Cambridge C4 Growth Zone 3 Waters Assessment

September 2020

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EXECUTIVE SUMMARY

This report outlines the proposed stormwater solution to enable the development of the Cambridge C4 Residential Growth Cell. Water supply and wastewater servicing options are also presented. The receiving environment for the C4 growth cell is a large, steeply incised gully which runs adjacent to the site along the eastern edge of the proposed development area. An unnamed stream runs through the gully in a northerly direction towards the Waikato River. The ecology assessment states this unnamed stream is vulnerable to changes in hydrological conditions resulting from development of C4. The stream outlets to the north via an existing culvert under Cambridge Road and then flows around the Aotearoa Industrial Park before connecting to the Waikato River approximately 1500m downstream from the Cambridge Road culvert (C4 gully outlet).

The intention is to drain treated runoff from the growth cell to the unnamed tributary within the C4 gully, but the connection will not be directly to the stream. The preferred option is to outlet to the gully via appropriate outfall design and spread diffuse flow across the wide gully floor.

There are several options to manage stormwater to meet the design level of service and guiding principles outlined in the Regional Infrastructure Technical Specification, NZS 4404 2010, NZBC Clause E1, as well as the overarching management philosophies promoted in the Waikato Regional Stormwater Management Guideline (January 2018).

The development of C4 will result overtime in an increase in the impervious area due to the creation of buildings, hardstand, and roads. New impervious surfaces generate significant increases in peak flow, timing, and volume of runoff during a rain event. A typical residential subdivision is likely to result in an increase in total metals, total suspended sediments, nutrients, hydrocarbons and an increase in temperature as well as gross pollutants generated from those surfaces especially during high frequency rain events (first flush events) following prolonged dry spells.

The design philosophy will seek to implement water sensitive principles which can be integrated into the layout and landscape. The intention is to manage stormwater as close to the point of origin as possible, to minimise collection and conveyance infrastructure and to ensure no adverse impacts downstream. It is noted these impacts can be flow related (i.e. flooding or scour) and/or water quality related. The options presented in this report offer solutions which will achieve the following:

1. Protect and enhance the downstream receiving environment including fish passage in accordance with the Vision and Strategy of the Waikato-Tainui Environmental Plan.
2. Outline capacity and servicing requirements for water and wastewater.
3. Water efficiency measures and retention of stormwater on private lots and within public road reserves.
4. Recommend pre-treatment and soakage to manage water quality and primary flow up to the 10 year + cc event prior to discharge to the gully.
5. Manage normal and potentially high contaminate load profiles.
6. Help to maintain baseflows within the C4 gully stream using soakage.
7. Appropriate location and sizing of stormwater infrastructure to enable staging development.
8. Managing secondary flow paths up to the 100 year + cc event safely within the development to the gully floor outlet point.
9. Hydraulic modelling and risk assessment to assess need for flood attenuation.
10. Stability protection of the gully side from uncontrolled overland flow.
11. Avoidance of adverse impacts from flooding downstream.

Soakage testing concludes the growth area is favourable to use infiltration for stormwater management. The flood risk assessment concludes increases in runoff due to creation of new impervious surfaces has less than minor effect downstream within the gully (due to the significantly large storage area) and below the Cambridge Road culvert. The proposed solutions for stormwater management at C4 are:

1. Pre-treatment + soakage on residential lots.
2. Road drainage via reticulated network to soakage trenches within the road reserve or alternatively to communal soakage basins with forebay for pre-treatment.
3. Planted swales for park/reserve edge roads where feasible.
4. Both primary and secondary flows conveyed to the gully with appropriate outlet to encourage dispersal and fan out across the gully floor to stream.
5. Construction of each gully outlet stormwater outlet structure is likely to require a concrete manhole stilling well, combined riprap and gabion protection and potentially a directionally drilled HDPE pipe. The outlet structure will provide velocity reduction of stormwater discharges to the gully environment.
6. RITS water quality volume and initial abstraction volumes will be managed via pre-treatment and soakage systems within the development.
7. Flood attenuation basins to limit post development peak flows to predevelopment peak flows are not required due to the storage and buffering effect of the large gully directly adjacent to the C4 growth area.

1 INTRODUCTION

Te Miro Water Consultants (TMW) have been engaged by Waipa District Council to provide a Three Waters Assessment to support the C4 Structure Plan. The C4 growth cell is located to the south of Cambridge as shown in Figure 1.

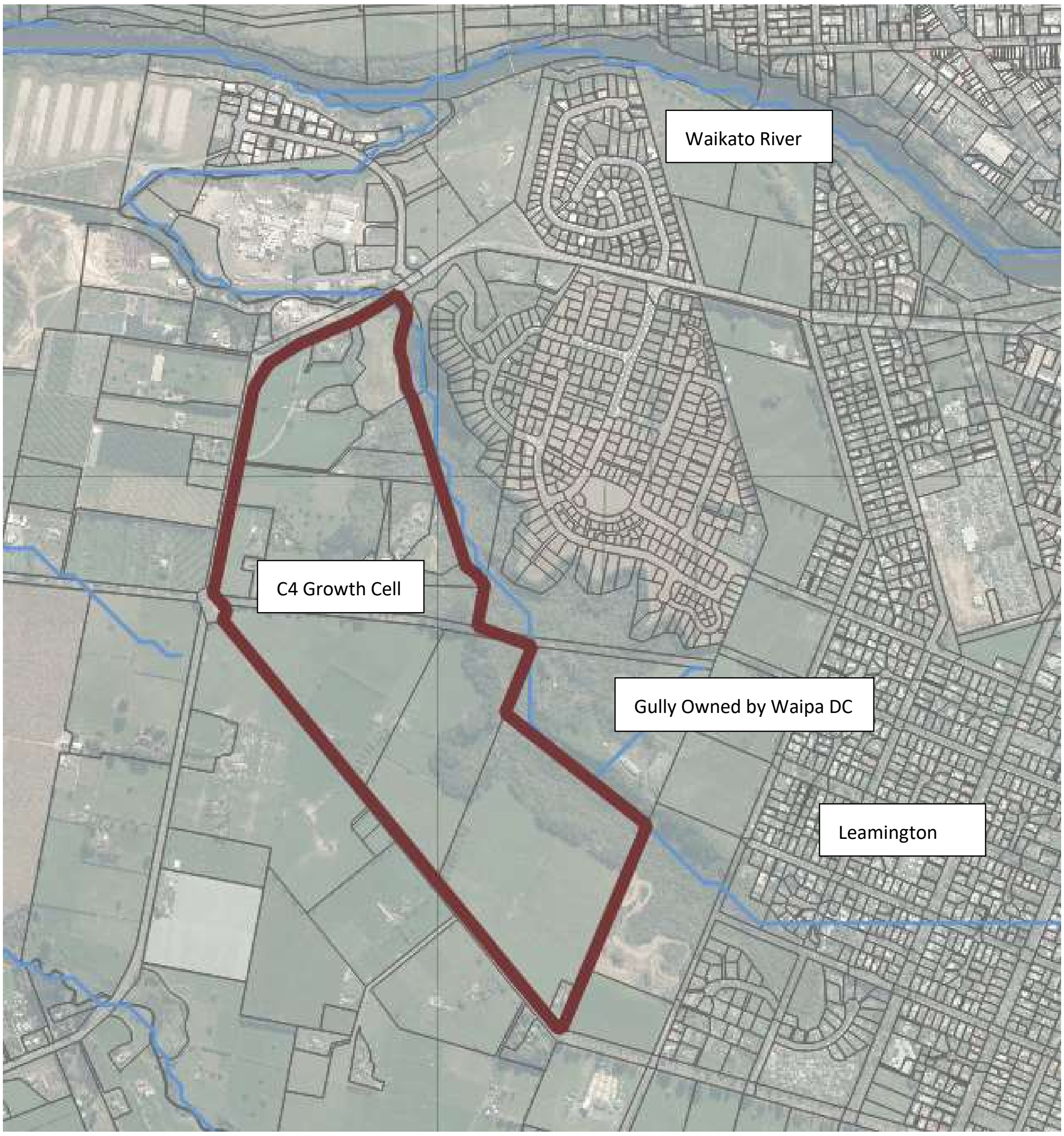


FIGURE 1 LOCATION OF C4 GROWTH CELL

1.1 BACKGROUND AND PURPOSE

The Structure Plan objectives are to determine the urban form, use and way infrastructure can be efficiently, and cost effectively developed to facilitate residential development (~800 dwellings). The C4 growth cell is one of 11 growth cells currently identified for Cambridge as shown in Figure 2.

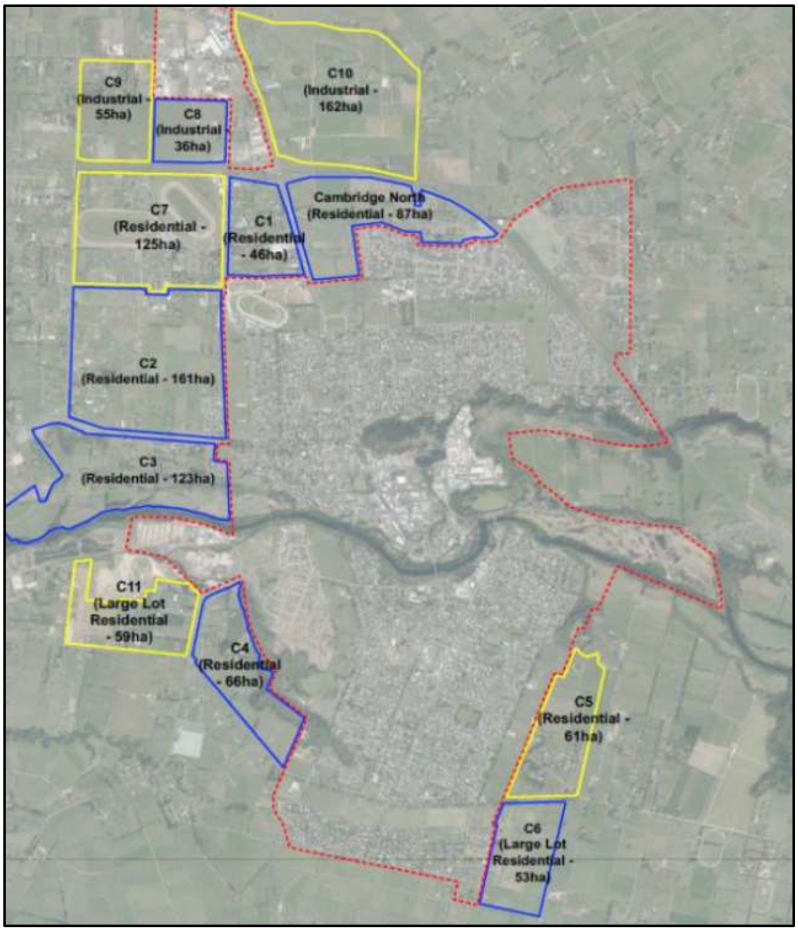


FIGURE 2 - CAMBRIDGE GROWTH CELLS

TMW have worked closely with the wider project team (WDC staff, Planning, Transport, Geo-technical etc) to determine key requirements and constraints to inform the three waters assessment.

A project start-up meeting was held with Robin Walker at WDC on 21 August 2019. The key issues identified during the meeting were:

- Overall objective for WDC is to seek ways to provide fish passage from the Waikato River up into the upper section of the C4 gully
- Consent monitoring conditions attached to the Arnold Street Stormwater outlet
- Monitoring outlet from the historic landfill
- Water supply and wastewater currently being master planned for Cambridge. The results of the master plan will influence the final solution for C4

The Three Waters Assessment will cover:

- Existing catchment conditions.
- Stormwater management options including flood modelling.
- Water supply options.
- Wastewater servicing options.
- Summary and conclusions.

1.2 DEVELOPMENT STAGING

Ideally sequencing and timing of development within C4 takes place in a coherent and efficient manner that is coordinated with the economic development of trunk 3 waters services. Council funding of infrastructure development will be generally in accordance with the programme in Waipa's Strategic Plan. A distributed stormwater solution is encouraged with more than 1 outlet to the gully to allow various pockets of land to be unlocked discreetly from one another. In this sense

There are 4 larger landowners within the growth cell as well as a cluster of rural residential property owners.

- The gully is owned by Waipa District Council which could help when requiring permission to construct any stormwater devices/outlets.
- Currently there is no detailed urban layout and the order of development is unknown. Less reliance on 'end of line' large scale communal devices will help promote development staging in a flexible manner reducing the need for multi-party ownership to form agreements to build infrastructure.

2

ASSESSMENT OF EXISTING CATCHMENT FEATURES, CONSTRAINTS, RISKS AND OPPORTUNITIES

The following section assesses the features, constraints, risks, and opportunities for the C4 growth cell. a summary is provided in the table below.

Features

1. C4 has two distinct landscape typologies:
   - a. A flat remnant river terrace at 2 broad levels where growth cell development is proposed to occur and;
   - b. A 20m deeply incised gully adjacent to the entire length of the C4 terrace;
2. An unnamed tributary within the gully floor draining via culvert under Cambridge Road to a channel around existing industrial area before discharging to the Waikato River;
3. The gully has been identified as:
   - a. ecologically significant with sensitivity to some scour and erosion
   - b. heavily vegetated with exotic and native plantings;
4. Two existing urban stormwater outfalls are present:
   - a. Draining the recent Cambridge Park sub-division and
   - b. Draining approximately half of the existing Leamington urban area.

Constraints

- Pipe outlet and velocity control at the base of the gully
- Water supply and wastewater trunk infrastructure
- Multiple land ownership

Risks

- Geo technical stability along gully edge and setback zone
- Reliance and positoning of public soakage systems and their on going operation and maintenance
- Timing of development aligning with construction of 3 waters trunk infrastructure and WWTP upgrades

Opportunities

- Public access through gully and connectivity with existing resdential areas
- Stream enhancement within the gully and downstream within the industrial estate
- Fish passage under Cambridge Road
- Amenity stormwater basins/wetland within public reserves
- Reserve edge roadside swales

A site walkover was undertaken on July 18th, 2019 to assist in understanding the catchment and determining objectives for the three waters design at the site. Site photos of key catchment features and different perspectives are provided in Appendix 1.

2.1 EXISTING LANDUSE CATCHMENTS AND TOPOGRAPHY

Distinct catchment and topography items include:

- The C4 structure plan area sits within a predominantly flat, well drained rural area. The existing land use is rural grazing and there is a small pocket of rural residential living. An existing aerial map and contour plan is provided in Appendix 2 and Appendix 3.
- The catchment is defined by Lamb Road and Cambridge Road to the west and north respectively.
- The total catchment area is effectively the C4 growth cell (66ha).
- The steeply incised gully (~20m deep) represents what was once a much larger tributary channel of the Waikato River. This gully now acts as a local drainage system. The gully floor is filled with dense shrubland at approximately 42mRL.
- The upper terrace which covers the developable area has a ground level of approximately 64mRL.

2.2 EXISTING OVERLAND FLOWPATHS

A review of the contour plan, aerial photos, and site visit observations, as well as consideration of nearby developments provides the following overland flow path assumptions:

- There are no obvious surface drainage networks connecting the site to the gully or farm drains within the site or culvert connectivity under Lamb Road to the west.
- The rural residential subdivision on Silverwood Lane have on lot soakage devices.
- Currently stormwater runoff would either pond on the farmland within shallow depressions and soak away during storms up to the ~10 year ARI design storm event. Storms greater that the 10 year ARI may run off overland into the gully and stream.
- Existing secondary flow paths generally follow the gradual fall of the land, being from the south-west to north east towards the gully. The site visit did not reveal any obvious ephemeral channel dissecting the grazing land to the gully edge – supporting the assumptions that most of the catchment ponds and/or disperses via soakage across the flat terrace.

A high level map of overland flow paths and contributing catchments is provided in Figure 3.

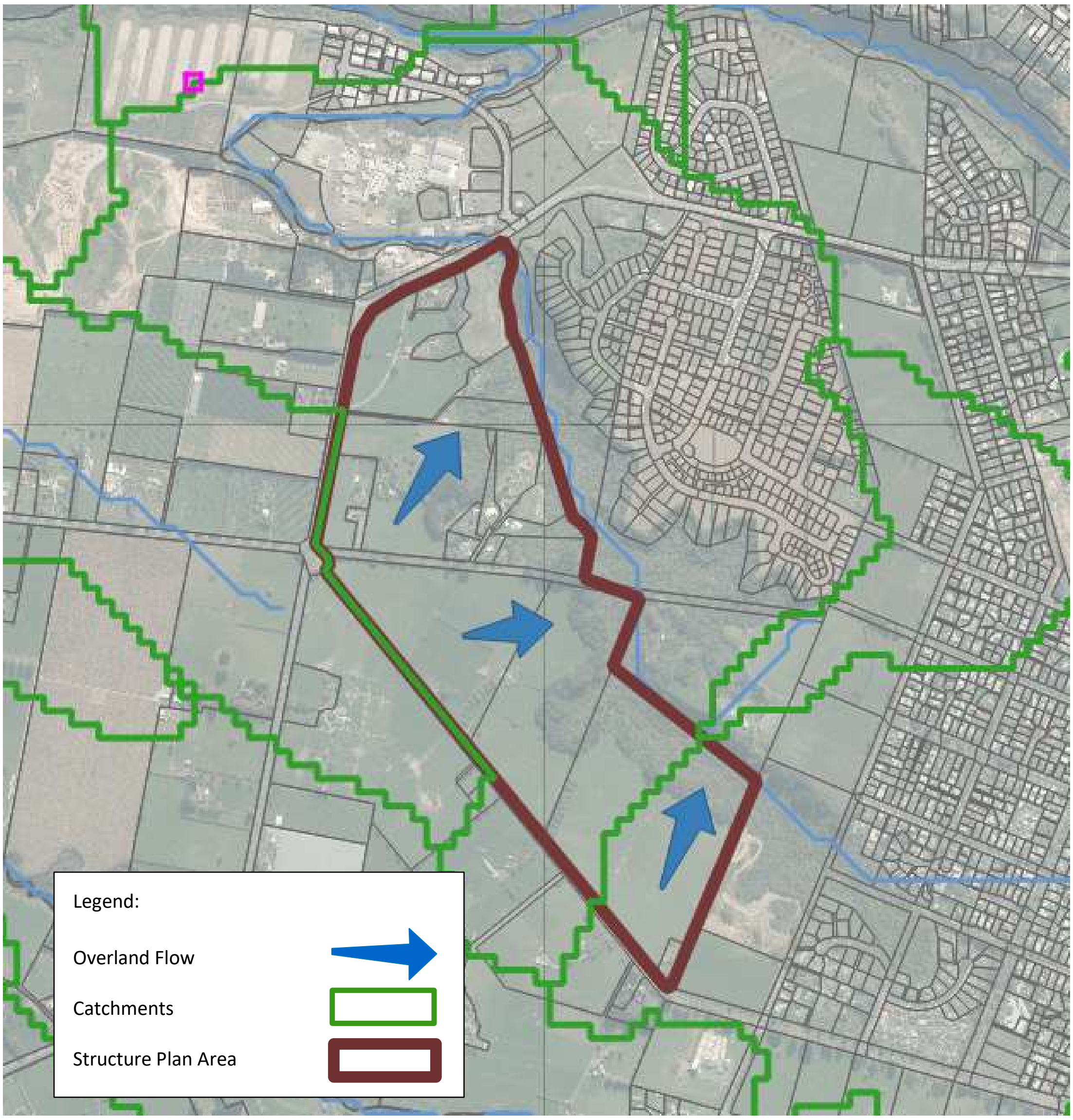


FIGURE 3 GENERAL OVERLAND FLOW PATHS AND CONTRIBUTING CATCHMENTS

2.3 EXISTING FLOOD IMPACT ASSESSMENT

A 1D/2D hydrological and hydraulic model (Infoworks ICM) of the 100 year ARI existing and post development scenario was developed to understand the present flood hazard within the gully and at the culvert outlet under Cambridge Road and immediately downstream through the industrial development. The purpose of this assessment was to determine the existing base case and in turn inform the stormwater management as part of the structure plan. The model build report and flood maps are presented in Appendix 6 and summarised as follows:

- Hydrology inputs such as rainfall depth, catchment land use type, impervious coverage etc are in accordance with the Regional Infrastructure Technical Specification (HCC, 2018) (RITS) and TR2020/07 and 06 (WRC,2020).
- The model estimates the pre-development hydrology conditions for the development area (C4) and wider catchment (existing land use which is a mix of urban and rural) which outlets to the gully including specific downstream constraints (culvert, road, 1D confined channels etc).

- The culvert under Cambridge road is included in the model with culvert details (diameter, length, invert levels etc) and 1D channel sections upstream and downstream obtained by site survey.
- Existing catchment runoff volumes are loaded directly to the basin model to derive peak waters levels within the gully and peak flows and levels at the culvert outlet.

The results of the flood modelling are summarised as:

- The expansive and deeply incised gully system will be the receiving environment for the development.
- The flat gully acts as a large attenuation basin with a fixed hydraulic control being the existing culvert and causeway on Cambridge Road.
- The downstream landuse is industrial/commercial, which is lower risk than residential landuse, notwithstanding the lower risk, the objective is to not create adverse impacts from the C4 development by increasing flows and water levels downstream.
- Other than C4 there is no future planned growth within the C4 stream catchment thereby reducing the issue of ‘cumulative impacts’ from a series of future unattenuated storm flows.

