REPORT

WAIMEA WATER AUGMENTATION COMMITTEE

Lee Valley Dam Feasibility Investigations
Geotechnical Investigation Report

Report prepared for:

WAIMEA WATER AUGMENTATION COMMITTEE

Report prepared by:

TONKIN & TAYLOR LTD

Distribution:

WAIMEA WATER AUGMENTATION COMMITTEE	3 copies plus 1 pdf
TONKIN & TAYLOR LTD – Wellington	1 copy
TONKIN & TAYLOR LTD (FILE)	1 copy

December 2009

T&T Ref: 24727.204

Table of contents

Executive summary

Geotechnical feasibility investigations have been carried out for a dam in the Lee River in a staged programme as part of the Waimea Water Augmentation Committee's (WWAC) Phase 2 feasibility study.

The issues addressed during the geotechnical investigations are:

- overall geological and geomorphic setting of the potential reservoir and dam
- the seismic hazard and preliminary seismic design criteria
- current and full reservoir groundwater regime
- dam embankment foundation conditions – strength, compressibility, erodibility and permeability
- dam foundation, abutment and spillway excavation stability
- current and full reservoir hill-slope stability
- availability and suitability of local materials for use in construction of the dam (bulk fill, core material, filters, drainage material and rip-rap).

Background

Stage 1 of the Phase 2 geotechnical feasibility assessment comprised geological and engineering geological mapping of the project area. The Stage 1 study assessed geotechnical issues at eight different potential dam sites in the upper Lee Valley, from Ch10,500 m to Ch12,400 m upstream of the Lee River/Wairoa River confluence. Engineering evaluation of the various dam sites identified a site at Ch11,010 m as being the most economical but noted that potential risks affected that site, including poor quality rock on the right abutment, potential large scale slope instability on the left bank upstream of the dam site, and lack of suitable core material and high quality rockfill.

Sampling and testing of borrow materials was carried out on selected sites as part of Stage 2A. This found that suitable plastic clay core material for an earth embankment dam was not locally available and the local bedrock would provide a lower quality rockfill, although alternative rockfill sources were available.

Stages 2 and 3 of the geotechnical 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 Ch11,010m. WWAC subsequently endorsed the recommendation to investigate an alternative site located between Ch12,100 m and Ch13,000 m.

Stage 4 involved geotechnical investigations between Ch12,100 m and Ch13,000 m. On the basis of preliminary engineering geological mapping and consideration of earthworks volumes, a site at Ch12,430 m was subsequently selected for drilling investigations.

Geotechnical Investigations Related to Proposed Dam Site at Ch12,430 m

The Stage 4 investigations covered the reach of the Lee River between Ch12,100 m and Ch13,000 m and were carried out in two parts. The first involved engineering geological mapping of river and track exposures and formation of access tracks to the potential dam site. This was followed by subsurface investigations including test pitting and drilling.

The investigations for the site included:

- Defect mapping and logging of 700 m of existing and excavated track batters;
- Excavation and logging of two test pits in the vicinity of the potential dam site;
- Drilling and logging of 140 m of core in five drill holes at the potential dam site and on the spillway alignment;
- Kinematic defect plots for 171 rock defect measurements,
- 32 packer (water pressure) tests.

Engineering Geology

Bedrock at the dam site is predominantly greywacke, locally inter-layered with argillite and is rated as being generally fair quality. Bedding layers are closely spaced and are the predominant defect. The bedding dips generally at 30-60° downstream beneath the dam footprint. Other rock defects are present and include orthogonal sets of sheared zones that are mapped at 10 to 50 m spacing, and joints that are often spaced at 1 to 3 m but which seldom persist for more than 10 m.

The defect orientation beneath the dam site is considered favourable with respect to stability, and no bedrock instability has been noted beneath the dam footprint. However, deep-seated rock relaxation is evident downstream of the left abutment of the dam and locally, on the steeper right abutment slopes.

Soils, locally up to 12 m deep, consisting of slope derived silt and sand and alluvial gravel, overlie bedrock on the left abutment, upstream of the dam centreline. On the right abutment scree and colluvium is generally less than 2 m thick but is locally up to 5 m thick along the axes of steep gullies that are spaced across the slope at about 50 m.

Packer tests and groundwater response in the boreholes during drilling indicate that rock mass permeability is likely to vary between 1×10^{-7} to 1×10^{-6} ms^{-1}, with higher permeability being associated with relaxed rock and adjacent to sheared zones.

No active faults have been identified in the immediate vicinity of the dam site. Active fault traces have been mapped 8 km to the west, within the Waimea Flaxmore fault system, and the Alpine Fault (Wairau Segment) is located 21 km east of the site. The Alpine Fault (Wairau Segment) is likely to pose the main seismic threat to the dam.

No active large landslides have been identified in the potential reservoir footprint. However, solifluction deposits, that blanket the lower level reservoir slopes, are subject to shallow slumping and erosion. It is anticipated that groundwater levels will be raised by the reservoir inundation, and local instability associated with solifluction slopes can be expected.

Implications for Dam Design

The investigations indicate that geological conditions at the proposed dam site (Ch 12,430m) are generally suitable for a Concrete Faced Rockfill Dam, and that suitable construction materials exist in the vicinity.

The key conclusions from this phase of work with respect to dam design and construction are as follows:

Foundation Preparation

- There is a degree of variability in rock mass quality requiring local subexcavation and or special treatment of poor quality rock associated with crushed, shattered, sheared or dilated rock.
- Special treatment will be required to mitigate piping/erosion of fines within sheared zones.
- Soil stripping depths on the left abutment will be up to 12 m and on the right abutment will be up to 5 m.
- Provision will be required for local stabilisation of temporary slopes on the right abutment.

Cut Slope Stability

- Excavations for the plinth will daylight local wedge blocks and undercut scree infilled gullies. These deposits will be unstable when cut to slopes steeper than 40°.
- The orientation of principal defects is favourable for the provisional spillway alignment, although sheared zones and joints in some locations will limit the maximum stable batter angles in some rock to 45°.
- Poor quality rock in the upper section of batters will require batters to be no steeper than 40°, and in soil to be no steeper than 36°.

Leakage Potential

- Low permeability can generally be expected in unweathered to slightly weathered rock below the zone of surface relaxation, but there is a potential for high leakage along rock defects and within rock affected by deep seated relaxation.
- Provision will be required for grouting and or near surface foundation treatment.

Construction Materials (for concrete faced rockfill dam)

- It is likely that suitable sources of rockfill can be sourced from either the spillway cut or local alluvial deposits within the potential reservoir footprint.
- Rockfill properties are likely to be strongly influenced by the degree of compaction.
- Poor quality rockfill will be produced from moderately weathered to highly weathered relaxed rock within the spillway excavation. This may not be suitable for rockfill.
- Riprap >600 mm may need to be imported.

- Aggregates for concrete/filters and drainage are likely to be sourced from local alluvium within the potential reservoir footprint. However the durability of fines may limit use for some filters.
- Local solifluction deposits can be used for non plastic fines applications.

Recommendations for further investigations that should be undertaken as part of the design process include:

- Further investigations are required to assess the range of rock strength and defect orientations. This should include uniaxial compression testing of rock core and drilling to accurately locate principal sheared zones.
- Monitoring water levels and flows in ephemeral streams around the dam, including observations of seepage and groundwater emergence after rainfall should be carried out for a full cycle of seasons prior to design.
- Further mapping, drilling and test pitting should be carried out to delineate potential permeable zones or aquitards beneath the dam in the abutments.
- Further systematic mapping of defects should be carried out from test excavations and oriented drill core on both abutments and the spillway.
- Defect strength testing should be carried out including testing clay/silt sheared seams and joint and bedding plane surfaces.
- Further detailed engineering geological mapping of the full reservoir should be undertaken during the detailed design phase, and attention should be given to stability modelling of those slopes with elevated risk of slope failure in order to quantify the volumes of landslide debris that could be generated.
- A site specific seismic assessment, that considers this potential scenario of an earthquake generated within the Waimea-Flaxmore Fault system as well as the Alpine Fault, should be carried out as part of the detailed design stage.
- Further attention should be given to potential seepage paths and the groutability of the rock mass. This should include continuous packer testing in additional abutment drillholes, including testing in inclined holes to intercept steeply dipping defects in the right abutment.
- Dispersion and erosion testing should be carried out on shattered and sheared zone material to assess the potential for internal erosion of fines.
- Further delineation of soil depths and rock classes will be required to optimise the foundation stripping depths for construction.
- Trial excavations and compaction trials on rockfill will be required for Class 1, 2 and 3 rock types and alluvium. Laboratory testing should be carried out on as-compacted soils to assess grading and permeability characteristics.
- Specific attention should be given to testing durability of the fill and filter materials under cycles of shaking, freeze thaw and wetting.

1 Introduction

This report presents the results of geotechnical investigations completed as part of the Waimea Water Augmentation Committee's (WWAC) feasibility study for a potential dam on the Lee River in Tasman District.

1.1 Background

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 Figure 24727.204-F1.

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, from chainage (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 Ch11,010 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 suitability of core material and high quality rockfill.

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 rockfill, although alternative rockfill sources were available.

Stages 2 and 3 of the geotechnical investigation programme comprised surface mapping, test pitting and drilling passed on a dam site at Ch11,010 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 Ch11,010. WWAC subsequently endorsed the recommendation to investigate an alternative site located between Ch12,100 m and 13,000 m.

Stage 4 involved geotechnical investigations between Ch12,100 m and 13,000 m. Those investigations are outlined in this current report. On the basis of preliminary engineering geological mapping and consideration of earthworks volumes a site at Ch12,430 m was subsequently selected for drilling investigations.

---

[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.

These investigations have been independently peer reviewed by Dr Trevor Matuschka of Enginering Geology Ltd. Copies of his peer review reports are appended in Appendix I.

1.2 Project Description

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, and together they form the Waimea River.

The reservoir will be impounded by an embankment dam at a location of Ch12,430 m. The dam would be approximately 52 m high and 210 m wide at crest level. The location and preliminary layout of the dam and reservoir is shown in Figure 24727.204-F2. The storage reservoir will have a top water level of RL197 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 drawoff.

A concrete faced rockfill dam, constructed with approximately 380,000m^3 of locally sourced rockfill is proposed. Structures associated with the dam include a main spillway, auxiliary overflow spillway, diversion conduit that will be utilised after construction as an irrigation off take, and sluice.

1.3 Purpose and Scope of the Report

The purpose and scope of this report is to describe the investigations that have been carried out to provide geotechnical information to assess the feasibility of a dam at Ch12,430 m on the Lee River. It has drawn on specific investigations carried out as part of Stage 4 and utilises information gathered during Stages 1 to 3 of the geotechnical feasibility study as described in Section 1.1.

Specifically, the issues addressed within this report are:

- overall geological and geomorphic setting of the potential reservoir and dam,
- the seismic hazard and seismic design criteria,
- current and full reservoir groundwater regime,
- dam embankment foundation conditions – strength, compressibility, erodibility and permeability,
- dam foundation, abutment and spillway excavation stability,
- current and full reservoir hill-slope stability,
- availability and suitability of local materials for bulk fill, core material, filters, drainage material and rip-rap.

1.4 Reference Documents

The following documents have been referred to in the preparation of this report.

1. Johnston, M.R. 1982: Sheet N28BD – Red Hills 1st ed. Geological Map of New Zealand 1:50000 With Notes. Wellington New Zealand DSIR.

2. Rattenbury, M. S.; Cooper, R. A.; Johnston, M.R. 1998: Geology of the Nelson Area. Institute of Geological and Nuclear Sciences 1: 250 000 Geological Map 9.

3. Tonkin & Taylor 2007: Geology of the Lee River Catchment and Environs of Potential Dam Sites. Unpublished T&T reference 24727.200, dated December 2007.

4. Gerstenberger, M., Langridge, R., McVerry, G., King, A. and Stirling, R. 2003: Proposed Wairau Valley Hydro Development Earthquake Design Spectra IGNS Ltd Client Report 2003/140.

5. Fraser, J.G.; Nicol, A.; Pettinga, J.R.; Johnston, M.R. 2006: Paleoearthquake investigation of the Waimea-Flaxmore Fault System, Nelson, New Zealand. Proceedings of New Zealand Geotechnical Society 2006 Symposium, Nelson, February 2006.

6. Houlsby, A.C. 1976: Routine interpretation of the Lugeon Water-Test. Auarterly Journal of Engineering Geology and Hydrogeology, Nov 1976, Vol 9, No. 4, p 303-313.

7. Bieniawski, Z.T. 1989: Engineering Rock Mass Classifications. Publ. Wiley, New York 251 pp.

8. Hoek, E. and Diedenchs, M.S. 2006: Empirical estimation of rock mass modulus. International Journal of Rock Mechanics and Mining Sciences, 43, 203-215.

9. Read, S. and Richards, L. 2008: Design inputs for stability assessment of dams on New Zealand greywackes. IPENZ Proceedings of Technical Groups 33/1 (LD).

10. Bieniawski, Z.T. and Orr, C.M. 1976: Rapid site appraisal for dam foundation by geomechanics classification 12th ICOLD. Mexico. Q46. R32.

11. Romana, M. 2003: DMR (Dam Mass Rating). An adaption of DMR geomechanics classification for use in dam foundations. International Congress on Rock Mechanics organised by ISRM, Sudafrica, September 2003.

12. AS/NZS/1170

13. Houlsby, A.C. 1992: Grouting in Rock Masses. In Engineering in Rock Masses. Bell, F.G. 1992: Butterworth Heinemann, London.

14. Weaver, K.D., Bruce, D.A. 2007: Dam foundation grouting, Rev. ed. ASCE 473 pages.

15. Fell, R., MacGregor P. and Stapledon, D. 2005: Geotechnical engineering of dams. A.A. Balkema Publishers The Netherlands.

16. Barton, N and Kjaernsli, B 1981: Shear Strength of Rockfill, Journal of the Geotechnical Engineering Division ASCE Volume No. 6 & 7 July 1981.

17. Hunter, G and Fell, R. 2003: Rockfill Modulus and Settlement of Concrete Face Rockfill Dams. Journal of Geotechnical and Geo-environment Engineering ASCE, October 2009.

1.5 Abbreviations Used in this Report

The following abbreviations and meanings are used in the report.

HW	highly weathered

MW	moderately weathered
SW	slightly weathered
UW	unweathered
HW-MW	highly to moderately weathered
ML	low plasticity silt
UCS	unconfined compressive strength
SZ	sheared zone
BGL	below ground level
GL	ground level
MCE	maximum credible earthquake
CRF	concrete faced rockfill
DH	drillhole
TRB	true right bank looking downstream
TLB	true left bank looking downstream
Ch	chainage
RL	reduced level

Further explanation of terms is provided in the engineering log terminology sheet included in Appendix D.

2 Site Investigations

As noted in Section 1.1 geotechnical field investigations have been carried out in a number of stages. Investigations predating the current 2008 - 2009 investigations focussed around previously proposed embankment locations downstream of the current proposed site. The different stages of investigation and their purpose are listed below.

2.1 Phase 1 Investigations 2006-2007

This pre-feasibility investigation included walkover surveys and test pitting at potential dam sites located between Ch10,800 m and 11,300 m, and assessment of potential borrow materials at sites between Ch19,800 m and 12,400 m. As part of this investigation the following relevant testing was undertaken:

- Geologic and geomorphic mapping,
- Excavation of 19 test pits.

2.2 Phase 2 Stage 1 Investigations (October – December 2007)

This investigation was undertaken by Tonkin & Taylor and Dr Mike Johnston and involved geological and engineering geological mapping in the Lee Valley and its tributaries specifically between Ch10,500 m and 12,400 m, and structural mapping of bedrock extending upstream to Ch14,200 m. This investigation included assessment of slope stability, assessment of fault hazard and seismic risk, and assessment of potential borrow sites within the upper Lee Valley.

2.3 Phase 2 Stage 2A Investigations (January – February 2008)

This investigation involved evaluation of potential borrow sites for sources of earthfill and rockfill. It involved sampling and laboratory testing for grading and plasticity.

2.4 Phase 2 Stages 2 and 3 Investigations (April – June 2008)

This investigation included assessment of the engineering geology of a dam site at Ch11,010 m, assessment of the stability of a landslide on the left bank between Ch11,700 m and 11,900 m, and an assessment of a potential hard rock borrow site on the left bank at Ch10,200 m. As part of this investigation the following relevant measurement and testing was undertaken:

- Excavation and logging of 18 test pits in the vicinity of the dam site and landslide,
- Drilling and logging of three triple-tube rotary drill holes (DH1 – DH3) at the potential dam site,
- Drilling and logging of one triple-tube rotary drill hole (DH4) at a potential hard rock borrow site,
- Rock defect measurements,
- 132 point load strength tests on drill core and excavated rock samples,

- 33 packer (water pressure) tests,
- Monitoring groundwater levels in piezometers installed in drill holes.

2.5 Phase 2 Stage 4 Investigations July 2008 – February 2009

Stage 4 investigations covered the reach of river between Ch12,100m and 13,000m and were carried out in two parts. The first involved engineering geological mapping of river and track exposures and formation of access tracks to the potential dam site. Track cutting and mapping commenced in late July 2008, but was suspended following loss of access to the site due to extreme wind storms and subsequent emergency logging activities. Access was restored in October 2008.

Investigations covered the reach of river between Ch12,100 m and 13,000 m.

Drill hole locations were determined after review of the mapping phase. Drill hole locations were constrained due to the difficulty in forming access. Factors affecting access were:

- requirement to minimise effect on plantation trees,
- steep slopes and very strong rock on the right bank downstream of Ch12,250 m,
- steep slope and scree on left slope at Ch12,350 m,
- two river crossings.

As part of this investigation stage the following relevant measurement and testing was undertaken:

- Excavation and logging of two test pits in the vicinity of the potential dam site,
- Defect mapping and logging of 700 m of existing and excavated track batters,
- Drilling and logging of 140 m of core in five triple-tube rotary drill holes (DH5–DH9) at the potential dam site and on the spillway alignment as follows:

Drill hole No	**Easting (GPS)**	**Northing (GPS)**	**RL**
DH 5	2523325	5970630	213
DH 6	2523308	5970720	201
DH 7	2523408	5970639	182
DH 8	2523491	5970783	194
DH 9	2523511	5970685	153

- Kinematic defect plots for 171 rock defect measurements,
- 32 packer (water pressure) tests.

The geological conditions on the site are presented on a series of plans and cross sections (Figures 24727.204-F6-F9) which are attached in Appendix A. Logs and photographs of the cores from drill holes from the investigations are presented in Appendix B. Lugeon water pressure testing and falling head permeability tests were carried out in the boreholes during drilling. These permeability test results are included Appendix C.

Test pit logs and Excavation face logs results are presented in Appendix D along with an explanatory sheet outlining the terms and symbols used.

Appendix E contains stereographic plots of rock defects.

Appendix F contains a selection of photographs of geological and geomorphic features at the site.

Test locations were estimated from high resolution aerial photographs and 0.5 m contour plans or surveyed using GPS only, so drill hole and pit locations and levels are approximate (±2m).

3 Geological Conditions

3.1 Regional Geological Setting

The regional geology is summarised in Figure 24727.204 –F3. The Richmond Ranges east of the Waimea Basin are composed of Upper Mesozoic-Lower Paleozoic rocks and are dominated by sandstone, siltstone and mudstone (greywacke and argillite) although igneous and metamorphic rocks (schists) also occur. The geology is divided into several north-east trending belts of rock (terranes) that are controlled by major faults. Rocks belonging to the Caples terrane occur within and predominantly upstream of the project area.

The Alpine Fault (Wairau segment) is located in the Wairau Valley, immediately to the south and east of the Richmond Ranges. This is an active fault, and a major tectonic boundary that separates the Pacific and Australian crustal plates. The Waimea-Flaxmore Fault System, that splays off the Alpine Fault near St Arnaud, forms the boundary between the Richmond Ranges and the Waimea Basin. Active fault traces are present within this fault system.

Field mapping carried out for the Phase 2 study generally confirmed the general distribution of rock types and regional structure that is shown on the 1:50,000 regional geology map [Ref 1] (see Figure 24727.204-F4). Greywacke, included in the Rai, Star and Ward Formations that form the Caples terrane, is the predominant rock type outcropping in the upper Lee Valley. A mixed assemblage of dolerite, basalt, serpentinite and greywacke form the Patuki Melange that is mapped downstream of the project area. Small areas of Croisilles Melange, that also include basalt, dolerite and serpentinite are locally contained within bedrock of the Caples terrane in the upper Lee catchment. [Ref 2].

Tonkin & Taylor report "Geology of the Lee River Catchment and Environs of Potential Dam Sites" [Ref 3] (T&T reference 24727.200, dated December 2007) provides a more detailed description of the regional setting and underlying bedrock structure.

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 are documented in www.geonet.org.nz are as follows:

Table 1 – Historical earthquakes

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.

Research by GNS 2003 [Ref. 4] indicates that the latest estimate of the recurrence interval for displacement on the Wairau Fault is 1,600 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 M7.6 event.

Based on the coincidence of the elapsed time and recurrence interval, and the coincidence of accumulated strain and single event displacement history, GNS have concluded that there is a relatively high risk of such an 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. 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).

The seismic hazard presented by the Waimea Fault has been assessed by Fraser et al, 2006 [Ref. 5]. They carried out trenching of Quaternary terrace surfaces at the mouth of the Wairoa Gorge that have been displaced by the Waimea Fault. Three fault displacements have been determined within the last 18,000 years with an average recurrence interval of 6,000 years. A magnitude M7.0 earthquake has been estimated for rupture of the Waimea Fault.

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 Com) as a relatively minor one and there is no evidence that it is active.
- *Faults adjacent to the Croiselles Melange*: Several north-east trending lineations are associated with the Croiselles 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.

3.3 Geomorphology

3.3.1 Introduction

The Richmond Ranges rise to over 1,721 m altitude in the headwaters of the Lee River, Figure 24727.004-F1. Apart from the mountain tops, slopes are steep and are covered in indigenous forest. In the vicinity of the project are and in the lower reaches, where the ridge crests are approximately up to 1,000 m above the Lee River, the slopes are largely planted in exotic forestry, Figure 24727.204-F2.

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 argillite (and fault weakened rock) has been progressively eroded. Natural slope angles are influenced by the underlying bedrock lithology and also by aspect, with greywacke generally forming steeper and more uniform slopes than the Melange lithologies, and south facing slopes being generally steeper than north facing slope.

3.3.2 Dam Site Slope Features

Surface features are shown on Figure 24727.204 – F5.

The general valley profile swings from a northerly trend upstream of Ch12,400 m to a north-west alignment downstream from Ch12,400 m to about 11,800 m. Weaker rock, associated with the Anslow Fault downstream and to the west, and stronger rock upstream and to the east, probably influenced this changing alignment as the river has downcut over the last few million years.

There is a pronounced kink in the alignment of the river between Ch12,350 and 12,550 m (the proposed dam site) and the present river bed is now aligned more to the west than the general slope at higher levels. This alignment is parallel to surface lineaments along gully lines to the north-west and north of the dam site and follows the trend of north-west dipping rock exposed on the left bank and a similar orientated sheared zone in the river bank downstream of the site. The section of valley at the proposed dam site is the narrowest within the project area. At this location the valley is broadly V shaped in profile; the floor of the valley is 50 m wide and the slopes rise at 35 to 45°.

The left abutment slope is a truncated spur at the northern end of a long gently plunging ridge. A rock bluff rises to about 30 m above the river and the slope is locally steep, but elsewhere the slopes on the left bank seldom exceed 35° and there are few natural outcrops. The crest of the ridge is at RL240 m at the dam site but the ridge is narrow and plunges to the valley floor at Anslow Creek at Ch12,200 m.

The slopes on the right abutment rise at 35 to 45° to about RL300 m and rise at a slightly flatter gradient to the ridge crest at RL420 m. The right abutment slope, particularly downstream of Ch12,500 m is characterised by numerous rock outcrops along spur lines and scree deposits within shallow gullies.

3.3.3 Reservoir Slope Features

Main Valley

Upstream of the dam site the valley is aligned in a northerly direction. The valley floor widens upstream of Ch12,650 m through to Ch13,400 m where Waterfall Creek enters the valley. A gently inclined alluvial fan at the mouth of Waterfall Creek overlies a broad flat terrace in the main valley.

The right bank slopes upstream of Waterfall Creek to Ch14,000 m are characterised by actively eroding bluffs (>45°) rising to between 40 and 50 m above the river bed. Upslope of the bluffs between Ch13,500 m and Ch14,000 m a landslide deposit, partly overtopped by solifluction deposits, extends onto a terrace remnant approximately 40 m above the river. Higher slopes are inclined at 20° to 26° and are extensively blanketed by solifluction deposits.

On the left bank, between the dam site and Ch13,500 m the ridge rises to RL500 m. An extensive apron of solifluction deposits lying at 20 to 35° blankets the lower slopes up to about RL260 m. Bedrock slopes above this are inclined at 34 to 40°. A gully with gentle gradient falling to the north, just below and parallel to the ridge crest, forms a prominent lineament that is also evident crossing ridge lines to the south.

Slopes further upstream, and on the northern side of the Flat Creek arm are generally steep (38 to 42°) and contain rock bluffs. In contrast, the southern slopes of Flat Creek are more gently inclined (30-34°) and are characterised by few outcrops.

Upstream of Ch14,500 m the river is entrenched in a narrow gorge with steep bluffs rising to about 100 m above river level on both sides of the valley. These bluffs have not been inspected, but from aerial photographs examined, they appear to be stable.

Large landslides have formed in a variety of rock types in the head of creeks draining into the Lee River upstream of the reservoir, but are beyond the likely reservoir extent and have not been inspected.

Waterfall Creek Arm

Waterfall Creek enters the main valley on the true right side. Within the extent of the reservoir it is V shaped in profile. The side slopes above the reservoir level are inclined at 38 to 42 ° on the northern side and 31 to 41° on the southern side. The slopes are planar in profile but are incised by narrow steep sided gullies spaced at 100 to 200 m. Gullies on the northern side are actively eroding. Flatter topography, inferred to be a landslide (LS2) infills a tributary gully above the upstream end of the reservoir on the southern side of Waterfall Creek. Upstream and east of the reservoir, Waterfall Creek is significantly asymmetric in profile (northern slopes 38 to 41° and southern slopes 12 to 29°), and the southern side of the valley is inferred to be a large bedrock landslide that is buttressed against the northern slope.

3.4 Reservoir and Dam Site Geology

3.4.1 Rai Formation

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.

Bedded sequences dominate the Rai Formation and although individual beds vary considerably in thickness they are typically spaced at 100 mm. Bedding throughout the area dips predominantly to the north-west and meso folding within the sequence is common, particularly within the argillaceous rocks. Individual bedding layers are not continuous over large distances. They appear to have been sheared prior to metamorphism. This original bedding plane shear has been healed by quartz recrystallisation during metamorphism (annealed). However, a preferred weakness exists along bedding and subsequent phases of tectonic deformation and local deformation of slopes by creep and/or seismic shaking has led to localised reshearing along bedding.

3.4.2 Star Formation

The Star Formation, dominated by indurated massive to poorly bedded greywacke, has been mapped within the proposed reservoir near Ch14,500 m and forms much of the upper left bank slopes of the reservoir. It also provides the main armour rock within the active river bed.

3.4.3 Patuki Melange

The Patuki Melange outcrops in the Lee River downstream of the dam site and forms the higher slopes to the west of the study area. It consists of blocks of indurated gabbro dolerite and basalt rock, ranging from less than 1 m to over 1 km in size, in a serpentinitic matrix. Investigations carried out during Stage 2 revealed a high variability in rock quality and weathering over short distances.

3.4.4 Croiselles Melange

The Croiselles Melange is mapped locally on the ridges above the right bank upslope of the reservoir and in the upper catchment of Waterfall Creek. It consists of blocks of ultramafic and mafic rocks and siltstone, enclosed within a serpentinite or sedimentary matrix. It is commonly characterised by widespread instability.

3.4.5 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.

Low Level Terrace Gravel

Low level terrace deposits on the right bank are preserved between Ch11,700 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 Ch12,800 m and 13,350 m.

Low level terraces are preserved on the left bank between Ch12,100 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 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 and unconsolidated. The deposits vary in thickness from one to three metres.

Mid Level and High Level Terrace Gravel

Mid Level terrace deposits up to 6 m thick occur locally 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. Isolated high level deposits, some at 40 m above the river bed and occasional deposits at 60 m above the river bed, 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 RL210-215 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.

3.4.6 Slope Deposits

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 Ch12,700 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 not observed below the level of the mid level terraces (i.e. in the lowest 10-20° of slope).

Solifluction deposits are stratified soil deposits, layered parallel to the slope. They are dominatedby 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 redeposited 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.

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, and outcrops and in narrow gullies on steep slopes (greater than 35°and up to 50°). They are 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 GRAVEL, unweathered to slightly weathered.

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 evident on the left bank, 500 m downstream of the proposed dam site, between Ch11,700 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.

An ancient and eroded earthflow deposit that contains debris derived from Croiselles Melange has been mapped on the right bank at Ch13,600 m and 14,000 m overlying a rock bench and high level alluvium at RL210 m. A large landslide deposit incorporating Croiselles Melange and Rai Formation is also inferred upstream of the reservoir extent in Waterfall Creek.

Rockfall deposits are locally evident at the foot of bluffs, mainly Ch13,300 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.

3.5 Groundwater

Groundwater information at the proposed dam site is limited to water levels measured in piezometers installed in the five drill holes at the dam site, from earlier drilling at Ch11,010 m and observations of emergent surface flows locally on the slopes.

Groundwater profiles are indicated on the cross sections in Figures 24727.204-F7, F8 and F9. Groundwater profiles have been plotted assuming a simple unconfined groundwater regime, with the permanent groundwater table being a subdued reflection of topography. Locally, perched groundwater is inferred within the solifluction and alluvial gravel deposits that overlie bedrock.

Insufficient groundwater monitoring has been carried out to assess the range of seasonal fluctuation of groundwater within either bedrock or in overlying soils.

The permanent groundwater table is located within slightly weathered rock and the interface between unweathered and slightly weathered rock is inferred to represent a low level of previous groundwater levels. Groundwater is emergent at river level. On the right abutment slope it rises at a grade of 1V:3H and is at 20 m depth BGL at DH8 (GL at approximately RL190 m).

The groundwater profile is steeper on the left abutment, particularly behind the north-east facing slope between Ch12,350 m and 12,450 m. It is at 10 m BGL in DH5 (GL at RL214 m approximately) and falls at a gradient of 1V:2H to the east through DH7 (cross section F). At DH 6, groundwater is at 15m depth and must fall steeply to the river. In cross section C the gradient is inferred to be 1H:1V. It is not clear whether this is a result of very low permeability unweathered rock or the presence of steeply dipping sheared zones that may act as an aquitard.

Flow paths have not been established, and it has been assumed that flow is generally in a downslope direction, at right angles to the general slope contour. However, the relatively elevated groundwater level in DH6 suggests that there might be a northerly (or north-west) component of flow along the line of the ridge that forms the left abutment.

The general groundwater profile observed at this site is consistent with observations at Ch11,010 m and a similar regime is inferred within the reservoir slopes that are underlain by Rai Formation greywacke. Local variations are likely, particularly near the terrane boundary faults.

4 Geotechnical Properties at Proposed Dam Site Ch12,430 m

This section describes the results of the geotechnical investigations undertaken for proposed dam in the vicinity of Ch12,430 m. The field investigations have been detailed in Section 2.

4.1 Rock Types

Bedrock 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.

4.1.1 Extent of Outcrop

The distribution of outcrop and exposure in tracks is shown on Figure 24727.204-F6. It is well exposed in the vicinity of the dam foundation on the right bank at river level from Ch12,240 m to 12,525 m and in the left bank between 12,160 m and 12,220 m, 12,240 m and 12,320 m, 12,350 m and 12,405 m (in a bluff rising 20 m above the river) and between 12,545 m to 12,650 m. It is also well exposed in forestry access tracks, notably:

- on the right bank slopes at RL280 m to RL300 m upslope from Ch12,380 m to 12,600 m,
- on the left bank at RL215 m (Lee Valley Road) and RL190 m to RL205 m between Ch12,400 m and 12,700 m.

Elsewhere outcrop is intermittently exposed mainly along spur lines.

4.1.2 Structure

At the dam site bedrock consists of a sequence of greywacke sandstone and argillite beds generally dipping at between 30 to 60° towards the north-west. There is a progressive steepening of dip from upstream to downstream in the river exposures between Ch12,550 m (35 to 45°) to 12,340 m (50 to locally 90 °). From Ch12,340 m downstream to 12,220 m the bedrock dips in the opposite direction; that is to the south-east, at 40 to 60°. No major fold axis has been identified and the reverse dip seen in the river exposures is not evident in outcrop at higher elevations. Locally, within the zone of south-east dipping rock, there are poorly formed chevron and kink folds and irregular quartz veins.

4.2 Intact Rock Strength

No specific strength testing, such as unconfined compressive strength (UCS), has been undertaken during this phase. During the previous investigation phase it was established from point load testing on rock core that undisturbed SW Rai Formation bedrock core had point load index Is values between 1.33 and 2.36 which may be the equivalent of an unconfined uniaxial strength between 30 and 52 MPa for fracturing parallel to and normal to bedding respectively.

Core from rock that had been tectonically disturbed or influenced by nearby sheared zones had point load index Is values between 0.63 and 0.9 which may be equivalent to an unconfined uniaxial strength between 14 and 20 MPa for fracturing parallel to and normal to bedding respectively.

However, the size of core and influence of microfractures on core strength has conservatively influenced these results.

Point load testing on 20 samples of excavated rock that varied from highly weathered to slightly weathered rock had point load index Is values between 2.4 and 3.0 which may be the equivalent of an unconfined uniaxial strength between 52 and 67 MPa for fracturing parallel to and normal to bedding respectively. When broken down by weathering grade the UCS is estimated as follows:

SW rock:	66 MPa parallel bedding	90 MPa perpendicular bedding
MW rock:	24 MPa parallel bedding	53 MPa perpendicular bedding

These results are consistent with field assessment of rock strength.

4.3 Rock Mass Defects

Rock mass defects have been mapped and plotted on stereographic projections for the right and left abutment slopes. These are included in Appendix E . Rock mass defects include bedding, joints and sheared zones. Typical of greywacke rocks, there is a broad scatter of defect orientations. However, five sets of defect orientations are recognised. The defect orientations are shown in the cross sections in Figures 24727.204-F7–F9.

4.3.1 Bedding

Bedding is the prominent defect set and is generally spaced at 100 mm but may range from less than 10 mm to more than 1 m (uncommon). Bedding beneath the dam generally dips to the north-west at between 30° and 60°. However, near the downstream toe of the dam the dip is locally subvertical. Between the downstream toe of the dam and the Anslow Fault bedding dips south-east at 40° to 80°. This bedding dip reversal is only evident in river exposure. Bedding defects are tight in unweathered to slightly weathered rock. However, the rock readily splits along micro fractures that lie parallel to bedding, and in weathered exposures bedding defects are more closely spaced and are open. Thin silt films are occasionally present on bedding surfaces.

4.3.2 Joints

There are four major prominent and persistent joint sets joint sets as follows:

- *Joint Set A.* Moderately steep (40-50°) north-east dipping set. This controls the bluff on the true left bank at Ch12,350 m;
- *Joint Set B.* Steep (60-80°) south-west dipping joints that form the upstream face of prominent outcrops on the true right bank upslope of Ch12,500 to 12,550 m;
- *Joint Set C.* Moderately steep to steep (40-60°) east dipping joints, mapped at several locations in the river exposures beneath the dam footprint, but not evident elsewhere;
- *Joint Set D.* Generally steep (70°) south-east dipping joints that are conjugate to bedding.

Joint spacing varies from 1 to 5 m. Joints are typically open in MW rock and tight in SW and UW rock, although many joints that are infilled with quartz veins are locally open even in UW rock. Joint wall surfaces are typically smooth to slightly rough. Individual joints seldom persist for more than 3m, but are stepped across bedding or sheared zones.

No specific asperity or waviness measurements have been taken, but they are observed to show meso scale waviness.

4.3.3 Sheared Zones

Sheared zones have been mapped, logged in core or inferred by surface lineaments. The most significant are shown in Figure 24727.204-F6 and on the cross sections. They form a generally orthogonal pattern of zones of weakness beneath the dam footprint. Sheared zones are mainly mapped parallel to bedding and Joint Sets A and B although other orientations are also evident locally. The most common are bedding parallel sheared zones where argillaceous beds have been sheared and crushed between more competent sandstone beds. Bedding plane sheared zones vary from 20 mm thick to about 1 m wide incipient zones of shatter containing clay crushed seams. Persistent sheared zones are spaced at 10 to 50 m intervals.

Sheared zones (SZ) mapped or inferred in the vicinity of the dam site include the following:

- *SZ3.* The most persistent bedding parallel sheared zone is observed in DH6 in the left abutment spillway area at a depth of 23 m (BGL). This sheared zone can be correlated with breaks in outcrop at river level and a 300mm weathered clay crushed seam exposed in a track cutting downslope of DH8. In DH6 this sheared zone corresponds with the base of dilated rock and the transition from MW to SW rock. SZ3. It will be intercepted in the spillway excavation and will be close to the downstream toe of the dam.

- *SZ4.* A subvertical healed zone of shattered rock that is evident in river exposure at Ch12,360 m. May be a sheared fold axis. Is is not evident at higher elevations.

- *SZ12.* This is a bedding parallel sheared zone defined by several thin crushed seams and shattered core between 28 and 30 m BGL in DH7 and inferred to correlate with a 0.5 m break in outcrop in the river exposure at Ch12,520 m. This feature would lie beneath the upstream shoulder of the dam.

- *SZ1, SZ10, SZ11, SZ13.* SZ1 is well exposed in the river at Ch12,270 m, and inferred by an eroded slot in bedrock in the DH8 access track. It includes between 1 and 2 m of altered, crushed rock contained within footwall and hanging wall clay gouge seams that dip to the north-east at 45 to 55°. SZ13 is logged as a steeply dipping crushed zone at 24.7 to 26.5 m in DH7. SZ10 and SZ11 are inferred based on breaks in outcrop and subdued slope features upstream of the dam site left abutment.

- *SZ8.* This is a prominent sheared zone 0.3 to 0.5 m wide that dips north-west at 75° and is exposed in the river bank outcrop at Ch12,450 m. Its continuation on the left abutment is suggested by a linear gully infilled with solifluction and colluvium. On the right abutment it is correlated with a steep sheared zone in the forestry access track at RL300 m.

- *SZ6.* This is a narrow sheared zone that dips west at 40°. It is identified in two track batter exposures and will be intercepted in the spillway cut.

- *SZ2, SZ5.* SZ2 is a zone of healed sheared seams identified in rock outcrop upslope of the dam that may correlate with a similar oriented sheared zone at RL300 m upstream of the right abutment, identified as SZ5.

Thin sheared, crushed and shattered seams up to 20 mm wide are spaced at 5 to 10 m intervals in weathered outcrops and in the drillcore. A thin silt or sand crush seam is sometimes present and is generally less than 2 mm. Clay seams are rare.

4.4 Rock Weathering

The greywacke is unweathered in river exposures, but varies from SW to HW in surface exposures elsewhere on the abutment slopes. The depth of weathering varies around the site and in the drillholes. Chemical weathering, primarily due to oxidation above the groundwater table, has leached, altered and redeposited minerals (notably iron), reduced the intact rock strength and altered the colour from blueish grey through to brown. The change in intact rock strength between UW and MW is minor, but there is a significantly lower intact rock strength in HW rock.

In the SW rock, defects are often iron stained, and rarely contain silt, but are not noticeably weaker than UW defects. In MW rock, defect surfaces are discoloured and altered, are more open, and have regular infilling of cemented iron oxide and silt. Joint wall strength is lower than the UW to SW rock and joints are more closely spaced. This leads to higher permeability and lower rock mass strength in MW rock than SW or UW rock.

Associated with the weathering process is a progressive dilation of the rock mass from UW through to HW.

4.4.1 Rock Weathering on Left Abutment

The rock is highly weathered in portions of the Lee Valley Road to the west of the ridge crest and in the gully that will form the auxiliary spillway. In other track exposures (0 to 3 m BGL) the rock is typically moderately weathered to locally highly weathered near sheared zones, where bedding is closely spaced, and where finer (argillaceous) lithology is present. Weathering profiles in drillholes are as follows:

- *DH5:* MW rock extends to 10 m BGL and is SW to the base of the hole at 21 m depth.
- *DH6:* HW rock is present to 7 m depth, MW rock extends to 23 m and below this the rock is mainly SW to the base of the hole at 34 m.
- *DH7*: Rock head is at 10 m, the rock is MW to SW to 18 m, SW to 24 m and UW to the base of the hole at 32 m.

