Appendix A1: Stormwater Servicing Assessment

Trices Road Private Plan Change

Stormwater Servicing Assessment

Trices Road Rezoning Group

21 June 21

e2 environmental CONSULTING ENGINEERS

Quality Control

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Reviewed by	Zeean Brydon	Date Issued	21 June 2021
Approved by	Andrew Tisch	Revision No.	6
Doc Name/Location	S:\Projects\20037 311 Trices Road PC\Deliv\rpt 210621 Trices PC SW Servicing (Final) R6.pdf		

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Project No. 20036

CONTENTS

1 INTRODUCTION

e2Environmental Ltd (e2) have been engaged by Trices Road Rezoning Group to design a stormwater system to serve the proposed plan change development of approximately 28 hectares of land bounded by Trices Road, Birchs Road and Hamptons Road, in Prebbleton (the site).

1.1 Setting

The general topography of the site falls to the south east at an average grade of 1 in 300 in a north west to south east direction with an outfall to a drain east of the site. Land-use has largely been for grazing and rural lifestyle living, including a horse track. Figure 1 below shows an aerial view of the site where these features can be seen.

Figure 1. Aerial View Of The Site, Parcels, Contours And Existing Drainage Routes

1.2 Published Geology and Soil Classifications

The institute of Geological & Nuclear Science Geological Map for "Christchurch"[1] reports ground geology beneath the site to be grey river alluvium deposits of the Springston Formation.

The S-Map soil information reports these river alluvium deposits to be split into two primary classifications, deep poorly drained silts in the eastern portion of the site and loam in the west portion. The silty loam soils occupy the Lincoln_2, Wilowby_7, and Flaxton_1 areas shown in Figure 2 below; whereas, the soils on the rest of the site are considered to be shallow well drained loam (Eyre_2 soils). The loam soils cover roughly 14.6 ha of the site.

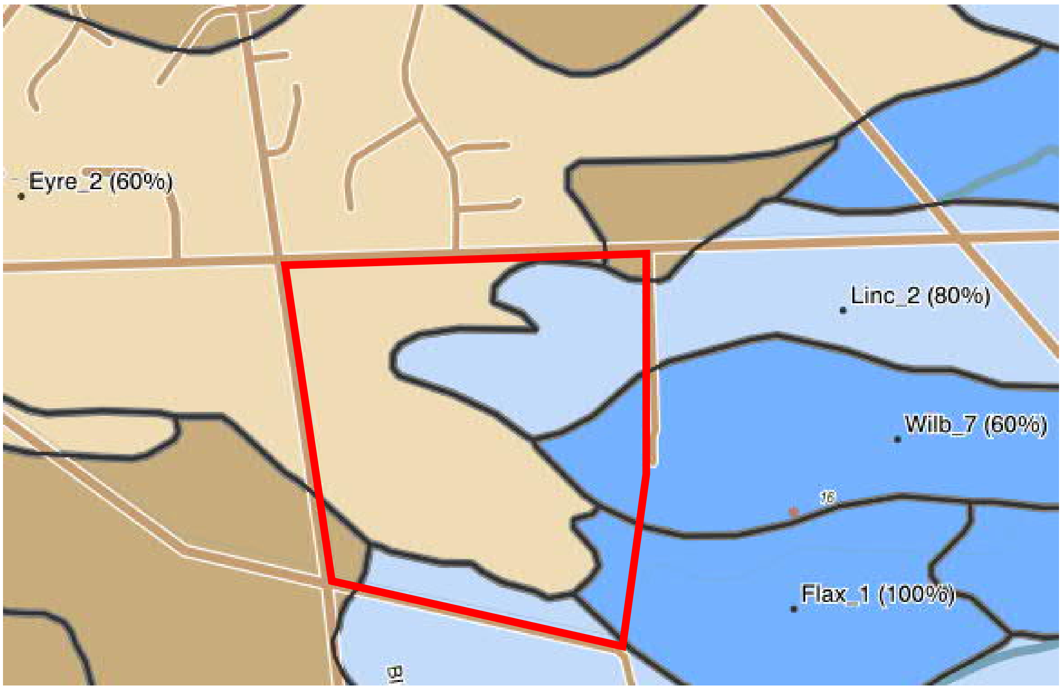


Figure 2. S-Map image for the Trices Road Site, Prebbleton

1.3 Ground conditions

Fraser Thomas Ltd (FTL) have undertaken geotechnical investigations (July 2020). These investigations included eight shallow hand augers and five cone penetrometer tests (CPTs) spread across the various properties within the site.

The hand augers were limited to depths ranging from 0.2m to 1.3m below ground level (i.e. BGL) due the refusal on gravels soils. CPT investigation depths were also limited by the underlying gravels but they reached, in general, slightly deeper depths ranging from 0.0m up to 3.8m BGL. Two CPT investigations, CPT 1 and 2, refused near the ground surfaces due to shallow gravels soils, which is the reason for the reported 0.0m penetration depth.

Although the geotechnical investigation depths were limited, the observations indicated that the site was underlain by alluvial sediments of the Springston Formation. The soil

[1] Forsyth, P.J; Barrel, D. J. A.; Jongens, R. (compliers) 2008. Geology of the Christchurch area. Institute of Geological & Nuclear Sciences 1:250 000 Geological map 16. 1 sheet + 67 p. Lower Hutt, New Zealand. GNS Science.

column was described as comprising 0.2m to 0.4m *topsoil* overlying *clayey silts*, then *silty sands and sands*, before reaching the *sandy gravels and gravelly sands* below.

Two test pits (TP1 and TP2) were later excavated by e2 in the centre of the proposed stormwater management area (see Figure 3 below) on 1st October 2020 to confirm ground soakage rates for stormwater disposal. The ground conditions observed within these test pits were largely consistent with the previous geotechnical investigations except that the shallow *clayey silts* were not encountered. A summary of the ground conditions observed by e2 is provided in Table 1 below.

Layer	Soil Description[1]	Depth to top (m BGL)[2]	Typical layer thickness (m)
1	Fine SAND, with some silt; dark brown. Loosely packed, dry to moist, poorly graded [TOPSOIL]	0.0	0.2 - 0.3
2	Silty fine SAND; light grey, mottled orange. Loosely packed, moist, poorly graded.	0.2 - 0.3	0.4 - 0.5
3	Sandy fine to coarse GRAVEL, minor cobble, trace silt; brownish grey. Tightly packed, moist, poorly graded, rounded to subrounded; sand, medium to coarse; cobble, rounded to subrounded.	0.6 - 0.8	+1.7[3]

Note:

1. Soil descriptions in accordance with the NZ Geotechnical Society Inc. Field Description of Soil and Rock (December 2005)
2. The depth to soil layers, and their thickness decreased to the east of the east, by the soils were largely similar in description.
3. This is the minimum layer thickness confirmed onsite to the depth of the soakage pit excavations (ranging from 2.3 to 2.6 m BGL). It is possible that the layer extent continues with depth.

