Wairarapa Irrigation Development Study

Prepared for
The Masterton Business Enterprise

Report No 4438/1

July 2001

MONTGOMERY WATSON

LINCOLN ENVIRONMENTAL
A division of Lincoln Ventures Ltd

Wairarapa Irrigation Development Study

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EXECUTIVE SUMMARY

This study is the first stage in the formulation of a long-term strategy for co-ordinated development of the Wairarapa Plain's water resources in a way that is acceptable to the communities on the Wairarapa Plain.

The main focus of this study was to:

- Develop an overall assessment of water demand for abstractive needs.
- Identify water surpluses and shortfalls by comparing the estimated demand with the available surface and groundwater resources.
- Develop preliminary designs for water supply schemes that could reliably meet long-term water demand.

Areas where irrigation development could proceed through private development (groundwater and localised surface water takes) were identified. The area requiring co-operative development of infrastructure was also identified, assuming potential water demand throughout the entire district was to be met. Total irrigation water demand was determined and compared to the available supply from groundwater and rivers. The storage capacity required to overcome water supply shortfalls was then estimated.

The main conclusion of the study was that there is enough water in the Wairarapa Plain's surface water and groundwater resources to:

- Provide for all existing and foreseeable non-irrigation water needs.
- Reliably irrigate all areas on the Wairarapa Plain that require irrigation for reliable production.
- Protect instream values of rivers in the Wairarapa Plain.
- Comply with the requirements of the Wellington Regional Council (WRC) Fresh Water Plan.

This conclusion is conditional on the construction of at least two surface water supply schemes to service areas of the Plain that cannot be adequately supplied from groundwater resources or adjoining rivers. The schemes would be supplied from several of the rivers in the district, and must include storage reservoirs to meet irrigation demand when summer river flows are low.

The total irrigable area in the Wairarapa Plain is approximately 122,000 hectares. The area of irrigable land that requires co-operative infrastructure for reliable irrigation is 34,500 ha.

To meet irrigation 90% of the time, assuming well-managed spray irrigation techniques, requires:

- Maximum on-farm design flow rates that range from 0.5 ℓ/s/ha for intensive pastoral farming to 0.21 ℓ/s/ha for grape production.
- Seasonal water deliveries ranging from 160 mm per year in some western areas up to 420 mm in the drier eastern areas.

Estimates of the average annual volume of water recharging groundwater show that there is sufficient groundwater available in the Wairarapa Plain aquifers to sustain the potential irrigation demand. In some areas, however, this water cannot be pumped at the rate needed to meet the irrigation demand, because of the hydrogeological properties of the aquifers. This, rather than the cost of pumping or the annual recharge, limits the area that can be supplied with irrigation water from aquifers. Water for municipal and most industrial uses can be met from groundwater because the pumping rates for these uses can be matched more easily to the aquifer's capacity to supply.

Various options for three water supply schemes were considered, and preliminary designs and costings were made for the most feasible option for each scheme. The short-listed options for each scheme are:

- *Opaki* – This is the northernmost scheme and supplies 7,500 ha of irrigable land in the upper Wairarapa Valley area. Irrigation demand is first supplied by run-of-river from the Kopuaranga River and then from reservoir storage when demand cannot be supplied directly by run-of-river. A 26 MCM (million cubic metres) reservoir is required to provide acceptable supply reliability. A suitable reservoir site is located at Highfield. The capital cost for this scheme has been estimated at $2,440/ha.

- *Greytown, Carterton & Opaki Combined* – This centrally located scheme supplies 28,500 ha of irrigable land in the upper and central Wairarapa Valley areas. Irrigation demand is first supplied by run-of-river from the Waiohine, Upper Ruamahanga and Waingawa Rivers and the Atiwhakatu Stream. When demand cannot be supplied directly by run-of-river, the shortfall would be met from a storage reservoir. A 38 MCM reservoir is required to provide acceptable supply reliability. A suitable reservoir site is located at Black Creek. Should this option be instigated, then an independent scheme to supply Opaki alone would not be required. The capital cost for this scheme has been estimated at $2,368/ha.

- *Martinborough* – This is the southernmost scheme and supplies 6,000 ha of irrigable land near Martinborough. Irrigation demand is first supplied as much as possible by run-of-river from the Huangarua River and pumped run-of-river from the Lower Ruamahanga River. When demand cannot be supplied directly by run-of-river, the shortfall would be met from a storage reservoir. A 9 MCM reservoir is required to provide acceptable supply reliability. A suitable 'turkey's nest' reservoir site is located at Te Muna. The capital cost for this scheme has been estimated at $5,500/ha.

1 INTRODUCTION

1.1 Purpose

This study is the first stage in the formulation of a long-term strategy for co-ordinated development of the Wairarapa Plain's water resources in a way that is acceptable to the communities on the Wairarapa Plain.

The purpose of Stage 1 of this study was to define the water sector "big picture" by comparing water supply and potential water demand in the Wairarapa Plain, in order to quantify water shortfalls and surpluses, and then to develop options for overcoming the water shortages.

1.2 Objectives

The objectives of the study were to:

(1) Identify the areas in the Wairarapa where:
- Irrigation is necessary to ensure production goals can be met at an acceptable level of reliability.
- Community irrigation schemes are likely to be required to meet the potential irrigation water demand.

(2) Develop conceptual designs for community irrigation schemes that are physically feasible, and likely to be economically feasible.

(3) Identify the most attractive community irrigation scheme options.

1.3 Scope

Agricultural drought is a frequent occurrence in the Wairarapa and is limiting the rate of development of higher value land-uses. Many areas are expected to benefit from irrigation, including those areas suitable for vineyards, dairy farming, and specialist seed production. Unfortunately, many land owners do not have ready access to a reliable supply of irrigation water. Summertime flows in the main rivers are not significant and are not large enough to meet all the potential demands. Some form of community development is therefore likely to be required to make full use of the rivers and of available groundwater. To address the demand, water resource development needs to be undertaken strategically to ensure optimum use of surface water and groundwater resources, and to protect the unity and cooperation evident between the communities that have an interest in irrigation development in the Wairarapa.

The scope of the study was therefore to assess the overall water balance for the Wairarapa Plain, taking into account medium- to long-term potential water demand and the capacity of groundwater and surface water to meet that demand. Where water shortages were identified, the study was to develop water supply schemes that would reliably meet water demand, and assess at pre-feasibility level their technical feasibility and cost.

The technical outputs from the study were to include:

- Ninety percentile water demand maps (as a flow rate for engineering design purposes).
- Seasonal water use maps (as a depth for water allocation purposes).
- Maps showing areas that can physically source water from either groundwater or directly from surface water, and remaining areas requiring scheme water.
- Summary of available groundwater and surface water resources.
- Summaries of water supply surpluses and limitations, and storage requirements.
- Summary of water allocation issues.
- Description of water supply scheme options developed to match water supply and demand, and their expected technical and economic feasibility.
- Summary of key environmental issues.
- Recommended development strategy.

2 DETERMINING THE POTENTIAL DEMAND FOR WATER

2.1 Introduction

Water demands within the Wairarapa Plain can be grouped into the following four main categories:

- Irrigation
- Stock water
- Commercial/industrial
- Public water supply/municipal
- Aesthetic/recreational

A review of consented groundwater takes held by the WRC (as of January 2001) indicates that 94% of the instantaneous groundwater takes within the Wairarapa Plain are currently allocated to irrigation. In addition, 39% of all consented surface water takes are allocated to irrigation. It is expected that future irrigation takes will represent an even greater proportion of the out-of-stream demand for water.

The quantum of water currently allocated for the above uses is summarised in Table 1:

Table 1: Water allocation summary

Total instantaneous takes (ℓ/s)		
Water source	**Surface water**	**Groundwater**
Irrigation	2,950	3,186
Stock water	1,635	0
Commercial/industrial	882	58
Public water supply/municipal	894	152
Aesthetic/recreational	1,250	0
Total	**7,611**	**3,396**

Given the dominance of irrigated agriculture as a consumer of water, the main focus of the water demand estimation stage of this study was on the potential demand for water for irrigation.

2.2 Irrigation Water Requirements

2.2.1 Area of Potentially Irrigable Land

The first stage in defining the potential irrigation demand was to delineate an area where irrigation is technically feasible and likely to be economically justified over the long term.

The inland boundaries of the area technically able to be irrigated were based on ground slope. Irrigation of slopes greater than 7 degrees is technically difficult. Areas of the Wairarapa with ground slopes less than 7 degrees were mapped using the NZ Land Resource Information System as a base data source. The resulting potentially irrigable land area is bounded by the base of the Rimutaka and Tararua Ranges to the west, the Aorangi Range to the south, the Wairarapa Eastern Hills to the east and Mount Bruce to the north. The resulting area used in the assessment of irrigation water requirements is shown in Figure 1.

Figure 1: Boundary of irrigable area

2.2.2 Potential Irrigated Land Uses

In order to assess potential irrigation demand, it is necessary to assume a distribution of land uses throughout the area. In doing so, it is unnecessary, and often counter-productive, to define in detail specific crops, their location and land area. The purpose in defining a land-use pattern is to provide a basis for calculating water demand, and not to prescribe a future land-use pattern. It is better to err on the safe side by assuming greater areas of high water use crops, and under-estimating the area of low water use crops.

Intensive livestock production is by far the predominant use of irrigated land in the Wairarapa Plain. This is due to the relatively high proportion of shallow and stony soils with relatively low water-holding capacities. These soils are much less versatile in terms of their suitability for cropping, but respond very well under irrigation for intensive livestock production. Irrigation of pasture for intensive livestock farming has the largest seasonal water use per hectare of any agricultural land use in New Zealand.

Well-defined areas around Martinborough and Opaki are in vineyards. It has been assumed that the provision of reliable water supplies will result in additional land being developed as vineyards, but that these areas will continue to be localised around Martinborough and Opaki. Grapes have a lower water demand than intensive livestock production and therefore the overall water requirements for these areas will be less.

Figure 2 and Table 2 present and summarise the areas assumed to be in each land-use category.

Table 2: Irrigable land area within the Wairarapa Plain study area by land use

Irrigation land type	Irrigable area (ha)
Intensive pasture	118,000
Grapes	4,000
Total	122,000

Figure 2: Potential irrigated land use

2.2.3 Irrigation Methods and Management

On-farm irrigation decisions – such as whether to irrigate, how much water to apply, and the frequency of irrigation – all influence the overall demand for irrigation. The irrigation methods and management parameters used to estimate water demand are summarised below. These parameters are typical of those used to achieve high levels of production in other east coast areas of New Zealand.

Pasture:	Irrigation return period:	7 days
	Irrigation-on trigger:	25% of soil water holding capacity
	Irrigation-off trigger:	100% of soil water holding capacity
	Sprinkler irrigation:	Application depth varied to suit soil water storage capacity at the time of irrigation
Grapes:	Irrigation return period:	1 day
	Irrigation-on trigger:	70% of soil water holding capacity
	Trickle irrigation:	Fixed depth application of 8 litres per vine per day, 2,250 vines per ha.

2.2.4 Irrigation Efficiency

The efficiency of the irrigation was assumed to be 75%. This level of efficiency is achievable with average management of well-designed spray irrigation systems. With very good management of such systems, higher levels of efficiency are being achieved.

2.2.5 Percentage of Area Irrigated

In practise, only a portion of the area of flat land that can be irrigated is actually irrigated, due to the presence of sealed areas, housing, shelter belts and other non-irrigated land uses. It is unlikely that more than 75% of the area within a scheme boundary would be irrigated. The assessment of water demand is therefore based on the assumption that the irrigated area is 75% of the gross scheme area.

2.2.6 Irrigation Demand Assessment Method

Irrigation demand is a function of climate, soil, crop, irrigation equipment, and irrigation management characteristics. To properly account for how these factors determine irrigation demand, a computer model was used to estimate daily crop water use, soil water depletion, and irrigation applications over 28 irrigation seasons from 1972/73 to 1999/2000. This process also provided an estimate of groundwater recharge volumes over the whole study area.

The study area was divided into many hydrological areas, based on soil, climate and land-use characteristics, to take account of their spatial variation through the area.

In addition, the WRC has divided the Wairarapa Plain into 30 groundwater management zones (Figure 3). Each groundwater management zone contained one or more of the hydrological areas referred to above. The groundwater management zones were used as the basis for aggregating irrigation demand, and for assessing the sustainability and hydraulic feasibility of meeting irrigation demand from groundwater.

The data used to estimate irrigation demand is described in detail in Appendix A. Irrigation demand estimates were made using Lincoln Environmental's Irrigation Scheduling and Demand Model, which is described in Appendix B. The 28-year record of historical climate data that was used for the estimates was long enough to include a range of summertime conditions – from severe droughts to periods when little irrigation would have been needed.

2.2.7 Estimated Irrigation Water Demand

The total amount of water required per irrigation season varies from year to year. The probability that specific water volumes would be able to meet total season demand was determined by estimating water demand over the 28 seasons for which climate data is available. Maps of the seasonal water use that meets requirements 50% and 90% of the time for each hydrological area are shown in Figures 4 and 5.

The results for each hydrological area and groundwater management zone are presented in Appendix C.

Figure 3: Wairarapa Plain groundwater management zones

Figure 4: 50 percentile seasonal water use

Figure 5: 90 percentile seasonal water use

2.2.8 Design On-Farm Flow Rates

The peak unrestricted water requirement for irrigation is 42.8 m^3/s for the entire Wairarapa Plain study area.

Peak flow rates were determined from the daily irrigation demand time-series that was estimated using the irrigation simulation model. Peak irrigation demand depends primarily on evapotranspiration and irrigation efficiency. It determines the flow rate the irrigation system must deliver if irrigation is to keep up with crop water demand. Crop water demand varies during a season, and from season to season. It is usually uneconomic to design irrigation systems to provide a flow rate that meets irrigation demand 100% of the time. The extra investment required to increase the design flow rate from the 90% level, for example, to the 100% level is rarely matched by increased economic benefits. The 90 percentile design application rate for each hydrological area is shown in Figure 6.

The design flow rate used in estimating scheme water demand was 0.5 ℓ/s/ha for pasture and 0.21 ℓ/s/ha for grapes. These values are similar to those used on irrigation systems installed in New Zealand in recent years, and provide farmers with the ability to maximise crop production in all but extremely dry years.

The design rate for grape irrigation is set by the choice of application rate of 8 ℓ per vine per day, and a planting density of 2,250 vines per hectare. This rate is now common in Marlborough, having reduced from 12 ℓ per vine per day.

Figure 6: 90 percentile annual application rate

2.3 Other Consumptive Water Requirements

Non-irrigation consumptive water uses may be classified as:

- Stock water takes
- Commercial/industrial takes
- Public water supply/municipal and domestic takes
- Aesthetic/recreational takes

A review of consented groundwater takes held by the WRC (as of January 2001) has been made to quantify the non-irrigation consumptive water uses. This is summarised in Appendix D.

2.3.1 Stock Water Takes

The territorial local authorities operate a system of open water races to supply stock water to farmland within the study area. Water to service the system is obtained from six intake sites sourcing water from the Mangatarere, Tauherenikau, Ruamahanga, Waingawa and Waiohine Rivers. Together, these stock water consents allow a maximum take of 1.64 m^3/s.

2.3.2 Commercial/Industrial Takes

Existing industrial permits to take water within the Wairarapa Plain area allocate a total of 0.86 m^3/s from surface water and 0.02 m^3/s from groundwater. This allocation is distributed throughout the study area for various uses, including gravel washing, and other minor industrial uses.

