Appendix G
NOR – Richmond South Water Supply Reservoir

Geotechnical Assessment

14 May 2024

Tasman District Council - Water Sector
Private Bag 4
7050 Richmond

Richmond South Reservoir - RC Geotech support letter

G3445.59

Dear David Burn and Graham Rimmer,

The purpose of this letter is to support the Notice of Requirement and resource consent application for the revised option for the Richmond South Reservoir currently being proposed.

The Table below provides a comparison between the previous reservoir layout addressed in the Geotechnical Assessment Report (GAR - 5-G3141.09, Issue 2, 17/02/22) and the most recent proposed reservoir option (option 1):

Features	Previous reservoir	New reservoir
Diameter	25.7 m	22 m
High	10.2 m	8.67 m
Volume	4300 m^3	2500 m^3
Platform elevation	+61 m RL	+62.35 m RL
Seismic Design life	100 years	50 years
Durability Design life	100 years	100 years
Importance level	3	3
ULS Return period	1/2500 years	1/1000 years
ULS PGA	0.60 g	0.43 g
ULS Magnitude	6.1	6.1

<u>Key GAR construction recommendations in regard the previous proposed reservoir development</u>

The GAR reservoir constructability recommendations highlighted the need of the following features to be considered in the detailed design phase:

- Undercutting of approximately 2 m of soft surficial silts and Moutere gravels to form a level platform beneath the reservoir.

WSP
Wellington
L9 Majestic Centre
100 Willis Street
Wellington 6011, New Zealand
+64 4 471 7000
wsp.com/nz

- At the eastern reservoir platform edge, a geogrid tied back gabion basket retaining wall, up to 3 m high, with top geogrids (15 m long) that extend back beneath the reservoir footprint,

- Backfilling with well compacted granular fill to form a suitable platform for founding the reservoir,

- At the western reservoir platform edge, a cantilevered soldier pile wall embedded into the Moutere gravels is proposed. Bored concrete encased timber piles could be considered; however it is likely that groundwater will be encountered which may mean that a casing is required to keep the hole open. An alternative pile option is driven universal steel columns, which could penetrate the Moutere gravels without the need for augering. This will be considered further during detailed design.

<u>Update of the previous construction recommendations in regard the new proposed reservoir development.</u>

The previous recommendations made for the GAR are still applicable for the new proposed reservoir development, as follows: We have added in brackets additional advice and our currently inferred understanding of the reasons that these recommendations were made.

- Undercutting soft surficial silts and Moutere gravels to form a level platform beneath the reservoir, thickness of excavation needs to be revised in regard the platform elevation (to provide consistent settlement performance across the tank footprint),

- At the eastern reservoir platform edge, a geogrid tied back gabion basket retaining wall (height to be adapted with new platform elevation), with top geogrids (geogrid length to be defined during detailed design phase) that extend back beneath the reservoir footprint (to ensure seismic instability of the eastern slope below does not affect the reservoir founded close to the top of the existing slope)

- Backfilling with well compacted granular fill to form a suitable platform for founding the reservoir, (to provide consistent settlement performance across the tank footprint)

- At the western reservoir platform edge, the soldier pile wall may be able to be replaced with an unsupported cutting with due care to ensure the adjacent trees are retained.

The above recommendations were formulated during the elaboration of the GAR and are based on inputs which need to be confirmed during detailed design phase.

Regards

4 NOS / 2024

Vivien Hespel

Senior Geotechnical Engineer

Greg Saul

Technical Principal – Chartered Geotechnical Engineer (CPEng)

Project Number: 5-G3141.09

Richmond South Low Level Watermain and Reservoir

22 February 2022

CONFIDENTIAL

Geotechnical Assessment Report

Contact Details

Helen Hendrickson

WSP
Morrison Square
Level 1
77 Selwyn Place
Private Bag 36, Nelson Mail Centre
Nelson 7010
+64 3 548 1099
+64 27 201 3146
helen.hendrickson@wsp.com

Document Details:

Date: 22/02/2021
Reference: 5-G3141.09
Status: Issue 2

Prepared by

pp.
Helen Hendrickson/Will Jones

Reviewed by

22 / 02 / 2022

Greg Saul

Approved for release by

Darren Rodd

Document History and Status

Revision	Date	Author	Reviewed by	Approved by	Status
A	21/08/2020	RW/HH	GS	DR	DRAFT
0	11/09/2020	RW/HH	GS	DR	Issue 1
1	17/02/2022	WJ/HH	GS	DR	Issue 2

Revision Details

Revision	Details
A	DRAFT FOR DISCUSSION
0	ISSUE 1
1	ISSUE 2 – UPDATED FOR SINGLE TANK OPTION

Contents

List of Figures

List of Tables

Disclaimers and Limitations

- This report ('Report') has been prepared by WSP New Zealand Limited ('WSP') exclusively for Tasman District Council ('Client') in accordance with the Contract 1116 Secondary Professional Services, Variation 2, dated 21 April 2020.

Permitted Purpose

- This Report has been prepared expressly for the purpose of confirming the potential geotechnical issues for the Richmond Reservoir at 520 Hill St Richmond to be addressed in design. WSP accepts no liability whatsoever for the use of the Report, in whole or in part, for any purpose other than the Permitted Purpose. Unless expressly stated otherwise, this Report has been prepared without regard to any special interest of any party other than the Client.
- WSP accepts no liability whatsoever for any use of this Report, in whole or in part, by any party other than the Client. Unless WSP agrees otherwise in writing, any use or any reliance on this Report by a third party is at its sole risk without recourse to WSP. Third parties must make their own enquiries and obtain independent advice in relation to any matter dealt with or any conclusion expressed in this Report.

Qualifications and Assumptions

- The services undertaken by WSP in preparing this Report were limited to those specifically detailed in the Agreement and the Report and are subject to the scope, qualifications, assumptions and limitations set out in the Report and/or otherwise communicated to the Client. Except as otherwise stated in the Report and to the extent that statements, opinions, facts, conclusion and/or recommendations in the Report ('Conclusions') are based in whole or in part on information provided by the Client and other parties ('Information'). The Information has not been and have not been verified by WSP and WSP accepts no liability for the reliability, adequacy, accuracy and completeness of the Information.
- The data reported and conclusions drawn by WSP in this Report are based solely on information made available to WSP at the time of preparing the Report. The passage of time; unexpected variations in ground conditions; manifestations of latent conditions; or the impact of future events (including (without limitation) changes in policy, legislation, guidelines, scientific knowledge; and changes in interpretation of policy by statutory authorities); may require further investigation or subsequent re-evaluation of the Conclusions.

Use and Reliance

- This Report should be read in its entirety and must not be copied, distributed or referred to in part only. The Report must not be reproduced without WSP's prior approval in writing. WSP will not be responsible for interpretations or conclusions drawn by the reader of the Report. This Report (or sections of the Report) must not be used as part of a specification for a project or for incorporation into any other document without WSP's agreement in writing.

Disclaimer

- No warranty, undertaking or guarantee whether expressed or implied, is made with respect to the data reported or the Conclusions drawn. To the fullest extent permitted at law, WSP, its related bodies corporate and its officers, employees and agents assumes no liability and will not be liable to any third party for, or in relation to any losses, damages or expenses (including any indirect, consequential or punitive losses or damages or any amounts for loss of profit, loss of revenue, loss of opportunity to earn profit, loss of production, loss of contract, increased operational costs, loss of business opportunity, site depredation costs, business interruption or economic loss) of any kind whatsoever, suffered on incurred by a third party.

1 Introduction

WSP has been commissioned by Tasman District Council to provide a scheme design for a new reservoir on the property at 520 Hill St South, Richmond, and pipeline connecting to Cupola Crescent.

This report presents a desk study review of the available geotechnical information, the results of site investigations carried out and a summary of the geotechnical issues which will need to be addressed in design.

This current report is an updated version of the Issue 1 report (dated September 2020) to reflect the changed reservoir configuration from two reservoirs to one.

1.1 Purpose

The purpose of this report is to;

- Confirm ground conditions through onsite investigations;
- Identify potential seismic and geotechnical hazards which may impact the proposed development;
- Report on potential issues which will need to be addressed in design;
- Identify potential issues which may impact constructability.

2 Project Scope

The project has three components. These are

- Construction of a new concrete reservoir (4300 m^3) and formation of an access road around the reservoir
- A water supply pipeline running from the reservoir to Cupola Crescent
- An accessway running from Hill Street South to the property boundary at 520 Hill Street South

A new DN450 PE (PE100, PN12.5) water line is proposed in the approximate alignment displayed in Figure 1. Details of the pipe size and alignment have not been confirmed and the information provided in this report on the pipeline alignment is intended to support detailed design. It is expected that the pipeline is to be constructed beneath the ground with a minimum proposed cover of 1.2 m and maximum depth approximately 2.1 m below existing ground level. The pipeline will be a buried PE pipe and therefore fully restrained. Hence no thrust blocks are required with the exceptions of the connection points for the rising main section to the existing network and the reservoir offtake.

