REPORT

WAIMEA WATER AUGMENTATION COMMITTEE/TASMAN DISTRICT COUNCIL

Waimea Water Augmentation Component 2 - Storages Assessment

Report prepared for:

WAIMEA WATER AUGMENTATION COMMITTEE/TASMAN DISTRICT COUNCIL

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1 Introduction

In November 2004, Tonkin and Taylor Ltd (T&T) was commissioned by the Waimea Water Augmentation Committee (WWAC) and Tasman District Council (TDC) to undertake Phase 1 of a feasibility study of water storage in the upper parts of the Wairoa/Lee catchments in Tasman District. The specific brief was to address the recurrent water shortages experienced on the Waimea Plains and to investigate enhancing water availability for consumptive and environmental/community/aesthetic benefits downstream on the Waimea Plains and surrounds.

The basic principle behind the project has been to develop an augmentation scheme that would capture river flows (leaving an appropriate residual flow in the river), store the water in a reservoir, and then allow release of that stored water into the river system during periods of high water demand and/or low natural river flows to augment those supplies, either directly or via a recharging of the groundwater system.

The project is multi-disciplinary and Phase 1 (preliminary) feasibility has extended over a three year period. It has four main components:

1. water demand and availability analysis
2. identification of storage options, and water delivery methods and costs
3. environmental assessment, and economic analysis of scenarios with and without augmentation
4. water allocation for optimisation of water use, environmental/community benefits/funding.

This report addresses Component 2 – identification and engineering assessment of storage options. The following work has been undertaken in this component:

- identification of a range of potential storage sites
- assessment of the broad-scale physical, engineering, and environmental constraints to refine the list to small number of practical storage sites
- refinement of hydrological, physical, engineering, and environmental issues and conducting a Workshop with WWAC to determine up to three possible storage options
- preliminary geotechnical investigations of preferred site
- preliminary hazard analysis for preferred site
- preliminary estimation of capital costs of preferred choice

Clarification Note: The NZMS topographic map names the branches of the Wairoa River as follows:

- Left Branch – this is the eastern branch
- Right Branch – this is the western branch

This naming is opposite to the usual convention of referring to right and left branches (or banks) to reflect the orientation when facing **downstream**. To avoid confusion in this report we have endeavoured to make it clear by including reference to the east or west in our descriptions.

2 Assessment of Potential Storage Sites

2.1 Selection Process

The assessment of potential storage sites was undertaken in a staged way, starting with a large number of sites, and gradually narrowing the list down through an assessment of engineering, environmental and social factors.

The assessment was undertaken in conjunction with WWAC and its Project Manager.

2.2 Initial Site Identification

In December 2004 Tonkin and Taylor completed a preliminary scan of possible storage (and infill) options in and adjacent to the study area, but excluding those below about 5 million m^3 size (see **Appendix 1** for letter report and map of sites).

A preliminary map of possible storage (and infill) arrangements was prepared. Most but not all were inspected on Thursday 9 December 2004, access to some being barred by locked gates (Sites 3, 4, 11, 13, 14).

A comprehensive approach to identifying possible storages was adopted in this first instance, with the next step being to cull the list down to a few of the apparently most competitive options and to undertake a preliminary fatal flaws review including broad environmental and cultural issues, hydrological characteristics, and indicative costs. Before doing that exercise, however, WWAC's views were sought on options which may not be viable from WWAC's perspective.

The Wai-iti River source or storages in the Wai-iti catchment are excluded from the present brief from WWAC. However, apart from Upper Wai-iti storage possibilities previously identified, lower Wai-iti possibilities were identified that the team included in the initial report to WWAC for completeness. Any storage in the Wai-iti can be utilised to supplement the Waimea aquifer (maybe needing a transfer to the Wairoa at the gorge) and extend out the area of new irrigation, which is included in the brief.

We noted that the best solution may involve more than one storage and it was kept in mind that, at least conceptually, storage for consumptive use could be separated out from storage for riparian flow enhancement, plus adding further flexibility to potential solutions.

It was felt at this stage of the project (without any input of environmental or cultural issues) that the off-river type storages deserved strong focus because they may involve considerably fewer environmental issues, have considerably reduced diversion and spillway requirements, are all in open country, have much better storage characteristics than any of the on-river storages, and are in the Moutere Gravel Formation (with the exception of site 5A) which is favourable for earth dam construction, therefore leading to reduced dam costs. Pumped transfer systems for infill is the corresponding negative, but a reasonable amount of local catchment capture is achievable without excessive pumping. If an on-river dam is part of the solution and add-on hydro is cost effective, the whole solution may be self sufficient in energy and avoid future energy price shocks.

The following were considered by the team to be top contenders:

- Pigeon Valley North (Site 2) with infill ex Pigeon Valley South (Sites 1A or 1B) and Wai-iti (in principle also by gravitating Wairoa water in a pipe from the Pig Valley Saddle).
- Church Valley (Site 5) with infill ex Wairoa, and integrating Pig Valley Stream (site 5A) via two stage pumping and intermediate storage (potential to recover around 40 – 50% of pump energy).
- a storage on the Lee River (Site 10A or 10B and/or 11), ideally with cost effective hydro

Those which appeared likely to be ranked lowest and without considering ecological impacts, are:

- No. 4 - poor storage
- No. 7 - berry farm and camp area at head of storage may be constraints
- No. 8 - some 8 or so residences affected plus a reasonably well used stretch of road
- No. 9 - to achieve reasonable storage, impinges closely on twin bridges area
- No. 10A - impact on TDC recreational area an issue?
- No. 12 - relatively poor storage and high cost
- Nos. 14/15 - poor storage

The site immediately downstream of the Wairoa – Lee confluence was not included on the basis that it would have large impact on infrastructure, would be comparatively quite expensive because of diversion and spill requirements and possibly could also be a problem to consent.

Site 13, the preferred site from the previous study by MWH, would result in an expensive dam to achieve the scale of storage anticipated to meet ultimate objectives or contribute significantly to required storage. It is noted that diversion and flood passage requirements would be substantial and there is no apparent convenient source of earthfill nearby. The likely dam type would be concrete or concrete faced rockfill because of material availability considerations, the tight site and risk exposure during diversion.

With the exception of two or three sites, the potential storages were drawn on the basis of about a 40 m dam height. While somewhat arbitrarily chosen, this height enabled a quick visual appreciation of comparative storage potential. Most of the useable storage is near the top of the dam.

WWAC's technical team discussed the options presented, and requested the removal of seven sites (see Section 2.3 below).

2.3 Selection of Short-list

Following on from assessment of the initial 18 sites, the remaining sites were:

- Sites 1A and 1B – Pigeon Valley South
- Site 2 – Pigeon Valley North
- Site 3 – Unnamed tributary of the Wai-iti River
- Site 4 – Teapot Valley

- Site 10A – Lower Lee
- Site 10A – Middle Lee
- Site 11 – Upper Lee
- Site 13 – Wairoa Forks
- Site 14 – Right Branch Wairoa (western)
- Site 15 – Left Branch Wairoa (eastern)

A ranking exercise of the above sites was undertaken, based on a range of initial technical and environmental criteria. These included:

- storage characteristics
- geological/seismic risk
- reservoir filling
- constructability
- hazard potential
- power generation potential
- flexibility for staging
- cultural acceptability
- land use
- effect on infrastructure
- aquatic ecology
- terrestrial ecology
- recreation
- archaeology

The following five sites ranked highest:

- Site 2 – Pigeon Valley North
- Site 10A – Middle Lee
- Site 11 – Upper Lee
- Site 13 – Wairoa Forks
- Site 15 – Left (eastern) Branch Wairoa

On 5 April 2005, a report setting out the characteristics of the above sites was discussed during two workshop sessions, firstly with a Technical Group of WWAC, and secondly with the formal Committee that evening (see **Appendix 2**).

A broad ranking exercise was undertaken by the Technical Group and was subsequently endorsed by WWAC. The ranking process assessed each of the five sites according to general criteria covering environmental, engineering, and consentability/public acceptance issues. The outcome was as follows:

- Site 11 (Upper Lee) – highest (best) ranking
- Site 2 (Pigeon Valley)
- Site 15 (Left (eastern) Branch Wairoa)
- Site 10B (Middle Lee) and Site 13 (Wairoa Forks) – lowest equal ranking

There was little difference between Sites 2 and 15 in terms of their relative ranking.

Accordingly, Sites 11, 2, and 15 were selected by WWAC for ongoing investigation.

2.4 Site 2 Assessment

Site 2 was then investigated further to determine relative costs (see **Appendix 3**), and WWAC representatives also met with Pigeon Valley residents to gauge community response. On the basis of the results of both those processes, Site 2 was eliminated from further investigation at that stage. Accordingly Tonkin and Taylor was instructed by WWAC to continue a comparative assessment of Site 11 (Upper Lee) and Site 15 (Left (eastern) Branch Wairoa.

2.5 Comparative Assessment of Two Sites

A preliminary and comparative investigation was undertaken for Sites 11 and 15, as a basis for WWAC to choose a preferred option to continue pre-feasibility investigations for. The report covering this stage of the project comprises **Appendix 4.**

The assessment focussed on those aspects that contributed to a *comparative* assessment. However, as a basis for comparison, the key aspects that drive the comparison were also set out, as well as providing comments on those issues where no comparative assessment had yet been undertaken (eg cultural impact).

The key comparative issues are summarised in tabular form below.

Feature	**Site 11 - Lee**	**Site 15 – Left Branch Wairoa**
Water demand	No difference	No difference
Storage capacity	No difference	No difference
Reservoir area (hectares)	Larger	Smaller
Materials availability	Majority available on-site	Expect most to be imported
Construction access issues	Effects less apparent	Effects more apparent
Operating regime	Effects more apparent	Effects less apparent (Right Branch Wairoa provides buffering)
Sedimentation potential	No significant difference	No significant difference
Downstream hazard potential	Lower	Higher
Required design standard	No difference	No difference
Comparative cost (base price)	No significant difference (within current order of accuracy)	No significant difference (within current order of accuracy)
Cost (piped delivery)	Lower cost	Higher cost
Land tenure	More owners (7)	Fewer owners (4)
Land administered by DOC	Less	More
Potential electricity generation	Little difference (1.2 MW; 6.8 GWh/annum)	Little difference (1.7 MW; 9.8 GWh/annum)
Aquatic ecology	Little difference	Little difference
Water quality (effect of	No issue	Potentially an issue

Feature	Site 11 - Lee	Site 15 – Left Branch Wairoa
ultramafic geology in catchment)		
Indigenous vegetation	More significant	Less significant
Blue duck (whio)	No difference	No difference
Cultural impact	Difference not known at this stage	Difference not known at this stage
Trout fishing	Little difference	Little difference
Informal recreation	Little difference	Little difference
Kayaking	No issues	An issue – development would be opposed
Access to Richmond Forest Park	Lesser effect	Greater effect
Archaeology/heritage values	No difference apparent at this stage	No difference apparent at this stage
Community preference	Not gauged	Not gauged
Enhancement opportunities	No difference apparent at this stage	No difference apparent at this stage

It appeared from the above table (without weightings being applied to any criteria) that the Lee may have more positive points than the Wairoa.

A Draft Discussion Document on the investigations was presented to WWAC at its meeting on 19 July 2006. Following that meeting, WWAC undertook further consultation with various components of the community, and considered the issues further at a second WWAC meeting on 21 August 2006. At that second meeting, WWAC took the decision to focus further investigations on Site 11 Lee River as the preferred option for possible storage.

The sections of this report on the Storages Assessment that specifically relate to dam site and reservoir location focus on the preferred site selected by WWAC; ie Site 11, Upper Lee.

Appendix 4 contains all information relating to the comparative assessment between Sites 11 and 15, including all collated information on the alternative site (Site 15 Left (eastern) Branch Wairoa River).

3 Site 11 – Upper Lee River – Engineering Characteristics

3.1 Location/Site Topography

Site 11 is within the main stem of the Lee River. The storage dam would be an instream dam.

Initially a site approximately 700m downstream from the confluence with Anslow Creek was identified as being suitable for a storage dam. However, following on-site geotechnical investigations (see Section 3.4), an alternative site approximately 250m downstream was identified as potentially being more suitable (see Figure 3.1).

The head of the reservoir would extend upstream for approximately 3.5 km, and would incorporate the lower reaches of Anslow Creek and Waterfall Creek. Initial delivery would be by release into the river.

