

Appendix 4: Stormwater Servicing Assessment

Trices Road Private Plan Change

Stormwater Servicing Assessment

Trices Road Rezoning Group

9 November 2020



Quality Control

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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, Willowby_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.

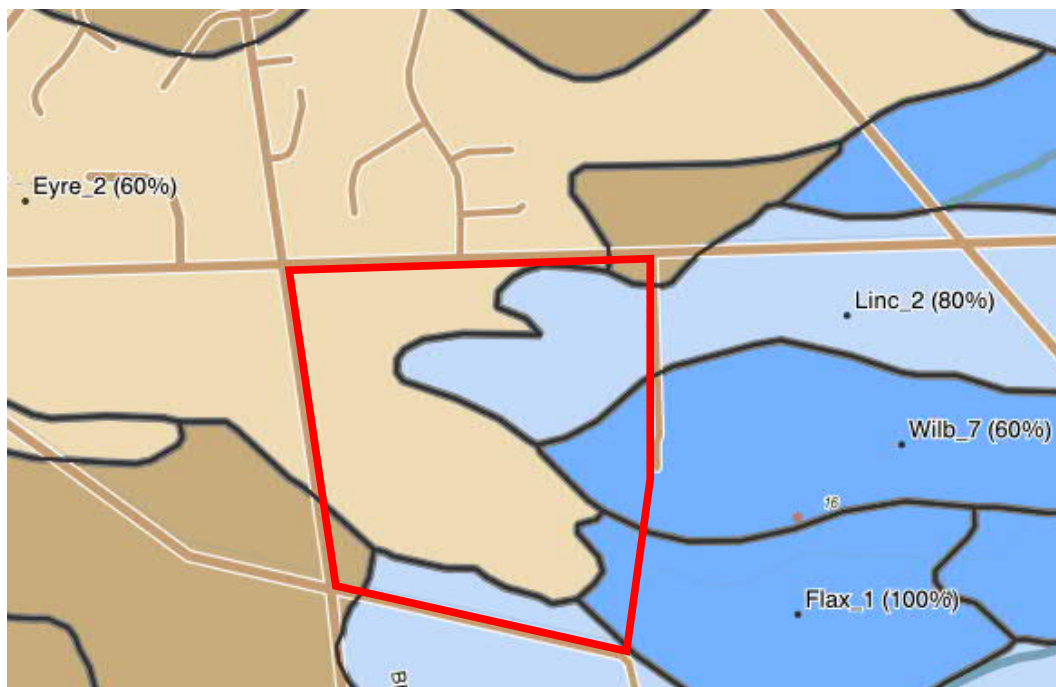


Figure 2. S-Map image for the Trices Road Site, Prebbleton

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 to the refusal on gravelly 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 gravelly 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. (compilers) 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 report².

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 Ngā Arawai Rēpo (WWDG) (2012) Part A and B for treatment, hydrological and hydraulic 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 ≤ 50 yr ARI event based on a weighted average runoff coefficient 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 $3,800\text{m}^3$ 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 $6,200\text{m}^3$ 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 subdivision 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 overlies 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 approximately 1.32 km from the natural outfall noted on eastern boundary of the site (see Figure 4 above).