A concept layout of the project layout is shown in Figure 1.

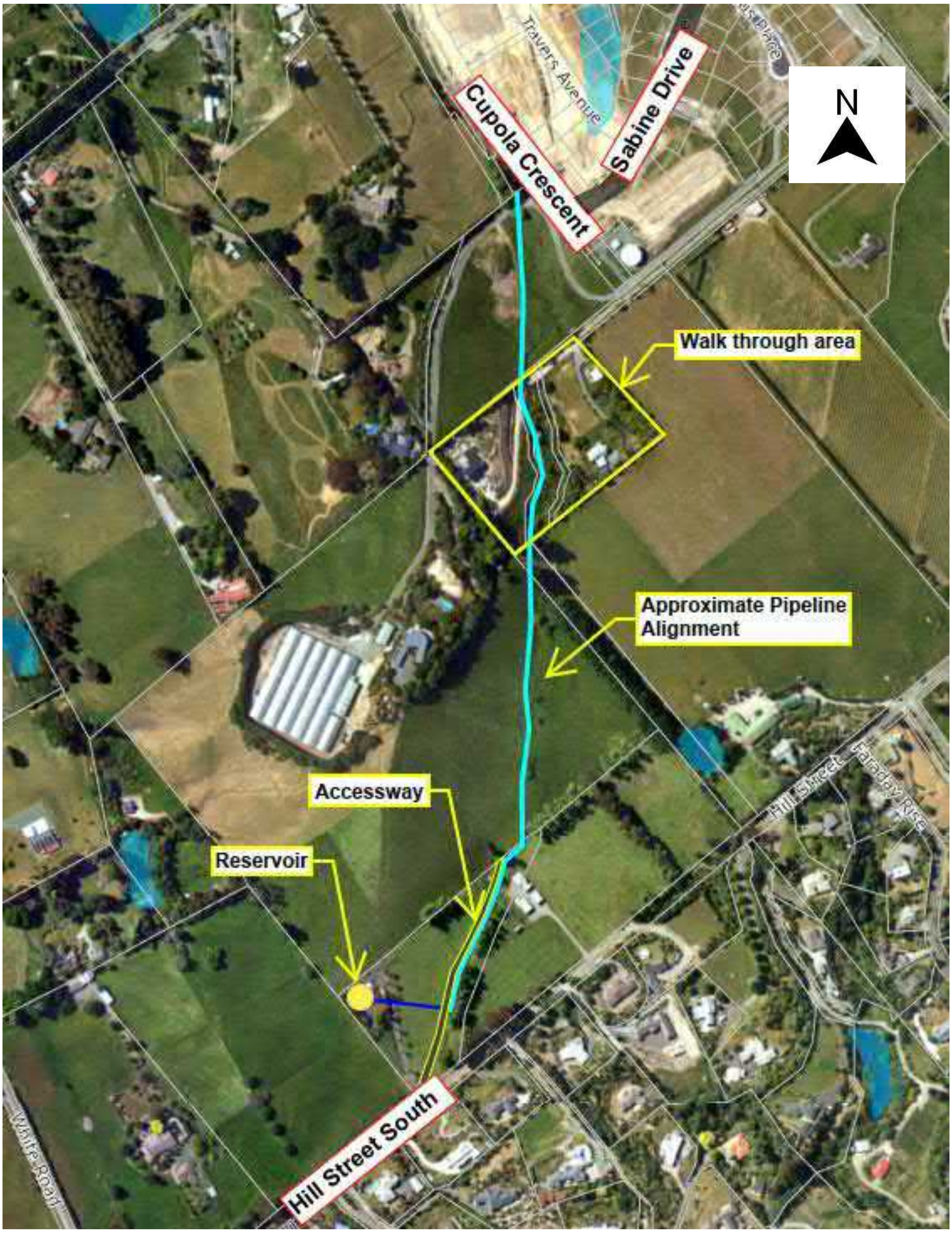

Figure 1 - Richmond Reservoir Site Layout.

3 Site Description

3.1 Reservoir Site

The proposed reservoir site is generally flat and currently occupied by a standalone residential dwelling and garage. The site is at the top of a ridgeline and the surrounding ground slopes downwards to the east and west. The ridgeline trends North-South and the site is in a low saddle. Access to the reservoir site will be along the existing driveway.

The proposed DN450 water supply line pipeline runs downslope east of the proposed reservoir (the 'hillside pipeline'), before following an existing stream to Cupola Crescent (the 'main pipeline').

Access along the southern 100 m of the pipeline is located to the east and downslope of the reservoir site.

3.2 Pipeline Alignment

The general alignment of the main pipeline approximately follows the existing stream between the bottom of the hillside pipeline to the connection point along Cupola Crescent (Figure 2 and Figure 3). The slopes on both sides slope downward towards the stream. In general, the slopes to the western side of the pipe alignment range from approximately 2H:1V to 4H:1V. The slopes on the eastern side of alignment are approximately 20:1.

The catchment of the stream consists of an approximate catchment area of 30 ha and consist of the Richmond Hills as well as the surrounding farmlands. The stream consists of a defined channel of approximately 1 m width and < 1.5 m deep at the 520 Hill Street South site and from the walkthrough inspection area and northward. Through the farmland (approximately Chainage 180 – 320 m), the stream is less defined and swampy.

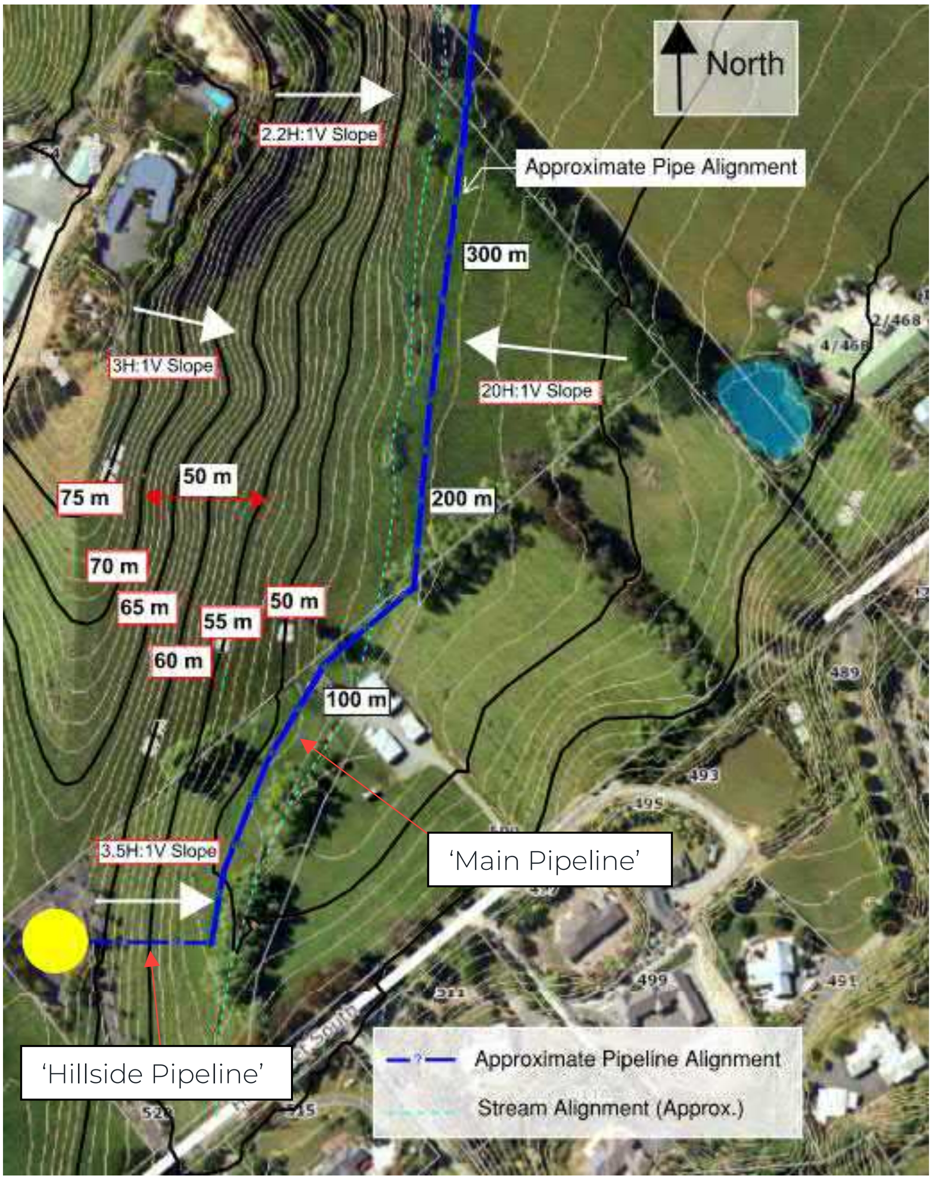

Figure 2. Approximate Richmond South Pipe Alignment (Southern Section & Reservoir Site)