3.2 Catchment Hydrology

Key hydrological parameters for this dam site are as follows:

- Catchment area to dam site = 84 km^2
- Estimated mean flow ≈ 3.6 m^3/s, equivalent to 113 million m^3 per annum
- Estimated mean annual low flow (MALF) ≈ 470 l/s , which is also assumed to be the minimum residual flow required at the foot of the dam
- Estimated 7-day 5-year low flow ≈ 360 l/s
- Maximum controlled dam release = 2070 l/s, based on 1100 l/s Appleby Bridge residual, full future allocations and 22,000 m^3/day future regional supply (see Component 1 Report)
- 10 year AEP flood peak 270 m^3/s
- 100 year AEP flood peak 400 m^3/s
- 10,000 year AEP flood peak 650 m^3/s

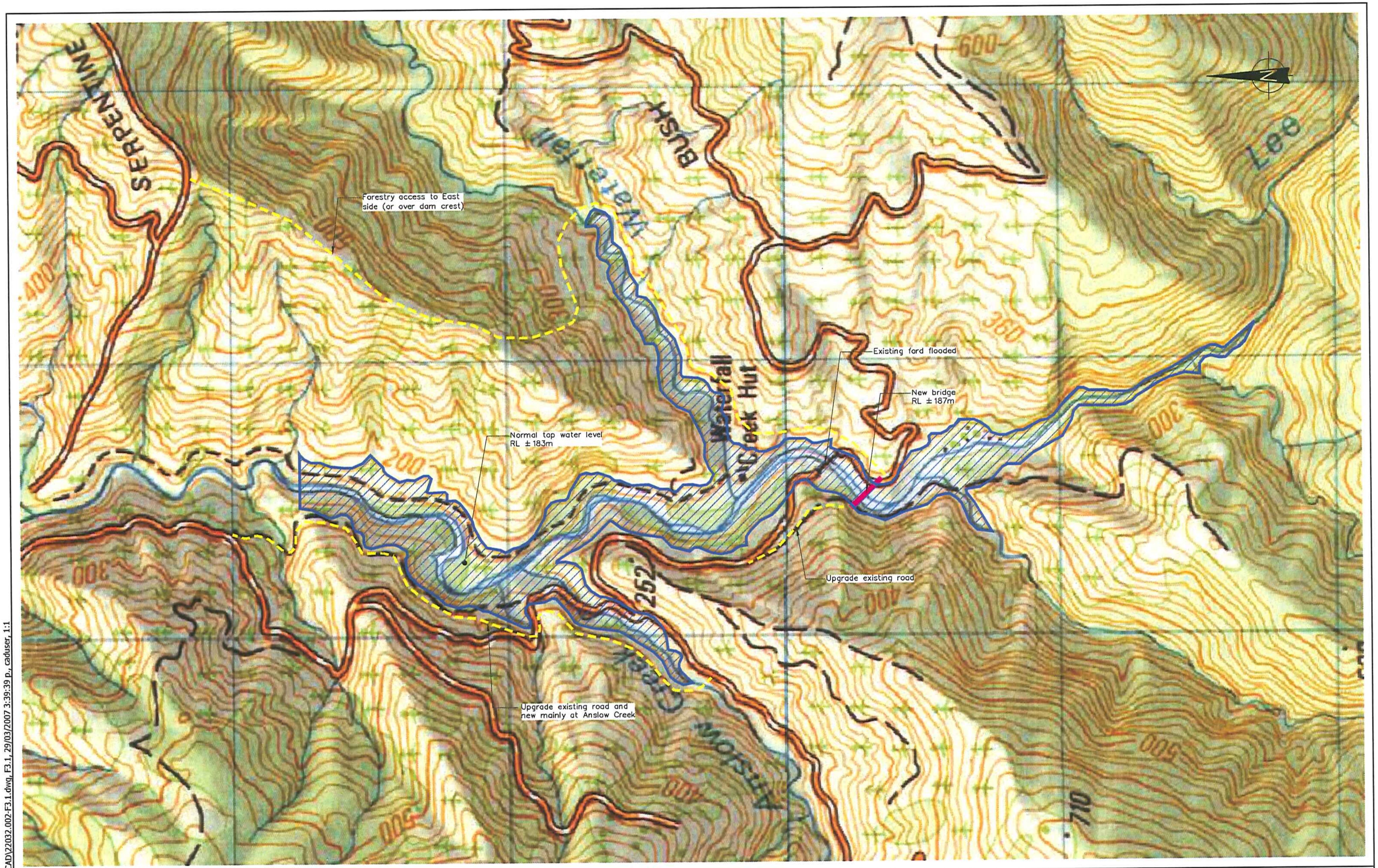
Further details of catchment hydrology are contained in the **Component 1 Report: Water Demand and Availability**.

3.3 Dam/Reservoir Characteristics

The brief received from WWAC following consideration of water demand and security of supply issues (see Component 1 Report) was to provide for a dam capable of creating a reservoir with storage capacity of 13 million m^3 of water (12 million m^3 live storage plus provision of 1 million m^3 dead storage).

The site is considered (on present information) to be best suited to earthfill dam construction.

Based on the 20 m contour data currently available, the indicated top water level is RL 183 m to store 13 million m^3 of water. However, more accurate survey data could result in a modest change in normal top water level. About 4 m (the amount depending on design optimisation of spillways) extra height will be required to enable passage of floods and provide safety freeboard to the dam. Thus the dam crest will be at about RL



SCALE 1: 12,500

0 100 200 300 400 500 600 700 800 (m)

NOTE:

1. Map sourced from Land Information New Zealand, Crown copyright reserved.

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LEE RIVER STORAGE (13 MCM)
INDICATIVE RESERVOIR
EXTENT AND ROADING

FIG. No. Figure 3.1

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187 m. The inferred stream level is at around RL 138 m, thus giving a total height for the dam of some 49 m. (Nelson City's Maitai dam, by comparison, is about 40 m high.)

After allowing for flood rise and a potential riparian/reservoir access margin, the plan area of the reservoir will cover around 90 hectares. For water quality reasons, vegetation below normal top water level will need to be cleared plus a small margin, prior to inundation.

Figure 3.1 illustrates the reservoir at normal top water level along with anticipated new roading to maintain access up valley and connections to existing main forestry tracks. The existing ford near the head of the reservoir may need to be replaced in some way and some 4.5km of new roads/tracks are required.

3.4 Geology and Geotechnical Conditions

3.4.1 Introduction

Tonkin & Taylor conducted a preliminary engineering geological investigation to provide information for a geotechnical prefeasibility study for the potential dam within the Lee River valley. Two potential dam sites were investigated, within the vicinity of grid square E2523000 N5971000 (NZMG). The locations of the two potential dam sites, labelled Site 1 and Site 2 can be seen on Figures 1 and 2 in **Appendix 5.**

Initially it was considered that Site 1 would provide a better layout. However during site investigation work, remnant gravel terrace materials of reasonably extensive depth were encountered in test pits on the true right (eastern) abutment, which led to a reconsideration of site. The downstream site (Site 2) is therefore the currently preferred site.

The investigations were undertaken during May and June 2006 at the two dam sites (Site 1 and Site 2) and two areas upstream from the dam sites where potential sources of fill material had been identified. The investigation involved geological and geomorphic mapping, subsurface pitting and Scala probing of the dam sites and potential borrow areas.

The location of the investigations is shown on Figures 1 and 2 attached in **Appendix 5**. The location and extent of the investigations at the dam sites was limited to that appropriate for prefeasibility, as well as by the dense vegetation and steep slopes in the area and further investigations will be required for full feasibility and preliminary design. The test pit logs, Scala penetrometer sheets and an engineering terminology sheet are included in **Appendix 5**. A description of the investigations carried out is as follows:

The investigation for Site 1 was undertaken over four days between 3 May and 6 June 2006. The investigation consisted of:

- A walk over survey of the potential dam site;
- Excavation and logging of 12 test pits;
- Excavation of three (3) shallow trenches/scrapes adjacent the river bed;
- Nine (9) Scala penetrometer tests at selected test pit locations.

The investigation for Site 2 was undertaken over four days between 20 and 23 June 2006. The investigation consisted of:

- A walk over survey of the potential dam site and abney profiling of the slopes;

- Excavation and logging of three (3) test pits;
- Face logging of the cuttings on the access tracks on and adjacent to the site;
- One (1) Scala penetrometer test at TP18.

The investigation for two potential borrow (fill source) areas, labelled F1 and F2 was carried out on 18 May 2006. The investigation consisted of:

- A walk over survey of the potential borrow sites;
- Excavation and logging of four (4) test pits;
- Two (2) Scala penetrometer tests.

3.4.2 General site description

The general area of the potential dam sites is covered in pine forest with access along logging roads. The site is at the foot of the Richmond Ranges and is characterised by steep sided valley slopes with patchy outcrops of rock and scree covered slopes. The basement rock of the area is described in more detail in Section 3.4.3 but is generally composed of altered mudstones and siltstones, generally termed 'Greywacke'.

3.4.3 Geological setting

A review of the geological setting of the dam site was undertaken by Dr Mike Johnston on behalf of T&T. A copy of his report is attached in **Appendix 5**. Figure 5 in **Appendix 5** shows the geological setting of the site and the location of the major faults in the region. The main findings of Dr Johnston's report were as follows:

- The regional geology of the area is Upper Mesozoic to Lower Paleozoic rocks dominated by sandstone, siltstone and mudstone sedimentary rock and minor igneous and metamorphic rocks;
- There are two major fault systems near the dam site, the Waimea-Flaxmore Fault System and the Alpine Fault;
- The Waimea-Flaxmore Fault System comprises a number of faults of which the closest is the Whangamoa Fault located 2.5 km west of the dam sites;
- The Alpine Fault is located about 23 km south-east of the dam sites. This is a major fault which delineates the boundary between the Pacific and Australian plates;
- No active faults have been located at the dam sites.

3.4.4 Dam site conditions

The steep slopes and thick vegetation on the site coupled with the limited prefeasibility scope of investigation severely limited access to undertake subsurface investigation particularly on the abutments of the dam sites.

The principal soil and rock units encountered at the sites are from oldest to youngest:

- Siltstone and mudstone (greywacke) bedrock of the Rai Formation
- Upper Terrace - sandy gravels
- Middle Terrace - gravels, sandy gravels, silty gravels and silty clays
- River Bank Terrace - sandy, silty and clayey gravels, sandy silts and silty clays
- Residual soil - gravely silts/silty gravels

- Colluvial slope deposits - gravely silts/silty gravels
- River bed gravels - coarse gravels, cobbles and boulders
- Fill - associated with access roads.

3.4.4.1 Site 1

3.4.4.1.1 Site 1 Surface conditions

The Lee River valley at this location (see Figures 1 and 3 in **Appendix 5**) has an asymmetrical profile consisting of a moderately steeply inclined (average 23°) eastern slope and a steeply inclined (average 33°) western slope.

There are three clearly defined terraces on the eastern slope: the River Bank Terrace which is approximately 4 m above the current river level (arl); the Middle Terrace at 15 m arl and the Upper Terrace at 40 m arl. The approximate extents of these terraces are shown on Figure 1 in **Appendix 5**.

The current river channel is relatively broad and armoured with rounded gravels, cobbles, boulders, with scattered outcrops of the underlying siltstone bedrock. Coarse sand and pebble beaches occur on the inside bends of the channel.

The river is fordable by foot under normal flow conditions and is less than 1.0 m deep at its wider stretches.

The Riverbank Terrace on the eastern slope has surface water flowing into it from feeder gullies and along the terrace until it exits into the river below. Marshy ground and lush vegetation indicate this area is usually wet.

The slope below the Riverbank Terrace on the western side of the valley drops away very steeply to the Lee River.

Hill slope vegetation on both the eastern and western slopes consists of indigenous forest and grassland for an approximate distance of 20 m to 40 m upslope of the river bank. Beyond this point the predominant species are gorse, broom, grasses and exotic plantation forest.

3.4.4.1.2 Site1 Subsurface conditions

The subsurface conditions encountered at Site 1 are summarised on geological section Figure 3 in **Appendix 5)**. Investigation logs are also attached in **Appendix 5**.

River Bed

Along both eastern and western margins of the current river bed, laminated to thinly bedded Rai Formation siltstones regularly outcrop at the surface. These siltstones exhibit extensive local folding. The bedrock is moderately to slightly weathered from the top of the rock surface, weak to moderately strong and closely jointed. Overlying the bedrock is up to 1.0 m of fluvial sand and gravel, cobbles and boulders (B axis up to 500 mm).

West Bank

The subsurface ground conditions on the western side of the Lee River can be divided into two sections, one above and one below the RL 150 m contour. Below RL 150 m, the general profile as seen in TP1 and TP4 to TP7 consists of a thin layer of leaf fall material overlying up to 1.5 m of fluvial silt, sand and rounded gravel. These materials are interpreted as river terrace gravel. Underlying these gravel are Rai Formation siltstones.

Above the RL 150 m contour subsurface conditions on the western side (TP2 and TP3) consist of up to 2.9 m of colluvial soils comprising silty and clayey, angular gravel, silty clays and clayey silts. This colluvium overlies bedrock.

Bedrock is folded, closely jointed and locally demonstrated flexure slip related shearing. A relatively low angle (32°) dextral shear (dipping at 284° into the abutment) was also identified on the road cut adjacent to TP3.

East Bank

On the eastern (right bank) side of the river, the River Bank Terrace deposits are alternating sands, silts, clays and gravel TP12 and TP16. These fluvial deposits can be in excess of 3.2 m deep.

Bedrock was only encountered in one test pit (TP 15) on the eastern side of the terrace at 2.5 m depth below surface level. TP 15 is located below RL 145 m on the slopes at the rear of the River Bank Terrace. A bedrock exposure was observed at shallow depth (< 0.5 m) (Location A, Figure 1 in **Appendix 5**). The siltstones at this location are bedded, closely jointed and folded; the fold was matched in a river bank on the opposite (western) side of the river (Location B, Figure 1 in **Appendix 5**).

Further up the left abutment, a second terrace referred to as the Middle Terrace is identified approximately 15 m above river level. TP13 on the terrace showed alternating gravels, sands and clays to at least 2.3m depth. A third terrace, the Upper Terrace is approximately 40 m above river level. TP14 was excavated on the terrace; it encountered 1.3 m of silty clay with gravel which is inferred to be colluvium from the slopes above. This is underlain by silty clay, gravely clay and sandy gravel to the limit of excavator reach at 3.2 m.

