area of 951 ha.

10. The total irrigated area of 38 mm soils is therefore 951/0.485 = 1960 ha. This value is consistent with the division of areas (reviewer’s calculations) that sum to a total of 6116 ha irrigated area for the Landcare results in columns A to H of *Demand-Phase2.xls (Landcare 1983).* After subtracting the WEIS irrigated area (951 ha), the Landcare total irrigated area becomes 5165 ha. This value can be compared with the total irrigated area for the irrigable area assumptions of 5600 ha plus 250 ha of Rabbit Island, which is a total of 5850 ha and becomes 0.8 x 5860 = 4680 ha irrigated.

11. The total water demand calculated for the Landcare 6116 ha irrigated is 31.15 million m^3, but the calculations in columns O and R of the worksheet imply that the Landcare total does not include urban and industrial use. I am not able to reconcile these results.

Water augmentation

12. I have derived an overall view of flow augmentation requirements by reference to the document *Irvines Minimum Flow Phase 2.xls (Main 1983)* and doing some additional calculation.

13. The predicted groundwater demand for future use is at an average rate of 930 L/s and a peak rate of 2750 L/s. The principal groundwater supply is from river loss at rates up to 1650 L/s, depending on the state of groundwater levels. The difference between groundwater demand and supply requires the use of groundwater storage in the aquifers up to a peak of 13.04 million m^3. This storage value is nearly the same as that for the proposed dam. For, say, 6000 ha of unconfined aquifer with storativity of 0.06 (Fenemor, 1988), the required average change in groundwater level would be about 3.6 m.

14. The river loss rate, between Irvines and Nursery-Appleby, depends on the differences between river level and groundwater levels along the river. Essentially, this means that the loss rate would depend mainly on groundwater levels in the Upper Aquifer. Under dry-year conditions, groundwater demand would have a dominating influence on the state of groundwater levels and hence a correlation between loss rate and demand could be expected.

15. The worksheet *LossFn* in *Irvines Minimum Flow Phase 2.xls* is fitted to the upper envelope of calculated river losses plotted against groundwater demand, and appears to be a good fit for groundwater demand in the range 1000 – 2500 L/s. The worksheet *ChrtAppBoost* in *Phase 2 functions.xls* illustrates the modelled effect of augmentation on flow at Appleby.

16. Operation of augmentation flow has the aim of replenishing groundwater via river loss whilst also maintaining minimum flow of 1100 L/s at Nursery-Appleby. The system constraints are to supply sufficient river loss to groundwater, which reduces as the groundwater level rises, and not lose excessive water down river past Appleby. Feasible feedback signals for system operation (in order of desirability, in my opinion) would be river flow at Appleby, groundwater demand, and supply flow at Irvines. These have been investigated in *Phase 2 functions.xls.*

17. The flow augmentation function shown in Figure 3 of the WWAC memo is a good pragmatic use of available data and selection of feedback signals. It refines the relationship between augmentation and Irvines flow by including the effect of groundwater demand. When the augmentation scheme is put into operation, further refinement may be achievable if real-time flow monitoring at Appleby is installed.

Dam storage and security

18. Within the scope of my brief, I have not reviewed the simulation and calculation of dam storage nor the statistics of extreme values that are used for quantifying security of water supply.

Reference

Fenemor, A.D. (1988): A Three-dimensional Model for Management of the Waimea Plains Aquifers, Nelson, Publication No. 18 of the Hydrology Centre Christchurch.

DRAFT

Waimea Water Augmentation Project

Review of Storage Volumes and Drought Security Standard

Vince Bidwell, Lincoln Ventures Ltd
7 July 2008

Introduction

This document is the final of three review notes. It addresses two points of clarification that were raised in my review note of 19 June 2008, and reports on my examination of the dam storage computations and estimation of drought security.

Clarifications

1. Paragraph 5 of my previous review note (19/06/2008) refers to a scale factor that is applied to Wairoa River flows to convert these to a synthetic record for the Lee River. I was unclear about the value of this scale factor. David Leong has directed me to the document *Lee-57to08-1100-x22.Phase2.11010.xls*, in which the factor is clearly specified as having a value of 0.2346.

2. In paragraph 11 I stated that I had difficulty reconciling two methods of estimating the water demand. I have subsequently discussed this issue with David Leong. We have agreed that there are small differences in approach and results that we cannot reconcile to the last digit. However, these differences are small and, in the context of the derivation of the augmentation rules, do not significantly affect the storage decision.

Storage volumes and drought security

3. I have examined the calculations of storage volume predictions in the document *Lee-57to08-1100-x22.Phase2.11010.xls*, and I am satisfied that these are appropriate and correct.

4. I have examined the calculations in document *Storage DFA-x22.Phase2.11010x.xls* for the Generalised Extreme Value (GEV) approach to estimating the storage required for specified return periods. These calculations appear to be complete and correct.

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

Unit 7C, Rosedale Office Park, 331 Rosedale Road
Albany, Auckland, New Zealand
PO Box 301054, Albany, North Shore 0752
Phone: 64 9 486 2546 Fax: 64 9 486 2556

Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON 6011

10 November 2009

Attention: Sally Marx

Dear Sally,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER DAM: CATCHMENT AND FLOOD HYDROLOGY
AND DAM BREAK ANALYSIS AND HAZARD ASSESSMENT
EXTERNAL PEER REVIEW REPORT NO.6

We have undertaken review of the following documents:

- Section 4 Catchment Hydrology from Waimea Water Augmentation Phase 2-Water Resource Investigations Report (T&T Ref:24727.100)
- Section 5 Flood Hydrology from Waimea Water Augmentation Phase 2-Water Resource Investigations Report (T&T Ref:24727.100)
- Lee River Dam: Dam Break Analysis and Hazard Assessment (T&T Ref:24727.304)

The catchment hydrology has been updated from the Phase 1 study. A flow recording station was installed on the Lee River upstream of Waterfall Creek on 20 April 2007 and this has assisted with improving the accuracy of flow estimates at the proposed dam site. In addition, the proposed dam site has moved upstream to CH12,430m and this has been accounted for in the update of catchment water balance, mean flows and low flow analysis.

Flood hydrology for the proposed dam site (CH12,430m) has also been updated as part of the Phase 2 studies. Three methods have been used to compute design floods for a range of return periods. Synthetic flood hydrographs were compared to the flood hydrograph computed using the conventional catchment rainfall-runoff model. They were comparable in terms of both peak flow and overall flow volume. The 48 hour duration storm is predicted to be the critical in terms of reservoir routing. An estimate of the probable maximum flood is also provided.

We consider that catchment and flood hydrology have been thoroughly assessed and will provide an adequate basis for final design.

A dam break analysis has been undertaken to assist with determining the potential impact classification (PIC) of the dam and to provide information for the emergency action plan (EAP). This information is required by the Building (Dam Safety) Regulations 2008. The dam break analysis has been conducted for a 'sunny day' failure as incremental damages have been assessed likely to be greater than for a flood-induced failure scenario. This is often the case and we consider it also likely to be the case for the Lee River Dam. A rigorous approach has been used to assess the effects of a dam break. The results show quite clearly that the Lee River Dam should be categorised as high PAR. This arises from the modelling that shows approximately 260-300 properties would be at risk of flooding from water depths in excess of 0.5m. We consider that the dam break analyses have been undertaken in accordance with current accepted practice and we concur with the conclusion that the dam should be categorised as high PIC.

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

Appendix B: Waimea Water Augmentation Project Feasibility Study: Phase 2 Modelling Report, GNS Science Consultancy Report 2008/185, August 2009

GNS SCIENCE
TE PŪ AO

DISCLAIMER

BIBLIOGRAPHIC REFERENCE

Hong, T.; Zemansky, G. 2008. Waimea water augmentation project feasibility study: phase 2 modelling report, *GNS Science Consultancy Report* 2008/185. 88p.

CONTENTS

FIGURES

TABLES

EXECUTIVE SUMMARY

The effects of water abstraction from groundwater and surface water (i.e., principally the Waimea East Irrigation Scheme pumping) on groundwater levels and Waimea/Wairoa River flow are an important issue for the Tasman District Council (TDC) with respect to water management in the Waimea Plains. Over the last few years, especially during the summer season, water users in the Waimea Plains have experienced severe restrictions in their water use through drought management measures emplaced to cope with declining streamflow.

To enhance water availability for both regional uses such as irrigation and environmental, community, and aesthetic benefits downstream on the Waimea Plains, a feasibility study titled the Waimea Water Augmentation Project (WWAP) led by the Waimea Water Augmentation Committee (WWAC) was initiated. The objective of the WWAP is to study the feasibility of water storage in the upper parts of the Wairoa-Lee catchments, which would release water into the river systems during low flow periods occurring in the summer.

This report addresses Phase 2 of the WWAP feasibility study. Phase 2 was intended to determine the augmented river flow release required from storages in the upper Wairoa-Lee catchments to maintain specified river flows at the downstream area in the Waimea River for two different drought years (year 1982/1983 and the more severe 2000/2001 drought year) and an average year (2004/2005). The Waimea Plains groundwater-river interaction model developed by GNS Science in collaboration with TDC was used in this study. This groundwater-river interaction modelling for Phase 2 of the WWAP feasibility study has been structured into the following four stages:

(1) Stage 1 Update and recalibrate the Waimea Plains groundwater-river interaction model by using new surveyed river cross-section data for the Wai-iti, Wairoa, and Waimea Rivers, a new rainfall-recharge model, and a new groundwater abstraction model.

(2) Stage 2 Evaluate the effect of water abstraction on river flows at the Nursery-Appleby Bridge location on the downstream end of the Waimea River and undertake scenario simulations to evaluate the effects of streambed changes in the Wairoa/Waimea River on groundwater resources.

(3) Stage 3 Calculate future water demand for the estimated 5,906 ha of expected irrigable area in the Waimea Plains and other water uses. Compute daily minimum river flow augmentation at Irvine-Wairoa Gorge required to maintain a minimum of 1,100 L/sec flow rate at Nursery-Appleby Bridge (while meeting unrestricted abstractive future demand in the period of 1 July through 30 June of the following year) for the 1982/1983 and 2000/2001 drought years and the 2004/2005 average year with the new calculated future water demand and undertake forward scenario simulation to confirm that the proposed river augmented water release regime at Irvine-Wairoa Gorge provided by Tonkin and Taylor Ltd will meet downstream requirements of maintaining a minimum 1,100 L/sec flow rate at Nursery-Appleby Bridge.

(4) Stage 4 Determine the “zone of effect” by analyzing groundwater level change in terms of river flow to enable a map to be created for the zone of river recharge to the aquifer.

- In modelling Stage 3, the future water demand model for the total 5,906 ha of expected irrigable area in the Waimea Plains was developed including urban water demand (for the 100 year horizon), irrigation demand for 250 ha of irrigable area on Rabbit Island, and outside irrigation demand in the Brightwater/Wakefield and Redwood Valley of 300 ha. A pasture crop was assumed over a range of soil types having 38, 78, or 130 mm soil moisture holding capacity. The future water demand model was based on daily climate data for the period of 1 July through 30 June of the following year for the 1982/1983 and 2000/2001 drought years (2000/2001 being somewhat more severe) and the 2004/2005 average year. Future daily peak water demand is estimated to be 3,351 L/sec, including all direct surface water abstraction (the Waimea East Irrigation Scheme or WEIS and 255 L/sec of Future Regional Supply) for the total 5,923 ha of irrigable area in the model domain (the actual area modelled marginally exceeded the expected area involved due to the irregular shape of the area and the fit of rectangular model cells.
- In modelling Stage 3, the model calculated that the daily average augmented flow required at Irvine-Wairoa Gorge to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge for the driest part of the 2000/2001 drought year (1 February through 31 March 2001), considering future water demand, would be 2,822 L/sec (including the WEIS). Similarly, for the 1982/1983 drought year (with the driest part of it also being the 1 February through 31 March time frame) the calculated minimum flow at Irvine-Wairoa Gorge needed to maintain the target minimum flow of 1,100 L/s at Nursery-Appleby Bridge was calculated to be 2,774 L/sec. River flow at Irvine-Wairoa Gorge is predicted to be able to maintain river flow above 1,100 L/s at Nursery-Appleby Bridge considering future water demand at all times of the 2004/2005 year average year without augmentation.
- It is predicted that minimum augmented river flows at Irvine-Wairoa Gorge of 2,474 L/sec (including WEIS) would be required to maintain a minimum flow of at least 600 L/sec at Nursery-Appleby Bridge, compared to 2,822 L/s to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge, for the 2000/2001 drought year.
- The model predicts that when 50% of the full unrestricted future water demand applies, river flow at Nursery-Appleby Bridge would increase by about 622 L/sec compared with average flow when full future water demand occurs in the 1 February to 31 March 2001 period (driest part of the 2000/2001 drought year), indicating that a significant decrease in the occurrence of zero flow, or very low flows of less than 250 L/sec at Nursery-Appleby Bridge, would occur.
- Forward simulation was undertaken to assess whether the augmented river flow at Irvine-Wairoa Gorge developed by Tonkin and Taylor Ltd could maintain a minimum 1,100 L/sec flow at Nursery-Appleby Bridge for the 1982/1983 and 2000/2001 drought years. Forward simulation results indicated that the proposed augmented river flows would maintain river flow above 1,100 L/s at Nursery-Appleby Bridge on most days, but not all days, in the critical periods of late March 1983 and April 2001. Some minor recalculation of the proposed augmented river flow would be necessary to achieve flows above 1,100 L/sec at Nursery-Appleby Bridge at all times.

1.0 INTRODUCTION

The primary sources of recharge to the shallow unconfined aquifers of the Waimea Plains are precipitation, surface water from the Waimea River and its upstream tributaries, and irrigation drainage. These aquifers supply irrigation water to agricultural operations of the Waimea Plains and domestic and industrial water to the Richmond urban area. In recent years, the relationship between water abstraction and low-flow conditions in the Waimea River has become an important issue in the Waimea Plains for water allocation and management. In particularly, low river flows at the downstream area of the Waimea River in drought years are a significant concern for water management because: (1) water shortage has significantly reduced production of irrigated crops, prompted water rationing in the urban area, and has also affected the Wairoa, Lee, and Waimea Rivers as well as the coastal springs that are highly valued by the community and local iwi; and (2) salt water may intrude up the river and into nearby groundwater thereby impacting domestic water supplies.

The Waimea Plains are clearly water-short for a 250 L/s minimum flow at Nursery-Appleby Bridge. Recent studies conducted by GNS Science show that water resources have been over-allocated by 22% (597 L/s) for the conditions of a 1:10 year drought (Hong, 2003). Therefore, the "Waimea Water Augmentation Project" (WWAP) was undertaken to study the feasibility of water storage in the upper parts of the Wairoa-Lee catchments feeding the Waimea River. The objective of this project was enhancement of water availability for regional use and for environmental, community, and aesthetic benefits downstream on the Waimea Plains. It was led by the Waimea Water Augmentation Committee (WWAC). The WWAC represents irrigation interests in the Waimea Plains, but also has representation from the Tasman District Council (TDC), the Nelson City Council, the Fish & Game Council, the Department of Conservation, and local iwi groups. One of the main goals of the WWAP feasibility study was to investigate potential storage options in the upper Wairoa-Lee catchments. Such storage options could potentially be used to release water into the Lee-Wairoa-Waimea River system for irrigation and other uses during dry weather conditions. Phase 1 was undertaken in 2006. Phase 2 was an extension of Phase 1 and consisted of groundwater-river modelling intended to gain a better understanding of the relationships and interactions between groundwater and the associated surface water system of the Waimea Plains. In particular, a major objective of Phase 2 was to quantify residual surface water flow in the downstream area of the Waimea River after groundwater abstraction for two different dry years and the average year.

2.0 PURPOSE OF THIS PROJECT

The purpose of Phase 2 of the WWAP was to determine the augmented river flow release required from storage in the upper Wairoa-Lee catchments to maintain specified river flows in the downstream area of the Waimea River for two different dry years (year 1982/1983 and year 2000/2001) and an average year (2004/2005 year). The two dry years were reportedly on the order of a 1 in 20 year drought and a 1 in 24 year drought respectively, based on a previous low flow analysis by TDC. However, with storage incorporated in the river system, the severity of these dry years in terms of reliance on storage-based augmentation will be different. The Waimea Plains groundwater-river interaction model (Hong, 2000; Hong, 2003; and Hong 2006) developed by GNS Science in collaboration with TDC was used in this study.

The river water augmentation modelling in this project was structured into the following four stages:

Stage 1 Update the Waimea Plains groundwater-river interaction model:

(1) Update the river model structure (Stream package) in the existing Waimea Plains groundwater-river interaction model by using the new surveyed river cross section data of the Wai-iti, Wairoa, and Waimea Rivers provided by TDC; (2) Update the rainfall-recharge model of the Waimea Plains groundwater-river interaction model by using new data provided by Landcare Research; (3) Update the groundwater abstraction model in the Waimea Plains groundwater-river interaction model with new data surveyed by TDC; and (4) Re-calibrate the Waimea Plains updated groundwater-river interaction model using historical data sets of river flows and groundwater levels.

Stage 2 Evaluate the effect of water abstraction on river flows, particularly in the downstream area of the Waimea River (i.e., Nursery-Appleby Bridge) and undertake scenario simulations to evaluate the sensitivity of the groundwater system to bed changes in the Wairoa/Waimea River.

Stage 3 Determine the augmented river flow release required from storages:

(1) Compute future water demand in the Waimea Plains based on climate data for the one year dry periods of 1982/1983 and 2000/2001 and the one year average period of 2004/2005 year (1 July through 30 June of the following year) and using an irrigation water balance model developed by Landcare Research for a pasture crop over a range of soil types (38 mm soil moisture holding capacity; 78 mm soil moisture holding capacity; and 130 mm soil moisture holding capacity); (2) Compute the daily minimum river flow augmentation at Irvine-Wairoa Gorge required to maintain a minimum 1,100 L/s flow rate at Nursery-Appleby Bridge while meeting unrestricted abstractive future demand in the periods of 1982/1983, 2000/2001, and 2004/2005; (3) Deduce the daily minimum river flow augmentation at Irvine-Wairoa Gorge required to maintain a minimum 600 L/s flow rate at Nursery-Appleby Bridge with unrestricted future water demand for the 2000/2001 year; (4) Repeat modelling for partial abstractive future water demands equivalent to 75% and 50% of the full unrestricted future water demand for the 2000/2001 year; (5) Undertake forward scenario simulation to confirm that the proposed river augmented water release regime at Irvine-Wairoa Gorge provided by Tonkin and Taylor Ltd. will meet downstream requirements of maintaining a minimum 1,100 L/s flow rate at Nursery-Appleby Bridge.

Stage 4 Determine the “zone of effect” by analyzing the groundwater level change in response to change in river flow in order to map the zone of river recharge to the aquifer.

3.0 GROUNDWATER RESOURCES IN THE WAIMEA PLAINS

The Waimea Plains cover an area of 75 km^2 and are located at the coastal margin of the Waimea Catchment, adjacent to the town of Richmond. The Waimea Plains are formed of late Quaternary terrestrial terrace and floodplain gravels deposited by the Waimea River and its major tributaries, the Wairoa River to the east and the smaller Wai-iti River to the south. The soils of the Waimea Plains are highly productive, with the principal source of water for irrigation, domestic, industrial and urban supply being groundwater from the various aquifers that underlie the area.

Three major aquifers (Figure 1) have been delineated under the Waimea Plains and are named the Lower Confined Aquifer (LCA), Upper Confined Aquifer (UCA) and the Appleby Gravel Unconfined Aquifer (AGUA). There are also minor aquifers called Hope Minor Confined and Unconfined Aquifers (HU). Figure 2 represents the three dimensional hydrogeology of the Waimea Plains.

3.1 Appleby Gravel Unconfined Aquifer (AGUA)

The Appleby Gravel Unconfined Aquifer underlies the floodplains of the Wai-iti, Wairoa and Waimea Rivers and the delta of the Waimea River. The AGUA is up to 15 m thick, with the water table averaging 2 to 3 m below ground level. The AGUA is underlain by the Hope Gravel, with the Hope Gravel being a clay-bound gravel. The contact with the Hope Gravel is less distinct in the Wai-iti Valley, where the AGUA is less permeable than in other areas of the Waimea Plains. Transmissivity values in the Wai-iti area range from 2,000-3,500 m^2/day. The AGUA is most permeable in the youngest gravel, adjacent to the Wairoa and Waimea Rivers. Transmissivity values of 20,000 m^2/day have been measured adjacent to the Waimea River and in the delta part of the plains. The AGUA is in contact with the Upper Confined Aquifer, and in lateral contact with marine gravel and sand in the Waimea delta region. Recharge to the AGUA occurs from the precipitation, surface waters, and irrigation drainage. The main river recharge zones are between the Wairoa Gorge and Brightwater township, upstream of the State Highway 60 Bridge near Appleby and downstream of Spring Grove on the Wai-iti River.

3.2 Hope Minor Confined and Unconfined Aquifer (HU)

East of the Appleby Gravel deposits, minor water-bearing lenses occur in gravel fans in the Hope Gravel, derived from the eastern hills. These unconfined and confined aquifers are seldom more than 0.5 m thick and exist to a depth of about 15 m. Laterally they are discontinuous, so drawdowns due to pumping are high.

Recharge is only from rainfall, and associated runoff, because the aquifers are above the level of any river influence. Pumpage from this aquifer is primarily for domestic use and small-scale irrigation. Water levels and yields decline markedly in this aquifer in summer.

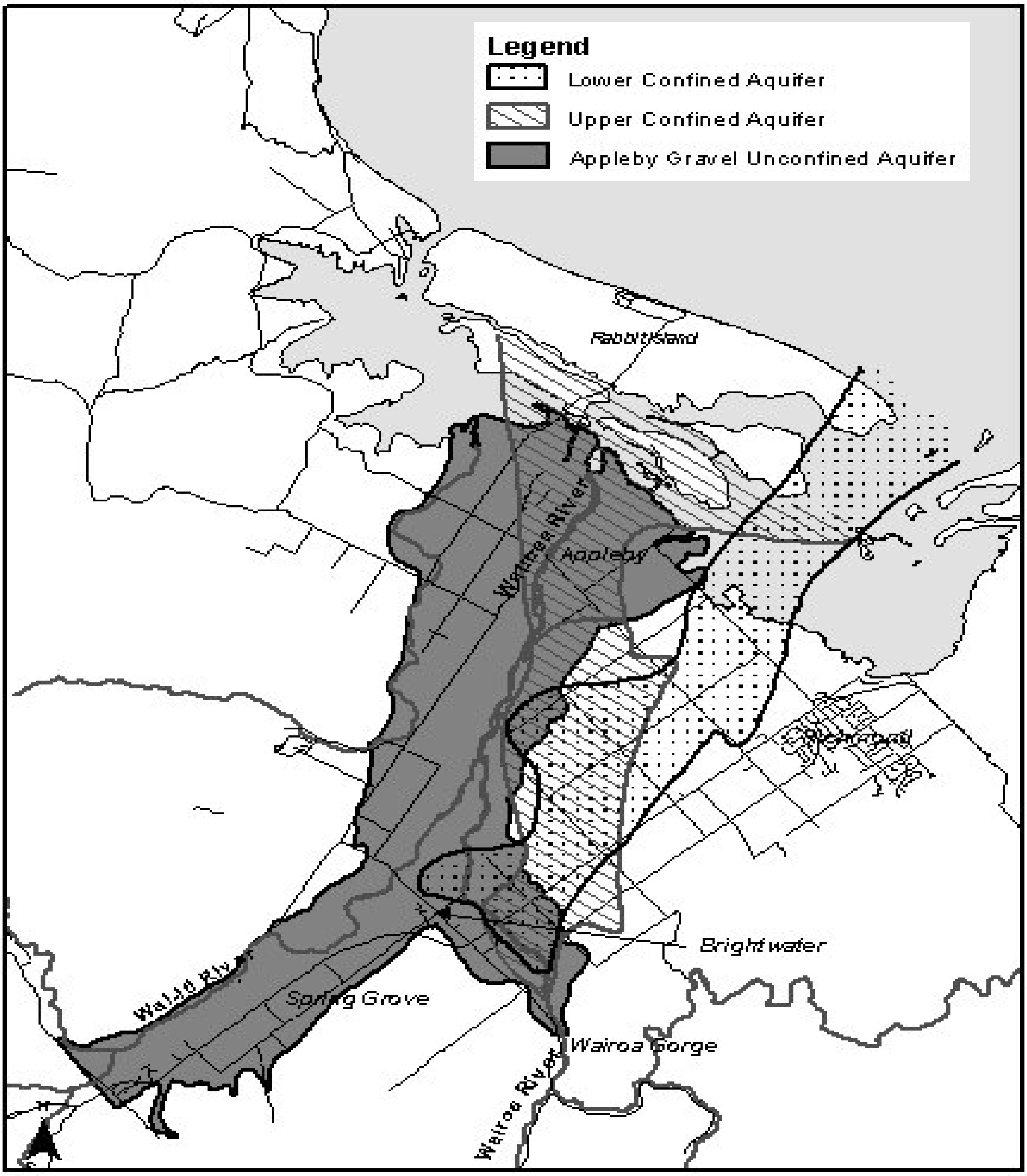

Figure 1. Major aquifers of the Waimea Plains.

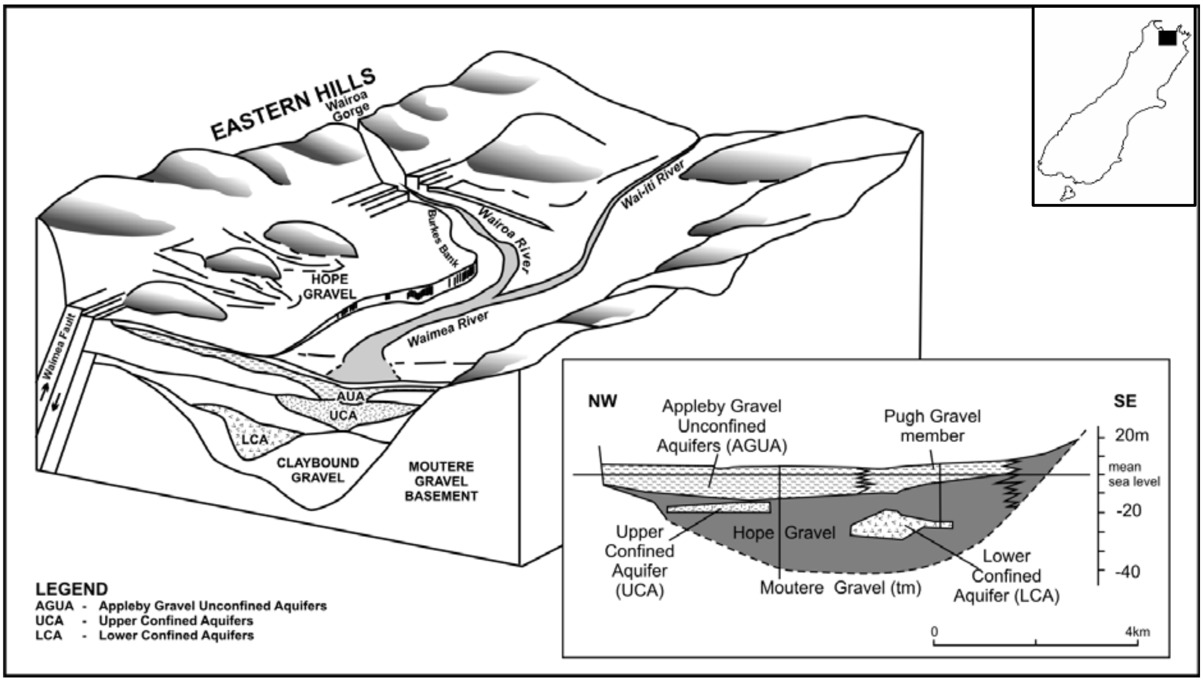

Figure 2. Three-dimensional hydrogeology of the Waimea Plains.

3.3 Upper Confined Aquifer (UCA)

The Upper Confined Aquifer (UCA) consists of clean river gravel deposited within the clay-bound Hope Gravel and accumulated on a degradation surface in the valleys of the Wairoa and Waimea Rivers.

The UCA extends from its recharge zone near the Wairoa Gorge towards the coast at Rabbit Island in the depth range from 18-32 m. The upper confining layer is ruptured within the recharge zone and also from Appleby northwards, providing a hydraulic connection with the overlying AGUA. Transmissivity values for the UCA range from 600 to 1300 m^2/day. Highest yields in the UCA are obtained along the western edges of Burkes Bank. Recharge occurs from the Wairoa River via the Appleby Gravel and in winter from the Hope Aquifers via the gravel fans. The latter source is confirmed by the high nitrate levels measured in the UCA and by the flow directions derived from a winter piezometric survey.

3.4 Lower Confined Aquifer (LCA)

The Lower Confined Aquifer (LCA) is lithologically similar to the Upper Confined Aquifer. It extends from the Wairoa Gorge to beyond the entrance of the Waimea Inlet east of Rabbit Island. The LCA is from 30-50 m deep and is recharged near the Wairoa Gorge. Recharge occurs in winter from the gravel fans, which recharge the UCA from the eastern hills. Seawater intrusion is a potential concern in this aquifer because large pumping wells are near the coast, the aquifer extends under the Waimea inlet, and the nature of the seaward contact of the aquifer is unclear. Pump testing shows a transmissivity range of between 200–1,600 m^2/day.

4.0 WATER MANAGEMENT ZONES IN THE WAIMEA PLAINS

Groundwater resources occur in the Appleby Gravel Unconfined Aquifer (AGUA), Upper Confined Aquifer (UCA) and Lower Confined Aquifer (LCA) under the Waimea Plains and aquifers within the river terraces associated with Wai-iti River. Groundwater recharge is predominantly by surface water losses to groundwater (Waimea/Wairoa/Wai-iti Rivers) and direct rainfall recharge from infiltration of precipitation. The area in the Waimea Plains is predominantly irrigated by groundwater taken from the AGUA, UCA, and LCA and, therefore, irrigation drainage also contributes to aquifer recharge.

The water management zones and hydrological monitoring sites (Table 1) for both surface water and groundwater on the Waimea Plains are shown in Figure 3. Water allocation limits (Table 2) in the Waimea Plains are quoted in the proposed Tasman Resource Management Plan (TRMP 2001). Subsequently, following the 2000/2001 drought, TDC has removed allocation limits as an interim measure and all new permits have non-complying status. These allocations exclude dams and include the water take for the Waimea East Irrigation Scheme (WEIS). The water take for the WEIS is a surface water take from the Wairoa River and is included in the Reservoir Zone. Water allocation limits for each management zone are based on the principle that up to a 35% reduction in the water availability can be expected during a 1 in 10 year drought (pers. comm., Joseph Thomas, TDC).

Under the TRMP, an allocation limit of 2,699 L/sec was set for summer abstraction from either surface or groundwater on the Waimea Plains (November through April inclusive). A minimum flow in the Waimea River of 500 L/sec at the Nursery-Appleby Bridge was proposed for the summer months (November to April inclusive) and 1,000 L/sec for the winter months (May to October inclusive). Most allocation (85%) was from the AGUA with a further 6 % (147 L/sec) from the UCA and 9% (230 L/sec) from the LCA. Under this WWAP, allocation limits and minimum flows will differ from those of the TRMP and, therefore, TRMP revision will be appropriate.

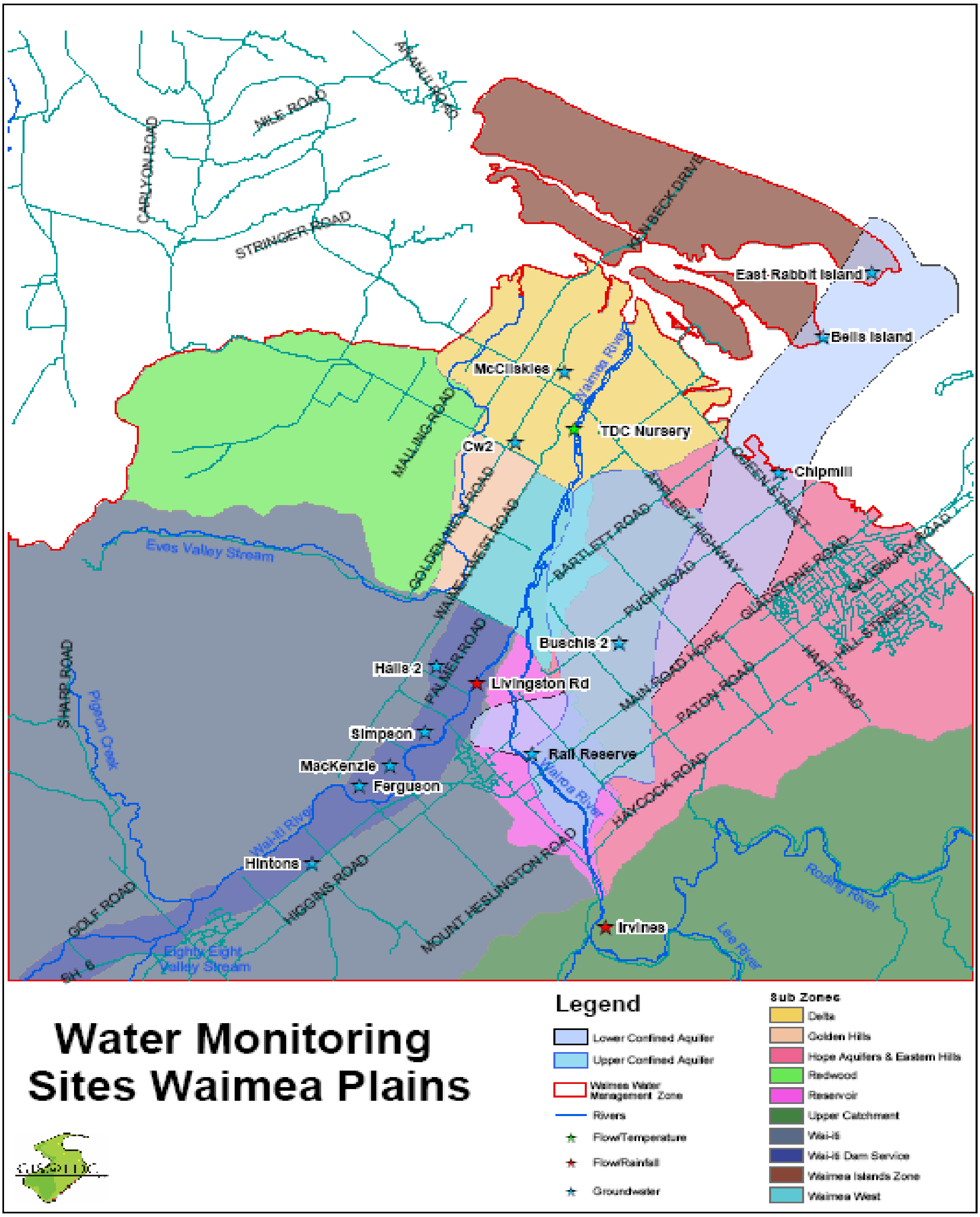

Figure 3. Water management zones and hydrological monitoring sites in the Waimea Plains (Hong, 2005).

