42197300/01/01
Prepared for:
Flaxbourne Community Irrigation Ltd
Prepared by URS New Zealand Limited
## DOCUMENT PRODUCTION / APPROVAL RECORD

<table>
<thead>
<tr>
<th>Issue No.</th>
<th>Name</th>
<th>Signature</th>
<th>Date</th>
<th>Position Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prepared by</td>
<td>Allen Ingles</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Checked by</td>
<td>Sioban Hartwell</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Approved by</td>
<td>Ron Flemming</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

## Document Production / Approval Record

### Report Name:
Flaxbourne Irrigation Scheme Preliminary Design Report

### Sub Title:
Report No. 42197300/01/01

### Status:
Client Contact Details:
Flaxbourne Community Irrigation Ltd
c/o John Patterson
Executive Finesse Limited
P O Box 744, BLENHEIM 7240

### Issued by:
URS New Zealand Limited
273 Cashel Street
Christchurch 8011
PO Box 4479, Christchurch 8140
New Zealand
T: +64 3 374 8500
F: +64 3 377 0655

© Document copyright of URS New Zealand Limited.
This document is submitted on the basis that it remains commercial in confidence. The contents are the intellectual property of URS New Zealand and are not to be provided or disclosed to third parties without the express written permission of URS New Zealand. No use of concepts, designs, drawings, specifications, plans etc. included in the document shall be permitted unless and until they are the subject of a written contract between URS New Zealand and the addressee. URS New Zealand accepts no liability of any kind for any unauthorised use of the contents of this document and URS New Zealand reserves the right to seek compensation for any such unauthorised use.

### Document Delivery:
URS New Zealand provides this document in either printed format, electronic format or both. URS New Zealand considers the printed version to be binding. The electronic format is provided for the client's convenience and URS New Zealand requests that the client ensures the integrity of this electronic information is maintained. Storage of this electronic information should at a minimum comply with the requirements of the Electronic Transactions Act 2002.
# TABLE OF CONTENTS

**EXECUTIVE SUMMARY** ........................................................................................................... 1

1 INTRODUCTION ......................................................................................................................... 3

2 SCHEME DESCRIPTION AND LAYOUT ...................................................................................... 5

2.1 Scheme Description .................................................................................................................. 5

2.2 Scheme Layout .......................................................................................................................... 6

2.2.1 Awatere River Intake ........................................................................................................... 6

2.2.2 Pump Station ......................................................................................................................... 6

2.2.3 Rising Main ............................................................................................................................ 7

2.2.4 Storage Dam ......................................................................................................................... 7

2.2.5 Distribution Network ............................................................................................................ 7

2.2.6 Control System ...................................................................................................................... 7

3 HYDROLOGICAL ASSESSMENT ................................................................................................. 9

3.1 Model Input and Assumptions ................................................................................................. 9

3.1.1 Irrigation Demand ............................................................................................................... 9

3.1.2 Water Availability from Awatere River .............................................................................. 10

3.1.3 Storage Dam .......................................................................................................................... 11

3.2 Intake Shutdown ....................................................................................................................... 12

3.3 Model Scenarios ...................................................................................................................... 13

3.4 Water Balance Results ........................................................................................................... 13

3.4.1 Discussion .............................................................................................................................. 14

3.5 Climate Change Implications .................................................................................................. 16

4 GEOTECHNICAL INVESTIGATIONS ......................................................................................... 17

4.1 Regional Geology ...................................................................................................................... 17

4.2 Regional Seismicity ................................................................................................................... 17

4.3 Preliminary Seismic Hazard Assessment ................................................................................ 18

4.3.1 Probabilistic hazard assessment based on the NZNSHM ................................................... 18

4.4 Site Investigations .................................................................................................................... 19

4.4.1 Stage 1 February 2008 .......................................................................................................... 19

4.4.2 Stage 2 June 2008 .................................................................................................................. 19

4.5 Reservoir and Damsite Geology .............................................................................................. 21

4.5.1 Quaternary Stratigraphy .................................................................................................... 21

4.5.2 Tertiary Stratigraphy ............................................................................................................ 21

4.5.3 Greywacke ........................................................................................................................... 22

4.6 Foundation Conditions ........................................................................................................... 22

4.7 Embankment Dam Material Suitability and Availability ....................................................... 22

4.7.1 Core material ......................................................................................................................... 22

4.7.2 Shoulder material ............................................................................................................... 23
8.3 Valves ........................................................................................................... 46
9 STORAGE DAM ................................................................................................. 47
9.1 General Criteria ............................................................................................ 47
9.2 Dam Site Location and Layout ...................................................................... 47
9.3 Potential Impact Category Assessment ........................................................ 47
  9.3.1 NZSOLD Potential Impact Categories .................................................... 48
  9.3.2 Potential Incremental Consequences of Failure ...................................... 49
  9.3.3 Building (Dam Safety) Regulations 2008 Methodology ......................... 50
  PIC Classification– Design Criteria .................................................................. 51
  9.3.4 51
9.4 Diversion Flood and Diversion Works .......................................................... 51
9.5 Alternative Dam Types .................................................................................. 52
  9.5.1 Roller compacted concrete (RCC) .......................................................... 53
  9.5.2 Hardfill .................................................................................................... 53
10 COST ESTIMATES ........................................................................................... 55
  10.1 Cost Estimate Assumptions ....................................................................... 55
  10.2 Summary of the Cost Estimates .................................................................. 58
11 LIMITATIONS ................................................................................................ 60

TABLES
Table 3-1 Water demand requirements .................................................................. 10
Table 3-2 Schemé Re liability ................................................................................ 14
Table 3-3 Dam Storage ......................................................................................... 16
Table 5-1 Summary of existing Resource Consents ............................................ 26
Table 5-2 Summary of Resource Consents required under the Wairau/Awatere Resource Management Plan ......................................................................................... 27
Table 6-1 Gallery Intake Critical issues ................................................................. 36
Table 6-2 Hydraulic design Parameters ............................................................... 37
Table 7-1 Option 1 Pipe Schedule ....................................................................... 41
Table 7-2 Option 2 Pipe Schedule ....................................................................... 42
Table 9-1 Potential Impact Categories for Dams in Terms of Failure Consequences (from NZSOLD (2000), Table III.1) ......................................................... 48
Table 9-2 Ward Dam Dimensions (120 m RL Reservoir Option) ......................... 49
Table 10-1 Summary of Costs ............................................................................. 59

FIGURES
Figure 2-1 Scheme Layout .................................................................................. 5
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
</table>

ABBREVIATIONS
EXECUTIVE SUMMARY

The proposed Flaxbourne Community Irrigation Scheme services approximately 2200 ha of land, in the Faxbourne area Grassmere to just south of Ward township. The scheme consists of an intake and pump station at the Awatere River which pumps water to a storage dam on the southern side of Lake Grassmere on Flaxbourne Station via a 600 – 750mm diameter rising main. Water will be pumped to storage during the winter period. During the irrigation season water will be pumped to the service areas from the river when water is available with the shortfall being provided from the storage dam when the abstraction from the Awatere River is restricted or demand cannot be met from the river alone.

A dam capable of providing up to 2.6M m$^3$ of usable storage can be built at the proposed dam site. The dam would be a zoned earth embankment dam and can be constructed using materials sourced from within the storage area and area immediately adjacent. The majority of the dam can be constructed with minimal modification of the insitu materials, the exception to this being the filter and drainage material which will require a high level of processing to achieve the required grading.

The hydrological assessment of the scheme, based on available Awatere flow data and climatic data, shows that a reliability of 97.3% can be achieved for 2200ha of service area with an 85%/15% viticulture/mixed cropping landuse, a dam of 2.6M m$^3$, and a transfer capacity of 450L/s. The assessment has assumed supply at full demand and has not considered managed restriction of supply during drier years. This is a refinement that could be investigated.

At present Class A abstractions from the Awatere River have a 98% reliability. Based on the assessment of the proposed scheme it would have a reliability of 97.3%.

Construction of the scheme will require a number of consents from the Marlborough District Council. A number of consents were obtained a part of the previous work carried out by Ward Water Company (Waterco) however the majority of these have lapsed, are about to lapse or would require modification due to changes in design criteria. As per the Waterco scheme it is possible that these consents may be non-notified if written approval can be obtained from all neighbours and affected parties.

The estimated cost for the construction of the scheme is $38,838,500 with an estimated annual operation maintenance cost of $967,500 per year. The total capital cost consists of a construction cost of $28,813,700, other owner’s cost of $3,071,400 and a contingency allowance of $7,153,400.
INTRODUCTION

Flaxbourne Community Irrigation Ltd (FCIL) is a community group set up to investigate and promote the development of a community irrigation scheme for the Flaxbourne area in Marlborough. FCIL commissioned the investigation of a scheme damming and utilising water from the Flaxbourne River but at present there is some doubt about the reliability of the scheme. (Engineering Feasibility study Report, Tonkin & Taylor July 2013).

Ward Water Company (Waterco), a subsidiary of the Yealands Group carried out investigation for the development of an irrigation scheme, using water from the Awatere River and servicing approximately 2,500 ha of land for vineyards in the Grassmere and Ward area. A preliminary investigation was completed in 2008 (Ward Water Irrigation Preliminary Design Report April 2008) along with consenting for the various elements of the scheme and geotechnical site investigation for the dam. The scheme did not proceed further due to economic circumstances. Peter Yealands released the findings from the 2008 investigation to FCIL in early 2014.

In August FCIL engaged URS NZ Ltd to update the 2008 Yealands report for a revised scheme for the Flaxbourne area, including a distribution network servicing identified properties in the Flaxbourne area and updating the cost estimates for the revised scheme.

This document reports the results of the preliminary design review of the revised scheme to allow confirmation of the scheme viability. It includes a proposed layout for the scheme and consideration of additional scenarios that may warrant consideration for optimisation should the scheme proceed to detailed design.
2 SCHEME DESCRIPTION AND LAYOUT

2.1 Scheme Description

The Awatere River is the most significant source of water for larger scale irrigation in the Seddon, Grassmere and Ward area. Abstractions are restricted during the summer irrigation period due to low flows within the river. Three levels of abstraction consent currently exist for the river (Class A, B and C) based on the flow at the Awapiri recorder with Class A providing the highest level of reliability of supply. Class A has been fully allocated and there is a limited quantity of Class B available that FCIL has applied for. There is still a significant quantity of Class C available, however this is prone to restriction for a large proportion of the irrigation season. The FCIL scheme uses a combination of Class B & C water, supplemented with stored water, to provide irrigation water at an appropriate reliability.

The proposed scheme is to provide irrigation to 2200 ha of land in the area between Grassmere and Ward based on a water demand suitable for the establishment and operation of vineyards and a limited area of mixed cropping. The scheme will take water from the Awatere River and pump it to storage in a dam located on Flaxbourne Station, on the south side of Lake Grassmere, when water is available and there is no irrigation demand. During the irrigation season, water for irrigation will be taken from the Awatere River, when flows are sufficient to allow abstraction, with demand supplemented by water from the dam. When restrictions on abstractions from the Awatere River are imposed, due to reduced flows, irrigation demand will be met using water from the storage dam. Figure 2-1 below shows the layout of the proposed scheme.

Figure 2-1 Scheme Layout
Construction of the scheme would require the intake, transfer line and dam to be completed prior to the winter season to allow filling of the dam for the irrigation season. Construction of the distribution system would extend over the winter season with completion and commissioning required prior to the irrigation season commencing.

Irrigation demand is not expected to match the design demand for the first few years of operation as properties and vineyards are developed and established.

The proposed scheme will have a design life of 100 years and comply with all relevant NZ Standards and Regulations.

2.2 Scheme Layout

The key aspects of the scheme layout are as follows;

- An intake structure in the Awatere River near Seaview.
- A pump station adjacent to the river intake to pump water to the storage dam or directly to irrigation areas.
- A rising main to transport water from the river to the storage dam or to the various irrigation areas.
- A storage dam with a storage capacity of 2.6 M m³.
- A distribution network for the service area receiving flow from the transfer line and/or the storage dam.
- A control system for the pumps and distribution valves to enable remote control of the overall system from a central control room.

2.2.1 Awatere River Intake

The intake will consist of an infiltration gallery of perforated pipes contained within a graded gravel surround. Initial design has indicated that multiple galleries will be required to meet peak demand while ensuring high infiltration rates do not lead to rapid blinding of the system with sediment. The multiple galleries will drain to a collection sump from which the water is pumped.

The Awatere River is a braided river system within a larger incised flood channel. A surface intake for the irrigation scheme was not considered as the river has a high sediment load and flood events in the river can result in the relocation of the main flow channel. A surface intake would require a high level of maintenance to maintain flows and a treatment system for sediment removal.

2.2.2 Pump Station

The pump station will be located on the true right bank of the river above the main channel. It will consist of multiple pumps. At least one of the pumps will be controlled by a variable speed drive to allow variation in flows, as required when restrictions in abstraction from the river are imposed. The pump station will have sufficient capacity to pump the design flow to the storage dam without additional boosting and will provide the 450 L/s to supplement water from the dam to meet the peak irrigation demand to the service areas.
2.2.3 Rising Main

The rising main will transport water from the Awatere River to the storage dam and to the irrigation areas. A more detailed description of the pipeline system and alignment is provided in Section 7.

2.2.4 Storage Dam

The storage dam will be located on Flaxbourne Station to the south of Lake Grassmere on a site identified by Waterco in 2008. The 2008 investigation indicated a dam providing up to 2.6M m$^3$ of live storage could be constructed on this site. Available survey indicates that it would be possible to increase the size of the dam structure but there may be other constraints to the maximum dam size in the storage catchment. Further survey work would be needed to confirm this.

While a number of options are discussed, the dam is most likely to be an earth embankment structure constructed using materials obtained within or adjacent to the storage area of the dam. Further detail of the dam including proposed footprints and typical sections are provided in Section 9.

Geotechnical investigations at the site have identified two faults in the vicinity of the dam. While neither of these pass through the proposed footprint of the dam, these will need to be examined further to assess the return period and potential for movement.

2.2.5 Distribution Network

The distribution network is a piped system providing water to the demand area from the transfer pipeline and or the storage dam. The preliminary design for the network is based on the demand and property information provided. Pipe installation is assumed to be located within the road easements and water will be delivered to the farm gate at a minimum head of 20m.