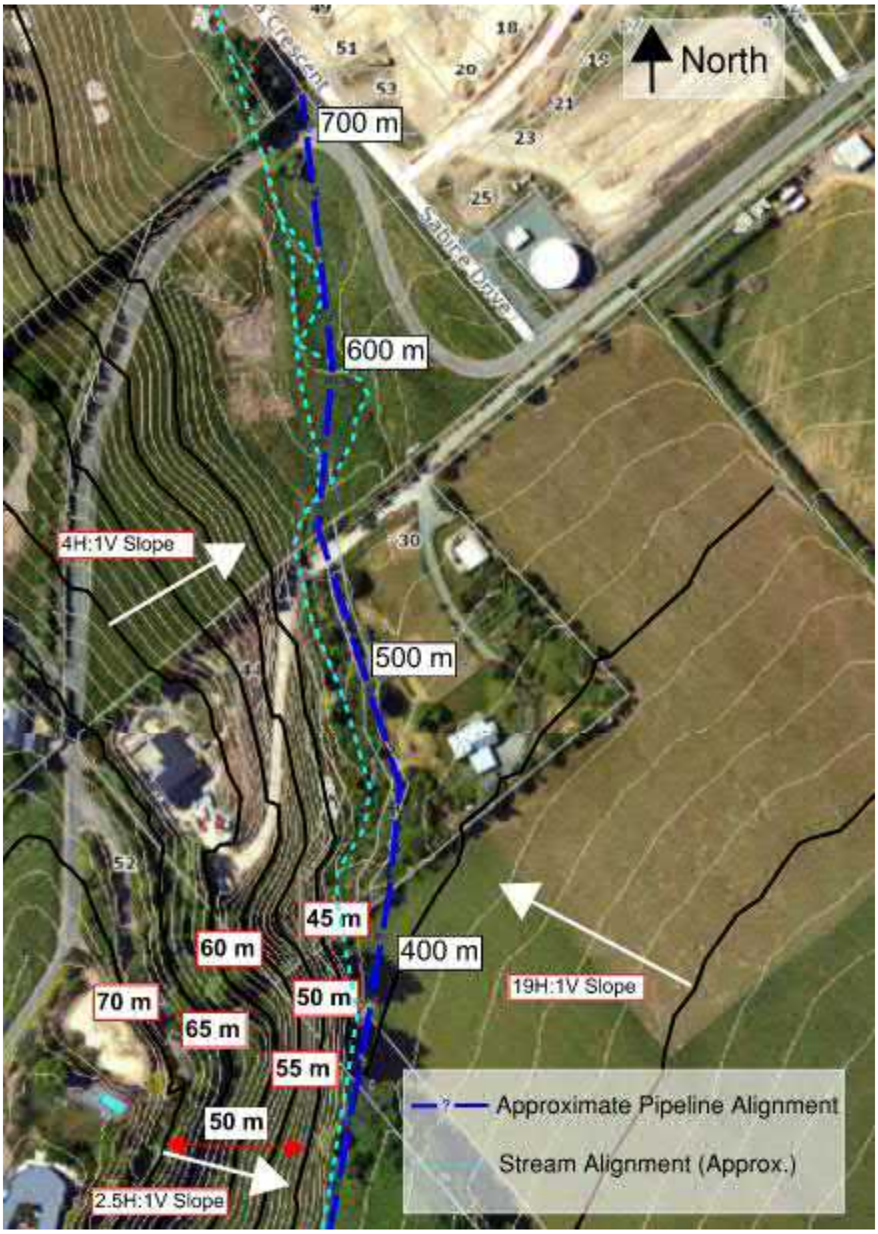

Figure 3 - Approximate Richmond South Pipe Alignment (Northern Section).

4 Existing Information

4.1 Published Regional Geology

The published geological map (Rattenbury et al, 1998) shows the site to be underlain by two surficial geological units as shown in Figure 4.

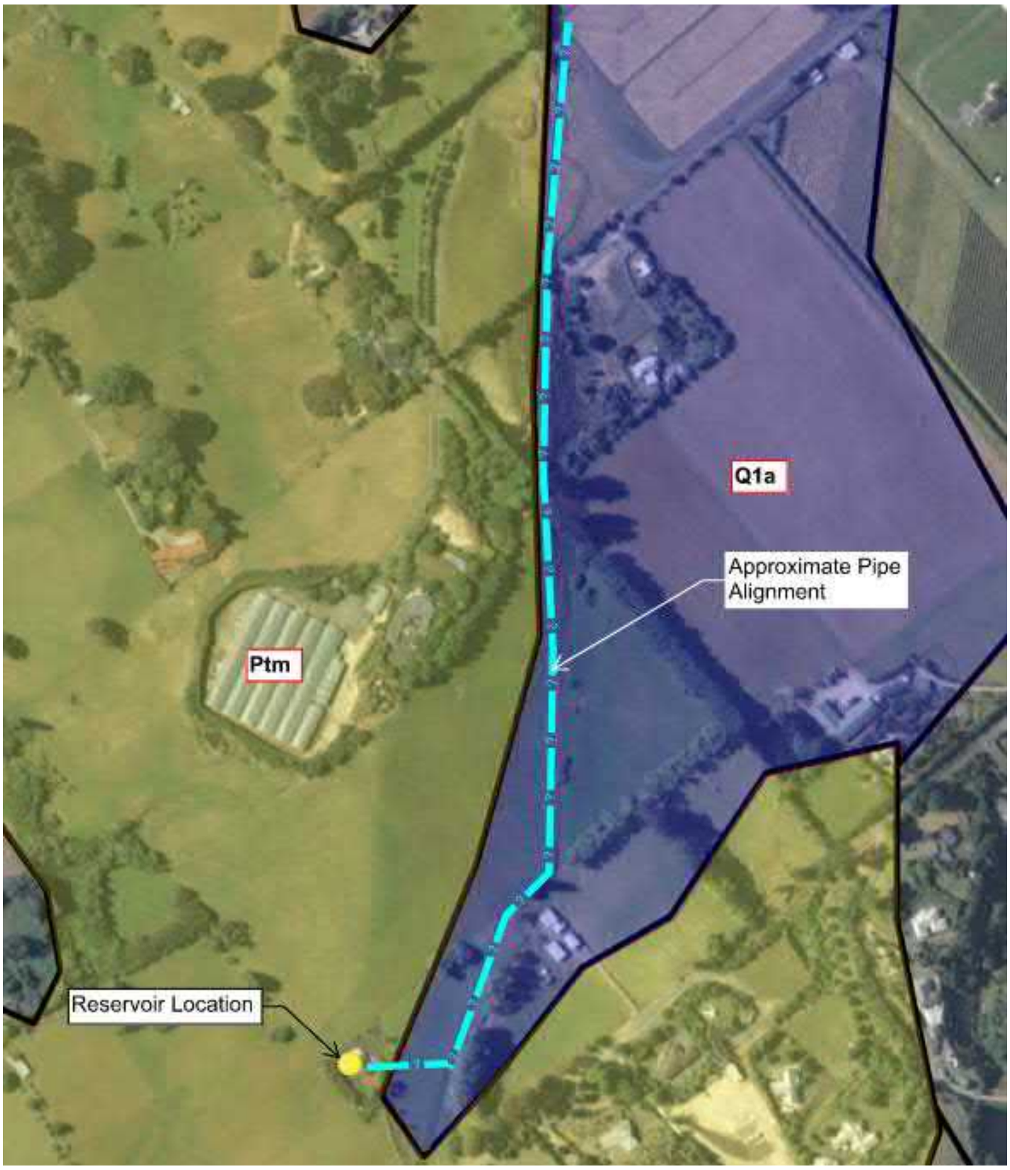

Figure 4 - Published Geological Map. Retrieved 22/6/20 from https://data.gns.cri.nz/geology/

The existing dwelling and location of the proposed reservoir is underlain by the Moutere Gravel Formation of the Tadmor Group. This formation contains poorly to moderately sorted clay-bound gravel dominated by quartzofeldspathic sandstone clasts (PTm).

The flats to the east are underlain by alluvial deposits consisting of poorly sorted gravels forming alluvial fans screes and colluvial deposits (Q1a). Depending on the final alignment, the main pipeline may be located on these materials.