4.4.2 Rock Weathering on Right Abutment

HW rock is only locally present adjacent to sheared zones in the forestry track upslope of the abutment at RL300 m. At this elevation most of the 3 m deep exposure is MW but becomes HW upstream of Ch12,500 m. At lower level on the abutment slope the rock exposed in outcrop is mainly SW. Weathering profiles in drill holes are as follows:

DH8 : The rock is MW to 7 m, SW to 23 m, and UW to SW to the base of the hole at 38 m.

DH9: The rock is SW to 5 m and UW to the base of the hole at 15 m.

4.5 Rock Mass Permeability

Water levels were monitored during and after drilling, 32 Packer tests were carried out, and falling and rising head permeability tests were carried out in DH5, DH6 and DH7. Plots of Packer tests, and water levels during and after drilling are reported in Appendix

C. Standpipe piezometers were installed in each drill hole to enable ongoing water level monitoring.

Single Packer water pressure testing was carried out at on average at 3 m intervals, each test being over a 1.5 m zone. Maximum test pressures varied between 50 and 80% of overburden pressure. Results are reported using the criterion developed by Houlsby (1976) [Ref 6]. In packer testing the drill hole is sealed at various intervals and water is injected under pressure. The water loss under various pressures is measured and from this a Lugeon coefficient is determined (one Lugeon being one litre loss per metre per minute at 10 bars pressure). The Lugeon coefficient is used to determine grouting requirements. 1 Lugeon approximates a permeability of 10^{-7} m/sec.

Twenty-two tests (60%) gave a Lugeon (Lu) value of 3 or less. Eight tests (27%) were in the range Lu 4-10 and four tests (13%), all in DH6, were in excess of Lu 10.

Test results have been reviewed for each hole and the following trends have been identified.

- Six tests (all DH6) were carried out in MW dilated rock, average Lu=12,
- Eleven tests were carried out in MW (not dilated) to SW rock, average Lu=2,
- Fourteen tests were carried out in UW-SW rock, average Lu=5.

Further review of rock core where high test results were encountered in the UW-SW rock revealed:

- In DH6, Lu 12.3 was linked to steeply dipping closely spaced joints,
- In DH7, Lu 5 (23.5-25 m) was linked to a steeply dipping sheared zone,
- In DH8, Lu 6 (24.5-26 m) was linked to open partly quartz veined joint, Lu 11 (27.5-29 m) was linked to steep joints above a bedding plane sheared zone, and Lu 9 (30.5-32 m) was linked to a bedding plane sheared zone,
- In DH9, Lu 5 (4.5-6 m) was linked to a sheared zone, dip not clear.

On the basis of the packer testing it is concluded that the SW to UW rock has a permeability in the range of $1\text{x}10^{-7}$ to $1\text{x}10^{-6}$, with higher permeabilities being associated with steep joints and sheared zones. As steep joints and sheared zones are present throughout the foundation area it would be advisable for planning purposes to assume that the higher permeability rock will be encountered.

Laminar flow predominated in the testing. Turbulent flow is more commonly associated with open fissures while laminar flow is common with many small fissures. Observations of joints and bedding planes in core suggest that defect apertures are generally less than 0.2 mm.

Permeability determined from falling head and rising head tests in DH5, DH6, and DH8 are given in Table 2 as follows:

Table 2 – Permeability tests

Hole/test	**Test type**	**Permeability (ms^{-1})**
DH5	Falling head	$1.3\text{x}10^{-8}$
DH5(2)	Falling head	$1.0\text{x}10^{-8}$

DH6	Rising head	1.0×10^{-7}
DH6	Falling head	2×10^{-8}
DH8	Falling Head	3×10^{-8}

These indicate rock mass permeability to be one order of magnitude less than that inferred from Lugeon values. An order of magnitude difference in results from differing techniques is not uncommon and it is recognised that the packer test is carried out at pressures approaching those that will be created from the reservoir and the other tests are at lower pressures.

Further investigations of the groundwater regime before and during the design phase should include:

- Monitoring water levels and flows in ephemeral streams around the dam, including observations of seepage and groundwater emergence after rainfall. This should be carried out for a full cycle of seasons prior to design.
- Further mapping, drilling and test pitting to delineate potential permeable zones or aquitards beneath the dam in the abutments.
- Further lugeon water pressure tests and rising and falling head tests in standpipe piezometers to assess foundation and defect permeability.
- Packer testing should be carried out in inclined holes to intercept the steeply dipping joints such as those linked to the higher Lu values in DH8.

4.6 Rock Mass Classification

4.6.1 Rock Mass Rating

Geotechnicial characteristics of the drill core have been classified in accordance with the RMR system of Bieniawski (1989) [Ref. 7] and are reported in Table G1 in Appendix G. Rock at this site is classed as either Poor – Class IV, or Fair - Class III quality under the RMR system.

4.6.2 Rock Mass Strength and Rock Mass Deformation Modulus

RocLab software program has been used for determining rock mass strength parameters, based on the generalized Hoek-Brown failure criterion. Rock Mass deformability modulus value (Ei) is estimated using a modulus reduction value; (Erm) is determined using the modified Hoek and Diederichs (2006) [Ref. 8] equation. These are summarised in Table G2 in Appendix G.

The rock mass deformation modulus, Erm, has been calculated for three levels of rock mass disturbance. Erm D=0 refers to the modulus below the zone of rock disturbed by excavation. Erm D=0.2 is the modulus of rock disturbed by careful dam foundation preparation (as would be carried out for the plinth). Erm D=0.7 is the modulus of rock that is carefully blasted or ripped as may be carried out for general foundation preparation. The depth to which the lower modulus applies will vary from 1-3 m for carefully prepared rock in the valley floor to in excess of 5 m on the moderately steep abutments where some gravity relaxation may accompany ripping and/or careful blasting.

4.6.3 Lee Dam Rock Classes

Three general classes of rock that are applicable to the potential dam site have been determined, based on the field observations. These classes, outlined in Table 3, are based on the RMR system, GSI and Rock Mass modulus. The locations of rock in these classes are shown on the cross sections.

Table 3 - Rock Class classification used for Lee Dam

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**
Lugeons	Range	1. - 10	1. - 10	1 ->40
	Average	3	3	10
Permeability (ms^{-1})		10^{-7}	10^{-7}	10^{-5}
Erm (undisturbed) (D=0)	**Range**	**3000 - 7000**	**1000 - 3000**	**100 - 1000**
	Average	**4000**	**2000**	**500**
c (KPa)*	Range	400-700	300-400	200 - 300
	Recommended	500	300	200
Φ'*	Range	48 -53	42 - 48	35 - 42
	Recommended	50	45	37
pd		2.7	2.7	2.7

*Relevant to slope heights in the order of 50 m. Defect strength likely to have more influence on stability for lower height slopes.

Rock mass rating and modulai for each drill hole is included in Appendix F.

Contours of the top of Class 1 and top of Class 3 rock are shown in Figure 24727.204-F9.

4.6.4 Defect Strength

No testing has been carried out to assess the defect strength. However, it is noted that the greywacke at the site is dominated by a siliceous (quartz) mineral assemblage. Low strength and/or plastic clays have not been identified in either crushed or sheared zones, or as a weathering product. Existing cut batters in MW to HW rock are generally stable at 45° slopes.

Further work will be required to assess strength of sheared zones and joints in areas where they need to be considered in design. However, for preliminary assessment, based on field observations, the following base fiction angles are proposed:

Joints and Bedding planes: UW-SW – Φ'=36°

Joints and Bedding planes: MW-HW - $\Phi'=33°$

Sheared Zones: UW-MW - $\Phi'=26°$

Asperities and waviness will increase the base friction angle by between 5° and 10°.

4.7 Soil Types

4.7.1 Alluvial Gravel

The distribution of alluvial gravel at the potential dam site is shown on Figure 24727.204-F3. Two low level terrace deposits exist on the right bank of the river. These contain gravelly SAND, and sandy coarse GRAVEL that is inferred to be 2 to 5 m thick. On the left abutment an elevated terrace gravel deposit overlies a rock bench at RL170 m. These gravels are slightly to moderately weathered. Generally 1 to 2 m of unweathered sand and gravel overlies rock in the active river bed. Locally, along the inferred location of SZ11, more than 3 m of gravel infills a 5 m wide eroded slot in the river bed. All gravels are unconsolidated (loose to medium dense). No permeability tests have been carried out.

4.7.2 Slope Deposits

On the left abutment, up to 5 m of gravelly SAND (solifluction) overlies the gravel deposit and rock bench at RL170 m. Soil elsewhere on the left abutment consists of gravelly SAND colluviums that is generally less than 1 m thick.

Soil on the right abutment consists of unweathered, poorly graded GRAVEL, within scree deposits that blanket steeper slopes (>35°) down slope of rock outcrops and infill shallow steeply plunging gullies to 4 m depth. Elsewhere the slope is mantled by generally less than 1 m of gravelly SAND colluviums.

5 Geotechnical Issues

5.1 Dam Foundation Stability

The rock mass strength is considered to be generally lower than other South Island Greywackes. The broad scaller of defects is consistent with other Greywacke sites reported by read and Richards (2008) [Ref. 9].

The principal defects identified as underlying the potential foundation of the dam at dam site Ch12,430 m are shown on Cross section I, Figure 24727.204-F7. These are bedding plane sheared zones and sheared zones aligned with Joint Set A. Principal bedding plane sheared zones (notably SZ3 and SZ12) strike parallel to the dam crest and dip downstream at between 35 and 50°. This orientation is considered favourable to very favourable for dam foundations by Bieniawski & Orr 1976 [Ref. 10] and Romana, (2003) [Ref. 11].

No significant gently inclined dipping weak sheared zones, that dip either upstream or downstream, have been identified, however other defect sets may locally affect stability.

Sheared zones SZ1, SZ11 and SZ10 strike at 90° to the dam axis and dip at 50° towards the right abutment. Joint Set B dips steeply towards the left abutment. Both defect sets are important with respect to abutment stability (see Section 5.1.1) and can form release surfaces for foundation movement along low angle sheared zones if any exist. One joint, located in river exposure at Ch12,440 m dips downstream at 25 to 30°, and SZ6, located high on the left abutment has an oblique dip downstream of 20 to 25°.

5.1.1 Abutment Stability

Left Abutment

There is evidence of localised existing shallow instability on a portion of a steep soil-blanketed slope above the rock bluffs at Ch12,400 m and it is likely that the bluff has formed as a result of planar failure along joints of Joint Set A. These persistent joints and inferred sheared zones dip to the north at 45 to 55°. The bluff itself is considered to be currently stable and the steep groundwater gradient within the slope suggests that there is no significant slope relaxation along these joints.

Right Abutment

There is no evidence of large scale instability within the rock that would form the right abutment. The slope angle is generally between 35° and 42°. Some incipient rock relaxation is suggested by the weathering profile and slightly depressed groundwater gradient, and small (mm) offsets of joints are evident in outcrops. However, bedding plane dip and the orientation of other principal sheared zones is favourable with respect to stability.

Immediately upstream of the abutment, (upstream of Ch12,500 m) where the slope is inclined to the west, the combination of north-west dipping bedding planes and steep north-east east dipping joints (Joint Sets B and D) provide kinematic orientation for localised wedge failures.

The plinth excavation is likely to intersect at least two shallow gullies that are infilled with up to 4 m of scree deposits overlying localised dilated rock. The deposits will be unstable at slopes steeper than 40°.

5.1.2 Spillway Stability

The main spillway excavation is provisionally aligned in a northerly direction and the auxiliary spillway excavation will be aligned in a north-west direction. Cut heights will be in the order of 40 m high.

Rock quality and the major defect alignment is shown in Figure 24727.204-F9. A large portion of the excavation would be in dilated rock (Class 3) that in DH6 is logged down to 23 m BGL. The base of the Class 3 rock is inferred to follow the trace of SZ3. Structure contours on the trace of SZ3 are shown in Figure 24727.204-F9. In DH5 the transition from Class 3 to Class 2 or better rock is at 5 m depth.

The orientation of SZ3, and also SZ6, which are mapped to be exposed in the spillway excavation are favourable with respect to planar sliding stability. However, dental treatment including mesh, bolting and/or shotcrete will be required to prevent erosion of the sheared zone and undermining of overlying rock. Batter design will also need to reflect the weaker rock overlying SZ3.

There is a risk of wedge or planar sliding of sheared zones parallel to Joint Set A (SZ10 and SZ11). To mitigate potential batter instability batter design should be based on 40° slopes in Class 3 rock and 45° slopes in Class 2 or 1 rock. Steeper slopes would require systematic rock support in the more closely jointed sections of the excavation to mitigate wedge slides. Further joint mapping will be required for the detailed design.

The floor of the spillway is expected to comprise Class 2 rock upslope/upstream of Ch12,400 m and Class 3 downsteam. Local subexcavation and replacement with dental concrete and/or pinning with dowels may be required beneath the concrete liner in the Class 3 foundation section. The floor of the auxiliary spillway will extend in a down-stream direction through Class 2 and Class 3 rock and into soils at the western end. The Class 3 rock and soils will scour when flooded.

5.2 Reservoir Stability

5.2.1 Existing Stability

The preliminary review of the existing stability of slopes upstream of the potential dam site has identified no active large landslides that would extend into the reservoir area. Small areas of active erosion are noted in the heads of many gullies, and locally, small volume rockfall is evident downslope of rock bluffs. Existing landslides in the Waterfall Creek catchment are remote from the reservoir and the one large landslide deposit mapped on the right bank upstream of Waterfall Creek is largely eroded and now blanketed by younger slope deposits.

A large landslide in Rai Formation greywacke downstream of the dam site has developed where bedrock defects are unfavourably oriented (notably bedding strikes parallel to the slope) and where river erosion has formed a high (150 m) slope that is inclined at 45°-50˚. No similar slope features are evident within the reservoir and, in general, slopes underlain by Rai Formation greywacke lie at between 35 and 42°.

Solifluction deposits that are well exposed in road batters upstream of the dam blanket the lower portion of slopes. There is historical evidence of local shallow landslips when the slopes have been deforested, but no evidence of deep seated instability. These deposits (which are probably in excess of 10,000 years old) are not overlain by rockfall or bedrock landslide debris.

5.2.2 Reservoir Induced Instability

It is anticipated that the inundation of the valley to form the reservoir will raise groundwater levels by up to 45 m around the perimeter of the impoundment and this will have a local destabilising effect on slopes. During operation, reservoir levels are likely to fluctuate by up to 25 m over several weeks.

The slopes that will be most affected are those that are blanketed by thick soil deposits and those where local instability is already evident. Solifluction deposits, particularly those upstream of the dam on the left bank are likely to experience surface erosion and shallow instability within the zone of drawdown and extending upslope of the maximum operating level. This may disrupt the forestry access road into Flat Creek. Elsewhere soil deposits will be locally eroded by wave action within the normal operating zone but it is unlikely that landslips will extend significantly above top water level.

5.2.3 Earthquake Induced Landslides

The valley slopes will, from time to time, experience ground shaking associated with seismic events that is of a similar magnitude to that experienced in the past. The absence of landslide debris overlying solifluction or terrace deposits in the area to be inundated suggests that these slopes have not failed due to large scale instability during earthquakes during the last 10,000 years. However, as a result of the presence of the reservoir, groundwater levels will be higher than in the recent geological past and this may increase the risk of slope failure during shaking. Preliminary stability modelling of the western, left slope between Ch13,000 m and 13,500 m suggests that discrete downslope movements are only likely during MCE events. Rapid large scale collapse of slopes into the reservoir is not considered to be a likely failure scenario.

Further detailed engineering geological mapping of the full reservoir should be undertaken during the detailed design phase, and attention should be given to stability modelling of those slopes with elevated risk of slope failure in order to quantify the volumes of landslide debris that could be generated. If any areas are identified that may present a significant engineering risk, mitigation measures such as buttressing or drainage could be carried out during the construction phase.

5.3 Fault Rupture and Seismic Shaking

No active faults have been identified in the immediate vicinity of the potential dam site and the risk of direct fault rupture affecting the dam structure is considered to be extremely low.

Two active faults have been mapped in the region: the Wairau Segment of the Alpine Fault and the Waimea Fault.

The Alpine Fault, 21 km south-east of the site, has an assessed recurrence interval of 1,600 years and a major earthquake on this fault is estimated to be an M7.6 event. The Waimea Fault, which is part of the Waimea–Flaxmore fault system, is located 8.5 km west of the site and is assessed to have a recurrence interval of 6,000 years. A major earthquake on this fault is provisionally estimated to be an M7.6 event.

Unlike the Waimea-Flaxmore Fault System, the Alpine Fault will likely rupture along its entire length in the Wairau Valley, and possibly even further afield, and the level of ground shaking at the dam site will be MM VIII, bordering on MM IX. The level of ground shaking from rupture on a component of the Waimea-Flaxmore Fault System will produce similar levels of ground shaking at the potential dam site to those from the Alpine Fault in the Wairau Valley.

No evidence of Quaternary movement has been identified in previous mapping of faults, such as the Whangamoa Fault (3 km west – although an active trace has been identified 40 km further north), Wards Pass Fault (2.5 km south) and Totara Saddle Fault (4 km north) and they are not classed as active. However, this does not mean that the potential for future fault rupture on these faults can be ruled out.

A 50 mm offset of the hanging wall of the Anslow Fault has been observed in Anslow Creek, small (<20 mm) offsets have been observed in sheared seams high above the location of the right abutment of the potential dam and locally, in surface outcrops, there are occasional offsets (<10 mm) across joint surfaces. These features are attributed to secondary effects of ground shaking arising from fault movement remote from the site. Whether these are old tectonic features or are due to secondary effects of recent (geologically) faulting is not clear.

The New Zealand code AS/NZS/1170 [Ref. 12] provides a standard procedure for estimating seismic ground response based on ground conditions and design criteria. Part 5 of the code specifically excludes dams for the application of the method set out in the code. However, the basis of the code is compatible with the common methods of assessing earthquake hazards for dams and the Building (Dam Safety) Regulations 2008 that support the Building Act 2004 indicate that the AS/NZS/1170 method represents the minimum requirement for dams.

AS/NZ 1170 requires consideration of near fault factors for faults located within 20 km of the site under consideration. However, the Waimea Fault is not listed as a major fault in the standard and the Wairau Segment of the Alpine Fault is 21 km from the site.

In the absence of more detailed studies, the values derived from the NZS/1170 method for the Operating Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE) are appropriate for this study as follows;

- OBE - 1 in 150 year return period; and
- MDE - 1 in 10,000 year return period.

The site is classified as a Class B-Rock shallow soil site. With further assessment and strength testing of the foundation rock this is likely to be upgraded to Class A-Rock.

Table 5 - Peak ground accelerations for design seismic events

Criterion	**Value**
Hazard factor (NZ1170.5)	0.34
Dams on rock foundations – concrete gravity dams	**Peak Ground Acceleration**
Operating Basis Earthquake (OBE)	0.20 g

Maximum Design Earthquake (MDE)	0.70 g

Note: Initial estimates of design earthquakes (NZ1170.5)

5.4 Liquefaction Potential

Up to 10 m of unconsolidated sand and gravel are mapped on the potential left abutment area beneath the upstream shoulder of the dam, and locally elsewhere alluvial gravel underlies the dam footprint.

It has been assumed that these materials will be largely removed as part of the foundation preparation (at least beneath the upstream shoulder), and the dam will be founded on rock. No specific assessment of liquefaction potential has therefore been carried out.

5.5 Leakage Potential

The Lugeon permeability testing results indicate that permeability in Class 1 and Class 2 rock is generally within the range of 1-5 Lu. However, defects will provide higher permeable pathways through the near surface rock that will require grouting or near surface foundation treatment.

Class 3 rock downstream of the left abutment has high permeability above SZ3 (Lu 1-40), and there is a potential for significant leakage where Class 3 rock underlies the upper left abutment.

Bedding parallel sheared zones may have moderate permeability parallel to the shears (Lu 5-10) but rock mass permeability through the rock between sheared zones is very low (<1Lu). SZ 8 will be intersected by the plinth low on both abutments and the surface trace will extend below the upstream shoulder. Other bedding parallel defects will also be intersected by the plinth at higher levels. Bedding is inclined downstream at 35˚ to 70˚ and is not likely to be a potential source of leakage around the abutment.

Joint Sets A and B will provide preferential leakage paths both under the dam and around the abutments as they strike parallel to the valley sides. There is a possibility that there has been some stress relief in the right abutment leading to the opening of joints producing locally moderate permeability (Lu 5-12) and there is a risk of individual seepage paths through the foundations and abutments associated with these joints. If water losses can be tolerated then the need for a grout curtain is reduced. However consequences of leakage may include piping and erosion of fines in sheared zones, and elevated pore water pressures in the slope downstream of the abutment.

Houlsby (1992) [Ref. 13] recommends that where some water losses are acceptable, single row grouting should be carried out where testing indicates Lu 5-10. Where piping needs to be prevented, grouting should be considered where Lugeons are 3 or more.

Until further Lugeon testing is carried out foundation permeability should be viewed conservatively. Test results indicate that apart from the right abutment, it is likely that defect apertures are less than 0.2 mm which is the lower level where cement grouting is often effective. Furthermore, Weaver and Bruce (2007) [Ref 13] cited data from a number of projects where small Lugeon values (Lu <2-5) usually indicated ungroutable rock. Therefore, at the proposed dam site while provision should be made for foundation grouting, the effectiveness of a grouting programme will need to be evaluated along with other options to reduce hydraulic gradients and reduce the risk of piping of clay and silt in sheared zones. Other options include: dental concrete treatment, widening the plinth, and blanketing the upstream shoulder. Solifluction deposits, located in close proximity to the dam will provide cohesionless, low permeability fine soils that can both reduce hydraulic gradients and assist in sealing seepage paths.

The depth and extent of the curtain grouting will have to be determined by additional geotechnical investigations during the detailed design stage of the dam.

Due to the anisotropic strength of the foundation rock, and often strongly developed fissility, care will need to be taken during any grout programme to control pressures to avoid excessive hydraulic fracturing.

5.6 Foundation Preparation

The investigations have indicated that site conditions are suitable for a concrete faced rockfill dam. The rock mass quality is generally fair and foundation permeability is within acceptable limits.

There is a degree of variability in rock mass quality and provision will need to be made for specific foundation treatment along the line of the plinth and beneath the upstream shoulder, particularly where areas of crushed, shattered and sheared rock are present. This will present construction challenges on the right abutment where the existing slopes are moderately steep to steep.

The major defects include sheared zones that contain clay and silt. Dispersion/piping characteristics of the clay and silt in these sheared seams, and the risk of joint silt infill erosion will need to be verified prior to detailed design of specific treatment works.

5.6.1 General Site Preparation

Preliminary site preparation should involve removal of unconsolidated deposits from the dam footprint. Contours of rock head are shown on Figure 24727.204- F7. Beneath the upstream shoulder on the left abutment up to 12 m of solifluction and alluvium will need to be removed. Temporary batters upstream of the dam for this area will need to be battered at no steeper than 34° to permit safe working in the plinth excavation.

Elsewhere, stripping depths will vary from 0 to 5 m. Much of this work should be achievable using bulldozer/excavator and truck techniques. However, access for construction plant will be difficult to achieve above RL175 m on the right abutment. Provision may need to be made to form an access bench at about RL200 m to facilitate foundation preparation in the upper right abutment. Stripping of soils within the shallow gullies on the upper left abutment will produce construction batters in the soil. Temporary support, involving soil nail and mesh, or equivalent measures will need to be provided.

5.6.2 Plinth Foundation

Additional excavation of rock will be required along the plinth line. Careful excavation with excavators ripping and utilising rock breaking hammers will be adequate in Class 3 and much of the Class 1 and 2 rock. Localised blasting may be required to remove strong surface irregularities.

The depth of excavation and minimum requirements for rock quality and permeability are a function of the hydraulic head and design plinth width balanced against the cost of preparation. For a standard minimum plinth width of 3 m it is possible to achieve a satisfactory hydraulic gradient in the upper levels of the plinth (above RL180m) within Class 2 rock where the HW rock, that is typically less than 2 m thick, is removed.

Within the floor of the valley SW to UW Class 1 rock is at about RL149 m and rock may need to be sub-excavated to between 1 and about 5 m depth. Alternatively other measures to limit seepage should be considered. However, provision will be needed in the valley floor to excavate to at least 5 m depth along the location of SZ11which is inferred where rock head drops steeply over a 5 m wide zone in the river channel.

On the lower level of the left abutment (between RL155 m and 170 m rock) where the surface rock is SW-MW Class 2, excavation may need to extend to the Class 1 rock which is at a depth of 2 to 6 m below rock head.

The right abutment is steep and there are probably three scree-infilled shallow gullies where rock head is 3 to 5 m deep. The steep slope will make deep sub-excavation very difficult and excavation to Class 2 rock will produce a stepped profile between the shallow gully and spur lines. Abrupt vertical or overhanging slopes along the plinth line may need to be removed by drilling and blasting to trim to an acceptable shape and provide a foundation that can be grouted. Blast vibrations will need to be carefully controlled to avoid undue overbreak and dilation along defects.

To avoid very high and steep temporary cuts along gully lines that will require systematic bolting, provision should be made to limit the depth of excavation on spur lines and to backfill these hollows along gully lines with dental concrete on which to build the plinth. More extensive grouting and seepage control will be required along the gully lines.

Final preparation of rock along the plinth line will involve hand excavation and water blasting. Localised zones of poor quality rock that are associated with sheared zones will be intersected in the plinth foundation. These will require special treatment that will include:

- flushing fines with water and slush grouting,
- widening the plinth width,
- placing a filter layer over rock downstream of the plinth,
- extending dental concrete downstream of the plinth.

5.6.3 Diversion

The diversion location is shown on Figure 24727.204-F2. Upstream of the dam crest the diversion will be located predominantly within the active river channel, and downsteam of the centreline it will be located predominantly to the north of the right bank of the river. The invert of the diversion excavation will be at about RL149 m.

No significant geotechnical risks are identified. Top of rock will vary from locally lower than RL149 in the central river channel to RL161 beneath the downstream face of the dam. Crushed zones, associated with bedding plane shears SZ12 and SZ4, and a crush zone associated with SZ8 will strike oblique to the diversion line. Localised foundation treatment involving over-excavation of crushed and weak rock and backfilling with dental concrete will be required to reduce the risk of localised differential settlement.

Downstream of the dam centreline a temporary box cut, up to 12 m deep, on the north side and 5 m deep on the south side, will be formed. Joint Set A, parallel to SZ1 and SZ11 will locally induce overbreak on the south side if the cut is formed at steeper than 1H:1V.

Defects are favourably oriented for the northern wall of the box cut, and temporary slopes of .25H:1V can be planned, with provision for local mesh and bolting of relaxed rock, that may exist within the top 3 m of the cut.

A temporary diversion channel into the left bank will be required to facilitate diversion construction. All excavations will be below the ground water table and high inflows will need to be catered for, especially upstream of the dam centreline, during excavation and cleanup of foundation rock.

6 Construction Materials

6.1 Construction Requirements and Sources

The general requirements for geological construction materials for a CFR dam at site Ch12,430 m are set out in Table 6.

Table 6 – Construction material requirements

Type	Approx Volume	Desired features/properties	On-site Availability
Rockfill	400,000m³	Medium-high strength, compatibility, high permeability	Yes
Filter Aggregate	5000m³	0.1-<20mm, stability compatibility	Yes - processing required
Bedding Aggregate	30000-40000m³	<75mm crusher run	Yes - processing required
Concrete Aggregate	10000m³	UW sand and fine-medium gravel, rounded clasts preferred to angular	Yes - processing required
Drainage Aggregate	500-2000m³	Durable, very strong, compatible grading, higher k than drained rock	Yes - processing required
Rip Rap	9000-12000m³	>300 mm ->1 m, durable, UW, angular, non tabular/pla°tey	Not confirmed for >600mm
Blanket	5000-10000m³	Non-plastic stable fines, low permeability	Yes
Plastic fines	?	Plastic clay	No
General Fill	?	Non specific, well graded	Yes

Figure 24727.204-F10 shows the location of potential borrow material sources that are on-site (300 m downstream to 1 km upstream of the dam location) and offsite sources where suitable aggregate or fill material is currently available.

6.1.1 On-site Sources and Volumes

Spillway Excavation

The spillway excavation will yield a minimum of 350,000 m³ of rockfill that will consist of moderately weathered to unweathered greywacke. Overburden that is 1-4 m thick and consisting of soil and highly weathered rock would be suitable as a general fill.

Other potential sites exist within the proposed reservoir footprint, notably near Ch14,500 m where Star Formation greywacke is mapped. This rock is likely to produce a coarser rockfill than Rai Formation.

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 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) and this coarser size fraction forms an imbricated armour layer on the river bed.

Solifluction deposits

Four large soliflction 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.

6.1.2 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 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, 3.5 km from the site 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.

6.2 Properties of On-site Sourced Materials

Limited sampling and testing was carried out as part of Stage 2 of the study. Table 7 summarises the properties for ripped greywacke bedrock, alluvial gravel and solifluction deposits.

Table 7 – Material properties results

Sample /Source	Gravel %	Sand %	Silt %	Clay %	Fines %	D_{50}	D_{10}	D_{60}/D_{10}	PL	LL	PI
1. HW-MW Greywacke	74	26	12	4	16	16	.05	300	NP	35	NP
2. SW-MW Greywacke	82	14	<6	-	<6	16	.3	90	-	-	-
3. UW-SW Greywacke	97	3	-	-	-	50	8	9	-	-	-

4. Alluvium	72	16	5								
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

6.3 Rockfill

6.3.1 Site Quarried Rockfill

Preliminary grading tests on SW to MW rock and MW-HW rock were carried out on samples that that been ripped and lightly track rolled. The resulting aggregate is angular, typical of quarried rock but gradings are finer than is reported as normally used in bulk rockfill zones beneath the upstream or downsteam shoulders of CFR dams. Fell et al [Ref. 15] propose that for rockfill after compaction:

- not more than 15% below 1.18 mm;
- not more than 5% below 0.075 mm;
- maximum particle size to be no more than the compacted layer thickness;
- water not to pond on the compacted surface.

HW greywacke is unlikely to satisfy this criteria. However, SW to UW greywacke is likely to satisfy the criteria. The MW greywacke may be suitable, but further evaluation is required.

Small grainsize is not necessarily an adverse feature as it is commonly associated with higher rockfill strength. Durability of the rock (change in characteristics on wetting/drying cycles) does need to be further assessed, particularly the MW–HW Class 3 rock and more argillaceous lithology. Field observations suggest that exposure to air, frost, and wetting cycles may lead to a progressive strength loss and disintegration of aggregate. This will impact on settlement and permeability in areas where Class 3 rock and argillaceous rock is placed within the dam.

6.3.2 Alluvium

The alluvium is a finer deposit than the ripped greywacke rockfill and is dominated by sub rounded coarse gravel clasts and sub angular sand and fine gravel. It is a bedded deposit and locally includes large bodies of gravelly Sand interbedded with the sandy Gravel. No strength testing of aggregate has been carried out, but from visual assessment the clasts are very strong and are considerably stronger than equivalent sized ripped greywacke.

6.3.3 Rockfill Properties

Shear strength of rockfill derived from greywacke and alluvium has been assessed following the methodology of Barton and Kjaernsli (1981) [Ref. 16]. Rockfill modulus has been assessed using the methodology of Hunter and Fell (2003) [Ref. 17]. The results are summarised in Table 8.

Table 8 - Rockfill shear strength and modulai *

Feature	Rockfill Source			
Source material	**Class 1**	**Class 2**	**Class 3**	**Alluvium**
Density (t/m^3)	2.2	2.1	2.0	2.1
Porosity (%)	19	21	26	22
UCS (MPa)	50	35	20	100
D_{50} (mm) rolled	40	20	15	15
D_{80} (mm)	100	50	40	80
φ' (σ'n=0.2MPa)	49	47	39	53
φ' (σ'n=0.8MPa)	44	41	34	48
Erc (σ'n =0.2MPa)	126	110	99	>200
Erc (σ'n =0.8MPa)	90	75	63	>170
Erf (1.5:1)	119		84	227
Permeability	>10^{-3}	>10^{-3}	10^{-5}-10^{-4}	>10^{-3}

*Based on limited testing

Further testing will be required as part of the design stage to verify the grading and strength of the proposed rockfill. Ideally this will involve compaction trials, and laboratory and field testing of strength of the compacted aggregate. Further testing will also need to be carried out to establish the effects of weathering on aggregate durability.

6.3.4 Aggregate for Concrete

No specific testing has been carried out to assess concrete aggregate suitability. However, the alluvium is likely to be a suitable source. Alluvium particle shape and durability will be better than the quarried greywacke, but there may be insufficient fines in the alluvium. On-site screening and blending will be required to produce the necessary grading.

Particle shape will need to be reviewed as many fine gravel and sand clasts are platey in appearance. Existing gravel pits are located downstream on the same river system and there are no reported alkalai-aggregate reactions from those sites.

6.4 Filter Materials

Insufficient testing has been carried out to assess the range of gradings in the alluvium, but it is likely to produce aggregate that can be effectively screened to produce a range of filter sizes. With there being some doubt regarding the durability of quarried greywacke, it is not considered to be suitable for producing filter aggregate.

6.5 Rip Rap

The defect spacing of the greywacke restricts breakout size which will limit the suitability of much of the excavated rock won from the spillway cut for rip-rap. Alternative bedrock sources may exist upstream of the dam where Star Formation outcrops.

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.

High strength and durable (up to 1.2 m diameter) rip rap is available from Taylors Quarry.

6.6 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.

7 Conclusions and Recommendations

Feasibility level geotechnical investigations have been carried out for a dam at Ch 12,430 m on the Lee River.

Engineering geological mapping, test pitting, drilling and water pressure testing has been carried out at the proposed dam site. In addition the investigations have drawn on existing reports and earlier phase investigation results from dam site investigations 1,400 m downstream of the current preferred location. Engineering geological mapping, test pitting, drilling and water pressure testing has been carried out at the proposed dam site.

The investigations indicate that geological conditions at the proposed dam site are generally suitable for a Concrete Faced Rockfill Dam.

The key conclusions from this phase and recommendations for future work are outlined below.

Bedrock

- The dam foundations are predominantly greywacke, locally interlayered with argillite. Bedrock dips generally at 30-60° downstream beneath the dam footprint.
- Bedding is closely spaced and is the predominant defect.
- There is a broad scatter of joint orientations that are collectively grouped into four joint sets.
- Sheared zones have been mapped at 10 to 50 m spacing and form an orthogonal pattern beneath the dam. Bedding plane sheared zones are the most common.
- The rock is rated as generally fair quality.

Further investigations are required to assess the range of rock strength and defect orientations. This should include uniaxial compression testing of rock core and drilling to accurately locate principal sheared zones.

Permeability and Groundwater

- The rock mass permeability is assessed to vary between 1 x 10^{-7} to 1 x 10^{-6} m^{-1}.
- Further investigations of the groundwater regime before and during the design phase should include:

Monitoring water levels and flows in ephemeral streams around the dam, including observations of seepage and groundwater emergence after rainfall should be carried out for a full cycle of seasons prior to design.

Further mapping, drilling and test pitting should be carried out to delineate potential permeable zones or aquitards beneath the dam in the abutments.

Dam Foundation Stability

- The defect orientation beneath the dam is considered favourable with respect to stability.

Left Abutment Stability

- Deep seated rock relaxation is evident downstream of the dam.
- No bedrock instability is evident beneath the dam footprint.

Right Abutment Stability

- Existing slopes are locally steep and some incipient rock relaxation is evident.
- The orientation of major defects is favourable with respect to deep seated stability.
- Excavations for the plinth will daylight local wedge blocks and undercut scree infilled gullies. These deposits will be unstable when cut to slopes steeper than 40°.

Spillway Stability

- The orientation of Principal defects (notably SZ6 and SZ3) is favourable for the provisional spillway alignment.
- Sheared zones and joints aligned with (Joint Set A, e.g. SZ10) will limit the maximum batter angles in Class 1 and 2 rock to 45°.
- Poor quality (Class 3) rock in the upper section of batter will require batters no steeper than 40°, and in soil to be no steeper than 36°.

Further systematic mapping of defects should be carried out from test excavations and oriented drill core on both abutments and the spillway.

Defect strength testing should be carried out including testing clay/silt sheared seams and joint and bedding plane surfaces.

Reservoir

- No active large landslides have been identified in the reservoir.
- Groundwater levels will be raised by inundation and local instability associated with solifluction slopes can be expected.

Further detailed engineering geological mapping of the full reservoir should be undertaken during the detailed design phase, and attention should be given to stability modelling of those slopes with elevated risk of slope failure in order to quantify the volumes of landslide debris that could be generated. If any areas are identified that may present a significant engineering risk, mitigation measures such as buttressing or drainage should be carried out during the construction phase.

Fault Rupture and Seismic Shaking

- No active faults have been identified in the immediate vicinity of the dam site.
- The Alpine Fault is likely to pose the main seismic threat to the dam.

A site specific seismic assessment, that considers this potential scenario of an earthquake generated within the Waimea-Flaxmore Fault system as well as the Alpine Fault, should be carried out as part of the detailed design stage.

Leakage Potential

- Lugeon values of 1-5 Lu can generally be expected in Class 1 and Class 2 rock, but there is a potential for high leakage along rock defects.
- Provision will be required for grouting and or near surface foundation treatment.

Further attention should be given to potential seepage paths and the groutability of the rock mass. This should include continuous packer testing in additional abutment drillholes, including testing in inclined holes to intercept steeply dipping defects in the right abutment.

Dispersion and erosion testing should be carried out on shattered and sheared zone material to assess the potential for internal erosion of fines.

Foundation Preparation

- There is a degree of variability in rock mass quality requiring local subexcavation and or special treatment of poor quality rock associated with crushed, shattered, sheared or dilated rock.
- Special treatment will be required to avoid piping/erosion of fines within sheared zones.
- Soil stripping depths on the left abutment will be up to 12 m and on the right abutment will be up to about 5 m.
- Provision will be required for local stabilisation of temporary slopes on the right abutment.

Further delineation of soil depths and rock classes will be required to optimise the foundation stripping depths for construction.

Construction Materials

- It is likely that suitable sources of rockfill can be sourced from either the Spillway Cut or local alluvial deposits.
- Rockfill properties are likely to be strongly influenced by the degree of compaction.
- Poor quality rockfill will be produced from Class 3 rock within the spillway excavation. This may not be suitable for rockfill.
- Riprap >600 mm may need to be imported.
- Aggregates for concrete/filters and drainage are likely to be sourced from local alluvium. However the durability of fines may limit use for some filters.
- Local solifluction deposits can be used for non plastic fines applications.

Trial excavations and compaction trials on rockfill will be required for Class 1, 2 and 3 rock types and alluvium. Laboratory testing should be carried out on as-compacted soils to assess grading and permeability characteristics.

Specific attention should be given to testing durability of the fill and filter materials under cycles of shaking, freeze thaw and wetting.