2.2.6 Control System

The scheme will be controlled from a central Communication will be provided to pump stations, automated valves, flow meters and level recorders to allow monitoring and control of all aspects of the scheme operation. Communication will be either via radio link to the equipment or could also be via fibre optic cabling installed with the pipeline.
3 HYDROLOGICAL ASSESSMENT

The 2008 URS report included a water balance model of the proposed irrigation scheme. The model included multiple demand areas, an intake on the Awatere River and a storage pond. This model has been updated to reflect the current aspirations of Flaxbourne Community Irrigation Ltd. (FCIL) scheme and reliability has been tested for a number of scenarios.

3.1 Model Input and Assumptions

The hydrology for this project is based on the gauged flow of the Awatere River and data from the Virtual Climate Station (VCS) network for rainfall and evaporation (node - P163116). This data enables the previous modelling to be extended to 2014 allowing irrigation reliability to be evaluated over a 25 year period (1/06/1989 to 31/05/2014). Testing over multiple seasons is essential when evaluating storage dependent irrigation supply schemes to ensure climatic variation can be captured.

The model uses daily demand downscaled from average monthly water demand, incorporates available abstraction from the Awatere River and models the live storage volume for defined input to determine supply reliability and shortfall.

The model consists of the following components:

- Water demand (based on monthly average demand supplied by FCIL);
- Rainfall (used in the calculation of the irrigation demand);
- Water abstraction from Awatere River (based on flow records, consent conditions and suspended sediment levels);
- Water storage dam (variable for different scheme options); and,
- Evaporation losses from the dam.

Rainfall on to the dam is not considered as its influence is believed to be negligible; this scenario is conservative.

3.1.1 Irrigation Demand

Average monthly demands have been provided in Table 3-1 for viticulture and mixed cropping land use types.

Cumulative daily demands have been calculated based on a total irrigable area (a model variable) and an apportioning of this land as 85% viticulture and 15% mixed landuse. Daily irrigation demand is estimated as the total daily demand minus the effective rainfall for the day. Irrigation is required between October and April.
### Table 3-1  Water demand requirements

<table>
<thead>
<tr>
<th>Month</th>
<th>Viticulture (m$^3$/ha/month)</th>
<th>Mixed Cropping (m$^3$/ha/month)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>790</td>
<td>1,003</td>
</tr>
<tr>
<td>February</td>
<td>510</td>
<td>348</td>
</tr>
<tr>
<td>March</td>
<td>230</td>
<td>259</td>
</tr>
<tr>
<td>April</td>
<td>0</td>
<td>46</td>
</tr>
<tr>
<td>May</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>June</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>July</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>August</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>September</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>October</td>
<td>0</td>
<td>269</td>
</tr>
<tr>
<td>November</td>
<td>140</td>
<td>801</td>
</tr>
<tr>
<td>December</td>
<td>580</td>
<td>1,125</td>
</tr>
</tbody>
</table>

#### 3.1.2 Water Availability from Awatere River

Daily mean flow records for the Awatere River have been provided by Marlborough District Council (MDC) for 1977 to 2014. During the early years of this record there are substantial gaps. For this study the period of 1/06/1989 to 31/05/2014 has been selected as it has a low percentage of missing data.

For the previous study minor gaps within the flow Awatere river flow record were closed using a synthesised hydrograph based on a developed cross-correlation with recorded flow data from the adjacent Waihopai River catchment. For each of the gaps a correlation was developed using flow data for days immediately before and after each gap. This method provided acceptable correlations and in addition the number of days with gaps in the flow record is minor compared to the total record used. For new data (post 2007) minor gaps have been in-filled using linear interpolation.

Water available for abstraction is based on the recorded Awatere flow and Marlborough District Council (MDC) rules and regulations of allowable abstractions as shown below.

- **Class A** – 1000L/s when the flow is above 2300L/s at the Awapiri flow recorder progressively reducing to 0 L/s at a flow of 1450 L/s.
- **Class B** - 2600 L/s when the flow is above 5600 L/s at the Awapiri flow recorder progressively reducing to 0 L/s at a flow of 2300 L/s (flow sharing); and,
- **Class C** - 67 % of any flow in excess of 5600 L/s at the Awapiri flow recorder, no upper limit.

FCIL has a consent application in for 50L/s of Class B water and is looking at options for obtaining additional Class B water.

Within the model, variables have been included to define the maximum Class B and Class C takes for FCIL. Using these allocations rules a flow rate of water available for abstraction has been calculated.

Previously the Class C rule was not implemented within the water balance model. In this revision an option to utilise this rule is included. For most scenarios, the influence of this rule is negligible.
The flow duration curve compiled from the 25 years of data for the Awatere River, Figure 3-1, shows that Class A minimum flow of 1450 L/s is exceeded 99.9 % of the time, and the flow of 2300 L/s for Class A’s full take is exceeded 98.0 % of the time.

Figure 3-1  Awatere Flow Duration Curve.

3.1.3 Storage Dam

Based on preliminary design, an active storage volume of 2.6 Mm³ was found to be feasible, based on a dam crest height of RL 120 m and a Full Supply Level (FSL) of RL 118 m. This includes additional 300,000 m³ of storage created by material excavation for construction of the embankment.

The dead storage level was assumed to be on RL 100 m which equates to a water volume of approximate 230,000 m³ of water that cannot be used because it is below the offtake level. This should be reviewed during further design to determine whether this volume can be reduced.

The reservoir storage volume curve shown on Drawing SK010 (see Appendix F) indicates that approximately 200,000 m³ of storage can be gained for every 1 m increase in reservoir level. The topographical survey carried out for the preliminary design did not extend above RL 120 m, however, subject to the results of additional survey that will need to be carried out, it appears that there is an opportunity to gain a further 5 m of storage. This may require a low “saddle” dam along the low lying eastern ridge, but this could be a low cost homogeneous dam structure.
There is also an opportunity to examine the assumed 2m freeboard during detailed design, with a view to increasing the FSL by possibly 0.5 m. This would need to take into account wave run-up in high winds and remaining free-board following a significant seismic event.

The evaporation losses from the reservoir surface area were estimated using the daily evaporation data and a stage area relationship developed based on the contour data.

Two additional storage scenarios were also tested but would need additional survey to confirm their feasibility these are:

- Total storage volume of 3.21 Mm$^3$ - representing current dam height + 3 m; and,
- Total storage volume of 3.72 Mm$^3$ - representing current dam height + 5 m.

3.2 Intake Shutdown

High levels of suspended sediment are known to occur in the Awatere River, particularly when there is high intensity localised rainfall within the catchment. This type of rainfall causes erosion and bank collapse. Due to the localised nature of this rainfall, there is no clearly definable relationship between suspended sediment and the Awatere River (Figure 3-2).

During times when suspended sediment levels are high, the intake will need to shut down to prevent damage to infrastructure and clogging of irrigation systems. Little information is available detailing the likely length and occurrence of shutdowns. For the Blind River Irrigation Scheme, shutdown periods due to dirty water total 46 and 86 days for two consecutive irrigation seasons (2008 – 2009 and 2009 – 2010). No other information is available.

To incorporate some accounting for shutdowns due to dirty water a flow limit for the Awatere River is included. When the river exceeds this limit the intake is closed for two consecutive days. A flow limit of 20 m$^3$/s was selected as it provides an average of 5 days shutdown per month during the irrigation season.

\[
y = 0.3203x^2 - 2.3296x
\]

\[R^2 = 0.7873\]
3.3 Model Scenarios

At this stage of the study the exact details of the irrigation scheme and consent conditions are unconfirmed. We have investigated a number of scheme scenarios varying dam storage volume, take consent conditions and land use mix. Irrigation area has also been adjusted for several scenarios so that an acceptable reliability is achieved.

Three storage scenarios have been tested:
1) 2.60 Mm$^3$;
2) 3.21 Mm$^3$ - dam height increased by ~3 m; and
3) 3.72 Mm$^3$ - dam height increased by ~5 m.

Three take scenarios have been tested:
1) Max take 250 L/s – 250 L/s Class B;
2) Max take 450 L/s – 250 L/s Class B and 200 L/s Class C; and,
3) Max take 450 L/s – 450 L/s Class B.

Two land use scenarios have been tested:
1) 85% viticulture and 15% mixed cropping; and
2) 100% viticulture.

The following is assumed for all scenarios:
- Dead storage volume of 0.23 Mm$^3$;
- No Class C flow sharing; and,
- Intake closed for two day (minimum) due to suspended sediment when Awatere River flow exceeds 20 m$^3$/s – equivalent to an average of 5 days shutdown per month of the irrigation season.

3.4 Water Balance Results

The scenarios outlined in Section 3.3 have been evaluated and are summarised in Table 3-2 below (a more detailed table of results is provided in Appendix A). The following descriptors are reported for each scenario:

1) Time based reliability – ‘days of shortfall’ divided by ‘days of demand’;
2) Time based reliability for the viticulture irrigation season only (for scenario comparison) – ‘days of shortfall’ divided by ‘days of demand’ for the viticulture irrigation season;
3) Volumetric based reliability – total volume supplied divided by ‘total volume requested’;
### Table 3-2 Scheme Reliability

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Irrigated Area (ha)</th>
<th>Landuse</th>
<th>Max Take</th>
<th>Total Storage Vol (L/s)</th>
<th>Time Based Reliability (%)</th>
<th>Time Based Reliability - Viticulture Period (%)</th>
<th>Volumetric Reliability (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>2200</td>
<td>85%/15%</td>
<td>250</td>
<td>2.6</td>
<td>92.08</td>
<td>89.92</td>
<td>91.30</td>
</tr>
<tr>
<td>2</td>
<td>1835</td>
<td>85%/15%</td>
<td>250</td>
<td>2.6</td>
<td>97.32</td>
<td>96.27</td>
<td>97.42</td>
</tr>
<tr>
<td>3</td>
<td>2200</td>
<td>85%/15%</td>
<td>450</td>
<td>2.6</td>
<td>97.34</td>
<td>96.3</td>
<td>96.76</td>
</tr>
<tr>
<td>5B</td>
<td>2475</td>
<td>85%/15%</td>
<td>450</td>
<td>3.21 (+ ~3 m)</td>
<td>97.32</td>
<td>96.27</td>
<td>97.28</td>
</tr>
<tr>
<td>6B</td>
<td>2700</td>
<td>85%/15%</td>
<td>450</td>
<td>3.72 (+ ~5 m)</td>
<td>97.32</td>
<td>96.27</td>
<td>97.4</td>
</tr>
<tr>
<td>8C</td>
<td>2315</td>
<td>100%/0%</td>
<td>450</td>
<td>2.6</td>
<td>96.32</td>
<td>96.32</td>
<td>96.73</td>
</tr>
</tbody>
</table>

#### 3.4.1 Discussion

Based on the modelling carried out, provided the scheme’s intake and transfer pipe capacity are increased to 450 L/s and FCIL are able to gain consent to take a minimum of 250 L/s Class B water, a time based reliability of 97% or greater is achievable using a dam with 2.6 Mm³ of storage (tested over 25 years). This is reported as scenario 3 in Table 3-2.

If the transfer pipe capacity is limited to 250 L/s, consents are gained for 250L/s of Class B water and dam storage is 2.6 Mm³ then to achieve a reliability of greater than 97% the irrigated area must be reduced 1835 ha, a reduction of 365 ha.

With the inclusion of more dam storage (2.6 Mm³ baseline with up to ~1.1 Mm³ additional) a reliability of 99.74% can be achieved for the same service area. Alternatively the irrigation area can be increased to 2700 ha (scenario 6B) or the transfer form the Awatere River reduced to 260 L/s (250 L/s Class B, 10 L/s Class C) while still maintaining a reliability of greater than 97%. If the irrigation demand is adjusted to 100% viticulture then for pipe capacity of 450 L/s (250 L/s Class B and 200 L/s Class C), and a storage capacity of 2.6 Mm³, the irrigated area can be increased to 2315 ha whilst maintaining the same reliability as the 85% viticulture/15% mixed cropping scenario (time based reliability for viticulture period only). Similarly, if the irrigation area is held at 2,200 ha then the transfer capacity can be reduced to 375 L/s whilst maintaining the same reliability as the 85% viticulture/15% mixed cropping scenario. This is reported as scenario 8C and 8D in Table 3-2.

Modelling also assessed availability of additional water for on farm storage. This showed that for a transfer pipeline of 450 L/s on average there would be 3.69 Mm³ available for on farm storage after the dam (2.6Mm³) had been filled, however this varies significantly from year to year as does the reliability and in 2009 no water would have been available.

When evaluating storage based irrigation schemes there is a direct relationship between the storage volume and the irrigation reliability. As the live storage volume of the reservoir is increased in the model, the scheme reliability increases and days with a water shortfall decrease. Days with shortfall are defined as days with irrigation demand but with no water available or insufficient (from river and/or dam) to fully meet demand.
The time reliability, as an indicator for reliability of supply, is the relationship between total days of demand and the days on which the demand is fully met (expressed as percentage) within the 25 year model period. This reliability takes no account the magnitude of shortfall. Volumetric reliability is also provided; a second indicator of reliability. This is the relationship between the total volume of water demand and the total volume of water supplied.

The irrigation season depends on the landuse mix. For example, viticulture requires water November through March whereas mixed cropping requires water October through April (Table 3-1). To enable a comparison of landuse scenarios (e.g. 85% Viticulture and 15% mixed cropping vs 100% mixed cropping) a third indicator of reliability is reported (see Appendix A). This is time based reliability (as defined before) for the viticulture irrigation season only. The standard time based reliability is biased by the number of demand days. For 100% viticulture the irrigation season and number of demand days is less than an 85%/25% mix (viticulture/mixed cropping); therefore for the same number of days shortfall, the time based reliability is poorer.

In general, there comes a point in the irrigation season demand cannot be met by run-of-the-river water alone and water from storage is required. When this occurs stored water depletes rapidly and in some years all stored water is used. As irrigation demand reduces later in the season and river flows increase the dam is gradually refilled. Figure 3-3 visualises the stored water volume in the dam for scenario 3. This shows the overall behaviour of the storage based system. In particular long periods with drawdown (demand not met from river) can be seen and these events are the main driver in order to establish the reliability levels for different scheme options.

![Figure 3-3 Storage volume for scenario 3 – 2.6 Mm³ dam storage](image)

The number of consecutive days with shortfall is a very useful indicator in order to understand the different reliability levels for a given model scenario. Figure 3-3 shows an example of how
dam storage affects the number of consecutive days of shortfall. Consideration of what is an acceptable period is needed; this will depend on crop type. No consideration of management measures restricting demand has been considered in the modelling.