4.2 Review of Previous Reports

Previous site investigations and reporting have been undertaken throughout the various stages of the project on this site and a site 150 m to the north at 38 Hart Road which was originally proposed for the same development. These reports are;

- Stantec (2018a) Geotechnical Factual Report. Richmond South Reservoirs (38 Hart Road, Richmond). Stantec Ltd, Richmond, New Zealand.
- Stantec (2018b) Geotechnical Factual Report. Richmond South Reservoirs (520 Hill Street). Stantec Ltd, Richmond, New Zealand.

The relevant logs and location plan from the Stantec Geotechnical assessment is displayed in Appendix E.

4.2.1 Stantec (2018a) 38 Hart Road.

Two boreholes were drilled at a location 150 m north of the currently proposed reservoir site. The boreholes were drilled to 19.5 m and 10 m below ground level respectively and standard penetration tests (SPT's) were undertaken at 1.5 m intervals. Piezometers were installed in both bores.

The boreholes show the site to be underlain by 0.3 – 0.4 m of topsoil with firm silty clay and sand to between 1.0 and 1.5 m depth underlain by dense to very dense clayey gravels (Moutere Gravels) to at least 19.5 m depth.

Raw SPT N values are generally shown to range between 40 and 50+ in the Moutere Gravels.

Groundwater was encountered at 6.9 m and 2.2 m below ground level in the boreholes on 9 April 2018 using measurements from Casagrande standpipe piezometers. Screen piezometer zones were set between 14.5 and 19.5 m for BH01 and 4.7 and 9.7 m for BH02.

4.2.2 Geotechnical Factual Report. Richmond South Reservoirs (520 Hill Street, Richmond).

Four test pits were excavated within the bounds of the currently proposed site to confirm the underlying geological conditions.

The test pits show the site to be underlain by a 0.2 to 0.3 m thick layer of topsoil with firm silty clay to between 1.0 and 1.4 m depth underlain by clayey gravels (Moutere Gravels) to at least 2 m depth.

Scala penetrometer testing was undertaken in the test pits. Scala penetrometer blow counts are reported as being in the range 1 – 6 blows / 100 mm within the firm silty clays and in the range of 5 to 20 + within the gravels.

Groundwater was not encountered in any of the pits.

4.2.3 WSP (2021) Design Life Comparison Geotechnical Memorandum

A Geotechnical memorandum was prepared with the purpose of providing advice to TDC around the implications of adopting a 100 year seismic design life over a 50 year design life. The report utilised the ground and site models developed at the time (i.e - the two tank option models) to compare the additional lateral displacements and whether it would drive the need for slope support, particularly in the eastern slopes. The report concluded that

- A geogrid reinforced gravel raft is required under the reservoirs to protect them from damage due to horizontal ground displacement. This is required regardless of the design life adopted.
- The vehicle accessway to and around the reservoirs may suffer cracking and not be traversable by vehicles following a ULS event. Cracks may be filled post event to quickly restore vehicle access however consideration should be given to geogrid stabilisation of the accessway in detailed design.
- The potential for vertical and horizontal displacement are considered in the selection of alignment and detailed design for the pipeline.

4.3 Active Faults

The locations and characteristics of active faults in the region have been obtained from the GNS Active Faults Database and the National Seismic Hazard Model (Stirling et. al, 2012).

The Alpine Fault is the most significant active fault to the Tasman region and is mapped as being 36 km south east of the site and is recorded in Stirling et al (2012) as having an average return period of 620 years and being capable of producing earthquakes of Magnitude, M_w = 7.7.

The Waimea – Flaxmore Fault system splits off the Alpine Fault at Lake Rotoiti and extends NNE along the eastern Tasman region towards Nelson City. The system consists of several faults including the Waimea, Flaxmore, Heslington and Whangamoa Faults. (Johnston, 2013).

The Waimea Fault is recorded in Stirling et al (2012) as having a return period of 5,570 years and being capable of producing earthquakes of magnitude 7.0. At the closest point, it is located 0.5 km south east of the site. The active faults map shows a gap in the fault trace within 0.5 km of the site. It is not known where the fault trace extends beyond the gap. There is no information available on the other faults in the Waimea – Flaxmore Fault system.

There are no active faults are mapped within the site and the likelihood of surface rupture affecting the site is assessed as low. However, due to the close proximity of the site to the Waimea and Alpine Faults, there is potential for strong ground shaking in the event of a rupture of these faults.

4.4 Liquefaction Hazard in Nelson and Tasman

An assessment of potentially liquefiable soils in the Nelson and Tasman region is presented in Johnston (2013).

This report identifies several geological deposits in the region which may be potentially vulnerable to liquefaction based on their age, composition, deposition and depth relative to the water table. The extents of these deposits are mapped within the region.

The recent alluvial deposits located underneath the main pipeline alignment are identified by Johnston (2013) and assessed by WSP as being potentially liquefiable where loose non-plastic silts and sands are encountered below the water table level.

The liquefaction potential of the Moutere Gravels is identified by Johnston and assessed by WSP as being low due to the dense nature of the materials.

5 Site Investigations

Site investigations in the form of two boreholes and eight test pits have been undertaken by WSP within the property at 520 Hill Street South and along the pipeline alignment to confirm the ground conditions.

In addition to this, a walkover inspection was completed in an area along the pipeline which was inaccessible by an excavator.

The locations of the WSP investigations undertaken are presented in Appendix A.

5.1 Boreholes

Two boreholes (WSP_BH01 and WSP_BH02) were drilled by CW Drilling between 22nd June and 29th June 2020 to using Hanjan D8 rig and PQ triple tube coring. The boreholes were drilled to 10 m and 20 m depth respectively.

A piezometer was installed in WSP_BH02 for the purpose of groundwater monitoring.

Core samples retrieved from boreholes were photographed and logged onsite by either a WSP Engineering Geologist or Geotechnical Engineer.

The logs of the boreholes are presented in Appendix B.

5.2 Test Pits

Eight test pits were completed along the alignment of the pipeline and access road. A summary of the test pits is presented in Table 1.

Table 1 - Test Pit Summary

Test Pit ID	Purpose	Achieved Depth (m)	Date Completed
TP01	Subdivision road and pipeline (South Section)	2.0 (Target depth reached)	22/5/20
TP02	Subdivision road and pipeline (South Section)	2.1 (Target Depth Reached)	22/5/20
TP03	Subdivision road and pipeline (South Section)	2.1 (Target Depth Reached)	22/5/20
TP04	Earthworks design for fill slope	2.1 (Target Depth Reached)	22/5/20
TP05	Earthworks design for fill slope and Hill side pipeline	2.1 (Target Depth Reached)	22/5/20
TP06	Pipeline (North Section)	3.8 (Target Depth Reached)	21/7/20
TP07	Pipeline (North Section)	3.3 (Target Depth Reached)	21/7/20
TP08	Pipeline (North Section)	2.7 (Excavator refusal)	21/7/20

All test pits were excavated using a 14-tonne excavator which was provided by Fulton Hogan. Soils recovered from test pits were logged onsite by either a WSP Engineering Geologist or Geotechnical Engineer.

The test pit logs are presented in Appendix C.

6 Ground and Groundwater Conditions

6.1 Ground Conditions

Ground conditions encountered onsite and engineering soil parameters for use in preliminary geotechnical assessment are presented in Table 2.

Table 2. Ground Parameters at Reservoir Site and under Hillside to the East

Soil Unit	Depth (m)	Thickness (m)	Description	Engineering Design Parameters
1	0 – 0.1	0.1	TOPSOIL	N/A
2	0.1 – 1.3	1.0 – 1.2	SILT / Sandy SILT (Soft to firm)	γ= 19 – 20 kN/m^3 Φ = 24 - 26° c' = 2 – 3 kPa S_u = 12 - 50 kPa
3	1.1 – 20 (end of borehole)	18.7+	Clayey GRAVEL / Sandy GRAVEL. (Medium dense to very dense) [MOUTERE GRAVELS]	γ= 19 – 20 kN/m^3 Φ = 30 - 32° c' = 20 – 50 kPa E = 80,000 – 100,000 kPa S_u = 100 - 200 kPa
F	0 – 5.3	1.2 – 5.3	Engineered Imported Fill (Well graded, compacted granular fill)	γ = 20 – 21 kN/m^3 Φ = 35° c' = 0 kPa E = 50,000 kPa

Table 3. Ground Parameters at Main Pipeline Alignment

Soil Unit	Depth (m)	Thickness (m)	Description	Engineering Design Parameters
1	0	0.1 – 0.6	TOPSOIL	N/A
2	0.3 – 0.6 0.8 – 2.4	0.4 – 1.2 0.3 – 1.7	SILT / Sandy SILT (Soft to firm) Sandy CLAY/Gravelly CLAY (Soft to firm)	γ= 19 – 20 kN/m^3 Φ = 24 - 26° c' = 2 – 3 kPa S_u = 12 - 50kPa
3	0.9 – 1.7	0.3 – 1.8	Clayey GRAVEL / Sandy GRAVEL. (Medium dense to very dense) [MOUTERE GRAVELS]	γ= 19 – 20 kN/m^3 Φ = 30 - 32° c' = 3 – 5 kPa E = 80,000 – 100,000 kPa S_u = 100 - 200 kPa
4	2.7 – 3.1	0.3 – 0.7	Blue SILTS (Soft to firm) [ALLUVIAL]	γ= 17 – 18 kN/m^3 Φ = 24 - 26° c' = 2 – 3 kPa S_u = 12 - 25 kPa
5	3.0**	0.3	Blue silty GRAVELS (only encountered in TP07) [ALLUVIAL]	γ= 19 – 20 kN/m^3 Φ = 30 - 32° c' = 3 – 5 kPa E = 40,000 kPa

6.2 Groundwater Conditions

6.2.1 Surface Water

Flowing water was observed in the stream which runs alongside the main pipeline alignment and a pond with water in it is located 200 m to the north west.