Table 1. Soakage Testpit soil descriptions by e2

Figure 3 Soakage Tests north of Hamptons Road in the Stormwater Management Area (SMA)

1.4 Groundwater

The hand augers and test pits completed at the site did not encounter groundwater. The geotechnical report by FTL inferred groundwater to be at a depth of 2.5m based on CPT pore pressure profiles.

Existing bores/wells on the site near the soakage test locations suggest the groundwater levels could be 3.6 to 4.0m BGL based on initial readings. These wells and initial groundwater readings are summarised in Table 2 below.

Well number	Initial Groundwater Depth (m BGL)
M36/7441	3.70
M36/3133	3.60
M36/3138	No reading
M36/3134	3.90
M36/3766	3.83
M36/3767	4.00
M36/5524	3.61

Table 2 ECan Well Log Records

1.5 Ground soakage results

e2 performed soakage tests on the 1st October 2020 inside the two test pits that were excavated, refer Figure 3 for their approximate locations. The soakage tests were completed in accordance with the methods described by the BRE Digest 365 Soakaway design guidance report[2].

Each test pit was filled with water to around 1.0m in depth and the drop in water level was recorded over time. Three soakage tests were completed in the first test pit (TP1), and two in the second test pit (TP2) to establish the soakage potential of the insitu gravels when well saturated with time. The results for all tests are shown in Table 3 below.

Test #	Soakage rate (mm/hr)	
	TP1 Width: 1.25m, Length: 1.6m, Depth: 2.6m	TP2 Width: 1.25m, Length: 1.7m Depth: 2.3m
1	300	40
2	160	40
3	135	No test

Table 3. Field soakage results, Trices Road, Prebbleton

The location of TP2 is to the east side of the proposed Stormwater Management Area (SMA) within the poorer drainage silty loam soils described by the S-Map and this may explain the significantly lower ground soakage rate observed compared to the soakage in TP1 which was located at the edge of the loam soils to the west of the proposed SMA.

Soakage rates between the investigations (located 105m apart) should, in theory, have been similar given they are both in the same soil category (see–Figure 2). The difference is assumed to be due TP2 being further east and possibly influenced by the imperfectly drained soils east of the site.

[2] BRE Digest 365 Soakaway Design Method for Falling Head Soakage Tests

2 STORMWATER SERVICING

2.1 Standards and Codes of Practice

The development is to be built to Selwyn District Council's (SDC's) Engineering Code of Practice (2010) and reference applicable parts of the Christchurch City Council Waterways, Wetlands and Drainage Guide, Ko Te Anga Whakaora mö Ngä Arawai Rëpo (WWDG) (2012) Part A and B for treatment, hydrological and hydralulic modeling. The following requirements are assumed:

- First flush water quality depth of 25mm
- Primary pipe conveyance of the 10% AEP capacity
- Where discharging stormwater to a surfacewater body in the Halswell River catchment , attenuate post development discharges for all events up to the 60hr 50 yr ARI duration to pre development rates.
- Where possible discharge clean roof water to land via soak holes and ensure secondary flow paths are to the road and SMA.
- All buildings shall be constructed so that there is 300mm from 50yr-ARI floodwater levels to finished floor levels.

2.2 Conceptual Stormwater Servicing

- Zonal runoff coefficient has been applied at C=0.54 for the ≤50yr ARI event based on a weighted average runoff coefficent based on the proposed zonings of the development.
- Primary reticulation conveys runoff to flow splitter structures in the SMA
- First flush runoff is detained for a minimum of 24 hours and discharged to surface water.
- The first flush volume is calculated at 3,800m³ based on WWDG Chapter 6 Eqn 6-2.
- To attenuate all storms up to and including the 60 hr duration 50yr ARI event an additional 6,200m³ storage is estimated to be required. This can be achieved in basins in the SMA area, if the basins are around 1m deep, with a 180mm orifice outlet provided to the waterway on the boundary (calculations are attached in Appendix B).

3 COMMENTRY ON PRELIMINARY SMA SIZING

3.1.1 Stormwater Estimates

The rational method has been used to estimate the stormwater attenuation volume requirements.

For catchments ≥ 7 ha it tends to over predict volumes of runoff. Hence this method is considered to be conservative and at the upper end of stormwater attenuation requirements for the site. The basin areas required in the SMA are detailed on plans in Appendix A.

3.2 Stormwater Reticulation

We anticipate that a network of stormwater pipes will be installed in the road reserves to collect runoff from roads (and roofs where necessary) and convey it to the SMA. The geotechnical report indicates groundwater is likely at depth in the underlying gravels and stormwater pipes are anticipated be able to be installed above groundwater levels.

3.3 Secondary Flow and Outfalls

It is anticipated that all roads will be used as secondary flow paths and will convey stormwater through the site to the SMA. The SMA is to be built in the lower part of the site and connect via a single outfall to an existing drain on neighbouring land (adjoining to the east – see Figure 4 below).

The peak discharge from the subdivsion will be kept below pre-developent discharge rates by the use of an attenuation basin. Calculations indicate that it is feasible to provide sufficient attenuation to achieve attenuation of all storms up to and including the 60 hour 50 yr ARI storm event. This assumes a weighted zonal runoff coefficient of 0.54. We therefore consider that the preliminary attenuation volume calculated is conservative high.

3.4 Soakage to Land

Approximately 14.6ha of land on the western side of the site is loam soil which the geotechnical report states is only 0.3 - 1.3m deep and overlys site gravels. Our intial soakage testing indicates that in the south east the soakage is not very high (in the location of the SMA). Potentially further investigation may show that soakage at higher rates are achievable on other areas of the site, which would result in a reduction of attenuation volume in the SMA.

The initial testing shows that relying on discharge to land in infiltration basins is not likely to be feasible because the low soakage potential. The possible consequence is the basins don’t empty sufficiently fast enough to drain the volumes within 48 hours after a storm event resulting in plant die-off.

3.5 Outfalls

There is an existing waterway, Crosslands Drain (which is part of SDC Stormwater Network) 190m to the south of Hamptons Road and Hamptons Road, itself drains to a shallow ephemeral drain located east of the site (Lot 1 DP365486 owned by the Drinnans) see Figure 4 below.