2.4 RECEIVING ENVIRONMENT

A site visit report by NIWA provides an assessment of the receiving environment within the gully. A summary of key items that feed into the three waters design objectives is provided below:

- Habitats in the upper reaches of the C4 Stream are likely capable of supporting black mudfish, banded kōkopu and giant kōkopu.
- The fallen trees and overhanging vegetation create cover and pool habitat that is preferred by banded kokopu and giant kōkopu (Baker & Smith 2007).
- The ephemeral wetland habitats and seeps within the broad gully floor are the preferred habitats of black mudfish. In addition, habitats suitable for both eel species, īnanga, smelt and common bullies were also present.
- There are concerns about potential fish passage impediments in the C4 Stream that may be preventing these species from colonising the upper reaches and there are no records of these species in the C4 Stream from the NZFFD (although survey cover is minimal).
- Longitudinal changes in the habitat quality of the C4 Stream were evident with the lower sites being the most degraded. Below Cambridge Road, the C4 Stream had poor riparian and canopy cover, evidence of stock damage and large sections of homogenous habitat.
- This lower habitat quality suggests that any impacts from the C4 Growth Cell Development and associated stormwater inputs will be greatest upstream of the lake where instream habitat diversity, stable banks and mature riparian buffer existed.

Overall, the ecological integrity of the C4 Stream cannot be fully understood without an updated survey sampling the range of habitats present, including the lake, to determine the fish communities utilising the different habitat types.

2.5 GROUNDWATER

Two piezometers were installed (3 September 2019) to 20m depth by Perry Geotech Ltd, one each in the northern and southern section of the development area. Three levels have been taken following installation, another on 16 September and 26 September 2019.

- Post installation (settled groundwater) depths range from 11m to ~15m depth for Piezometer 1 and 2 respectively.
- The development is located directly west of the deeply incised gully. The groundwater levels across the site are reflected by the depth of the gully with the soils draining towards the gully floor at a 1 in 10 gradient. Shallow groundwater encountered in the CPT holes are indicative of perched groundwater in wetter winter months.
- Localised perched water table encountered at approximately 4m depth.

2.6 EXISTING SOIL CAPACITY FOR SOAKAGE

A further site investigation including stormwater disposal testing was undertaken by Mark T Mitchell on October 14th and 15th 2019. The purpose of the study was to determine and evaluate the sub surface conditions within the site and assess the feasibility for on-site stormwater disposal within the C4 Growth Cell. The findings are presented in a report by Geocon Geotechnical Ltd (Mark Mitchell associate company) dated 31 October 2019.

Falling head permeability testing was carried out within the upper terrace zone at 4 locations as shown in Appendix 7 (Drawing No. 16064-20). The subsurface conditions within the test bore holes revealed:

- There is 200mm of topsoil overlying silt (loam) to between 0.4m to 0.8m depth.
- The silt underlain by gravelly fine to coarse grained sand to at least the base of the 1.5m to 3.0m deep bore holes.
- Groundwater was not encountered within the bore holes during the spring site investigation.

The results represent the theoretical soil hydraulic conductivity or ability of that soil medium to transmit water flows under a simulated water level head. The results are summarised as:

- Five of the six tests revealed consistent hydraulic conductivity (k) with values between 1.1m-2.8m per day or on average between 46mm/hr and 117mm/hr.
- The other test (A2 at 3.0m deep test) provided inconsistent results. This is likely to be on account of:
    - Heavy rainfall in the days prior to testing.
    - Perched water above silt lenses which are exposed in the gully branch located south of the test site.
    - The possibility of some deeper sands being very dense which limited pore space availability.

The results may not be fully representative of the full capacity of the silts and further testing is to be carried out such as with a ring permeater in the base of the proposed stormwater devices.

2.7 GEOLOGY

The C4 Growth Cell Geotechnical Report (Mark T Mitchell Ltd, September 2019) notes the area to be characterised by an upper alluvial terrace with covers most of the development area and a lower terrace in the northern portion of the site. The key issues related to stormwater management are summarised below:

- Bore hole information carried out across the site indicate the presence of free draining sand soils encountered to at least 0.4m to 1.0m depth. Therefore, all collected stormwater can be captured and detained within each proposed residential lot. Road areas could be discharged to a siltation pond which releases the water further to the base of the gully.
- Upper terrace: low groundwater, silt loam to 12m depth underlain by fine to coarse sands.
- Lower terrace: Uncontrolled filling overlying loose to dense fine sands. Absence of filling in holes in the north of this area. Groundwater encountered 1.9m to 6.0m below existing ground level.
- High to severe liquefaction damage on the lower terrace which could impact on any communal basin or swales.
- Building line restriction (BLR) of 8m in the north of the site and 14m in the southern portion from top of slope of the gully edge. The BLR has implications for the location of any excavated basins/swales for communal soakage devices.
- The 8m (Northern area) and 14m (Southern area) are applied between top of slope of the steeper banks (slope angles range between 20 and 55 degrees) and proposed house foundations, pools, and wastewater/stormwater fields. In addition, no retaining walls such as to form stormwater basins sides are to be constructed within the gully or gully edge.
- Upper terrace natural soils consist primarily of Loam, overlying alluvial deposits fine to coarse sands. Taupo pumice encountered in the northern extent of the subject area.

3 ASSESSMENT OF STRUCTURE PLAN IMPACTS

The following section outlines the expected impacts on three waters resulting from a change in land use from rural to urban as shown in Figure 4 below.

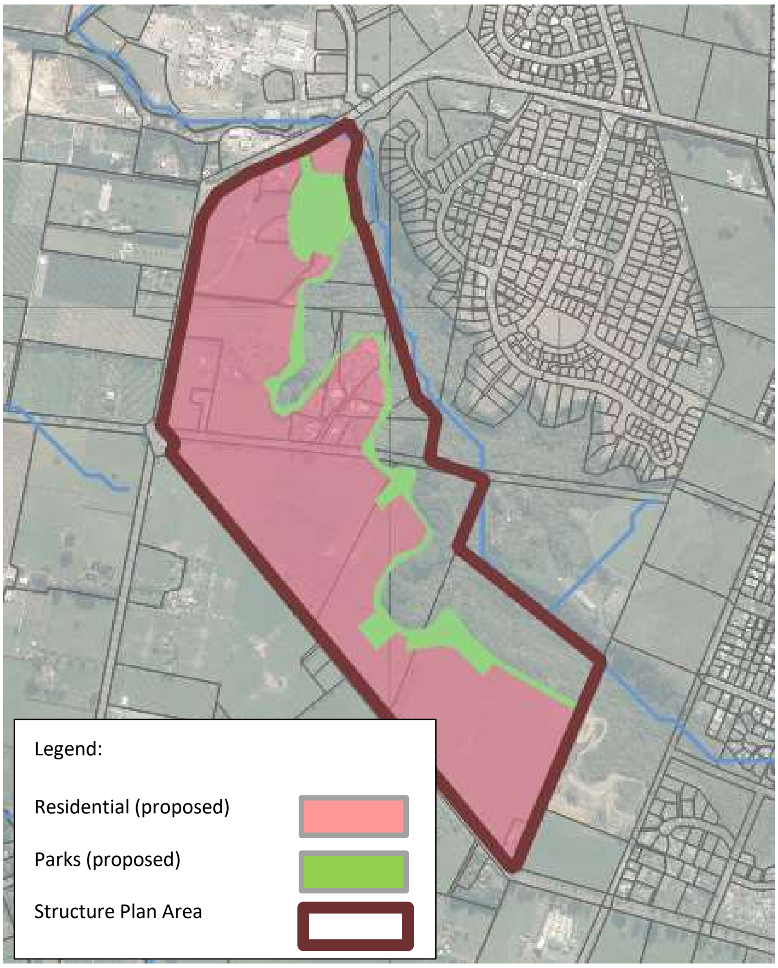


FIGURE 4: PROPOSED LAND USE

3.1 POTENTIAL STRUCTURE PLAN FLOOD IMPACTS

The existing scenario flood model was updated to incorporate the developed structure plan area (post development scenario). The post development scenario assesses the impacts of peak water level and flow within the gully and the culvert outlet under Cambridge Road and immediately downstream within the channel from residential development. The model build is presented in Appendix 6 (and Section 2.4) with summary as follows:

- The model estimates the post-development hydrology conditions for the development area in addition to the wider catchment (remaining as existing land use) which outlets to the gully including specific downstream constraints such as road culvert.
- The model included unattenuated post development hydrology conditions based on the C4 residential zoning landuse discharging to the gully and existing wider catchment (Cambridge Park and Leamington sub catchments). The model does not consider on site soakage.

- Other than C4 there is no future planned growth within the C4 stream catchment thereby reducing the issue of 'cumulative impacts' from a series of future unattenuated storm flows.

The results of the comparison between the pre and post development scenarios are presented below. The results demonstrate that the unattenuated post development flood level within the gully increases a maximum of 100mm. Further details provided in Appendix 6.

TABLE 1 ASSESSMENT OF PRE AND POST FLOOD LEVELS AND PEAK FLOWS

LOCATION	PRE – DEVELOPMENT FLOWS		POST – DEVELOPMENT FLOWS	
	Flow (m^3/s)	Level (mRL)	Flow (m3/s)	Level (mRL)
XS 1	20.20	47.30	20.20	47.32
XS 2	20.56	42.28	20.78	42.38**
XS 3	7.05*	42.27	7.19*	42.36
XS 4	7.13	40.16	7.26	40.18
XS 5	7.13	39.98	7.26	39.98

* Flows reduce at XS 4 and XS 5 due to the backwater and throttle effects of the Cambridge Road culvert.

** Maximum difference of 100mm may be partly due to the direct loading of lumped catchment runoff in the vicinity of XS 2.

In summary, the results of the comparison between the unmitigated pre and post development hydrologic and hydraulic modelling show the impacts of unattenuated flows to the gully do not have significant impacts on level or flow. The Cambridge Road culvert has a throttling effect with floodwater backing up to utilise the existing significantly large flood storage capacity within the gully. The largest increase in the order of 100mm is shown within the mid-section of the gully. However, this increase is almost unnoticeable at the gully edge. This conclusion is like the earlier Cambridge Park sub division which undertook hydraulic modelling and concluded a less than minor impact from unattenuated flows to the gully.

3.2 VOLUME AND WQ CHANGES

Volume impacts and water quality changes are expected due to the new development. However, all storm events up to the 10 year will be managed within the development using pre-treatment and soakage devices (private and public working together). Potential erosive flows within the gully will thereby be eliminated with only flood flows entering the gully via stilling outlets and rip rap basin with elongated gabion wall acting as a weir (between 10m and 20m wide) to disperse flow out across the gully floor. Section 5.5 provides an example of a stilling manhole outlet.

3.3 GROUNDWATER RECHARGE

Groundwater recharge is expected to continue via soakage devices. At detailed design, once the final location of each device is known, site specific soakage testing will be undertaken and potentially mounding assessment to ensure no adverse impacts from soakage to ground.

3.4 RECEIVING ENVIRONMENT IMPACTS

The NIWA ecological assessment highlighted the following risks to the receiving environment because of the structure plan change. It is noted that the assessment was based on the premise that existing waterways within the gully system are to be used for stormwater discharge (water and wastewater is contained at treated elsewhere):

- The hydrological regime of stream sand wetlands is altered due to development effecting freshwater habitats and species.
- Typically, urban development reduces baseflows to streams and increase both the peak flow and volume entering the watercourse as well as the timing of those flows.
- Erosion and contaminants associated with urban development can impact fish ecology. It is important that the stormwater management plan minimises additional contaminant inputs to the C4 stream.
- The culvert under Cambridge Road being an impediment to fish passage.
- No known hydrological data exists for the C4 stream, however maintaining existing flow regime following development is a preferred option to ensure no adverse impacts on stream habitat.

3.5 ASSESSMENT OF RECEIVING ENVIRONMENT USING WRC MATRIX

An assessment of impacts to the receiving environment has been undertaken based on the matrix approach (based on WRC guidelines TR2020/06 and 07 (WRC, 2020).

To undertake the assessment, the C4 site was delineated into proposed developed catchments. The purpose of this was to allocate points in relation to outlet location and the associated source control target and the low impact design (LID's) target.

The proposed structure plan residential and green space area; LID/source control assessment catchments and proposed discharge locations are presented in Figure 5.

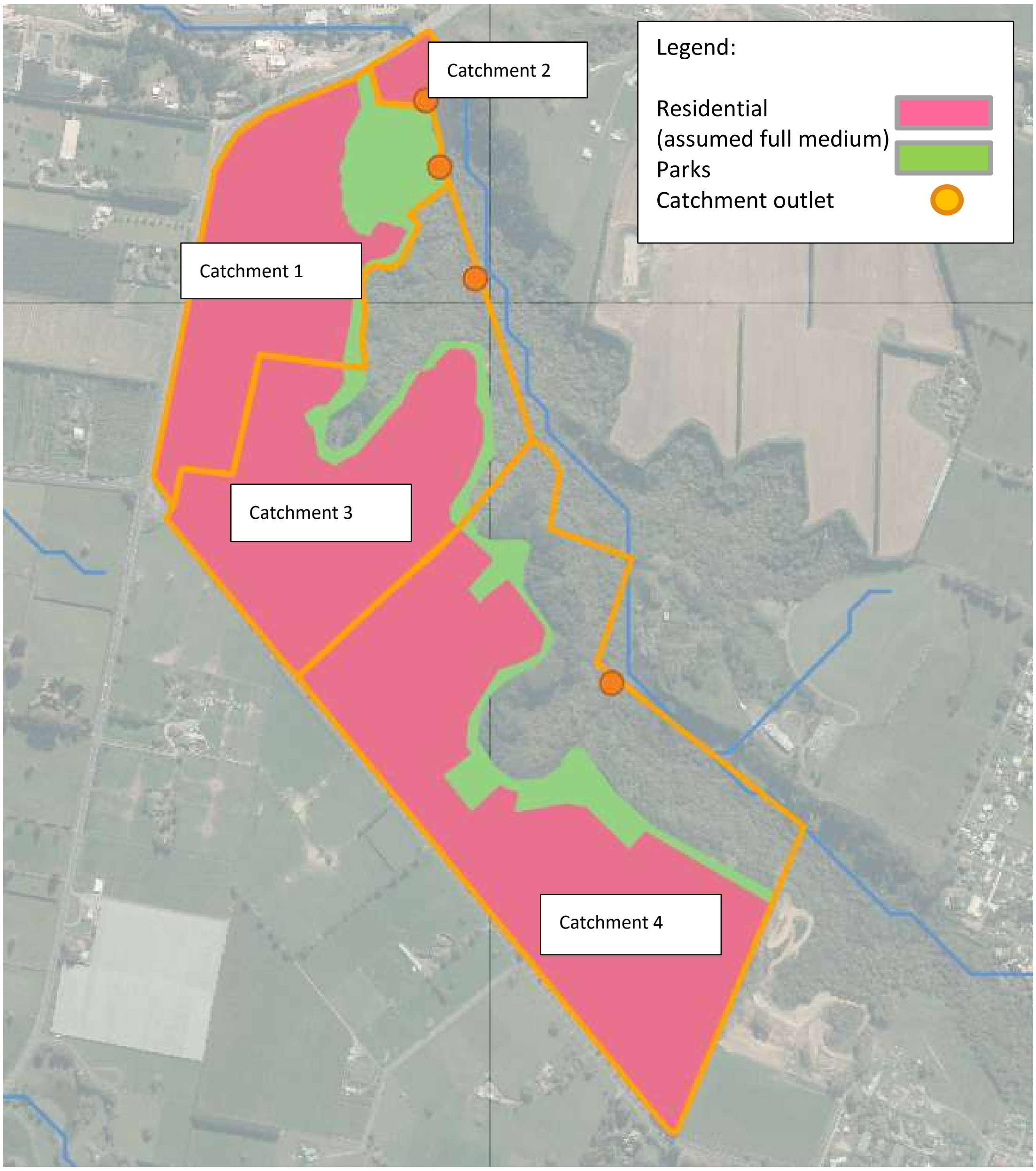


FIGURE 5 PROPOSED C4 DEVELOPMENT (RESIDENTIAL AND GREEN SPACE) AND LIDS/SOURCE CONTROL CATCHMENTS

Each catchment discharging from C4 has been assessed compared to the receiving environment. Table 2 presents the points associated with each catchment. These points and key receiving environment features are also shown in Figure 5.

TABLE 2 MINIMUM SOURCE CONTROL AND LIDS POINTS APPLIED TO EACH CATCHMENT

Catchment	Design criteria for the site	No existing natural features to protect			Justification
		Source control target	LID devices target	Total target	
Catchment 1	• Water quality treatment required • Volume control required	4	3	9	• Waterway is not present within the catchment • Ephemeral Waterway is located downstream of catchment • Erosion risk considered high • Flood risk considered low • Downstream environment considered to have significance
Catchment 2	• Water quality treatment required	4	2	8	• Waterway is not present within the catchment • Discharging into an area with a constant water level (erosion risk considered low) • Flood risk considered low • Downstream environment considered to have significance
Catchment	Design criteria for the site	Existing natural features to protect			Justification
Catchment 3	• Water quality treatment required • Volume control required	6	3	12	• Waterway is not present within the catchment • Ephemeral Waterway is located downstream of catchment • Erosion risk considered high • Flood risk considered low • Downstream environment considered to have significance
Catchment 4	• Water quality treatment required	6	3	12	• Waterway is not present within the catchment

	• Volume control required				• Ephemeral Waterway is located downstream of catchment • Erosion risk considered high • Flood risk considered low • Downstream environment considered to have significance

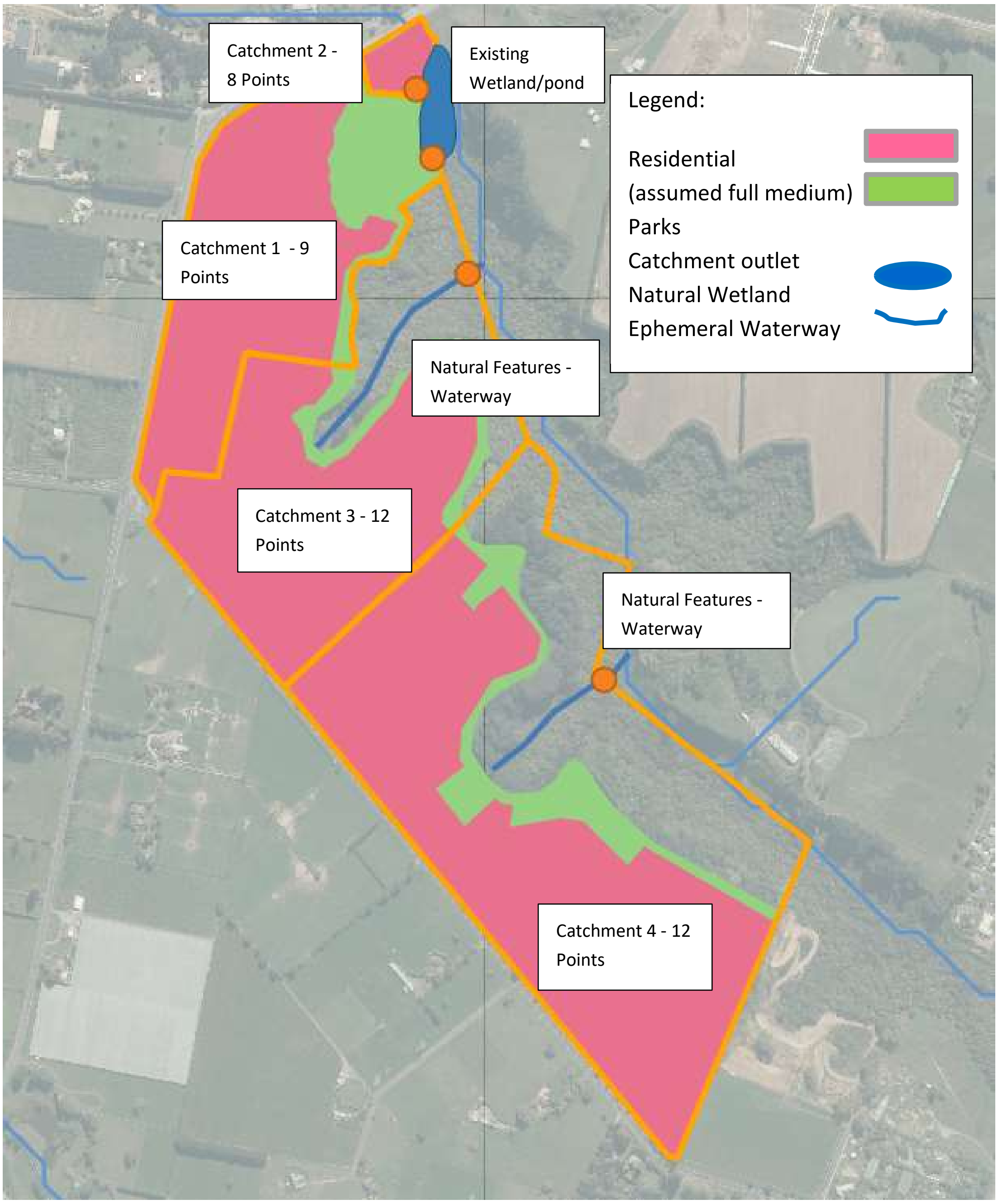


FIGURE 5: LID POINTS AND RECEIVING ENVIRONMENT

4 MITIGATION OF STRUCTURE PLAN IMPACTS

Following the assessment of effects of the proposed C4 structure plan on the receiving environment, the following Table 3 outlines the provisions that that shall be applied to the C4 Growth Area. The provisions are in accordance with the RITS (HCC, 2018) and the TR2020/06 and 07 (WRC, 2020) compliance documents.

