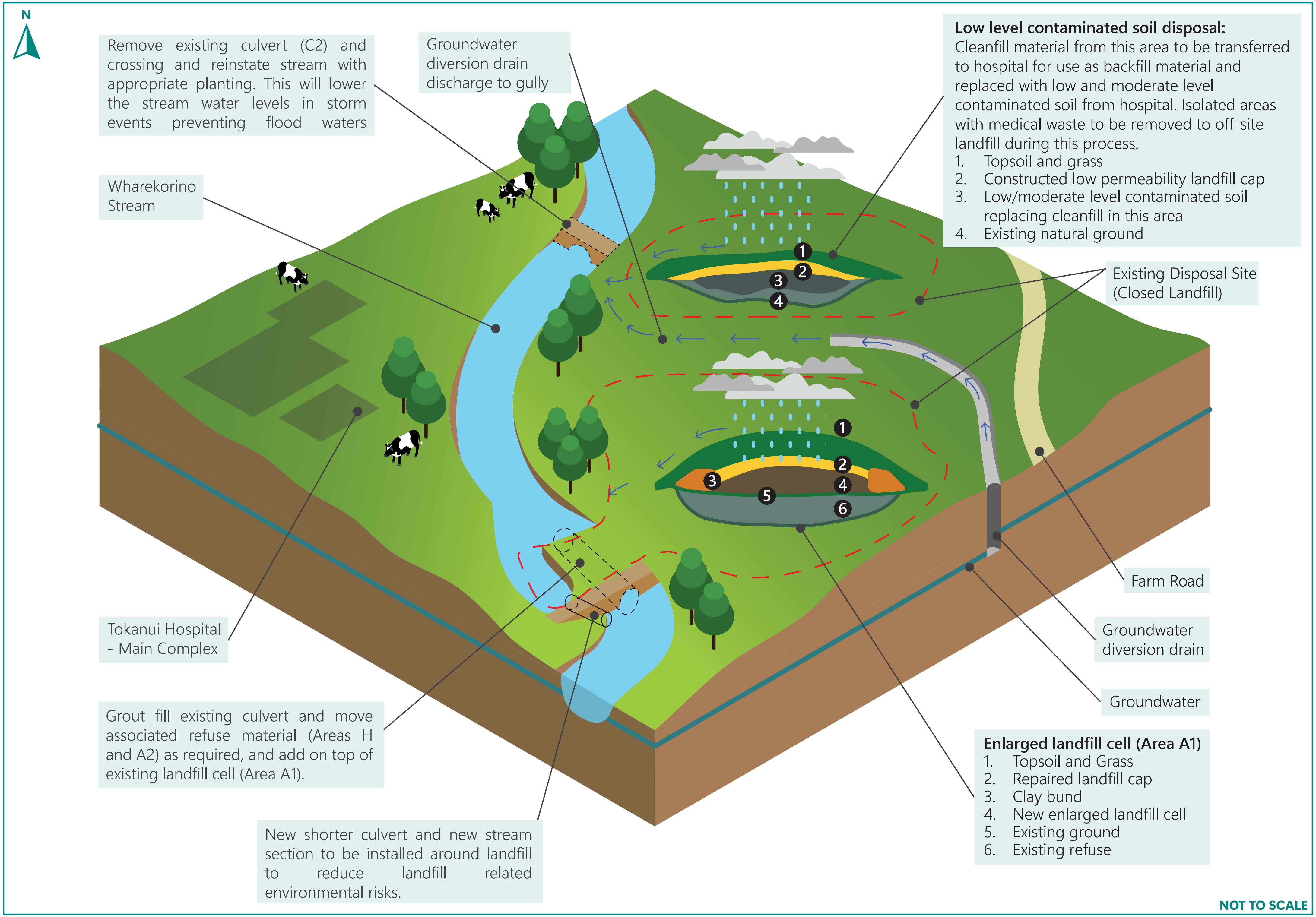
APPENDIX SF5: SCHEMATIC OF DISPOSAL SITE REPAIR/UPGRADE WORKS


N
Remove existing culvert (C2) and crossing and reinstate stream with appropriate planting. This will lower the stream water levels in storm events preventing flood waters
Groundwater diversion drain discharge to gully
Low level contaminated soil disposal:
Cleanfill material from this area to be transferred to hospital for use as backfill material and replaced with low and moderate level contaminated soil from hospital. Isolated areas with medical waste to be removed to off-site landfill during this process.
1. Topsoil and grass
2. Constructed low permeability landfill cap
3. Low/moderate level contaminated soil replacing cleanfill in this area
4. Existing natural ground
Wharekōrino Stream
Existing Disposal Site (Closed Landfill)
Farm Road
Groundwater diversion drain
Groundwater
Tokanui Hospital - Main Complex
Grout fill existing culvert and move associated refuse material (Areas H and A2) as required, and add on top of existing landfill cell (Area A1).
New shorter culvert and new stream section to be installed around landfill to reduce landfill related environmental risks.
Enlarged landfill cell (Area A1)
1. Topsoil and Grass
2. Repaired landfill cap
3. Clay bund
4. New enlarged landfill cell
5. Existing ground
6. Existing refuse
NOT TO SCALE

APPENDIX SF6: FTL MEMO (26 JUNE 2025), EFFECT OF CHANGES IN IMPERVIOUS AREAS FROM HOSPITAL DEMOLITION WORKS ON STREAM FLOWS

Fraser Thomas Limited
21 El Kobar Drive : East Tamaki : AUCKLAND 2013
PO Box 204006 : Highbrook : AUCKLAND 2161
NEW ZEALAND
TEL +64 9 278 7078 : E-MAIL sfinnigan@ftl.co.nz

Fraser Thomas
ENGINEERS • RESOURCE MANAGERS • SURVEYORS

MEMORANDUM

Date: 26 June 2025 33097

From: Y Goh, S Finnigan

Subject: Former Tokanui Hospital – Effect of Changes in Impervious Areas from Hospital Demolition Works on Stream Flows

To: Emily Buckingham (emily.buckingham@slrconsulting.com)

1.0 INTRODUCTION

This memo summarises our calculations of the effect of changes in impervious areas from hospital demolition works on stream flows.

2.0 CATCHMENTS

This assessment has been undertaken for the Wharekōrino Stream, based on the catchment areas shown below, with the downstream catchment boundary being taken as Te Mawhai Road. The hospital is almost entirely located within the western catchment of the stream (160ha), while the existing disposal site is contained within the southern catchment (440ha) of the stream.

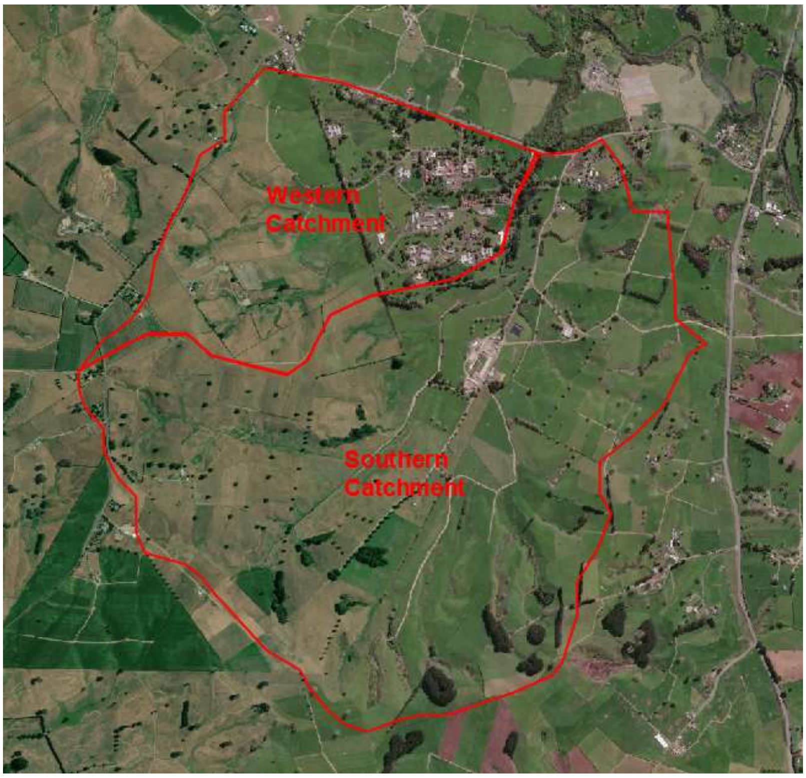


Figure 1: Catchment Locations

3.0 IMPERVIOUS AREA CHANGES

The hospital demolition works will result in the removal of approximately 6.2ha of impervious areas, of which 2.7ha of existing roading will be converted to farm gravel tracks. The landfill repair and upgrade works will not result in any changes in impervious area in the southern catchment.

Table 1: Impervious Areas – Pre and Post demolition works

Land Cover	Soil Group	CN	Western catchment (ha)		Southern catchment
			Pre-demo (ha)	Post-demo	Pre and post demo
Pasture	C	74	149.1	157.3	440
Impervious		98	10.9	0.0	
Gravel		89	0	2.7	0
Total			160.0	160.0	440

Notes:

1. Hospital existing impervious area made up of 3.65ha of buildings, 1.02ha of paving and 6.20ha of roading, totalling 10.87ha. Post-development, the entire site will be pervious except for 2.7ha of gravelled farm access tracks.
2. Other impervious areas outside of the site in the contributing catchments are relatively small and have been ignored in this assessment.

4.0 EFFECT ON BASE FLOWS

Stream base flows refers to the sustained, consistent flow of water that is not directly caused by recent rainfall. It is primarily sustained by groundwater discharge seeping into the stream channel over an extended period. It helps keep the stream flowing, even during dry periods. Base flows may also be referred to as low flows.

The hospital demolition works will reduce the impervious area from 10.9ha of paved surfaces to 2.7ha of gravelled farm track (considered as an impervious surface in this assessment). This means the pervious (grassed) cover across the western catchment will increase from 93.2% to 98.3%, an increase of 5.1%, which is significant.

In this situation, the increased pervious coverage means there will be increased infiltration through the reinstated topsoiled grassed areas (8.2ha), which will increase lateral groundwater flows towards the stream and hence help to increase base flows in the stream. The actual increase in base flows is likely to be less than the 5.1% increase in pervious cover.

5.0 EFFECTS ON STREAM FLOWS

HEC-HMS modelling has been used to calculate the effect of these changes in impervious areas on peak flows for different storm events, comprising the following:

- 1/3 2yr 24h storm – this event is 1/3 of the two year storm event (22.5mm, meaning it has a probability of occurring approximately once every 8 months. It is representative of common flows in the stream.
- 2yr, 10yr and 100yr 24h storms – these storms are representative of the 2yr (68mm), 10yr (101mm) and 100yr (151mm) storm events, which have a probability of 50%, 10% and 1% occurring in any given year.

The results are summarised in Table 2. This shows that combined peak flows and volumes at Te Mawhai Road are estimated to decrease by 2.5-3% for an 8 month storm and 0.8-1.5% for other more significant storm events. The actual decrease is likely to be less than this, as will be partially offset by increased base flows into the stream.

6.0 CONCLUSIONS

Overall, the reduction in impervious cover in the western catchment as a result of the site demolition works is expected to result in a small increase (up to 5%) in base flows (low flows) in the stream. There is estimated to be small decreases in stream flows and volumes at Te Mawhai Road for storm events, ranging from up to 2.5-3% for an 8 mth storm event to up to 0.8-1.5% for 2-100 year storm events.