Table 1. Hydrological monitoring sites in the Waimea Plains.

Site number	Area	Site Name	Grid reference	NZMG Easting	NZMG Northing
Groundwater Level					
1330108	Redwood	Redwood Lane	N27:1654-8942	2516541	5989419
1330127	Wai-iti	Simpson	N27:1786-8202	2517861	5982024
1330128	Wai-iti	MacKenzie	N27:1721-8138	2517213	5981376
1330129	Wai-iti	Ferguson	N27:1667-8096	2516666	5980960
1330247	Waimea	Hintons	N28:1580-7940	2515800	5979400
1330375	Waimea	Halls 2	N27:1807-8334	2518071	5983344
1331014	Waimea	Bells Island	N27:2510-8990	2525100	5989900
1331069	Waimea	McCliskies	N27:2040-8923	2520400	5989230
1331098	Waimea	CW2	N27:1950-8780	2519500	5987800
1331105	Waimea	Rail Reserve	N27:1980-8160	2519800	5981600
1331119	Waimea	Chipmill	N27:2430-8720	2524300	5987200
1331238	Waimea	Buschls 2	N27:2140-8380	2521400	5983800
1331255	Waimea	East Rabbit Island	N27:2600-9120	2526000	5991200
River Flow					
57517	Wai-iti	Belgrove	N28:0650-7260	2506500	5972600
57520	Wai-iti	Livingston Rd	N27:1880-8300	2518800	5983000
57521	Wairoa	Irvines	N28:2160-7820	2521600	5978200
57523	Waimea	TDC Nursery	N27:2057-8808	2520573	5988085
57524	Roding	Caretakers	O27:3182-8329	2331819	5983289

Table 2. Water allocation limits set by TDC for water management zones in the Waimea Plains (Hong, 2006).

Water Management Zones	Current allocation limits (L/sec)
Waimea Zones	
Wai-iti	105
Reservoir	826
Upper Catchments (Wairoa, Lee and Roding Rivers)	3
Waimea West	178
Hope and Eastern Hills	97
Golden Hills	113
Delta	1000 (subject to condition)
Upper Confined Aquifer	147
Lower Confined Aquifer	230
Total	2699

5.0 UPDATED GROUNDWATER-RIVER INTERACTION MODEL OF THE WAIMEA PLAINS

The groundwater-river interaction model of the Waimea Plains includes the following assumptions (Hong, 2000):

1. A three-layered aquifer system consisting of the Appleby Gravel Unconfined Aquifer (AGUA), Upper Confined Aquifer (UCA), and the Lower Confined Aquifer (LCA).
2. The Waimea groundwater flow model with a uniform grid of 210 m x 225 m cells in the horizontal plane (65 rows by 60 columns), see Figure 4, Figure 5, and Figure 6.
3. Boundaries of the Waimea Plains aquifer system consisting of constant head cells and general head cells.
4. Major water sources in the model being precipitation and surface water recharge (Lee/Wairoa River, Waimea River, and Wai-iti River) inflow.

The U.S. Geological Survey's (USGS's) numerical model for simulating groundwater flow, MODFLOW-96, implemented using the Groundwater Modelling System (GMS) pre- and post-processor, is used to represent the conceptual model of the Waimea Plains. The following MODFLOW packages were used to represent the Waimea Plains groundwater-river interaction model:

- Basic package
- Block-Centered Flow package
- Well package
- General Head Boundary package
- Drain package
- Recharge package
- Stream package
- Slice Successive Overrelaxation package
- Output package

The groundwater-river interaction model of the Waimea Plains simulates flow in the AGUA (1st layer), the UCA (2nd layer), and the LCA (3rd layer). The model grid for the AGUA (1st layer) is shown in Figure 4 with a topographic map. Figures 5 and 6 show the model structure of the UCA (2nd layer) and the LCA (3rd layer), respectively. Figure 3 shows water management zones and hydrological monitoring sites (Table 2) for surface water and groundwater on the Waimea Plains.

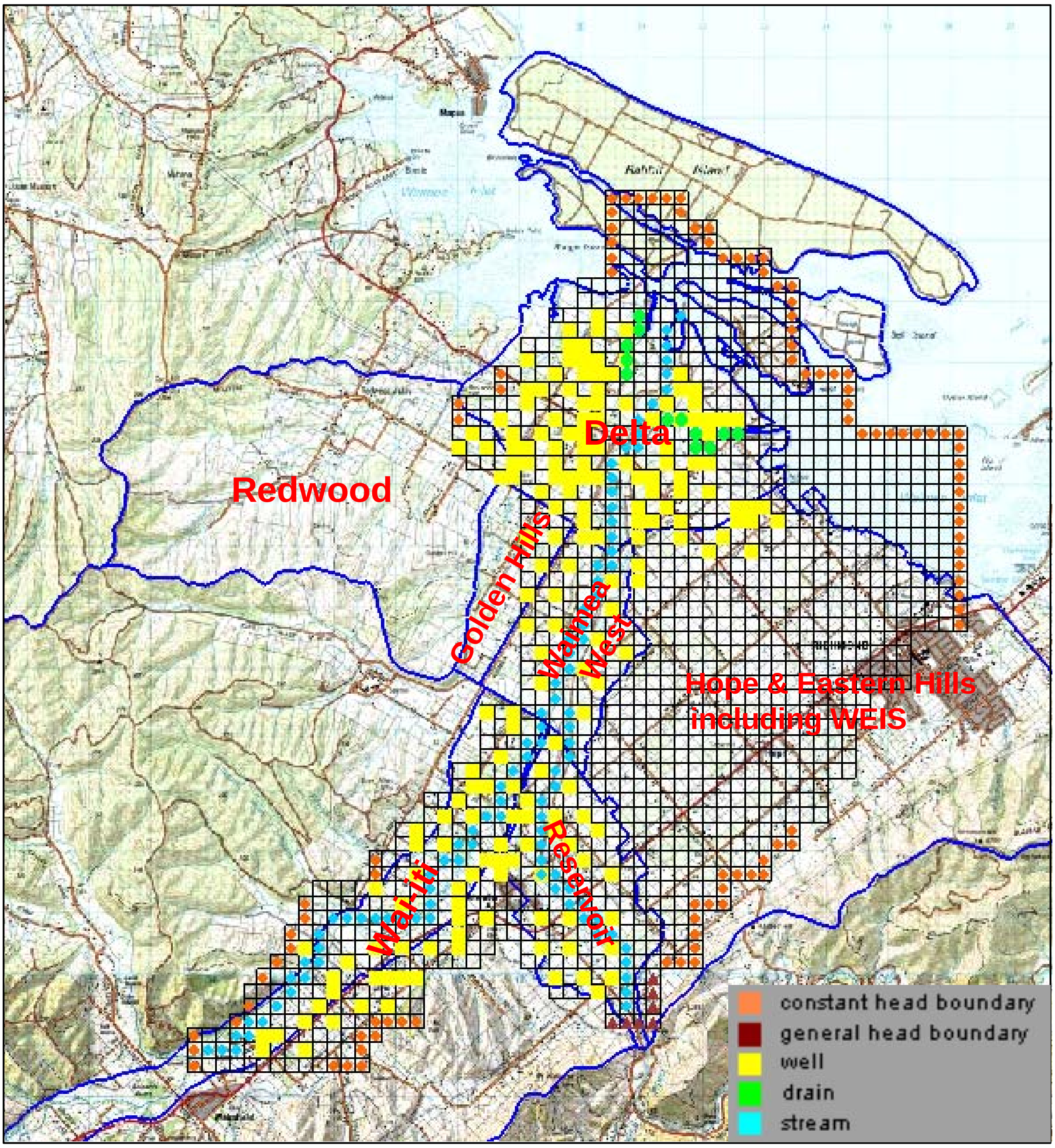

Figure 4. Water management zones for the AGUA and Hope Minor Confined and Unconfined Aquifer (HU) and top layer of groundwater flow model.

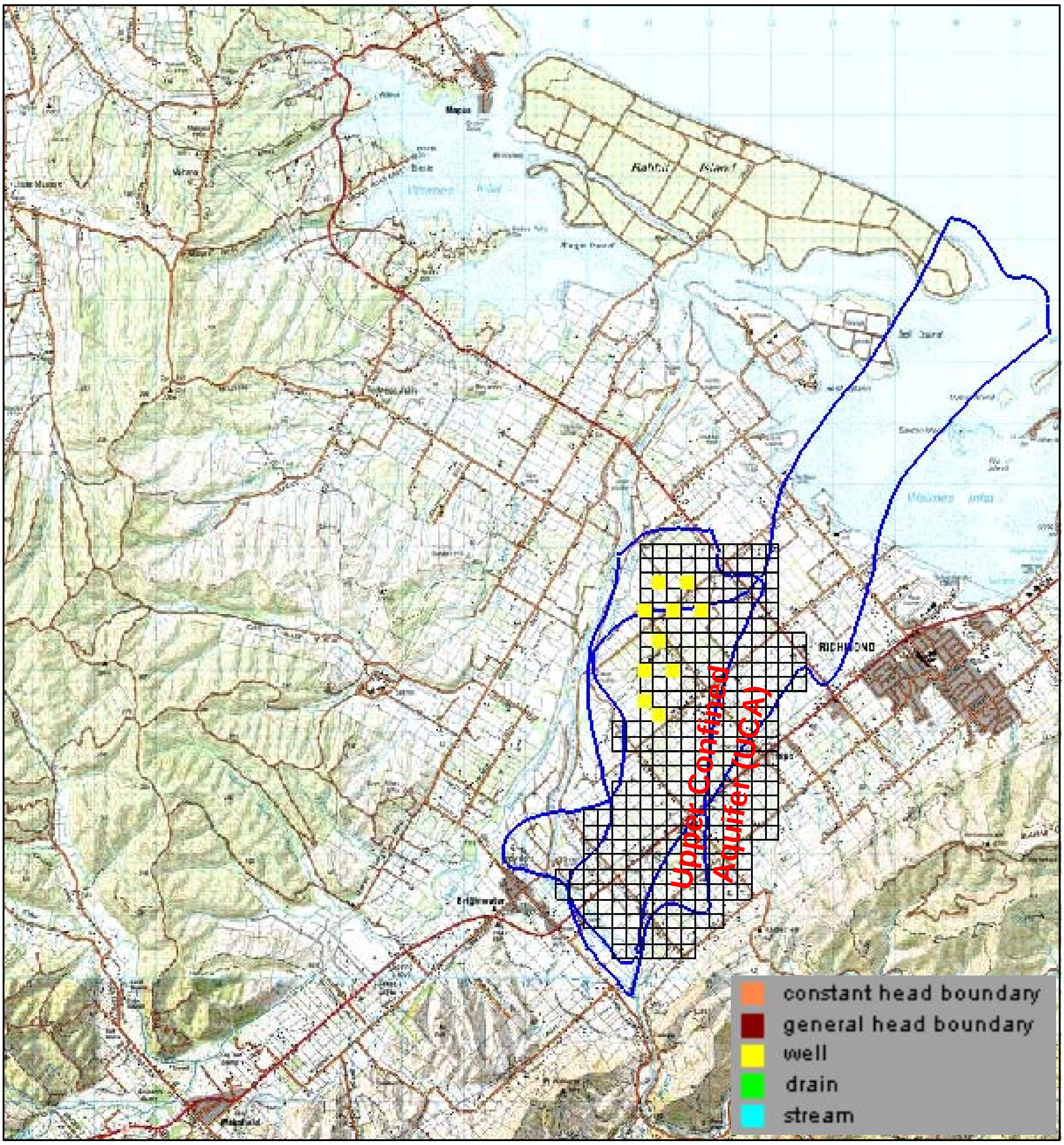

Figure 5. UCA water management zone and second layer of groundwater flow model.

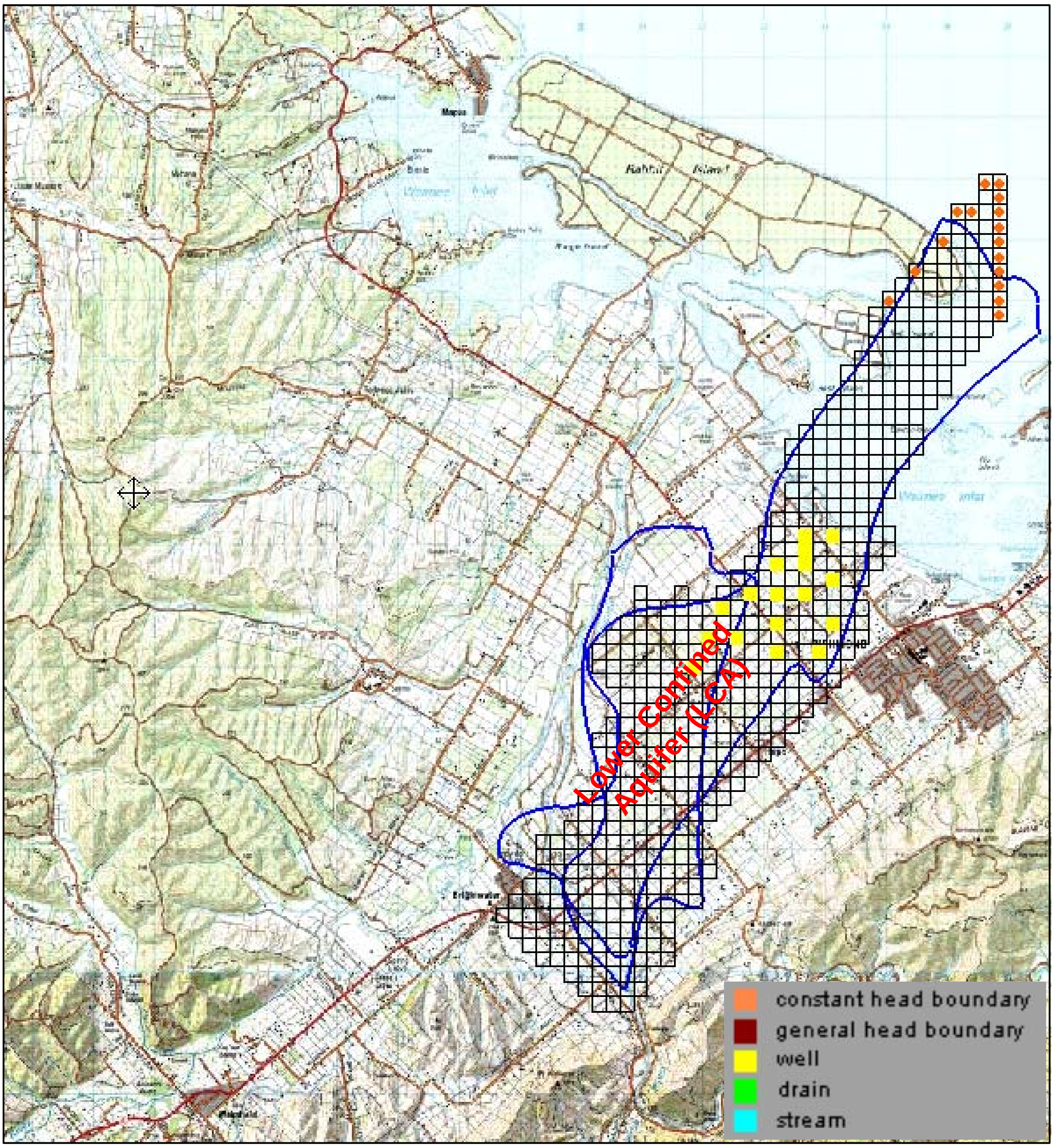

Figure 6. LCA water management zone and third layer of groundwater flow model.

5.1 Groundwater-river interaction model in the Waimea Plains

The rivers are a main recharge source for the unconfined aquifer and indirectly for the two confined aquifers in the Waimea Plains. Because the groundwater system is directly linked to the river, groundwater abstractions, particularly during summer, reduce river flows. Therefore, developing a well-calibrated transient groundwater-river interaction model is vital to assess water allocation in the Waimea Plains. The Wai-iti River is frequently dry in its lower reaches in summer. The historical data show consistent flow losses from the Wairoa River Gorge to the downstream area of the Waimea River at the Nursery-Appleby Bridge due to groundwater abstraction during summer. Figure 7 shows the strong interaction between river and the shallow unconfined aquifer in the Waimea Plains in a severe dry year

(2000/2001). In a severe dry year with actual groundwater usage (2000/2001), it has been observed that river flow loss from the Wairoa River and Waimea River is approximately 1,300 L/sec (in the period of February 2001 to April 2001) (Figure 7).

The MODFLOW Stream package (or the stream-routing package) is used to model the interaction between aquifers and rivers in the Waimea Plains. The Stream package is a combination of a known flux and head-dependent flux boundary. It is similar to the MODFLOW River package since it allows flow into and from a stream. However, it is more sophisticated than the River package because it considers the flow rate in the stream and limits the leakage between the aquifer and the stream accordingly. This package increases stream flow in areas of gaining stream segments (reaches) and reduces flow by taking water out through riverbed seepage in losing reaches (Prudic, 1989 and Hong, 2000). This package also calculates the stream stage needed by other MODFLOW modules by using a set of required data whereas the River package uses a pre-assigned stage value. Because of its versatility, the Stream package requires intensive preparation and more input parameters than the River package (Hong, 2000).

GNS studies (Hong, 2000; Hong 2003, Hong 2006) in collaboration with TDC have used the Stream package to simulate observed groundwater levels and river flows more accurately than the River package. Recently, the Stream package of the Waimea Plains groundwater-river interaction model has been updated with 2005 (Wairoa and Waimea) and 2007 (Wai-iti) river survey data. An intensive field river survey of up to 38 cross-sections in the Wairoa, Wai-iti, and Waimea Rivers was undertaken in 2005 and 2007 to generate the data required for the Stream package. Figure 8 shows the location of 37 cross-sections surveyed in the Wairoa, Wai-iti, and Waimea Rivers. Figure 9 displays selected river cross-sections for the Wairoa and Waimea Rivers based on 2005 survey data. Selected river cross-sections for the Wai-iti River based on 2007 survey data are also shown in Figure 10.

Tables 3 and 4 summarize river cross-section width and minimum streambed elevation information for the Wairoa and Waimea Rivers based on 2005 and 1997 survey data, respectively. A similar summary is presented in Table 5 for the Wai-iti River based on 2007 survey data. River stage has been recorded at the Wairoa River Gorge since 1957 and for the Wai-iti River since 1976 and these gaugings have allowed the derivation of stage-flow ratings. River flows at the Irvine-Wairoa Gorge and for the Wai-iti River are major inputs to the Stream package.

Streambed elevations measured in 1984 were used for the 1982/1983 year simulation. Streambed elevations measured in the 2005 survey for the Wairoa River Gorge and Waimea River and streambed elevations measured in 2007 for the Wai-iti River were used for the 2000/2001 year and 2004/2005 year simulations. Figure 11 shows the 1997 and 2005 surveyed data of the river cross-section and river bed levels in the Wairoa River and Waimea River. Survey data indicate streambed elevations decreased on average 0.21 m between 1997 and 2005 in the Wairoa River while increasing on average 0.16 m during the same period in the Waimea River (Tables 3 and 4 and Figure 11). 2007 survey data of river cross-section and minimum streambed levels for the Wai-iti River are shown in Figure 12.

Streambed conductance values were estimated to be in the range of 370,000-40,000 m^2/day for the Wairoa River and 42,000-11,000 m^2/day for the Waimea River. Manning's roughness coefficient was estimated by NIWA from a description of the physical characteristics of the streambed (Table 6). Stream channel slope was estimated from streambed elevation measurements (Table 6).

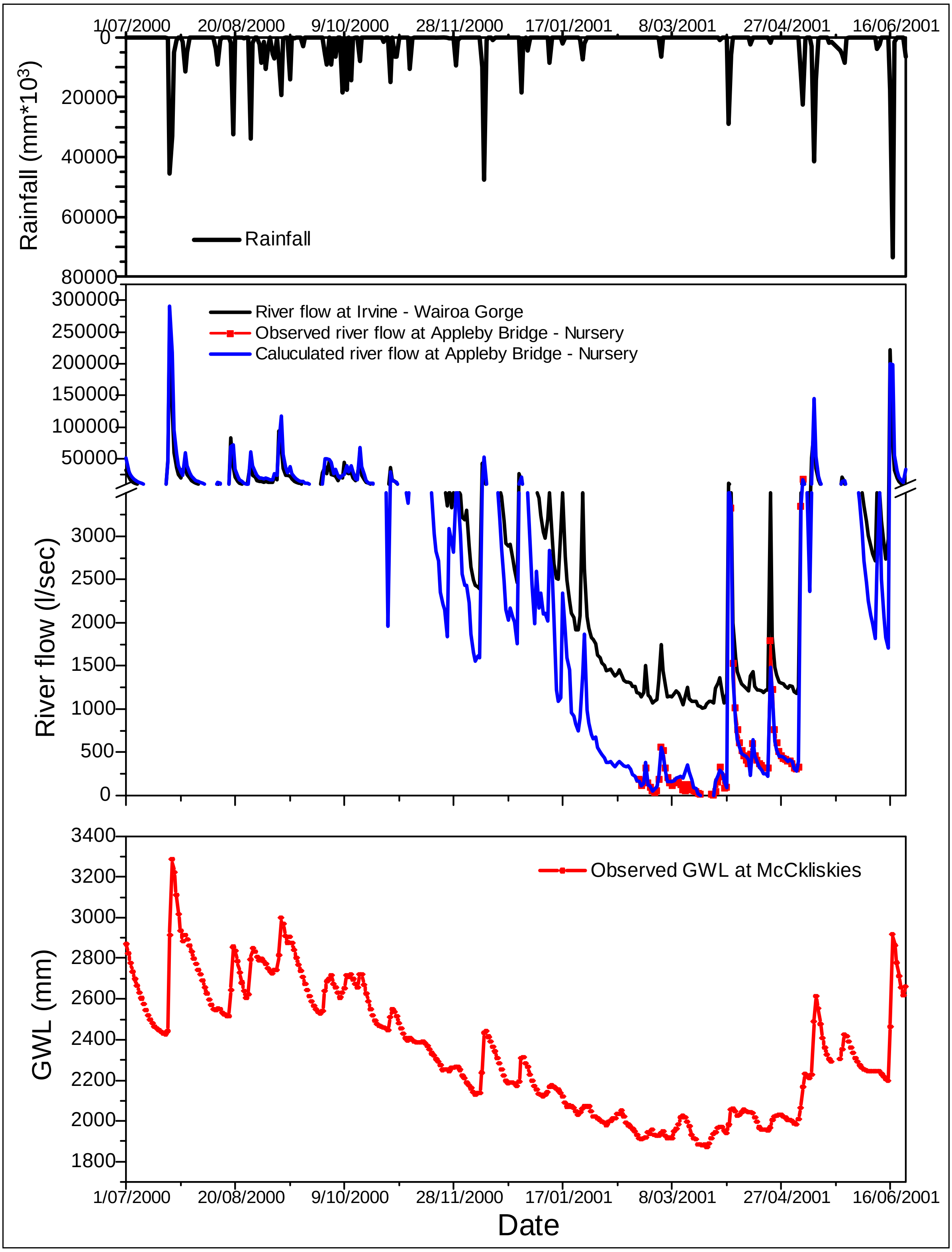

Figure 7. Observed and calculated river flows after model calibration for the Waimea River at Nursery-Appleby Bridge in the period from 01-01-2001 to 30-04-2001.

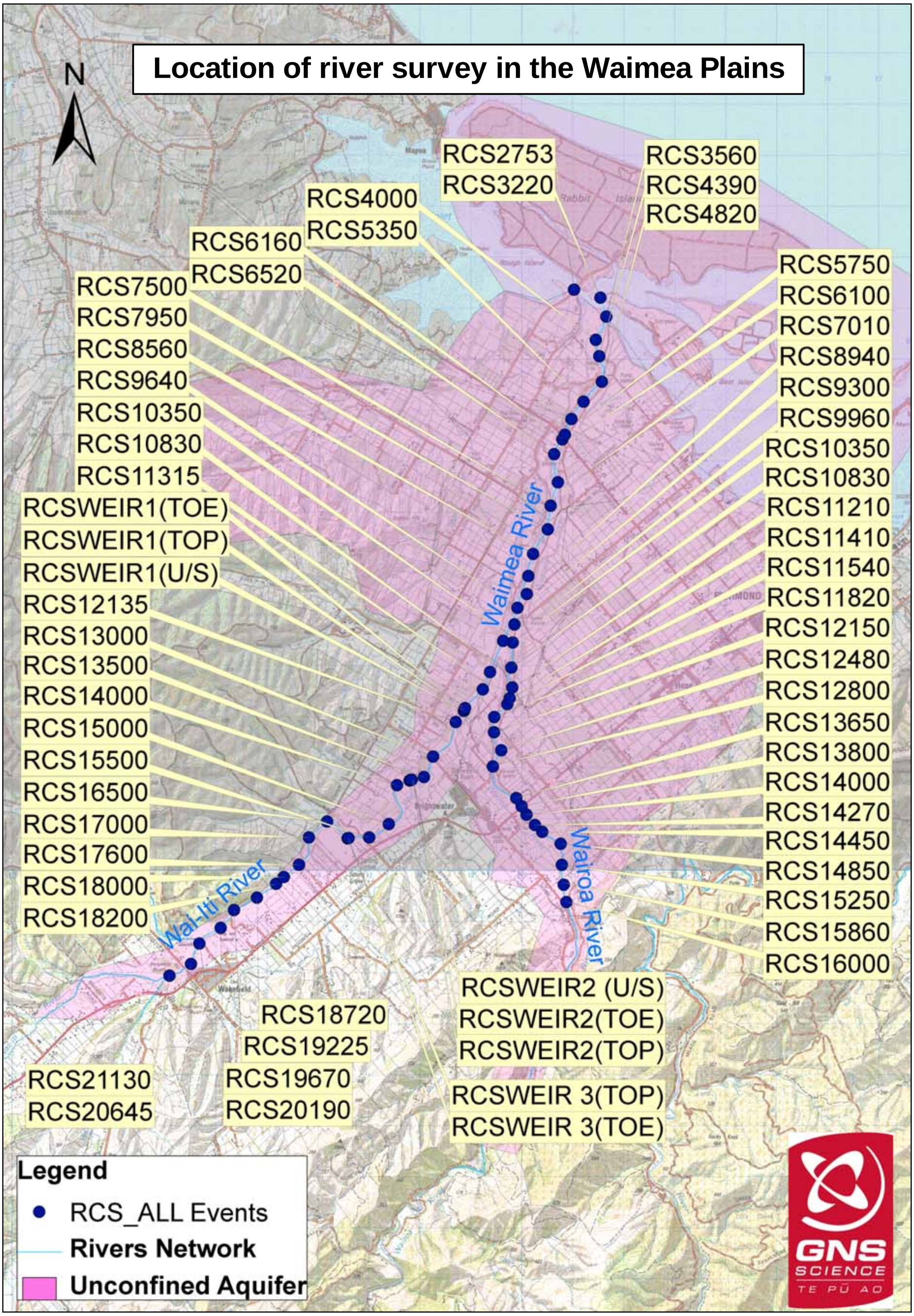

Figure 8. Locations of river cross-sections surveyed in the Wairoa, Wai-iti, and Waimea Rivers.

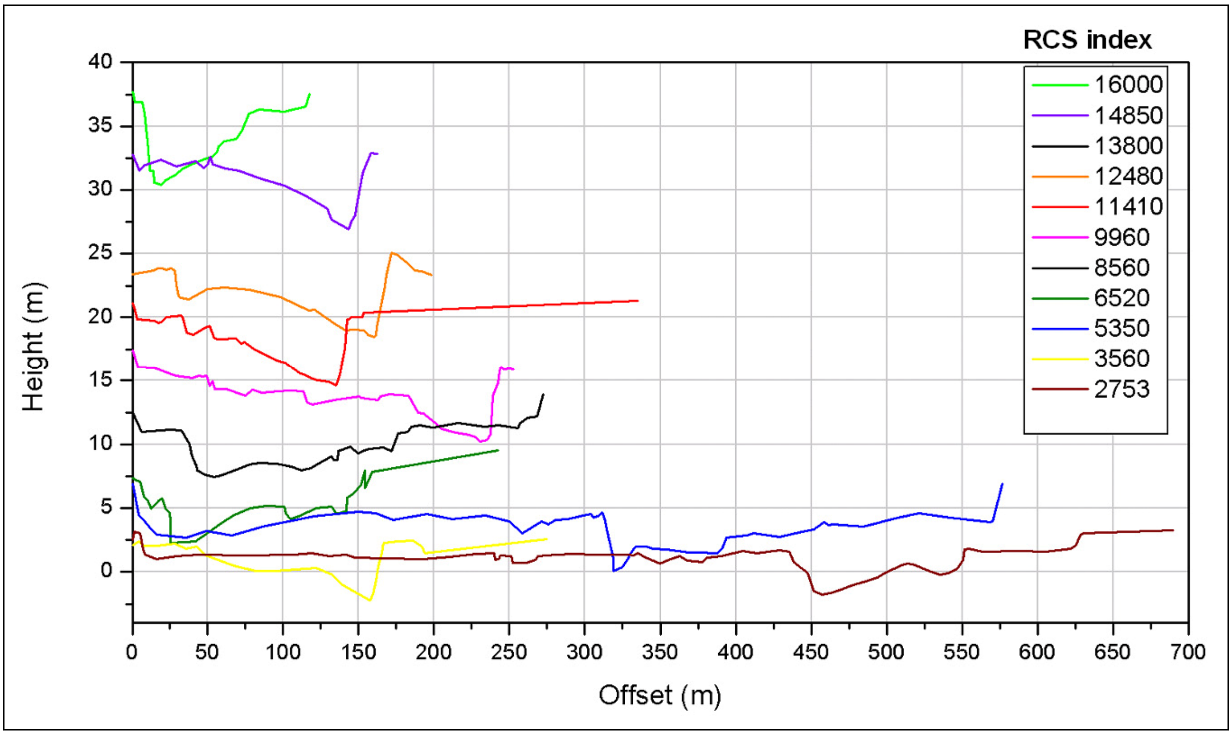

Figure 9. Selected Wairoa and Waimea Rivers cross-sections (2005 survey data).

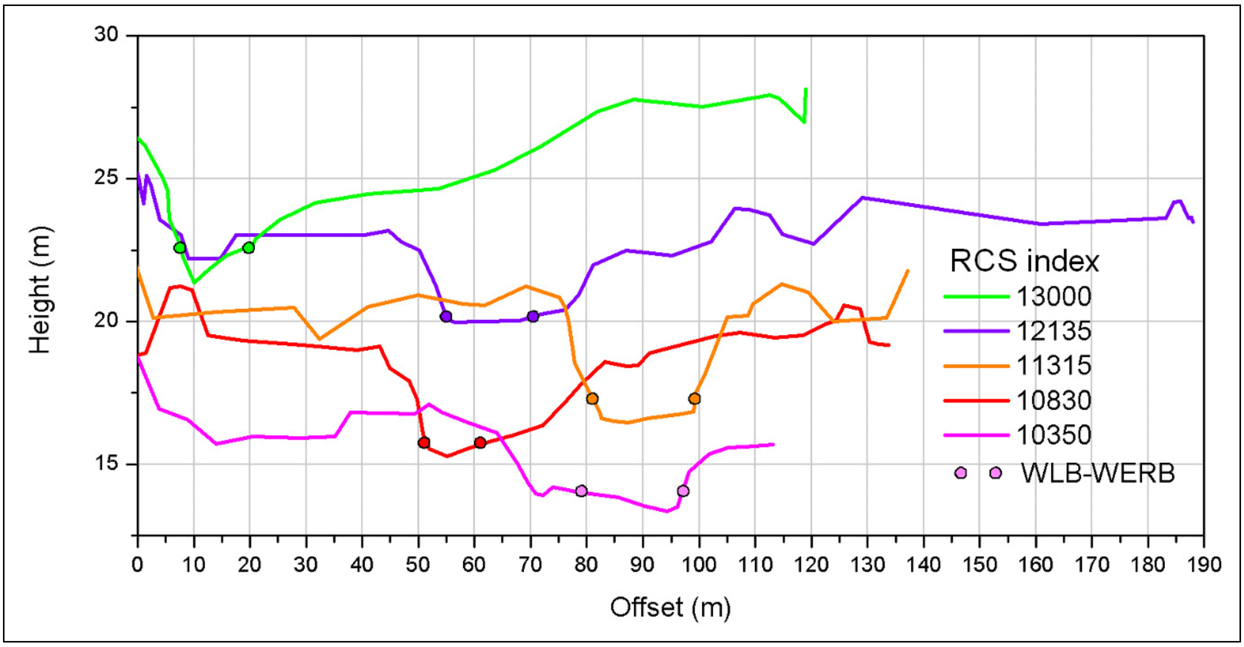

Figure 10. Selected Wai-iti Rivercross-sections (2007 survey data).