8 Applicability

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

TONKIN & TAYLOR LTD

Environmental and Engineering Consultants

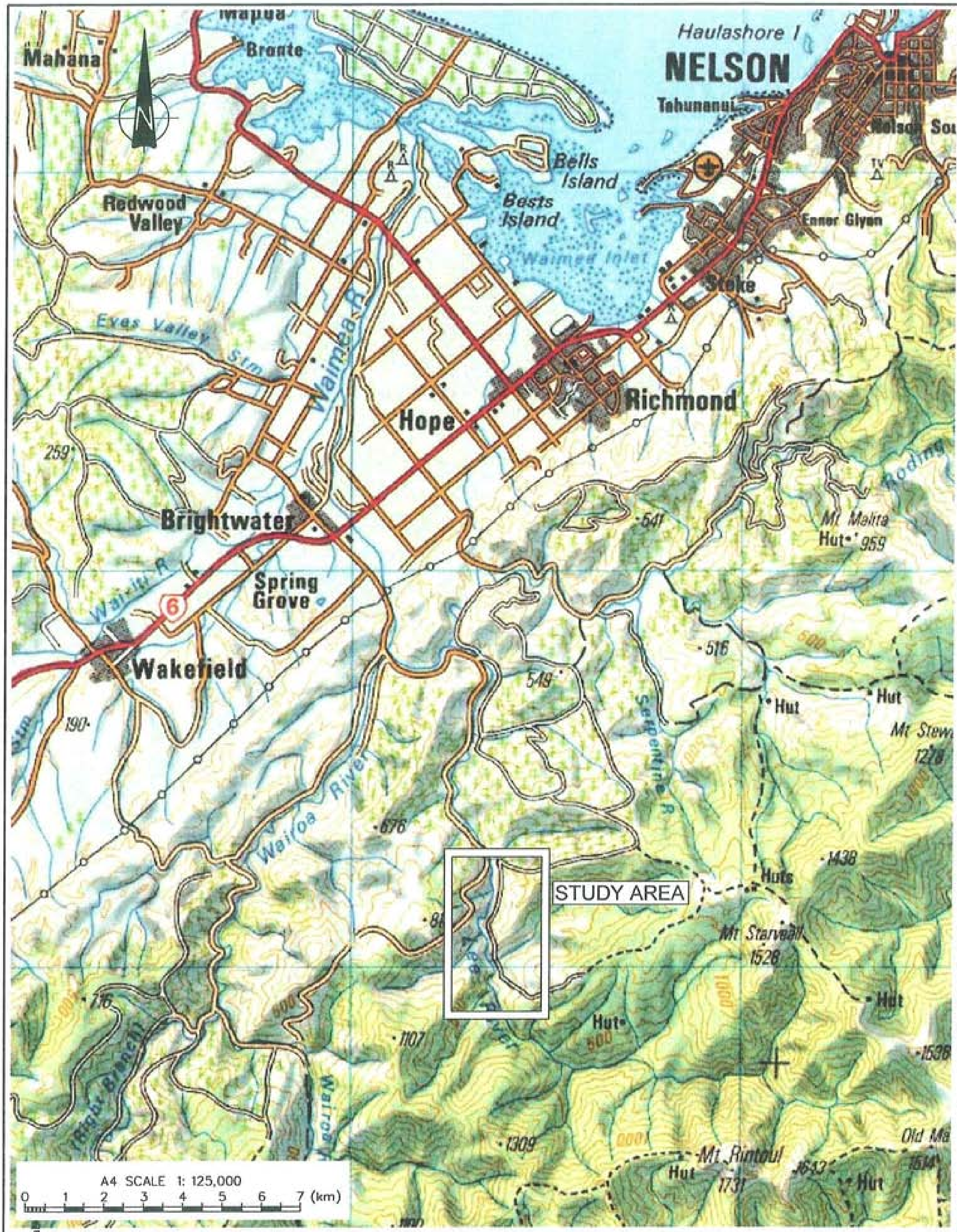
Report prepared by:	Authorised for Tonkin & Taylor by:
Mark Foley	Gary Smith
Senior Engineering Geologist	
Project Manager	Project Director

Reviewed by: Bernard Hegan

p:\24727\24727.204\workingmaterial\mf lee feasibility report\finaldraft revised for final issue.doc

Appendix A: Tonkin & Taylor Figures

- **Figure 24727.204-F1 - Study Area**
- **Figure 24727.204-F2 - Location and Layout of Dam and Reservoir**
- **Figure 24727.204 F3 - Regional Geology Summary**
- **Figure 24727.204-F4 - Geological Setting**
- **Figure 24727.204-F5 – Dan and Reservoir Geomorphology**
- **Figure 24727.204-F6 - Dam Site Geology**
- **Figure 24727.204-F7 - Cross Sections H-H & I-I**
- **Figure 24727.204-F8 - Cross Sections F-F & G-G**
- **Figure 24727.204-F9 - Rock Quality – Spillway and Plinth**
- **Figure 24727.204-F10 – Potential Borrow Site**



Tonkin & Taylor
Environmental & Engineering Consultants

☐ Auckland ☐ Hamilton ■ Nelson ☐ Wellington ☐ Christchurch ☐ Whangarei

DRAWN	EVL	Jul.09
DRAFTING CHECKED	✓	11/09
APPROVED		

CADFILE : \\24727.204-F 1.dwg

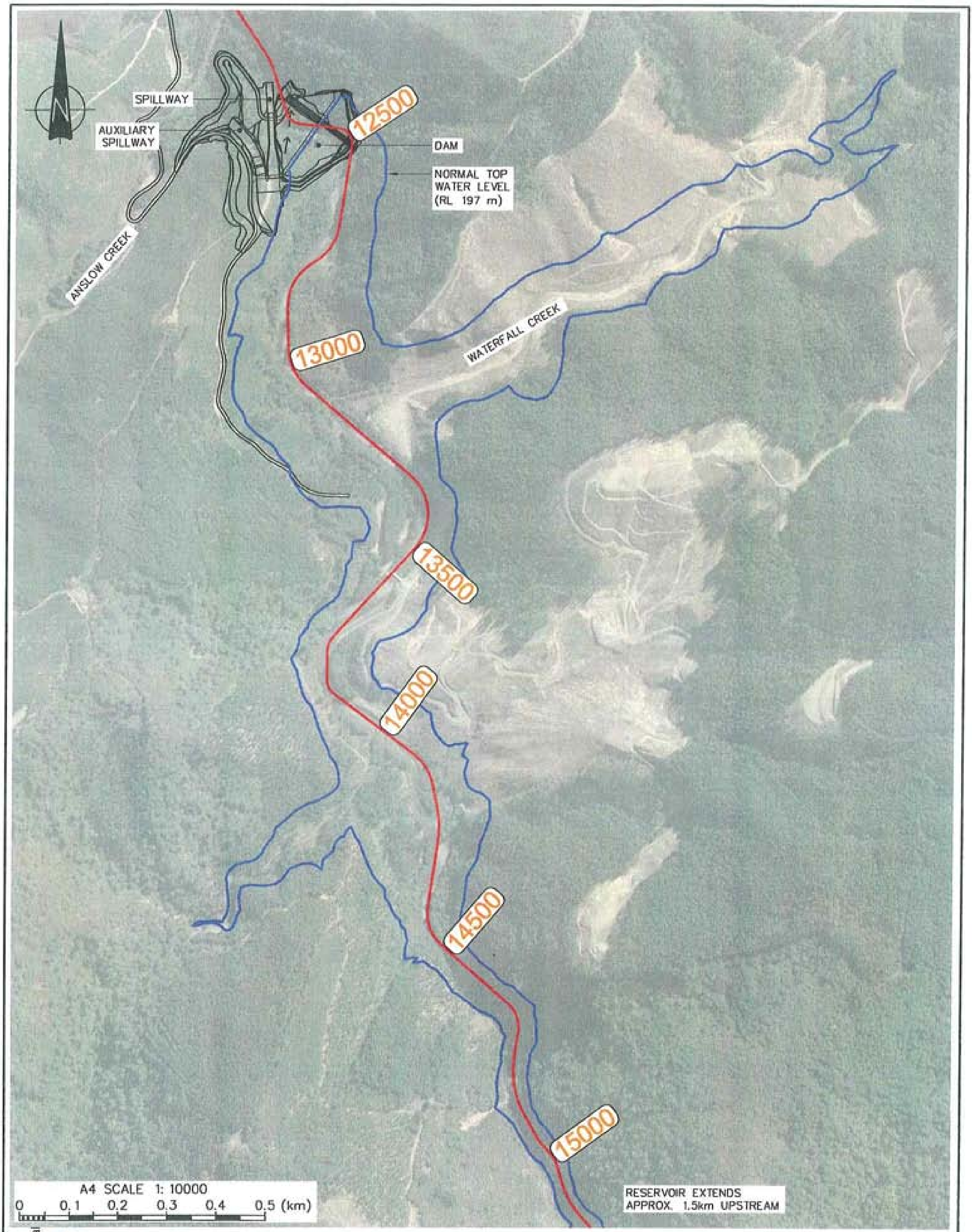
SCALES (AT A4 SIZE) 1: 125,000

PROJECT No. 24727.204

WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA WATER AUGMENTATION – PHASE 2
LEE RIVER
Study Area

FIG. No. 24727.204–F1

REV. 0

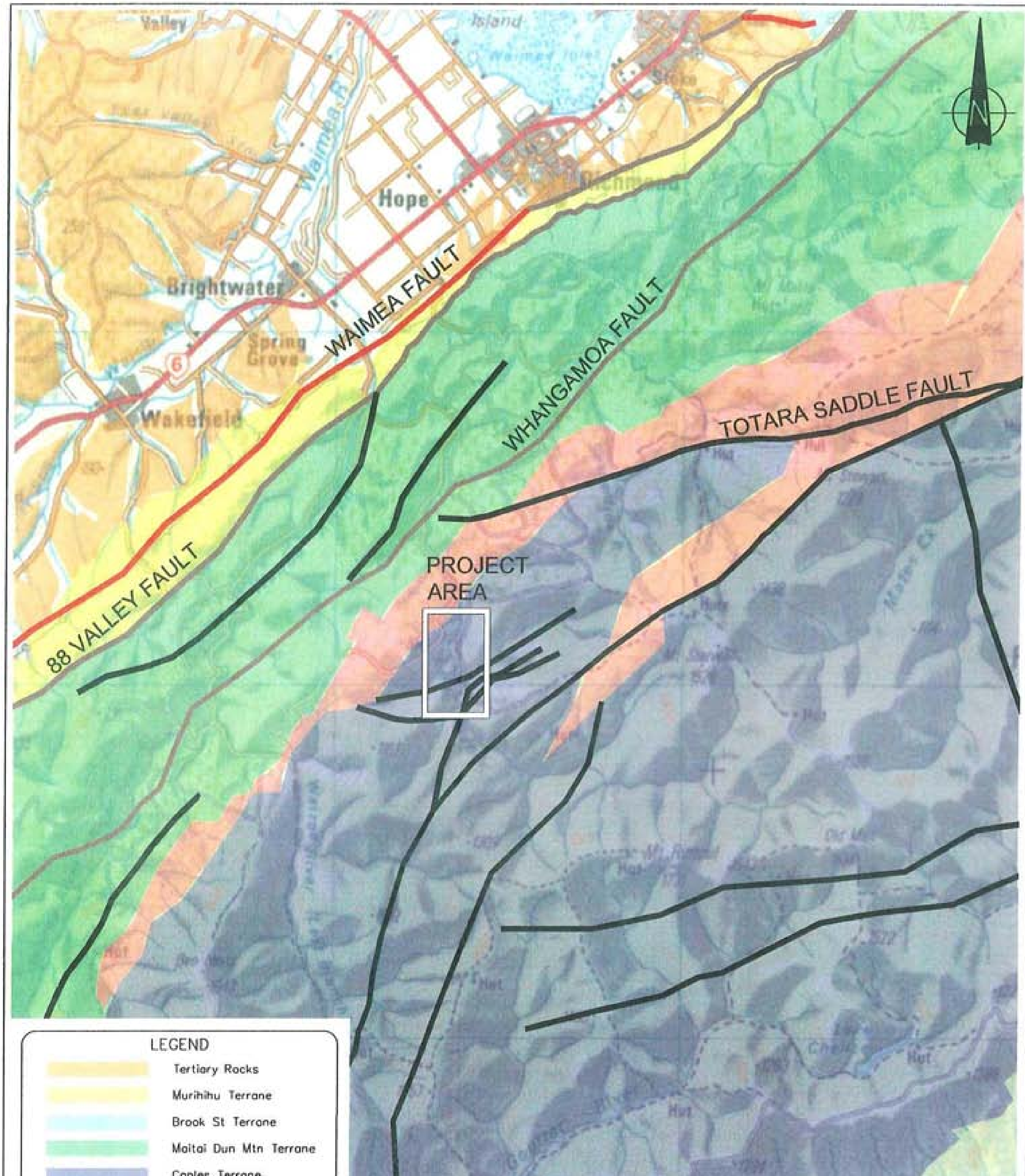


Tonkin & Taylor
Environmental & Engineering Consultants
☐ Auckland ☐ Hamilton ■ Nelson ☐ Wellington ☐ Christchurch ☐ Whangarei

DRAWN	EVL	Jul.09
DRAFTING CHECKED		11/09
APPROVED		
CADFILE : \\24727.204-F2.dwg		
SCALES (AT A4 SIZE) 1: 10,000		
PROJECT No. 24727.204		

WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA WATER AUGMENTATION – PHASE 2
LEE RIVER
Location and Layout of Dam and Reservoir

FIG. No. 24727.204-F2 REV. 0



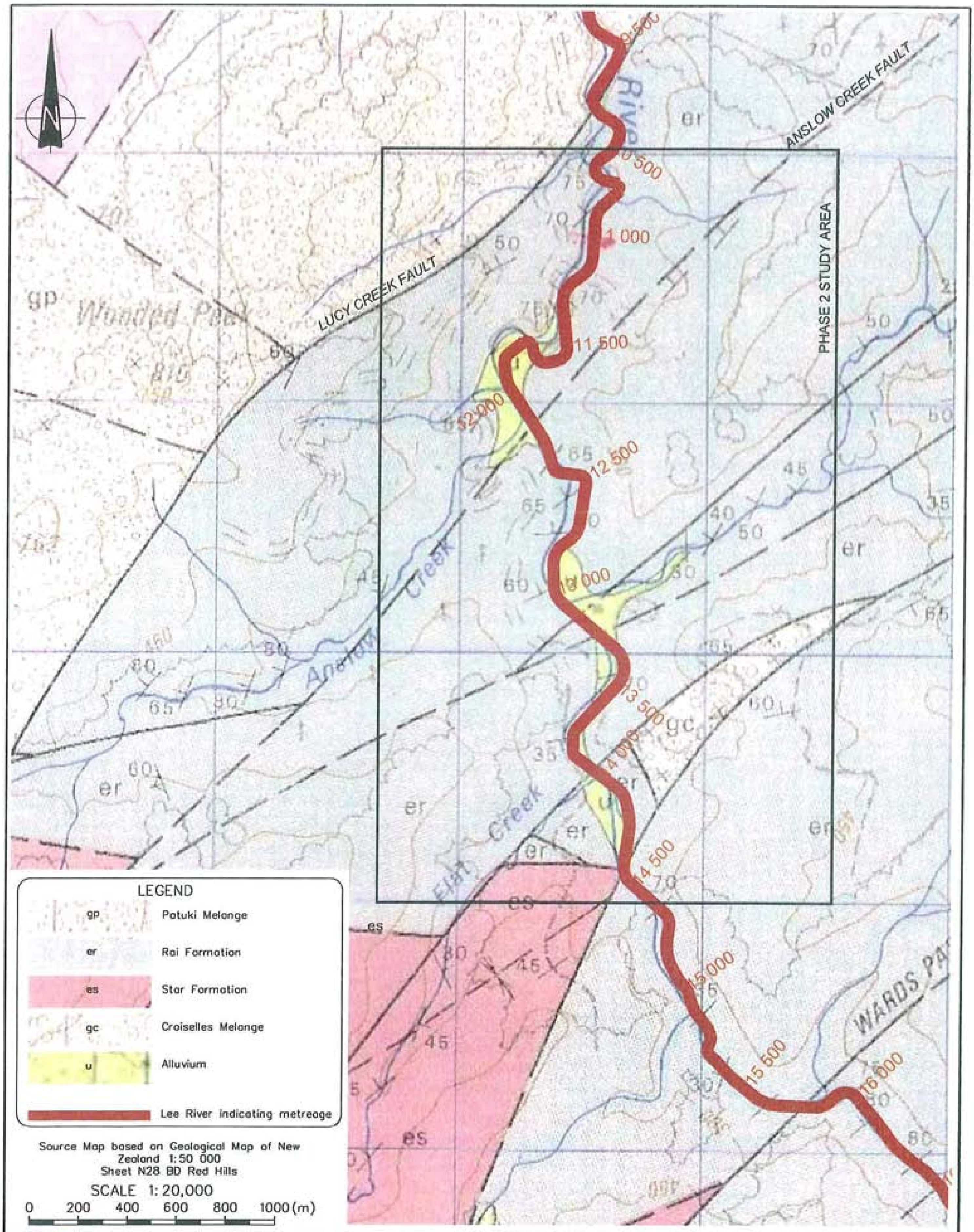
DRAWN	EVL	Jul.09
DRAFTING CHECKED		
APPROVED		
CADFILE: \\24727.204-F3.dwg		
SCALES (AT A4 SIZE) 1: 200,000		
PROJECT No. 24727.204		

Tonkin & Taylor
Environmental & Engineering Consultants
Auckland, Hamilton, Nelson, Wellington, Christchurch, Whangarei

WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA WATER AUGMENTATION – PHASE 2
LEE RIVER
Regional Geology Summary

FIG. No. 24727.204-F3

REV. 0



DRAWN	EVL	Jul.09
DRAFTING CHECKED	Y	11/09
APPROVED		

Tonkin & Taylor
Environmental & Engineering Consultants
☐Auckland ☐Hamilton ■Nelson ☐Wellington ☐Christchurch ☐Whangarei

CADFILE : \\24727.204-F4.dwg

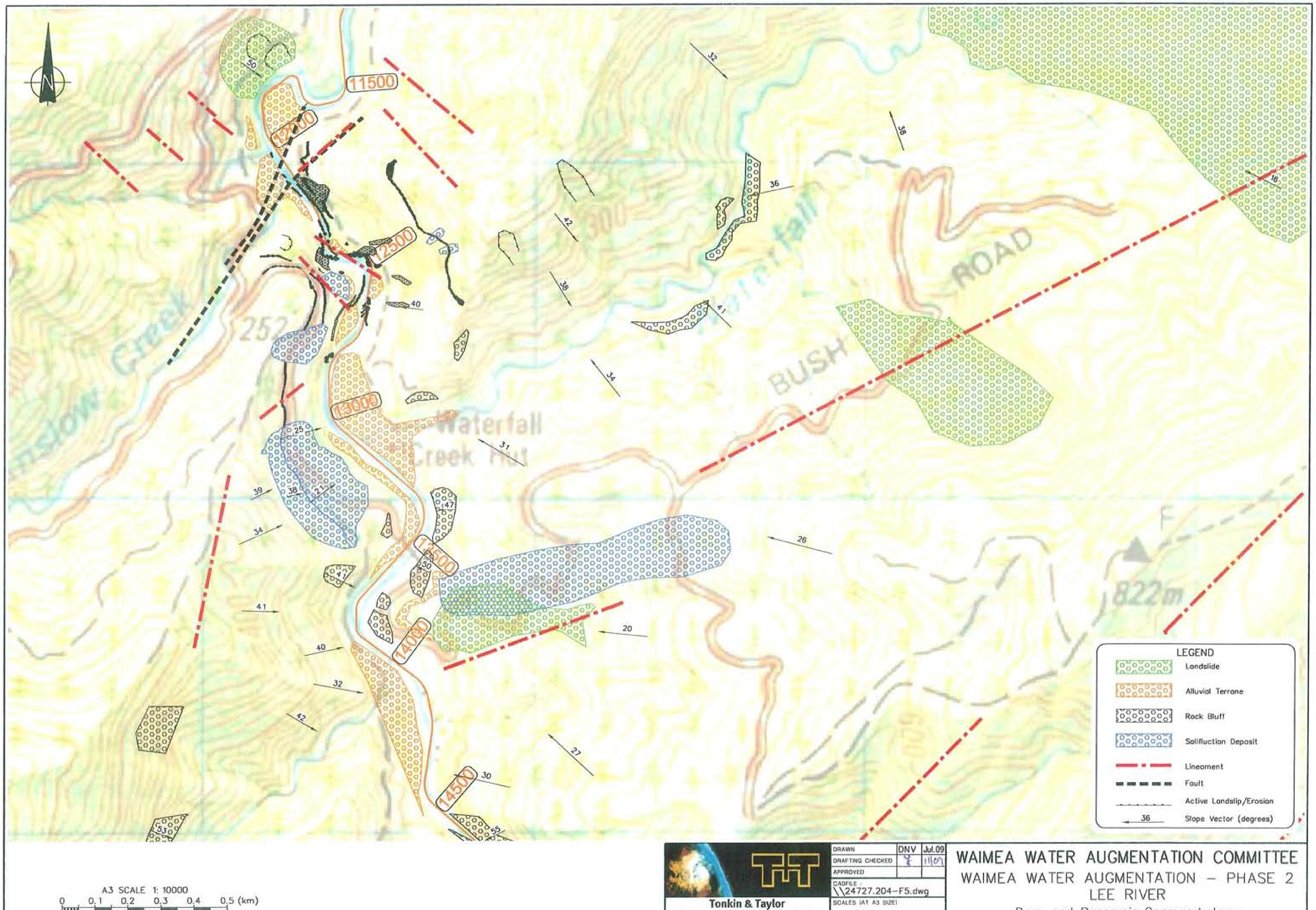
SCALES (AT A4 SIZE) 1: 20 000

PROJECT No. 24727.204

WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA WATER AUGMENTATION – PHASE 2
LEE RIVER
Geological Setting

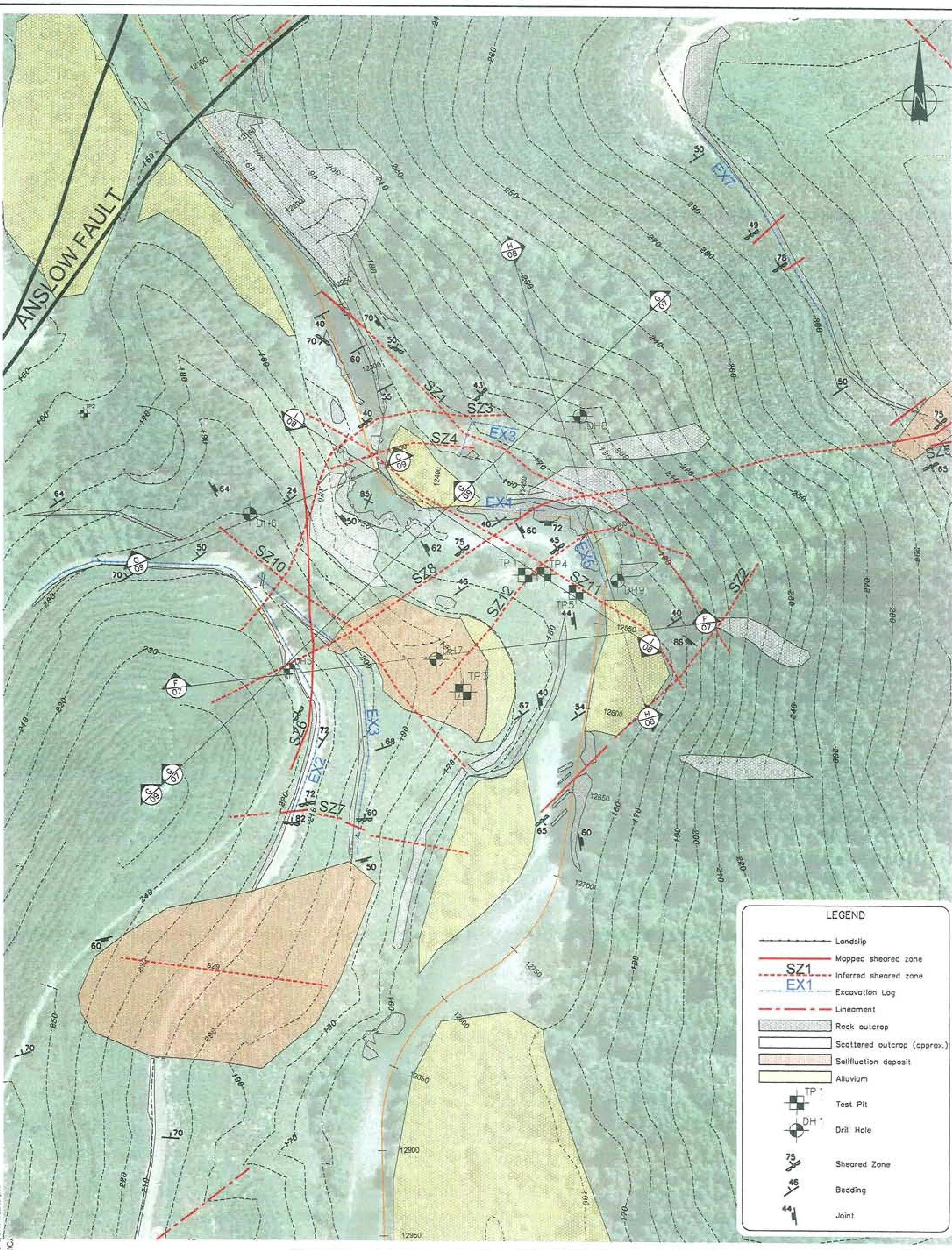
FIG. No. 24727.204–F4

REV. 0



P:\24727\WorkingMaterial\CAD\01 DWGS\204 GEOSTAGE4\24727.204-F5.dwg, F5, 27/11/2009 8:26:33 a.m., dnv, 1:1

Tonkin & Taylor Environmental & Engineering Consultants Auckland, Hamilton, Nelson, Wellington, Christchurch, Whangarei	DRAWN: DNV Jul.09 DRAFTING CHECKED APPROVED CADFILE: \\24727.204-F5.dwg SCALES (AT A3 SIZE): 1: 10,000 PROJECT No. 24727.204	WAIMEA WATER AUGMENTATION COMMITTEE WAIMEA WATER AUGMENTATION – PHASE 2 LEE RIVER Dam and Reservoir Geomorphology FIG. No. 24727.204–F5 REV. 0



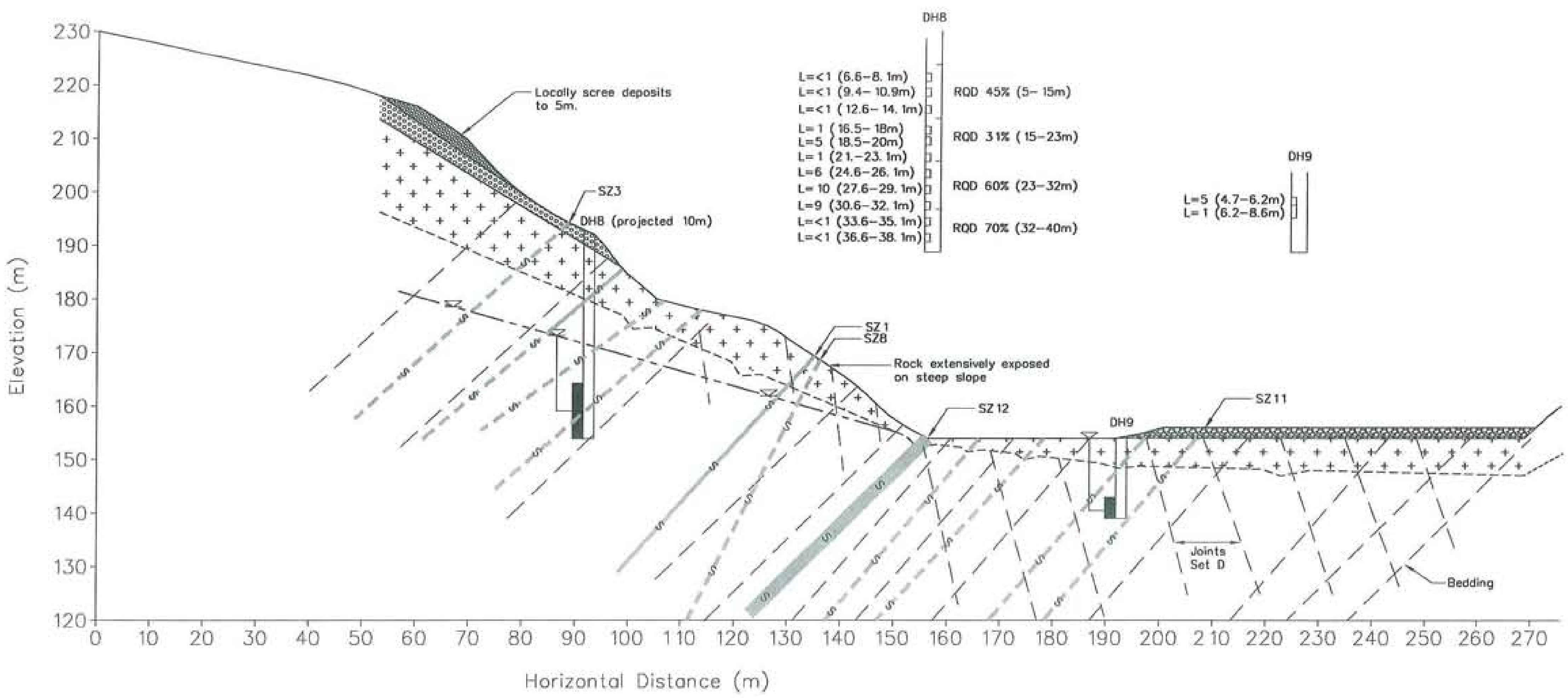
DRAWN	DNV	Sep.09
DRAFTING CHECKED		11/09
APPROVED		
CADFILE: 24727.204-F6.dwg		
SCALES (AT A3 SIZE) 1:2000		
PROJECT No. 24727.204		

Tonkin & Taylor
Environmental & Engineering Consultants
□Auckland □Hamilton ■Nelson □Wellington □Christchurch □Whangarei

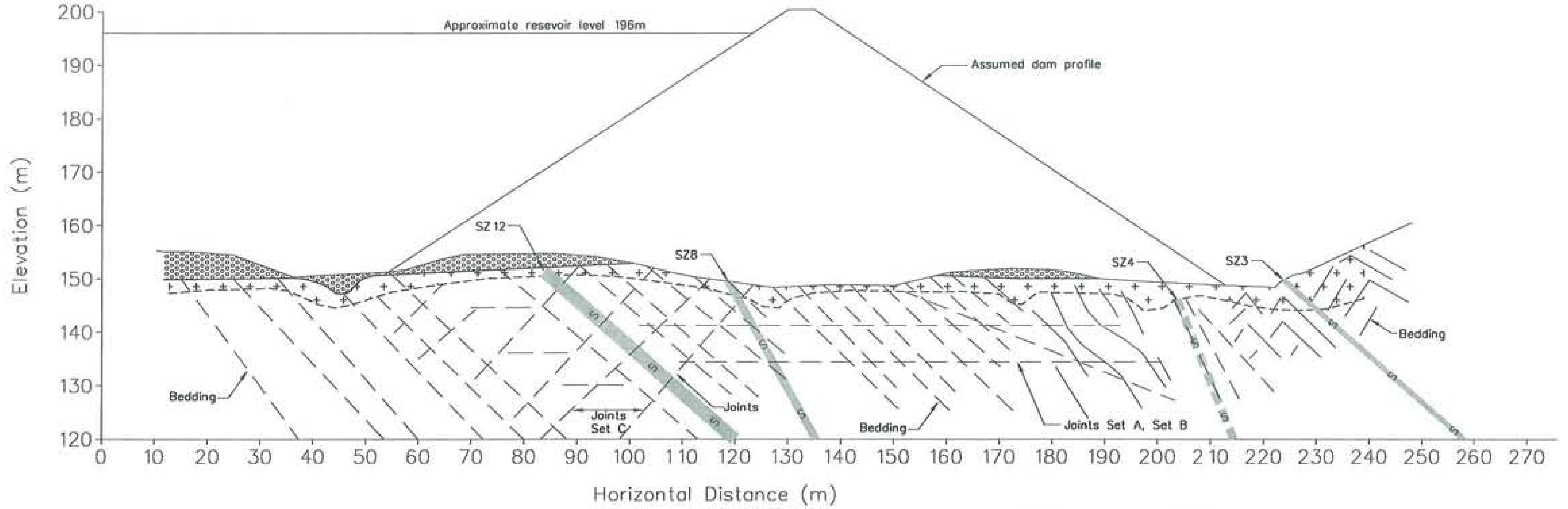
WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA WATER AUGMENTATION – PHASE 2
LEE RIVER
Dam Site Geology

FIG. No. 24727.204-F6 REV. 0

P:\24727\WorkingMaterial\C/



SECTION H/07
SCALE 1:1000



SECTION I/07
SCALE 1:1000

LEGEND

DH7	Drill hole
	Water table
	Sheared zones
	Inferred sheared zones
	Inferred joint
	Piezometer and water level
	Alluvium
	Solifluction/colluvium
	Class 1 rock - UW-SW, R3-R4, RMR>50, Erm>3000
	Class 2 rock - SW-MW, R2-R3, RMR 40-50, Erm 1000-3000
	Class 3 rock - MW-HW, R2, RMR<40, Erm<1000

A3 SCALE 1:1000
0 5 10 15 20 30 40 50 (m)

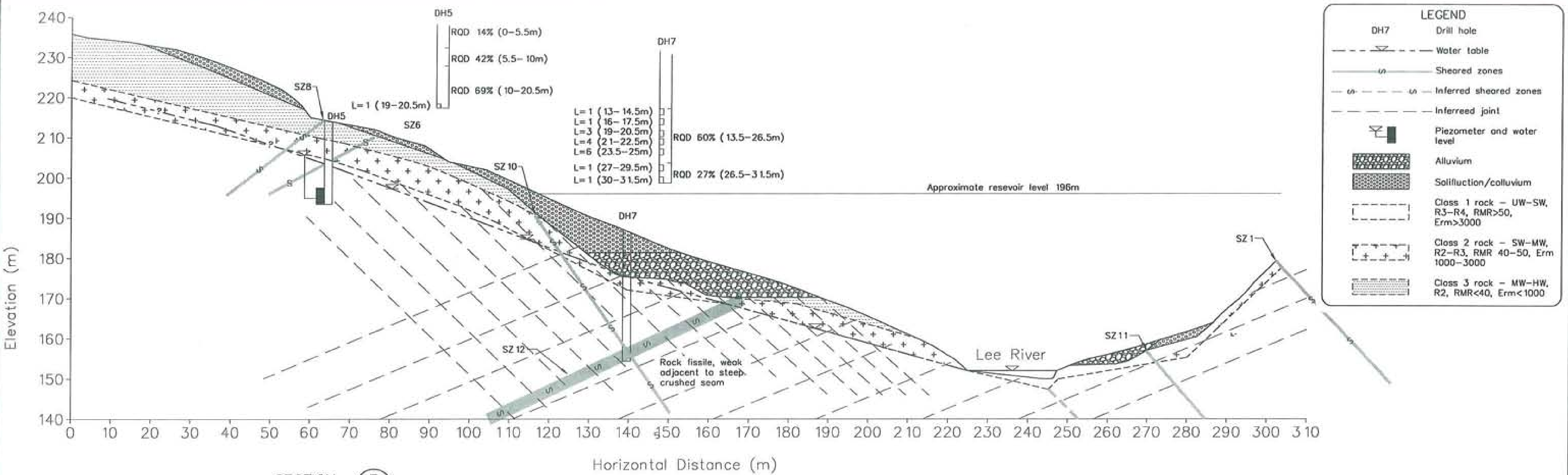
P:\24727\WorkingMaterial\CAD\01 DWGS\204 GEOSTAGE4\24727.204-F7_F9.dwg, F7, 27/11/2009 8:40:35 a.m., dnv, 1:1

Tonkin & Taylor
Environmental & Engineering Consultants
☐Auckland ☐Hamilton ■Nelson ☐Wellington ☐Christchurch ☐Whangarei

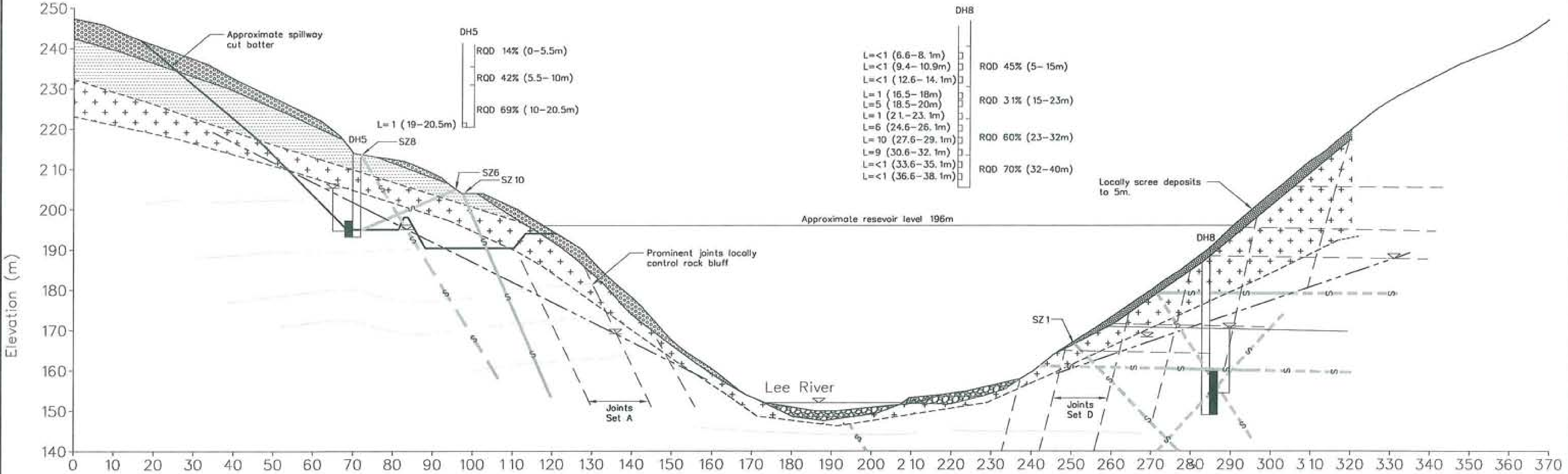
DRAWN	DJA	Jul.09
DRAFTING CHECKED	F	11/09
APPROVED		
CADFILE: \\24727.204-F7_F9.dwg		
SCALES (AT A3 SIZE) 1:1000		
PROJECT No. 24727.204		

WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA 2 - FEASIBILITY
LEE RIVER
Cross Sections H-H & I-I

FIG. No. 24727.204-F7
REV. 0



SECTION F/07
SCALE 1:1000



SECTION G/07
SCALE 1:1000

A3 SCALE 1: 1000
0 5 10 15 20 30 40 50 (m)

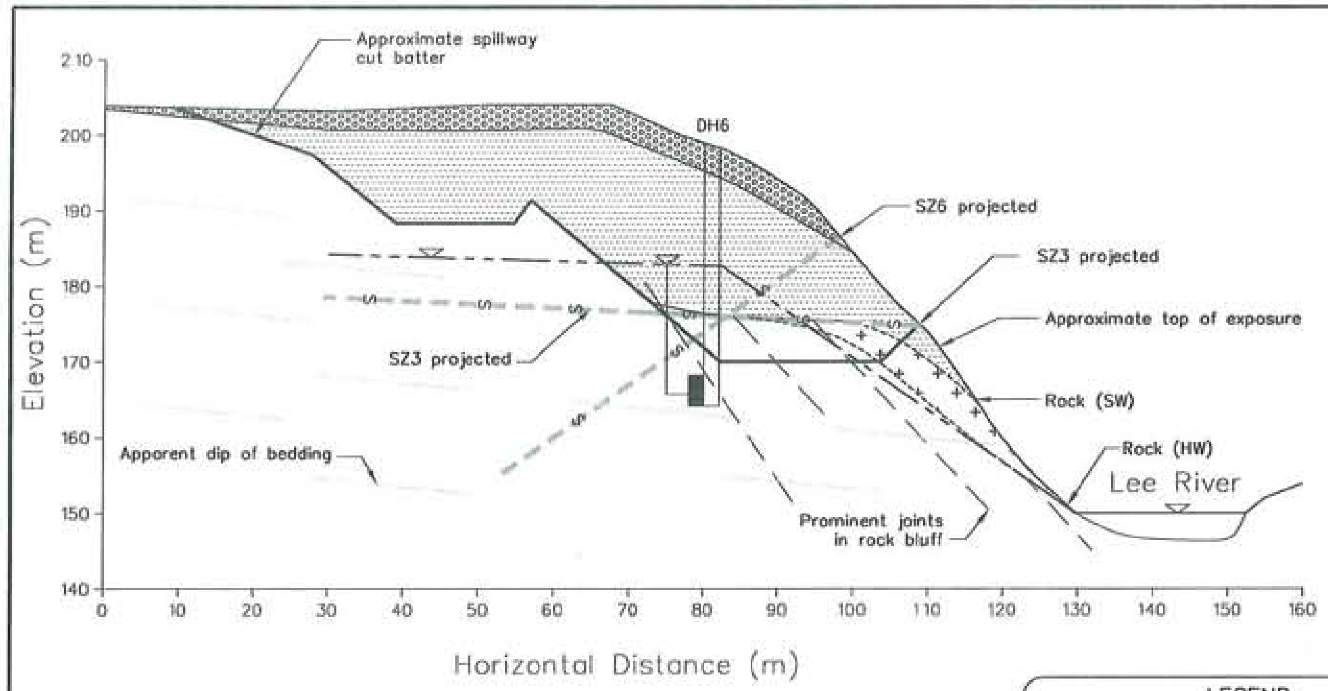
P:\24727\WorkingMaterial\CAD\01 DWGS\204 GEOSTAGE4\24727.204-F7_F9.dwg, F8, 27/11/2009 8:40:46 a.m., dnv, 1:1