3.4.4.1.3 Site 1 Groundwater conditions

Groundwater was encountered in several of the test pits. On the eastern bank, seepage was encountered between 0.85 m and 2.5 m depth below surface level, in TP12 – TP14 and TP16. High inflow of approximately 0.5 litres per minute was observed in TP16 at 1.9 m depth below ground surface. These 4 test pits are all located below the RL 140 m contour.

On the western bank, seepage was encountered in TP1 0.65 m and 1.4 m and in TP4 at 1.3 m depth below ground surface. Higher inflows (between 2.5 and 5 litres per minute) were observed in TP4 and TP5 between 2.3 m and 2.4 m depth below ground surface. These test pits were all located below the RL 150 m contour on the River Bank Terrace.

Our observations suggest that there is perched groundwater in the surface soils and terrace gravels on both sides of the river. The permanent groundwater table is likely to correspond with river level at the toe of the slope and rise away from the river under both abutments.

The terrace deposits will be wet and require drying if used for fill. The wet conditions could cause saturation and softening of any fill placed over these areas.

3.4.4.2 Site 2

3.4.4.2.1 Site 2 Surface conditions

The Site 2 also has an asymmetrical profile (see Figures 1 and 4 in **Appendix 5**).

River Bed

The active river bed consists of a relatively broad channel with steep river banks. The western riverside bank is steeper than the eastern bank and bedrock outcrops along both banks of the river.

The river is fordable by foot under normal flow conditions 40 m upstream of Site 2, and is less than 1.0 m deep at its wider stretches. Its depth at the dam centreline has not been proved.

West Bank

The western (left) abutment slope above RL 143 m is a relatively uniform profile, and is moderately steep (average 27°) slope with localised steepening at terrace edges and minor slump scarps. There is evidence of small, shallow (less than 1.0 m deep) slumps. Below RL 143 m, the slope is steeply to very steeply inclined (30° to 60°) and bedrock outcrops at the surface.

East Bank

The eastern (right) bank is characterised by a steep scree covered slightly concave mid to lower slope. The average slope angle for this section of slope is approximately 36°. There is evidence of dilation and down slope creep of the near surface rock immediately under the surface soils in the investigation track on this abutment. Above RL 174 the slope flattens to a moderately steep 22°. The base of the slope is wet marshy ground with several small stream flows.

Hill slope vegetation on both the eastern and western slopes consists of indigenous forest and grassland for an approximate distance of 10 m to 40 m upslope of the river bank. Beyond this point the predominant species are gorse, broom, grasses and exotic plantation forest. This forest consists of mature pine trees on the western side of the river and immature pine on the eastern side.

3.4.4.2.2 Site 2 Subsurface conditions

The subsurface conditions encountered at Site 2 are summarised on the geological section (Figure 4 in **Appendix 5**). Investigation logs are attached in **Appendix 5**.

River Bed

The river occupies the entire river bed and there are rock exposures on the river banks on both sides. The rock is moderately strong bedded siltstone with bedding dipping towards the north-west.

West Bank

Due to access difficulties, no test pitting was undertaken on the western slope at Site 2. Observations on the existing road batter on the upper abutment indicate up to 3.5 m thickness of silty clay and clayey gravel colluvial soils overlying bedrock.

East Bank

TP18 at RL 132 m on the outer edge of the Riverbank Terrace revealed topsoil over 1.15 m of firm to stiff clayey silt colluvial material overlying up to 0.4 m of loose to medium dense, rounded, fluvial sandy gravel. Underlying the fluvial gravel at 1.7 m depth was weak to moderately strong siltstone bedrock.

TP19 was located at the rear of the Riverbank Terrace and was excavated into the toe of the slope above (between RL 131 m and RL 134.5 m). This pit was 4.5 m deep and did not encounter bedrock. Loose, angular silty and clayey gravel, was encountered between 0.1

m and 3.2 m depth below surface. This material is interpreted to be either scree or landslide debris. We observed smooth slickensided surfaces at approximately 3.7 m depth below ground surface. This indicates movement of the overlying layer. Underlying the scree/debris was a 0.4 m thick layer of wet, high plasticity clay interpreted as fluvial overbank deposits. The pit terminates in medium dense silty clayey, rounded gravel (terrace gravel).

The cut batters along the investigation access track between Sites 1 and 2 (see Site photographs P1 to P3) revealed a mix of bedrock derived colluvial soils and reworked fluvial terrace gravel. These overburden soils were generally thicker in the mid slope, and thinned sharply (to ≤0.5 m) above the RL180 m contour, along the margins of side gullies, and on spur and ridge lines. Bedrock observed in these batters consisted of highly weathered, closely jointed sometimes shattered, closely folded (inter limb angle 50° to 61°), weak to moderately strong, laminated and thinly bedded siltstones. Photographs P10 and P11 show down slope displacement of the soil and near surface rock along a prominent set of down slope dipping joints. Joints were commonly open, with up to 10 mm separation. Some of this dilation is digger induced. Trees on this slope are often bowed indicating down slope creep of surface scree material. The depth of this movement has not been confirmed but is believed to be shallow.

TP17 at RL 200 m showed topsoil overlying up to 0.65 m of completely to highly weathered, angular, clayey silty gravel interpreted as a residual soil. Below the residual soil layer at 0.9 m depth, a very weak, blocky, highly weathered siltstone was encountered. This siltstone became more competent and less weathered with depth. The bedding in the bedrock at this location was orientated 104°/76°NE (strike and dip). The rock is closely jointed and joints are open and are infilled with silt.

There is evidence of creep movement on the slope below RL 180 m and the near surface slope materials are considered to be marginally stable.

3.4.4.2.3 Site 2 Groundwater

Groundwater was not encountered in the Site 2 investigation, although on the eastern slope wet soils were observed below 2.9 m in TP19. No test pitting was conducted on the western slopes of Site 2, so no subsurface groundwater information is available.

A similar permanent groundwater table is inferred as at Site 1.

3.4.5 Construction materials

A preliminary assessment has been carried out to identify potential sources for:

- Bulk fill
- Low permeability/core fill
- Transition zone material
- Armour rock/rip rap
- Drainage aggregates.

3.4.5.1 Bulk/general fill

The underlying greywacke bedrock will provide a suitable source of bulk fill. Test pits and exposures indicate a thin layer of extremely weathered/highly weathered rock usually less than 1 m in thickness which will provide well graded gravely silt and silty gravel fill. With depth, the rock will become less weathered and harder and provide more granular (rocky) bulk fill. Deep borrow excavation may encounter slightly weathered or unweathered rock which will require ripping or blasting.

3.4.5.2 Low permeability fill

The investigations have focussed on identifying suitable surface soils for low permeability/core material. Two potential borrow areas (F1 and F2 on Figure 2 in **Appendix** 5) were specifically investigated as a source for the low permeability fill material for the dam. In addition two other potential low permeability sources were identified in cut batters along the haul road in the vicinity of the west (left) abutments of the two potential dam sites. These areas are identified as F3 and F4 on Figure 1 in **Appendix** 5.

The subsurface conditions encountered at the borrow areas F1 to F4 are as follows:

3.4.5.2.1 Fill area F1

Site F1 is located on the side slopes on the west/left bank of a ridge upstream of the dam sites. It consists of a gentle to moderately steep (15-22°) slope. This ridge has been modified by formation of a logging access road and is vegetated in young pine forest. The logging track showed patchy outcrops of rock within varying thicknesses of residual soil and colluvium.

Two test pits showed varying ground conditions. The upslope test pit (TP8) revealed a 0.6 m layer of clayey gravel side-cast material overlying up to 1.1 m of low plasticity silty clay with root contamination throughout the layer which could make the material unsuitable as core material. This layer is assessed to be residual soil derived from the underlying Rai Formation siltstones which were encountered from 1.7 m depth. The siltstone was highly weathered, very weak and excavated as clayey gravel.

Test pit 9 (down slope) revealed 0.5 m of low plasticity silty clay side-cast material overlying 0.7 m of low plasticity gravely clay colluvial materials. Underlying the colluvium was fluvial clayey gravel to at least 3.2 m depth. This unit was predominantly medium to coarse gravel with cobbles and large boulders up to 1.5 m.

3.4.5.2.2 Fill area F2

Site F2 has a slightly broader and gentler profile than F1 and consists of an elongate gently inclined slope (average 12°) extending along the ridge above the road batter to a point approximately 125 m upslope of the road batter.

The two test pits (TP10 and TP11) were in the road batter and on top of the ridge respectively. The test pits showed approximately 0.7 m thick layer of silty clay with minor gravel below the topsoil. Underlying these surficial soils was a 0.3 m to 0.9 m thick layer of very weak, highly weathered siltstones. The highly weathered siltstones graded into weak, moderately weathered siltstones at 1.9 m and 1.0 m depth respectively and were thinly bedded, closely jointed and blocky. These siltstones excavated as angular, coarse silty gravel.

3.4.5.2.3 Fill areas F3 and F4

Sites F3 and F4 are located on the western (left) bank of the Lee River at Site 2 and Site 1 respectively. The hill slopes at these locations are moderately steep (26° average), forested (down slope of the haul road) and clear felled (upslope of the haul road) slopes.

Access to the sites is via the existing forestry haul roads.

At the F3 and F4 locations between 2.5 and 4.5 m of low plasticity colluvium and residually weathered siltstone was observed in cut batters along the road. In TP3 at F4, the residual soil was highly weathered siltstone. No test pit excavations were conducted at the F3 location.

3.4.5.2.4 Approximate low permeability fill volumes

From the limited test pits and observations along the track batters, we have made an estimate of the likely quantities of fine grained material available for dam construction (see Table 3.1 below) which, subject to later confirmation, are expected to provide sufficient material for the envisaged dam. *These values are preliminary estimates only and further investigations are required to verify these volumes and the permeability/suitability of these soils.*

Table 3.1: Estimated Quantities of Low Permeability Fill

Borrow Area no.	F1	F2	F3	F4
Available Material Volume (m^3)	8,000	29,000	65,000	23,000

Note 1: This assumes 100% recovery.

Careful earthworks will be required to recover this low permeability fill. This will involve removing topsoil and root contaminated surface soil then scalping the thin zone of surface soil and extremely weathered from the less weathered rock below. *This will probably not be practical on slopes steeper than 3H: 1V without progressive blading to temporary stockpiles downslope. Given the above it is expected that 40% to 80% of the material will be winnable depending on the stripping and excavation methods used.*

3.4.5.2.5 Recommendations

Based on the results of our preliminary field investigations we consider that further investigation of low permeability fill sources should initially concentrate on the F3 and F4 sites.

Laboratory testing should be carried out on residual soil and extremely weathered rock from the Fill Sites prior to confirming their suitability as sources of core material.

3.4.5.3 Transition zone material

A source of fill is required with a grading midway between the low permeability/core fill and the general bulk fill. The grading of this material is critical to ensure compatibility and prevent piping.

The transition zone material will be derived from either locally screened durable terrace deposits/rock or imported gravel/rock. *Laboratory gradings will be required to confirm the compatibility of these materials with the other fill types.*

3.4.5.4 Armour rock/rip rap

An assessment was made as to the likely quantities of gravels available for use as rip rap. This is based on two methods: a sample count of the number of cobbles and boulders visible in the river bed; and an estimation of the total volume of gravel in the vicinity of the dam site.

Two distinct sources of rip rap have been identified:

- The current riverbed ;
- The Riverbank, Middle and Upper Terraces identified on the eastern (right) slopes of the valley.

(The approximate extent of these terraces is shown on Figure 1 in **Appendix 5**.)

3.4.5.4.1 River bed boulder count

During our site walkover a visual assessment of the number of small boulders and cobbles was undertaken on a typical 2 m x 2 m section of the river bed. A similar count over a larger 10 m x 10 m section of the river bed was also undertaken for the larger boulders (B-axis >400 mm). These quantities have been extrapolated for the 250 m x 40 m section of river bed in the vicinity of the dam sites, and assumes similar distribution of cobbles and boulders to that of the chosen area. The results of this survey are summarised in Table 3.2 below:

Table 3.2: Number of cobbles and boulders in riverbed

Material Type	Number of Large boulders (≥ 400 mm/10,000m^2)	Number of Small boulders (≥ 200 mm/10,000m^2)	Number of Large cobbles (≥ 100 mm/10,000m^2)
River bed	2,100	40,000	87,500

These rocks were predominantly strong to very strong. However a proportion (10 to 30%) of fissile rocks are present. These rocks have planes of weakness parallel to bedding and the more fissile rock will slake and breakdown. *The thickness of the rip rap layer may need to be increased to allow for deterioration of a proportion of the rock.*

3.4.5.4.2 Total volume of river gravels

The total volume of potential rip rap material was also estimated. The areal extent of river gravel material was estimated from our site walkover (see Figure 1 in **Appendix 5**) and the depth of the material was estimated from the test pits. The total volume of gravels (assuming 100% recovery) is estimated to be as follows:

- River bed and Riverbank Terrace 20,000 m^3
- Middle and Upper Terraces 3,000 m^3

3.4.5.5 Drainage aggregate

Drainage aggregates could be selected and screened from the river bed or terrace gravel deposits or sourced from a local quarry.

3.4.6 Geotechnical design issues

The following geotechnical design issues have been identified at the dam sites.