TABLE 3 STORMWATER MANAGEMENT PROVISIONS FOR THE C4 GROWTH AREA

Storm event (ARI)	Provision	Guidance
All events	First flush – pre-treatment prior to soakage	Regional SW Guidance, RITS on-site water efficiency measures,
1/3 2yr	Water quality treatment provided by soakage	TP 10, RITS and Regional SW guidance
2yr	Soakage disposal on private lots to manage runoff from roof and driveway areas (catchments 1, 3, 4). Limited soakage within Catchment 2	RITS
10yr	Primary drainage conveyance within the residential development with pipe network and swale network for park edge roads	RITS
10yr	Soakage disposal within public devices (Final Site Testing to Confirm) for road runoff and spill from private lot soakage (see typical sizing tables)	RITS, NZBC E1, Regional SW Guidance
100yr	Safely manage secondary flows through the site via road/green network. No people or property at risk	RITS, NZBC E1, NZS 4404 and Regional rainfall runoff guidance
100yr	Controlled outlet to the gully floor and with appropriate erosion controls no peak flow attenuation requirements (as per flood risk assessment)	RITS and Regional SW Guidance

4.1 MITIGATION OF RECEIVING ENVIRONMENT EFFECTS

The following section outlines how the development within each structure plan catchment 1-4 can mitigate the effects on the receiving environment. The proposed options are indicative only and are subject to concept and detailed design as the staging of development is currently unknown. The options do however provide evidence that achieving the required outcomes is practical and feasible. Key mitigation concepts are presented in Figure 6.

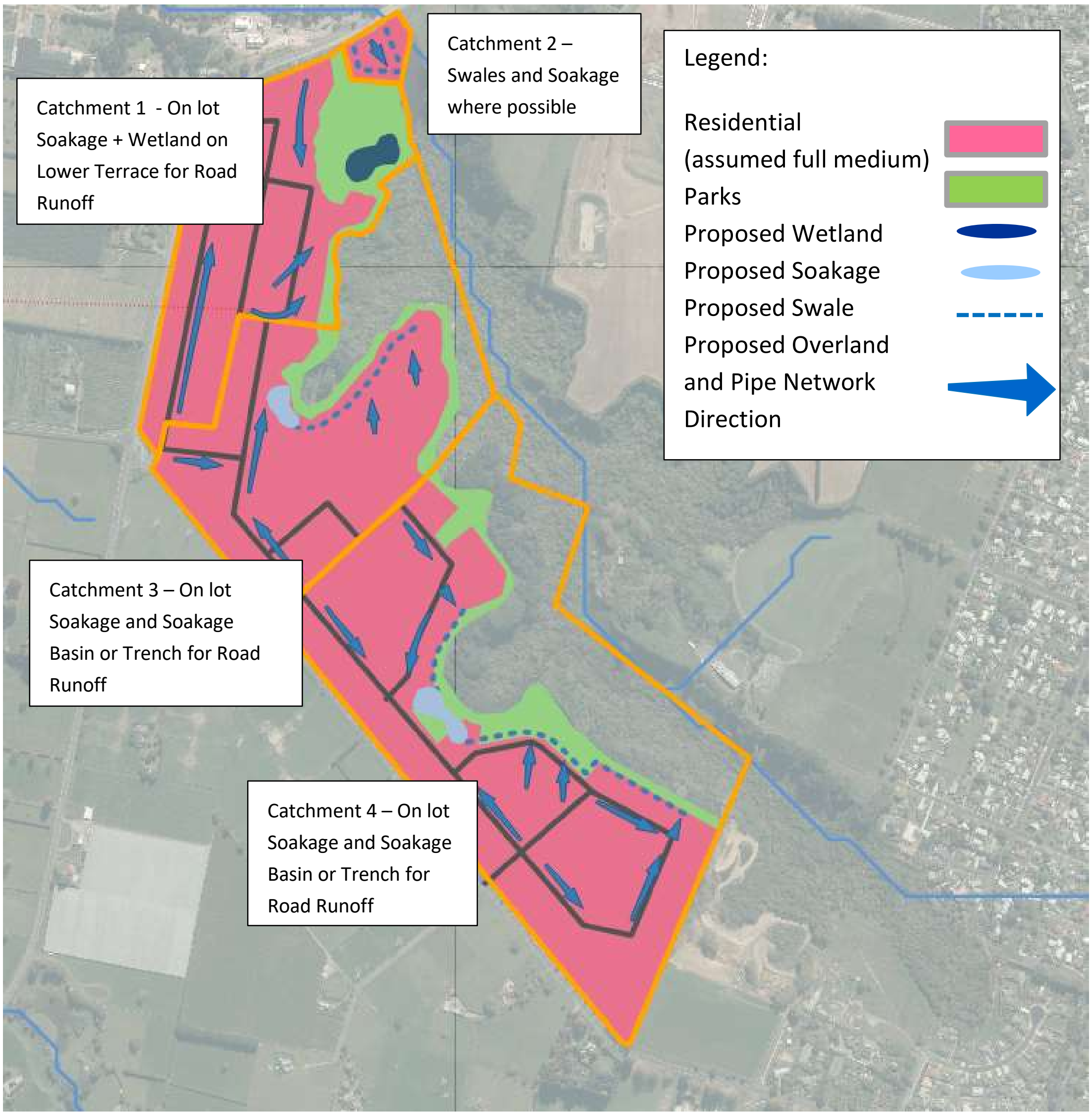


FIGURE 6: KEY STORMWATER MITIGATION CONCEPTS

4.2 CATCHMENT 1

The key source control toolbox options for Catchment 1 are presented in Table 4 with assessment undertaken on the proposed solution with area and percentage calculation provided in Appendix 8. The key outcomes for this catchment include:

- Private on lot soakage (up to 2 year) is considered favourable due to conditions on the upper terrace.
- A wetland is proposed at the base of the catchment (lower terrace) due to likely unfavourable soakage conditions and to tie in with the urban design principals with regards to the use of the open space and neighbouring stream/pond features.
- Urban design can allow for green areas due to size of developable area.
- Lot areas and site disturbance can be reduced due to size of the developable area.
- As this catchment is expected to discharge to the gully with some ecological significance, volume control up to the 10 year is considered valid.

The proposed approach for this catchment is:

- SOURCE CONTROL:
    - Utilise inert building materials
    - Reducing the total impervious surface of the site by avoiding development in or near the gully.
    - Reducing the site disturbance through utilising conventional lot sizes and confining the development to the terrace.
- LIDS CONTROL:
    - Soakage for private lot runoff (roof only) up to the 2-year ARI.
    - Adoption of wetland to treat and attenuate runoff from driveways and public roads up to the 10 year event.
    - High flows will bypass the wetland.

TABLE 4 CATCHMENT 1 SOURCE CONTROL OPTIONS

Decision leaders	Source Control – Minimum of 4 points	Proposed solution	Minimal Solution	Minimal Solution
		Toolbox Option 1	Toolbox -Option 2	Toolbox -Option 3
Developer Lead	Water re-use - *Flow detention only is 1 point in houses by use of rain tanks*	0	Rain Tanks are used for flow detention - 1	0
Developer/Council Lead	Site disturbance reduced from a conventional development approach *· 10 % reduction from a conventional development is 2 points.*	2	2	2
Developer/Council Lead	Impervious surfaces reduced from a traditional approach. Impervious surfaces reduced from a conventional development approach *- 5% reduction is 2 points.* *-10% reduction is 3 points.*	Current expected lot coverage - 3	Larger reduction in lot sizes to account for the open space - 0	Smaller reduction in lot sizes to account for the open space - 5% -2
Developer Lead	Use of building or site materials that do not contaminate *Residential roofs, gutters, down spouts made of non-contaminant* *leaching materials is 1 point.*	1	1	1
Council Lead	Protection and future preservation of existing native bush areas *Protection, preservation and, if needed, enhancement of native bush* *areas that exceed 10% of the site is given 2 points.*	Green space is not planted out -0	Green space is not planted out -0	Green space is not planted out -0
TOTAL SOURCE CONTROL		6 (out of min 4)	4 (out of min 4)	5 (out of min 4)
LIDS – Minimum of 3 points		Proposed solution	Minimal Solution	Minimal Solution
		Toolbox Option 1	Toolbox -Option 2	Toolbox -Option 3

Developer Lead	On lot devices to reduce runoff volume *Meeting the capture and infiltration requirements of the initial abstraction volume is given 2 points.* *· Meeting the capture and infiltration requirements for the site water quality storm is given 3 points.* *· Meeting the capture and infiltration requirements for the 2-year ARI event for the site is given 6 points.*	Properties to capture the 10/2 year ARI Soakage of dwelling (assumed 80% of area) = 4 points	Houses to capture the WQ Soakage (assumed 85% of area) = 2.5 points	Houses to capture the WQ Soakage (assumed 85% of area) = 2.5 points
Council Lead	Public devices to reduce runoff volume *Meeting the capture and infiltration requirements of the initial abstraction volume is given 2 points.* *· Meeting the capture and infiltration requirements for the site water quality storm is given 3 points.* *· Meeting the capture and infiltration requirements for the 2-year ARI event for the site is given 6 points.*		Public soakage basin/trench to capture the 10/2-year ARI Soakage (assumed 15% of area) = 0.5 points	Public soakage basin/trench to capture the 10/2-year ARI Soakage (assumed 15% of area) = 0.5 points
Developer Lead	Swales and filter strips *All impervious surfaces draining to swales and filter strips that have capacity for treating the water quality event and conveying the 2-year ARI event is given 3 points.*			Assume swales can capture 1/3 of development runoff = 1 point
Council Lead	Wetland *Meeting the water quality design storm criteria is given 2 points.* *Meeting extended detention and peak control requirements is given an additional 2 points.*	Treatment of Road and driveways – 1 point		
Council Lead	Urban design values *Stormwater management is designed to be an integral and well considered part of the urban design.*	A design narrative is developed for the vegetation parts of this site – 1 point	A design narrative is developed for the vegetation parts of this site – 1 point	
TOTAL SOURCE CONTROL		6 (out of min 3)	4 (out of min 3)	4 (out of min 3)
TOTAL POINTS		12 (out of min 9)	9 (out of min 9)	9 (out of min 9)

4.3 CATCHMENT 2

The key source control toolbox options for Catchment 2 are presented below with assessment undertaken on the proposed solution with area and percentage calculation provided in Appendix 7. The key outcomes for this catchment include:

- Catchment is lower in the gully and therefore soakage is considered less favourable than upper terraces.
- It is expected that there may be no specified green areas due to size of developable area.
- It is expected that the lots and site disturbance in this area will be of a conventional nature due to size.
- As this catchment is expected to discharge directly into a permanent waterway with large flood capacity, volume and peak discharge are considered not required.

The proposed approach for this catchment is:

- SOURCE CONTROL:
    - Utilise inert building materials
    - Water reuse (if soakage is not feasible) for private lots
    - Reducing the total impervious surface using permeable pavements
- LIDS CONTROL:
    - Soakage for private driveway runoff up to the 2-year ARI.
    - Adopt swales to convey flows.

Catchment 2 Table

Decision leaders	Source Control – Minimum of 4 points	Proposed solution	Minimal Solution	Minimal Solution
		Toolbox Option 1	Toolbox -Option 2	Toolbox -Option 3
Developer Lead	Water re-use	Site use for garden watering - 2 points.	Flow detention is adopted on houses - 1 point	Site use for garden watering and for non-potable inside waters uses including laundry and toilets - 3 points
Developer Lead	Use of building or site materials that do not contaminate *Residential roofs, gutters, down spouts made of non-contaminant leaching materials is 1 point.*	1	1	1
Council Lead	Impervious surfaces reduced from a traditional approach. *Impervious surfaces reduced from a conventional development approach* *5% reduction is 2 points.* *10% reduction is 3 points.*	Permeable pavements on all roads - 2.5 percent of catchment - 1 point	Permeable pavements on all roads - 5 percent of catchment - 2 point	0

TOTAL SOURCE CONTROL		4 (out of min 4)	4 (out of min 4)	4 (out of min 4)
LIDS – Minimum of 2 points		Proposed solution	Minimal Solution	Minimal Solution
		Toolbox Option 1	Toolbox -Option 2	Toolbox -Option 3
Developer Lead	On lot devices to reduce runoff volume *Meeting the capture and infiltration requirements of the initial abstraction volume is given 2 points.* *· Meeting the capture and infiltration requirements for the site water quality storm is given 3 points.* *· Meeting the capture and infiltration requirements for the 2-year ARI event for the site is given 6 points.*	Driveways capture the 2-year soakage – 1 point		
Developer Lead	Swales and filter strips *All impervious surfaces draining to swales and filter strips that have capacity for treating the water quality event and conveying the 2-year ARI event is given 3 points.*	Assume swales can capture 100% of development runoff = 3 point	Assume swales can capture 100% of development runoff = 3 point	
Developer lead	Bioretention (including tree pits) *Meeting the capture and retention requirements of the initial abstraction volume is given 2 points.* *Meeting the capture and retention requirements for the site water quality storm is given 3*			Site capture and retention requirements for the 2-year storm for all roads and driveways – 3 points

	points. *Meeting the capture and retention requirements for the 2-year storm for the site is given 6 points.*			
	Urban design values *Stormwater management is designed to be an integral and well considered part of the urban design.*		A design narrative is developed for the vegetation parts of this site – 1 point	A design narrative is developed for the vegetation parts of this site – 1 point
TOTAL SOURCE CONTROL		4 (out of min 3)	4 (out of min 3)	4 (out of min 3)
TOTAL POINTS		4 (out of min 8)	4 (out of min 8)	4 (out of min 8)

4.4 CATCHMENT 3

The key source control toolbox options for Catchment 3 are presented below with assessment undertaken on the proposed solution with area and percentage calculation provided in Appendix 7. The key outcomes for this catchment include:

- Soakage is considered favourable (up to the 2 year) due to conditions of the upper terraces.
- Urban design has ability to allow for green areas due to size of developable area.
- Lot areas and site disturbance can be reduced due to size of the developable area.
- As this catchment is expected to discharge to the mid gully with some ecological significance and potential for enhancement, volume control up to the 10 year is recommended.

The proposed approach for this catchment is:

- SOURCE CONTROL:
    - Reducing the total impervious surface of the site by avoiding development in or near the gully.
    - Reducing the site disturbance through utilising conventional lot sizes and confining the development to the terrace.
- LIDS CONTROL:
    - Utilise inert building materials
    - Soakage for private on lot runoff up to the 2-year ARI.
    - Public soakage device (basin/trenches) for road runoff and spill from private lots up to the 10-year ARI.

Catchment 3 Table

Decision leaders	Source Control – Minimum of 6 points	Proposed solution	Minimal Solution	Minimal Solution
		Toolbox Option 1	Toolbox -Option 2	Toolbox -Option 3
Developer/Council Lead	Site disturbance reduced from a conventional development approach · *10 % reduction from a conventional development is 2 points.*	2	2	2
Developer/Council Lead	Impervious surfaces reduced from a traditional approach. Impervious surfaces reduced from a conventional development approach *- 5% reduction is 2 points.* *-10% reduction is 3 points.*	Current expected lot coverage - 3	Larger reduction in lot sizes to account for the open space - 0	Smaller reduction in lot sizes to account for the open space - 5% -2
Developer Lead	Use of building or site materials that do not contaminate. *Residential roofs, gutters, down spouts made of non-contaminant leaching materials is 1 point.*	1	1	1
Council Lead	Existing streams and gullies (including ephemeral streams) are protected and enhanced *Preservation and protection of natural streams and gullies is 3 points.*	3	3	3

Council Lead	Protection and future preservation of existing native bush areas *Protection, preservation and, if needed, enhancement of native bush areas that exceed 10% of the site is given 2 points.*	2	2	2
TOTAL SOURCE CONTROL		11 (out of min 6)	8 (out of min 6)	9 (out of min 6)
LIDS – Minimum of 3 points		Proposed solution	Minimal Solution	Minimal Solution
		Toolbox Option 1	Toolbox -Option 2	Toolbox -Option 3
Developer Lead	On lot devices to reduce runoff volume *Meeting the capture and infiltration requirements of the initial abstraction volume is given 2 points.* *· Meeting the capture and infiltration requirements for the site water quality storm is given 3 points.* *· Meeting the capture and infiltration requirements for the 2-year ARI event for the site is given 6 points.*	Properties to capture the 10/2-year ARI Soakage (assumed 85% of area) = 5 points	Houses to capture the WQ Soakage (assumed 85% of area) = 2.5 points	Houses to capture the WQ Soakage (assumed 85% of area) = 2.5 points
Council Lead	On lot devices to reduce runoff volume *Meeting the capture and infiltration requirements of the initial abstraction volume is given 2 points.* *· Meeting the capture and infiltration requirements for the site*	Public soakage basin/trench to capture the 10/2-year ARI Soakage (assumed 15% of area) = 1 points	Public soakage basin/trench to capture the 10/2-year ARI Soakage (assumed 15% of area) = 1 points	Public soakage basin/trench to capture the 10/2-year ARI Soakage (assumed 15% of area) = 0.5 points

	water quality storm is given 3 points. *· Meeting the capture and infiltration requirements for the 2-year ARI event for the site is given 6 points.*			
Council Lead	Urban design values *Stormwater management is designed to be an integral and well considered part of the urban design.*		A design narrative is developed for the vegetation parts of this site – 1 point	
TOTAL SOURCE CONTROL		6 (out of min 3)	4 (out of min 3)	3 (out of min 3)
TOTAL POINTS		17 (out of min 8)	12 (out of min 8)	12 (out of min 8)

4.5 CATCHMENT 4

The key source control toolbox options for Catchment 4 are presented in the table below with assessment undertaken on the proposed solution with area and percentage calculation provided in Appendix 7 . The key outcomes for this catchment include:

- Soakage is considered favourable due to conditions of the upper terraces.
- Urban design has ability to allow for green areas due to size of developable area.
- Lots areas and site disturbance can be reduced due to size of the developable area.
- As this catchment is expected to discharge to gully with some ecological significance, volume control up to the 10 year is considered valid.

The proposed approach for this catchment is:

- SOURCE CONTROL:
    - Protection of gullies, streams, and natural open bushland.
    - Reducing the total impervious surface of the site by avoiding development in or near the gully.
    - Reducing the site disturbance through utilising conventional lot sizes and confining the development to the terrace.
- LIDS CONTROL:
    - Utilise inert building materials.
    - Soakage for private on lot runoff up to the 2-year ARI.
    - Public soakage device (basin/trenches) for road runoff and spill from private lots up to the 10-year ARI.

Catchment 4 Table

Decision leaders		Proposed solution	Minimal Solution	Minimal Solution

	Source Control – Minimum of 6 points	Toolbox Option 1	Toolbox -Option 2	Toolbox -Option 3
Developer/Council Lead	Site disturbance reduced from a conventional development approach *· 10 % reduction from a conventional development is 2 points.*	2	2	2
Developer/Council Lead	Impervious surfaces reduced from a traditional approach. Impervious surfaces reduced from a conventional development approach *- 5% reduction is 2 points.* *-10% reduction is 3 points.*	Current expected lot coverage - 3	Larger reduction in lot sizes to account for the open space - 0	Smaller reduction in lot sizes to account for the open space - 5% -2
Developer Lead	Use of building or site materials that do not contaminate. *Residential roofs, gutters, down spouts made of non-contaminant leaching materials is 1 point.*	1	1	1
Council Lead	Existing streams and gullies (including ephemeral streams) are protected and enhanced *Preservation and protection of natural streams and gullies is 3 points.*	3	3	3

Council Lead	Protection and future preservation of existing native bush areas *Protection, preservation and, if needed, enhancement of native bush areas that exceed 10% of the site is given 2 points.*	2	2	2
TOTAL SOURCE CONTROL		11 (out of min 6)	8 (out of min 6)	9 (out of min 6)
LIDS – Minimum of 3 points		Proposed solution	Minimal Solution	Minimal Solution
		Toolbox Option 1	Toolbox -Option 2	Toolbox -Option 3
Developer Lead	On lot devices to reduce runoff volume *Meeting the capture and infiltration requirements of the initial abstraction volume is given 2 points.* *· Meeting the capture and infiltration requirements for the site water quality storm is given 3 points.* *· Meeting the capture and infiltration requirements for the 2-year ARI event for the site is given 6 points.*	Properties to capture the 10/2 year ARI Soakage (assumed 85% of area) = 5 points	Houses to capture the WQ Soakage (assumed 85% of area) = 2.5 points	Houses to capture the WQ Soakage (assumed 85% of area) = 2.5 points
Developer Lead	Swales and filter strips *All impervious surfaces draining to swales and filter strips that have capacity for treating the water quality event and conveying the 2-year ARI event is given 3 points.*	Public soakage basin/trench to capture the 10/2-year ARI Soakage (assumed 15% of area) = 1 points	Public soakage basin/trench to capture the 10/2-year ARI Soakage (assumed 15% of area) = 1 points	Public soakage basin/trench to capture the 10/2-year ARI Soakage (assumed 15% of area) = 0.5 points

	Urban design values *Stormwater management is designed to be an integral and well considered part of the urban design.*		A design narrative is developed for the vegetation parts of this site – 1 point	
TOTAL SOURCE CONTROL		6 (out of min 3)	4 (out of min 3)	3 (out of min 3)

5 IMPLEMENTATION

The following section outlines the proposed implementation with high-level sizing of devices to demonstrate applicability moving to the next stages.

A preliminary summary of the C4 stormwater concept design is provided below. It is noted that the concept design needs to be integrated with wider urban design elements and planning considerations. However, embedding water sensitive design principles to manage stormwater as early as possible in the design process is smart and follows international best practice.