Tonkin & Taylor
Environmental & Engineering Consultants
Auckland Hamilton Wellington Nelson Christchurch Whangarei

DRAWN	DJA	Jul.09
DRAFTING CHECKED	F	11/09
APPROVED		
CADFILE: \\24727.204-F7_F9.dwg		
SCALES (AT A3 SIZE) 1: 1000		
PROJECT No. 24727.204		

WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA 2 – FEASIBILITY
LEE RIVER
Cross Sections F-F & G-G

FIG No. 24727.204-F8 REV. 0

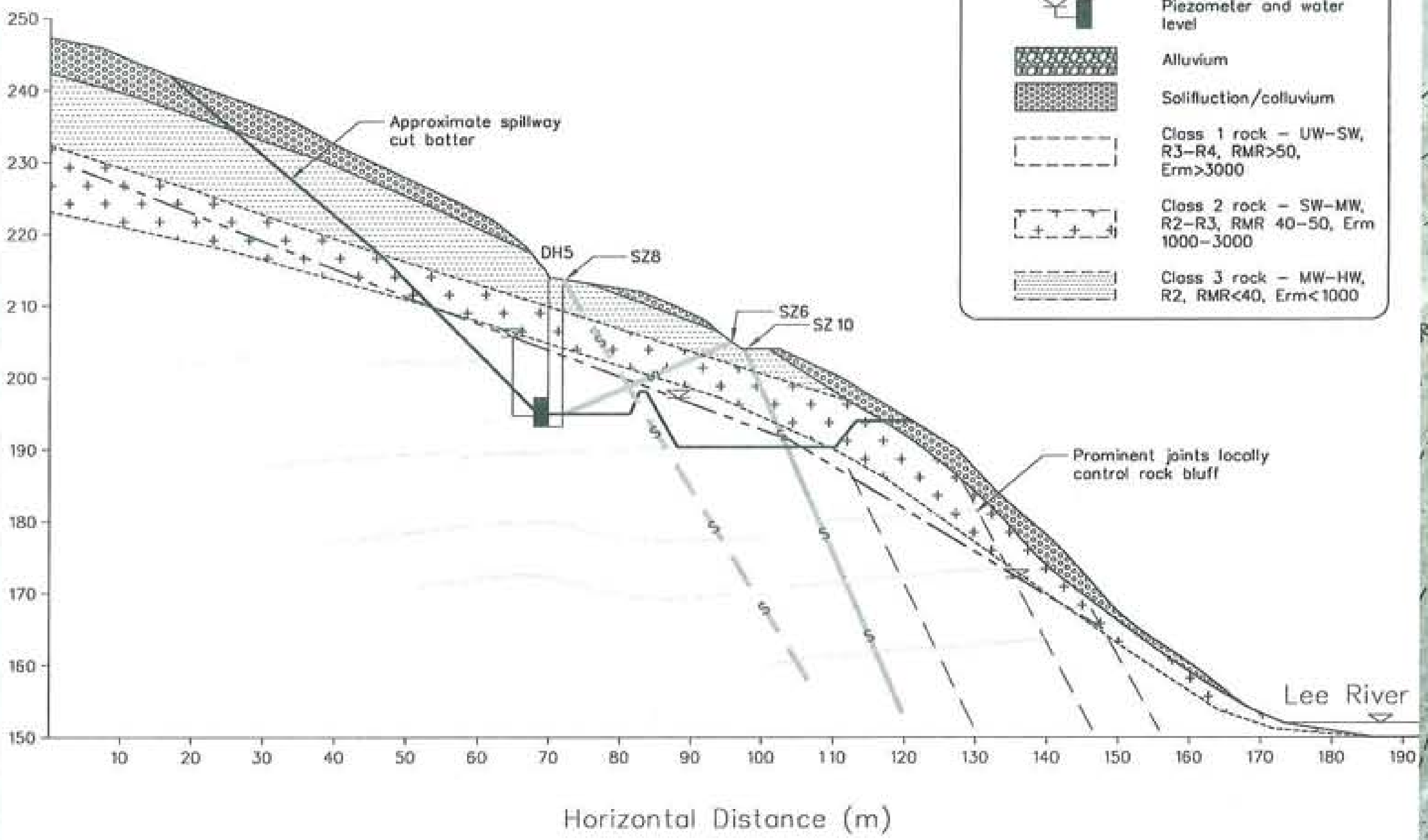


A3 SCALE 1: 1000
0 5 10 15 20 30 40 50 (m)

SECTION C
SCALE 1: 1000

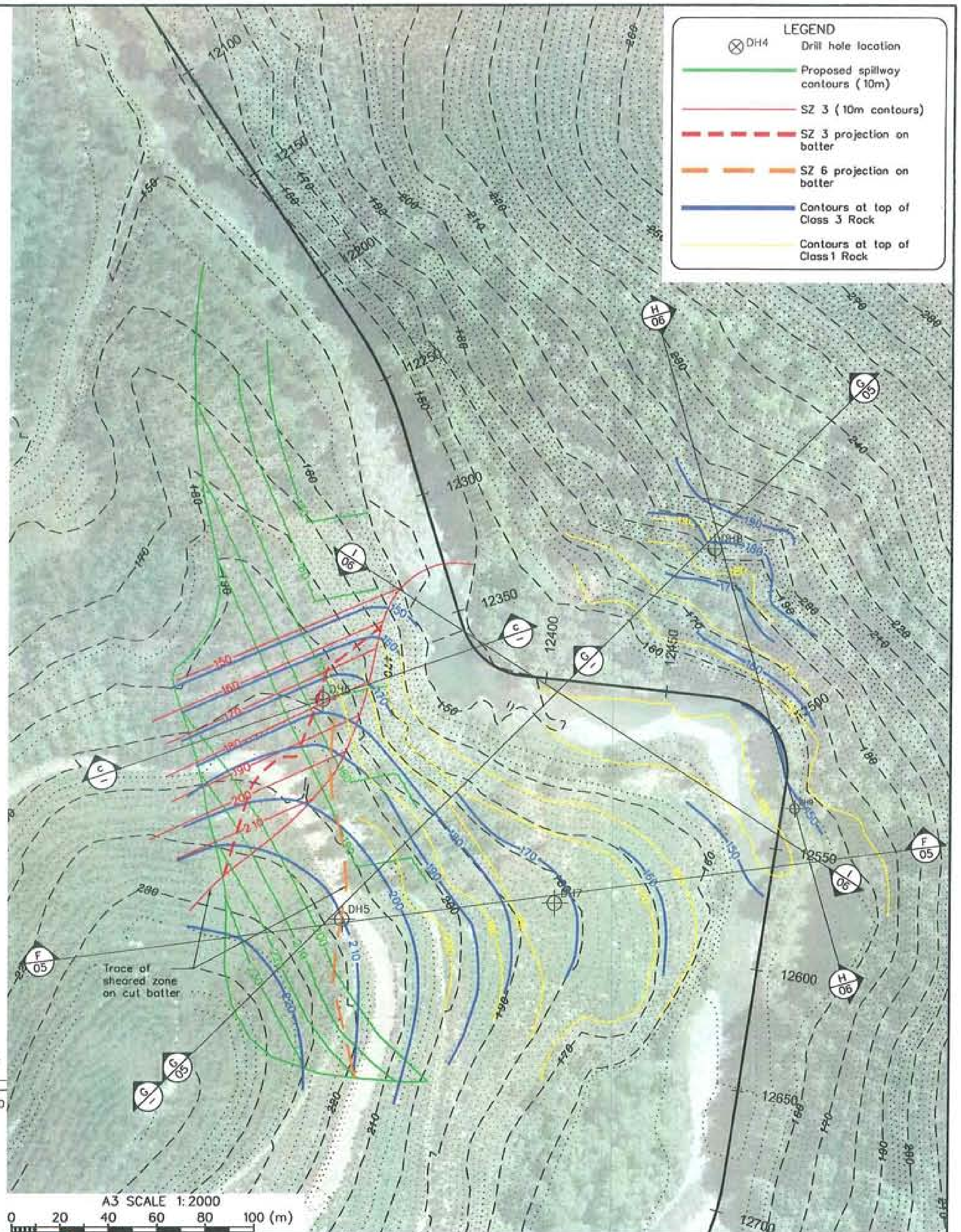
LEGEND

DH7	Drill hole
	Water table
	Sheared zones
	Inferred sheared zones
	Inferred joint
	Piezometer and water level
	Alluvium
	Solifluction/colluvium
	Class 1 rock – UW-SW, R3-R4, RMR>50, Erm>3000
	Class 2 rock – SW-MW, R2-R3, RMR 40-50, Erm 1000-3000
	Class 3 rock – MW-HW, R2, RMR<40, Erm<1000



A3 SCALE 1: 1000
0 5 10 15 20 30 40 50 (m)

SECTION G
SCALE 1: 1000



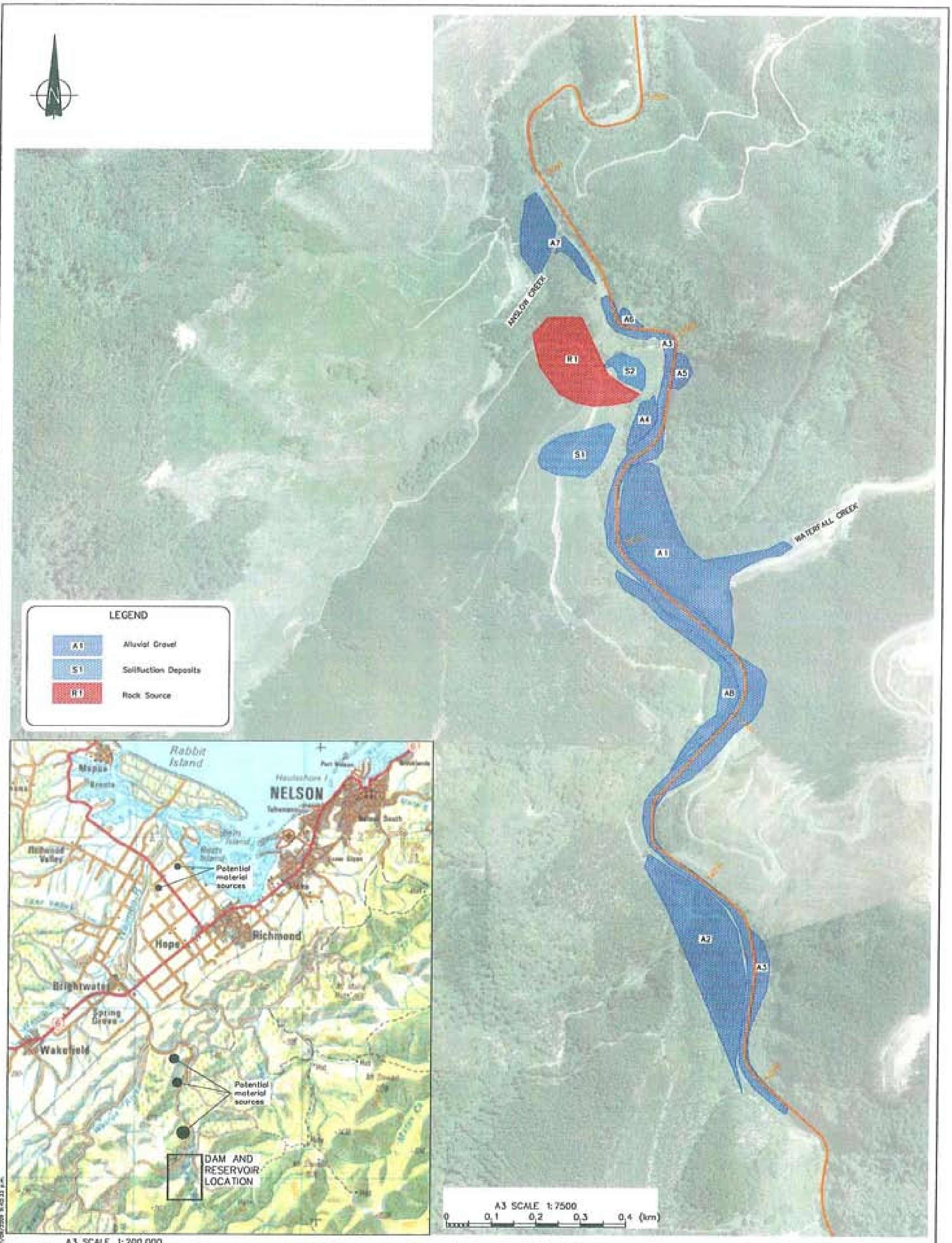
A3 SCALE 1: 2000
0 20 40 60 80 100 (m)

DRAWN	DJA	Jul.09
DRAFTING CHECKED		
APPROVED		
CADFILE: \\24727.204-F7_F9.dwg		
SCALES (AT A3 SIZE) AS SHOWN		
PROJECT No. 24727.204		

Tonkin & Taylor
Environmental & Engineering Consultants
Nelson
Auckland Hamilton Wellington Christchurch Whangarei

WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA 2 – FEASIBILITY
LEE RIVER
Rock Quality – Spillway and Plinth

FIG. No 24727.204-F9 REV. 0



Tonkin & Taylor
Environmental & Engineering Consultants
Auckland, Hamilton, Nelson, Wellington, Christchurch, Whangarei

DRAWN	DNV	Sep.09
DRAFTING CHECKED		
APPROVED		
CADFILE: 24727.204-F 10.dwg		
SCALES (AT A3 SIZE) 1:7500		
PROJECT No. 24727.204		

WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA WATER AUGMENTATION – PHASE 2
LEE RIVER
Potential Borrow Sites

FIG. No. 24727.204-F 10 REV. 0

Appendix B: Drill hole Logs

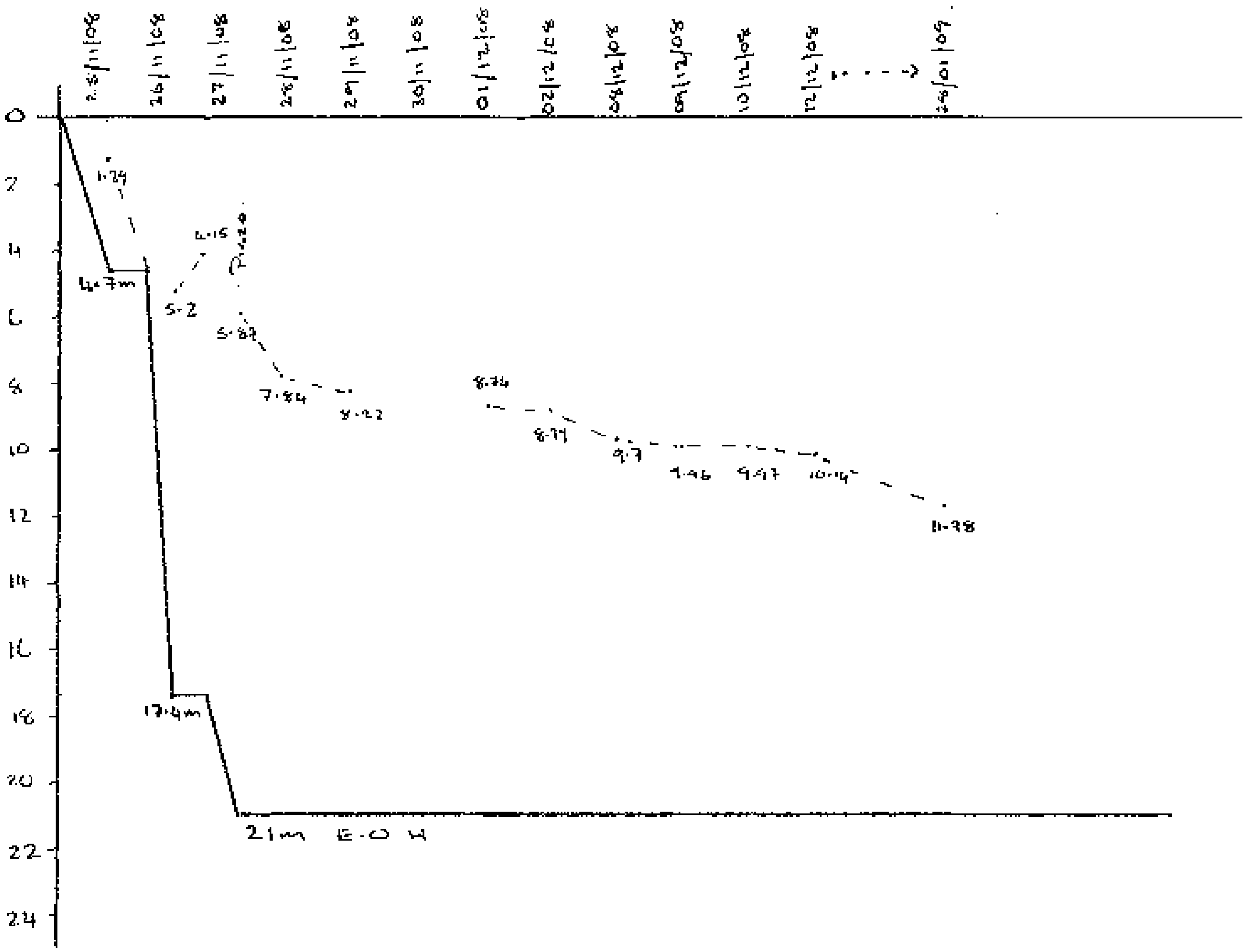
Tonkin & Taylor Job No: 24727-901 File: Sheet No: 1 of 1

Office: NELSON Project: LEE DAM Description: DH5

Computed: JXWW 03/12 2008 WATER LEVEL

Checked: 20 DATA

DH5 Drilling depth / Water vs Time.



Tonkin & Taylor

Office: NELSON

Computed: MRF 20

Checked: 20

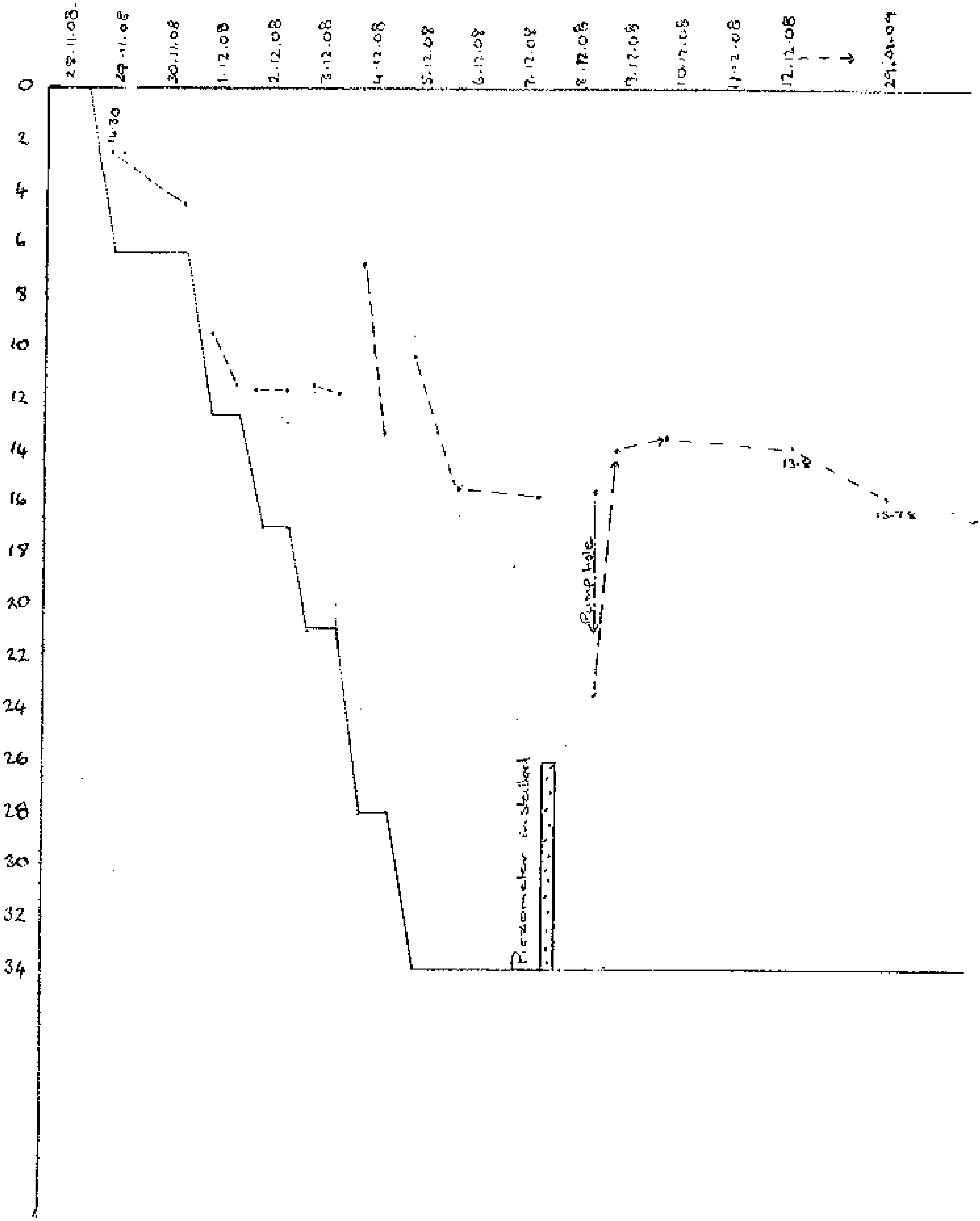
Job No: 24727-401 File:

Project: Lee Dam

Sheet No: 1 of 1

Description: DH6 WATER LEVEL DATA.

DH6 Drilling Depth / Water Depth vs Time



Tonkin & Taylor

Office: NELSON

Computed: JKWW **20**

Checked: **20**

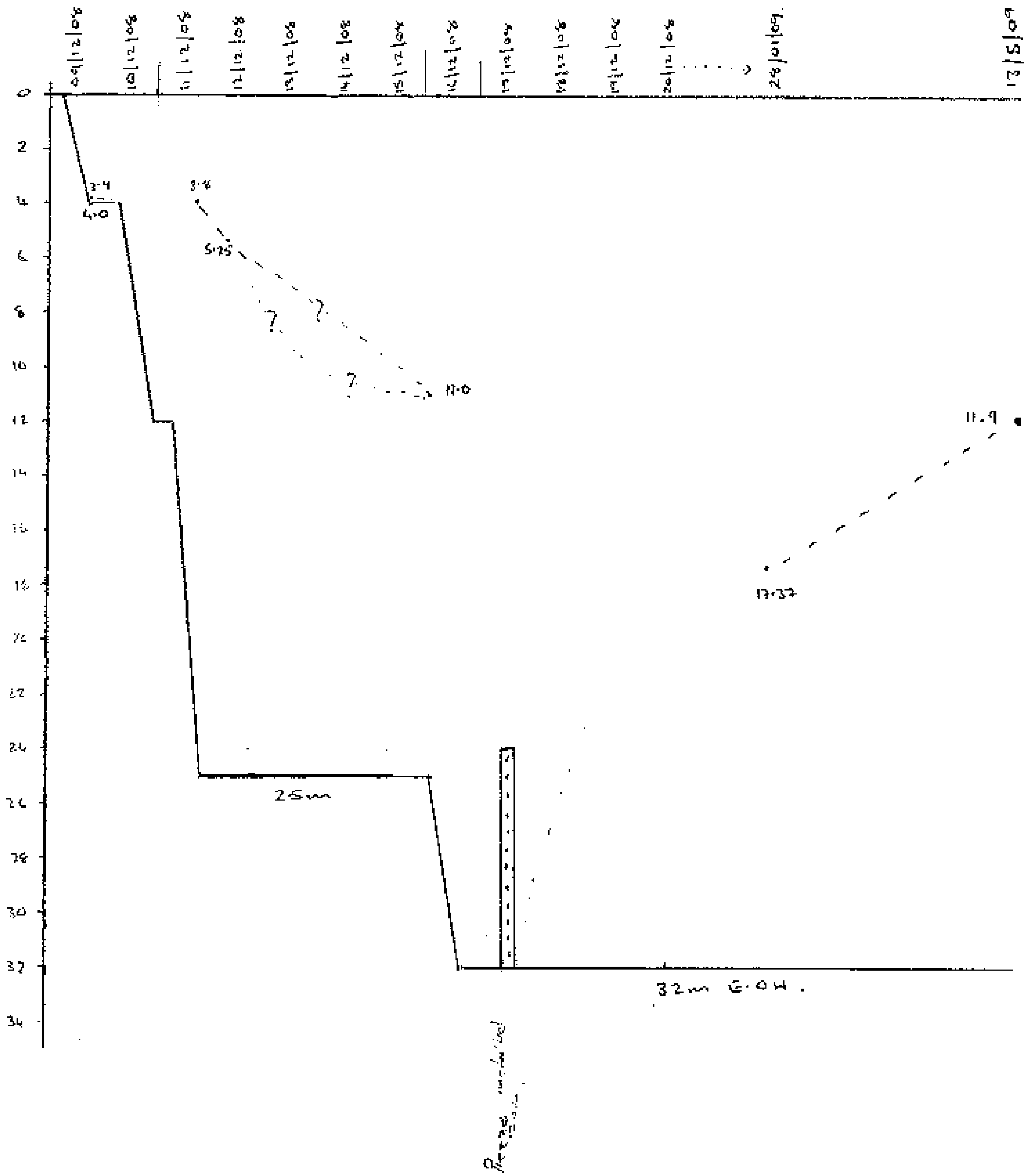
Job No: 20727.901 **File:**

Project: LEE DAM

Sheet No: 1 of 1

Description: DH 7 WATER LEVEL DATA

DH 7 Drilling depth / Water vs Time.



Tonkin & Taylor

Office: NELSON

Computed: JXWW 20/01 2009

Checked: 20

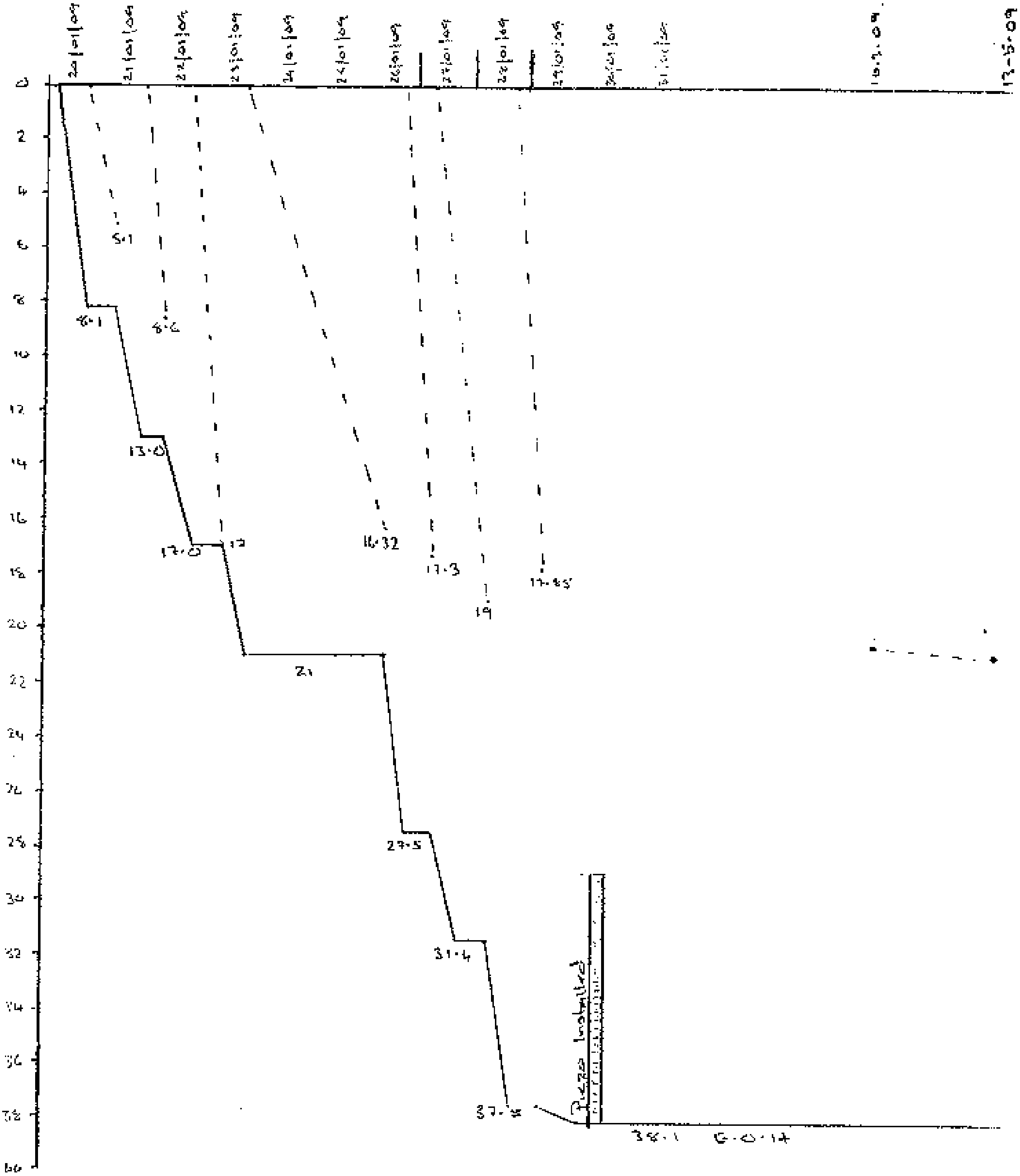
Job No: 24727.901 File:

Project: LEE DAM

Sheet No: 1 of 1

Description: DH8 WATER LEVELS.

DH8 Drilling depth/water level vs time



Tonkin & Taylor

Office: NELSON

Computed: JXWW 05/02 2009

Checked: 20

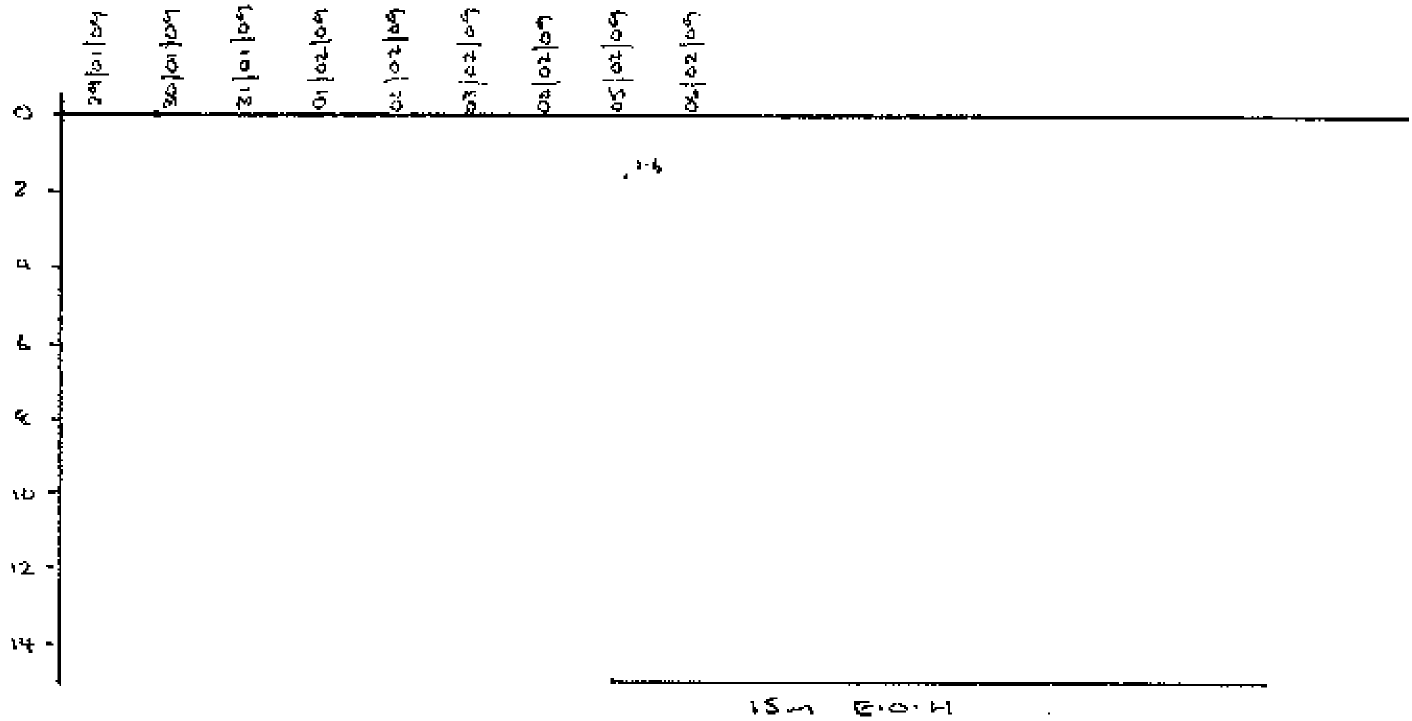
Job No: 26737.901 File:

Project: Lee Dam

Sheet No: 1 of 1

Description: DH9 Water Level Data.

DH9 Drilling depth/Water vs Time



Log Scale 1:50 @ A3.

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH5

PROJECT 24727-901 FEATURE Left Abutment LOCATION Lee River Valley
GRID REF CO-ORD DATUM
ANGLE FROM HORIZONTAL 90° DIRECTION H.A.D. GROUND H.A.D. COLLAR +0.55

DESCRIPTION OF CORE (weathering, hardness, strength, colour, rock or soil type, defect spacing, lithological features (bedding, foliation, minerology, texture, cement, etc...); stratigraphic name)	DEPTH H.A.D. (m)	ROCK DEFECTS / (OR SOIL DESCRIPTION)	R.O.D. %	WATER LEVEL
Residual siltstone	0	Sandy gravel, orange-brown tightly packed, fine, angular in a silty sand matrix with some clay.	0	
Rai Formation siltstone R1-R2, very closely spaced steeply dipping defects excavates as a GP silty gravel. No core lengths ≥ 100mm. Darkgrey & brown	1–2	J ~ 30°, T, SC, SW, CV silt; J ~ 20°, T-VT, SC-SW; J ~ 60°, T-O, SC-SW, CV-silt; J ~ 12°, VT, SC, silt; SZ 30°, O	21%	16:00 hrs 25/11
Lost core	2.3	Lost core	0	
Rai Formation siltstone R1-R2 closely spaced, defects, discreet SZ ≤ 100mm excavates as silty gravel with trace clay & much sand more competent than above. Dark grey & brown	3	Recovered as clean fine gravel	0	
Rai Formation siltstone Grey-green & dark grey laminated closely spaced joints, minor limonite staining on joint surfaces R2-R3	4–5	B ~ 60°, T-O, SC-WV, FeSt-silt; J ~ 72° VT, SC-PL, CN; J ~ 70° ⊥ to B, VT, SC-WV; J ~ 22°, O, PL-SC, CV-silt; SZ? ~ 20°, O, SC-P, CV	35% / 44%	08:00 hrs 27/11; 08:00 hrs 26/11; 17:42 hrs 26/11
Clayey silt, LP, grey-green	5.5		0	17:00 hrs 27/11
Rai Formation siltstone, R3 laminated, green, grey & red-brown, yellow limonite staining closely spaced defects. Thin bedding // shears about 6.5m.	6–7	B ~ 80, VT, SC-SW, CV-silt; J ~ 39°-60°, T-O, SC-SW, silt-FeSt; B ~ 59°, T, SC-SW, CV-silt-FeSt; SZ as above ≤ 10mm. Many drilling breaks.	64% / 62%	Piezo 0800 28/11; 10:00 29/11
Lost core	8	Lost core	0	
Recovered as coarse gravel R1	8.2	J 60°, O, SC-SW, silt-FeSt; J 22°, T, SW, silt	0	13:42 01/12; 12:00 02/12
Siltstone, R3-R4 increased limonite staining	9–10	B ~ 62°, T, SW-SC, silt-FeSt; J ~ 22°, T, SC-CV-silt; J ~ 58°, VT, SW-SC, silt-FeSt	40% / 72%	
Rai Formation siltstone R3-~~R4~~ green grey, closely spaced steeply dipping defects heavy limonite staining on defect surfaces	11–15	B ~ 40° to 60°, VT, SC-SW, FeSt; J ~ 78°, T, SC-SW, silt-FeSt; J ~ 30~60, VT, SC-SW, FeSt. Many drilling breaks	86% / 100% / 100% / 87%	

Box 1 · Box 2 · Box 3 · Box 4 · Box 5

Column headings: Rock weathering (SW MW HW), Rock hardness (H MH MS S), Point load test (MPa), Core loss /lift %, Depth H.A.D. Core size casing, Graphic log, Fracture log (spacing of natural fractures), Date/Depth, R.Q.D. %, Water level, Drill water loss % (0–100), Water pressure tests – Lugeons – or – permeability 10^{-7} m/s (0.01, 0.1, 1, 10, 100, 1000)

DRILLER: CW (Kane)
STARTED: 25/11/2008
FINISHED: 27/11/2008
DRILL: Skid Rig

ROCK WEATHERING	ROCK HARDNESS
UW – Unweathered	VH – Very hard
SW – Slightly Weathered	H – Hard
MW – Moderately Weathered	MH – Moderately hard
HW – Highly Weathered	MS – Moderately soft
CW – Completely Weathered	S – Soft
	VS – Very Soft

FRACTURE LOG: Spacing of Natural Fractures (cm: 100, 50, 10, 5, 1, 0.1); Fractures/m of core (1, 2, 10, 20, 100, 1000)

EXPLANATION:

LOGGED: JKWW
DATE: 26/11/08
TRACED:
CHECKED:
ORIGINAL VERTICAL SCALE: 1:50
PROJECT: 24727-901
HOLE NO: BH
LENGTH: 21m
CORE BOXES: 8
SHEET 1 OF 2 DRG NO

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH5

PROJECT: LEE DAM — FEATURE: LEFT ABUTMENT — LOCATION: LEE RIVER VALLEY
GRID REF: — CO-ORD: — DATUM:
ANGLE FROM HORIZONTAL: 90° — DIRECTION: — H.A.D. GROUND: — H.A.D. COLLAR: + 0.55

DESCRIPTION OF CORE (Weathering, hardness, strength, colour. Rock or soil type, defect spacing, lithological features (bedding, foliation, mineralogy, texture, cement, etc...); stratigraphic name.)	DEPTH H.A.D. (m)	ROCK DEFECTS (Prominent joints, bedding, seams, veins. Shatter, shear & crush zones, foliation schistosity, attitude, width, spacing, smoothness) (OR SOIL DESCRIPTION)	DATE/DEPTH	R.Q.D. %	WATER PRESSURE TESTS – Lugeons
RAI FORMATION SILTSTONE R4, laminated, Very closely spaced steeply dipping joints.	to 16	B ~ 40°, T, SC-PL, Fest J ~ , VT, SC-SW, Fest J ~ , T, ST-PL, Fest		87%	
CORE LOSS SILTY CLAY HP GREY	16	CORE LOSS SILTY CLAY HP GREY	16.50m		
As above.	16–17.6	As above many drilling breaks silt on all jnt surface.	26/11/08	87% 95%	
As above decreasing core quality, split core still has heavy limonite staining on joint surfaces. Core washed out @ 19.4m	17.6–19.9	J ~ 20°, T, WV, Fest-silt. B ~ 40°, T-VT, SC-SW, Fest J ~ 84°, T-O, ST, PL, Fest Drill induced shatter J ~ 30°, VT, SC-SW, Fest. RQD 19.4 to 19.8m ~ 67%		35% 29%	
SILTSTONE, R4, laminated, light grey with light limonite staining light shatter.	19.9–20.3	B ~ 50, VT, SC-SW, Fest (light) J ~ 40° T, SC-SW, Fest (") J ~ 10°, VT, SW			0.3
LOST CORE	20.3–20.7	WASHED OUT		30%	
	20.7–21	Drilling disturbed	27/11/08		
	21	E.O.H 21m			