### Table 3-3 Dam Storage

<table>
<thead>
<tr>
<th>Dam Storage (M m³)</th>
<th>Time Based Reliability (%)</th>
<th>Max. Consecutive Shortfall (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.6</td>
<td>97.34</td>
<td>27</td>
</tr>
<tr>
<td>3.21</td>
<td>98.83</td>
<td>18</td>
</tr>
<tr>
<td>3.72</td>
<td>99.74</td>
<td>11</td>
</tr>
</tbody>
</table>

The results shown in Table 3-2 are intended to support the decision making process for scheme development. The applied model could be extended and refined for various storage and irrigation management options in order to support a scheme optimisation phase, if required.

The model period of 25 years of continuous data is considered sufficient to predict the irrigation reliabilities and storage requirements for a preliminary design level. However, the period is not long enough to predict long-term trends and climate variations.

### 3.5 Climate Change Implications

Climate change is a subject that has many contributing factors and many unknowns. Expected climate change in New Zealand can be broadly summarised as having an increase in westerly winds, an increase in temperature and a tendency for rainfall to increase in the south and west and to decrease in the north and east.

From these future projections, the proposed irrigation scheme located in the Marlborough Region on the north-east of the South Island is likely to experience increased temperatures and/or dry conditions. The magnitude of these changes in the next 20-30 years is expected to be similar to a ‘hot’ year at present (MFE, 2004).

No climate change assessment has been carried out however the 25 year record does include the years of 1997/98, 2000-2002 and 2007/08 which were known to be relatively extreme years. It is assumed that these events are similar to the predicted ‘hot/dry’ conditions where river flow will be lowest and irrigation demand high. For the baseline scenario (Scenario 3 – Table A1), where the dam is 2.6 M m³ and the maximum take is 450 L/s, the dam empties completely for these years and is only just refilled prior the next season.

### References

4 GEOTECHNICAL INVESTIGATIONS

4.1 Regional Geology

The north-eastern Marlborough region encompasses an area of active deformation that is associated with the main boundary zone between the Australian and Pacific plates. The plate motion is partitioned onto the Marlborough fault system which includes a series of northeast striking dextral strike-slip faults, including the Wairau, Awatere, Clarence and Hope Faults.

The basement rocks in the Marlborough area are dominated by Mesozoic (ca. 135 – 195 Ma) sedimentary rocks of the Torlesse terrane which include indurated sandstones and mudstones, usually referred to as greywacke. Unconformably overlying the greywacke to the south of Lake Grassmere (near the site area) is a sequence of marine siltstones, sandstones and mudstones of Late Miocene and Early Pliocene age (ca. 10 – 2 Ma), forming the Upton and Starborough Formations (Awatere Group), respectively. These deposits have been folded into the Post Pliocene Ward Syncline which has a general north-northeast orientation.

4.2 Regional Seismicity

The close proximity of the site to the major tectonic boundary of the Pacific and Australian plates means that this is a seismically active area. We have reviewed web-based databases maintained by GNS (New Zealand Active Faults Database (NZAFD)). The faults that contribute most to the seismic hazard of the scheme include the Hope, Clarence and Awatere Faults, and the Jordan Thrust Fault, all of which generate earthquakes with magnitude greater than Mw 7 every 300 to 2000 years. The London Hills Fault and a recently discovered, unnamed fault are located within close proximity of the dam site. These faults are discussed in more detail below.

**London Hill Fault**

This fault passes across the south-eastern end of the proposed reservoir, striking in a north to northeast direction, and forming the contact between greywacke and Tertiary sediments. Mapped outcrops of greywacke and Tertiary rocks were recognised in the field which helped determine the approximate location of the fault. No recent fault scarp was observed.

The NZAFD indicates that the fault has an estimated rupture recurrence interval of 3500 to 5000 years, however there is no available data indicating the last rupture event, slip rate or single event displacement.

**Unnamed Fault**

Recent inspections carried out by Geotech Consulting Ltd, 2007, indicate the presence of a fault north of the proposed dam site which exhibits an obvious scarp in the field. However the continuation of the fault trace south, adjacent to the proposed dam, was more difficult to discern. Inferred projection of the fault was made from the observation of subtle lineaments in aerial photographs and was shown to pass southwest of the proposed dam alignment. The current investigations confirm that this fault passes close to the dam but not through the dam footprint.
**Hog Swamp Fault**

The Hog Swamp Fault which is located approximately 12km northwest of the proposed dam site and crosses the pipeline route, near the Otuwhero River. This is a northeast striking, dextral strike slip fault. According to NZAFD, no recent events have been recorded, however the fault may have been active in 1966 during the Seddon earthquake (Adams & Lensen, 1970). No slip rates or single event displacement events have been established. The recurrence interval for this fault is considered to be in the order of 5,000 to 10,000 years.

### 4.3 Preliminary Seismic Hazard Assessment

A review of the NZSOLD dam design guidelines indicates that a dam is required to be designed for two levels of earthquake:

- **The Operating Basis Earthquake (OBE)** which should cause either no damage or minor repairable damage to a dam represents ground motions that have an annual exceedence probability (AEP) of 1/150 years.

- **The Maximum Design Earthquake (MDE)** is the maximum level of ground motion for which a dam should be designed. The dam must be designed to withstand the MDE without severe damage (i.e. without catastrophic release of the reservoir). The MDE can be determined probabilistically in the case of low or medium PIC dams, but is usually evaluated deterministically for high hazard dams. Selection of the MDE is based on the potential impact classification (PIC).

#### 4.3.1 Probabilistic hazard assessment based on the NZNSHM

The New Zealand National Seismic Hazard Model (NZNSHM) predicts the seismic hazard throughout New Zealand using a probabilistic seismic hazard assessment method (Stirling et al. 1999, 2007). The model predicts future seismic hazard by considering historical seismicity as well as geological data characterising active faults. Estimates of peak ground acceleration predicted for the Ward dam site by the NZNSHM are presented in Table 4-1. These data indicate that the scheme is located in an area with a high seismic hazard and this will need to be recognised in the design of the scheme, particularly in the dam design.

<table>
<thead>
<tr>
<th>Return Period (years)</th>
<th>Stirling et al. (2007) pga (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>150</td>
<td>0.36</td>
</tr>
<tr>
<td>475</td>
<td>0.67</td>
</tr>
<tr>
<td>1000</td>
<td>0.84</td>
</tr>
</tbody>
</table>

**Figure 4-1** Estimates of peak ground acceleration (pga) for the Ward area

As mentioned above, selection of the MDE is based on the PIC. For “High” PIC dams the MDE is usually the Maximum Credible Earthquake (MCE), which is defined as the largest reasonably conceivable earthquake that appears possible along a recognised fault or within a geographically defined tectonic province, or a 1 in 10,000 AEP event. For “medium” and “low” PIC dams, the MDE usually has ground motions less severe than the 1 in 10,000 event (NZSOLD, 2000).
The NZNSHM estimates are adequate for preliminary design of the dam, but given the 2013 earthquakes in the Cook Straight and Grassmere region, a site specific assessment of seismic hazard should be carried out during the detailed design stage.

Selection of the seismic design loads is based on the Potential Impact Classification (PIC) of the dam. A higher the potential impact classification results in higher return periods for the seismic design criteria. This is covered in more detail in Section 8.

4.4 Site Investigations

Two stages of site investigations have been carried out, comprising geological mapping and test pits in the first stage followed by three deep boreholes and additional test-pits to identify construction materials.

4.4.1 Stage 1 February 2008

An extensive walkover survey was undertaken in early February 2008 to locally map the area proposed for the reservoir. Outcropping greywacke, Tertiary and Quaternary deposits were observed within the proposed dam footprint and surrounding catchment area. Based on the walkover survey, various locations across the site were identified for possible subsurface investigations.

The subsurface investigations included the excavation of test pits which was undertaken over a two day period in mid February 2008. Test pitting aimed to determine the depth of soils, the thickness of the gravel terraces and to determine the suitability of soils as a potential source for use in construction of the proposed dam.

A total of 12 test pits were excavated across the site, using a (35 ton) excavator. The test pits were excavated to either refusal, or to the limit of the excavator reach. One test pit (TP10) was terminated due to water inflow and collapse of the test pit. The depth of the test pits ranged from 2.7 to 7.1m below ground level.

Bulk and disturbed soil samples were collected from the majority of the test pits and selected samples were laboratory tested. Test pit logs and photographs are presented in Appendix B.

On a separate site visit in late February 2008, a trench (TR01) was excavated close to the footprint of the proposed left abutment of the dam to determine whether a fault, originally identified by Geotech Consulting Ltd, 2007 downstream of the proposed dam, continued through the dam footprint.

4.4.2 Stage 2 June 2008

Additional site investigations were undertaken at the site between 10 and 20 June 2008. These included the drilling of 3 boreholes and the excavation of 13 test pits and 2 trenches.

4.4.2.1 Test Pits

The test pits were excavated within gravel terraces, located south of the dam site beyond the extent of the eastern arm of the proposed reservoir, to investigate potential source volumes of gravels for use as dam construction materials. The pits were excavated using a CAT 325B (30 ton) tracked excavator with 1.5m wide toothed bucket.
Two separate gravel terraces were investigated. The closest terrace to the reservoir was investigated with 6 test pits to the limit of reach of the excavator (from 7.0 to 7.1m below ground level). The test pits encountered alluvial deposits consisting of silty, sandy and clayey gravels and cobbles with less abundant bouldery gravels. Bedrock was not encountered during test pitting however based on outcrop distribution it is anticipated to be present from approximately 10 to 15m below ground level. The gravels, cobbles and boulders consist largely of unweathered, rounded to subangular, greywacke. Some angular greywacke gravel and mudstone cobbles are also present. The deposits are largely dry to moist however become moist to wet with depth. Visual assessment of the larger size fraction material suggests that it is likely to be suitable for crushing for manufacture of filter and drainage material. The volume of gravel deposits present within this terrace is estimated to be approximately 230,000m$^3$.

The terrace located further to the south was investigated with 5 test pits, 3 of which were extended to the limit of reach of the excavator (6.6m – 7.6m) and two to refusal on greywacke bedrock (6.7m & 5.7m). The deposits within this terrace are alluvial in origin, and consist of silty / clayey and sandy gravels and cobbles with some medium to large boulders. The coarse size fraction consists of largely unweathered, rounded to angular greywacke, with some weathered, angular mudstone gravel and cobbles. The finer size fraction material within the gravels consists of a silty and sandy matrix with some clay. Deposits become moist to wet after approximately 2.0m depth. Greywacke bedrock is present in the southern most test pits (TP22, TP23), and becomes shallower toward the south. An estimate of the volume of available materials in this terrace is given as 160,000m$^3$.

4.4.2.2 Trenches

A trench was excavated in the left abutment of the dam to determine the presence or absence of the ‘unnamed fault’. The trench was 5m wide, and included two 2m benches and was excavated to a depth of 2.0m. The trench was approximately 40m long and extended from RL120 to RL95. Surficial soils consist of silty clay with a trace of fine to medium sand. Weathered Tertiary Siltstone is present from approximately 1.0m below ground level and becomes a reasonably uniform unweathered grey siltstone from approximately 2.0m below ground level. The grey siltstone was consistent, both lithologically and structurally, along the length of the trench at that level and no evidence of the presence of a fault was observed. An additional trench was excavated on the hillside opposite this first trench due to an observed gravelly deposit which was discovered during mapping. The trench exposed a consolidated gravelly layer with silty sand and sandy clay matrix. The layer is interpreted to be an isolated alluvial deposit, however will be required to be removed during preparation of the foundation of the dam.

4.4.2.3 Boreholes

Three boreholes were drilled within or very close to the proposed dam foundation, to 25 to 30m depth using a Wayman Hydriil 1000 drill rig. Standard Penetration tests (SPT) were conducted within the upper 5m of each borehole to gain an understanding of the consistency of the subsurface soil and water pressure testing (Packer Tests) was undertaken at regular intervals throughout drilling to assess the rock mass permeability of the foundation bedrock. Surficial soils are considered to be firm to very stiff, low plasticity, silty clays. Up to 3m of weathered siltstone underlies surficial soils in BH1 & BH2, however the weathered profile of rock in BH3 extends to a greater depth (up to 11m), due to its location in a topographical low
in the valley bottom. The siltstone in all three boreholes which continues to depth (25-30m) is largely massive, unweathered, very weak to weak, grey siltstone. Discontinuities in the rock mass are typically moderately to very steeply inclined and are considered to be tight. Water pressure testing indicates that the rock formation is generally of low permeability (0-2 lugeons) especially at depth of >5m and therefore provides confirmation of the low permeability of the foundation rock. Water takes were only elevated near the top of borehole BH2 and the values indicate that the discontinuities within this section of rock may have been dilated as a result of water pressure during the test.

4.5 Reservoir and Damsite Geology

Investigations carried out by URS, along with information from Geotech Consulting Ltd (2006) indicate that the proposed reservoir and damsite areas encompass both Tertiary and Quaternary stratigraphic sequences.

4.5.1 Quaternary Stratigraphy

The Quaternary deposits which infill part of the catchment area are present upstream of the proposed embankment as a terrace approximately 150m wide and 200m in length. The test pits indicate that the terrace consists of a surficial silty soil deposit up to 1.5m thick, followed by a sequence of alluvial clayey silt and sandy silty gravels and cobbles interbedded with silty and gravelly clay.

Additional smaller terraces were observed along the right abutment. Up to 2.6m of clayey silt / silty clay is present along the eastern boundary of the terrace, this thins to 0.5m toward the western edge of the terrace. Sandy, silty sandy, and sandy clayey gravels and cobbles occur below loess deposits to depths of 4.8m to 6.4m.

The samples tested showed the gravels contain between 10% and 50% fines (combined silt and clay particles), and between 15% and 40 % sand. The silt/clay materials are of low activity and low to medium plasticity.

4.5.2 Tertiary Stratigraphy

Excavations into the right abutment of the proposed dam site indicate up to 1.5m of silty clay soils overlying Tertiary siltstone often referred to as “papa”. The clay soils are interpreted to be a weathered profile overlying the parent siltstone, though some colluvial material was observed and thin deposits are also possible. The left abutment exhibited about 1.0m of colluvium overlying weathered Tertiary siltstone. An additional test pit excavated within a saddle at a slightly lower elevation on the left abutment encountered a thicker colluvium / loess deposit (up to 3.0m) overlying residual Tertiary siltstone.

Test pits confirm the presence of weathered Tertiary siltstone in the central high spot in the dam alignment from between 1.0 and 2.0m below ground level. Surficial soils include silty clay / clayey silt with angular, fine gravel to cobbles.