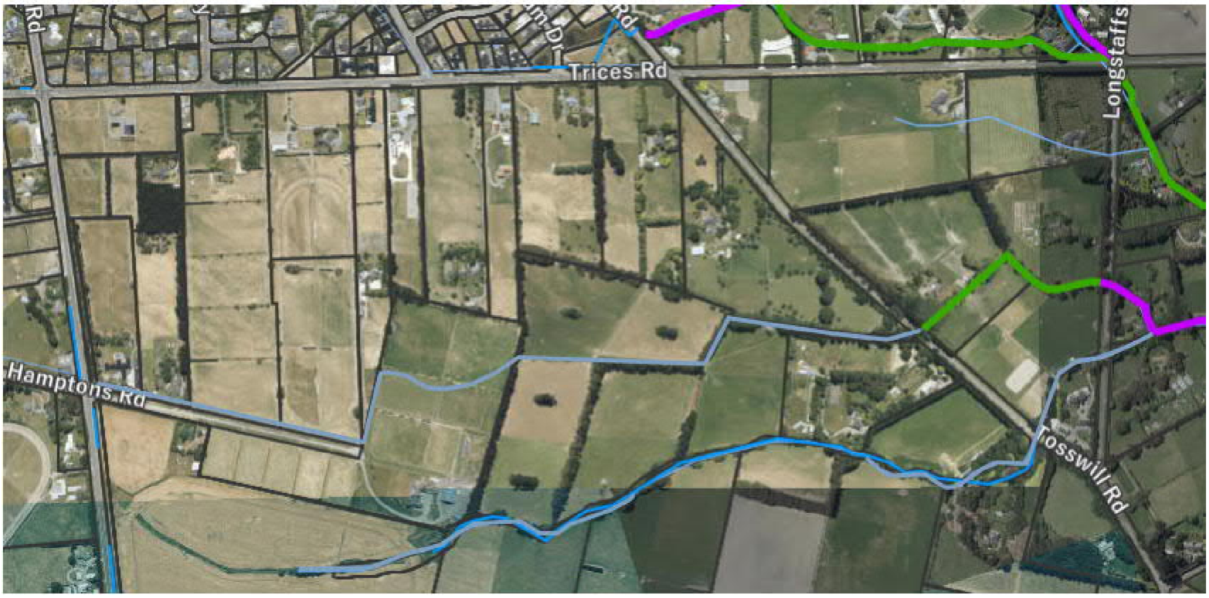


Figure 4 Wider Drainage Network

The drain serving Hamptons Road is ephemeral next to the block and eventually joins up with Crosslands Drain near the intersection of Tosswill Road and Longstaffs Road approximatley 1.32 km from the natural outfall noted on eastern boundary of the site (see Figure 4 above).

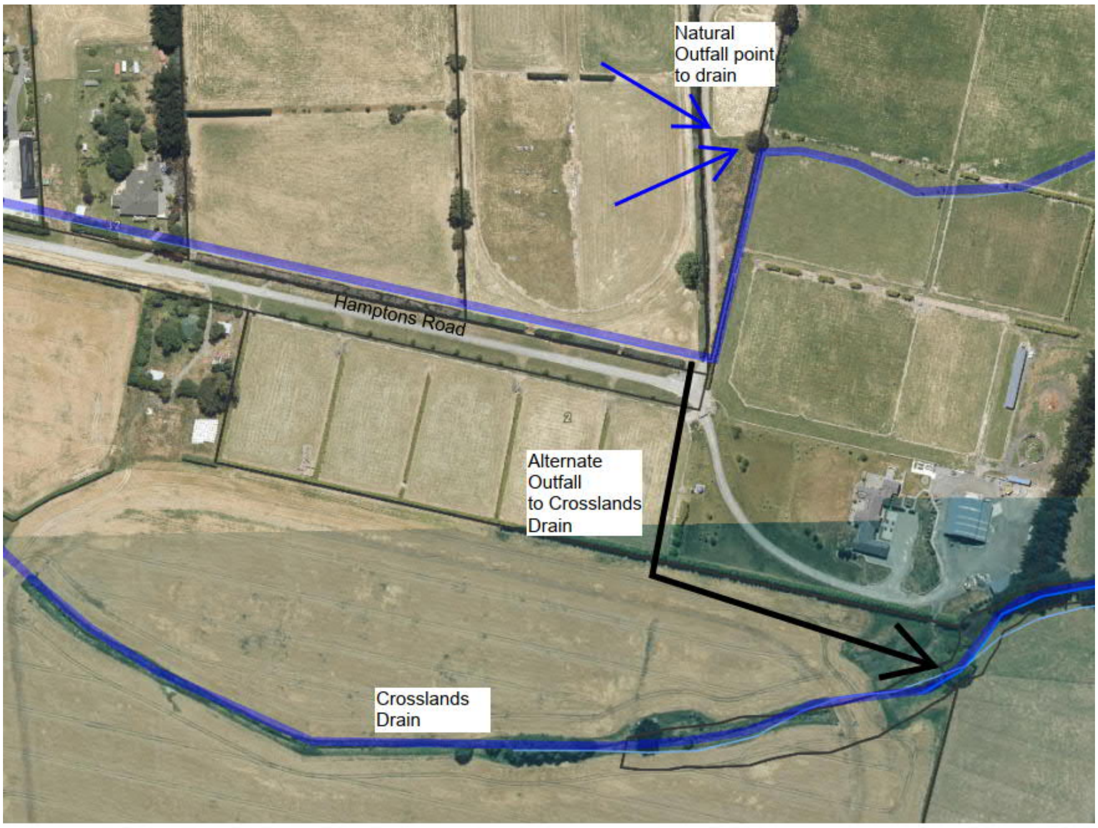


Figure 5 Outfalls from Site to SDC Stormwater Network

The simplest servicing solution is to utilise the natural drainage outfall into the Drinnans land. The invert of drain at this location is nearly 0.8m below exising ground levels. This means stormwater attenuated in the proposed basins of the SMA will drain freely into this receiving drain. The rate of discharge from the SMA will be controlled by a outlet structure so that there is no change in peak post-development flow to the existing drain network.

An alternative outfall considered was to estabish a connection immediately to the south into Crosslands Drain. We note that he invert of the drain is elevated relative to the proposed basins in the SMA which would result in a 1:500 gradient (measured invert to invert).

This makes outfalling to this waterway difficult because any existing backwater effects might hold back discharges, and large pipes or a large shallow channels would be required to convey flows away from the site. Stormwater would need to be conveyed through or under Hamptons Road to connect to Crosslands Drain and a right to drain water easement would be required. However, in its favour is that SDC are developing land to the south and might be ammenable to a connection into their proposed Birchs Road Reserve (see Figure 6 below).

Figure 6 SDC Proposed Birchs Road Reserve Masterplan

We consider that discharging to the existing drain to east of the site is the most practicable stormwater servicing solution for this development.

3.6 Further Work

We recommend that during the subdivision consent phase a hydraulic and hydrological model is built of the primary stormwater network and basins to accurately model the stormwater system and confirm the land required in the SMA for stormwater management.

We also recommend more wide spread soakage testing across mutiple locations to assess how much roof soakage is possible. This can be fed into the hydrolocigal model when optimising the basins in the SMA.

4 CONCLUSIONS

Preliminary sizing of the stormwater basins in the proposed SMA to serve the entire 28ha requires approximately 10,000m³ of attenuation including the first flush runoff volume.