Table 2: Peak Flow and Volume Results

Catchment	Item	1/3 of 2yr			2yr			10yr			100yr		
		Pre	**Post**	**Change**	**Pre**	**Post**	**Change**	**Pre**	**Post**	**Change**	**Pre**	**Post**	**Change**
South	Peak flow (m^3/s)	1.17	1.17	0.00%	11.54	11.54	0.00%	22.67	22.67	0.00%	41.83	41.83	0.00%
	Volume ($1000m^3$)	13.38	13.38	0.00%	115.04	115.04	0.00%	220.53	220.53	0.00%	400.4	400.4	0.00%
West	Peak flow (m^3/s)	0.51	0.46	-9.80%	4.73	4.49	-5.07%	9.13	8.78	-3.83%	16.58	16.10	-2.90%
	Volume ($1000m^3$)	5.42	5	-7.75%	44.33	42.28	-4.62%	83.92	80.87	-3.63%	150.75	146.54	-2.79%
Combined	Peak flow (m^3/s)	1.670	1.62	-2.99%	16.26	16.02	-1.48%	31.80	31.44	-1.13%	58.41	57.93	-0.82%
	Volume ($1000m^3$)	18.8	18.34	-2.45%	159.37	157.32	-1.29%	304.45	301.4	-1.00%	551.14	546.94	-0.76%

APPENDIX SF7: FTL MEMO (21 AUGUST 2025), EVALUATION OF FISH PASSAGE FOR PROPOSED CULVERT 3

Fraser Thomas Limited
21 El Kobar Drive : East Tamaki : AUCKLAND 2013
PO Box 204006 : Highbrook : AUCKLAND 2161
NEW ZEALAND
TEL +64 9 278 7078 : E-MAIL sfinnigan@ftl.co.nz

Fraser Thomas
ENGINEERS • RESOURCE MANAGERS • SURVEYORS

MEMORANDUM

Date: 21 August 2025 33097

From: Y Goh, S Finnigan

Subject: Former Tokanui Hospital – Evaluation of Fish Passage for Proposed Culvert 3 (V4)

To: Emily Buckingham (emily.buckingham@slrconsulting.com), Nicola Pyper (nicola.pyper@slrconsulting.com)

1.0 INTRODUCTION

This document evaluates the water velocity and depth within the proposed culvert 3 (C3) design and whether it satisfies the fish passage criteria according to the NIWA New Zealand Fish Passage Guidelines Version 2.0. This has been done for both normal (mean) and storm flows, using both spreadsheet and "stream simulation" culvert design software as the most appropriate solution for assessing movement of fish or other aquatic organisms through culverts.

This document summarises the analysis for:
- Section 2 - Proposed culvert details;
- Section 3 – Adopted maximum velocity and minimum depth thresholds;
- Section 4 - Fish passage under actual "mean stream" flows;
- Section 5 - Fish passage in accordance with the NIWA Fish Passage Guidelines

2.0 PROPOSED CULVERT 3 DETAILS

The existing culvert is 1200mm diameter, with an inlet IL of 29.5mRL and outlet IL of 29.44mRL (NZVD2016).

The proposed culvert 3 will be a 1350mm diameter pipe, with a gradient of 0.49% and 41m long. It will be embedded by 340mm (25%) within the stream bed to facilitate fish passage. The culvert itself will have an inlet IL of 29.86m and outlet IL of 29.66m (NZVD2016), while the invert with embedment will be 30.2mRL with outlet IL 30.0mRL.

The proposed culvert 3 has been sized to match the flow area of the existing culvert (1200dia, 1.13m^2) taking the embedment depth into account (1350dia, less 340mm deep embedded area = 1.15m^2) to match the existing situation as close as possible, so as to hold flood waters back for optimum flood relief benefits for the landfill. This has been done using the "stream simulation" culvert design as the most appropriate solution for assessing the movement of fish or other aquatic organisms through culverts.

3.0 MAXIMUM THRESHOLD VELOCITY AND MINIMUM THRESHOLD DEPTH FOR ĪNANGA (*GALAXIAS MACULATUS*)

In the New Zealand Fish Passage Guidelines (Version 2.0), īnanga (*Galaxias maculatus*) is used as a benchmark species for assessing fish passage. Although īnanga have not been recorded in the vicinity of Culvert 3, a conservative approach was adopted, and the culvert design was assessed using īnanga as the target species, as stated in SLR Consulting's "Former Tokanui Hospital Culvert Remediation Fish Passage Evaluation report" (August 2025). The report specifies a maximum velocity threshold based on the prolonged swimming speed of īnanga (0.44m/s). The NIWA guideline also specifies a minimum depth requirement of 150 mm for fish passage. Therefore, these thresholds – a maximum velocity of 0.44m/s and a minimum depth of 150mm were applied to assess the feasibility of fish passage through Culvert 3.

4.0 FISH PASSAGE UNDER ACTUAL "MEAN STREAM" FLOWS

4.1 Determination of Mean Flows

Three methods have been used to estimate the mean flow in the Wharekōrino Stream by the landfill at the Tokanui Hospital. These are briefly described below:

(a) **Catchment Analysis**: Average annual runoff analysis, based on catchment area and annual average rainfall data for the area (30 year average over period 1991-2020 inclusive) from the nearby Waikeria NIWA rainfall gauge site.

(b) **NZRiversMaps**: NIWA NZRiversMaps website, which includes various estimates of stream and river flows across the entire New Zealand river network (https://shiny.niwa.co.nz/nzrivermaps/). This website states that the mean flow was estimated using the Hydrology of Ungauged Catchments method (Ref: *Booker DJ, Woods RA. 2014. Comparing and combining physically-based and empirically-based approaches for estimating the hydrology of ungauged catchments*).

(c) **WRC Flow Gauging**: Waikato Regional Council actual flow gauge data on the Pūniu River at the Bartons Corner Rd bridge, approximately 4.9km NNW, downstream of the site. Mean flow data from this site was then applied to the Wharekōrino Stream on a pro rata catchment area basis.

4.2 Comparative Results

Mean flow comparative results are summarised below.

Table 1: Mean Flow Comparative Results

Method	Wharekōrino Stream (by landfill)	Pūniu River (Bartons Corner Rd bridge)
Catchment Analysis	62.4 L/s	7.4 m^3/s
NZRiversMaps	105 L/s	14.6 m^3/s
WRC Flow Gauging	121 L/s	14.32 m^3/s

Note: Wharekōrino Stream catchment area at landfill = 0.44km^2; Pūniu River catchment area at Bartons Corner Rd bridge = 518.7km^2

The WRC Flow Gauging method is considered the most appropriate to use, as it is based on actual flow measurements. This gives a mean flow for Culvert 3 of 121L/s.

4.3 Depth and Velocity for Mean Flow

Using the adopted mean flow of 121L/s, the proposed culvert 3 was analysed for fish passage performance, starting with no tailwater and incrementally increasing tailwater until the culvert was nearly full. As the culvert is embedded by 340mm, tailwater depth was measured from the embedded level. These results are summarised in Table 2.

Table 2: Velocity and depth at increasing tailwater depth.

Tailwater depth (m)	Outlet Velocity (m/s)	Depth (m)	Threshold met
0.0	1.04	0.18 (inlet control)	No
0.2	1.04	0.18 (inlet control)	No
0.3	1.04	0.18 (inlet control)	No
0.4	0.24	0.22 (outlet control)	Yes
0.6	0.16	0.40 (outlet control)	Yes
0.8	0.12	0.60 (outlet control)	Yes
1.0	0.11	0.80 (outlet control)	Yes

At tailwater depths below 0.4m, the culvert operates under inlet control and the velocity increases above the threshold required for īnanga passage (0.44m/s from SLR Fish Passage Evaluation Memo). At tailwater depths of 0.4m and greater, the culvert operates under outlet control and the thresholds for both depth and velocity are met.

This indicates that when the water level exceeds roughly half the culvert diameter (~0.4m tailwater depth) conditions become feasible for fish passage. This assumes that all water travels through the culvert with a uniform (constant) velocity profile both horizontally and vertically across the water profile. In reality, the velocity profile is non-uniform and increases with distance from the culvert wall (0m/s) to a maximum in the centre of the culvert at the water surface. Hence, fish passage is likely to be feasible over a greater depth and velocity range than summarised in Table 2 based on mean flow.

If the tailwater depth is 1.01m or greater, the 1.35m diameter culvert is in a fully submerged condition and the calculated average velocity is 0.11m/s (=$0.121m^3/s/1.15m^2$). Fish passage remains feasible under these conditions, as the velocity is below the maximum threshold for īnanga.

4.4 Expected Flow Depths in Culvert 3

Observations from multiple site visits are that Culvert 3 is typically either fully drowned or almost drowned, under both normal and storm flow conditions. This is shown in Figure 1 below, where the top of culvert 3 at the inlet end is just visible.

Figure 1: Culvert 3 Inlet – 29 Feb 2024

Based on an estimated conservative water depth in Culvert 3 of 1m, this gives a water level of 30.5m RL (29.5m IL +1.0m) (NZVD2016).

For the proposed culvert, the effective base of the culvert (at top of embedded material) will be 30.2mRL at the inlet and 30.0mRL at the outlet. Hence, based on a "normal" water level of 30.5m RL from site observations, this means the water depth in the proposed culvert will normally be 0.3m and 0.5m at the culvert inlet and outlet respectively. Under these conditions, the calculated flow velocities in Table 1 are less than 0.44m/s which is sufficient for īnanga. Hence, it is expected that the proposed culvert will provide adequate fish passage during normal flow conditions.

5.0 FISH PASSAGE IN ACCORDANCE WITH NIWA FISH PASSAGE GUIDELINES

According to the New Zealand Fish Passage Guidelines Version 2.0 (June 2024), section 4.5.3, the bank-full flow is defined as the upper flow threshold (Q_H) and ⅓ of the bank-full flow is defined as the lower flow threshold (Q_L) for verifying fish passage in the culvert. Survey data and site observations shows that the stream channel is about 3m wide and 1m deep at the top of bank at the culvert 3 inlet end.

Section 4 demonstrates that fish passage is feasible under actual mean stream flows, while this section assesses fish passage under bank-full flow. Culvert water velocities are typically calculated as mean cross-sectional water velocities, and these mean velocities, often exceed fish swimming velocities and endurance. The guideline states that the best practical way to account for the velocity distribution within the culvert is to look at depth-averaged vertical slices within the culvert at the fish passage design flow. Section 4.5.6 of these guidelines provides a systematic methodology for determining the depth-averaged velocity of vertical slices of the flow within the culvert (Figure 2).

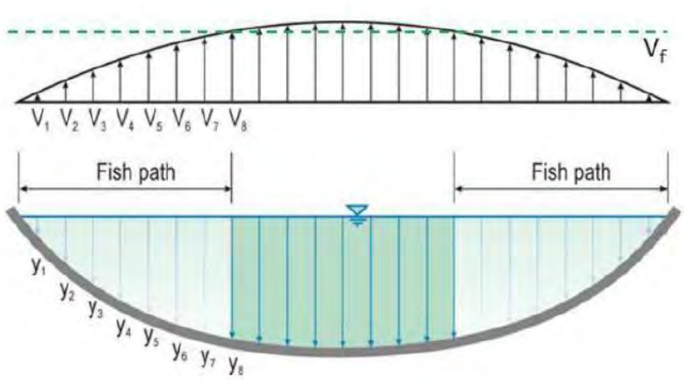


Figure 2: Culvert water velocity distribution with depth averaged velocities in vertical slices.