Within the eastern stream channel a test pit was excavated into the valley floor. Residual siltstone at this location was very shallow (0.4m) and provided a unique colouration and odour (blue/grey and sulphurous odour). The soil was moist to wet and seepage was observed
entering the pit from 2m below ground level. The siltstone is extremely weak from 1.9 to at least 6.4m, due largely to the water saturation.

The two knolls located south and west of the main river terrace were investigated and encountered up to 2m of clayey silty loess followed by residual siltstone.

The weathered Tertiary material has been assessed and is classed as non-dispersive and of low to medium plasticity.

4.5.3 Greywacke

Greywacke was observed during geological mapping to the southwest of the proposed dam site. The greywacke can generally be described as competent fine sandstone interbedded with mudstone.

4.6 Foundation Conditions

Geological mapping and sub-surface investigations indicate that the dam site is underlain by Tertiary age siltstone or mudstone. This material could be described as a ‘weak’ rock (following the terms of the New Zealand Geotechnical Society guidelines) which probably has an unconfined compressive strength of between 5 and 20 MPa. This rock has poorly developed bedding and joints and no sheared surfaces were observed in outcrop or in the test pits or trench. The weak rock is covered by clay soils to a typical depth of 1 to 2 m, but up to 5 m of soft clay soil was present in the valley floor.

No evidence was observed of a fault crossing the dam foundation, although the foundation should be mapped during construction to confirm that no faults exist in the foundations.

The foundation material is expected to have a low permeability, which should act to limit the flow of seepage below the dam.

Based on our observations to date the site is suitable for construction of a zoned earth embankment dam. Our investigations indicate that the foundation would not support the higher bearing stresses imposed by a roller compacted concrete (RCC) dam.

4.7 Embankment Dam Material Suitability and Availability

Embankment dams are constructed from locally available soil materials placed into various “zones” within the dam according to the purpose they will serve in providing for the reservoir retention and dam stability. The available sources for these materials are described below.

4.7.1 Core material

The clay soils formed by weathering of the Tertiary rock appear suitable for use as a core material. The soils have a high fines content, are of low plasticity and have a moisture content at or below the plastic limit. Testing indicates that these soils are also non-dispersive i.e. not prone to internal erosion by water seepage. A large quantity of these soils has been identified within the reservoir footprint.

An estimated volume of these materials within the reservoir catchment is 270,000m³, which assumes 3m of surficial clay soils within the catchment including the area below current
terrace gravels south of the dam site. Excluding the area below the terrace would reduce the volume to approximately 180,000m$^3$. The alluvium with high fines content from the terrace within the catchment area may provide an additional source of core material, although it would probably require selective excavation.

4.7.2 Shoulder material

Shoulder material is required to provide adequate support for the core under all operating conditions. The alluvium underlyng the terrace is gravel dominated, but contains a high proportion of fines. This alluvium could be used as shoulder material provided adequate filter and drainage elements are included in the design along with appropriate slope angles. Greywacke exposed in the reservoir could be a source of good quality shoulder material, and greywacke derived alluvium exposed above reservoir level in the upper catchment could also be used. Bedrock greywacke would require quarrying and processing, but this operation should enable good quality material to be won, with an effectively unlimited available volume.

Shoulder materials should comprise strong free draining fill. Investigations indicate that there are sufficient quantities of suitable materials. However, all of the options for winning shoulder material would probably require some level of processing. The available materials include:

- The alluvial gravel in the reservoir catchment has a high proportion of fines and would require screening to provide a suitable free-draining material.
- Alluvial gravel near the eastern arm of the proposed reservoir appear to have a lower proportion of fines. They would possibly require screening, but should provide a higher yield than those in the rest of the catchment. The estimated volume of these deposits may be up to 380,000m$^3$ fill part of the catchment area.

Quarried greywacke rock could provide a large volume of material of good quality but would be expensive to win.

4.7.3 Filter and drainage material

Filter and drainage material will need to be manufactured to achieve the gradings that are required for those functions. Quarried greywacke rock or greywacke gravel are expected to be suitable input materials for the processing into a wide range of gradations.

The required volume of combined filter and drainage blanket material is 28,300m$^3$.

Investigations indicate that suitable materials could be manufactured from greywacke gravels, cobbles and boulders from the upper-valley alluvium. The volume of suitable materials available from this source should exceed the required volumes for construction. Any remaining material could be incorporated into the Zone 3 shoulder.

4.7.4 Rip-rap wave protection

Rip-rap for wave protection on the upstream shoulder of the dam will comprise greywacke boulders of varying size. An evaluation of the required size range based on the expected wave heights will be required.

Greywacke may be won from exposures within the reservoir via rock ripping or drilling and blasting. Given that some very large greywacke boulders were present in alluvial deposits
located south of the dam site beyond the eastern extent of the proposed reservoir, these could provide a small additional source rock for use as rip rap.

4.8 Geotechnical Assessment of Pipeline Route

In mid-March 2008 an engineering geologist conducted a walk over survey of the pipeline route to identify predominant soil and/or rock cover along the proposed route. Outcrops were observed in road and farm track cuttings. At this time it is understood that a trench with approximate dimensions of 1.2m width and 2m depth will accommodate the pipeline.

The route can be divided into two broad material types. The region is characterised by siltstones of Tertiary age and younger fluvial and glacial derived gravels. The majority of the pipeline will be trenched into gravels, with siltstone mostly encountered across the Spence Farm and Flaxbourne Station.

The siltstone is generally, although not always, capped by hard, dry, silty clay (loess) up to 1m thick. The siltstone outcrops observed were generally extremely weathered and had engineering properties of dry, hard clay. In some locations the siltstone was more competent with obvious structure remaining although being discoloured and highly weathered. It should be noted that as these observations are not based on test pitting. It is currently unknown how quickly the siltstone will become competent with depth.

The gravels observed were variable in size and distribution, which is expected as different areas relate to different rivers and glacial outwash periods. In general all the gravels were well rounded and particle size varied from fine gravel up to cobble sized particles. Silt lenses and layers were often interbedded within the gravels. The gravels were generally clast supported (i.e. not supported by a matrix of silt or sand), densely packed and the clasts were typically slightly weathered to fresh.

On the Marfell’s Beach Road alignment the pipeline is mostly within glacial outwash gravels and silts, although sections of siltstone are likely to be encountered as observed in drainage paths just above sea level.

Geotechnical issues affecting the pipeline

It is assumed that groundwater seepage will occur in the trench in the vicinity of Lake Grassmere where the land is generally at or slightly above water level.

The pipeline will traverse several creeks and drainage paths, the most significant being a deeply incised creek (approx 3 m deep) between Reservoir Road and Cable Station Road. Numerous culverts are present under the roads along the route, and it is assumed these areas will encounter groundwater seepage.

Another area where groundwater seepage may occur was identified in the lower (southern) reaches of the Spence’s farm, approximately 400m from the farm house.

The excavation of the trench will encounter hard dry silty clay, densely packed gravels and potentially weathered siltstone. Excavation is likely to be slow and will require a suitably sized excavator.
As the materials are expected to be relatively fine grained they will be suitable for backfilling around the pipe. In areas where the gravels are too coarse to use as backfill, finer material should be able to be borrowed from a local source.
5 PLANNING COMPLIANCE SUMMARY

5.1 Introduction

Since 2008 the Marlborough District Council has finalised the Wairau/Awatere Resource Management Plan (WARMP), with only minor changes to the parts of the Proposed WARMP relevant to the proposed scheme. Resource consents have been obtained for parts of the scheme and some of these, in relation to the abstraction of water and the physical works to extract water, have lapsed. The relevant planning requirements and remaining consent authorisations are discussed below.

5.2 Summary of existing resource consents

The resource consents currently held in relation the proposed scheme are summarised in Table 5-1 below.

<table>
<thead>
<tr>
<th>Resource consent number</th>
<th>Activity</th>
<th>Status of resource consent</th>
</tr>
</thead>
<tbody>
<tr>
<td>U080564</td>
<td>To construct a 38m high dam, to dam up to 2,800,000m³ of water and excavate approximately 2,000m³ of soil and rock.</td>
<td>Active. Consent will lapse July 2015.</td>
</tr>
<tr>
<td>U041096</td>
<td>To abstract C Class water from the Awatere River (max rate 556 L/s and 4,022,938m³/year) for storage.</td>
<td>Active. Consent lapses 31 March 2017.</td>
</tr>
<tr>
<td></td>
<td>To abstract B Class water from the Awatere River (max rate 200 L/s) for irrigation use.</td>
<td>Not active. Consent has lapsed</td>
</tr>
<tr>
<td></td>
<td>To construct a gallery and intake structure in the riverbed of the Awatere River.</td>
<td>Not active. The portion of the consent to divert water and construct a gallery and intake structure expired 19 January 2012.</td>
</tr>
</tbody>
</table>
5.3 Summary of Resource Consents Required

A summary of the resource consents that are likely to be required for the proposed scheme are provided in Table 5-1 below.

Table 5-2 Summary of Resource Consents required under the Wairau/Awatere Resource Management Plan

<table>
<thead>
<tr>
<th>Consent Type</th>
<th>Activity requiring resource consent</th>
<th>Relevant Rule / Standard</th>
<th>Activity Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pump Station</td>
<td>Land Use</td>
<td>Chapter 30 Clause 1.7.3.1 and 30.4.2</td>
<td>Discretionary</td>
</tr>
<tr>
<td></td>
<td>Land Use</td>
<td>Chapter 27 5.1.1.1</td>
<td>Permitted</td>
</tr>
<tr>
<td>Pipeline</td>
<td>Land Use</td>
<td>Chapter 27, Clause 1.8.2</td>
<td>Discretionary</td>
</tr>
<tr>
<td></td>
<td>Water Permit</td>
<td>Chapter 27, Clause 1.7.2</td>
<td>Discretionary</td>
</tr>
<tr>
<td></td>
<td>Land Use</td>
<td>Chapter 30, Clause 1.7.3.1, 1.7.3.2 and 30.4.2</td>
<td>Discretionary</td>
</tr>
<tr>
<td></td>
<td>Discharge Permit</td>
<td>Chapter 27, Clause 1.10.2</td>
<td>Discretionary</td>
</tr>
<tr>
<td>Dam and</td>
<td>Land Use</td>
<td>Chapter 27, Clause 1.6.2</td>
<td>Discretionary</td>
</tr>
<tr>
<td>Reservoir</td>
<td>Water Permit</td>
<td>Chapter 27, Clause 1.2.3 or 1.2.4</td>
<td>Discretionary or Non-complying</td>
</tr>
<tr>
<td>Quarry</td>
<td>Land Use</td>
<td>Chapter 30, Clause 30.4.1 or 30.5.1</td>
<td>Discretionary or non-complying</td>
</tr>
</tbody>
</table>

The pump station(s) and pipeline will generally be permitted activities, subject to standards, except where noted above in table 5.1. The identification of the above resources consents has been confirmed with Guy Boddington, Marlborough District Council, in 2008.

Note that whilst a number of applications are required, we envisage that they would be dealt within a single Assessment of Effects on the Environment for lodgement with Marlborough District Council. In addition, a number of the resource consents may not need to be exercised depending on conditions encountered at the time of the works; for example, diversions at river crossings will not be required if there is no surface flow. However, we consider all the consents listed should be sought as a conservative measure.

5.4 General Overview

The proposed Scheme falls within the Marlborough District Council (MDC) area and is subject to the Wairau/Awatere Resource Management Plan (WARMP). This document contains the objectives, policies and methods for both District and Regional planning functions and duties under the Resource Management Act 1991 (RMA).
Chapter 27.5 Clause 1 sets out the rules relating to utilities and designations. Clause 5.1.1.1 specifies the utilities that are permitted throughout the District, subject to various standards found within that chapter and other chapters of the Plan. As relevant to the Scheme, the list of permitted utilities includes:

- Underground pipe networks for the conveyance of water, and any ancillary equipment
- Utility buildings and buildings ancillary to utilities
- Reservoirs and supply intakes for the reticulation or provision of public water supply.
- Community irrigation and stock water races and public drainage channels

Therefore the proposed pipeline is generally considered a permitted activity, except in certain instances where various standards may not be met (further discussed in Section 5.4). Pump stations, being a utility building, will also be permitted subject to standards (refer Section 5.3). The proposed reservoir is not a permitted activity as it will not be for the provision of public water supply (refer Section 5.5).

Other Chapters of relevance to the Scheme include:

- Chapter 127 General Rules Having Application in All Zones, Rivers, Riverbeds and Lakes – Various rules and standards in this chapter apply, relating to damming, river crossings, and discharges.
- Chapter 30 Rural 3 & 4 Zones - The entire Scheme falls within the Rural 4 zone and various standards apply.

The various rules and standards of the WARMP, as it relates to the conceptual design, are explored in the following sections. For convenience, the sections have been split into the Pump Station, Pipeline and Dam/Reservoir. Not considered in the assessment is the existing resource consent, U041096, which covers the water take, use of that water for irrigation and storage, and activities in the bed of the Awatere River.

### 5.5 Infiltration Gallery

Excavating in the bed of the Awatere River is unlikely to be a significant consenting hurdle. Infiltration gallery systems are numerous in the valley, with significant works in the river bed (including the actively flowing channel) common. However, any application to undertake such works will require an assessment of the effects of the works on water quality and downstream abstractions. Consideration may also be required for works to occur outside fish spawning periods (if applicable in this river system).

Works are likely to be required to divert the main channel during the installation process and this is likely to require an assessment of the effects of the diversion on river bank erosion (i.e. ensuring that the diversion does not cause erosion to the banks of the river) and downstream property (including intakes). This effect is most likely to be addressed through the provision of a management plan.

---

1 Note that, in contrast to most other District Plans, the term “Utility” is not defined in the WARMP. In this case the list of permitted utilities under Clause 5.1 in Chapter 2.5 provides a de facto list of what is considered a utility.
It is recommended that a consenting strategy is formulated early on, with discussions with the MDC and neighbouring parties to identify any significant constraints on the consenting and construction programme. It is possible that some additional investigations could be required to satisfy consenting requirements. These investigations are likely to involve the installation of a test bore in the bank of the river and monitoring wells to confirm hydraulic conductivity of the material. However, a sensitivity analysis of the K values used in the modelling may avoid the necessity of this testing ahead of the consenting.

Note: the previous intake gallery was consented but has lapsed. The consent was processed on a non-notified basis. As the proposed take has increased in size it is expected that limited notification would be required.