2.3.3 Public Water Supply/Municipal and Domestic Takes

Existing consents allow up to 0.86 m^3/s of surface water and 0.15 m^3/s of groundwater to be taken for public water supply. In addition to these consented takes, small domestic takes of up to 2.5 ℓ/s or 20 m^3/d (both groundwater and surface water) are defined as permitted activities that do not require consents under the WRC Regional Fresh Water Plan (WRC, 1999). These permitted takes are very small, given that the maximum rate of 20 m^3/day is equivalent to an average rate of 0.23 ℓ/s.

2.3.4 Aesthetic/Recreational Takes

Water takes for aesthetic or recreational purposes are an important component of the overall water demand requirements. Aesthetic or recreational takes in the Wairarapa Plain area include takes for wetland enhancement (habitat management) and as sources for artificial lakes (both privately and publicly owned). Existing aesthetic/recreational takes allocate a total of 1.25 m^3/s, sourced completely from surface water resources.

3 DEFINITION OF WATER SUPPLY ZONES

Before matching water demand with water supply, an assessment was required to assess how, and from where, each water source is likely to be accessed. Community irrigation schemes will only be required in areas where individual farms do not have affordable access to water. For farms irrigated on an individual basis, water must be taken from groundwater or from riparian corridors where it is feasible to have individual access to streams or rivers.

For the purposes of this study, the Wairarapa Plain has been divided into areas of groundwater supply zones and surface water supply zones. Groundwater supply zones are areas of land that have affordable access to irrigation water supplied from groundwater or riparian corridors. Surface water supply zones are areas of land that do not have affordable access to irrigation water from groundwater or riparian zones and must therefore source water from community irrigation schemes.

3.1 Groundwater Supply Zones

The main issues to be addressed with respect to access to reliable groundwater were development risk (would the bore yield sufficiently for irrigation?) and economics (at what pumping depth does a groundwater supply become uneconomic?). We addressed these issues by evaluating groundwater development patterns in the Wairarapa Plain and by assessing the actual construction costs of recently-developed wells.

3.1.1 Overview of the Wairarapa Plain Aquifer System

The hydrogeology of the Wairarapa Plain is complex. The plain has been in-filled by marine deposits (particularly at the lower end of the plain), which have been overlain with layers of alluvial deposits by the Ruamahanga River and its tributaries.

Generally, the lower Wairarapa Plain comprises thin sand and gravel aquifers separated by thick layers of fine grained sediments, principally clay. Aquifer transmissivities here are comparatively high, as the aquifers are comprised of reworked sands and gravels. The western areas of the plain consist of alluvial fan aquifers and, as such, comprise poorly sorted sands and gravels, with typically lower transmissivities than experienced in the lower plain. Shallower aquifers adjacent to the Ruamahanga River are composed of recently deposited gravels and generally have higher transmissivities. Many of the Wairarapa's aquifers tend to be very heterogeneous, containing thin bands of higher conductive material.

At deeper depths, aquifers tend to become more confined and are essentially separated from the shallow unconfined aquifers. Confined flowing artesian aquifers exist towards the lower plain and in a few other areas. Groundwater discharge via springs and surface seepage is a significant contributing source to many Wairarapa streams.

Close to the rivers, shallow unconfined aquifers are generally present, and are usually hydraulically connected to rivers. River infiltration forms a significant proportion of recharge to these aquifers. The lower yielding aquifers remote from the main rivers are recharged solely from rainfall infiltration.

Groundwater follows a general flow path down plain towards the coast. The groundwater system is dynamic, with groundwater levels at any time dictated by the balance of inflows and outflows.

Aquifers in the Wairarapa Plain are monitored with 18 automatic recorder sites and 36 monthly manual measuring sites. More frequent monitoring occurs in high use aquifers, with little or no monitoring occurring in low use aquifers.

3.1.2 Physical Availability of Groundwater

Prior to the 1970's, groundwater use in the Wairarapa Plain was limited to rural domestic and stock water takes. With pressure on surface water resources in the 1970's, groundwater takes increased significantly in numbers and in importance, and extensive groundwater studies were carried out. By 1989, 25 million m^3/y was allocated to groundwater takes. This has since increased to 50 million m^3/y in 2000, from 250 consented takes.

Most groundwater development has occurred at depths less than 80 m, and much of the water abstracted is from shallow unconfined or semi-confined aquifers. Deeper bores have tended to be installed in the lower plain area, with shallower bores scattered throughout the entire plain, but concentrated towards the middle and upper areas.

Modelling shows that aquifer inputs (rain, irrigation and river recharge) are significantly higher than the abstract demand from the Wairarapa aquifers. There is therefore little reason to suggest that there are any areas where groundwater will not be available. Bore specific capacities tend to be low throughout the plain. Cases where a "shortage" of water has been experienced are likely to be a function of well hydraulics and low specific capacity, rather than a true shortage of water in terms of the available resource.

3.1.3 Economic Availability of Groundwater

Based on discussions with farm advisors in mid and Central Canterbury, it was estimated that annual groundwater supply costs of $200-300 per hectare (which equates to wells of 100-200 metres depth) are likely to be acceptable to intensive pastoral farmers. These figures are consistent with current practice. It is likely that a similar acceptable cost would also apply to the Wairarapa region.

Because water underlies all of the Wairarapa Plain area and is generally available at shallow depths (typically within 20 m of the surface), groundwater is relatively cheap to abstract – significantly cheaper than the acceptable figure mentioned above. Economic factors will therefore not need to be considered in the decision about which areas should be irrigated from groundwater and which from surface supplied schemes.

3.2 Surface Water Supply Zones

The WRC groundwater management zones in which groundwater could not provide an adequate irrigation supply were defined as surface water supply zones – that is, with irrigation supplied from rivers.

The surface water supply zones selected are discussed further in Section 5.3.

4 QUANTIFICATION OF WATER AVAILABILITY

The procedures used to quantify available water are described here. The results of these procedures are presented in the appendices referred to. The ability of the quantified available water to meet demand is discussed in Section 5.

4.1 Groundwater

The reservoir characteristics of groundwater systems cushion the effects of low rainfall periods, and allow abstractions to sometimes exceed rates of natural recharge.

The availability of groundwater for abstraction is determined by:

- Inputs into the aquifers.
- The hydraulic capacity of the aquifer to support existing and future takes.
- Statutory allocations, including environmental concerns (e.g. sustaining spring and stream flows, preventing saltwater intrusion), as covered under water allocation consents and policies.

For the purposes of this study, we have assumed that all existing and potential users will have the same priority.

4.1.1 Aquifer Inputs

Groundwater flowing under the Wairarapa Plain is recharged from rivers, rainfall, and losses from stock water systems.

Conservative estimates of river recharge rates for each groundwater management zone have been made by Butcher (1995) and are summarised in Appendix E.

Rainfall recharge has been calculated from daily simulations of the soil and rainfall data used to assess irrigation demand. Summaries of groundwater recharge from rainfall as simulated are provided in Appendix F.

For this study, we have excluded recharge from stock water system losses, both because of the conservative approach taken, and because these losses are small and difficult to measure.

4.1.2 Aquifer Hydraulic Capacity

In addition to determining the overall volume of water available in the aquifers for irrigation, it is important to determine whether this water can actually be extracted. This requires an assessment of the hydraulic performance of the aquifer and of the bores abstracting the water.

Details of the assessment procedure and results are provided in Appendix G.

4.1.3 Statutory Allocations

The requirements of the WRC Fresh Water Plan limit the availability of groundwater. However, the assessments completed in this study incorporate tighter allocation limits then the Plan, and therefore further consideration of this Plan is not required.

4.2 Surface Water

The availability of surface water for abstraction is determined by:

- The natural flow in the rivers.
- Environmental flow requirements.
- Existing users.
- The water allocation regime.

Environmental flow requirements have two components:

- The flow regime required to maintain the river or stream's life-supporting capacity and its inherent characteristics; and
- The regime desired to support in-stream uses such as recreational, cultural, and amenity uses.

Environmental flow requirements and allocation regimes for Wairarapa rivers have been established through public processes and are formalised in the WRC's Regional Fresh Water Plan (WRC, 1999) and in previous allocation plan documents.

These allocation plans have been followed where possible. It has been assumed that half of the available river flow during supplementary allocation flow periods may be used for irrigation. The allocation plans for the four rivers simulated have been summarised in Appendix A.

River flow data has been obtained from NIWA and from the WRC (Wairarapa Division) for use in the assessment of surface water supply (Appendix A).

5 MATCHING WATER SUPPLY TO DEMAND

The ability of the Wairarapa Plain water resources to meet potential water demands in each water supply zone was assessed by:

1) Identifying areas where there is sufficient rainfall to meet the simulated irrigation demand, and additional irrigation is therefore not necessary.
2) Comparing seasonal water demand (irrigation + non-irrigation) in each groundwater management zone with the capacity of the zone's groundwater resources to meet demand on a sustainable basis.
3) Based on that comparison, defining those groundwater management zones as groundwater supply zones or surface water supply zones.
4) For each surface water supply zone, comparing daily time series of demand with the mean daily flow available from the relevant river source.

5.1 Necessity of Irrigation

Some soil types simulated in reliable rainfall areas require very little or no irrigation. An assessment of the necessity for irrigation was completed by looking at the ninety-percentile seasonal irrigation demand.

On soils with high water holding capacities (120-200 mm), a total seasonal water application of 250 mm or less is about two irrigation applications per season. This represents quite a low utilisation of irrigation equipment, and is probably uneconomic. With reference to Table 3, 170 and 200 mm water holding capacity soils in areas with more than 500 mm rain during the irrigation season are probably not worth irrigating. Similarly, 120 mm water holding capacity soils in areas with more than 700 mm rainfall during the irrigation season are probably not worth irrigating.

Table 3: 90 percentile seasonal irrigation demand

Rain	Average soil water holding capacity (mm)					
(mm)	50	75	100	120	170	200
400	418	391	367	373	301	-
450	418	391	367	373	301	284
500	393	385	338	345	279	251
550	400	346	303	300	250	220
600	384	335	292	279	219	191
700	384	335	292	279	219	191
800	348	267	265	200	-	162

Although some areas do not need irrigation most of the time, they have been included in all scheme water demand assessments to allow for a worst case scenario. The contribution to total demand from these areas is very small.

A summary of the areas requiring and not requiring irrigation is provided in Figure 7.

Figure 7: Summary of areas requiring irrigation (based on 90% irrigation demand)

5.2 Groundwater Zones

5.2.1 Water Availability

For each WRC groundwater management zone, the amount of water available in the aquifers was assessed and compared to the potential abstraction. This was done by matching the 50% annual demand (Appendix C) with the 50% annual rainfall/irrigation recharge (Appendix F), river recharge (Appendix E) and non-irrigation consumptive uses (Appendix D) to quantify the amount of water available for irrigation purposes. The results from this water balance assessment are presented in Table 4.

Table 4 shows that, in all groundwater management zones, the potential demand simulated is significantly less than the available water in the aquifer. The ability to irrigate within the Wairarapa Plain area is therefore not limited by the amount of water available in the aquifers.

Due to insufficient data, it has been assumed that all aquifers within each management zone act as one in terms of the water balance (i.e. that there is some vertical connection between overlying and underlying aquifers).

Table 4: Summary of aquifer water balance by WRC groundwater management zone

Groundwater management zone	Area (ha)	50% annual demand (10^6 m^3/y)	50% annual recharge (10^6 m^3/y)	Estimated river recharge (10^6 m^3/y)	Non-irrigation uses (10^6 m^3/y)	Net excess (10^6 m^3/y)
Ahikouka	1,174	1.86	5.73	0.0	0.00	3.87
Battersea	3,937	6.99	14.65	0.0	0.00	7.66
Carterton	1,481	1.98	6.12	0.0	0.53	3.62
East Taratahi	2,071	4.57	12.42	0.0	0.00	7.85
Fern Hill	3,072	8.82	12.56	0.0	0.00	3.74
Fernridge	537	0.68	3.41	0.0	0.00	2.73
Greytown	1,763	2.22	10.72	17.0	0.00	25.51
Hodders	961	1.52	5.26	0.0	0.00	3.77
Huangarua	2,360	7.09	8.53	0.0	0.00	1.44
Lower Plain	30,225	39.00	111.55	0.0	0.01	72.54
Mangatarere	908	1.31	6.05	0.0	0.00	4.73
Martinborough Tcs	3,277	8.64	10.37	0.0	0.00	1.73
Masterton	2,152	2.73	4.54	0.0	0.00	1.81
Matarawa	3,893	3.15	32.74	0.0	0.00	29.59
Middle Ruamahanga	4,038	10.69	14.57	0.0	0.19	3.68
Moroa	535	1.06	3.01	0.0	0.00	1.95
Opaki	2,215	4.23	10.56	0.0	0.00	6.33
Parkvale	5,042	10.68	23.16	0.0	0.00	12.48
Pirinoa Terraces	7,728	13.14	34.57	0.0	0.00	21.43
Rathkeale	1,290	2.15	6.22	0.0	0.17	3.91
Riverside	1,939	4.57	8.74	0.0	0.00	4.17
South Featherston	2,569	3.16	18.30	0.0	0.00	15.13
Tauherenikau	1,882	3.21	8.67	13.0	0.15	18.31
Tawaha	3,605	9.87	12.31	3.4	2.23	3.61
Te Maire Ridge	2,682	7.62	10.92	0.0	0.00	3.30
Te Ore Ore	2,787	6.77	8.70	6.3	0.30	7.93
Upper Opaki	2,164	2.89	10.12	0.0	0.00	7.23
Upper Plain	3,509	6.96	19.96	4.7	0.44	17.26
West Taratahi	2,088	4.46	15.41	0.0	0.00	10.95
Woodside	6,164	10.73	40.87	0.0	0.00	30.14
Other	13,977	31.32	66.69	0.0	0.00	35.37
Total	122,024	224.07	557.4	44.4	4.02	373.7

5.2.2 Aquifer Hydraulic Capacity

Although the assessment discussed above (Section 5.2.1) concluded that there is sufficient water available in Wairarapa Plain aquifers to sustain the potential irrigation demand, an assessment was needed to be made to determine if this water could physically be extracted from the aquifer. This was completed for each known aquifer in each WRC groundwater management zone. A description of the assessment procedure and the results are presented in Appendix G.

Aquifers were rated in terms of their hydraulic feasibility (i.e. could the aquifer support the pump rate required) as either "YES", "LIKELY", "UNLIKELY" or "NO". A summary of the best-case scenario for each groundwater management zone is presented in Figure 8. This represents the availability of groundwater from any of the aquifers within the respective zone.

Aquifers with a rating of "YES" or "LIKELY" were considered acceptable in terms of aquifer hydraulical feasibility. That is, pumping the required abstraction rate to adequately irrigate the land is likely to be possible. No additional assessment in terms of alternative irrigation water supply is necessary for these areas.

Aquifers with a rating of "UNLIKELY" or "NO" were considered unacceptable in terms of aquifer hydraulic feasibility. That is, pumping the required abstraction rate to adequately irrigate the land is not likely to be possible (or only at restricted abstraction rates). Additional assessment in terms of alternative irrigation water supply (such as from surface water) is necessary for these areas.

Five zones with a hydraulic assessment of "YES" or "LIKELY" were re-classified as "UNLIKELY" or "UNLIKELY (50%)", based on data limitations. This is discussed in Appendix G.

Areas where groundwater is available but cannot be adequately extracted (i.e. rated as "UNLIKELY", "NO", or "UNLIKELY (50%)") will require irrigation sourced from surface water. These areas are show in Figure 9.