Figure 3 shows examples of vertical slices over a typical velocity distribution within a rectangular channel. The depth-averaged velocity within the vertical slice outlined in red would be much lower than the vertical slice in yellow. The vertical slice outlined in red would also have a significantly lower water velocity than the mean cross-sectional water velocity.

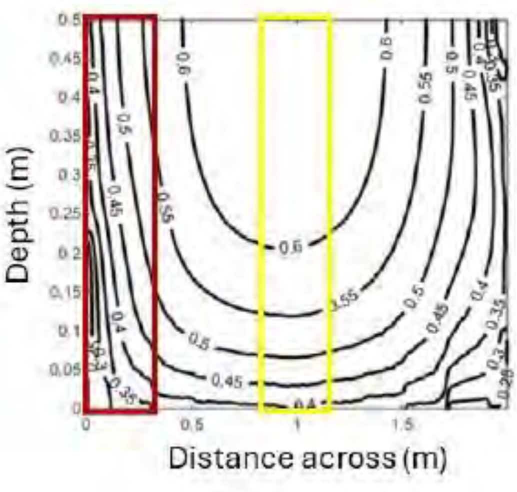


Figure 3: Cross-sectional water velocity distribution within a rectangular channel with example vertical slices. Modified from Cassan et al. (2019).

5.1 Scenario 1: Open channel/culvert not fully submerged - Depth-averaged water velocity analysis using HY8 low flow hydraulics

Based on the HECRAS 2D flood modelling results, the blue line below shows the 'bankfull' flow (Figure 4). At this water level, the bankfull flow is 1.35m^3/s. In this case, Q_H = 1.35m^3/s and Q_L = 0.45m^3/s (calculated from

1.35/3). For these conditions, V_H = 1.39m/s and V_L = 0.56m/s – these are estimated <u>average velocities</u> calculated from modelled flow depths in HY-8 software when the culvert is partially embedded and water is flowing freely and still behaves as an open channel.

It should be noted that the stream bed gradient through this area is relatively flat, with an estimated bed gradient of 0.3%, while water levels at culvert 3 are controlled by existing trees/vegetation within the stream itself below Te Mawhai Road and the Te Mawhai Rd culvert (C1). Removal of culvert C2 between C1 and C3 has been modelled to help reduce stream flood levels upstream of C2 but is unlikely to have an effect on normal stream water levels, due to the stream bed gradient, existing vegetation/trees and C1 all restricting flow, causing water to back up through this area. This is supported by FTL staff who have visited the Tokanui site regularly over the last two years, including during summer, only ever having seen the top of C3 once, with it just visible above the water level, meaning the water depth would have been around 1.-1.1m under low flow conditions, further supporting the above comments (refer photo in Figure 1).

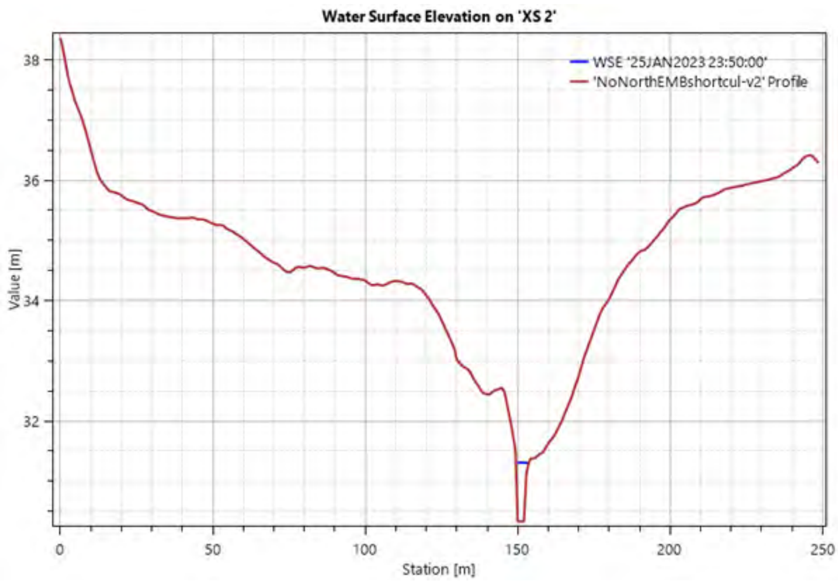


Figure 4: Cross section of stream at culvert 3 and the water level during 'bankfull' flow.

As stated above, C3 has been observed to be submerged under normal flow conditions. Hence, a tailwater depth of 800mm from the stream bed was included in the HY8 low flow hydraulics analysis. In this case, the culvert still behaves as an open channel as the culvert is not fully submerged yet. Figures 5 and 6 show culvert cross-section slices for half of the culvert span and would mirror the same way to the right. The NIWA guideline states that the minimum width of the fish passable vertical slice should be 150mm. About 200mm from both sides of the culvert during low flow and 170mm during high flow meet the threshold for īnanga. The velocity beyond that becomes too fast for īnanga as the slices approach the centre.

Low Flow Results			
Distance from wall (m)	0.00 - 0.20	0.20 - 0.41	0.41 - 0.61
Slice Average Velocity (m/s)	0.04	0.57	0.69
Slice Depth (m)	0.62	0.62	0.62
Threshold	Threshod Met	Too Fast	Too Fast

Figure 5: Low flow results

High Flow Results			
Distance from wall (m)	0.00 - 0.17	0.17 - 0.35	0.35 - 0.52
Slice Average Velocity (m/s)	0.09	1.42	1.71
Slice Depth (m)	0.76	0.76	0.76
Threshold	Threshold Met	Too Fast	Too Fast

Figure 6: High flow results

5.2 Scenario 2: culvert fully submerged – Average velocity analysis

Since the culvert was modelled to be embedded 340mm in the stream bed, the remaining height within the 1350mm culvert is 1010mm. Therefore, when a tailwater of 1010mm depth or more was incorporated, the culvert became fully submerged and behaved as a pipe flowing under pressure. Hence, HY8 low flow hydraulics is not applicable anymore.

In this case for the same peak flows as in section 5.1, the average velocities of turbulent flow within a fully submerged culvert, during low and high flow are v_L (average) = 0.39m/s and v_H (average) = 1.17m/s respectively – calculated from the continuity equation (Q=Av). These average velocities are lower than those calculated in section 5.1. The velocity on the wall of the pipe is zero and increases to maximum as it approaches the centre of the pipe (Figure 7). The HY-8 software is not able to analyse the pipe velocity distribution in this situation. However, based on the calculated average velocities, it is expected that fish would be able to swim along the walls of the pipe during low flow but possibly not during high flow based on the typical velocity distribution shown in Figure 7.

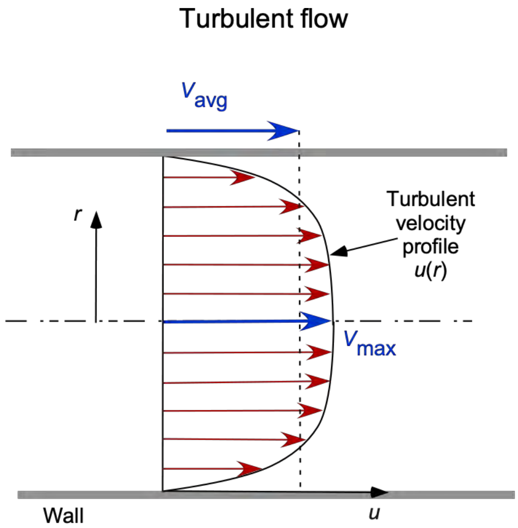


Figure 7: Turbulent velocity profile.

From observing the current permanent ponded/pool condition with slow flowing water at culvert 3, usual flows should be lower than the bankfull modelled flow that was used in this analysis. Therefore, the frequency of a potentially suitable fish passage condition should be higher. High flow conditions should be temporary and of short duration, and fish should be able to pass once the flow reduces.

5.3 Scenario 3: Culvert Fully Submerged and Under Inlet Control

This scenario is when the pipe is flowing full due to storm flows, irrespective of the downstream tailwater level and the analysis is based on a 1200mm culvert, presuming the culvert has the same flow area as the 1350mm diameter with 340mm embedded. Desktop calculations indicate that the culvert would be flowing full for peak flows in excess of 2m^3/s. At a peak flow of 2m^3/s, the average velocity through the culvert would increase to approximately 1.77m/s.

The Fish Passage guidelines (Section 4.5.4) refer to bankfull flow being approximately 50% of the year flood flow (approximately a 1.5 year storm). Hence, on this basis, the 2-year storm would be 2.7m^3/s. Hence, the 2m^3/s criteria is estimated to around the 1.8 year storm or a storm that on average might occur once every 22 months. This means it is a relatively infrequent event and would be expected to be of relatively short duration.

6.0 FISH PASSAGE MITIGATION MEASURES

Design of Culvert 3 and new stream has incorporated several features to promote fish passage. These are summarised in this section:

(a) Embedment of proposed culvert by 340mm (25% of diameter);
(b) Stream alignment to follow a gentle meander and to be naturalised including regular 150-300mm deep pools, rocks and logs.
(c) Low flow channel.
(d) Riparian planting.
(e) Resting pools at the culvert inlet and outlet, with the outlet pool also functioning as an energy dissipation basin for storm events.

These features are shown on the landfill drawings (33097/LF300 series).

7.0 SUMMARY AND RECOMMENDATIONS

The culvert design allows for embedding the 1350mm diameter culvert by 340mm, which is an approved method of providing for fish passage, from the NIWA Fish Passage Guidelines. Fish passage has then been assessed for the embedded culvert for a range of flow conditions.

For mean flow conditions (121L/s), Culvert 3 is expected to be feasible for fish passage, based on both depth and velocity thresholds. This also applies even when the culvert is fully submerged. This is significantly lower than the modelled 2-year lower bank-full flow threshold (QL) of 450L/s (see section 5). This is to be expected as the mean flow represents average conditions, while the 2 year storm flow represents a storm event which has a 50% probability of occurring in any one year.

In the bank-full storm scenario, when culvert 3 behaves as an open channel, the “vertical slices” hydraulics analysis shows that it satisfies the minimum width of 150mm, maximum water velocity 0.44m/s and minimum water depth 150mm for īnanga to pass during low and high flow conditions. When the inlet and outlet of culvert 3 are fully submerged due to high tailwater conditions, the culvert behaves as a pressurised pipe with turbulent flow. In this case, the average pipe velocities calculated indicate that fish should be able to swim along the walls of the pipe during low flow, but possibly not during high flow. The culvert will also be fully submerged due to high flow conditions, for events equivalent to approximately a 1.8 year (22 month) storm or more extreme storm events. In this situation, average velocities through the culvert will increase, but for a relatively short duration and fish should be able to pass once the flow reduces.

