REPORT

Waimea Water Augmentation Committee

Lee Valley Dam
Detailed Design Geotechnical Investigation Report

Volume **1**

Tonkin & Taylor

ENVIRONMENTAL AND ENGINEERING CONSULTANTS

REPORT

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Volume 1

Report prepared for:

Waimea Water Augmentation Committee

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Table of contents

Volume 1 of the Report:

Volume 2 of the Report:

Appendix A:	Drawings
Appendix B:	Test pit logs
Appendix C:	Seismic survey report
Appendix D:	Drill hole logs
Appendix E:	In situ test data for test pads
Appendix F:	GNS Science Site Specific Seismic Assessment
Appendix G:	Cross section drafted during the course of the investigations (For information only)
Appendix H:	Lugeon test results
Appendix I:	Piezometer monitoring data
Appendix J:	Spillway stability analyses
Appendix K:	Zeolite analysis report
Appendix L:	Laboratory test data
Appendix M:	Alluvial borrow test pit logs
Appendix N:	Alluvial gravel suitability as concrete aggregate

Executive summary

A dam up to 52 m high is currently proposed to store water in the Lee River headwaters for augmenting flows in the Waimea River and to provide irrigation on the Waimea Plains. The proposed dam is located at Chainage 12,430m upstream from the confluence between the Wairoa and Lee rivers. This report presents the findings of the detailed design geotechnical investigation for the site.

The proposed dam site is located within Rai Formation greywacke sandstone and siltstone/mudstone basement rocks within the Caples Terrane. The site is flanked by the Gordon Range to the west and the Richmond Range to the east.

The dam site is very likely to have been affected by significant earthquake shaking in the past as nearby active faults include the Waimea Fault 8.5 km to the NW and the Wairau segment of the Alpine Fault 20 km to the SE.

The geology underlying the dam site comprises variably weathered, weak to very strong, very closely spaced to closely spaced jointed fine sandstone and siltstone of the Rai Formation. The rock has been separated into three rock classes based on rock mass condition. These are summarised below:

- Class 1 – Unweathered, strong to very strong, blocky to very blocky rock mass
- Class 2 – Slightly weathered, moderately strong to strong, very blocky rock mass
- Class 3 – Moderately to highly weathered, weak to moderately strong, very blocky rock mass

Defects in the rock are dominated by bedding and three sets of joints. Bedding within the dam footprint and spillway dips predominantly to the northwest, but locally also dips to the southeast. Crushed and sheared zones are present within the rock mass and are often orientated sub-parallel to bedding.

Foundation requirements for most of the dam require that the foundations are stiffer than the rock fill. This will be achieved by excavation to an appropriate grade of rock. Foundations prepared by bulldozers and or excavators using mainly blade and light ripping will generally satisfy the required stiffness criteria. In places the excavation will need to be deepened. Higher quality rock will also need to be exposed in the vicinity of the downstream toe of the dam for the purposes of anchoring mesh which will be required to prevent flood scour.

Preparation of the plinth will require excavation to at least Class 2 and preferably Class 1 rock.

Preparation of the plinth foundation on the right abutment will require cut slopes of approximately 10V:1H. Wedge failures in two orientations may occur on the right abutment. Face mapping will be required as the plinth excavation is progressively undertaken and support requirements will need to be designed for the specific defects encountered.

Preparation of the plinth on the left abutment will require low height cut slopes of up to 1V:0.8H. Local drapes of wire netting may be required to contain loose blocks of rock falling from these cuts. Local drapes of wire mesh may also be required to contain loose blocks rolling down the cut faces of the diversion channel.

No indications of deep seated instability have been identified above the right or left abutments. However the right abutment is steep and construction of the plinth will require careful staging.

Spillway cuts will be up to 60 m high and generally within rock classes 1 and 2 will mostly be cut at 1V:1H. Spot bolting may be required to stabilise shallow failures on these slopes. Depending on blasting and the presence of sheared zones and dilated rock, some dental concrete may be required for the floor of the spillway. Excavation methodology must give due consideration to minimising dilation of the rock mass adjacent to finished profiles.

Permeability of the rock within the dam site was found to be variable and as such a grout curtain will be required beneath the plinth excavation to increase the seepage path and thereby reduce the leakage potential around and beneath the dam. Grout holes will need to be both vertical and angled on the abutments to treat the defects observed. Sheared zones of 100mm or greater thickness are likely to be encountered at regular intervals along the plinth excavation foundation and will require individual treatment.

On-site construction materials comprise sandstone and mudstone/siltstone rock of the Rai Formation, alluvial gravel and solifluction deposits.

Excavation of the rock has been assessed with rippability trials and seismic refraction profiles carried out on the site. These indicate that Class 3 rock is easy ripping, Class 2 rock is difficult to rip and Class 1 rock is very difficult ripping or unrippable. Transitions between the different rock classes were found to be irregular and there could be changes in rippability both laterally and vertically during excavation.

Compaction trials on Class 2 and 3 rocks indicate that rock fill is likely to have a Young's Modulus of between 20 and 30 MPa. Laboratory testing carried out before and after compaction yielded similar grain size distributions suggesting that the site sourced rock fill is not prone to construction breakdown. If sufficient volumes of Class 1 and Class 2 rock are not available to complete the embankment some of the Class 3 rock when selectively excavated may be suitable for construction. Further trials at the time of construction are recommended to confirm this.

Alluvial borrow material is present adjacent to the Lee River and on terraces above the main channel. These materials are dominated by river gravels of which 62,000 m^3 may be located within 2 km upstream of the dam. The alluvial material contains up to 30% boulders and may be processed to yield rip-rap and filter materials for dam construction.

Solifluction deposits around the dam site can be processed to provide blanket materials if required.

Nearby off-site sources of aggregate and rip-rap are also available.

The stability of slopes around the reservoir is not likely to be significantly affected by the impounded water level within the reservoir. There are, however, areas of steep ground mantled with rock scree that could mobilise during earthquake shaking. In this respect the tree cover (exotic or native) should be maintained as much as possible to provide stability to the soil mantle. In addition to this, an OBE earthquake or an extreme long rainfall event combined with a full reservoir could mobilise one of two areas (or both) of previous instability. These have the potential to generate waves within the reservoir. Beach creation and wave action may also cause localised instability of superficial deposits during reservoir operation.

1 Introduction

1.1 General

This report summarises the detailed design site investigations undertaken for the proposed Lee River Dam, Tasman District. It is being developed by a community backed committee known as the Waimea Water Augmentation Committee (WWAC). The dam is intended for use as an irrigation dam to provide drought security to the Waimea Plains. The dam's purpose is water augmentation for irrigation and community water supply. The dam is intended to supplement the Lee River's natural flows to provide a constant residual flow as well as an irrigation flow. The proposed dam site is located on the Lee River approximately 40 minutes by car to the south of Nelson. The Lee River is a tributary of the Waimea River.

1.2 Background and previous work

Between 2004 and 2006 Tonkin & Taylor Ltd (T&T) evaluated a number of options to provide water storage for long-term irrigation and community supplies in the Waimea Basin on behalf of WWAC. The outcome of that Phase 1 study was to focus feasibility investigations on a dam site located in the upper Lee River catchment as the preferred option for possible water storage. The general location of the project area is shown in figures 1 and 2 and in Drawing 27425-GEO-01 (Appendix A).

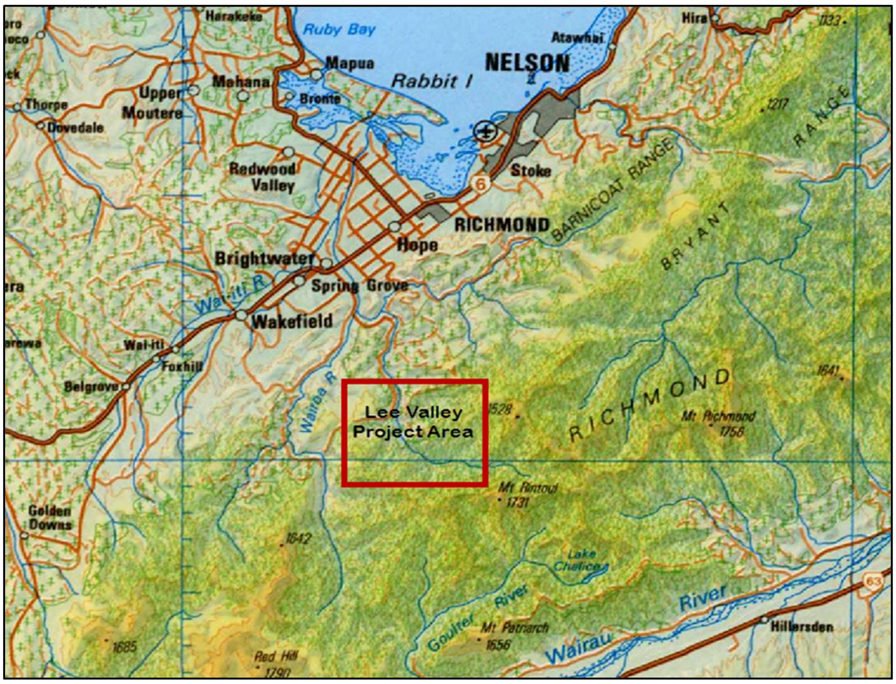


Figure 1: Location of the proposed dam site

The Lee Valley dam site is accessed by forestry roads off Lee Valley Road as shown on Figure 2.

In 2007 WWAC initiated Phase 2 of the study. Geotechnical feasibility investigations have been carried out in a staged programme as part of Phase 2.

Stage 1 of the Phase 2 geotechnical feasibility assessment comprised geological and engineering geological mapping of the project area. The Stage 1 study (T&T ref. 24727.200 dated December

2007) assessed geotechnical issues at eight different potential dam sites in the upper Lee Valley, between Ch 10,500 m[1] to 12,400 m upstream of the Lee River/Wairoa River confluence. As part of the Stage 1 work a preliminary assessment of 17 potential construction material borrow sites within the area was also carried out. Engineering evaluation of the various dam sites (T&T ref. 24727.301 dated December 2007) identified a preferred site at Ch 11,010 m as being the most economical but noted that potential risks affecting that site included poor quality rock on the right abutment, a potential large scale slope instability on the left bank upstream of the dam site, and lack of suitable of core material and high quality rock fill.

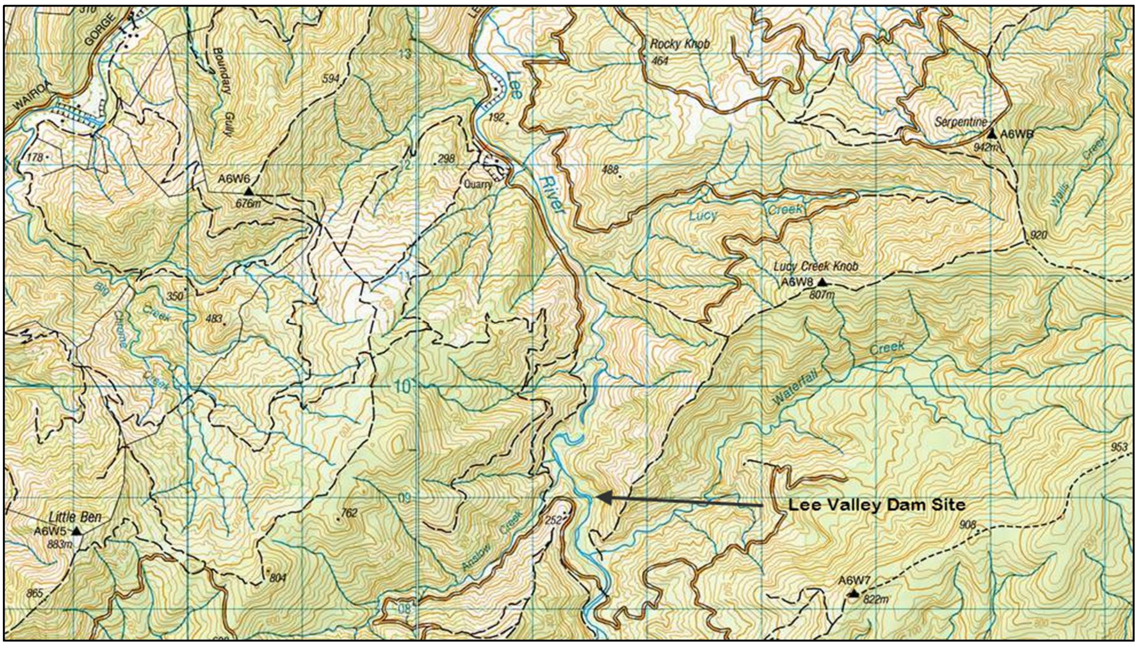


Figure 2: Location of the proposed dam site on the Lee River.

Sampling and testing of borrow materials was carried out on selected sites as part of Stage 2A (T&T ref. 24727.203 dated February 2008). This found that suitable plastic clay core material for an embankment dam was not locally available and the local bedrock would provide a lower quality rock fill, although alternative rock fill sources were available.

Stages 2 and 3 of the geotechnical investigation programme comprised surface mapping, test pitting and drilling focussed on a dam site at Ch 11,010 m. That investigation is documented in the draft “Geotechnical Feasibility Report – Phases 2&3 Lee Valley Dam site” (T&T ref. 24727.201 dated June 2008). That investigation confirmed that poor quality rock extended to significant depth on the right abutment and a need to provide for stabilisation of the left bank landslide. When viewed cumulatively, it was considered that these issues had an adverse effect on potential cost and programme in relation to a dam at Ch 11,010. WWAC subsequently endorsed the recommendation to investigate an alternative site located between Ch 12,100 m and 13,000 m.

Stage 4 involved geotechnical investigations between Ch 12,100 m and 13,000 m. Those investigations are documented in “Lee Valley Dam Feasibility Investigations – Geotechnical Investigations Report”, dated December 2009”. On the basis of preliminary engineering

[1] A river location referencing system has been set up for the reach of the Lee River relating to this study. Distances are referred to in metres upstream of the Lee River/Wairoa River confluence.

geological mapping and consideration of earthworks volumes a site at Ch 12,430 m was subsequently selected for drilling investigations (see Figure 2 for selected location).

This report discusses the detailed design investigations which have included further drilling, test pitting and seismic surveys at, and around the Ch 12,430 m site. The full scope of the detailed design investigations is discussed in Section 1.6 below.

1.3 Proposed Dam

The project comprises the construction of a dam and $13Mm^3$ reservoir in the upper Lee Valley approximately 200 m upstream of Anslow Creek. The Lee River is one of two major tributaries of the Wairoa River which drains the Richmond Range east of the Waimea Plains. The Wairoa River is then joined by the Wai-iti River, and together they form the Waimea River.

The reservoir will be impounded by a concrete faced rock fill dam (CFRD) at a location of Ch 12,430 m. The dam would be approximately 52 m high and 210 m wide at crest level. The location and preliminary layout of the dam is shown in Figure 3. The storage reservoir will have a top water level of RL197.2 m and will extend approximately 3.7 km upstream of the dam. Arms of the reservoir will extend approximately 1 km into Waterfall Creek on the right bank, and 350 m into Flat Creek on the left bank. The reservoir will be drawn down to about RL171 m during periods of river augmentation draw off.

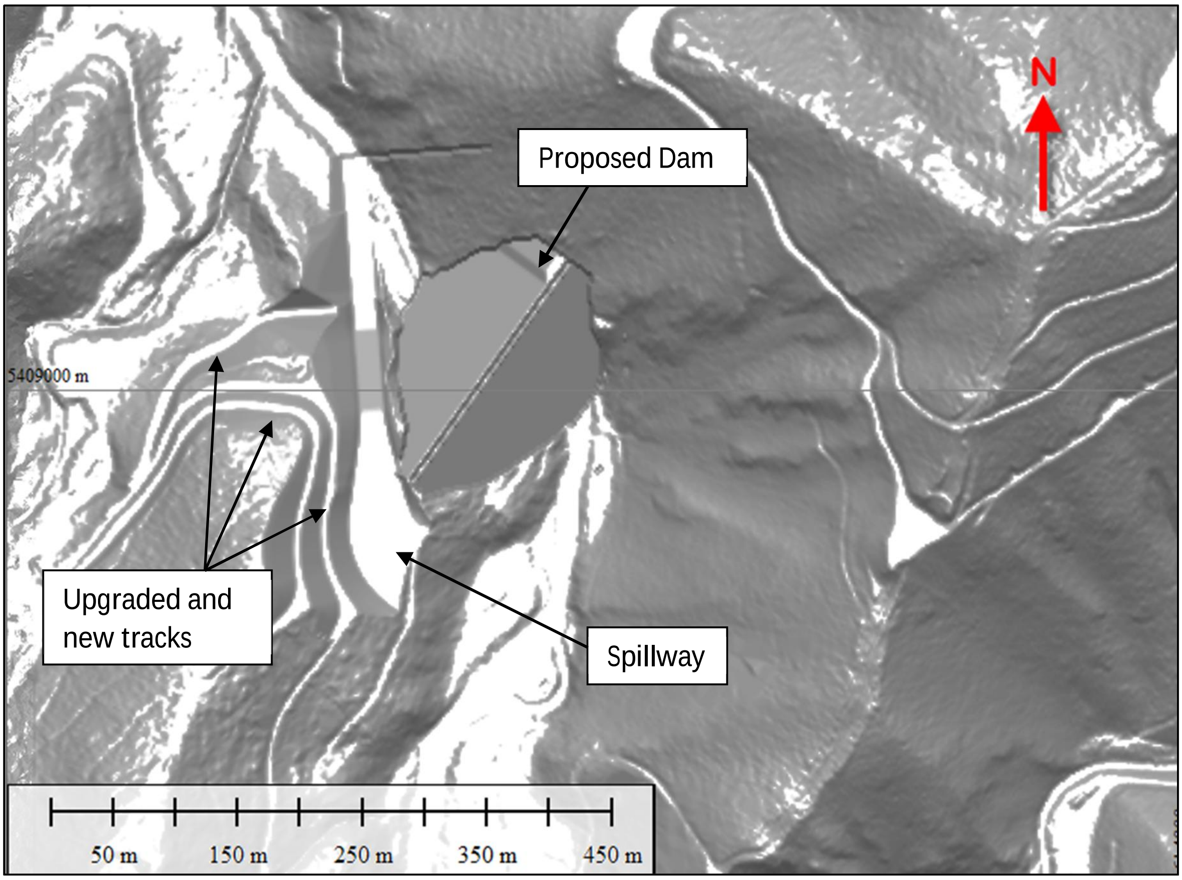


Figure 3: Location and layout of the proposed dam.

The proposed concrete faced rock fill dam will be constructed from approximately 430,000m^3 of locally sourced rock fill. Structures associated with the dam include a spillway, a sluice, and a diversion conduit that will be utilised after construction as the irrigation off take.

1.4 Scope of investigations

Investigations in this detailed design stage have addressed specific items identified following issue of the Feasibility Report. Our scope of works for this stage was proposed as follows:

Right Abutment Plinth Excavation
Further investigation to include:

- Define accurate ground levels so that rock defects and rock head depths can be accurately plotted along the line of the plinth excavation.
- Drill two investigation holes with continuous packer testing to approximately 20 m below plinth level. Some further bench cutting will be required from Track C to accurately position a hole over the plinth excavation.
- Further defect mapping will be carried out and plotted on track logs. Stereographic plots of defects will be prepared to enable a kinematic stability assessment to be undertaken.
- Data will be presented on a new cross section along the line of the plinth to show excavated rock conditions, specific plinth line defect combinations, and rock quality (RMR and GSI classifications).

Left Abutment Plinth Excavation
Further investigation work is to include:

- Two to three holes will be drilled with continuous packer testing to approximately 20 m below plinth level.
- Stereographic plots of left abutment defects from track and test pit logs will be prepared to enable a kinematic stability assessment to be undertaken.
- Data will be presented on a new cross section along the line of the plinth to show excavated rock conditions, specific plinth line defect combinations, and rock quality.

Centre Plinth Excavation/Starter Dam
Further investigation work will include:

- Drilling on the lower left abutment to extend to assess whether sheared zone in TP8 extends at shallow depth beneath the plinth excavation, mapping defect orientations on riverbank exposure along line of plunge pool will be carried out and data will be plotted stereographically to enable a kinematic stability assessment to be undertaken.
- Trial pad excavations on the lower left abutment will expose rock to 4m depth across the dam footprint. Rock quality characterisation will be carried out.

Seismic Refraction Survey:
Further investigation work will include:

- Locating a plinth line drill hole at approximately RL 200m to assess depth, permeability and rock mass modulus characteristics of poor quality, low velocity rock beneath the upslope portion of the plinth line.
- Excavation and compaction trials of the lower velocity, poor quality rock to assess suitability and properties of rock fill derived from this source.
- Refining the depths and characteristics of the rock quality classes and providing an updated 3D model of rock quality on the left abutment.

Reservoir Mapping

Further investigation work will include:

- Extending the proposed LIDAR survey to fully cover the slopes identified with elevated slope instability risk to delineate extent of potential/existing instability.
- Further mapping and test pit excavation near the toe of the left bank where disturbed rock has been identified,
- Prepare slope models to assess the sensitivity of each area of stability to reservoir filling.

1.4.1 Trench and test pit excavation

Trench and trial pit excavations were carried out between February 2011 and August 2012 using 22 and 34 tonne excavators. The trench and pit locations are presented in Table 1 and the locations are shown on the drawings 27425-GEO-03 to 05 in Appendix A. The test pit logs for those excavated during 2011 and 2012 are reproduced in Appendix B.

Left Abutment Plinth Line Test Pits TP 1A, TP1B and TP1C (2011)

Test pits TP1A, TP1B and TP1C were excavated close to the line of the left abutment plinth line.

TP1A was excavated between RL 201 to RL 182m. It encountered highly weathered rock that was easy to excavate at less than 1 m depth upslope of RL 192m. Downslope of RL 192m the rock head drops steeply (50^{o}) and the pit encountered silt and gravelly sand to the maximum digger reach of 6 m.

TP1B was excavated between RL 179 and RL 169m. Gravel and sand was encountered at the upslope end of the pit. Downslope of RL 175m rock was at less than 1 m depth. Near surface rock was moderately weathered and closely jointed but stronger and more difficult to excavate than the rock encountered in TP1A.

The silt and gravelly sand encountered in the downslope end of TP1A and the upslope end of TP1B taken in conjunction with the 10m thickness of similar materials encountered DH7 (2009) (gravel base at RL 175m) and TP3 (2009) indicate the presence of an old channel eroded into the bedrock to an approximate depth of RL 172m between the two trenches.