Table 3. Wairoa and Waimea Rivers cross section information (2005 survey data).

Wairoa and Waimea Rivers survey data (2005)				
River name	**RCS No.**	**CS ID**	**Minimum river bed levels (m, above mean sea level)**	**River cross section width (m)**
Wairoa River	16000	1	30.399	21.835
Wairoa River	15680	2	30.819	48.822
Wairoa River	15250	3	30.316	37.986
Wairoa River	14850	4	27.493	19.404
Wairoa River	14450	5	25.773	18.803
Wairoa River	14270	6	25.29	31.022
Wairoa River	14000	7	25.699	29.604
Wairoa River	13800	8	24.236	29.643
Wairoa River	13650	9	21.992	9.992
Wairoa River	13250	10	21.288	34.37
Wairoa River	12800	11	20.611	26.508
Wairoa River	12480	12	18.438	37.8
Wairoa River	12150	13	18.634	16.388
Wairoa River	11820	14	15.146	13.931
Wairoa River	11540	15	14.582	14.999
Wairoa River	11410	16	14.617	16.271
Wairoa River	11210	17	14.397	35.779
Wairoa River	10830	18	14.689	10.489
Confluence	10350	19	10.564	37.527
Waimea River	9960	20	10.216	47.503
Waimea River	9640	21	11.672	47.503
Waimea River	9300	22	11.351	29.857
Waimea River	8940	23	10.188	24.13
Waimea River	8560	24	7.456	19.789
Waimea River	7950	25	5.748	32.117
Waimea River	7500	26	4.21	42.939
Waimea River	7010	27	3.703	34.641
Waimea River	6520	28	2.213	38.418
Waimea River	6160	29	2.228	43.004
Waimea River	6100	30	2.325	31.662
Waimea River	5750	31	1.708	36.141
Waimea River	5350	32	1.491	31.141
Waimea River	4820	33	-0.379	43.387
Waimea River	4390	34	-0.512	57.235
Waimea River	4000	35	-1.900	95.491
Waimea River	3560	36	-2.244	95.491
Waimea River	3220	37	-1.175	62.662
Waimea River	2753	38	-1.804	51.18

Table 4. Wairoa and Waimea Rivers cross section information (1997 survey data).

Wairoa and Waimea Rivers survey data (1997)				
River name	**RCS No.**	**CS ID**	**Minimum river bed levels (m, above mean sea level)**	**River cross section width (m)**
Wairoa River	16000	1	30.94	30.86
Wairoa River	15680	2	30.99	51.26
Wairoa River	15250	3	30.30	43.39
Wairoa River	14850	4	26.68	19.59
Wairoa River	14450	5	25.62	29.46
Wairoa River	14270	6	26.25	32.70
Wairoa River	14000	7	26.01	28.39
Wairoa River	13800	8	24.21	20.83
Wairoa River	13650	9	22.30	12.71
Wairoa River	13250	10	21.49	16.38
Wairoa River	12800	11	21.03	40.25
Wairoa River	12480	12	17.35	16.66
Wairoa River	12150	13	18.38	41.53
Wairoa River	11820	14	15.81	29.90
Wairoa River	11540	15	15.13	31.77
Wairoa River	11410	16	14.75	23.12
Wairoa River	11210	17	14.58	38.71
Wairoa River	10830	18	14.00	20.07
Confluence	10350	19	13.19	39.37
Waimea River	9960	20	11.98	23.68
Waimea River	9640	21	10.49	52.03
Waimea River	9300	22	8.90	19.34
Waimea River	8940	23	8.65	21.53
Waimea River	8560	24	7.57	68.65
Waimea River	7950	25	5.63	35.09
Waimea River	7500	26	5.04	41.19
Waimea River	7010	27	4.27	45.49
Waimea River	6520	28	2.86	20.07
Waimea River	6160	29	1.15	23.27
Waimea River	6100	30	1.84	32.66
Waimea River	5750	31	0.55	34.34
Waimea River	5350	32	0.29	32.23
Waimea River	4820	33	-0.35	34.65
Waimea River	4390	34	-0.22	81.51
Waimea River	4000	35	-3.02	65.04
Waimea River	3560	36	-2.95	95.04
Waimea River	3220	37	-0.76	53.64
Waimea River	2753	38	-1.21	84.84

Table 5. Wai-iti River cross section information (2007 survey data).

Wai-iti River survey data (2007)				
River name	**RCS No.**	**CS ID**	**Minimum river bed levels (m, above mean sea level)**	**River cross section width (m)**
Wai-iti	21130	1	60.94	11.10
Wai-iti	20645	2	58.81	14.07
Wai-iti	20190	3	55.68	9.34
Wai-iti	19670	4	52.51	13.09
Wai-iti	19225	5	49.87	13.27
Wai-iti	18720	6	47.25	5.63
Wai-iti	18200	7	45.50	21.52
Wai-iti	18000	8	44.15	32.50
Wai-iti	17600	9	42.60	25.49
Wai-iti	17000	10	38.59	12.81
Wai-iti	16500	11	37.51	31.31
Wai-iti	16000	12	33.37	23.51
Wai-iti	WEIR 3(TOP)	13	34.55	16.12
Wai-iti	WEIR 3(TOE)	14	33.58	13.09
Wai-iti	15000	15	29.28	10.99
Wai-iti	14000	16	26.03	18.89
Wai-iti	WEIR2 (U/S)	17	25.50	20.34
Wai-iti	WEIR2(TOP)	18	25.20	11.30
Wai-iti	WEIR2(TOE)	19	23.10	15.83
Wai-iti	13500	20	23.71	13.24
Wai-iti	13000	21	21.37	12.78
Wai-iti	12135	22	19.97	17.12
Wai-iti	WEIR1(U/S)	23	19.04	11.80
Wai-iti	WEIR1(TOP)	24	19.09	11.64
Wai-iti	WEIR1(TOE)	25	17.17	16.60
Wai-iti	11315	26	16.45	14.66
Wai-iti	10830	27	15.28	10.54
Wai-iti	10350	28	13.36	18.53

Table 6. Manning's roughness coefficient and stream-channel slope.

River name	Manning's roughness coefficient	Slope
Wairoa	0.025	0.003
Wai-iti, and	0.01	0.0025
Waimea	0.02	0.002

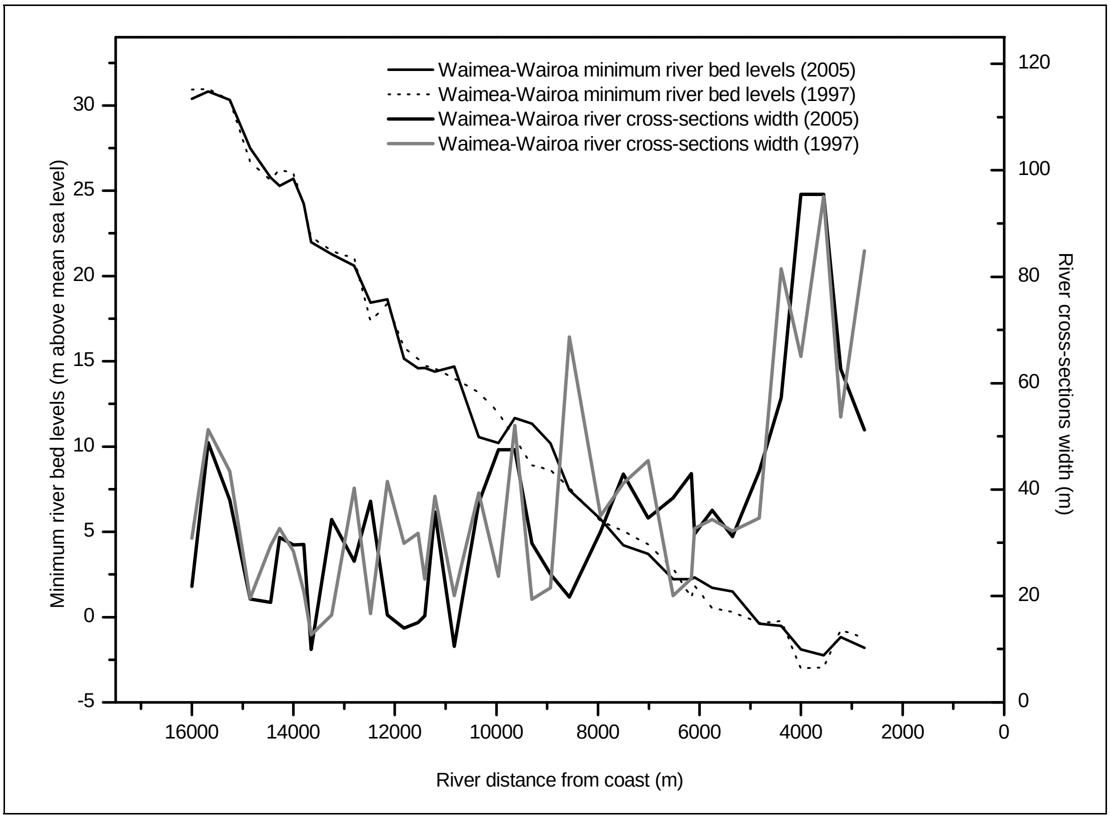

Figure 11. River cross-section width and streambed bed elevation in the Wairoa and Waimea Rivers (1997 and 2005 survey data).

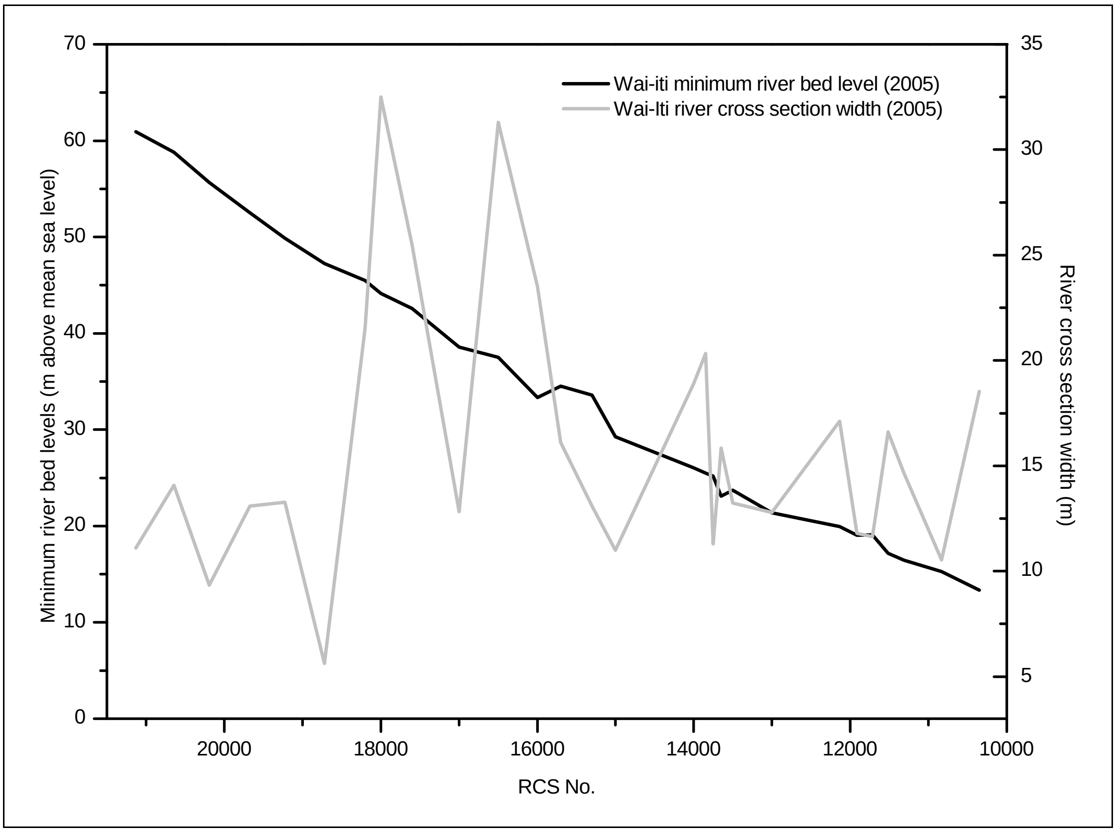

Figure 12. River cross-section width and streambed elevation in the Wai-iti River (2007 survey data).

5.2 Rainfall recharge model in the Waimea Plains

A daily soil water balance model was developed by Landcare Research to calculate the rainfall recharge to the AGUA. Daily observed evapotranspiration at Nelson airport was used. Water-holding capacities for the three main soil groups (38mm, 78mm, and 130mm) in the Waimea Plains were applied. The single main crop type (pasture) was also used in the model. A crop rooting depth of 600 mm was assumed. Figure 13 shows the soil types in the Waimea Plains overlaid on the Waimea Plains groundwater-river interaction model.

For each of the three soil groups, a rainfall recharge model was run on a daily basis for the period of 1st July to 30th June of the following year for two different drought years (1982/1983 and 2000/2001) and the average year condition (2004/2005). The calculated recharge for three soil groups was weighted according to the percentage of each soil group over the AGUA area to give a composite recharge depth for application at every cell in the model. Table 7 shows a statistical summary of the rainfall recharge models developed. For example, the annual seasonal recharge rate (%) for pasture with 38 mm soil moisture holding capacity for the 1982/1983 year for the non-irrigated area was estimated at 35%, compared to 25% with 130 mm soil moisture holding capacity. Figure 14 shows the rainfall recharge estimation by Landcare Research for the 2000/2001 drought year over a range of soil types in non-irrigated area.

Direct calibration of the daily soil water balance model is impossible due to a lack of data from actual measurement of recharge. Nevertheless, the model has been found to be reasonably valid for New Zealand soil and climate conditions by comparing lysimeter measurements in the Canterbury Plains with simulated recharge.

Table 7. Statistical summary of the rainfall recharge models developed.

Seasonal recharge rates (% of rainfall) for irrigated area			
Soil types	1982/1983	2000/2001	2004/2005
38mm	37	41	40
78mm	32	36	33
130mm	29	34	27
Seasonal recharge rates (% of rainfall) for non-irrigated area			
Soil types	1982/1983	2000/2001	2004/2005
38mm	35	38	38
78mm	30	34	29
130mm	25	30	24
Seasonal rainfall			
Rainfall (mm)	664	681	1004

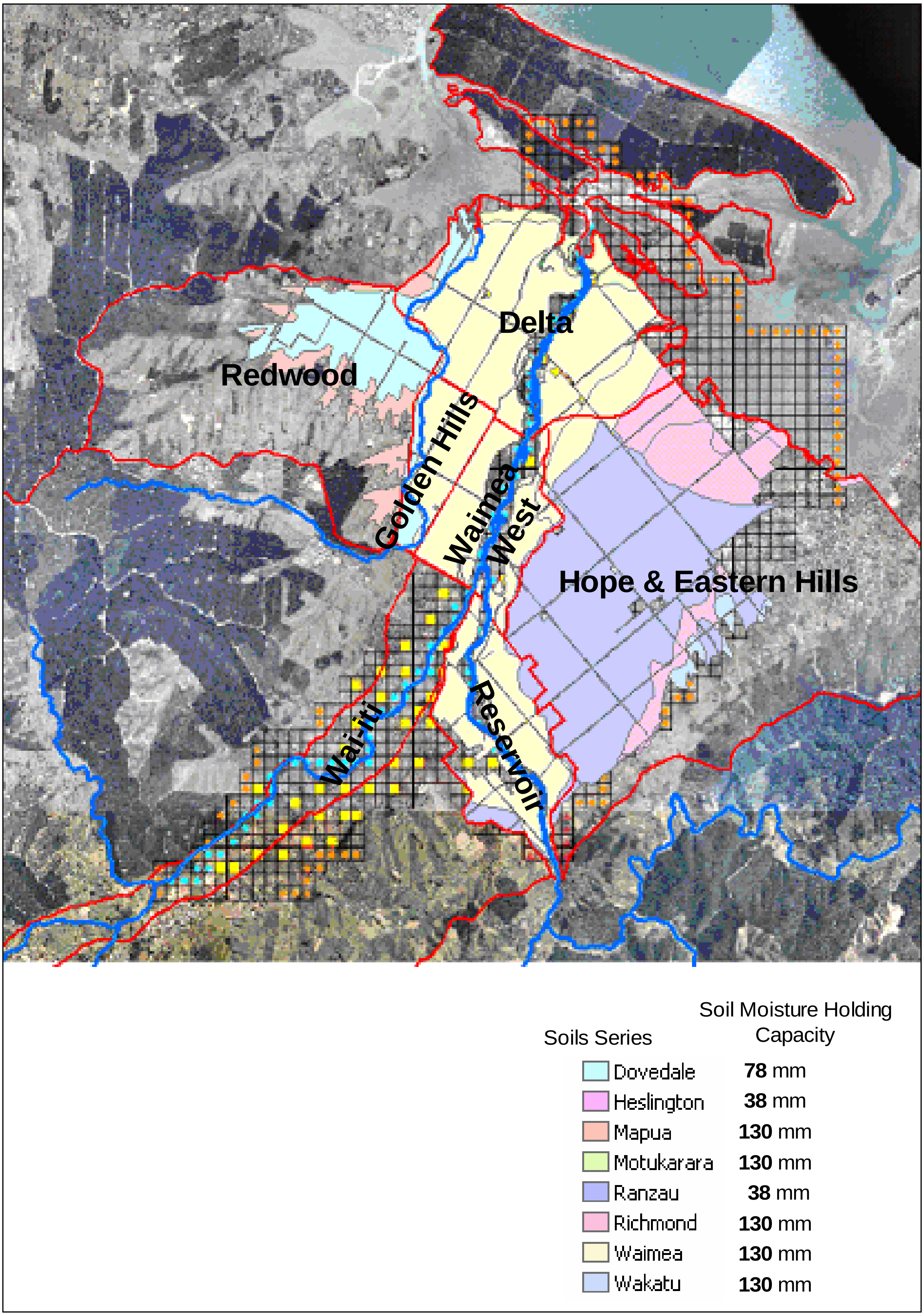

Figure 13. Soil types in the Waimea Plains.

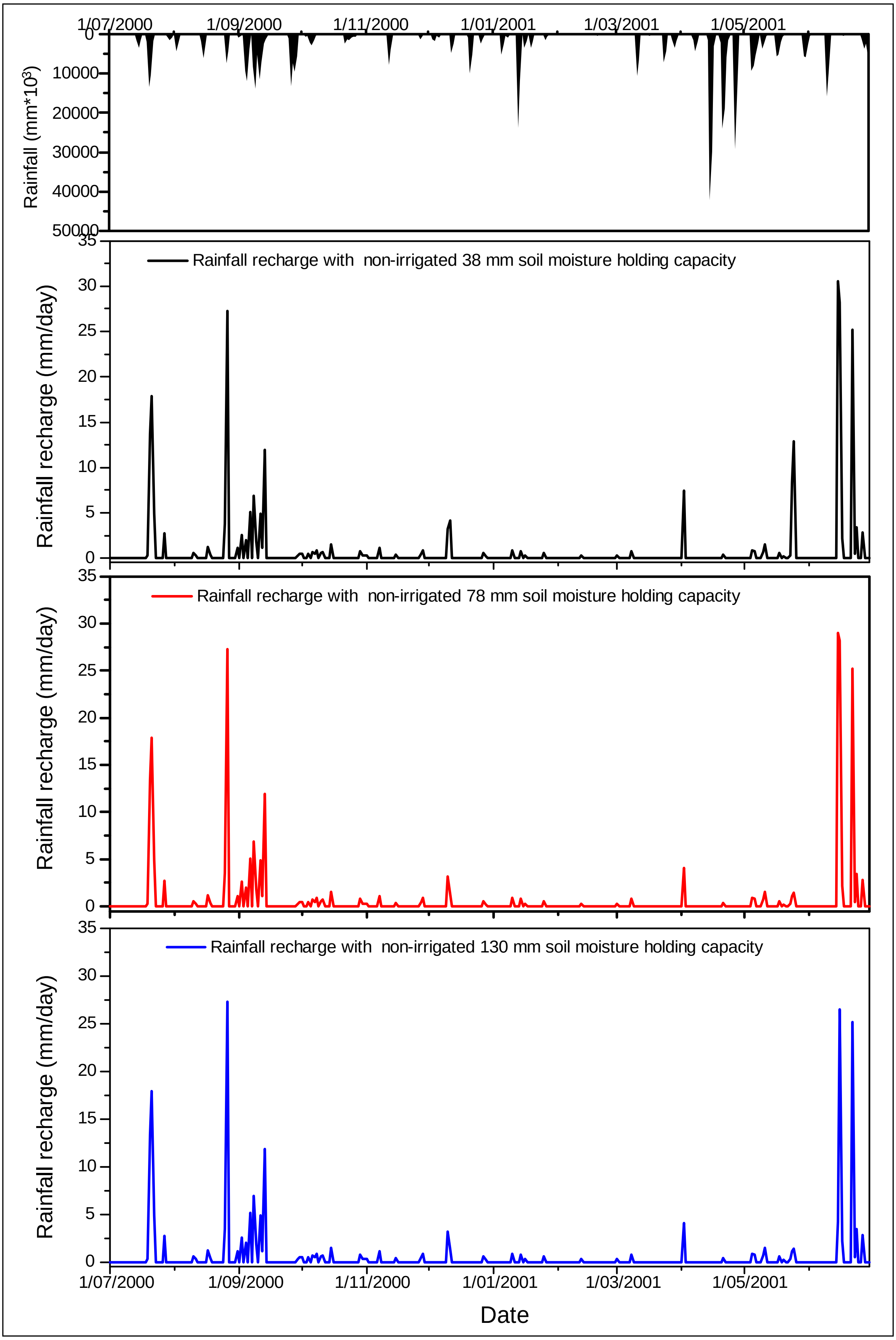

Figure 14. Rainfall recharge estimation for the 2000/2001 year.

5.3 Groundwater abstraction model in the Waimea Plains

The primary function of the Waimea Plains groundwater-river interaction model was to assess the effect of groundwater abstraction on river flows and groundwater levels. To do this, actual abstraction must be measured accurately.

Water metering at pumping wells started in 1979, and by the 1990s full metering of water usage was achieved. Since 1979, the water management authority (TDC) has required all water users to install water meters at every pumping well. Weekly meter readings are required. A total of 335 pumping wells in the Waimea Plains have been metered with an automatic metering system. The split for these is 128 wells in the Delta zone, 89 wells in the Wai-iti zone, 29 wells in the UCA, and 24 wells in the LCA. This historical groundwater abstraction monitoring gives an indication of water usage for various crops and climate conditions and allows an estimate of pumpage from all water management zones in the Waimea Plains. Groundwater abstraction data is most accurate in the irrigation season, especially during dry summers.

The largest amount of groundwater abstraction data (75-84%) comes from the AGUA and mainly from the Delta zone (40-50%) with around 5-10% coming from the UCA and 10-16% from the LCA. The current water allocation limit of 2,699 L/sec (Table 2) has been set for summer (November to April inclusive) abstraction, including direct surface water pumping on the Waimea Plains. Daily groundwater abstraction models for the existing demand scenario were developed based on actual water usage data using the Well package. Each well was allocated to the nearest node in the model grid and well pumpage was given as daily pumpage at every node in the model. In the Delta zone, some wells are very close to each other and, in this case, the pumpage of the wells located in the same grid cells was summed up as one well. Daily peak water usage was modelled at 2,166 L/sec including 402 L/sec of direct surface water pumpage for the 2000/2001 year, 2,008 L/sec for the 1982/1983 year, 1,521 L/sec (including 308 L/sec of direct surface water pumping) for the 1 in 10 drought year (1991/1992), and 1,430 L/sec (including 339 L/sec of direct surface water pumping) for the average year (2004/2005 year). Figures 14 and 15 show total actual groundwater abstraction in the Waimea Plains for the 2004/2005 year (average year) and 2000/2001 drought year, respectively.

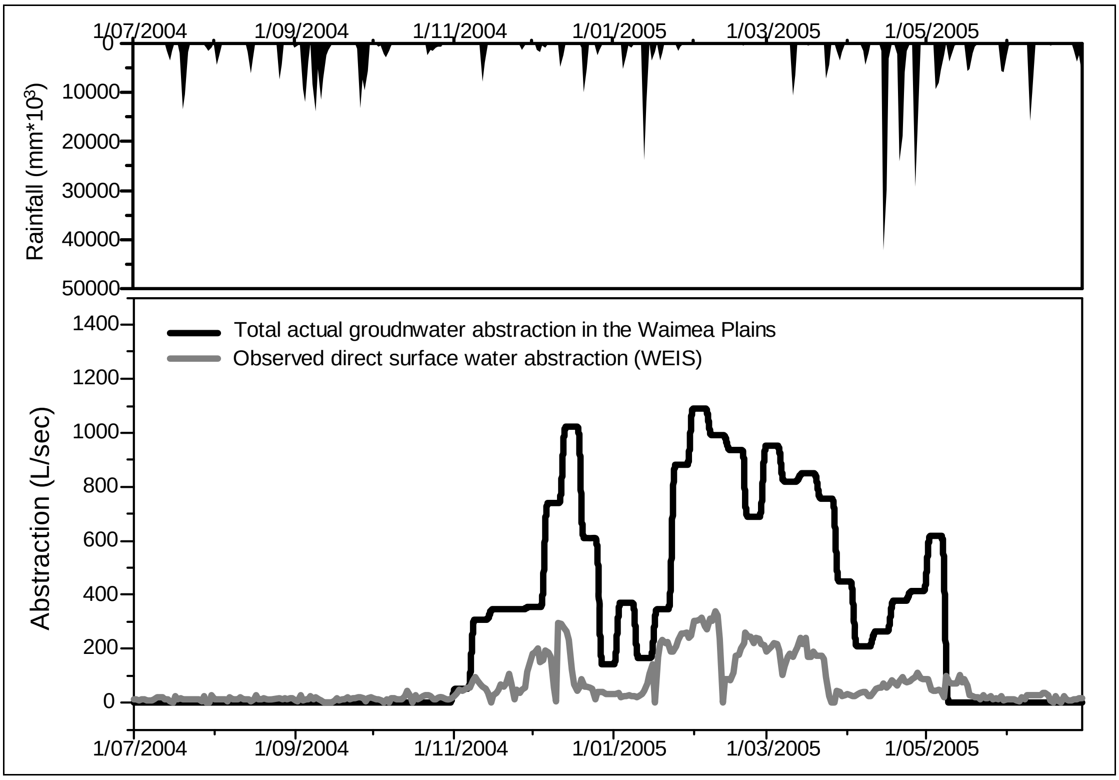

Figure 15. Total actual groundwater abstraction and observed direct surface water abstraction in the Waimea Plains in the period of 1 July 2004 to 30 June 2005 (average year).

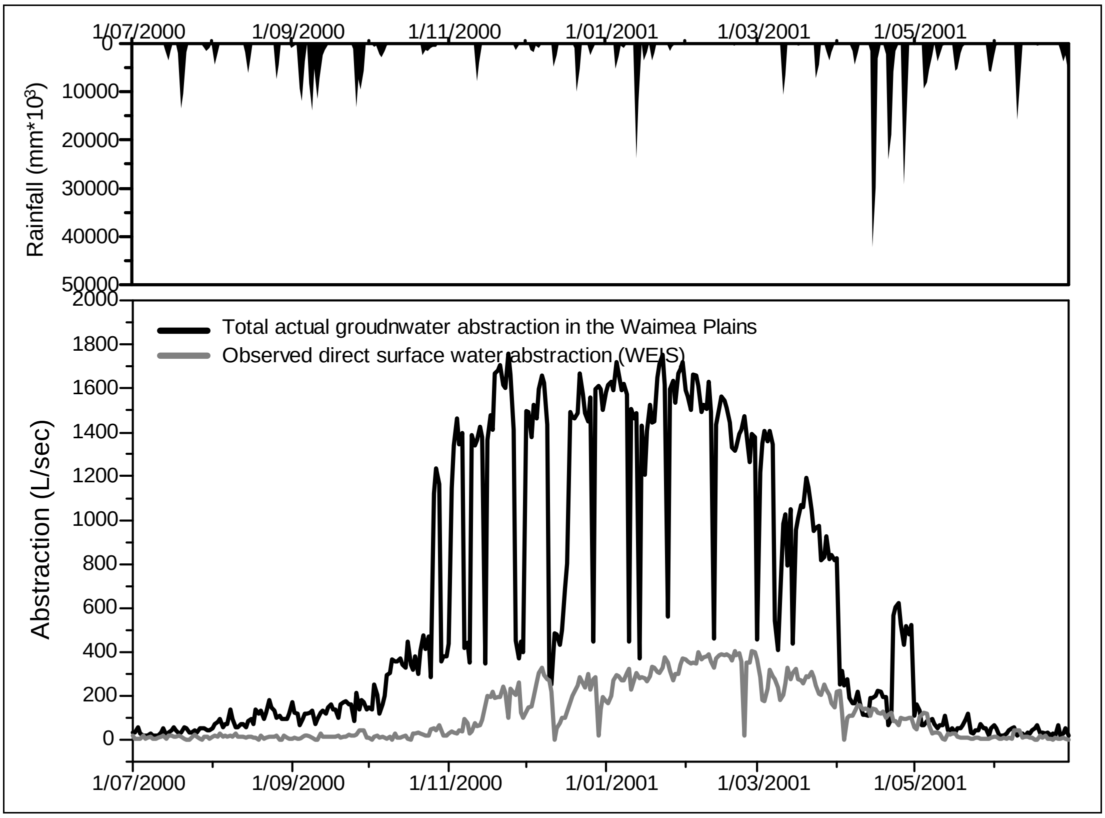

Figure 16. Total actual groundwater abstraction and observed direct surface water abstraction in the Waimea Plains in the period of 1 July 2000 to 30 June 2001 (2000/2001 drought year).

5.4 Recalibration of the Waimea Plains groundwater-river interaction model with updated river survey data

The Stream package of the Waimea Plains groundwater-river interaction model has been upgraded with new river survey data as described in Section 5.1. River stage and streamflow records from the Wairoa River Gorge and Wai-iti at Brightwater Bridge were used as inputs for calibration of the Stream Package.

Water level recorders operate in 11 observation wells on the Waimea Plains. In order to assess the effect of groundwater abstraction on river flows, particularly at the downstream area in the Waimea River, CW2 and McCliskies (Figure 3) were chosen as calibration points for groundwater level. Nursery-Appleby Bridge, located at the downstream area in the Waimea Plains, was also chosen as a river flow calibration point to evaluate the effect of groundwater abstraction on river flows.

The updated Waimea Plains groundwater-river interaction model was run for the period of 1 July 2000 through 30 June 2001 (2000/2001 drought year) to recalibrate the model and ensure it could reproduce river flow and aquifer response to groundwater abstraction under drought conditions. The calibration was aimed at matching observed Waimea River flow at Nursery-Appleby Bridge with calculated river flow as well as groundwater levels at the two observation well calibration points (CW2 and McCliskies). Using pump test data and considering the geology involved, the hydraulic conductivity for the AGUA was divided into several zones and further adjusted during the calibration process.

The spatial distribution of hydraulic conductivity in the AGUA and transmissivity in the UCA and LCA are shown in Figure 17. The AGUA is most permeable in the youngest gravels, adjacent to the Wairoa and Waimea Rivers. Hydraulic conductivity values of 1,000 – 3,000 m/day have been measured adjacent to the Waimea River and in the delta part of the plains of the AGUA. Hydraulic conductivity values used in the Wai-iti area range from 200-1,000 m/day. Typical transmissivity values used for the UCA range from 600 to 2,000 m^2/day. Pump testing indicated a transmissivity range of between 200–2,000 m^2/day in the LCA. Therefore a similar range of transmissivity was implemented into the model for that aquifer. Vertical leakance was also adjusted during the calibration process. The vertical leakance varied in the range of 1.0-8.0 day^{-1} within the AGUA. In contrast to the AGUA, relatively high values of vertical leakance, in the range of 500-5,500 day^{-1}, were considered appropriate for the UCA and a constant leakance value of 3 day^{-1} was used within the LCA. Streambed conductance values were estimated in the ranges of 37,000-40,000 m^2/day for the Wairoa River and 42,000-11,000 m^2/day for the Waimea River. Calibration runs were begun in winter and steady-state heads were used as initial conditions. Figure 18 shows the final fit between observed and simulated groundwater level fluctuations and river flows. There is good agreement in it between measured and simulated groundwater levels at two observation wells. Historic groundwater level records indicate that groundwater levels decrease during the irrigation season in the period of November to April and recover to equilibrium levels by August each winter. The model makes good predictions of drawdowns at the two groundwater wells involved during the irrigation season as well as winter recovery of groundwater levels from rainfall recharge. Model calculations are also a good match with the observed data set of river flows at Nursery-Appleby Bridge, particularly during the irrigation season when groundwater abstraction is increased (Figure 18).