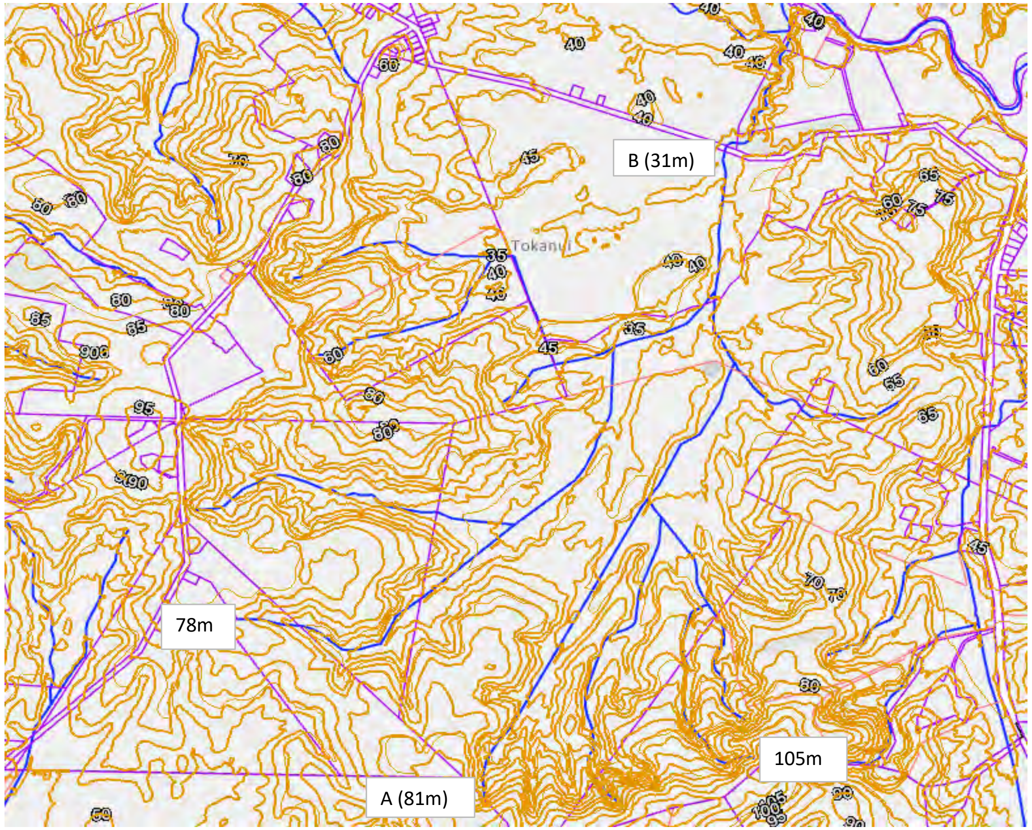
The culvert and new stream design also feature a number of other features to promote fish passage, including the stream alignment having a gentle meander, naturalisation using 150-300mm deep pools, rocks and logs; low flow channel; riparian planting; and resting pools at the culvert inlet and outlet, with the outlet pool also functioning as an energy dissipation basin for storm events. Collectively, these features together with the culvert embedment should ensure that fish passage along the new stream and culvert 3 is possible under most conditions, except during certain storm events, which will be of short duration.

Stream Flow Assessment

Method 1 (Catchment Runoff Basis)

	Landfill	Bartons Br	
Catchment area	440	51870	ha
Catchment slope	Moderate		
Land use	Mainly pasture		
Runoff coefficient	0.4	0.4	heavy clay soil type, 5-10% gradient (measured off contour plans)
Annual rainfall	1118.7	1118.7	mm
Average annual runoff	1968912	2.32E+08	m3/yr
Average flow per day	5394	635912	m3/d
Average flow per day	62.4	7360	L/s

https://waikatomaps.waikatoregion.govt.nz/Viewer/?map=8d6d6fda779b4e59951953ae97d0ec4a



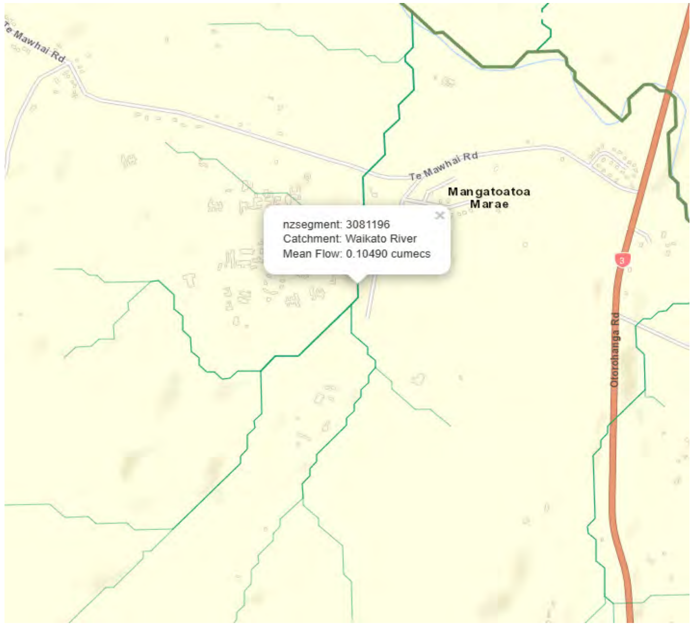
Distance AB = 2860m Gradient = 1.7% (along stream)

Measured overland flow gradients going anticlockwise: Generally in range 5-10%

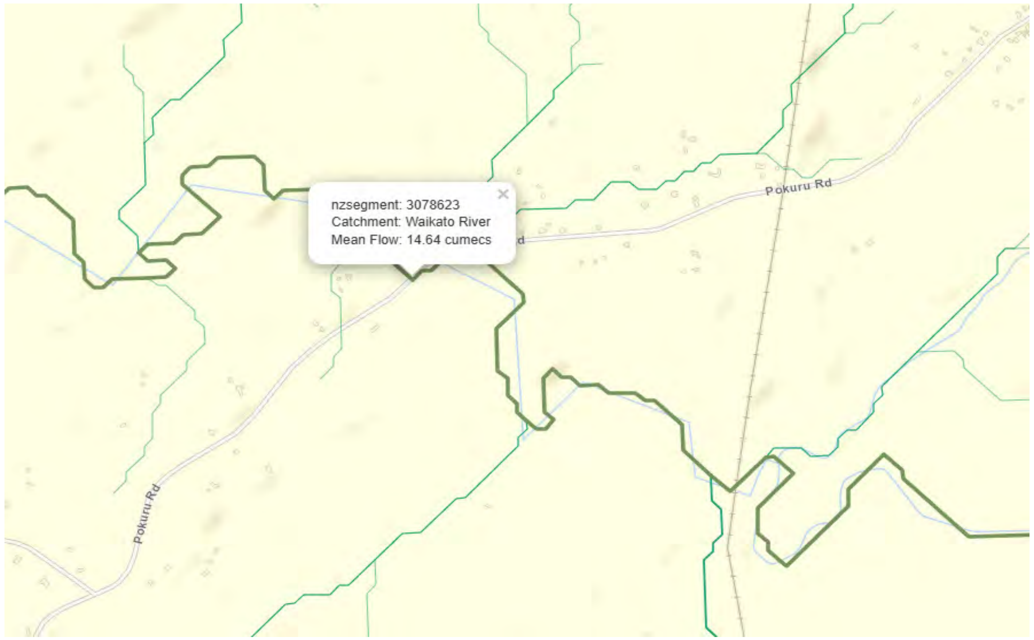
Line	1	2	3	4	5	6
%	5.6	5.6	7.5	5.6	9.3	9.5

Method 2 (NIWA River Maps)

Location	Mean Q	
Wharekorino at Te Mawhai Rd (includes hospital catchment)	124.7	L/s
Wharekorino at landfill	104.9	L/s
Puniu (at Bartons Bridge)	14.64	m3/s

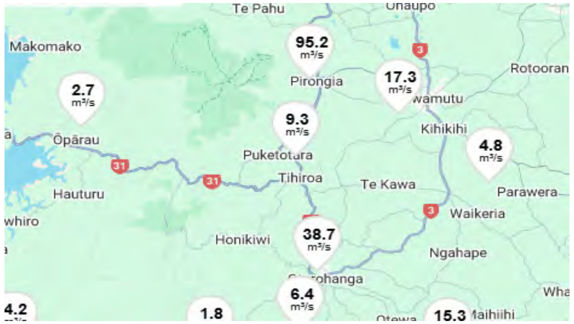


Puniu River, Pokuru Road (Barton's Bridge)



Method 3 (from comparison with WRC flow gauges)

Location	Catch Area (km2)	Mean Q (m3/s)	Spec Q (m3/s/km2)
Puniu (at Bartons Bridge)	518.7	14.32	0.028
Mangatutu Stream	122	3.79	0.031
Pro rata basis			
Wharekorino at Te Mawhai Rd	6	0.166	
Wharekorino at Landfill	4.4	0.121	

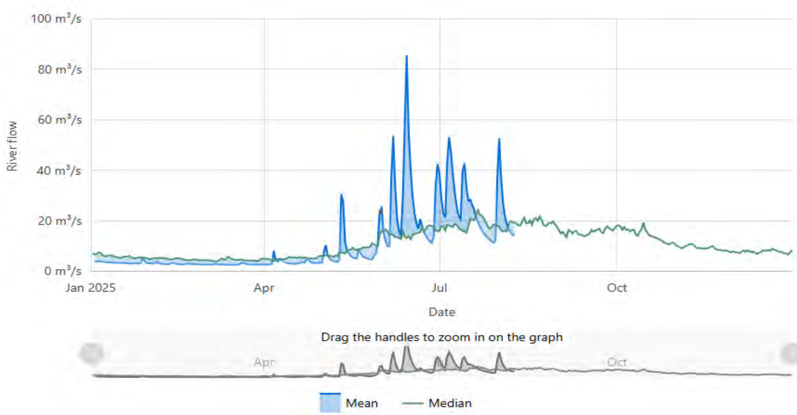


Site information

Site name:	Puniu River - Bartons Corner Rd Br
Site number	818
Station number	2

Additional Information

Mean flow (m³/s)	14.32
Continuous data since	06/05/1985
Highest recorded flow (m³/s)	322.19 on 16/10/1989
Lowest recorded flow (m³/s)	2.14 on 15/02/1992
Data download restriction	No
Altitude (m) above mean sea level (amsl)	30.95
Catchment size (km²)	518.70
Map reference	NZTM:1801295 - 5788365

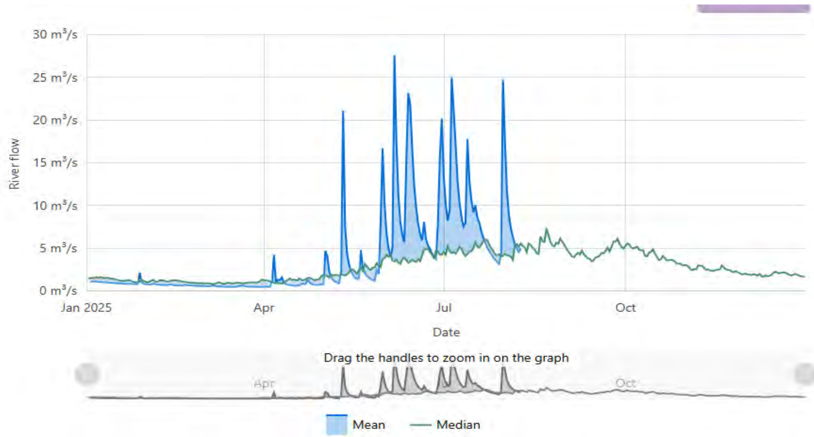


Site information

Site name:	Mangatutu Stm (Waikeria) - Walker Rd Br
Site number	476
Station number	7

Additional Information

Mean flow (m³/s)	3.79
Continuous data since	08/06/2004
Highest recorded flow (m³/s)	38.19 on 01/01/2024
Lowest recorded flow (m³/s)	0.48 on 27/03/2008
Data download restriction	No
Altitude (m) above mean sea level (amsl)	40
Catchment size (km²)	122.00
Map reference	NZTM:1810109 - 5780575