FIRST FLUSH:

- First flush events will be managed at source via a series of pre-treatment devices prior to discharge for all catchments 1, 2, 3 and 4. Pre-treatment for on lot devices is recommended to ensure the long-term performance of the device by removing the coarse grain fragments and any large litter items. Examples include rainwater harvesting, leaf diverters, sumps, filter stops and porous surfacing. The RITS provides for on-site water efficiency measures which include a variety of pre-treatment options which shall be applied within the C4 growth area as part of building consent.
- First flush events may also be managed at source via water reuse (for Catchment 2) where soakage is unlikely to be viable.
- Green networks are encouraged within the development integrated with overland flow paths, park edge swales and planted soakage basins for amenity and passive recreational use.
- First flush events from the road network will be managed via pre-treatment devices prior to discharge to ground (soakage) for Catchment 3 and 4. Pre-treatment of public soakage devices is recommended to ensure the long-term performance of the final adopted soakage devices by removing the coarse grain fragments and any large litter items. Examples include sediment forebays built within larger soakage basins, catch pit inserts/chamber sumps, grass filter strips and planted swales.
- First flush events from the road network (Catchment 1) will be managed via pre-treatment prior to entering the wetland on the lower terrace. This could be a sediment forebay within the wetland.

PRIMARY/WQ AND EDV STORM RUNOFF:

- Soakage up to the 2 year ARI event will occur on lot for Catchments 1, 3, and 4 (noting that the small catchment 2 will soak driveway runoff only due to proposed water reuse and expected low soil permeability). This will reduce the size of the public infrastructure (drainage network, soakage basins and wetland) needed to manage and treat runoff.

- Primary flows from road runoff, including spill above the 2 year from private lots, up to the 10yr ARI will be conveyed using pipes or swales to soakage devices either communal planted basins or trenches within the road reserve (catchments 3 and 4).
- The soakage up to the 10 year (incorporating the WQ and EDV volumes) removes the potential for adverse impacts of increased contaminant and temperature discharge as well as scour erosion and sedimentation within the C4 Stream receiving environment.
- Water Quality and EDV volumes (Catchment 1) are conveyed to a wetland. The wetland and EDV treatment remove the potential for adverse impacts of increased contaminant and temperature discharge as well as scour erosion and sedimentation within the C4 Stream receiving environment. High flows bypass the wetland and discharge to the natural water body that forms part of the receiving environment.
- Water Quality and EDV volumes (Catchment 2) are recommended to be conveyed to swales. The swales remove the potential for adverse impacts of increased contaminant and temperature discharge within the C4 Stream receiving environment. Primary flows up to the 10 year for road and dwellings are also conveyed by swales and discharge to the natural water body that forms part of the receiving environment.

SECONDARY FLOW:

- Secondary flows up to the 100yr ARI + climate change (CC) event must be managed and safely conveyed within the subdivision to protect pedestrians, road users and building floor levels (meeting freeboard requirements). This requirement also covers New Zealand Building Code 50yr ARI design standard to protect buildings from flood inundation.
- No requirements for flood attenuation and peak flow control is required due to capacity of the downstream network as demonstrated by flood modelling.

5.1 SOAKAGE SIZING

Soakage disposal will form a key aspect of the stormwater solution for all catchments 1,2,3 and 4. Soakage is supported by the geotechnical review and by the stormwater disposal hierarchy outlined in the RITS. Soakage disposal is also a practical option which provides multiple benefits for the development to be implemented within both the public and private realm, including:

- Maintains the natural hydrological outcomes for the catchment (10-year pooling and soaking to ground and flows above the 10-year discharging from the site).
- Avoids the potential adverse effects on the stream receiving environment of smaller more frequent storm events up to the 10yr ARI event.
- Assists in reducing peak flows from larger storm events up to the 100yr ARI.
- Maintains base flows to the stream environment.
- Coupled with appropriate pre-treatment captures and treats contaminant runoff from impervious surfaces.
- Soakage at source reduces infrastructure requirements such as size of the stormwater primary pipe network.

The following section presents the recommended soakage approach for both public and private devices.

5.1.1 PRIVATE DEVICES

Private on-lot soakage devices considered are a viable option due to:

- The geology, soil type and residential land use and in accordance with the stormwater hierarchy promoted in the RITS.
- It is noted Cambridge Park sub-division (opposite C4) adopted on lot soakage up to the 2yr ARI event to good effect and many parts of Leamington also use on lot soakage devices prior to discharge to the C4 gully.

Private devices are recommended to have the following design considerations:

- Capture runoff from all impervious areas including roof and driveway for catchments 3 and 4.
- Capture runoff from driveways only for Catchments 2 due to specification for water reuse in this catchment.
- Capture runoff from roof only for Catchment 1 due to specification for wetland treatment in this catchment.
- Separate configurations could be adopted for clean roof water and driveway runoff using side by side soakage chambers.
- Driveway areas could also be porous (permeable pavers, porous concrete) thereby negating the need for a separate soakage device adjacent to the driveway within the lot boundary.
- Roof areas could firstly drain to a detention tank for re-use prior to out letting to the soakage device.

Given most regular rain events will be captured and returned to ground on site, there will likely be minimal actual runoff to the public network. This would only occur for events greater than the on lot device design which is recommended at a 2 year ARI event. Consideration therefore should be given to adopting kerb outlet from each lot to reduce the need or size and therefore cost of expensive storm water pipe infrastructure.

5.1.2 PUBLIC DEVICES

Stormwater runoff from the public road reserve will be managed separately to runoff from private for events less than the 2yr ARI above which the lots with spill into the public conveyance network. Options are summarised below:

- Ruoff from road pavement could be collected via traditional kerb and channel to catchpit inlets and then to a pipe network or to a park edge swale via flush or drop kerbs.
- A swale network can potentially provide treatment, conveyance, and soakage prior to discharge to a soakage basin or wetland. Due to the size of the devices it is unlikely that the site runoff can be managed by swales only, however, the use of swales will reduce the size of the end of line soakage basins and provide excellent pre-treatment benefits.
- Swales can be either side of the road, on one side (reduce need for driveway crossings) or they could be designed independently of the road network within larger green corridors linking the development.

- Disposal to ground in soakage basins (likely to be preferred by WDC over trenches) would need careful consideration as to their location, depth and runoff loading given the geotechnical constraints and set back requirements outlined in the Mark Mitchel report. Basin sizes however maybe be relatively modest to treat runoff from just the road corridor.

The following indicative sizing table is provided to assist WDC, developers and lot builders

Table 5 Soakage size estimates – assuming 100% void ratio (ie. no gravel filled devices). Sizes are considered conservative due to relatively low soakage rate (site testing may show higher soakage rates).

Catchments	Contributing Impervious Area	Assumed Soakage Rate	Assumed Storm Event	Soakage Area and Volume	Approximate Overall Device Areas
Catchment 1 – on lot (roof only)	3.7 ha (assumed 151 lots)	70mm/hr	2 year	10.3 m^2 10.3 m^3 (4.3 x 2.4 x 1m) per lot	10.3 m^3 per lot
Catchment 2 – on lot (driveway only)	0.11 ha (assumed 11 lots)	70mm/hr	2 year	6.0 m^2 2.5 m^3 (5 x 1.2 x 0.5) per lot	2.5 m^3 per lot
Catchment 3 – on lot (roof and driveways)	6.2 ha (assumed 178 lots)	70mm/hr	2 year	14.3 m^2 14.3 m^3 (5 x 2.8 x 1m) per lot	14.3 m^3 per lot
Catchment 3 – public system (road and footpaths)	1.88 ha	70mm/hr	10 year plus 10-year overflow from lots	1174 m^2 1996 m^3 (32 x 37 x 1.7m)	Basin: Device depth (3 m) Device Area 2430 m^2 Trenches: 250 m (base width 1.5 metre) Depth 0.5 metre
Catchment 4 – on lot (roof and driveways)	10.1 ha (assumed 289 lots)	70mm/hr	2 year	14.3 m^2 14.3 m^3 (5 x 2.8 x 1) per lot	14.3 m^3 per lot
Catchment 4 – public system (road and footpaths)	3.1 ha	70mm/hr	10 year plus 10-year overflow form lots	1880 m^2 3190 m^3 (50x 38 x 1.7)	Basin: Device depth (3 m) Device Area 3410 m^2 Trenches: 550 m (base width 1.5 metre) Depth 5 metre

The following assumptions have been implemented in the estimation of the soakage device volumes and areas:

1. 100% runoff from dwelling impervious areas and 90% from road surfaces.

2. Infiltration is through the device base only and based on the average values from the Geocon Report.
3. Storage required is based on volume lost to ground (over storm duration) and live storage within the device (assuming 100% void space ie. tank/'milk crate' systems).
4. Approximately 1m deep device have been assumed for on lot devices (i.e. soakage manhole/tanks) or 1.5m overall depth assuming 0.5m cover.
5. Approximately 3m deep devices have been assumed for public devices (i.e. soakage basins. The total device area is based on 1 in 4 slopes.
6. Approximately 0.5m deep devices have been assumed for soakage trenches or 1m total depth with 0.5m cover.
7. Public systems are based on critical 10 year storm durations from 10 minutes to 48hrs.
8. Private systems are based on critical 2 year storm durations from 10 minutes to 48hrs.

5.2 WETLAND DESIGN

The following Table 6 estimates the size of the wetland for Catchment 1 in the lower terrace.

TABLE 6 WETLAND AREA

Catchments	Contributing Impervious Area	Volume (WQ/2 + EDV+FB) (m^3)	Surface Area (4% of catchment) and 20% for Batters/maintenance (m^3)	Estimate of (m)	Estimate of Length (m)
Catchment 1	3.4 ha (assumed road and driveway)	1230	1662	20	80

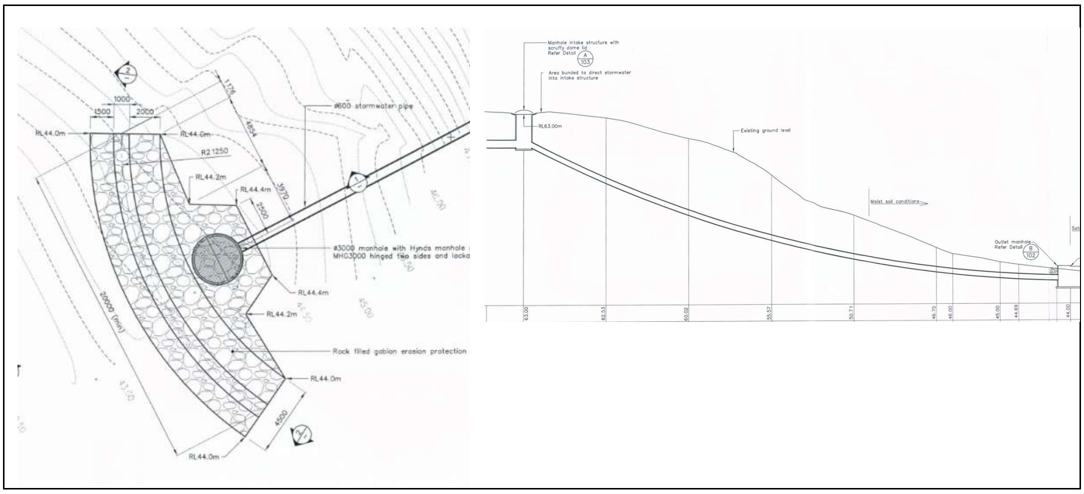
5.3 PROPOSED OUTLET DESIGN – CATCHMENTS 1, 3 & 4

Both primary and secondary flows will be conveyed to the gully base and then flows will be dispersed and fan out across flat gully to stream channel.

High velocities are expected within the pipe down the gully and at the outlet. Construction of each gully outlet structure will therefore involve the placement of a concrete manhole stilling well, combined riprap and gabion protection and potentially a directionally drilled HDPE pipe. The outlet structure will provide velocity reduction of stormwater discharges to the gully environment.

A similar outlet is recommended to that currently used for the adjacent Cambridge Park development. A selection of screen shots from the design drawings (Tonkin and Taylor, 2008) and photos from the authors site visit of the Cambridge Park outlet are provided below.

The stilling manhole is surrounded by rip rap with a gabion wall providing a ~20m wide weir for spills to fan out into the gully.


1000
1500
2000
1176
4854
3970
2500
RL44.0m
RL44.0m
R2 1250
RL44.2m
RL44.4m
ø600 stormwater pipe
ø3000 manhole with Hynds manhole
MHG3000 hinged two sides and locka
RL44.4m
RL44.2m
20000 (min)
Rock filled gabion erosion protection
RL44.0m
4500
RL44.0m
Manhole intake structure with scruffy dome lid
Refer Detail A 103
Area bunded to direct stormwater into intake structure
RL63.00m
Existing ground level
Moist soil conditions
Outlet manhole
Refer Detail B 102
63.00
62.53
60.02
55.57
50.71
46.75
46.00
45.00
44.69
44.00

6 WATER SUPPLY

6.1 BACKGROUND REVIEW

The following documents were reviewed during the preparation of the water supply section:

- The Waikato LASS Regional Infrastructure Technical Specifications (RITS)
- The Waipa District Development and Subdivision Manual
- Opus Waipa District Wide Water Supply Strategy (DWWSS), 2014

Consultation with WDC staff (Robin Walker) has also been undertaken regarding existing infrastructure and programmed upgrades.

Following review of the first draft of this report, a meeting was held with the WSP Waipa Master Plan Team (Rebecca Francis, Jorge Munoz Santamaria and Mark De Lange) on the 17th March 2020 to discuss the Cambridge master plan water supply and wastewater modelling inputs and outputs and how these interrelate with the assumptions in the C4 growth cell model.

6.2 EXISTING NETWORK

The C4 area falls just outside the Cambridge municipal water supply network. There is an existing 150mm diameter PVC water supply pipe running around the Western side of C4 along Lamb Street and Cambridge Road. This supply is currently serviced by the Pukerimu Water supply scheme and is a low pressure “trickle-feed” supply that will not meet the requirements of a new residential development. WSP have included this pipe within their masterplan model network with a single demand node for the C4 growth cell.

The WDC municipal reticulation borders on the Western boundary of the C4 area, supplying the areas of Leamington to the South and Cambridge Park to the north. This network is supplied from the Karapiro Water treatment plant and conveyed to Leamington (Browning Street) in twin 375mm diameter trunk mains. The supply then flows through the Leamington network before crossing over the Victoria Street Bridge to Northern Cambridge.

It appears that the municipal network has been extended from the Pope Terrace/ Cambridge Road Roundabout to supply Aoteoroa Park/ Matos Segedin Drive area.

There is also capacity on the existing Cambridge Pipe Bridge across the Waikato River to take an additional new water pipe, however, preliminary modelling by WSP indicated that this would have minimal impact on the water networks (including C4) south of the Waikato River.

6.3 DESIGN FLOWS

The DWWSS 2014, states that WDC use a rate 261 L/person/day. This correlates with the RITS daily domestic rate of 260 L/person/day for residential subdivisions. The current peak factor for Cambridge was found to be 1.69 in 2014 and it was WDC’s intention to maintain this peak factor. This is significantly lower than the RITS requirement of a peak factor of 5.

The WSP masterplan flows for the C4 cell have been determined using the demand projection for 2050. This was determined with the Peak Day model demand as a base for calculation and using the NZ1-16239247-DRAFT Gateway Approval 4 - Population Forecast Report figures, which stated the number of people per growth cell in 2050. This projection has resulted in a lower expected population of 1830 people and an average daily demand of 5.6l/s and a peak demand of 15l/s with a peak factor of 2.4. As can be seen from Table 5 the projected populations result in significantly lower flow rates than the requirements of the RITS and should be addressed as part of the additional masterplan modelling.

For the purposes of this assessment we have adopted the RITS requirements as a more conservative approach

6.4 NORMAL PEAK DEMAND

Water supply design flows based on the RITS are summarised in Table 7 below.

Table 7: RITS WATER SUPPLY DEMAND

CATCHMENT AREA (Ha)	POPULATION EQUIVALENT	PEAK FACTOR	AVERAGE DAILY FLOW (m3/D)	DOMESTIC FLOW RATE (l/s)	PEAK FLOW RATE (l/s)	FIRE FLOW RATE (l/s)
65	2925	5.00	760.50	8.80	44.01	51.41

The DWWSS 2014 identified the current peak factor for Cambridge residential areas as 1.69. The report identified this peak factor as suitable for future forecasting. If this peak factor is applied to the flow rates listed in Table 7 above, the peak flow rate and fire flows will reduce to 14.88 l/s and 33.93 l/s respectively.

6.4.1 FIRE FIGHTING DEMAND

The WDC Water Supply Bylaw 2013 states that Council is under no obligation to provide an on-demand supply for fire protection purposes at any particular flow or pressure or maintain existing pressures or flows. It is noted that this is in contradiction to Section 6.2.3.3 of the RITS which states that “Council’s standard design meets the FW2 firefighting requirements at the street boundary for residential areas and provides FW3 for other zones.”

It is aspirational to supply a minimum of an FW 2 Water Supply Classification within the reticulated network. The feasibility of this will be tested once the outstanding information about the existing network has been provided. PAS NZS 4509:2008 states that FW2 requires 25 l/s to be provided from a maximum of 2 fire hydrants. The fire demand should be applied on top of 60% of the peak flow.

The practical reasoning for providing an FW 2 supply is that if a building is fitted with sprinklers, then those may be supplied by the network, and subsequently the fire service upon attendance at the fire, also from the network. Even if the reticulation network does not meet the head requirements to meet FW2 flows the reticulation will need to be sized to ensure that FW2 requirements can be met using a fire tender pump.

6.4.2 WATER SUPPLY NETWORK ALLOCATION

WSP confirmed the Cambridge masterplan model includes the existing 150mm diameter pipe as a single point demand. This line runs along Lamb Street and Cambridge Road within the C4 growth cell. The WSP model was run for a period of 24 hours with a peak factor of 2.4 and the meeting with WSP indicated there most likely is capacity to supply the C4 growth cell, however more specific modelling around the C4 cell is required.

6.5 IMPACTS OF STAGING AND TIMING

The development of C4 will most likely be phased, with sales of each phase determining the development of the next phase. As the land has multiple landowners this will also impact the development staging if some owners are not willing to develop their property at the same stage as others.

WDC’s intention is to extend the water network from the Cambridge Park roundabout along Cambridge Road towards the C4 growth cell. This may not align with actual development stages and it would be worthwhile investigating the option of supply from the Leamington side as well for a portion of the C4 zone. Ultimately this would be a preferred looped supply feeding C4 from Leamington and Cambridge Park.

The extent of phasing will also be influenced by the final source of water supply and base capacity that will be identified in the master plan report due in 2020.

6.6 PROPOSED WATER SUPPLY NETWORK

An initial draft reticulation concept to service the development is included in Appendix 5. This draft network is based on a preliminary development layout that mimics block sizes of the neighbouring suburbs. The water model network has been analysed with 150mm diameter pipes on both sides of the road and analysed using EPANet.

The WSP master plan model did not model the C4 area in isolation nor any connections points to C4, only the ring main that would supply C4 and the predicted demand is included in the masterplan model. WSP did however confirm that in its current configuration the network would be able to provide a supply pressure of 300kPa at the Leamington and Pope Terrace ends of the ring main. Our C4 network model includes the ring main from Leamington to Pope Terrace and assumes a connection pressure of 300kPa.

We have modelled the water demand on what we perceive to be the usable areas within the C4 growth cell. This usable area excluded the gulley areas of the C4 growth cell and resulted in a total area of 49.5Ha. The design flows used for initial modelling are in Table 8:

Table 8: DESIGN WATER SUPPLY FLOWS

USABLE CATCHMENT AREA (Ha)	POPULATION EQUIVALENT	LOTS	PEAK FACTOR	AVE. DAILY FLOW (m3/D)	DOMESTIC FLOW RATE (l/s)	PEAK FLOW RATE (l/s)	FIRE FLOW RATE (l/s)
49.5	2228	825	5.00	579.28	6.70	33.52	45.11

The results of our C4 model indicate that if the network can provide a constant supply pressure of 300kPa at Pope Terrace and Leamington (as indicated in the WSP model) there would be sufficient residual pressure within C4 during peak flows. Under fire flow conditions, however, the residual pressures within the network will drop below the RITS requirement of 200kPa.

Should the supply pressures fall below 300kPa the pressure within C4 will drop below 200kPa under normal flow conditions.

Discussions with WSP highlights the need for additional modelling of the C4 growth cell in isolation to determine what upgrades would be needed to ensure the viability of the C4 growth cell in the future. Additional modelling is also required to address the higher demand and peak flow rates as specified in the RITS

6.7 LONG TERM WATER DEMAND

It is well recognised that as growth continues, the demand for water will also increase, sometimes reaching close to the limits of sources of supply.

The figure below shows that the 2050 projected minimum pressures are currently projected to be low with C4 being less than 10m (100kPa) and Leamington and Pope terrace (Cambridge Park) being between 10-20m (100 – 200kPa). This illustrates that without upgrades the existing network will be unable to sustain the growth cells.