5.6 Pump Station

The pump station will be a ‘building’ for the purposes of the WARMP. To be a permitted activity the following conditions apply:

- Maximum gross floor area of 65m² and maximum height of 5 m (Chapter 2.5, Clause 5.1.2.6)
- If the building has a ground floor area greater than 15m² and/or a height over 2 m, the building needs to be set back from any road boundary by a distance not less than half the height of the structure (Chapter 2.5, Clause 5.1.2.8)
- Sites containing a building with a ground floor area greater than 15m² and/or a height over 2 m require a landscaped area of 2 m on the road boundary (Chapter 2.5, Clause 5.1.2.11)
- Noise shall not exceed the following parameters when measures at the boundary of any land zoned Residential or Rural Residential or within the notational boundary of any dwelling (Chapter 30, clause 1.4.1):
  - 55 dB A L10 0700 to 2200 hrs Monday to Saturday
  - 45 dB A L10 0800 hrs to 1900 hrs Sunday
  - 75 dB A Lmax At all other times

It is likely the above standards can be met subject to design.

Electricity supply to the pump station is a permitted activity provided the line and any transformer does not exceed a voltage of 110 KV and a capacity of up to 100 MVA per circuit. The proposed line will be 11kV and approximately 1 MVA and will therefore not require a consent.

The land use consent for the infiltration gallery installation (U041096) only applies to the bed of the river. Any excavation between the bed and the pump station is subject to Clause 1.7.3.1 in Chapter 30, which states that no excavation shall occur within 8 metres of any permanently flowing river. Resource consent for a discretionary activity under Clause 30.4.2 is likely to be required on this basis.
5.7 Pipelines

As noted in Section 5.2 above, pipelines for conveying water are generally considered to be a permitted activity, subject to compliance with various rules and standards. Relevant in this case are:

- No excavation within 8.0 m of permanently flowing rivers or wetlands (Chapter 30, Clause 1.7.3.1)
- On land greater than 20 degrees where excavation exceeds 1000 m³ in any two year period (Chapter 30, Clause 1.7.3.3)
- Temporary diversion of water in watercourses during trenching activities at river crossings (Chapter 27, Clause 1.7.1)
- Discharges of sediment to water from excavation works in the river bed (Chapter 27, Clause 1.10.1.3)
- Structures to be in riverbeds less than 3 m width (Chapter 27, Clause 1.8.2)

Resource consents may be required with respect to the above, except that there are no slopes over 20 degrees on the pipeline route.

All roads, state highways, and the rail corridor within the project area are designated under Part 8 of the RMA. The permission of the relevant requiring authorities – being Marlborough District Council, Transit New Zealand, and New Zealand Railways Corporation – is required wherever the pipeline is within or intersects the designations. Even with the permission of the requiring authorities, Ward Irrigation does not acquire any rights afforded to the designations and is still subject to the rules and standards of the underlying zoning.

5.8 Reservoir and Dam

Reservoirs for the provision of public water supply are a permitted utility in terms of Rule 5.1.1.1 in Chapter 27. However, if the proposed dam is considered a private scheme consent will be required as a discretionary activity under Rule 5.1.3.

Construction of a dam and the associated damming of water is a permitted activity under Rule 1.6.1 of Chapter 27, subject to conditions. The proposed dam will not meet a number of the conditions, principally in relation to the impoundment volume (20,000 m³ permitted; 2,000,000 – 2,800,000 m³ proposed) and dam height (4 m permitted; 35 m proposed). Resource consent is required as a discretionary activity under Clause 1.6.2. Being a discretionary activity, the assessment matters which can be considered by MDC are essentially unrestricted, although the WARMP does give some guidance as to what may be considered, such as effects on ecological, amenity, and recreational values. At the proposed site any effects on such values from construction are likely to be minor or negligible and should not present any significant issues to the granting of resource consent.

Establishment of a temporary quarry at the dam site is subject to the land disturbance provisions of the Rural 4 zone (30.1.6). It is likely that these provisions can be complied with, but in any event the effects of the quarry would be considered alongside the discretionary activity resource consent for the dam construction.
The damming of the existing watercourse amounts to a full take of that water, albeit that in this case the volumes are very small and at times non-existent. The damming/taking of water requires resource consent as a discretionary activity (27.1.2.3) or a non-complying activity (27.1.2.4) depending on the volume of take required. Note that MDC has verbally advised us that there is a policy (apparently unwritten) of notifying all water take applications. In this instance we consider there are good grounds to argue for non-notification, however, at this stage there remains a risk that MDC may still choose to use the public or limited notification processes with respect to the water damming/take.

Emergency discharge bywash from the dam will be a permitted activity under Chapter 127 Clause 1.10.1.1, on the basis that the receiving waters are Class F and subject to compliance with the following standards:

- The discharge contains less than 0.3 g/m$^3$ free chlorine;
- The discharge contains less than 50 g/m$^3$ suspended solids;
- No erosion is caused at or downstream of the discharge point;
- The discharge shall not alter the natural course of its receiving river or stream;
- The discharge point shall be maintained in a condition such that it is clear of debris and structurally sound;
- The water clarity standards will be met after reasonable mixing; and
- The discharge will not cause flooding on private land.

The WARMP does not specifically identify potential downstream impacts in the event of dam failure, but these would undoubtedly be considered during the resource consent process. Construction of the dam to internationally accepted safety standards, coupled with obtaining the written approval of downstream landowners and occupiers, should negate a large number of potential consenting issues.

The dam will be classified as a large dam under the Building Act 2004 and as such will require a Building Consent prior to construction commencing. This is a separate consenting process which is described under Section 8.

5.9 Fill Source

Should gravel from a river bed source be required for dam and reservoir construction, resource consent will be required as a discretionary activity. Impacts to be considered in any application will include effects on river hydraulics, flooding and erosion, ecology, natural character and recreational values.

Gravel sourced from private land for the dam and reservoir construction would require resource consent as a discretionary activity under Clause 30.4.1, unless the quarry is on land above 1000m, in which case the activity would be non-complying under Clause 30.5.1.

---

2 Class F – managed for fishery purposes, the primary objective being water quality of a standard where fish can be safely consumed
Earthworks associated with a quarry would be a permitted activity where all the conditions of Chapter 30, Clause 1.7.3 are met.

5.10 **Notification vs Non-notification**

There are reasonable grounds to argue that the adverse effects of the proposed scheme are minor and that public notification is not required. To ensure the greatest likelihood of MDC adopting the non-notification route, the written approval of the following parties should be obtained prior to lodgement of the applications:

- All private landowners and occupiers directly affected by the pipeline route and pump stations
- All landowners and occupiers downstream of the reservoir and dam ('downstream' includes those potentially affected in a dam break scenario)
- Department of Conservation (both as potentially affected landowner and as conservation advocate)
- Fish and Game New Zealand

Consultation with Tangata Whenua is also advised to ensure an efficient consenting process.

5.11 **Consent Processing Timeframes**

Once the resource consent applications have been lodged, and should they follow the non-notified process, up to three months should be allowed for before a decision is released by MDC. This allows for the standard 20-working day timeframe within the RMA as well as any delays arising out of further negotiations with Council and affected parties (for example, in relation to mitigation) and responding to any minor further information requests. Once a non-notified decision is released, works could effectively commence.

Should the resource consents follow the limited or full notification processes, up to six months should be allowed for before a decision is released by MDC. This allows for the standard timeframes in the RMA as well as any delays as described above. Assuming the consents are granted, and provided no appeals to the Environment Court are made by Ward Irrigation or any submitter within the 15 working day period following the decision, works could commence at that time.
6 RIVER INTAKE

6.1 Introduction

The preliminary river intake design undertaken for Ward Water Company (Waterco) by URS in 2008, included an infiltration gallery as the preferred method of abstraction with the gallery for an abstraction 200 L/s. Preliminary assessment for the FCIL scheme assumes the gallery would be constructed in the same location, see Figure 6-1 and would require the infiltration gallery system to be increased to a proposed abstraction rate to 450 L/s.

Figure 6-1 Intake Layout

6.2 Background

In March 2008 URS produced a draft report for Waterco which included a preliminary design for the river intake. The preliminary design for the intake concluded that an infiltration gallery which consisted of multiple perforated pipes lain within a graded filter pack was the preferred option. This option was selected to deliver a maximum flow rate of 200 L/s as consented under UO41096. Initial assessment for Flaxbourne Irrigation Scheme indicates the design would need to be updated it for the purpose of abstracting up to 450 L/s from the Awatere River.

The preliminary design recommended the installation of an infiltration gallery beneath the bed of the Awatere River (i.e. a bed mounted intake system). The design acknowledged the potential issues associated with abstractions from the Awatere, particularly the high suspended sediment load associated with the highly erodible tertiary deposits of Papa (i.e. mudstone).

The preliminary design referred to discussions held with the Marlborough District Council (MDC) regarding performance issues with gallery systems installed in the river valley, noting that the fines content in the river has caused clogging and reduction in yields. Discussions were also held with John Butt of Butt Drilling (who had installed a number of gallery systems in
the Awatere catchment) on his experiences with installation of gallery systems in the Awatere River valley. A broad review of 36 consented abstractions from the Awatere River was also undertaken. The review identified the intake type (and size if available) and commented on any maintenance issues. Infiltration trenches (i.e. gallery) were most common, with trenches of between 60-70m in length. The comments indicated that these systems did not require any maintenance. However, the review did not make any comment on reduction in yield or reliability associated with these systems.

Inspection of the upstream Yealands Group intake and a limited intrusive investigation was also undertaken. Six test pits were excavated to a depth of 2.5 m to characterise the sub-surface lithology. Material samples were collected and sent to materials laboratory for particle size distribution analysis (PSD). Based on the PSD results (using the Kozeny-Carmen empirical equation) an average permeability of approximately 75 m/day was derived.

The infiltration gallery was designed to meet the hydraulic design parameters detailed in Driscoll (1986). Specifically:

- The pipe is installed in trenches 4m below the river surface (i.e. 3 m below bed level)
- The river is approximately 1m deep on average
- 300 mm diameter PVC pipe (with 4 pipe branches installed under the river, each designed to deliver an equal proportion of the total yield) designed to maintain an axial velocity less than the design criteria of 0.9 m/s
- The filter pack around the perforated pipe comprises medium to coarse gravel with a d30 of 24 mm and a coefficient of variance of < 2.5
- An impermeable membrane is placed above the screen (approx. 300mm) in the filter pack to promote horizontal flow and avoid short circuiting and sedimentation of the filter pack
- The slot size is between 5-10 mm to give an open area of approximately 11%
- The active flow channel width is approximately 30 m
- The infiltration pipe directly under the main channel delivers the main proportion of the yield

Based on the inferred hydraulic characteristics a yield of 50 L/s per lateral for the 30 m under the active channel was calculated. However, the design included a total length of pipe of 150 m per lateral (to extend from bank to bank), with a connection pipe and sump which connects to a further pipe of 150 m taking water from the infiltration pipes to the pump house.

It was recommended that further investigations should be undertaken during the detailed design phase to confirm the hydraulic parameters adopted in the preliminary design.
6.3 Review of Preliminary Design

The preliminary design has been reviewed against the design guidelines produced by Aqualinc Research Limited for the Marlborough District Council. The design has also been updated to reflect the proposed increase in the abstraction rate (i.e. 450 L/s).

The review of the design is in two parts: the first addresses the selection of the river bed infiltration gallery system given the proposed location and environmental setting; and the second addresses the hydraulic design parameters of the proposed system.

6.3.1 River Bed Infiltration Gallery System

The selection of the river bed infiltration gallery system was based on designing a system that would meet the required yield and water quality objectives (i.e. low fines content). These two factors lead the preliminary design to the recommendation for a multi-lateral river bed infiltration gallery system.

During the 2008 investigations URS discussed performance issues with MDC, Butt Drilling, and inspected the Yealands Group intake located immediately upstream of the proposed intake location. The report identified that constructing canals from the main stem of the river did not address the sedimentation issue, with the Montana intake noted as experiencing significant issues with high sediment load entering the intake pond.

Achieving the required yield of 200 L/s effectively eliminated the use of vertical well systems due to the limited saturated thickness of the aquifer and the likelihood of low permeability sediments located away from the Awatere River bed (Davidson and Wilson, 2011). Direct river intakes were excluded based on the requirement for high water quality (i.e. low TSS) to avoid clogging and maintenance issues associated with irrigation drippers and spray nozzles. The proposed increase in the abstraction rate also reduces the number of options available, with the river bed infiltration gallery, offline infiltration gallery (i.e. bank gallery), and direct river take (managed for suspended sediment) more likely to provide the required yield.

The preference for a river bed system over an embankment system was not discussed in the Report. However, it is likely to be associated with the available land area and the potential performance issues associated with the intake pipework located away from the main channel. It was noted that the Yealands intake was operating successfully with a 300 m lateral running under the river, yielding approximately 80 L/s, with a further 80 L/s derived from through flow in the upstream gravel bund, whilst infiltration through the embankment materials to the Yealands intake canal was considered to be relatively low.

The selection of the river bed infiltration gallery system is consistent with the selection criteria and approach outlined in Aqualinc (2014a). Table 1 outlines critical issues to address when selecting an intake method. Based on the assessment undertaken in the preliminary design it is likely that the river bed infiltration gallery system is the best suited for the environment. However, the report also correctly acknowledges that there are residual risks associated with any system adopted.

### Gallery Intake Critical Issues

<table>
<thead>
<tr>
<th>Question</th>
<th>Assessed</th>
<th>Not Assessed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Can the infiltration method achieve the required yield?</td>
<td>Yes, based on the preliminary site investigations.</td>
<td></td>
</tr>
<tr>
<td>Is there a viable alternative?</td>
<td>No viable alternatives that provide required yield and water quality.</td>
<td></td>
</tr>
<tr>
<td>Will the installation have a detrimental impact on the environment?</td>
<td></td>
<td>Not assessed but remains a critical issue to address. Will be clarified during the consenting process.</td>
</tr>
<tr>
<td>Does water quality prevent the use of an infiltration gallery?</td>
<td>Water quality is poor under high flows, so river bed gallery selected with a vertical flow barrier to prevent sedimentation.</td>
<td></td>
</tr>
<tr>
<td>Do geotechnical problems prevent the use of an infiltration gallery?</td>
<td></td>
<td>Not assessed, but method is likely to address key geotechnical constraints.</td>
</tr>
<tr>
<td>What would the consequences of a flood event be in the installation?</td>
<td></td>
<td>Not assessed specifically. However, installation 3m below bed level addresses scour, and collector pipework on stable bank largely addresses washout.</td>
</tr>
</tbody>
</table>

### 6.4 Hydraulic design Parameters

Note: the updated design is still considered to be preliminary, with further in-situ testing and hydraulic design to be undertaken during subsequent phases of work.