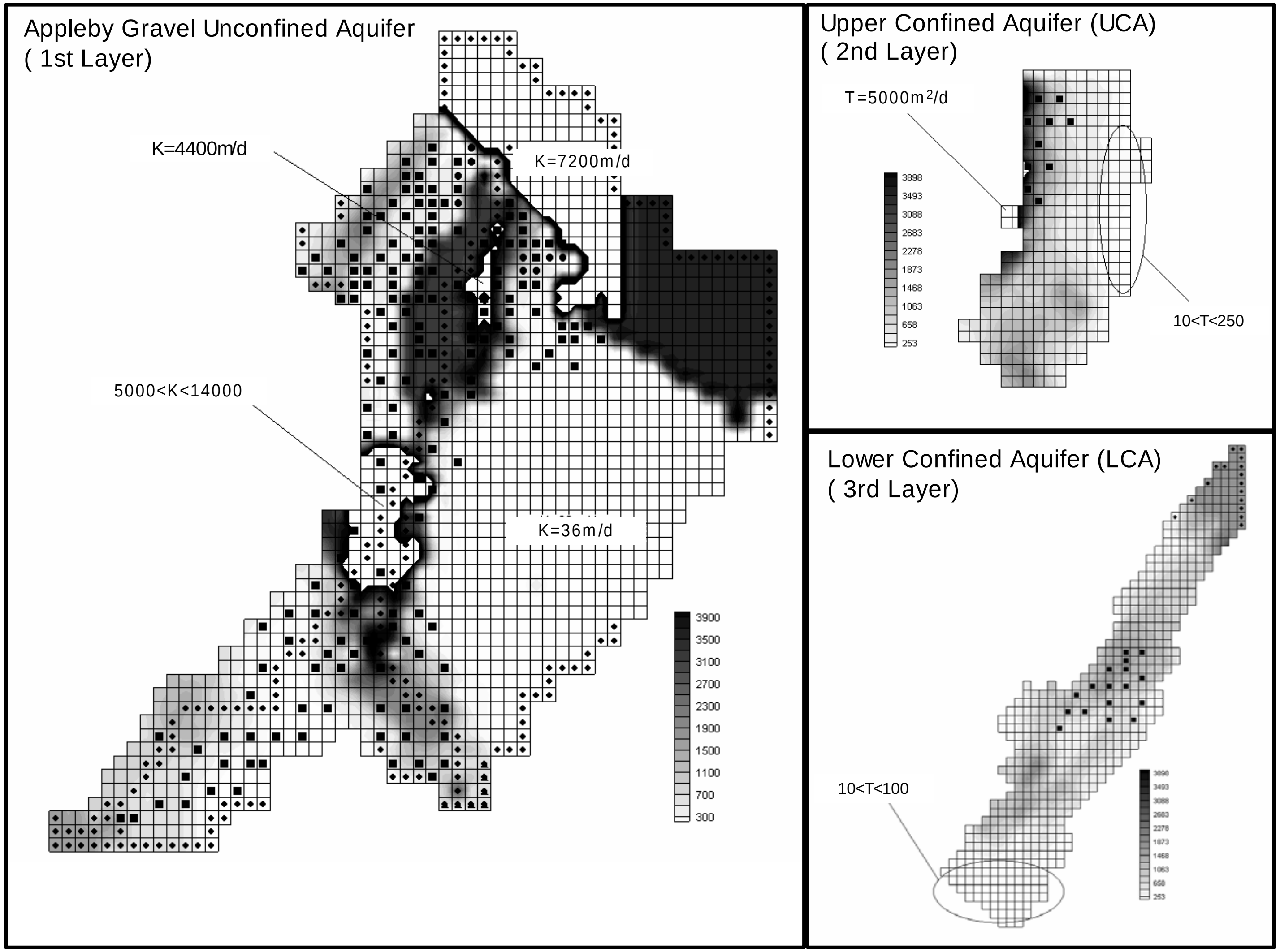

Figure 17. Spatial distribution of hydraulic conductivity and transmissivity used in the 3-layered Waimea Plains groundwater-river interaction model.

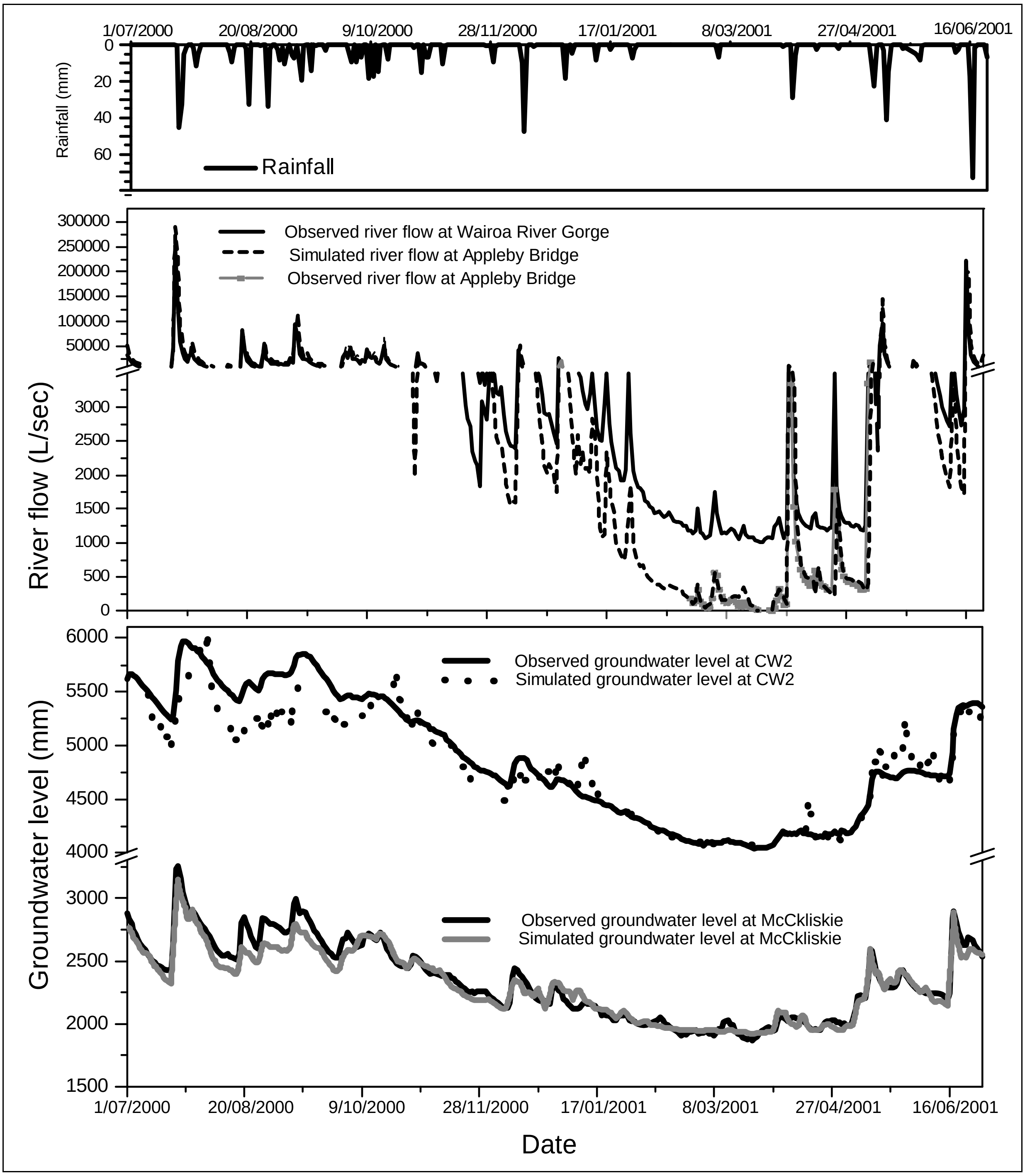

Figure 18. Model calibration results for groundwater levels and river flows in the period from 1-07-2000 to 30-06-2001.

6.0 MODELLING RESULTS OF UPDATE WAIMEA PLAINS GROUNDWATER-RIVER INTERACTION MODEL

6.1 Analysis of river flow losses/gains between downstream and upstream gauging sites

Groundwater abstractions, particularly during summer, reduce river flows significantly because the rivers (Wairoa and Waimea Rivers) are a major recharge source for the unconfined aquifer and indirectly feed the two confined aquifers in the Waimea Plains. Recent GNS Science research (Hong, 2006) shows that a significant increase in river flow loss between the upstream gauging site (Irvine-Wairoa Gorge) and the downstream gauging site (Nursery-Appleby Bridge) during the summer season is predicted if groundwater abstraction is increased.

In order to evaluate the effect of groundwater abstraction on river flows without upstream flow augmentation, the updated Waimea Plains groundwater-river interaction model was used to:

1. Predict Waimea River flow rate at Nursery-Appleby Bridge river for two different drought conditions (years 1982/1983 and 2000/2001) and an average year (2004/2005);
2. Calculate the frequency of 100 L/sec, 250 L/sec, 600 L/sec, and 1,100 L/sec flows in the Waimea River at Nursery-Appleby Bridge.

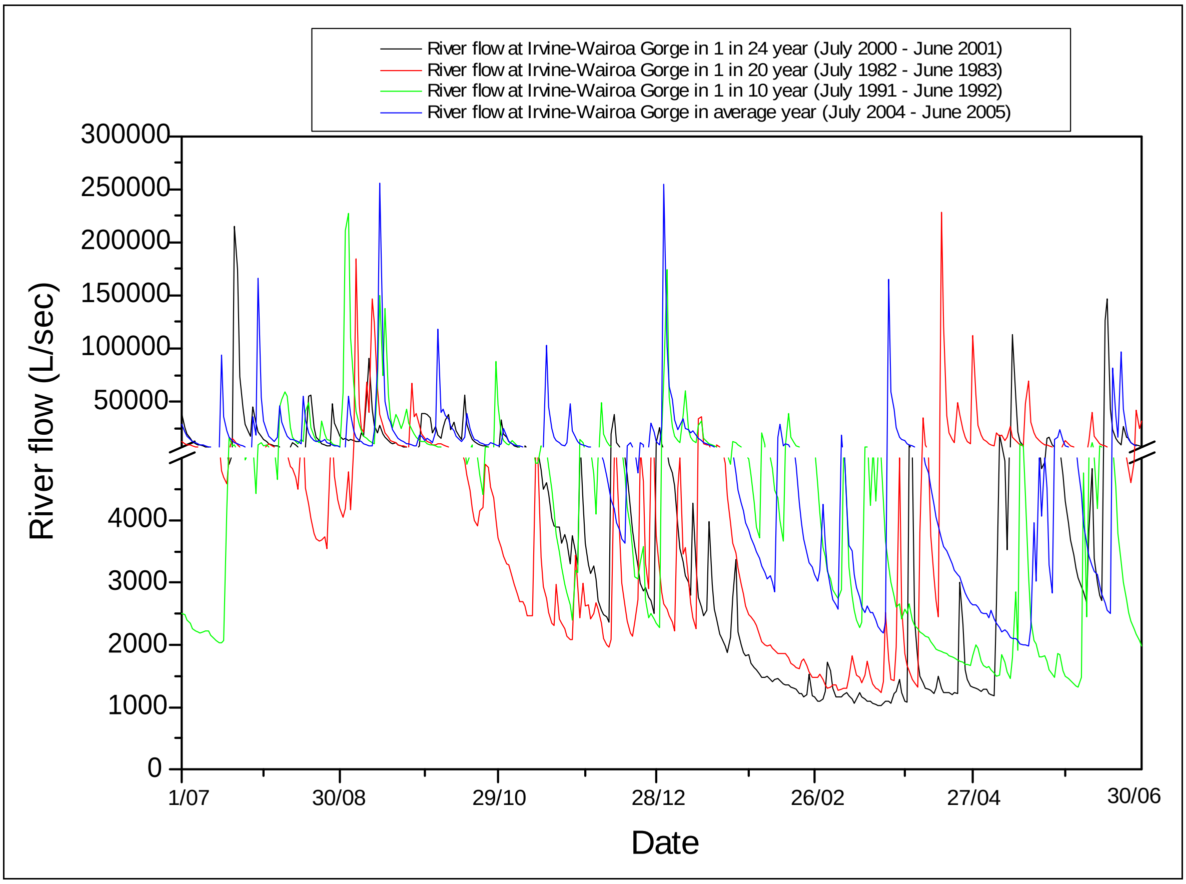

Figure 19. Observed river flow at Irvine-Wairoa Gorge in the period from 1 July to 30 June for drought years and the average year.

Figure 19 displays the Wairoa River flow at Wairoa River Gorge for the period from 1st July to 30th June for the 1991/1992 year, the 1982/1983 year drought, the 2000/2001 year drought, and the 2004/2005 average year. Table 8 summarises observed rainfall and daily average river flow at Wairoa River Gorge for the period February to March for two drought years and the average year for the same late summer period and the fall. For example, the observed rainfall was 8 mm and average river flows at Wairoa River Gorge and at Appleby Bridge were 1,285 L/sec and 171 L/sec, respectively, for the drought period February 2001 to March 2001 while average river flow at Wairoa River Gorge in the period February 2005 to April 2005 (average year) was approximately 6.2 times higher than the average river flow in the drought period February 2001 to March 2001.

The model calculated that for the 2000/2001 drought year, the average flow loss between Wairoa River Gorge and Nursery-Appleby Bridge would be 1,010 L/sec (i.e., 1,285 – 187 L/sec) with actual water usage. In contrast, the average observed flow at Wairoa River Gorge for the February-April 2005 time frame during the average year (2004/2005) was less than the average observed flow at Nursery-Appleby Bridge (Table 8). This is due to a flooding event in late March 2005 in the Waimea River. However, observed and calculated flows at Nursery-Appleby Bridge are less than observed flows at Wairoa River Gorge on most days in this period (Figure 20). The model calculates an average flow of 8,984 L/sec at Appleby Bridge in the period of 1 February to 30 April 2005, compared to observed flow of 9,135 L/sec. The average flow loss between Wairoa River Gorge and Appleby Bridge in the period of 15 April 2005 to 15 May 2005 (the driest period for that year and a period excluding the 23 March 2005 flood event) is calculated at 197 L/sec (with actual water usage), compared to 258 L/s of observed average flow loss.

Figure 20 shows that the Wairoa River flow at Irvine-Wairoa Gorge in the 2004/2005 year is not less than 2,000 L/s during the dry part of the year from 1 January 2005 to 30 April 2005. The minimum river flow at Nursery-Appleby Bridge was higher than 1,500 L/sec for actual water usage in the same period. However, in a severely drought year like the 2000/2001 one (assuming actual pumping usage), the observed river flow at Irvine-Wairoa Gorge decline below an average flow of 1,500 L/sec in the period February to April (Figure 21), and the Waimea River at Nursery-Appleby Bridge was predicted to be dry or have only very small flow. From Figure 22, it is apparent that the river at Nursery-Appleby Bridge in a drought, the period such as from 1 January1983 to 30 April 1983, would not go dry but would only have very low flow (assuming actual pumping usage). However, there were no flow records available at the Nursery-Appleby Bridge stream gage site for this period.

Table 9 summarises the number of days when the calculated river flow is less than 100, 250, 600, or 1,100 L/sec at Nursery-Appleby Bridge for the average year (2004/2005) and two different drought years (1982/1983 and 2000/2001). The model calculates that river flow at Nursery-Appleby Bridge will not be at less than any of those flows in the average year with actual water usage, but that for the drought years of 1982/1983 and 2000/2001 flows would be dry or very low (i.e., would be less than flows of 100, 250, 600, or 1,100 L/sec on 3, 12, 33, and 65 days for the 1982/1983 year and on 17, 55, 86, and 94 days for the 2000/2001 year (with actual usage).

Table 8. Average flows at Irvine, Wairoa River Gorge and Nursery-Appleby Bridge in the Waimea River for three drought years and the average year in the period of February and March.

Period		Average river flow		
	Total rainfall in period at Irvine (mm)	Observed daily mean flow at Irvine-Wairoa Gorge (L/sec)	Observed flow at Nursery-Appleby Bridge (L/sec)	Calculated flow at Nursery-Appleby Bridge with actual water usage* (L/sec)
February 1983 – March 1983 (Drought Year)	37	1756	not observed	746
February 2001 – March 2001 (Drought Year)	8	1285	171	187
15 April 2005 -15 May 2005 (Average Year)	4	2633	2375	2436
February 2005 – April 2005 (Average Year)	184	7980	9135	8984

* flows are calculated by the Waimea Plains groundwater-river interaction model.

Table 9. Number of days calculated river flow less than 100, 250, 600, or 1,100 L/sec at Nursery-Appleby Bridge for average year and two drought years.

Year	Simulation	Number of days where river flow is less than 100 L/sec	Number of days where river flow is less than 250 L/sec	Number of days where river flow is less than 600 L/sec	Number of days where river flow is less than 1,100 L/sec
2004/2005 (Average Year)	Actual water usage	0	0	0	0
1982/1983 (Drought Year)	Actual water usage	3	12	33	65
2000/2001 (Drought Year)	Actual water usage	17	55	86	94

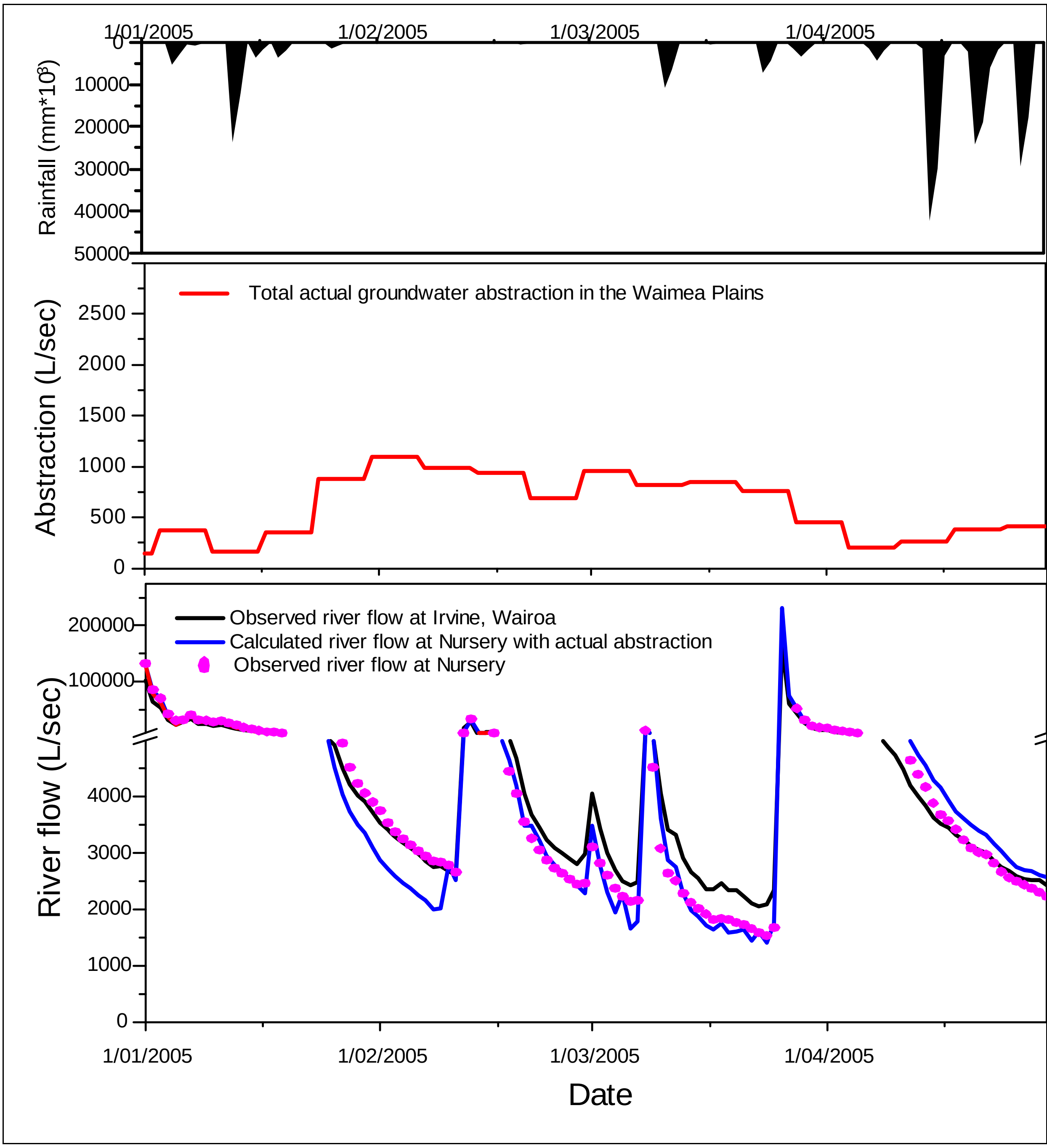

Figure 20. Waimea River flow calculations at Nursery-Appleby Bridge in the average year period of 1 January 2005 to 30 April 2005.

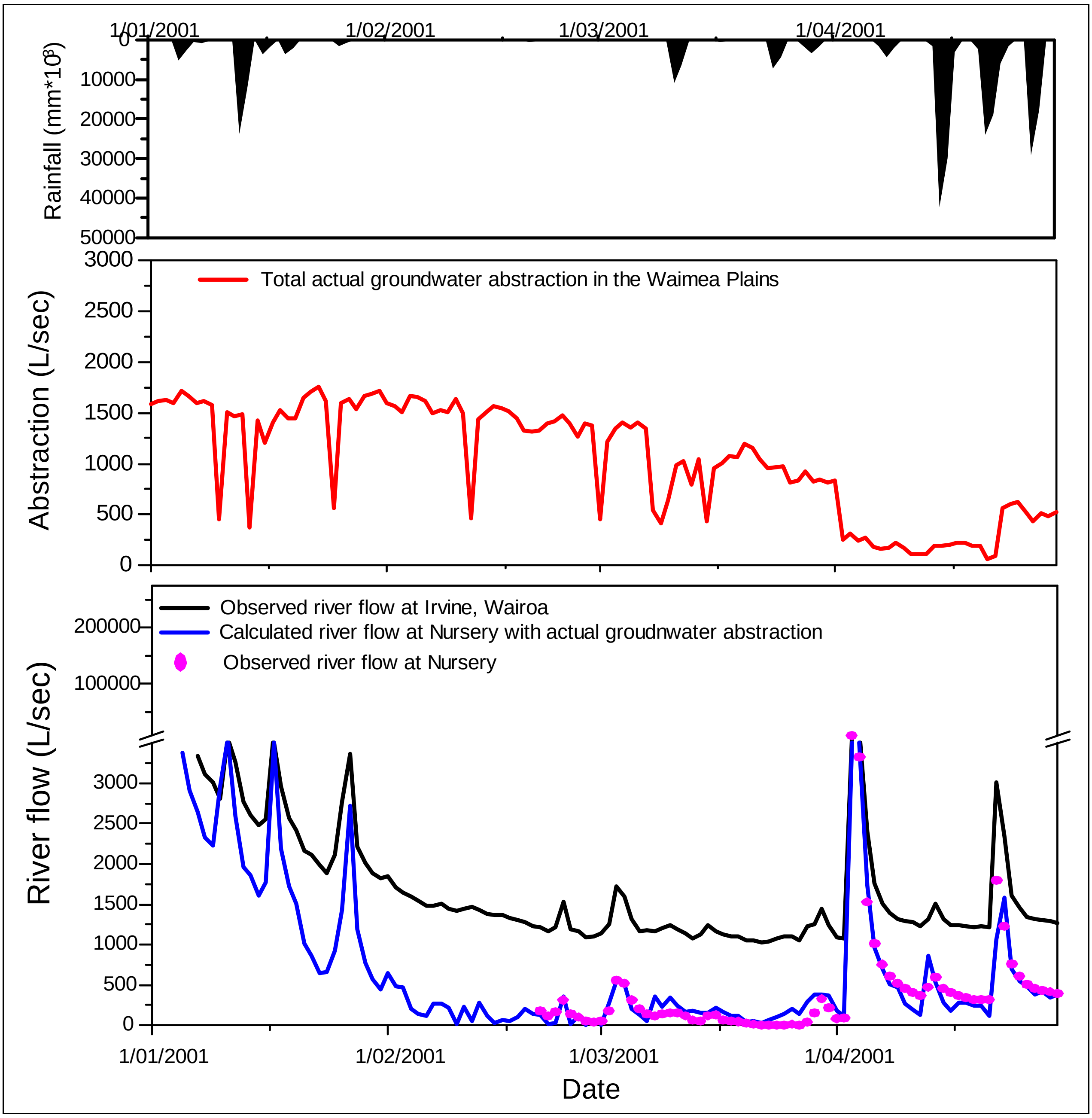

Figure 21. Waimea River flow calculations at Nursery-Appleby Bridge in the drought year period of 01 January 2001 to 30 April 2001.

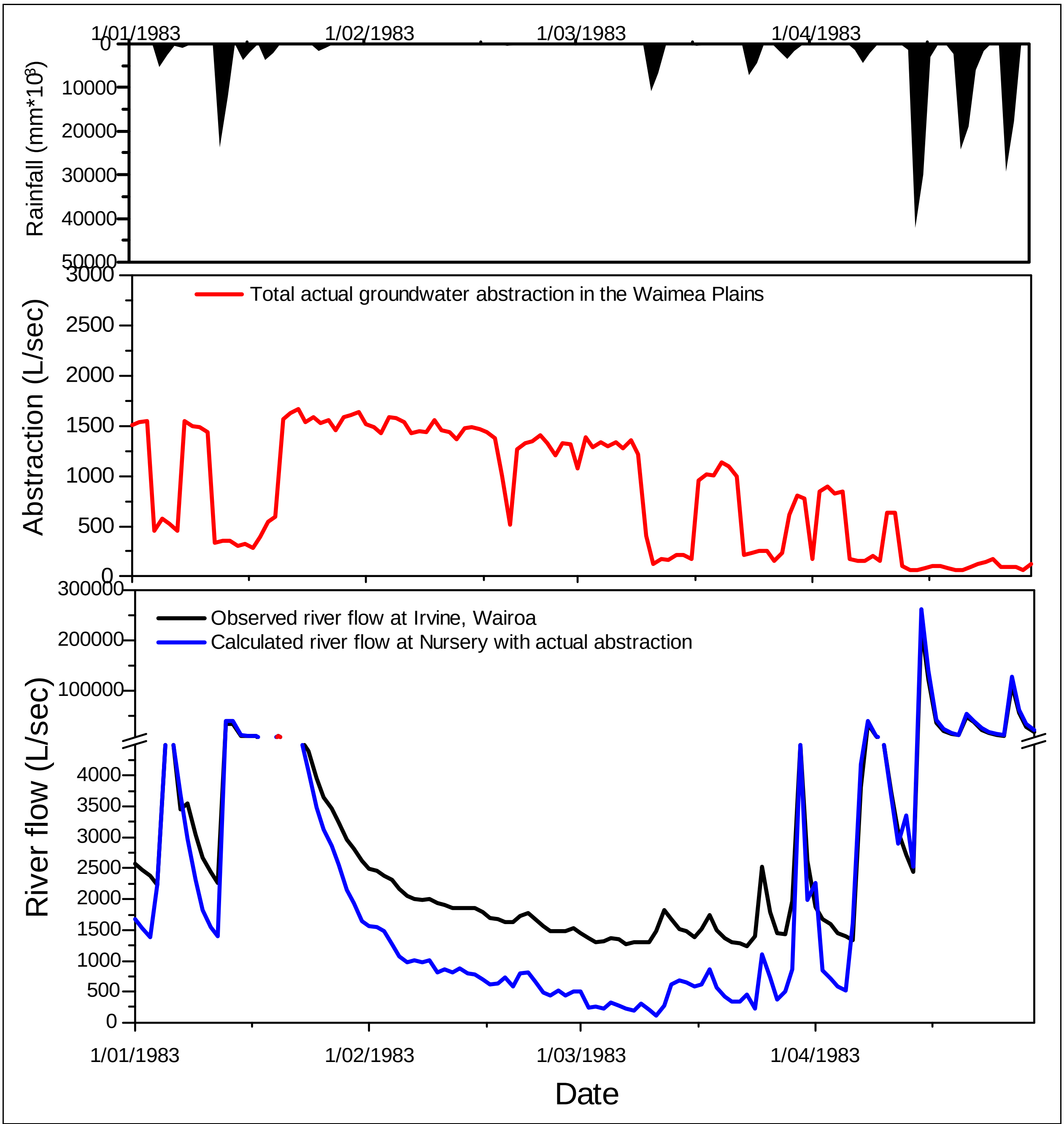

Figure 22. Waimea River flow calculations at Nursery-Appleby Bridge in the period 1 January 1983 to 30 April 1983 (Note: There were no observed flow data available for this period at the Nursery- Appleby Bridge stream gage site).

6.2 Effect of river bed change on groundwater levels

Several studies (Grant, et al., 1990; Harvey and Bencala, 1993; Bouwer and Maddock, 1997) have suggested that water exchange between streambeds and adjacent aquifers is influenced by the change of streambed topography. Variation in streambed topography and the resulting variation in river slope influences the potential water volume exchange between a stream and its associated adjacent aquifer and could be a significant control on river recharge to the AGUA in the Waimea Plains. Recharge to the aquifer flow path decreases where hydraulic gradient between river and aquifer decreases.

A sensitivity analysis was undertaken to evaluate the effects of river bed changes in the Wairoa/Waimea River system on groundwater levels for the 2000/2001 drought year, two scenarios were used in this simulation:

1. 0.5 m depletion (i.e., decrease) of streambed topography in the Wairoa/Waimea River.
2. 1 m depletion of streambed topography in the Wairoa/Waimea River.

In order to assess the effect of streambed topography changes on river recharge to aquifers, particularly groundwater levels at the downstream area in the Waimea River, McCliskies (Figure 3) was chosen as the indicator site for groundwater levels because it is very close to Waimea River and the groundwater levels at that site are influenced significantly by river recharge.

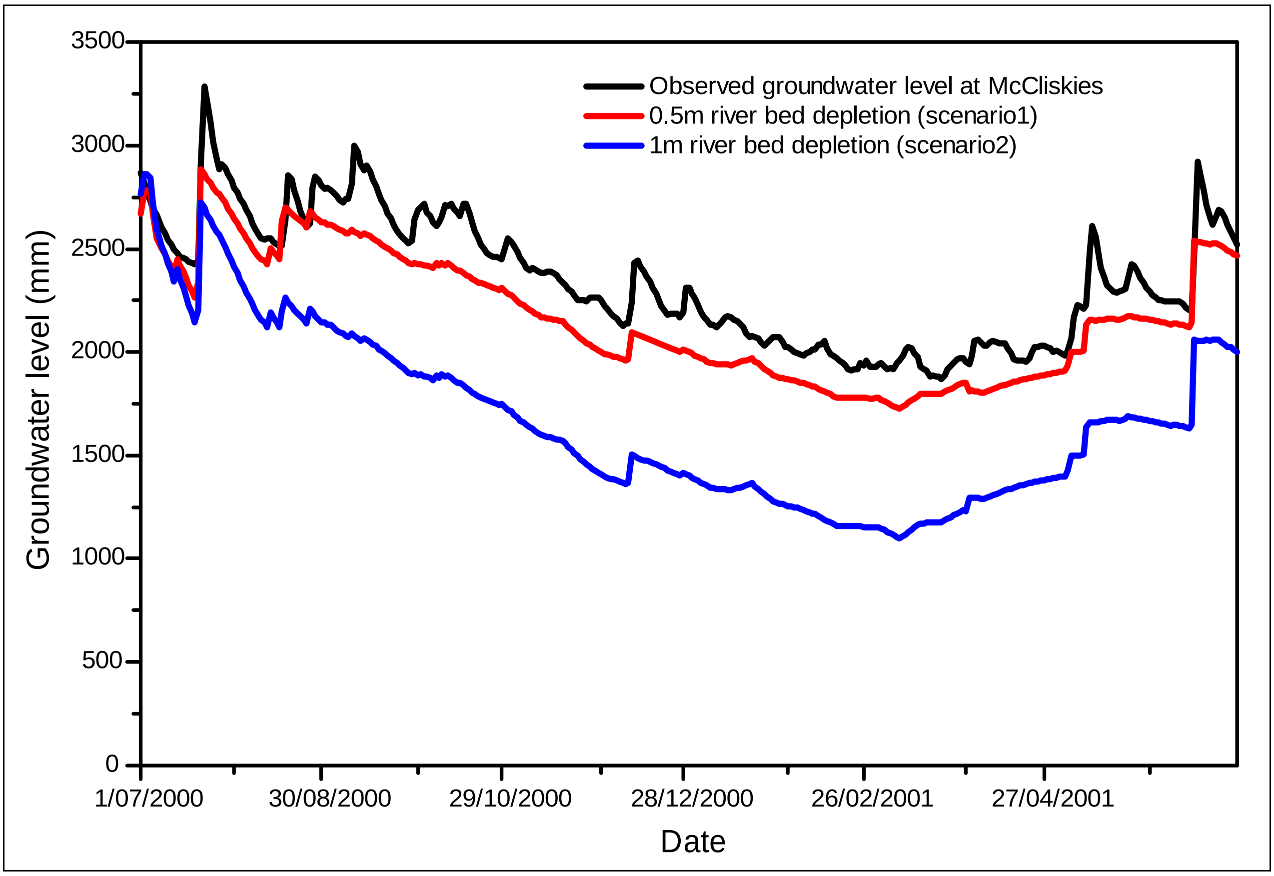

Figure 23. Sensitivity of groundwater level change to river bed depletion.