Site contour maps (TOTSM, 2020) show the stream level to be approximately 12 m below the reservoir site and the pond to be 25 m below the site.

At the time of investigations there was no surface water observed at the reservoir site itself or in the paddocks alongside the stream which the pipeline alignment runs through.

6.2.2 Groundwater

The depth of groundwater encountered in the boreholes and test pits are displayed in Table 4. The location of WSP Borehole 2 was undertaken at approximately the same level as the proposed 2500 m^3 reservoir.

Table 4. Groundwater Table Level

Test	GWL (m. bgl)	Existing Ground Level (m)
BH01	1.10	61.9
BH02	3.26	62.6
TP01 (2.0 m)	Not encountered	51.50*
TP02 (2.1 m)	2.0 m	50.75*
TP03 (2.1 m)	2.0 m	48.75*
TP04 (2.1 m)	Not encountered	53.50*
TP05 (2.1 m)	Not encountered	56.50*
TP06 (3.8 m)	3.7	46.75*
TP07 (3.3 m)	1.1	46.75*
TP08 (2.6 m)	2.1	41.00*

*Test levels approximated from Top of the South Maps contours

7 Geotechnical Assessment

7.1 Site Subsoil Class

The reservoir site is underlain by Moutere Gravels which extend to 1000's of metres below ground level in some locations and have shear wave velocities in the range 600 – 1200 ms^{-1} (MacMahon et al, 2017).

In this assessment, it is assumed that the gravels underlaying the reservoir site are > 150 m thick and have shear wave velocity over this depth of 1000 ms^{-1} or lower giving the site a low amplitude natural period of greater than 0.6 s.

Based on the available geotechnical information, it is expected that the site is subsoil class D – deep or soft soils in accordance with the requirements of NZS 1170.5.

7.2 Design Seismic Loadings for Geotechnical Assessment

Design seismic loadings for the reservoir site stability and liquefaction assessment have been determined using unweighted peak ground accelerations in accordance with the Waka Kotahi New Zealand Transport Agency Bridge Manual 3rd Edition (2018) with the following key inputs

- Importance level 3
- 100-year design life
- Site subsoil class D
- Return periods in accordance with the requirements of NZS 1170.0

On 7th December 2021, the New Zealand Geotechnical Society (NZGS) released an updated Module 1 – Earthquake Geotechnical Engineering Guideline. The guideline provides interim peak ground accelerations recommended for use in design and identifies that the PGAs provided for site subsoil class C are also appropriate for site subsoil class D sites. These are given in brackets in Table 5.

We have evaluated the sensitivity to the interim PGAs provided by the NZGS and established that the liquefaction hazard and a slope displacement are not sensitive to the interim PGAs.

The PGA adopted in design should be confirmed by TDC.

The design seismic loads are summarised in Table 5.

Table 5 - Design Seismic Loadings (Class D Site) for Liquefaction and Slope Stability

Design Case	Return Period	Effective Magnitude	Peak Ground Acceleration (g)*
SLS	1 in 25 years	6.1	0.08 (0.10g NZGS**)
ULS	1 in 2500 years	6.1	0.60 (0.74g NZGS**)

*Peak Ground Accelerations in Accordance with NZTA Bridge Manual.
**Interim PGAs from NZGS (2021) are given in brackets.

7.3 Liquefaction Assessment

Potentially liquefiable soils include loose or soft sands, gravels and non-plastic silts below the water table. Liquefaction susceptible soils are typically Holocene age and recently deposited.

Liquefaction causes the soil to undergo a partial to complete loss of shear strength. It results in settlement of the soil through densification and ejection of liquefied soils to the surface and lateral spreading if it occurs near a free face or water body. It can cause total and differential settlement and tilting of structures. The severity of the effects of liquefaction depend the intensity and duration of ground shaking, soil density, plasticity, composition and depth of the groundwater table.

7.3.1 Reservoir Site and Eastern Hillside

The liquefaction potential of the soils at the reservoir site and eastern hillside slope has been evaluated using the standard penetrometer (SPT) based method presented in Boulanger and Idriss (2014) with fines content estimated based on the soil descriptors in accordance with NZGS (2005).

Liquefaction was not predicted in either of the serviceability limit state or ultimate limit state seismic events due to the dense nature of the gravels.

As the water table in the side slopes was observed to be below the top of the dense gravels, liquefaction is not expected to occur in the side slopes.

7.3.2 Main Pipeline Alignment (Alongside Stream)

Potentially liquefiable soils were encountered in test pits TP06 and TP07 (North Section of main pipeline) below the water table.

Both of these test pits encountered non-plastic blue silts at a depths > 2.7 m, as displayed in Figure 5. A summary of the potentially liquefiable layers and materials are displayed in Table 6.

Table 6 - Potentially Liquefiable Layer Summary

Test Pit	Depth	Soil Description	Assessment
TP06	3.7 to end of hole at 3.8 m	Sandy SILT; bluey grey, partially orange in small areas. Moist, soft, low-plasticity, silt sticks to hands. At 3.3 small amounts of vegetation noticed in hole, seepage starts occurring at 3.7 m. Note, soil layer starts at 3.1, however groundwater not encountered until 3.7 m. Test pit terminated in this layer so potentially liquefiable soil extends below this depth.	Potentially liquefiable below GWT at 3.7 m.
TP07	2.7 - 3.0 m	SILT with some gravels; blue. Saturated, tightly-packed, very soft, low-plasticity. Gravels, fine, sub-angular to angular.	Potentially liquefiable silt beneath groundwater table of 1.1 m.
TP1-5 & TP8		No liquefiable materials encountered.	Soils encountered below GWL at 2.0 m depth or deeper are dense gravels.

Figure 5. Potentially Liquefiable Blue Silts

7.3.3 Effects of Liquefaction on Pipelines

Effects of liquefaction on buried structures such as pipelines, include total and differential settlement, changes in pipe gradients, buoyancy (which can exacerbate differential settlement), pipe rupture and silt intrusion. Liquefaction can also cause settlement and tilting of above ground structures such as connecting pump stations and cause damage at pipeline connection points.

Lateral spreading movement of the ground towards a free face such as the stream can occur and cause shearing, bending or rupture of the pipeline.

The severity of liquefaction impacts on the pipeline will depend on the proximity of the pipeline relative to the potentially liquefiable layers and free faces and the thickness and lateral extents of the layers.

The potential for liquefaction and lateral spreading impacting the pipeline will need to be considered in design. However, current investigations suggest a 2.7 to 3.0 m thick non-liquefiable surficial crust below the flats and significant spreading is unlikely as the liquefiable soils are located approximately 1.2 m below the stream invert. If the pipe is located within the crust and free draining backfill is used, then only minor adverse liquefaction effects would be expected. Lateral spreading effects could be mitigated by moving the pipe away from the stream.

Further site-specific investigations in the form of cone penetrometer tests and / or boreholes with laboratory testing are required to confirm the extent and thickness of liquefiable materials and potential for damage near potentially liquefiable soils. It is recommended that the potential for instability in the surrounding slopes due to liquefaction is considered in design.

7.4 Bearing Capacity and Settlement of Reservoir

7.4.1 Bearing Capacity - Reservoir

In this assessment, the reservoir has been assumed to be 26 m in diameter founded 0.5 m below the surrounding ground level. The applied load from the full reservoir is estimated to be 110 kPa. There may be higher loads applied to the ring beam under the walls or under columns. This needs to be considered in detailed design.