TP1C was excavated between RL 167 and RL 158m. Rock head was deeper than upslope and alluvial gravel overlaid a buried rock bench at RL 158m. The rock was moderately weathered to locally slightly weathered and was more difficult to excavate than the rock in the upslope pits.

Table 1: Trench and test pit locations

Investigation	Northern End			Southern End		
	E	N	RL	E	N	RL
TP1 (2008)	1613469	5408991	156.7	n.a	n.a	n.a
TP2 (2008)	1613214	5409085	161.3	n.a	n.a	n.a
TP3 (2008)	1613433	5408923	178.9	n.a	n.a	n.a
TP4 (2008)	1613480	5408992	154.4	n.a	n.a	n.a
TP1A (2011)	1613375	5408931	201	1613421	5408920	182
TP1B (2011)	1613439	5408932	192	1613459	5408943	169
TP1C (2011)	1613489	5408969	167	1613467	5408950	158
TP2 (2011)	1613356	5409074	149.7	1613346	5409069	160.4
TP3 (2011)	1613415	5409066	158.0	1613432	5409053	159.0
TP4 (2011)	1613423	5408977	165.4	1613415	5408967	175
TP5 (2011)	1613519	5408976	152.6	1613539	5408964	158.4
TP6 (2011)	1613537	5408960	158.2	1613538	5408948	157.6
TP7 (2011)	1613531	5408926	155.4	n.a	n.a	n.a
TP8 (2011)	1613510	5408940	151.4	n.a	n.a	n.a
TPR1 (2011)	1613389	5408918	196.1	1613393	5408909	194.6
TPR2 (2011)	1613450	5408945	174.9	1613454	5408938	171.6
TPR3 (2011)	1613479	5408970	157.7	1613483	5408964	157.8
TPR4 (2011)	1613305	5409035	198.5	1613314	5409033	198.8
TPR5 (2011)	1613269	5408926	234.5	1613275	5408923	236.9
TPR6 (2012)	1613324	5409007	203.8	1613329	5409001	202.8
TPR7 (2012)	1613357	5408942	204.7	1613360	5408938	205.9
TPO (2012)	1613441	5408933	176.0	1613442	5408927	176.5
TPP (2012)	1613320	5409037	195.7	1613313	5409035	198.3
TPQ (2012)	1613274	5409016	195	1613279	5409018	196
TPS (2012)	1613229	5408995	200.9	1613232	5408996	200.6
TPT (2012)	1613307	5408998	208.9	1613309	5408995	209
TPV (2012)	1613374	5408911	201.7	1613375	5408904	200.8

Test Pits TP 2 to TP8 (2011)

TP2 was excavated across the line of the plunge pool and exposed rock to a height of 6 m above the river. The near surface rock was slightly weathered siltstone/mudstone and varied from moderately strong to very strong. On the slope the rock mass was dilated and defects were open. The siltstone/mudstone dipped steeply upstream. Riverbank exposure was traced for at approximately 50 m downstream of the pit and consisted of siltstone/mudstone, with no significant crushed or sheared zones evident.

TP3 was excavated over the alignment of the downstream section of the diversion. It exposed dilated rock at shallow depth in the upstream half of the trench but no rock on the downstream half of the trench (depth >5 m, approximately lower than RL 154m).

TP4 was excavated between RL 165 and RL 175m on the left abutment close to the dam centreline. It exposed highly weathered to moderately weathered, thinly bedded siltstone/mudstone at shallow depth. Bedding dipped steeply to the south-west. The rock mass was dilated.

Test pits TP5, TP6, TP7 and TP8 were excavated upstream of the dam plinth line. TP5 and TP6 were located downslope of a steep shallow gully. TP5 encountered deep alluvium (within a few metres upstream of rock exposure) and TP6 encountered scree deposit overlying displaced and dilated rock, indicating poorer rock quality immediately upstream of the dam location. TP 7 was located at the base of a valley slope and encountered alluvium and river gravels overlying strong, slightly weathered rock. TP8, located 50 m upstream of the plinth excavation, encountered shattered rock adjacent to a moderately inclined downstream dipping crushed zone.

Right Abutment Investigations (Track B, Track C, drilling platforms for DH10, DH11)
In contrast to the left abutment the right abutment is locally very steep with an irregular slope profile with the exposures highlighting the variable rock quality with lithology (siltstone/mudstone lithology being of poorer quality locally and greywacke sandstone lithology being better quality). Crushed/sheared zones such as SZ8 occur along the contacts between the two lithologies. However, while locally the intact rock is very strong, overall the rock mass is dilated and wedge blocks were noted on the right abutment that will daylight in cut faces and in the plinth excavation. There are potential wedge failure combinations and it is likely that rock breakout will be irregular and stepped.

Rock Rippability Test Pits on the Left Abutment (TPR1 to TPR7 and TPO to TPV)
TPR1 to TPR7 and TPO to TPV test pits were excavated at various locations around the left abutment to assess the rippability of rock and to calibrate the seismic lines. TPR1 to TPR7 were excavated during October and November 2011 using a 22 tonne digger. TPO to TPV were excavated during August 2012 using a 34 tonne digger. The pits encountered varying amounts of colluvial and solifluction derived soils overlying the highly to moderately weathered rock. This highly to moderately weathered rock was found to be easily excavatable using the 34 tonne diggers. TPO to TPV tended to terminate on strong to very strong, slightly to moderately weathered rock that was difficult to excavate with the 34 tonne digger.

1.4.2 Geophysical investigations

Seismic refraction surveys were carried out on the left abutment, SL 1 was located sub-parallel the line of the proposed spillway, SL 2 was along the left abutment plinth line. The seismic surveys were carried out by Opus Consultants (Appendix C). The locations of the seismic lines are shown on drawings 27425-GEO-03 to 05. The plots from both surveys are reproduced figures 4 and 5.

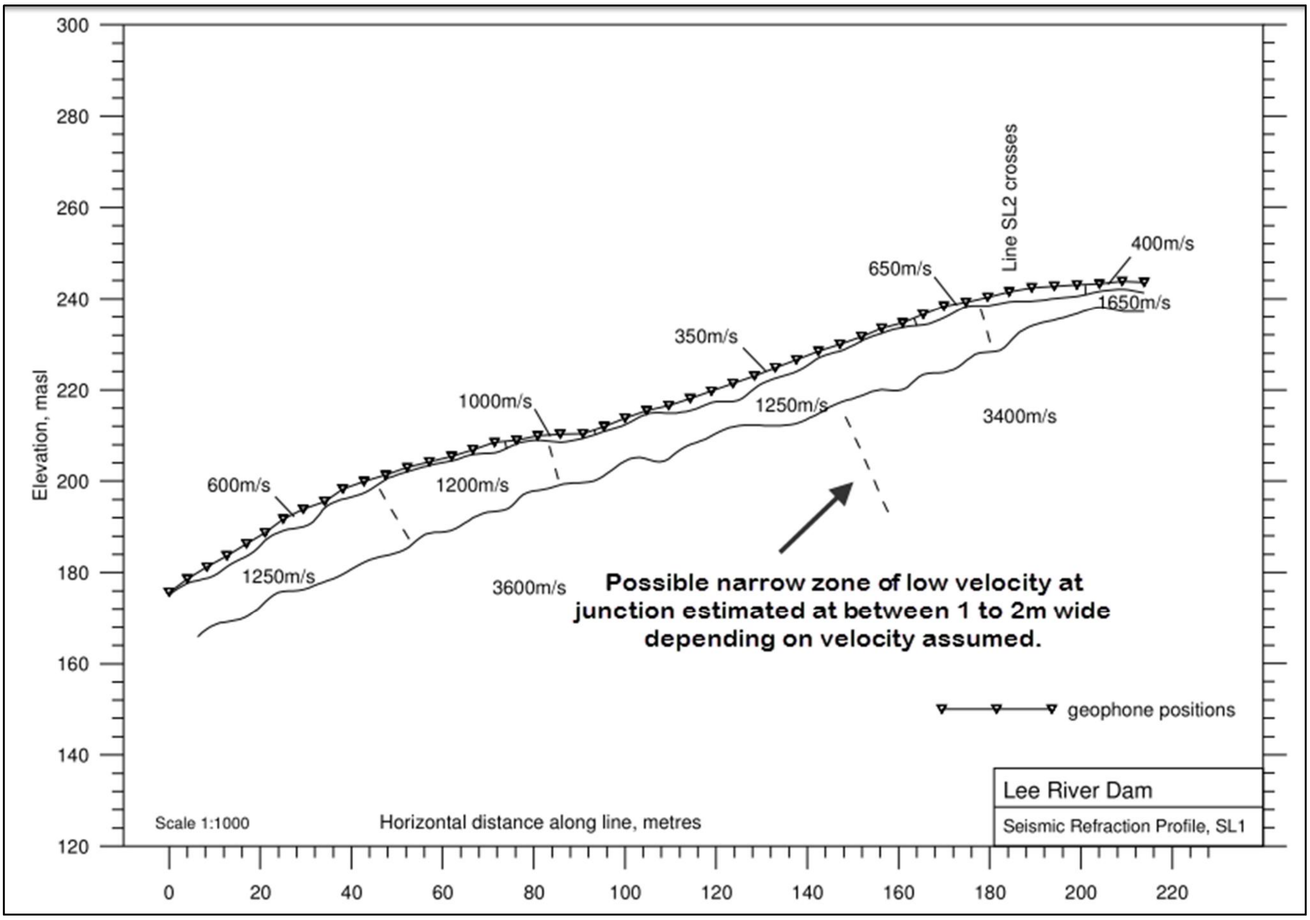


Figure 4: Seismic Line SL 1 (Spillway)

SL 1 located close to the spillway centreline shows a 10 to 20 m thickness of lower velocity rock ranging from 1200 m/s to 1250 m/s which is interpreted as either a dilated and/or weathered rock mass. This is underlain by a higher velocity zone of 3400 m/s on the upper slopes and a 3600 m/s zone on the lower slopes. A 1.5 millisecond time step was observed in the bedrock at the junction between the 3400 m/s and 3600 m/s velocities at about Ch 150 m on the survey line. This could be interpreted as a 2m wide zone with a velocity of 1000 m/s or a 1m wide zone with velocity of 600 m/s.

The second line SL 2, located along the left abutment plinth line, shows 8 to 10 m of lower velocity materials overlying bedrock. There is no obvious indication of “terrace gravels” in the seismic data. However, they were identified in the feasibility investigations, to be present near the downslope end of the 1000 m/s zone between Ch 50 m and Ch 90 m on the survey line.

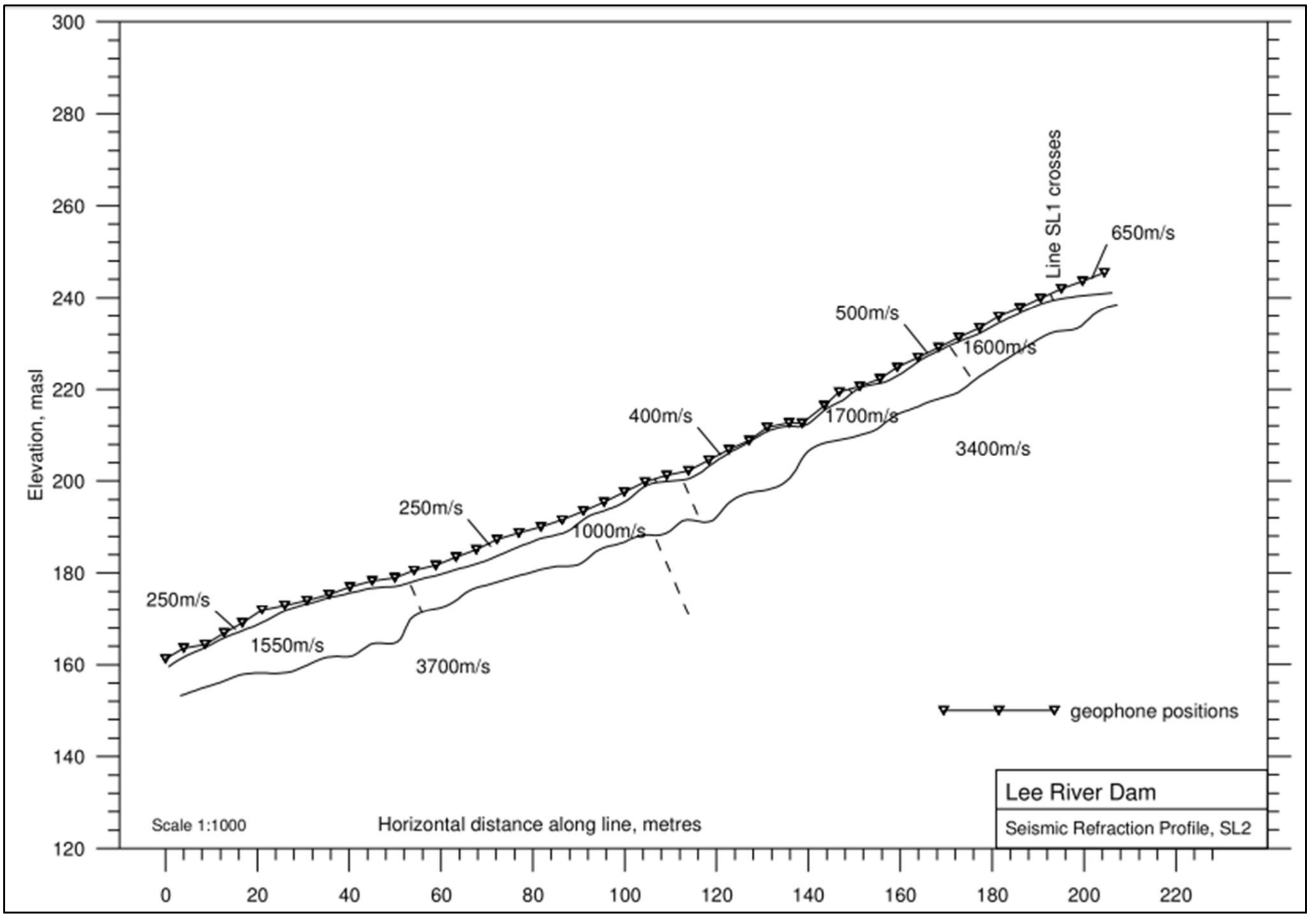


Figure 5: Seismic Line SL2 left abutment plinth line

1.4.3 Drilling and in-situ rock permeability testing

The diamond drilling programme for the detailed design investigations was started in November 2011 and was completed February 2012. Seven drill holes were completed (DH10 onwards), three on the right abutment and four on the left abutment. All drill holes were drilled with HQ wireline gear and logged at a scale of 1:25 in accordance with New Zealand Geotechnical Society guidelines (2005). Water pressure testing was carried in all drill holes and core from the holes was stored in core boxes and placed in lay-flat plastic sheeting. Casagrande type piezometers were installed in drill holes. The locations of the drill holes are shown on Drawing 27425-GEO-04. The detailed design drill holes are summarised in Table 2. The drill hole logs are provided in Appendix D.

Table 2: Summary of detailed design drill holes.

Drill hole	Coordinates (NZTM)		RL m	Angle	Direction	Length	Piezometer screen details	
	mE	mN					Number	m
DH5	2523325	5970630	213	90°	n.a.	21.0m	DH5/1	14-21
DH6	2523308	5970720	201	90°	n.a.	34.0m	DH6/1	28-34
DH7	2523408	5970639	182	90°	n.a.	32.0m	DH7/1	25-31
DH8	2523491	5970783	194	90°	n.a.	38.1m	DH8/1	32-38
DH9	2523511	5970685	153	90°	n.a.	15.0m	DH9/1	9-15
DH10	1613513	5409059	189.5	60°	N130°	48.0m	DH10/1	14-20
							DH10/2	29-33
DH11	1613519	5409025	162.1	52°	N326°	19.0m	DH11/1	13-19
DH12	1613453	5408944	170.1	90°	n.a.	18.5m	DH12/1	3-10
							DH12/2	15-18.2
DH12A	1613453	5408941	171.3	50°	N260°	19.5m	n.a.	n.a.
DH13	1613398	5408915	189.9	90°	n.a.	20.1m	DH13/1	5.4-11.4
							n.a.	n.a.
DH13A	1613413	5408918	185.7	55°	N224°	24.4m	n.a.	n.a.
DH14	1613518	5408991	153.1	50°	N235°	23.5m	n.a.	n.a.

1.4.4 Field compaction trials

Field compaction trials were carried out at two locations labelled TPR 2 and TPR 3 respectively on Drawing 27425-GEO-04. Test Pad 1 was constructed using Class 3 rock and Test Pad 2 was constructed using Class 2 rock. The pads were 2.6 m and 1.0 m thick respectively and constructed in 300 to 400 mm lifts. The rock fill was compacted following each lift by eight to ten passes of a 7.5 tonne vibrating smooth drum roller. The compaction was tested following each lift using a nuclear densometer and Clegg hammer (see Appendix E). The testing carried out on the pads is described in sections 1.6.4.1 to 1.6.4.3.

1.4.4.1 Plate bearing tests

A plate bearing test was carried out on each test pad sometime after compaction with the reaction load provided by a 22.5 tonne digger.

1.4.4.2 Density

To calculate the density of the locally sourced rock fill material a sample was taken from Test Pad 1. The soil was removed to create a cylinder that was 800 mm diameter by 630 mm deep. This material was weighed and the bulk density calculated as 2.25 ± 0.1 t/m^3.

1.4.4.3 Permeability

A falling head test was carried out following the removal of the sample for density testing and grading. The test involved lining the excavation with PVC and filling it with water. The PVC sheet was quickly removed and the water took four minutes to completely drain. Using formulae set out in British Standard BS5930 the permeability of the compacted rock fill has been calculated as 1.4×10^{-3} m/s.

1.4.5 Reservoir mapping

A reconnaissance level stability review has been undertaken of slopes around the reservoir margin. The review included a walkover of the upper reaches of Waterfall Creek, and the left bank slopes of the Lee valley upstream of the dam where extensive solifluction deposits blanket moderately steep slopes around the reservoir margin.

Twelve areas are identified as having potential for slope instability following reservoir impoundment. They are shown on Drawing 27425.200-GEO-09. They include the following, where there is a greater likelihood of future slope instability impacting on the reservoir:

- Left bank of reservoir 500 m to 800 m upstream of dam. Prominent lineaments are recognised near the crest of the slope, and more recent shallow instability is evident from the LiDAR survey.
- Left bank of reservoir between 1100 and 1400 m upstream from dam. A debris slide has been identified at this location with a slope gradient that steepens towards the valley floor.
- Scree deposit immediately upstream of the dam right abutment. Although the volume is not large this deposit is in close proximity to the diversion and intake.
- Upstream end of Waterfall Creek arm of reservoir. Recent forestry operations have cleared the Waterfall Creek slopes of vegetation of plantation exposing areas underlain by disturbed Rai Formation greywacke and Croisilles Melange. Large areas of instability were noted but they were generally shallow, however while the geomorphology is complex the areas are at the upstream end of the reservoir.

1.4.6 LiDAR and site survey

Topographical information used in the geotechnical investigation comprised:

i. Light Detection and Ranging data (LiDAR) provided by New Zealand Aerial Mapping Ltd (NZAM), who also supplied ortho-corrected photos from digital imagery captured at the same time as the LiDAR on 18 May 2011

ii. Ground survey data provided by Staig & Smith Ltd, undertaken progressively between February 2011 and March 2012

The LiDAR covers an 11 km^2 area of interest, as indicated in Figure 6. The data was supplied in terms of New Zealand Transverse Mercator (NZTM) and Nelson 1955 vertical datum. NZAM undertook comprehensive manual editing of the raw data, including changing the classification of points into and out of the ground point dataset by reference to the orthophotos, removing returns from water bodies, and adding supplementary points around water bodies and under bridges. The accuracy of the processed data was checked using ground survey at a bridge over the Lee River supplied by Staig & Smith Ltd. The standard deviation statistic for this site was 0.04m.

The ground survey undertaken by Staig & Smith Ltd was generally located within or very close to the proposed dam footprint, except for a cross section at Waterfall Creek (a tributary to the Lee River) and the survey at a bridge over the Lee for LiDAR verification and establishment of control on a hill. The ground survey locations are shown in cyan in Figure 6.

Aside from verification of LiDAR, the ground survey was used for locating and orienting geotechnical investigations such as drill holes, seismic refraction survey lines, trenches, tracks and test pits. The ground survey also included cross sections of the Lee River which provided topographical information where LiDAR could not be captured due to the water body. Early data was originally supplied in New Zealand Map Grid and approximate Nelson 1955 vertical datum, but was reissued (along with all subsequent data) in NZTM with corrected Nelson 1955 vertical datum.

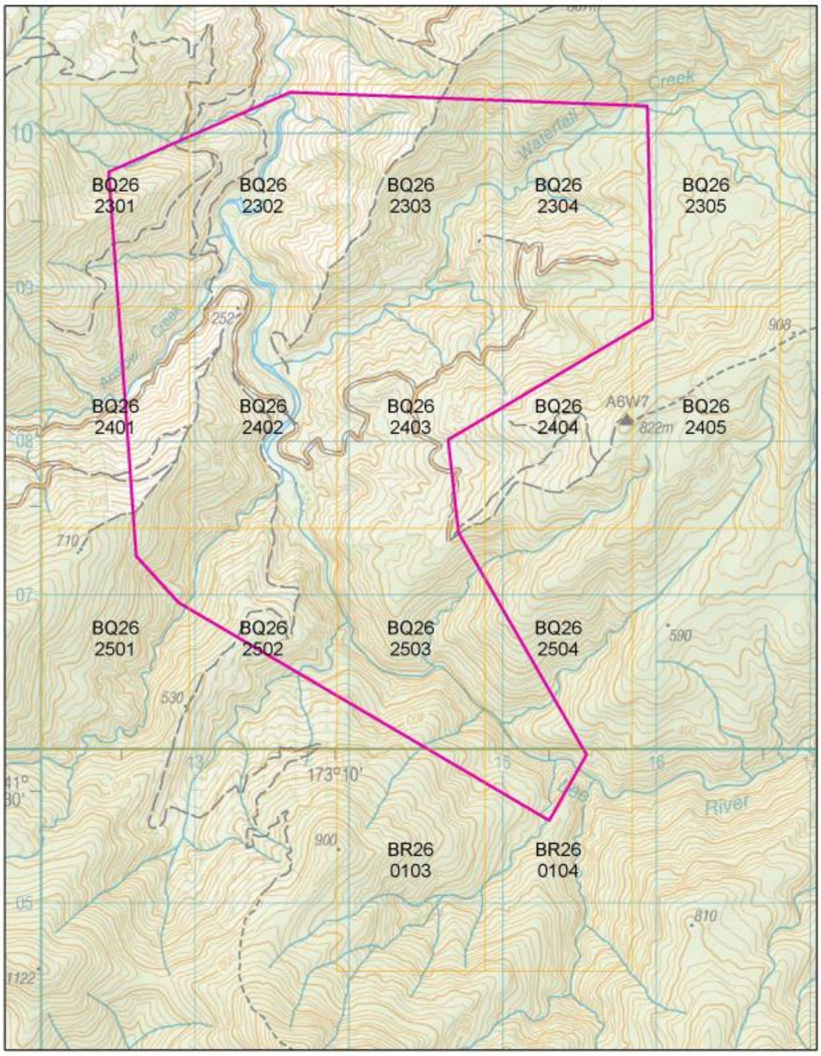


Figure 6: Area covered by LiDAR shown as rectangles labelled BQ26,2301 to BR26,0104.