The following section outlines the hydraulic design parameters adopted in the preliminary design. Table 2 summarises the calculated hydraulic parameters adopted compared to the design requirements detailed in Aqualinc (2014b). The design has also been updated to reflect the increase in the proposed abstraction rate.

Pipe sizing is indicative only. The detailed design will need to consider the appropriate pipe configuration (i.e. larger pipes with fewer laterals vs smaller pipes with multiple laterals). In addition, the safety factor applied to the design should consider the length (i.e. total bed width or half bed width) and number of pipes used in the infiltration gallery.
Table 6-2  Hydraulic design Parameters

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Preliminary Design (200 L/s)</th>
<th>Updated Design (450 L/s)</th>
<th>Aqualinc (2014b)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe Diameter (mm)</td>
<td>300</td>
<td>450</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>% Open Area</td>
<td>13%</td>
<td>10%</td>
<td>&lt;18-20%</td>
<td>met</td>
</tr>
<tr>
<td>Entrance velocity (m/s)</td>
<td>0.015</td>
<td>0.015</td>
<td>&lt;0.03</td>
<td>met</td>
</tr>
<tr>
<td>Axial velocity (m/s)</td>
<td>0.006 (per lateral is 0.71m/s)</td>
<td>0.35</td>
<td>&lt;0.9</td>
<td>met</td>
</tr>
<tr>
<td>Depth of pipe</td>
<td>3 m bgl</td>
<td>3 m bgl</td>
<td>&gt;1.5 m bgl</td>
<td>met</td>
</tr>
<tr>
<td>Screen length per lateral (effective)</td>
<td>~30m / lateral</td>
<td>~30m / lateral</td>
<td>Based on Driscoll</td>
<td></td>
</tr>
<tr>
<td>Required screen length</td>
<td>120 m</td>
<td>220 m</td>
<td>Based on Driscoll</td>
<td></td>
</tr>
<tr>
<td># of laterals</td>
<td>4</td>
<td>8</td>
<td>Based on Driscoll</td>
<td></td>
</tr>
<tr>
<td>Spacing between laterals (m)</td>
<td>3</td>
<td>3</td>
<td>Based on Driscoll</td>
<td></td>
</tr>
<tr>
<td>Infiltration area (effective)</td>
<td></td>
<td>750 m²</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Increasing the lateral length to ~80m provides for additional open area to address channel migration (to half of the bed) and clogging of the river bed material and filter pack. The intake design is based on an average horizontal K value of 76 m/d (0.0009 m/s). The K was based on an analysis of the grain size distribution from the preliminary site investigations. Further work should be undertaken as part of the detailed design phase to confirm the hydraulic conductivity of the aquifer (through pump test).

The entrance velocity has been set at 0.015 m/s to be half of the upper limit of 0.03 m/s as set out in Driscoll (1986). The velocity influences the calculated open area through the back calculation of the yield and pipe length. The slot width, spacing, and number of slots per meter has been calculated to maintain the entrance velocity. These parameters need to be confirmed with the provider of the slotted pipe during detailed design.

The axial velocity for each lateral (i.e. 30m length) has been calculated. This indicates that the velocity is below the threshold defined in Driscoll (1986).

6.5 Discussion

The preliminary design was progressed on a conceptual understanding of the Awatere River catchment and the success (or otherwise) of existing infiltration galleries located in the catchment. The preference of a river bed infiltration gallery over other methods appears to be well supported given the water quality issues in the Awatere River.

The extent of the infiltration gallery pipework requires further consideration. The 2008 assessment proposed four laterals extending the full width of the river bed (i.e. 150m per...
lateral). However, the increased rate of take has resulted in an increase in the pipe diameter and the number of laterals to meet the hydraulic design requirements. This will affect the cost and risk associated with maintaining the system. It has been assumed some in-stream river works to maintain a flow channel to the true right bank of the river in the area of the proposed intake will be required periodically. This reduces the requirement to install each lateral across the full width of the bed. Generally, intake pipes of between 60-70m are used in the valley. For this phase of the works, costings have been undertaken using a safety factor of 2.5 applied to the length of pipe (i.e. pipe length of 80 m). The hydraulic design has also included a safety factor of 2 for the through screen velocity (i.e. set at 0.015 m/s c.f. an upper design limit of 0.03 m/s).

There are a risk associated with river bank erosion and movement of the main channel to expose pipework and collection sumps. The detailed design should consider the potential for bank stabilisation works to be undertaken in this area.

The 2008 work did not quantify the potential improvements in water quality associated with a river bed infiltration gallery. However, the design did include an impermeable liner to be placed above the gallery pipework to restrict/prevent vertical infiltration and promote horizontal movement of water into the intake. This was proposed following discussions with John Butt, who is experienced in installing infiltration galleries in the Awatere River catchment.

Aqualinc (2014b) note that galleries in the Marlborough District experience a reduction in yield due to falling water levels or because of clogging. Aqualinc (2014b) also notes that many of the galleries installed in the Awatere River system are designed to intercept horizontal flow by using polyvinyl sheeting to prevent vertical flow (recharge) to overcome the clogging of the infiltration pack.

The effectiveness of the impermeable liner to prevent silt ingress to the infiltration chamber is uncertain. In the short term it is likely to provide some benefit to a newly installed gallery system. However, the grading of the gavel pack around the gallery pipework is considered to be more critical in preventing sediment movement into the infiltration chamber. Further investigations into pipe installation and impermeable liner should be undertaken during the detailed design phase.

Alternatives such as diversion channels and the use of sedimentation ponds to lower the suspended sediment concentration in the water before passing it over a gallery intake are considered impracticable given the limited area for construction. Furthermore, the suspended sediment is comprised of mudstone (clay) which is unlikely to settle out without additives (i.e. flocculants). Given the proposed rate of abstraction, the use of flocculants is considered to be impracticable (technically and costs).

### 6.6 Additional Investigation at Detailed Design Stage

In order to optimise the final design of the proposed infiltration gallery, such that the design yield can be achieved at minimum cost and with minimum risk from sedimentation problems, some further investigation is proposed to be carried out at the site for detailed design. This will broadly comprise:

- Installation of a pumping well.
- Installation of a number of monitoring points.
- Completion of a short pumping test.
- Analysis of the data to define hydraulic parameters.
- Optimisation of the preliminary design.
7 TRANSFER PUMP STATIONS AND RISING MAIN

7.1 Rising Main

7.1.1 Proposed Alignment

Several options for the alignment of the rising main were investigated for the Waterco scheme including a beach alignment and crossing the saltworks ponds before selecting the proposed alignment which follows Reserve Road, Blind River Loop Road, Kaparu Road S.H.1, and Marlells Beach Road as shown in Appendix C. This alignment has been retained. Pipes will be buried at shallow depth (typical 900mm cover), deeper for stream and road crossings as required.

7.1.2 Materials

Two material options have been considered based on the required pressure ratings, glass reinforced plastic (GRP) and steel. GRP pipes are corrosion resistant and have significantly lower supply cost rate than steel, therefore, it is the recommended material to use. Steel pipes are not corrosion resistant and would require additional corrosion protection if it is the preferred material to be used. Protection will be in the form of coating or wraps including special low abrasion backfill. Bulk corrosion protection of the transfer pipeline has not been considered at this time.

Piping within the pump stations will be galvanised mild steel for ease of fabrication and fitting of headers and associated valves, flow meters and pressure gauges.

7.1.3 Pipe Size Selection

Pipe diameters have been selected for the scheme based on the maximum flow for each section when utilising water from the Awatere River, or the dam. A nominal pipe diameter of 600 mm was selected for the 18.3 km section between Awatere River and the distribution system to transfer a maximum flow of 450 L/s. For the 5 km long pipeline between the distribution system and the storage dam, a nominal diameter of 750 mm was chosen to transfer a maximum flow of 675 L/s.

The pressure rating for the pipes was calculated based on the static pressures that were determined from the output pressure of the pump stations.

7.2 Pump Stations

7.2.1 Pump from Awatere River to Dam

Pumping is required to transfer the flow from Awatere River to the storage dam. Two options were analysed:

Option 1: Awatere River Pump Station

Option 2: Awatere River and Booster Pump Station
7.2.1.1 **Option 1: Awatere River Pump Station**

The proposed Option 1 is to size the pumpstation at Awatere River so that it has enough pressure to pump a maximum flow of 450 L/s directly to the dam. The duty head for this pump station is 150 m which will require approximately 960 kW power to operate. As a result of the considerably large pressure head, the pressure rating of the transfer pipe line will be relatively high (up to PN20). Table 7-1 below shows a summary of the pipe quantities and pressure rating for Option 1.

**Table 7-1** Option 1 Pipe Schedule

<table>
<thead>
<tr>
<th>Pressure Rating</th>
<th>150 mm Dia.</th>
<th>600 mm Dia.</th>
<th>750 mm Dia.</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>PN6</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>PN9</td>
<td>-</td>
<td>200</td>
<td>741</td>
<td>941</td>
</tr>
<tr>
<td>PN12</td>
<td>-</td>
<td>2200</td>
<td>100</td>
<td>2300</td>
</tr>
<tr>
<td>PN16</td>
<td>-</td>
<td>5000</td>
<td>300</td>
<td>5300</td>
</tr>
<tr>
<td>PN20</td>
<td>353</td>
<td>10883</td>
<td>3900</td>
<td>15136</td>
</tr>
</tbody>
</table>

Figure 7-1 below shows a long-section of the transfer pipeline with the scheme hydraulic grade line for Option 1. As can be seen there is a high point towards the dam where the hydraulic grade line falls below the ground level elevation of approximately RL 120m however it is expected that a minor change in alignment to the north will avoid this feature and the associated undesirable negative pressures in the pipeline.
7.2.1.2 Option 2: Awatere River and Booster Pump Station

Option 2 proposes a smaller pump station at Awatere River Station to provide enough pressure head to pass over the two high points between the river and the distribution system. A booster pump station is then required to deliver the flow over the high point just before the dam. The two pump stations proposed require the same pump duty head of 80 m that has an operating power of approximately 512 kW each. The advantage of this option is lower pipe pressure rating is required for the pipelines which will save capital costs in the pipeline construction. The maximum rating of PN12 is required for the section between river and the distribution system and PN16 for the 5 km long section between the distribution system and dam. Table 7-2 below shows a summary of the pipe quantities and pressure rating for Option 2.

Table 7-2 Option 2 Pipe Schedule

<table>
<thead>
<tr>
<th>Pressure Rating</th>
<th>150 mm Dia.</th>
<th>600 mm Dia.</th>
<th>750 mm Dia.</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>PN6</td>
<td>-</td>
<td>4700</td>
<td>741</td>
<td>541</td>
</tr>
<tr>
<td>PN9</td>
<td>-</td>
<td>2700</td>
<td>100</td>
<td>2800</td>
</tr>
<tr>
<td>PN12</td>
<td>353</td>
<td>10683</td>
<td>3700</td>
<td>14936</td>
</tr>
<tr>
<td>PN16</td>
<td>-</td>
<td>-</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>PN20</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The booster pump station is located on a minor road approximately 400 m south of Marfells Beach Road. The pump station location ensures accessibility for future operation and maintenance. Figure 7-2 below shows the location of the booster pump station.

Figure 7-2 Pump Station Location

Figure 7-3 below shows a long-section of transfer pipeline with the scheme hydraulic grade line of Option 2.
7.2.1.3 Recommended Option – Option 1 Awatere River Pump Station

Due to the availability of the 11kV power at the Awatere river site it is recommended that a single high pressure pump station be constructed at the site. Although Option 2 incurs cost savings for the pipeline construction due to the lower pressure ratings, this saving will be minor compared to the higher capital and operational and maintenance costs incurred for the extra booster pump station required.

7.2.2 Pump from Dam to Distribution System

The proposed dam has a bottom outlet water level of RL100m, and a top water level of 120m. The proposed pump will supply a flow of 675 L/s at a pressure head of 30 m to overcome the high point in the pipeline. The highpoint is approximately 650 m away from the dam at an elevation of approximately RL 120m.

Electrical Power and Control Requirements

High voltage power supply is available at the booster station location, however the existing 11kV lines from State Highway One are the property of Dominion Salt Works. Permission will be required from them and an agreement formed for the purchase of power or an alternative supply line constructed. We have assumed that an agreement can be negotiated and the existing lines to Spence can be utilised for the booster station. If an agreement cannot be negotiated with Dominion Salt Works, a new 11 kV line would be required to S.H.1 which would incur a high cost to the scheme. Considering the number of days the pump station will operate (a maximum of forty days in dry years but less during normal years), a diesel powered pump may be a more cost effective alternative, particularly if access to power cannot be negotiated. This will incur higher operational and maintenance costs but these are not high compared to the savings in capital cost.

Marlborough Lines advise that the existing 11kV power supply to the Awatere River pump station has sufficient capacity for the proposed new pump station, and can be utilised providing the transformer is upgraded. A lines contribution will be payable to Marlborough.
Lines. A contribution will also be payable if electric pumps are selected for the booster station, despite the local lines being property of Dominion Salt.

Variable Speed Drives (VSDs) are required by Marlborough Lines for soft starting of the pumps, and an allowance has been made for these accordingly. They also require that motors be less than 100kW each. Budget figures for Programmable Logic Controllers (PLCs), switchboards, wiring, programming, and installation have also been allowed for at the booster station (the diesel option includes telemetry for the booster station), and the Awatere River pump station.

Power to supply PLC controls including level sensors at the dam is required. This is likely to be supplied from Flaxbourne.

Communications are proposed using radio links and repeaters, which is cost effective compared to fibre optic cabling installed in the trenching alongside the pipeline.
8 DISTRIBUTION NETWORK

8.1 Hydraulic Model

A hydraulic model of the concept distribution network was developed using DHI Mike Urban which is GIS integrated water modelling software. The hydraulic model is based on 20 m contour data that is generally available from Land Information New Zealand. This information has been used to determine the expected water pressure at each of the farm offtakes. Refer to Appendix D for the network layout showing the proposed alignment, pump station, farm offtakes and pipe pressure ratings and sizes.

8.1.1 Water Demand

The locations of the offtakes were determined from the irrigable area GIS shapefile obtained from MDC and were positioned at the property boundaries. The flow at each offtake was determined from the schedule of irrigable areas for each property ID provided. The schedule consisted of the property ID, irrigable area, a nominal value of 225 m$^3$/ha, and any extra volume required at a property. The total volume of water required at each property was calculated and the total flow of 675 m$^3$/s was distributed, on pro-rata basis, to reflect the water volume required at each property.