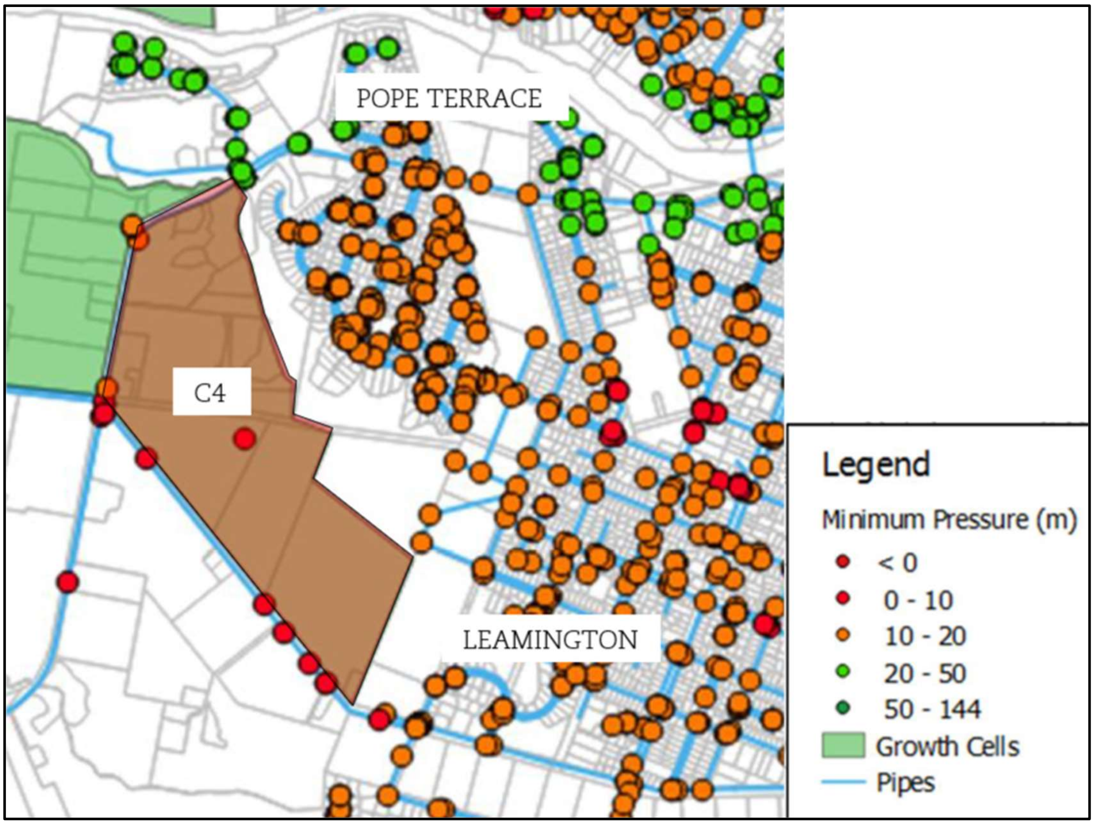


FIGURE 7 MINIMUM PRESSURES 2050 (*EXTRACTED FROM WSP MEMO TO WDC 23/09/2019)*

Some steps to mitigate this and to aid in promoting best practice in water sensitive design, water reuse, where appropriate, should be considered. If some, or all, of that water can be harvested and stored, then it can be used to offset the treated potable demand. This water can be used for non-potable building water services such as garden irrigation and toilet flushing.

The most economic time to introduce the infrastructure to enable harvesting and reuse is at the initial building development point.

The implementation of individual water metering has also shown to reduce domestic water consumption.

7
WASTEWATER

7.1 BACKGROUND REVIEW

The following documents were reviewed during the preparation of this section:

- The Waikato LASS Regional Infrastructure Technical Specifications (RITS)
- The Waipa District Development and Subdivision Manual
- NZS 4404:2010 Land Development and Subdivision Infrastructure
- Wastewater Treatment and Disposal Activity Management Plan 2015 - 2025
- Opus Wastewater Issues Report, 2013
- Opus C7 Growth Cell – Wastewater Assessment, 2017

Correspondence with WDC staff (Robin Walker) was also had regarding existing infrastructure and proposed upgrades. A meeting was also held with WSP master plan team (Rebecca Francis, Jorge Munoz Santamaria and Mark De Lange) in March 2017 to discuss their master plan and the impacts on the C4 growth cell.

7.2 EXISTING NETWORK

Currently all the wastewater generated within Cambridge is conveyed by a gravity network to the Wastewater Treatment Plant (WWTP) on the Southern bank of the Waikato River on the western border of the urban limit. The wastewater generated from the northern part of Cambridge crosses the Waikato River on the pipe bridge, west of the River Garden residential development. The gravity main across the pipe bridge was recently upgraded to a 700mm diameter CLS pipe. The northern network then joins the southern network and flows in a 600mm diameter gravity main to the WWTP.

This portion of pipe is known to surcharge and was recommended to be upgraded by 2025 in the 2013 Cambridge and Te Awamutu Wastewater Master Plan. With the current rates of development within Cambridge it is expected that the 2020 master plan will advance this upgrade.

The Aotearoa Park gravity network connects to this portion of the trunk main. Due to the surcharging, the gravity flows are collected in a wastewater pump station (WWPS) on Matos Segedin Drive and pumped 50m into a manhole on the trunk main upstream of the WWPS.

The proposed connection point for a gravity network from the C4 growth cell has been identified as the WWPS on Matos Segedin Drive. This WWPS may have spare capacity to accommodate a minor portion of the C4 development but will require major upgrades to meet the demands of the full development.

7.3 DESIGN FLOWS

Section 5.2.4.2 of the RITS sets out the following criteria for the calculation of wastewater flows:

- Domestic average daily flow is 200 litres per person per day.
- Infiltration allowance is 2,250 litres per hectare per day.
- Surface water ingress allowance is 16,500 litres per hectare per day.
- Peaking factor based on Table 5.2.
- Population equivalent as per Table 5.3. For General Residential this is 45 persons per hectare.
- Gross contributing land area upstream of the wastewater pipe is defined as the total catchment area, excluding reserve land, but including land within legal road boundaries

Average daily flow

ADF = (infiltration allowance x catchment area) + (water consumption x population equivalent)

Peak Daily Flow

PDF (l/s) = ((infiltration allowance x catchment area) + (peaking factor x water consumption x population equivalent))/86400

Peak inflow and infiltration factor

PIIF (l/s/ha) = infiltration allowance + surface water ingress

Peak wet weather flow

PWWF (l/s) = ((infiltration allowance x catchment area) + (surface water ingress x catchment area) + (peaking factor x water consumption x population equivalent))/86400

The wastewater design flows have been based on the RITS and are summarised in Table 9. We have also included the WSP master plan information.

Table 9: RITS WATER SUPPLY DEMAND

	AREA (Ha)	POPULATION EQUIVALENT	AVERAGE DAILY FLOW (m3/D)	PEAK DAILY FLOW (l/s)	PEAK WET WEATHER FLOW (l/s)	EMERGENCY STORAGE m3
C4 Growth Cell	65.0	2925	731.25	20.65	33.06	274.22
C4 Usable Area	49.5	2228	557.00	15.73	25.19	208.87
C4 WSP Masterplan	66.0	1830	-	14.0	26.6	-

For the purposes of this report we have adopted the more conservative wastewater flows from the entire C4 Growth Cell.

7.3.1 WASTEWATER NETWORK ALLOCATION

The master plan modelling carried out by WSP have identified the discharge from the C4 growth cell to be in the same manhole that the Matos Segedin WWPS discharges to. Their model shows that while this part of the network does surcharge, there is sufficient capacity for the C4 flows.

There is also capacity within the wastewater treatment plant to treat the effluent produced by the C4 zone.

7.4 PROPOSED WASTEWATER NETWORK

On-site wastewater treatment and soakage is not considered to be feasible for this site based on the anticipated volume of wastewater that will be generated.

The topography of the site is essentially three relatively flat terraces, with a steep drop down to the Aotearoa Park network on Matos Segedin Drive. There is also a large gully to the East of the site. The gulley area has been excluded from the wastewater network as we believe it will not be developed.

The preferred solution would be to drain the whole area by gravity, however as the site is generally flat there is a chance some of the pipes may be quite deep. In the situation where the gravity network becomes impractical because of extreme depths and/or significant earthwork changes the possibility of using wastewater pump stations has also been addressed as an option.

7.4.1 GRAVITY NETWORK

To accurately assess the depth limitations of a gravity network, an initial wastewater network concept was developed to service the site, this can be found in Appendix 5. This was based on a very preliminary layout that we created using similar block sizes of the neighbouring suburbs.

The site (excluding the gulley) is generally flat and for this assessment we have assumed that there will not be extensive earthworks carried out on the site other than filling in some localized areas and possibly smoothing out some of the terrace drops.

Generally, we found most of the pipeline depths to be in the 2-4m depth range. There were some deeper sections where the pipe depths were over 6m deep. We believe that in these cases further investigation in the pipeline route will result in a shallower route. An earthworks design that compliments the gravity network by falling towards the north will also reduce the pipe depths.

The network we developed shows that it is possible to create a gravity network that will be able to connect to the Matos Segedin WWPS. The network does however run through the C4 from South to North. Any development in the Southern portions of C4 would require consent from the other landowners to allow the gravity main to run through their property. Running the gravity main along Lamb St and Cambridge Road (avoiding traversing the northern properties) will result in very deep pipelines and is not feasible.

The current network to the Matos Segedin WWPS consists of a very small network of 150mm diameter pipes discharging into the WWPS. A 150mm diameter pipe normally has a Peak Wet Weather Flow (PWWF) capacity of about 14 l/s. This is assumption can be justified by the information provided by WDC that the Matos Segedin Drive WWPS has a PWWF of 8.8 l/s and pump duty of 10 l/s. As the C4 Growth Cell has an expected PWWF of 33.06 l/s, the pipe network along Matos Segedin Drive from the Cambridge Road intersection will need to be upgraded to accommodate the additional flows.

The Matos Segedin Drive WWPS does appear to be able to accommodate some additional flows with additional cycles and minor upgrading of the pumps. Ultimately the restriction is the capacity of the existing 80mm diameter rising main. The RITS restricts the flow velocity in the rising main to a maximum of 3 m/s, with pumps sized to match the PWWF. At 3 m/s this would have a maximum flow capacity of 14 l/s, 5.2 l/s above the current PWWF of 8.8 l/s. This additional capacity would only cater for about 170 additional lots.

Once the Capacity of the Matos Segedin WWPS is exceeded it would need to be completely upgraded for increased capacity, increased emergency storage, and upgraded pumps and rising main. As this would be a significant capital cost, the option of discharging from the C4 zone directly into the gravity network at an alternative location has been investigated.

To avoid having to upgrade the Matos Segedin WWPS a second gravity option of connecting to the wastewater network to the West of the River Gardens development was investigated. This option is possible however there will be some large sections of pipeline in excess of 7m depth that would most likely make it economically unfeasible.

Based on the limitations of the Matos Segadin WWPS and finding an economically feasible gravity main the possibility of a gravity network is considered unsuitable for the C4 growth cell.

7.4.2 COMBINED NETWORK

With the gravity network being unsuitable, requiring multiple landowner consents and costly upgrades to the Matos Segadin WWPS, a combined sewer network with smaller gravity networks feeding a central wastewater collection point with a pump station discharging into the existing gravity network is a possible solution.

Depending on earthworks there may be multiple WWPS. These pump stations could be operating in a “chain” with all the C4 WWPS discharging to a central, larger pump station that discharges into the same discharge point as the Matos Segadin WWPS (Figure 8). This system also allows for phased development with the first pump station being the collector and constructed with the rising main. Subsequent phases requiring a pump station will then discharge to the collector pump station.

The possibility of discharging to the Leamington WW network was investigated however there is insufficient capacity available.

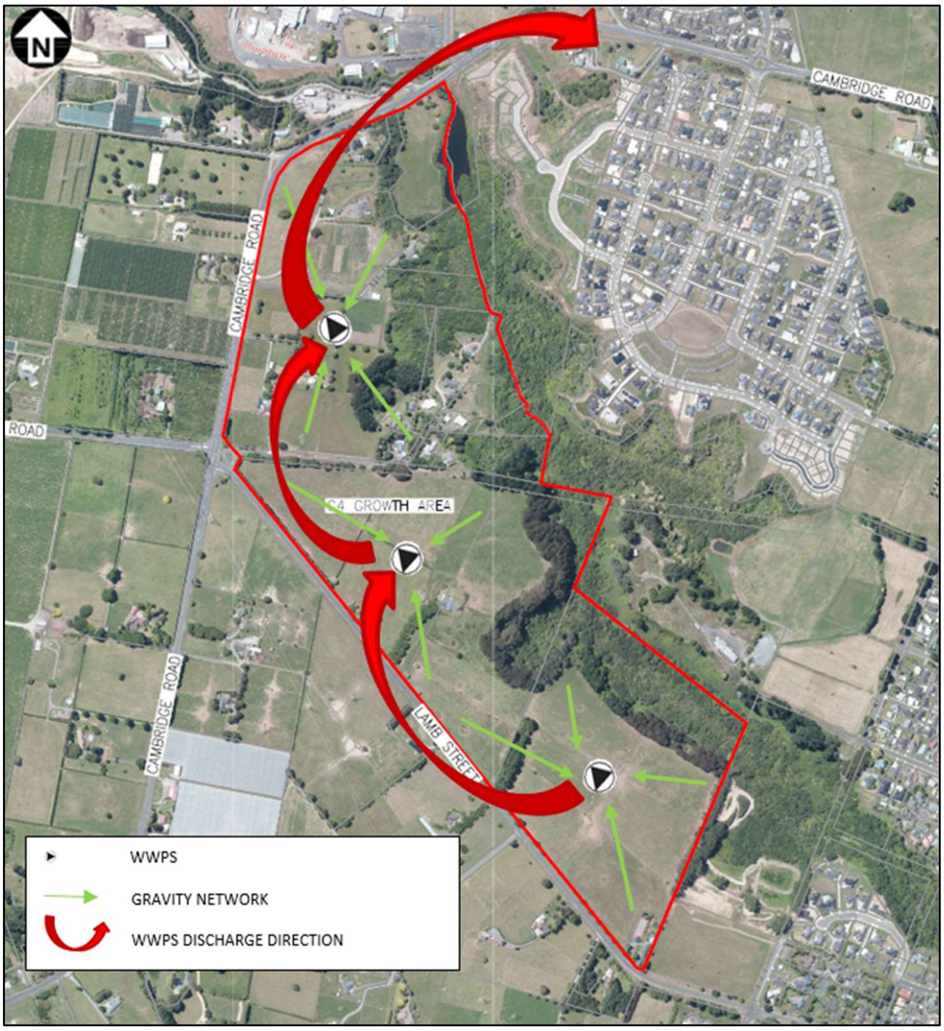


FIGURE 8 WWPS CHAIN

The use of WWPS's would mitigate the dependency of the network to travers through the C4 properties as the rising mains can be laid within the road reserves. This option is also in line with the WSP master plan assumptions and has the most flexibility in terms of phasing and earthwork modelling

8 SUMMARY WATER SUPPLY AND WASTEWATER

A summary of the recommendations from this report in respect of servicing the C4 Growth Cell, water and wastewater follows:

1. The water supply and network for the C4 growth cell needs to be modelled in detail as part of the master plan to study the impact of C4 in isolation as well as the point/s of connection. Further modelling will identify the impact of the growth cell in relation to the neighbouring networks as well as identify any upgrades required for the development of the growth cell.
2. A pumped wastewater network discharging into the gravity network upstream of the Matos Segadin WWPS is the preferred scheme for the C4 Growth cell.

3. Even if the water and wastewater infrastructure can provide for the development, water efficiency measures such as rain water harvesting and grey water recycling are well established technologies in New Zealand and can provide benefits in the form of reduced demand on water supply and wastewater treatment.

9 THREE WATERS CONCLUSIONS

9.1 STORMWATER

1. The ecology report highlights the C4 stream to be vulnerable to changes in hydrological conditions resulting from new development within C4. The geo-technical report indicates groundwater conditions that are favourable for disposal of stormwater via soakage techniques. The gully edge is however susceptible to erosion from uncontrolled surface flows and infiltration within the building setback line.

2. Peak flows above the 10 year will increase to the gully compared to the existing landuse, no adverse effects are expected on flood risk or stream habitat due to the significant storage capacity and existing culvert control under Cambridge Road as well as diffuse flows through heavily vegetated gully floor prior to flows reaching the stream. Above the 10 year, the gully stream will be out of bank.

3. Currently there are several options to manage stormwater using the principles of water sensitive design - the primary objective is however to utilise soakage techniques as the preferred approach to treat water quality and manage the primary 10 year flow in accordance with the stormwater disposal hierarchy in the RTIS. Soakage devices are proposed within each private lot which will be controlled using the WDC stormwater management bylaw. Public road reserves can be serviced using a range of techniques which include rain gardens overflowing to soakage devices, communal basins, infiltration swales, trench soakage and porous manholes. These options will be discussed with WDC and will need to be integrated with the urban design layout and roading network.

4. Currently 4 stormwater outlets are proposed within the gully floor. Flows above the final soakage design up to the 100 year + cc event will be conveyed safely within the development roading network and greenspace and are likely to be piped down the gully side to the outlet. Secondary flows must be controlled to the outlet to avoid erosion of the gully sides and outlet erosion control measures such as a stilling basin and flow dispersion implemented within the gully floor. The main stream is approximately 60m-100m from the proposed gully outlet points allowing some distance for dispersal of high flows within the existing storage area.

9.1.2 THE PREFERRED SOLUTIONS ARE:

- Private soakage disposal on each lot
- Communal soakage basins or trenches in public reserves to manage road runoff
- Primary flow reticulated to each soakage device
- Secondary flows conveyed within road or public greenspace reserves to drop structure prior to outlet to the basins floor via erosion control and energy dissipation basins

9.2 WATER SUPPLY

1. Additional modelling around the C4 Growth cell needs to be carried out and included in the Waipa Masterplan Modelling to confirm connection points, and capacity upgrades. WDC will also need to confirm timelines for any upgrades that will influence the development of this zone.

9.3 WASTEWATER

1. Options for pumped and gravity networks and discharge point have been identified as possible wastewater solutions, with the wastewater treatment plant having adequate capacity to treat all generated waste from the C4 development. The master plan model identified a discharge point with adequate capacity for the C4 growth cell.

2. The preferred wastewater option is gravity networks within the C4 growth cell, pumped along the road reserves to the gravity manhole upstream of the Matos Segadin WWPS. The number of pumps and extent of the gravity networks will be determined at the detailed design phase.

3. WDC need to include the wastewater generated from the C4 Growth Cell into their Masterplan models to determine capacity within the existing network. If there is insufficient capacity WDC will need to provide timelines for the upgrades.

10
LIMITATIONS

10.1 GENERAL

This report is for the use by Waipa District Council and should not be used or relied upon by any other person or entity or for any other project.

This report has been prepared for the project described to us and its extent is limited to the scope of work agreed between the client and Te Miro Water Limited. No responsibility is accepted by Te Miro Water Limited or its directors, servants, agents, staff or employees for the accuracy of information provided by third parties and/or the use of any part of this report in any other context or for any other purposes.

APPENDIX 1 SITE PHOTOS

Photo 1: Top Terrace C4 Existing Greenfield – Looking North

Photo 2: Top Terrace Existing Well Drained Horse Grazing

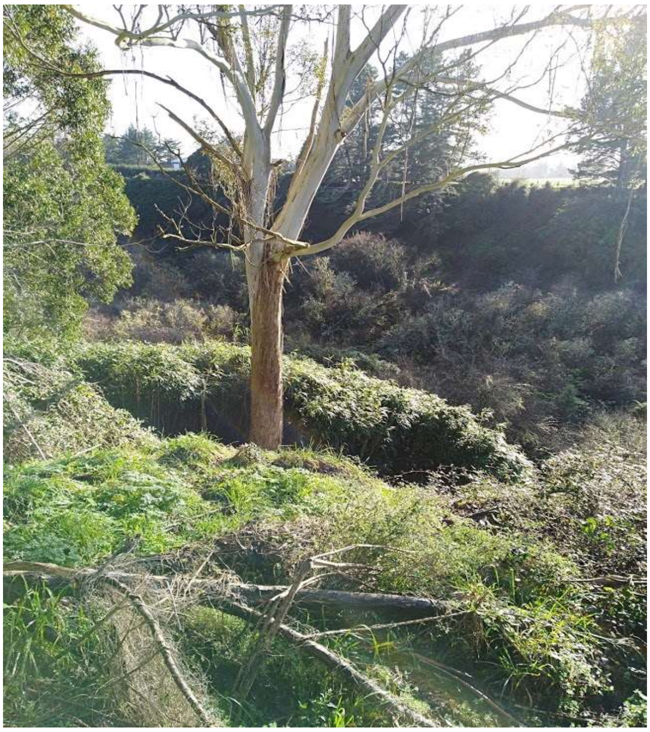


Photo 3: View East Across C4 Gully Receiving Environment

Photo 4: View North Along Gully Towards Outlet

Photo 5: Existing Pond Looking South up gully from Cambridge Rd

Photo 6: Pumice Deposits Gully Wall

Photo 7: Submerged Culvert Inlet Under Cambridge Road

Photo 8: Submerged Culvert Outlet Under Cambridge Road

APPENDIX 2 PLAN CHANGE AREA

READ DRAWINGS CAREFULLY #MEASURETWICECUTONCE

DRAWING SCALE BAR (IN MILLIMETRES)

LEGEND

- 🟥 GROWTH CELL EXTENTS
- ─ CADASTRAL BOUNDARIES

NOTES:

1. CADASTRAL BOUNDARIES SOURED FROM LINZ DATA SERVICE (AUGUST 2019)
2. AERIAL PHOTO SOURCED FROM WAIKATO REGIONAL COUNCIL DATA SERVICE (AUGUST 2019)