Figure 23 shows the results of this sensitivity analysis. From Figure 23, it is clear that the influence of streambed topography change on river recharge to the McCliskies site is significant. Calculated groundwater levels at the McCliskies site were significantly decreased in direct proportion to streambed depletion in the Waimea River.

Groundwater levels at the McCliskies site in the period of 1 November 2000 to 31 March 2001 were observed to average 2.13 m (Table 10). Calculated average groundwater levels at the McCliskies site for the same period would decrease to 1.96 m (a decline of approximately 17 cm) for 0.5 m streambed depletion. However, this increases to a calculated decline of 0.8 m in average groundwater levels for the 1 m streambed depletion scenario. This result indicates that river recharge to the AGUA in the Waimea Plains would be strongly influenced by river bed depletion exceeding 0.5 m because the hydraulic gradient between the river and the aquifer would be substantially decreased and, as a result, river recharge fluxes from the Waimea River to the AGUA would be similarly reduced. This reduction in recharge to the AGUA would, obviously, have important consequences for water allocation in the catchment. Reduction in groundwater levels would, of course, reduce available aquifer storage and yield while increasing susceptibility to seawater intrusion.

Table 10. Effect of streambed depletion on groundwater levels at McCliskies site.

McCliskies Site	Annual average groundwater levels (m)	Average groundwater levels between November 2000 and March 2001 (m)	Monthly average groundwater levels (m)				
			Nov. 2000	Dec. 2000	Jan. 2001	Feb. 2001	Mar. 2001
Observed (m)	2.34	2.13	2.36	2.25	2.12	1.98	1.94
0.5 m streambed depletion	2.25	1.96	2.15	2.03	1.96	1.84	1.81
1 m streambed depletion	1.72	1.33	1.57	1.42	1.33	1.19	1.16

7.0 RIVER FLOW AUGMENTATION MODELLING TO FLOW AT NURSERY-APPLEBY BRIDGE WITH FUTURE WATER DEMAND

This analysis was intended to determine river flow release requirements from storage in the upper Wairoa-Lee catchments by re-running the upgraded Waimea Plains groundwater-river interaction model developed by GNS Science in collaboration with TDC. Specifically, it was intended to establish the augmented river flow rate at Irvine-Wairoa Gorge needed to maintain a minimum of 1,100 L/s flow rate at the Nursery-Appleby Bridge location in the Waimea River based on future water demand for two different drought years (1982/1983 year and 2000/2001) and an average year (2004/2005 year).

This river water augmentation modelling has been structured into the following five modelling steps:

(1) Step 1 Compute future water demand in the Waimea catchment based on climate data for the period of 1 July through 30 June for the drought years 1982/1983 and 2000/2001 and the average year 2004/2005, assuming an irrigation water balance model developed by Landcare Research for a pasture crop over a range of soil types (38, 78, and 130 mm soil moisture holding capacity).

(2) Step 2 Compute daily minimum river flow augmentation at Irvine-Wairoa Gorge required to maintain a minimum 1,100 L/s flow rate at Nursery-Appleby Bridge while meeting unrestricted abstractive future demand in the period of 1 July through 30 June for the drought years 1982/1983 and 2000/2001 year and the average year 2004/2005.

(3) Step 3 Calculate daily minimum river flow augmentation at Irvine-Wairoa Gorge required to maintain a minimum 600 L/s flow rate at Nursery-Appleby Bridge (as opposed to a flow of 1,100 L/sec) with unrestricted future water demand for the drought year 2000/2001.

(4) Step 4 Repeat modelling for partial abstractive future water demands equivalent to 75% and 50% of full unrestricted future water demand for the drought year 2000/2001.

(5) Step 5 Undertake forward scenario simulation to confirm that the proposed river augmented water release regime at Irvine-Wairoa Gorge proposed by Tonkin and Taylor Ltd will meet downstream requirements of maintaining a minimum 1,100 L/s flow rate at Nursery-Appleby Bridge.

7.1 Future water demand in the Waimea Plains

Table 11 summarises the annual water requirement (mm/year) calculated by Landcare Research for pasture and a range of soil types. For example, the annual irrigation requirement for pasture with 38 mm soil moisture holding capacity for the 2000/2001 drought year is estimated at 539 mm/year, compared to 345 mm/year for the 2004/2005 average year. Irrigation demand profiles for pasture and a range of soil types based on daily climate data for the 1982/1983 and 2000/2001 drought years and the 2004/2005 average year for the Waimea Plains are shown in Figures 24, 25, and 26, respectively.

Table 11. Annual irrigation requirement (mm/year) calculated by Landcare Research for a pasture and a range of soil types in the period of 1 July through 30 June.

Crop type: pasture	Soil moisture holding capacity related to soil type		
	38 mm	**78 mm**	**130 mm**
2000/2001 (Drought Year)	539 mm/year	522 mm/year	515 mm/year
1982/1983 (Drought Year)	522 mm/year	511 mm/year	502 mm/year
2004/2005 (Average Year)	345 mm/yr	288 mm/yr	231 mm/yr

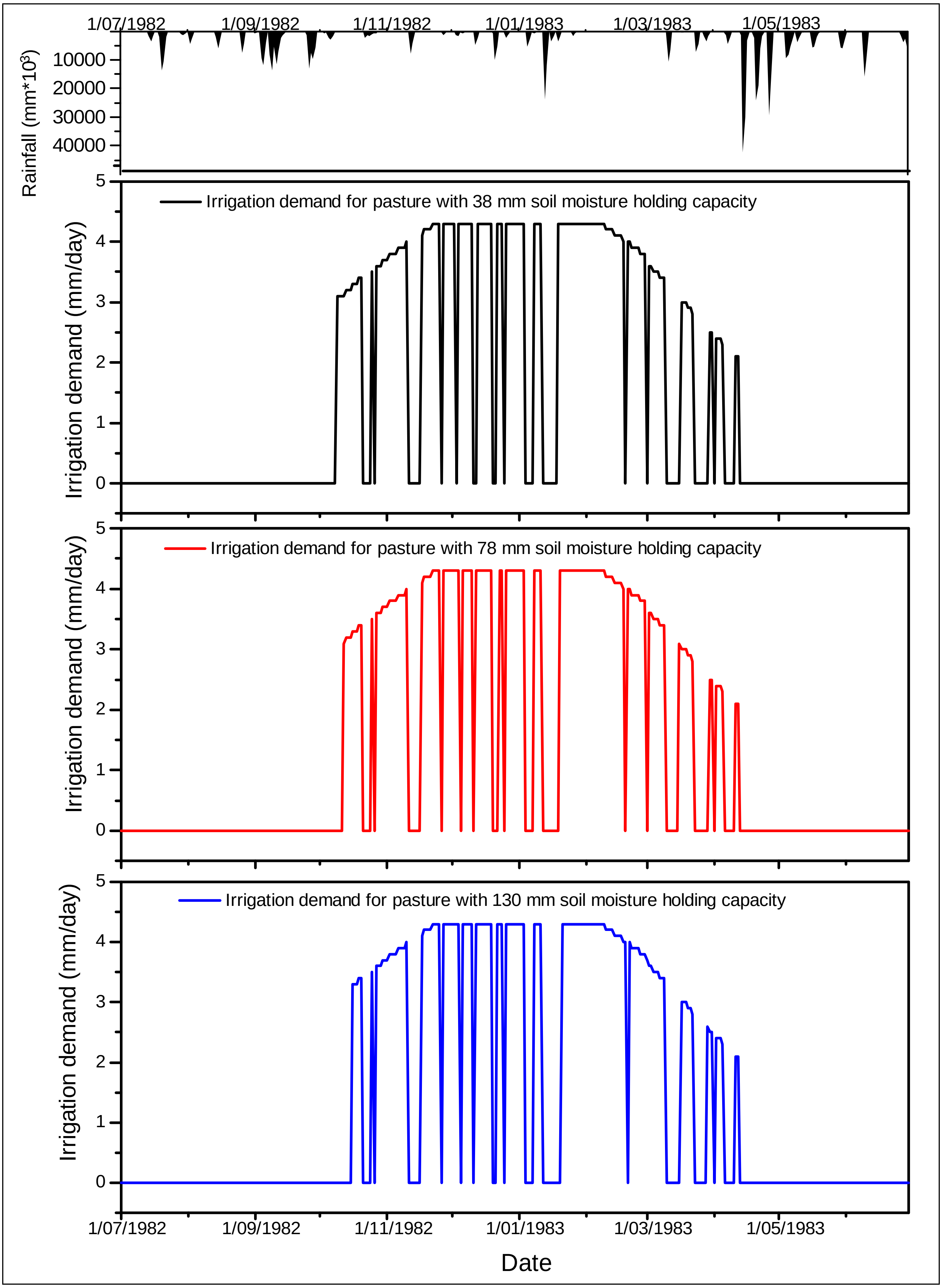

Figure 24. Daily irrigation demand for pasture and a range of soil types based on climate data for the July 1982 to June 1983 drought year in the Waimea Plains.

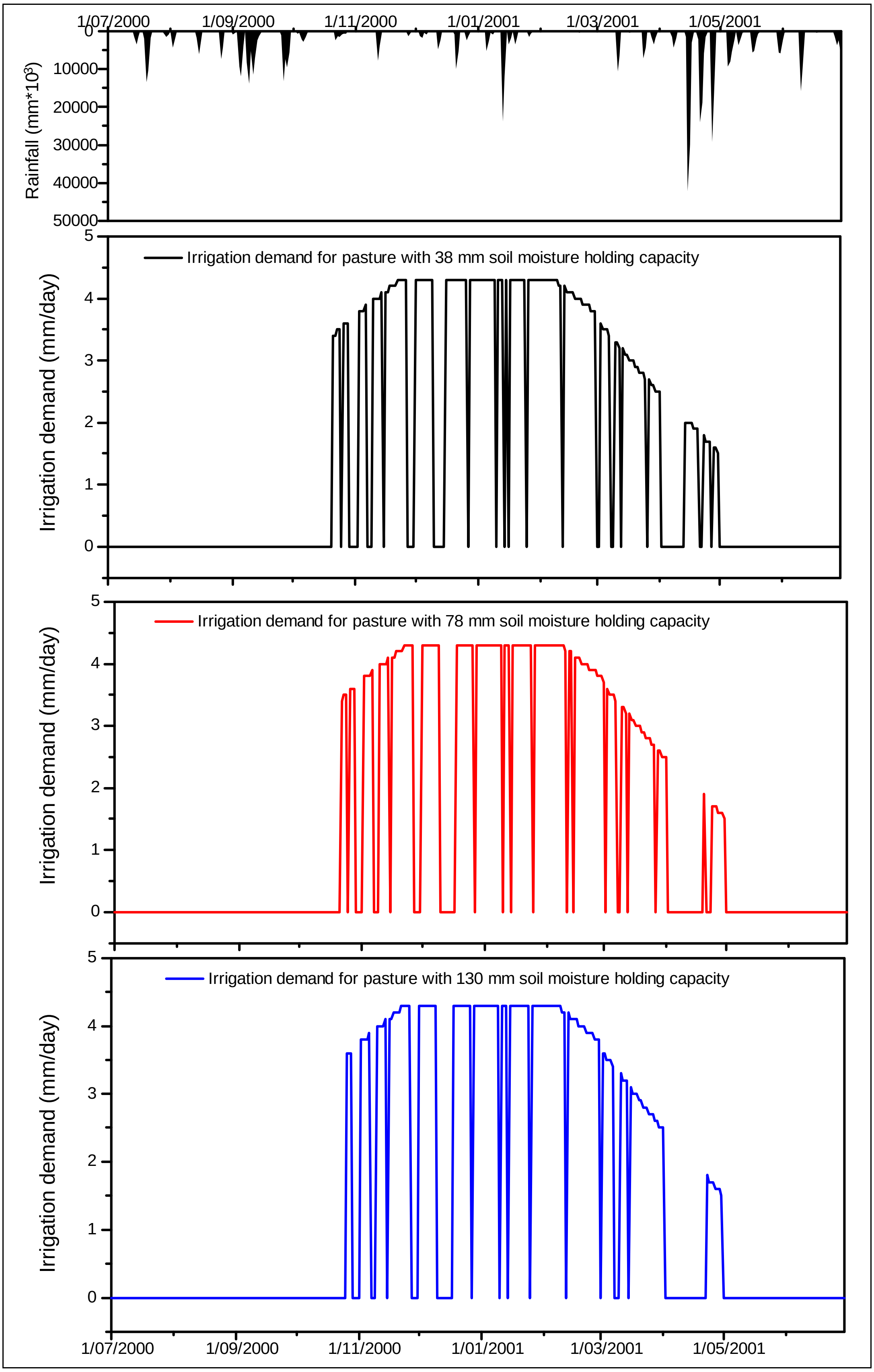

Figure 25. Daily irrigation demand for pasture and a range of soil types based on climate data for the July 2000 to June 2001 drought year in the Waimea Plains.

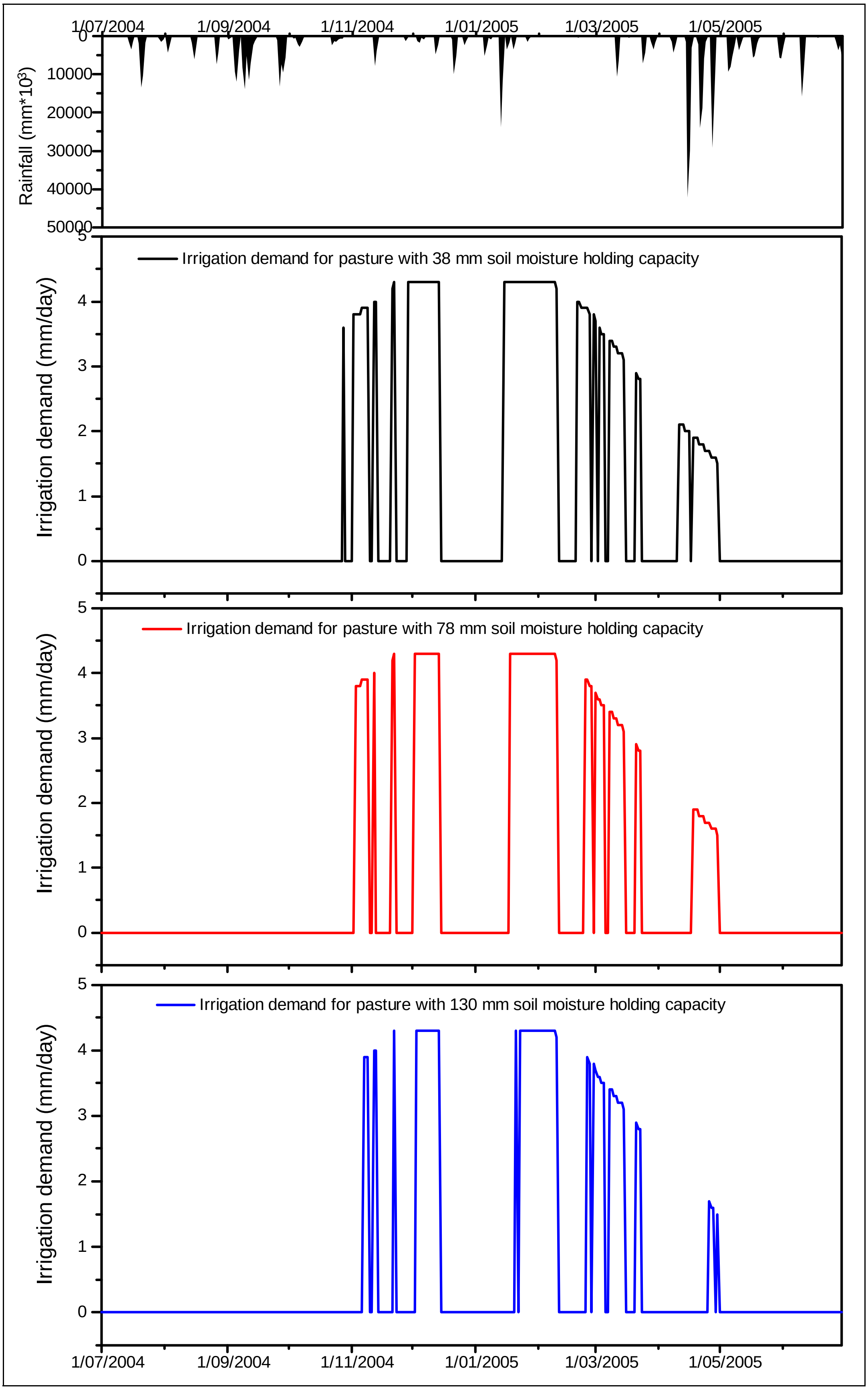

Figure 26. Daily irrigation demand for pasture and a range of soil types based on climate data for the July 2004 to June 2005 average year in the Waimea Plains.

Based on the irrigation demand displayed in Figures 24-26, the irrigation demand for each cell in the Waimea Plains groundwater-river interaction model was calculated by the following formula:

*Irrigation demand per day in each cell = 5.47 ha * daily irrigation demand (mm/day) for pasture and each soil type * crop mix (%) * 0.8 (irrigated portion of the land)* Eqn-1

In equation one (Eqn-1), the factor 0.8 represents the "irrigated" part of the land. The non-irrigated 20% portion of the land (a figure utilized by TDC) allows for that portion of the total land used for townships, roads, and river reserves. John Bealing of AgFirst recommends that the irrigated portion be decreased by an additional 5% if individual rural property (including houses, gardens and farm sheds) is considered. Adoption of the 25% non-irrigated portion figure for land in Eqn-1 would make water requirements correspondingly lower. Eqn-1 may also be used for a mix of crop types. In this case, since all land is assumed to be in pasture, crop mix is 100% (i.e., a factor of 1).

Table 12 shows the areas of each soil type within the Waimea Plains and the allocation per hectare. This allows calculation of the water requirement, given rainfalls at different times of the year. Figure 13 shows soil types in the Waimea Plains overlaid on the Waimea Plains groundwater-river interaction model. In Figure 13, the red lines are the water management zones defined by TDC.

Table 12. Irrigable areas by soil type in the Waimea Plains.

Soil Type	Area Ha		Allocation Limits[1] m^3/ha/week	Comments
	Total	**Irrigable**		
Mapua sandy loam	258	200	190	Foothills around the Redwood Valley
Dovedale gravelly loam	525	520	250	Mostly found in the Redwood Valley area.
Richmond clay loam, silt loam & Wakatu silt loam	643	610	270	Towards the estuary and the foothills along Patons Road.
Waimea silt loam & sandy loam	2,137	2,130	300	Alongside the Waimea River
Ranzau Soils	1,649	1,640	350	Waimea Plains
Totals	**5,212**	**5,100**		
Motupiko loams (Wai-iti)	1180	480	350	Wai-iti Valley (Brightwater to Wakefield)
Totals	**6,392**	**5,580**		

[1] These limits are taken from the TRMP, Chapter 31, Fig 31.1D

Eqn-1 provides an estimate of the daily irrigation water required for each cell in the Waimea Plains groundwater-river interaction model in the period 1 July to 30 June of the following year. For example, the irrigation water requirement for a model cell with 130 mm soil moisture water holding capacity on 1 February 1983 would be:

Daily irrigation water requirement= [5.47 (ha) * 4.3 (mm/day) pasture * 1 (for crop mix) * 0.8] = 187.5 m^3/day Eqn-2

Future water demand for each management zone defined by TDC in the Waimea Plains can be calculated using Eqn-1 in a manner similar to that shown by Eqn-2 for the 1982/1983 drought year, 2000/2001 drought year, and 2004/2005 average year.

Tables 13-15 show calculated future water demand. Tables 13-15 also show the approximate areas associated with each of the water management zones in the Waimea Plains groundwater-river interaction model. About half of the Hope aquifers and Eastern Hills water management zone (Figure 3 and 11) are covered by the Waimea East Irrigation Scheme (WEIS), and much of the remainder by either the unconfined gravels alongside the Waimea River, or by the UCA and LCA.

In Tables 13-15, there is an irrigation allowance in the Brightwater/Wakefield area of 300 ha to service some of the lower Wai-iti zone, in addition to the current allocation already allowed for this zone. There is also an irrigation allowance in the Redwood Valley area of 476 ha to service the Redwood Valley Zone. This is in addition to the existing 73 ha irrigation for the Redwood Valley Zone. As agreed during the WWAC Technical Group meeting on 7 October 2007 at TDC, there is an allowance for 250 ha of irrigable land area in Rabbit Island for future land use change (possibly recreational/reserve use) included in the future water demand calculation. Abstraction locations in Redwood Valley and the Brightwater/Wakefield area within the model domain are displayed in Figure 27.

Urban water demand in WWAP Stage 1 was calculated at 45,000 m^3/day maximum daily demand for a 50 year horizon. The WWAC agreed on a 100 year horizon with a maximum daily value of 63,795 m^3/day for urban water supply (Table 16). The location of urban water abstraction in the model domain is displayed in Figures 28-29. Most abstraction for urban water supply is from the AGUA. Relevant locations within the AGUA are shown in Figure 28. Figure 29 shows abstraction locations for the Richmond-Cargill Wellfield urban water supply from the LCA. Table 16 summarizes the urban water supply location and maximum daily demand for a 100 year horizon at each well, compared to the 50 year horizon implemented in WWAP Stage 1. Figure 30 shows time series of total urban water demand caculated for the 100 year horizon in the Waimea Plains.

The expected irrigable area is 5,906 ha, including 250 ha on Rabbit Island. A total irrigable area of 5,923 was modelled in the Waimea Plains groundwater–interaction model (Tables 13-15). The difference results from limitations in coverage of irregular land boundaries by rectangular model cells. Considering irrigation demand calculated by Landcare Research for the total 5,906 of irrigable area modelled, future daily peak water demand was estimated by the model to be 3,351 L/s for the Waimea Plains model area (any year). Predicted annual total water demand volume for the 1982/1983 drought year was 41,200,000 m^3 compared to 41,900,000 m^3 for the 2000/2001 drought year and 30,300,000 m3 for the 2004/2005 average year (see Tables 13, 14, and 15, respectively).

Model calculated future water demands in the Waimea Plains based on climate data for the period of 1 July through 30 June of the following year for the 1982/1983 drought year, 2000/2001 drought year, and 2004/2005 average year are shown in Figures 31-33. Total future water demand displayed in Figures 31-33 includes direct surface water abstraction (WEIS and Future Regional Supply), irrigation demand for Rabbit Island, urban water demand (including industrial) for the 100 year horizon, and abstraction for outside valley (i.e., new Redwood Valley and Brightwater/Wakefield).

In summary, a new groundwater abstraction model for future water demand in the Waimea Plains groundwater-river interaction has been created using the existing groundwater abstraction wells in the model displayed in Figures 4-6. New abstraction wells were created to simulate the effect of groundwater abstraction for the outside valley and urban water supply in the model (see Figures 27-29). The new groundwater abstraction model for future water demand does not include direct surface water abstraction from rivers, namely WEIS that is taken directly from the Wairoa River just below Irvine-Wairoa Gorge and the constant abstraction of 254.6 L/s for Future Regional Supply.

Table 13. Predicted peak daily water demand and expected irrigable area by water management zones in the Waimea Plains for the 1982/1983 drought year.

Management Zone		Expected irrigable Area (ha)	Modelled irrigable area (ha)	Current Allocation Limits (L/sec)	Predicted peak daily water demand (L/sec)	Predicted annual sum of water demand (m^3)
Upper Catchments		minimal		3	minimal	
Reservoir		580	591	826	234	2,380,000
Waimea West		385	388	178	154	1,560,000
Hope & Eastern Hills*		2,170	2,232	97		
	WEIS		1,177		471	4,960,000
	Upper Confined		361	147	143	1,510,000
	Lower Confined		564	230	228	2,400,000
	Hope aquifer		131		52	530,000
Delta		1,246	1,246	1,000	495	5,010,000
Golden Hills		300	300	113	119	1,210,000
Redwood		625	613			
	Redwood Valley		115		46	470,000
	Outside irrigation in the Redwood Valley, 476ha		427		169	1,710,000
	Redwood Valley existing , 73ha		71		28	290,000
Outside irrigation in the Brightwater/ Wakefield, 300ha (Wai-iti Aldourie Rd to Wakefield)		300	301	105	119	1,210,000
Future urban water demand (100 year projection)					738	8,940,000
Future regional supply					255	8,030,000
Future Rabbit Is. demand		250	252		100	1,030,000
TOTALS		**5,906**	**5,923**	**2,699**	**3,351**	**41,240,000**

* Hope & Eastern Hill is covered by and Waimea East Irrigation Scheme (WEIS), UCA, LCA, Hope aquifer.

Table 14. Predicted peak daily water demand and expected irrigable area by water management zones in the Waimea Plains for the 2000/2001 drought year.

Management Zone		Expected irrigable Area (ha)	Modelled irrigable area (ha)	Current Allocation Limits (L/sec)	Predicted peak daily water demand (L/sec)	Predicted annual sum of water demand (m^3)
Upper Catchments		minimal		3	minimal	
Reservoir		580	591	826	234	2,440,000
Waimea West		385	388	178	154	1,600,000
Hope & Eastern Hills*		2,170	2,232	97		
	WEIS		1,177		471	5,110,000
	Upper Confined		361	147	143	1,560,000
	Lower Confined		564	230	228	2,470,000
	Hope aquifer		131		52	550,000
Delta		1,246	1,246	1,000	495	5,140,000
Golden Hills		300	300	113	119	1,240,000
Redwood		625	613			
	Redwood Valley		115		46	480,000
	Outside irrigation in the Redwood Valley, 476ha		427		169	1,760,000
	Redwood Valley existing , 73ha		71		28	300,000
Outside irrigation in the Brightwater/ Wakefield, 300ha (Wai-iti Aldourie Rd to Wakefield)		300	301	105	119	1,240,000
Future urban water demand (100 year projection)					738	8,940,000
Future regional supply					255	8,030,000
Future Rabbit Is. demand		250	252		100	1,050,000
TOTALS		**5,906**	**5,923**	**2,699**	**3,351**	**41,910,000**

* Hope & Eastern Hill is covered by and Waimea East Irrigation Scheme (WEIS), UCA, LCA, Hope aquifer.

Table 15. Predicted peak daily water demand and expected irrigable area by water management zones in the Waimea Plains for the 2004/2005 average year.

Management Zone		Expected irrigable Area (ha)	Modelled irrigable area (ha)	Current Allocation Limits (L/sec)	Predicted peak daily water demand (L/sec)	Predicted annual sum of water demand (m^3)
Upper Catchments		minimal		3	minimal	
Reservoir		580	591	826	234	1,150,000
Waimea West		385	388	178	154	720,000
Hope & Eastern Hills*		2,170	2,232	97		
	WEIS		1,177		471	3,320,000
	Upper Confined		361	147	143	1,010,000
	Lower Confined		564	230	228	1,610,000
	Hope aquifer		131		52	270,000
Delta		1,246	1,246	1,000	495	2,320,000
Golden Hills		300	300	113	119	570,000
Redwood		625	613			
	Redwood Valley		115		46	260,000
	Outside irrigation in the Redwood Valley, 476ha		427		169	790,000
	Redwood Valley existing , 73ha		71		28	160,000
Outside irrigation in the Brightwater/ Wakefield, 300ha (Wai-iti Aldourie Rd to Wakefield)		300	301	105	119	560,000
Future urban water demand (100 year projection)					738	8,940,000
Future regional supply					255	8,030,000
Future Rabbit Is. demand		250	252		100	580,000
TOTALS		**5,906**	**5,923**	**2,699**	**3,351**	**30,290,000**

* Hope & Eastern Hill is covered by and Waimea East Irrigation Scheme (WEIS), UCA, LCA, Hope aquifer.

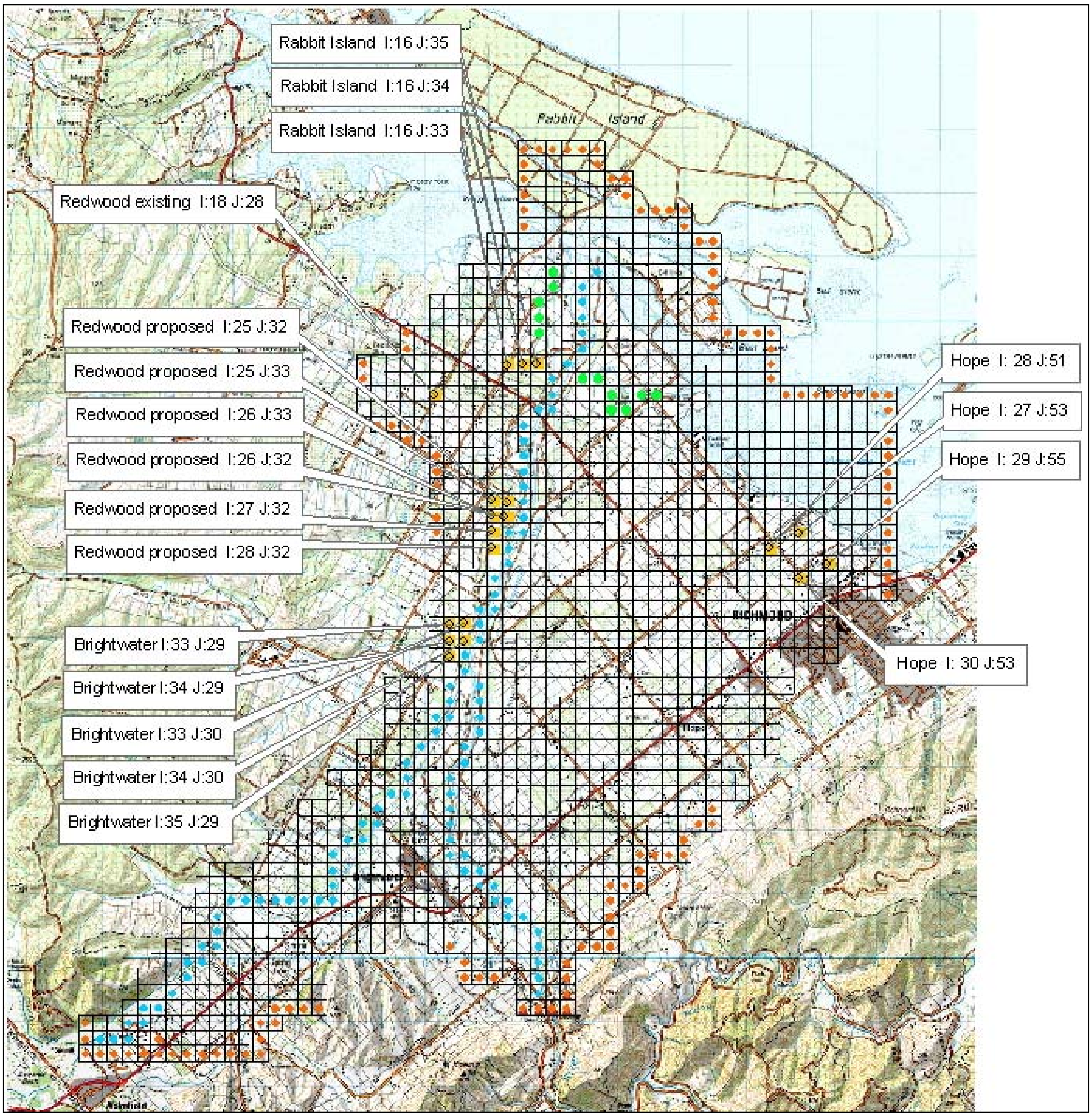

Figure 27. Outside valley pumping locations.