Figure 8: Summary of aquifer hydraulic assessment rating (best-case scenario)

Figure 9: Areas requiring community irrigation scheme water

5.3 Surface Water Supply Zones

For areas where groundwater cannot be adequately extracted or does not exist (i.e. areas requiring community irrigation scheme water, Figure 9), an assessment has been made to supply these areas from surface water resources, i.e. rivers.

The groundwater management zones classified as needing scheme water are as follows:

- Battersea (50% only)
- East Taratahi
- Fern Hill
- Fernridge
- Huangarua
- Martinborough Terrace
- Matarawa
- Moroa (50% only)
- Opaki
- Pirinoa Terraces
- Rathkeale
- Te Maire Ridge
- Upper Opaki
- Te Ore Ore outlier
- Upper Plain
- Woodside
- West Taratahi

The boundary of irrigable area shown in Figure 1 is larger than the WRC groundwater management zones shown in Figure 3. The areas outside the WRC groundwater management zones (defined as "outliers") were included with the nearest groundwater zone adjacent to each outlier, with the exception of the following areas:

- Ponatahi Plain (referred to as "Ponatahi")
- Longbush Plain (referred to as "Longbush")
- The upper Ruamahnaga catchment, Mt. Bruce (referred to as "Bruce")

Due to the size of these areas, they were allocated their own surface water "zone" for assessing irrigation demand and surface water supply availability. In addition, areas which needed very little irrigation were also included to allow for the worst case scenario.

Each of the surface water supply zones was independently simulated using Lincoln Environmental's Irrigation Scheduling and Demand Model to determine the irrigation demand, by WRC groundwater management zone. This model is described in Appendix B. A time series for the irrigation demand flow was obtained for each zone.

To determine which zones were to be supplied from which river, the zones were initially grouped into the following river supply groups:

<u>Lower Ruamahanga River</u>
- Huangarua
- Martinborough Terrace
- Pirinoa Terraces
- Te Maire Ridge

<u>Mangatarere River</u>
- Battersea
- Matarawa
- Moroa
- Woodside

<u>Upper Ruamahanga River</u>
- Bruce
- Opaki
- Rathkeale
- Upper Opaki
- Te Ore Ore outlier

<u>Waingawa River</u>
- East Taratahi
- Fern Hill
- Fernridge
- Longbush
- Ponatahi
- Upper Plain
- West Taratahi

The natural flow data for each of these rivers was obtained from NIWA and from the WRC. Where required, this flow data was naturalised by adding on all non-irrigation takes to the recorded flow data. This was only necessary for rivers where the flow recorder was positioned downstream of takes from the river. It was assumed that all existing irrigation takes were negligible, as the total area of current irrigation (as per the WRC consents database) was less than 7 % of the total possible area of irrigation as simulated. This was necessary as the existing irrigation take schedule was unknown. Only the non-irrigation takes needed to be added on, which were assumed to be a constant take throughout all years.

The natural river flow was then matched with existing allocation plans to determine a time series of the flow available for irrigation. The three up-plain rivers (Mangatarere, Upper Ruamahanga and the Waingawa Rivers) each which flow "independent" of each other. The Lower Ruamahanga River, however, is contributed to by the three up-plain rivers. The available flow for the Lower Ruamahanga was therefore determined by subtracting the upstream actual irrigation take (for each of the three up-plain rivers) from the naturalised flow. The likely irrigation (take on any day) for each of the three up-plain rivers was taken as the lesser of the irrigation flow demand (as simulated) or the available river flow for each river.

The daily time series of the available flow for each of the four river supply groups was matched with the daily time series for the total irrigation demand by taking the ratio of irrigation demand to available flow. The results of these matchings are provided in Appendix H. The ratio was restricted to an upper limit of 1. A ratio of 1 implies the available water in the river can supply the full irrigation demand. A ratio of zero means no irrigation demand can be supplied at all (this is usually a function of the allocation plan rather than insufficient water being available in the river). A ratio between zero and 1 indicates that only a portion of the required irrigation demand can be supplied by the available flow in the river.

The ratios provided in Appendix H indicate that the Mangatarere and Waingawa rivers are unable to supply the irrigation demand for the zones allocated to them. The Upper and Lower Ruamahanga rivers can supply the irrigation demand to the zones allocated to them much more reliably than the other two up-plain rivers, but still not completely.

Because none of the rivers used as water sources for surface supply can fully meet irrigation demand, some form of storage will be required to provide a fully reliable supply of irrigation to the Wairarapa Plain study area.

5.4 Total Catchment Storage

5.4.1 Calculation of Total Storage Requirements

There is sufficient water in Wairarapa to fully irrigate the entire area within the boundary shown in Figure 1, but short-term storage will be required to hold surpluses from one time period for use in other times of shortfalls. Computer modelling, using daily time series of supply and demand for water, including the operation of storage facilities, was used to determine storage options. Water supply was calculated by using a daily time series of water available for abstraction from the Upper and Lower Ruamahanga, Waingawa and Mangatarere rivers (added together), after allowing for existing takes. Simulations were run over 26 years from the 1972/73 season to the 1998/99 season. Any surplus water was routed to the storage if it was needed to restore the storage to full capacity. When the supply from the rivers was insufficient to meet demand, the shortfall was met by drawing water from storage.

Because of the way the storage reservoirs have been assumed to operate, demand is always met while there is water in the reservoir. The frequency and duration of periods in which the reservoir is empty are therefore a measure of supply reliability.

5.4.2 Acceptability of Supply

There is no simple rule that determines an acceptable reliability of supply. The required reliability of supply differs with farm type and with the financial risk profile of a farm. Assessment of whether there is enough water for irrigation requires a judgement as to how often and by how much irrigation takes can be restricted before the reliability of supply becomes unacceptable. Overall figures that present the percentage of time that full demand can be met often hide key information about supply reliability, such as the length of time for which water is unavailable. For example, a 30-day period without water will pose far greater problems than will fifteen 2-day periods without water, spread throughout the irrigation season. The timing of restrictions is also important, particularly with intensive cropping, where loss of water prior to mid-January will create significant problems.

Where it has been necessary to apply reliability criteria (for example, in assessing the required storage volume), we have used the following criteria:

- The water supply was considered to be sufficiently reliable if no more than one irrigation was missed. This usually meant that a loss of water for more than about a week was unacceptable. Repeated on/off cycles further reduced the assessed reliability.

5.4.3 Total Storage Requirements

Figure 10 shows the volume of water held in storage for three storage capacities.

An 80 MCM (million cubic metres) storage enabled full irrigation on all but two occasions in the 26 years simulated: from 5 February to 12 March 1973 and from 8 to14 April 1973.

A 100 MCM storage enabled full irrigation requirements to be met on all but one occasion, from 18 February to 12 March 1973.

A 120 MCM storage enabled full irrigation requirements to be met on all occasions in the 26 years simulated.

Based on the supply reliability criteria discussed in Section 5.4.2, the 120 MCM storage met reliability criteria in all of the 26 seasons simulated.

Figure 10: Storage reservoir volumes for three reservoir capacities

6 INDIVIDUAL SCHEME CONCEPTS

6.1 Individual Scheme Selection

The land requiring scheme water is all land that cannot be adequately irrigated from groundwater, as depicted in Figure 9. Individual surface water supply zones have been grouped into four scheme zones. One scheme zone has been selected for each district. The scheme zones have been selected to encompass the majority of the areas requiring scheme water, but have logical boundaries such as major rivers, roads and foothills. The extent of the four scheme zones are shown in Figure 11 and may be summarised as follows:

Opaki	Bruce and outlier Fernridge outlier Opaki Rathkeale and outlier Upper Opaki and outlier Upper Plain
Carterton	East Taratahi Fern Hill Matarawa and outlier Upper Plain West Taratahi and outlier Woodside outlier
Greytown	Battersea (50%) Moroa Te Maire Ridge Woodside and outlier
Martinborough	Huangarua and outlier Martinborough Terraces and outlier Pirinoa Terraces and outlier

Surface water zones Ponatahi, Longbush and Te Ore Ore outlier have not been included in any scheme zone due to their location and the difficulty of supplying water to them. It is possible that these zones may be supplied directly from the streams within each zone.

Figure 11 shows scheme zones encompassing some areas that can be adequately sourced from groundwater. These areas were included within each zone only to simplify the management of the zone. The water requirements and storage assessments exclude all areas that can be adequately sourced from groundwater, even if they are within a scheme boundary.

Figure 11: Scheme supply zones

6.2 Individual Scheme Description

A brief description of each scheme area and the proposed sources of water follows. In all cases, water available for irrigation first supplies the scheme area by run-of-river, and any remaining available water is diverted into storage, if needed.

6.2.1 Opaki

Opaki is the northernmost scheme, located in the Masterton District. The land use simulated in this area is intensive pasture, except for a small allocation of grapes just north of Masterton. The total area of land requiring scheme water is 10,000 ha. However, the area east of the Waipoua River primarily comprises life style blocks that are unlikely to be irrigated. A reduced area of 7,500 ha was therefore included in the options considered.

A suitable reservoir site near the Opaki supply zone was selected. This site is located at Highfield (Figure 11). A maximum storage volume of about 35 MCM can be obtained at this site. Water can be supplied from the Kopuaranga River or the Upper Ruamahanga River.

Options considered for this supply zone will be discussed in more detail in Section 6.3.1.

6.2.2 Carterton/Greytown

Due to the close proximity of the Carterton and Greytown supply zones, these areas have been combined and supplied from a single reservoir. The Carterton and Greytown supply zones occupy a significant portion of the central Wairarapa Plain irrigable land and are located in the Carterton and Greytown districts, respectively. The land use simulated in these areas is intensive pasture, with no allowance for grapes. The total area of land requiring scheme water is 11,000 ha for Carterton and 10,000 ha for Greytown: scheme water can be supplied from the Waiohine River, Atiwhakatu Stream and Waingawa River.

A suitable reservoir site near the Carterton and Greytown supply zones was selected. This site is located at Black Creek (Figure 11). A maximum storage volume of about 58 MCM is achievable at this site.

The option of also supplying the Opaki supply zone from the Black Creek reservoir was considered. This and additional options considered for the Carterton/Greytown supply zone will be discussed in more detail in Section 6.3.2.

6.2.3 Martinborough

Martinborough is the southernmost scheme. The land use simulated in this area is intensive pasture, with grapes around and southwest of Martinborough. The total area of land requiring scheme water is 6,000 ha.

Two suitable reservoir sites near the Martinborough supply zone were selected. One site is located at Te Muna (Figure 11). A maximum storage volume of about 9 MCM can be obtained with a "turkey's nest" dam at this site. A "turkey's nest" dam is a dam constructed by excavating a hole in the ground to form a reservoir at ground level. Excavated material from the hole is then used to form embankments around the perimeter of the hole (above ground level) to further increase the reservoir capacity in a cut-and-fill type operation.

The other suitable reservoir site is located at Hautotara (Figure 11). A maximum storage volume of about 41 MCM can be obtained at this site. Water for the area can be supplied from the Huangarua River or the Lower Ruamahanga River.

An option of separating the Martinborough supply zone into two areas divided by the Martinborough-Pirinoa and Ponatahi Roads was considered. The northern portion of the supply zone (800 ha) was supplied directly from pumping the Lower Ruamahanga River and the southern portion of the supply zone (5,200 ha) was supplied by the Huangarua River and an up-plain reservoir.

Options considered for this supply zone will be discussed in more detail in Section 6.3.3.

6.3 Supply Zone Options

A brief description of the options considered for each supply zone follows. The term "run-of-river" refers to water which is taken from the river to supply some or all of the immediate irrigation demand directly (i.e. without first being stored in a reservoir).

6.3.1 Opaki Supply Zone – Highfield Storage Options

(1) The full Opaki supply zone (10,000 ha) was first supplied as much as possible by run-of-river from the Kopuaranga River. The excess available water from the Kopuaranga River was then diverted into a storage reservoir, which in turn met Opaki demand when it could not be supplied directly by run-of-river. A 26 MCM reservoir was required to provide acceptable supply reliability. The irrigation supply could be distributed throughout the supply zone via piped reticulation.

(2) The reduced Opaki supply zone (7,500 ha) was supplied in the same manner as the full supply zone described above. An 18 MCM reservoir provided no loss of supply.

6.3.2 Carterton/Greytown/Opaki Supply Zones – Black Creek Storage Options

(1) The full Carterton supply zone (11,000 ha), Greytown supply zone (10,000 ha) and Opaki supply zone (10,000 ha) were first supplied as

much as possible by run-of-river from the Waingawa River and Atiwhakatu Stream combined. The excess available water from these rivers was diverted into a storage reservoir which in turn met Carterton, Greytown and Opaki demand when it could not be supplied directly by run-of-river. A 70 MCM reservoir provided acceptable supply reliability. The irrigation supply was conveyed to each supply zone by open channels and could be distributed by either piped or open channel distribution systems.

(2) The full Carterton supply zone (11,000 ha) and Greytown supply zone (10,000 ha) was first supplied as much as possible by run-of-river from the Waingawa River and Atiwhakatu Stream combined. The excess available water from these rivers was harvested into a storage reservoir, which in turn met Carterton and Greytown demand when it could not be supplied directly by run-of-river. A 46 MCM reservoir provided acceptable supply reliability. The irrigation supply was conveyed to each supply zone by open channels and could be distributed by either piped or open channel distribution systems. The Opaki supply zone was assumed to have an independent scheme and was not included in this option. (This option was considered as a means to reduce the required reservoir capacity.)

(3) 80% of the Carterton supply zone (11,000 ha × 0.8), Greytown supply zone (10,000 ha × 0.8) and Opaki supply zone (10,000 ha × 0.8) was first supplied as much as possible by run-of-river from the Waingawa River and Atiwhakatu Stream combined. The excess available water from these rivers was then diverted into a storage reservoir, which in turn met Carterton, Greytown and Opaki demand when it could not be supplied directly by run-of-river. A 45 MCM reservoir provided acceptable supply reliability. The irrigation supply was conveyed to each supply zone by open channels and could be distributed by either piped or open channel distribution systems. (This option was considered as a means to reduce the required reservoir capacity.)

(4) The full Carterton supply zone (11,000 ha), Greytown supply zone (10,000 ha) and reduced Opaki supply zone (7,500 ha) was first supplied as much as possible by run-of-river from the Waingawa River and Atiwhakatu Stream combined. The excess available water from these rivers was then diverted into a storage reservoir, which in turn met Carterton, Greytown and Opaki demand when it could not be supplied directly by run-of-river. A 65 MCM reservoir provided acceptable supply reliability. The irrigation supply was conveyed to each supply zone by open channels and could be distributed by either piped or open channel distribution systems.

(5) The full Greytown supply zone (10,000 ha) was first supplied independently as much as possible by run-of-river from the Waiohine River. Likewise, the reduced Opaki supply zone (7,500 ha) was first supplied independently as much as possible by run-of-river from the Upper Ruamahanga River. The remaining Greytown and Opaki demands and the full Carterton supply zone (11,000 ha) was then supplied as much

as possible by run-of-river from the Waingawa River. The remaining Greytown, Opaki and Carterton demand was then supplied by run-of-river from the Atiwhakatu Stream. The excess water available from this river was then diverted into a storage reservoir, which in turn met Carterton, Greytown and Opaki demand when it could not be supplied directly by run-of-river. A 38 MCM reservoir was required to provide acceptable supply reliability. The irrigation supply was conveyed to each supply zone by open channels and could be distributed by either piped or open channel distribution systems.