3.4.6.1 Earthquake shaking

A review of the earthquake risks was undertaken by Dr Mike Johnston (see attached in **Appendix** 5). Dr Johnston's main findings are as follows:

- On the Waimea-Flaxmore Fault System the maximum credible earthquake is likely to be M=6.9 and M=7.1 (Richter Scale). This would result in ground shaking as measured on the Modified Mercalli Scale of MM=IX to MM=VIII.
- On the Alpine Fault the maximum credible earthquake is likely to be in the range M=7.3 to 7.7 (Richter Scale) and the level of ground shaking would be similar to the Waimea-Flaxmore Fault with MM=VIII to IX.

We have separately assessed earthquake risks for proposed developments in Nelson and for a M=7 earthquake event in Nelson, peak ground accelerations of 0.7 g are considered likely. A similar or higher level of ground acceleration can be expected at this site.

A site-specific seismic hazard assessment of the dam has not yet been carried out and likely peak ground accelerations for this site will need to be assessed.

3.4.6.2 Slope instability

3.4.6.2.1 Natural slopes

Our investigations to date have highlighted that the right bank of Site 2 within the dam footprint is underlain by scree and possibly landslide deposits. The scree/landslide extends approximately 250 m along the river bank and the highest point is 60 m above the river level. It covers the entire right abutment of Site 2 and extends onto the downstream abutment of Site 1. Figure 4 in **Appendix 5** illustrates inferred foundations at the nominal dam centreline.

The toe of the scree/landslide overrides the River Bank Terrace and there is evidence of displacement in this area (TP19). The surface soils on the abutment are inferred to be marginally stable. The depth of this material has not been established.

The upper slopes of the right bank at Site 2 show evidence of creep displacement along down slope dipping defects in the near surface rock. This needs further investigation and will need to be considered in the design of both temporary and permanent excavations in rock.

It is possible that other parts of the dam reservoir will be blanketed with scree/landslide deposits which have not yet been investigated. We have identified on aerial photographs at least four areas within the proposed reservoir where the geomorphic expression is similar to the dam Site 2 right abutment. Further geotechnical investigations will be required to assess the risk of slope instability within the reservoirs.

3.4.6.2.2 Cut batters

For preliminary design purposes the following permanent batter slopes are recommended. The right abutment of Site 2 will require careful investigation because of the evidence of down slope creep in the surface soils and near surface rock.

Soil Type	Maximum Batter Angle
Surface soils	1.5H: 1V up to 3m. Specific design greater than 3m.
Rock	1H: 1V up to 5m. Specific design greater than 5m.

3.4.6.3 Dam foundations

3.4.6.3.1 Compressibility

It is expected that the foundations of the dam will be taken down through compressible surface soils including residual soils colluvium river terrace gravel deposits and river bed gravels and founded on bedrock. Any compressible zones within the rock, i.e., crushed or sheared zones are likely to be of limited lateral extent. Consolidation will be limited to closure of dilatant defects and most of this will occur during construction. Compressibility of the foundation materials is therefore not considered to be an issue.

3.4.6.3.2 Foundation permeability

The right bank of dam Sites 1 and 2 are underlain by highly permeable materials consisting of scree/landslide debris, river bed and river terrace gravels. The preliminary design assumes these materials will be removed.

The right bank of dam Site 2 is underlain by an unknown depth of scree/landslide debris. Surface exposures indicate this to be poorly-graded, loosely packed silty gravel derived from the Rai Formation Siltstones. This material is likely to be highly permeable.

The coarse gravel, cobble and boulder deposits on the current river bed are highly permeable.

The right bank of dam Site 1 is underlain by river terrace gravels. These terraces are greater than 3.2 m thick in places and will require sub excavation to the bedrock to achieve an adequate cut-off. The depth of the terraces was not proven during our investigations and further investigation will be required to establish the depth of terrace deposits. Sub-excavation of the materials will be required at this dam site to control leakage.

In the valley floor, the bedrock exposures appear relatively tight. However, bedding strikes upstream/downstream and there may be bedding planes or crushed/sheared zones parallel to bedding along which groundwater flow could occur. This will need to be assessed by detailed investigation of the rock structure at the selected dam site.

There is also evidence of dilation in the bedrock on the abutments of the dam probably due to a combination of stress relief and down slope creep. The extent and depth of these zones will need to be established and the cut off/core of the dam will need to be excavated through this zone to achieve an effective cut off. In addition, provision will need to be made for assessing whether foundation grouting will be required.

3.4.6.4 Fill materials

From our preliminary assessment of the site, fill materials could be obtained from the following sources:

- General fill: It is expected that the spurs at any of the borrow sites could be quarried to produce a well graded fill material and rockfill.
- The local surface soils and the extremely weathered siltstone are a potential source of low permeability core fill material. The suitability of these materials needs to be confirmed. *Careful consideration also needs to be given as to how these materials will be separated and recovered.*
- The riverbed and terrace deposits will be suitable for rip rap but will need to be screened and sorted.

- Screened river bed or terrace deposits may be suitable for use as drainage aggregates. Grading tests will need to be performed to establish suitability. Alternatively drainage aggregates can be sourced from nearby quarries.

Further investigations will need to be carried out to confirm the suitability and quantity of these potential sources of fill materials.

3.5 Preliminary Layout

Figure 3.2 illustrates the preliminary layout that appears at this stage to be best suited to the site.

The preliminary general layout has features similar to those adopted for Nelson City's Maitai Dam. However, diversion and flood flows for the Lee site are larger than those at Maitai. Another point of difference is the capacity of the supply release (consumptive plus environmental) which at around 2m³/s for the Lee, which is higher than at the Maitai Dam. It should be noted that the preliminary layout (and associated costing) does not provide for any additional large controlled release, which may possibly be required for the likes of flushing macrophytes from the riverbed in prolonged dry spells.

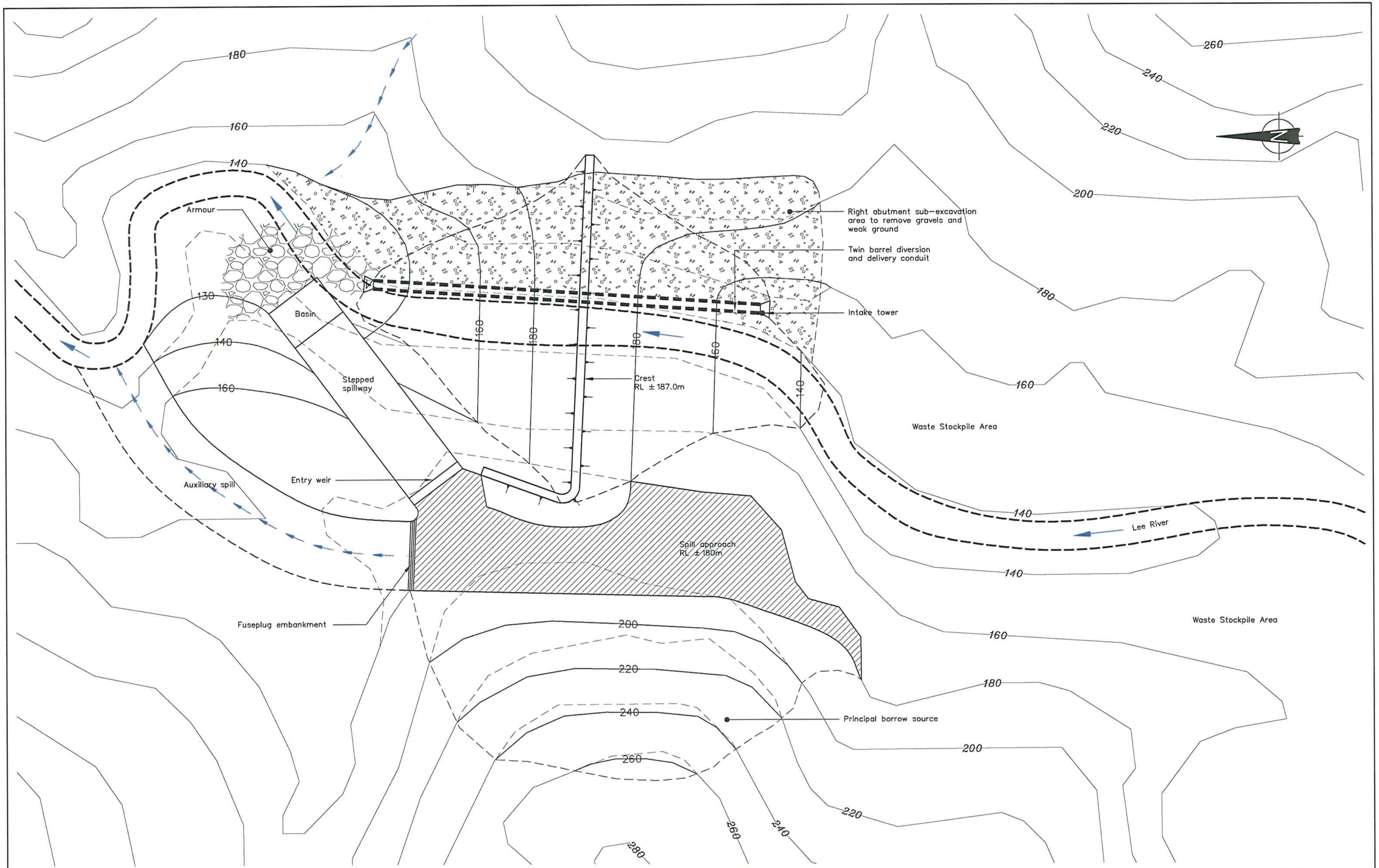
Principal components of the dam works are:

- a diversion culvert with twin barrels each around 2.4 m wide x 3.6m high on the right bank terrace, likely founded on rock or densely compacted fill after subexcavation of terrace gravels, initially to pass diversion flows during construction, and later with one barrel carrying controlled discharges
- an intake tower with entry ports at about 8-10 m vertical intervals to allow highest quality water to be selected for controlled discharges (note that final requirements will depend on design standards set in consents and detailed analysis involving coffer dam heights and available flood attenuation via upstream storage)
- the zoned embankment dam structure, including internal filters and drainage and wave armour or riprap over higher levels on the upstream face
- a left abutment concrete-lined stepped spillway closely similar to that used on the Opuha irrigation dam, with terminal basin, directing flow back into the river via an armoured zone and taking flow from up to about a 100 year flood event prior to calling on the auxiliary spillway
- a left abutment auxiliary spillway channel with fuse plug embankments excavated at 1.25:1 slope up the ridge line (matching steepest natural slopes locally), the excavated material going into the embankment.

3.6 Construction Materials – Requirements and Potential Sources

The principal requirements for construction materials are as follows:

- concrete for structures, some 3500 m³
- dam fill:
  - filter material: approx 15,000 m³
  - riprap: approx 9000 m³
  - low permeability fill: approx 45,000 m³



DRAWN	EAA	Apr.07
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APPROVED		
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SCALES (AT A3 SIZE) 1:2000		
PROJECT No. 22032.002		

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LEE RIVER STORAGE (13 MCM)
SCHEMATIC LAYOUT OF DAM

FIG. No. Figure 3.2

REV. 0

- bulk fill: 450,000 m^3

Figure 3.3 provides an indicative embankment section which illustrates the different earthfill materials.

3.6.1 Material sources

Section 3.4.5 commented on the characteristics of various material types and likely sources. From the preliminary investigations it appears that the terrace gravels at the right abutment (except perhaps as a natural drainage zone under part of the downstream shoulder of the dam) will need to be removed to provide a suitable foundation. Slip debris here will also need to be removed.

Based on the preliminary investigation work, these gravels appear to have potential to be processed and used as filters in the dam. However, after detailed investigation, that may prove not to be the case and a proportion of the required filter material may need to be imported from established quarries. This possibility has been factored into preliminary costing.

There is judged to be sufficient boulder material within the river bed at and upstream of the dam site, to satisfy riprap needs.

As shown on Figure 3.2, there is a substantial excavation required at the left abutment to provide for spillways and this area will be the prime source of borrow material (particularly bulk dam fill) for the dam embankment. The weathered veneer here will provide a source of low permeability material (possibly involving some blending with less weathered material), with the balance obtained from fully proven borrows as identified in the geotechnical investigations. These indicated in the order of 125,000m^3 gross of low permeability material available, compared to 45,000m^3 required.

It is expected that aggregates for concrete will be imported from established quarries, but depending on the quality of the right abutment gravels and extent of processing required for them, a contractor may consider using the local gravels and establishing a concrete plant on site.