Properties 535882, 533822 and 181294 are located close to the transfer line; therefore, it is reasonable for the water to be taken directly from the transfer line.

8.1.2 Assumptions

The pipeline selection and booster pump station sizing has been designed to provide a minimum pressure head to the farm offtake of 20 m.

The pipeline friction loss factors used for the Mike Urban model are as follows:

a) Pipe roughness coefficient, $k = 0.25$ mm

b) Minor loss allowance, $K = 1.5$ / km

8.1.3 Results

Refer to Appendix E for the summary of offtake flows and pressures at each property when the network is running at full demand.
8.1.4 Proposed Pipeline Alignment

The preliminary alignment was based on the layout provided by Tonkin and Taylor in AutoCAD format. This alignment and the extent of the network was amended to service to the properties and the specified demands identified in the schedule provided.

It has been assumed that the pipelines will be constructed in the road verge within the road easement. Water will be piped to the identified properties and an offtake provided at the farm gate at a minimum head of 20m, this may be greater depending on the location and elevation of the offtake.

8.1.5 Pipe Size and Pressure Rating

The pipe sizes were selected to allow a maximum head loss of 5 m per 100 m of pipe length and a minimum pipe diameter of 150 mm were used. Long sections were developed for all pipe runs and the pipe pressures at every 100 m intervals were calculated to confirm the pressure ratings for the pipes.

It has been assumed that pipes 375mm dia or smaller will be High Density Polyethylene (HDPE) with larger pipes being fibreglass. The materials for these pipes are imported and prices vary due to exchange rates and petroleum prices. The selection of material should be investigated further at detailed design and tendering of the work could include pricing of more than one option to achieve the most cost effective solution.

8.2 Booster Pump Station

In order to deliver a minimum pressure head of 20 m to the farm gate, the distribution network requires a booster pump station. The booster pump is required to deliver a design flow of approximately 360L/s delivered at a head of 80 m. The concept layout for the network locates the pump station to the west of SH1 near the SH1/Ward Beach Road intersection at an elevation of 39 m.

8.3 Valves

The distribution network will require air relief valves at intervals along the pipelines to allow the removal of entrained air and scour valves at low points to allow draining of the line for maintenance and repair or removal of sediment. The number of air relief and scour valves were assessed by analysing general 100 m interval longsections produced from Mike Urban. Air valves were placed along the pipeline at all high points and at approximately every 600 m along rising grades or flat runs where there are no significant high points or grade changes. Scour valves were placed at low points of the pipeline.
9 STORAGE DAM

9.1 General Criteria

The dam site is located on an un-named stream that flows north across Flaxbourne Station into Clifford Bay. This location was selected in the preliminary design as being the most likely to store the required 2-3 Mm³. No other dam sites have been considered as part of the current study.

The maximum reservoir full supply level is generally constrained by the topography along the eastern side of the reservoir, where the lowest ground surface level is in the order of RL 120-125 m. Based on this, a maximum dam crest level of RL 120 m was selected with a FSL of RL 118 m, giving a usable storage volume of 2.6 Mm³.

There is an opportunity to gain further storage by raising the dam a few metres; however additional topographical survey will be required around the reservoir rim before this can be confirmed. A low point is known to exist along the eastern rim of the reservoir, but if this is a constraint it may simply require a low saddle dam to allow the reservoir level to be raised. Approximately 200,000 m³ of additional storage may be gained per metre increase in storage level.

9.2 Dam Site Location and Layout

The dam alignment and footprints are shown in drawing SK010 and SK011 (provided in Appendix F) for the RL 120 m dam crest level option. Some preliminary work was carried out on optimisation of the alignment to minimise dam volumes and construction costs in 2008, however it is expected that this would be looked at if the project was to proceed further. The location and layout has not been revisited as part of this project.

The storage volume to water surface elevation curve for the reservoir is shown on Drawing SK010. See Appendix F

9.3 Potential Impact Category Assessment

Dams are required to be classified in terms of the potential consequences of failure, in terms of loss of life, socio-economic losses, financial losses and environmental losses. The Potential Impact Category (PIC) classifies the dam in terms of these consequences for the specific dam, which in turn provides the design parameters that the dam design must comply with.

In the Preliminary Design Report this PIC assessment was carried out using the methodology set out in the NZSOLD Dam Safety Guidelines, 2000.

Since 2008 when the Preliminary Design Report was issued, there has been a statutory change in the requirements and methodology for determining the Potential Impact Category (PIC) of dams.

Dams are classified as “Buildings” under the Building Act 2004, and the Building Amendment Act 2013 defines a Large Dam as a dam that has a height of 4 or more and holds 20,000 or more cubic metres of water. A large dam requires a Building Consent before construction commences.

The Building (Dam Safety) Regulations 2008 are not due to come into force until 1 July 2015, however regulators have required compliance with this document for some years now when
applying for a building consent. These Regulations set out a more stringent methodology for determining PIC, which is applied below.

The Regulations are underpinned by the NZSOLD Dam Safety Guidelines 2000, which are currently under revision to bring them in line with the legislation.

The following sets out a revised PIC assessment using both the NZSOLD Guidelines and the Building (Dam Safety) Regulations methodologies.

### 9.3.1 NZSOLD Potential Impact Categories

The New Zealand Society of Large Dams (NZSOLD) Dam Safety Guidelines provide an “initial screening” classification in terms of dam height and storage volume as follows:

- **Very Low PIC**: height < 4 m; reservoir level < 3 m; volume < 20,000 m$^3$.
- **Low PIC**: height < 10 m and less than 6 m if storage exceeds 50,000 m$^3$.
- **Medium PIC**: height 10 – 20 m, but < 15 m storage level if volume exceeds 1,000,000 m$^3$.
- **High PIC**: where these parameters are exceeded.

These guidelines imply that the dam should be High PIC. However, the guidelines also indicate that these parameters should not control the PIC assessment where the consequences of failure are not consistent with the initial screening.

Table 9.1 sets out the PIC assessment guidelines in terms of consequences:

<table>
<thead>
<tr>
<th>Potential Impact Category</th>
<th>Potential Incremental Consequences of Failure</th>
<th>Socio-economic, Financial, &amp; Environmental</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>Fatalities</td>
<td>Catastrophic damages</td>
</tr>
<tr>
<td>Medium</td>
<td>A few fatalities are possible</td>
<td>Major damages</td>
</tr>
<tr>
<td>Low</td>
<td>No fatalities expected</td>
<td>Moderate damages</td>
</tr>
<tr>
<td>Very Low</td>
<td>No fatalities</td>
<td>Minimal damages beyond owner’s property</td>
</tr>
</tbody>
</table>

1. Refer to “New Zealand Dam Safety Guidelines” published by NZSOLD, November 2000
Table 9-2  Ward Dam Dimensions (120 m RL Reservoir Option)

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Water Depth</td>
<td>33 m</td>
</tr>
<tr>
<td>Maximum Height</td>
<td>40 m</td>
</tr>
<tr>
<td>Maximum Height above Foundation</td>
<td>38 m</td>
</tr>
<tr>
<td>Water Storage Volume</td>
<td>2,600,000 m³</td>
</tr>
</tbody>
</table>

Figure 9-1  Dam Dimension Measurements

While the physical dimensions of the proposed dam (shown in Table 9-2) indicate that, based on the initial screening assessment it should be categorised as High PIC, the potential consequences of failure has the over-riding influence in specifying the PIC. These are discussed as follows:

9.3.2 Potential Incremental Consequences of Failure

“Incremental” consequences refer to those over and above those which might have occurred for the same natural event or conditions, had the dam not failed.

Risk to Structures

The downstream structures which may come under some risk are Cape Campbell Road and possibly some farm buildings located 0.7 - 1.0 km downstream. The lower catchment area is uninhabited. Two stream valleys extend downstream of the dam site, one aligned toward the northwest and the other heading due north. The northwest valley extends for approximately 2 km downstream, following Cape Campbell Road, before reaching Lake Grassmere, whereas the north valley continues for approximately 4km to a small bridge at the coast at Marfells Beach. In the event of an embankment failure it is anticipated that structural damage will be entirely confined to within these two valleys.

- Cape Campbell Road, would be overtopped for its entire length across these valleys and put out of service.

Depending on the location of any dam breach the resulting flood flow could flow down either the northwest or north valley damaging unoccupied farm buildings or the small bridge at Marfells beach.

Risk to Human Life
Under the NZSOLD Guidelines, population at risk is not defined, with consequences to human life being qualitatively defined in terms of “Fatalities”. The term Population at Risk (PAR) will be brought in with the Building (Dam Safety) Regulations 2008, which are discussed further below. These regulations define PAR as the number of people likely to be affected by inundation greater than 0.5 m in depth. With no residences downstream of the dam, the PAR would be zero.

It is anticipated that maintenance personnel and or farmers could be working in the vicinity of the dam at any time. Other people who may be downstream of the dam include itinerant travellers using the road to Marfell’s beach.

No probability assessment of the likelihood of an itinerant worker or traveller being located downstream of the dam at the time of a failure has been carried out at this stage, but this would need to be carried out for detailed design to confirm the PIC assessment. Under the NZSOLD methodology itinerant workers were not considered at risk.

**Socio-economic & Environmental Risk**

The socio-economic risk associated with a dam failure is minimal given that there are no residences or major public infrastructure in the path of any dambreak flow. In addition, the environmental consequences of a dambreak may only include erosional damage to the valleys downstream, which would be no worse than Moderate in Table 8.1.

Therefore on the basis of the consequences and solely using the NZSOLD Guidelines, it could be argued that the dam is a Low PIC classification. However, with the change in legislation, the more stringent methodology for assessing PIC in the Building (Dam Safety) Regulations 2008 must now be used, as described below.

### 9.3.3 **Building (Dam Safety) Regulations 2008 Methodology**

Section 4 of the Regulations sets out the methodology for dam classification using Tables 1 and 2 in Schedule 1. The following is a preliminary desktop assessment of the PIC.

Table 1 is used to assess the potential damage level under the categories: Residential Houses, Critical or Major Infrastructure, Natural Environment and Community Recovery Time. The damage level is categorised from Minimal to Catastrophic. Our assessment under each of these categories is:

- **Residential Houses**: No known houses downstream – Minimal
- **Critical or Major Infrastructure**: flooding of farmland, potential damage to farm buildings, fences and roads, silt spread across paddocks, loss of dam and irrigation to community for at least 1 year – Major
- **Natural Environment**: short term damage to farmland, creeks etc – Moderate
- **Community Recovery Time**: Loss of irrigation for at least one season while dam re-built - Major

The Assessed Damage Level is therefore Major.
Table 2 is used to factor in Population at Risk (PAR). A “sunny day” dam failure could place at risk people driving past the dam site to Marfell’s Beach or farm workers below the dam. Our assessment is that the PAR is greater than zero but unlikely to result in more than 1 life lost.

Using this methodology, the dam would be classified as Medium PIC, although given the absence of residences or major infrastructure, we suggest the PIC could be at the low end of Medium.

9.3.4 PIC Classification – Design Criteria

For the Medium PIC classification outlined above, the dam design should be based on the following parameters:

**Design flood**

The Incremental Damage Flood (IDF) is between 1 in 1,000 and 1 in 10,000 Annual Exceedence Probability (AEP). Given the small reservoir catchment area the spillway could be sized for the larger event at minimal cost penalty. **Seismic Design**

Seismic design requires two levels of shaking to be assessed. Under the Operating Basis Earthquake (OBE) which has a 1 in 150 AEP, the dam may suffer minor damage but must remain operational.

Under the Maximum Design Earthquake (MDE), some damage is permitted, which may even take the reservoir out of service for repairs, but the damage must not lead to catastrophic failure.

For a Medium PIC classification, the MDE should be between a 1 in 1,000 AEP and 1 in 10,000 AEP event. Based on our assessment of PIC being at the lower end of the medium range, a 1 in 1,000 AEP seismic event would be appropriate for the MDE, resulting in a peak ground acceleration (PGA) of 0.84 g (after Stirling et al, 2007). Given the 2013 Cook Straight and Grassmere earthquakes, we recommend a site specific seismic hazard assessment be carried out at detailed design stage to confirm the design loadings.

9.4 Diversion Flood and Diversion Works

The flood magnitude used for the diversion design during construction is dependent on the consequences of overtopping. Depending on the type of dam, this is typically between 1 in 10 AEP and 1 in 100 AEP. For this dam the consequences of failure are unlikely to be loss of life or major damage to infrastructure and should be confined to cleanup and re-build costs for the dam.

We suggest a diversion design flood of a 20 year AEP, with a flow of 9 m³/sec. This is a relatively small flow and will be controlled during construction by a small cofferdam upstream of the dam footprint, with flood flows being conveyed downstream of the dam via a temporary pipe located within the dam foundation.

It may be possible to convert the diversion pipeline to the permanent intake/outlet arrangement. The configuration, size and specification of the diversion and intake/outlet structure will be determined during detailed design.
9.5 Alternative Dam Types

Three dam types have been considered during the preliminary design stage:

- Zoned embankment dam
- RCC (roller-compacted concrete) dam
- Hardfill dam

Typical cross-sections for each of the dam types are shown on Drawing SK013 in Appendix F.

Zoned Embankment Dam

Zoned embankment dams are the most common type of cross section for a dam of this size in New Zealand and typically comprise the zones shown in Figure 9-2.

The zoned embankment dam would generally consist of the following material types: a central impervious core, a downstream pervious filter zone and drainage blanket, and strong well-drained shoulders to support the dam. A layer of rip rap would be placed on the upstream face to protect it from wave erosion. Depending on the engineering characteristics of the available materials, a transition zone may be required between the shoulder and filter to prevent migration of materials within the dam. The materials that have been identified within the vicinity of the dam site are described in Section 4.

The preliminary dam design includes a core with 0.5H:1V side slopes. The core will comprise low permeability material sourced from the weathered Tertiary mudstone within the reservoir. The natural moisture content of this material appears to be dry of the optimum moisture content and will need conditioning prior to placement. This will involve breaking down the larger particles and moisture conditioning to ensure that maximum compaction can be achieved. A vertical downstream face on the core could be considered during detailed design to reduce the volume of the core and filter drain.

A one metre wide filter drain separating the downstream shoulder from the core has been assumed. This filter drain intercepts any seepage passing through the core and prevents migration of core material should a crack occur in the core. The filter material will comprise a fine, specifically graded sand manufactured by screening and/or crushing greywacke rock or gravels.