6.3.3 Martinborough Supply Zone – Te Muna/Hautotara Storage Options

(1) The full Martinborough supply zone (6,000 ha) was first supplied as much as possible by run-of-river from the Huangarua River and pumped water from the Lower Ruamahanga River simultaneously. The excess available water from Huangarua River and pumped Lower Ruamahanga River was diverted into a storage reservoir, which in turn met the Martinborough demand when it could not be supplied directly by run-of-river. A 9 MCM reservoir provided no loss in supply. The irrigation supply was distributed throughout the supply zone via piped reticulation.

(2) Option 2a – The Martinborough north supply zone (800 ha) was supplied by direct pump supply from the Lower Ruamahanga River. However, reliability of supply was not sufficient, as there were times when no water was available from the Lower Ruamahanga River.

Option 2b – The Martinborough south supply zone (5,200 ha) was first supplied as much as possible by run-of-river from the Huangarua River. Any excess available water from Huangarua River was diverted into a storage reservoir, which in turn met Martinborough south demand when it could not be supplied directly by run-of-river. However, reliability of supply was not sufficient with a 9 MCM reservoir.

(3) The full Martinborough supply zone (6,000 ha) was first supplied as much as possible by pumped run-of-river from the Lower Ruamahanga River and then from run-of-river from the Huangarua River. Any excess available water from the Huangarua River was diverted into a storage reservoir, which in turn met the Martinborough demand when it could not be supplied directly by run-of-river. A 9 MCM reservoir provided no loss of supply. The irrigation supply was distributed throughout the supply zone via piped reticulation.

(4) The full Martinborough supply zone (6,000 ha) was first supplied as much as possible by pumped run-of-river from the Lower Ruamahanga River. The remaining Martinborough demand that could not be met directly by pumped run-of-river was met from a reservoir filled by the remaining pumped available water from the Lower Ruamahanga River. The Huangarua River was not utilised in this option. A 9 MCM reservoir provided acceptable supply reliability. The irrigation supply was distributed throughout the supply zone via piped reticulation.

(5) The full Martinborough supply zone (6,000 ha) was first met as much as possible by run-of-river from the Huangarua River. The remaining Martinborough demand was then met as much as possible by pumped run-of-river from the Lower Ruamahanga River. The remaining Martinborough demand that could not be supplied directly by run-of-river was supplied from a storage reservoir. The storage reservoir was first filled by any excess available run-of-river from the Huangarua, and then if needed from pumped run-of-river from the Lower Ruamahanga. A 9 MCM reservoir provided acceptable supply reliability.

This option included a "trigger-on" for pumping from the Lower Ruamahanga when the reservoir volume reached a specified percentage of the full reservoir volume. This option was designed to minimise pumping requirements and to minimise the cost of the rising main and pump station.

6.4 The Short-Listed Solutions

6.4.1 Opaki Supply Zone – Highfield Storage Option

The preferred solution for a stand-alone Opaki scheme is Option 2, discussed in Section 6.3.1. This solution minimises the storage volume required, while maintaining acceptable supply reliability to the reduced Opaki area.

A preliminary scheme design is provided in Figure 12.

6.4.2 Carterton/Greytown/Opaki Supply Zones – Black Creek Storage Option

The preferred solution to supply the combined Carterton, Greytown and Opaki supply zones is Option 5, discussed in Section 6.3.2. This solution minimises the storage volume required by maximising the use of natural run-of-river supply and maintains acceptable supply reliability to the three zones. Should this solution be instigated then an independent scheme to supply Opaki alone (Section 6.4.1) would not be required.

A preliminary scheme design is provided in Figure 13.

6.4.3 Martinborough Supply Zone – Te Muna/Hautotara Storage Option

The preferred solution to supply the Martinborough supply zone is Option 5 discussed in Section 6.3.3. This solution minimises the pumping requirements by maximising the use of natural run-of-river supply and maintains an acceptable supply reliability.

A preliminary scheme design is provided in Figure 14.

Figure 12: Highfield storage option – preliminary scheme design

Figure 13: Black Creek storage option – preliminary scheme design

Figure 14: Te Muna/Hautotara storage option – preliminary scheme design

7 PRELIMINARY ASSESSMENT OF TECHNICAL FEASIBILITY

7.1 Summary of Key Technical Issues

This study has identified a number of issues affecting the technical feasibility of the proposed water supply scehmes. The key risks concern the proposed reservoirs, which are required to provide storage for the schemes. The key risks are summarised here:

- The proposed Highfield Dam is located within close proximity of the active West Wairarapa Fault, which introduces significant dam safety issues. Other schemes should be considered in preference to those involving the Highfield Reservoir, as they are likely to involve lower technical risk.

- The proposed Black Creek Dam is located within close proximity to known faults. Although these faults are not known to be active, their presence introduces dam safety concerns. Detailed assessment of this risk will be necessary before the dam can proceed.

- The optional dam for Martinborough, the Hautotara Dam, may be founded on limestone and deep alluvial foundations. Limestone is not considered desirable in dam foundations, because it is potentially soluble in water and cavities can form, undermining the dam foundations and introducing excessive seepage. Deep alluvial foundations could lead to excessive seepage unless potentially expensive cut-off measures are undertaken. These deposits are also potentially liquefiable. The alternative Te Muna Reservoir is considered technically superior to this site.

Additional detail on the technical issues affecting the scheme components is given in the following sections.

7.2 Highfield Storage Option

7.2.1 Highfield Dam

Key Parameters

The key parameters for the proposed Highfield Dam are:

- Maximum reservoir volume = 35 MCM
- Target available storage = 18 MCM
- Design floods
  - Mean annual flood = 23 m^3/s
  - $Q_{20} = 46\ m^3/s$
  - $Q_{100} = 62\ m^3/s$
  - $Q_{200} = 69\ m^3/s$
- Diversion inflow
  - $Q_{20} = 46\ m^3/s$
- Dam layouts
  - Structural height = 23 m
  - Batter slopes = 1V:2.5H

The flood flows at this dam site have been calculated using McKerchar & Pearson (1989).

Site Description and Geology

The Highfield Dam is located on an unnamed stream, between the Ruamahanga and Kopuaranga Rivers (at map reference NZMS 260 T26:395-345). At the dam site the stream is incised in the plain floor, which is approximately 500 m wide. The abutments rise at modest slopes of approximately 3H:1V to 4H:1V. The proposed dam will have a crest length of approximately 800 m.

The proposed reservoir area extends up to 4km upstream of the dam and would flood the existing Highfield, Perrymead and Val Dor settlements. The reservoir would also flood part of Jacksons Line (road), which continues to the north above the reservoir level.

The site is logged on the New Zealand Geological Map – Sheet 12. The site is shown as being underlain by either Coquina Limestone, calcareous sandstone/siltstone or calcareous mudstone (Wanganui series sediments). These materials are likely to be overlain by weathered material and possibly loess deposits. Alluvial deposits are also indicated in the streams. Immediately downstream of the proposed dam, terrace and flood plain deposits are noted.

The West Wairarapa Fault passes within a few hundred metres (or closer) of the proposed dam site. This fault is known to be active and was the source of the 1855 earthquake in Wellington.

Embankment Design Concept and Material Sources

In order to provide the required 18 MCM storage, a reservoir with a full supply level (FSL) of approximately RL 211 is required. To achieve this, a dam up to approximately 23 m high is required across the plain.

The most economical type of dam for this site is likely to be an earthfill embankment, provided that sufficient quantities of suitable fill material can be found in close proximity of the dam site. All earth fill materials would preferentially be borrowed from within the reservoir to enhance storage volume. It is likely, however, that a significant quantity of material would be borrowed from the terrace and flood plain deposits downstream of the dam.

It is envisaged that the dam will consist of a low permeability core, higher permeability shells, internal chimney, blanket drains and rock rip-rap protection on the upstream face. Rock rip-rap consists of large rocks placed on the face of the embankment to prevent erosion by water action. The core of the dam would most likely be constructed from locally borrowed loess or flood plain silt deposits, while the shells could be constructed from similar materials, but are more likely to consist of terrace gravel deposits. Unless sand deposits of suitable grading can be found locally, materials for the internal drains would need to be imported to the site. It should be possible to quarry rock locally for rip-rap. Depending, however, on joint spacing in the rock, some larger size rock may need to be imported to the site.

It is not expected that a significant depth of alluvial materials will be present, and significant cut-off works are not envisaged for the site. It will be important to site the dam on the Wanganui series sediments rather than the terrace gravels present further downstream.

Additional investigations will have to be performed in the next phase of work to identify and confirm the location of the dam and the availability and suitability of the onsite materials (especially the availability of low permeability materials, and the suitability of the downstream sand deposits for use as filter materials).

Diversion Works and Spillways

It is envisaged that a conventional diversion conduit would be provided at the Highfield Dam site, with an upstream coffer dam and intake and downstream energy dissipation structure. Construction of these structures is expected to be relatively straightforward. The diversion works would provide the basis for outlet works in the final operation of the reservoir.

A spillway structure would be provided on the Highfield Dam to pass flood flows up to a return period of approximately 200 years. The current cost estimate assumes a gated spillway, with a lined channel and stilling basin. Other options, such as a "Morning Glory" tower, would be considered at the next stage of investigations. An auxiliary spillway could be constructed at relatively low cost on the left abutment. This spillway would enable the probable maximum flood to be passed safely, in conjunction with the primary spillway, and would also act as a back-up if, for any reason, the primary spillway was blocked.

Detailed studies on the hydrology of the plain would be required at the next stage of investigations.

Foundations

It is anticipated that the dam foundations will generally consist of weathered Wanganui series sediments. It is not anticipated that these materials would be unduly compressible, and they should provide adequate foundations for the dam. It is noted, however, that potentially soluble limestone may be encountered at the dam site and within the reservoir. This is a significant technical risk to the project, and will be necessary to undertake detailed geotechnical investigations and geological mapping at the next stage of the project to identify the presence of this material and evaluate its significance to the project.

Seismic Issues
The dam site is located in an area of high seismicity. Detailed seismic assessment of the known faults located in the vicinity of the proposed dam site will be required, in particular the active West Wairarapa Fault, in the next phase of work. The peak ground accelerations generated by the fault will be extremely high. The estimated magnitude of peak ground accelerations for the proposed dam site under the Maximum Credible Earthquake is likely to be in excess of 1 g. Movement of secondary features that may underlie the dam, such as shear zones, may be triggered by a fault rupture. These issues present the greatest technical risk to the project and need to be addressed by detailed investigation and mapping at the next stage.

However, it should be possible to site the dam so that it is not directly affected by fault rupture. The proposed batter slopes should be capable of withstanding the effects of the potential seismic loading, and it is unlikely that liquefiable zones exist within deposits under the footprint of the dam. This will have to be confirmed through future investigations and analyses in the next phase of design work.

Construction
Construction of the proposed embankment dam is expected to take two summer construction seasons. It is envisaged that construction of the dam would be relatively straightforward, with conventional construction techniques used throughout the project.

7.2.2 Kopuaranga Intake

Key Parameters
The key parameters for the Kopuaranga River intake are:

• Maximum diverted flow	1.0 m^3/s
• Flood with 100-year return period	175 m^3/s (approx.)
• River bed width	75 m (assumed)
• River bed slope	1:125

The flood flow at this site has been estimated using 50-year return period floods estimated from data recorded on the Kopuaranga River.

Site Description
The intake site was not visited during this study, and this site description is based on observations from 1:50,000 topographical map NZMS 260 T25. The proposed intake site is located at co-ordinate T25:420-364. At this location the Kopuaranga River flows around a bend, indicating that the outside of the bend is resistant to erosion. It is therefore anticipated that the river is well confined at this location.

On the true right bank the ground rises relatively steeply, at slopes of approximately 3H:1V, while on the left bank a relatively flat area is observed.

Intake Concept

The intake would consist of the following:

- A low (up to 2 m high) rock fill weir across the plain to divert water to the intake and provide control to water levels at the intake.
- A reinforced concrete sluice/intake channel on the right bank, with walls up to 4 m high and 20 m long.
- A sluice gate in the sluice/intake channel to provide control for water levels at the intake screens and to keep the intake clear of bed load.
- A screened intake in the right wall of the sluice/intake channel, leading the water into the downstream conduit (Ø 0.9 m pipe).
- A sluice gate immediately downstream of the intake screen to allow further control of water into the intake conduit.
- A small embankment between the intake structure and the right bank.
- From the intake structure, water is delivered to a race. This race will generally be constructed on the right bank of the river and run southeast for a distance of approximately 2.5 km. It delivers water to a pumping station to be pumped into the Highfield Reservoir.

The weir will likely be founded on alluvial gravels, and appropriate measures will be required to control seepage under the structures.

Residual flow and fish passage would be provided through the sluice/intake channel. If environmental investigations show it is required, a fish screen will be provided in the conduit after the intake gate.

Intake Operation

Water abstraction will normally be controlled by the gates in the sluice/intake channel and the intake conduit. During extreme floods, it is expected that the intake will be shut down for short periods to avoid transportation of sediment through the intake. Although it is anticipated that the water lost due to these shutdowns will not be significant, this needs to be considered in further detail at the next stage of design.

Intake Construction

During construction, the river will be diverted by earthmoving machinery away from the site. Dewatering of the foundation will be required, and temporary sheet piling may be necessary. Rock may need to be imported to the site to construct the weir, unless a suitable nearby source can be identified. Good access to the site is available from the existing Kaiparoro Road.

7.2.3 Highfield Pump Station

Key Parameters

Water is abstracted from the Kopuaranga River and is delivered to the foot of the Highfield Dam by a race. From here, it can be either released immediately to the irrigation distribution system or pumped into the Highfield Reservoir as storage by the Highfield pump station.

- Maximum flow 1 m^3/s
- Average flow 0.24 m^3/s
- Head 19 m
- Number of pumps 4

Description

At this pre-feasibility study stage, a pump station containing 4 KSB pumps of around 75 kW output is envisaged. At a latter stage of investigations, other configurations would be considered to optimise the pump station.

The pumps would be housed in a specialised pump station building constructed near the Jacksons Line Bridge, approximately 500 m upstream of the proposed Highfield Dam site. The pump station would consist of:

- A hydraulic structure on the race to divert water to the pump station.
- A reinforced concrete wet well that is sufficiently deep to provide adequate head to the pump suction lines under all flow conditions.
- A reinforced concrete dry well to house the pumps.
- A lightweight (corrugated iron or similar) building constructed on top of the reinforced concrete structure, to house controls.

The structure would be mostly buried, and would therefore be only partially visible and have a relatively low visual impact. Overall dimensions of the structure will be in the order of 12 m × 8 m × 5 m high.

Surge protection is an issue that needs to be addressed during the feasibility stage. At this stage, the cost estimate allows for fitting all pumps with variable speed controllers. These will be necessary to control the pump capacity over the wide flow range required, but will also be effective in dealing with most common transient problems.

Power Supplies

We have assumed that power at 11 kV will be available from an existing 11 kV line within 2 km of the pump station.

Operating Cost

The annual operating costs are estimated as follows:

Energy charges 0.6 gWh @ 4c/kWh	$24,000
Transmission/distribution charges @ $70/kW/annum	$17,500
Other operating costs	$25,000
Total costs/annum	$66,500

It is possible that negotiations with the various potential suppliers could result in significantly different supply arrangements

7.3 Black Creek Storage Option

7.3.1 Black Creek Dam

Key Parameters

The key parameters for the proposed Black Creek Dam are:

- Maximum reservoir volume = 58 MCM
- Target available storage = 38 MCM
- Design floods
  - Mean annual flood = 29 m^3/s
  - Q_{20} = 58 m^3/s
  - Q_{100} = 78 m^3/s
  - Q_{200} = 93 m^3/s
- Diversion inflow
  - From Black Creek catchment Q_{20} = 58 m^3/s
- Dam layouts
  - Structural height = Up to 45 m
  - Batter slopes = 1V:2.5H

The flood flows at this dam site have been calculated using McKerchar & Pearson (1989).