Box 6
Box 7
Box 8

DRILLER: CW (KANE)
STARTED: 24/11/2008
FINISHED: 27/11/2008
DRILL: HQ SKID RIG

ROCK WEATHERING	ROCK HARDNESS
UW – Unweathered	VH – Very hard
SW – Slightly Weathered	H – Hard
MW – Moderately Weathered	MH – Moderately hard
HW – Highly Weathered	MS – Moderately soft
CW – Completely Weathered	S – Soft
	VS – Very Soft

FRACTURE LOG
Spacing of Natural Fractures: 100 50 10 5 1 0.1
Fractures/m of core: 1 2 10 20 100 1000

EXPLANATION:

LOGGED: JXWW
DATE: 27/11/2008
TRACED:
CHECKED:
ORIGINAL VERTICAL SCALE: 1:50 @ A3
PROJECT: 24727-901
HOLE NO: BH5
LENGTH: 21m
CORE BOXES: 8
SHEET 2 OF 2
DRG NO

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH6

PROJECT: Lee Dam Investigation FEATURE: Left Abutment LOCATION: ……
GRID REF: …… CO-ORD: …… DATUM: ……
ANGLE FROM HORIZONTAL: 90° DIRECTION: …… H.A.D. GROUND: …… H.A.D. COLLAR: ……

DESCRIPTION OF CORE (weathering, hardness, strength, colour, rock or soil type, defect spacing, lithological features (bedding, foliation, mineralogy, texture, cement, etc...); stratigraphic name)	DEPTH H.A.D. (m)	ROCK DEFECTS (prominent joints, bedding, seams, veins, shatter, shear & crush zones, foliation schistosity, attitude, width, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc...)	R.Q.D. %
Lost core		Lost core	
Rai Formation Sandstone R1 – R2, light grey & orange-brown, very blocky, very closely spaced steeply dipping intersecting defects	0–2	J ~ 79°, O, SC–PL, CV–FeSt. B? ~ 60°, T–O, SC–P/SL, CV–FeSt. J ~ 50°, T–O, SC–PL, CV–silt. J ~ 20°, VT, SC–PL, silt. Shattered & recovered as a coarse gravel	12%, 22%
Shattering	2–2.5	Highly shattered.	24%
Recovered as silty gravel with clay regularly spaced sticks of ≤120mm sandstone.	2.5–3.5		0, 77%/0
Shatter	3.5–4.8	B? 50–60°, T–O, SW, CV–silt. J ~ 70°, VT–T, SW, CV–silt–FeSt	25%, 12%
Sandstone, R2 – R3, light brown & grey, blocky, closely spaced defects	4.8–5.9	J ~ 78, T, SC–ST, silt. J ~ 42°, T, SC–SW, silt. Some shatter, many micro fractures	0, 0
Recovered as a silty gravel	5.9–6.1	GM.	0
Rai Formation Siltstone R3, laminated, blocky, closely spaced defects. 30° stepped joint & fracturing around annealed shatter clasts.	6.1–7.8	J ~ 87°, T, SC, PL, silt. J ~ 30°, T, ST, silt–CV. B ~ 42°, T–VT, SC–SW, silt–CV. B ~ 52°, T, SC–SW, silt–water. J ~ 72°, T, SW–PL, silt – water	63%, 0, 50%
Siltstone, R2, laminated very blocky, very closely spaced defects.	7.8–8.9	J ~ 19°, T, SW, silt	16%
Shattered siltstone. Possible hydraulic fracture ?	8.9–10.9	Shattered rock average clast ~ 7mm × 20mm probably opened by drilling. Defects as above	0, 0, 0
As above, R3. Water along bedding plane lamination annealed shatter, blocky, closely spaced defects	10.9–13.2	B ~ 30–40° – T, SC–SW, silt. J ~ 50° – T, SC–PL, silt/CN. J ~ 50° – T–O, SW, FeSt–silt. As above increased limonite.	33%, 0, 0, 23%
Siltstone recovered as a clean gravel.	13.2–14.5	Many drilling breaks.	0, 20%
Lost core	14.5–14.7	Lost core	0
Siltstone R4	14.7–15	Many drilling breaks	

DATE/DEPTH: 6.2m 28/11/2008; 12.5m 01/12/2008

WATER LEVEL: 17:00 28/11; 11:00 hrs 01/12; 17:00 hrs 01/12; 09:00 hrs 02/12

DRILL WATER LOSS %: Drillers Estimate (5%); (60%)

WATER PRESSURE TESTS – Lugeons ~~or PERMEABILITY 10^{-7} m/s~~: Test 1 – 2; Test 2 – 30; Test 3 – 10

Core size, casing: HQ

Box 1, Box 2, Box 3, Box 4, Box 5, Box 6

DRILLER: CW
STARTED: 28/11/2008
FINISHED: 08/12/2008
DRILL: Skid rig

ROCK WEATHERING	ROCK HARDNESS
UW – Unweathered	VH – Very hard
SW – Slightly Weathered	H – Hard
MW – Moderately Weathered	MH – Moderately hard
HW – Highly Weathered	MS – Moderately soft
CW – Completely Weathered	S – Soft
	VS – Very Soft

FRACTURE LOG: Spacing of Natural Fractures (cm) 100 50 10 5 1 0.1; Fractures/m of core 1 2 10 20 100 1000

EXPLANATION:

LOGGED: JXW
DATE: 02-12-2008
TRACED: ……
CHECKED: ……
ORIGINAL VERTICAL SCALE: 1:50 @ A3
PROJECT: 24723.901
HOLE NO: BH6
LENGTH: 34m
CORE BOXES: 14
SHEET 1 OF 3 DRG NO ……

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH6

PROJECT Lee Dam FEATURE Left Abutment LOCATION Lee River Valley
GRID REF CO-ORD DATUM
ANGLE FROM HORIZONTAL 90° DIRECTION H.A.D. GROUND H.A.D. COLLAR

Depth H.A.D. (m)	DESCRIPTION OF CORE (weathering, hardness, strength, colour, rock or soil type, defect spacing, lithological features (bedding, foliation, minerology, texture, cement, etc...); stratigraphic name)	ROCK DEFECTS (prominent joints, bedding, seams, veins, shatter, shear & crush zones, foliation schistosity, attitude, width, spacing, smoothness) (OR SOIL DESCRIPTION)	R.O.D. %	Water pressure tests – Lugeons
15–16	SHATTERED SILTSTONE, R2 laminated with closely spaced, steeply dipping defects, many micro fracture becoming more weathered down hole. Grey-green.	Recovered (GP) gravel; B~30°, T-O, SC-SW, Fest; J~80°, T, SC-PL, Fest-qz; J~60, T-O, SC-PL, Fest	100/1200; 200/400	Test 4: 18
17–18	SHATTERED ROCK	Recovered as clean gravel (GP)	0/300; 0/300; 40/100; 20/300	Test 5: 1
18	LOST CORE	Lost core	0	
18.5–20	SHEARED & SHATTERED very steeply dipping partially annealed bedding parallel shears. SHATTERED SILTSTONE, defects consistent with previous. R2 – R3	SZ~68°, T, PL-ST, CW-Fest, qz; J~36°, T, SC-SW, silt Fest, qz; J~76°, T-O, WV, CV-Fest	0/100; 360/?	
20–21	RECOVERED AS CLEAN GRAVEL. SILTSTONE, grey-green. laminated some shatter R2	GP - med-coarse; J~70, VT, PL annealed qz; B~50, T, SC-SW, Fest	0; 120/?	Test 6: 9
21	LOST CORE	Lost core	0	
21–22	RECOVERED AS CLEAN GRAVEL. SHATTERED SILTSTONE R1-R2	GP - coarse; J~32°, SW, T-O, CV-Fest; J~30, T, SC-WV, CV-Fest	0; 0; 120/400	
22	LOST CORE	Lost core	0/?	
22–23	SHATTERED/CRUSHED SILTSTONE R0, intact core has properties of a fine silty gravel, loose to medium dense.	GP-GM coarse	0; 0/500	
23–24	SHEAR ZONE, bedding parallel, argillaceous siltstone	BSZ~40°, T-O, SW-P, silt; J~32°, T-O, SW-SC, silt; J~58°, T-O, SW-SC, silt-Fest; J~22°, T-VT, SC, CN	66%	Test 7: 2
24–26	INTERBEDDED SILTSTONE (dark grey) SANDSTONE, (green-grey), R4 closely spaced, steeply dipping defects, thickly laminated (≥20mm)	B~30°, T-VT, SW-SC, Fest; some drilling breaks; annealed sub V defect P ≥ 50mm; SZ 25.4 to 25.9m (R2); BSZ~40°, SW-SC P, CN-Fest; J~60°, VT-O, ST, qz annealed; J~50°, T-VT, SW-SC, CN-Fest	860/300; 200/?; 500/600 56%	
26–27		J~50, O, SC, Fest; Annealed perpendicular stress fractures	52%	Test 8: 1
27–27.5	CRUSH ZONE R0-R1	Crush.	880/?	
27.5–30	SILTSTONE, laminated, R4 green-grey, superficial yellow limonite staining on defects; improving core quality.	J~78°, SW-ST, T-O, Fest; B~50°, VT-closed, SW-SC, Fest; Many drilling breaks	250/?; 64%; 900/1000	Test 9: 12.3

Date/Depth: 08/12/08 09/w/? (15 m); 03/12/08 21.1m
Water level / Drill water loss: 15-20% Drillers estimate
Core boxes: Box 7, Box 8, Box 9, Box 10, Box 11

DRILLER: C.W. (KANE)
STARTED: 28/11/2008
FINISHED: 08/12/2008
DRILL: SKID

ROCK WEATHERING
UW – Unweathered
SW – Slightly Weathered
MW – Moderately Weathered
HW – Highly Weathered
CW – Completely Weathered

ROCK HARDNESS
VH – Very hard
H – Hard
MH – Moderately hard
MS – Moderately soft
S – Soft
VS – Very Soft

FRACTURE LOG
Spacing of Natural Fractures (cm): 100 50 10 5 1 0.1
Fractures/m of core: 1 2 10 20 100 1000

EXPLANATION:

LOGGED: JKNW
DATE: 04/12/2008
TRACED:
CHECKED:
ORIGINAL VERTICAL SCALE: 1:50 @ A3
PROJECT: 24727-901
HOLE NO: DH 6
LENGTH: 34m
CORE BOXES: 14
SHEET 2 OF 3 DRG NO

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH6

PROJECT: LEE DAM .... FEATURE: LEFT ABUTMENT .... LOCATION: LEE RIVER VALLEY

GRID REF. .... CO-ORD. .... DATUM ....

ANGLE FROM HORIZONTAL: 90° .... DIRECTION .... H.A.D. GROUND .... H.A.D. COLLAR ....

DESCRIPTION OF CORE (WEATHERING, HARDNESS, STRENGTH, COLOUR. ROCK OR SOIL TYPE, DEFECT SPACING, LITHOLOGICAL FEATURES (bedding, foliation, mineralogy, texture, cement, etc...); STRATIGRAPHIC NAME.)	ROCK WEATHERING (SW MW HW)	ROCK HARDNESS (H MH MS S)	POINT LOAD TEST (MPa)	CORE LOSS /LIFT % (5 10 50)	DEPTH H.A.D. Core size, casing m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures) (50 10 5 1 cm)	ROCK DEFECTS: PROMINENT JOINTS, BEDDING, SEAMS VEINS. SHATTER, SHEAR & CRUSH ZONES, FOLIATION SCHISTOSITY attitude, width, spacing, smoothness (OR SOIL DESCRIPTION) consistency, compactness, water content, group symbol etc...	DATE/DEPTH	R.O.D. %	WATER LEVEL (Date)	DRILL WATER LOSS % (0–100)	WATER PRESSURE TESTS – Lugeons or PERMEABILITY – 10^{-7} m/s (0.01 0.1 1 10 100 1000)
Interbedded RAI FORMATION SILTSTONE & Fine SANDSTONE Rk. with closely spaced steeply dipping joint sets, superficial limonite staining on joints	SW				31	50°, 60°, 88°		B ~ 50°, sw-sc, VT-closed, cm-fest J ~ 60, P-closed, sw-sc, cm-fest J ~ 88, VT, sc-P, fest Shallower 2 defects are partially qz annealed.		380/400 67 / 1200			12.3 — Box 12
laminated.					32			Many drilling breaks		800/1200			TEST 10
shear					33			Defects as above.		250/500			2 — Box 13
								Many Drilling Breaks.	34m 08/12/2008	780/1000			
					34			E.O.H 34m.					— Box 14

DRILLER: C.N. (Kane)

STARTED: 28/11/2008

FINISHED: 08/12/2008

DRILL: SKID RIG

ROCK WEATHERING
- UW – Unweathered
- SW – Slightly Weathered
- MW – Moderately Weathered
- HW – Highly Weathered
- CW – Completely Weathered

ROCK HARDNESS
- VH – Very hard
- H – Hard
- MH – Moderately hard
- MS – Moderately soft
- S – Soft
- VS – Very Soft

FRACTURE LOG

Spacing of Natural Fractures: 100 50 10 5 1 0.1

Fractures/m of core: 1 2 10 20 100 1000

EXPLANATION:

LOGGED: JXWW

DATE: 08/12/2008

TRACED: ....

CHECKED: ....

ORIGINAL VERTICAL SCALE: 1:50 @ A3

SHEET 3 OF 3 .... DRG NO ....

PROJECT: 24727.001

HOLE NO: DH6

LENGTH: 34m

CORE BOXES: 14

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH 7

PROJECT: Lee Dam
FEATURE: Left Abutment (Low Bench)
LOCATION:
GRID REF:
CO-ORD:
DATUM:
ANGLE FROM HORIZONTAL: 90°
DIRECTION:
H.A.D. GROUND:
H.A.D. COLLAR:

Column headings: DESCRIPTION OF CORE (WEATHERING, HARDNESS, STRENGTH, COLOUR, ROCK OR SOIL TYPE, DEFECT SPACING, LITHOLOGICAL FEATURES (bedding, foliation, mineralogy, texture, cement, etc...); STRATIGRAPHIC NAME.) | ROCK WEATHERING (SW MW HW) | ROCK HARDNESS (H MH MS S) | POINT LOAD TEST (MPa) | CORE LOSS /LIFT % | DEPTH H.A.D. m | Core size, casing | GRAPHIC LOG | FRACTURE LOG (Spacing of natural fractures) cm | ROCK DEFECTS (PROMINENT JOINTS, BEDDING, SEAMS VEINS. SHATTER, SHEAR & CRUSH ZONES, FOLIATION SCHISTOSITY attitude, width, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc...) | DATE/DEPTH | R.O.D. % | WATER LEVEL Date | DRILL WATER LOSS % 0–100 | WATER PRESSURE TESTS – Lugeons ~~or PERMEABILITY 10^{-7} m/s~~ (0.01, 0.1, 1, 10, 100, 1000)

Depth (m)	Description of core	Rock defects (or soil description)
0 – ~4.5	SOLIFLUCTION, yellow-brown, CLAYEY SILT with much fine angular platey gravel grading to SILTY SANDS with much gravel & some clay	TEST PIT BACKFILL. Low plasticity ML-GM. Non plastic SM-GM
~4.5 – ~7.1	ALLUVIUM. SILTY GRAVEL, yellow-brown HW, fine to coarse, some cobbles, rounded to subrounded in a non plastic sandy-silt matrix with trace clay. well sorted, poorly graded individual layers. Well graded over whole unit	TEST PIT BACKFILL. GM-GW. Washed core.
~7.1 – ~7.3	LOST CORE	LOST CORE
~7.3 – ~10.2	SILTY GRAVEL, yellow-brown to red-brown, HW, medium to cobble, rounded to subrounded in either a non plastic silty-sand or low plasticity clayey-silt matrix	GM-GW. Boulder & many cobbles
~10.2 – ~14.2	RAI FORMATION SILTSTONE R0, green-grey, R4, laminated, closely spaced, steeply dipping defects.	CRUSH / SHEAR. Many drill breaks (high bit pressure). B~60, SC-SW, VT~closed, CN. J~80, SC-PL, T-O, silt. Washed.
~14.2 – 15	As above MW, R3-R2, heavy FeSt.	J ~75°, SC-WY, T, FeSt-silt. J ~26°, SC-SW, T, FeSt. Split core heavy Fe staining

DATE/DEPTH: 4m 09/12/08

R.O.D. %: N/A; N/A; N/A; N/A; N/A; N/A; N/A; 0/200; 150/300; 650/1200 54%; 600/600 100%; 290/300; 430/400

WATER LEVEL: 1700hrs 09/12 – 11/12; 0800hrs 12/12

DRILL WATER LOSS: Drillers estimate 5–10%

WATER PRESSURE TESTS: Test 1 – 5; Test 2 – 1; Test 3 – 1

Box 1; Box 2; Box 3.

Graphic log annotations: 75; 26

DRILLER: CW (Kane)	ROCK WEATHERING	ROCK HARDNESS	FRACTURE LOG	LOGGED: J.X	PROJECT: 24727.901
STARTED: 09/12/2008	UW – Unweathered	VH – Very hard	Spacing of Natural Fractures (cm): 100 50 10 5 1 0.1	DATE: 12/12/2008	HOLE NO: DH 7
FINISHED:	SW – Slightly Weathered	H – Hard	Fractures/m of core: 1 2 10 20 100 1000	TRACED:	LENGTH: 32m
DRILL: Skid Rig	MW – Moderately Weathered	MH – Moderately hard		CHECKED:	
	HW – Highly Weathered	MS – Moderately soft		ORIGINAL VERTICAL SCALE: 1:50 @ A3	CORE BOXES: 10
	CW – Completely Weathered	S – Soft		SHEET 1 OF 3	DRG NO
		VS – Very Soft			

EXPLANATION:

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH7

PROJECT: Lee Dam
FEATURE: Left Abutment
LOCATION: Lee River Valley
GRID REF: ...
CO-ORD: ...
DATUM: ...
ANGLE FROM HORIZONTAL: 90°
DIRECTION: ...
H.A.D. GROUND: ...
H.A.D. COLLAR: ...

Depth (m)	DESCRIPTION OF CORE (Weathering, hardness, strength, colour. Rock or soil type, defect spacing, lithological features (bedding, foliation, minerology, texture, cement, etc...); stratigraphic name.)	ROCK DEFECTS (Prominent joints, bedding, seams, veins. Shatter, shear & crush zones, foliation schistosity; attitude, width, spacing, smoothness) (OR SOIL DESCRIPTION)	R.Q.D. %
~15.0–15.4	Shear Zone R2	SZ ~ 80°, SC-SW, O, Fest. J ~ 40, SC-PL, T, CN-Fest.	0/500
~15.4–18.3	Rai Formation SILTSTONE R4 MW-SW, dark green-grey, laminated, closely spaced, tight steeply dipping defects, micro fractures, qz annealed joints. SPLIT CORE FROM 16.1m. R3.	J ~ 58°, SC-PL, T, CN. B ~ 42°, SC-SW, VT-T, SC. J ~ 22°, SC-SW, C, CN. Some drill breaks. B ~ 42°, SC-WW, VT, Fest. 60° Drilling break CN. J ~ 80°, SC-PL, T, Fest, slight silt/CN. J ~ 38°, SC-PL, T, Fest.	400/500; 32%; 53%
~18.3–18.4	Shear Zone R1	Excavated as GP BSZ	0
~18.4–24.0	Rai Formation SILTSTONE R4 SW, blue grey - dark green grey, laminated, closely spaced, moderately steep to steeply dipping defects, qz annealed defects, occ Fest. Core becomes more competent.	Much drilling disturbance. J, 24°, SW, ST, C, qz annealed. B ~ 70° SW-SC, T-VT, CN-Fest. OLD SHEAR ZONE annealed micro fractures. B ~ 38°-44°, SW-SC, T-VT, Fest silt. J ~ 60° ~ T/partially annealed, SC-PL, Fest-CN. Many drilling breaks. B ~ 36°, SC-SW, VT-T, silt-fored. Many annealed micro fractures.	73%; 73%; 0/400; 55%; 100%
~24.0–24.8	SILTSTONE, Argillaceous, R4-R3, dark green-grey & black, high fissility	Some Drilling breaks. B ~ 29°, SC-SW, T, CN-silt.	33%
~24.8	CLAYEY SILT & compacted 10mm thick	CLAY SILT, LP, 8mm blue-grey	
~24.8–24.9	CORE LOSS	CORE LOSS	
~24.9–26.5	CRUSH ZONE in argillaceous rock R2. Rock has properties of a weakly cemented sandy-silt	CRUSH ZONE. SZ 70°, SW-SW, PL, O, crush. B ~ 42°, SC-SW, T-VT, CN	51%
~26.5–26.8	CORE LOSS	CORE LOSS	
~26.8–27.7	Interbedded argillaceous siltstone & laminated siltstone.	B ~ 30°, SW-SC, C. J ~ 42°, SW-SC, T/partially annealed SC	33%
~27.7–27.8	CORE LOSS	CORE LOSS	
~27.8–29.0	Micro fracturing. SILTSTONE, laminated, blue-grey R3-R4, blocky, 3 sub V qz annealed joint sets, mod-steep to steeply inclined sets // & ⊥ to bedding. Closely spaced, some fine sandstone, bedding // crush zones	B ~ 40°, SW-SC, VT-C, silt-CN. Drilling breaks & shatter. 150mm Recovered as GP gravel.	31%
~29.0–29.1		LOST CORE	
~29.1–29.9		CRUSH ZONE	0%
~29.9–30.0	CORE LOSS	CORE LOSS	

WATER PRESSURE TESTS – Lugeons (or PERMEABILITY – 10^{-7} m/s)

Test	Lugeon	Core box
Test 4	0.75	Box 4
Test 5	2.75	Box 5
Test 6	4.5	Box 6
Test 7	5.3	Box 7
Test 8	1	Box 8, Box 9

DRILL WATER LOSS: DRILLERS ESTIMATE 5 - 10%

DRILLER: CW (KANE)
STARTED: 09/12/2008
FINISHED: ...
DRILL: SKID RIG

ROCK WEATHERING
UW – Unweathered
SW – Slightly Weathered
MW – Moderately Weathered
HW – Highly Weathered
CW – Completely Weathered

ROCK HARDNESS
VH – Very hard
H – Hard
MH – Moderately hard
MS – Moderately soft
S – Soft
VS – Very Soft

FRACTURE LOG
Spacing of Natural Fractures: 100 50 10 5 1 0.1
Fractures/m of core: 1 2 10 20 100 1000

EXPLANATION:

LOGGED: JXWW
DATE: 07/01/2009
TRACED: ...
CHECKED: ...
ORIGINAL VERTICAL SCALE: 1:50 @ A3
PROJECT: 24727.401
HOLE NO: DH 7
LENGTH: 32
CORE BOXES: 10
SHEET 2 OF 3
DRG NO ...

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH7

PROJECT: Lee Dam — FEATURE: Left Abutment (Site 3) — LOCATION: LE
GRID REF — CO-ORD — DATUM
ANGLE FROM HORIZONTAL: 90° — DIRECTION — H.A.D. GROUND — H.A.D. COLLAR

DESCRIPTION OF CORE (WEATHERING, HARDNESS, STRENGTH, COLOUR. ROCK OR SOIL TYPE, DEFECT SPACING, LITHOLOGICAL FEATURES (bedding, foliation, minerology, texture, cement, etc...); STRATIGRAPHIC NAME.)	ROCK WEATHERING (SW MW HW)	ROCK HARDNESS (H MH MS S)	POINT LOAD TEST (MPa)	CORE LOSS /UFT % (5 10 50)	DEPTH H.A.D. (Core size, casing) m	GRAPHIC LOG	FRACTURE LOG (Spacing of natural fractures) (50 10 5 1 cm)	ROCK DEFECTS: PROMINENT JOINTS, BEDDING, SEAMS VEINS. SHATTER, SHEAR & CRUSH ZONES, FOLIATION SCHISTOSITY attitude, width, spacing, smoothness (OR SOIL DESCRIPTION) consistency, compactness, water content, group symbol etc...	DATE/DEPTH	R.O.D. %	WATER LEVEL (Date)	DRILL WATER LOSS % (0–100)	WATER PRESSURE TESTS – Lugeons ~~or PERMEABILITY 10^{-7} m/s~~ (0.01 0.1 1 10 100 1000)
RAI FORMATION, SANDSTONE, R2-R3 laminated, blocky interbedded shattered siltstone lenses, annealed sub V defects closely spaced. steeply dipping defects // & ⊥ to laminations. blue-grey & dark grey.					31			CORE LOSS crush zone with intact core between ... & ... long Shatter along ... fractures. B ~ 32°, SC-SW, T, CN		38%			0.3
SANDSTONE R4, laminated closely spaced bedding // defects					32			Drilling breaks & partially annealed defects.					
					33			E.O.H 32m					
					34								
					35								

Box 10

DRILLER: CW (KANE)
STARTED: 09-12-2008
FINISHED:
DRILL:

ROCK WEATHERING
UW – Unweathered
SW – Slightly Weathered
MW – Moderately Weathered
HW – Highly Weathered
CW – Completely Weathered

ROCK HARDNESS
VH – Very hard
H – Hard
MH – Moderately hard
MS – Moderately soft
S – Soft
VS – Very Soft

FRACTURE LOG
Spacing of Natural Fractures: 100 50 10 5 1 0.1
Fractures/m of core: 1 2 10 20 100 1000

EXPLANATION:

LOGGED: JKWH
DATE: 07/01/2009
TRACED:
CHECKED:
ORIGINAL VERTICAL SCALE: 1:50 @ A3
SHEET 3 OF 3

PROJECT: 24727-901
HOLE NO: DH7
LENGTH: 32
CORE BOXES: 10
DRG NO

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH 8

PROJECT: LEE DAM
FEATURE: RIGHT ABUTMENT (Site 3)
LOCATION: LEE RIVER VALLEY
GRID REF: ...
CO-ORD: ...
DATUM: ...
ANGLE FROM HORIZONTAL: 90°
DIRECTION: ...
H.A.D. GROUND: ...
H.A.D. COLLAR: ...

DESCRIPTION OF CORE (WEATHERING, HARDNESS, STRENGTH, COLOUR. ROCK OR SOIL TYPE, DEFECT SPACING, LITHOLOGICAL FEATURES (bedding, foliation, minerology, texture, cement, etc...): STRATIGRAPHIC NAME.)	DEPTH H.A.D. (m)	ROCK DEFECTS: PROMINENT JOINTS, BEDDING, SEAMS VEINS. SHATTER, SHEAR & CRUSH ZONES, FOLIATION SCHISTOSITY attitude, width, spacing, smoothness (OR SOIL DESCRIPTION) consistency, compactness, water content, group symbol etc...	WATER PRESSURE TESTS – Lugeons
SCREE, HW, loose, grey-brown, angular, platey, fine to medium (large cobble in outcrop) in a non plastic, silty sand matrix	0.0–3.0	Washed Core	
Lost Core	3.0	Lost Core	
FLUVIAL GRAVEL		GM	
LOST CORE		LOST CORE	
RAI FORMATION, Interbedded siltstone & fine Sandstone, thickly laminated, R3–R2, closely spaced bedding // & ⊥ joint sets	4.0	Many drill breaks core probably dilated in this zone, very disturbed core.	
More competent R3	5.0–6.0	B~42°, T, SC-SW, silt - Fe st. J~30°, O, SC-PL, silt (3mm) B~60, VT, SC, silt	
As above R3 some qz annealed micro fracturing	7.0	Many drilling breaks. J~20°, SC-SM, T, silt - Fe st B~22° SW-SC, T; silt - cv	Test 1: 0.044
Core Loss	8.0	Core Loss	
As Above.	9.0	B? 42°, SW, SC, T-O, silt (1-2mm)	Test 2: 0.19
Core Loss	10	Core Loss	
As above closely spaced natural defects apparent R3 – R4		Many drilling breaks along microfractures. SZ~39°, SW-PL, T-O, silt - cv	
SILT GOUGE	11	SANDY SILT, with trace clay, slight plasticity	
		B~28°, SW-SC, T, cv, Fe st J~22°, SW-SC, T, cv-silt	
Core Loss		Core Loss	
SILTSTONE / fine SANDSTONE, R4. thickly laminated, closely spaced, moderately steep to steeply dipping defects, 3 main sets.	12–13	J~72°, SC-ST, VT, trace silt B~52°, SW-SC, T-VT, trace silt J~30°, SW-SC, T, trace silt common joint B~48°, SW-SC, T, silt (5mm)	Test 3: 0.03
	14	Drill induced shatter recovered as GP.	
becomes more siltstone dominant, many micro fractures R3 - R4	15	Partially annealed shatter zone B~40°, SW-SC, T-VT, silt. Many drilling breaks.	

Box 1; Box 2; Box 3; Box 4 (5-2)

DATE/DEPTH: 20/01/09; 13:30 ... 8.1m; 14:30 hrs 21/01 13.0m

R.O.D. % (core recovered / run): N/A; N/A; 5/700; 0/350; 0/700; 120/600; 355/600; 300/1050; 330/600; 50/... ; 140/350; 55%; 710/1300; 220/1050; 280/1400; 300/1400; 0/700; 36%; 360/1400; 300/1400; 255/... ; 185/400; 530/1100

ROCK WEATHERING: UW – Unweathered; SW – Slightly Weathered; MW – Moderately Weathered; HW – Highly Weathered; CW – Completely Weathered

ROCK HARDNESS: VH – Very hard; H – Hard; MH – Moderately hard; MS – Moderately soft; S – Soft; VS – Very Soft

FRACTURE LOG: Spacing of Natural Fractures (100, 50, 10, 5, 1, 0.1); Fractures/m of core (1, 2, 10, 20, 100, 1000)

DRILLER: CW (James)
STARTED: 20/01/2009
FINISHED: 29/01/2009
DRILL: MAGGOT

EXPLANATION:

LOGGED: JXWN
DATE: 20-01-2009
TRACED: ...
CHECKED: ...
ORIGINAL VERTICAL SCALE: 1:50 @ A3
PROJECT: 24727.901
HOLE NO: DH8
LENGTH: 40
CORE BOXES: 12
SHEET 1 OF 3
DRG NO ...

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH8

PROJECT: Lee Dam — FEATURE: Right Abutment — LOCATION: Lee River Valley

GRID REF. ... CO-ORD. ... DATUM ...

ANGLE FROM HORIZONTAL: 90° — DIRECTION ... H.A.D. GROUND ... H.A.D. COLLAR ...

DESCRIPTION OF CORE (weathering, hardness, strength, colour, rock or soil type, defect spacing, lithological features (bedding, foliation, mineralogy, texture, cement, etc...); stratigraphic name)	Depth H.A.D. (m)	ROCK DEFECTS (prominent joints, bedding, seams, veins, shatter, shear & crush zones, foliation schistosity; attitude, width, spacing, smoothness) (OR SOIL DESCRIPTION) (consistency, compactness, water content, group symbol etc...)	R.O.D. %	Water Pressure Tests – Lugeons
Rai Formation, Interbedded SILTSTONE & fine SANDSTONE, R4, blue-grey, laminated, closely spaced, steeply dipping opposed defects, many drilling breaks along previously annealed defects	15–16.7	J ~ 58°, VT, SW – SC, CN J ~ 59°, T-VT, SW-SC, silt B ~ 60, T, SW – SM, silt. some drill shatter.	19%	Test 4
SHEAR ZONE R0 – R1	16.7	Fine – Med Sandy gravel		
As above but fresher & more shattered, closely – very closely spaced defects	16.7–18.5	Drilling breaks SPLIT CORE some shatter		0·21
As above but fresher and more shatter, closely to very closely spaced defects R3 – R4	18.5–20	J ~ 60°, T, SW – SC, silt. J – 40°, T-VT, SW-SC, CN – Fest B ~ 38°, T-O, SW – SC, silt – CN (2-3mm).	26%	Test 5 — 5
	20–21	Recovered as GP, coarse gravel Drilling breaks adjacent to steep Fest joint Thin Shear seam	30%	
Increasing limonite staining	21–22.7	Drilling breaks adjacent to steep Fest. joint	0%	Test 6 — 1
End of shatter	22.7			
Interbedded SILTSTONE & fine SANDSTONE, R4, blue grey with yellow limonite staining closely spaced, steeply dipping defects, thickly laminated some annealed defects.	22.7–24.5	B ~ 34°, SW-SC, T – VT, silt – Fest. D ~ 80°, PL-SM, VT, Fest B ~ 64°, SW-SC, VT, Fest Jts planar 55° UW Bedding Plane Shear Micro Fractures. V tight qtz vein – locally broken by drilling	30%	
As above improving R4 core quality, decreasing limonite staining, qz annealed defects. – Steeply inclined	24.5–27.5	B – 48°, SW-SC, T, silt. Open partly healed SW joint/qtz vein. B – 44°, SW-SC, T, Fest – CN J ~ 38°, SW-SC, T, silt	79% 80%	Test 7 — 5·9
As ABOVE but more shatter	27.5–29.3	J ~ 50°, SW-SC, T, silt Much drilling shatter B ~ 72°, WV, SC, T-O, silt – Fest J ~ 30-38° SW-SC, O, silt – Fest.	41%	Test 8 — 11
SHEARED ZONE 100mm rock R0 sticks then 100mm layers of fine gravel, ...	29.3–30	SHEARED ZONE	18%	

Driller's estimate 30% — Driller's estimate 40-45%

Box 4 — Box 5 — Box 6 — Box 7 — Box 8 — Box 9 30-2

DRILLER: CW (Josh)	ROCK WEATHERING UW – Unweathered SW – Slightly Weathered MW – Moderately Weathered HW – Highly Weathered CW – Completely Weathered	ROCK HARDNESS VH – Very hard H – Hard MH – Moderately hard MS – Moderately soft S – Soft VS – Very Soft	FRACTURE LOG Spacing of Natural Fractures: 100, 50, 10, 5, 1, 0.1 Fractures/m of core: 1, 2, 10, 20, 50, 100	LOGGED: ... DATE: 20/01... TRACED: ... CHECKED: ... ORIGINAL VERTICAL SCALE: 1:50 @ A3
STARTED: 20/01/09 FINISHED: 29/01/09 DRILL: MAGGOT	EXPLANATION:			PROJECT: 24727.901 HOLE NO: DH8 LENGTH: 40 CORE BOXES: 12

SHEET 2 OF 3 — DRG NO ...

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH 8

PROJECT 24727-901 LEE DAM ...... FEATURE UPPER RIGHT ABUTMENT ...... LOCATION LEE RIVER VALLEY

GRID REF. .... CO-ORD. .... DATUM ....

ANGLE FROM HORIZONTAL 90° ...... DIRECTION .... H.A.D. GROUND .... H.A.D. COLLAR ....

DESCRIPTION OF CORE (weathering, hardness, strength, colour. Rock or soil type, defect spacing, lithological features (bedding, foliation, mineralogy, texture, cement, etc...); stratigraphic name.)	DEPTH H.A.D. (m)	ROCK DEFECTS (prominent joints, bedding, seams, veins. Shatter, shear & crush zones, foliation schistosity; attitude, width, spacing, smoothness (OR SOIL DESCRIPTION) consistency, compactness, water content, group symbol etc...)	DATE/DEPTH	R.O.D. %	WATER PRESSURE TESTS – Lugeons or PERMEABILITY – 10^{-7} m/s
BEDDING PLANE SHEAR R1		S2		0/100	Box 9
END OF SHEAR ZONE R1-R2					
CORE LOSS		CORE LOSS		34%	Test 9
RAI FORMATION, Interbedded, SILTSTONE & fine SANDSTONE, R4, blue-grey, very thinly bedded, thickly laminated, steeply dipping, closely spaced defects.	31	B ~ 29°, SW-SC, T, silt-minor FeSt		305/1000	9
		J ~ 64°, SW-ST, T-VT, silt		26%	
	32	Many drilling breaks			
		B ~ 50°, SW-ST, T, FeSt-CN			
Quartz annealed shatter				79%	
As above but more competent some bedding plane shears ≤ 10mm th @ approx 34.4m R4.	33	J ~ 60°, SW-SC, VT, silt.		790/1000	Box 10
		B ~ 38°, SW-SC, VT-C, silt-CN			Test 10
Fresh rock.	34	Many drilling breaks in fresh rock.		64%	0.03
		B ~ 39°, SW-SC, C, CN		1440/2100	
Argillaceous SILTSTONE	35	BS2 ~ 49°, WV-SC, T-VT, silt			
RAI FORMATION SILTSTONE, R4, Thickly laminated, dark blue-grey closely spaced, steeply dipping predominantly bedding // defects some micro fracturing, much drilling disturbance. 5 to 8mm calcite bands @ 36m & 36.5m.		B ~ 42°, WV-SC, VT-C, silt		61% 490/800	
	36	calcite band ~ 18° sub // to bedding		97%	Box 11
		B ~ 62°, SW-SC, VT-C		580/600	Test 11
	37	Many drilling breaks.	28/01/09 33.5m	50% 500/1000	0.06
		J ~ 40°, SC-ST, C, ca-annealed			
		B ~ 60°, SW-SC, VT-C, "	29/01 36.2m	67% 400/600	
	38	Many drilling breaks			Box 12
		E.O.H 38.1m			
	39				
	40				

Column headings also on form: ROCK WEATHERING (SW MW HW); ROCK HARDNESS (H MH MS S); POINT LOAD TEST (MPa); CORE LOSS /LIFT %; Core size, casing; GRAPHIC LOG; FRACTURE LOG (Spacing of natural fractures) cm; WATER LEVEL Date; DRILL WATER LOSS % 0–100.

DRILLER: CN (Josh)	ROCK WEATHERING	ROCK HARDNESS	FRACTURE LOG	LOGGED: JXMW	PROJECT: 24727-901
STARTED:	UW – Unweathered	VH – Very hard	Spacing of Natural Fractures (cm): 100 50 10 5 1	DATE: 29/01/09	HOLE NO: DH 8
FINISHED: 29/01/2009	SW – Slightly Weathered	H – Hard		TRACED:	LENGTH: 40m
DRILL: Maggot	MW – Moderately Weathered	MH – Moderately hard	Fractures/m of core: 1 2 10 20 100 1000	CHECKED:	
	HW – Highly Weathered	MS – Moderately soft		ORIGINAL VERTICAL SCALE:	CORE BOXES: 12
	CW – Completely Weathered	S – Soft		SHEET 3 OF 3	DRG NO
		VS – Very Soft			

EXPLANATION:

TONKIN & TAYLOR LTD
CONSULTING ENGINEERS

DRILL HOLE LOG

HOLE No. DH9

PROJECT: Lee Dam FEATURE: Lower Right Abutment LOCATION: Lee River Valley
GRID REF. .... CO-ORD. .... DATUM ....
ANGLE FROM HORIZONTAL .... DIRECTION .... H.A.D. GROUND .... H.A.D. COLLAR ....