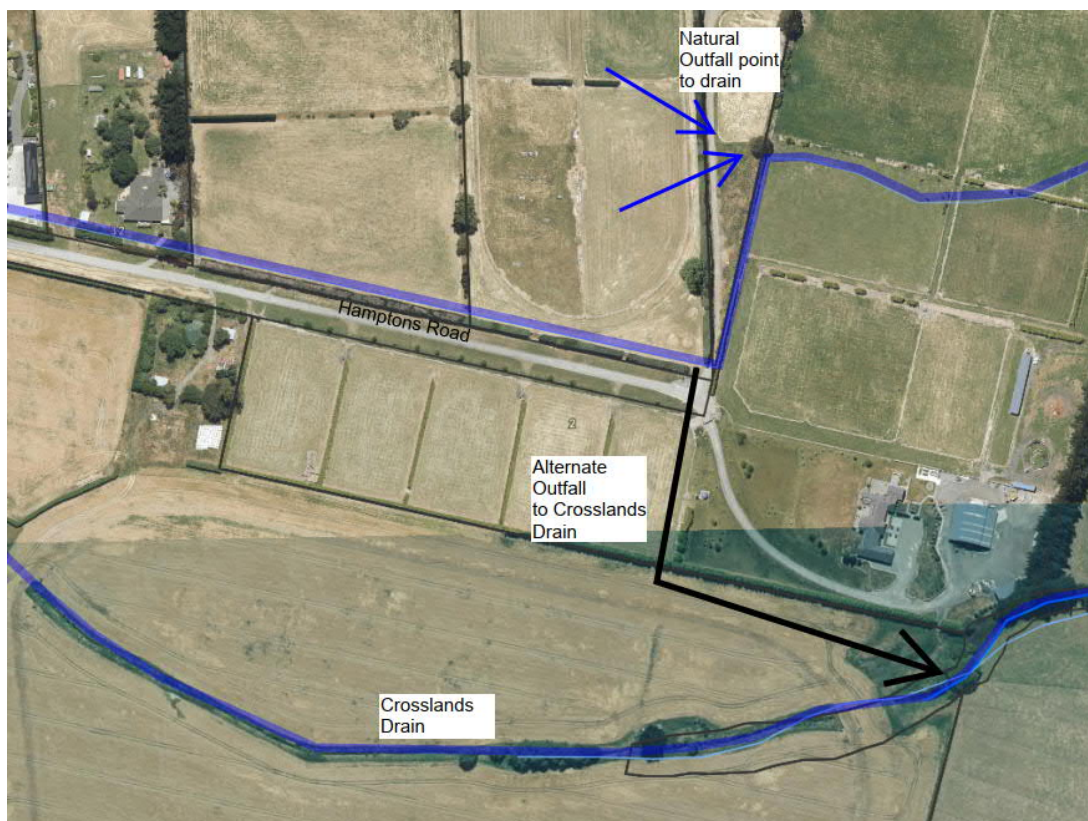


Figure 5 Outfalls from Site to SDC Stormwater Network

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 existing 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 establish a connection immediately to the south into Crosslands Drain. We note that the 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 amenable to a connection into their proposed Birchs Road Reserve (see Figure 6 below).