2 Geomorphology

2.1 Regional geomorphology

The Richmond Ranges rise to over 1,721 m altitude in the headwaters of the Lee River, which apart from the mountain tops, are covered in indigenous forest. In the lower reaches, where the ridge crests are approximately up to 1,000 m above the Lee River, steep slopes are largely planted in exotic forestry, with narrow terraces flooring the valley in pasture.

The Lee River valley is an antecedent feature. It flows in a generally northerly direction and follows a sinuous course reflecting a likely original meander pattern that has been generally preserved as the eastern ranges have been uplifted. Within the project area, the current meander pattern is influenced by erosion resistant greywacke lithologies that have restricted lateral migration of the river while weaker siltstone/mudstone (and fault weakened rock) has been progressively eroded. Natural slope angles are influenced by the underlying bedrock lithology and locally aspect, with greywacke generally forming steeper and more uniform slopes than the Melange lithologies, and south and west facing slopes being generally steeper than north and east facing slopes. In addition to this the lower valley slopes are generally steeper than those at higher elevations which may be due to an increase in the rate of down cutting (erosion) by the Lee River.

2.2 Dam site slope features

2.2.1 Alluvial Gravels

Alluvial gravels form a thin veneer over rock in the bed of the Lee River, underlie low (2-4 m above the river) terraces beside the river and are mapped in isolated terrace remnants on the valley sides at heights of up to 60 m above the river. They are described as follows.

2.2.1.1 Low Level Terrace Gravel

Low level terrace deposits on the right bank are preserved between Ch 11,700 m and 12,000 m, 12,300 m and 12,420 m, 12,540 m and 12,600 m and in a wide fan deposit at the confluence of the Lee River and Waterfall Creek between Ch 12,800 m and 13,350 m.

Low level terraces are preserved on the left bank between Ch 12,100 m and 12,300 m.

The deposits consist of sandy GRAVEL, with less than 20% finer than coarse silt size. They include rounded boulders dominated by very strong hard green, grey and purplish-red volcaniclastic greywacke, rarely more than 0.8 m across. Clasts of weaker, finer-grained lithologies, such as argillite, are less abundant and are considerably smaller in size. Gravel clasts are typically unweathered. The deposits vary in thickness from one to three metres.

2.2.1.2 Mid Level and High Level Gravel

Mid Level gravel deposits are observed at two heights above the present valley floor. Upper-Mid Level gravel deposits up to 6 m thick occur on a poorly preserved rock bench about 15 to 20 m above river bed level, and are preserved at RL170 m on the left bank at the dam site. A less well preserved gravel layer is also locally preserved on the left bank 5 to 8 m above the river level (RL155 to 158m, Lower-Mid Level Gravel).

Isolated high level deposits, some at 40 m above the river bed and occasional deposits at 60 m above the river bed at RL200 m are preserved within the valley. At the dam site a gravel deposit is locally preserved on the left bank in the Lee Valley Road at RL 210-215 m.

The deposits consist of silty GRAVEL. Gravel clasts are moderately to highly weathered sandstone, well rounded and yellow or brown in colour. The fines fraction varies from sand to silt, with some clay. These deposits are generally capped by 1 to 6 m of slope deposits.

2.3 Reservoir Slope Deposits

2.3.1 Solifluction Deposits

Solifluction deposits are the product of periglacial physical erosion of bedrock through repeated freeze-thaw cycles.

Solifluction deposits are extensively distributed on the slopes in the Lee Valley. They locally form mappable units in excess of 10 m thick where they infill fossil gullies and form apron deposits below steep bedrock slopes. Large deposits of solifluction are mapped on the left abutment of the dam site and on the left bank upstream of the dam between Ch 12,700 m and 12,800 m and 13,000 m to 13,200 m. No large deposits have been mapped on the right bank near the dam site. Solifluction deposits are well preserved above RL170m overlying Upper-Mid Level Gravel but are not observed below the level of the Lower Mid-Level terraces (i.e. in the lowest 5 to 8 m of slope).

Solifluction deposits are stratified soil deposits, with the layering lying parallel to the slope. They are dominated gravelly SAND and sandy (fine) GRAVEL with some silt and traces of clay. Fines, when present, classify as low plasticity silt (ML). These soils are very stiff to dense. They are yellow brown in colour and the coarse fraction clasts are moderately weathered. Poorly graded fine to medium GRAVEL layers are occasionally present. These layers are highly porous and contain some re-deposited clay that binds the gravel clasts. The poorly graded gravel layers are loose.

Groundwater seepage is often observed within the solifluction deposits near or at the interface with the underlying bedrock.

2.3.2 Colluvium and Scree

Colluvium and scree deposits are formed by on-going slope erosion. In contrast to the solifluction deposits that are mainly preserved within gullies or as discrete mappable bodies, colluvium deposits are widespread and generally form a thin veneer less than 2 m thick over bedrock on slopes up to about 40°. Scree deposits are common downslope of rock bluffs, gullies and outcrops and in narrow gullies on steep slopes (greater than 35°and up to 50°), but are discrete and of limited lateral extent.

Colluvium deposits are gravelly SANDS and gravelly SILTS; gravel clasts are typically slightly weathered and include angular bedrock (scree) clasts and rounded alluvial clasts. Scree deposits are mainly medium grained GRAVEL, unweathered to slightly weathered at the surface, but will likely be more broadly graded at depth.

2.3.3 Landslide Deposits

Landslide deposits, derived from bedrock or soil slide or flow are not widespread within the immediate vicinity of the dam site or within the margins of the reservoir but do occur within the broader Lee River catchment.

A large bedrock landslide deposit in Rai Formation greywacke is present on the left bank, 500 m downstream of the proposed dam site, between Ch 11,700 and 11,900 m. This landslide has developed on a steep slope (45-50°) where bedrock defects are unfavourably oriented, and where the toe of the slope is actively eroded by a river meander. The main access road into the dam site crosses this landslide.

An ancient and eroded earthflow deposit that contains debris derived from Croisilles Melange has been mapped on the right bank at Ch 13,600 m and 14,000 m overlying a rock bench and high level alluvium at RL210 m. Large landslide deposits incorporating Croisilles Melange and Rai Formation is also inferred upstream of the reservoir extent in Waterfall Creek.

Rock fall deposits are locally evident at the foot of bluffs, mainly between Ch 13,300 m and 13,800 m on the right bank.

Landslide deposits derived from recent slippage involving solifluction, colluvium and scree are common within steep gullies and on slopes cut to form forestry roads but are rare on the vast majority of slopes. Landslides identified around the reservoir are further discussed in Section 8.

3 Regional Geology

3.1 Regional Geological Setting

The Lee Valley dam site lies between the Gordon Range to the west and the Richmond Range to the east upstream from the confluence of the Lee and Wairoa rivers.

Lee River flows north through NE-SW trending belts of rock (terranes) separated by faults (see Drawing 27425-GEO-02). The terranes consist of indurated sandstone, siltstone and mudstone (greywacke and argillite) of the Upper Palaeozoic to Lower Mesozoic aged Caples Group in the project area. A small area of Croisilles Melange, an assemblage of basalt, dolerite, and serpentinite are locally contained within the Caples Terrane in the upper Lee catchment.

3.2 Faulting and Seismicity

A number of large historical earthquakes would have been felt at the potential dam site. The magnitude and level of ground shaking at the dam site associated with recorded events documented in www.geonet.org.nz are summarised in Table 3.

Table 3: Significant historic earthquakes likely to have been experienced at the site

Earthquake	Date	Magnitude	Felt Intensity
Marlborough	1848	M7.8	MMVII
Murchison	1929	M7.8	MM VII-MMIII
Inangahua	1968	M7.1	MMV-MMVI

Peak ground accelerations for these events would have been in the range <0.15g for MMV, 0.15g-0.25g for MMVII and 0.25g-0.45g for MMVIII.
The GNS New Zealand Active Faults database http://maps.gns.cri.nz/website/af/viewer.htm indicates that seismic hazard at the site is dominated by the Alpine Fault (Wairau Segment) located 21 km to the south-east of the site and the Waimea Fault located 8.5 km to the north-west of the dam site (see Figure 7).

A site specific seismic assessment for the proposed Lee Valley dam carried out by GNS Science (included in Appendix F) estimates the recurrence interval for displacement on the Wairau Fault is 2,500 years. A major earthquake associated with this fault could result in both lateral and vertical offsets and severe ground shaking in the vicinity of the fault. The associated earthquake is estimated to be an M_w 7.8 event.

Many segments of the faults in the Waimea–Flaxmore fault system are active, with the ground on the south-eastern side of the major faults being uplifted. The major faults in the system are, from northwest to southeast, Flaxmore, Waimea, Eighty-eight and Whangamoa (see Drawing 27425-GEO-02. The Whangamoa Fault is approximately 3.5 km west of the potential dam site but in this region it is not classed as an active fault. Active traces are associated with the Waimea Fault that is located at the western end of the Wairoa Gorge (8.5 km from the dam site).

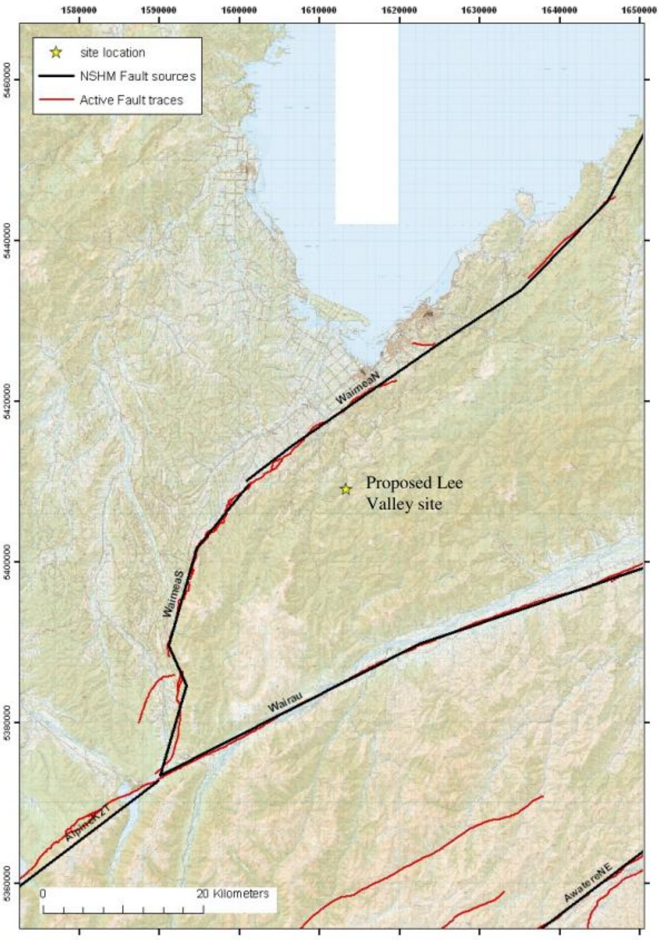


Figure 7: The locations of the Waimea Fault and Wairau section of the Alpine Fault in relation to the dam site (taken from GNS site specific seismic assessment, see Appendix F for full report).

The Waimea Fault is represented as two sources, those being the Waimea North and Waimea South faults. The seismic hazard presented by these faults has been also been assessed by GNS Science. An earthquake associated with the Waimea North Fault is estimated to be a M_w 7.4 event with a recurrence interval of 9600 years. An earthquake associated with the Waimea South Fault is estimated to be a M_w 7.0 event with a recurrence interval of 5600 years.

There are several other faults mapped within the Richmond Ranges. The following faults have been reviewed as part of this study as being in regional proximity to the proposed dam, but are not considered to be active, (M Johnston pers comm).

- *Lucy Creek Fault*: It forms the boundary between the Caples Terrane rocks and Patuki Melange. The contact is generally poorly exposed and varies from between 35 and 200m wide. It is offset by other faults.
- *Anslow Fault:* The Anslow Fault is best exposed in Anslow Creek adjacent to a culvert on the main forestry access road to the dam site. At this locality there is a zone of crushed Rai Formation rocks about 30 m wide. It is inferred to splay into two or more segments north-east of the Lee Valley. The fault is assessed (M Johnston pers comm) as a relatively minor one and there is no evidence that it is active.

- *Faults adjacent to the Croisilles Melange*: Several north-east trending lineations are associated with the Croisilles Melange and it appears that several landslides have originated where serpentinitic rocks are sheared out along faults.
- *Wards Pass and Totara Saddle Faults*: The Wards Pass Fault is a relatively major fault with a well developed crushed zone and has been traced from the Alpine Fault northwards into the Wairoa catchment where it crosses the Lee River 3.5 km upstream of the potential dam site. North of the dam site the fault has not been identified. Approximately 3 km north of the proposed dam site is the Totara Saddle Fault, which trends ENE and appears to be the most south-western part of the Queen Charlotte Fault Zone. Neither the Wards Pass nor the Totara Saddle Fault displays evidence indicating that it is active.
- *Intraformational Faults within the Rai Formation:* Several crushed and sheared zones, trending both north-east and north-west, are recognised within the Rai Formation in the vicinity of the project area. They are aligned parallel to the major tectonic faults and also are common at lithological contacts.

4 Damsite Geology

4.1 Stratigraphy and rock types

The Rai Formation is the foundation bedrock at the proposed dam site and is the predominant bedrock exposed in the reservoir. It consists of Palaeozoic age, moderately strong to strong jointed greywacke (well indurated fine sandstone) and argillite (well indurated siltstone and mudstone) that is commonly fissile. There is only limited exposure of mudstone sequences. The bend in the Lee River at the dam site appears to be controlled by a NE to SW trending thick bed of sandstone that outcrops on the right bank of the river immediately upstream of the bend.

Bedded sequences of both well indurated sandstone and siltstone/mudstone dominate the Rai Formation and individual beds vary considerably in thickness. Bedding throughout the area dips predominantly to the north-west and meso-folding within the sequence is common, particularly within the argillaceous rocks. Individual beds may not be continuous over large distances.

The Rai Formation has been subject to two periods of major tectonism and appears to have undergone deformation including shearing prior to metamorphism during the first period (Rangitata Orogeny). Thus original bedding planes have been sheared then re-healed by quartz recrystallisation during this phase of metamorphism (annealed). However, a preferred weakness still existed along bedding planes and subsequent phases of tectonic deformation (Kaikoura Orogeny) and local deformation of slopes by creep and/or seismic shaking has led to localised re-shearing along bedding especially where mudstone (argillite) beds are present.

4.2 Structure

The rock mass structure at the dam site consists of a sequence of Palaeozoic aged indurated greywacke sandstone and siltstone/mudstone (argillite) beds generally dipping at moderate to steep angles to the north-west within the dam and spillway foundations and at moderate to steep angles to the south-east just within the downstream toe in the vicinity of the plunge pool. The change in dip direction may be due to the presence of a synform or sheared synform axis trending north-east to south-west although the fold axis has not been located.

Crushed and sheared zones are present within the rock mass, many zones are along the contacts between sandstone lithology and siltstone/mudstone lithology where the siltstone/mudstone has suffered deformation by shearing during tectonic movement where located between beds of massive sandstone.

4.3 Rock material description

The Rai Sandstone of Ward Formation (Late Permian aged; see Figure 8 taken from ref: Nelson QMap) rock mass consists typically of slightly weathered, moderately strong to strong intact material, light grey to grey muddy fine to medium sandstone, with zones of finely laminated dark grey to black siltstone/mudstone. The siltstone/mudstone comprises up to 50% of the rock mass. Bedding is typically steeply inclined with a generally consistent NE to SW strike. The siltstone/mudstone dominated and thinly interbedded sequences have minor zones of bedding parallel thin sheared zones and partings where they are in contact with the sandstone and poorly persistent orthogonal defect sets.

The Rai Sandstone has been subjected to at least two phases of tectonism resulting in a complex occurrence of defects both macroscopic and microscopic in scale and, in this respect, is similar to many other NZ greywacke rocks.

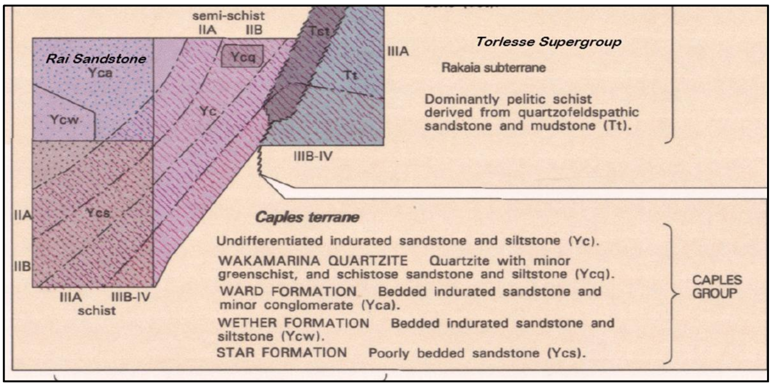


Figure 8: Stratigraphic relationship of Rai Sandstone with other NZ Greywacke rocks (copied from Nelson QMap).

For the purposes of this project the Feasibility study separated the rock underlying the site into three rock classes based on weathering and strength. These are:

- Class 1 – Unweathered to slightly weathered, strong to very strong, grey brown siltstone/mudstone and sandstone
- Class 2 – Slightly weathered, moderately strong to strong, grey brown siltstone/mudstone and sandstone
- Class 3 – Highly to moderately weathered, weak to moderately strong, green to brown siltstone/mudstone and sandstone

Our interpretation of the depths to the various rock types are provided in the drawings in Appendix A. Various other cross sections drafted during the course of the site investigations have also been provided for information only in Appendix G. All of the cross sections are interpretations based on the investigations carried out on site and should be treated as indicative only. The geology may vary away from the exploratory hole positions. In addition, seismic refraction is often used to aid the delineation of boundaries of similar rock mass properties. However, seismic refraction averages these properties and it cannot delineate small or local variations in the rock mass. It must be appreciated that the geology encountered during the construction works may vary from that shown on our sections. .

4.4 Rock mass defects

Defects in the rock mass are defined by ISRM (1978) as any mechanical discontinuity having zero or low tensile strength. They include open, filled or healed joints (no visible displacement); weak bedding planes; and faults.

Faults are defined by ISRM (1978) as having visible displacements ranging from a few millimetres to hundreds of metres. Within the study area faults refer to large scale features that are mappable at a scale of 1:20,000 to differentiate them from outcrop scale discrete crushed and sheared zones which have been observed throughout the rock mass.

Representative defect orientation data were collected during the engineering geological mapping and are presented in the following sections. Defects with persistence of less than 3m were not sampled.

Rock mass defects measured from both outcrop and test pit excavations have been plotted on stereographic projection covering the area for both the left and right abutment plinth excavations and the spillway excavation (Figure 9). Mean defect orientations are summarised in Table 4.

The most comprehensive defect data was collected on the right abutment plinth line where the rock is well exposed in the river bed, local steep faces and in the plinth and drill hole investigation access cuts. The defect data has been used for a kinematic stability review of the right bank plinth and spillway excavations which are reported in a following section.

Table 4: Combined Mean Defect Data from Dam Site.

Defect Set	Dip →Dip Direction*	Notes
Bedding	55°→317°	Bedding parting
Sets A/C	42°→070°	Structural joint
Set B	88°→068°	Structural joint
Set D	55°→141°	Structural joint conjugated to bedding

*Dip angles and directions are mean values based on lower hemisphere stereonet projections. Actual dip and dip directions vary across the site.

Rock mass defects (not oriented) were logged in all drill cores and dip magnitudes along with joint surface condition and joint frequency recorded.

Of note was that silt infilling within the defect apertures was recorded in DH10 (angled hole) down to 40m depth, DH 11 to 10m depth, DH 14 to 23.5m depth, DH 12/12A to 17m depth, and DH 13/13A to 19m depth.

This observation is relevant to both the Lugeon testing and grouting requirements. The presence of silt indicates that the defects have both open aperture and wash-out potential. The open aperture may be as a result of valley stress relief within a mature valley profile that has undergone down cutting in geologically recent times.