Site Description

The proposed Black Creek Dam site is located approximately 400 m downstream of the existing Black Creek Road Bridge. At this location, Black Creek is incised in the plain floor, which is 50-60 m wide. The true right abutment rises at a slope of approximately 2.5H:1V, while the left abutment rises at a lesser slope of approximately 4H:1V. The proposed dam will have a crest length of approximately 500 m.

Upstream of the dam site, the plain rises gradually until reaching a peak at a saddle approximately 2 km upstream of the dam site. Beyond this saddle, natural drainage is to the Wakamoekau Creek.

The proposed reservoir area is currently used for farming. Access is provided by Black Creek Road and Falloon Settlement Road, which are orientated approximately north-south along the plain. An airstrip is also located within the proposed reservoir.

Engineering Geology Assessment

This site assessment is made on the basis of observations from geological maps and geological mapping undertaken by an engineering geologist for a previous study of the site. Detailed investigations have not been undertaken and will be necessary at the next stage of the project.

The site is logged on the New Zealand Geological Map – Sheet 12. The Black Creek site is close to the boundary of a Coquina Limestone or calcareous sandstone/siltstone unit and alternating argillite and greywacke unit. The boundary appears to be associated with reverse faulting. Downstream of the Black Creek Dam site, flood plain and terrace deposits are noted.

At the dam site the plain floor is approximately 50-60 m wide, and greywacke rock is exposed 150 m downstream of the dam crest. Assuming there is no river channel fault, the bedrock under the alluvium is unlikely to be deeper than 10-20 m below the current river levels and may be considerably less (5 m).

Greywacke forms both abutments at river level. However, younger Tertiary soft sandstone and siltstone may exist under the old gravels at higher levels on the west abutment. The river plain follows the common fault trend in the area. There is no major fault of comparable geological importance to the Black Creek East or West Faults running through the site (this is indicated by the existence of greywacke both sides of Black Creek). However, the rock in the Black Creek Road batter is highly fractured in places, and there may be shear zones in the buried greywacke under the alluvium. Shear zones exist on this trend in the river further down Black Creek near Upper Plain Road, and a fault or system of shears may be present parallel to the river and under the dam site.

If this is the case, there does not appear to have been any significant active vertical uplift movement across the plain (e.g. uplifting the high eastern side with respect to the lower western side). If this had occurred, the uplift would have distorted the old Waingawa River gradient, as preserved in the 60,000 year old Waingawa Terrace in the Waingawa Plain downstream of the site.

Sighting along this terrace from upstream of Black Creek shows the typical smooth Waingawa River gradient continuing across Black Creek without any obvious abrupt change caused by a cross-cutting active fault (this gradient will be quantified by survey in the next phase of work). Therefore, if shears do exist in the river plain parallel to Black Creek, they are unlikely to be active in the primary geological sense. If shears are found, it may still be necessary to design the dam for the possibility of minor secondary movements induced on the shears by local adjustment to possible movement of nearby active faults. Given the plan to build an earth dam, which is by nature a flexible structure, incorporating the necessary features to provide for possible secondary movement is feasible without huge cost implications.

Laboratory testing of collected samples will be carried out in the next phase of work to assess suitability of construction materials. The work will then be refined and extended as required in the design stage investigations.

The proposed reservoir area is free of large-scale pre-existing instability (e.g. comparable to that in the Waingawa Plain upstream of Black Creek). There will be issues of possible new slumping and slope failure associated with creating the new reservoir and raising pore water pressures in the existing stable slopes. Based on the appearance of the existing slopes, such failures will tend to be small and shallow (say, maximum 50 m wide and <5 m deep).

The consequence of any such induced slope failure will be minor, given the gentle slope angles and shape of the reservoir. A wave resulting from a slump of material into the reservoir will be widely dissipated, rather than channelled downstream by a long narrow reservoir shape to overtop the dam. The maximum wind-generated design wave will probably be higher than that arising from slope failure.

Seepage of water out of the reservoir will be limited by the relatively low permeability of the Upper Tertiary sandstone underlying the Black Creek Basin. In addition, the abundant springs around the basin slopes indicate saturation at depth and groundwater movement generally towards the basin.

Embankment Design Concept and Material Sources

In order to provide the required 38 MCM storage, a reservoir with a full supply level (FSL) of RL 284 is required. To achieve this, a dam up to 45 m high is required across Black Creek. In addition, a dam is required across the previously mentioned saddle, to prevent water from flowing into the Wakamoekau Creek. This dam would be approximately 15 m high and would have a crest length of approximately 800 m.

The most economical type of dam for both sites is likely to be an earthfill embankment, provided that sufficient quantities of suitable fill material can be found near the dam sites. All earth fill materials would preferentially be borrowed from within the reservoir to enhance storage volume. Loess deposits have been identified in the Upper Black Creek area adjacent to the reservoir margin, which should be suitable for embankment construction. It may also be possible to use the Upper Tertiary sandstone and siltstone. If sufficient quantities of suitable material are not available within the reservoir, the flood plain and terrace deposits downstream of the Black Creek dam site would be utilised.

It is envisaged that the dams will consist of a low permeability cores, higher permeability shells, internal chimneys, blanket drains and rock rip-rap protection on the upstream faces. The cores would most likely be constructed from locally borrowed loess or flood plain silt deposits, while the shells could be constructed from similar materials but are more likely to consist of terrace gravel deposits. Unless sand deposits of suitable grading can be found locally, materials for the internal drains would need to be imported to the site. It should be possible to quarry rock locally for rip-rap. Depending, however, on joint spacing in the rock, some larger size rock may need to be imported to the site.

As discussed previously, significant depths of alluvial materials are not expected, and significant cut-off works are not envisaged for the sites.

Additional investigations will have to be performed in the next phase of work to identify and confirm the availability and suitability of the on-site materials, especially the availability of low permeability materials, and the suitability of the downstream sand deposits for use as filter materials.

Diversion Works and Spillways

It is envisaged that a conventional diversion conduit would be provided at the Black Creek Dam site, with an upstream coffer dam and intake and downstream energy dissipation structure. Some rock excavation is likely to be necessary; otherwise, construction is expected to be relatively straightforward. The diversion works would provide the basis for outlet works in the final operation of the reservoir.

A spillway structure would be provided on the Black Creek Dam to pass flood flows up to a return period of approximately 200 years. The current cost estimate assumes a gated spillway with a lined channel and stilling basin. Other options, such as a “Morning Glory” tower, would be considered at the next stage of investigations. An auxiliary spillway could be constructed at relatively low cost through a natural saddle in the northeast corner of the reservoir. This spillway would enable the probable maximum flood to be passed safely, in conjunction with the primary Black Creek spillway, and would also act as a back-up if for any reason the primary spillway was blocked.

Detailed studies on the hydrology of Black Creek would be required at the next stage of investigations.

Foundations

It is anticipated that the dam foundations will generally consist of competent greywacke, although Tertiary deposits and gravel deposits may be encountered at higher elevations within the footprint of the dam. Foundation preparation would involve stripping of the dam foundations back to rock, with some rock shaping and possible grouting.

It is necessary to undertake detailed geotechnical investigations during the next phase to confirm the foundation conditions. These investigations must also consider the potentially soluble Coquina Limestone mapped near the site. Consideration must also be given to the presence of limestone within the reservoir.

Seismic Issues

The dam sites are located in an area of high seismicity. Detailed seismic assessment of the known faults located in the vicinity of each proposed dam site will be required in the next phase of work. Of particular concern are the faults logged within the proposed reservoir area. It is likely that local faulting is the largest technical risk associated with the dams.

The estimated magnitude of peak ground accelerations for the proposed dam sites under the Maximum Credible Earthquake is likely to be in the order of 0.7-0.9 g. The proposed batter slopes should be capable of withstanding the effects of the potential seismic loading, and it is unlikely that liquefiable zones exist within deposits under the footprints of the dams. However, this will have to be confirmed through future investigations and analyses in the next phase of design work.

Construction

Construction of the proposed embankment dams is expected to take two summer construction seasons. It is envisaged that construction of the dams would be relatively straightforward, with conventional construction techniques used throughout the project.

7.3.2 Atiwhakatu Stream Intake

Key Parameters

The key parameters for the Atiwhakatu Stream intake are:

- Maximum diverted flow 5.5 m^3/s
- Atiwhakatu Stream mean flow 3.7 m^3/s
- Mean annual flood 119 m^3/s
- Design flood 760 m^3/s
- River bed width 20 m

Site Description

The intake site is on the left bank of the Atiwhakatu Stream, approximately 2 km upstream of its junction with the Waingawa River. At this point, the Atiwhakatu Stream is well confined and sweeps around a rock bluff, the rock being described as slightly weathered, closely jointed, strong greywacke and argilite.

Intake Concept

The Atiwhakatu Stream is hydraulically steep and carries high suspended and bed loads. The following intake concept was selected as being the most appropriate for the site:

- A reinforced concrete weir approximately 7 m high across the river channel to divert water to the intake and provide control to water levels at the intake.
- A spillway approximately 16 m wide cut into the rock bluff on the right bank. This spillway, in combination with the weir, will be able to pass the design flood.
- Twin open-screened culvert incorporated with the concrete weir.
- A shaft gate to isolate one of the two culverts.
- A conduit (Ø 1.8 m pipe) underneath a short section of embankment to transfer water from the intake to the settling basin.
- A radial gate at the end of the conduit.
- From the intake, the water is delivered to a race over a low weir. The race would carry water approximately 1,500 m downstream to a pipe, which would carry water another 500 m downstream to join a pipeline from the Waingawa River intake.

In general, the reinforced concrete structure will be founded on greywacke rock. This will require excavation of alluvial materials in the stream bed.

Intake Operation

Water abstraction will normally be controlled by the intake gates. During extreme floods, it is expected that the intake will be shut down for short periods to avoid transportation of sediment through the intake. Although it is anticipated that the water lost due to these shutdowns will not be significant, this needs to be considered in further detail at the next stage of design.

Intake Construction

During construction, the river will be diverted away from the site, using a diversion channel through the bluff on the right bank of the river, in conjunction with upstream and downstream coffer dams. Once diversion is complete, the intake site would be stripped back to rock to allow construction of the concrete weir on rock foundations. Following construction of the permanent works, the river would be diverted through the intake structure, allowing the diversion cut to be sealed.

7.3.3 Waingawa River Intake

Key Parameters

The key parameters for the Waingawa River intake are:

- Maximum diverted flow 4.4 m^3/s
- Waingawa River mean flow 10.9 m^3/s
- Mean annual flood 251 m^3/s
- Design flood 1,340 m^3/s
- River bed width 50 m
- River bed slope 1:125

Site Description

The intake is located at the site of the existing Waingawa water supply intake, approximately 700 m upstream of confluence of the Waingawa River and Atiwhakatu Stream. At this location, the Waingawa River is near the end of a broad anticlockwise curve. Steeply rising rock (greywacke) cliffs are present around the outside of the curve, which confine the river to its current location. On the true left, the banks rise less steeply, and gravel beaches have formed.

Intake Concept

The existing Waingawa water supply intake consists of a simple submerged intake and pipe at right angles to a delivery pipe, which runs steeply up a rock face before being taken to a pump station building as a buried pipeline. A similar concept would be used for the Waingawa River intake for the Black Creek storage option.

Intake Operation

Water from this intake is discharged directly into the Carterton/Greytown Main Distribution Channel and is not stored in the Black Creek Reservoir.

Key Parameters for Waingawa River Intake Pump Station

- Maximum flow 4.4 m^3/s
- Average flow 0.2 m^3/s
- Head 12 m
- Number of pumps 6

Construction

At this pre-feasibility study stage, a pump station containing 6 KSB pumps of around 150 kW output is envisaged. At a later stage of investigations, other configurations would be considered to optimise the pump station.

The pumps would be housed in a specialised pump station building constructed on the true right bank of the Waingawa River. This would consist of:

- A screened intake to the river, with a 1.8 m diameter delivery pipe to the pump station similar to the existing water supply intake.
- A reinforced concrete wet well that is sufficiently deep to provide adequate head to the pump suction lines under all flow conditions.
- A reinforced concrete dry well to house the pumps.
- A lightweight (corrugated iron or similar) building constructed on top of the reinforced concrete structure, to house controls.

The structure would be mostly buried, and would therefore be only partially visible and have a relatively low visual impact, similar to the existing water supply pump station. Overall dimensions of the structure will be in the order of 20 m × 12 m × 7 m high.

Surge protection is an issue that needs to be addressed during the feasibility stage. At this stage, the cost estimate allows for fitting all pumps with variable speed controllers. These will be necessary to control the pump capacity over the wide flow range required, but will also be effective in dealing with most common transient problems.

Power Supplies

We have assumed that power at 11kV will be available from an existing 11 kV line within 6 km of the pump station, and that the cost of this power supply extension would be shared with the Black Creek pump station (see Section 7.3.4).

Operating Cost

The annual operating costs are estimated as follows:

Energy charges 0.4 gWh @ 4c/kWh	$16,000
Transmission/distribution charges @ $70/kW/annum	$57,500
Other operating costs	$25,000
Total costs/annum	$98,500

It is possible that negotiations with the various potential suppliers could result in significantly different supply arrangements

Pipeline

From the pump station, a pipeline would deliver the water to the Carterton / Greytown Main Distribution channel. This pipeline would also carry flows from the Atiwhakatu intake. Preliminary design on the pipeline indicates that a 1.8 m diameter pipe is required to limit head loss in the pipe to acceptable levels. It is envisaged at this stage that the pipe would be steel, with a 6 mm wall thickness. Other materials, however, would be considered at a later date.

7.3.4 Black Creek Pump Station and Pipework

Key Parameters for Pump Station

Water from the Atiwhakatu Stream intake is pumped into the Black Creek Reservoir by the Black Creek pump station. The key parameters for the pump station are:

- Maximum flow 5 m^3/s
- Average flow 0.67 m^3/s
- Head 19 m
- Number of pumps 6

Description

At this pre-feasibility study stage, a pump station containing 6 KSB pumps of around 200 kW output is envisaged. At a later stage of investigations, other configurations would be considered to optimise the pump station.

The pumps would be housed in a specialised pump station building, likely constructed on the true right bank of the Waingawa River, downstream of its confluence with the Atiwhakatu Stream. Tther locations for the pump station would, however, also be possible, and the final location would be selected to best suit the final layout of pipework. This would consist of:

- Appropriate pipework and valves to take the correct amount of water from the combined Atiwhakatu Stream and Waingawa River pipeline.
- A reinforced concrete wet well that is sufficiently deep to provide adequate head to the pump suction lines under all flow conditions.
- A reinforced concrete dry well to house the pumps.
- A lightweight (corrugated iron or similar) building constructed on top of the reinforced concrete structure, to house controls.

The structure would be mostly buried, and would therefore be only partially visible and have a relatively low visual impact. Overall dimensions of the structure will be in the order of 20 m × 12 m × 7 m high.

Surge protection is an issue that needs to be addressed during the feasibility stage. At this stage, the cost estimate allows for fitting all pumps with variable speed controllers. These will be necessary to control the pump capacity over the wide flow range required, but will also be effective in dealing with most common transient problems.

Power Supplies

We have assumed that power at 11kV will be available from an existing 11 kV line within 6 km of the pump station, and that the cost of this power supply extension would be shared with the Waingawa pump station (see Section 7.3.3).