A blanket drain will be constructed beneath the downstream shoulder to control any seepage through the dam and foundation and prevent pore pressures from building up within the
downstream shoulder. The preliminary design assumes that the blanket drain will have an average thickness of 1 m and consist of greywacke-derived sandy gravel.

The shoulders incorporate 2.5H:1V side slopes based on the assumption that good quality greywacke gravels can be incorporated into the dam shoulders. The shoulders are expected to include most of the available gravely alluvium from the within the reservoir footprint as well as some greywacke derived alluvium from upstream of the reservoir. These gravels may need to be screened to remove excess fines.

Preliminary slope stability modelling confirms that the dam has an adequate factor of safety under static loads at full storage. Under the relatively high peak ground acceleration of 0.84 g (1000 year AEP) deformation and cracking may be expected. The current design incorporates defensive measures such as broad shoulders and zones of filter to accommodate the likely deformations. Further modelling will be required during detailed design to refine the dam design. In particular, this modelling will confirm the optimum combination of materials within the dam cross section, to ensure that the dam meets its design criteria. The modelling will also confirm any design features that may be required to limit deformation such as additional buttressing of the shoulders.

Additional modelling will also optimise the dam alignment to choose the most efficient site in terms of dam volume and reservoir volume. The assumed alignment has a straight crest across two valleys, which minimise the dam crest length and maximises the reservoir volume. There may be an opportunity to reduce the material volume in the dam by using a “dogleg” alignment to follow natural higher ground, but this needs to be optimised against reservoir volume and dam height.

9.5.1 Roller compacted concrete (RCC)

The RCC dam is gravity dam constructed with roller compacted concrete, which is a low strength concrete placed by bulk earthmoving equipment and rolled in layers. A waterproof barrier is usually required on the upstream face and can be constructed using conventional concrete. The advantages of the RCC configuration include the smaller cross sectional area, and rapid construction which reduces costs which compared to a conventional concrete dam. Significant reduced costs also result from incorporation of the spillway in the crest of the dam.

Foundation conditions for RCC usually require sound rock and the material encountered during the preliminary site investigations are not believed to be strong enough to support an RCC dam. Additional site investigations will be required to confirm whether adequately strong rock is present at a practical depth. If adequate foundation conditions cannot be achieved, this type of dam will not be feasible. Good quality concrete aggregate within close proximity of the dam site is required to make this option economic, otherwise RCC would be an expensive option for this site.

Few RCC dams have been built in New Zealand, but this method has been widely used in recent years throughout the World.

9.5.2 Hardfill

The hardfill dam is a special type of RCC dam which utilizes a larger cross-sectional area and lower strength, less costly RCC mix. The symmetrical cross section also gives greater stability than a conventional RCC dam in areas where there is a high seismic hazard.
A waterproof barrier is required on the upstream face, which could be constructed using either conventional concrete, shotcrete or a geomembrane.

This is a relatively uncommon dam type, without precedent in New Zealand, although over 100 have been constructed worldwide.
10 COST ESTIMATES

The cost estimates for the construction of the various elements of the irrigation scheme have been based on rates provided by suppliers, previously tendered rates for similar work items and budget prices based on discussions with contractors experienced in carrying out similar work items.

The quantities have been based on volumes for the preliminary design of the major work items.

An initial assessment of costs showed that a high level of processing of onsite materials would be required for construction of a roller compacted concrete dam (RCC) and a hardfill dam. As it has been confirmed that an earth embankment dam can be constructed with limited processing of on site materials, the increased cost of processing material more than offsets any volume reductions for the RCC and hardfill dams these options were dropped from further consideration.

Other Owner Costs have been allowed for covering: detailed design, resource consents, building consent for the dam, construction management and Owner’s contingency. No allowance has been made for Owner internal costs, prospectus, community consultation or financing, which we would expect FCL to be in the best position to estimate.

10.1 Cost Estimate Assumptions

The following assumptions have been adopted for the quantities, dimensions, materials and rates used for the development of the construction costs and the operation and maintenance costs.

10.1.1 Preliminary and General (P&G) (15% of Construction Cost)

A preliminary and general allowance of 15% of the construction cost has be allowed for:

- Insurance of works.
- Mobilisation and site establishment.
- Temporary works and traffic control; and
- Health and Safety and construction supervision.

10.1.2 Infiltration Gallery

The costing for the infiltration gallery assumes the following.

- The gallery will consist of 8 No 80m long PVC screen pipes of 450 mm dia.
- The pipes would be installed at 3m spacing across the river with two banks of four pipes each draining to 750mm dia. pipe transporting flows to the transfer pump sump.
- The screen pipes will have a minimum cover of 3m below ground with a filter bed thickness of 300mm below the pipes and 1m above the pipes.
• Construction of the gallery in the river bed will require construction of a channel to divert Awatere flows around the works.

10.1.3 Awatere River Pump Station

The costing for the pump station assumes the following:
• The pump station will consists of the pumps (as identified in section 7) pumping form a concrete pump sump.
• The pumps and controls will be located in a simple structure at the identified location.
• There is HV electrical supply to the Yealands pump station just upstream of the site but it is unlikely this would be sufficient to operate an additional pump of the size proposed. Therefore costs have assumed HV cabling at a rate of $50/m from the junction with Seaview Rd and Reserve Rd and an additional transformer.
• The HV cable and the transformer may be subsidised by Marlborough Lines but no allowance for this has been made.

10.1.4 Rising Main Awatere to Dam

The Costing for the transfer line to the dam assumes the following:
• The pipeline will be located in the road verge within the road easement along the alignment shown.
• The pipeline will be constructed using fibreglass pipes with a typical depth of cover of 900mm.
• The length of pipe and various pressure rating have been defined using the topographical maps for the area with a 20m contour interval.
• Traffic management has been allowed for as part of the P&G items.
• Stream, road and rail crossings have been allowed for with extra over items.
• Pipe costs for both the transfer line and the distribution network have been developed from prices prepared for the Tarras and North Otago Irrigation Company (NOIC) schemes. These projects have not progressed to construction but the NOIC rates underwent independent review.

10.1.5 Zoned Embankment Dam

The Costing for the dam assumes the following:
• Based on the geotechnical information available the materials for the dam construction can be sourced from within the dam catchment or the area adjacent and costings have assumed this will be the case. The mobilisation of the processing plant and establishment of a quarry is covered under the the P&G items for the dam.
• Consenting cost for the dam and a $50k that may be required by the Council is identified separately from the construction costs.
---

- Quantities are insitu volumes.
- The dam will be filled and discharge via the same pipe.
- An emergency drawdown facility will be included on the downstream side of the dam. This will be large diameter gate valve that can be opened discharging water into the channel running down Cape Campbell Road to the salt works. This would only be used in extreme circumstances and no stream works have been allowed for.
- The outlet structure for the dam would consist of a pipe from a floating pontoon leading down to a flexible connection with the pipeline running through the base of the dam. This ensures that water is always drawn from the surface. No detailed assessment of the system has been carried out and the costing includes a provisional sum of $300k for this item.
- Prices for the manufacture of material have been developed from tendered rates for the Central Plains Irrigation Scheme currently under construction.

**10.1.6 Distribution System**

The costing for the distribution network assumes the following:
- The pipeline will be located in the road verge within the road easement along the alignment shown.
- The pipeline will be constructed using HDPE and fibreglass pipes with a typical depth of cover of 900mm.
- The length of pipe and various pressure rating have been defined using the topographical maps for the area with a 20m contour interval.
- Traffic management has been allowed for as part of the P&G items.
- Stream, road and rail crossings have been allowed for with extra over items.
- A single take off point has been allowed for at each property identified in the demand schedule.
- A booster pump is required within the network. It is located beside S.H.1 and it is assumed that an HV supply would be available near the finalised location. Cost has allowed for a short length of cabling and a transformer. No subsidy by Marlborough Lines has been allowed for.

**10.1.7 O & M costs**

The annual electrical costs for the system have been based on the electrical requirement for the average pumped volume (both transfer and booster pump) for the 25 year period. Power has been assumed to cost 18c/kWh.

Maintenance costs for the pipework are based on a percentage of the construction cost based on a 100 year life and annual repair of damage faults and inspection of valves.
Maintenance costs for Variable Speed Drives (VSDs) and controls are a percentage of the construction cost based on replacement every 15 years with minor additional annual maintenance.

Maintenance cost for the pumps are a percentage of the construction cost based on replacement every 25 years.

Maintenance for the infiltration gallery assume a cost of $5000/year for maintaining of the channel and bank protection in the area of the intake with ten yearly maintenance to maintain performance costing of excavating down to and replacing the surface layer of the filter material.

10.1.8 Design investigation and Construction management cost

Design Investigation and Construction Management costs have been included as percentage of the construction total for costing purposes.

10.1.9 Resource Consents

Resource consenting cost have been based on the consents required as identified in Section 5 and the work carried out for the consenting of the Waterco Scheme.

10.1.10 Building Consents

Construction of the dam comes under the Building Act (2004) and requires a building consent. Typically local authorities do not have the expertise in house to assess these consents and monitor the construction and this is contracted out and charged to the applicant. The cost provided is based on fees for similar scale projects.

10.1.11 Contingency

A contingency sum of 25% of the construction cost has been allowed for in the estimate to cover variations in design, quantities and additional items not directly measured and costed at this early stage of design.

10.2 Summary of the Cost Estimates

The summary cost estimate table is shown in Table 10-1 below, with a detailed breakdown of the cost estimates provided in Appendix G. These show that the scheme total capital cost of construction estimate is $38,838,512 and an annual operation maintenance cost estimate of $967,500 per year. The total capital cost consists of a construction cost of $28,613,700, other owner’s cost of $3,071,400 and a contingency allowance of $7,153,400.

A summary of the costs is shown in Table 10-1:
Table 10-1  Summary of Costs

<table>
<thead>
<tr>
<th>CAPITAL COSTS</th>
<th>COST TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>CONSTRUCTION COSTS</strong></td>
<td></td>
</tr>
<tr>
<td>P&amp;G</td>
<td>$ 3,717,223</td>
</tr>
<tr>
<td>INFILTRATION GALLERY</td>
<td>$ 422,304</td>
</tr>
<tr>
<td>PUMP STATION (AWATERE RIVER)</td>
<td>$ 1,718,686</td>
</tr>
<tr>
<td>TRANSFER PIPELINE</td>
<td>$ 7,742,113</td>
</tr>
<tr>
<td>ZONED EMBANKMENT DAM</td>
<td>$ 7,746,541</td>
</tr>
<tr>
<td>DISTRIBUTION NETWORK</td>
<td>$ 7,151,846</td>
</tr>
<tr>
<td>COMMISSIONING</td>
<td>$ 115,000</td>
</tr>
<tr>
<td><strong>CONSTRUCTION TOTAL</strong></td>
<td>$ 28,613,712</td>
</tr>
<tr>
<td><strong>OTHER COSTS</strong></td>
<td></td>
</tr>
<tr>
<td>Detailed Design &amp; Investigations (5% of CC)</td>
<td>$ 1,430,686</td>
</tr>
<tr>
<td>Resource Consents</td>
<td>$ 60,000</td>
</tr>
<tr>
<td>Building Consent</td>
<td>$ 100,000</td>
</tr>
<tr>
<td>MDC Quarry Fee</td>
<td>$ 50,000</td>
</tr>
<tr>
<td>Construction Management (5% of CC)</td>
<td>$ 1,430,686</td>
</tr>
<tr>
<td><strong>OTHER COST TOTAL</strong></td>
<td>$ 3,071,371</td>
</tr>
<tr>
<td><strong>RECOMMENDED CONTINGENCY</strong></td>
<td></td>
</tr>
<tr>
<td>Dam</td>
<td>$ 2,233,797</td>
</tr>
<tr>
<td>Transfer Pump Station and Pipeline</td>
<td>$ 2,854,309</td>
</tr>
<tr>
<td>Distribution Network</td>
<td>$ 2,085,322</td>
</tr>
<tr>
<td><strong>TOTAL CONTINGENCY</strong></td>
<td>$ 7,153,428</td>
</tr>
<tr>
<td><strong>SCHEME CAPITAL COST TOTAL</strong></td>
<td>$ 38,838,512</td>
</tr>
</tbody>
</table>

| **ANNUAL OPERATION AND MAINTENANCE**               | $/YEAR          |
| Operation                                         | $ 607,000       |
| Maintenance                                       | $ 360,473       |
| **ANNUAL OPERATION AND MAINTENANCE COST TOTAL**    | $ 967,473       |
11 LIMITATIONS

URS New Zealand Limited (URS) has prepared this report in accordance with the usual care and thoroughness of the consulting profession for the use of Flaxbourne Community Irrigation Ltd. and only those third parties who have been authorised in writing by URS to rely on this Report.

It is based on generally accepted practices and standards at the time it was prepared. No other warranty, expressed or implied, is made as to the professional advice included in this Report.

It is prepared in accordance with the scope of work and for the purpose outlined in the proposal letter dated 6 August 2014.

Where this Report indicates that information has been provided to URS by third parties, URS has made no independent verification of this information except as expressly stated in the Report. URS assumes no liability for any inaccuracies in or omissions to that information.

This Report was prepared between 25 August and 12 November 2014 and is based on the conditions encountered and information reviewed at the time of preparation. URS disclaims responsibility for any changes that may have occurred after this time.

This Report should be read in full. No responsibility is accepted for use of any part of this report in any other context or for any other purpose or by third parties. This Report does not purport to give legal advice. Legal advice can only be given by qualified legal practitioners.

Except as required by law, no third party may use or rely on this Report unless otherwise agreed by URS in writing. Where such agreement is provided, URS will provide a letter of reliance to the agreed third party in the form required by URS.

To the extent permitted by law, URS expressly disclaims and excludes liability for any loss, damage, cost or expenses suffered by any third party relating to or resulting from the use of, or reliance on, any information contained in this Report. URS does not admit that any action, liability or claim may exist or be available to any third party.

Except as specifically stated in this section, URS does not authorise the use of this Report by any third party.

It is the responsibility of third parties to independently make inquiries or seek advice in relation to their particular requirements and proposed use of the site.

Any estimates of potential costs which have been provided are presented as estimates only as at the date of the Report. Any cost estimates that have been provided may therefore vary from actual costs at the time of expenditure.
URS is a leading provider of engineering, construction, technical and environmental services for public agencies and private sector companies around the world. We offer a full range of program management; planning, design and engineering; systems engineering and technical assistance; construction and construction management; operations and maintenance; and decommissioning and closure services for power, infrastructure, industrial and commercial, and government projects and programs.