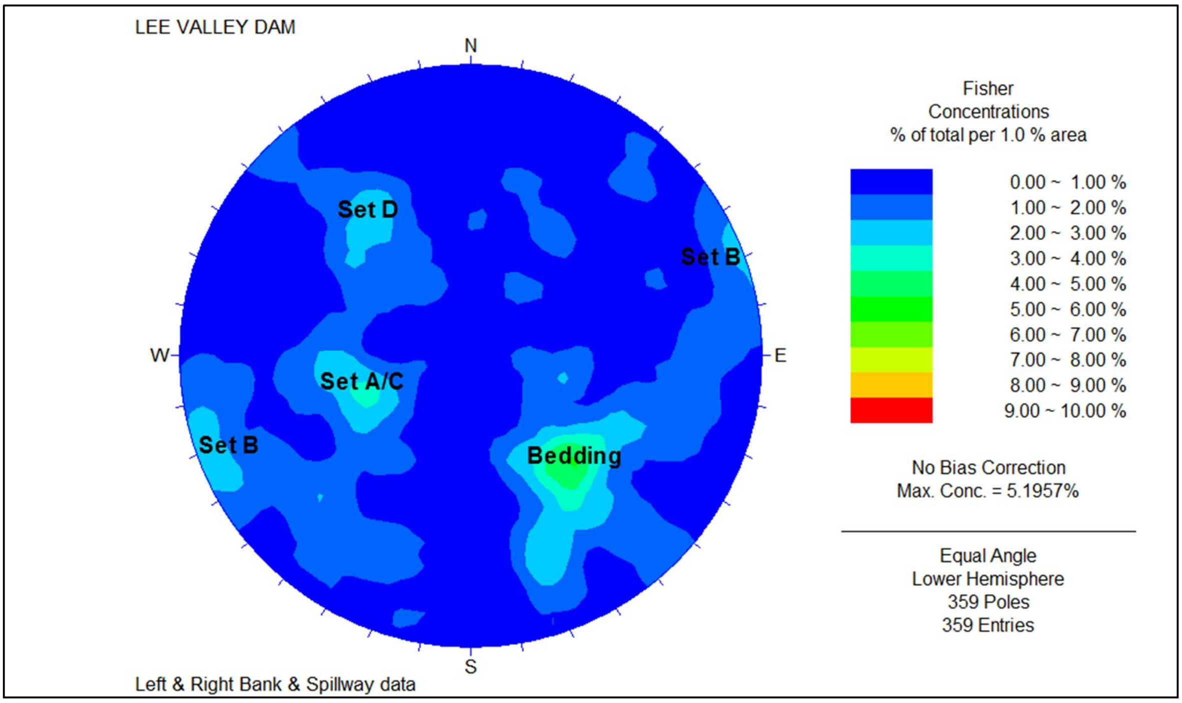


Figure 9: Stereonet projection of all defect data collected along the plinth and spillway alignments.

The defect patterns also give some indication on the use of Hoek-Brown failure criteria (see Figure 10, taken from Read et al., 2003).

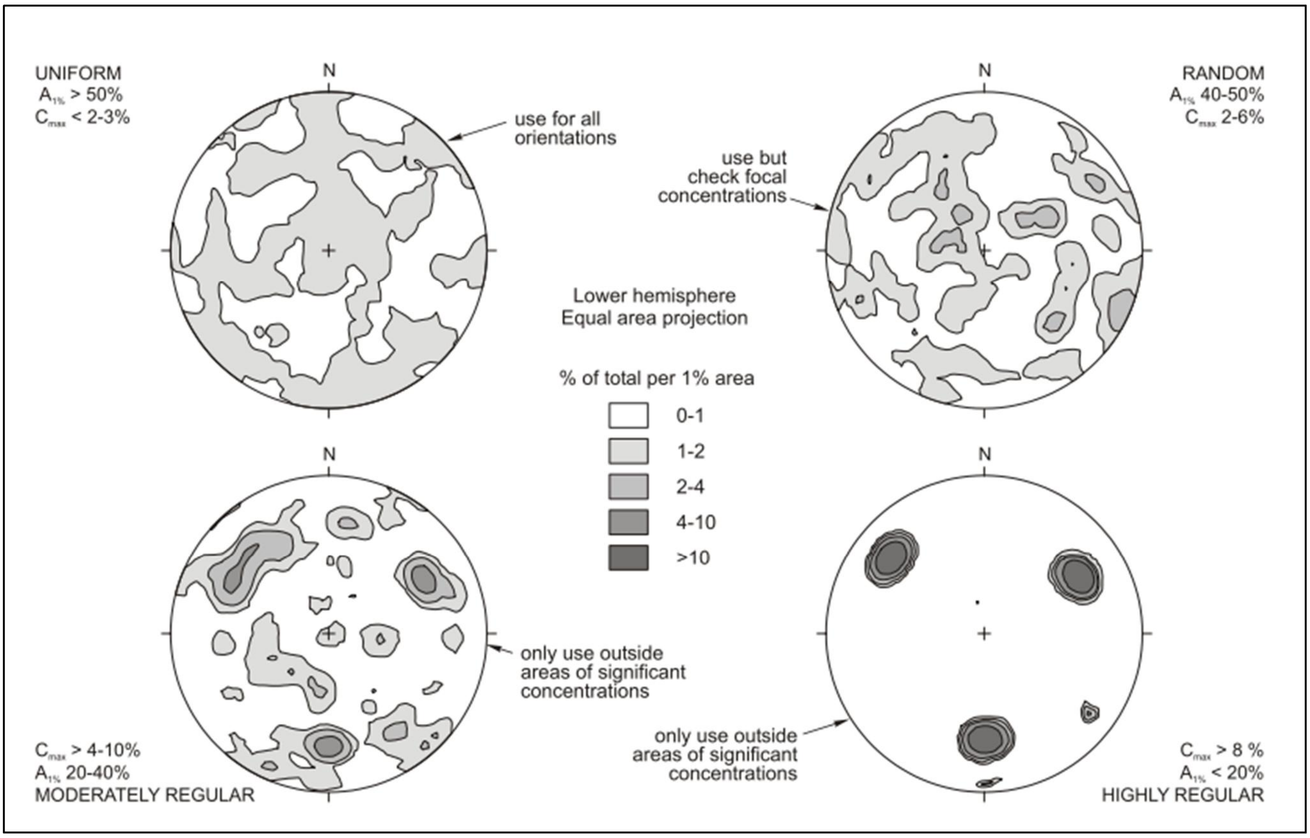


Figure 10: Guidelines for use of Hoek-Brown failure criteria based on defect patterns (from Read et al., 2003)

The defect pattern at Lee Valley is moderately regular indicating that both kinematic analyses (defect orientations, defect intersections where there are areas of significant concentrations) and Hoek-Brown failure criteria (outside areas of significant concentrations) are required.

4.4.1 Bedding

Within the dam footprint bedding (as seen by the interbedded siltstone/mudstone) dips at moderate to steep angles to the northwest. The variation in dip is thought to be due to local crumpling of the bedding by tectonic forces creating a cascading effect.

Just within the downstream dam footprint and within the spillway flip bucket/plunge pool area the bedding abruptly changes dip direction to the southeast. The reason for this change is not known. No structural features that control this change in bedding attitude have been identified in exposures at the site, but the simplest explanation would be the presence of a synform or sheared (and healed) synform axis.

4.4.2 Crushed and sheared zones

A number of sheared zones were identified in the Feasibility Investigations Report based on mapping, drill core logging or inferred from topography. Most sheared zones were bedding parallel where siltstone/mudstone beds have been sheared and crushed between more competent sandstone beds and these zones varied from 20mm thick to about 2m wide incipient zones of shattered rock containing clay filled crushed seams. The sheared zones are difficult to trace laterally as they bifurcate and anastomose along strike. The Feasibility Investigations Report noted that persistent sheared zones are spaced at 10 to 50 m intervals.

The Feasibility Investigations Report identified thirteen sheared zones of which six (SZ1, SZ3, SZ8, SZ10, SZ12 and SZ13) could be present in the plinth excavation and three (SZ6, SZ8, and SZ10) could lie within the spillway excavation. Further sheared and crushed zones not observed during this study may be exposed during the site works.

The present phase of drilling investigations was focused on the plinth excavation/plinth line and DH10 drilled as part of this phase intersected a significant sheared zone at 31.6 m which may indicate that one of the zones mapped in outcrop on the right bank of the river crosses the right bank plinth excavation at Chainage 220m. This sheared zone in DH10 comprises 100mm of plastic clayey gouge with angular rock fragments throughout. This is typical of crushed zones in greywacke terrain; the gouge is usually well graded with a low clay content, which gives rise to a low plasticity. The amount of clay present in the gouge zones increases towards the most recently active planes of shearing. The gravel content usually consists of angular, interlocking blocks of mudstone/siltstone. Photographs of this sheared zone is presented in figures 11 and 12.

Figure 11: DH10 sheared zone at 31.6 m as recovered.

Figure 12: DH10 Shear zone with plastic clayey gouge and rock fragments at 31.6m

4.5 Rock mass characteristics

4.5.1 Drill core RQD

Rock Quality Designation (RQD) was measured during drilling as it is an input parameter for most rock mass classification systems (Deere, D.U. et al, 1967). RQD is defined as the percentage of intact core pieces longer than 10cm in the total length for each core run as shown below (Figure 13).

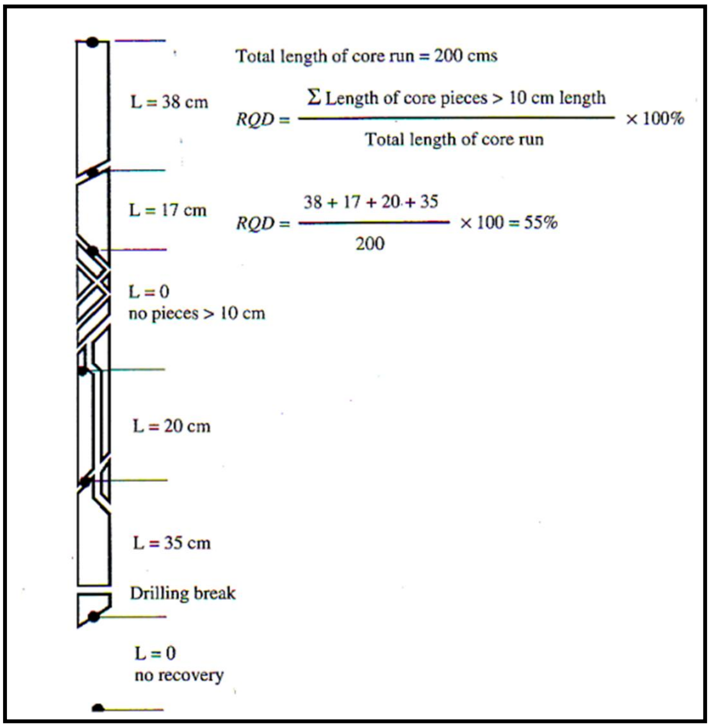


Figure 13: Graphic description of showing the method for measuring Rock Quality Designation (taken from Deere et al., 1967).

RQD data for each drill hole is presented in figures 14 to 20 and also shown on the drill hole logs along with the frequency of naturally occurring fractures.

New Zealand greywacke rock mass is typically well jointed and Cook (2001) noted from detailed scan-line mapping of New Zealand greywacke that joint persistent was short, typically less than 0.5m, due to the highly fractured mass in which it occurs. He further commented that while individual joints intersect and truncate each other and appear to be irregularly oriented, stereographic analysis may indicate that only two or three dominant joints sets are present.

On a smaller scale the rock mass may contain micro-fractures with a persistence of less than 5cm.

Together, the varying scales of defect spacing usually combine to give core retrieved from a greywacke rock mass a low RQD. Consequently, RQD by itself is not a particularly good indicator of overall rock mass quality and this aspect has been thoroughly researched in New Zealand by S.A.L. Read, R.L. Richards and others whose works are discussed in the following section.

RQD values varied widely in the recently drilled drill holes as shown in figures 14 to 20 below. The drill hole order runs from extreme left abutment to right abutment.

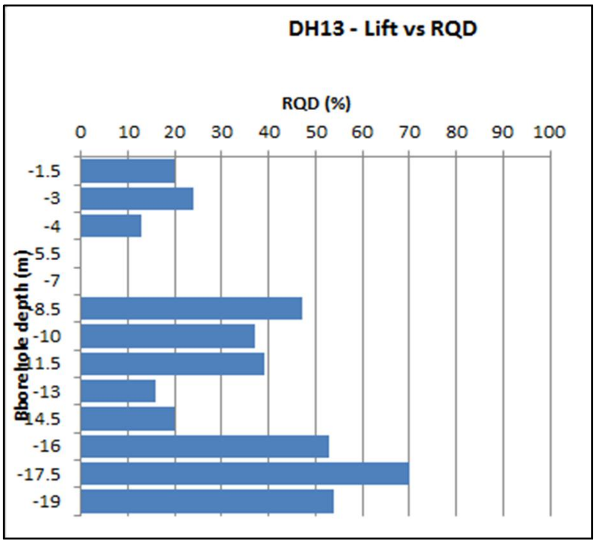


Figure 14: RQD versus depth for DH13

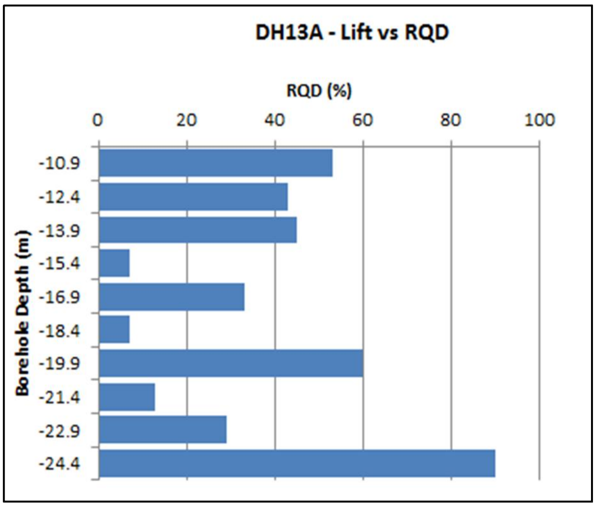


Figure 15: RQD versus depth for DH13A

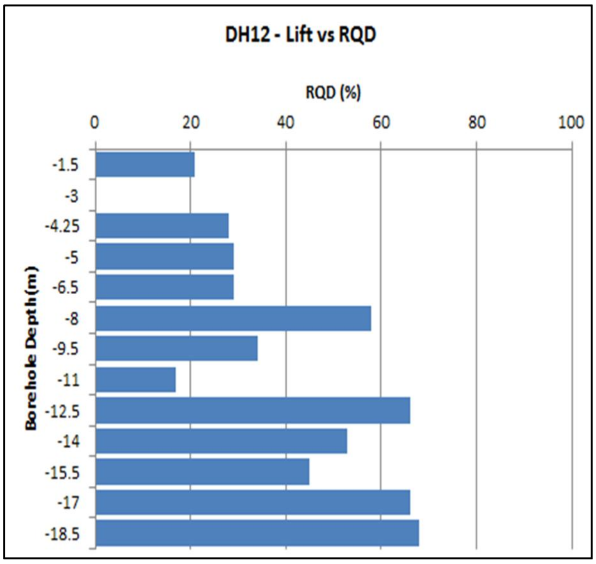


Figure 16: RQD versus depth for DH12

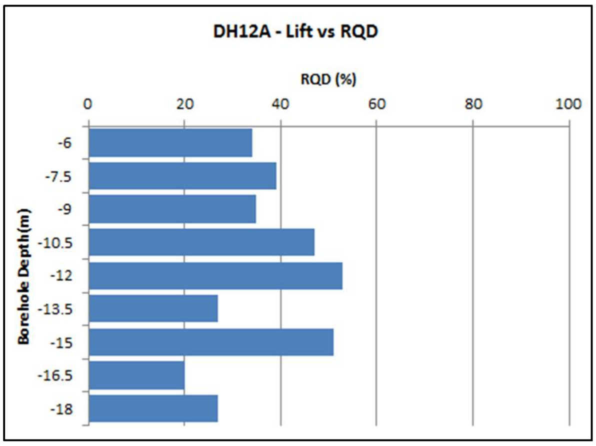


Figure 17: RQD versus depth for DH12A

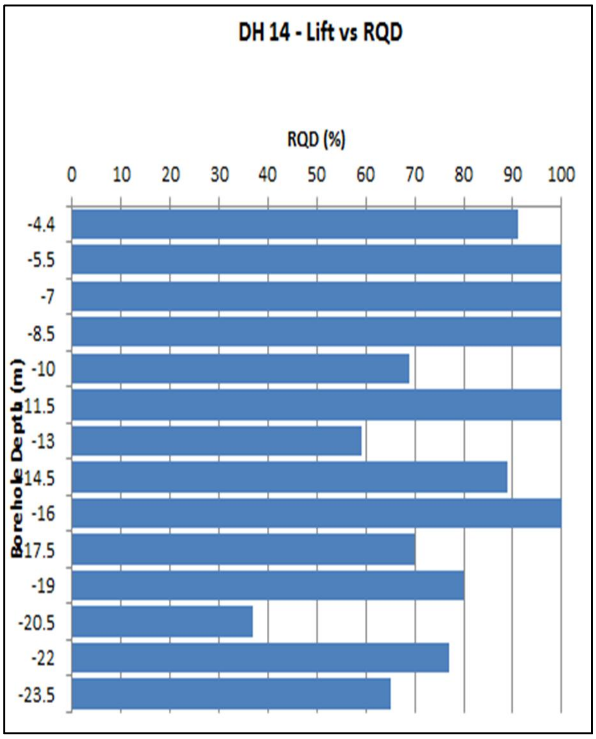


Figure 18: RQD versus depth for DH14

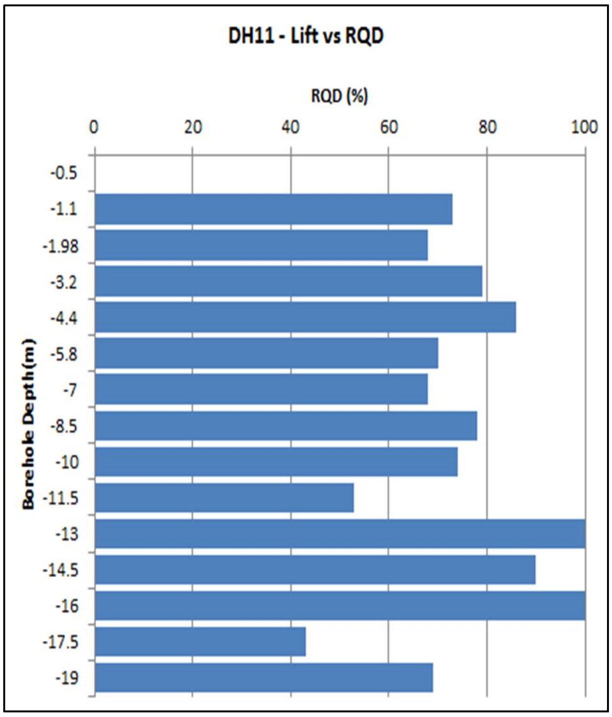


Figure 19: RQD versus depth for DH11

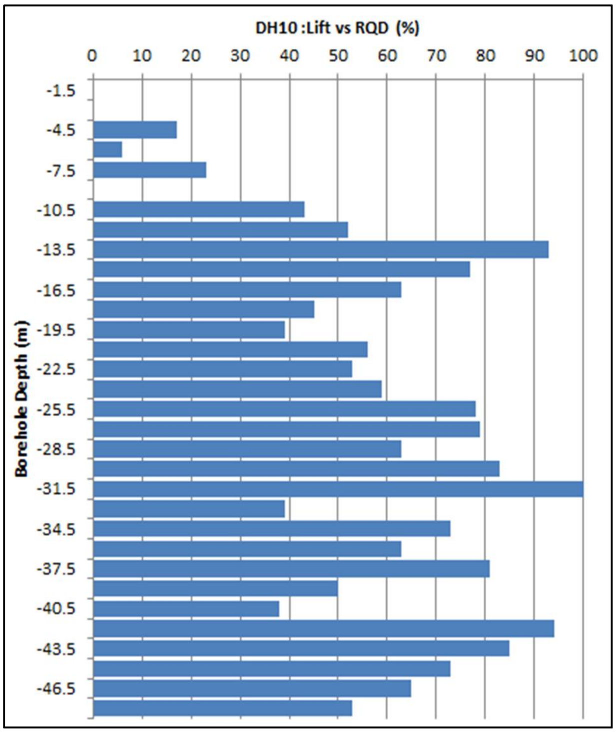


Figure 20: RQD versus depth for DH110

4.5.2 Rock mass GSI

Geotechnical characteristics of the rock mass have been classified using a modified Geological Strength Index (GSI) adapted for poor quality rock masses (Sonmez and Ulusay, 1999). Sonmez and Ulusay state that Bieniawski's (1993) RMR classification scheme requires unrealistic rating adjustments in very poor quality rock masses. The modified GSI classification has a Structural Rating (SR) based on the volumetric joint count and a Surface Condition Rating (SCR) based on scoring weathering, joint roughness and joint infilling. Both SR and SCR may be estimated from drill core. Another advantage of GSI is that it is an input parameter for Hoek-Brown failure criteria.

A rock mass classification system for New Zealand greywacke (Table 5) has been developed based on the characterisation of the rock mass quality (Read et al., 1998 and Read et al., 1999, Read et al., 2000). The classification recognises five classes of greywacke rock mass ranging through very blocky to foliated/laminated/sheared in an unweathered rock mass.

The classification system provides a basis for assessing rock mass conditions to be encountered in foundation excavations. The informal descriptive classification developed by Read is presented below.

It is of interest to note that, weathering aside, the range of rock strength for the five classes is:

- Class 1 UCS range is 100 to >250 MPa
- Class 2 UCS range is 50 to 100 MPa
- Class 3 UCS range is 20 to 50 MPa
- Class 4 UCS range is 20 to 50 MPa
- Class 5 UCS range is 20 to 50 MPa

That is, strength taken alone is not diagnostic of any particular class.

Table 5: rock mass classification system for New Zealand unweathered greywacke (after Read et al., 2000).

Descriptive classification for New Zealand greywacke rocks (after Read et al. 2000).

Class	Lithology	Strength	Defects	Comments
I	Homogeneous or faintly bedded medium-grained sandstone. Fine-grained sandstone with some widely spaced interbeds of mudstone.	Extremely strong to very strong	Joint spacing > 150mm, typically 200 – 300 mm, surfaces rough to smooth. Sheared, crushed or shattered zones generally absent.	Little indication of major tectonic deformation in rock mass.
II	Fine or very fine-grained sandstone with mudstone laminae. Interbedded sandstone and mudstone. Mudstone / sandstone with coarse podding.	Very strong to strong	Joint spacing 60 – 200mm, surfaces rough to slickensided. Minor narrow (< 300mm wide) sheared, crushed or shattered zones.	Rock mass may contain minor very widely spaced zones of sheared and crushed rock.
III	Mudstone with extensive recrystallisation. Interbedded sandstone and. mudstone, often with podding and some veining.	Strong to moderately strong	Joint spacing < 100mm, surfaces smooth to slickensided. Narrow (< 300mm wide) sheared, crushed, or shattered zones.	Characterized by closely spaced defects (may be shattered) or recrystallised rock mass.
IV	Interbedded sandstone and mudstone, often with extensive podding. Mudstone or very fine sandstone with extensive veining.	Strong to moderately strong	Joint spacing < 60mm, surfaces smooth to clay-lined. Sheared with crushed zones (typically < 500mm wide), and may contain thin (< 25mm) gouge zones.	Characterised by very closely spaced fractures with sheared zones (i.e. shattered and sheared rock mass with some crushed zones associated with fault zones).
V	Mudstone or fine sandstone (rock material generally sheared and crushed).	Strong to moderately strong (or n/a)	Joint spacing < 20mm, surfaces slickensided to clay-lined. Generally sheared or crushed zones which contain gouge zones.	Characterised by very or extremely closely spaced fractures with crushed zones and gouges (i.e. crushed rock mass associated with major faulting).