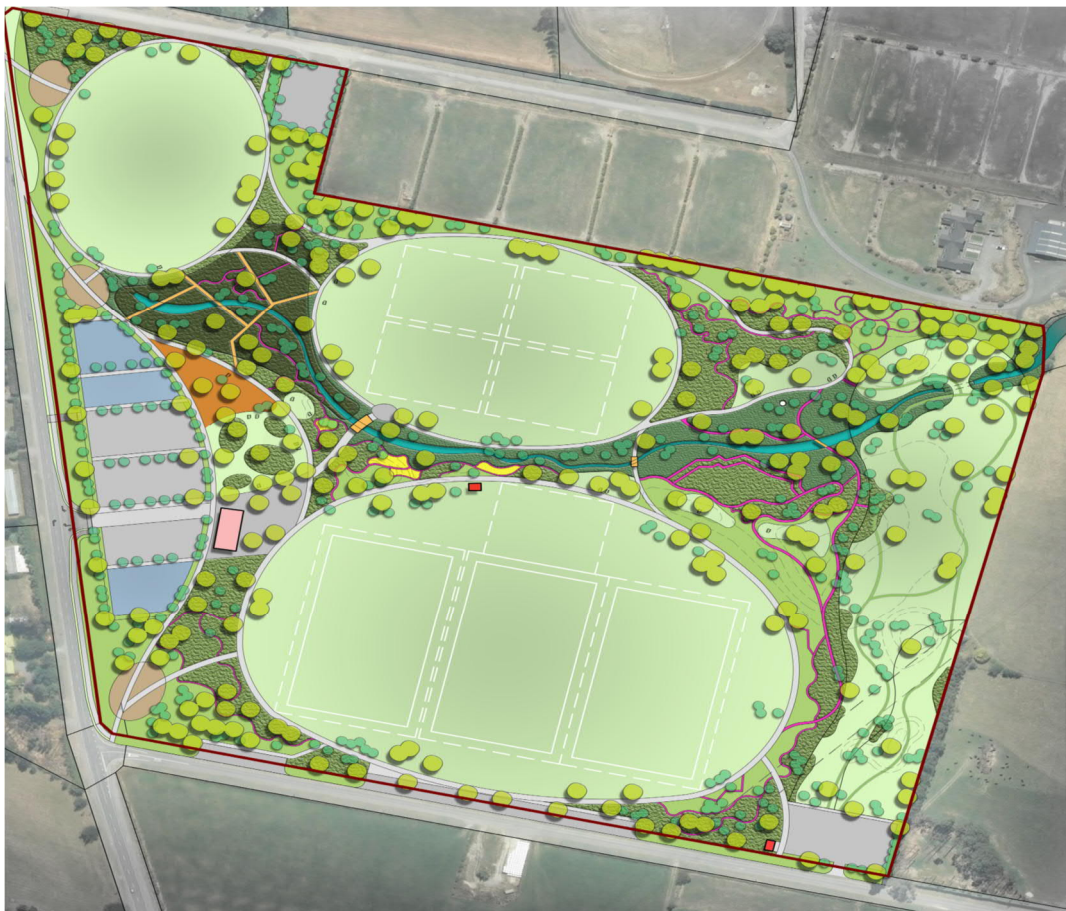


Figure 6 SDC Proposed Birchs Road Reserve Masterplan

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

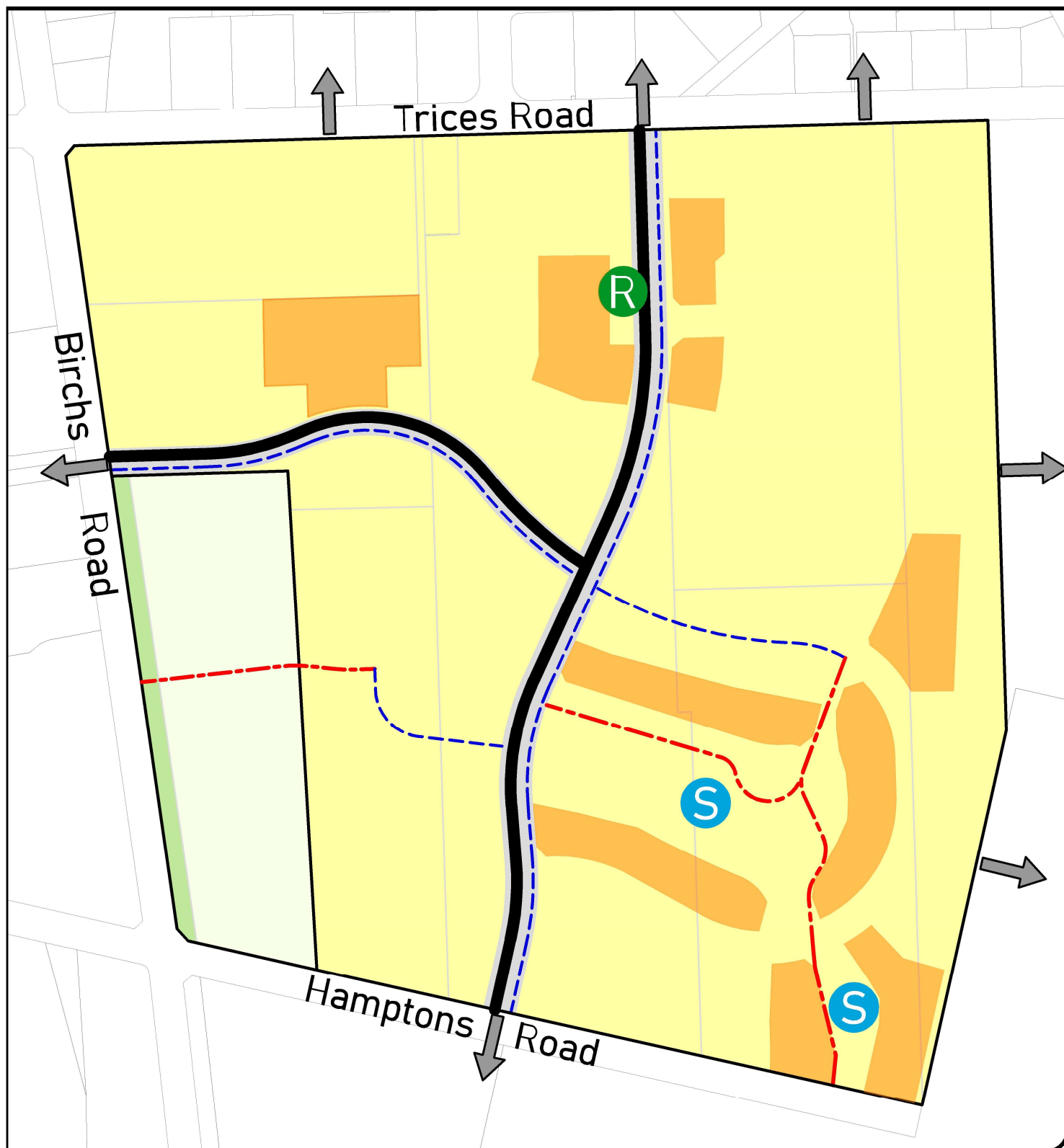
APPENDIX B – Stormwater Attenuation Calculations