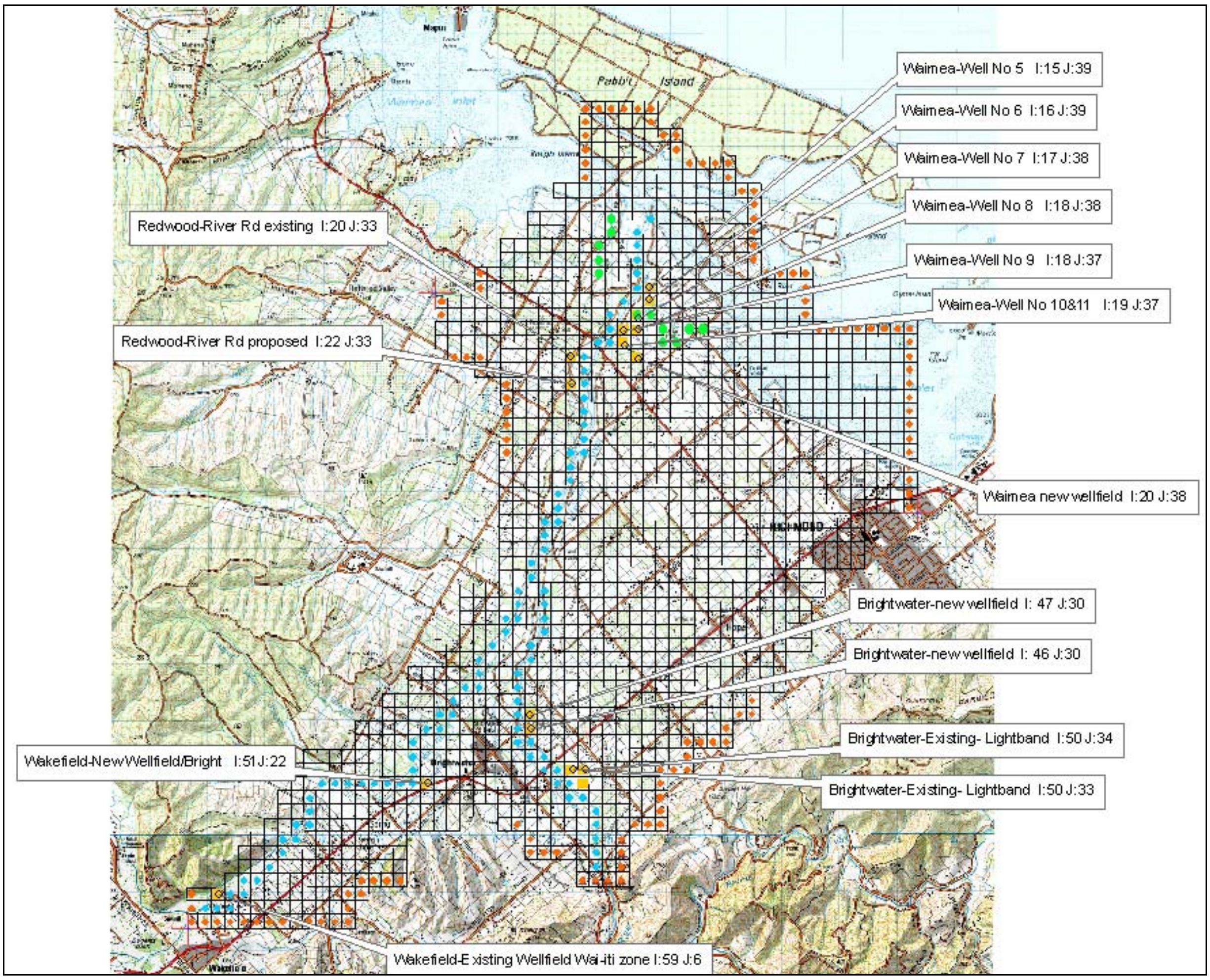

Figure 28. Pumping locations in the Appleby Gravel Unconfined Aquifer (AUGA) for the urban water demand.

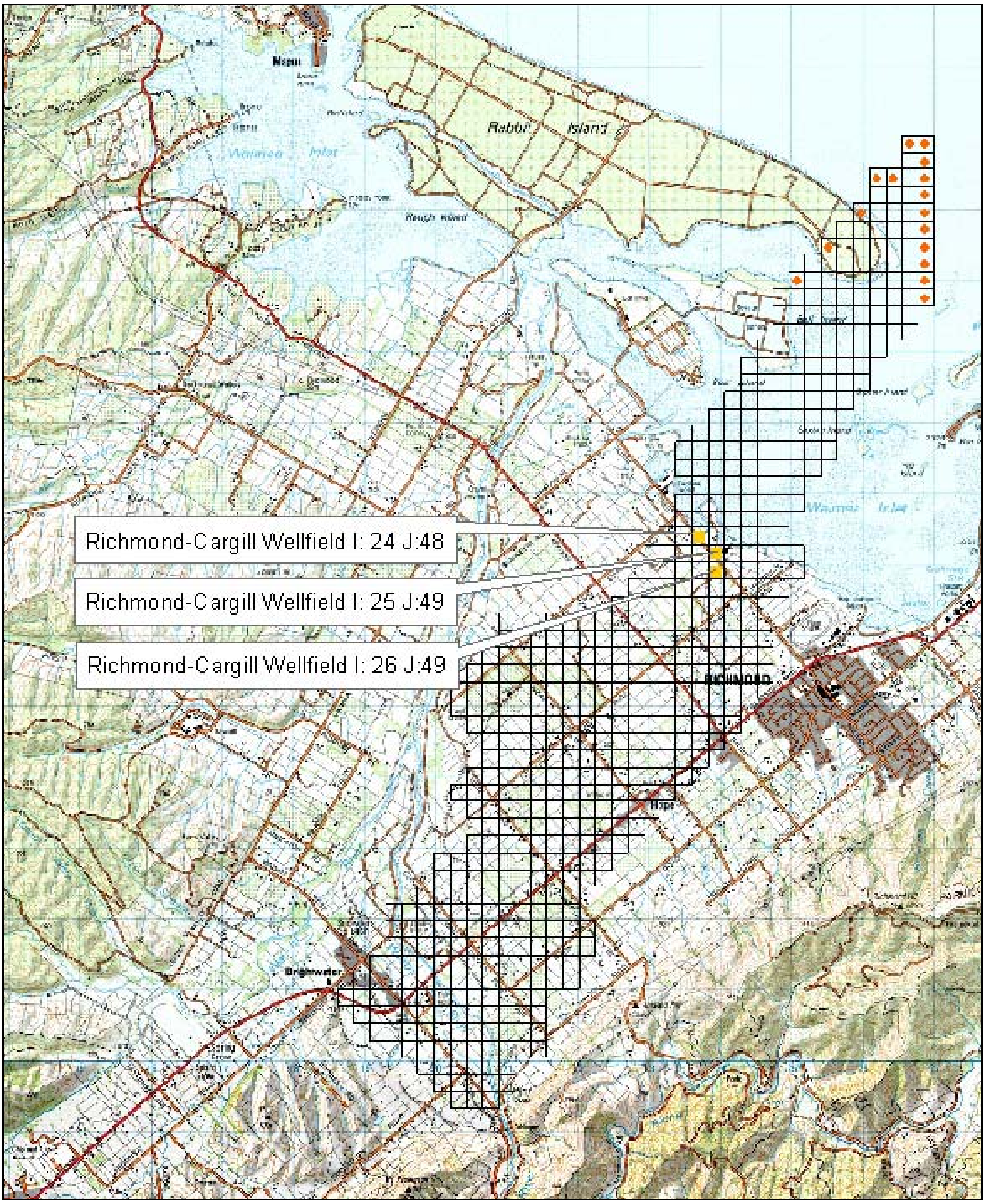

Figure 29. Pumping locations in the Lower Confined Aquifer (LCA) for the urban water demand (100 year projection).

Table 16. Urban water demand (100 year projection)

Area	Well Location	Aquifer	WWAP Feasibility Phase 1: Future Supply 50 years (m^3/day)	WWAP Feasibility Phase 2: Future Supply 100 years (m^3/day)	Comments
Richmond	Cargill	LCA	6458	**8663**	
	Appleby	LCA	815	**0**	Not used
Waimea & Mapua	Well No.5	AGUA	1324	**1776**	existing
	Well No.6	AGUA	1324	**1776**	existing
	Well No.7	AGUA	2218	**2975**	existing
	Well No.8	AGUA	2633	**3532**	existing
	Well No.9	AGUA	2633	**3532**	existing
	Well No.10 & 11	AGUA	5267	**13167**	new
	New Wellfield	AGUA	9821	**13167**	
Brightwater Hope	Existing Wellfield - Lightband	AGUA	2800	**3756**	
	New Wellfield	AGUA	602	**5899**	
Wakefield	Existing Wellfield in Wai-iti Zone	AGUA	1300	**1300**	No change from Wakefield supply 2056 prediction
	New Wellfield/Bright - in Wai-iti Zone	AGUA	2495	**2495**	No change from Wakefield supply 2056 prediction
Redwood Valley	O'Connors	AGUA		**0**	
	Golden Hills	AGUA		**0**	
	Redwood River Road existing	AGUA	683.82	**917**	
	Redwood River Rd Proposed	AGUA	626.18	**840**	
Total			41000	**63795**	

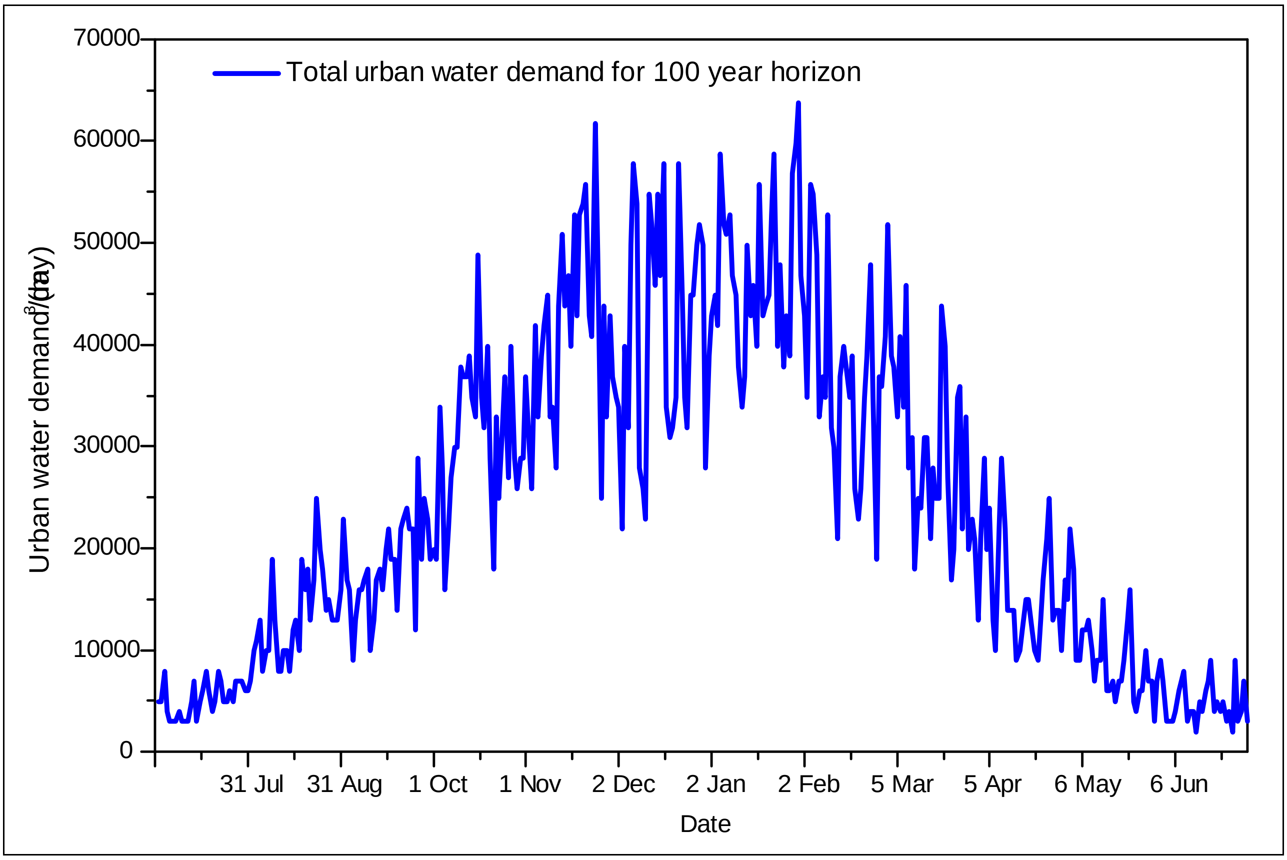

Figure 30. Time series of total urban water demand for 100 year horizon.

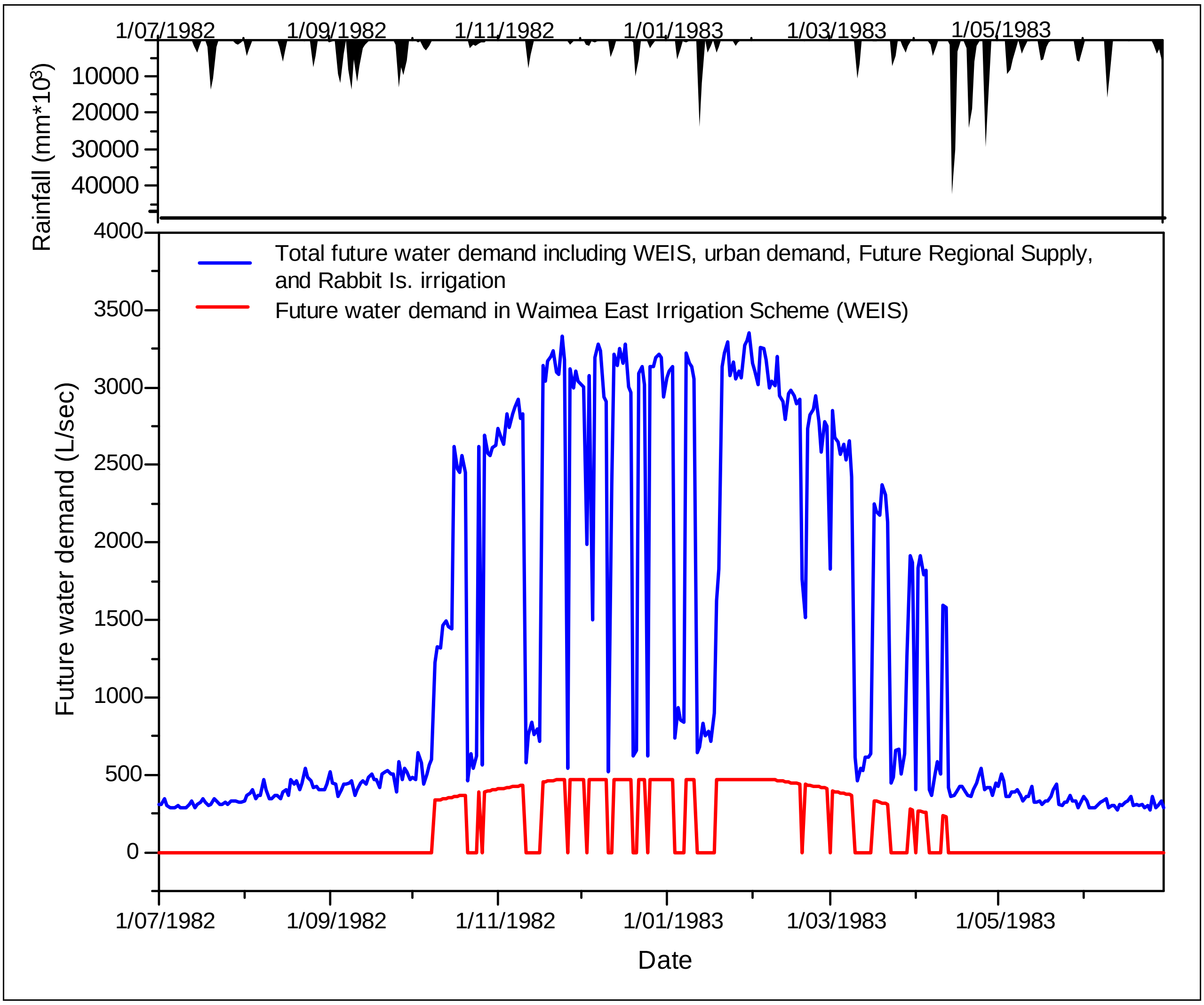

Figure 31. Time series of total future water demand calculated for the 1982/1983 drought year in the Waimea Plains.

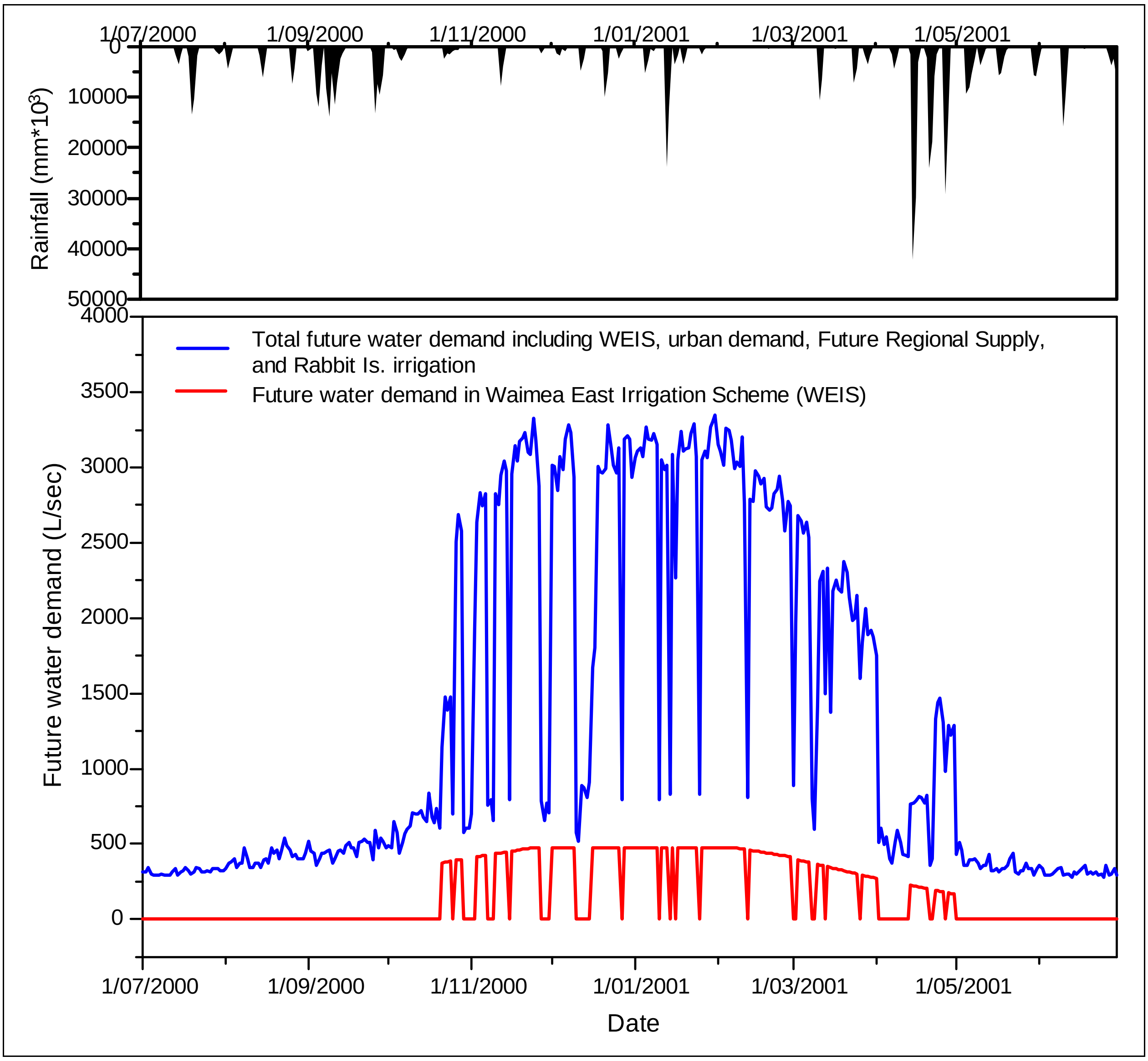

Figure 32. Time series of total future water demand calculated for the 2000/2001 drought year in the Waimea Plains.

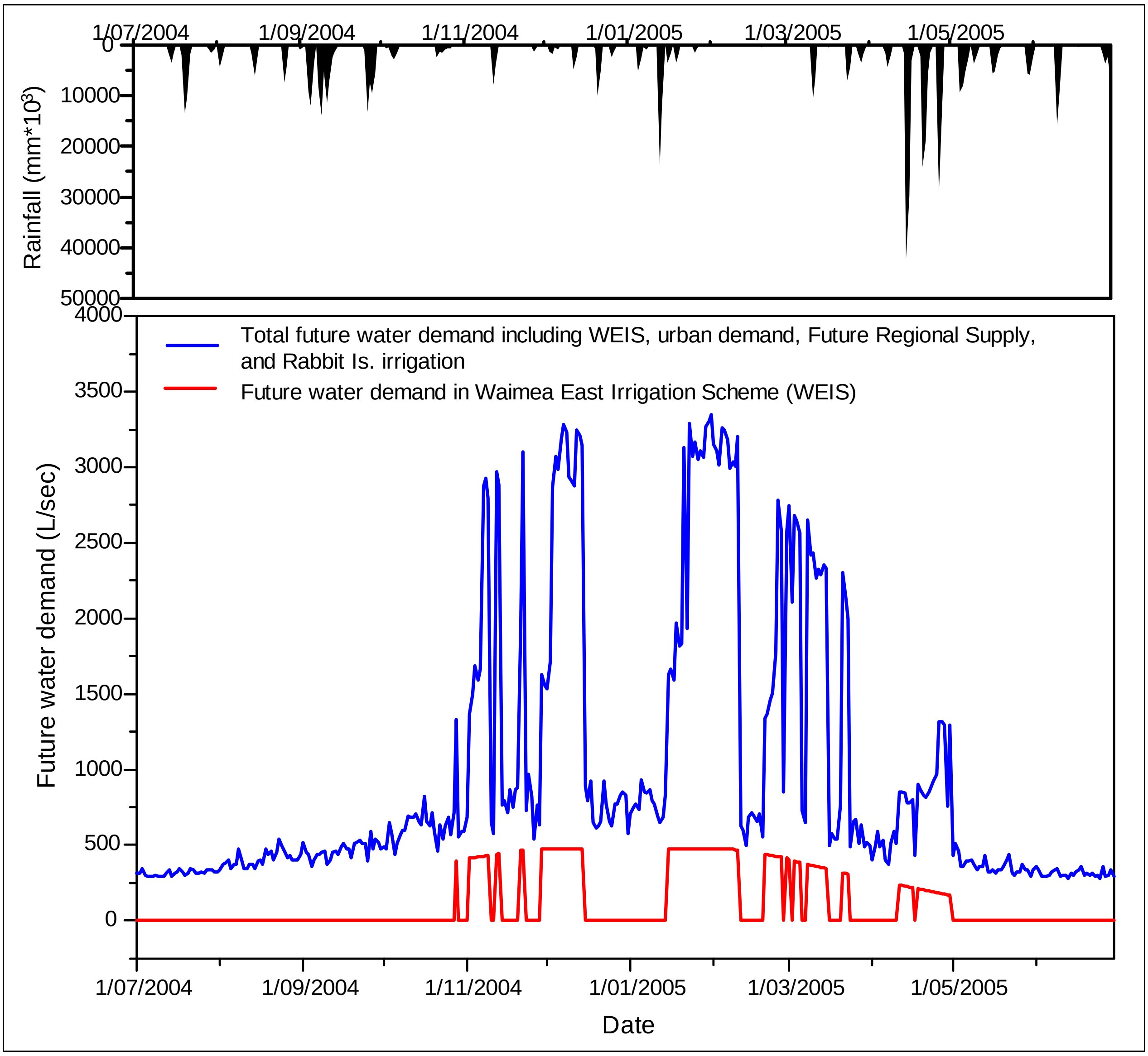

Figure 33. Time series of total future water demand calculated for the 2004/2005 average year in the Waimea Plains.

7.2 Effect of future water demand on river flows

This section evaluates the effect of future water demand on Waimea River flow as the basis for determining the required flow release from the Lee Dam to meet all water demands (consumptive and instream). The Waimea Plains groundwater-river interaction model was used to establish the frequency of 100, 250, 600, and 1,100 L/sec flows in the Waimea River at Nursery-Appleby Bridge for the 1982/1983 and 2000/2001 drought years and 2004/2005 the average year with pumpage at future water demand.

Table 17 summarises the number of days when the calculated river flow would be less than 100, 250, 600, and 1,100 L/s at Nursery-Appleby Bridge for the 1982/1983 and 2000/2001 drought years and the 2004/2005 average year. The model predicts that the river flow at Nursery-Appleby Bridge will be less than 100 L/sec on 67 days, less than 250 L/sec on 80 days, less than 600 L/sec on 98 days and less than 1,100 L/sec on 109 days for the 2000/2001 drought year if pumpage is equal to future water demand. There is a significant increase in the occurrence of zero flow, or very low flows of less than 250 L/sec, calculated for the Nursery-Appleby Bridge location for the 2000/2001 drought year, with pumpage equal to future water demand compared to actual water usage (see Figure 34).

The model calculates that river flow at the Nursery-Appleby Bridge location will not be less than 100 L/sec in the 1982/1983 drought year with actual water usage. Abstraction at future water demand is calculated to result in flow at the Nursery-Appleby Bridge location that is less than 100 L/sec on 30 days in the 1982/1983 drought year (Table 17), indicating a significant increase in the occurrence of zero flow, or very low flow conditions with future water demand. Pumpage at actual water usage is estimated to cause flow in the Waimea River at the Nursery-Appleby Bridge location to be below 250 L/sec on 7 days, below 600 L/sec on 29 days, and below 1,100 L/sec on 63 days for the 1982/1983 drought year (Table 17). The model calculates that flow at Nursery-Appleby Bridge will be less than 250 L/sec on 45 days, less than 600 L/s on 65 days, and less than 1,100 L/sec on 93 days if abstraction is at future water demand.

Calculated river flows at Nursery-Appleby Bridge for the 2004/2005 average year are shown in Figure 35. It is predicted that the average river flow at Nursery-Appleby Bridge will not be less than 1,100 L/sec even in the driest period of the 2004/2005 average year with current allocation limits.

The model calculates that in drought years like 2000/2001 and 1982/1983, low-flow conditions at Nursery-Appleby Bridge are likely to be more common if water usage is equal to future water demand, compared with actual water usage (Table 17).

Table 17. Number of days where calculated river flow is less than 100, 250, 600, and 1,100 L/sec at Nursery- Appleby Bridge for two drought years and the average year.

Year	Simulation	Number of days where river flow is less than 100 L/sec	Number of days where river flow is less than 250 L/sec	Number of days where river flow is less than 600 L/sec	Number of days where river flow is less than 1100 L/sec
2004/2005 (Average Year)	Actual water usage	0	0	0	0
	Future water demand	0	0	0	3
1982/1983 (Drought Year)	Actual water usage	0	7	29	63
	Future water demand	30	45	65	93
2000/2001 year (Drought Year)	Actual water usage	17	55	86	99
	Future water demand	67	80	98	109
2000/2001 (Drought Year)	75% of future water demand	40	57	91	100
	50% of future water demand	4	18	68	93

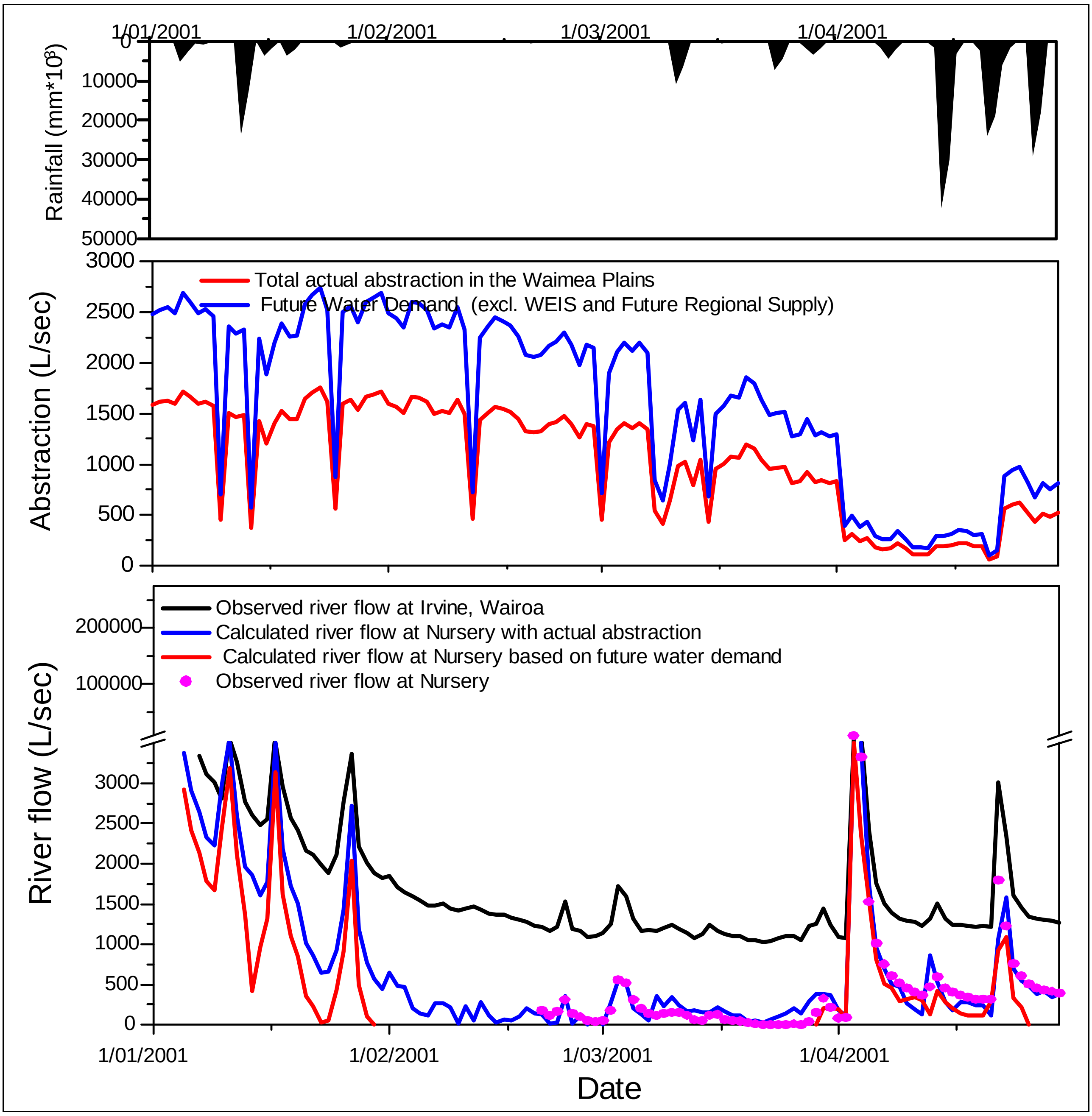

Figure 34. Observed river flow and calculated river flow at Nursery-Appleby Bridge with future water demand in the period from 1 January 2001 to 30 April 2001 drought year.

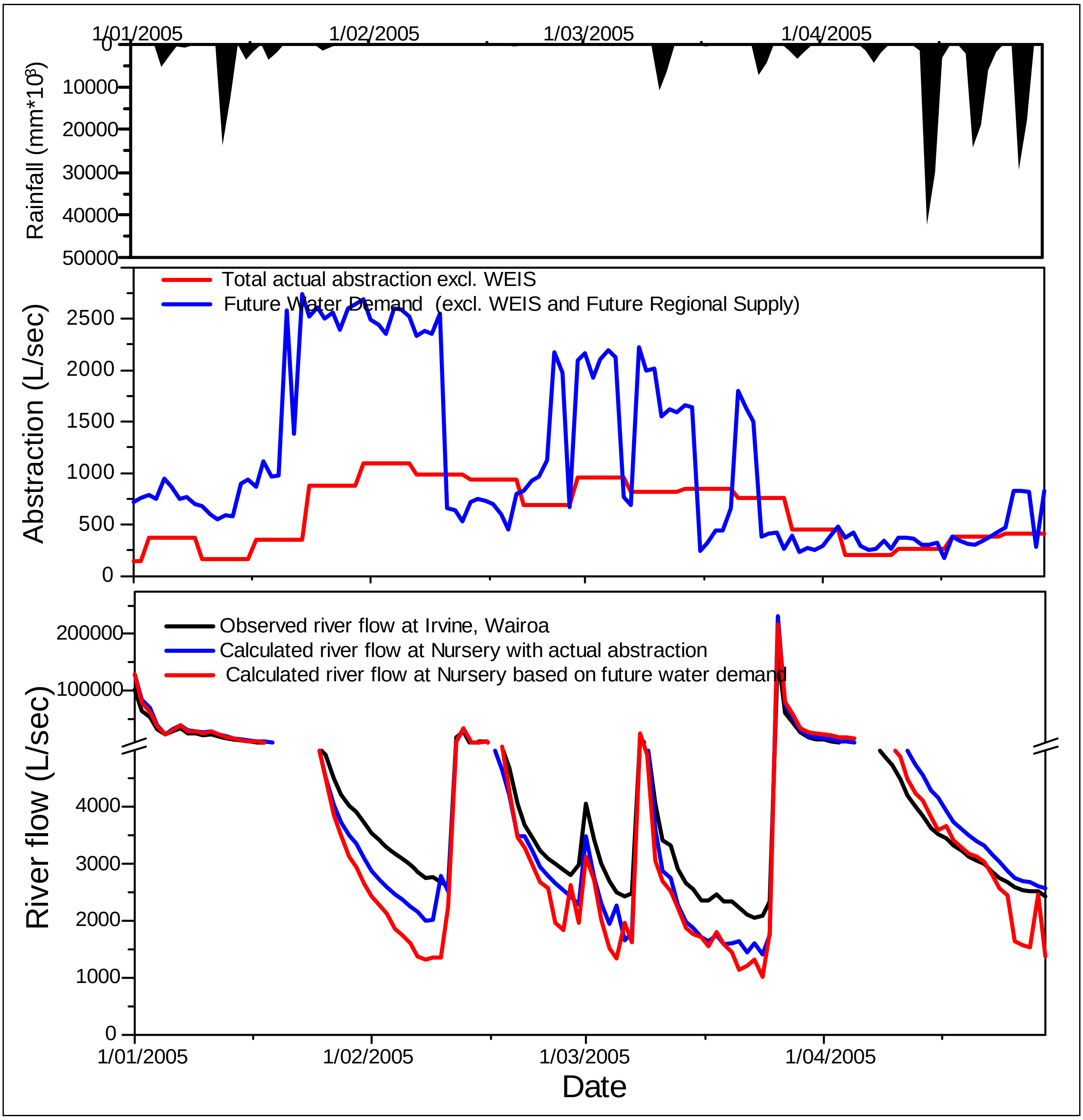

Figure 35. Observed river flow and calculated river flow at Nursery-Appleby Bridge with future water demand in the period from 1 January 2005 to 30 April 2005 average year.