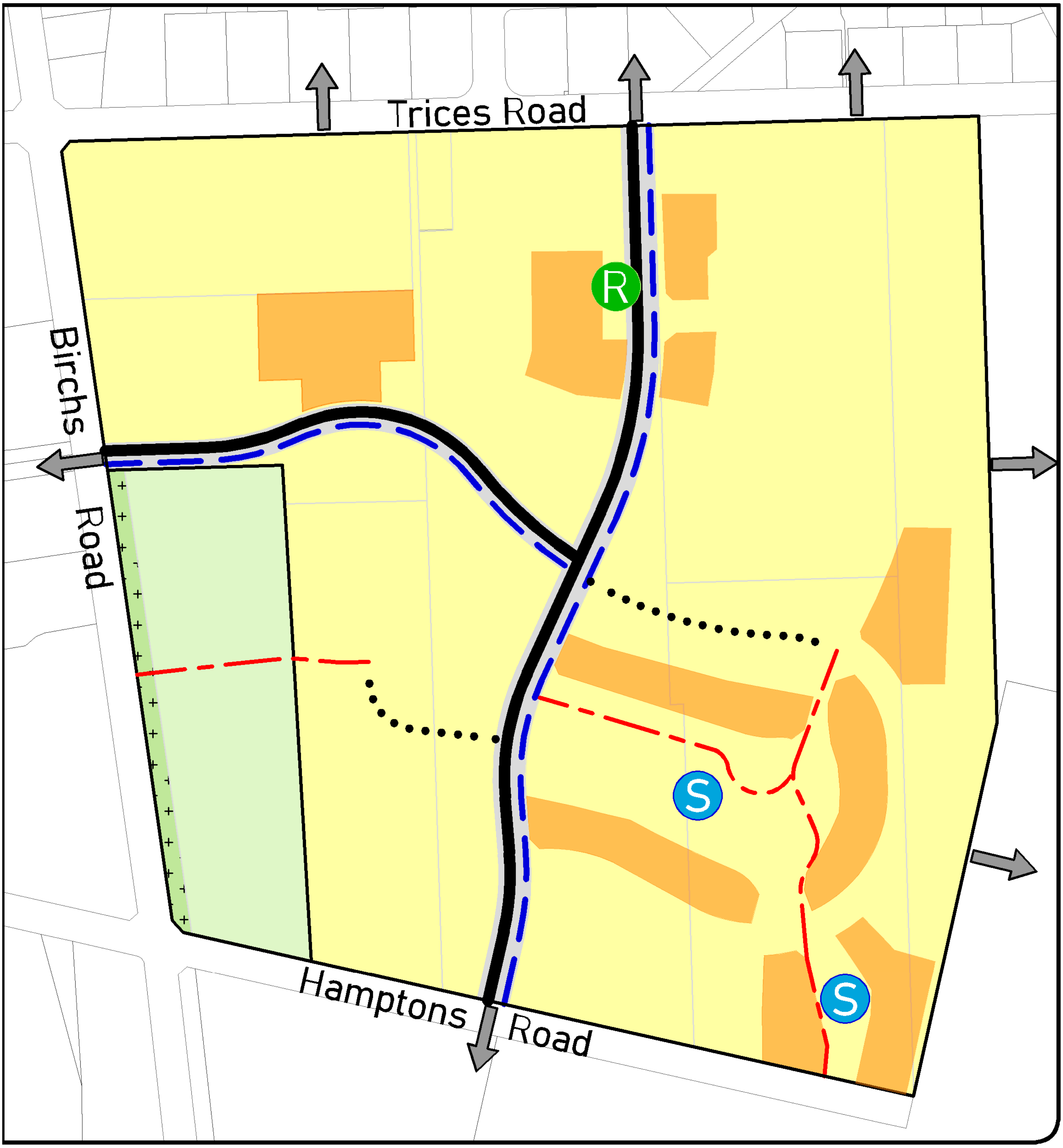
This volume is assessed to be conservative and is likely to reduce as a result of more accurate hydraulic stormwater modelling used to optimise the stormwater system, and if roof runoff is to discharge to land where feasible.

We conclude it is feasible to construct and build stormwater first flush and attenuation basins to mitigate the effect of the proposed stormwater runoff generated by the Trices Road Plan rezoning.

APPENDICES

APPENDIX A – Plan Change Scheme and Drawings

Prebbleton 5 Development Area
Operative District Plan



Indicative Primary Road
Indicative Local Road
Potential future road connection
Indicative separated shared pedestrian / cycle path (within road corridor)
Indicative shared pedestrian / cycle path (off road)
Landscaping strip (existing mature planting)
S Stormwater management area (size to be determined at time of subdivision)
R Reserve location (size to be determined at time of subdivision)

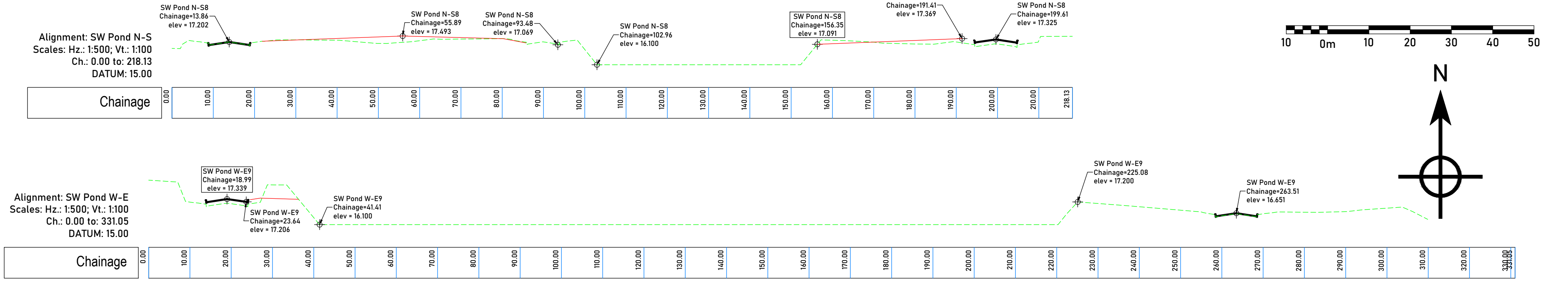
Living 3 Zone (Hamptons/Birchs Road)
Living Z Zone (Prebbleton)
Indicative Living Z Zone (Medium Density)

N
NTS

Cadastral information derived from Land Information
New Zealand's Digital Cadastral Database (DCDB)

Rev M - 19 May 2021

K:\4719G George and Jeffs - 311 and 321 Trices Road\CAD\4719G 20200615 Eng Concept.dwg : 05 Oct 2020 4:12 PM : A1 Planprof 250



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Trices Road Rezoning Group

Stormwater - Preliminary calculations

Address:

Consent No:

Project Phase:
Prelim Eng Design

No.	Revision	Appd	Date
#	##	##	##
#	##	##	##
#	##	##	##
#	##	##	##
#	##	##	##
#	##	##	##
A	Update road grades and levels - no section fill or cuts	caf	05/10/20

Scale (A1)	1:500
Reduced (A3)	1:1,000
Designed	CAF
Drawn	CAF
Checked	-
Date	05/10/2020