APPENDIX C – Geotechnical (excerpt only)

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APPENDIX A – Plan Change Scheme and Drawings

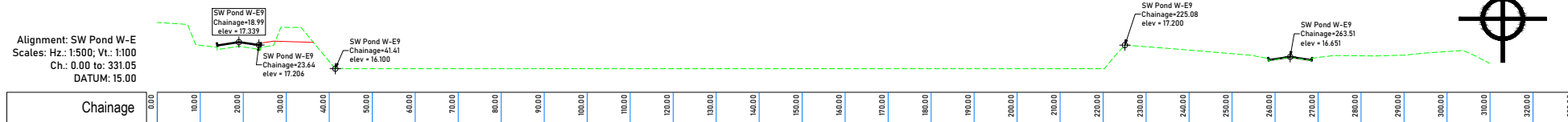
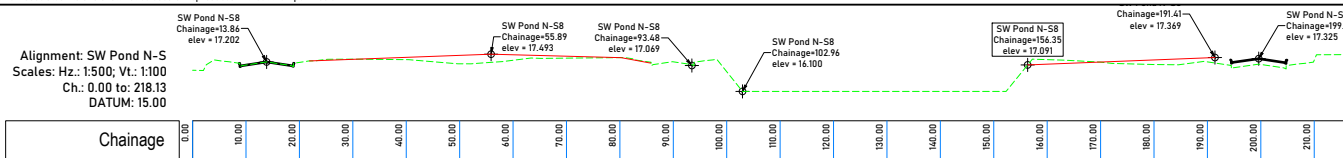
DEV-PR3 - Prebbleton 3 Development Area Operative District Plan



- Primary Road
- Potential future road connection
- Shared Pedestrian / Cycle Lane (on road)
- Shared Pedestrian / Cycle Lane (off road)
- Landscaping strip (existing mature planting)
- Stormwater management area (size to be determined at time of subdivision)
- Reserve location (size to be determined at time of subdivision)