7.3 Minimum river flows at Irvine-Wairoa Gorge needed to maintain minimum flow of 1,100 L/sec at Nursery-Appleby Bridge with future water demand

The Waimea Plains groundwater-river interaction model was used to calculate the river flow rate required at Irvine-Wairoa Gorge to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge in the Waimea River based on future water demand as shown in Section 7.1 for the period of 1 July through 30 June of the following year for the 1982/1983 and 2000/2001 drought years and the 2004/2005 average year.

Figure 36 shows the calculated river flows at Irvine-Wairoa Gorge needed to maintain a minimum river flow of 1,100 L/sec at Nursery-Appleby Bridge in the summer period from 1 January 2001 to 30 April 2001. The model calculates that a minimum flow at Irvine-Wairoa Gorge of 2,822 L/s would be needed to maintain flow at the Nursery-Appleby Bridge of at least 1,100 L/sec during the driest period of the 2000/2001 drought year (i.e., from 1 February through 31 March 2001) considering future water demand (Table 18).

The calculated minimum flow at Irvine-Wairoa Gorge needed to maintain the target minimum flow of 1,100 L/sec at Nursery-Appleby Bridge in the Waimea River for the 1 February through 31 March 1983 period of the 1982/1983 drought year is 2,774 L/sec (including the WEIS) taking future water demand into consideration (Table 18). Augmented river flow at Irvine-Wairoa Gorge to maintain a minimum flow 1,100 L/sec at Nursery-Appleby Bridge in the Waimea River based on a future water demand for the 1982/1983 year is shown in Figure 37.

Figure 38 shows calculated river flows at Irvine-Wairoa Gorge to maintain a minimum flow 1,100 L/sec at Nursery-Appleby Bridge in the Waimea River during the driest period of the 2004/2005 average year (1 January through 30 April 2005) taking future water demand into consideration. The unaugmented river flow at Irvine-Wairoa Gorge will maintain river flow above 1,100 L/sec at Nursery-Appleby Bridge at all times for the 2004/2005 average year with future water demand.

Table 18. Calculated minimum river flow at Irvine-Wairoa Gorge to maintain the target minimum flow of 1,100 L/s at Nursery-Appleby Bridge for the two different drought years and the average year

Year	Target minimum flows at Nursery-Appleby Bridge (L/s)	Future water demand		
		Calculated minimum river flow at Irvine-Wairoa Gorge, to maintain target minimum flow at Nursery-Appleby Bridge excluding WEIS (L/s)	Calculated minimum river flow at Irvine-Wairoa Gorge, to maintain target minimum flow at Nursery-Appleby Bridge including WEIS (L/s)	Calculated minimum river flow at Irvine-Wairoa Gorge, to maintain target minimum flow at Nursery-Appleby Bridge including WEIS and Future Regional Supply (L/s)
2000/2001 (Drought Year)	1100	2475	2822	3077
1982/1983 (Drought Year)	1100	2485	2774	3029
2004/2005 (Average Year)	1100	River flow at Nursery-Appleby Bridge would be above 1100 L/s at all time	Not applicable	Not applicable

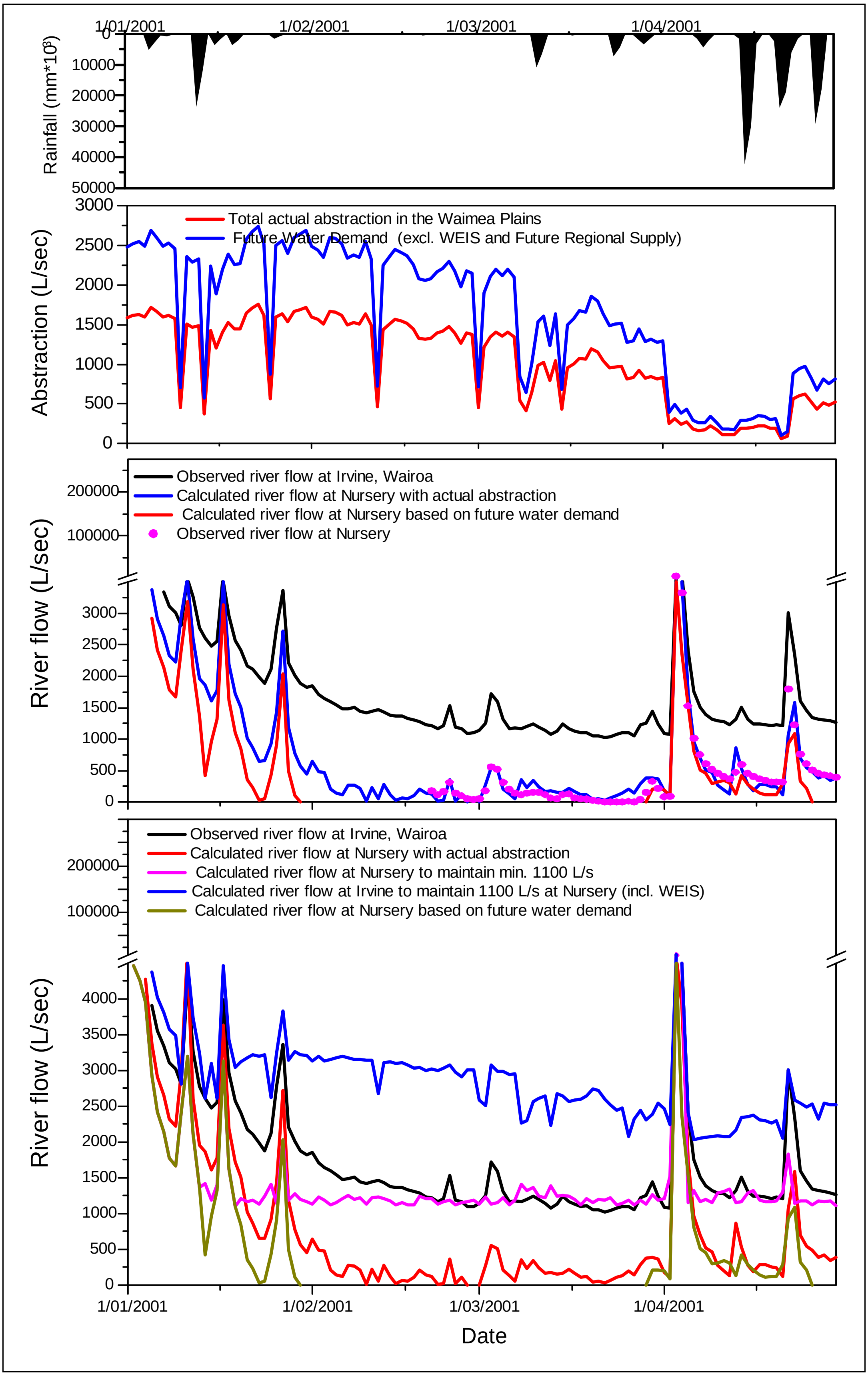

Figure 36. Calculated river flow at Irvine-Wairoa Gorge to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge in the Waimea River based on future water demand in the period from 1 January 2001 to 30 April 2001 drought year.

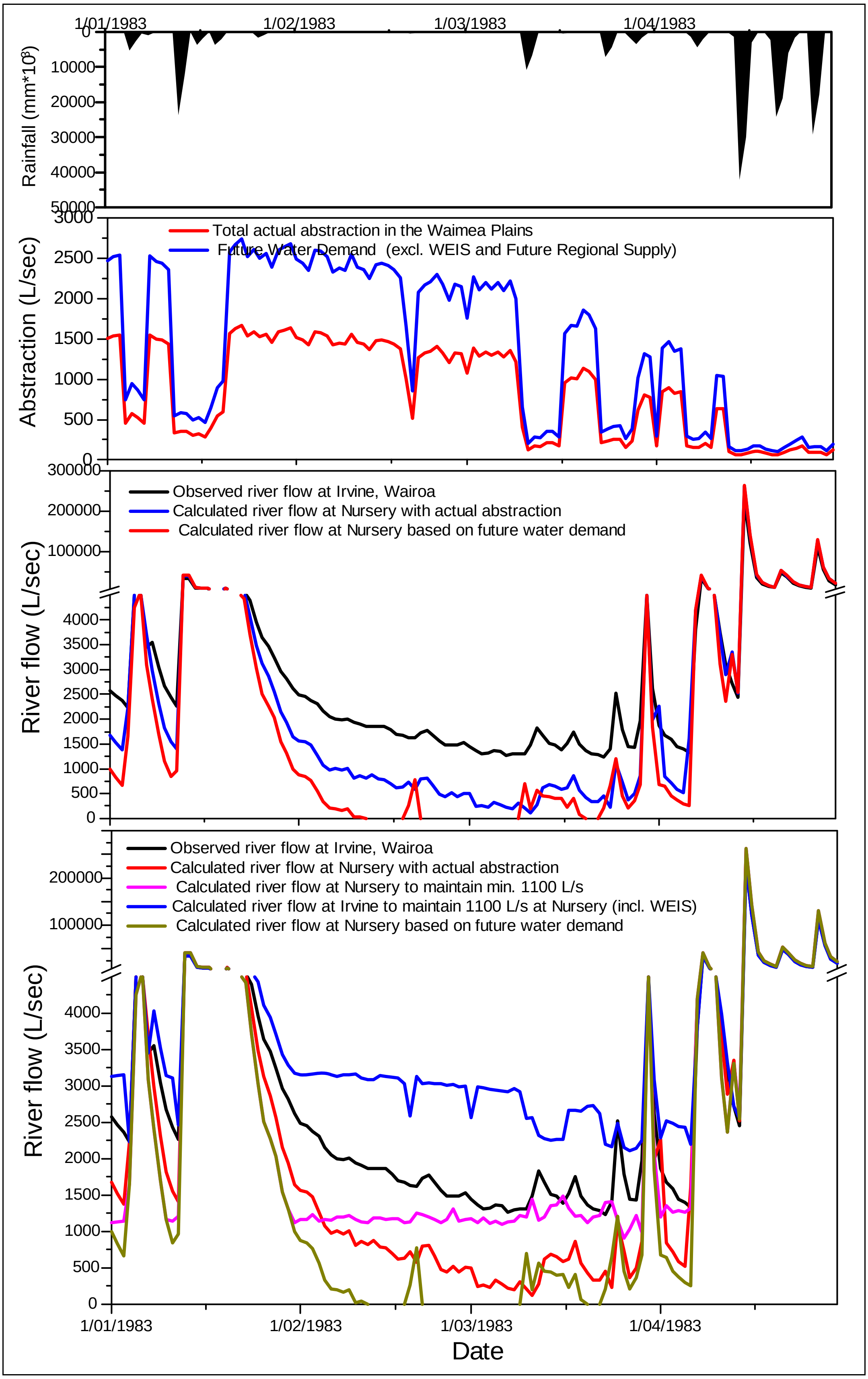

Figure 37. Calculated river flow at Irvine-Wairoa Gorge to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge in the Waimea River based on future water demand in the period from 1 January 1983 to 30 April 1983 drought year.

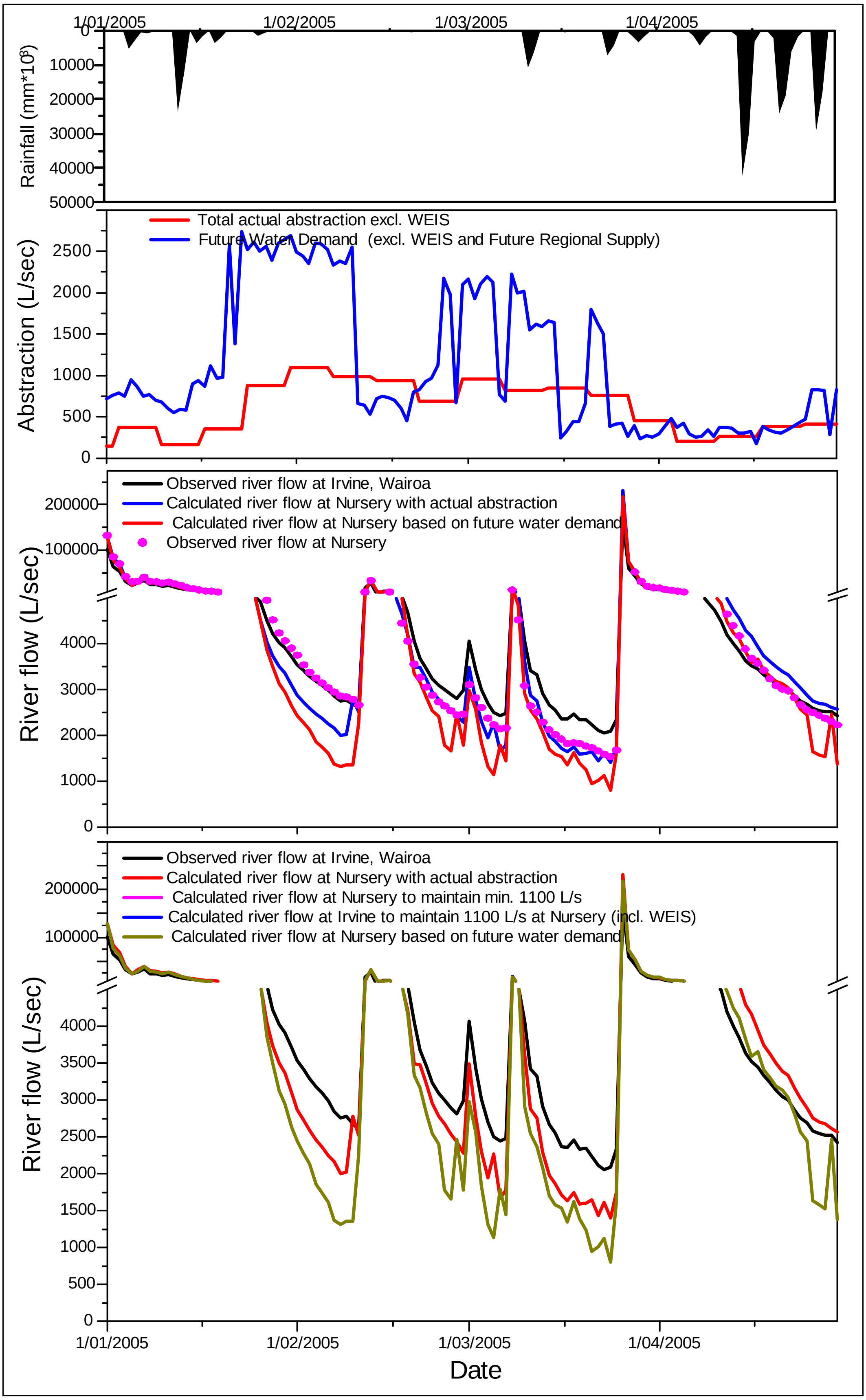

Figure 38. Calculated river flow at Irvine-Wairoa Gorge to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge in the Waimea River based on future water demand in the period from 1 January 2005 to 30 April 2005 average year.

7.4 Minimum river flows at Irvine-Wairoa Gorge needed to maintain minimum flow of 600 L/sec at Nursery-Appleby Bridge with future water demand

The Waimea Plains groundwater-river interaction model was used to establish augmented river flow rates needed at Irvine-Wairoa Gorge to maintain a minimum 600 L/sec flow at Nursery-Appleby Bridge in the Waimea River based on future water demand for the 2000/2001 drought year (driest part of the year during the period of 1 February through 31 March 2001).

The Waimea Plains groundwater-river interaction model calculated that the minimum flow at Irvine-Wairoa Gorge would need to be 2,116 L/sec (excluding the WEIS) to maintain a minimum flow of at least 600 L/sec the at Nursery-Appleby Bridge location. If WEIS take is included, the minimum flow at Irvine-Wairoa Gorge would be 2,474 L/sec (excluding 254.6 L/s of Future Regional Supply) to maintain a minimum flow of at least 600 L/sec at the Nursery-Appleby Bridge (Table 19 and Figure 37).

Table 19. Calculated minimum river flow at Irvine-Wairoa Gorge to maintain the target minimum flow at Nursery-Appleby Bridge for the 2000/2001 drought year.

Year	**Target minimum flows at Nursery-Appleby Bridge (L/s)**	**Future water demand**		
		Calculated minimum river flow at Irvine-Wairoa Gorge, to maintain target minimum flow at Nursery-Appleby Bridge excluding WEIS (1 February-31 March) (L/sec)	**Calculated minimum river flow at Irvine-Wairoa Gorge, to maintain target minimum flow at Nursery-Appleby Bridge including WEIS (1 February-31 March) (L/sec)**	**Calculated minimum river flow at Irvine-Wairoa Gorge, to maintain target minimum flow at Nursery-Appleby Bridge including WEIS and Future Regional Supply (1 February-31 March) (L/sec)**
2000/2001 (Drought Year)	1100	2475	2822	3077
2000/2001 (Drought Year)	600	2116	2474	2729

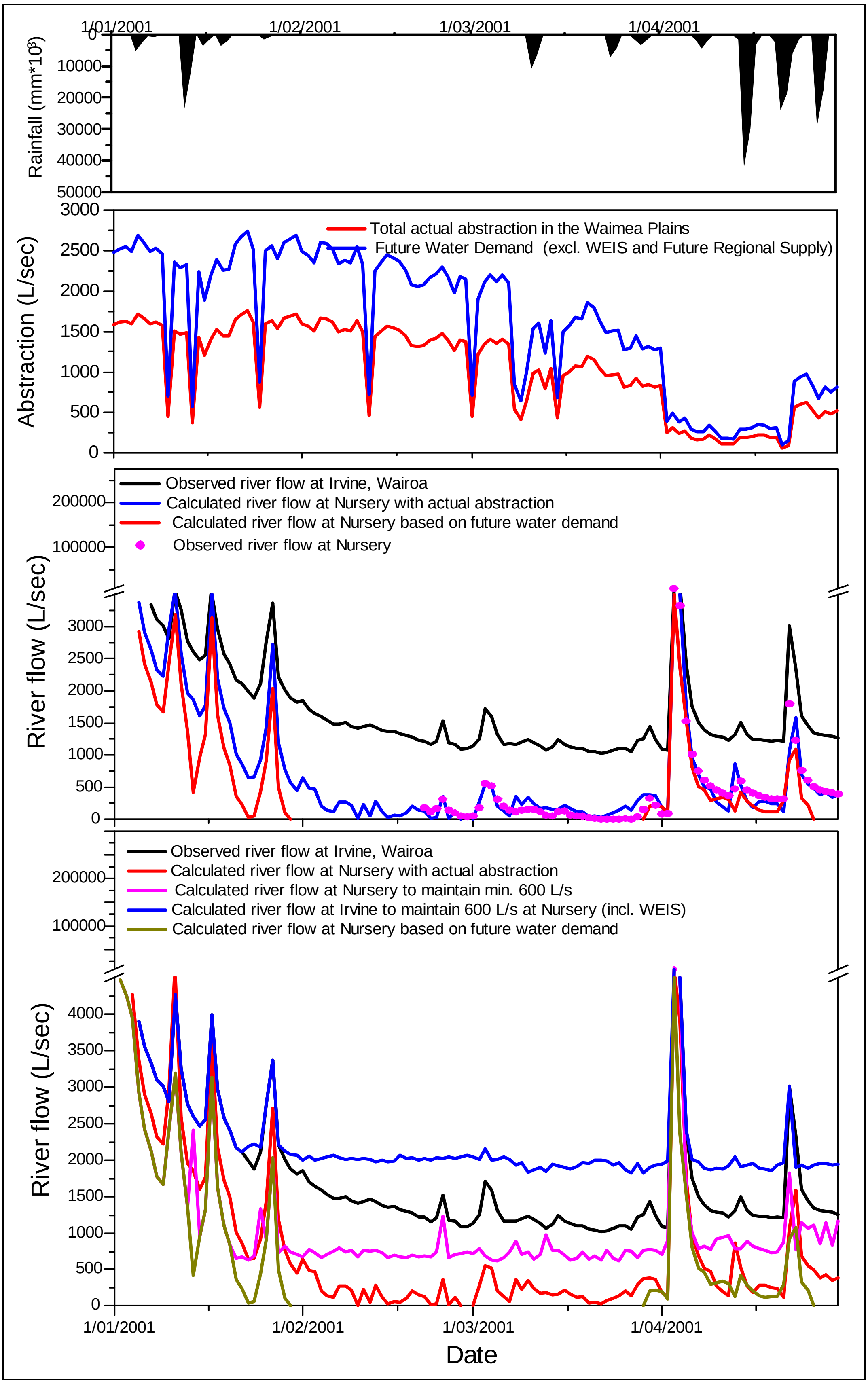

Figure 39. Calculated river flow ar Irvine-Wairoa Gorge to maintain a minimum flow 600 L/sec at Nursery-Appleby Bridge based on future water demand in the period 1 January 2001 to 30 April 2001 drought year.

7.5 Effect of 75% and 50% abstraction of the full future water demand for the 2000/2001 year on river flows

This section evaluates how partial abstraction of future water demand equivalent to 75% and 50% of the full unrestricted future water demand can influence river flows of the Wairoa/Waimea River.

Two scenarios of partial abstractive future water demands including surface water (i.e., WEIS and Future Regional Supply) are implemented:

- Scenario 1: partial abstractive future water demands equivalent to 75% of the full unrestricted future water demand for the 2000/2001 drought year;
- Scenario 2: partial abstractive future water demands equivalent to 50% of the full unrestricted future water demand for the 2000/2001 drought year.

Model calculations show that when 75% partial abstraction of the full unrestricted future water demand is applied, river flow at Nursery-Appleby Bridge:

- would be less than 100 L/sec on 40 days (compared to 67 days with pumpage at the full unrestricted future water demand), less than 250 L/sec on 57 days, less than 600 L/sec on 91 days, and less than 1,100 L/sec on 100 days in the period 1 February to 31 March 2001 (Table 17);
- would increase by about 121 L/sec (141-21 L/sec) compared with average flow when full future water demand occurs in the period 1 February to 31 March 2001 (Table 20).

Figure 40 shows river flow calculations at Irvine-Wairoa Gorge needed to maintain a minimum river flow of 1,100 L/sec at Nursery-Appleby Bridge in the summer period from 1 January to 30 April 2001 if pumpage is equal to 75% of the full unrestricted future water demand. When 75% partial abstraction of the full future water demand occurs, the minimum flow at Irvine-Wairoa Gorge would need to be 2,643 L/sec (including WEIS) to maintain a minimum flow of at least 1,100 L/sec at the Nursery-Appleby Bridge location, (compared to 2,822 L/sec with full unrestrictive future water demand) (Table 20 and Figure 40).

Model calculations also show that when 50% partial abstraction of the full unrestricted future water demand applies, river flow at Nursery-Appleby Bridge:

- would be less than 100 L/s on 4 days, less than 250 L/sec on 18 days, less than 600 L/sec on 68 days, and less than 1,100 L/sec on 93 days in the period 1 February to 31 March 2001 (Table 17);
- would increase by about 622 L/sec (643-21 L/sec) compared with average flow when full future water demand occurs in the period 1 February to 31 March 2001 (Table 20).

Figure 41 shows river flow calculations at Irvine-Wairoa Gorge needed to maintain a minimum river flow of 1100 L/sec at Nursery-Appleby Bridge in the summer period from 1 January to 30 April 2001 if pumpage is equal to 50% of the full unrestricted future water demand. The model calculation indicates that a significant decrease in the occurrence of zero flow, or very low flows of less than 250 L/sec, at Nursery-Appleby Bridge in the period of

1 February through 31 March 2001 drought year if pumpage is equal to 50% of the full unrestricted future water demand compared (see Table 17 and Figure 41). The Waimea Plains groundwater-river interaction model calculates that when 50% of the full future water demand occurs, the minimum flows at Irvine-Wairoa Gorge would need to be 2,341 L/sec (including WEIS) to maintain minimum flow of at least 1,100 L/sec at Nursery-Appleby Bridge, compared to 2,822 L/sec if pumpage is at the full unrestricted future water demand (Table 20).

Table 20. Effect of partial abstraction of the full future water demand on **c**alculated minimum river flow at Irvine-Wairoa Gorge to maintain the target minimum flow of 1100 L/s at Nursery-Appleby Bridge in the period of 1-02-2001 and 31-03-2001 (driest period) of the 2000/2001 year.

Year	**Abstraction**	**Predicted average river flow at Nursery-Appleby Bridge (L/s)**	**Future water demand**		
			Calculated minimum river flow at Irvine-Wairoa Gorge, to maintain minimum 1100 L/s flow at Nursery-Appleby Bridge excluding WEIS (L/s)	**Calculated minimum river flow at Irvine-Wairoa Gorge, to maintain minimum 1100 L/s flow at Nursery-Appleby Bridge including WEIS (L/s)**	**Calculated minimum river flow at Irvine-Wairoa Gorge, to maintain target minimum flow at Nursery-Appleby Bridge including WEIS and Future Regional Supply (L/s)**
2000/2001 year (1:24 drought year)	Full unrestricted future water demand	21	2475	2822	3077
2000/2001 year (1:24 drought year)	75% of full unrestricted future water demand	141	2295	2643	2897
2000/2001 year (1:24 drought year)	50% of full unrestricted future water demand	643	1993	2341	2596

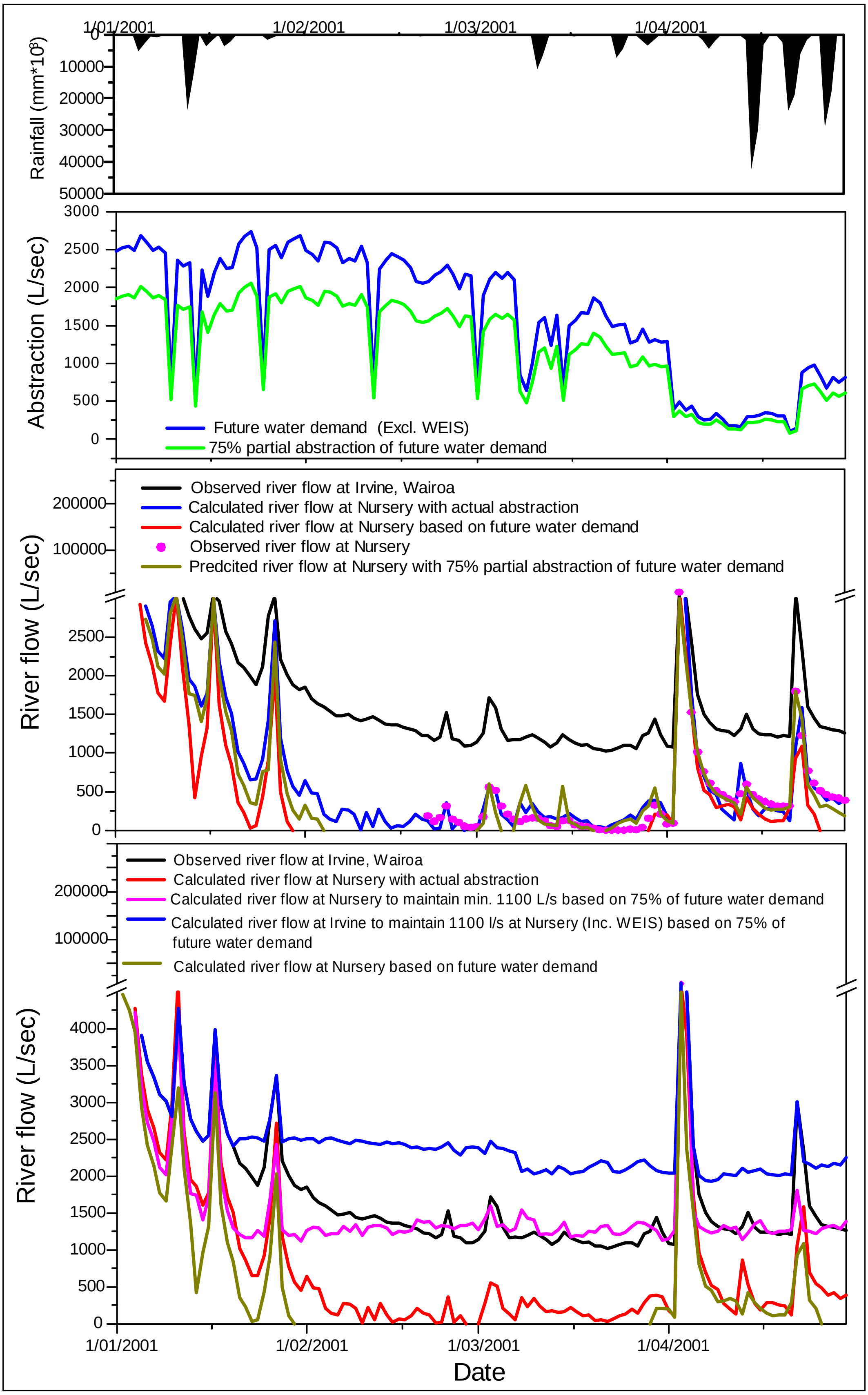

Figure 40. Calculated river flow at Irvine-Wairoa Gorge to maintain a minimum flow 1,100 L/s at Nursery-Appleby Bridge in the Waimea River based on 75% partial abstraction of future water demand in the period 1 January to 30 April 2001 drought year.

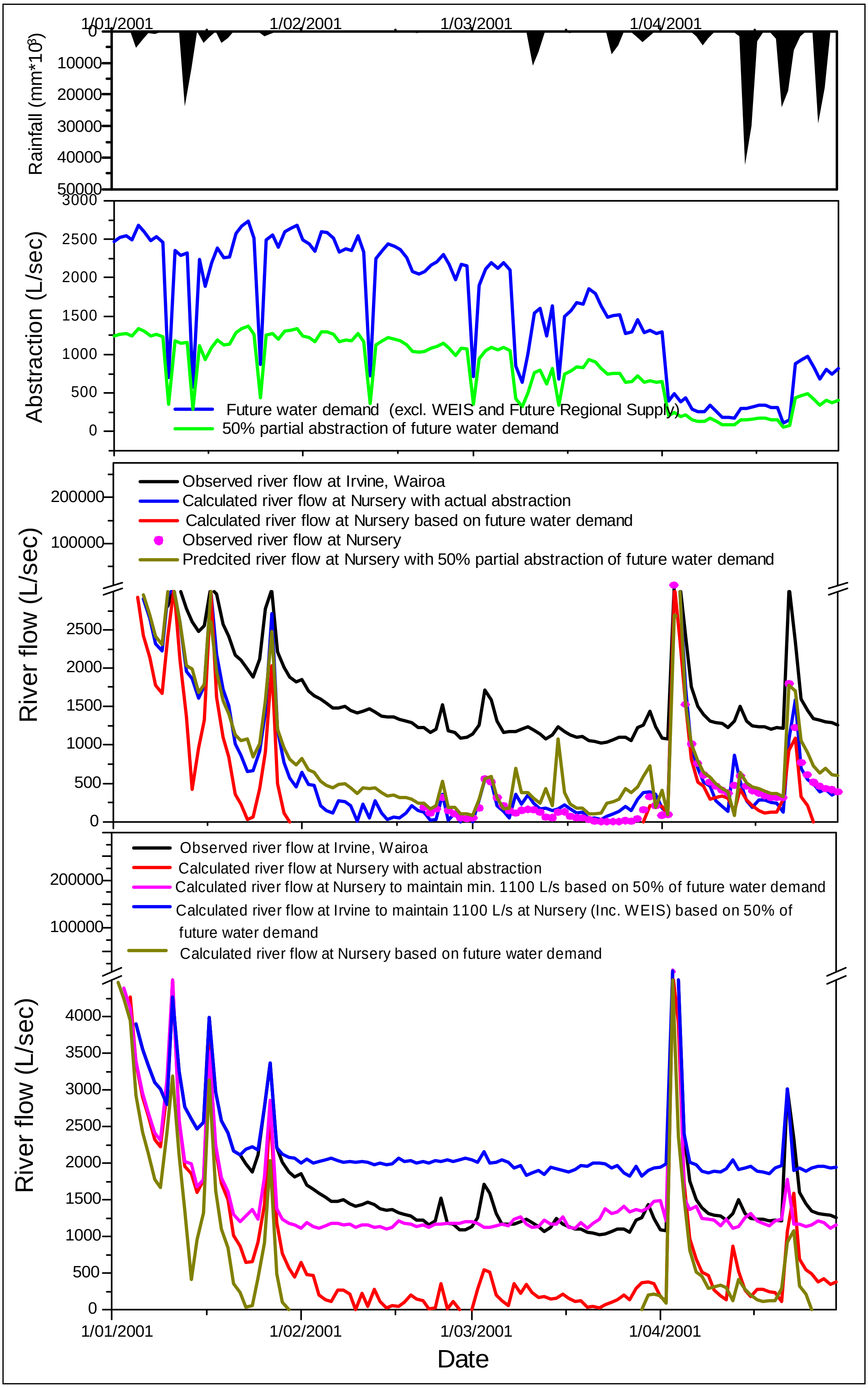

Figure 41. Calculated river flow at Irvine-Wairoa Gorge to maintain a minimum flow 1,100 L/s at Nursery-Appleby Bridge in the Waimea River based on a 50% partial abstraction of future water demand in the period 1 January to 30 April 2001 drought year.