Job No. 4719G
Sheet No. 1 of 1
Rev. A

APPENDIX B – Stormwater Calculations

Trices Road Property Group
Preliminary Stormwater Basin Sizing
Achieved using Rational Model Calculation
Rev 2 (21/10/2020)
Ref: SDC ECoP
~~Post Dev. Runoff C = 0.65 for 50% AEP for developed urban areas (5.12.3, 2010)~~ too conservative
Pre Dev. Runoff C =0.30 rural (assumed)
Residential GR zoning C assumed to be 0.56 in 50yr storm, in line with CCC RSDT (assuming 12 houses/ha)
Residential LLR zoning assumed to be between 0.36-0.4

CCC WWDG June 2020:

Table 21-5: Peak Flow Rate Runoff Coefficients versus AEP

District Zone	Storm AEP 20%	10%	5%	<=2%
Residential Suburban (RS)	0.38	0.42	0.44	0.47
Residential: Suburban Density Transition (RSDT)	0.47	0.51	0.53	0.56
Residential New Neighbourhoods (RNN) & Residential Medium Density (RMD)	0.56	0.60	0.63	0.65
	[illegible]	[illegible]	[illegible]	[illegible]

Basin 1m deep water, freeboard 0.3m but no incl in calcs
Ave ground level mid reach of SMA 17.1m, Outlet invert 16.1m RL to waterway (approx 1m for basin water depth)

First Flush Runoff Volume (WWDG Part B Eqn 6-2)

$$V_{ff} = 10 \times C_{ff} \times A_{total} \times d_{ff} \quad (m^3) \qquad \text{Eqn (6-2)}$$

Where Cff = 0.53 based on L2 zoning, A = 28.7ha, dff =25mm
Vff = 3,803 m³

	A	C	∑ C.A
General Res	25.9	0.56	14.50
Large Lot Res	2.8	0.4	1.12
			0.54 Weighted PC Zonal Runoff C

HIRDS V4 Intensity-Duration-Frequency Results
Sitename: Trices Rd
Coordinate system: WGS84
Longitude: 172.5164
Latitude: -43.5922
DDF

Model	Parameters	c	d	e	f	g	h	i
	Values:	-0.0112	0.516893	-0.01115	-0.00184	0.33994	-0.01375	2.241307
	Example:	Duration (h	ARI (yrs)	x	y	Rainfall Rate (mm/hr)		
		24	100	3.178054	4.600149	5.172378		

Rainfall intensities (mm/hr) :: RCP8.5 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	60
50	2%	97.1	68.4	55.6	38.8	26.5	13.8	8.84	5.52	3.39	2.95

Total Basin Top water area 1.05ha excl free board

Pond Analysis

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Catchment Parameters

Pond Treatment Area =	287,000	m²
Pre-development Runoff Coefficient =	0.30	(-)
Post-development Runoff Coefficient =	0.54	(-)

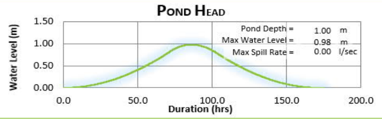
Design Conditions

Design Rainfall Zone =	Alternative Rainfall	
Design Storm Duration =	60 hr	
Design Storm Return Period =	50	yr

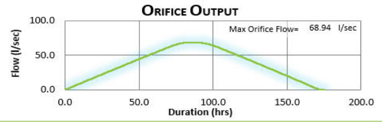
Pond Details

Pond Top Length =	105.0	m	Pond Depth =	1.00	m
Length Side Slope z_L =	2.5	$1v:z_Lh$	Pond Volume =	9,995.83	m³
Pond Top Width =	100.0	m	Orifice Size =	(Marley Nom. Size) 180	mm
Width Side Slope z_W =	2.5	$1v:z_Wh$	Orifice Type =	Short tube (C=0.80)	(-)

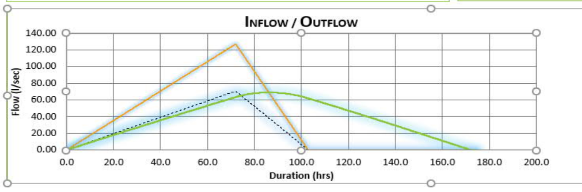
Pond Head



Orifice Output



Inflow / Outflow



Runoff Results

Pre-Developed Peak Flow =	70.61	l/sec
Pre-Developed Runoff Volume =	13073.05	m³
Without Tank		
Post-Developed Peak Flow =	127.10	l/sec
Post-Developed Runoff Volume =	23531.49	m³
With Tank		
Post-Developed Peak Flow =	68.94	l/sec
Post-Developed Runoff Volume =	23531.49	m³

NOTES: Storm Duration 60yr - Total runoff attenuation incl First Flush Basin vol of 3,800m³. Calculation over predicts runoff from catchment by Rational Method. Full hydraulic model recommended for Engineering Approval

Alternative Rainfall Data

CONSULTING ENGINEERS

Rainfall (mm/hr) *(copy and paste date from NIWA High Intensity Rainfall System V2 output)*

		Duration (hrs)	0.17	0.33	0.50	1	2	6	12	24	48	72
year	% AEP	Duration (mins)	10	20	30	60	120	360	720	1,440	2,880	4,320
1.58	63%											
2	50%											
5	20%											
10	10%											
20	5%											
30	3.3%											
40	2.5%											
50	2%		97.10	68.40	55.60	38.80	26.50	13.80	8.84	5.52	3.39	2.95
60	1.7%											
80	1.3%											
100	1%											

APPENDIX C – Geotechnical (excerpt only)

e2 environmental CONSULTING ENGINEERS

10 blows per 50 mm penetration, corresponding to a SPT ‘N’ value of generally greater than 50, corresponding to a very dense consistency.