- Living 3 zone
- Living Z zone (Low Density)
- Living Z zone (Medium Density)





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APPENDIX B – Stormwater Calculations

~~Post Dev. Runoff C = 0.65 for 50% AEP for developed urban areas (5.12.3, 2010) too conservative~~

Residential GR zoning C assumed to be 0.56 in 50yr storm, in line with CCC RSDT (assuming 12 houses/ha)

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

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)

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

Where $C_{ff} = 0.53$ based on L2 zoning, $A = 28.7\text{ha}$, $d_{ff} = 25\text{mm}$

$$V_{ff} = 3,803 \text{ m}^3$$

0.54 Weighted PC Zonal Runoff C

Sitenam: : Trices Rd

Longitude: 172.5164

DDF

Values: -0.0112 0.516893 -0.01115 -0.00184 0.33994 -0.01375 2.241307

Example: Duration (tARI (yrs) x y Rainfall Rate (mm/hr)

24	100	3.178054	4.600149	5.172378
----	-----	----------	----------	----------

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

POND ANALYSIS





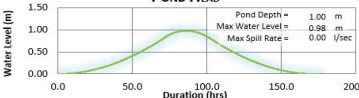
CATCHMENT PARAMETERS

Pond Treatment Area = 287,000 m²

Pre-development Runoff Coefficient = 0.30 (-)

Post-development Runoff Coefficient = 0.54 (-)

POND HEAD



Pond Depth = 1.00 m
 Max Water Level = 0.98 m
 Max Spill Rate = 0.00 l/sec

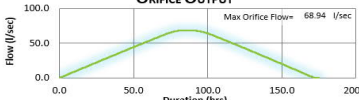
DESIGN CONDITIONS

Design Rainfall Zone = Alternative Rainfall

Design Storm Duration = 60 min

Design Storm Return Period = 50 yr

ORIFICE OUTPUT



Max Orifice Flow = 68.94 l/sec

POND DETAILS

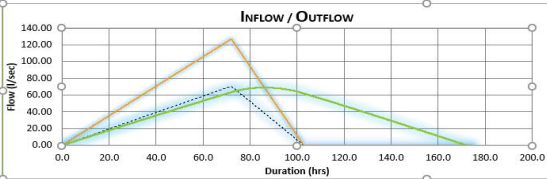
Pond Top Length = 105.0 m Pond Depth = 1.00 m

Length Side Slope z₁ = 2.5 1v:2h Pond Volume = 9,995.83 m³

Pond Top Width = 100.0 m Orifice Size = Marley Nom. Size 180 mm

Width Side Slope z₂ = 2.5 1v:2h Orifice Type = Short tube (C=0.80) (-)

INFLOW / OUTFLOW



RUNOFF RESULTS

Parameter	Value
Pre-Developed Peak Flow	70.61 l/sec
Pre-Developed Runoff Volume	13073.05 m ³
Post-Developed Peak Flow	127.10 l/sec
Post-Developed Runoff Volume	23531.49 m ³
Post-Developed Peak Flow (with tank)	68.94 l/sec
Post-Developed Runoff Volume (with pond)	23531.49 m ³

---- Pre Development Runoff
— Post Development Runoff (without pond)
— Post Development Runoff (with pond)

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

(copy and paste date from NIWA High Intensity Rainfall System V2 output)

[illegible]

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APPENDIX C – Geotechnical (excerpt only)