A	FOR INFORMATION	GCJ	23.09.19
REV	DESCRIPTION	BY	DATE

APPROVAL STATUS
FOR INFORMATION

DESIGNED BY L.MCCAFFREY	DATE 23.09.19
DRAWN BY G.JONES	DATE 23.09.19
APPROVED BY	DATE

CLIENT NAME
Waipa
DISTRICT COUNCIL

PROJECT NAME
C4 STRUCTURE PLAN

DRAWING TITLE
SITE PLAN

PRODUCED BY
TE MIRO. WATER

DATE PRINTED: 25 September 2019

DRAWING SCALE
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REVISION No.
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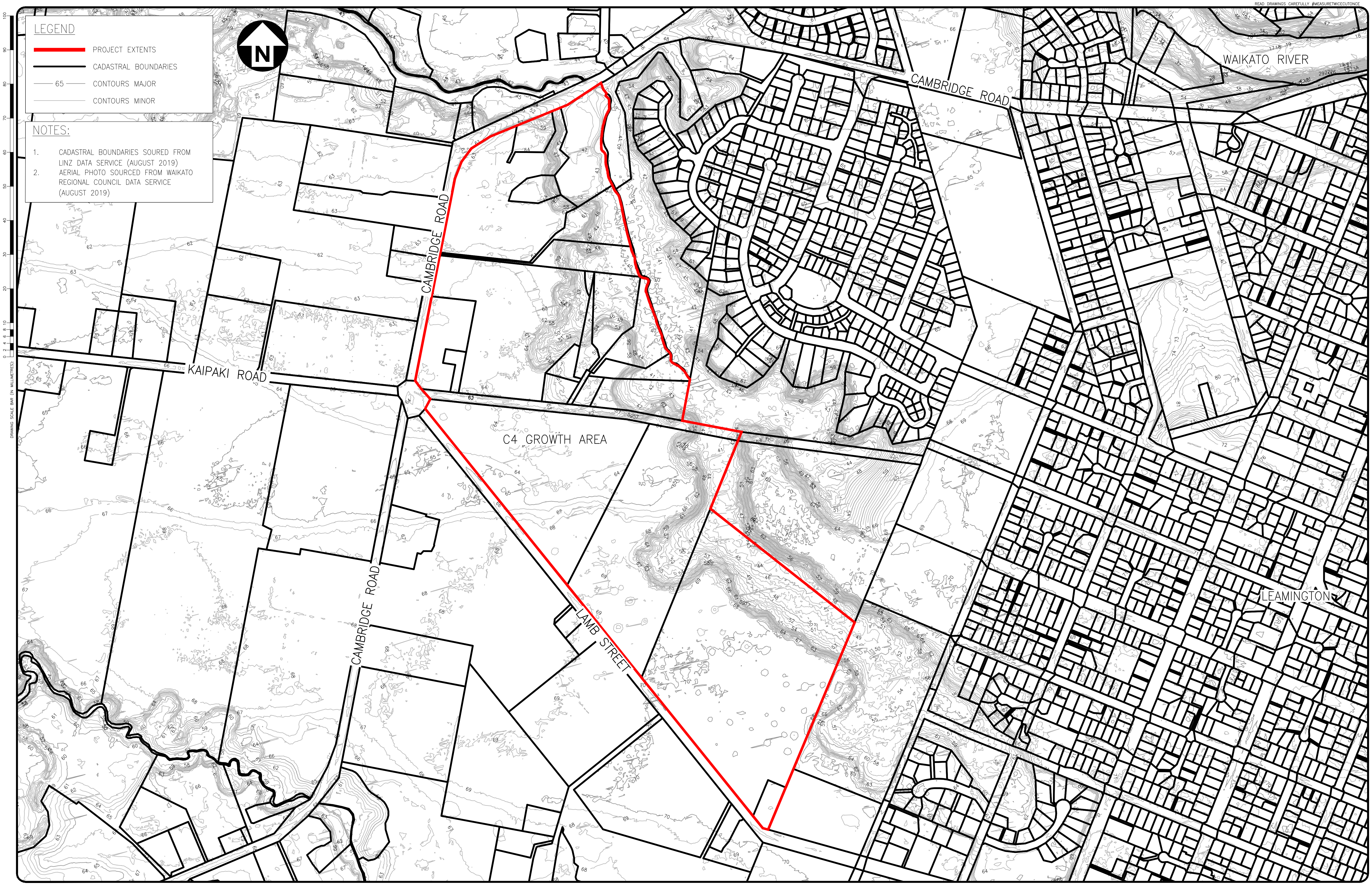
DISCIPLINE
CIVIL ENGINEERING

DRAWING No.
19014–SK–001

FILE NAME: 19014–SK–001.dwg

APPENDIX 3 EXISTING CONTOUR LEVELS

READ DRAWINGS CAREFULLY #MEASURETWICECUTONCE



LEGEND

- PROJECT EXTENTS
- CADASTRAL BOUNDARIES
- 65 CONTOURS MAJOR
- CONTOURS MINOR

NOTES:

1. CADASTRAL BOUNDARIES SOURED FROM LINZ DATA SERVICE (AUGUST 2019)
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REV	DESCRIPTION	BY	DATE

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PROJECT NAME
C4 STRUCTURE PLAN

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EXISITING CONTOURS PLAN

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TE MIRO. WATER

DATE PRINTED: 25 September 2019

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DISCIPLINE CIVIL ENGINEERING
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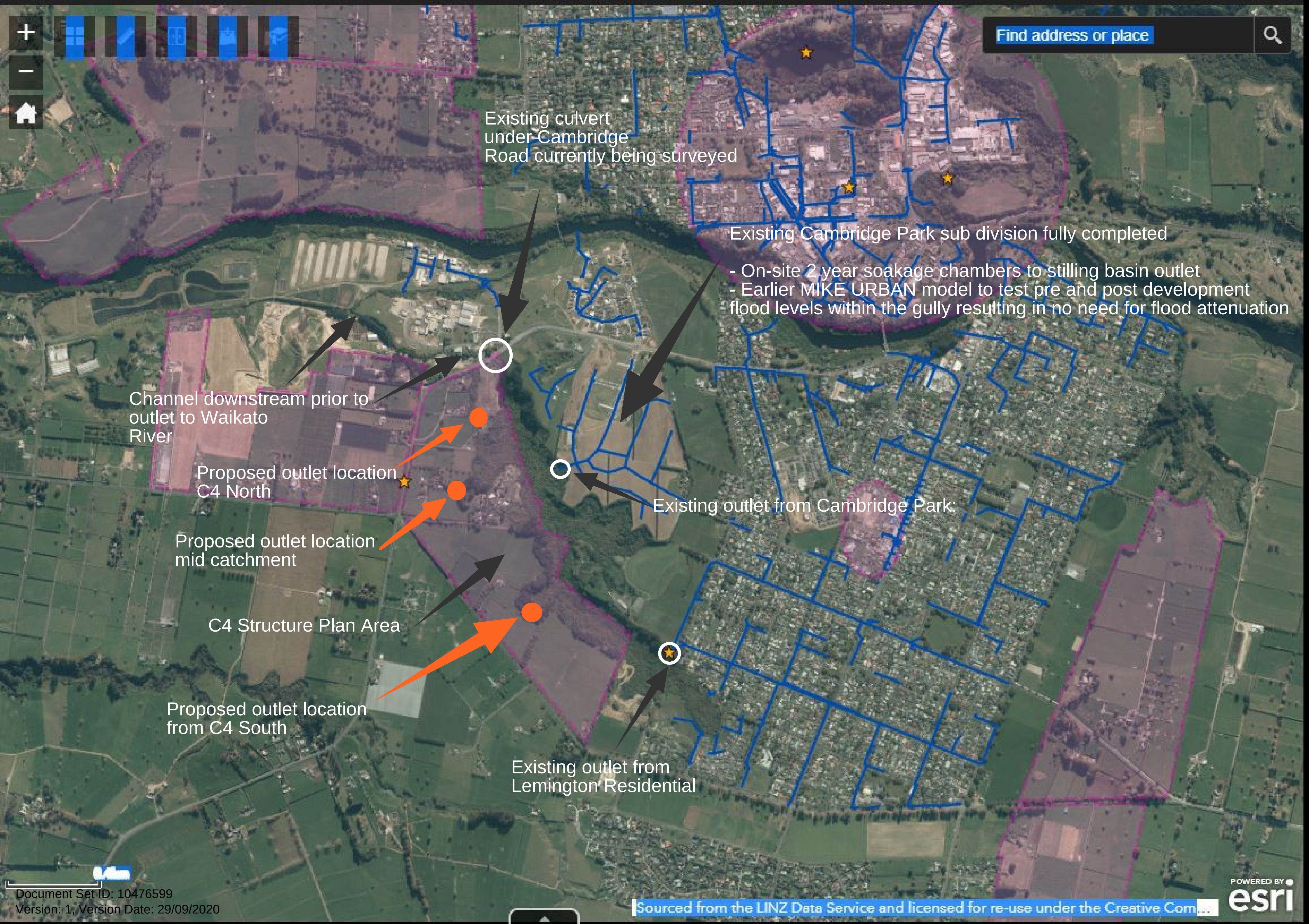
APPENDIX 4 INDICATIVE STORMWATER PLAN



Existing culvert under Cambridge Road currently being surveyed

Existing Cambridge Park sub division fully completed

- On-site 2 year soakage chambers to stilling basin outlet
- Earlier MIKE URBAN model to test pre and post development flood levels within the gully resulting in no need for flood attenuation

Channel downstream prior to outlet to Waikato River

Proposed outlet location C4 North

Existing outlet from Cambridge Park

Proposed outlet location mid catchment

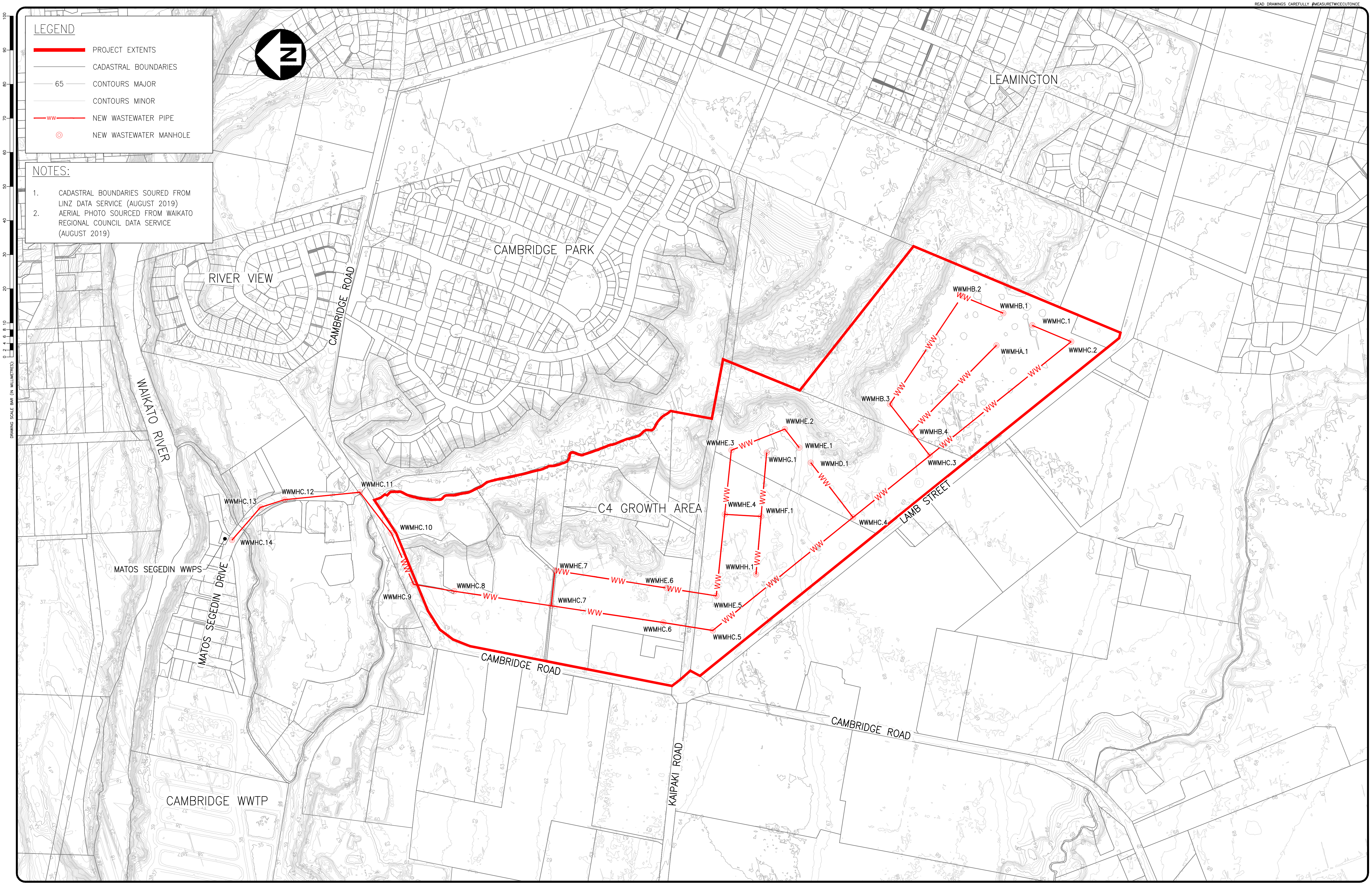
C4 Structure Plan Area

Proposed outlet location from C4 South

Existing outlet from Lemington Residential

APPENDIX 5 WATER AND WASTEWATER PLANS

READ DRAWINGS CAREFULLY #MEASURETWICECUTONCE



LEGEND

▬	PROJECT EXTENTS
—	CADASTRAL BOUNDARIES
65	CONTOURS MAJOR
—	CONTOURS MINOR
—WW—	NEW WASTEWATER PIPE
◎	NEW WASTEWATER MANHOLE

NOTES:

1. CADASTRAL BOUNDARIES SOURED FROM LINZ DATA SERVICE (AUGUST 2019)
2. AERIAL PHOTO SOURCED FROM WAIKATO REGIONAL COUNCIL DATA SERVICE (AUGUST 2019)

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DESIGNED BY M.FARRELL DATE DD.MM.YY
DRAWN BY AXZ DATE 24.09.19
APPROVED BY TBC DATE DD.MM.YY

CLIENT NAME
Waipa
DISTRICT COUNCIL

PROJECT NAME
C4 STRUCTURE PLAN

DRAWING TITLE
WASTEWATER NETWORK
DRAFT PLAN
OPTION A

PRODUCED BY
HG

DATE PRINTED: 3 October 2019

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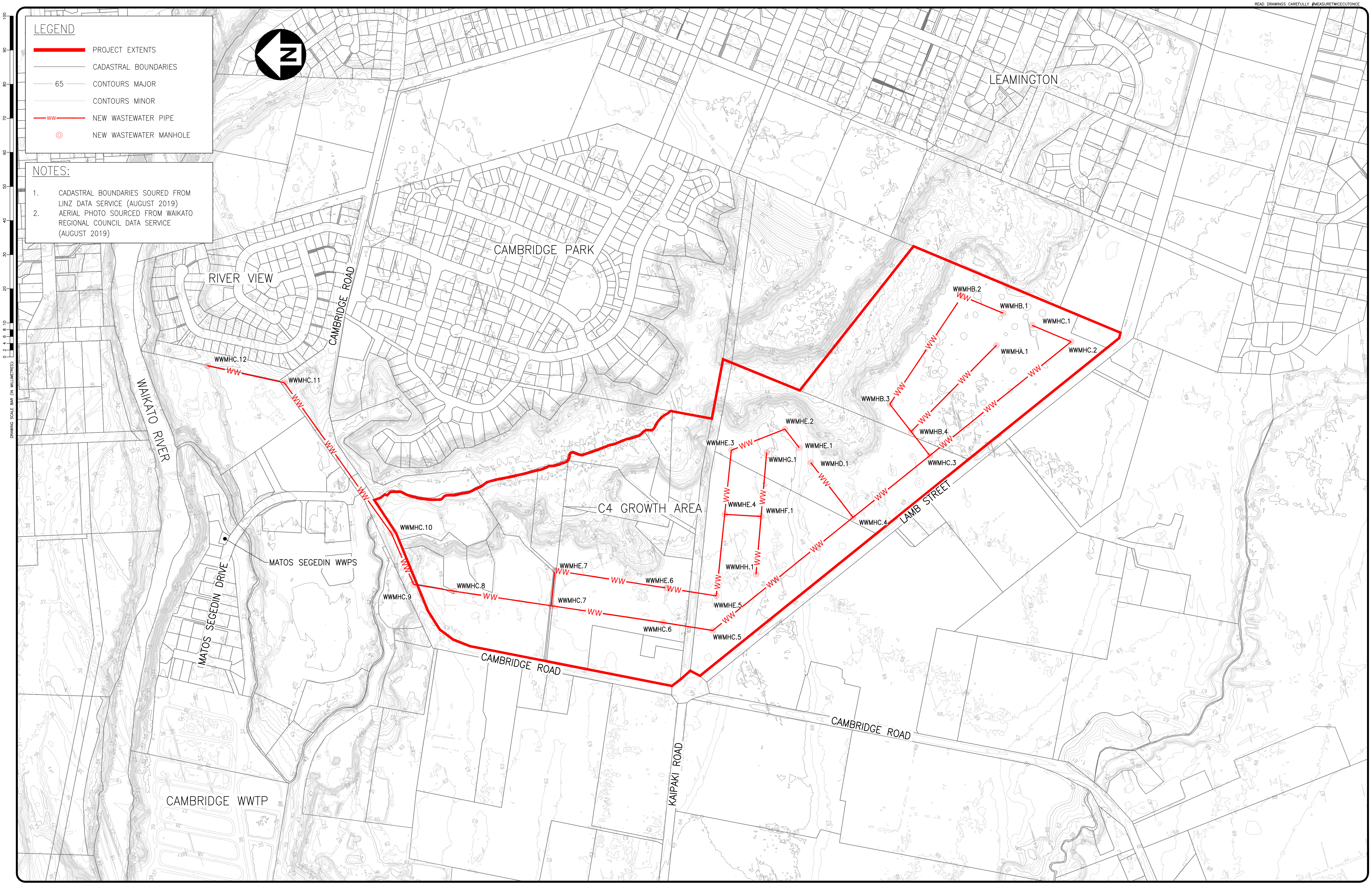
REVISION No. A
DISCIPLINE CIVIL ENGINEERING
DRAWING No. 19014–SK–501

FILE NAME: 19014–SK–501.dwg

READ DRAWINGS CAREFULLY #MEASURETWICECUTONCE



LEGEND

🟥	PROJECT EXTENTS
—	CADASTRAL BOUNDARIES
65	CONTOURS MAJOR
—	CONTOURS MINOR
—WW—	NEW WASTEWATER PIPE
◎	NEW WASTEWATER MANHOLE

NOTES:

1. CADASTRAL BOUNDARIES SOURED FROM LINZ DATA SERVICE (AUGUST 2019)
2. AERIAL PHOTO SOURCED FROM WAIKATO REGIONAL COUNCIL DATA SERVICE (AUGUST 2019)

A	FOR INFORMATION	AXZ	24.09.19
REV	DESCRIPTION	BY	DATE

APPROVAL STATUS
FOR INFORMATION

DESIGNED BY
M.FARRELL
DATE
DD.MM.YY

DRAWN BY
AXZ
DATE
24.09.19

APPROVED BY
TBC
DATE
DD.MM.YY

CLIENT NAME
Waipa
DISTRICT COUNCIL

PROJECT NAME
C4 STRUCTURE PLAN

DRAWING TITLE
WASTEWATER NETWORK
DRAFT PLAN
OPTION B

PRODUCED BY
HG

DATE PRINTED: 3 October 2019

DRAWING SCALE
1:7500

REVISION No.
A

DISCIPLINE
CIVIL ENGINEERING

DRAWING No.
19014-SK-502

FILE NAME: 19014-SK-502.dwg

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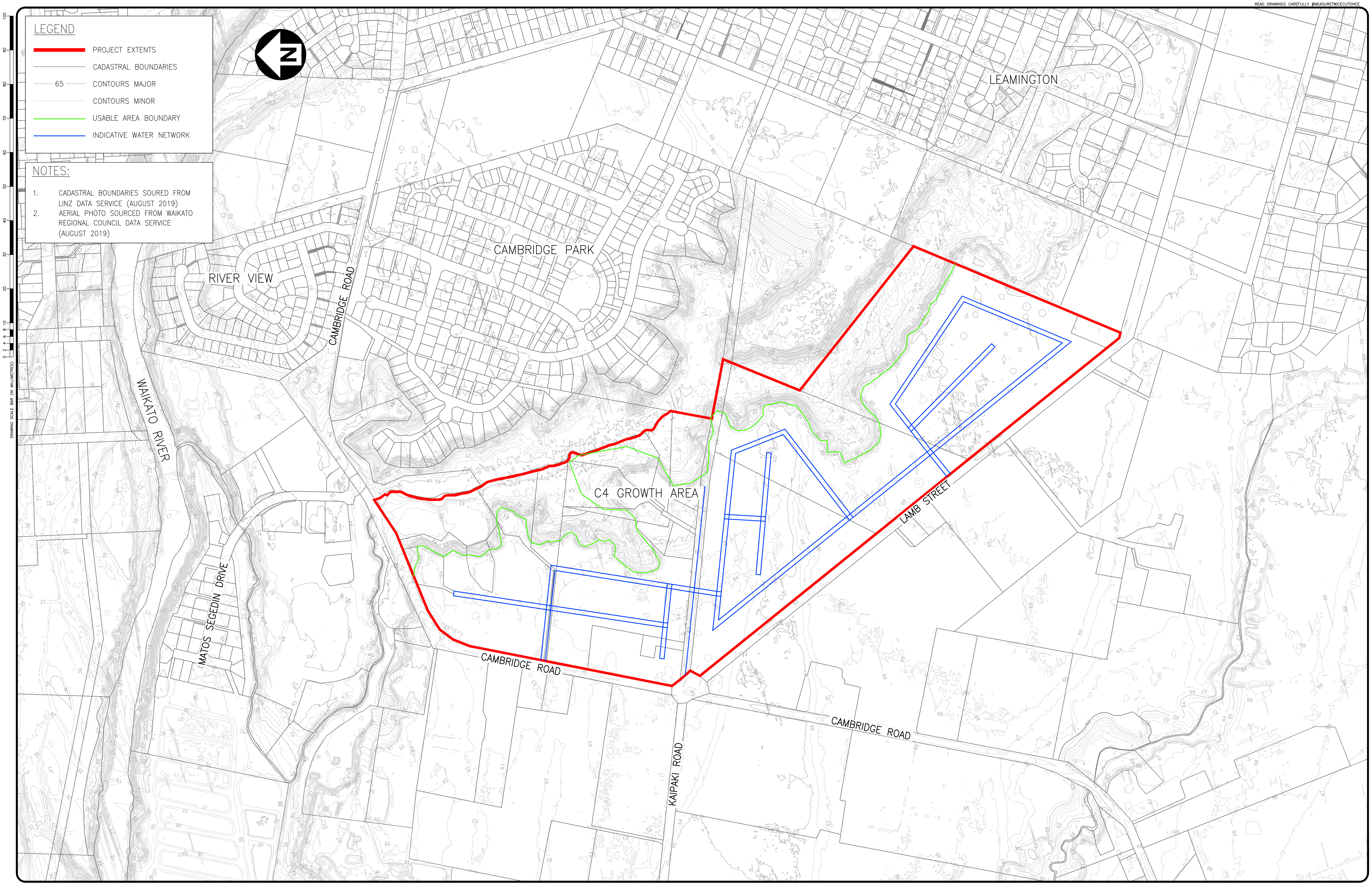