Borehole BH01 and BH02 show the reservoir locations to be underlain by 1.1 to 1.5 m of soft silt. These silts are considered unsuitable for founding the reservoir on due to the soft and variable nature and potential for long term settlement. It is recommended that the silts are removed underneath the reservoir extending for a minimum of 2 m outside the perimeter. Undercutting should be completed to a uniform level platform across the footprint to minimise the potential for variation in strengths underneath the reservoir.

Well-engineered fill will need to be placed to achieve the required site levels and bearing capacity without excessive settlement. It is recommended that the fill within the vicinity of the reservoir is an imported well graded free draining granular fill and is well compacted to minimise the potential for differential settlement. Close monitoring and testing will be required to ensure this is achieved in construction.

A strength reduction factor of 0.45 should be applied to account for variability in the underlying soil material properties.

A long term static ultimate bearing capacity of > 300 kPa has been determined for the reservoir, giving a design bearing capacity of 135 kPa.

Seismic loading is unlikely to affect foundation bearing capacity, but this will need to be considered in design

7.4.2 Static Settlements

Due to the granular and dense nature of the underlying soils, settlements are expected to occur relatively quickly as the reservoir is filled and emptied with limited potential for ongoing settlement.

Immediate static settlements underneath the reservoir assuming a rigid reservoir footing are expected to be in the order of 15 to 30 mm under the reservoir with up to 20 mm of differential settlement expected between the centre and edges of the reservoir.

7.5 Slope Stability

7.5.1 Reservoir Site

The potential for slope instability at the reservoir site has been assessed using the software package Slope/w 2021 by GeoStudio using limit equilibrium methods and the material parameters outlined in Table 2.

Potential slope displacements have been assessed using Newmark based methodologies as presented in Ambrasey and Srbulov (1995) and Jibson (2007). 50th percentile probability of exceedance displacements have been estimated.

The Newmark sliding block methodology adopted for our analysis assumes rigid, perfectly plastic behaviour. This assumption is not completely accurate as it does not account for changes in material strength during an earthquake or deformation within the sliding block itself. Slope displacement results are expected to have an accuracy of 50% to 200% of the values given.

Cases assessed in the analysis are presented below in Table 7.

Table 7: Slope stability cases assessed.

Design Case	Description
1. Static	Static slope stability using drained soil parameters. A 90 kPa surcharge applied across the tank base (9 m of water head) with an additional 46 kPa applied to 2 m wide perimeters footings to account for static loading.
2a. Crane access individual wheel tracks	Static slope stability using drained soil parameters. Localised crane wheel loading imparted when the crane travels adjacent to the crest of the geogrid tied back retaining wall. 13.2 tonne of loading per axle has been assumed. No surcharge from the tank itself has been considered.
2b. Crane access 15 kPa surcharge	As per case 2a but with a 15 kPa surcharge applied across the access way in front of the tank rather than localised wheel loadings.
2c. Crane lifting 26 kPa surcharge	Static slope stability using drained soil parameters. 26 kPa surcharge applied over an 8 m length to account for the load imparted while the crane is lifting at maximum 100 tonne capacity. 66 tonne self-weight (tare) of the crane also accounted for. Surcharge offset 5 m from the crest of the retaining wall. No surcharge from the tank itself has been considered.

Design Case	Description
3a. SLS (0.08 g)	A pseudo-static seismic analysis using undrained strengths for cohesive soils and drained parameters for cohesionless soils. SLS seismic loading (0.06g). A 90 kPa surcharge applied across the tank base (9 m of water head) with an additional 106 kPa applied to 2 m wide perimeter footings to account for inertial seismic loading imparted by the tank during a seismic event.
3b. ULS (0.60 g)	As per case 3a but with ULS seismic loading (0.60g) and a 106 kPa applied to 2 m wide perimeters footings.

Two cross sections through the tank have been assessed. The locations of the sections are shown in Appendix F. Section 1 considers the shortest distance between the tank edge and the top of the retaining wall (6 m). Section 6, however, considers a greater off set between the tank and the retaining wall (9.4 m), but a geogrid tied back retaining wall with an increased retained height (~2 m).

The potential for stretch and differential settlement across the tank footprint in design has been evaluated by comparing the predicted displacements at different positions within the tank footprint. The assessed locations are:

- Global movement of entire site (behind tank – typical surface 1 and surfaces to the left)
- Slip surfaces affecting the western half of the tank (LHS inner tank – between typical surfaces 1 and 2)
- Slip surfaces affecting the eastern half of the tank (RHS inner tank – between typical surfaces 2 and 3)
- Slip surfaces affecting the soil in east of the tank (In front of tank – typical surface 4 and any surfaces to the right).
- For the purpose of slope stability relating to the hillside pipeline, slip surface 5, below the retaining wall has also been considered.

These are displayed Figure 6.

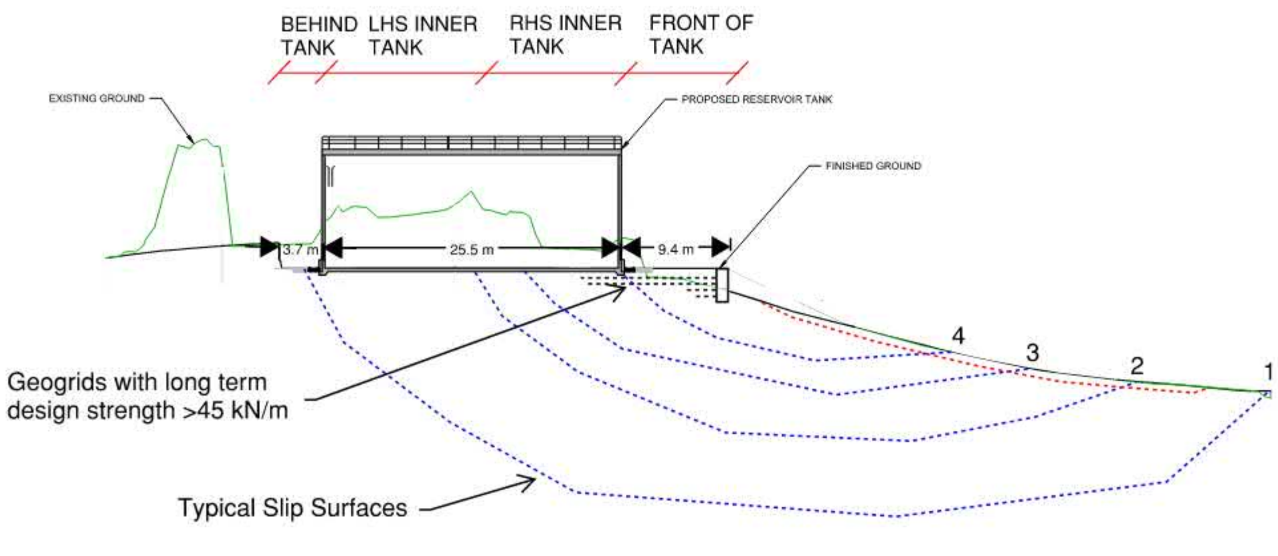

Figure 6. Tank Assessment Locations (Section 6 shown).

The stabilising effect of the eastern retaining wall and associated geogrids has been included in global stability analysis. Geogrids with a long term design strength of 45 kN/m have been modelled.

Slope stability analysis has been performed on the eastern side of the ridgeline only as it is steeper (and therefore more critical) than the slope to the west of the ridgeline. It is possible for seismic displacement to occur on either or both slopes and exacerbate stretching across the reservoir footprint. It is recommended that the potential for slope movement in both directions is considered in reservoir design.

The outcomes of the slope stability analysis are presented in Table 8 and Table 9 Slope/w outputs are presented in Appendix D.

Table 8 - Slope Stability Assessment Outcomes (Long Section 1)

Case	Behind Tank (Slip Surface 1)	LHS Inner Tank (Slip Surface 2)	RHS Inner Tank (Slip Surface 3)	In front of tank (Slip Surface 4)	Side slope below retaining wall (Slip Surface 5)
1. Static (FoS) [Critical Acceleration (g)]	3.04 [0.52]	2.87 [0.54]	2.85 [0.67]	3.75 [1.04]	1.66 [0.16]
2a. Crane access individual wheel tracks FOS	Not Applicable	Not Applicable	Not Applicable	5.03	Not Applicable
2b. Crane access 15 kPa surcharge FOS	Not Applicable	Not Applicable	Not Applicable	6.94	Not Applicable
2c. Crane lifting 26 kPa surcharge FOS	Not Applicable	Not Applicable	6.54	Not Applicable	Not Applicable
3a. SLS (0.08g) FOS Displacement (mm)*	2.38 Nil	2.37 Nil	2.72 Nil	4.01 Nil	1.26 Nil
3b. ULS (0.6g) FOS Lateral Displacement (mm)*	0.93 < 5 mm	0.92 <5 mm	1.08 Nil	1.47 Nil	0.44 80** mm

* Ambrasey and Srbulov (1995) and Jibson (2007)
** Range of deformation (-50% to +100%)

Table 9 - Slope Stability Assessment Outcomes (Long Section 6).