Comparison

Method	Mean Q (L/s)
Catchment	62.4
NIWA Rivers	104.9
WRC flow gauges	121.5

Average velocity – culvert not fully submerged

$$V_L = \frac{Q_L}{A_L} = \frac{0.45\ m^3/s}{0.81\ m^2} = 0.56\ m/s$$

$$V_H = \frac{Q_H}{A_H} = \frac{1.35\ m^3/s}{0.97\ m^2} = 1.39\ m/s$$

Average velocity – culvert fully submerged

$$V_L = \frac{Q_L}{A_L} = \frac{0.45\ m^3/s}{1.15\ m^2} = 0.39\ m/s$$

$$V_H = \frac{Q_H}{A_H} = \frac{1.35\ m^3/s}{1.15\ m^2} = 1.17\ m/s$$

HY-8 Culvert Analysis Report

Culvert Data: Culvert 3

Culvert Data Summary - Culvert 3

Barrel Shape: Circular

Barrel Diameter: 1350.00 mm

Barrel Material: Concrete

Embedment: 340.00 mm

Barrel Manning's n: 0.0130 (top and sides)

Manning's n: 0.0130 (bottom)

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall (Ke=0.5)

Inlet Depression: None

Low Flow Hydraulic Computations

Low Flow Hydraulic Discharges

Low Flow 0.45 cms

High Flow 1.35 cms

Peak Flow 2.00 cms

Threshold

Max Threshold Velocity 0.44 m/s

Min Threshold Depth 0.15 m

Threshold for both Flows Threshold is Met

Model Parameters

Number of Divisions in Flow Top Width: 6

Low Flow Threshold Results

Highest Average Velocity in Culvert 0.56 m/s

Depth in Culvert at Highest Average Velocity 0.62 m

Top Width in Culvert at Highest Average Velocity 1.22 m

Selected Velocity Adjustment Curve Curve 1: Culvert Embedment is greater than 20%

Width of Slice along Culvert Water Surface 0.20 m

Threshold at Low Flows Threshold is Met

High Flow Threshold Results

Highest Average Velocity in Culvert 1.40 m/s

Depth in Culvert at Highest Average Velocity 0.76 m

Top Width in Culvert at Highest Average Velocity 1.04 m/s

Selected Velocity Adjustment Curve Curve 1: Culvert Embedment is greater than 20%

Width of Slice along Culvert Water Surface 0.17 m

Threshold at High Flows Threshold is Met

Low Flow Results

Distance from Wall (m)	1.00	1.67	2.34	3.00	3.67	4.34	5.01	5.67	6.34	7.01	7.68	8.34	9.01	9.68	10.35	11.01	11.68	12.35	13.02	13.68
Slice Velocity (m/s)	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68
Slice Depth (m)	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68	13.68
Threshold	Threshod	TooF	TooF																	

	Met	ast	ast																	

High Flow Results

Distance from Wall (m)	**0.86**	**1.43**	**2.00**	**2.57**	**3.14**	**3.71**	**4.28**	**4.85**	**5.42**	**5.99**	**6.56**	**7.13**	**7.70**	**8.27**	**8.84**	**9.41**	**9.98**	**10.55**	**11.12**	**11.69**
Slice Velocity (m/s)	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69
Slice Depth (m)	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69	11.69
Threshold	Threshold Met	Too Fast	Too Fast																	

APPENDIX SF8: FTL MEMO (20 AUGUST 2025), ESTIMATED EFFECT OF LANDFILL UPGRADE WORKS ON BORON DISCHARGES

Fraser Thomas Limited
21 El Kobar Drive : East Tamaki : AUCKLAND 2013
PO Box 204006 : Highbrook : AUCKLAND 2161
NEW ZEALAND
TEL +64 9 278 7078 : E-MAIL sfinnigan@ftl.co.nz

Fraser Thomas
ENGINEERS • RESOURCE MANAGERS • SURVEYORS

MEMORANDUM

Date: 20 August 2025 33097

From: S Finnigan

Subject: Former Tokanui Hospital: Estimated Effect of Landfill Upgrade Works on Boron Discharges (V2)

To: Emily Buckingham (emily.buckingham@slrconsulting.com)

1.0 INTRODUCTION

This memo summarises our assessment of the estimated likely reduction in boron discharges to the Wharekōrino Stream as a result of improved landfill capping and groundwater diversion from the landfill, in response to a question raised by Mana Whenua during a site meeting to discuss the landfill repair and upgrade works.

2.0 BACKGROUND INFORMATION

Long-term monitoring (consent compliance) of groundwater downgradient of the landfill, and surface waters in the Wharekōrino Stream upstream and downstream, has indicated that dissolved boron levels from groundwater sampled at monitoring bore P2 exceed ANZECC's trigger levels for the protection of 95% of freshwater species. This results in elevated total boron concentrations in mid-stream and downstream samples of the Wharekōrino Stream, but still well within the ANZECC 95% trigger thresholds. These findings are depicted in Figures 1 and 2 below.

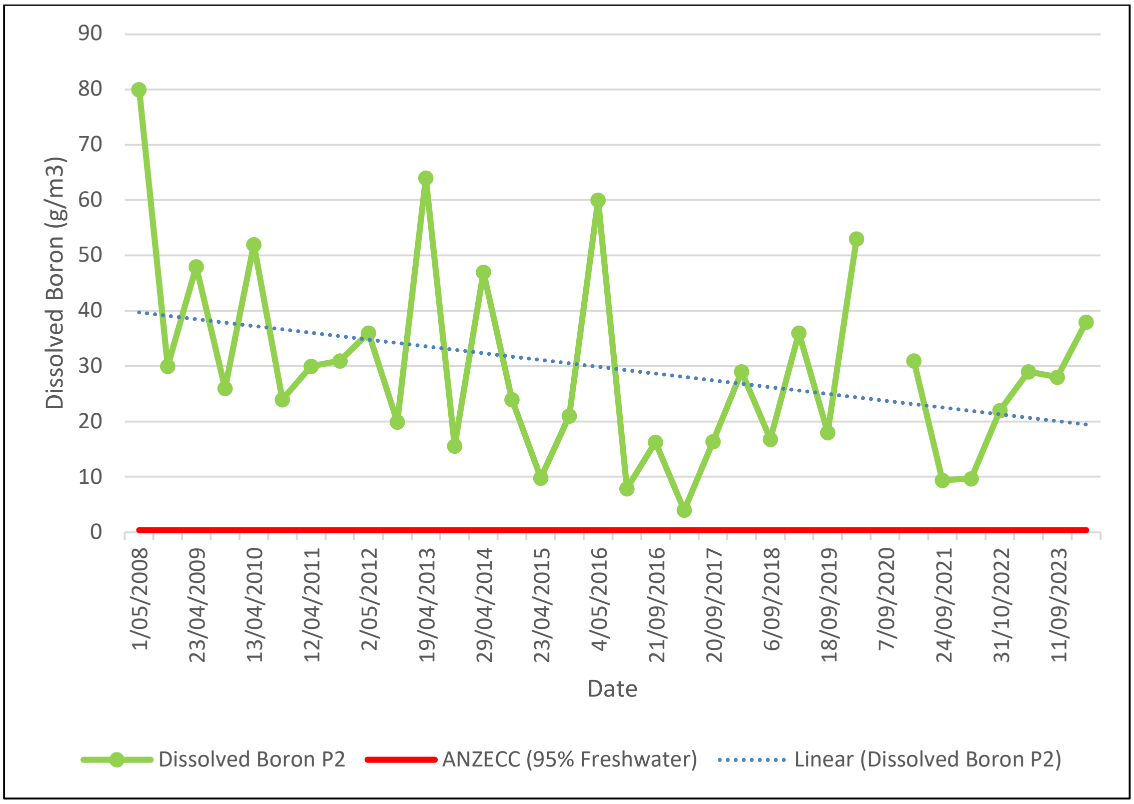


Figure 1: Long term dissolved boron levels (g/m^3) within the landfill (P2)

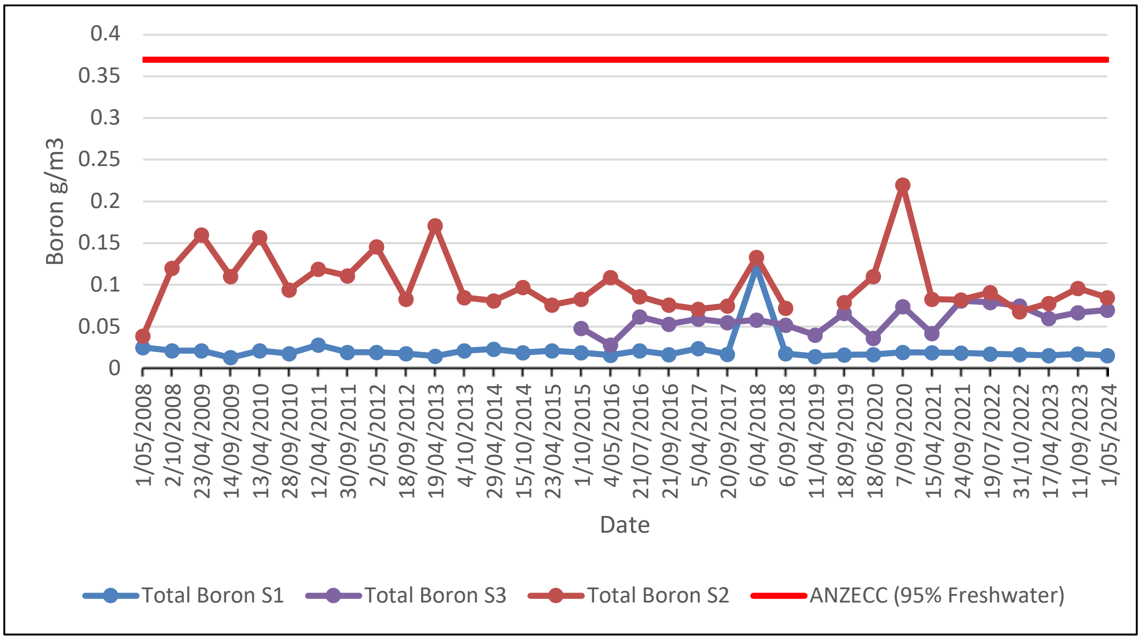


Figure 2: Long-term total boron levels (g/m³) upstream (S1), midstream (S3), and downstream (S2) of the landfill in comparison to the ANZECC trigger value for protection of 95% of aquatic species. Note that the ANZECC boron trigger referred to above comes from the 2000 guidelines and was increased in 2021 to 0.94mg/kg (approximately 3 times higher) based on errors identified in the 2000 derivation and new data becoming available.

Boron leaching to groundwater and the stream currently occurs via two main pathways:

(a) Shallow groundwater from upgradient travelling through the refuse to the stream.

(b) Rainfall infiltrates through the landfill and passes through the refuse, and then combines with shallow groundwater going to the stream.

The current situation and these pathways are illustrated in Figure 3 below.