3.7 Preliminary Operating Regime

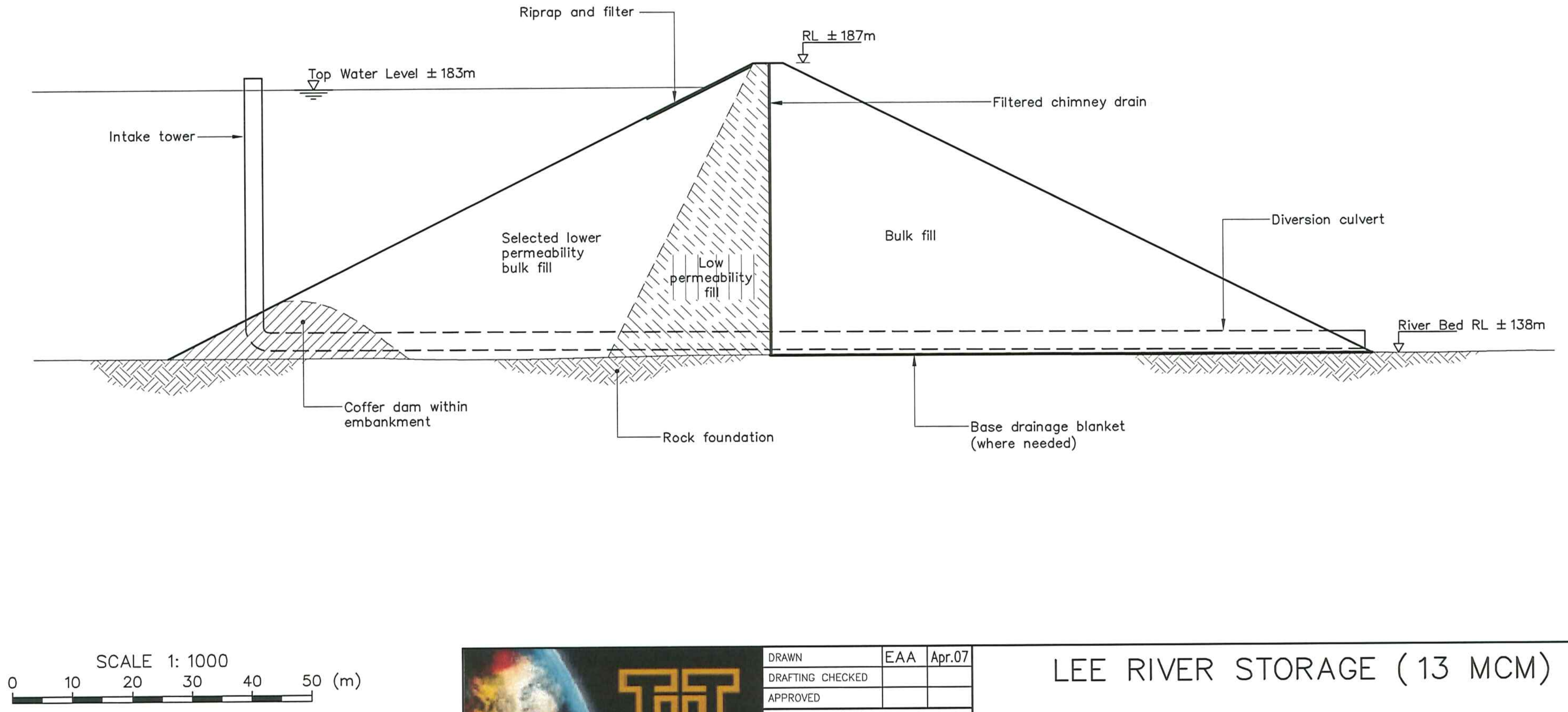
A preliminary assessment of the likely operating regime has been undertaken, and is reported on in the Component 1 report.

3.8 Potential Effects on Existing Infrastructure

Site access up the Lee Valley is relatively straightforward and should present few difficulties for safe operation of construction traffic. Some upgrading of the final 3km of private access roading may be required.

TDC's Manager Property reports (April 2006) that there is legal road reserve extending on both sides of the river along the extent of the reservoir, including the main tributaries of Anslow Creek and Waterfall Creek. These roads would need to be stopped and would become esplanade reserve, provided that agreement could be reached with the affected parties. However legal access to several properties would disappear and the practicalities of obtaining alternative access would need to be investigated in later stages of the project.

Based on current information the following formed roads would potentially be affected by the proposed storage:



DRAWN	EAA	Apr.07	LEE RIVER STORAGE (13 MCM)
DRAFTING CHECKED			INDICATIVE EMBANKMENT SECTION
APPROVED			
CADFILE : 22032.002–F3.3.dwg			
SCALES (AT A4 SIZE) 1: 1000			
PROJECT No. 22032.002			FIG. No. Figure 3.3 — REV. 0

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- Approximately 4-5km of formed road on the true left side of the river. However this road is largely private access to forest land, with public access being restricted by locked gate.
- Anslow Road - approximately 500m of this forestry road would be potentially affected by the reservoir. This road provides access to forestry land, and to Bush Road. The latter potentially provides access to Richmond Forest Park.
- Ford and forestry access road upstream of Waterfall Creek

3.9 Alternative Distribution System – Piped Delivery

An assessment has been made of the potential for piping the abstractable flow from the dam to the existing Waimea East Irrigation Scheme pump station at the end of the Wairoa Gorge. This piped flow has been assessed at being approximately 1 m^3/s excluding environmental and groundwater component.

The purpose of this assessment is to put this possibility into perspective, relative to releasing all flow into the river at the dam and then recovering that part of the demand not delivered through the groundwater system.

The static head available has been assessed as being 130m (calculated from the lowest drawoff level being at about RL 155m and the delivery point at about RL 25m). It has been assumed that the pipeline substantially remains in the road reserve, apart from a more difficult river edge alignment near the old cement works where the road is locally high. There would also be a need to cross the Lee River at the bottom end.

To keep the pipeline below the hydraulic grade line, the controlling friction gradient for the pipeline would be about 1 in 88 over the upstream 5 km of pipeline. To deliver 1 m^3/s, a pipeline of about 750mm diameter is required for the first 5 km, reducing then to 600mm diameter.

3.10 Construction Period

It is estimated that the construction period would be at least two years. To achieve the shortest construction period, diversion works would need to be built prior to the first summer earthworks season and timing arranged to allow full earthworks seasons.

3.11 Potential for Electricity Generation

There is potential for generating electricity as part of the scheme. The preliminary assessment of hydro potential is based on:

- For peak output, net head when the reservoir is full and an eyeball estimate of the economic peak generation flow based on flow duration characteristics and prior studies undertaken elsewhere
- Energy based on expected economic plant factor, tempered by long-term "average" head which is influenced by drawdown of storage for consumptive use, again drawing on prior detailed studies elsewhere and modelling of reservoir level duration.

In drought conditions the reservoir will be drawn down too low to meet the turbine design characteristics, so while most of the controlled release can pass through the generator (assuming irrigation pick-up is all downstream), a small proportion will not. On the other hand, when the reservoir is full and spilling, the generator can operate continuously at full capacity.

The indicative economic potential has been assessed as being 1.2 MW capacity and 6.8 GWh/annum average.

4 Preliminary Dam Breach Hazard Assessment – Site 11 Lee

4.1 Introduction

4.1.1 Purpose of study

A preliminary dam breach analysis was undertaken by Tonkin and Taylor based on the proposed dam site, dam type, and size (including reservoir capacity) for Site 11 Lee River.

As a preface, it is worthwhile stating the purpose and significance of carrying out a dam breach analysis for any proposed dam: such analyses are undertaken within the dam industry primarily to help assessment of downstream hazard potential, which in turn guides the setting of standards to adopt for dam design, construction, operation, and ongoing monitoring and maintenance. The analyses are hypothetical and entirely divorced from the chances of a dam failure ever occurring.

Another reason in the New Zealand context is that the Resource Management Act requires consideration of an effect of low probability but high potential impact. The key point which has to guide any decision under the Resource Management Act is the low probability of occurrence, and ensuring that the probability is indeed extremely small related to the degree to which the potential impact is "high". Consent conditions can be specified (i.e. adoption of NZSOLD Guidelines) to achieve this objective and meet the requirements of the Act.

This analysis has been undertaken for the above purposes.

4.1.2 Potential impact classification

The current issue of the New Zealand Dam Safety Guidelines (November 2000) adopts a potential impact classification (PIC) system to determine the appropriate design standards for the dam (for earthquake loading and safe flood passage) and the level of rigour applied to site investigations, construction, commissioning and on-going maintenance and surveillance. The consequences of failure, specifically the downstream harm and damage potential, are the main determinant for assessing the potential impact classification. Table 4.1 shows the definitions of Potential Impact Category adopted by the Guidelines.

Table 4.1 **Potential Impact Categories for Dams in Terms of Failure Consequences (NZ Dam Safety Guidelines, Nov 2000)**

Potential Impact Category (PIC)	Potential Incremental Consequences of Failure	
	Life	Socio-economic, financial & environmental
High	Fatalities	Catastrophic damages
Medium	A few fatalities are possible	Major damages
Low	No fatalities expected	Moderate damages
Very Low	No fatalities	Minimal damages beyond owner's property

Interpretative details on definition of the potential impact categories and their significance in terms of dam safety requirements are contained in "Regulations for the Dam Safety Scheme: Discussion Document" released by the Department of Building and Housing in May 2006. Figures 6 and 7 from this document, reproduced below as Tables 4.2 and 4.3 respectively, provide an interpretation of both the classification categories and descriptors of 'catastrophic', 'major', 'moderate' and 'minimal' damages. It should be noted that these tables are currently going through some change but they provide a snapshot indication of likely categories.

Table 4.2 Incremental Consequences for PIC Categories for Use in Dam Classification Regulations

Population at risk (PAR)	Severity of damage and loss			
	Minimal	**Moderate**	**Major**	**Catastrophic**
0	Low	Low	Medium	High
1-10	Low (see notes 1 and 3 below)	Low (see notes 3 and 4 below)	Medium (see note 4 below)	High
11-100	(see note 1 below)	Medium (see notes 2 and 4 below)	High	High
More than 100		(see note 2 below)	High	High

The shaded area indicates the classification or PIC that should be chosen.

Note 1: With a PAR of five or more people, it is unlikely that the severity of damage and loss will be 'minimal'.
Note 2: 'Moderate' damage and loss would be unlikely where PAR exceeds 100.
Note 3: Change to 'medium' PIC where the potential for one identifiable life being lost is recognised or where the loss of itinerant lives is reasonably likely.
Note 4: Change to 'high' PIC where it is reasonably likely two or more non-itinerant lives will be lost.

In the Discussion Document, the population at risk is defined as all those people who would be directly exposed to flood waters within the natural flood or dam break affected zone if they took no action to evacuate. An inundation depth of 0.3 metres or higher can be used as an indication of the area where population is at risk. This definition allows the incremental population at risk to be considered for flood conditions.

The Discussion Document states that in estimating the population at risk, consideration needs to be given to:

- groups of dwellings
- camping areas and occupancy rates
- allowance for itinerants, such as authorised fishers, trampers, picnickers, casual visitors or people travelling across the floodplain

- river crossings and bridges
- occupation of schools, factories, retirement homes, hospitals, institutions, and commercial and retail areas.

Table 4.3 Incremental Damage Descriptors Associated with Table 4.2 for Use in Dam Classification Regulations

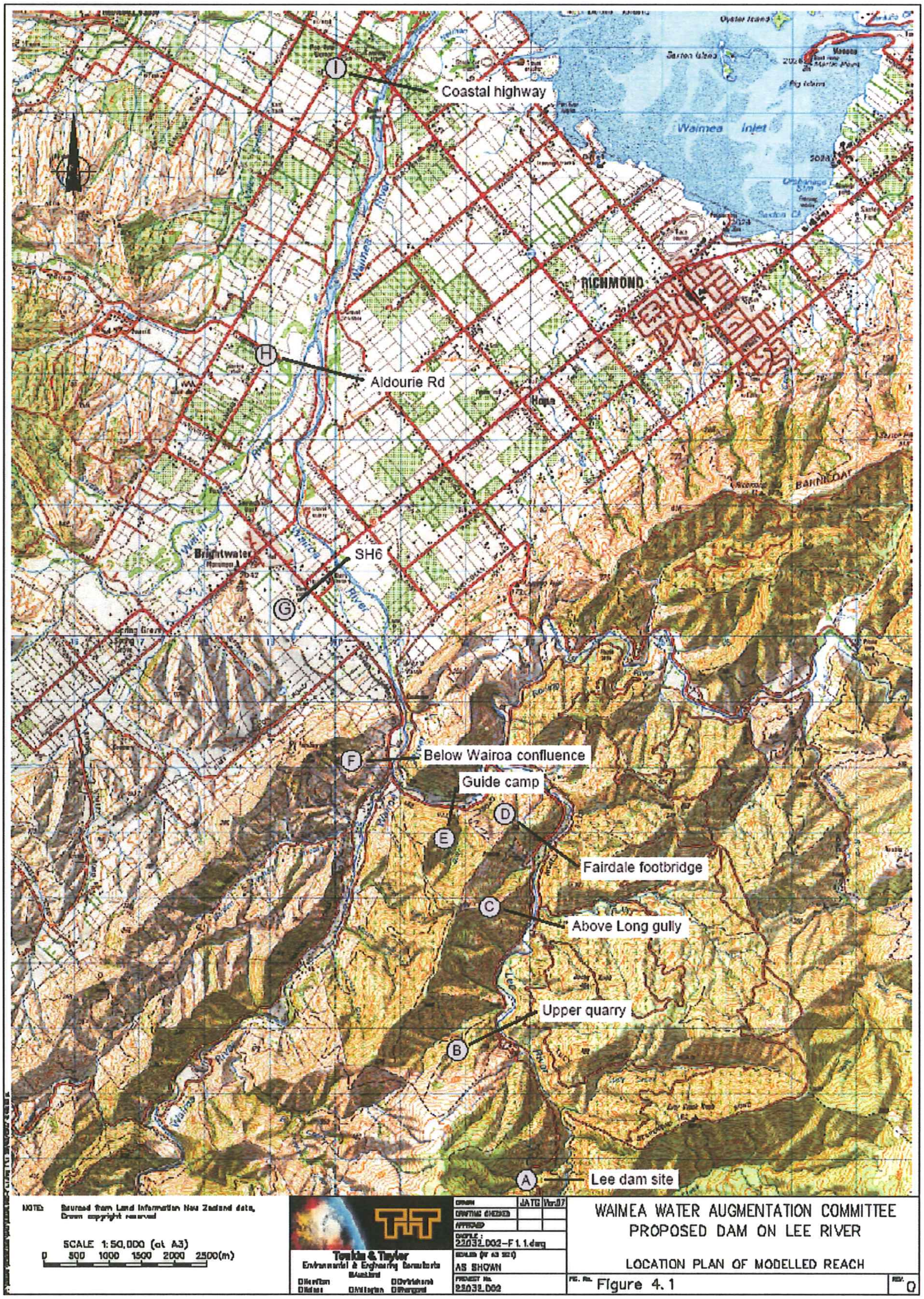
Descriptor	Residential	Costs: socio-economic and financial	Environment	Recovery time
Catastrophic	More than 50 houses destroyed	Greater than $10 million	Permanent widespread ecological damage	Many years
Major	4-49 houses destroyed and a number of houses damaged	$1-10 million	Heavy ecological damage and costly restoration	Years
Moderate	1-2 houses damaged	$100,000-$1 million	Significant but recoverable ecological damage	Months
Minimal	No damage	Less than $100,000	Short-term damage	Days to weeks

4.2 Background to Lee Dam Breach Analysis

The following sub-sections describe an approximate and preliminary assessment of the potential flood hazard in the event of a breach of the potential storage dam on the Lee River, which is a tributary of the Wairoa and Waimea Rivers.