Depth (m)	DESCRIPTION OF CORE (weathering, hardness, strength, colour, rock or soil type, defect spacing, lithological features (bedding, foliation, mineralogy, texture, cement, etc...); stratigraphic name)	ROCK DEFECTS (prominent joints, bedding, seams, veins, shatter, shear & crush zones, foliation schistosity, attitude, width, spacing, smoothness) (OR SOIL DESCRIPTION)	R.O.D. %	WATER PRESSURE TESTS – Lugeons or PERMEABILITY 10^{-7} m/s
0–3	No Core. Drilling into Scree & Dilated rock			
3–3.6	Core Loss	Core Loss	17%	
3.6–5.4	Rai Formation Siltstone, R3, laminated, closely spaced moderately steep to steeply dipping defects, light-bluegrey with qz & calcite veining & discreet bedding plane shears between 10mm & 150mm thick	S2/J ~ 18°, SW-PL, O, silt gouge S2 ~ 60°, SW-SC, O, silt gouge J ~ 50, SW-SC, O, silt B ~ 50, SW-SC, VT, CN	33%	Test 1: 5
5.4	SHEARED ROCK	SHEARED ROCK		
5.4–9	Above becomes more competent R4. no bedding plane shears to 9.0m intact sticks up to 300 mm.	B ~ 30°, SW-SC, T-VT, silt J ~ 32°, SW-PL, VT, CN-silt B ~ 30°, SW-SC, VT-C, CN-silt Many drilling breaks J ~ 70°, SW-ST, T-VT, CN, Ca B ~ 40°, SW-SC, VT-T, Ca-CN	52%	Test 2: 1
9–11.7	As above, becoming less weathered occasional thin S(10mm) bedding plane shears	B(S2?) 64°, SW-SC, T, silt-Ca Drilling breaks B ~ 61°, SW-SC, T-VT, Ca-CN B ~ 44°, SW-SC, VT, silt	65%	Test 3: 0.1
11.7–15		B ~ 50°, SW-SC, T, silt J ~ 80, SW-ST, VT, silt B/S2 ~ 30, SW-SC, T-O, silt gouge B ~ 50, SW-SC, VT-C, silt, C (Drill breaks) Quartz band ~ 10°, PL-SC, T, silt gouge B ~ 58°, SW-SC, T, silt B ~ 64°, SW-SC, T, silt	64%	Test 4: 0.1

Core loss column: 42% (3–4 m); Flush return ~ 100%

Water level: 1.6m 05/02/09

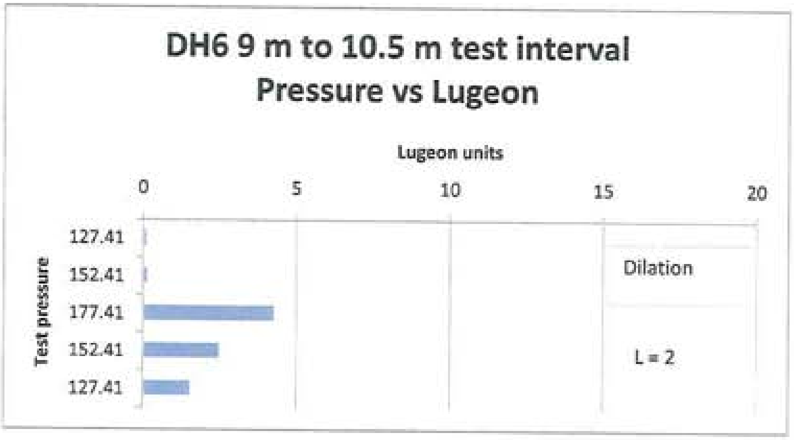
Box 2 (9 m), Box 3 (11.7 m), Box 4 (14.3 m), Box 5 (15 m)

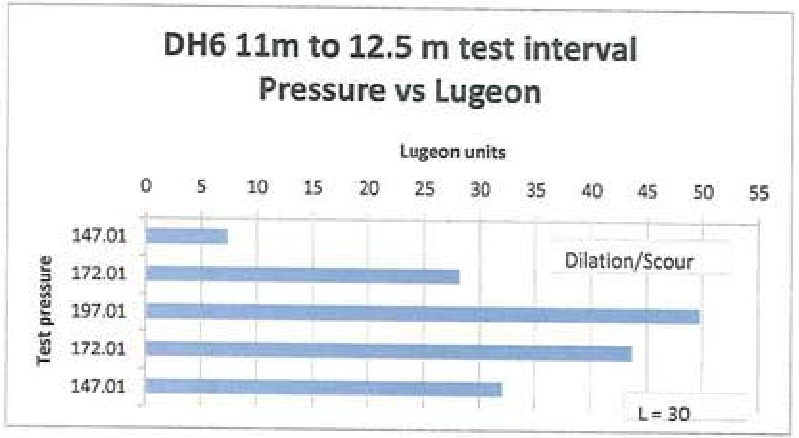
DRILLER: CW (Josh)	ROCK WEATHERING	ROCK HARDNESS	FRACTURE LOG	LOGGED: JXWW	PROJECT: 24723-901
STARTED: 30/01/2009	UW – Unweathered	VH – Very hard	Spacing of Natural Fractures (cm): 100 50 10 5 1 0.1	DATE: 05/02/2009	HOLE NO: DH 9
FINISHED: 05/02/2009	SW – Slightly Weathered	H – Hard	Fractures/m of core: 1 2 10 20 100 1000	TRACED:	LENGTH: 15m
DRILL: Maggot	MW – Moderately Weathered	MH – Moderately hard		CHECKED:	
	HW – Highly Weathered	MS – Moderately soft		ORIGINAL VERTICAL SCALE: 1:50 @ A3	CORE BOXES: 4
	CW – Completely Weathered	S – Soft		SHEET 1 OF 1	DRG NO
		VS – Very Soft			

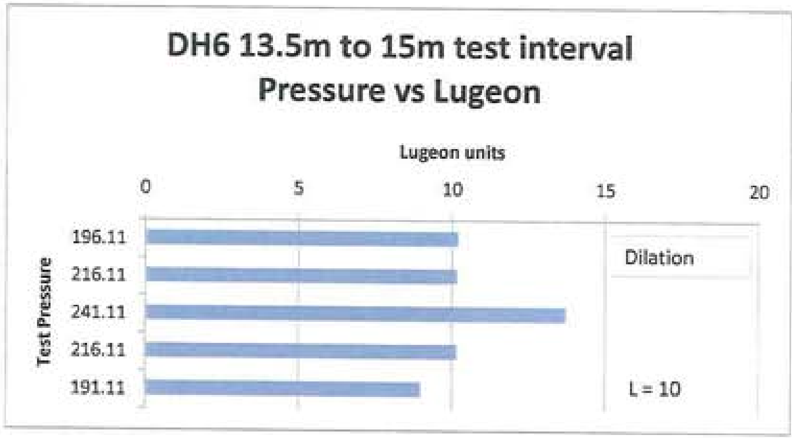
EXPLANATION:

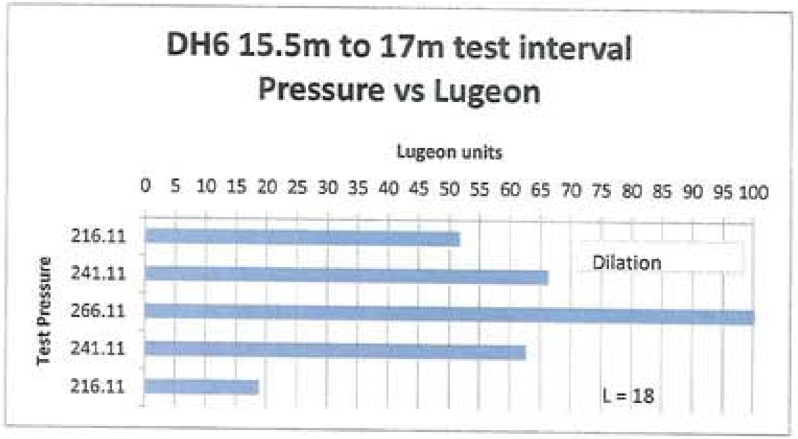
Appendix C: Packer and Permeability Tests

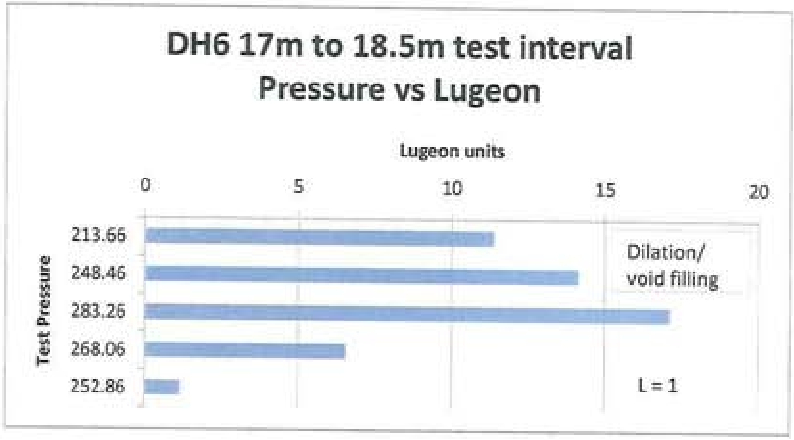
Lee Dam Investigation: Drill hole 6 Lugeon testing

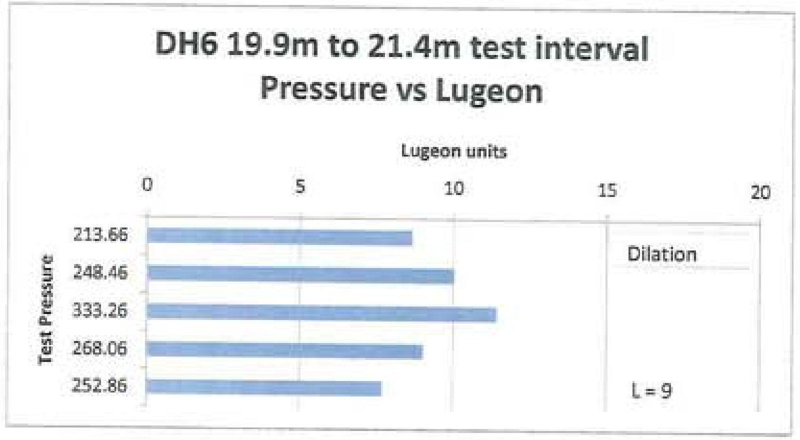


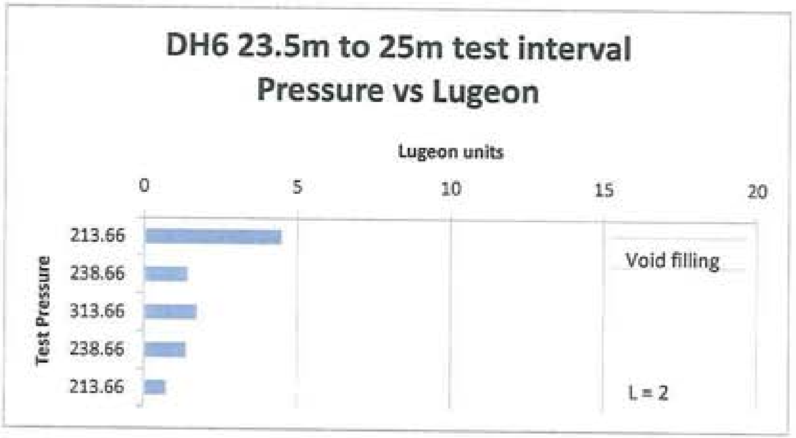


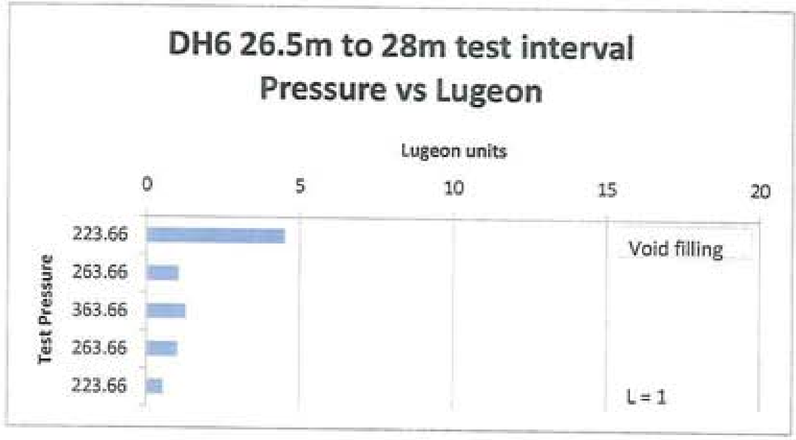


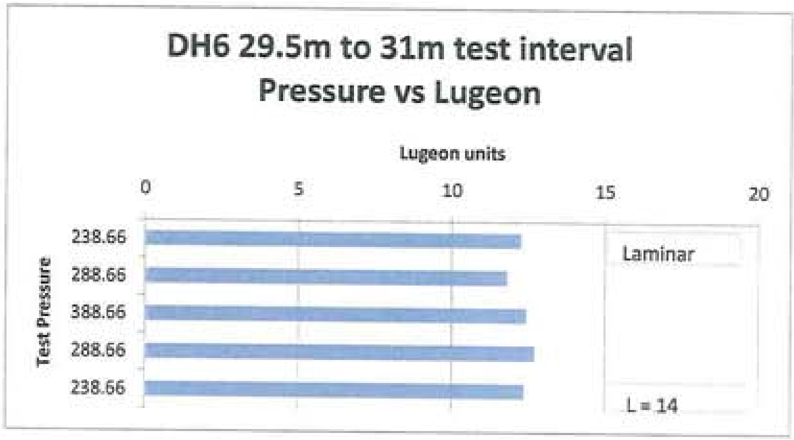


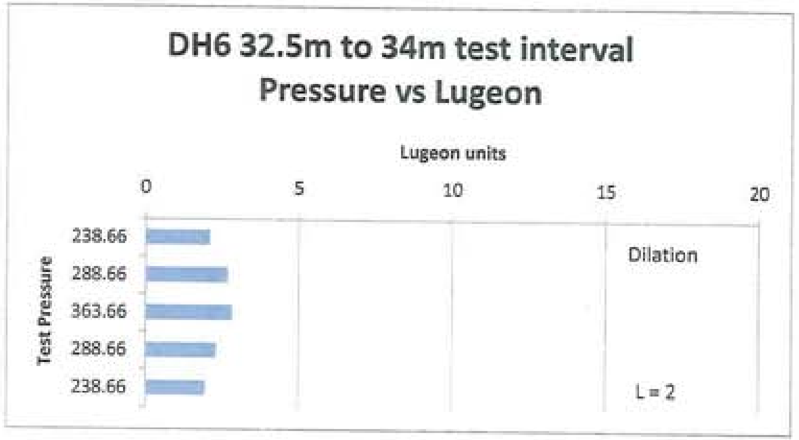

DH6 17m to 18.5m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
213.66
248.46
283.26
268.06
252.86
Dilation/
void filling
L = 1


DH6 19.9m to 21.4m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
213.66
248.46
333.26
268.06
252.86
Dilation
L = 9

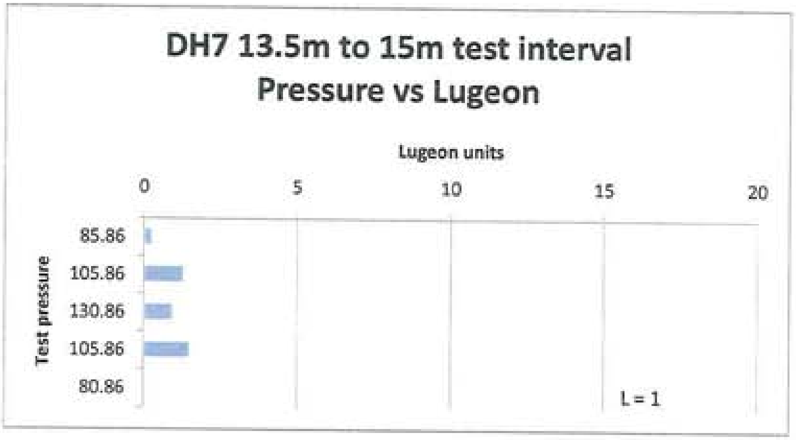

DH6 23.5m to 25m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
213.66
238.66
313.66
238.66
213.66
Void filling
L = 2

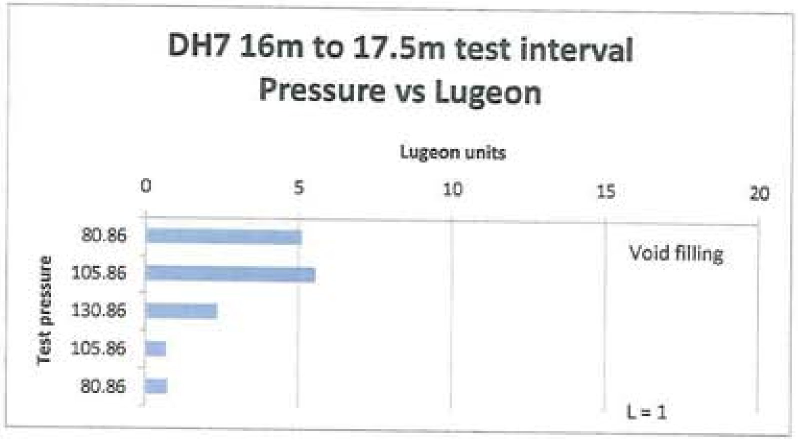

DH6 26.5m to 28m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
223.66
263.66
363.66
263.66
223.66
Void filling
L = 1

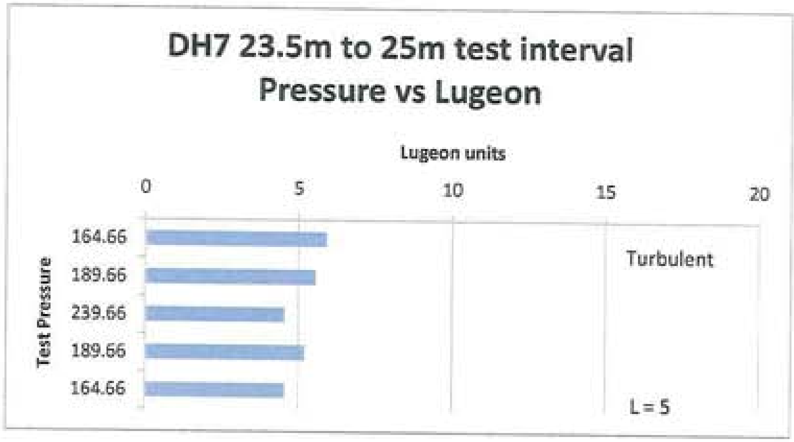

DH6 29.5m to 31m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
238.66
288.66
388.66
288.66
238.66
Laminar
L = 14


DH6 32.5m to 34m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
238.66
288.66
363.66
288.66
238.66
Dilation
L = 2

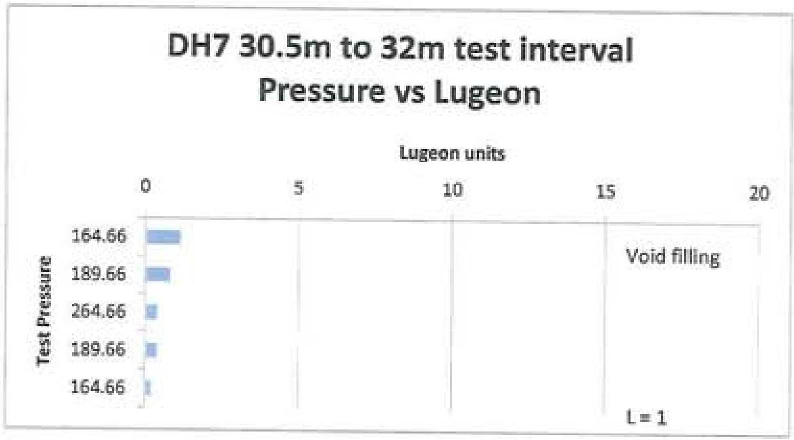
Lee Dam Investigation: Drill hole 7 Lugeon testing



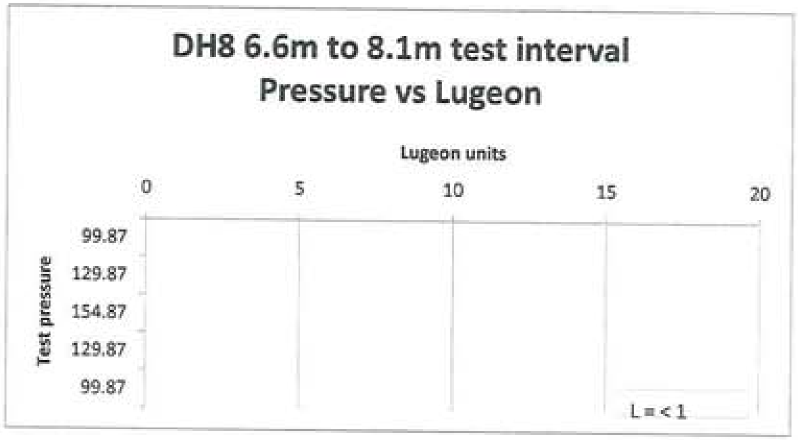


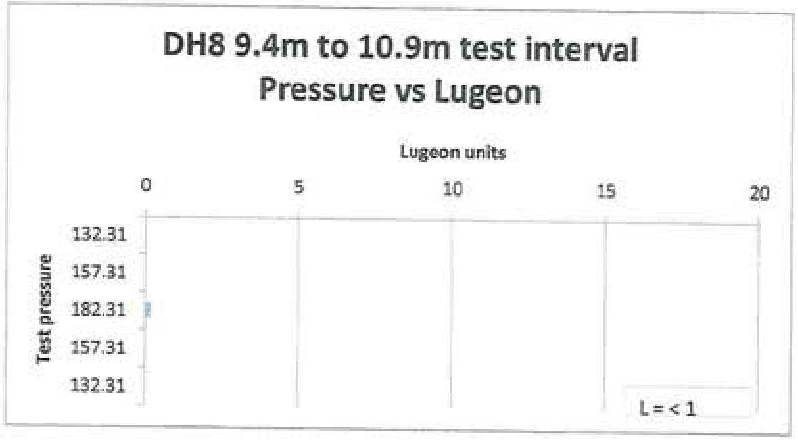

DH7 23.5m to 25m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
164.66
189.66
239.66
189.66
164.66
Turbulent
L = 5

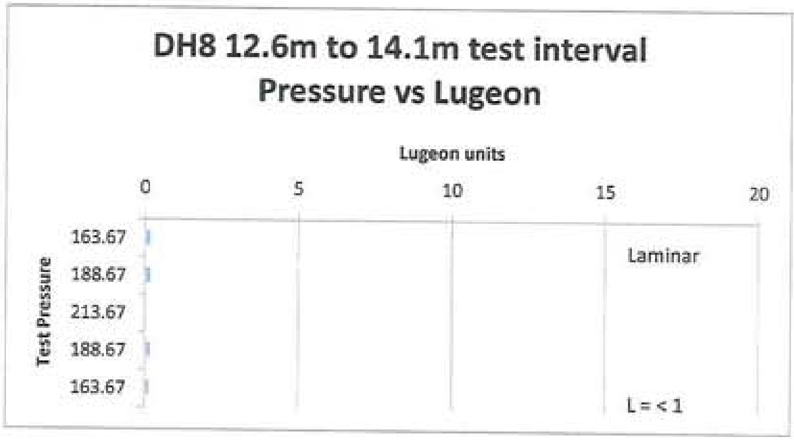

DH7 27.5m to 29m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
164.66
189.66
264.66
189.66
164.66
Void filling
L = 1

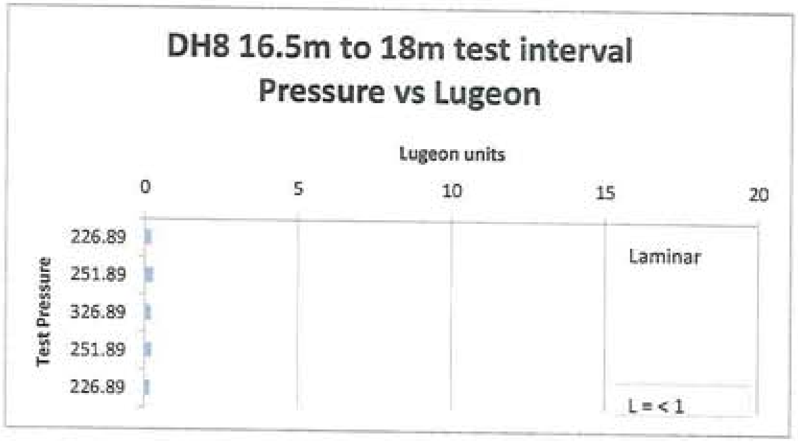

DH7 30.5m to 32m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
164.66
189.66
264.66
189.66
164.66
Void filling
L = 1

Lee Dam Investigation: Drill hole 8 Lugeon testing



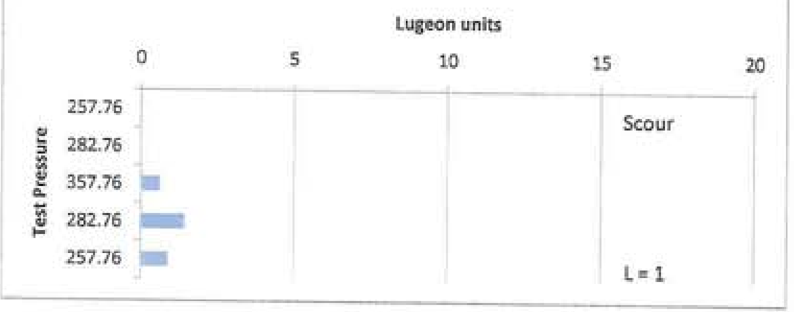




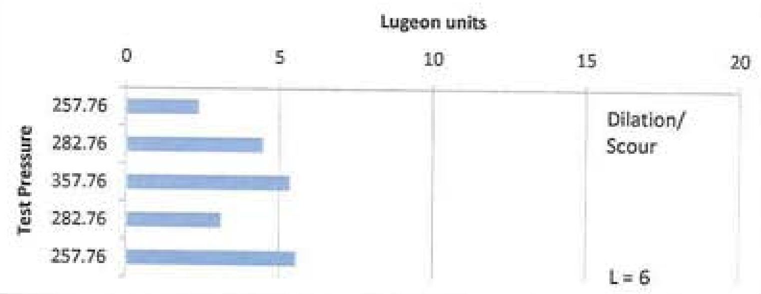


DH8 18.5m to 20m test interval
Pressure vs Lugeon

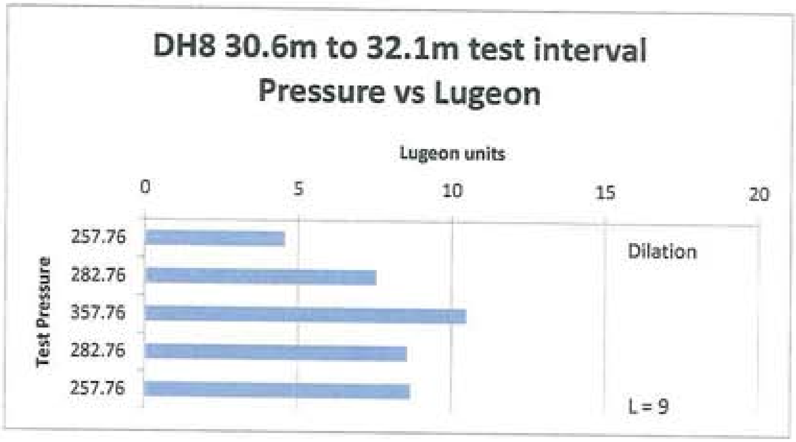
DH8 21.6m to 23.1m test interval
Pressure vs Lugeon

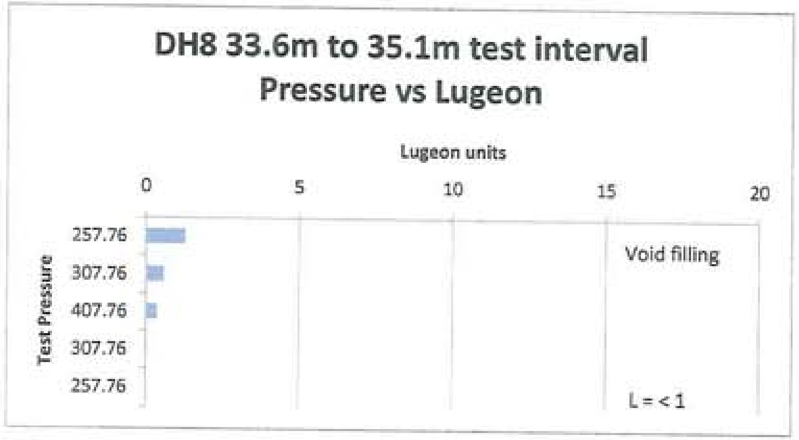


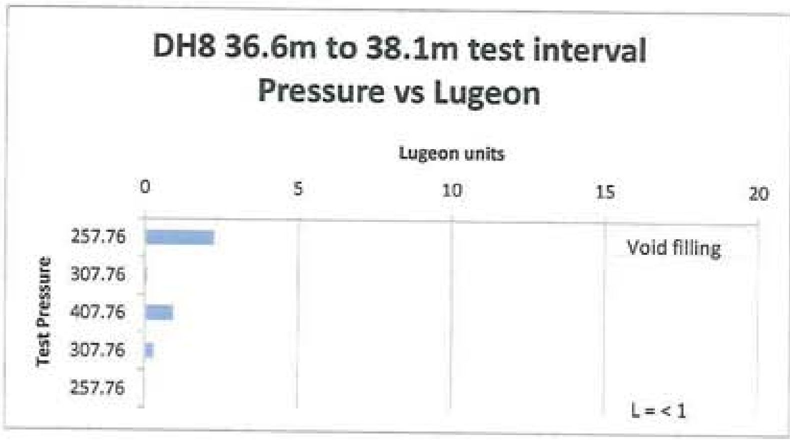
DH8 24.6m to 26.1m test interval
Pressure vs Lugeon



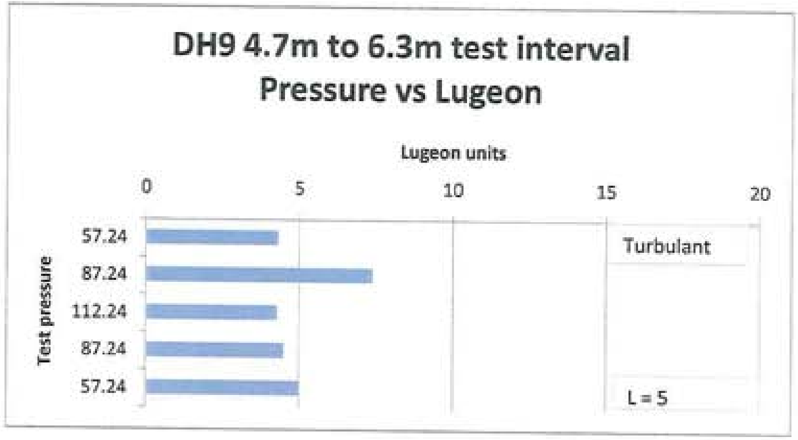
DH8 27.6m to 29.1m test interval
Pressure vs Lugeon

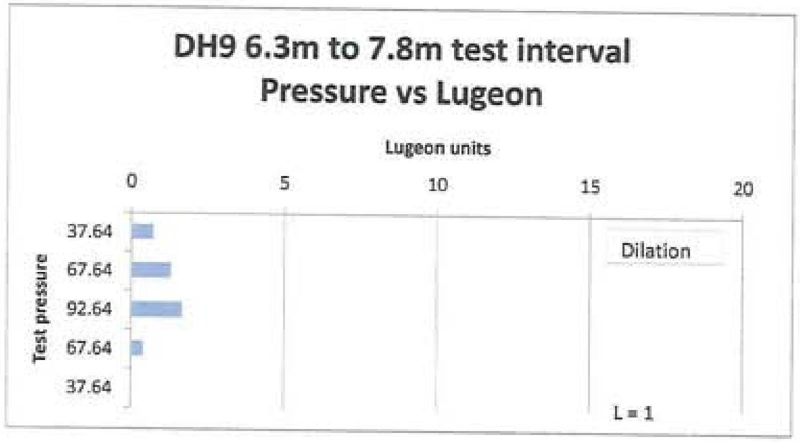

DH8 30.6m to 32.1m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
257.76
282.76
357.76
282.76
257.76
Dilation
L = 9

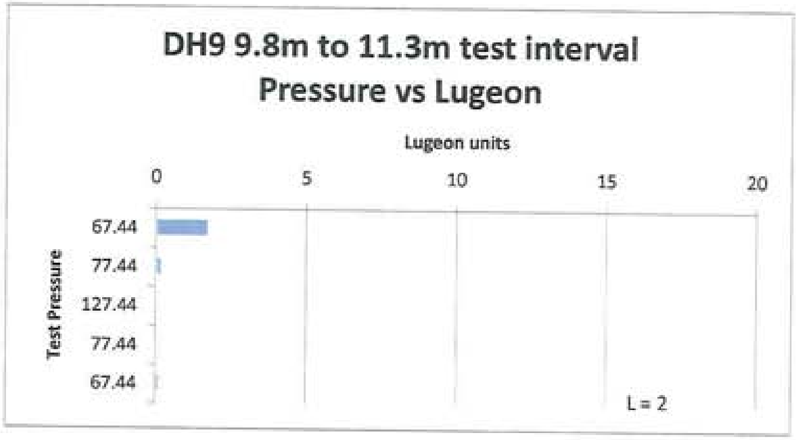

DH8 33.6m to 35.1m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
257.76
307.76
407.76
307.76
257.76
Void filling
L = < 1

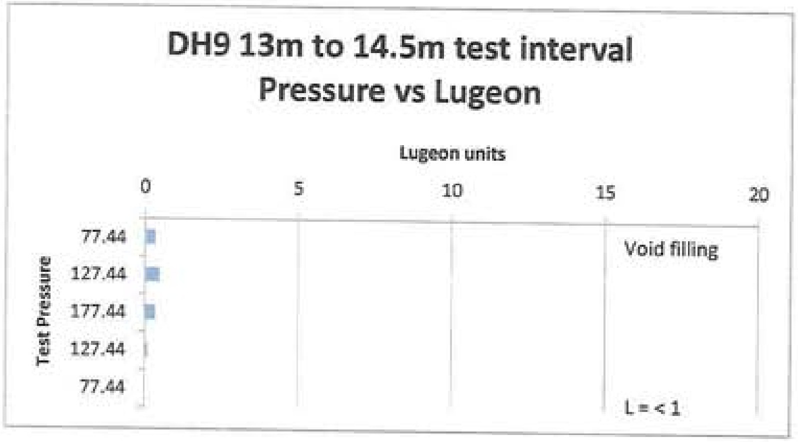

DH8 36.6m to 38.1m test interval
Pressure vs Lugeon
Lugeon units
0
5
10
15
20
Test Pressure
257.76
307.76
407.76
307.76
257.76
Void filling
L = < 1

Lee Dam Investigation: Drill hole 9 Lugeon testing









Appendix D: Test Pit and Excavation Logs

TONKIN & TAYLOR LTD.

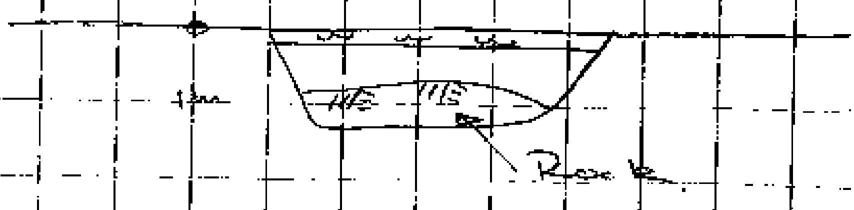
EXCAVATION LOG

EXCAVATION NO: TP 1
SHEET 1 OF 1

PROJECT: [illegible] Dam
LOCATION: [illegible] left abutment
JOB NO: 24727.901
CO-ORDINATES: [illegible] 24727 20456
EXPOSURE TYPE:
HOLE STARTED: 17.7.2008
EQUIPMENT:
HOLE FINISHED: 17.7.2008
RL:
OPERATOR:
LOGGED BY: [illegible]
DATUM:
EXCAVATION DIMENSIONS:
CHECKED BY: MRF

EXCAVATION AND TESTS						ENGINEERING DESCRIPTION						GEOLOGICAL	
PENETRATION 1 2 3	SUPPORT	WATER	SAMPLES, TESTS PP = Pocket Penetrometer q_u kPa SV = Shear Vane Corrected kPa	RL (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	SOIL NAME, PLASTICITY OR PARTICLE SIZE CHARACTERISTICS, COLOUR, SECONDARY AND MINOR COMPONENTS	MOISTURE CONDITION	SHEAR STRENGTH OR RELATIVE DENSITY	ESTIMATED SHEAR STRENGTH kPa	ORIGIN TYPE, MINERAL COMPOSITION, DEFECTS, STRUCTURE	UNIT
							OL	SILT, non plastic, organic brown, with some fine gravel	M	L		Topsoil	
							GM GC	silty GRAVEL with some clay and sand, orange brown, some boulders	M	F, St		ALLUVIUM	
					1			SILTSTONE, greenish grey SW-MW R3-R4, closely spaced steep defects 134/47S 228/41 NW 260/86N 216/57 NW				GREYWACKE RAI FORMATION	

SKETCH



TONKIN & TAYLOR LTD.

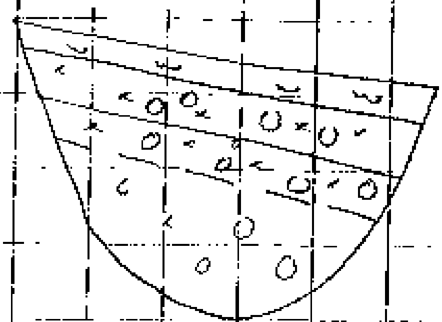
EXCAVATION LOG

EXCAVATION NO:	TP2
SHEET	OF

PROJECT: Lee Dam	LOCATION: Anslow Creek		JOB NO: 24727.701
CO-ORDINATES: Refer Figure 24727.701-F6	EXPOSURE TYPE: Test pit	HOLE STARTED:	17.7.09
RL:	EQUIPMENT:	HOLE FINISHED:	17.7.09
DATUM:	OPERATOR:	LOGGED BY:	JKWW
	EXCAVATION DIMENSIONS:	CHECKED BY:	MAF

PENETRATION 1 2 3	SUPPORT	WATER	SAMPLES, TESTS (PP = Pocket Penetrometer q_u kPa; SV = Shear Vane Corrected kPa)	RL (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	SOIL NAME, PLASTICITY OR PARTICLE SIZE CHARACTERISTICS, COLOUR, SECONDARY AND MINOR COMPONENTS	MOISTURE CONDITION	SHEAR STRENGTH OR RELATIVE DENSITY	ESTIMATED SHEAR STRENGTH kPa	ORIGIN TYPE, MINERAL COMPOSITION, DEFECTS, STRUCTURE	UNIT
							OH	Silt, non plastic, brown, organic grading into LP clayey silt	M	L		Topsoil	
							GM / ML	gravelly SILT, angular, coarse gravel clasts in low plastic sandy silt matrix, rare clay	M	F / St		Colluvium	
					1		GM / GW	Silty GRAVEL, fine to cobble size, with clayey silt matrix, some sand, gravel HW-MW	M	MD		Alluvium	
	1/2	inflow below 1.5m	Pit sides caving in below 1.5m										
					2		GM / GW	- Change to bouldery GRAVEL, brownish grey, with some silt and sand	W	MD / D			
					3								
					4			EOP at 4.0m, many large boulders, hard digging					

SKETCH



TONKIN & TAYLOR LTD.

EXCAVATION LOG

EXCAVATION NO: TP 3
SHEET 1 OF 1

PROJECT: Lee Dam | LOCATION: 12600 - Left bank | JOB NO: 24727.901

CO-ORDINATES: Refer Figure 24727.20k-F6

RL:

DATUM:

EXPOSURE TYPE: Test Pit

EQUIPMENT: 20T

OPERATOR: TAYLORS

EXCAVATION DIMENSIONS: 6m x 1m

HOLE STARTED: 8/10/08

HOLE FINISHED: 8/10/08

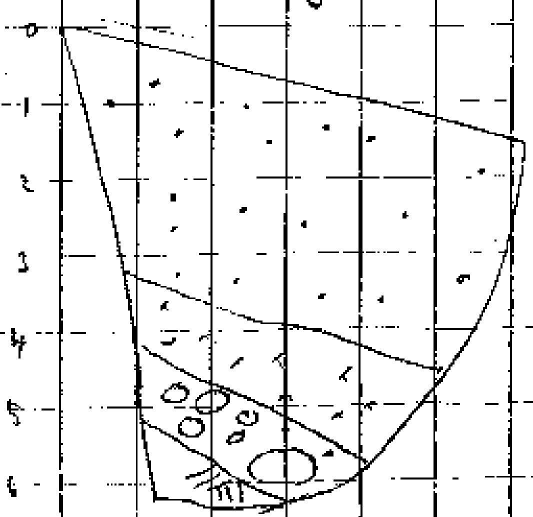
LOGGED BY: MRC

CHECKED BY:

DEPTH (m)	EXCAVATION AND TESTS: SUPPORT / WATER / SAMPLES, TESTS (PP – Pocket Penetrometer q_u kPa; SV – Shear Vane Corrected kPa)	CLASSIFICATION SYMBOL	SOIL NAME, PLASTICITY OR PARTICLE SIZE CHARACTERISTICS, COLOUR, SECONDARY AND MINOR COMPONENTS	MOISTURE CONDITION	SHEAR STRENGTH OR RELATIVE DENSITY	GEOLOGICAL: ORIGIN TYPE, MINERAL COMPOSITION, DEFECTS, STRUCTURE	UNIT
0 – 3.2		SP	SAND, Silty, non plastic, yellow, with some fine gravel. Sand, gravel particles are subangular, sourced greywacke	M	MD	Solifluction	
3.2 – 4.2	Nil / Nil	ML	SILT, with minor gravel, yellow brown low plasticity.	M	F	Alluvium (flood plain or lake)	
4.2 – 5.3	Minor wall collapse	GW	GRAVEL, coarse, with some boulders, and sand, grey	M	MD	Alluvium	
5.3 – 6.1			SANDSTONE – indurated, Moderately strong, MW	M		RAI FORMATION GREYWACKE	

End of Pit at 6.1m Hard digging

SKETCH



TONKIN & TAYLOR LTD.