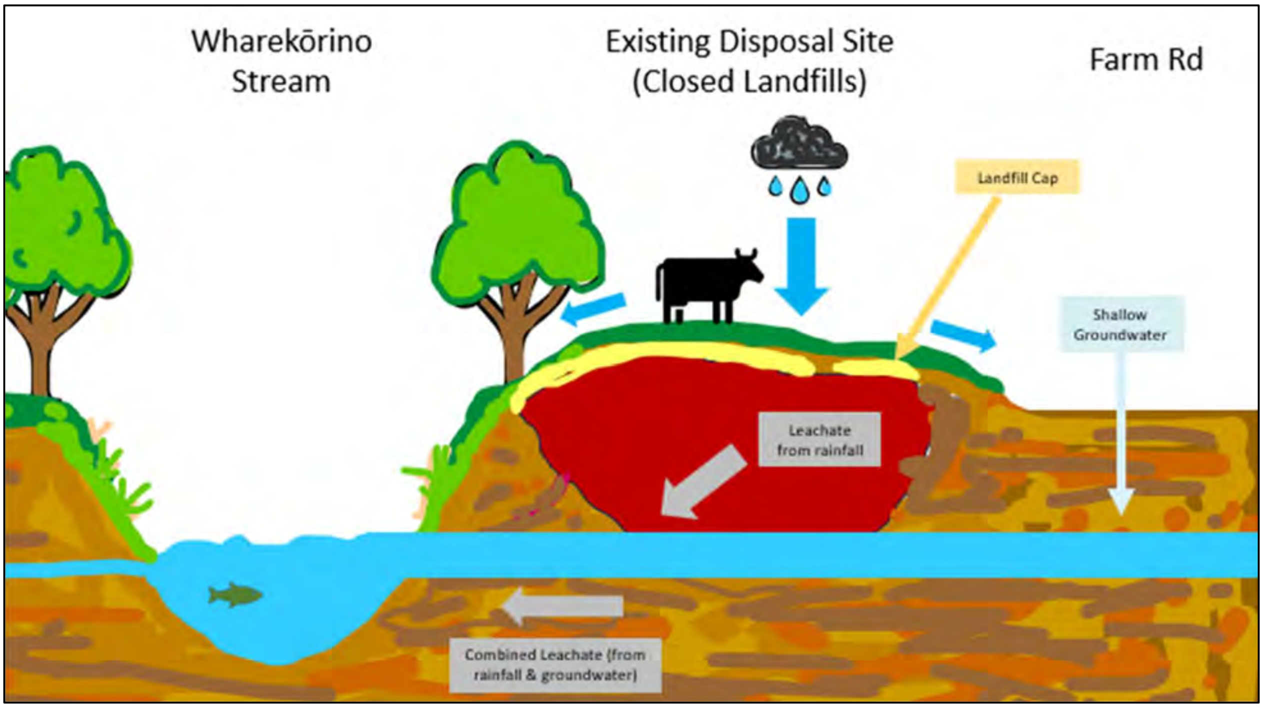


Figure 3: Current Boron Leaching Pathways to Stream

The proposed landfill upgrade and repair works will address both these issues, by:

(a) Installing an improved GCL (geosynthetic clay liner) cap that will significantly reduce rainfall infiltration into the landfill and hence leachate generation from rainfall.

(b) Installing a groundwater cutoff drain upgradient of the southern landfill area that will intercept shallow groundwater and divert it around the landfill to the stream. This will significantly reduce (and potentially eliminate) shallow groundwater contact with refuse in the southern landfill area.

The upgrade situation and the effects of this on boron leaching pathways to the stream are illustrated in Figure 4 below.

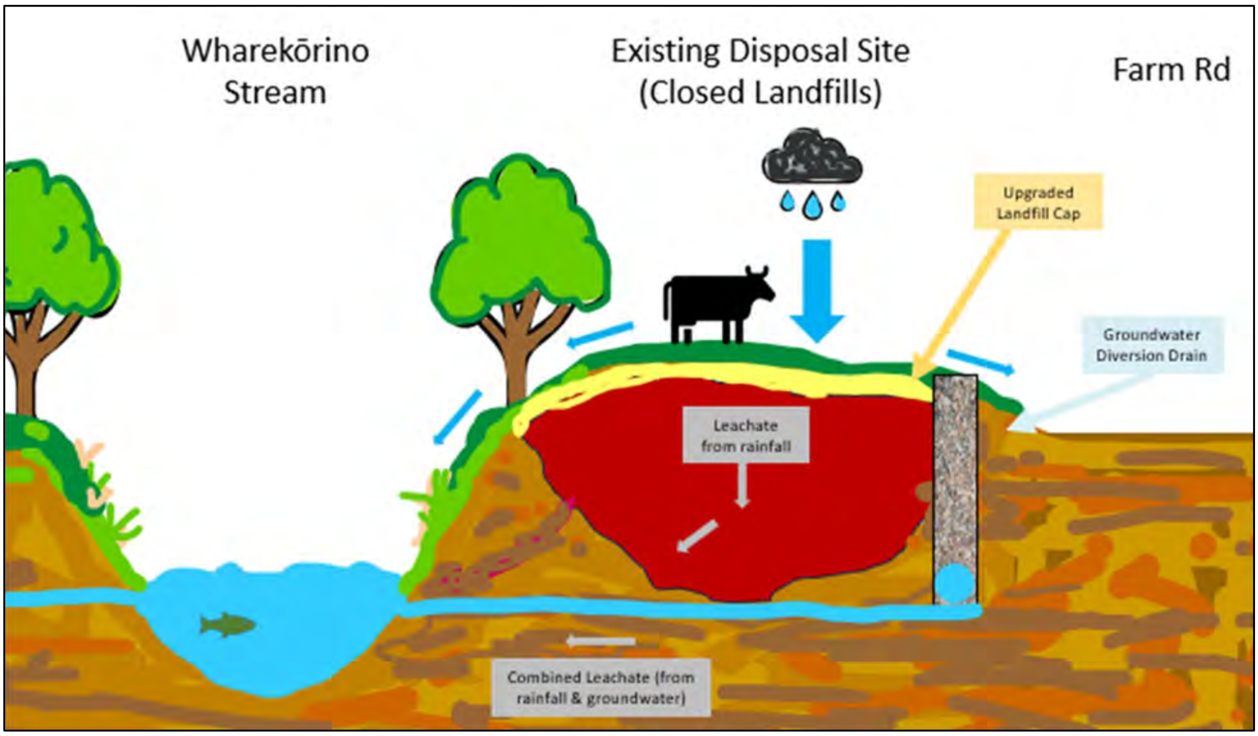


Figure 4: Post-Upgrade Works, Reduced Boron Leaching Pathways to Stream

3.0 METHODOLOGY

The methodology used in this assessment is set out below:

1. Estimate annual average stream flows in the Wharekōrino stream by the landfill, using two methods:
   (a) Catchment analysis, using average annual rainfall data (1,119mm) from the nearby NIWA Waikeria rain gauge – 30-year record (1991-2020).
   (b) Using Waikato Regional Council Pūniu River flow gauge data for mean stream flows to estimate corresponding stream flows in the Wharekōrino Stream on a pro rata area basis (440ha catchment vs 518.7km^2 catchment).
2. Analysing the long-term stream monitoring data over the last 10 years, calculating the difference in boron concentrations between upstream and downstream sampling locations for each sampling event and then calculating the average change (increase) in boron concentration in the stream.

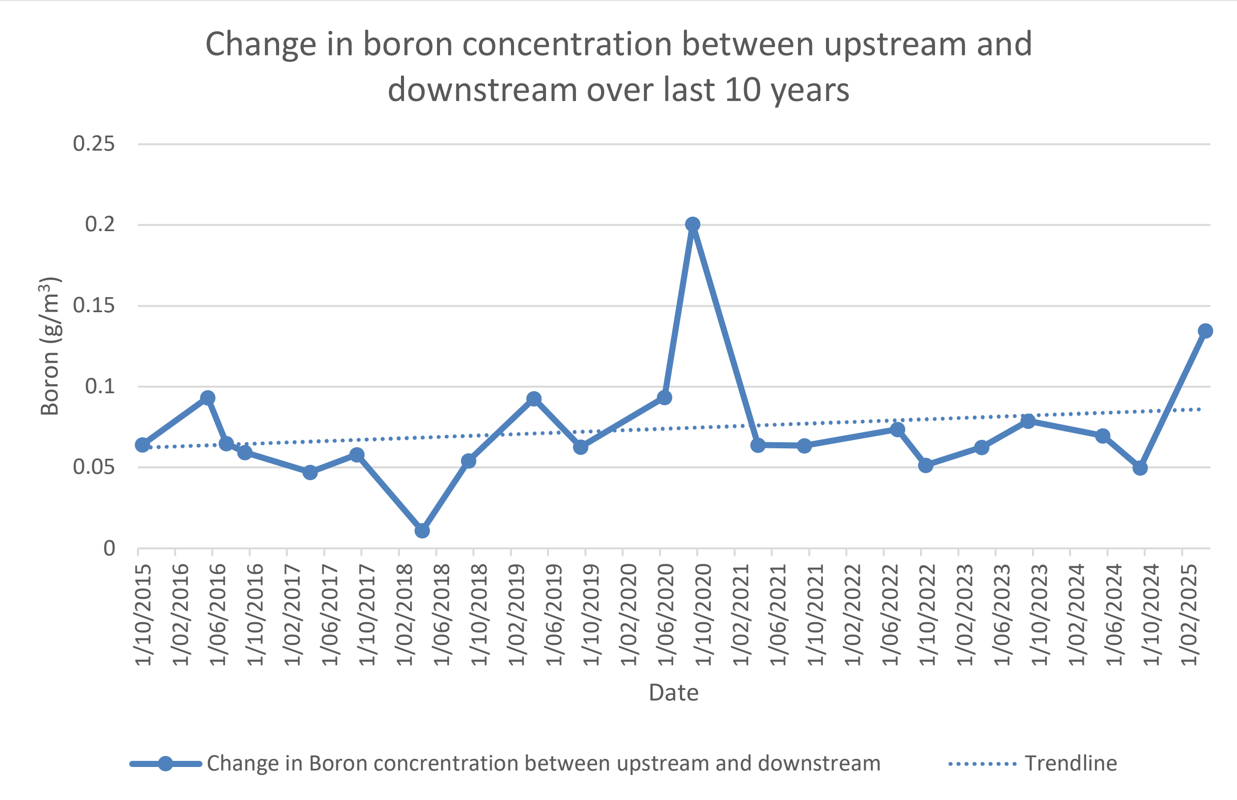


Figure 5: Change in boron concentration upstream and downstream of landfill over the last 10 years

3. Estimating the annual average boron discharge to the stream (kg/yr) as average annual stream flow x average change in concentration for each stream flow method.
4. Calculating the long-term boron concentration from monitoring data in the groundwater monitoring well, P2, and assuming this is representative of the average boron concentration in the groundwater discharge to the stream from the entire landfill.
5. Hence, calculate the average annual leachate flow to the stream as boron discharge to stream/boron concentration in groundwater.
6. Calculate the estimated annual average leachate volume from rainfall onto the landfill, based on measured landfill areas from site investigations, average annual rainfall (Waikeria rainfall gauge) and estimated infiltration rates of 10-25%.
7. Hence, calculate the estimated leachate volume from shallow groundwater flowing through the landfill by difference.
8. Check whether this makes sense by comparing estimated groundwater leachate volumes with theoretical volumes based on Darcy's Law (Q = kiA), where k = refuse permeability, i = hydraulic gradient of groundwater and A = cross-sectional area of groundwater in contact with refuse. Literature values of refuse hydraulic conductivity vary between around 1 x 10^{-3}m/s and 1x10^{-9}m/s (source: LANDSS). For Tokanui, the permeability is considered likely to be at the high end of this range, or even more permeable, based on the age of the landfill, anecdotal information on refuse placement and site observations. The hydraulic gradient was estimated by plotting groundwater level information on representative cross-sections through the landfill. Figure 6 shows a cross-section through the southern landfill area. This information was also used to estimate the average depth and width of groundwater in contact with refuse, thereby determining the cross-sectional area.
9. The effect of the landfill upgrade and repair works on leachate flows and boron discharges to the stream was then estimated, as follows:
   (a) Assume capping reduces the leachate infiltration rate to 2-5% and calculate the revised leachate volume from rainfall.
   (b) Assumed the groundwater intercept trench reduces the amount of groundwater in contact with refuse by 80-90%.
   (c) Calculate the revised average annual leachate volume as the sum of rainfall and groundwater-derived leachate.