Classification based on rock mass in the unweathered (fresh) or fresh-stained state.
Podding refers to the disruption of bedding into irregular lenses or pods. Recrystallisation refers to recementation of the rock mass and is often accompanied by veining.

The informal classifications of Read are shown overlaid on a Geological Strength Index (GSI) chart in Figure 21.

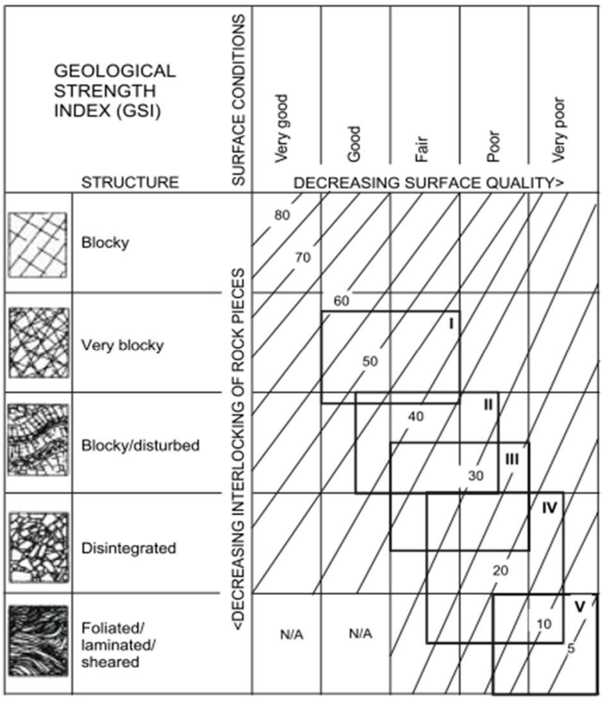


Figure 21: Read et al. (1998 & 1999) unweathered greywacke rock classes overlain on a GSI chart.

The three classes of rock mass recognised and described in the Feasibility Investigation Geotechnical Report are shown on the modified GSI chart in Figure 22 and they are broadly similar to those of Read et al (1998 & 1999) except that they take into account rock mass weathering. In a following section of this report the three rock mass classes are used to describe potential sources of rock fill for the embankment. It is important to realize that Class 2/3 material may be either unweathered blocky/disturbed/crushed rock or moderately weathered very blocky rock or a mixture of the two. However their performance as rock fill may not be comparable.

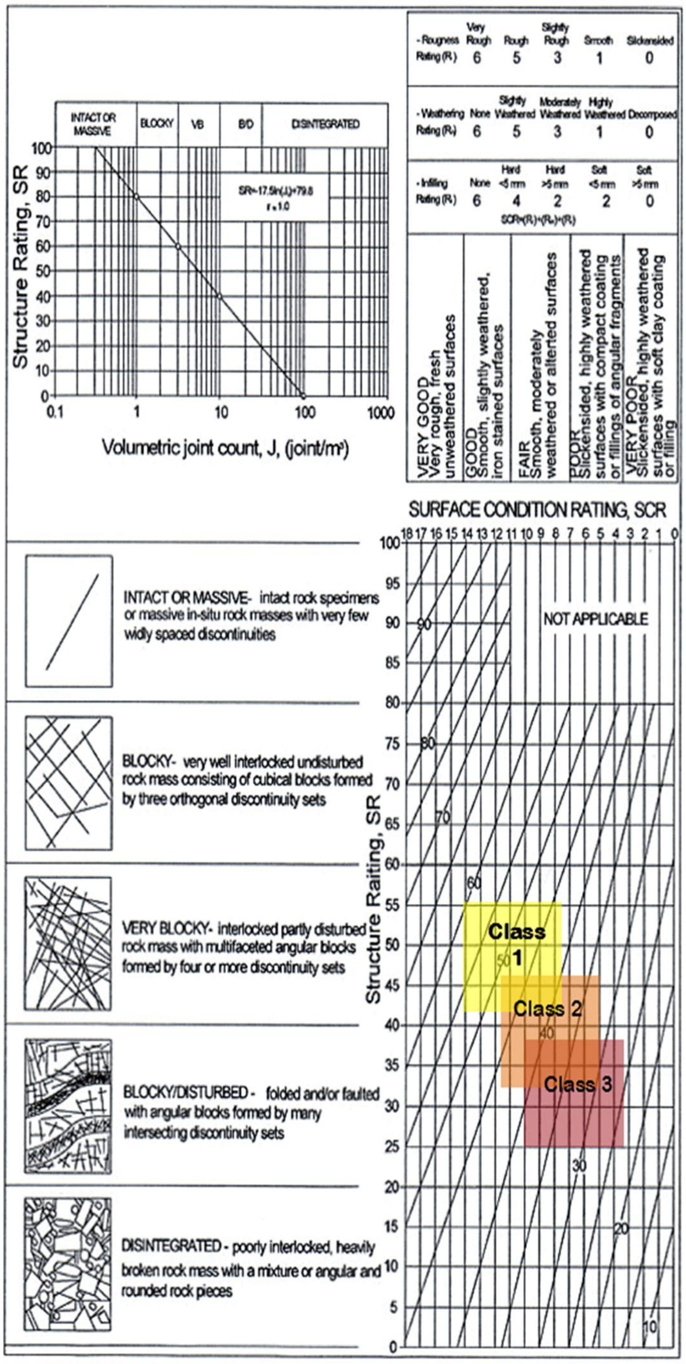


Figure 22: The three greywacke rock classes identified at the proposed Lee River dam site overlain on a GSI chart.

The rock mass classification developed for Lee River during the feasibility investigations is presented in Figure 23. The RMR classification was based on core recovered from DH 5 to DH 9.

Feature		Class 1	Class 2	Class 3
Intact Strength MPa	Range	30-70	20-50	10-20
	Average	50	35	15
Weathering		UW - SW	MW - SW	HW-MW
ROCK MASS RATING		**50-60**	**41-50**	**30-40**

Figure 23: Part of Table 3 taken from Lee Valley Dam Geotechnical Feasibility Report showing RMR for the different rock classes.

RMR and GSI for rock below the plinth excavation foundation level were assessed from the core logs of the relevant detailed design drill holes and the results are presented in Table 6. At each locality these rocks are overlain by varying thicknesses of lower Class 2 and Class 3 rock which will need to be removed during the foundation preparation.

Table 6: RMR and GSI for the rock encountered in holes drilled for the detailed design investigations.

Drill Hole	RMR	GSI	Class
DH13/13A	54	49	Lower Class1/ Upper Class2
DH 12/12A	55	50	Lower Class1/ Upper Class2
DH14	62	57	Upper Class 1
DH11	58	53	Class 1
DH10	58	53	Class 1

4.5.3 Rock mass strength and deformation moduli

No direct measurement of rock mass modulus was undertaken during the present phase of investigations. Rock mass deformation modulus was estimated using RocLab with input parameters based on field observations and is presented in Table 7. An intact modulus ratio of 1:400 was selected as mid-range for greywacke type rock.

Table 7: Rock mass strength and deformation moduli based on field observations

Class	UCS (MPa)	GSI	m_i	D	Modulus Ratio	E_{rm} (GPa)
1	30 to 70	40 to 60	12	0	400	1.9 to 14.5
2	20 to 50	30 to 50	12	0	400	0.65 to 6.1
3	10 to 20	20 to 40	12	0	400	0.18 to 1.2

There is a wide range and overlap in the determined modulus values for the classes 1 to 3.

Richards and Read (2006) report a large range in modulus ratio for New Zealand greywacke (see Figure 24).

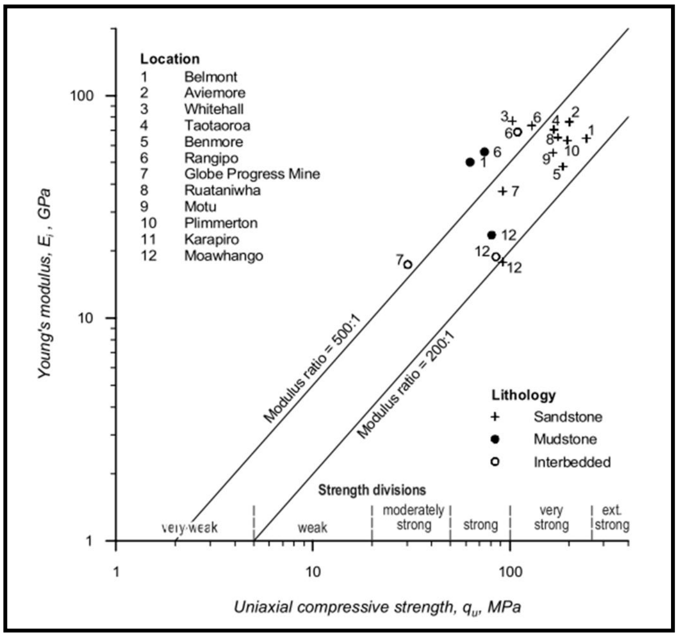


Figure 24: Measured modulus ratios for NZ Greywacke (from Richards and Read, 2006)

4.5.4 Intact rock strength

Unconfined compressive strength testing was undertaken on core samples during August 2012 and the results are summarised in Table 8.

Table 8: Unconfined compressive strength (UCS) test results

Drill Hole	Depth (m)	UCS (MPa)	E_i (MPa)	Modulus ratio	Failure Mode	Lithology
DH10	9.23-9.38	23.5	3770	160	Shear on defect	SW Siltstone
DH10	12.05-12.18	23.8	5623	236	Shear on defect	SW Siltstone
DH10	21.98-22.11	57.7	-	-	Axial	UW Siltstone
DH10	25.10-25.23	41.3	-	-	Shear	SW Siltstone
DH10	37.78-37.91	88.2	-	-	Axial	UW Siltstone/Sandstone
DH10	45.75-45.87	62.5	-	-	Axial	UW Siltstone
DH11	1.49-1.67	81.7	-	-	Axial	SW Sandstone
DH11	6.50-6.63	96.3	-	-	Axial	SW Sandstone
DH11	13.33-13.46	123.8	-	-	Axial	UW Sandstone
DH11	14.55-14.68	67.1	-	-	Axial	UW Siltstone
DH12	7.83-7.96	6.6	1362	206	Shear on defect	SW Siltstone
DH12A	11.85-11.98	50.6	-	-	Shear	UW Siltstone/Sandstone
DH13	9.53-9.66	21.1	3816	181	Shear on defect	MW Siltstone

DH13	10.05-10.18	8.0	1674	209	Shear on defect	MW Siltstone
DH13A	21.05-21.18	37.7	-	-	Shear	SW Siltstone
DH14	6.65-6.78	16.3	2648	162	Shear on defect	UW Siltstone
DH14	11.24-11.37	60.5	-	-	Axial	UW Sandstone/Siltstone
DH14	13.98-14.11	46.1	-	-	Shear	UW Sandstone/Siltstone
DH14	17.35-17.48	143.2	-	-	Axial	UW Sandstone/Siltstone
DH14	19.58-19.70	23.5	3598	153	Shear on defect	UW Sandstone

(Axial strain was not measured where samples were tested on concrete machine because of high strength)

This range in UCS strength is typical for indurated greywacke rock. Note that both axial (brittle) failure through the rock fabric and shear along defects within the core are recorded. Again this is typical for greywacke rock. Also of interest is that, where recorded, the modulus ratio is lower than depicted in Figure 24. This is likely to be due to the failure of the rock on defects and micro defects during these tests and they are unlikely to reflect the modulus of the intact rock. Due to the capabilities of the equipment the modulus could not be measured on the stronger rock samples.

4.5.5 Defect strength

No defect strength testing has been undertaken at Lee Valley Dam. Shear box testing on a similar suite of greywacke rocks at Rangipo (Bryant J. M. 1977: Shear Strength of Joints in Rangipo Rocks: MWD Central Laboratories Report No 2-77/2) gave the following results:

- Sandstone (rough joints) — $Ø_{average}$ = 33° to 38° (at 4 MPa σ_n)
- Sandstone (smooth joints) — $Ø_{average}$ = 27°
- Siltstone — $Ø_{average}$ = 35° to 38°
- Fault gouge (clay and silt with crushed rock gravel) — $Ø_{average}$ = 40° up to 2 MPa normal stress

  $Ø_{average}$ = 32° from 2 MPa to 5 MPa normal stress

The values given above have been used for wedge failure analysis. This is considered conservative and is based on small shear box samples. In practice this friction angle would have an 'i' angle added (to account for joint roughness) taking it up to between 40° and 42° as is seen in the field.

4.5.6 Rock mass seismic velocity

Rock mass seismic velocity and variability is discussed in Section 1.4.2

4.6 Permeability

4.6.1 Rock material permeability

The intact rock material in the plinth foundation excavation for the dam is indurated moderately strong to strong greywacke sandstone with interbedded indurated moderately strong to strong siltstone/mudstone. The matrix of both the sandstone and the siltstone/mudstone is recrystallized by low grade metamorphism. Porosity measurements (n%) reported for Rangipo Power Project greywacke sandstone and mudstone (Kaweka Terrane, part Torlesse Super group) by Bryant (Bryant 1977) averaged at 0.25% for sandstone and 0.34% for interbedded sandstone/siltstone/mudstone. For the purposes of this study the intact rock material permeability is taken as less than 10^{-10} m/sec.

4.6.2 Rock mass permeability

Rock mass permeability was assessed in each of the investigation drill holes by undertaking a Lugeon test at the completion of each core run (1.5m intervals). The maximum test pressures used were the lesser of overburden pressure or reservoir pressure (P_{MAX}). The Lugeon test was carried out as a "pressure loop" as shown in Table 9.

Table 9: Lugeon test carried out as a "pressure loop".

Test Stage	Description	Pressure Step
1st	Low	0.50·PMAX
2nd	Medium	0.75·PMAX
3rd	Maximum (peak)	PMAX
4th	Medium	0.75·PMAX
5th	Low	0.50·PMAX

Table 10 describes the condition of the rock mass defects associated with the different Lugeon values.

Table 10: Rock mass defects with different Lugeon values.

Lugeon Range	Classification	Hydraulic Conductivity Range (cm/sec)	Condition of Rock Mass Discontinuities	Reporting Precision (Lugeons)
<1	Very Low	$<1\times10^{-5}$	Very tight	<1
1-5	Low	1×10^{-5} - 6×10^{-5}	Tight	<1
5-15	Moderate	6×10^{-5} - 2×10^{-4}	Few partly open	±1
15-50	Medium	2×10^{-4} - 6×10^{-4}	Some open	±5
50-100	High	6×10^{-4} - 1×10^{-3}	Many open	±10
>100	Very High	$>1\times10^{-3}$	Open closely spaced or voids	>100

The data has been presented as plots of Lugeon values versus test pressure as suggested by Houlsby (1976) and flow loss versus test pressure as suggested by Camilo Quinones-Rozo (2010). The interpretation presented by Quinones-Rozo is reproduced below in Figure 25. The interpretation presented by Houlsby (1976) for dilation results is reproduced as Figure 26.

Lugeon values for the seven drill holes drilled as part of the detailed design investigations are presented in Table 11. Full test results may be found in Appendix H. Where high flows were measured during testing a pressure correction for head loss through the packer has been applied. While most of Lugeon tests showed laminar flow behaviour with low to moderate Lugeon values some tests indicated dilation of the rock mass with high Lugeon values being recorded. In these cases Camilo Quinones-Rozo (2010) suggests that the high Lugeon value be adopted as it most closely represents the water pressures that will be present during operation of the dam. However, Houlsby (1976) recommends that the high Lugeon value recorded at the maximum test pressure be ignored. Where dilation has occurred the representative lugeon value shown in Table 11 has been selected following the Houlsby (1976) method. Full details of the tests are provided in Appendix H.

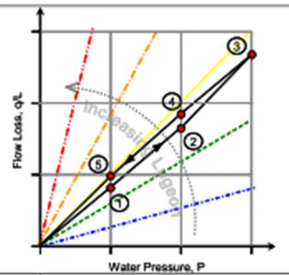
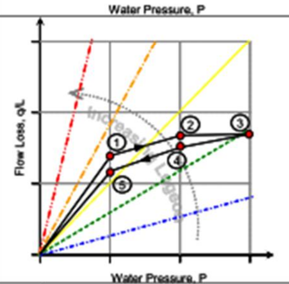
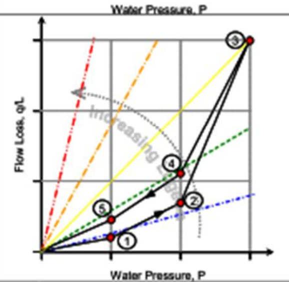
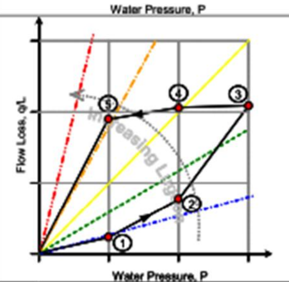
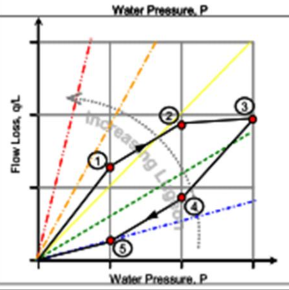
BEHAVIOR	WATER LOSS VS PRESSURE PATTERN	DESCRIPTION	REPRESENTATIVE LUGEON VALUE
LAMINAR		All Lugeon values about equal regardless of the water pressure	Average of Lugeon values for all stages
TURBULENT		Lugeon values decrease as the water pressures increase. The minimum Lugeon value is observed at the stage with the maximum water pressure	Range of Lugeon values observed at water pressures expected during operation. If water pressure expected during operation is unknown use the value corresponding to the medium water pressure (2nd or 4th stage)
DILATION		Lugeon values vary proportionally to the water pressures. The maximum Lugeon value is observed at the stage with the maximum water pressure	Range of Lugeon values observed at water pressures expected during operation. If water pressure expected during operation is unknown use the value corresponding to either low or medium water pressures (1st, 2nd, 4th, or 5th stage)
WASH-OUT		Lugeon values increase as the test proceeds. Discontinuities' infillings are progressively washed-out by the water	Highest Lugeon value recorded (5th stage)
VOID FILLING		Lugeon values decrease as the test proceeds. Either non-persistent discontinuities are progressively being filled or swelling is taking place	Use final Lugeon value (5th stage), provided that presence of non-persistent discontinuities and/or occurrence of swelling is confirmed by observation of rock core.

Figure 25: Interpretation of Lugeon data (from Camilo Quinones-Rozo, 2010)

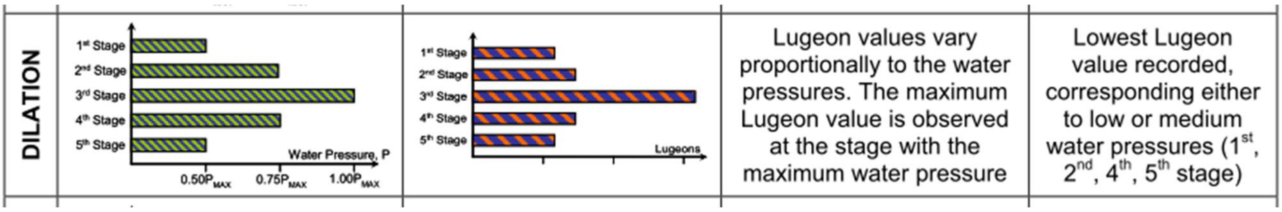


Figure 26: The interpretation of dilation lugeon values as presented by Houlsby (1976)

Table 11: Summary of Lugeon values for the detailed design drill holes. Lugeon values are for each 1.5m lift. Note that DH 12 & 13 are for 0.7m test lengths. Drill holes are ordered in the positions they are located from left to right across the valley.

DH13	DH13A	DH12	DH12A	DH14	DH11	DH10
571	1.2	10.5	1.5	29.3	7.6	1.3
529	15.6	6.9	1.1	0	8.9	10.2
498	70.3	1.3	9.6	1	67.1	3.6
3	116.7	16.9	27.3	32.7	68.6	3.5
3.9	9.7	2.6	19	33	1.5	16.5
7.9	5.4	11.3	13.5	1	70.9	95.4
2.9	6.8	5.2	9.6	45	1.3	2
17.3	1.7	5	19.8	23.2	9.8	127.5
2.3	5.7	1.3	8.9	2.1	1.6	37.4
3.5				1.8		127.4
				0.9		1.4
				10.5		9
						8.6
						1.1
						5.5
						10.9
						60
						9.6
						0.9
						1.1
						9.7
						0.9
						0.7
						0.4
						0.8
						0.4
						1
						16.4

	>100	very high
	50 - 100	high
	15 - 50	medium
	5-15	moderate
	>5	low

As noted previously silt infilling within the defect aperture was recorded in DH10 (angled hole) down to 40m depth, DH 11 to 10m depth, DH 14 to 23.5m depth, DH 12/12A to 17m depth, and DH 13/13A to 19m depth. The typical water losses versus pressure pattern for wash out conditions as shown in Figure 25 is not observed in the Lugeon test data indicating that either the silt infilling within the defects system is not continuous or that at the flow rates achieved during the testing did not erode the silt.

4.7 Groundwater

Groundwater standing levels were measured during drilling and Casagrande type standpipe piezometers are installed in drill holes DH5, 6, 7, 8, 9, 10, 12 and 13. Standing water levels during drilling were:

DH10	13 to 14.5m (measured down the angled hole) later falling to 36m measured on the angle.
DH11	1.8m (measured down the angled hole) falling to 8.2m measured on the angle
DH12	3.0m falling to 9.5m
DH12A	7.5m measured down the angle
DH13	6.0m
DH13A	9.4m measured down the angle
DH14	1.1 m measured down the angle

Groundwater levels in drill holes DH5 to DH7 have been measured by Tasman District Council since December 2009 and are provided in Appendix I. Groundwater measurements taken on 16 August 2012 are provided in Table 12. The measurements show that the groundwater rises sharply away from the river with topography. The difference in measurements between the two piezometers in DH10 also indicate the presence of possible perched water tables or compartmentalised defect bounded aquifers (at least locally) within the rock strata.