7.6 Forward scenario simulation for Tonkin & Taylor's augmented river flows at Irvine-Wairoa Gorge

From the river augmentation modelling results at Irvine-Wairoa Gorge described in Section 7.3 (to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge), Tonkin & Taylor Ltd established a preliminary river flow augmentation regime at Irvine-Wairoa Gorge. Tonkin & Taylor Ltd provided the augmented river flow data at Irvine-Wairoa Gorge, including direct surface water abstraction (WEIS and Future Regional Supply) to GNS Science on 26 June 2008 for the 1982/1983 and 2000/2001 drought years.

GNS Science then undertook a forward simulation of the Waimea Plains groundwater-river interaction model, using this derived river flow information, to assess the ability of the proposed augmented river flow at Irvine-Wairoa Gorge to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge.

Figure 42 shows rainfall, future water demand, and observed river flows at Irvine-Wairoa Gorge and Nursery-Appleby Bridge. The forward simulation result of calculated river flow at Nursery-Appleby Bridge is also shown with observed river flow at Irvine-Wairoa Gorge and augmented river flow at Irvine-Wairoa Gorge for the period of 1 January through 30 April 2001 (drought year). The Waimea Plains groundwater-river interaction model calculated that the river flow at Nursery-Appleby Bridge would average 1,280 L/sec, based on Tonkin and Taylor's preliminary river flow augmentation regime. Tonkin and Taylor's regime has an average flow over the same period of 2,467 L/sec (excluding direct surface water abstraction of WEIS and Future Regional Supply) at Irvine-Wairoa Gorge between 1 February and 30 March 2001 (the driest period of the 2000/2001 drought year). River flow calculated for the Nursery-Appleby Bridge would be an average of 1,222 L/sec at Tonkin and Taylor's preliminary river flow of 2,125 L/sec (excluding direct surface water abstraction of WEIS and Future Regional Supply) between 5 April and 5 May 2001. However, the model calculates that river flow at Nursery-Appleby Bridge would be below 1,100 L/s on the following days:

- 26 April, 2001: 1,028 L/sec;
- 27 April, 2001: 1,067 L/sec;
- 28 April, 2001: 991 L/sec;
- 29 April, 2001: 1,001 L/sec;
- 30 April, 2001: 952 L/sec.

Forward simulation results show that the preliminary Tonkin and Taylor augmented river flow derived for Irvine-Wairoa Gorge will maintain river flow above 1,100 L/sec at Nursery-Appleby Bridge at most times, but not all times in the period April 2001. Some minor recalculation of the proposed augmented river flow would therefore be necessary to achieve flows at or above 1,100 L/sec at Nursery-Appleby Bridge at all times.

GNS Science also undertook a forward simulation of the Waimea Plains groundwater-river interaction model to assess the ability of this river flow augmentation regime to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge from 1 February through 31 March 1983 (the driest period of the 1982/1983 drought year). Results from this forward simulation are shown in Figure 43. The Waimea Plains groundwater-river interaction model calculated, based on this preliminary river flow augmentation regime having an average flow for the same period of 2,535 L/sec, that the flow at Nursery-Appleby Bridge would average 1,246 L/sec (excluding direct surface water abstraction of WEIS and Future Regional Supply). However, the model calculates that river flow at Nursery-Appleby Bridge would be below 1,100 L/sec on the following days:

- 19 February, 1983: 1,060 L/sec;
- 20 February, 1983: 819 L/sec;
- 11 March, 1983: 1,001 L/sec;
- 12 March, 1983: 991 L/sec;
- 13 March, 1983: 1,052 L/sec;
- 25 March, 1983: 873 L/sec;
- 26 March, 1983: 989 L/sec;
- 27 March, 1983: 1,038 L/sec.

Forward simulation results show that Tonkin and Taylor's preliminary augmented river flow regime for Irvine-Wairoa Gorge will maintain river flow above 1,100 L/s at Nursery-Appleby Bridge on most days, but not all days in the period March 1983. Some minor recalculation of river flow augmentation would be necessary to achieve flows at or above 1,100 L/s at Nursery-Appleby Bridge at all times.

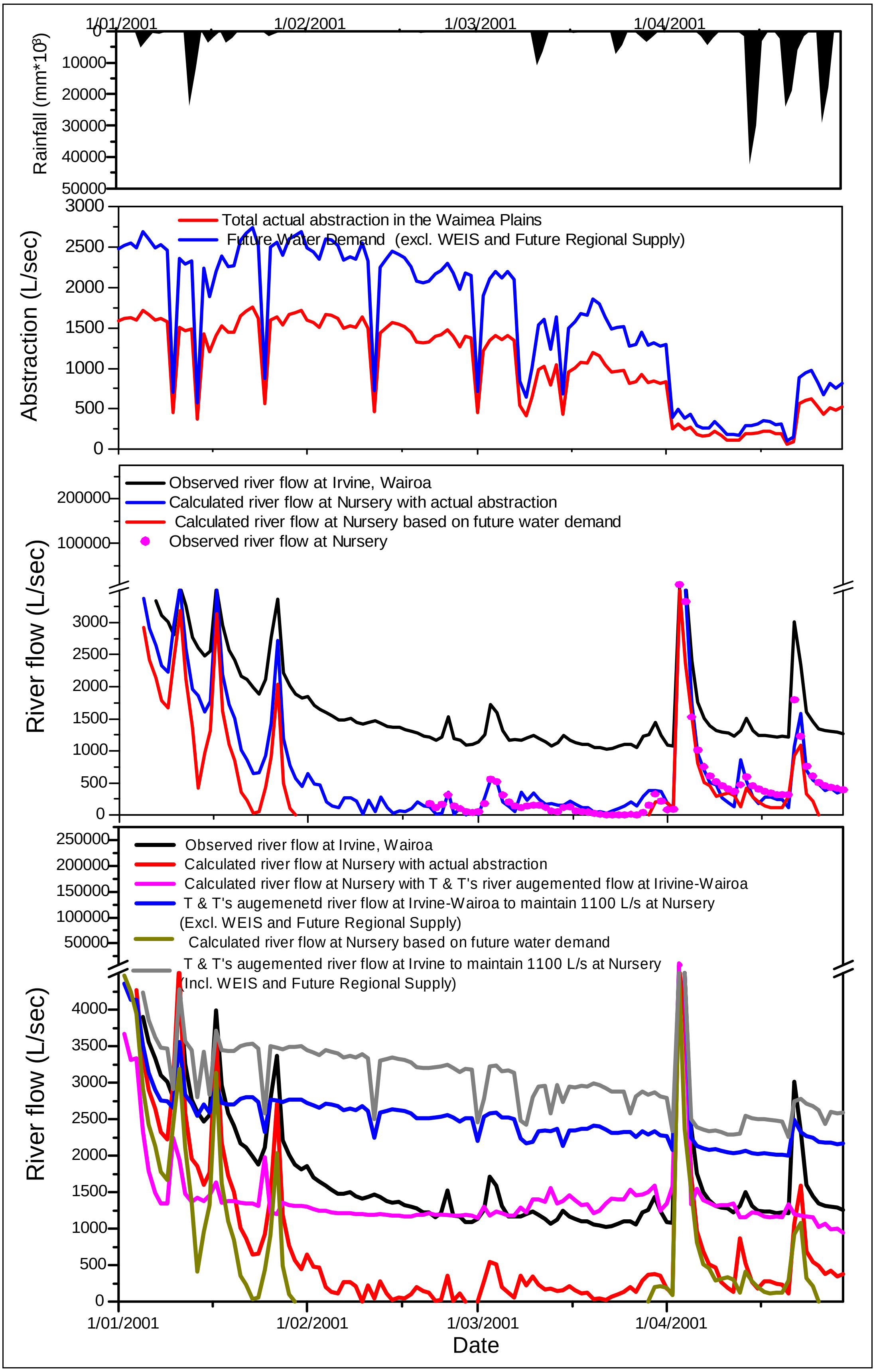

Figure 42. Forward simulation result to maintain a minimum river flow of 1,100 L/sec at Nursery-Appleby Bridge based on preliminary augmented river flow at Irvine-Wairoa Gorge in the period from 1 January to 30 April 2001 drought year.

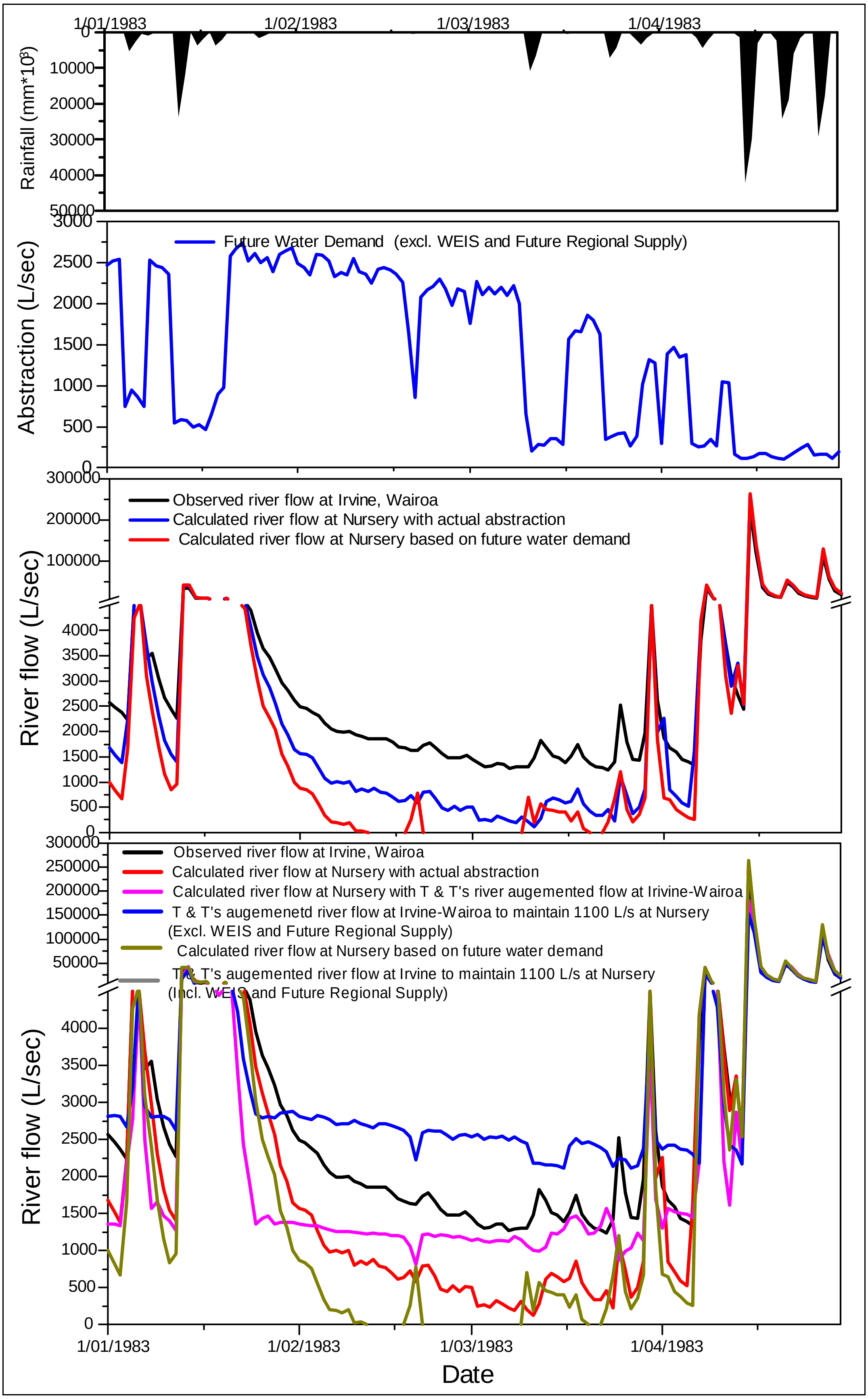

Figure 43. Forward simulation result to maintain a minimum river flow of 1,100 L/sec at Nursery-Appleby Bridge based on the preliminary augmented river flow at Irvine-Wairoa Gorge for the period 1 January through 30 April 1983 drought year.

8.0 ZONE OF RIVER RECHARGE TO AQUIFER

Groundwater recharge is predominantly by surface water losses from the Waimea and Wairoa Rivers to the associated aquifer over the summer period in the Waimea Plains. It is important to determine the "zone of effect" of this recharge by analyzing groundwater level changes in response to changes in river flow. Doing so allows mapping of the zone of river recharge to the aquifer and thereby determining those areas where water supply abstraction can best be made.

In order to identify the effect of river flows on the zone of river recharge to aquifers in the Waimea Plains, particularly at the downstream area in the Waimea River, the period of 1 January through 30 April 2001 in the 2000/2001 drought year was chosen for assessment. This was because that period is the driest period in a severe drought year without rainfall events. Thus, it is possible to ignore the effect of rainfall recharge on groundwater level changes and identify more clearly the effect of river flows on the zone of river recharge to the shallow aquifer in the Waimea Plains.

The following three steps were followed in this assessment:

1. Model augmented river flows at Irvine-Wairoa Gorge required to maintain minimum residual flows of 1,100 and 600 L/sec at Nursery-Appleby Bridge (as described in Section 7.3) for the 2000/2001 drought year).
2. Calculate the contour plot for average differences in groundwater level across the Waimea Plains based on the two minimum residual flows of 1,100 and 600 L/sec at Nursery-Appleby Bridge for the period 1 January through 31 March 2001.
3. To determine the zone of river recharge to the aquifer, generate the river recharge zone boundary line on the map of the Waimea Plains groundwater-river interaction model domain by setting a threshold head difference of 0.1 m (10 cm) and plotting locations of model areas where the difference in head exceeds that threshold for the two minimum residual flow regimes at Nursery-Appleby Bridge.

Figure 44 shows the contour plot of the average groundwater level difference (increase) between augmented river flow at Irvine-Wairoa Gorge for a minimum 1,100 L/sec at Nursery-Appleby Bridge and for historic river flow at Irvine-Wairoa Gorge in the period of 1 January through 31 March 2001. A threshold groundwater level difference of 10 cm has been set. The locations of model cells where differences in groundwater levels exceed 10 cm are plotted via contours to identify the effect of river flows on the zone of river recharge of the aquifer. The model shows that the average groundwater level adjacent to the river between survey locations RCS 5350 and RCS 8940 (see Figures 44 and 8) in the Waimea River would rise in the range of 20 to 34 cm for the 1 January to 31 March 2001 period if the augmented river flow at Irvine-Wairoa gorge for maintaining a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge is implemented. The most active zone of river recharge to the aquifer is the river network between RCS 5750 and RCS 8560 (see Figures 44 and 8), where average groundwater level is predicted to increase in the range of 30 to 34 cm.

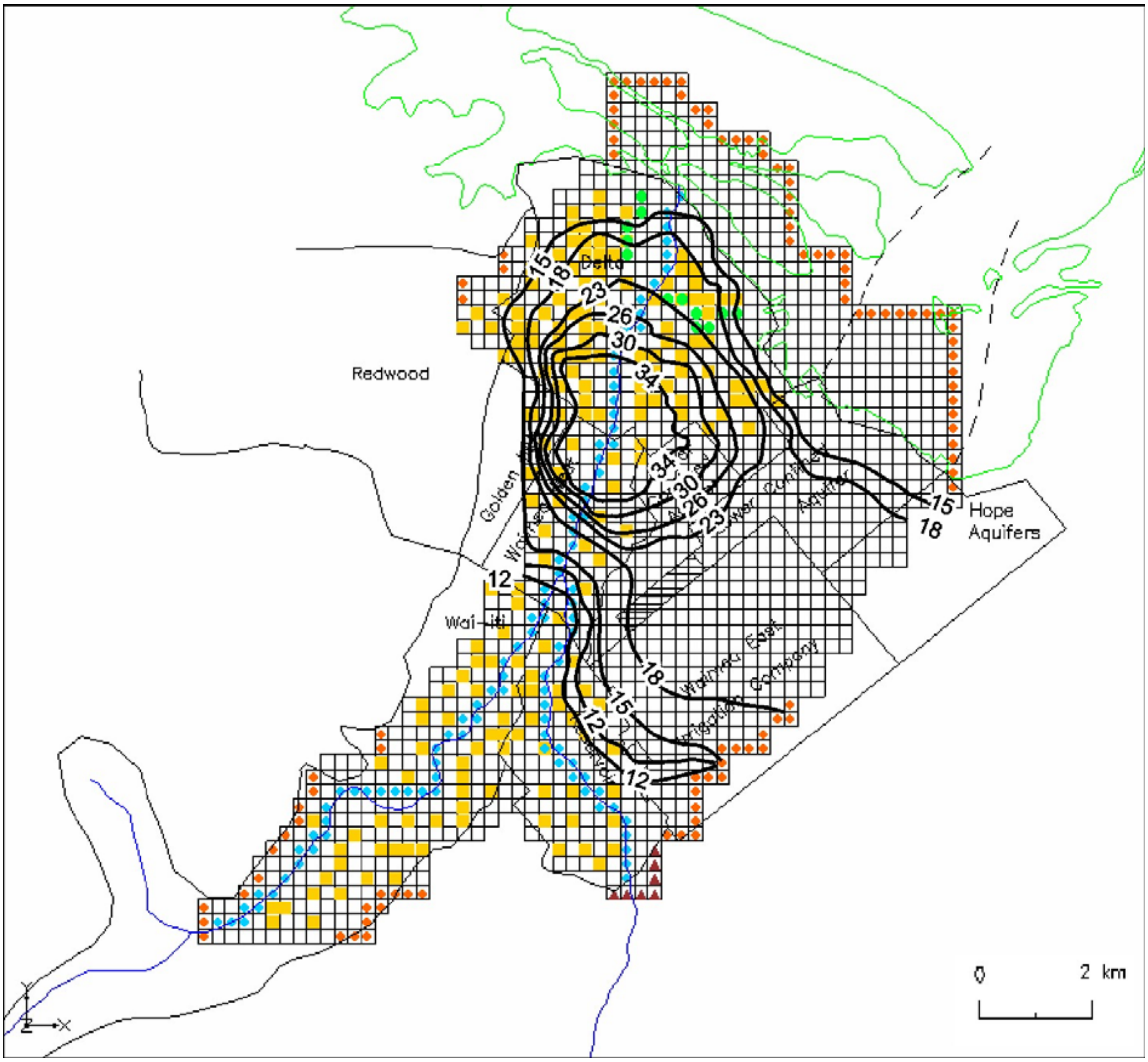

Figure 44. Zone of river recharge to the aquifer (contour plot of average groundwater level increase in cm across the Waimea Plains using augmented river flow to maintain 1,100 L/sec at Nursery-Appleby Bridge compared to without augmentation during the period of 1 January through 31 March 2001 drought year).

Figure 45 shows the contour plot of the average groundwater level difference (increase) between augmented river flow at Irvine-Wairoa Gorge for a minimum 600 L/sec at Nursery-Appleby Bridge and for historic river flow at Irvine-Wairoa Gorge in the period of 1 January through 31 March 2001. As expected, the contour plot in figure 45 shows that the average groundwater level difference (increase) with augmented flow at Irvine-Wairoa Gorge for a minimum 600 L/sec at Nursery-Appleby Bridge is smaller compared to augmented river flow for a minimum of 1,100 L/sec. The model predicts that the average groundwater level adjacent to the river between survey locations RCS 5350 and RCS 8940 (see Figures 45 and 8) in the Waimea River would rise in the range of 10 to 23 cm for the 1 January to 31 March 2001 period compared to the 20 to 34 cm range for the higher augmented river flow necessary to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge. The most active zone of river recharge to the aquifer is the river network between RCS 5750 and RCS 8560. Average groundwater levels there would increase in the range of 20 to 23 cm during this period (see Figures 44 and 8).

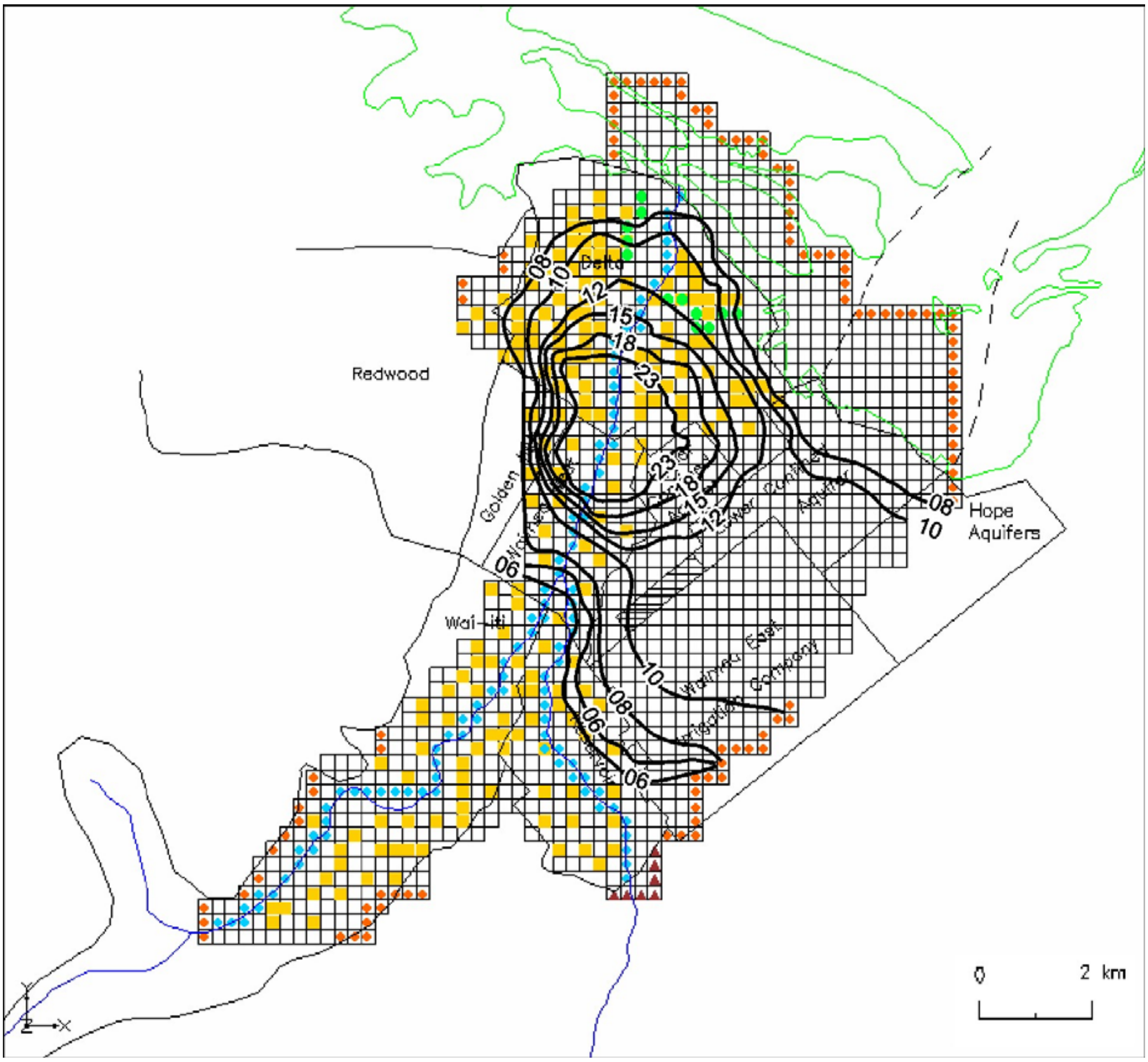

Figure 45. Zone of river recharge to the aquifer (contour plot of average groundwater level increase in cm across the Waimea Plains using augmented river flow to maintain 600 L/sec at Nursery-Appleby Bridge compared to without augmentation during the period of 1 January through 31 March 2001 drought year).

The effects of flow augmentation on the UCA, LCA, and Hope Minor confined and unconfined aquifers were not quantitatively addressed in this modeling program. However, based on general information about these aquifers (see Section 3.0), the following effects are likely:

1. Hope Minor confined and unconfined aquifers – Since these minor water-bearing lenses are laterally discontinuous, not connected to any river system, and are recharged solely from rainfall and associated runoff, there would be no effect on these aquifers from flow augmentation of the Waimea River.

2. UCA – Since the UCA is recharged from the vicinity of Wairoa Gorge and its upper confining layer is ruptured within the recharge zone and also from Appleby northwards (thereby providing hydraulic connection with the AGUA), changes produced in the AGUA would be transmitted to the UCA. Therefore, it is likely that flow augmentation would have positive effects on the UCA (i.e., resulting in increased head and resistance to seawater intrusion compared to not having flow augmentation).

3. LCA – Since the LCA is recharged from the vicinity of Wairoa Gorge it is likely that flow augmentation would also have positive effects on the LCA (i.e., resulting in increased head and resistance to seawater intrusion compared to not having flow augmentation).

9.0 SUMMARY

This Phase 2 assessment was undertaken to assess the feasibility of augmenting flow in the Wairoa-Waimea River system from storage in the upper parts of the Wairoa-Lee river catchments to enhance water availability for regional use and for environmental and community benefits. The Waimea Plains groundwater-river interaction model was used for this assessment with data for two different drought years (1982/1983 and the more severe 2000/2001 drought years) and an average year (2004/2005).

Results include:

- The Waimea Plains groundwater-river interaction model developed by GNS Science has been upgraded and recalibrated to match both groundwater levels and river flows simultaneously. To achieve this, the Stream package in the Waimea Plains groundwater flow model has been updated with new river cross section data for the Wai-iti, Wairoa, and Waimea Rivers surveyed by TDC. The rainfall-recharge model has also been updated with a new data set provided by Landcare Research. The model was then calibrated with observed of groundwater levels at two sites (CW2 and McCliskies, sites 1331098 and 1331069, respectively) and Waimea River flow at Nursery-Appleby Bridge (site 57523) using a daily time step. The groundwater levels and river flows simulated by the model matched observed data well.
- A sensitivity analysis was undertaken to analyse the effect of river bed changes on recharge of the adjacent aquifer in the Waimea River. The result indicates that river recharge to the AGUA in the Waimea Plains would be strongly influenced if river bed depletion exceeds 0.5 m. This is because the hydraulic gradient between river and aquifer decreases substantially and, as a result, river recharge flux from the Waimea River to the AGU is reduced (see Section 6.2).
- Future water demand in the Waimea Plains based on climate data for the period of 1 July through 30 June of the 1982/1983 and 2000/2001 drought years, and the 2004/2005 average year was developed in daily time step based on an irrigation water demand calculated by Landcare Research for a pasture crop over a range of soil types: 38, 78, and 130 mm soil moisture holding capacity (see Section 7.1). Urban water demand is included based on the assumption of 63,795 m^3/day of maximum daily demand for the 100 year horizon. Water demand for outside irrigation in the Brightwater/Wakefield area and Redwood Valley of 300 ha and 250 ha of irrigable area on Rabbit Island was also included in future water demand. Expected irrigable area was 5,906 ha (including 250 ha irrigable area on Rabbit Island). A total 5,923 ha of irrigable area was modelled in the Waimea Plains groundwater–interaction model. Future daily peak water demand was estimated to be 3,351 L/sec including all direct surface water abstraction (WEIS and Future Regional Supply) for the total 5,923 ha of modelled irrigable area in the model domain. Predicted annual sum of water demand for the 1982/1983 drought year was 477,295 m^3, compared to 485,159 m^3 for the 2000/2001 drought year. The predicted annual sum of water demand for the 2004/2005 average year was 350,646 m^3.

- The Waimea Plains groundwater-river interaction model was used to establish minimum augmented river flows at Irvine-Wairoa Gorge needed to maintain a minimum 1,100 L/sec flow at Nursery-Appleby Bridge based on future water demand for two different drought years (the 1982/1983 year and the more severe 2000/2001 drought year) and the 2004/2005 average year. For the 2000/2001 year, the model calculated that minimum flow at Irvine-Wairoa Gorge required would be 2,822 L/s (including WEIS) to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge during the driest period of 1 February through 31 March 2001 based on future water demand. The calculated minimum flow for the 1 February through 31 March 1983 period (the driest part of the 1982/1983 drought year) at Irvine-Wairoa Gorge needed to maintain the target minimum flow of 1,100 L/sec at Nursery-Appleby Bridge in the Waimea River is 2,774 L/sec (including WEIS) based on future water demand. The river flow at Irvine-Wairoa Gorge was predicted to maintain river flow above 1,100 L/s at Nursery-Appleby Bridge at all times for the 2004/2005 average year with future water demand .
- The Waimea Plains groundwater-river interaction model was used to establish minimum augmented river flows at Irvine-Wairoa Gorge needed to maintain a minimum 600 L/sec flow at Nursery-Appleby Bridge in the Waimea River based on future water demand calculated for the 2000/2001 drought year. It was predicted that minimum augmented river flows at Irvine-Wairoa Gorge needed to be 2,474 L/sec (including WEIS) to maintain a minimum flow of at least 600 L/sec at Nursery-Appleby Bridge, compared to 2,822 L/sec to maintain a minimum flow of 1,100 L/sec at Nursery-Appleby Bridge.
- Two scenarios were simulated to evaluate how partial abstractive future water demands equivalent to 75% and 50% of the full unrestricted future water demand can influence flows of the Wairoa/Waimea River for the 2000/2001 drought year. Model calculations indicate that when 75% of the full unrestricted future water demand was applied, river flow at Nursery-Appleby Bridge would increase by about 121 L/sec compared with average flow when full future water demand occurred in the period 1 February to 31 March 2001. It was predicted that for 50% of the full unrestricted future water demand, river flow at Nursery-Appleby Bridge would increase by about 622 L/sec compared with the average flow for full future water demand in the period 1 February to 31 March 2001. The Waimea Plains groundwater-river interaction model also calculated that when 50% of the full future water demand occurred, minimum river flows at Irvine-Wairoa Gorge would need to be 2,341 L/sec (including WEIS) to maintain a minimum flow of at least 1,100 L/sec at Nursery-Appleby Bridge, compared to 2,822 L/sec for pumpage at the full unrestricted future water demand. This model calculation indicates that a substantial decrease in the occurrence of zero flow, or very low flow of less than 250 L/sec, at Nursery-Appleby Bridge for the period of 1 February through 31 March 2001 drought year, occurs if pumpage is equal to 50% of the full unrestricted future water demand compared to full future water demand.
- GNS Science has undertaken a forward simulation to assess whether the preliminary augmented river flow at Irvine-Wairoa Gorge computed by Tonkin and Taylor Ltd could maintain a minimum 1,100 L/sec flow at Nursery-Appleby Bridge in the 1982/1983 drought year or the more severe 2000/2001 drought year. The calculated river flow at Nursery-Appleby Bridge would be an average flow of 1,222 L/s with an average augmented river flow of 2,125 L/sec (excluding direct surface water abstraction of WEIS and Future Regional Supply) between 5 April and 5 May 2001. Forward simulation results show that the preliminary augmented river flow at Irvine-Wairoa Gorge will maintain river flow above 1,100 L/sec at Nursery-Appleby Bridge for most days, but not all days in the

late-April 2001 period. The model also calculated that, based on the preliminary augmented river flow for Irvine-Wairoa Gorge of 2,535 L/sec, the average flow at Nursery-Appleby Bridge would be 1,246 L/sec (excluding direct surface water abstraction of WEIS and Future Regional Supply) between 1 February and 31 March 1983 (the driest period of the 1982/1983 drought year). However, the model calculated that river flow at Nursery-Appleby Bridge would be below 1,100 L/sec on eight days in the period of 1 February through 31 March 1983. Some minor recalculation of the proposed augmented river flow would be necessary to achieve flow above 1,100 L/s at the Nursery-Appleby Bridge at all times for these drought years.

10.0 ACKNOWLEDGEMENTS

The assistance, advice, and helpful comments of Joseph Thomas (Tasman District Council), Dr. Tim Davie (formerly of Landcare Research and now with Environment Canterbury), and John Bealing (AgFirst) are gratefully acknowledge. Similarly, Paul White (GNS Science) made valuable comments and suggestions. David Leong (Tonkin & Taylor Ltd) provided several useful comments during review of the initial draft of this report.

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www.gns.cri.nz

Principal Location

1 Fairway Drive
Avalon
PO Box 30368
Lower Hutt
New Zealand
T +64-4-570 1444
F +64-4-570 4600

Other Locations

Dunedin Research Centre
764 Cumberland Street
Private Bag 1930
Dunedin
New Zealand
T +64-3-477 4050
F +64-3-477 5232

Wairakei Research Centre
114 Karetoto Road
Wairakei
Private Bag 2000, Taupo
New Zealand
T +64-7-374 8211
F +64-7-374 8199

National Isotope Centre
30 Gracefield Road
PO Box 31312
Lower Hutt
New Zealand
T +64-4-570 1444
F +64-4-570 4657

Appendix C: Probable Maximum Precipitation calculation for Lee River Dam catchment

PROBABLE MAXIMUM PRECIPITATION

Synoptic Scale

Step 1		5968600	mN	
		2527000	mE	
Catchment Area		77.20	km2	
Step 2				
Convergence Component 24 hr 100 year, C (figure 1)		137	mm	
T/C Orographic Factor (Figure 2)		2.50		
Significance Level (Table 1)		1%		
Convergence Ratio (Table 1)		3.05		
Sea Level Convergence PMP (FAFPx)		418	mm	
Storm Intensification Factor (m, Table 2)		0.61		
24 hour Index PMP	**811**	**873**	**mm**	***From catchment averaged value in GM (CMBB)**
Step 3				
Barrier Elevation (mAD)		550		
Barrier Correction		**89.6%**		
Step 4 - Areal Reduction		**96.7%**		
Step 5		**756**	**mm**	