The dam site assumed for this phase of investigations (Site 11 Lee) is located about 11.5 km upstream of the confluence of the Lee and Wairoa Rivers, and about 12.5 km upstream of the Wairoa Gorge. Brightwater and Wakefield are the closest towns and lie to the north and northwest of the dam site respectively. Figure 4.1 is a plan showing the site location and surrounding area.

The dam currently being considered is an earthfill embankment which would impound a full storage of 13 million m^3 with a normal top water level of about RL 183 m, which is about 46 m above the river bed level. This current analysis is based on those essential characteristics.


Coastal highway
Waimea Inlet
RICHMOND
Aldourie Rd
Brightwater
SH6
Below Wairoa confluence
Guide camp
Fairdale footbridge
Above Long gully
Upper quarry
Lee dam site
A
B
C
D
E
F
G
H
I
NOTE: Sourced from Land Information New Zealand data, Crown copyright reserved
SCALE 1:50,000 (at A3)
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Environmental & Engineering Consultants
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WAIMEA WATER AUGMENTATION COMMITTEE
PROPOSED DAM ON LEE RIVER
LOCATION PLAN OF MODELLED REACH
Figure 4.1
0

4.3 Modelled Dam Breach Scenarios

As is normal practice for dam breach analyses, a "sunny day" failure scenario (resulting from an event such as an extreme earthquake) rather than a flood-induced failure, has been considered. In terms of incremental damages from a dam failure, the former has significantly greater potential consequences. Part of the reason for this is that there is little or no warning of the sunny-day event, whereas the flood-induced failure occurs in the context of a major meteorological event with consequently more warning and heightened awareness for potential dam failure. Warning and evacuation time can dramatically influence the loss of life in such an event. Procedures developed by the US Bureau of Reclamation based on case history indicate that the loss of life can vary from 0.02% of the population-at-risk when the warning time is 90 minutes to 50% of the population-at-risk when the warning time is less than 15 minutes.

Despite this, given the extensive stopbanking in the lower reaches of the river (Waimea River floodplain), a dam failure induced by an extreme flood event could result in significant incremental damages (i.e. damages over and above that caused by the flood by itself without the dam failing). That is, the flow surge released on dam failure added to a natural flood flow would likely result in overtopping and breaching of the stopbanks. This secondary or consequential failure scenario has not been considered in the current assessment but should be investigated in subsequent more detailed hazard studies in subsequent phases of the overall investigation programme.

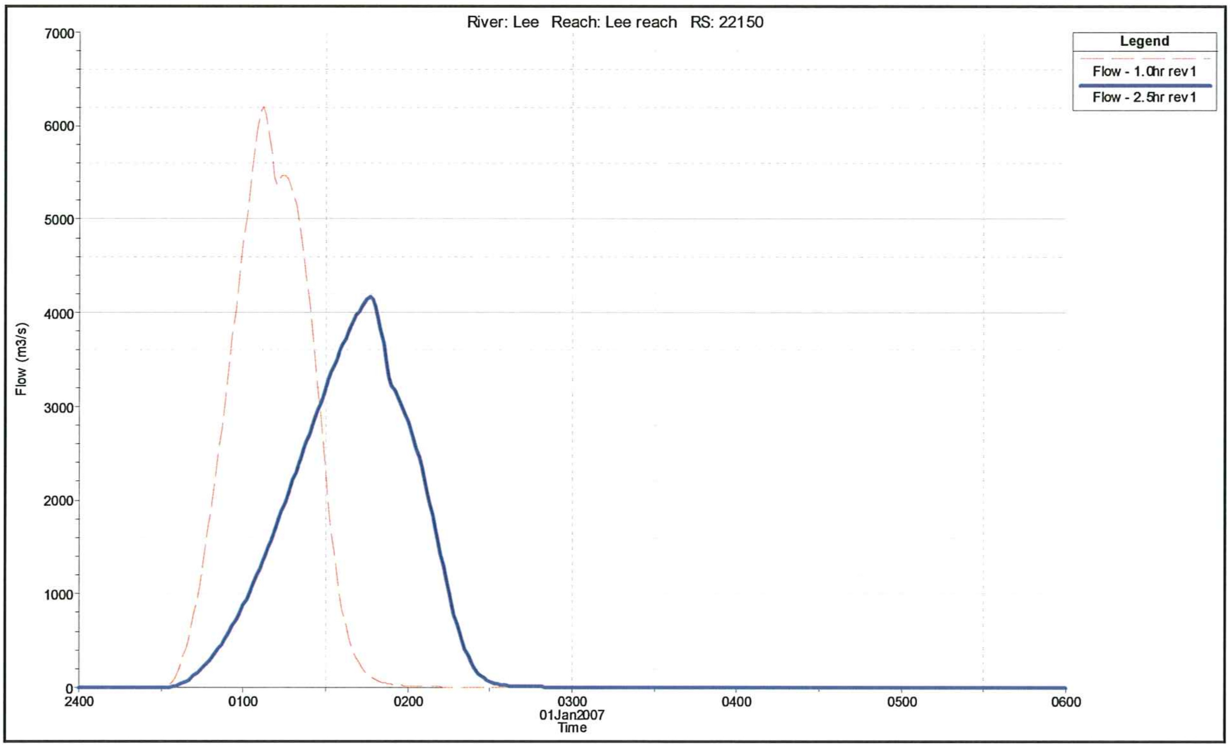
For this phase of the investigation, two breach options have been considered, both based on a trapezoidal breach in the dam wall over the full height of the dam, with 0.5H : 1V side slopes and a 30 m bottom width (full river bed width). The first, more severe scenario (Upper Bound Case) assumed that the period to develop the full breach was 1.0 hours. This is a very short time (worst-case) for a well-constructed earth embankment built using from high strength greywacke material as proposed here. The Base Case scenario considered a more moderate (and perhaps more credible) failure scenario, which assumed the full breach developed over 2.5 hours. The reservoir was assumed to be full but not spilling at breach initiation. These two breach scenarios are expected to encompass the potential flood wave from a full reservoir in the event of dam failure.

The Upper Bound Case (1 hour failure) resulted in a peak discharge at the base of the dam of 6,210 m^3/s. In the Base Case (2.5 hour failure) the simulated peak flow was about a third lower at 4,170 m^3/s.

In comparison, estimates of peak dam outflow using empirical formulae range widely from 2,700 m^3/s up to 10,000 m^3/s depending on the method used. Excluding the outliers, the majority of empirical estimates lie between 4,000 m^3/s and 7,000 m^3/s. Thus, the modelled peak flows (4,170 m^3/s: Base Case and 6210 m^3/s: Upper Bound Case) are in good agreement with the empirically calculated estimates. At the dam site, these flows are an order of magnitude greater than the natural 100 year return period flood (about 400 m^3/s).

Figure 4.2 shows the outflow hydrograph from the dam for the two modelled scenarios.

Figure 4.2 Lee River Storage Dam – dam breach outflow hydrographs
(dashed red line – 1 hour failure, bold blue line – 2.5 hour failure)



4.4 Hydraulic Modelling

The HEC-RAS hydraulic modelling package, produced by the US Army Corp of Engineers, was used for routing the flood wave down the Lee River to the Wairoa Gorge and to the sea. A very coarse representation of the valley downstream of the dam was set up in the model based on the following information:

- LINZ 1:50,000 topographic maps N27 & N28
- limited measurements of the Lee valley cross-section at the guide camp (Paretai Lodge) using a laser rangefinder and barometer on 8 June 2006
- other field observations from site visits of 7 February 2006 and 8 June 2006 (notes, photographs (see **Appendix 6**), etc.)

It should be noted that the available data for setting up the model is very limited and there is considerable uncertainty in the ground surface representation, particularly in the flatter and wider floodplain areas downstream of the Wairoa Gorge. Nevertheless, the modelling should still give a reasonable indication of the flood wave attenuation down the valley and the peak discharges at various locations. Since the narrower sections of the valley have generally been used in the model representation of the river channel, the model would likely under-predict the attenuation in the flood wave as it progresses downstream and over-predict the maximum flood heights. Thus, the peak water level and peak flow results are likely to be conservative (i.e. high, tending towards worst-case).

Table 4.4 summarises the simulated peak discharges and approximate peak water levels at several locations downstream of the dam site. These locations, labelled A to I, are shown in Figure 4.1.

At the confluence of the Wairoa and Lee Rivers (map ref. F), the peak flow of 4,830 m^3/s in the 1 hour dambreak and 3,620 m^3/s in the 2.5 hour dambreak is, respectively, 3.1 and 2.3 times as great as the natural 100 year return period flow (1570 m^3/s). Such flows are likely of the same order of magnitude as the Probable Maximum Flood. Further downstream, the model predicts significant attenuation in the peak flow in the 5 km reach between the Gorge exit and Aldourie Road (map ref. H). Peak flows in both scenarios are between 1800 m^3/s and 2200 m^3/s in the lowermost reach from Aldourie Rd to the sea, which would be comparable with the natural 100 year return period flood.

Figures 4.3 and 4.4 plot the flood wave hydrographs at locations A through to I for the 1 hour (Upper Bound Case) and 2.5 hour (Base Case) dam failure scenarios respectively, and illustrate the attenuation of the flood peak as the floodwave travels downstream from the dam site to the coast.