Operating Cost

The annual operating costs are estimated as follows:

Energy charges 1.3 gWh @ 4c/kWh	$52,000
Transmission/distribution charges @ $70/kW/annum	$83,000
Other operating costs	$25,000
Total costs/annum	$160,000

It is possible that negotiations with the various potential suppliers could result in significantly different supply arrangements

Pipeline
The pipeline is required to deliver water from the pump station to the reservoir and to carry flows released from the reservoir to the Carterton/Greytown Main Distribution Channel. A pipe diameter of 1.5 m has been selected to keep head loss in the pipe to acceptable levels. At this stage, it is envisaged that the pipe will be steel, with a wall thickness of 6 mm. Other pipe materials would, however, be considered at a later stage.

Because of the difference in elevation between the reservoir and the Main Distribution Channel (up to 42 m), it is necessary to pass the released flows through an energy dissipating structure, probably an impact basin, before discharging into the channel.

The pipeline must cross the Waingawa River; allowance has been made in the cost estimate for a pipe bridge.

7.3.5 Carterton/Greytown Main Distribution Canal

Key Parameters
The Carterton/Greytown Main Distribution Canal delivers water from the Atiwhakatu Stream and Waingawa River intakes and Black Creek Reservoir to the Tauherenikau River. Along the canal route, off-takes will be used to deliver water to the Carterton and Greytown supply zones.

The key parameters are:

• Capacity	7.6 m^3/s
• Length	29.3 km
• Grade	1:900
• Start elevation	RL 240 m
• End elevation	RL 95 m
• Flow velocity	1 m/s
• Design depth	1.2 m
• Base width	4 m
• Side slopes	2.5:1
• Freeboard	0.8 m

Site Description
The canal runs across relatively flat land at the foot of the Tararua Range. At the start of the route, natural ground slopes are as high as 1 in 50. This reduces, however, along the race route, and over the final 10 km the canal will generally follow the 100 m contour.

The canal route is shown as being underlain by alluvial and terrace deposits on the New Zealand Geological Map – Sheet 12. It is anticipated that the majority of the route would be capped by a low permeability layer. Higher permeability deposits are, however, likely where the canal route crosses existing streams.

The canal crosses a number of small streams, six of which would be considered significant. In addition, there are a number of road crossings along the route. It is envisaged that these would be crossed by a 1.8 m diameter steel pipe syphon.

Canal Design Concept

It is anticipated that construction of the canal would generally be a cut-to-fill operation in relatively impermeable materials, apart from stream fans. It is not expected that there would be any significant haul distances involved in the earthworks. A maintenance road, constructed on one of the berms, would be provided along the full length of the canal.

As it is expected that the canal would be generally constructed in a surficial layer of low permeability material, it is not anticipated that there will be any need for significant lining. Allowance has been made in the estimates for a small amount of localised lining with local material and filter underlay for small sections of the race constructed in alluvial stream deposits.

Flows into the canal would be controlled by the Black Creek outlet works. Flows and water depths along the canal would be controlled by the hydraulic characteristics of the canal and pipe stream crossings. The design gradient of 1 in 900 has been selected to keep water velocities below 1 m/s, as greater velocities would cause scour of the canal. The relatively steep natural gradient over the first half of the route necessitates a number of drop structures, consisting of rip-rap lined basins, to dissipate head along the route. The large number of crossings will also dissipate a significant amount of head.

Upstream of all piped stream crossings, emergency grassed spillways would be provided to cover abnormal conditions such as pipe blockage and extreme overland flow conditions. The entrances of all piped stream crossings would be protected by a coarse screen (with a nominal screen bar gap of 300 mm).

Construction

The area affected by the race earthworks would be stripped and a cut-to-fill operation undertaken, using conventional earthmoving machinery. This is expected to generally be in a local area. Streams that cross the race will be crossed by a pipe syphon. During the construction of the syphons, the streams will be diverted relatively easily, using suitable earthmoving machinery. Road crossings will potentially cause difficulties, and appropriate traffic management measures would need to be put in place.

7.3.6 Waiohine Intake

Key Parameters

The key parameters for the Waiohine River intake are:

- Maximum diverted flow 3.8 m^3/s
- Mean annual flood 950 m^3/s
- Flood with 100-year return period 1,800 m^3/s
- River bed width 300 m
- Rived bed slope 1:150

The flood flows at this site have been calculated using McKerchar & Pearson (1989).

Site Description

The intake site was not visited during this study, and this site description is made on the basis of observations from 1:50,000 topographical map NZMS 260 S26. The proposed intake site is located at co-ordinate S26:138-117. At this location, the Waiohine River sweeps around a broad curve. It is anticipated that the river is well confined at this location.

On the true right bank a terrace rises approximately 20 m, while on the left bank gravel flats are noted.

Intake Concept

The intake would consist of the following:

- A low (up to 3 m high) rock fill weir across the plain to divert water to the intake and provide control to water levels at the intake.
- A reinforced concrete sluice/intake channel on the right bank, with walls up to 6 m high and 30 m long.
- A sluice gate in the sluice/intake channel to provide control for water levels at the intake screens and to keep the intake clear of bed load.
- A screened intake in the right wall of the sluice/intake channel, leading the water into the downstream conduit (Ø 1.8 m pipe).
- A sluice gate immediately downstream of the intake screen to allow further control of water into the intake conduit.
- A small embankment between the intake structure and the right bank.
- From the intake structure, water is delivered to a race. This runs south for a distance of approximately 2 km and delivers water into the Carterton/Greytown Main Distribution Channel. The race must run along a small section of terrace and cross the Wairarapa Railway.

The weir will likely be founded on alluvial gravels, and appropriate measures will be required to control seepage under the structures.

Residual flow and fish passage would be provided through the sluice/intake channel. If environmental investigations show it is required, a fish screen will be provided in the conduit after the intake gate.

Intake Operation

Water abstraction will normally be controlled by the gates in the sluice/intake channel and the intake conduit. During extreme floods, it is expected that the intake will be shut down for short periods to avoid transportation of sediment through the intake. Although it is anticipated that the water lost due to these shutdowns will not be significant, this needs to be considered in further detail at the next stage of design.

Construction

During construction, the river will be diverted from the site by earthmoving machinery. Dewatering of the foundation will be required, and temporary sheet piling may be necessary. Rock may need to be imported to the site to construct the weir, unless a suitable nearby source can be identified. Good access to the site is available from the existing Waiohine Plain Road.

7.3.7 Opaki Main Distribution Canal

Key Parameters

The Opaki Main Distribution Canal delivers water from Black Creek Reservoir to the Waipoua River to feed the Opaki supply zone.

The key parameters are:

• Capacity	2.1 m^3/s
• Length	10.7 km
• Grade	1:500
• Start elevation	RL 240 m
• End elevation	RL 180 m
• Flow velocity	1 m/s
• Design depth	0.8 m
• Base width	2.0 m
• Side slopes	1.5:1
• Freeboard	0.7 m

Site Description and Geology

The canal initially runs for a distance of approximately 3 km to the southeast of Black Creek Reservoir across moderately sloping ground. At Burnetts Road, it then turns and runs on a northeast orientation, initially through a valley and then out onto the plain before reaching the Waipoua River.

The canal route is shown as being predominantly underlain by alluvial and terrace deposits on the New Zealand Geological Map – Sheet 12. It is anticipated that the majority of the area would be capped by a low permeability layer. Higher permeability deposits are, however, likely where the canal route crosses existing streams. Approximately 3 km of the route crosses an area that may be underlain by Coquina limestone or calcareous sandstone/siltstone. It would be anticipated that the race would be constructed in the weathered cap overlying this material. It will, however, be necessary to identify areas of potentially soluble material along the route in the next stage of investigations.

The canal crosses a number of streams and roads along the route. It is envisaged that these would be crossed by a 1.2 m diameter steel pipe syphon.

Canal Design Concept

It is anticipated that construction of the canal would generally be a cut-to-fill operation in relatively impermeable materials, apart from stream fans. It is not expected that there would be any significant haul distances involved in the earthworks. A maintenance road, constructed on one of the berms, would be provided along the full length of the canal.

As it is expected that the canal would be generally constructed in a surficial layer of low permeability material, it is not anticipated that there will be any need for significant lining. Allowance has been made in the estimates for a small amount of localised lining with local material and filter underlay for small sections of the race constructed in alluvial stream deposits.

Flows into the canal would be controlled by the Black Creek outlet works. Flows and water depths along the canal would be controlled by the hydraulic characteristics of the canal and pipe stream crossings. The design gradient of 1 in 500 has been selected to keep water velocities below 1 m/s, as greater velocities would cause scour of the canal. The relatively steep natural gradient of the route necessitates drop structures, consisting of rip-rap lined basins, to dissipate head along the route. The relatively large number of crossings will also dissipate a significant amount of head. To cross the Wakamoekau Stream plain and drop from an elevation of RL 220 to RL 185m, a 1.2 m diameter 700 m long syphon is required. This would discharge into an energy dissipation structure.

Upstream of piped stream crossings, emergency grassed spillways would be provided to cover abnormal conditions such as pipe blockage and extreme overland flow conditions. The entrances of all piped stream crossings would be protected by a coarse screen (nominal screen bar gap 300 mm).

Construction
The area affected by the race earthworks would be stripped and a cut-to-fill operation undertaken using conventional earthmoving machinery. This is expected to generally be in a local area. Streams that cross the race will be crossed by a pipe syphon. During the construction of the syphons, the streams will be diverted relatively easily, using suitable earth moving machinery. Road crossings will potentially cause difficulties, and appropriate traffic management measures would need to be put in place.

7.3.8 Ruamahanga Intake

Key Parameters
The key parameters for the Ruamahanga River intake are:

- Maximum diverted flow 2.1 m^3/s
- Flood with 100-year return period 500^+ m^3/s
- River bed width 300 m (approx)
- River bed slope 1:150

The flood flow at this site has been estimated by comparison with other rivers in the area.

Site Description
The intake site was not visited during this study, and this site description is made on the basis of observations from 1:50,000 topographical map NZMS 260 T26. The proposed intake site is located at co-ordinate T26:373-323. At this location, the Ruamahanga River splits into two channels, presumably because of the presence of rock. It is therefore anticipated that the river is well confined at this location. Gravel flats are noted immediately downstream of the proposed intake site.

Intake Concept
The intake would consist of the following:

- A low (up to 3 m high) rock fill weir across the plain to divert water to the intake and provide control to water levels at the intake.
- A reinforced concrete sluice/intake channel on the right bank, with walls up to 5 m high and 25 m long.
- A sluice gate in the sluice/intake channel to provide control for water levels at the intake screens and to keep the intake clear of bed load.

- A screened intake in the right wall of the sluice/intake channel leading the water into the downstream conduit (Ø1.2 m pipe).
- A sluice gate immediately downstream of the intake screen to allow further control of water into the intake conduit.
- A small embankment between the intake structure and the right bank.
- From the intake structure, water is delivered to a race. This runs south for a distance of approximately 2 km and delivers water into the Opaki supply zone.

The weir will likely be founded on alluvial gravels, and appropriate measures will be required to control seepage under the structures.

Residual flow and fish passage would be provided through the sluice/intake channel. If environmental investigations show it is required, a fish screen will be provided in the conduit after the intake gate.

Intake Operation

Water abstraction will normally be controlled by the gates in the sluice/intake channel and the intake conduit. During extreme floods, it is expected that the intake will be shut down for short periods to avoid transportation of sediment through the intake. Although it is anticipated that the water lost due to these shutdowns will not be significant, this needs to be considered in further detail at the next stage of design.

Construction

During construction, the river will be diverted from the site by earthmoving machinery. Dewatering of the foundation will be required, and temporary sheet piling may be necessary. Rock may need to be imported to the site to construct the weir, unless a suitable nearby source can be identified. Good access to the site is available from State Highway 2 to the west and off Bruce Road to the east.

7.4 Te Muna/Huangarua Storage Option

7.4.1 Te Muna Dam

Key Parameters

The key parameters for the proposed Te Muna Dam are:

- Maximum reservoir volume = 9 MCM
- Target available storage = 9 MCM
- Design floods
  - Mean annual flood = 6 m^3/s
  - Q_{20} = 12 m^3/s
  - Q_{100} = 17 m^3/s
  - Q_{200} = 19 m^3/s
- Diversion Inflow
  - Q_{20} = 12 m^3/s
- Dam layouts
  - Structural height = 11 m
  - Batter slopes = 1V:2.5H

The flood flows at these dam sites have been calculated using McKerchar & Pearson (1989).

Site Description and Geology

The Te Muna Reservoir is formed in a natural basin using a long, relatively low embankment to form a "turkey's nest" style storage. The embankment would dam an unnamed stream to the west of Huangarua River, which collects a number of smaller tributaries which drain a catchment to the south of the reservoir.

The proposed reservoir area extends approximately 1.75 km upstream of the dam. The land is currently used for farming, and it appears that at least two properties would need to be purchased as part of the project. The reservoir would also flood part of Martinborough Awhea Road, which would need to be realigned to avoid the reservoir.

The site is logged on the New Zealand Geological Map – Sheet 12. The site is shown as being underlain by alluvium. It is anticipated that this material would consist of a cap of low permeability material (flood plain silts), possibly underlain by more permeable sands and gravels.

A number of faults are mapped near the site. They are not, however, known to be active and do not pass through the reservoir area.

Embankment Design Concept and Material Sources

In order to provide the required storage, a reservoir with a full supply level of RL 85 is required, necessitating an embankment crest level of RL 87. The embankment would be a maximum of approximately 13 m high, with a crest length of approximately 800 m.

The most economical type of dam for this site is likely to be an earthfill embankment. Earth fill materials would probably be borrowed from outside the reservoir to avoid compromising the capping layer of low permeability material that is anticipated on the site. It is anticipated that a suitable borrow will be able to be established within a short distance of the dam site.

It is envisaged that the dam will consist of a low permeability core, higher permeability shells, internal chimney, blanket drains and rock rip-rap protection on the upstream face. The core would most likely be constructed from locally borrowed flood plain silt deposits, while the shells could be constructed from similar materials or gravel deposits. Unless sand deposits of suitable grading can be found locally, materials for the internal drains would need to be imported to the site. Materials for rock rip-rap will also need to be imported.

It is anticipated that low permeability material over the site will provide adequate control of seepage from the reservoir, and additional measures will not be necessary. This will, however, need to be confirmed by detailed investigations and modelling at the next stage.

Investigations will have to be performed in the next phase of work to identify the availability and suitability of near-site materials for embankment construction.

Diversion Works and Spillways
It is envisaged that a conventional diversion conduit would be provided at the Te Muna Dam site, with an upstream coffer dam and intake and downstream energy dissipation structure. Construction of these structures is expected to be relatively straightforward. The diversion works would provide the basis for outlet works in the final operation of the reservoir.

A spillway structure would be provided on the Te Muna Dam to pass flood flows up to a return period of approximately 200 years. The current cost estimate assumes a gated spillway, with a lined channel and stilling basin. Other options, such as a "Morning Glory" tower or a low-cost grass lined spillway, would be considered at the next stage of investigations. An auxiliary grass lined spillway could be constructed at relatively low cost on the embankment. This spillway would enable the probable maximum flood to be passed safely, in conjunction with the primary spillway, and would also act as a back-up if for any reason the primary spillway was blocked.

Detailed studies on the hydrology of the plain would be required at the next stage of investigations.

Foundations
It is anticipated that the dam foundations will generally consist of flood plain deposits. It is not anticipated that these materials would

be unduly compressible, and they should provide adequate foundations for the dam. This would, however, need to be confirmed by detailed investigations, as it is possible that loose material or even soft estuarine deposits could be encountered.