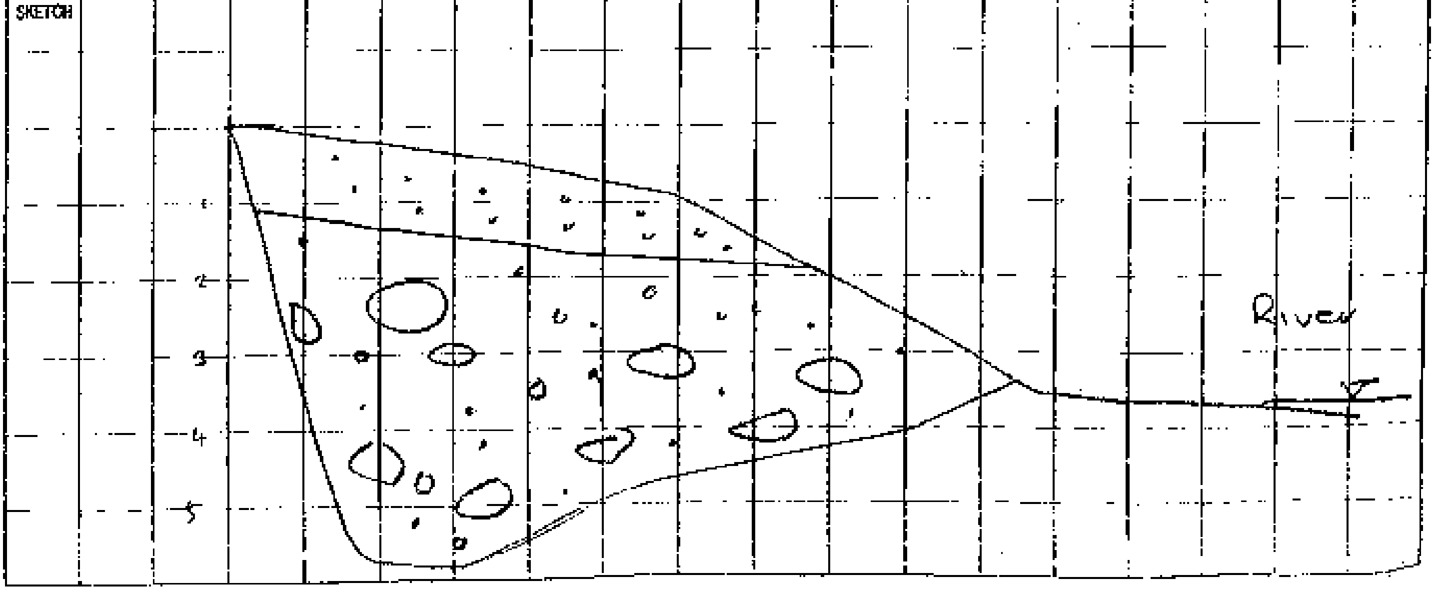
EXCAVATION LOG

EXCAVATION NO: TP 4.
SHEET 1 OF 1

PROJECT: Lee Dam.	LOCATION: Stn 12500 left bank	JOB NO: 24727.901
CO-ORDINATES: Refer Figure 24727.204-F6	EXPOSURE TYPE:	HOLE STARTED: 8/10/08
	EQUIPMENT: 20T.	HOLE FINISHED: 8/10/08
RL:	OPERATOR: Taylors	LOGGED BY: MRF
DATUM:	EXCAVATION DIMENSIONS: 6m x 1m.	CHECKED BY:

EXCAVATION AND TESTS: PENETRATION 1 2 3	SUPPORT	WATER	SAMPLES, TESTS (PP = Pocket Penetrometer q_u kPa; SV = Shear Vane Corrected kPa)	RL (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	ENGINEERING DESCRIPTION: SOIL NAME, PLASTICITY OR PARTICLE SIZE CHARACTERISTICS, COLOUR, SECONDARY AND MINOR COMPONENTS	MOISTURE CONDITION	SHEAR STRENGTH OR RELATIVE DENSITY	ESTIMATED SHEAR STRENGTH, kPa	GEOLOGICAL: ORIGIN TYPE, MINERAL COMPOSITION, DEFECTS, STRUCTURE	UNIT
					0–1		SP	SAND, medium, yellow with rare silt and fine gravel	M	L		Recent Alluvium.	
	Nil		Pit walls collapsing		1–5			GRAVEL, fine to coarse, with some sand and boulders, grey. gravel clasts sub rounded.		L – MD			
			Slight seepage below 4m		4								
								End of Pit at 5.7m. Hole Collapsing					

SKETCH



Tonkin & Taylor

Office:

Computed: MAR 6 2009.

Checked: 20

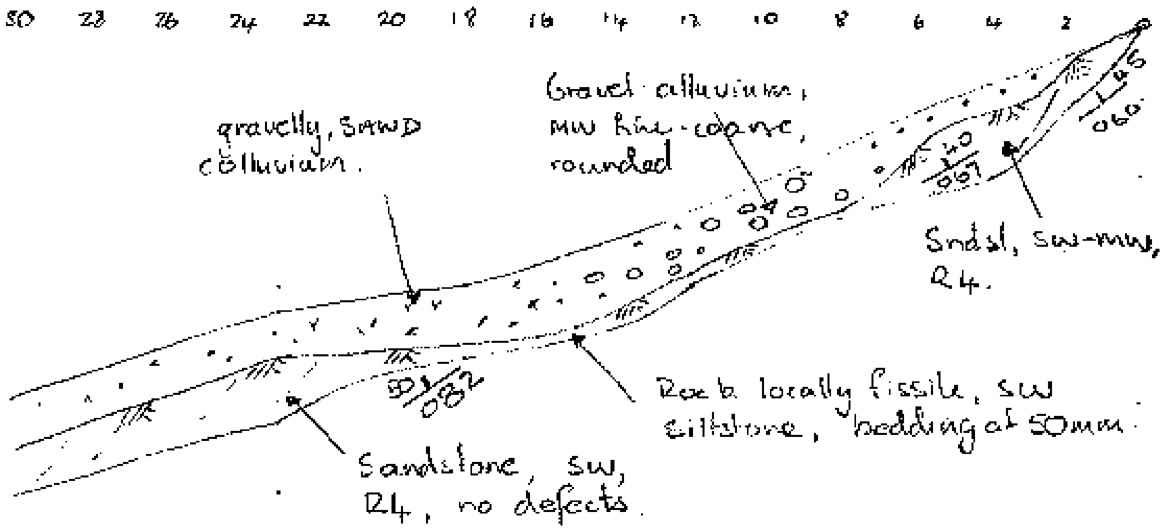
Job No: 24727.901 File: 24727.901

Project: LEE DAM

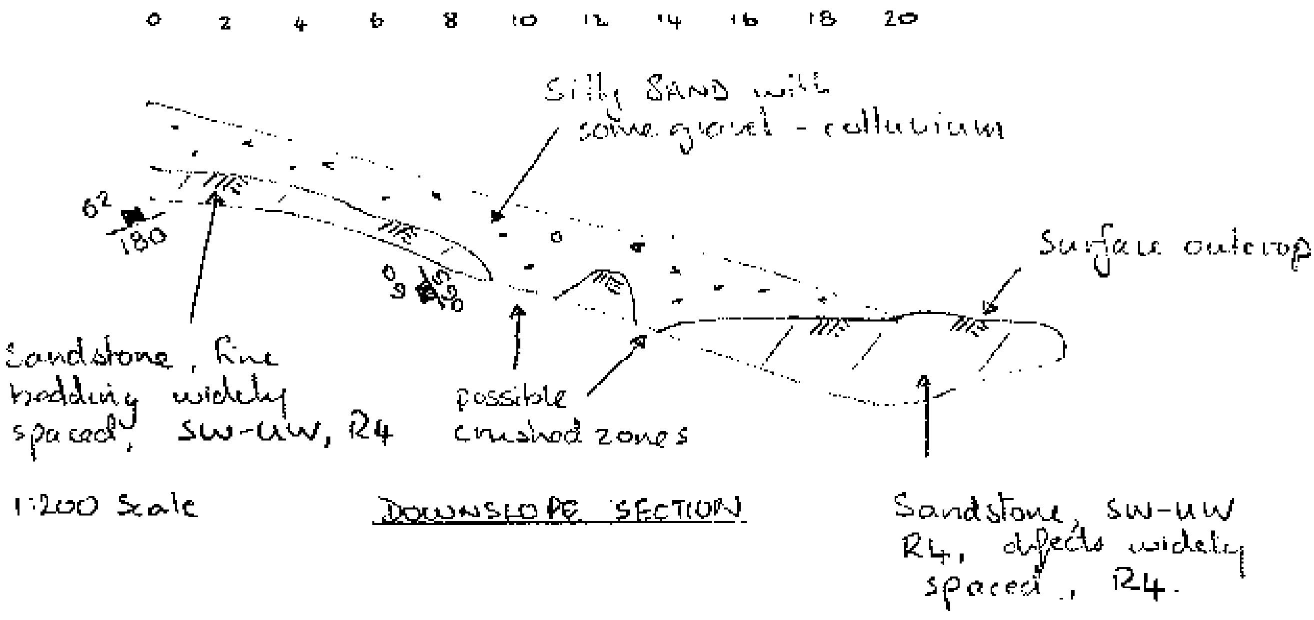
Sheet No: 1

Description: EX3.

Excavation Log EX3.

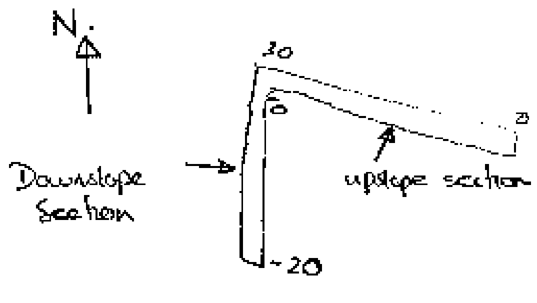


UPSLOPE SECTION 1:200 Scale



1:200 Scale DOWNSLOPE SECTION

SECTION VIEW OF TRACK BATTER

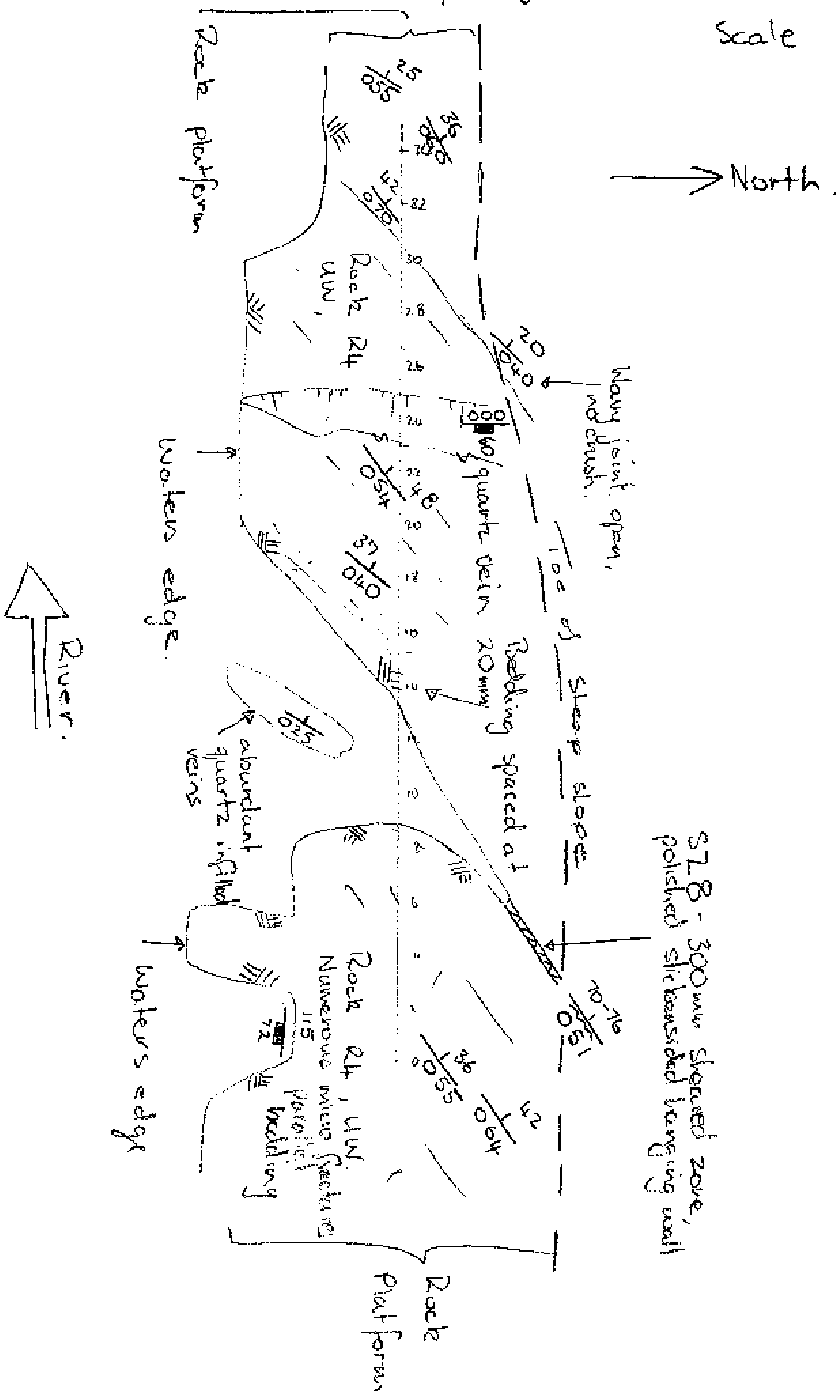


PLAN VIEW - NTS

Tonkin & Taylor		**Job No:**	**File:** 24727.901
Office:		**Project:**	
Computed:	MRF 7 2007	Exposure Log EX 4.	
Checked:	20	Refer Figure 24727.204-F6	

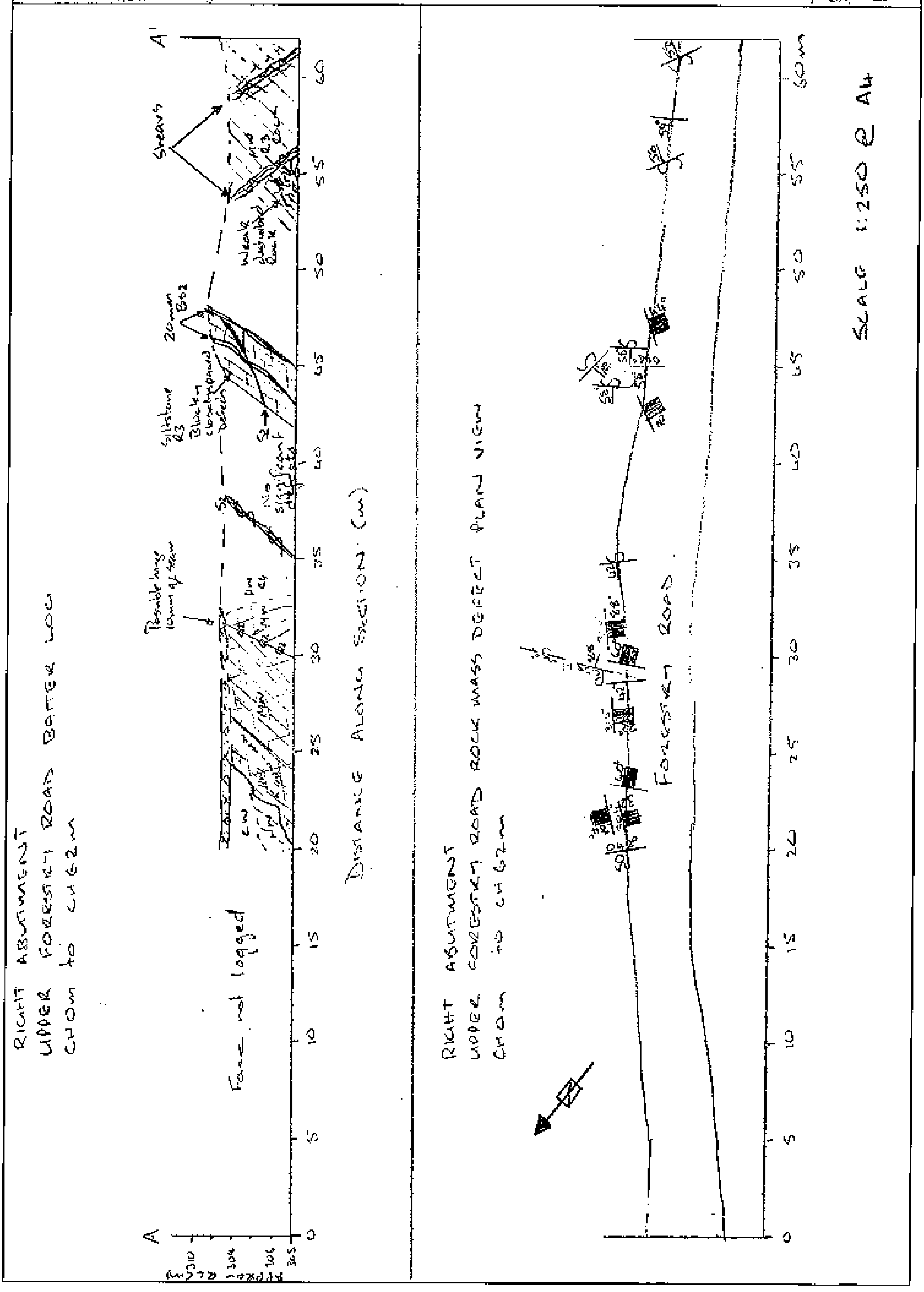
Sheet No: 1 of 1
Description:

Scale 1:200

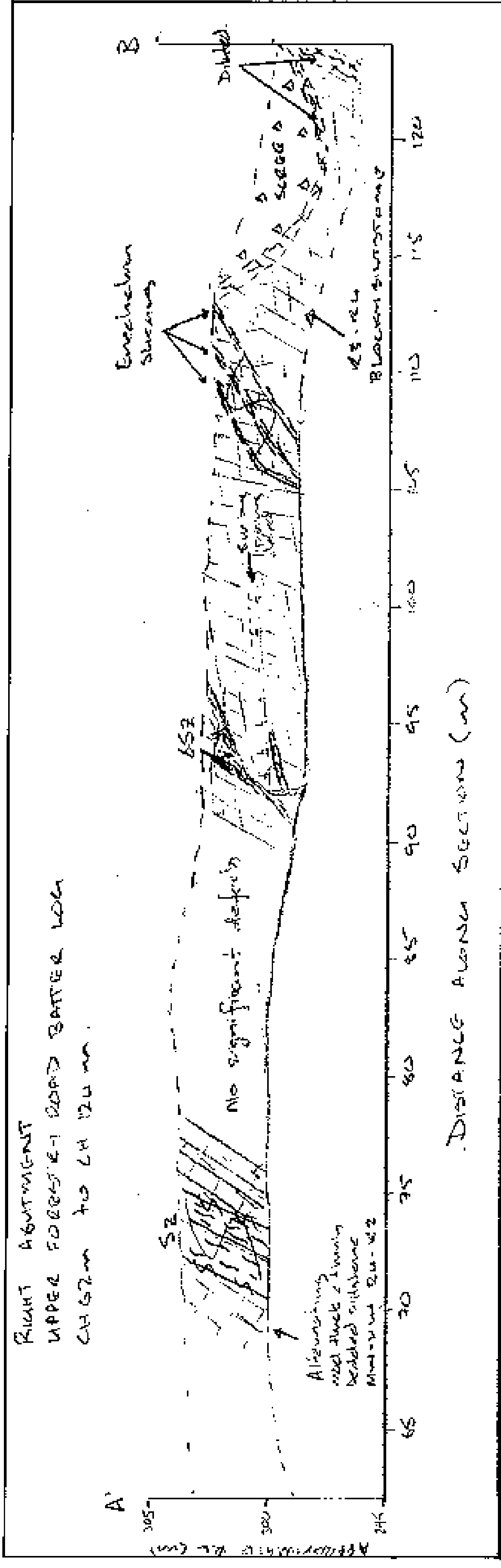


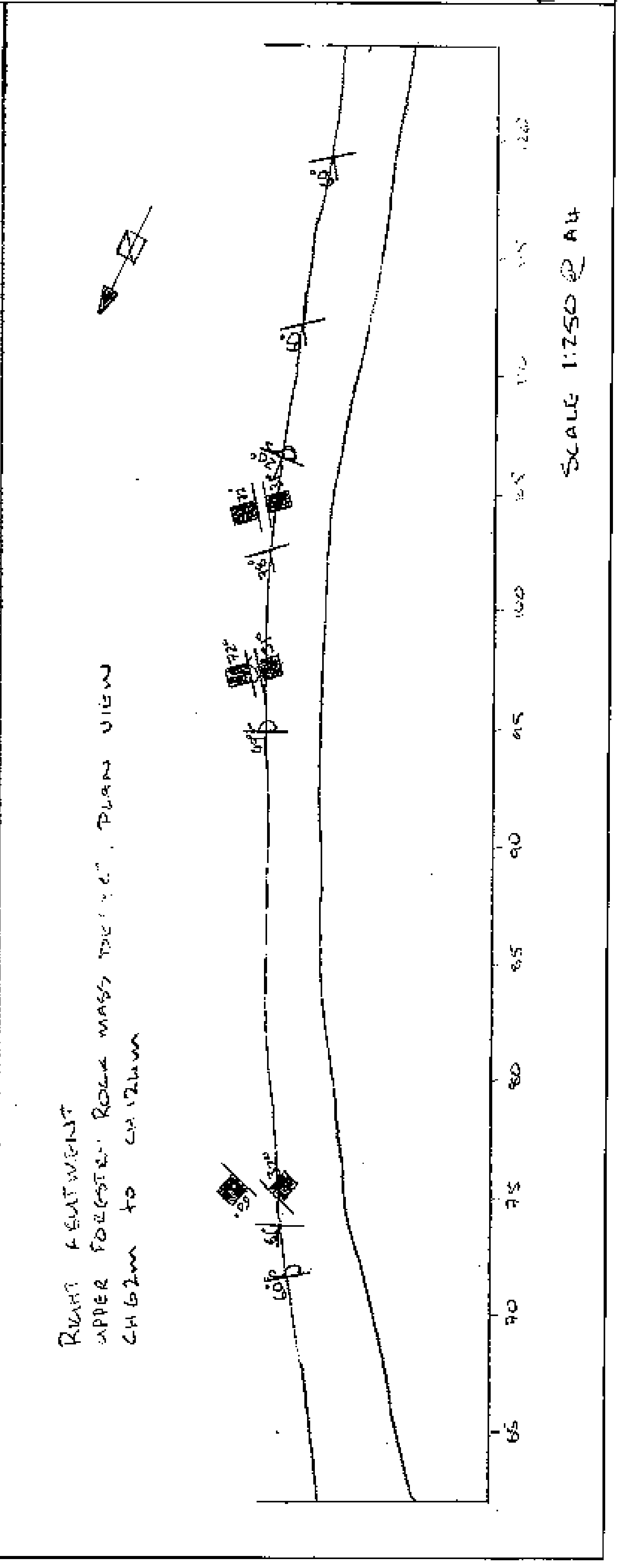
Tonkin & Taylor

Project: LEE RIVER DAM STAGE 4	Computed: JXWW 09/07/2009		Office: NELSON
	Checked:	20	Job No: 24727.200
Description: UPPER FORESTRY ROAD EXT. RIGHT ABUTMENT	Revised:	20	File:
	Checked:	20	Sheet No. 1 of 2



Tonkin & Taylor	Project: LEE RIVER DAM STAGE 4	Computed: [illegible] 13/07 2004		Office: NELSON
		Checked:	20	Job No: 20737.704
Description: UPPER FORESTRY ROAD EX7 RIGHT ABUTMENT		Revised:	20	File:
		Checked:	20	Sheet No. 2 of 2





Tonkin & Taylor

Office: NELSON
Computed: [illegible] 09/12 2008
Checked: 20

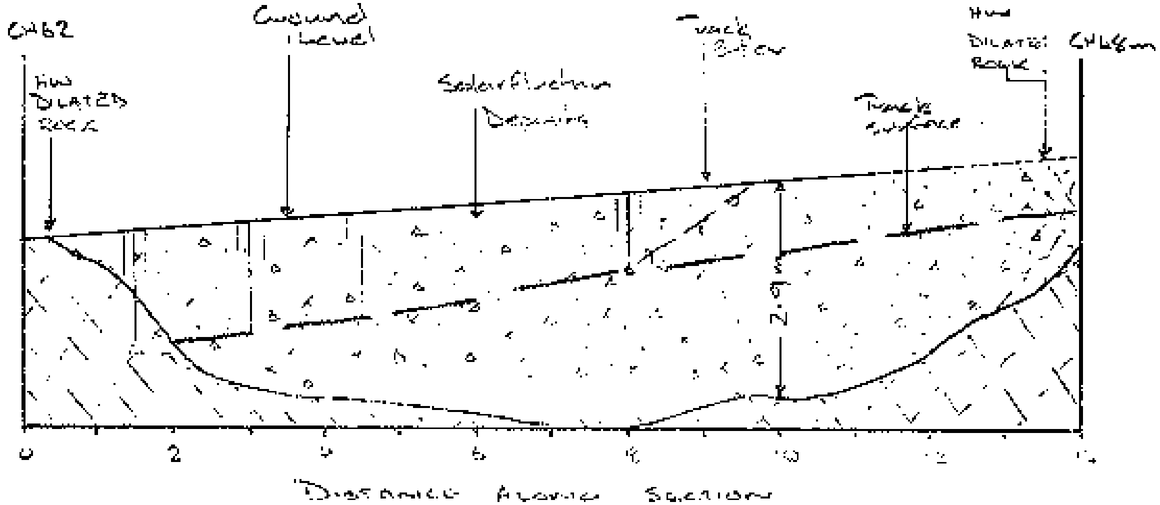
Job No: [illegible] **File:**
Project: [illegible]

Sheet No: 1 of 1
Description: Sketch

EXCAVATION LOG EX3 48m – 62m.

Approx coords:
2523353, 5970637

CROSS SECTION THROUGH CH48 TO CH62
ON LEFT ORIENTATION LOOKING UP TRACK



1:100

Notes: Rock is steeply dipping 50-65° to the northwest with an opposing southeast dipping joint set. Blocky, B2 to B3.

Rock varies in extent of weathering with improving spacing towards the centre of the section

No distinctive shear zones exposed in trench

CRP = Unable to Penetrate.

TONKIN & TAYLOR LTD.
ENVIRONMENTAL & ENGINEERING CONSULTANTS

ENGINEERING LOG TERMINOLOGY

DRILLING OR EXCAVATION

WATER

Water level on date shown

Water inflow

Water Outflow

CORE RECOVERY

Core recovered expressed as percentage of the length of the core run

METHOD/CASING

Shows drilling method and depth of casing

Common types:

- OB - open barrel
- W -- wash
- HQ3 - HQ triple tube coring
- PQ3 -- PQ triple tube coring

SAMPLE TYPE

(The length of the sample is indicated by the length of symbol)

- OPEN BARREL
- DOUBLE OR TRIPLE TUBE
- STANDARD PENETRATION TEST
- LARGE DIA. THIN WALLED TUBE
- SMALL DIA. THIN WALLED TUBE
- BULK SAMPLE

GRAPHIC LOG

The graphic log shows soil and rock substances, significant defects, and core loss. Soil and rock substances represented clear contrasting symbols consistent for each project.

TYPICAL SYMBOLS:

- ORGANIC MATERIAL
- CLAY
- SILT
- SAND
- GRAVEL
- MUDSTONE
- SILTSTONE
- SANDSTONE
- VOLCANIC ROCK
- NO CORE

TESTS

N=22	SPT uncorrected blow count for 300mm
ø 76kPa	Undrained shear strength as measured by field vane
⊠	Pressuremeter Test
*	Laboratory test(s) carried out:

Common types:

- LV - Laboratory vane
- AL - Atterberg limits
- UU -- Undrained triaxial
- PSD -- Particle size
- C' Ø' - Effective stress
- CONS - Consolidation
- DS - Direct Shear
- COMP - Compaction
- UN - Unconfined Compression
- IS -- Point Load

SOIL DESCRIPTIONS

CLASSIFICATION SYMBOL

Based on USBR Unified Soil Classification System Visual Method for field identification. Classification symbols based on Laboratory Method may differ.

Soil and rock descriptions generally follow the "Guidelines for the field descriptions of Soils and Rocks in Engineering Use" by the NZ Geomechanics Society (1985). Particular in-house abbreviations are given below.

MOISTURE CONTENT

- D - Dry, looks and feels dry
- M -- Moist, no free water on hand when remoulding
- W - Wet, free water on hand when remoulding

Moisture content may be compared to the plastic limit (PL) eg M > PL = moist, moisture content greater than the plastic limit

UNDRAINED SHEAR STRENGTH

		Cu (kPa)
VS	Very Soft	<10
S	Soft	10 to 25
F	Firm	25 to 50
St	Stiff	50 to 100
VSt	Very Stiff	100 to 200
H	Hard	>200
Fb	Friable	

RELATIVE DENSITY

		SPT -- Uncorrected
VL	Very Loose	0 to 4
L	Loose	4 to 10
MD	Medium Dense	10 to 30
D	Dense	30 to 50
VD	Very Dense	>50

ROCK DESCRIPTIONS

WEATHERING

UW	Unweathered
SW	Slightly Weathered
MW	Moderately Weathered
HW	Highly Weathered
CW	Completely/Extr. Weathered
RW	Residual Soil

FIELD STRENGTH

		UCS (MPa)	Point Load Index (MPa)
R0	Extremely Weak	0.25-1	N/A
R1	Very Weak	1 to 5	N/A
R2	Weak	5 to 25	N/A
R3	Moderately Strong	25 to 50	1-2
R4	Strong	50 to 100	2-4
R5	Very Strong	100 to 250	4-10
R6	Extremely Strong	>250	

P.T.O.

TONKIN & TAYLOR LTD.
ENVIRONMENTAL & ENGINEERING CONSULTANTS

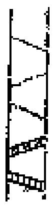
ENGINEERING LOG TERMINOLOGY

ROCK DESCRIPTIONS

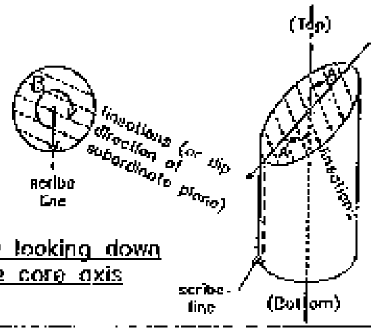
(Continued)

DEFECTS

Significant defects are shown graphically



B	BEDDING
J	JOINT
SZ	SHEARED ZONE
CZ	CRUSHED SEAM/ZONE
IF	INFILLED SEAM/ZONE
XD	EXTREMELY WEATHERED SEAM



View looking down the core axis

CODING

Typical Example:

2J60°, PL, SL, T, CV, stiff green CLAY

- 2 — No. in defect set
- J — Type
- 60° — Angle to core axis
- PL — Shape
- SL — Roughness
- T — Apeture
- CV — Infilling/Coating Type
- stiff green CLAY — Infilling Description (as per Soil Description)

Where possible, the relative angle between the defect and the bedding (B) is given, where B = angle from the dip direction of the bedding, to the dip direction of the defect, in a clockwise direction when looking down the core axis.

SHAPE

TERM	CODE
Planar	PL
Slightly curved	SC
Curved	CV
Irregular	IR
Stepped	ST
Wavy	WV

ROUGHNESS

DESCRIPTION OF JOINT SURFACE	CODE
Slickensided	SL
Smooth	SM
Defined Ridges	DR
Small Steps	ST
Rough	R
Very Rough	VR

After Piteau (1973)

APERTURE

TERM	SYMBOL	DESCRIPTION (Separation)
Very Tight	VT	less than 0.1mm
Tight	T	0.1mm to 1.0mm
Open	O	1.0mm to 10.0mm
Very Open	VO	more than 10mm

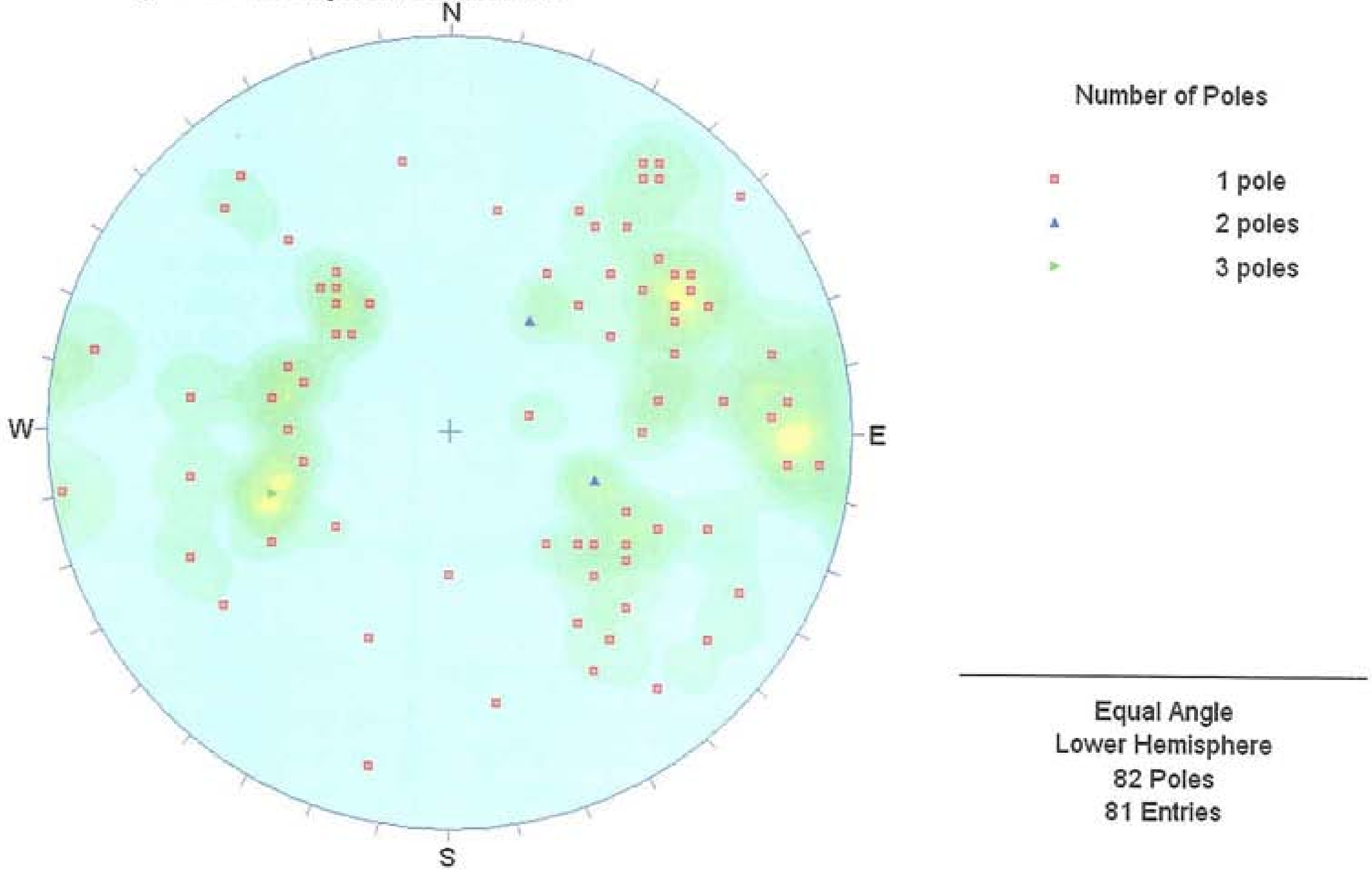
After Bieniawski (1973)

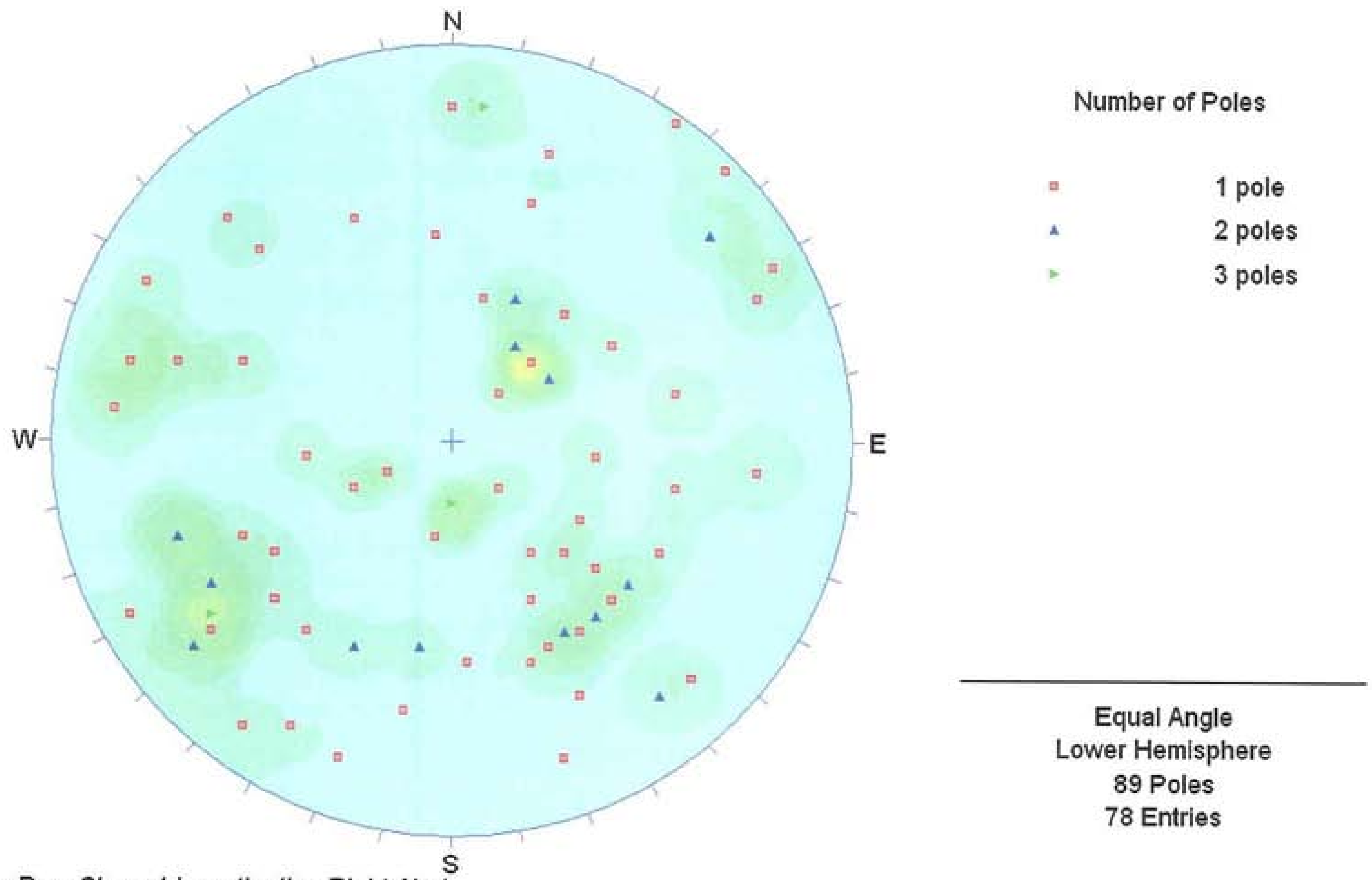
INFILLINGS AND COATINGS

Clay Gouge	CG	Joints have openings between opposing faces of intact rock substance in excess of 1mm filled with clay gouge. Clay is generally described in terms of soil properties.
Clay Veneers	CV	Joints contain clay coating whose maximum thickness does not exceed 1mm. Note: Describe clay in terms of soil properties.
Penetrative Limonite	PL	Joint traces are marked in terms of well defined zones of slightly to moderately weathered ferruginised rock-substance within the adjacent rock.
Limonite Stained	FeSt	Joint surfaces are stained or coated with limonite, although the rock substance immediately adjacent to the joints is fresh.
Coated	CT SC	Joints exhibit Coatings other than clay or limonite, eg. Carbonate (CT) or silica (SC).
Cemented	CL CS CC	Joints are cemented with limonite (CL), silica (CS), or carbonates (CC).
Clean	CN	Joint surfaces show no trace of clay, limonite, or other coatings.