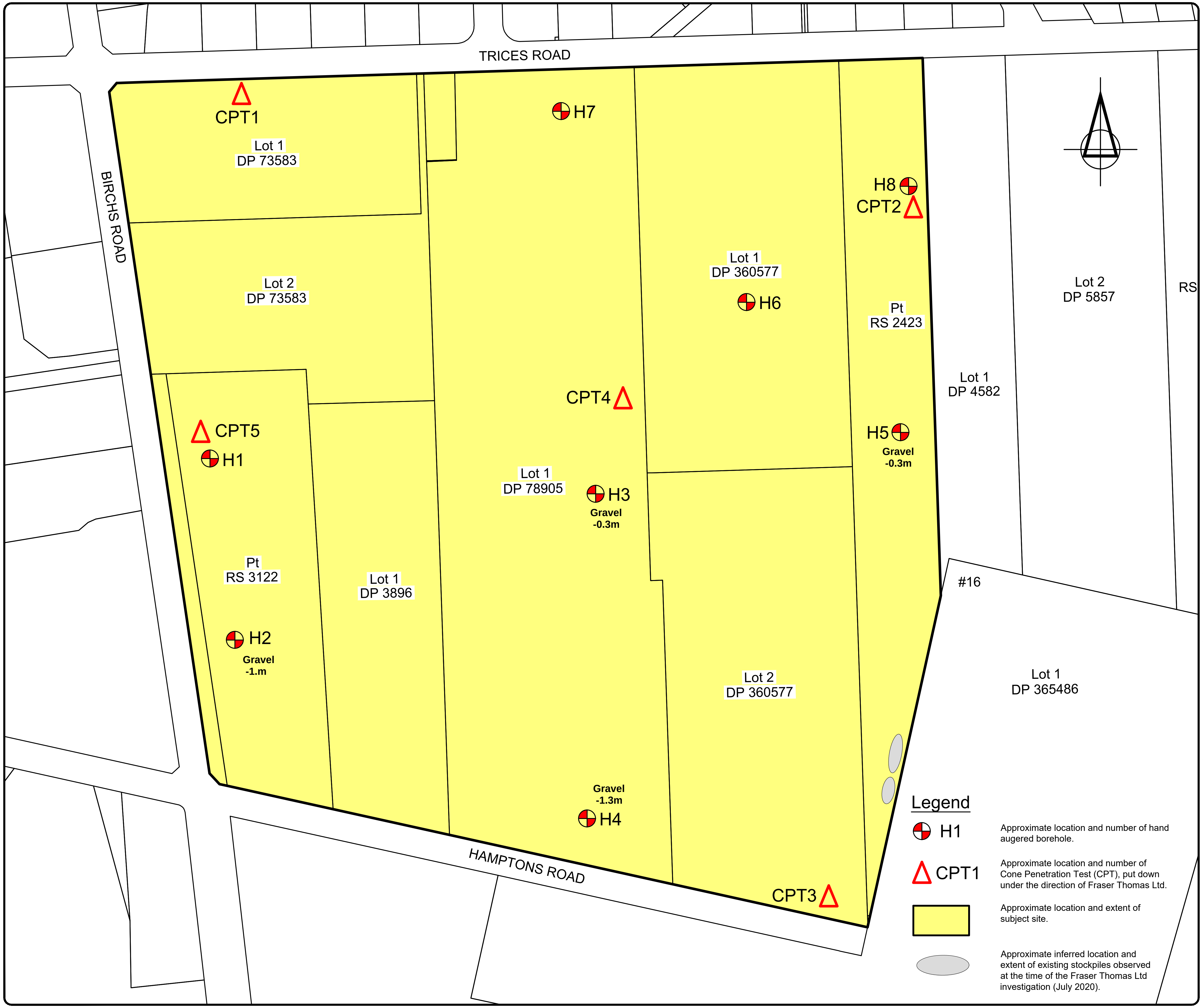
The logs of some existing machine excavated test pits have been sourced from the NZGD. The test pits are located at a site abutting the western site boundary. These logs indicate that gravels are encountered at shallow depth and are encountered to the extent of the test pits (i.e. 3.0 m depth). The logs indicate that cobbles were also encountered in the gravel soils.

The logs of existing water bore logs, put down within the subject site, have also been sourced from Environment Canterbury records.

The existing water bore logs indicate that sandy gravels are generally located at shallow depths, which is consistent with the subsoil conditions encountered at the subject site. The bore logs indicate that these sandy gravels generally extend to depths of greater than approximately 18 m below the ground surface. Based on the foregoing, it is, in our opinion, likely that the gravel soils underlying the site extend to significant depths below the existing ground surface.

8.4 GROUNDWATER

Based on the results of the borehole investigations undertaken at the site, and ground investigation information obtained from the NZGD, the groundwater level is inferred to be at a depth of approximately 2.5 m below the existing ground surface, for analysis purposes.



			APPROVED	DATE
SURVEYED			MVR	
DESIGNED				
DRAWN	JTG	13/07/2020		
CAD	GS	14/07/2020		
CHECKED				

REVISION	CHANGES	CHECKED	DATE
A	Issued for information	MVR	22/07/20

NOTES

1. This plan has been adopted from Quick map. The location and extent of the site boundaries and site features are therefore considered to be approximate only.

CLIENT

TRICES ROAD REZONING GROUP

PROJECT

TRICES ROAD AREA
PREBBLETON

TITLE

SITE PLAN

Fraser Thomas

ENGINEERS • RESOURCE MANAGERS • SURVEYORS

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DRAWING No G00417-01 REVISION A

SHEET 1 of 1

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Appendix B: Flood Hazard Assessment

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9 November 2020

Trices Road Rezoning Group
C/O Fiona Aston
Aston Consultants
PO Box 1435
Christchurch, 8140

Attn: Fiona Aston

Dear Fiona,

Trices Road Rezoning Group – Flood Hazard Assessment

e2Environmental Ltd (e2) have been engaged by Trices Road Rezoning Group to assess the Flood Hazard and appropriate mitigation to support the proposed Plan Change development of 28.7 hectares of land bounded by Trices Road, Birchs Road and Hamptons Road, in Prebbleton.

The flood hazard and anticipated mitigation have been assessed against the proposed District Plan and associated Selwyn District Council (SDC) flood maps which are expected to come into effect in 2022.

Flood Hazard

Review of the SDC Flood Maps[1] confirms that the site is not subject to:

- coastal flooding,
- flooding from the Waimakariri River,
- flooding from the Selwyn River.

There are no local waterways / water bodies within the site boundary and the only source of flooding is therefore pluvial flooding which occurs when the land can't absorb rainfall and excess water either runs off or ponds.

1 https://ecanmaps.ecan.govt.nz/portal/apps/webappviewer/index.html?id=57c74073c2f14a85ac0caf30073ae48a

In terms of the proposed District Plan the site is classified as being within the 'Plains Flood Management Overlay'.

Figures A1 and A2 in Attachment A provide an overlay of the current proposed masterplan and the 1 in 200 (0.5% AEP)[2] and 1 in 500-year (0.2% AEP) pluvial flood depths.

From review of the overlays it can be seen that:

- In both the 200-year and 500-year rainfall events there are:
    - o significant areas of the site that are free from flooding.
    - o significant areas of the site where the flood depth is < 200 mm.

- apart from isolated pockets of localised ponding there are three preferential overland flood flow paths (local drainage routes) through the site.

- Flood depths in the preferential flood flow paths (localised drainage routes) are typically < 500 mm.

- In the 200-year event, flooding > 500 mm depth is limited to a very small isolated area near the eastern boundary. The maximum flood in this area is 0.63 m

- In the 500-year event there are a few very small isolated pockets of flooding > 500 mm, the largest of these is again a small area near the eastern boundary where the maximum flood depth is 0.74 m.

The flood flow paths (localised drainage routes) are confirmed by review of the lidar (Figure 1 below).

Figure 1 – Lidar & Local Drainage Routes

[2] Annual Exceedance Probability, i.e. the probability that the event will be exceeded in any given year.

SDC were approached to confirm the flood hazard within the site (where the site flood hazard is assessed as being the flood depth measured in meters x flood velocity measured in meters / second). A high hazard area is defined in the proposed District Plan as being an area where the pluvial flood hazard is > 1.