Table 12: Groundwater measurements taken from piezometers on 16 August 2012.

Piezometer screen details m		**True depth to groundwater (mbgl)** 16 August 2012
DH5/1	14-21	9.17
DH6/1	28-34	13.04
DH7/1	25-31	8.0
DH8/1	32-38	27.69
DH9/1	9-15	0.98
DH10/1	14-20	11.58
DH10/2	29-33	23.51
DH11/1	13-19	6.88
DH12/1	3-10	4.01
DH12/2	15-18.2	5.05
DH13/1	5.4-11.4	5.3

5 Dam foundation stability

5.1 General foundation stability

The requirements for the general foundation for a concrete faced rock fill embankment dam are that it is founded on a rock foundation that is stiffer/stronger than the embankment rock fill. At the site, the highly weathered/moderately weathered and less weathered rocks (Class 2 & 3) are expected to form a suitable foundation. During investigations we were able to rip the overburden and Class 3 rock with relative ease using a 22 tonne excavator.

Higher quality rock will need to be exposed along the downstream toe for the foundation and anchorage of mesh protection. Such protection is required to prevent damage to the downstream face caused by flooding.

5.2 Plinth

The plinth (also referred to as toe slab) excavation requirements are for a rock that, following grouting, is tight and non-erodible. At the site this will require excavation to at least Class 2 and most likely to Class 1 rock in areas where the hydraulic gradient is high.

The foundation for the plinth excavation was investigated by digger pitting and trenching, cored diamond drilling and on the left abutment also by seismic refraction survey. Data from these investigations are shown on a developed section along the plinth excavation.

The plinth will be founded on indurated greywacke sandstone and interbedded siltstone/mudstone with bedding plane partings dipping at moderate angle to the NW and the other main defect sets orthogonal to bedding.

The depth of the plinth excavation shown on the drawing is based on a combination of data from drill hole core (weathering, defect condition, RQD) and the various rock velocities measured during the seismic survey. The seismic velocities are shown lying within boundaries of similar velocities and by the nature of the survey technique velocity must increase with depth. Seismic refraction measures average rock mass properties that affect the velocity and often these boundaries do not relate well with geological features recognisable in drill core (rock type, weathering, defect spacing, RQD or rock mass permeability).

5.3 Plinth Excavation Cut Stability

5.3.1 Right abutment

Excavation for the plinth requires significant cutting on steep slopes on the right abutment of the proposed dam (see Figure 27). The plinth platform has typically a width of 8m cut inside of the existing rock line as shown in Figure 28 at Section 227 on the right abutment. The rock face above the plinth platform has a design cut shown at 10V to 1H to minimise the height of cut.

Defect data collected on the right abutment was analysed with Dips (DIPS version 6.006) as described in Section 2.4 and great circles produced for the main defect sets. A wedge analysis using SWEDGE version 5.015 was then undertaken for the two main intersections of the defects sets which would be day-lighted by the cut slope as shown circled in Figure 29 for section 227 (the highest section of cut). The stability of wedges was determined assuming friction only shear strength parameters with Ø' = 33° (rough joints in sandstone).

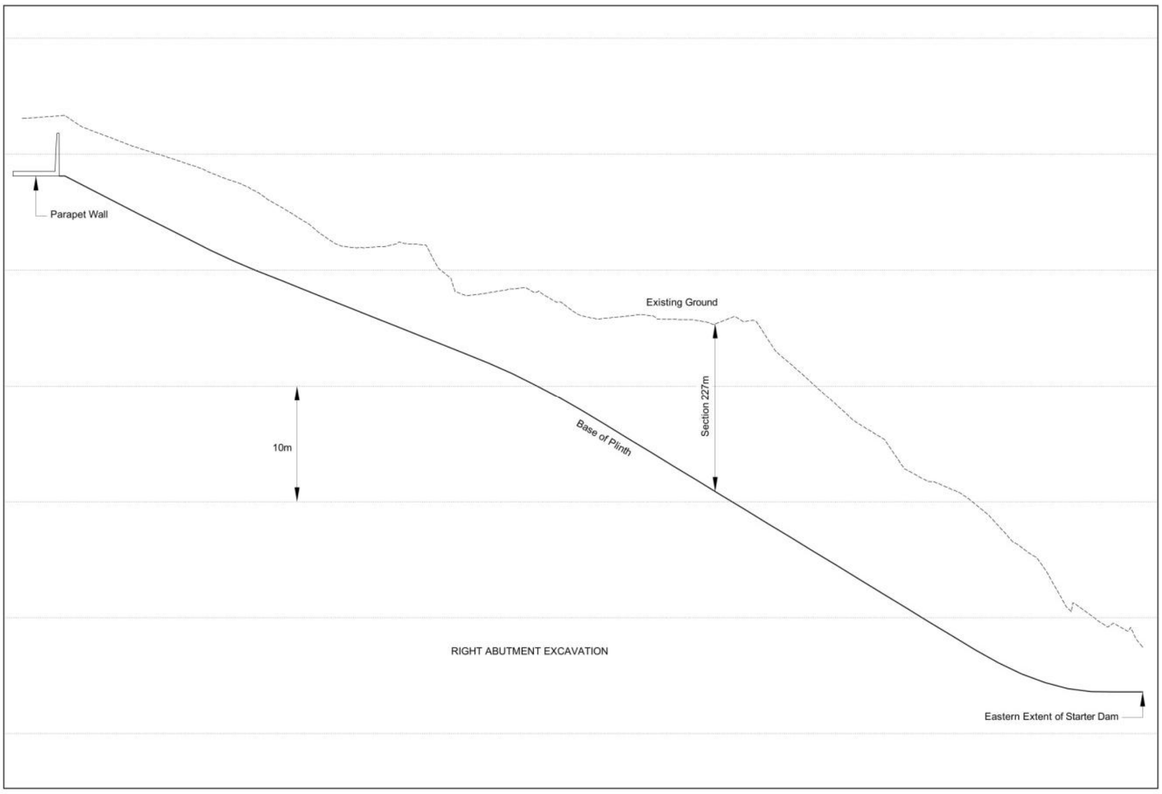


Figure 27: Developed section of cut for right abutment plinth

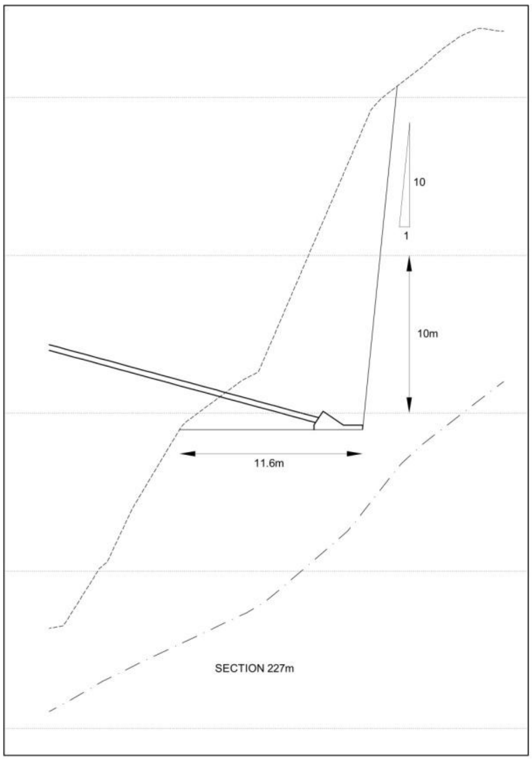


Figure 28: Plinth cut at Station 227 (right abutment)

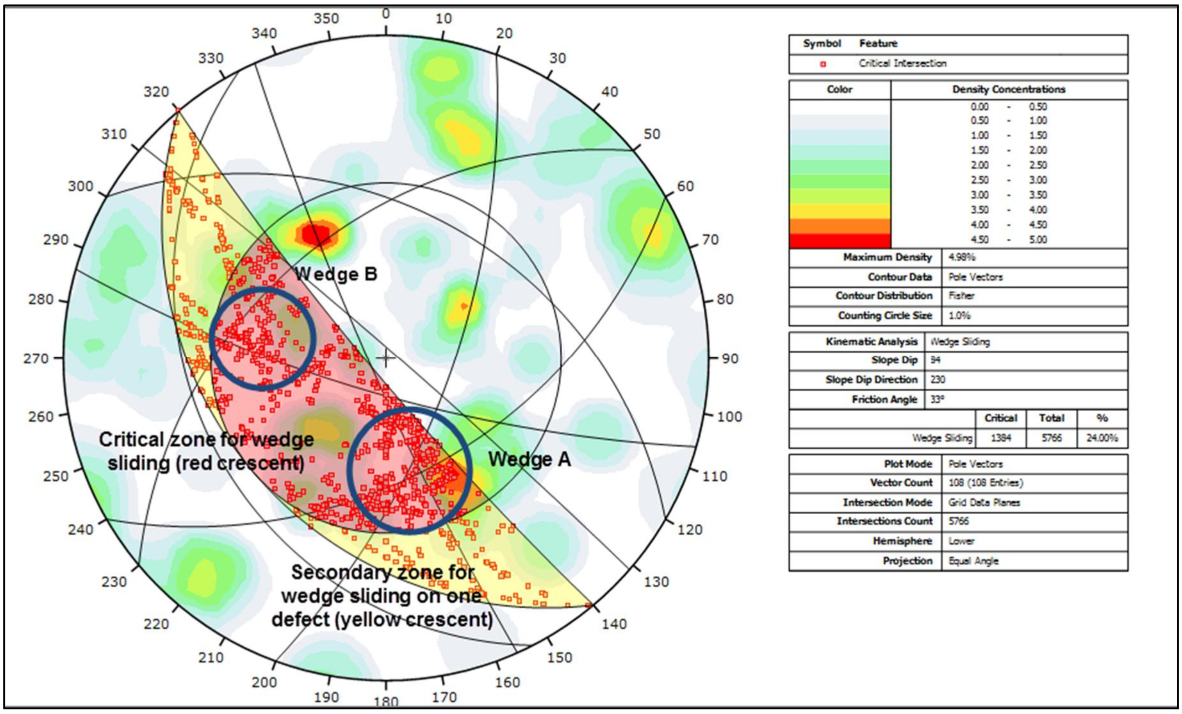


Figure 29: Stereonet of right abutment defect data showing potential wedges at intersections of major defect sets at section 227 of the plinth excavation.

The analyses indicate that a face pressure of 2.4 tonnes/sq m is required for wedge A and 1.0 tonnes/sq m for wedge B to achieve a static FOS = 1.5. However, face mapping will be required as the plinth excavation is progressively undertaken and support requirements will need to be designed for the specific defects encountered.

5.3.2 Left Abutment

The design for cut slopes for the left abutment plinth excavation is shown in Figure 30 for Station 77 where the cut is highest. The upstream face of the plinth excavation will be cut at 1V:0.8H (51°) for the lower 4.5m with the balance to ground surface cut at 1V:1.2H (40°). The trace of bedding plane defects is sub parallel to the cut batter and local drapes of wire netting may be required to contain loose blocks of rock.

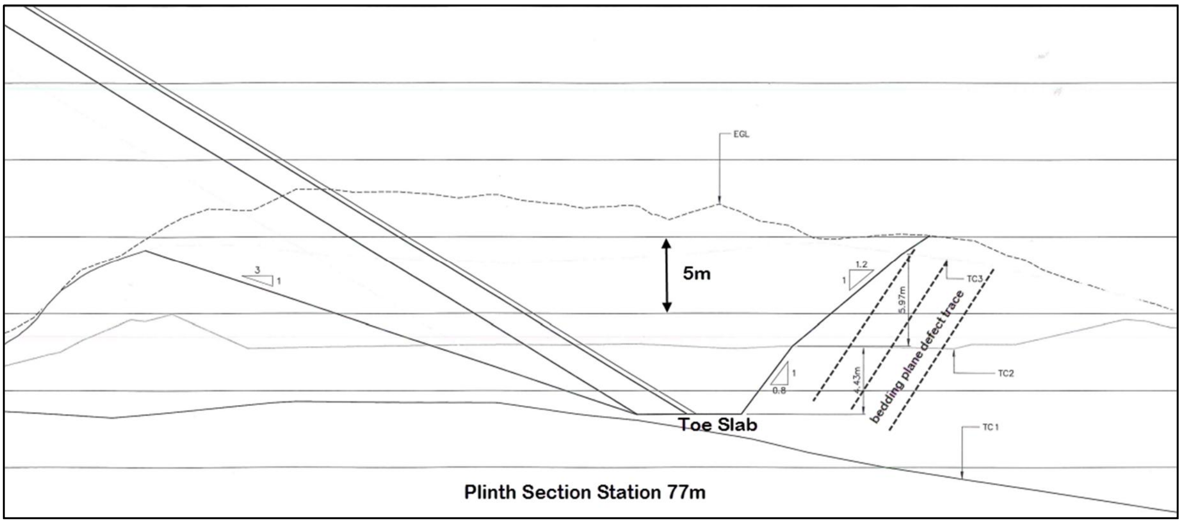


Figure 30: Plinth cut at Station 77.

5.4 Left abutment stability

The left abutment is located at the end of a north westerly trending ridge which will be partially truncated by the spillway cut and excavation for the dam footprint and plinth excavation. There are no recognised stability issues above the spillway cut. The stability of the spillway cut is discussed further in Section 7.

5.5 Right abutment stability

The slopes above the right abutment are moderately steep at 40° with bedrock exposed in ribs and scree in chutes between the ribs. The top of the ridge is 150m above reservoir level. The slopes have been recently clear felled of exotic (pine) trees and may be replanted in the near future. While steep there are no indications of deep seated instability (see Figure 31). However, TP7 excavated at the toe of the slope encountered a 7 m thickness of scree overlying disturbed/displaced bedrock. At an overall slope angle of 40° debris will be more inclined to slide/roll than bounce down the slopes while clear of vegetation.

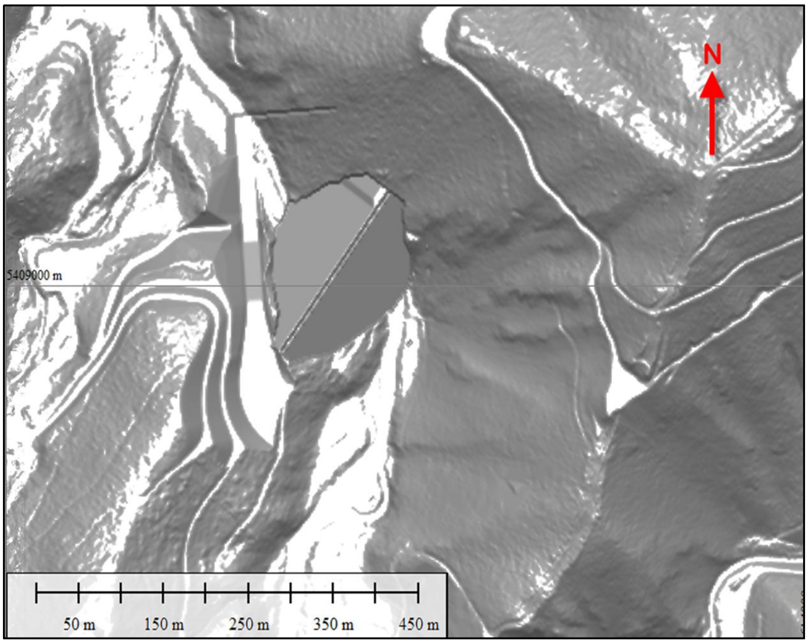


Figure 31: Final slope shape model following dam construction showing shallow instability features upstream of the right abutment.

5.6 Diversion stability

A cut into rock at the toe of the right bank of the river is proposed for the diversion culvert. At the present time the cut is designed at 1V:1H and reaches approximately 28m vertical height to eliminate differential settlement of the embankment when placed. Local drapes of heavy wire mesh/wire netting may be required to contain loose blocks of rock from rolling down the excavated face. Additional armouring may be required on the right bank of the diversion upstream of the dam where the channel excavation will possibly extend into scree and alluvium.

5.7 Leakage potential

The main leakage potential from the reservoir is likely to be in the vicinity of the dam, either under the dam (plinth) foundation or through the abutments.

The permeability of the rock mass under the plinth has been assessed by Lugeon testing and given the range of values measured it is expected to require grouting to reduce water flow.

The present excavation profile for the plinth will also remove known paleo-channels cut in to bedrock (such as that identified on the left abutment between trenches TP1A and TP1B) leaving only longer seepage paths either under the plinth or around the abutment grout curtains.

Based on our present understanding there is a potential for leakage along bedding parallel features (crushed and sheared zones) and joints which will daylight in the reservoir and within the dam footprint downstream of the grout curtain. The Lugeon testing in DH10 the vicinity of the sheared zone at 31.6 m returned high values (Lu >50) thus water could track around the grout curtain and the grouting program will require modification to ensure the zone is sealed off. While some uncertainty exists regarding the number and continuity of such sheared zones based on our current knowledge, the grouting will be designed as the plinth is prepared. As the plinth excavation proceeds, all zones of shearing or crushing will be mapped and a grouting design prepared to treat each one individually.

Northeast and southeast dipping joints were found to be open at some depths in drill holes and resulted in large lugeon values on the left abutment. Inclined grouting has been recommended on the left abutment to infill these joints. No other obvious leakage paths are known on this abutment.

6 Foundation preparation and stabilisation

6.1 Plinth excavation foundation

The plinth excavation will be subject initially to bulk excavation and clean-up and at the proposed depth will be sited below the majority of the weathered blocky, seamy rock. However, borehole cores indicate some clay/silt coated narrow aperture joints, bedding planes and minor sheared and crushed zones will occur just below foundation level. These features, where known, are shown on drawings 27425-GEO-06 and 07. The deepest joints infilled with silt that were logged in the boreholes are summarised in Table 13.

Table 13: Depth to lowest silt infilled defect identified in each drill hole.

Drill Hole	Depth to lowest silt infilled defect (angle)	Depth of defect below ground level (true)	Notes
DH10	36m	31m	Angle hole (60°) Length 48m
DH11	10.5m	8.2m	Angle hole (52°) Length 19.0m
DH12	10m	10m	Length 18.5m
DH12A	17.4m	13.3m	Angle hole (50°) Length 19.5m
DH13	19.3m	19.3m	Length 20.1m
DH13A	14m	11.5m	Angle hole (55°) Length 24.4m
DH14	23.4m	18m	Angle hole (50°) Length 23.5m

The frequency of encountering clayey sheared/crushed zones requiring individual treatment has been estimated from data collected from logs of excavations (Track's A, B, C, and Test Trenches TP1A, TP1B, TP1C) and drill holes along the plinth excavation where minor sheared zones (10mm to 50mm) were encountered every 5 to 10m and 100mm or greater thickness zones requiring individual foundation treatment occurred at 20m to 30m intervals of foundation length.

Emerson Crumb testing on silt/clay gouge sample from a sheared zone showed it to be non-dispersive. Crushed rock in the sheared zones that retains its rock structure is also interpreted as non-dispersive.

6.2 Embankment foundation

Following stripping, the embankment foundation away from the river channel will comprise an irregular to undulating surface composed of moderately weathered rock. Near the river channel the rock is expected to slightly weathered or unweathered but iron stained. The exposed rock in all areas will be jointed and contain narrow zones of crushed/sheared rock. Bedding plane defects dipping to the NW along with joint sets mainly orthogonal to bedding are expected.

Over the majority of the foundation surface no treatment is expected.

6.3 Grout curtain

Lugeon water pressure testing along the plinth excavation indicates a variable rock mass permeability. Grouting will be required along the alignment of the plinth excavation to control leakage beneath the foundation.

6.3.1 Left abutment grouting

The main defect sets that may control water leakage on the left abutment are bedding partings (and associated crushed zones) dipping 60°→320° and joints conjugated to bedding 55°→135° and normal to bedding 42°→070° as shown on the stereonet below (Figure 32). The more significant joint set is judged to be the 42°→070° set oriented at right angles to the plinth excavation. It should be noted that the dips and dip directions provided are mean values and variations are likely to occur on site. Drill holes DH12A and DH13A were oriented to pass through this defect set at more or less at right angles. Thus the Lugeon numbers from testing in these drill holes reflects the water leakage potential along this defect set.

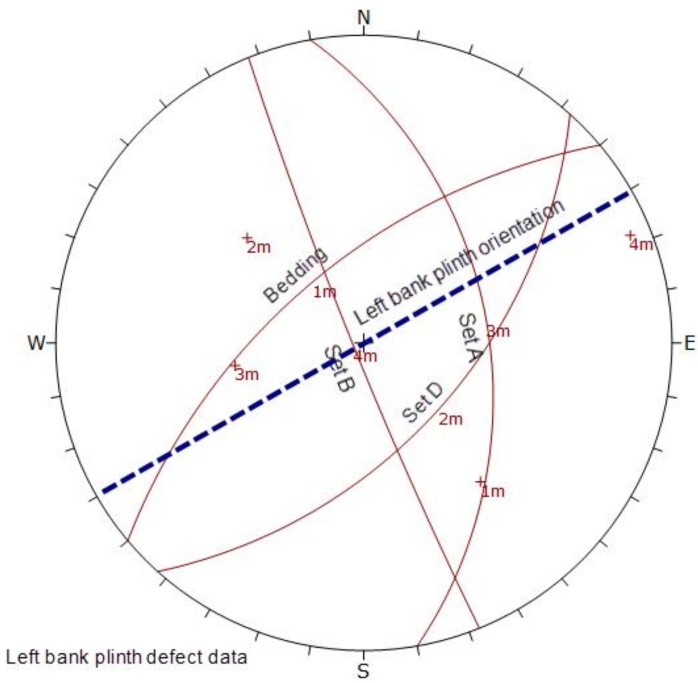


Figure 32: The mean orientations of the main defect sets on the left abutment plinth

Based on this information it is recommended that the grout holes have a similar orientation to DH's 12A & 13A, which is at moderate angles to the southwest.

6.3.2 Right abutment grouting

The right abutment plinth line has a north to south orientation and as the main defect sets are similar to those on the left abutment vertical grout holes will intersect both bedding plane defects and defects orthogonal to the bedding orientation.