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

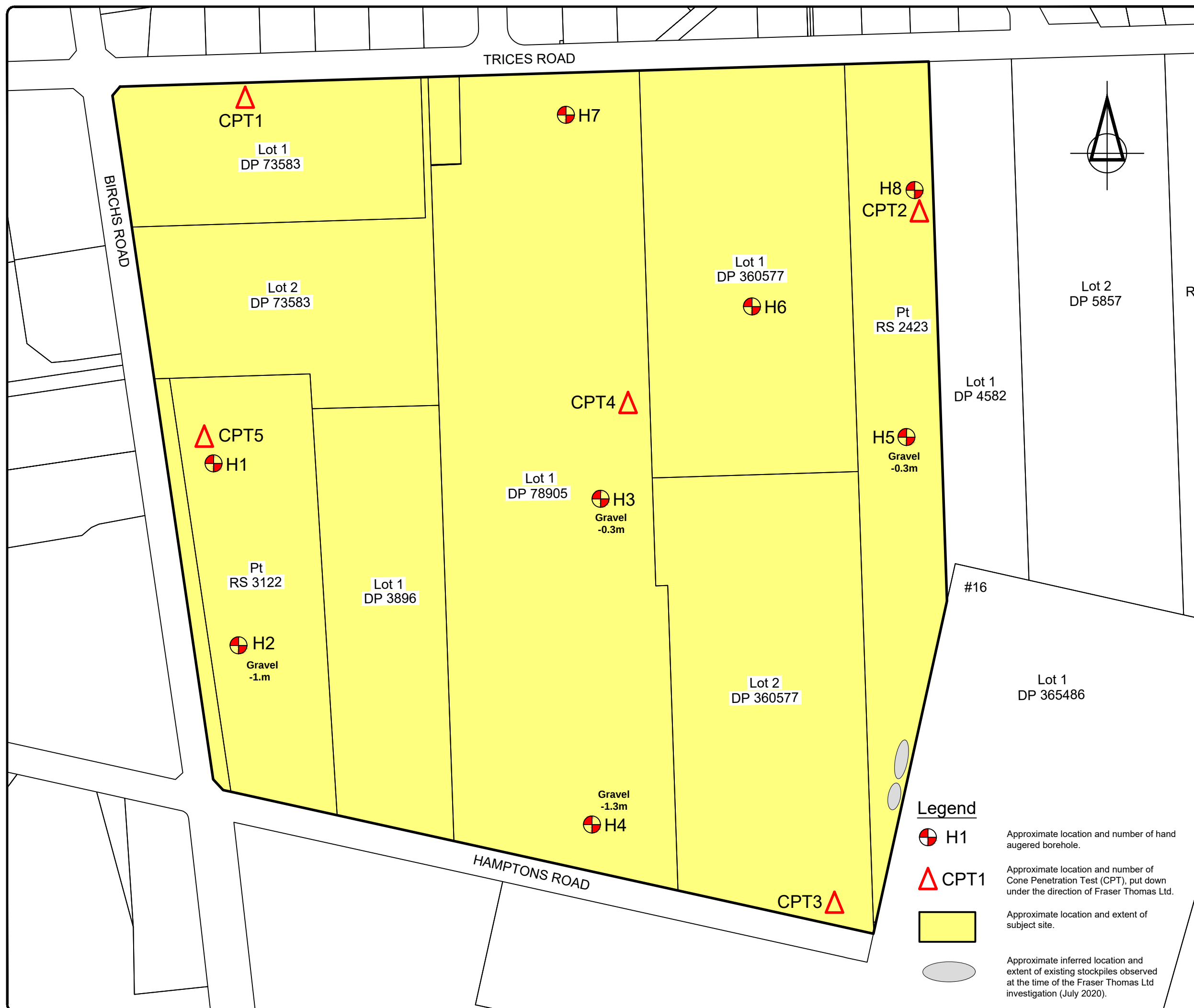
The logs of some existing machine excavated test pits have been sourced from the NZGD. The test pits are located at a site abutting the western site boundary. These logs indicate that gravels are encountered at shallow depth and are encountered to the extent of the test pits (i.e. 3.0 m depth). The logs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

[illegible]

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.

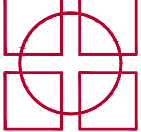
CLIENT

TRICES ROAD REZONING GROUP

PROJECT

TRICES ROAD AREA
PREBBLETON

TITLE
SITE PLAN



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