Case	Behind Tank (Slip Surface 1)	LHS Inner Tank (Slip Surface 2)	RHS Inner Tank (Slip Surface 3)	In front of tank (Slip Surface 4)
1. Static (FoS) [Critical Acceleration (g)]	3.15 [0.58]	3.00 [0.60]	2.98 [0.75]	3.98 [1.24]
2a. Crane access individual wheel tracks FOS	Not Applicable	Not Applicable	Not Applicable	2.68
2b. Crane access 15 kPa surcharge FOS	Not Applicable	Not Applicable	Not Applicable	4.19
2c. Crane lifting 26 kPa surcharge FOS	Not Applicable	Not Applicable	4.33	Not Applicable
3a. SLS (0.08g) FOS Displacement (mm)*	2..52 Nil	2.54 Nil	2.87 Nil	4.41 Nil
3b. ULS (0.6g) FOS Lateral Displacement (mm)*	0.98 Nil	1.00 Nil	1.18 Nil	1.67 Nil

* Ambrasey and Srbulov (1995), Jibson (2007)

It can be seen from the slope stability analysis that

- The static factors of safety for slip surfaces affecting the reservoir exceed the normally accepted value of 1.5 in all static cases.
- Lateral ground movement is not expected in serviceability limit state events.
- Differential lateral ground movement or stretch of up to 10 mm can be expected across the reservoir footprint during an ultimate limit state event. This may result in a similar amount of differential settlement. Detailed design of the tank foundation needs to consider the potential differential settlement and lateral displacement.
- Downslope ground movement may occur within shallow silt (Unit2) on the side slope east of the reservoir of up to 80 mm (and possibly up to 160 mm) in the ULS case. The hillside pipeline linking the reservoir to the main water pipeline will be located within this slope and we recommend that it is formed within the Moutere Gravels below the relatively soft silt/sandy silt layer, at a provisional depth of > 1.5m.

The displacements reported in Table 8 and Table 9 are horizontal only. The vertical component of these movements needs to be considered in detailed design.

7.5.2 Pipeline Alignment (including walk through area)

The natural slopes observed onsite and, in the area, typically stand at 25 – 30° with some cut slopes exceeding 45° degrees in localised areas. There was no evidence of significant instability observed in the walkover inspection or anywhere else along the pipeline alignment. An image of the typical walkthrough area is displayed in Figure 7.

Figure 7. Walk through area slope

Gravels, silts and sands were observed in the walls of the stream, as displayed in Figure 8. At the time of the inspection, water was flowing through the stream, indicating groundwater is likely to be relatively shallow.

Figure 8. Stream Bank Materials

Moutere Gravels were observed in the slopes forming the accessway to the property at 52 Cupola Crescent and it is expected that these gravels also underlay the majority of the walk through inspection area, refer to Figure 9. This geology is consistent with the test pits undertaken nearby.

Figure 9. Slope at 52 Cupola Crescent Accessway

Given the slopes onsite appeared relatively stable, it is expected that there shouldn't be any slope stability issues in static cases provided any new cuts are constructed at similar profile to the existing cuts.

Seismic slope stability will need to be considered in design of new cut slopes, but similar cut profiles should prove adequate.

7.6 Earthworks

Earthworks required at the reservoir site include

- Undercutting of approximately 2 m of soft surficial silts and Moutere gravels to form a level platform beneath the reservoir.
- Construction of a geogrid tied back gabion basket retaining wall, up to 3 m high, with top geogrids (15 m long) that extend back beneath the reservoir footprint. This will be located to the east of the reservoir and access track.
- Backfilling with well compacted granular fill to form a suitable platform for founding the reservoir on and the reservoir access road and side slope. It is recommended that fill is placed and compacted in layers of uniform thickness.
- Excavation associated with laying the pipeline and reinstatement of trenches.

The proposed earthworks at the reservoir site can be observed in the Tank Cross-Sections displayed in Appendix D. It may be possible to re-use some the gravels cut from underneath the reservoir in building up the access road provided moisture and workability tolerances can be met.

7.7 Retaining Walls

Two retaining walls at the site are discussed further below. Detailed design of both walls is required.

7.7.1 Eastern Downslope Retaining Wall

The current concept design shows a retaining wall of up to 3 m in height is required to support the access track around the eastern edge of the platform. This retaining wall will support the soil between the reservoir and the top of the slope and the reservoir access road.

A fill slope has been considered as an alternative to the eastern retaining wall. Large tree pits (measuring 1 m deep and 5 m wide comprising of loose organic soil) would have been required at 5 m centres high in the fill slope to allow for growth of screening trees amongst the dense fill slope material. Due to the surcharge imparted by large 100t cranes trafficking near the top of the fill slope and the presence of relatively soft soil within the tree pits, this option has been discounted.

A mechanically stabilsed earth (MSE) wall is proposed for the following reasons:

- A relatively high retained height (up to 3 m) and high seismic loads will likely make pure cantilevered walls unfeasible.
- The MSE wall can be constructed in parallel with filling operations.
- Extension of the top geogrids beneath the base of the reservoir (15 m long) will limit seismic induced stretch across the reservoir (discussed in section 7.5.1)

The MSE wall will need to be founded on the underlying Moutere gravels to satisfy bearing capacity requirements.

Seismic deformation and surcharge loading from heavy construction plant will need to be considered in detailed design.

A safety handrail around the top of the wall for personal safety would likely be required.

7.7.2 Western Upslope Retaining Wall

A retaining wall is also shown on the western side of the reservoir, up to ~2.5 m high. We understand that there are several established trees, which are intended to be retained, with driplines offset ~ 2 m from the top of the wall. As such, excavation behind the wall will need to be kept to a minimum.

A cantilevered soldier pile wall embedded into the Moutere gravels is proposed. Bored concrete encased timber piles could be considered; however it is likely that groundwater will be encountered which may mean that a casing is required to keep the hole open. An alternative pile option is driven universal steel columns, which could penetrate the Moutere gravels without the need for auguring.

7.7.3 Backfill

Granular free draining well graded backfill should be placed behind the walls and used to build the ground between the wall and reservoir up to design levels.

Reuse of Moutere gravels that will arise from cutting beneath the reservoir footprint could be considered as an alternative backfill material behind the walls. The Moutere gravels have a high fines content and therefore relatively low permeability, which could result in water pressure building up behind the walls. Active drainage such as a network of strip and subsoil drains would need to be provided at the interface between natural ground and fill. All water captured by the drains should be directed to an appropriate discharge point.

7.8 Water Stops

We understand that the pipeline will be founded between 1.2 m and 2.1 m below ground level and have gradients ranging from ~2 % (~ 700 m length when aligned adjacent to the stream) to ~29 % (in the 60 m length of hillside pipeline between the reservoir and adjacent stream). Groundwater was encountered between 1.1 m and 3.26 m below ground level during test pit and borehole investigations. As such, there is potential for groundwater flow along the pipe bedding material.

Groundwater flow through permeable bedding material can lead to the migration of soil particles and bedding scour, ultimately leading to loss of support beneath the pipeline and differential settlement.

Water stops, consisting of concrete collars, be formed along pipelines to significantly reduce the flow of water through bedding material.

Water stops should be formed along the main pipeline and hillside pipeline in accordance with the guidance provided in Nelson- Tasman Land Development Manual (NTLDM). Based on Table 5-16 of the NTLDM, water stops should have a maximum spacing of 12 m and 30 m for grades steeper than 6.5% and 2% respectively. The pipeline designers should consider whether closers spacings should be adopted for the hillside pipeline located on the side slope below the reservoir.

8 Constructability Considerations

8.1 Reservoir Site

8.1.1 Heavy Construction Plant and Traffic

It is expected that cranes and heavy construction traffic will be required to construct the reservoir and fill slopes. Concept drawings show crane pads to be located outside the reservoir footprint.

Consideration will need to be given to crane pad bearing capacity and stability of the nearby retaining wall and slopes in design and construction planning. The potential for applied loads from cranes will need to be taken into account in retaining wall design.

It is recommended that cranes or any plant which imparts a large load on the ground is positioned as far back from the edge of the slopes and retaining wall as practicable.

8.1.2 Groundwater

The groundwater table was generally encountered near the top of the Moutere gravels in site investigations and is likely to be encountered during undercutting for the reservoir platforms. It is not known if the ground water encountered in the piezometer in WSP_BH02 is the static groundwater table or a flow path across the top of the gravels and down the hill.

It is recommended that a test pit to 5 m depth is excavated at the reservoir location to confirm the groundwater regime at the reservoir site. The existing Stantec Test Pit TP04 conducted to a depth of 4.0 m did not find groundwater.