The SDC flood hazard overlay (Figure A3 in Attachment A) shows that the majority of flooding for the 1 in 500 year event with RCP8.5 Climate Change allowance[3], within the site, has a flood hazard of <0.2. The overlay shows only one isolated pocket of flooding near the northern boundary where the flood hazard increases to 0.3. The site flood hazard is therefore classified as low for infants and small children and all adults.[4]

Defences against Water

The site is not located within proximity to any existing flood defences / stop banks and as such the proposed development will not effect any existing defences against water.

Flood Mitigation

In accordance with the proposed District Plan (Natural Hazards - Policy 12) the site earthworks would be undertaken to ensure that the off-site flood hazard was not exacerbated as a result of the proposed development. It is considered that this would involve maintaining the three preferential overland flood flow paths through the site. It is anticipated that, where possible, the overland flow paths would be aligned to follow the road network.

Review of the indicative ODP overlay (Figures A1 and A2) shows a reasonable correlation between the proposed road network and existing preferential flow paths. Limited areas of overland flow are expected upstream and downstream of the road network to collect flows into the road network and to discharge them in a manner that mimics the existing situation. It is expected that development within these areas would be managed by formalising the overland flow route (if possible), setting finished floor levels, maintaining ground levels, requiring permeable fencing etc and possibly consent notices to maintain and protect the existing flood flow routes. The post development preferential flow paths would be identified and protected through the planning process.

It is considered that managing flood flow paths through the site in this manner would allow development (which may include raising ground levels / impermeable house foundations) outside of the flood flow paths without exacerbation of the offsite flood hazard.

The assessment shows that there are no areas of high flood hazard within the site boundary which would be inappropriate for development.

In accordance with the Proposed District Plan (Natural Hazards - Policy 10) all future residential dwellings would be provided with Finished Floor Levels set 300mm above the 200-year flood event, noting that the proposed District Plan provisions may alter through the submission and decision making process.

It is also important to consider access / egress to and from the site during flood events and as such the road network should be designed to be free from inundation during a 5-year (20% AEP) event and be passable in a 10-year event. Although it is anticipated that the 3 preferential flood flow paths will be maintained through the site using the road network it is considered that:

1) Constructing preferential flood flow paths through the site will allow the on-site flood hazard to be managed,

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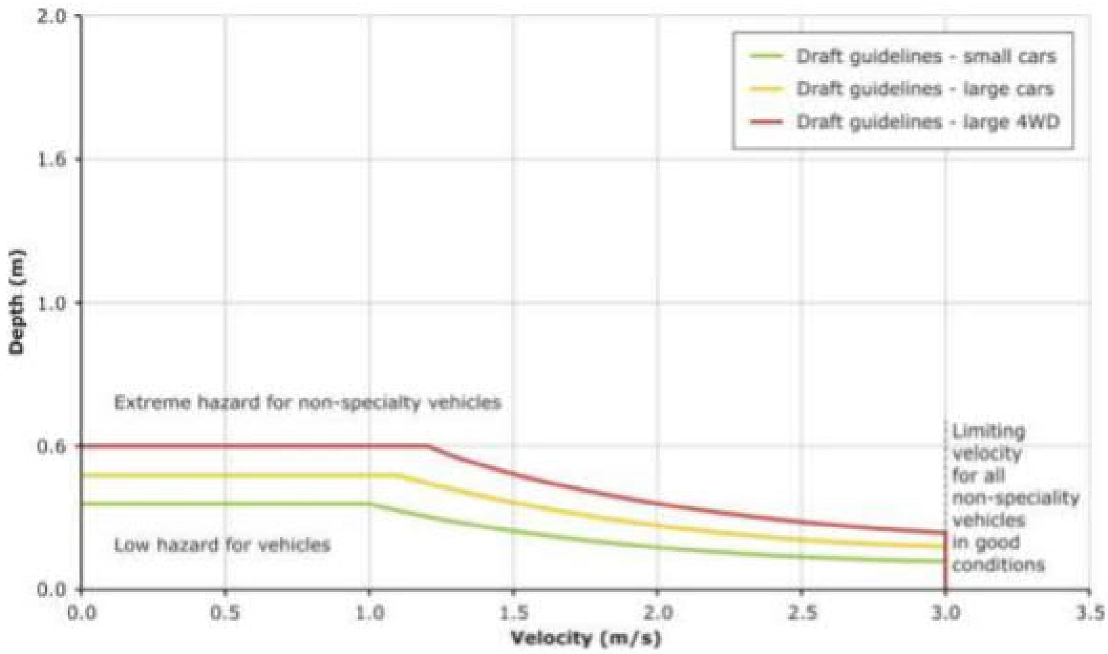
[3] The flood modelling assumes rainfall related to the worst-case climate change emission scenario Representative Concentration Pathway (RCP 8.5).
[4] DHI Regional Policy Statement Modelling for Selwyn District Council – District Plan, November 2019, Figure 5-1, pg. 13

2) The low flow hazard suggests that velocities are low which implies therefore that the 1 in 200 year and 1 in 500-year flood flow depths for the majority of the site (< 200 mm) would not present a hazard to small cars and site flood depths up to 600 mm depth (almost all of the site) would not present a hazard to emergency vehicles.[5]

Figure 2 – Flood Hazard to Vehicles



Source: Modelled after Shand et al. (2011)

Proposed District Plan Natural Hazards - Policy P11, which relates to the location of residential dwellings between a waterbody and any defence against water is not considered to be applicable to this development.

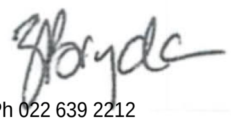
Conclusion

It is the conclusion of this assessment that the site flood hazard as identified within the SDC Flood Maps could be fully mitigated during the detailed design and subdivision process to allow the development of the Trices Road Rezoning Group Land for residential purposes.

Yours sincerely

Zeean Brydon

Associate Engineer



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[5] Australian Institute for Disaster Resilience, (2012). Technical flood risk management guideline: Flood Hazard, Guideline 7-3, Figure 9.