7 Spillway

7.1 General

The spillway involves a significant excavation which provides the bulk of the rock fill for the embankment construction and in doing so creates cut slopes up to 60 m high. The overall design concept is to bench the final batters as described in Section 7.3 below.

The present spillway location differs from that described in the Geotechnical Feasibility Report.

7.2 Geology

Investigations for the spillway cut have involved drilling two cored investigation holes, DH5 and DH6 during the Feasibility Investigations, and completing seismic refraction line SL1 during this phase of investigations. Defect data was also measured along access tracks and other digger excavations. These data show a similar range in defect orientations to those measured along the left abutment plinth line. The distribution of the rock mass classes within the cut has been estimated using the seismic refraction survey data and drill hole logs. Cross sections showing the interpretation of this distribution are shown drawings 27425-GEO-10 to 12. The depths to the different rock classes shown on the cross sections should be treated as indicative only and it must be appreciated that they may vary from what will actually be encountered on site. Seismic refraction is often used to aid the delineation of boundaries of similar interpreted rock mass properties. However, seismic refraction averages these properties and it cannot delineate small or local variations in the rock mass.

7.3 Spillway stability

The spillway cut will result in slopes up to 60 m high on the east facing slopes. The east facing cut slopes are proposed to be 1V:1.2H (40°) in the overlying soil and in part of the Class 3 rock and generally at 1V:1H (45°) in Class 1 and Class 2 rock. The batters are locally steepened to 1V:0.67H above the chute floor. The spillway cuts are for the most part excavated down to Class 1 and Class 2 rock.

Stability of the spillway cut has been has been checked by both kinematic and slope stability analyses. The following techniques have been used:

- Defect data and Dips version 5 software; and
- Slope stability analyses using Slide version 6 software.

Both generalized Hoek-Brown failure criteria and anisotropic shear strength parameters to model defects within the rock mass were used in the stability analyses.

The kinematic analyses show that with the available defect data the individual batters could be subject to very shallow wedge and planar type failures but the overall slope is unlikely to be affected. Allowance for spot bolting should be included to stabilise these features.

Slope stability analyses were carried out on spillway sections 940, 1000, 1100 using both generalised Hoek-Brown criteria and anisotropic shear strengths to model the dominant defect sets present in the slope for static conditions. Analysis was carried out with a range of groundwater conditions and earthquake shaking with OBE (0.16g) and MDE (0.48g) events.

The generalised Hoek-Brown parameters adopted for the slope stability analyses are provided in Table 14.

Table 14: Hoek-Brown parameters adopted for stability analysis.

Material Name	Unit Weight (kN/m3)	UCS (MPa)	GSI	M_i
Class 1 rock	24	50	45	10
Class 2 rock	24	35	35	10
Class 3 rock	24	15	25	10

For the anisotropic analyses the shear strength parameters adopted are provided in Table 15.

Table 15: Shear strength parameters adopted for anisotropic analyses.

Material Name	Unit Weight (kN/m^3)	Discontinuities		Rock Mass	
		Cohesion	Phi	Cohesion	Phi
Class 1 rock	24	10 kPa	33°	500 kPa	49°
Class 2 rock	24	10 kPa	33°	320 kPa	44°
Class 3 rock	24	10 kPa	33°	174 kPa	34°

These analyses show that the overall slopes for all three spillway sections are stable under all static and OBE conditions and shallow failure may occur if high groundwater conditions coincided with an MDE event. There may be small scale drop outs in lesser earthquake events but these will not affect the operation of the spillway.

The analysis outputs are presented in Appendix J.

7.3.1 Batter slopes

Batter slopes in the spillway cut will be formed in a closely jointed rock mass that may require blasting for excavation in all Class 1 rock and possibly in much of the Class 2 rock. In addition, ripping will leave a rough surface finish which, subject to any specific requirements that may exist in the specification, may not be desirable in the spillway. As a result the finished surface roughness will be strongly influenced by ripping or the blasting technique. Smooth wall or cushion blasting along closely spaced holes may be required to minimise roughness on the final surface. This will be particularly important on the lower batter that forms the spillway chute to avoid rock dilation and overbreak and to provide a smooth surface for the concrete lining.

7.4 Foundation treatment

The spillway chute is expected to be founded on Class 1 rock over most of its length. Areas of dilated joints and sheared/crushed zones are likely to be encountered and these may require treatment using dental concrete with passive bars to create a smooth surface for the concrete floor. The Contractor’s chosen method of excavation and production blasting must give due consideration to minimising dilation of the rock mass adjacent to finished profiles.

7.5 Flip bucket

The foundation for the flip bucket will be in predominantly interbedded sandstone and siltstone/mudstone dipping at moderate to steep angles to the SE with joints orthogonal to bedding. The rock mass exposed in TP2 located in the vicinity of the flip bucket is dilated. However, undilated Class 1 rock is exposed in the river bed also in this vicinity. The final excavated level is some 5m below the present river level and we infer Class 1 rock at this depth.

8 Reservoir stability

8.1 Introduction

Slope stability within the proposed reservoir has been assessed at reconnaissance level using LiDAR topographical/geomorphology data and walkover observations. A review of the size of impulse waves from slopes that have the potential to fail under extreme conditions (e.g. earthquake and or rainstorm) identified in this study was undertaken using ICOLD recommended methodology. Data for this relied on landslide boundaries being recognised from the LiDAR imagery and depth estimates based on field observations and judgement. The size of the generated wave was estimated using the methodology of Pugh (Pugh C.A., and Chiang W., 1986 "Landslide generated wave studies" Proc. of the Conf Water Forum '86 Lang Beach California p27-34 and Huber (Huber A., and Hager W.H., 1997 "Forecasting Impulse Waves in Reservoirs"). The results are summarised in Table 16.

Nine main areas of instability are identified in the main Lee Valley within the reservoir and three areas in Waterfall Creek. The areas are shown on Drawing 27425-GEO-09 and include previously slumped ground based on LiDAR contours, evacuated (relic) slips, and debris slides and chutes with steep rock faces subject to defect controlled failure above. Three areas (references 3, 6 & 7 in Table 16) are considered to have features that suggest a higher potential exists for future landslides that could generate large waves in the reservoir. These three areas have therefore been assessed in greater detail below.

Table 16: Areas of slope instability identified around the Lee Valley reservoir.

Location (upstream of dam)	Ref	Description	Material	Potential trigger	Likely rate of movement	Likely consequence
Left bank* N/A	1	Rock Slump	Displaced Rock, dilated and shattered	Extreme rainfall, Earthquake	Very rapid	Flooding at dam toe
Left Bank 200m	2	Erosion of earth slump	Silty SAND fine GRAVEL	Reservoir operation	Slow	Loss of road/sediment into reservoir
Left Bank 500m to 800m	3	Deep solifluction deposits	Debris on lower slope, solifluction material mid and upper slope, colluvium and dilated bedrock also upper slope	Reservoir operation/ Earthquake/ extreme rainfall event	Rapid following drawdown or earthquake	Impulse wave and debris into reservoir
Left Bank 900m	4	Rock fall/ rock slide	Rock	Reservoir operation/ earthquake	Rapid	Small impulse wave and debris into reservoir
Left Bank 1000m	5	Rock fall/slide	Rock	Reservoir operation/ earthquake	Rapid	Small impulse wave and debris into reservoir
Left Bank 1200m to 1300m	6 6B	Landslide debris/ rock slide	Landslide debris/rock	Reservoir operation/ earthquake	Rapid	Impulse wave and debris into reservoir

Left Bank 1400m	7	Rock Slide Debris slide	Rock and colluvium	Earthquake	Rapid	Small impulse wave and debris into reservoir
Left bank 1700m	8	Debris slide	Landslide debris	Earthquake/ drawdown	Rapid	Small impulse wave and debris into reservoir
Left bank 2200m	9	Rock slide/ fall	Rock debris	Earthquake	Rapid	Small impulse wave and debris into reservoir
Left bank 2300m	10	Rock fall/ debris	Rock debris	Earthquake	Rapid	Small impulse wave and debris into reservoir
Waterfall Ck 500m (both banks)	11	Relic colluvium Slump	colluvium	Extreme rainfall event	Rapid	Debris into reservoir
Waterfall Ck 800m right bank	12	Relic colluvium Slump	colluvium	Extreme rainfall event	Rapid	Debris into reservoir
Waterfall Ck 1200m	13	Relic colluvium Slump (3 areas)	colluvium	Extreme rainfall event	Rapid	Debris into reservoir

*Note: Landslide Ref 1 is downstream of the dam site.

8.2 Existing stability

8.2.1 Earth Slump/Earthflow Area 3

While no subsurface investigations have been carried out through this feature, the materials within the slide body are exposed in a roading borrow area where slope-parallel bedded solifluction and colluvial deposits (sand and fine angular gravel) are exposed. These deposits are estimated to be up to 10 m thick. The location of Earth Slump/Earthflow Area 3 is shown on Drawing 27425-GEO-09.

The LiDAR generated contours show the upper slopes as being generally smooth with a slope angle of 35°. Below RL 235m the contours become slightly irregular and below RL 200m they are distinctly irregular which is indicative of previous slope instability.

Bedrock is locally exposed in upper parts of the chutes and solifluction and colluvium overlie this to a depth of approximately 5 m. In the middle portion of the slope (around the access road at approximately RL 210 m) in situ solifluction and weathered scree is exposed in cuts up to 8 m in height. Down slope of RL200 m the slopes are hummocky and there is evidence of recent slope creep.

The solifluction/colluvium materials all have slope angles close to 37°, the angle of repose for frictional material, and the most likely mode of instability would be as an infinite slope failure due to a major rainstorm event, or ground shaking during an earthquake event. There is no observable topographic/geomorphic evidence of this type of failure occurring recently and it is surmised that internal natural drainage within these deposits is sufficiently well developed to dissipate rainfall induced pore pressures. In addition to this the slopes at the present time are covered with mature native trees that will have well developed root systems strengthening the soil mantle. Well -developed seepage flows from the toe of the slope out on to the terrace surface were noted during a site visit (August 2012) that followed felling and clearing of the trees in this area. The slumping identified here may have occurred during periods of global cooling when climate and vegetation would have been significantly different, or have been triggered by past large earthquakes.

The cross sectional area of the potential failure mass is approximately 1400 m^2 and the main slide bodies each have an approximate width of 60 m where the failure surface is at full depth. The slide volume is therefore in the order of 80,000 m^3 per slide.

We assess that reservoir filling will have a small adverse effect on slope stability, primarily arising from elevated groundwater during flood/rainfall events.

8.2.2 Debris Slump/Rock Slide Areas 6 and 7

Areas 6 and 7 are characterised by complex geomorphology, steep slope gradient and recent evacuated landslide scars with bedding and joints locally being inclined sub-parallel to slope.

Area 6 is divided into two parts, 6A and 6B, as shown on drawing 27425-GEO-09. Area 6A the upper part of Area 6B are both evacuated large plane or wedge failures and bedrock is extensively exposed. The lower half of Area 6B is blanketed by chaotic bouldery landslide debris that is exposed in the forestry access road. The landslide debris forms a wedge shaped deposit that thickens progressively to the south from the Area 6A/6B boundary to a maximum estimated thickness of 14 metres adjacent to the Area 6B/7 boundary where it abruptly terminates adjacent to weathered bedrock outcrop. The landslide debris thins downslope of the forestry road and in situ bedrock is exposed at the toe of the slope. It is likely that this landslide has developed along prominent bedding and joint defect sets that dip sub-parallel to slope (bedding dipping to the north east and joints dipping to the south east).

The landslide debris is a geologically young feature which is being actively incised by a steep flowing stream that drains a large catchment upslope of the landslide area. It is likely that perched groundwater is present within the landslide debris.

We estimate that the landslide deposit has a volume of approximately 84,000m^3

We have considered the potential for reactivation of the landslide debris and the potential for a first time failure of a possible wedge block underlying the debris.

We consider that it is possible following reservoir filling that under large scale earthquake shaking the landslide debris could be remobilised.

A first time failure of a deeper seated wedge block within the underlying bedrock is considered to be unlikely due to the inclination of the main rock defect sets and the overall slope inclination.

In Area 7 there is extensive evacuation of bedrock slides on the upper slopes, immediately downslope of steep bluffs. The middle section of slope is blanketed extensively by scree colluvium and local rock debris chutes. Weathered bedrock is exposed in the lower third of the slope. Bedrock dips downslope but is inclined at steeper than 50 degrees, while the slope angle varied from 25 to 40 degrees. There is no evidence on the lower slopes of past significant bedding plane slides.

We consider that that bedrock slides in Area 7 are likely to arise from local rock bluffs but that large volume (e.g. > 50,000m^3) bedrock slides are unlikely as the bedrock dips significantly steeper than the overall slope. We consider it unlikely that a landslide from Area 7 could result in a large wave within the reservoir.

8.3 Reservoir induced instability

8.3.1 Beach creation during normal reservoir operation

Ponding water behind the Lee Valley dam will both change groundwater levels adjacent to the reservoir and create a new beach around the perimeter of the reservoir. The reservoir will also be subject to fluctuations in water level during normal operations which, in turn, will influence nearby groundwater levels. While changes in reservoir induced groundwater levels are not expected to have a significant effect on stability, the creation of new beaches by wave induced erosion could lead to the development of a notch in the slopes, particularly where the solifluction/colluvial deposits form the shoreline. The initial over steepened back scarp developed as part of this notch could remove toe support and lead to local regression, by slumping, of the immediate slopes above. This slumping is likely to be confined to within 10 to 15m of the slope behind the beach.

8.3.2 Effect of reservoir on existing areas of instability

The results of the slope stability assessment indicate that the full reservoir is likely to have a small adverse effect on the main landslide areas identified in this study.

Of the three areas considered in more detail, our assessment indicates that it is unlikely that reservoir inundation will trigger large landslides. Two of the areas, Area 3 and Area 6, have characteristics that suggest a landslide could be triggered by large earthquake or rainfall events once the reservoir is full. Our assessment suggests that the likely volume of landslides from such scenarios is in the order of 80,000m^3 in Area 3 and 84,000m^3 at Area 6. Movement of the landslides into the reservoir would be rapid.

There other areas of very steep ground mantled with rock scree which could mobilize during earthquake shaking. These areas are identified in Table 16 and on Drawing 27425-GEO-09. In this respect we recommend that tree cover (both native and exotic) is maintained above reservoir level wherever possible to provide stability to the soil mantle.

8.3.3 Effect of Landslides into the full reservoir

While no subsurface investigations have been directly carried out, the slope stability assessment indicates that with a full reservoir, the OBE earthquake or an extreme rainfall event could mobilise one of the two areas (Areas 3 and 6 or possibly both) which are identified as more prone to slope failure. This eventuality has been modelled in accordance with ICOLD (2000) to determine the size of wave generated as the landslide mass enters the reservoir. The event modelled assumes full mobilization of one of the identified at-risk landslide areas. The results are presented in the Detailed Design Report Stage 3.

9 Construction materials

9.1 Requirements and sources

The general requirements for geological construction materials for a concrete faced rock fill dam at the site are summarised in Table 17. The estimated volumes of material available from onsite excavations are summarised in Table 18.

Table 17: Required volumes of rock classes required for construction.

Required Volumes		
Embankment Zone	Source	Placed Volume Required (m^3)
2A	Class 1 & 2	500
2B	Class 1 & 2	22,500
2B filter	Class 1 & 2	500
2C	Class 1 & 2	4,000
3A	Class 1 & 2	39,500
3B	Class 1 & 2	320,000
3C	Class 2	12,000
3D	Class 1 & 2	11,000
4	Class 1 & 2	23,000
Class 1 & 2 Total		433,000

Table 18: Estimated volumes of rock available from site excavations.

Excavation Volumes	
Rock type	Solid (m^3)
Class 2	249,300
Class 1	154,400
Total:	403,700
*Assumes Class 3 is cut to waste	

9.1.1 On-site Sources and Volumes

Spillway Excavation

We understand that the spillway excavation may yield a minimum of 350,000 m^3 of rock fill that will consist of slightly weathered to unweathered (Class 1& 2) greywacke. This is overlain by a 1 to >5 m thickness of soil and Class 3 rock which would be suitable as a general fill. Further evaluation of Class 3 rock may show that some of it is be acceptable for use in dam construction.

Alluvial Gravel Deposits

There are several low level deposits of alluvial gravel identified near the dam site. These deposits vary from 2 m to more than 5 m in thickness and have an estimated resource volume of 360,000 m^3. They are generally overlain by less than 0.5 m of silty sand. In addition, approximately 60,000 m^3 of generally unweathered gravel is present within the active river bed between the dam and Ch14,000 m. The river bed deposit contains a larger proportion of coarse gravel and boulder sizes (up to 600 mm). This coarser size fraction forms an imbricated armour layer on the river bed. There may be occasional unsuitable material in the alluvium such as weathered gravels, logs and silt lenses.

Solifluction deposits

Four large solifluction deposits are in close proximity to the dam site with an estimated total resource volume of 380,000 m^3. A volume of 30,000 m^3 of this material will be excavated from the left abutment of the dam site.

9.1.2 Off-site Sources

Patuki Melange

Patuki Melange includes a mixed rock assemblage that includes very strong diorite. The diorite has been locally quarried in the past from scree deposits below high cliffs at a site 2.3 km downstream of the dam site. Investigations during Stage 2 of the study revealed a high variability in rock quality and weathering over short distances and this option was discounted at that time due to the likely high overburden ratios that would arise from quarrying the diorite.

Taylor's Quarry

Taylor's Quarry is 3.5 km from the site and produces a high quality aggregate from limestone bedrock that is used in a range of applications. Quarry products include: rip rap – up to 1.2 m diameter, AP65 for road sub-base and general fill, and GAP 40 for roading aggregate.

Berketts Quarry

Berketts Quarry is 12 km from the site. It produces hard rock aggregates.

Appleby Gravel Pits

Two gravel pits near Appleby produce the majority of concrete aggregate for the Nelson region. The aggregate is river run, derived from the Waimea River.

9.2 Rock fill

9.2.1 Site quarried rock fill

9.2.1.1 General

As discussed previously the rock beneath the site has been categorised into three classes based on strength and weathering. Cross sections showing indicative depths to the various rock classes are provide on drawings 27425-GEO-06 to 07 and 27425-GEO-10 to 12. The use of these various rock types as construction materials is discussed in the following sections.

9.2.1.2 Petrographic analysis

Petrographic analysis of gravel material was undertaken on a sample of Class 2 greywacke sandstone at Auckland University to determine the presence or absence of zeolites, or specifically the swelling zeolite Laumontite (results are included in Appendix K). The report concluded that the greywacke (and thereby greywacke derived material) does not contain zeolites.

9.2.1.3 Extraction of site quarried rock

The strength of the different classes of rock were measured by point load testing (132 No.) during the feasibility study and by 20 unconfined compressive strength tests carried out during the current investigations. The calculated UCS values had quite broad ranges and there significant overlap in values for classes 1 and 2 rock (Table 19). These are summarised below:

Table 19: Unconfined compressive strength ranges of rock classes encountered on site.

Rock Class	UCS (MPa)
Class 1	60 to 143
Class 2	20 to 100
Class 3	10 to 20

The rippability of the rock has been examined based on the rock strengths above, seismic velocity of the different rock classes and on site observations. Within each rock class, massive sandstone rock is more difficult to rip than the same class of interbedded sandstone and siltstone.

Weaver (1975, see tables 20 and 21) and Minty and Kearns (1983) provide guidance on rock rippability using seismic velocity and other geological inputs. Guidance on rippability is also provided by Transit New Zealand in terms of the energy required to extract the rock. We are aware that ripping technology has improved since these the cited references were published and that some modern plant may be able to effectively rip rock which has a higher strength than is suggested here.

Table 20: Extract from Weaver (1975) showing rippability of rock estimated from seismic velocity.

Rock hardness description	Identification criteria	Unconfined compressive strength (MPa)	Seismic wave velocity (m/s)	Excavation characteristics
Very soft rock	Material crumbles under firm blows with the sharp end of a geological pick; can be peeled with a knife; too hard to cut a triaxial sample by hand. SPT will refuse. Pieces up to 3 cm thick can be broken by finger	1.7 to 3.0	450 to 1,200	Easy ripping
Soft rock	Can just be scraped with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of the pick point; has dull sound under hammer	3.0 to 10.0	1,200 to 1,500	Hard ripping
Hard rock	Cannot be scraped with a knife; hand specimen can be broken with pick with a single firm blow; rock rings under hammer	10.0 to 20	1,500 to 1,850	Very hard ripping
Very hard rock	Hand specimen breaks with pick after more than one blow; rock rings under hammer	20.0 to 70.0	1,850 to 2,150	Extremely hard ripping or blasting
Extremely hard rock	Specimen requires many blows with the geological pick to break through intact material; rock rings under hammer	>70.0	>2,150	Blasting

Table 21: Rippability rating chart taken from Weaver (1975).

Rock Class	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
Seismic Wave Velocity	>2,150	2,150 to 1,850	1,850 to 1,500	1,500 to 1,200	1,200 to 450
Rating	26	24	20	12	5
Rock hardness	Extremely hard rock	Very hard rock	Hard rock	Soft rock	Very soft rock
Rating	10	5	2	1	0
Rock weathering	Unweathered	Slightly weathered	Weathered	Highly weathered	Completely weathered
Rating	9	7	5	3	1
Joint spacing (mm)	>3,000	3,000 to 1,000	1,000 to 300	300 to 50	<50
Rating	30	25	20	10	5
Joint continuity	Non continuous	Slightly continuous	Continuous – no gouge	Continuous – some gouge	Continuous – with gouge
Rating	5	5	3	0	0
Joint gouge	No separation	Slight separation	Separation <1mm	Gouge <5mm	Gouge>5mm
Rating	5	5	4	3	1
Strike and dip orientation*	Very unfavourable	Unfavourable	Slightly unfavourable	Favourable	Very favourable
Rating	15	13	10	5	3
Total rating	100 - 90	90 – 70**	70 – 50	50 – 25	<25
Rippability assessment	Blasting	Extremely hard ripping and blasting	Very hard ripping	Hard ripping	Easy ripping
Tractor selection	-	DD9G/D9G	D9/D8	D8/D7	D7
Horsepower	-	770/385	385/270	270/180	180
Kilowatts	-	575/290	290/200	200/135	135

*Original strike and dip orientaition now revised for rippability assessment
** Ratings in excess of 75 should be regarded as unrippable without pre-blasting

The seismic velocities and rippability ratings of the different rock classes based on the method outlined by Weaver (1975) are provided in Table 22. These ratings indicate that Class 3 rock will be easy ripping, Class 2 hard to very hard ripping and Class 1 rock unrippable (blasting required for extraction). The rock classifications in accordance with NZTA guidelines (TNZ F/1) are also provided below.