Figure 4.3 Lee River Storage: Flow Hydrographs for 1 Hour Dam Failure

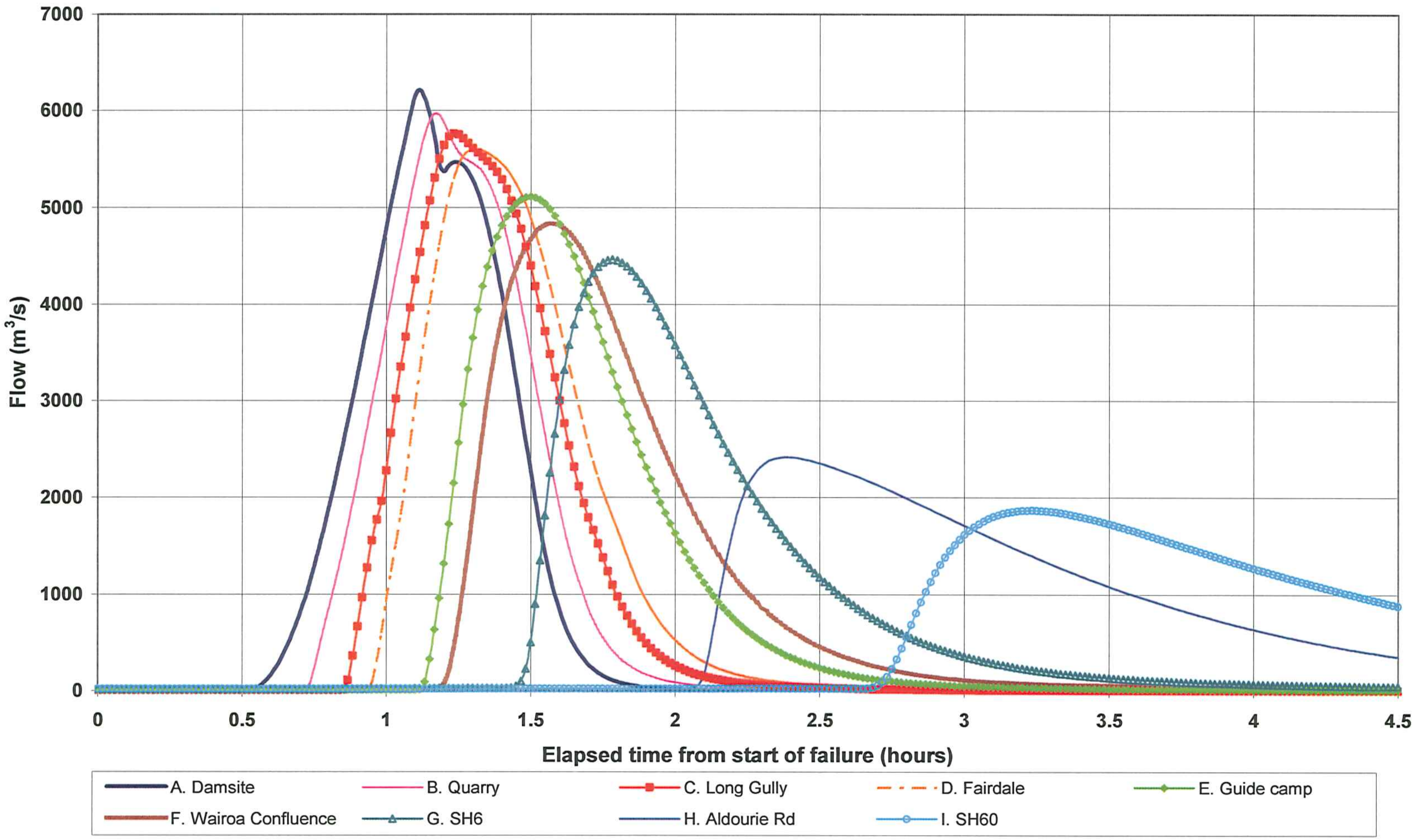


Figure 4.4 Lee River Storage: Flow Hydrographs for 2.5 Hour Dam Failure

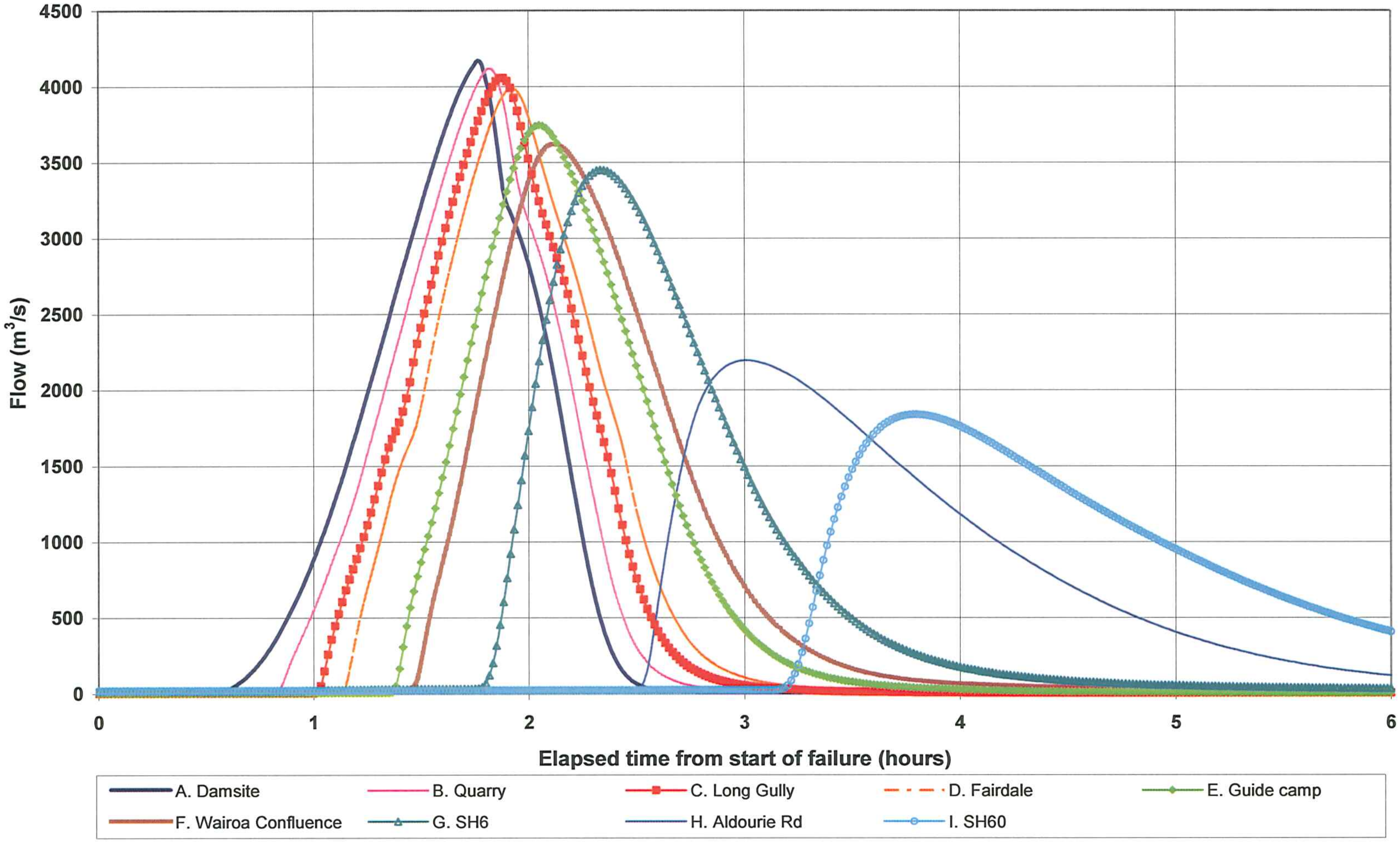


Table 4.4 Simulated Peak Discharges and Peak Water Levels from a Breach of the Proposed Storage Dam on the Lee River

[1]Map Ref.	[2]Location	Peak Flow 1 hour breach (m^3/s)	Peak Flow 2.5 hour breach (m^3/s)	Peak Water Level 1 hour breach (RL m)	Peak Water Level 2.5 hour breach (RL m)	Approx. Normal River Level (RL m)
A	At the foot of the dam	6210	4170	150.8	147.8	136
B	Adjacent to the upper quarry site	5960	4120	112.3	109.3	96
C	Above confluence with Long Gully	5760	4060	85.6	83.5	70
D	Fairdale footbridge	5600	3990	73.7	71.5	58
E	Guide camp (Paretai Lodge)	5110	3740	56.1	54.0	38
F	Below Wairoa River confluence (upstream of gorge)	4830	3620	51.0	49.0	33
G	Around SH6	4460	3450	27.3	26.9	23
H	Adjacent to Aldourie Rd	2410	2190	19.1	18.9	14
I	At about SH60 (coastal highway)	1870	1840	9.9	9.8	5

Notes
1 Figure 4.1 shows the location of the reference points.
2 Ground and water levels are very approximate and for modelling purposes only – based on interpretation of 1:50,000 topomaps

4.5 Description of Dam Breach Flowpath

4.5.1 Assessment of scenarios

In the Upper Bound Case (1 hour dam failure), the outflow from the reservoir increases rapidly, and very high flows last for about one hour. The reservoir effectively empties in 1 hour and 20 minutes.

In the first 4 km below the dam site (i.e. map ref. A to partway between B and C), where the river is confined in a narrow V-shaped valley and the river bed relatively steep (average slope 1 in 75), the peak flow velocity averages about 9 m/s, and the flood wave attains a maximum height some 15.5 m to 16.5 m above normal water level. Over the next 8.5 km, the river is similarly confined. However, the gradient gradually reduces to 1 in

400 (from 1 in 100 initially) over this distance, and there are several localised areas where the river valley widens out. The peak flow velocity ranges from 4 m/s to 7 m/s over this reach; with the higher velocity corresponding with the higher gradient and vice versa. Maximum flood heights increase inversely with the reduction in riverbed gradient, and reach a maximum of 18 m above normal river level just upstream of the guide camp (map ref. E). The flood front arrives at this location about 40 minutes after the start of the breach.

At the relatively constricted Wairoa Gorge (downstream of map ref. F), the peak flow velocity increases to over 7 m/s for a short distance. After the flood wave exits the gorge the maximum flow depth reduces rapidly to generally less than 6 m. Flow velocities are also substantially lower as the flow spreads out and fills up the floodplain confined between gravel terraces about 2 km apart. The peak flow reduces by about half (to about 2200 m^3/s) as a result of storage attenuation over the next 4.5 km. Downstream of this storage reach, stopbanks confine the main floodway to between 400 m and 700 m wide. However, based on currently available information it is unclear if the stopbanks are high enough (and of sufficient integrity) over this length to contain the floodwave without any breakout. The flood front arrives at Aldourie Road (map ref. H) some 1.5 hours from the start of the dam breach.

In the Base Case (2.5 hour dam failure), high outflows from the breach persist for about 1.5 hours, and the reservoir empties in about 2 hours. (It is worthwhile noting here that the breach development time assumed is greater than the time to empty, which suggests that a 2 hour dam failure, if such a case were modelled, would give very similar results.)

The flood flowpath in a 2.5 hour failure is not dissimilar to that in the 1 hour failure described above; while maximum flow depths and flow velocities are consistently lower, the areal extent of the inundation is expected to be only slightly smaller, excluding breakouts from stopbanked sections of the floodway.

For example, in the first 4 km below the dam site (i.e. map ref. A to partway between B and C), where the river is confined in a narrow V-shaped valley and the river bed relatively steep, the peak flow velocity averages just over 8 m/s and the flood wave attains a maximum height some 12.5 to 13.5 m above normal water level. Over the next 8.5 km, where the river is of lower gradient and has a few widish sections, the peak flow velocity ranges from 3.5 m/s to 6 m/s. The maximum flood height through this reach is up to 16 m above normal river level and occurs just upstream of the guide camp (map ref. E).

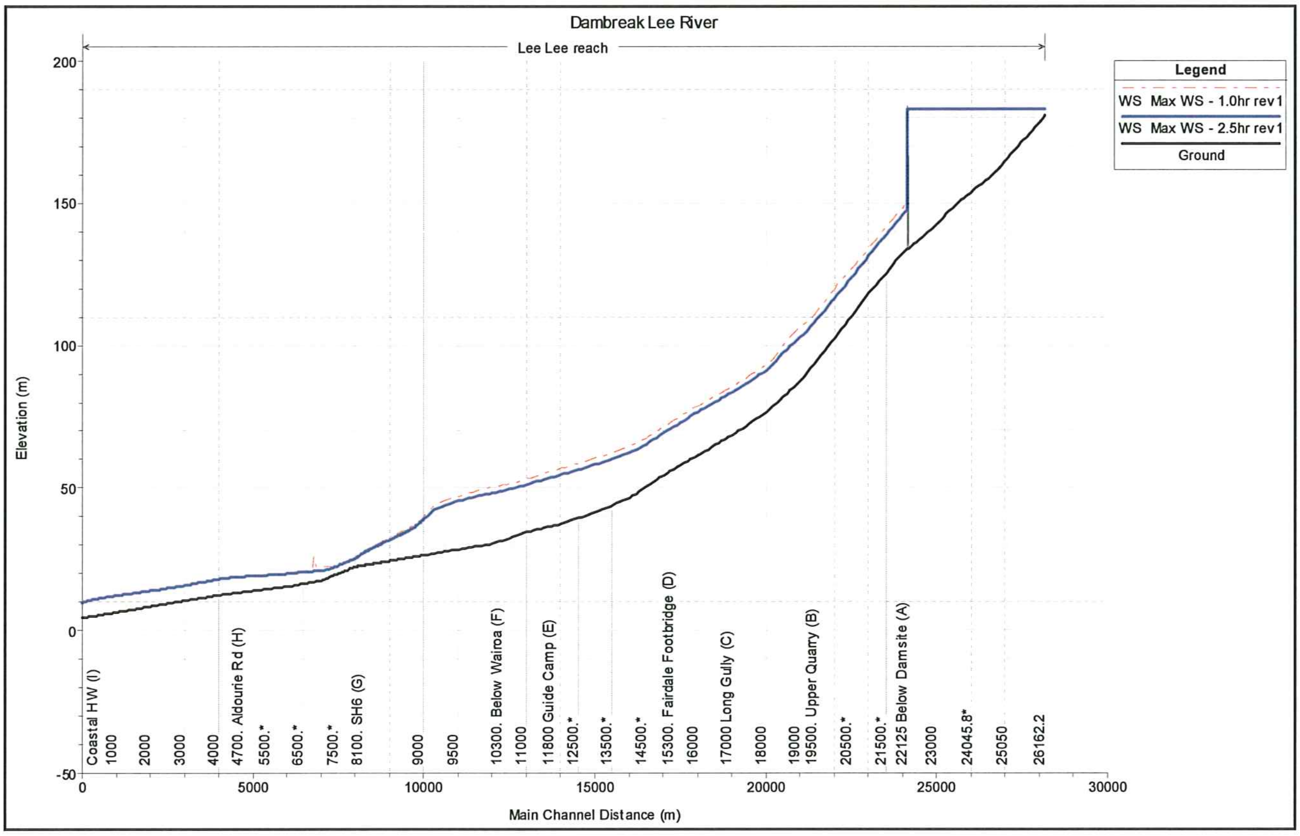
Because the breach development time is slower, there is a greater lag between the arrival of the flood wave and breach initiation compared with the 1 hour dam failure. At the guide camp, the flood front arrives after 55 minutes.

As the flood wave passes through the Wairoa Gorge (downstream of map ref. F), the peak flow velocity increases to over 6.5 m/s for a short distance. Maximum flow depths and flow velocities reduce significantly after the flood wave exits the Gorge. The peak flow reduces by about 35% as a result of storage attenuation in the 5 km or so downstream of the Gorge. It takes about 2 hours for the flood wave to reach Aldourie Road from the start of the breach.

Figure 4.5 plots the riverbed profile (longitudinal section) and the maximum water levels from a dam breach along the modelled reach for both the 1 hour and 2.5 hour failure cases.