(d) Assume that the average annual boron concentration in leachate remains the same as measured at P2.
(e) Hence, calculate the reduced boron discharge to the stream in kg/yr (reduced volume x same concentration).
(f) Calculate the percentage reduction in the boron discharge by comparison with the pre-works discharge.
(g) Calculate estimated revised boron concentrations in the stream, compared with the current situation.
(h) Estimate the time it will take for the reduced concentrations to be achieved, taking into account typical groundwater travel times.

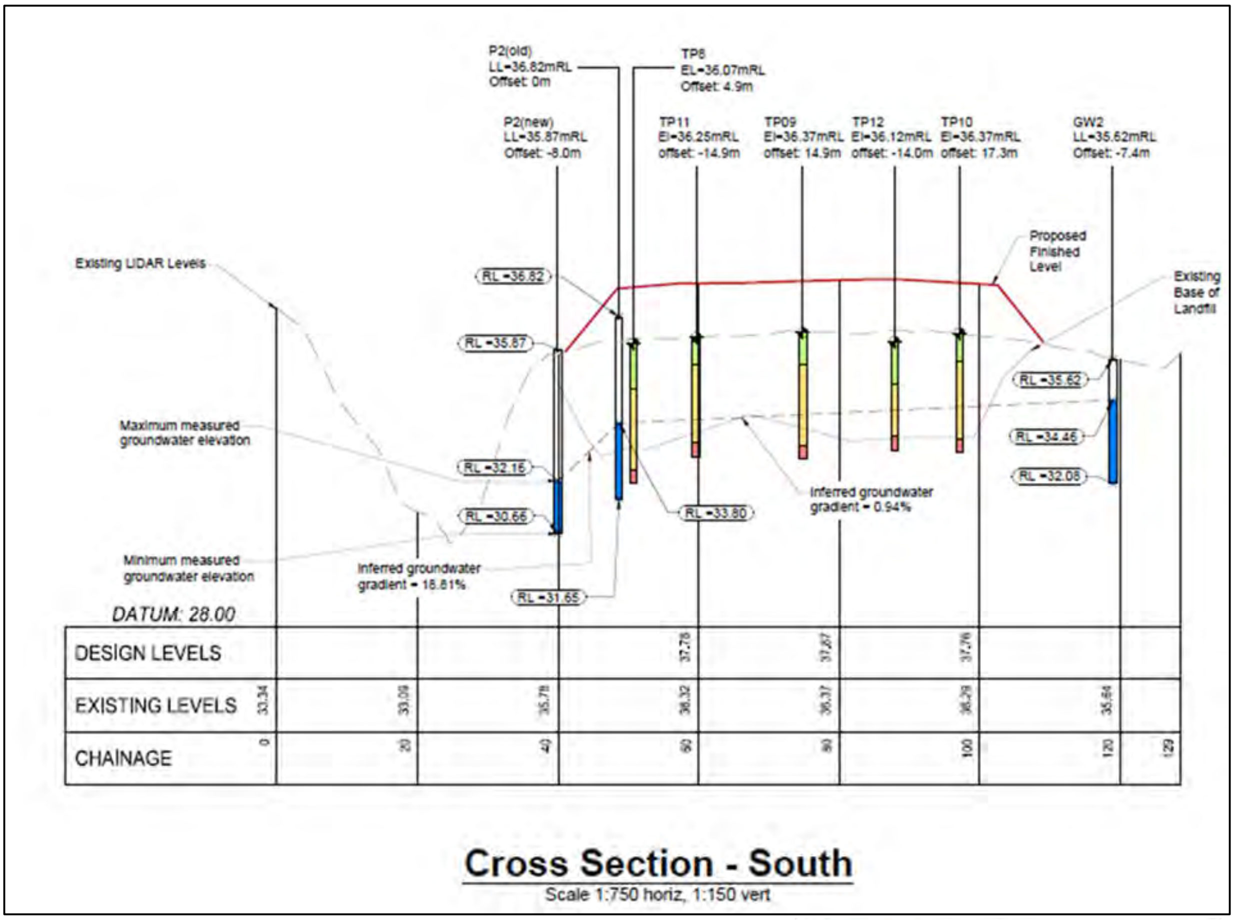


Figure 6: Cross-section – South showing refuse (yellow) and groundwater (blue) extents

4.0 RESULTS

4.1 Estimated Boron Discharge Reduction

The results are summarised in Table 1, with calculations attached in Appendix A. They show that current leachate discharges to the stream are estimated to be in the range of 5,787-11,260m^3/yr, representing 0.29% of average stream flows. The proposed landfill upgrade and repair works will reduce these to an estimated 468-2,700m^3/yr, equivalent to percentage reductions of 72-92% and equating to 0.02-0.07% of annual average stream flows.

Table 1: Summary of Boron Discharge Reduction Results

Item	Stream Flow Method 1 (from rainfall data)		Stream Flow Method 2 (from river flow data)	
Current Situation				
Average annual Stream Flow (m^3/yr)	1,970,261		3,833,394	
Average annual change in boron concentration in stream due to landfill (g/m^3)	0.0738		0.0738	
Average annual boron load to stream (kg/yr)	145		283	
Average Boron Concentration in GW Well, P2 (g/m^3)	25.1		25.1	
Annual average leachate flow to stream (m^3/yr)	5,787		11,260	
Rainfall infiltration rate through landfill (%)	10	25	10	25
Leachate from rainfall (m^3/yr)	1,709	4,273	1,709	4,273
Leachate from groundwater (m^3/yr)	4,078	1,514	9,551	6,987
Effect of Landfill Remedial Works				
Rainfall infiltration rate through landfill (%)	2-5	2-5	2-5	2-5
Leachate from rainfall (m^3/yr)	316-790	316-790	316-790	316-790
Groundwater diversion from landfill (%)	80-90	80-90	80-90	80-90
Leachate from groundwater (m^3/yr)	408-816	151-303	955-1,910	699- 1,397
Total reduced leachate discharge to stream (m^3/yr)	724-1,606	468-1,093	1,271-2,700	1,015-2,188
Reduction in Leachate Discharge to Stream	**72-88**	**81-92**	**76-89**	**81-91**

4.2 Estimated Revised Boron Concentrations in Stream

Actual and estimated annual average boron concentrations in the stream are summarised below and illustrated in Figure 7:

- Existing average annual upstream concentration = $0.0022 g/m^3$
- Existing average annual downstream concentration = $0.0957 g/m^3$
- Average estimated downstream concentration post-upgrade works:
  - Rainfall method = 0.00082-$0.0227 g/m^3$
  - Stream flow method = 0.00089-$0.0199 g/m^3$

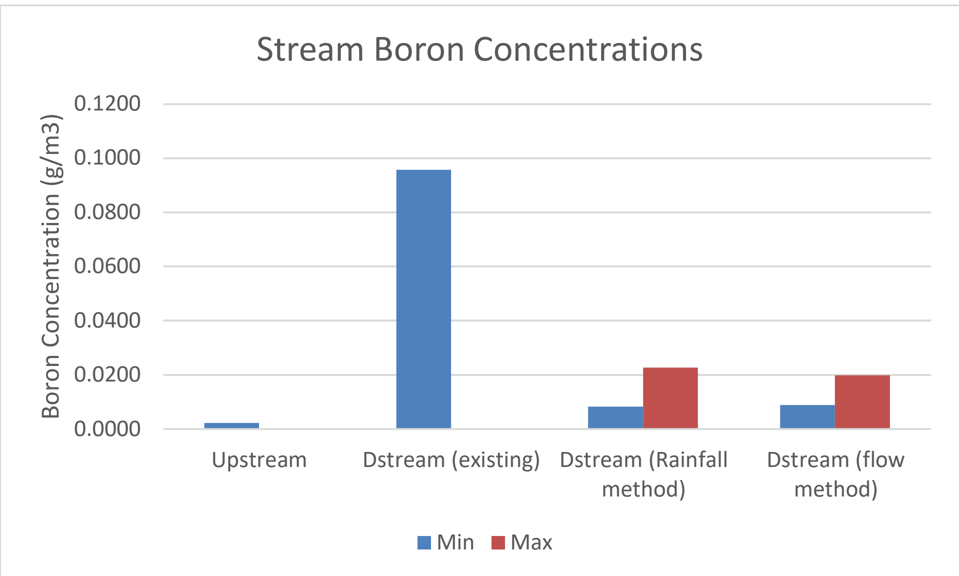


Figure 6: Estimated Downstream Boron Concentrations in Stream Compared with Existing Situation (Upstream and Downstream)

4.3 Estimated Time for Reduced Boron Concentrations to be Achieved

Groundwater travels relatively slowly through the ground, both horizontally and vertically, depending on a range of factors, including the hydraulic gradient and soil permeability. Estimated groundwater travel times have been calculated based on the following:

- A cross-section through the southern landfill, with groundwater having to travel 70m from monitoring well GW2 to the existing P2 well and then another 23m to the stream (refer Figure 6).
- An average hydraulic gradient between GW2 and P2 (old) of 0.94% and from P2 (old) to the stream of 10.4%.
- Variable permeabilities for groundwater to travel through refuse ranging from 0.01-0.001m/s and through natural ground, assuming relatively poor drainage conditions of 0.0001-0.000001m/s.

This gives an estimated groundwater travel time range of from 34 days to 7.2 years. Hence, it may take some months to years for the landfill upgrade and repair works to be reflected in corresponding reductions in boron concentrations reaching the stream.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Ongoing long-term consent compliance water quality monitoring has found that there are elevated dissolved boron levels within groundwater that passes through the landfill, leading to increased boron concentrations in the stream, but that the stream boron levels are well within ANZECC 95% freshwater trigger levels.

This assessment has found that boron discharges to the Wharekōrino Stream should significantly reduce by an estimated 72-92% as a result of proposed improved landfill capping and groundwater diversion from the landfill.