LEGEND

🟥	PROJECT EXTENTS
—	CADASTRAL BOUNDARIES
65	CONTOURS MAJOR
—	CONTOURS MINOR
🟩	USABLE AREA BOUNDARY
🟦	INDICATIVE WATER NETWORK

NOTES:

1. CADASTRAL BOUNDARIES SOURED FROM LINZ DATA SERVICE (AUGUST 2019)
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DRAWN BY AXZ	DATE 24.09.19
APPROVED BY TBC	DATE DD.MM.YY

CLIENT NAME
Waipa
DISTRICT COUNCIL

PROJECT NAME
C4 STRUCTURE PLAN

DRAWING TITLE
WATER SUPPLY NETWORK
DRAFT PLAN

PRODUCED BY
HG
DATE PRINTED: 3 October 2019

DRAWING SCALE
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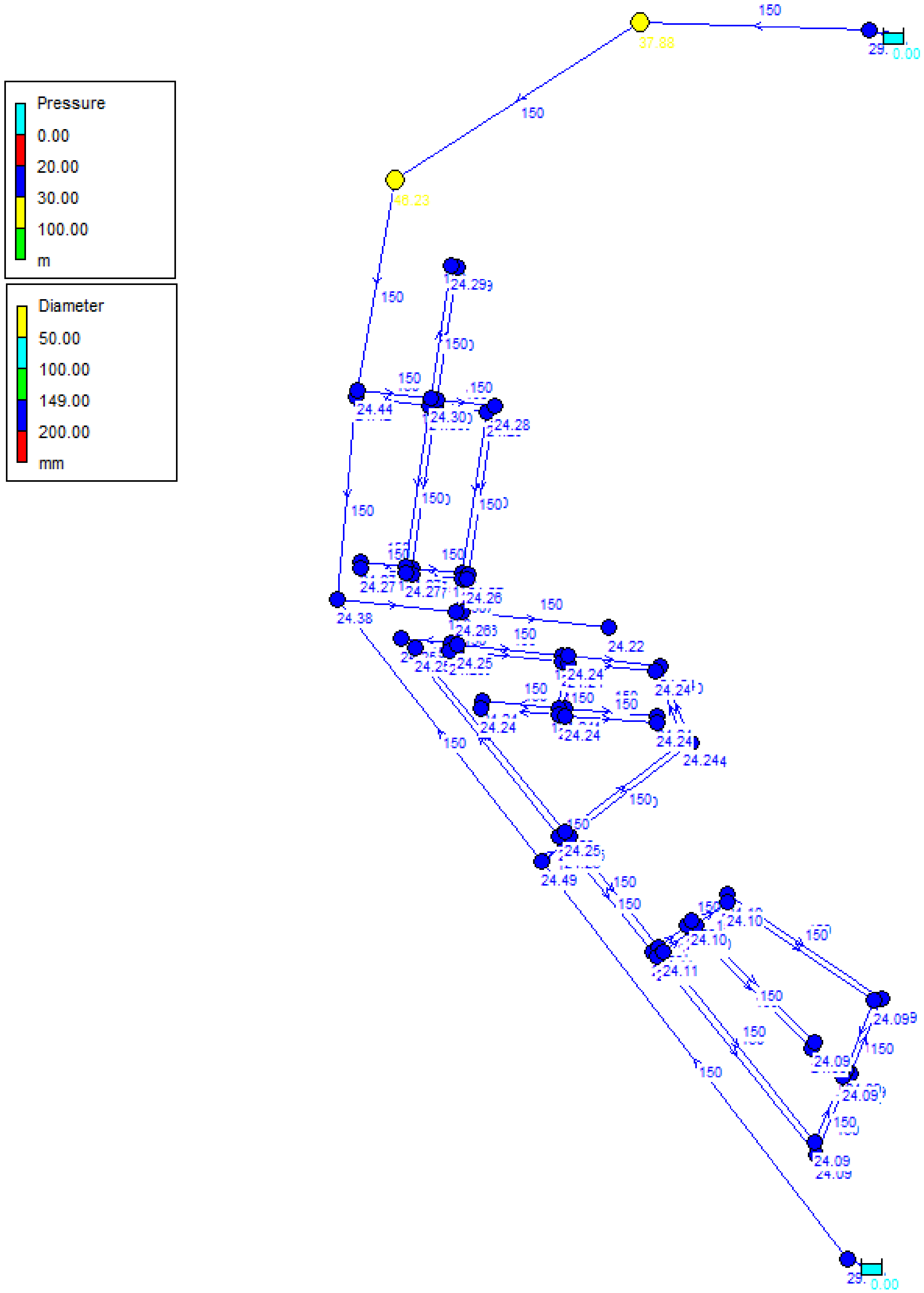
REVISION No.
A

DISCIPLINE
CIVIL ENGINEERING

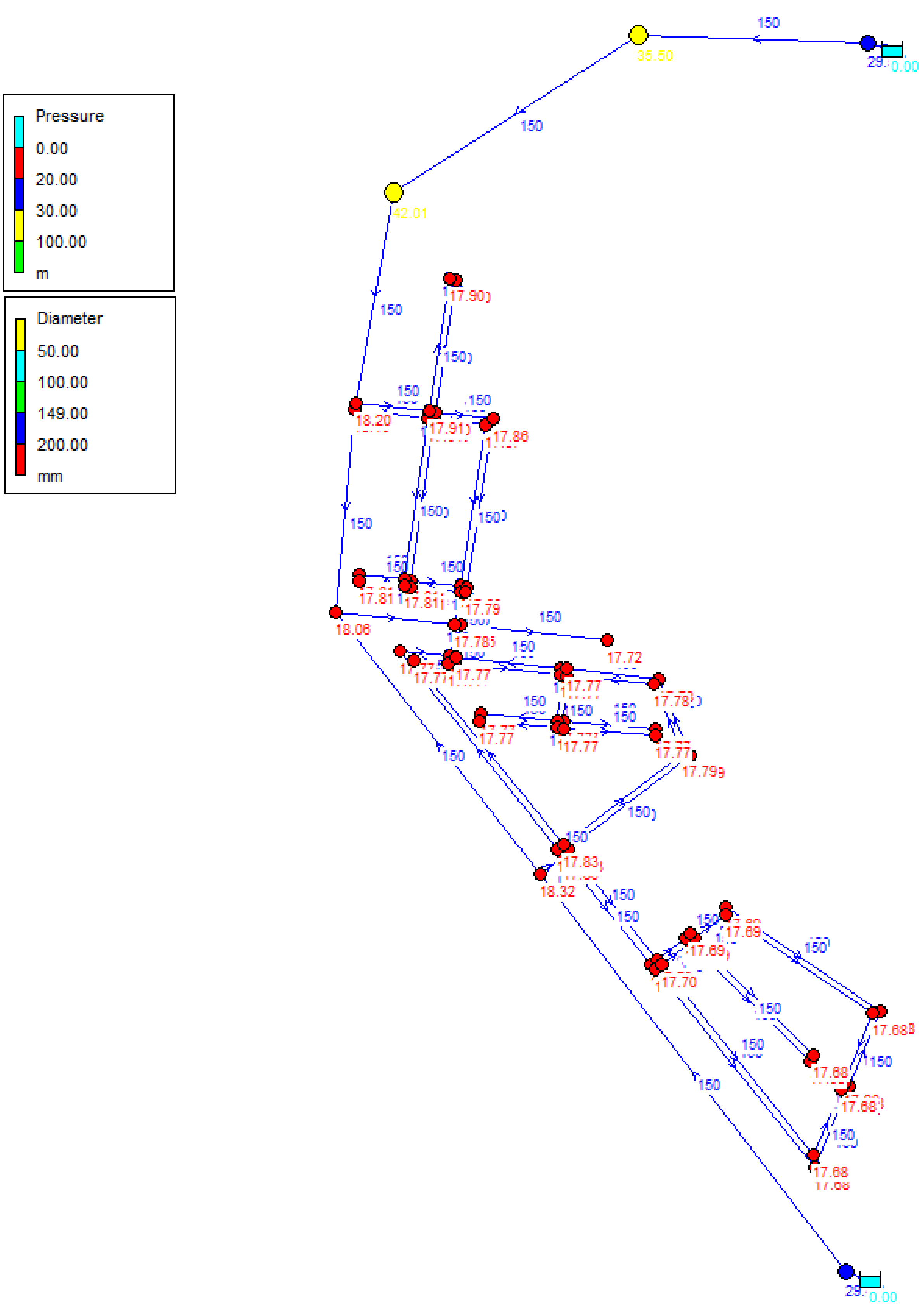
DRAWING No.
19014–SK–600

FILE NAME: 19014–SK–600.dwg

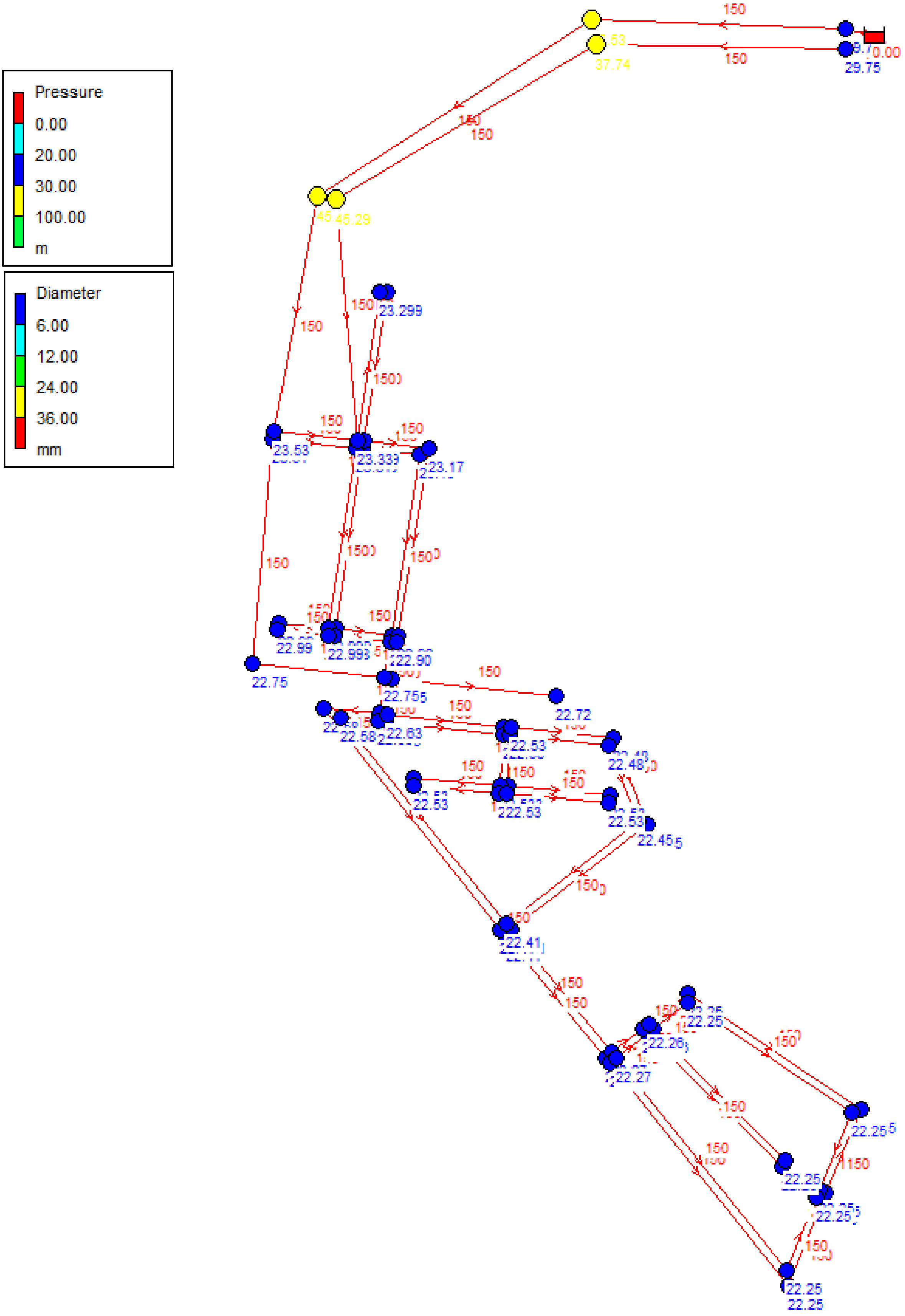
SINGLE NORTH AND SOUTH 150MM SUPPLY (Peak Flow)



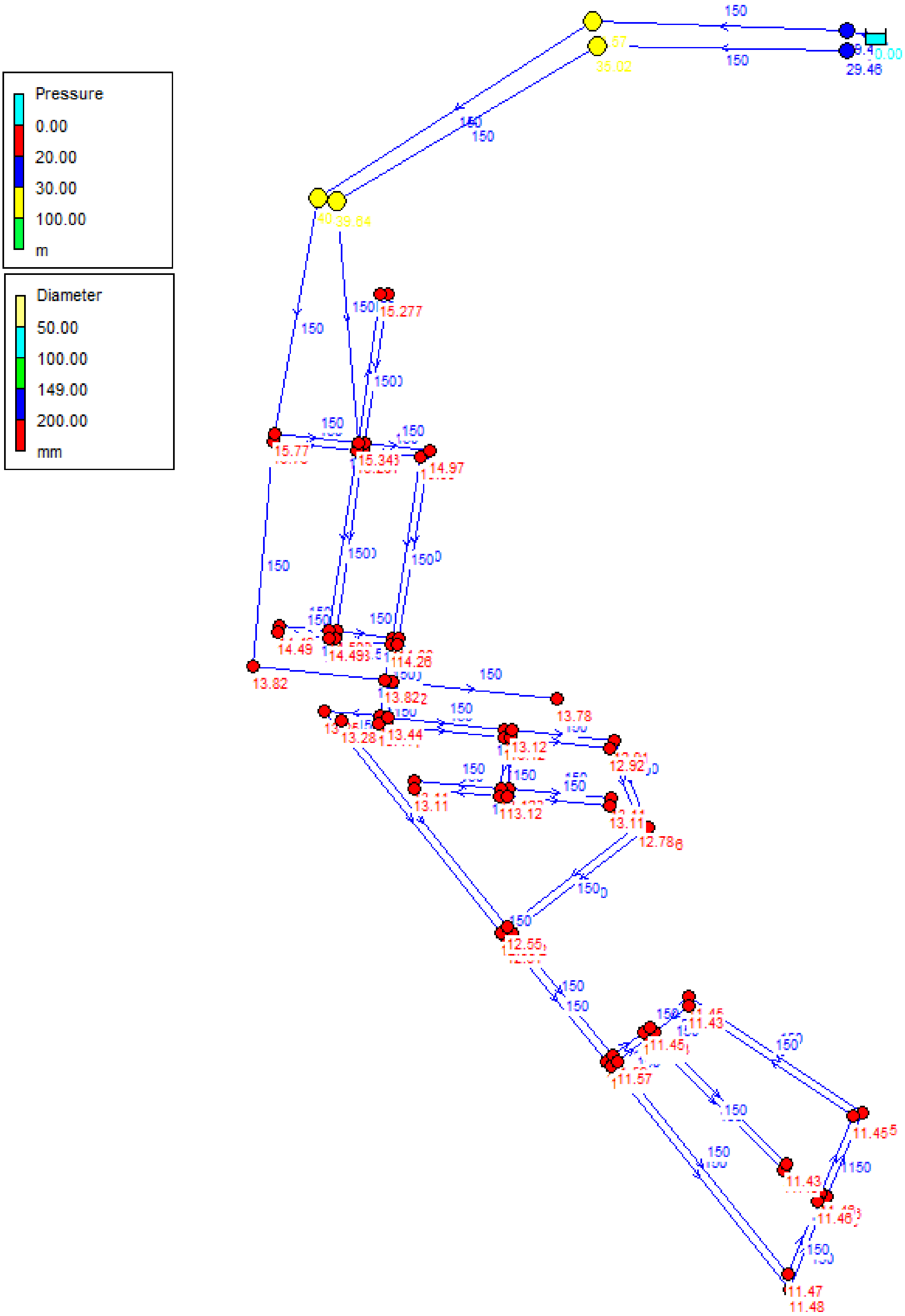
SINGLE NORTH AND SOUTH 150MM SUPPLY (Fire Flow)



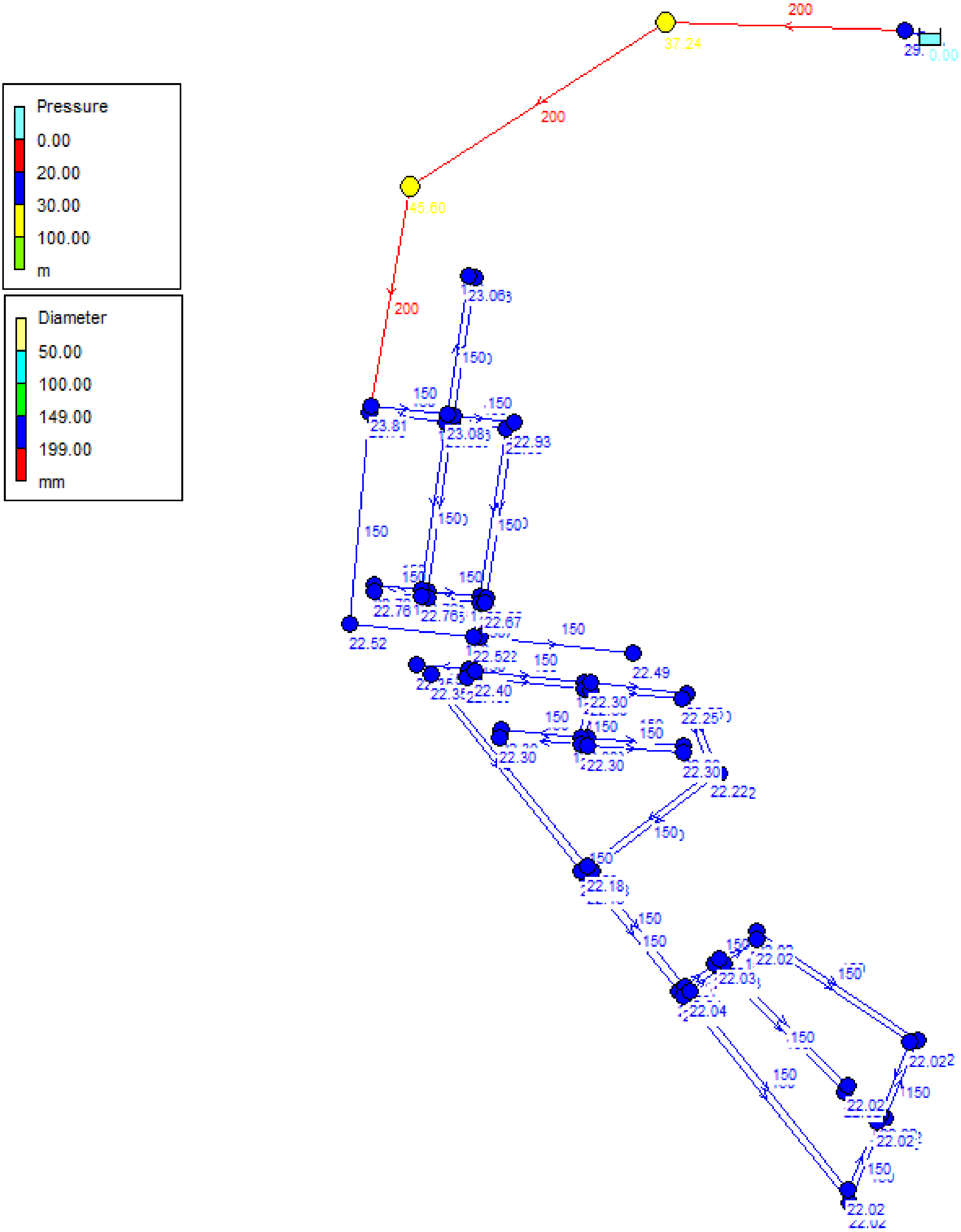
DOUBLE 150MM SUPPLY (Peak Flow)