It is envisaged that foundation preparation would simply involve stripping of topsoil and other unsuitable material from beneath the embankment footprint.

Seismic Issues

The dam site is located in an area of high seismicity. Detailed seismic assessment of the known faults in the vicinity of the proposed dam site will be required in the next phase of work. These issues present a technical risk to the project.

The estimated magnitude of peak ground accelerations for the proposed dam site under the Maximum Credible Earthquake is likely to be in the order of 0.7-0.9 g. The proposed batter slopes should be capable of withstanding the effects of the potential seismic loading; this will, however, have to be confirmed in the next phase of design work. It is possible that liquefiable zones exist under the footprint of the dam; this also needs to be considered in the next stage.

Construction

Construction of the proposed embankment dam may be possible in one summer construction season, although it may be prudent to allow for two construction seasons. It is envisaged that construction of the dam would be relatively straightforward, with conventional construction techniques used throughout the project.

7.4.2 Hautotara Dam

Key Parameters

The key parameters for the proposed Hautotara Dam are:

- Maximum reservoir volume = 42 MCM
- Target available storage 4 MCM
- Design floods
  - Mean annual flood 150 m^3/s
  - Q_{20} 300 m^3/s
  - Q_{200} 450 m^3/s
- Diversion inflow
  - Q_{20} 300 m^3/s
- Dam layouts
  - Structural height 18 m
  - Batter slopes 1V:2.5H

The flood flows at these dam sites have been calculated using McKerchar & Pearson (1989).

Site Description

The Hautotara Dam is situated just downstream of the White Rock Road crossing of the Huangarua River, commonly referred to as the "Banana Bridge". At this location, the Huangarua River runs through a plain approximately 500 m wide, with terraces rising sharply on both the left and right abutments.

The majority of the proposed reservoir extends only 500 m upstream of the dam. It would, however, reach as far as 1.5 km upstream of the dam in the Ruakokoputuna and Makara rivers, which are the tributaries of the Huangarua River. The flooded land does not appear that any existing dwellings would be affected by flooding of the reservoir. The reservoir would also flood the Banana Bridge, which appears to have a deck level of approximately RL 80.

Engineering Geology Assessment

The area is underlain by Pliocene to Pleistocene sediments, ranging from deep sea marine sediments to terrestrial gravels and silts. The geology has been mapped by Collen & Vella (1984) and is a field study area for Victoria University geology students.

The Pliocene to Lower Pleistocene Greycliffs Formation comprises a massive low permeability mudstone called the Mangaopari Mudstone. This is overlain by a Lower Pleistocene sequence of calcereous sandstones and Coquina Limestones known as the Pukenui Limestone.

The Pukenui Limestone is overlain, with local disconformity (a period of no deposition and possible minor erosion) by the Hautotara Formation. The Hautotara Formation comprises silty sands, sands and silts deposited under marine and terrestrial conditions.

The Te Muna Formation overlies the Hautotara Formation. Te Muna Formation is interpreted as a series of gravel aggradations separated by swamp or lake depositions. Accordingly, the lithologies vary from lacusterine silts to river gravels.

There are no known active faults within the dam footprint area. The nearest active fault is the Dry River Fault, approximately 4 km to the north. The whole area is undergoing active deformation, and minor faulting can be seen in an outcrop of Pukenui Limestone to the south of the Banana Bridge. There is a high level of confidence that there are no major active faults through the dam site, as a geophysical survey has been completed across the Huangarua Plain near Te Muna, and the limestone beds are a prominent marker which stand out in the cross-sections generated by the survey. The section runs from the foot of Windy Peak to the Dry River Fault in the west.

Extensive mapping has been carried out in the area but has not been published. Further review of this information will be required if the decision is made to proceed with a dam at this location.

The dam will be constructed over each of the three geological units. That is, it will be partially underlain by Pukenui Limestone, Hautotara silts and sands and Te Muna gravels, silts and sands.

Historically, many dams have been constructed over limestone bedrock. Limestone bedrock is prone to dissolution and, not surprisingly, a number of dams around the world have experienced the development of solution cavities which potentially can undermine the dam foundations and cause excessive seepage losses. This is particularly pronounced where short flow paths exist through dam abutments along pre-existing limestone dissolution channels. The Pukenui Limestone is present along the toe of the Windy Peak Range of hills and will underlie the dam.

The Pukenui Limestone forms a karst landscape in places. At the surface, there is extensive evidence of dissolution and redeposition.

Natural rainwater is slightly acidic because of absorption of carbon dioxide (CO_2) from the atmosphere. CO_2 and H_2O mix to form carbonic acid, a weak acid. Plant and animal decay, as well as animal respiration, produce CO_2 in the soil in the vadose zone. Rainwater further mixes with CO_2 in the soil, becoming more acid. (It is still only weakly acidic.) It picks up enough acidity in the soil to dissolve the calcite ($CaCO_3$ – calcium carbonate) that comprises limestone.

Karst morphology can occur on carbonates with less purity than 80%, but generally the purer the limestone, the better the development of karst morphology (Jennings, 1985). As the Pukenui Limestone is a series of calcareous sands and Coquina Limestones, it is less prone to development of massive internal drainage systems than purer massive limestones. Interbeds of material with a higher clastic content may limit solution path development. Thus solution paths are more likely to occur along bedding in the Pukenui Limestone. Karst topography has developed with dissolution and recrystallisation of limestone near the surface and development of small tomos and sink holes in places. However, the nature of the limestone does not allow development of major cave systems or dissolution channels.

Observations from near the site support this assessment. For example, no major dissolution features are visible in the exposure upstream of Banana Bridge. The only significant cave system in the area is developed in an older limestone formation 9 km to the south. The presence of limestone is also noted in investigation records for the Banana Bridge foundations. The presence of this material at shallow depths below river bed level indicates that it has resistance to dissolution.

It is anticipated that stream water in the area will have elevated levels of calcium, reducing its acidity and effectiveness in dissolving calcium carbonate. These waters are still likely to have a significant capacity to dissolve limestone, as the stream water is unlikely to be in equilibrium with the limestone, because only short reaches of the stream bed encounter limestone and contact time will be short. Water quality testing is recommended to assess this.

The bedding orientation of the limestone is not particularly favorable. Whilst the flow path distances are significant, there is a significant hydraulic gradient due to the head difference. It is possible that more detailed inspection of the geology of the area could identify a location where the limestone is effectively capped by the Hautotara formation. A dam location further downstream provides a longer flow path for leakage along the limestone bedding and should be investigated at the next stage.

The hydraulic head in the limestone may already be greater than the dam water elevation along this flow path. This is because the limestone has the form of a classic confined aquifer, with limestone beds recharged high on Windy Peak and possibly confined by overlying strata. This would need to be evaluated.

The presence of limestone within the foundation represents the most severe technical risk to the project. In the next stage of work, it would be necessary to:

- Obtain detailed geological maps of the area to determine the likely extent of limestone and assess its potential impacts on the dam and to determine if alternative sites further downstream are more suitable.
- Undertake water quality testing to assess the ability of the water to cause dissolution of limestone.
- Undertake detailed investigations to confirm the geological interpretation of the site, collect samples for testing, and study the existing groundwater regime.

Embankment Design Concept and Material Sources

In order to provide the required storage, a reservoir with a full supply level of approximately RL 86 is required, necessitating an embankment crest level of RL 88 m. The embankment would be a maximum of approximately 18 m high, with a crest length of approximately 600 m.

The most economical type of dam for this site is likely to be an earthfill embankment. It is envisaged that the dam will consist of a low permeability core, higher permeability shells, internal chimney, blanket drains, and rock rip-rap protection on the upstream face.

Abundant suitable material, consisting of silts, sands and gravels of the Hautotara and Te Muna formations, should be available near the site to construct the embankment. Unless sand deposits of suitable grading can be found locally, materials for the internal drains would need to be imported to the site. Depending on the suitability of locally available rock for rip-rap, it may be necessary to import rock to the site.

Plans for the Banana Bridge show a depth of alluvial material in the existing river of approximately 3 m. Removing and replacing this material with low permeability fill should therefore be a simple operation. Permeable sand and gravel deposits of the Hautotara and Te Muna formations will also probably underlie the dam, and some form of cut-off to restrict seepage flow will probably be necessary. Given the significant depth of cut-off that may be required, the most likely option is a low permeability slurry wall. Such walls are commonly built to depths of 30 m or more overseas, and the technique has been used in New Zealand successfully.

Investigations will have to be performed in the next phase of work to identify the availability and suitability of near-site materials for embankment construction; the presence, depth and permeability of sand and gravel deposits; and, as previously discussed, the presence and significance of limestone deposits.

Diversion Works and Spillways

At the proposed dam site, the Huangarua River has a catchment of approximately 140 km^2, meaning that flood flows in this reservoir are considerably higher than for other dams proposed for these schemes. Diversion and spillway costs are therefore high for the relatively small storage capacity of the reservoir.

It is envisaged that a conventional diversion conduit would be provided at the Huangarua Dam site, with an upstream coffer dam and intake and downstream energy dissipation structure. Construction of these structures is expected to be relatively straightforward. The diversion works would provide the basis for outlet works in the final operation of the reservoir.

A spillway structure would be provided on the dam to pass flood flows up to a return period of approximately 200 years. The current cost estimate assumes a gated spillway, with a lined channel and stilling basin. Although other options, such as a “Morning Glory” tower, would be considered at the next stage, the large capacity required may limit the suitability of these structures. An auxiliary grass lined spillway could be constructed at relatively low cost on the right abutment terrace. This spillway would enable the probable maximum flood to be passed safely, in conjunction with the primary spillway, and would also act as a back-up if for any reason the primary spillway was blocked.

Detailed studies on the hydrology of the plain would be required at the next stage of investigations.

Seismic Issues

Although there are no known active faults through the dam site, the site is located in an area of generally high seismicity. Detailed seismic assessment of the known faults in the vicinity of the proposed dam site will be required in the next phase of work.

The estimated magnitude of peak ground accelerations for the proposed dam site under the Maximum Credible Earthquake is likely to be in the order of 0.7-0.9 g. The proposed batter slopes should be capable of withstanding the effects of the potential seismic loading. This will, however, have to be confirmed through future investigations and analyses in the next phase of design work.

Foundations

Given the potential for silt and sand deposits beneath the dam, liquefaction must be considered a risk. Liquefaction occurs when loose, saturated, non-plastic soils are subject to shaking. The soil structure tends to collapse, which results in an increase in the fluid pressure and a sudden loss in strength in the foundation materials. It is possible that silts in the Hautotara Formation and Te Muna Formations are potentially liquefiable; detailed investigations are necessary at the next stage of work to confirm the vulnerability of these materials to liquefaction and to determine the most effective measures to mitigate this possible risk.

Construction

Construction of the proposed embankment dam may be possible in one summer construction season, although it may be prudent to allow for two construction seasons. It is envisaged that construction would be relatively straightforward, provided issues with the presence of limestone can be addressed, with conventional construction techniques generally used. Specialist equipment may need to be imported from overseas to undertake construction of a slurry trench to significant depth. Good access is available to the site from the existing White Rock Road and Martinborough Aweha Road.

Delivery Race

Water from the reservoir would be delivered to the distribution system by a race which initially runs along the left bank terrace of the Huangarua River and then runs northwest, approximately following Te Muna Road. The first section of race will require significant earthworks on the terrace, while the second section will require cuts of up to 15 m.

7.4.3 Huangarua Intake

Key Parameters

The key parameters for the Huangarua River intake are:

- Maximum diverted flow 4.0 m^3/s
- Mean annual flood 150 m^3/s
- Flood with 200-year return period 450 m^3/s
- River bed width 75 m
- River bed slope 1:150

The flood flows at this site have been calculated using McKerchar & Pearson (1989).

Site Description

The Huangarua River intake and pump station are located at the same site proposed for the Huangarua River Dam. The site is situated just downstream of the White Rock Road crossing of the Huangarua River, commonly refereed to as the "Banana Bridge". At this location, the Huangarua River runs through a plain approximately 500 m wide, with terraces rising sharply on both the left and right abutments. The main river channel is approximately 30 m wide.

Intake Concept

The intake would consist of the following:

- A low (up to 3 m high) rock fill weir across the plain to divert water to the intake and provide control to water levels at the intake.
- A reinforced concrete sluice/intake channel on the right bank, with walls up to 5 m high and 25 m long.
- A sluice gate in the sluice/intake channel to provide control for water levels at the intake screens and to keep the intake clear of bed load.
- A screened intake in the right wall of the sluice/intake channel, leading the water into the downstream conduit (Ø 1.8 m pipe).
- A sluice gate immediately downstream of the intake screen to allow further control of water into the intake conduit.
- A small embankment between the intake structure and the left bank.
- From the intake structure, water is delivered to a pump station, then into a race to the Hautotara Reservoir.

Drawings of the Banana Bridge indicate approximately 3 m of alluvial deposits in the river channel. It should therefore be possible to found the weir and structure on rock foundations. The potential presence of limestone at the intake site is not considered a major concern for this type of structure.

Residual flow and fish passage would be provided through the sluice/intake channel. If environmental investigations show it is required, a fish screen will be provided in the conduit after the intake gate.

Intake Operation
Water abstraction will normally be controlled by the gates in the sluice/intake channel and the intake conduit. During extreme floods, it is expected that the intake will be shut down for short periods to avoid transportation of sediment through the intake. Although it is anticipated that the water lost due to these shutdowns will not be significant, this needs to be considered in further detail at the next stage of design.

Intake Construction
During construction, the river will be diverted from the site by earthmoving machinery. Dewatering of the foundation will be required, and temporary sheet piling may be necessary. Rock may need to be imported to the site to construct the weir, unless a suitable nearby source can be identified. Good access to the site is available from the existing Martinborough Awhea Road and White Rock Road.

7.4.4 Huangarua Pump Station

Key Parameters
Water from the Huangarua River is pumped from the intake, at a reduced level of 70 m, into a race which delivers water to the reservoir.

- Maximum flow: 4 m^3/s
- Average flow: 0.55 m^3/s
- Head: 15 m
- Number of pumps: 6

Description
At this pre-feasibility study stage, a pump station containing 6 KSB pumps of around 300 kW output is envisaged. At a later stage of investigations, other configurations would be considered to optimise the pump station.

The pumps would be housed in a specialised pump station building constructed on the true left bank of the Huangarua River. This would consist of:

- An intake pipe connected to the Huangarua intake.
- A reinforced concrete wet well that is sufficiently deep to provide adequate head to the pump suction lines under all flow conditions.

- A reinforced concrete dry well to house the pumps, with 1 m freeboard over design flood levels in the river.
- A lightweight (corrugated iron or similar) building, constructed on top of the reinforced concrete structure, to house controls.

The structure would be excavated into the bank and would therefore be only partially visible and have a relatively low visual impact. Overall dimensions of the structure will be in the order of 20 m × 12 m × 7 m high.

Surge protection is an issue that needs to be addressed during the feasibility stage. At this stage, the cost estimate allows for fitting all pumps with variable speed controllers. These will be necessary to control the pump capacity over the wide flow range required, but will also be effective in dealing with most common transient problems.

Power Supplies

We have assumed that power at 11 kV will be available from an existing 11 kV line that runs adjacent to the site, but that 10 km of the line will require re-cabling.

Operating Cost

The annual operating costs are estimated as follows:

Energy charges 2.1 gWh @ 4c/kWh	$84,000
Transmission/distribution charges @ $70/kW/annum	$122,000
Other operating costs	$25,000
Total costs/annum	$231,000

It is possible that negotiations with the various potential suppliers could result in significantly different supply arrangements.