Table 22: Rippability rating of rock classes 1 to 3

Rock Class	Seismic velocity (m/s)	Rippability rating	Rippability (according to Weaver 1975)	Transit New Zealand F/1 rating
Class 1	3,300 to 3,700	85	Unrippable without pre-blasting	A
Class 2	1,000 to 1,700	65	Very hard ripping	R2
Class 3	<1000	45	Hard ripping	R1

There is some disparity between the seismic velocities outlined by Weaver (1975) and the UCS estimated from point load testing, e.g. Class 1 rock has a seismic wave velocity of 3300 to 3700 m/s, but UCS values range between 30 to 70 MPa, however the rippability assessment was supported by field observations during construction of the two test pads and excavation of test pits in rock around the left abutment. The two test pads described in section 9.2.2 below were constructed using material won on site using a 22 tonne excavator. Class 3 rock close to the surface could be easily excavated using a smooth bucket. Deeper Class 3 rock required a toothed (rock) bucket and was more difficult to rip. Class 2 rock was very to extremely difficult to rip using the 22 tonne excavator.

Further rippability assessments were carried out between 13 and 17 August 2012 using a 34 tonne excavator equipped with a rock bucket and a rock pick. Similar results to the excavation of the test pads were found. Class 3 rock was easily excavated with a rock bucket. Class 2 rock was difficult rip using the rock bucket, but was easy to rip using a rock pick. Class 1 rock was extremely difficult to rip to unrippable.

The size of machinery required to rip classes 1 and 2 rock was further assessed in accordance the methodology outlined in Minty and Kearns (1983). The assessments are provided in Table 23.

Table 23: Rock rippability based on Minty and Kearns (1983)

Rock class	Average seismic velocity (m/s)	Geological factor rating	Seismic velocity × geological factor rating	Weight of machine required to provide satisfactory rippability
Class 1	3,300 to 3,700	49	161,700 to 181,300	No plant big enough - Unrippable
Class 2	1,000 to 1,700	35	35,000 to 59,500	15 to 60 tonnes (45 tonnes for average seismic velocity)
Class 3	<1,000	Not assessed	Not assessed	Not assessed

These results from both Minty and Kearns (1983) and Weaver (1975) correspond well and indicate Class 2 is hard to very hard ripping and Class 1 rock is unrippable.

The excavation of the test pits also revealed that the weathering and strength profiles in the rock are variable and are not parallel to topography. Zones of difficult to excavate to Class 2 rock are present in areas dominated by Class 3 rock and vice versa. This indicates that the rippability of rock may change over short distances both laterally and vertically. A similar situation may also occur between classes 1 and 2 rock.

The rippability of the rock is also influenced by the spacing of joints and the orientation of bedding and joints relative to the direction of ripping. Given the difficulty that may be experienced in extracting Class 1 rock we consider that it would be prudent to undertake trials prior to the commencement of bulk excavation.

The selection of ripping vs blasting may also be determined by the finish required for the spillway. As mentioned previously, a smother profile may be achieved with controlled blasting techniques. Care will be required to avoid inducing dilation of the rock mass when blasting/excavating in close proximity to finished spillway profiles.

9.2.2 Particle size distributions of ripped rock

Particle size distributions were carried out on samples of classes 2 and 3 rock ripped from the left abutment. The results are summarised below in Table 24 and shown on Figure 33. The particle size distribution result carried out on Class 1 reported in the feasibility study has been reproduced in Table 25. Laboratory results are presented in Appendix L.

Table 24: Grain size distributions of classes 2 and 3 rock ripped and sampled on site.

Sample /Source	Gravel %	Sand %	Silt %	Clay %	Fines %	D 50	D_{10}	D_{60}/D_{10}	PL	LL	PI
Class 2 - MW Greywacke	93	7	0	-	0	50	3.5	20	-	-	-
Class 3 - HW-MW Greywacke	92	8	0	0	0	37.5	4	11	-	-	-
Class 3 - HW Greywacke*	77	19	NT	NT	4	8	0.3	53	NT	NT	NT

*Highly weathered greywacke tested during the detailed design geotechnical investigation. NT = Not tested

Table 25: Grain size distribution for Class **1** rock taken from the Feasibility Report.

Sample /Source	Gravel %	Sand %	Silt %	Clay %	Fines %	D 50	D_{10}	D_{60}/D_{10}	PL	LL	PI
Class1 – UW-SW Greywacke	97	3	0	-	-	50	8	9	-	-	-

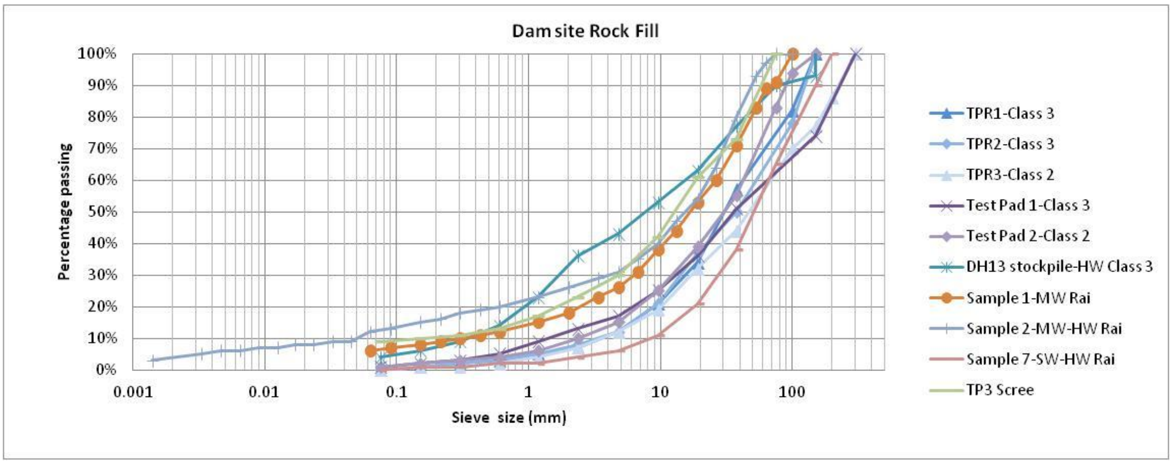


Figure 33: Particle size distributions for locally won Class 2 and Class 3 rock fill.

9.2.2.1 Solid density

The results of solid density testing for the different rock types and classes are provided in Table 26 below.

Table 26 – Solid density of rock sampled during the site investigations.

Rock Class	Sandstone (t/m^3)	Siltstone (t/m^3)
Class 1	2.71	2.69
Class 2	2.58	2.64
Class 3	2.28	2.45

9.2.2.2 Test Pads

Two test pads were constructed using site won Class 2 and 3 rock. The rock was extracted using a 20 tonne excavator. The upper portion of the Class 3 rock was easily excavated using a smooth bucket. This easily excavated material was discarded. The lower Class 3 rock was excavated with a rock bucket. Class 2 rock was partly penetrated using the rock bucket, but was extremely difficult to rip using the 20 tonne excavator.

The materials extracted were placed in two spoil heaps and clast strengths tested on site using a geological hammer. Test Pad 1 was constructed using dominantly Class 3 rock at the locations of test pit TPR2. Test Pad 2 was constructed using dominantly Class 2 rock at the location of test pit TPR3.

The material used for the pads was placed in 300 to 400 mm thick layers and compacted using a 7.5 tonne vibrating smooth roller. The compaction was tested between each lift using a nuclear densometer and a Clegg hammer.

Plate bearing tests carried out on these test pads indicate that the rock fill has a Young's modulus of 20 to 30 MPa. The particle size distribution data for compacted Class 2 and Class 3 rock is summarised in Table 27 and shown on Figure 33.

Table 27: Grain sizes of classes 2 and 3 rock following compaction

Sample /Source	Gravel %	Sand %	Silt %	Clay %	Fine s %	D 50 (mm)	D10 (mm)	D60/ D10
Class 2 – SW Greywacke	90	9	<1	<1	1	30	2.36	18.6
Class 3 - MW Greywacke	87	12	<1	<1	1	37.5	1.5	47

The particle size distributions for the compacted site-sourced rock fill are comparable to those for ripped, uncompacted rock. They indicate that site quarried rock fill is unlikely to break down considerably during construction.

Permeability and density testing was carried out on Test Pad 1. The compacted rock fill had a permeability of 1.4×10^{-3} m/s and a compacted bulk density of 2.25 ± 0.1 t/m^3.

9.2.3 Alluvial borrow material

9.2.3.1 Volume and characteristics

Twenty seven test pits (TPA1 to TPA27) have been excavated within 1,300 m upstream of the dam to assess the distribution, quality and quantity of the alluvial gravel as a potential fill and concrete aggregate source. The location of the test pits are shown on Drawing 27425-GEO-08 and the test pit logs are provided in Appendix M.

Test pitting has indicated that the alluvium resource consists of two main material types, gravel and silty sand. Gravel predominates and is widespread beneath the active river bed and underlying low level terraces that flank the valley floor. The gravel is predominantly a well graded fine to coarse GRAVEL with cobbles and boulders, some sand and with traces of silt. Visual assessments indicated that boulders comprised between 20 and 30% of the river gravel deposits. Occasionally, gravelly sand lenses occur within the gravels. Gravel deposits vary from less than 1 m to more than 5 m thick. The gravel is unweathered with very strong, sub rounded to rounded clasts.

Silty SAND overlies the gravel in many of the terrace deposits, this is a flood plain deposit and varies from 0.1 m to 1 m thick. This capping layer is absent in the active river bed.

The alluvium resource within 1,900 m of the dam is estimated to be 73,000 m^3 of which 62,000 m^3 is assessed to be gravel, and 8,500 m^3 as sand and 1,500 m^3 as silt/clay. Seventy percent of the alluvium resource or 36,000 m^3 is assessed to lie between 250 m and 600 m of the dam within a broad terrace at the mouth of Waterfall Creek.

Gravel grading tests were carried out on samples selected from the active river bed, Waterfall Creek Terrace and upstream of the Waterfall Creek Terrace.

Sand castle tests carried out on processed samples taken from TPA25 and 26 indicate that the fines within the gravel are likely to be cohesive. Permeability testing carried out on the same samples gave permeabilities of 1.07×10^{-3} m/s and 1.23×10^{-4} m/s respectively.

Grading characteristics are given in Table 28 and shown on Figure 34.

Table 28: Grain size summary of River Gravel samples

Source	Gravel and boulders* %	Sand %	Silt/Clay %	D_{50} (mm)	D_{10} (mm)	D_{60} (mm)	D_{80} (mm)	D_{60}/D_{10}
TPA2	85	14	1	60	1.2	95	195	79
TPA4	87	12	1	90	1.5	120	195	80
TPA15	87	10	3	50	1.5	95	195	63
TPA24	78	21	1	15	0.7	23	45	33
TPA25	92	8	1	120	4.5	195	400	43
TPA26	93	6	1	170	4.5	205	370	46

*Boulder fraction was scalped from samples prior to testing. The percentages shown have been calculated based on actual particle size distribution results and volume of boulders observed in the trial pits (approximately 30%).

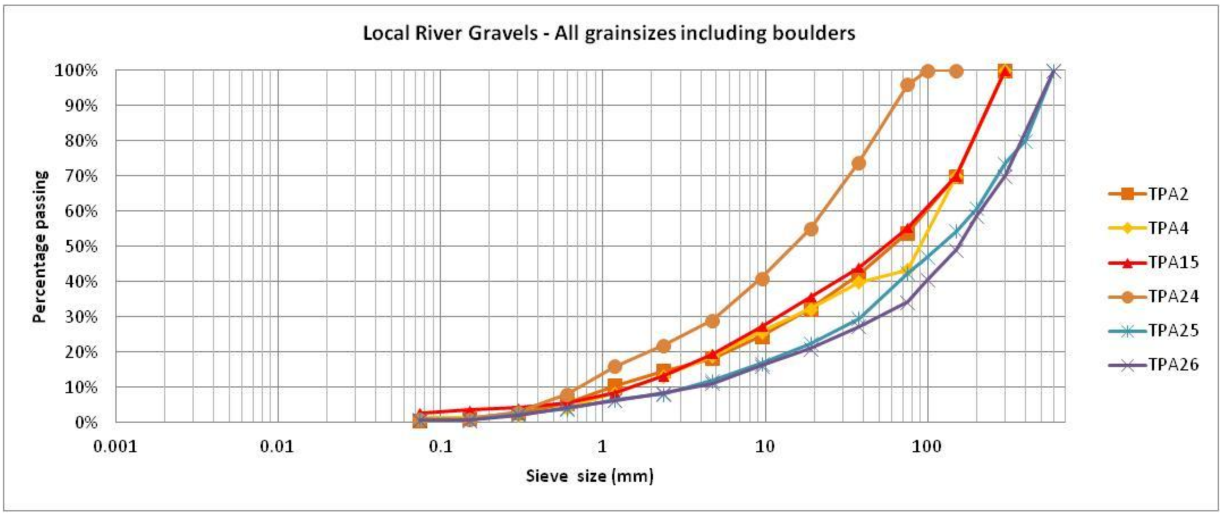


Figure 34: Particle Size Distributions for locally sourced alluvial gravel (recalculated to include boulder fraction).

Additional tests (results in brackets) undertaken to characterise the gravel suitability for aggregate include: Los Angeles Abrasion (18%), Crushing Resistance (>217kN), Sand Equivalent (47).

9.2.3.2 Alluvial gravel as concrete aggregate

Petrographic analyses were carried out on clasts from a gravel sample taken from the site (Black 2012, see Appendix N). The analyses were undertaken to determine the suitability of the gravel for use as concrete aggregate. The gravels are a range of rock types from a number of tectonic terranes that outcrop in the Lee Valley catchment and were separated into seven different clast types. The clasts were predominantly volcaniclastic sedimentary sandstones and siltstone/mudstones. The sand-sized fraction of the sample comprised mainly broken rock fragments of similar composition to the gravel. Of the seven samples analysed one contained some swelling clay.

Petrographic analyses conducted by Dr Philippa Black on a Lee River gravel (alluvium) sample from the site indicated that they have no potential for producing alkali-silicate reactions with concrete. Nor did they contain sulphides, organic matter, shell fragments or mica which may be detrimental to concrete. Professor Black noted that there is a tendency for pebbles and sand grains to be slightly elongated, and that one clast analysed contained swelling clay, but assessed the gravel sample as suitable for use as concrete aggregate. On-site screening and blending may be required to produce the necessary grading depending on the concrete design.

9.3 Filter materials

Filter materials to satisfy the grading requirements of the dam are likely to be sourced from screened alluvium or processed onsite quarried rock.

9.4 Blanket materials

Solifluction deposits are characterised by low plasticity or non-plastic fines and are suitable as blanket materials where some migration of fines to block seepage paths is desired. Particle size distributions from solifluction deposits sampled during the feasibility study are provided in Table 29.

Table 29: Grain size summary of solifluction deposits.

Sample /Source	Gravel %	Sand %	Silt %	Clay %	Fines %	D_{50} (mm)	D_{10} (mm)	D_{60}/D_{10}	PL %	LL %	PI %
5. Solifluction	47	47	6	-	6	1.4	0.1	20	-	-	-
6. Solifluction	48	34	14	4	18	1.9	.01	300	27	40	13

9.5 Rip rap

River alluvium contains high strength aggregate up to 600 mm diameter. The existing armour layer in the river bed is a potentially easily won source of 200-600 mm sized rip-rap and has an estimated volume of 21,000 to 22,000 m^3 based on field estimates.

High strength and durable (up to 1.2 m diameter) rip rap is also available from Taylors Quarry approximately 3.5 km from the site.

The sandstone from the Star Formation outcrops upstream from the site and was considered as a possible local source of rip rap materials. However, the steep topography in the area investigated is likely to make track formation and rock extraction uneconomic.

10 Conclusions and recommendations

A 52m high concrete faced rock fill dam (CFRD) dam is proposed in the Lee Valley to store water for augmenting flows in the Waimea River and to provide irrigation on the Waimea Plains. Various potential sites have been examined for the dam with the final investigations focussing on a site at CH 12,430 m upstream from the confluence between the Wairoa and Lee rivers. The investigations at the preferred location have comprised:

- Field mapping
- Seismic refraction investigations
- Borehole investigations and in situ testing
- Test pit investigations
- Laboratory Testing

The proposed dam site lies within greywacke rock of the Caples Group Rai Formation characterised by sedimentary rocks which have been subjected to low to medium grade metamorphism.

Active faults around the dam site include the Waimea Fault 8.5 km to the NW and the Wairau segment of the Alpine Fault 20 km to the SE.

The geology underlying the dam site comprises variably weathered, indurated, moderately strong to strong, closely spaced jointed, fine sandstone and siltstone. The measured unconfined compressive strength (UCS) of the rock lies between 6.6 and 143 MPa. The rock mass has been separated into three classes based on joint spacing (structure) and surface condition (weathering and condition of joint surfaces). These are summarised below:

- Class 1 – Unweathered, strong to very strong, blocky to very blocky rock mass
- Class 2 – Slightly weathered, moderately strong to strong, very blocky to disturbed rock mass
- Class 3 – Moderately to highly weathered, weak to moderately strong, blocky to disturbed rock mass.

Defects in the rock are dominated by bedding and three other sets of structural joints roughly orthogonal to bedding. Bedding within the dam footprint and spillway dips predominantly to the northwest, although there are local variations in dip and dip direction across the dam footprint.

Crushed and sheared zones are present within the rock mass and are mostly orientated sub-parallel to bedding and occur along sandstone/siltstone lithological boundaries. Crushed and sheared zones varied from 20 mm thick to 1m wide incipient zones of crushed rock containing clay seams. Persistent sheared zones are spaced approximately 10 to 50 m apart. Thirteen sheared zones have been identified by the investigations within the construction area.

For a concrete faced rock fill dam the concrete face along with the plinth excavation is the principal water retention structure and requires a foundation that is both incompressible and non-erodible. This will generally require excavation to at least Class 2 and possibly Class 1 rock and improvement by grouting.

Preparation of the plinth foundation on the right abutment requires cut slopes of approximately 10V:1H. Analyses of the joint orientations exposed on this abutment indicate that two types of wedge failure may occur. Slope support will be required and this will need to be confirmed by defect mapping and specific design as the rock is exposed during construction.

Preparation of the plinth on the left abutment will require low height cut slopes of up to 1V:0.8H. Local drapes of wire netting may be required to contain loose blocks of rock.

Local drapes of wire mesh may also be required to contain loose blocks rolling down the cut faces of the diversion channel excavation.

No indications of deep seated instability have been identified either on or above the right or left abutments. The right abutment is steep and construction of the plinth will require careful staging.

Foundation requirements for most of the dam require that the foundations are stiffer than the rock fill. This will be achieved by excavation to an appropriate grade of rock. Foundations prepared by bulldozers and or excavators using mainly blade and light ripping will generally satisfy the required stiffness criteria. In places the excavation will need to be deepened. Overall, the general foundation is likely to comprise an undulating surface over the majority of which no treatment is expected.

Higher quality rock will also need to be exposed in the vicinity of the downstream toe of the dam for the purposes of anchoring mesh which will be required to prevent flood scour.

Spillway cuts will be up to 60 m high and within rock classes 1 and 2. They will generally be cut at 1V:1H (with some local steepening above the chute floor). These slopes may be subject to shallow wedge and planar failures which may require spot bolting to stabilise. Not all rock in the spillway is likely to be rippable. Depending on blasting and the presence of sheared zones and dilated rock, some dental concrete may be required for the finished subgrade of the spillway. Excavation methodology must give due consideration to minimising dilation of the rock mass adjacent to finished profiles.

Permeability of the rock within the dam site was found to be variable and as such a grout curtain will be required beneath the plinth excavation to increase the seepage path and thereby reduce the leakage potential around and beneath the dam. Grout holes will need to be both vertical and angled on the abutments to treat the defects observed. Grout holes on the left abutment will need to be drilled at moderate angles to the southwest to seal the main defect sets. High permeabilities were observed in DH10 in the vicinity of the sheared zone at 31.6m. Sheared zones of 100mm or greater thickness are likely to be encountered every 20 to 30 m along the plinth excavation foundation and these will require individual grouting and/or dental treatment.

The stability of slopes around the reservoir has been investigated at a reconnaissance level. The results of this study show that the reservoir inundation will have little effect on existing areas of instability. There are, however, areas of steep ground mantled with rock scree that could mobilise during earthquake shaking or extreme rainfall events. In this respect the tree cover (exotic or native) should be maintained as much as possible to provide stability to the soil mantle.

When the reservoir is in use, an OBE or greater magnitude earthquake event or an extreme rainfall event could cause local landslides areas around the reservoir margin. These have the potential to generate waves within the reservoir. Beach creation and wave action may also cause localised instability of superficial deposits during reservoir operation.

On-site construction materials comprise sandstone and mudstone/siltstone rock of the Rai Formation, alluvial gravel and solifluction deposits.

Excavation of the rock has been assessed with rippability trials and seismic refraction profiles carried out on the site. These indicate that Class 3 rock is easy ripping, Class 2 rock is difficult to rip and Class 1 rock is very difficult ripping or unrippable. Transitions between the different rock classes were found to be irregular and there could be changes in rippability both laterally and vertically during excavation.

Compaction trials on Class 2 and 3 rocks indicate that rock fill is likely to have a Young's Modulus of between 20 and 30 MPa. Laboratory testing carried out before and after compaction yielded similar grain size distributions suggesting that the site-sourced rock fill is not prone to construction breakdown. If sufficient volumes of Class 1 and Class 2 rock are not available to complete the embankment some of the Class 3 rock when selectively excavated may be suitable for construction. Further trials at the time of construction are recommended to confirm this.

Alluvial borrow material is present adjacent to the Lee River and on terraces above the main channel. These materials are dominated by river gravels of which 62,000 m^3 may be located within 2 km upstream of the dam. The alluvial material contains up to 30% boulders and may be processed to yield rip-rap and filter materials for dam construction.

Solifluction deposits around the dam site can be processed to provide blanket materials if required. Nearby offsite sources exist where aggregate and armour stone can be sourced.

Nearby offsite sources exist where aggregate and armour stone can be sourced.

11 Applicability

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

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