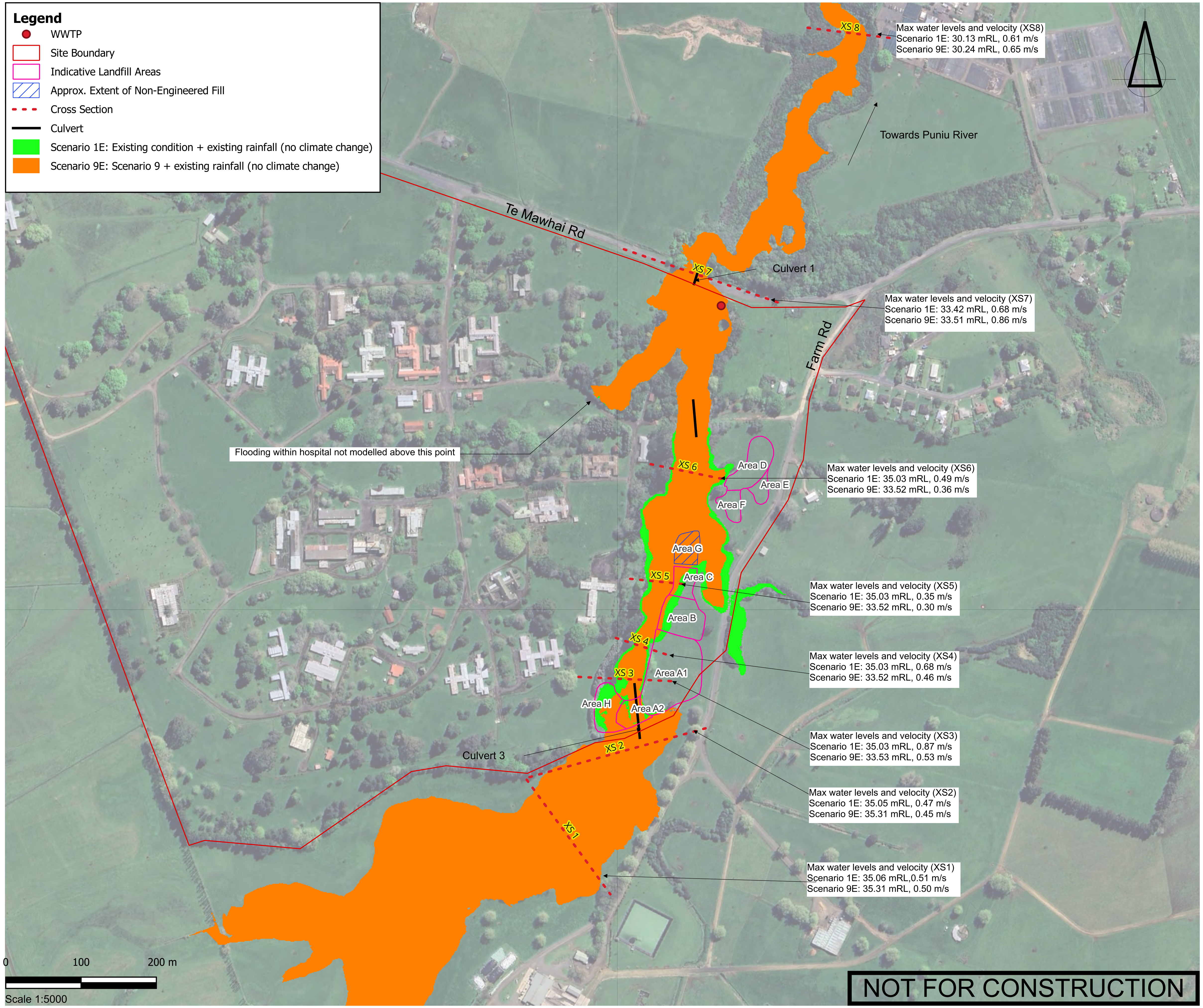
This should result in boron concentrations measured at the downstream sampling location in the stream reducing from their current annual average level of 0.0957g/m^3 to in the range of 0.00082-0.0227g/m^3. However, it may take somewhere between just over one month to an estimated 7 years for the landfill upgrade and repair works to be reflected in corresponding reductions in boron concentrations reaching the stream.

Drawings

Appendix A

Calculations

APPENDIX SF9A AND SF9B: UPDATED FLOOD MODELLING DRAWINGS M08 AND M09



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SURVEYED			APPROVED	DATE
DESIGNED			SF	
DRAWN	YG	11/11/2024		11/11/2024
CHECKED	SF	11/11/2024		

REVISION	CHANGES	CHECKED	DATE

NOTES

1. Tailwater not modelled for results shown in this drawing.

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PROJECT

FORMER TOKANUI HOSPITAL DEMOLITION AND REMEDIATION PROJECT

TITLE

100YR ARI + RCP 8.5 (2081-2100) FLOOD EXTENT OF EXISTING SCENARIO 1E VS 9E

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CHRISTCHURCH..........03 358 5936
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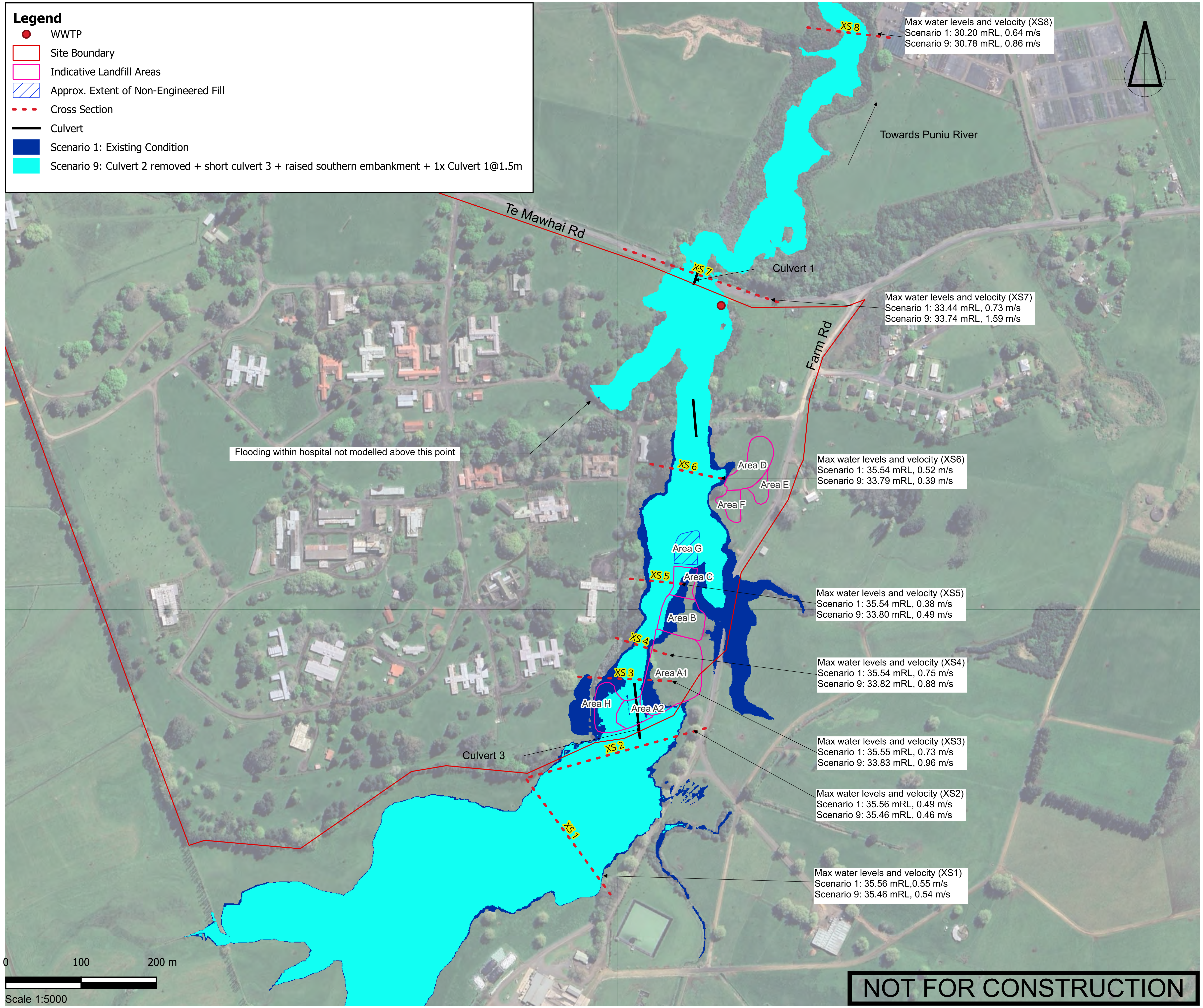

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NOTES

1. Tailwater not modelled for results shown in this drawing.

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100YR ARI + RCP 8.5 (2081-2100) FLOOD EXTENT OF EXISTING SCENARIO 1 VS 9

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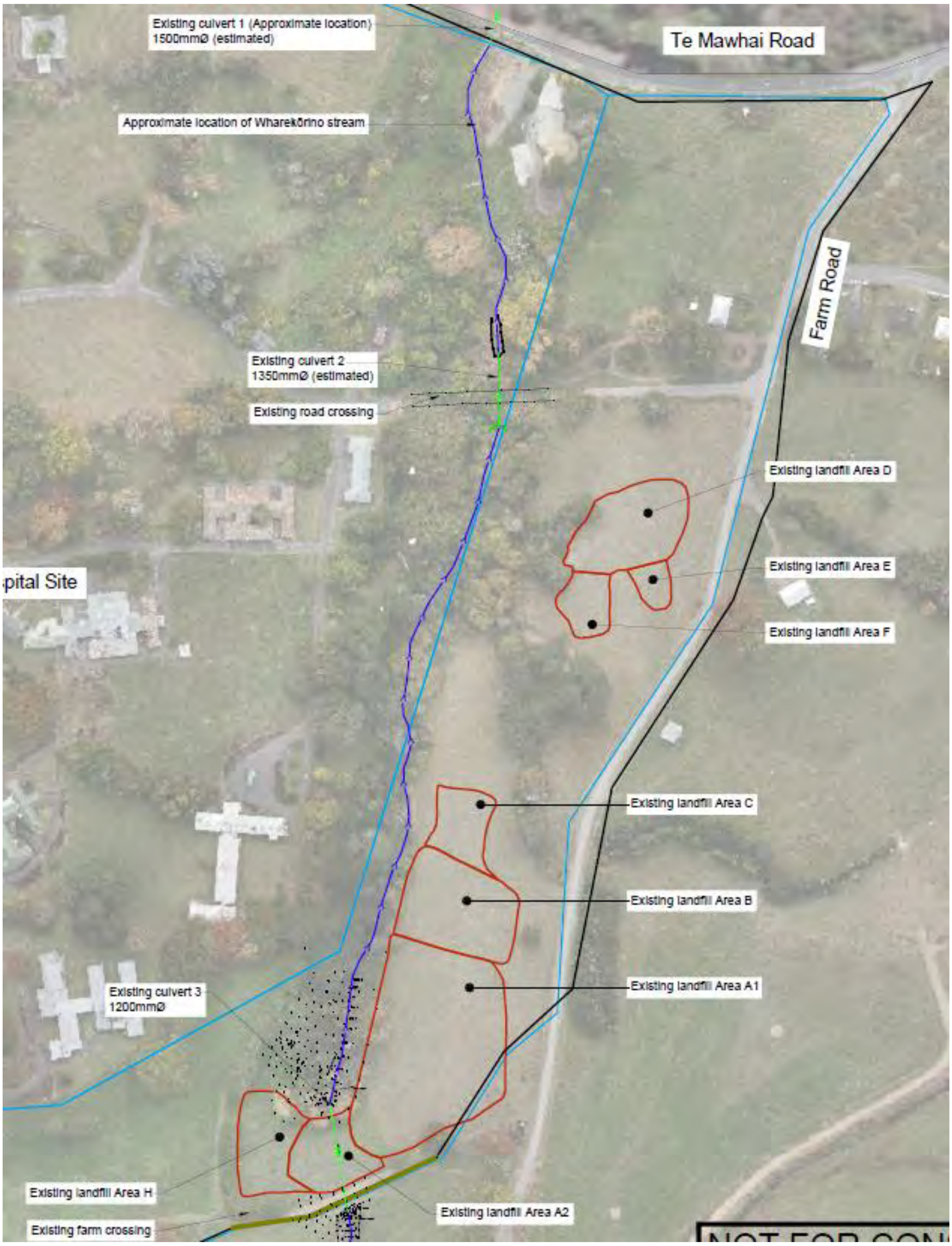
APPENDIX SF10: LATEST FIGURES 2 AND 7 (WRC S42A REPORT)

Updated Figure 2



Updated Figure 7