Appendix E: Stereographic Plots

Lee Dam Stage 4 Site Investigation, Left abutment





Lee Dam Stage 4 Investigation Right Abut

Appendix F: Rock Mass Rating and Modulus Assessment

Feature	DH5 0-5m	DH5 5-10m	DH5 10-20m	DH 6 0-5m	DH6 5-23m	DH6 23-34m	DH 7 10-15	DH7 15-25	DH7 25-32	DH 8 3-8m	DH8 8-13	DH 8 13-23	DH 8 23-38	DH 9 3-6m	DH 9 6-15
1. Strength	3	4	5	2	3	7	4	5	4	4	5	5	7	4	7
2. RQD	3	8	13	3	3	13	13	13	8	8	8	11	13	8	13
3. Discontinuity Spacing	5	10	10	5	6	8	8	8	8	8	8	8	8	8	10
4. Discontinuity Persistence	2	2	4	2	2	2	3	3	2	4	4	4	4	2	3
5. Discontinuity Separation	4	4	5	4	5	5	4	5	5	3	3	5	5	1	5
6. Discontinuity Roughness	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
7. Discontinuity Infilling	3	3	6	2	2	6	4	4	0	3	3	3	3	2	4
8. Discontinuity Weathering	2	3	5	3	4	5	3	5	6	3	4	5	6	5	6
9. Water	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10
10. Adjustment for Orientation	-2	-2	-2	-5	-5	-2	-2	-2	-2	-5	-4	-2	-2	-2	-2
ROCK MASS RATING	33	45	59	29	33	57	50	54	44	41	44	52	57	41	59
CLASS	IV	III	[illegible]	IV	IV	[illegible]	III	[illegible]	III	III	III	[illegible]	[illegible]	III	[illegible]
DESCRIPTION	POOR	FAIR	FAIR	POOR	POOR	FAIR	FAIR	FAIR	FAIR	FAIR	FAIR	FAIR	FAIR	FAIR	FAIR

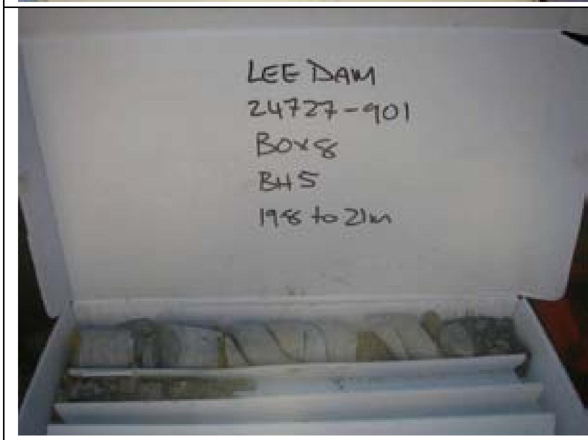
Rock Strength and Deformation Modulai

Feature	DH5 0-5m	DH5 5-10m	DH5 10-20m	DH 6 0-5m	DH6 5-23m	DH6 23-34m	DH 7 10-15	DH7 15-25	DH7 25-32	DH 8 3-8m	DH8 8-13	DH 8 13-23	DH 8 23-38	DH 9 3-6m	DH 9 6-15
Ei (intact modulus)	5250	10500	14000	7000	8750	17500	12250	14000	10500	12250	14000	15750	17500	12250	17500
Erm D = 0	314	[illegible]	[redacted]	569	992	3913	[illegible]	[redacted]	1190	[illegible]	[illegible]	[redacted]	[redacted]	2739	[redacted]
Erm D = .2	250	1249	3237	438	747	2916	1458	2333	897	1049	2333	1874	4046	2042	5481
Erm D = .7	159	611	1501	251	392	1370	713	1096	470	549	1096	917	1877	959	2584
c (KPa)*	200	300	500	250	300	535	400	480	330	350	480	446	618	450	728
Phi'*	35	42	50	39	42	50	46	48	44	45	49	48	51	48	53

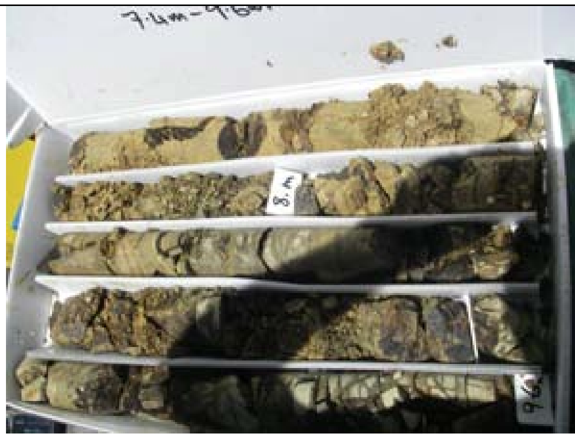
*Relevant to slope heights in the order of 50 m Defect strength likely to have more influence on stability for lower height slopes

Appendix G: Photographs of Geological and Geomorphic features

Drillhole 5: Core Boxes 1 – 4



Drillhole 5 Core Boxes 5-8



Drillhole 6: Core Boxes 1 – 4

Drillhole 6: Core Boxes 5-8

Drillhole 6: Boxes 9 - 13

Drillhole 6: Box 14

Drillhole 7: Core Boxes 4-7

Drillhole 7: Core Boxes 8-11

Drillhole 8: Core Boxes 1 - 4

Drillhole 8: Core Boxes 5-8

Drillhole 8: Boxes 9 - 12

Drillhole 9: Core Boxes 1-4

Drillhole 7: Core Boxes 8-11

Photo 1: View upstream of dam location and upper catchment

Photo 2: Dam Left abutment

Photo 3: View of left abutment slope

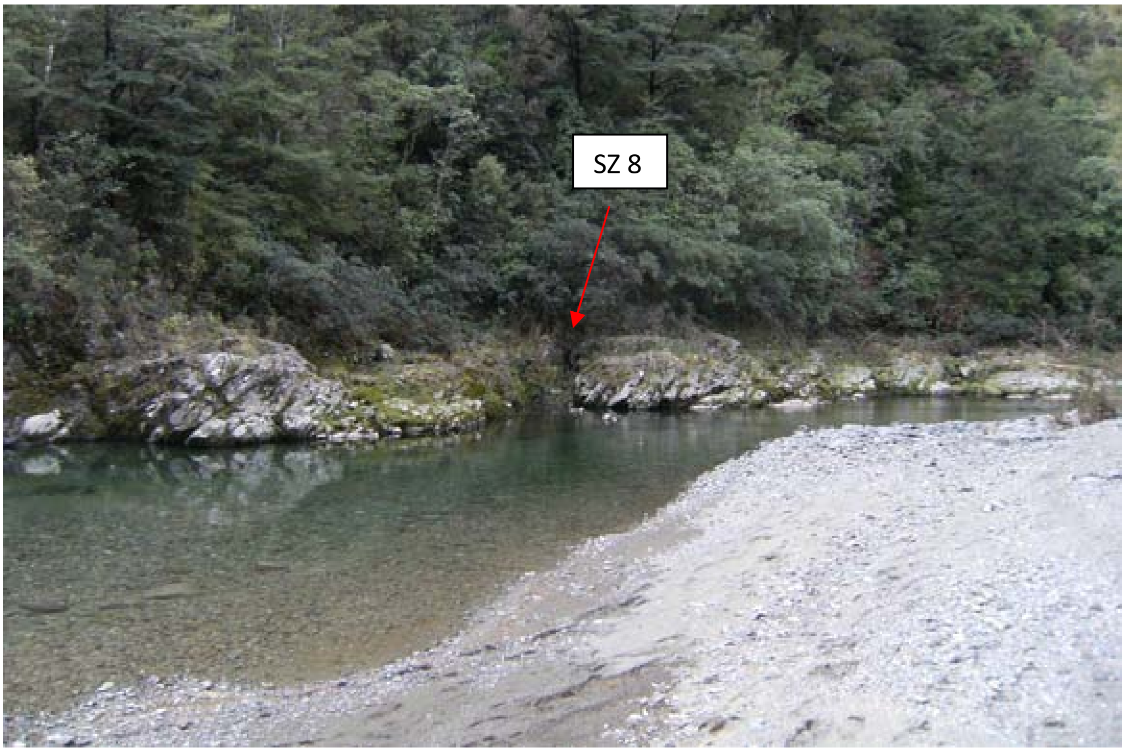


Photo 4: Rock exposed in right river bank beneath dam centreline

Photo 5: Greywacke exposed upstream of SZ8 (refer Photo 4)

Photo 6: Dam foundation, right bank of river, upstream of dam centreline

Photo 7: Greywacke exposed in river downstre3am of dam toe

Photo 8: Downstream of centreline, left abutment foundation

Photo 8a: Downstream left abutment near dam toe (closer view of Photo 8)

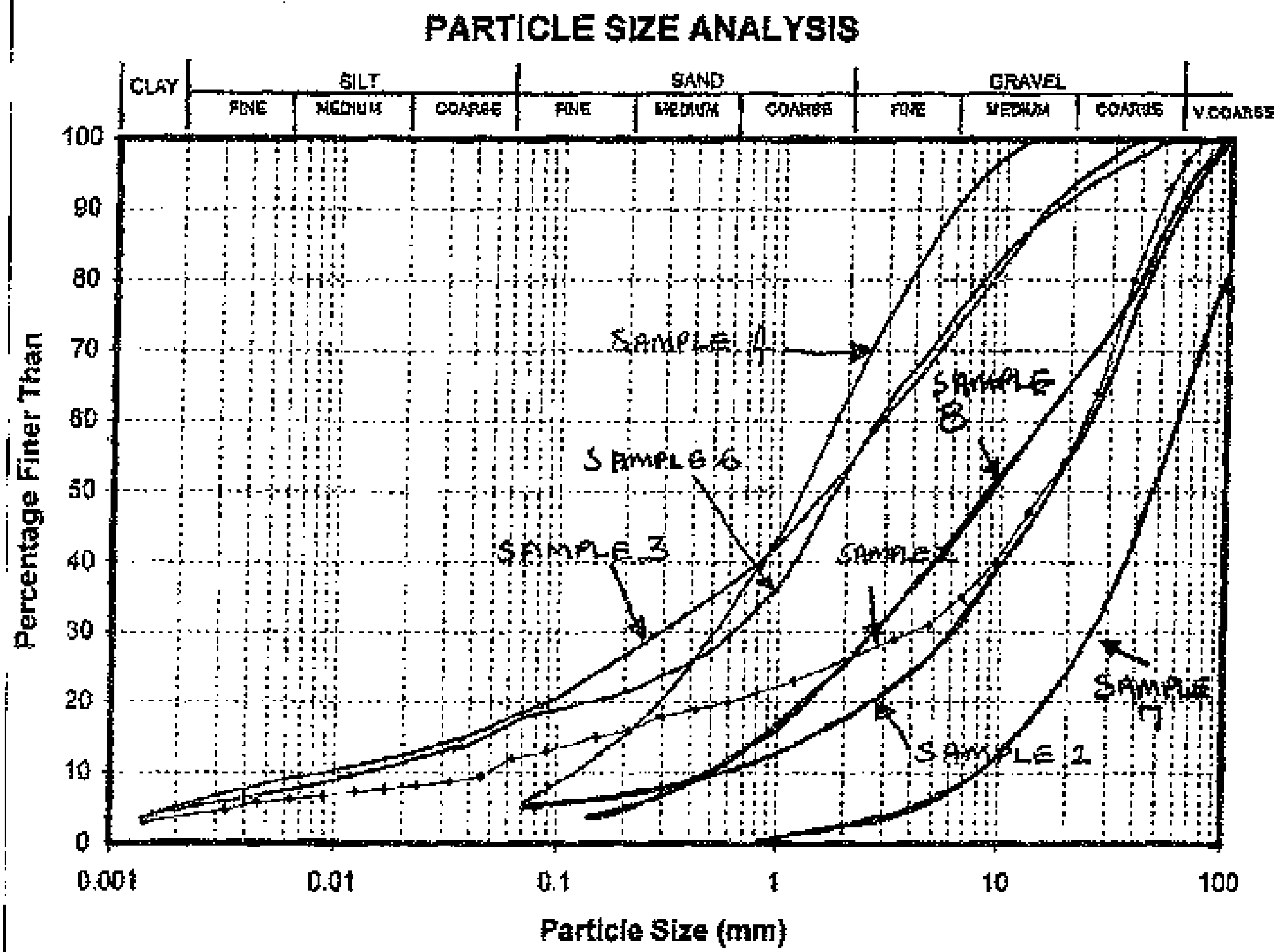
Photo 9: Laminated greywacke in river exposure in dam foundation

Photo 10: View downstream from dam toe

Photo 11: Moderately weathered, closely jointed rock exposed in track upslope of right abutment

Photo 12: Solifluction deposit infilling fossil gully, upstream of left abutment

Appendix H: Laboratory Test Results



SOURCE:

Solifluction	Samples 4 & 6
Croiselles Landslide	Sample 3
Terrace Alluvium	Sample 8
Rai Formation	MW-HW - Sample Z
Rai Formation	MW - Sample 1
Rai Formation	SW-HW - Sample 7

Tonkin & Taylor
Environmental & Engineering Consultants
Auckland Hamilton Nelson Wellington Christchurch Whangarei

DRAWN	EVL	Jun.03
DRAFTING CHECKED		
APPROVED		

CADFILE: \\24727.201_F7.dwg
SCALES (AT A4 SIZE) NOT TO SCALE
PROJECT No. 24727.201

WAIMEA WATER AUGMENTATION COMMITTEE
WAIMEA 2 – FEASIBILITY
LEE RIVER
Summary Gradings: Potential Borrow Materials

FIG. No. 24727.201-F8 REV. 0

CIVIL ENGINEERING LABORATORY SERVICES LTD

Independent Materials Testing Laboratory

Lab Ref No.: 08/323   28 MAY 2008

Client Ref No.:

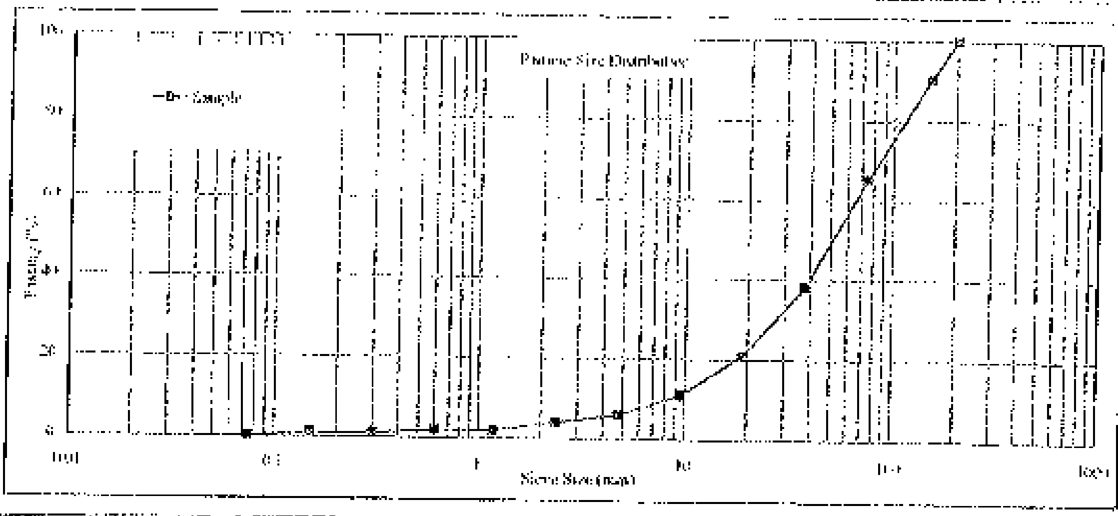
Order No.:

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project :	Lee Valley Dam	Date sampled :	Unknown
Location :	Lee Valley	Sampling method :	Unknown
Client :	M Lovell	Sampled by :	M Lovell
Contractor :	Tonkin & Taylor	Sample description :	Weathered ROCK [illegible]
Source :	Left Bank Spillway	Sample condition :	Damp

Particle Size Distribution

Sieve Size (mm)	Percentage Passing	
	Sample	Limits
200	100	
150	90	
75	65	
37.5	38	
19	24	
9.5	11	
4.75	6	
2.36	4	
1.18	2	
0.6	2	
0.3	1	
0.15	1	
0.075	0	



Test Methods		Notes
Particle Size Distribution	NZS 4407: 1991 Test 3.8.1	
Date tested :	28/05/08	
Date reported :	28/05/08	This report may only be reproduced in full
	Mark Zoeganl	
Designation :	Laboratory Manager	

PO Box 1424 NELSON, Unit 3/30 Echodale Place STOKE Ph: 03 547 0110 Fax: 03 547 0120
Mobile: 027 445 7071 Email: civillab@xtra.co.nz Web: www.cels.co.nz

Lab Ref No : 08/351

Client Ref No : -

Order No : -

CIVIL ENGINEERING LABORATORY SERVICES LTD

Independent Materials Testing Laboratory

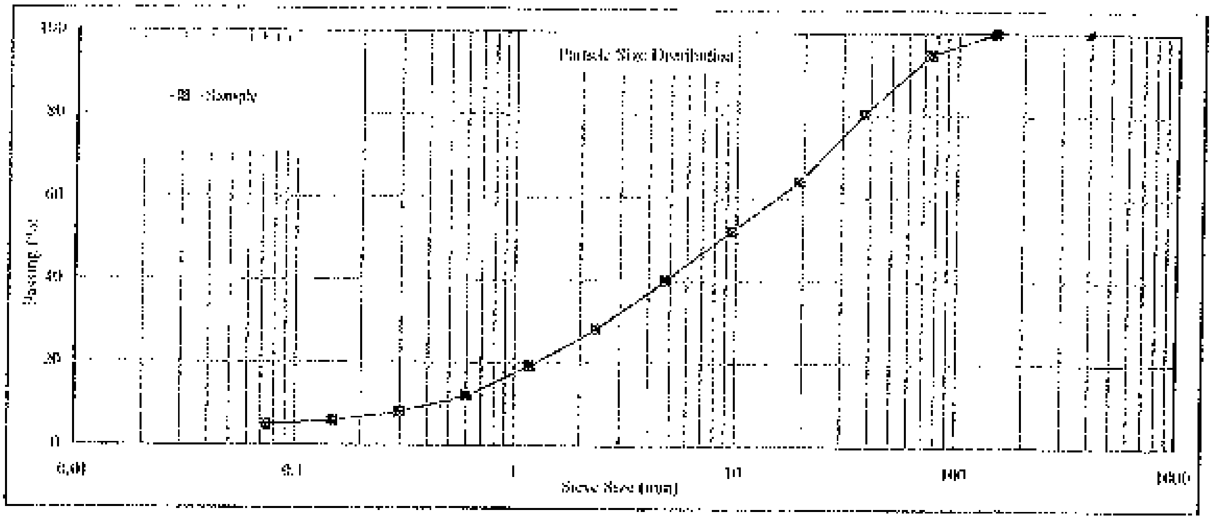
PARTICLE SIZE DISTRIBUTION TEST REPORT

05 JUN 2008

Project :	Lee Valley Dam	Date sampled :	20/05/08
Location :	Lee Valley	Sampling method :	NZS 4407: 1991 2.4.2
Client :	M Patel	Sampled by :	J Westerson
Contractor :	Tonkin & Taylor	Sample description :	Silty Sandy Gravel
Source :	Borrow	Sample condition :	Damp

Particle Size Distribution

Sieve Size (mm)	Percentage Passing: Sample	Percentage Passing: Limits
150	100	
75	95	
37.5	80	
19	64	
9.5	52	
4.75	40	
2.36	28	
1.18	19	
0.6	12	
0.3	8	
0.15	6	
0.075	5	



Test Methods		Notes
Particle Size Distribution	NZS 4407 : 1991 : Test 3.8.1	
Date tested :	30/05/08	
Date reported :	30/05/08	This report may only be reproduced in full
	Mick Zeewoldt	
Designation :	*Laboratory Manager*	

PO Box 1424 NELSON, Unit 3/30 Echodale Place STOKE Ph: 03 547 0110 Fax: 03 547 0120
Mobile. 027 445 7071 Email: civillab@xtra.co.nz Web. www.cels.co.nz

Appendix I: Peer Review Reports

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

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

Ref: 6387

Tonkin and Taylor Ltd
P O Box 5271
AUCKLAND

17 December 2007

Attention: Robin Dawson

Dear Robin,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER WATER STORAGE
EXTERNAL PEER REVIEW REPORT NO.1

Engineering Geology Ltd (EGL) have been engaged as a sub-consultant by Tonkin and Taylor Ltd (T&T) to provide external peer review of the Phase 2 geotechnical investigations and design of the proposed dam that will create a water storage reservoir on the Lee River. The water storage reservoir is being developed for the Waimea Water Augmentation Committee (WWAC). The scope of external peer review is outlined in Appendix A of the Services Agreement.

The objects of the peer review are to:

i) carry out peer review of preliminary geotechnical mapping and engineering design work associated with selecting the most appropriate location for the dam within the subject reach of river, and the selection of the most appropriate type of dam (including site visit)

ii) carry out peer review of the proposed second and third stage geotechnical investigations

iii) carry out peer review of the Stage 2 geotechnical report

iv) carry out peer review of the Stage 3 geotechnical report

v) carry out peer review of the preliminary dam design, cost estimate and associated reporting.

This is the first external peer review report by EGL, and covers the first objective listed above. Peer review has consisted of a site inspection by Trevor Matuschka on 28 November 2007 and review of the following documents:

- section 5 Methodology from the T&T proposal for Phase 2 studies
- Geology of the Lee River Catchment in the Environs of Potential Dam Sites (T&T Ref. 24727.200, 10 December 2007 draft).
- Project Feasibility Study Optimisation of Dam Location and Type (T&T Ref. 24727.301, 10 December 2007 draft)

Peer Review comments on the work to date follow:

1. Site Inspection

An inspection of the seven potential dam sites identified between CH10500 and CH12400 was undertaken by Trevor Matuschka on 28 November 2007. The inspection was undertaken in company with Mark Foley (Project Manager – T&T). Prior to the site inspection a briefing was provided on the geology of the area, the scope of and interpretation of geological investigations to date and implications for dam design. The various dam sites that are being considered and potential borrow areas and materials were inspected during the site visit.

2. Geological Investigations

The results of geological and engineering geological studies and a preliminary assessment of the geotechnical risks are documented in the T&T report 'Geology of the Lee River Catchment in the Environs of Potential Dam Sites' (Ref. 24727.200, 10 December 2007). Some investigations were undertaken as part of the Phase 1 (pre-feasibility) investigation, but they have been extended upstream as part of the Phase 2 study. This work constitutes Stage 1 of a three stage geotechnical investigation for the Lee Valley Dam.

We consider that the scope of investigations are appropriate for this stage of the study. The T&T report provides a good summary of the regional and local geology and seismic hazard of the site. It also includes assessment of geotechnical issues affecting the selection of the preferred dam site and the design of the various types of dams that are being considered. The geotechnical issues include slope stability, foundation conditions, assessment of the construction materials and seismic risk.

A summary of the geotechnical conditions as they are currently understood, at the seven potential dam locations, is provided. The studies to date indicate that the depth of weathering is shallow and bedrock would be expected to have acceptably low permeability, except at shallow depth where some grouting maybe required where rock relaxation may have occurred. Landslides are evident at two locations but they are not considered fatal flaws and could be mitigated as part of the dam design. No active faults have been identified at the potential dam sites. The availability of low permeability fill material for core material in an earth/rockfill embankment dam is limited. This has been recognized and further studies will be necessary to confirm the location and properties of such material.

3. Optimisation of Dam Location and Type

A study to determine the optimum dam location and type has been undertaken and is documented in the T&T report 'Feasibility Study Optimisation of Dam Location and Type' (T&T Ref.24727.301, 10 December 2007). This study considers the possibility of two different sized storages (13 and 16MM^3). The optimisation study has compared seven potential dam sites and three different dam types: earth/rockfill embankment, concrete faced rockfill dam (CFRD) and roller compacted concrete (RCC).

We consider that the seven potential dam sites selected by T&T are appropriate and cover the range of realistic possibilities. We are also in agreement with the three types of dams that have been considered. For each type of dam a concept design has been prepared as the basis for preparing construction cost estimates. Quantities have been determined from computer generated design models using ground contour data that was obtained from a recent survey. Generally the design concepts for the different dam types, including assumptions for diversion and spillway design are reasonable for the purposes of assessing options. Modifications to these designs can be expected as design is advanced for the selected option. We consider that it may be necessary to widen the core of the earth/rockfill embankment, and add an upstream filter zone, but there details should not add significant cost. Also the slopes of the shoulder associated with the CFRD options may require flattening to achieve acceptable performance under seismic loading conditions. A conservative approach has been adopted with respect to diversion requirements and this is appropriate. Unit rates for the key items used to determine the construction cost estimates are generally considered realistic. The rate for core material in an earth embankment involves some uncertainty at this time because locations and haul distances have not yet been confirmed. Analyses have been conducted to assess the sensitivity of the construction costs of the various dam types to variations in the unit rates, and they indicate a low sensitivity to the cost of core material.

The optimisation study concludes that an earth/rockfill embankment at CH11010 is the optimal solution from a cost perspective. The conclusion is the same for both the 13 and 16Mm^3 storages. Two other options (CH10540 and CH10880) are only about 6-8 percent more expensive, which is not that significant given the level of uncertainty associated with studies to date. However, these options are less favourable for diversion and spillway construction. For most sites, including the preferred (CH11010) the CFRD option is only marginally more expensive than the earth/rockfill embankment. Taking into account the uncertainties associated with the assumptions adopted in the optimisation study the difference in cost between the earth/rockfill embankment and the CFRD option is not significant.

We agree with the T&T recommendations that CH11010 is the preferred site. T&T conclude that an earth/rockfill embankment is the preferred dam type. Due to the uncertainty associated with suitable core material geotechnical investigations are recommended that focus on locating suitable core material, to confirm dam type suitability, prior to investigating the dam foundations. This is appropriate, however, we recommend that due to the very close cost difference between an earth/rockfill embankment the CFRD option, consideration could be given to also advancing studies for the CFRD option in parallel, particularly if there is an imperative to have the dam commissioned as quickly as possible. The CFRD option is a simpler design and has cost/time advantages with respect to diversion arrangements and should have a shorter construction period.

4. Summary and Conclusions

We consider that the geotechnical and engineering studies undertaken by T&T are appropriate for optimising the location and type of dam for the proposed water storage reservoir on the Lee River. We agree that the preferred site is CH11010. An earth/rockfill embankment is identified as the preferred dam type and recommendations for advancing consideration of this option are provided. We agree, however, we note that the cost difference between an earth/rockfill embankment and a CFRD is not significant and consideration could be given to also advancing studies for a CFRD in parallel, particularly if there is an imperative to commission the dam as quickly as possible.

Yours faithfully
ENGINEERING GEOLOGY LTD

Trevor Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

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

Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON

RECEIVED
28 FEB 2008
BY: SLM

26 February 2008

Attention: Sally Marx

Dear Sally,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER WATER STORAGE
EXTERNAL PEER REVIEW REPORT NO.2

This report covers review of the Tonkin and Taylor Ltd memo dated (ref: 24721.203) 20 February 2008. That memo reports on the results of additional sampling, testing and geological mapping undertaken to assess the characteristics of potential borrow material (earthfill and rockfill) for an earthfill (clay core) dam and a concrete faced rockfill dam (CFRD). The memo was accompanied by the laboratory transcripts of the test results on the different materials. I have reviewed both the test results and the memo.

The results indicate limited sources of plastic, low permeability material. Greater quantities of low plasticity earthfill were identified, but the study indicates that the available volumes of these materials are not great. It is possible to construct an earthfill dam with low plasticity earthfill, but particular attention is required to the design of filters and a high standard of construction would be required to provide assurance that the constructed dam met design specifications. The study has identified significant volumes of rock suitable for construction of a CFRD dam. Consequently Tonkin and Taylor has recommended that priority should be given to assessing the feasibility of a CFRD. I agree with this recommendation and the recommendation to develop a staged dam foundation investigation programme. I note that a CFRD has some advantages over an earthfill type dam including:

- it should be able to be investigated and designed quicker
- construction is less dependent on weather
- it can be constructed faster
- there are potential savings with flood diversion costs during construction because the design standards for the diversion conduit can be less onerous as a CFRD can sustain some overtopping during construction.

In summary, I consider that the approach that has been adopted to investigate potential sources of borrow is adequate for this stage of the project. The results indicate that priority should be given to proceeding with evaluating the feasibility of a CFRD. I consider the recommendations provided by Tonkin and Taylor for progressing with the feasibility of a CFRD are appropriate.

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

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

Ref6387

Tonkin and Taylor Ltd
P O Box 5271
AUCKLAND

20 June 2008

Attention: Robin Dawson

Dear Robin,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER WATER STORAGE
EXTERNAL PEER REVIEW REPORT NO.3

Tonkin and Taylor (T&T) have recently undertaken Phase 2 and 3 investigations for a potential dam on the Lee River. The results are presented in an interim report (Ref:24727.201, June 2008). Phase 2, geotechnical investigations covered assessment of:

i) the stability of the landslide on the left bank between 11,700 and 11,900m
ii) the engineering geology of a dam site at 11,010m
iii) a potential hard rock borrow site in the Putaki Melange, on the left bank at 10,200m

Phase 3 investigations comprised:

i) review of aerial photographs
ii) mapping bedrock geology exposures in the vicinity of the landslide, proposed dam and borrow area
iii) excavation and logging of 18 test pits at the landslide and proposed dam locations
iv) drilling and logging of four triple-tube rotary boreholes.

We previously reviewed and commented on the proposed scope of work for the Phase 2 and 3 investigations.

In addition to the interim report covering the Phase 2 and 3 investigations T&T have prepared a memo (Ref:24727.301, 17 June 2008) that provides interpretation the Phase 2 and 3 geotechnical investigations, comments on the implications of the results of the investigations and provides recommendations for advancing studies for the Lee River Dam. This memo identifies some issues that potentially affect the performance of the dam and will increase previous construction cost estimates for the dam at 11,010m. The issues are:

- the rock mass quality on the right abutment is poor to a large depth (>35m) and will likely result in the need for extensive foundation remedial works (additional excavation to remove relaxed rock and extensive grouting). The poor quality rock on the right abutment is believed to be due to the presence of a splay fault associated with the Anslow Fault

- up to approximately 14m of alluvial and colluvial materials blanket the left abutment which most likely will require removal

- active slope movement is evident upstream of the dam on the left bank and will likely require buttressing (T&T estimate the volume of buttress fill to be about 200,000m^3).

As a consequence of the above issues, T&T have undertaken a further site reconnaissance and have reviewed the geological ranking of the sites reported in the Project Feasibility Study Optimisation of Dam Location and Type report (December 2007). It is concluded that many of the dam sites considered in 2007 will be affected by the features identified by the Phase 2 and 3 investigations for a dam at 11,010m. The recent site reconnaissance and air photo interpretation indicates that higher quality rock would be expected between 12,100m and 13,000m. Consequently T&T now recommend that the Phase 2 Feasibility Study be extended to undertake a staged investigation of foundation condition at sites between 12,100m and 13,000m.

We have reviewed the data from the Phase 2 and 3 investigations and agree with the interpretation of the results and the assessment of the implications for the dam at 11,010m. The additional work that will be necessary at this site is such that the previous economic advantage with this site compared to between 12,100m and 13,000m is not longer valid. Consequently, we also agree with the recommendation to undertake a staged investigation of foundation conditions at sites between 12,100m and 13,000m. In addition, we note that as a result of the Phase 2 and 3 investigations, no suitable core material in the vicinity of the dam has been found. The closest source is approximately 18km downstream on private property. This has implications on the cost for a zoned earth/rockfill embankment, and supports the adoption of a concrete faced rockfill dam (CFRD).

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

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

Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON

9 April 2009

Attention: Sally Marx

Dear Sally,

RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE RIVER WATER STORAGE
ALTERNATIVE SITE (CH 12,000m - 13,000m)
EXTERNAL PEER REVIEW REPORT NO.4

Since July 2008 Tonkin and Taylor (T&T) have planned and undertaken geotechnical investigations of an alternative dam site located between CH12,000m and 13,000m. Investigations of this site were instigated because of limitations identified with the previously preferred site at CH11,010m.

Two stages of investigation have been conducted at the CH12,000m - 13,000m site as summarised below:

1. **Stage 1**

   This comprised surface mapping of track and river bank exposures, interpretation of the data, initial assessment of suitability of the site for a dam and recommendations for more detailed (Stage 2) investigations. The Stage 1 investigations are summarised in a T&T memo dated 28 October 2008.

2. **Stage 2**

   This comprised the drilling of five drillholes with packer permeability tests, further mapping of exposures in tracks formed to access the drillhole locations and interpretation of the results of the investigations. The Stage 2 investigations are summarised in a memo dated 25 February 2009. The initial scope of work for Stage 2 was for three drillholes. However, we recommended an additional drillhole in the spillway. A further drillhole was drilled as a result of encountering poor quality rock in the south abutment. In total, 5 drillholes were drilled.

The Stage 2 investigations have been undertaken by T&T in a staged manner, with on-going reporting of the results by the Engineering Geologist (Mark Foley) to the Internal Engineering Geology Peer Reviewer (Bernard Hegan), the Dam Designer (Robin Dawson), and the Internal Dam Design Peer Reviewer (Alan Pickens) as drilling progressed. T&T's Project Director for this particular work package, Senior Engineering Geologist Gary Smith, was involved in an overview capacity. We have participated in regular tele-conferences with the team to discuss the results, to confirm the depth of drilling and the locations of subsequent drillholes, and the need for additional drillholes to fill in gaps of knowledge.

The investigations indicate that a potential dam site exists between CH12,420m and 12,600m. T&T have reported that the rock mass quality is fair, but permeabilities are acceptably low for a dam allowing for some grouting of the near-surface rock. Defects are not generally unfavourably orientated.

We consider that T&T's memo dated 25 February 2009 on the Stage 2 investigations provides a good summary of the results and conclusions arising from the investigations to date. We confirm our agreement with T&T's conclusions that a potential dam site exists between CH12,420m and CH12,600m. The site is suitable for a CFRD or embankment dam, and possibly a RCC dam if the rock mass modulus can be proven to be consistently high across the site. We consider a CFRD is most suited, as low permeability core materials for an embankment dam are scarce. Additional investigations and analyses will be necessary to confirm the depth of excavation for the plinth associated with a CFRD, extent of foundation excavation, foundation treatment (grouting, rock anchors), stability issues during construction, rockfill properties and diversion structures. However, these are more matters of detail.

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

Unit 7C, Rosedale Office Park, 331 Rosedale Road
Albany, Auckland, New Zealand
PO Box 301054, Albany, North Shore 0757
Phone: 64 9 486 2546 Fax: 64 9 486 2556

Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON 6011

9 September 2009

Attention: Sally Marx

Dear Sally,

**RE: WAIMEA WATER AUGMENTATION COMMITTEE
LEE VALLEY IRRIGATION STORAGE DAM
ENGINEERING FEASIBILITY REPORT
EXTERNAL PEER REVIEW REPORT NO.5**

This report summarises the conclusions arising from our review of the engineering feasibility study of the proposed Lee Valley Irrigation Storage Dam undertaken by Tonkin and Taylor (T&T) and presented in a report titled "Lee Valley Storage Dam Engineering Feasibility Report" (Ref: 24727.303, September 2009). A supporting report "Geotechnical Investigations Report" (Ref: 28727.204, September 2009) has also been prepared and reviewed. The feasibility study is for a concrete faced rockfill dam located at CH12,430m. This site was selected following review of potential dam sites and preliminary geotechnical investigations. The original preferred site was at Chainage CH11,010m. However, geotechnical investigations revealed issues that had an adverse effect on potential construction costs and programme. We reviewed the outcome of those investigations and endorsed the conclusions and recommendations (see External Peer Review Report No 3 20 June 2008).

The engineering feasibility report is focussed on the preliminary design of the proposed dam. Sections 1-9 cover site selection, design standards and inputs. Section 10 covers the evaluation of options at the selected dam site (embankment type, flood diversion and routing and optimisation of spillway and dam crest parameters). Section 11 presents the arrangements of the embankment, spillway and outlet works. Section 12 Construction Methodology and Section 13 Capital Construction Cost Estimate are excluded from the review. We understand that the commencement of these sections is dependent on our endorsement of the preliminary design presented in the current issue of the engineering feasibility report. They will be completed in October. The engineering feasibility study report makes reference to accompanying technical reports, 'Dambreak Hazard Assessment' and 'Hydrological Assessment'. The results from a dambreak assessment are used to assess the potential impact category (PIC) of the design. This can be Low, Medium or High. The PIC guides the design standards for the dam. The design standards for a High PIC dam have been adopted by T&T in their preliminary design and we understand that early indications from the dambreak assessment support this assumption. We also consider this is the most likely outcome and endorse the assumption of the High PIC. We understand the dambreak assessment report will be made available to us shortly. Relevant hydrological information has been developed and used in the feasibility study. The interpretation of the hydrological data and development of design assumptions presented in the Feasibility Report appear reasonable. We understand the hydrological report will also be made available to us shortly.

We consider that engineering studies have been undertaken to a level that is appropriate to establish and confirm the feasibility of the proposed dam at CH 12,430m. We agree that geotechnical investigation studies indicate the site is suitable for a concrete faced rockfill dam. No issues affecting the feasibility of the proposed dam have been identified, and we consider that the geotechnical investigations undertaken are appropriate for this stage of feasibility assessment. There are some issues that need to be considered at the detailed design phase and they are summarised in 'Section 7 Conclusions and Recommendations' of the Geotechnical Investigation Report. The more significant issues include:

- further mapping, drilling and test pitting to confirm the range of rock strengths, orientation and strength of defects in the rock, particularly in the spillway area, and to delineate potential permeable zones (seepage pathways) or acquitards
- further packer testing to assess the location and permeability of potential seepage pathways and the groutability of the rock
- assessment of the dispersion and erosion vulnerability of shear zone material
- confirmation of foundation stripping depths
- undertake a site-specific seismic hazard study to confirm design earthquake ground motions and the potential for ground movement
- excavation and compaction trials to assess the grading and permeability of the potential rockfill. This needs to take into consideration the likely mix of rock types (greywacke and argillite) that will result during excavation of the spillway. This may lead to re-evaluation of the quantities of the different types of rockfill for dam construction
- assessment of durability of the fill and filter material under expected service conditions that include freeze/thaw/wetting and earthquake shaking

We endorse these conclusions, and agree that they are appropriate to investigate at the detailed design phase.

The proposed dam design (i.e. concrete faced rockfill dam) is well proven overseas, including in areas of high seismic hazard. There are many dams of this type that are considerably larger than that proposed on the Lee River. T&T's preliminary design is generally in accordance with established guidelines. The intake structure is not typical in that it is not a free standing structure. It is located on the upstream face of the dam. However, we understand that there is some historical precedence for this.

Issues that we recommend giving further consideration to at the detailed design phase include:

- zoning of the embankment to ensure that the principal design objectives (to minimise deformation under the face slab and to have a zone of high drainage capacity at low level) are achieved with the available construction materials. This may require some modification to the current zoning and use of alluvial materials to achieve drainage in critical areas
- confirm that sufficient freeboard has been allowed for in the maximum design flood event
- providing mesh reinforcement to the downstream shoulder so as to allow overtopping during construction which would allow a reduction in the size of the diversion culvert

In assessing the construction cost estimate we recommend allowing for:

- additional 100mm thickness of concrete in the face slab to allow for the difference between design and actual constructed thickness
- anchor bars and drainage holes beneath the ogee weir, spillway chute and flip bucket.

In summary, we agree that an appropriate level of geotechnical investigations and preliminary dam engineering design has been undertaken by T&T for this Feasibility Study, and that the results adequately demonstrate the feasibility of a concrete-faced rockfill dam at CH12,430m on the Lee River.

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng

Engineering Geology Ltd
CONSULTING GEOTECHNICAL, GEOLOGICAL AND EARTHQUAKE ENGINEERS

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

Ref: 6387

Tonkin and Taylor Ltd
P O Box 2083
WELLINGTON 6011

10 November 2009

Attention: Sally Marx

Dear Sally,

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

We have undertaken review of the following documents:

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

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

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

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

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

Yours faithfully
ENGINEERING GEOLOGY LTD

T Matuschka, CPEng