7.4.5 Ruamahanga Pump Station and Pipeline

Water from the Ruamahanga River is pumped from an intake at a reduced level of 20 m, through a 1.2 m diameter pipeline, to be either stored in the Te Muna Reservoir or distributed immediately for irrigation.

Two pump station capacities are considered, as the required capacity is dependent upon which other components are selected for this scheme.

Key Parameters

- Maximum flow — 1.3 or 2 m^3/s
- Average flow — 0.3 or 0.4 m^3/s
- Head — 77 or 90 m
- Number of pumps — 4 or 5

Description

At this pre-feasibility study stage, a pump station containing 4-5 KSB pumps of 400-550 kW output is envisaged. At a later stage of investigations, other configurations would be considered to optimise the pump station.

The pumps would be housed in a specialised pump station building constructed on the true left bank of the Ruamahanga River. This would consist of:

- A screened intake to the river, with a 2 m diameter delivery pipe to the pump station.
- A reinforced concrete wet well that is sufficiently deep to provide adequate head to the pump suction lines under all flow conditions.
- A reinforced concrete dry well to house the pumps, with 1 m freeboard over design flood levels in the river.
- A lightweight (corrugated iron or similar) building, constructed on top of the reinforced concrete structure, to house controls.

The structure would be excavated into the bank and would therefore be only partially visible and have a relatively low visual impact. Overall dimensions of the structure will be in the order of 20 m × 12 m × 7 m high.

Surge protection is an issue that needs to be addressed during the feasibility stage. It is possible that large air pressurised tanks will be required to accommodate surge in the pipeline. The requirements for this will be determined by the details of the route selected for the pipeline. At this stage, the cost estimate allow for fitting all pumps with variable speed controllers. These will be necessary to control the pump capacity over the wide flow range required, but will also be effective in dealing with most common transient problems.

Power Supplies

For the 1.3 m^3/s option, we have assumed that power at 11 kV will be available from an existing 11 kV line within 2 km of the pump station.

For a 2 m^3/s pump station we would not expect the existing 11 kV infrastructure to have sufficient capacity, and have made an allowance for supply to be taken at 33 kV.

Operating Cost

The annual operating costs are estimated as follows:

Option 1 (1.3 m^3/s)

Energy charges 2.5 gWh @ 4c/kWh	$100,000
Transmission/distribution charges @ $70/kW/annum	$115,000
Other operating costs	$25,000
Total costs/annum	$240,000

Option 2 (2 m^3/s)

Energy charges 3.2 gWh @ 4c/kWh	$128,000
Transmission/distribution charges @ $70/kW/annum	$175,000
Other operating costs	$25,000
Total costs/annum	$328,000

It is possible that negotiations with the various potential suppliers could result in significantly different supply arrangements.

Pipeline

The Ruamahanga Pipeline has a dual role, namely to deliver water from the Ruamahanga pump station to the Martinborough irrigation scheme and/or the Te Muna Reservoir and to distribute water from the Te Muna Reservoir to the irrigation scheme. The key parameters of the pipeline are as follows:

- Length 10.2 km
- Pipe diameter 0.9-1.2 m
- Start elevation RL 20 m
- End elevation RL 85 m
- Peak pumped flows 1.3-2 m^3/s
- Head loss at 1.3 m^3/s 12 m
- Head loss at 2 m^3/s 26 m
- Peak released flow 2.5 m^3/s

The initial 2 km of pipeline from the Ruamahanga pump station is only required to carry flows extracted from the Ruamahanga River and has been sized as 0.9 m diameter. Beyond this point, the pipeline must carry the released flows, and a larger pipe size of 1.2 m diameter is required to limit head loss in the initial section of pipeline and to provide pressure to the remainder of the distribution system. A non-return valve separates the two sections of pipeline.

At this pre-feasibility stage, it is envisaged that the pipe would be steel (5-6 mm wall thickness). At later stages, however, other options, such as HDPE, would be considered.

8 COST ESTIMATES

Capital cost estimates have been prepared for each of the components of the scheme. Estimates have been prepared by undertaking a conceptual design for each of the components, scheduling quantities based on these designs, and assigning rates to the items. Where possible, cost estimates derived in this manner have been checked against cost estimation formulae published by Southern Energy Group (1979). Montgomery Watson's extensive experience in the construction of similar projects also allowed for a further check of cost estimates.

The schedule of costs is presented in Appendix I. This schedule of costs does not include land acquisition, road reconstruction costs, or GST.

A number of assumptions were required to develop these pre-feasibility level estimates, namely:

- Assessments of storage volume and earthfill volumes were made from 1:50,000 scale topographical maps. No detailed survey has been undertaken, and the volumes can therefore only be considered approximate.
- A number of the intake sites were not visited, and detailed walkovers of the race routes were not undertaken. In the absence of site specific information, assessments were made from observations off 1:50,000 scale topographical maps.
- Apart from sites where previous studies had been undertaken, assessment of ground conditions was made from 1:250,000 scale geological maps. These maps are relatively coarse, and it is possible that localised ground conditions may introduce issues that have not been anticipated at this stage.

Overall, it is considered that these estimates are accurate to the order of ±30%. A contingency sum of 25% has been applied to all scheme components. This is considered a reasonable allowance for the level of uncertainty inherent in a pre-feasibility level assessment.

The overall cost estimates for each scheme are summarised here from Appendix I:

Scheme supply zone	Storage option	Total cost	Cost/ha
Opaki	Highfield	$18,300,000	$2,440
Carterton/Greytown/Opaki	Black Creek	$67,500,000	$2,368
Martinborough	Te Muna	$33,000,000	$5,500
	Hautotara	$36,500,000	$6,083

9 KEY ENVIRONMENTAL ISSUES

An overview of key environmental issues associated with the proposed water supply schemes follows:

- Taking water from rivers will reduce flow in these rivers. The taking of water simulated herein does, however, comply with WRC Fresh Water Plan (where applicable), although the river flows may not be redeemed at times of low flows.

- Reservoirs will be constructed for each scheme. These new water bodies will have associated landscape effects. The new water bodies will have associated habitats which will be dominated by the changing water levels of the reservoirs during various seasons (i.e. they will not be a stable or static ecological environment).

- The local communities and ecosystems will be affected by the construction, presence and operation of the intake structures, dams, and canal structures, including effects associated with:
  - Flooding by the new reservoir.
  - Changes in micro-climate associated with the reservoir.
  - Relocation of small settlements and associated infrastructure.
  - Nuisance effects from noise, dust, and traffic associated with the construction of the structures.
  - Dust nuisance from the edges of the reservoir lake and canals, particularly when reservoir levels are low.

- Discharge into the Tauherenikau River from the Carterton/Greytown/Opaki scheme will augment the flow in this river, leading to river ecological and morphological effects and changes to ecosystems and their components (both aquatic and terrestrial). In addition, issues of cultural and tangata whenua values will need to be addressed, particularly with regard to the mixing of water from two sources or more.

- In the short term, there will be disruption to farming activities from the water race construction. In the long term, the ease of access to portions of farms may become more difficult.

- Full development of the irrigable area able to be supplied from a surface water supply scheme would increase recharge of the Wairarapa Plain's groundwater systems. This is expected to help maintain flows in spring-fed streams and to maintain the accessibility and reliability of groundwater as a source. A reduction of surface drainage may, however, occur in low lying areas, due to elevated groundwater levels.

- There will be recreational and augmented tourism potential from the new reservoir sites.

- Excess irrigation scheme water may be used to augment identified wetland areas, should this be necessary.

10 DEVELOPMENT COST COMPARISONS WITH SIMILAR PROJECTS

A number of similar water supply projects are presently under investigation in New Zealand. These projects represent a new era of water supply development in that they depend on water storage to reliably match water supply to demand. In order to put the development costs of the proposed Wairarapa schemes into context, the costs of these projects are summarised in the table below:

Table 5: Estimated total annual cost of bulk water supply for East Coast schemes currently under investigation

Scheme	**Area served (ha)**	**Estimated total annual cost ($/ha/y)**
Canterbury – Selwyn district	85,000	182
Canterbury – Ashburton district	153,000	197
Marlborough – Southern Valleys	10,000	~320
Marlborough – Awatere Valley	8,000	$385 at the farm gate (includes distribution system cost)

The land area in Canterbury for which consents to take water for irrigation are held is approximately 400,000 ha. This area is twice the area that had consents to irrigate in 1985. Much of the area developed for irrigation since 1985 is supplied from groundwater.

The full cost of owning and operating a groundwater supply bore, pump and associated equipment depends primarily on the depth of the well and the pumping head. For areas of Canterbury west of State Highway 1 (where much of the diary farm development is occurring), the full cost of a groundwater supply for irrigation is typically in the range of $200/ha to $400/ha per year.

In order to compare the cost of groundwater supply with surface water supply, it is necessary to add the annual cost of owning and operating a water distribution system to the bulk water supply total annual costs presented in Table 5. Distribution system annual costs typically lie within the range $50/ha to $150/ha, depending on the type of system. Adding this cost to the bulk water supply costs results in estimated water supply scheme costs of the same order as those for a groundwater supply.

The financial benefits of irrigation to various land-uses in the Wairarapa Plain area have not been assessed. However, the benefits of irrigation in other east coast regions provide some indication of potential benefits in the Wairarapa Plain, although there may be differences due to variations in soil and climate characteristics.

Table 6: Estimated net benefit of water for irrigation, for a range of farm enterprises in Mid-Canterbury

	Location	**Water supply improvement**	**Enterprise**	**Increase in surplus: ability to pay for water (per ha)**
1	Border-dyke scheme	Improved water supply	Sheep/beef/mixed cropping	$220
2	Border-dyke scheme	Improved water supply	Dairy conversion	$832
3	Ruapuna	New irrigation	Livestock/cropping	$224
4	Ruapuna	New irrigation	Dairy Conversion	$657
5	Mayfield	New irrigation	Livestock/cropping	$207
6	Mid plains	New irrigation	Livestock/cropping	$319
7	River fringe	Improved water supply	Livestock/cropping	$146
8	Deep soils	New irrigation	Intensive cropping	$242
9	Seafield/Pendarves/Dorie	New irrigation	Livestock/cropping	$302
10	Seafield/Pendarves/Dorie	New irrigation	Dairy conversion	$1,112
11	Lyndhurst/Rokeby/ Hatfield	New irrigation	Intensive cropping/ stock trading	$305
12	Upper Highbank	New irrigation	Intensive cropping	$203

Differences in the capital value of land with water compared to land with no water is perhaps the best indication of the value of water for irrigation in the grape growing areas of Marlborough. At present, bare land without a reliable source of irrigation water sells for about $12,000/ha. Bare land with sufficient water for viticulture sells for about $50,000/ha. Land planted in grapes with a good water supply sells for about $140,000/ha. The ability to adequately irrigate therefore has significant impact on land capital value.

11 WATER SUPPLY SCHEME OWNERSHIP STRUCTURES

The most common ownership structures for irrigation schemes are "incorporated societies" and "limited liability companies". The major irrigation schemes that have been established over the past ten years have all been set up as companies. The most recent company, the Barrhill-Chertsey Irrigation Ltd, is a co-operative company.

Usually the shareholders of the water supply company are the water users. Both the Opuha and Waimakariri-Ashley schemes require a farmer to be a shareholder in order to be able to purchase water. Under this requirement, the farmers collectively carry some, or all of, the water supply companies business risk. Individually, they also carry the business risk of irrigation development on their own property.

In general terms, the Opuha and Waimakariri-Ashley schemes were financed by one third cash up-front and the balance by bank loan. The annual water charges cover loan servicing (principal and interest payments), plus operation and maintenance costs.

In some circumstances it might be appropriate to seek share capital costs and risk with other beneficiaries of irrigated agriculture. Processing companies, such as dairy factories and wineries, benefit for higher and more reliable production through better utilisation of their plant. Local communities benefit financially through servicing of a more economically active farming sector. Investment in water supply schemes by these sectors could be seen to be enlightened self-interest.

The two most difficult stages of financing water supply scheme development are the investigations stage (includes obtaining resource consents) and the initial years of scheme operation.

During the investigation and resource consenting stage the financial risks are usually considerable, and the financial support from prospective water users is usually limited to the enthusiasts. It is now common for this phase to be financed predominantly by a local authority (e.g. Ashburton District Council, Selwyn District Council, Christchurch City Council, Timaru District Council and Marlborough District Council). Once the project reaches the stage where a prospectus can be issued for financing detailed design and construction of the scheme, the financial risk begins to shift from the local authority to the potential water users.

The early years of operation of the water supply company are high risk years because a substantial part of the total investment in infrastructure will probably have been made, but water sales will not have built up to fully utilise the infrastructure. The rate of on-farm development generally controls the rate of growth in water sales. In some instances, it is economic to include hydroelectric generation in the scheme to make optimum use of the water supply system from day one. Another method of managing the financial risk during the early years is to carefully stage the development of water supply capacity to match the growth in water demand. This is probably more easily achieved where water is to be supplied via piped reticulation.

12 SUMMARY

(1) The total irrigable area in the Wairarapa Plain is approximately 122,000 hectares.

(2) There is enough water in the Wairarapa Plain's surface and groundwater resources to reliably irrigate the whole of the Wairarapa Plain, while maintaining instream river values. To achieve this requires the construction of at least two storage reservoirs and supply schemes to service areas of the plain that cannot be adequately sustained (in terms of irrigation) by groundwater resources.

(3) To meet irrigation demand 90% of the time, assuming well-managed spray irrigation techniques, requires:

- Maximum on-farm design flow rates for intensive pastoral farming of 0.5 ℓ/s/ha and 0.21 ℓ/s/ha for grape production.
- Seasonal water deliveries ranging from 160 mm per year in the western areas to 420 mm in the eastern areas.

(4) There is sufficient groundwater available in the Wairarapa Plain aquifers to sustain the potential irrigation demand. In some areas, however, this water cannot be physically extracted to supply the irrigation demand.

(5) The area supplied from groundwater is limited by the physical availability of groundwater in the district, rather than by the economics of pumping groundwater.

(6) The area of irrigable land that requires community irrigation scheme for reliable irrigation is 34,500 ha.

(7) Four options for surface water supply schemes have been presented, the costs of which range from $2,368 to $6,083 per hectare.

13 RECOMMENDATIONS

Based on the findings of the investigations undertaken herein, it is recommended that the following options be carried through for investigation in subsequent stages of the study:

- The option of supplying the Carterton, Greytown and Opaki supply zones from the Black Creek reservoir site, as this provides the most reliable water supply at the cheapest cost.
- The option of supplying the Martinborough supply zone from the Te Muna Reservoir site, as it is likely that the Hautotara site is be structurally feasible. The Te Manu Reservoir site is also the cheapest unit cost of both options.

The scale of the schemes investigated was designed to be very inclusive. As a result, areas have probably been included from which there may be only a small interest in irrigation development.

It is strongly recommended that a process of formal information sharing, awareness raising, and consultation occur, for the purpose of more tightly defining areas with a strong interest in irrigation development. The technical investigations that would follow could therefore be tightly scoped to obtain maximum value from the investment in those investigations.

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Appendix A:

Soil, Climate and Water Resource Data

Appendix B:

Computer Simulation Model

Appendix C:

Summary of Simulated Water Demands

Appendix D:

Summary of Non-Irrigation Consumptive Water Use

Appendix E:

Summary of Groundwater River Recharge

Appendix F:

Summary of Simulated Groundwater Recharge

Appendix G:

Aquifer Hydraulic Analysis

Appendix H:

Supply/Demand Matching

Appendix I:

Schedule of Costs