Attachment A Flood Overlays

Figure A1 – 1 in 200-year pluvial flooding

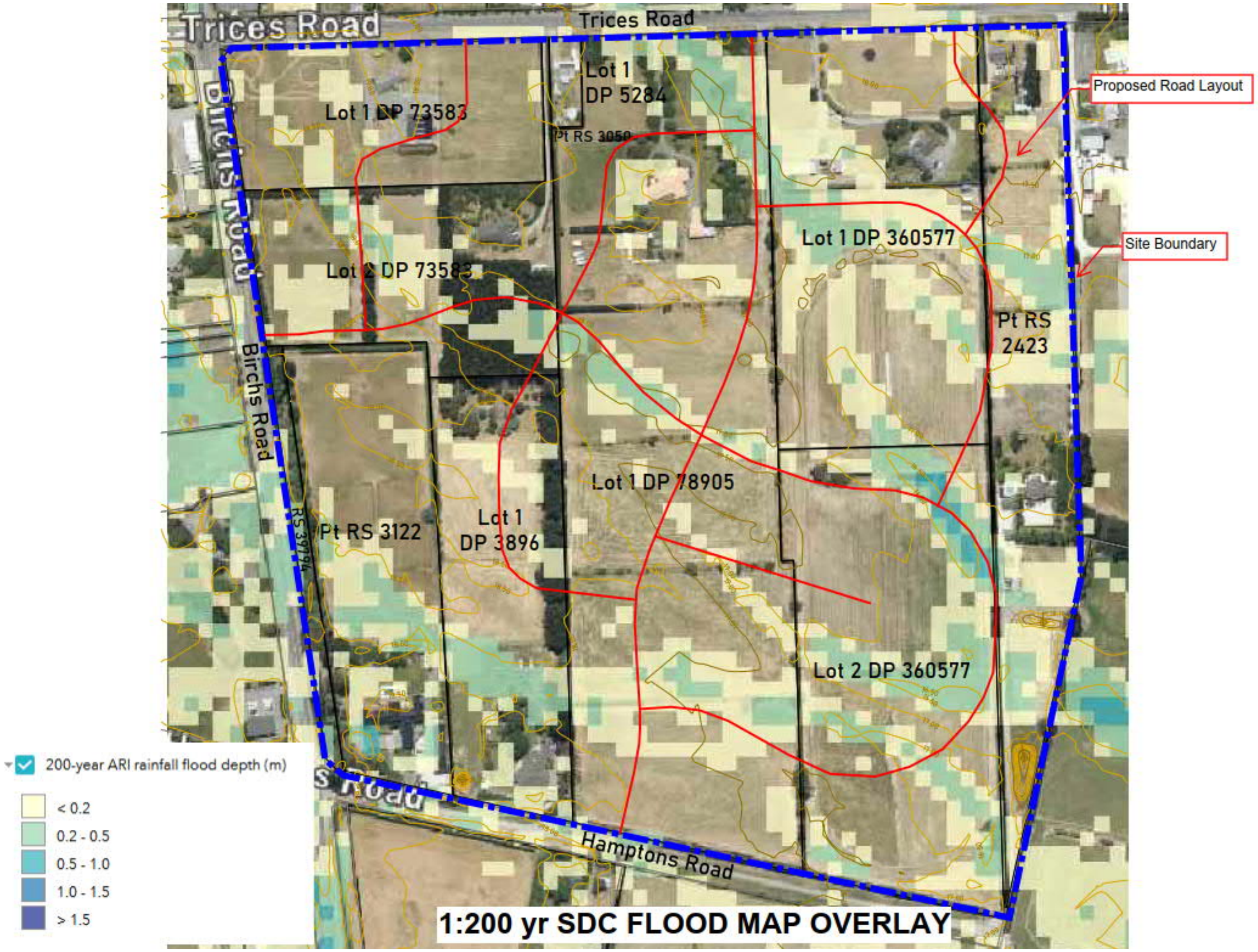

Figure A2 – 1 in 500-year pluvial flooding

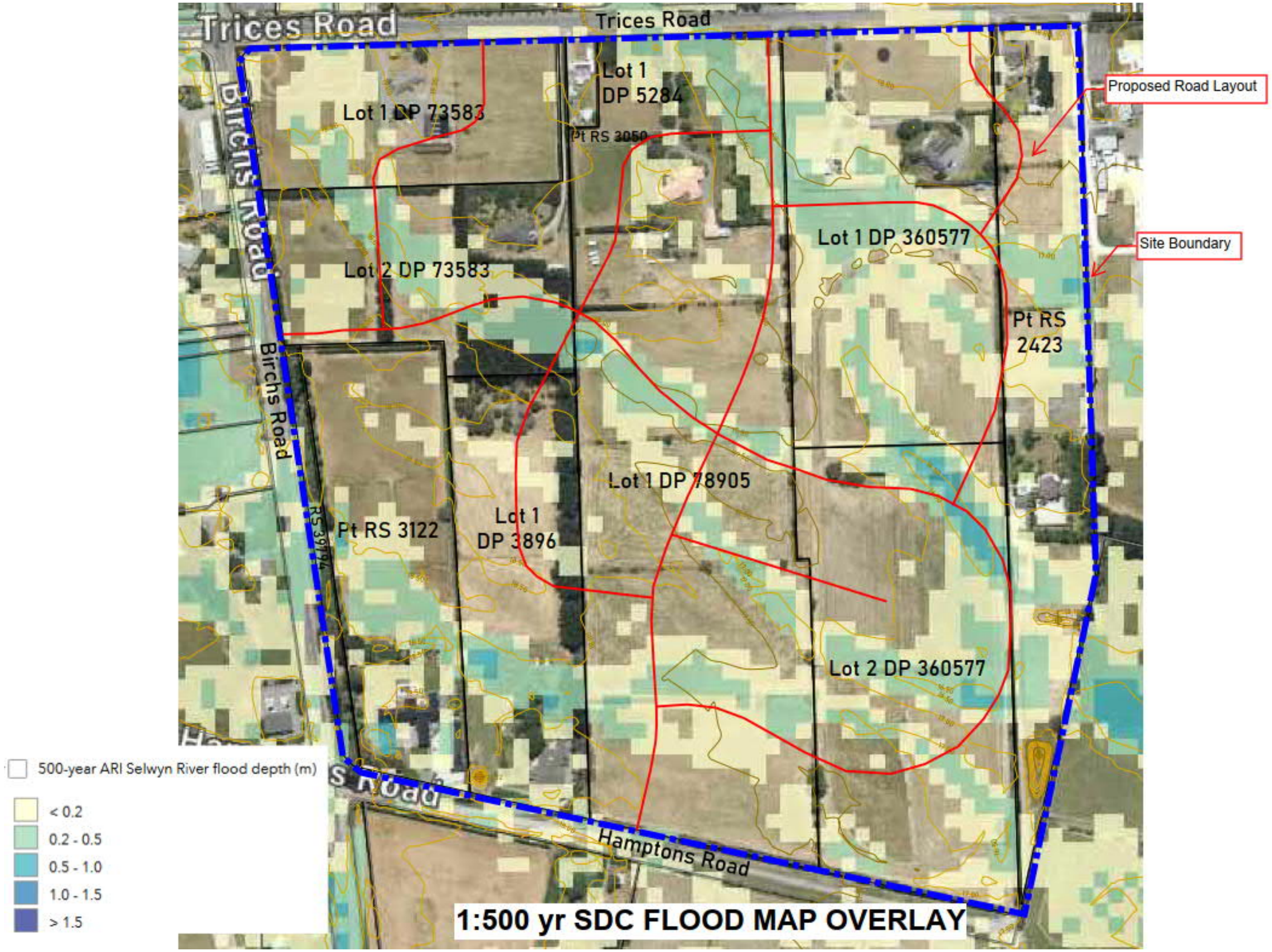

Figure A3 – 1 in 500-year Flood Hazard