The groundwater may be managed during construction through diverting it away from the site or formation of trenches or sumps around the perimeter of the excavation in order to maintain a dry working platform. Water could then be piped off site in a controlled fashion with appropriate controls and consents while meeting any other project requirements. Geofabric may be required to maintain separation of the compacted fill and underlying Moutere gravels.

Long term drainage of the fill should be incorporated by installing a drainage blanket and/or a network of subsoil drains with an appropriate discharge point.

8.2 Main Pipeline Alignment

8.2.1 Ease of Excavation

At test pits TP01 to TP07, the 14-tonne excavator was able to reach the target depth with relative ease.

At test pit TP08, the excavator refused in the gravels below 2.6 m depth indicating the ground is very hard. It can be expected that excavation through these gravels will be slow and a large capacity excavator may be required for excavation of gravels along the pipeline alignment.

8.2.2 Groundwater and Trench Collapse

At test pit TP07, the static groundwater depth was encountered at approximately 1.1 m depth. During the test pit, rapid seepage was noticed (refer to Figure 10) causing the hole to rapidly fill with water. It is possible that strong seepages will be encountered at other locations over the alignment and the Contractor should be made aware of this and groundwater control measures. Dewatering will be required during the pipeline construction in this area, if the pipeline is to be founded below the groundwater table.

Test pit TP06 also started showing significant signs of collapse in the clayey gravels at approximately 1.8 m. It is likely that temporary trench support in the form of shoring or trench shields will be required during construction in this area and other areas with similar conditions.

Figure 10. Groundwater at Test Pit TP07 showing spring in side wall

8.2.3 Soft Surficial Soils

The surficial soil near the existing stream (near TP06 and TP07) was extremely soft and saturated and the excavator encountered difficulty tracking in this area and became stuck. The contractor may need to form temporary access tracks through these areas possibly with geofabrics and running course to allow construction traffic to pass over top.

8.2.4 Use of Excavated Material as backfill

Tasman District Council Standard Specification 1112 Earthworks (Pipe Trenches) states the following material requirements for backfilling outside carriageways.

a) Unless otherwise instructed by the Engineer, material for backfilling outside carriageways shall be sound material excavated from the trench.

b) If, in the opinion of the Engineer, the site material is unsuitable, suitable material shall be imported by the Contractor at rates to be agreed in advance.

c) Under footpaths and vehicle crossings, the material in the upper 100mm shall be GAP 65 sub-base complying with Clause 2.1.1.

d) Under berms and outside road reserves, the material in the upper 100mm shall be topsoil selected from the trench excavation.

Based on these requirements and the rural location of the pipeline, it is expected that some of the material excavated from the trench may be suitable to be reused as backfill if the moisture content is below the optimum moisture content. Limited subsidence and low bearing capacity from the backfill can be expected.

The trench should be over filled then left for a period of time then rolled to minimise the potential for subsidence. Alternatively, gravel backfill and possibly a geogrid wrapped gravel raft may be constructed in areas of soft pipe subgrade.

Topsoil should be stock piled then respread upon trench reinstatement.

To reduce construction traffic loads on the pipeline, it is recommended that a running course of greater than 500 mm of compacted gravel is placed where the accessway crosses the backfilled trench.

8.3 Pipeline Alignment Up Hillside Slope

8.3.1 Ease of Excavation

All test pits in this region were excavated using a 14t excavator and reached the target depths (between 2.1 m and 3.8 m bgl) with relative ease.

8.3.2 Groundwater and Trench Collapse

Test pits located higher on the side slopes (TP04, TP05) were dry at a target depth 2m bgl and had encountered the Moutere gravels. Test pits lower down on the slope adjacent to the stream channel (TP02, TP03) encountered groundwater at 2 m bgl, approximately the same elevation as the Moutere gravels. The test pit sidewalls were relatively stable with no sidewall collapses.

8.3.3 Hillside Stability

There is potential for access tracks or other temporary works to affect hillside and hence reservoir stability during and after construction. This has not been addressed specifically in this report but needs to be considered in developing the construction methodology.

9 Conclusions and Recommendations

9.1 Reservoir and Hillside Pipeline

Based on the site investigations and geotechnical assessment carried out it is concluded that:

- The area where the reservoir is located is typically underlain by a layer of soft silts and gravels overlaying Moutere Gravels.
- The potential for liquefaction and hence lateral spread at the reservoir site is low
- The static ultimate bearing capacity of the reservoir foundations is assessed to be > 300 kPa if 1.5 m of silt is removed and replaced with engineered fill.
- There is potential for static differential settlement across the reservoir footprint of up to 20 mm as the reservoir is filled and emptied provided the fill underneath is compacted appropriately.

- Lateral stretch of up to 10 mm may occur across the reservoir footprint in a ULS event, provided that two layers of geogrid extend from the gabion retaining wall back beneath the reservoir footprint (~15 m long).
- Lateral ground movement in the ULS case of 80 mm (an up to 160 mm) can be expected in the shallow Unit 1 materials on the side slope. The hillside pipeline should be formed within the more stable Moutere gravels, nominally >1.5 m deep .
- A retaining wall is required on the eastern slopes to support crane loads.

It is recommended that the following items are considered in detailed design:

- The potential for liquefiable soils impacting the pipeline and stability of the surrounding slopes warrants further site investigations to confirm the potential impacts of liquefaction.
- The impact of potential lateral and vertical ground movements of the platform and eastern side slope on the pipeline.
- Bearing capacity for tanks once tank loads are available.
- Impact of bearing loads from the tanks and crane loadings on the nearby retaining wall and around slopes.
- A test pit to 5 m depth is excavated at the reservoir location to confirm the groundwater regime at the reservoir site. The existing Stantec Test Pit TP04 conducted to a depth of 4.0 m did not find groundwater.
- Potential for differential settlement across the reservoir footprint.
- Water stops are provided along the pipeline are provided as a minimum in accordance with the NTLDM and are considered more specifically for the steeper side slope section of pipeline east of the reservoir.
- The potential for re-use of Moutere gravels excavated from under the reservoir as fill behind the retaining wall.
- Surcharge loads from construction traffic and seismic loads are to be accounted for in retaining wall design.
- Any large construction loads from crane footings are positioned as far back from the edge of the slope as practicable and specific assessment of the safety of crane loading and bearing capacity should be carried out by the Contractor.
- The construction methodology needs to consider the potential for temporary and long term works to destabilise the hillside slopes and reservoir site itself.

9.2 Main Pipeline (Alongside Stream)

Based on the site investigations and geotechnical assessment carried out it is concluded that:

- Soft silts overlay clayey/sandy gravels (Moutere Gravels) for most of the pipe alignment. Alluvial silts and gravels were encountered at the northern end of the alignment.
- The groundwater level in the lower flats ranges between 1.1 m and 3.3 m below existing ground level.
- Temporary groundwater control through dewatering and trench support will likely be required when constructing the pipeline.
- Soft surficial soils are present along the main pipeline route. Geofabric and running coarse aggregate will likely be required to form temporary access tracks.

It is recommended that the following items are considered in detailed design:

- Water stops are provided along the main pipeline are provided as a minimum in accordance with the NTLDM, but with specific consideration of the steeper pipeline section east of the reservoir.

- Potentially liquefiable soils were encountered in a some of the test pits along the main pipeline alignment. Lateral spreading to the stream could affect the pipeline in a seismic event and this may warrant further investigation and assessment.
- A running course of greater than 500 mm of compacted gravel is placed where the access track crosses the backfilled trench.

10 References

Ambraseys, N and Srbulov, M (1995). Earthquake Induced Displacements of Slopes. Soil Dynamics and Earthquake Engineering. 14 (1995). 59-71. Great Britain

Austroads (2004) Pavement Design Manual.

Johnston, M. R (2013) Revised Preliminary Assessment of the Liquefaction Hazard in Tasman and Nelson.

Jibson (2007) displacement, based on regression models for estimating coseismic landslide displacement, Engineering Geology 91 (2007) 209-218

Rattenbury, M.S.; Cooper, R.A.; Johnston, M.R. (compilers) 1998: Geology of the Nelson area: scale 1:250,000. Lower Hutt: Institute of Geological & Nuclear Sciences Limited. Institute of Geological & Nuclear Sciences 1:250,000 geological map 9. 67 p. + 1 folded map

Stantec (2018a) Geotechnical Factual Report. Richmond South Reservoirs (38 Hart Road, Richmond). Stantec Ltd, Richmond, New Zealand.

Stantec (2018b) Geotechnical Factual Report. Richmond South Reservoirs (520 Hill Street). Stantec Ltd, Richmond, New Zealand.

Tasman District Council, Nelson City Council (2020) Nelson Tasman Land Development Manual Revision 1