Figure 4.5 Lee – Wairoa River Profile and Maximum Water Levels

(red line – 1 hour failure case, blue line – 2.5 hour failure case)



4.5.2 Comparison with natural events

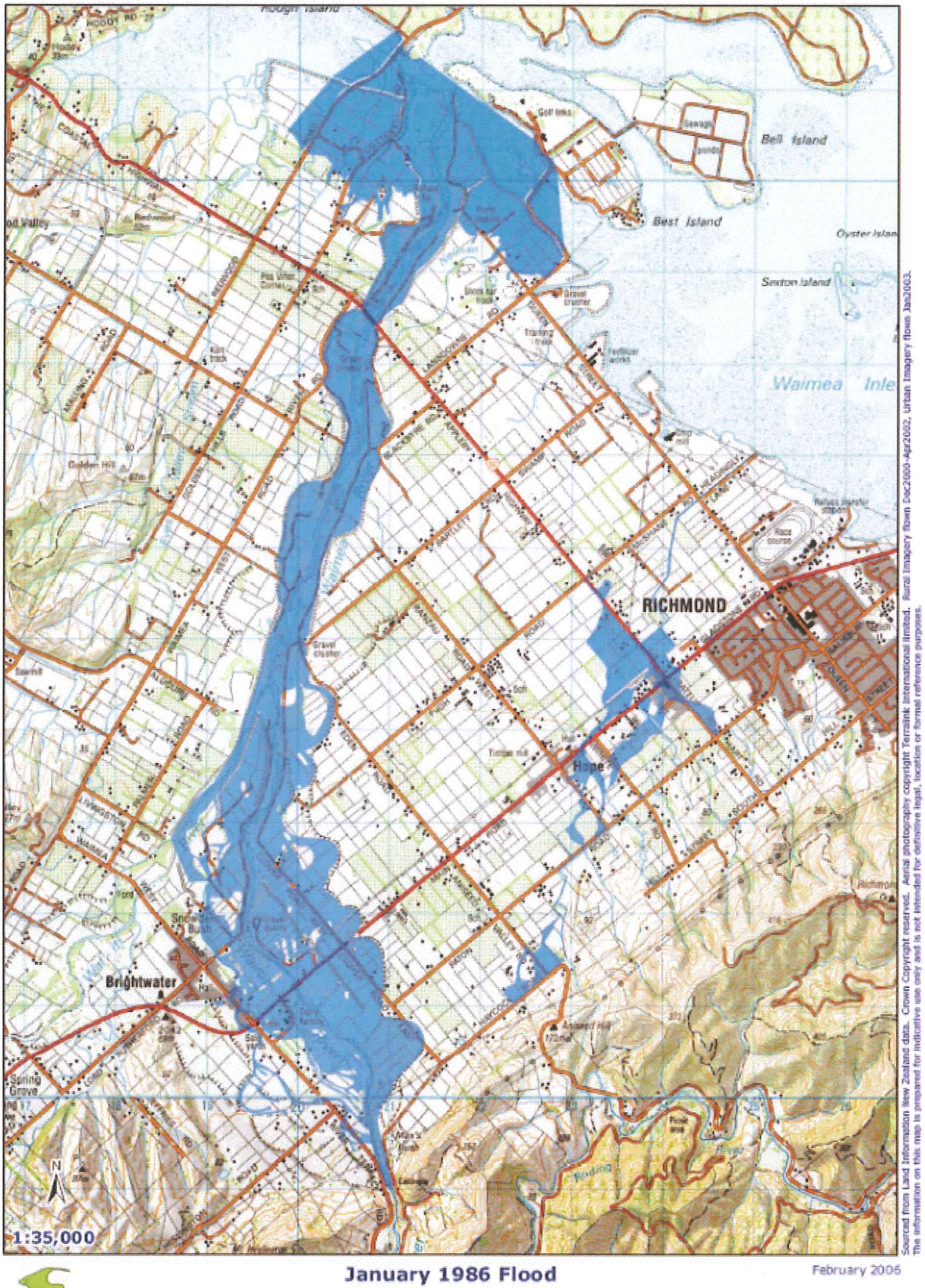
In terms of comparable historical (natural) flooding, the largest flood in the Wairoa River since continuous records began in November 1957 is the January 1986 flood. At the Wairoa Gorge, a peak flow of 1466 m^3/s was recorded. This flood has a return period of about 65 years. Note that the 100 year return period flood is estimated to be 1570 m^3/s at this location.

Figure 4.6 supplied by Tasman District Council maps the approximate extent of inundation below the Wairoa Gorge to the coast from the 1986 flood event. Floodplain flows (out-of-bank flows) were estimated to have commenced at about 900 m^3/s to 1000 m^3/s and eventually spread out to inundate the area shown (pers. comm. Verstappen)

It is interesting to note that during this event, the town of Brightwater appeared to be generally above the inundation level. However, under present day conditions with interim in-fill development and increased floodplain roughness, especially on the right bank, the inundation extents from a similar sized flood may well encompass Brightwater (pers. comm. Verstappen). Photographic records exist that show Brightwater was flooded in 1939 during a flood event presumably larger then the 1986 flood.

In comparison to these historic floods, the peak flows from the two dam failure scenarios considered here are some 2.5 to 3.3 times as great as that recorded in the 1986 flood (at the Wairoa Gorge). However, by the time the flood wave arrives at Aldourie Road some 5 km downstream, the peak flow from both scenarios would have attenuated to about 1.5 times the 1986 peak flood flow.

Figure 4.6 Extent of Inundation in the January 1986 Flood (Estimated by TDC)



4.6 Potential Impact Categorisation

4.6.1 Description of hazard and damage potential

The following is a discussion and assessment of the downstream hazard potential if the dam were breached. To put this discussion into proper context, it is essential to draw the distinction between hazard potential (that is the effect of the dam breach were it to occur) and the risk or chances of the dam breach actually occurring. As noted earlier, the failure risk for a dam engineered and built to appropriate standards (per NZSOLD Guidelines), as would be the case for this proposal, would be extremely low.

A dam breach creates a flood wave that rises very rapidly, has a high peak flow but is relatively short-lived. As such there would generally be a short lead time for warning and for people and any stock to be evacuated to safer ground. Within the floodpath, the area of extreme hazard to human safety is within the river bed, particularly where the river is situated within a narrow and steep sided valley, from which escape from the advancing flood wave would be difficult.

In the 13 km reach of river between the dam site and the Wairoa Gorge the river is relatively confined, and the maximum flood level above normal water level ranges from 12.5 m to 18 m (12.5 m to 16 m in the Base Case and 15.5 to 18 m in the Upper Bound Case). Therefore, several of the few building sites flanking the river, which may appear to be above natural flood inundation levels, could be at risk if the dam breach were to occur. Peak flow velocities are also very high, ranging from 3.5 m/s to 9 m/s. The available warning time would be very short – the flood front would arrive at the Wairoa Gorge in less than one hour after the start of the dam breach.

There are a few dwellings on both banks of the Lee River, mainly in its lower reaches between the lower quarry site (map ref. D) and its confluence with the Wairoa River (map ref. F).

The first dwelling below the dam site, as determined from vantage points from the Lee Valley Road and available mapping, is located on the left bank about 3.5 km downstream from the dam site, and comprises a farmhouse and several farm buildings. The next dwelling appears temporary in nature (a parked caravan) and is located about another one kilometre downstream. Down as far as the guide camp (Paretai Lodge, map ref. E), which is located about 10 km downstream from the dam site, there are a further 6 to 10 habitable buildings on the flanks of the Lee River. Between the guide camp and the confluence of the Lee and Wairoa Rivers, there are possibly 5 to 8 more dwellings sited close to the river. Detailed survey and more refined hydraulic modelling will be necessary to determine how many and to what extent these dwellings would be at risk from inundation in a dam breach. However, it is clear that several buildings (as distinct from dwellings) would definitely be at risk.

It is also noted that parts of Mead Road, which serves a number of properties on the right bank of the Lee River, are very low-lying (in the context of inundation from a dam breach) and would likely be severely damaged in the event of a dam breach. Thus, access to these properties would be cut-off.

At the guide camp, barometric levelling indicated that the floor level of the two-storey main accommodation block is some 13 m above normal river level. At this location, the predicted maximum flood height in dam breach is between 16 m and 18 m above normal river level. On this basis, it is likely that this facility would be at significant risk of being inundated to some depth.

Below the confluence of the Lee and Wairoa Rivers, many more habitable buildings are potentially located within the floodpath of a dam breach. It is anticipated that there will be widespread flooding and property damage, and this would include the township of Brightwater. However, except for the 1.5 km long reach between the confluence and the gorge exit, the general expectation is that flooding would not be to life-threatening levels in most areas. Detailed survey and sophisticated hydraulic modelling (2-dimensional) will be required in subsequent phases of the project to ascertain and map the hazard in the floodplain below the Wairoa Gorge.

4.6.2 Recommended potential impact category for Site 11 Lee

From the foregoing assessment of breach scenarios and discussion on damage potential, it appears that **if** a dam breach were to occur, there is potential for fatalities to occur, as well as very major damages. Accordingly, it is recommended that the potential dam on the Lee River be classified as a **High** Potential Impact Category dam. Design, construction, maintenance and ongoing monitoring should therefore be undertaken in accordance with the requirements set out in the NZSOLD Dam Safety Guidelines for a High Potential Impact Category dam.

4.7 Dam Break Avoidance or Mitigation

The Resource Management Act requires that any adverse effects on the environment from the proposed development, potential or actual, be avoided, remedied or mitigated. In this context, dam breach, as a potential adverse effect, may be avoided or mitigated as far as practicable by adopting the suitably conservative design standards for a High Potential Impact Category per the NZSOLD Dam Safety Guidelines.

5 Cost Estimate – Site 11 Lee

An assessment has been made of the raw capital cost of the potential scheme as described in Section 3. The indicative cost at this stage of the project is as follows:

- Between $20 - $25 million confidence limits. This range represents a range in contingencies of 10% to 30% respectively.

This provides for:

- Access road upgrade and contractors' working area
- Power supply/communications to site
- Stripping to waste/stockpile, dam footprint and borrow areas
- Rehabilitation of exposed borrow areas and dam surrounds
- Dam fill, drainage and armour (six sub-items)
- Extra allowance for special drainage zones adjacent
- Clearing of reservoir area
- Reservoir access road upgrading
- New bridge at head of reservoir
- Cofferdams/dewatering and flood risk allowance
- Plunge pool armouring
- Diversion/conveyance structure
- Diversion structure headwalls, furniture
- Temporary energy dissipation during construction
- Spillway works
- Access bridge over spillway
- Extra for spillway chute anchoring/drainage
- Dam instrumentation
- Intake tower
- Pipework valving, ventilation, screens and ancillaries ($2m^3/s$ max discharge)
- Miscellaneous small items allowance
- Contractor establishment, engineering and 10-30% contingency/uncertainty allowance.

This cost excludes:

- financing costs
- legal and developer administration
- land acquisition
- RMA process
- any hydro add-on costs
- extra flow release requirements for flushing purposes

Should the piped option be adopted (see Section 3.9) there would be an additional cost of about $6.5M (pipeline from the dam to the existing Waimea East Irrigation Scheme pump station at the end of the Wairoa Gorge).

This overall cost estimate is based on Phase I preliminary level investigations only – further refinement is dependent on the outcome of feasibility level investigations which requires more refined sizing of components as part of Phase II investigations. Major identifiable uncertainties at this stage include:

- The accuracy of contour information (influencing dam height and earthworks volumes)
- Any requirements imposed on diversion and flood capacities
- Outcome of any independent peer review
- The outcome of more detailed geotechnical investigations confirming abutment conditions (potential for right abutment geology to be more complex than indicated to date)
- Whether the right abutment gravels can be utilised for filters after processing or whether some filter material may need to be imported to site
- Whether new reservoir roading proves more extensive than assumed
- The requirement for mitigation measures involving significant cost
- Price fluctuations for fuel or other materials which affect schedule item prices
- Contractor availability/demand at the time of tendering which may impact pricing and/or programme
- Local (district) specific issues relating to contractor pricing and timing.

6 Applicability

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

TONKIN & TAYLOR LTD

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Authorised for Tonkin & Taylor by:

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John Grimston
Project Coordinator

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Appendix 1: **Letter Tonkin and Taylor Limited to WWAC/TDC dated 14 December 2004. (Job no 22032.002)**

Appendix 2: **Assessment of Water Storage Options – Waimea Catchment. Tonkin and Taylor Limited, April 2005. (Job no 22032.002)**

Appendix 3: Revised Approach for Transfer from Site 2 Storage. Letter Tonkin and Taylor to WWAC dated 1 July 2005. (Job no 22032.002)

Appendix 4: **Assessment of Two Alternative Water Storage Options – Upper Lee and Left (eastern) Branch Wairoa River – Waimea Catchment. Tonkin and Taylor Limited, August 2006. (Job no 22032.005)**

Appendix 5: Appendix to Preliminary Geology and Geotechnical Assessment:

- Photographs of Lee River Dam Site
- Figure 1 - Investigation Location Plan - Dam Sites
- Figure 2 - Investigation Location Plan - Borrow Areas
- Figure 3 - Geological Section Site 1
- Figure 4 - Geological Section Site 2
- Figure 5 - Geological Setting
- Report by Dr Mike Johnston
- Test pit excavations - TP1-TP19
- Scala penetrometer tests - SC1-SC14
- Representative batter log - B1
- Table 1 - Scrapes 1-3
- Engineering Log Terminology

Appendix 6: Preliminary Dam Breach Assessment - Photographs of Selected Sites