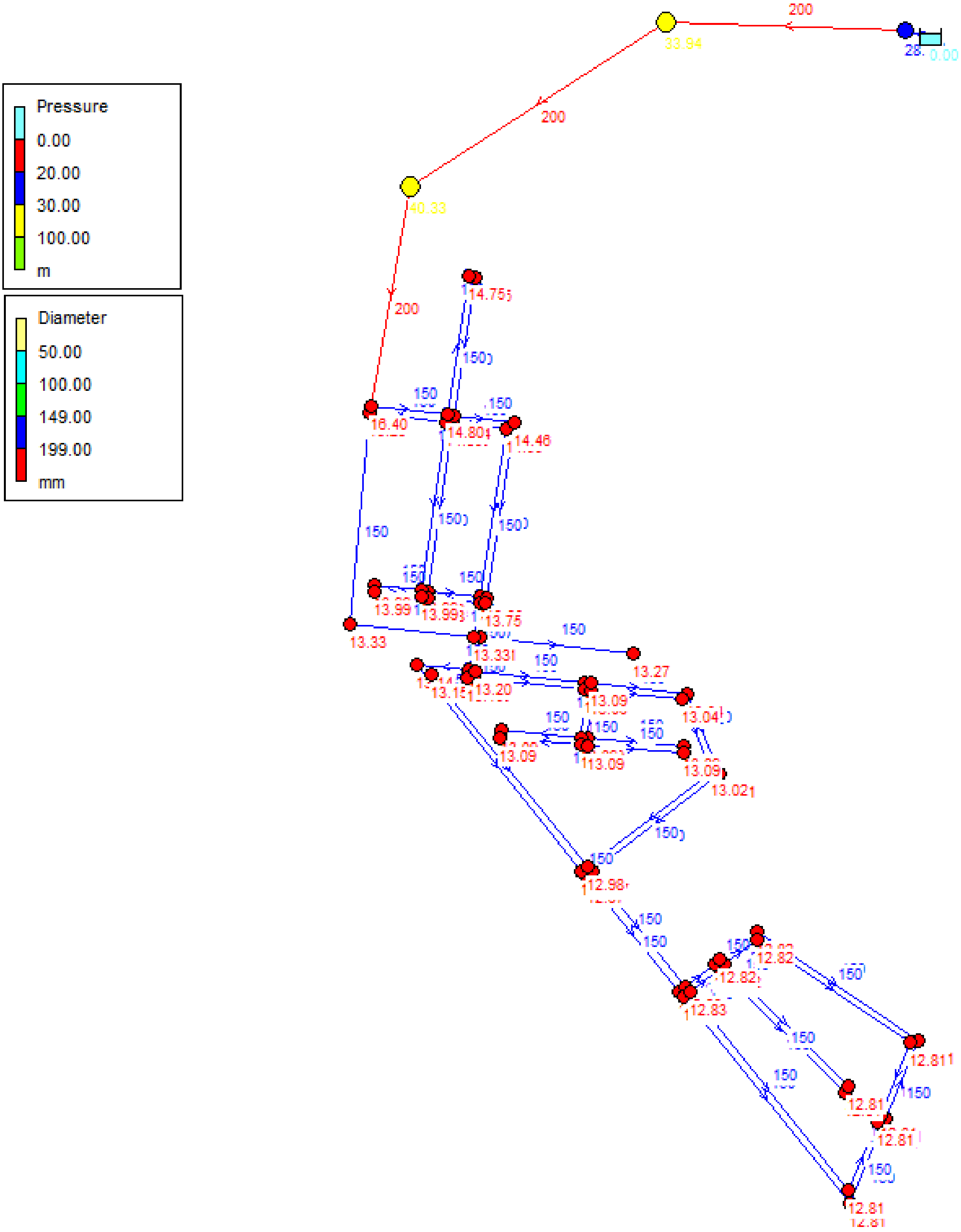
DOUBLE 150MM SUPPLY (Fire Flow)



SINGLE 200MM DIAMETER SUPPLY (Peak Flow)



SINGLE 200MM DIAMETER SUPPLY (Fire Flow)



APPENDIX 6 FLOOD MODEL BUILD

Technical Memo

C4 – MASTER PLANNING, CAMBRIDGE

Flood Risk Assessment

TO:	Mike Chapman – Te Miro Water Ltd	**HG PROJECT NO :**	1610-146182-01
FROM:	Saeed Ghavidelfar; Mona Liao	**DATE:**	20 December 2019

1.0 EXECUTIVE SUMMARY

Harrison Grierson was commissioned by Te Miro Water Ltd to carry out a flood risk assessment for 3 Waters Master Planning of C4 Growth Cell, located at the south western boundary of the Cambridge town (Figure1).

This flood risk assessment aims to inform Waipa District Council whether it is needed to undertake post development flow attenuation as a design consideration for this area. In this way, a coupled 1D-2D MIKE FLOOD model was developed to evaluate the flood impact of the C4 development site.

The results of the assessment showed that

- The C4 development may not have any adverse impact on the downstream since the expansive gully system adjacent to the growth area, which is owned by WDC and will be the receiving environment for the development, provides a natural storage area.
- The increase of water level in the gully and the maximum flow through the culvert downstream of the gully is marginal.
- There is no need to undertake a post development flow attenuation for C4 growth cell.

Fig 1. Cambridge Future Growth Cells (Future Growth Waipa 2050)

2.0 HYDRAULIC MODEL BUILD

A coupled 1D/2D DHI MIKE FLOOD model was developed for pre-development and post-development scenarios to assess the flood impact of residential development at C4 growth cell. For this assessment, the catchment was modelled in MIKE URBAN, while a river reach along with a culvert downstream of the gully was modelled in MIKE 11. These two models were coupled with a MIKE 21 model, representing the 2D surface, in MIKE FLOOD in order to present a fully coupled model which is capable of showing the changes of water level and flow across the catchment due to the changes of land use at the C4 growth cell.

To develop the model, initially an overland flow path analysis was carried to understand the full extent of the catchment. Then, the subcatchments through the area were delineated based on the OLFP analysis and the existing pipe network. Figure 2 shows the overland flow path, and the delineated subcatchments with the loading points.

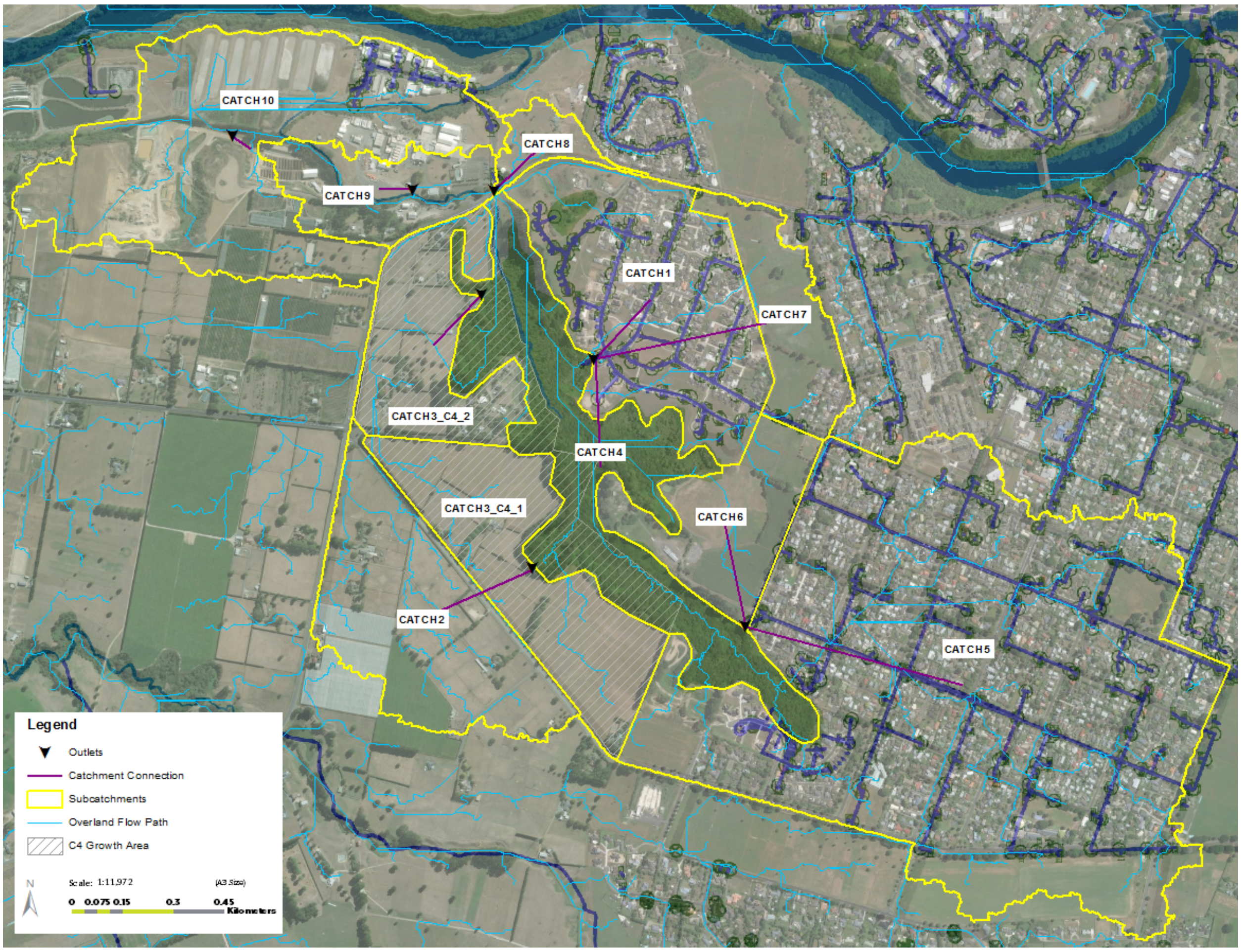


Fig 2. Subcatchments

To calculate stormwater runoff for each subcatchment, the model parameters including rainfall, Curve Number (CN), time of concentration, initial abstraction were estimated based on Waikato stormwater runoff modelling guideline (TR2018/02). The S-MAP soil database and aerials were used to identify the soil type and CN for each subcatchment. In general, the catchment is covered by a well-drained B type soil while in some areas more impervious C type soil is also available. Figure 3 shows the S-MAP soil classification across the site, while Table 1 presents the assigned CN for each soil type.

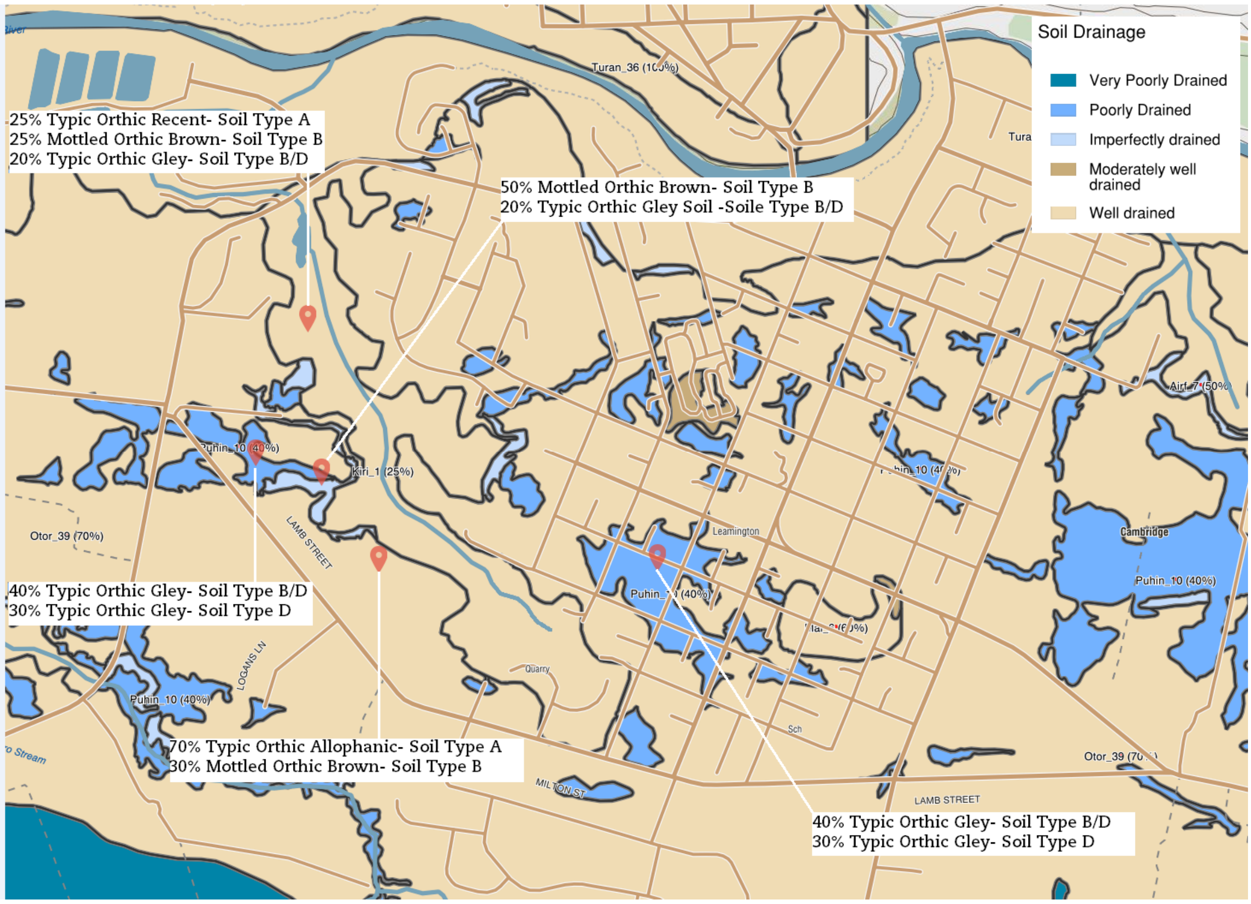


Fig 3. Soil type through the catchment (Source S-MAP)

TABLE 1: ESTIMATED CN

SOIL TYPE	PERVIOUS	IMPERVIOUS
Soil Type B	69	98
Soil Type C	79	98

Table 2 and 3 provide the subcatchment characteristics for pre-development and post-development scenarios, respectively.

TABLE 2: SUBCATCHMENT CHARACTERISTICS (PRE-DEVELOPMENT)

CATCH_ID	SOIL TYPE	IMPERVIOUS AREA (%)	TOTAL AREA (HA)	COMPOSITE CN	COMPOSITE INITIAL ABSTRACTION (MM)	TIME OF CONCENTRATION (HR)
CATCH1	B	60	42.66	86.4	2	0.45
CATCH2	B	10	34.56	71.9	5	0.93
CATCH3_C4_1	B	10	29.67	71.9	5	0.75
CATCH3_C4_2	B	10	21.19	71.9	5	0.75
CATCH4	B	5	42.59	70.45	5.3	0.55
CATCH5	C	60	155.46	90.4	1.3	1.18
CATCH6	B	10	18.84	71.9	5	0.58
CATCH7	B	10	14.18	71.9	5	0.78
CATCH8	B	10	4.50	71.9	5	0.63
CATCH9	B	50	14.76	83.5	2.5	0.36
CATCH10	B	50	61.12	83.5	2.5	0.61

TABLE 3: SUBCATCHMENT CHARACTERISTICS (POST-DEVELOPMENT)

CATCH_ID	SOIL TYPE	IMPERVIOUS AREA (%)	TOTAL AREA (HA)	COMPOSITE CN	COMPOSITE INITIAL ABSTRACTION (MM)	TIME OF CONCENTRATION (HR)
CATCH1	B	60	42.66	86.4	2	0.45
CATCH2	B	10	34.56	71.9	5	0.93
CATCH3_C4_1	B	60	29.67	86.4	2	0.47
CATCH3_C4_2	B	60	21.19	86.4	2	0.47
CATCH4	B	5	42.59	70.45	5.3	0.55
CATCH5	C	60	155.46	90.4	1.3	1.18
CATCH6	B	10	18.84	71.9	5	0.58
CATCH7	B	10	14.18	71.9	5	0.78
CATCH8	B	10	4.50	71.9	5	0.63
CATCH9	B	50	14.76	83.5	2.5	0.36
CATCH10	B	50	61.12	83.5	2.5	0.61

Design 24-hour rainfall depths are derived from HIRDS Version 4 for a 100yr ARI event. Site specific rainfall profile was generated using the alternating block method (Chicago nested rainfall method) based on the HIRDS v4 data. This standard 24-hour temporal rainfall pattern has a peak rainfall intensity at mid-duration while shorter duration rainfall bursts with a range of durations from 10 minutes to 24 hours are nested within the 24-hour temporal pattern.

Climate change is also accounted for in the post-development calculations using RCP 6.0 (2031-2050) as this is considered a medium to high prediction result. The climate change is only applied to the C4 subcatchments in the post-development scenario in order to allow for an accurate flood impact assessment for the development.

Table 4 shows the rainfall depths, while Figures 4 and 5 present 100yr ARI design storm for the existing and the future climate change scenarios.

TABLE 4: 24HR RAINFALL DEPTHS (MM)

RAINFALL EVENT	RAINFALL DEPTH (MM)- EXISTING	RAINFALL DEPTH- CLIMATE CAHNGE RCP 6
100YR	152	161

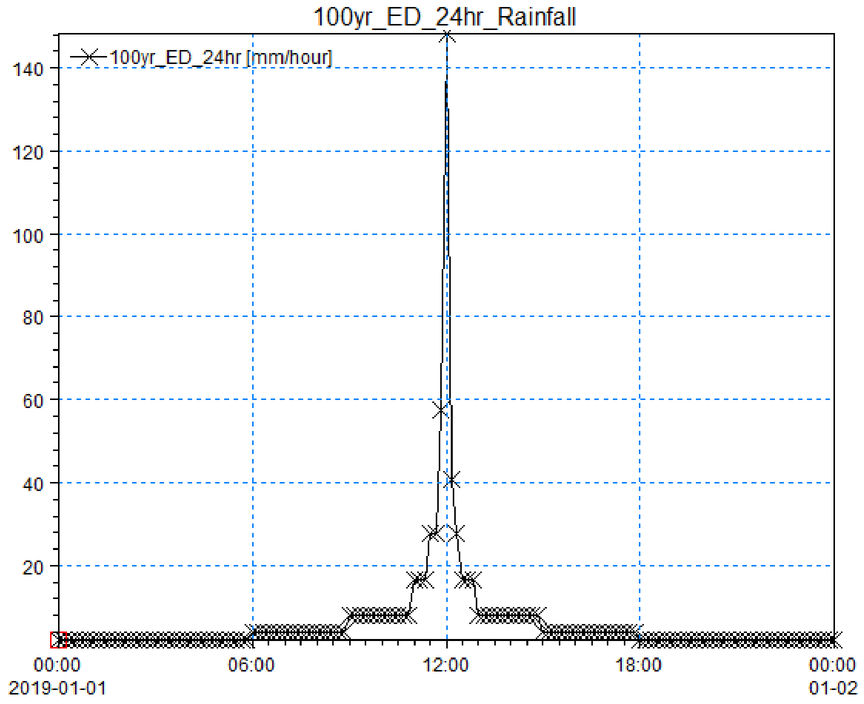


Fig 4. Design storm –existing scenario

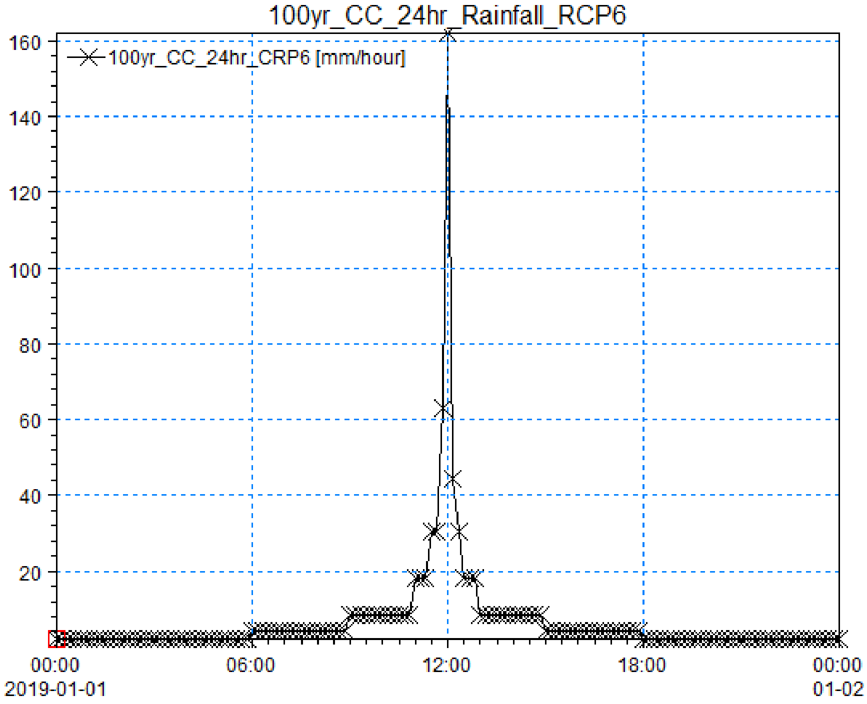


Fig 5. Design storm –climate change scenario

The culvert and a section of stream downstream of the gully, was modelled in MIKE 11 as 1D river reach (Figure 6). The river reach was coupled with MIKE 21 2D Surface in MIKE FLOOD. The culvert dimension and the ground level at two cross sections upstream and downstream of the site were obtained through a site survey. For other cross sections upstream and downstream of the site, the ground level was estimated based on the LiDAR and the survey cross sections.

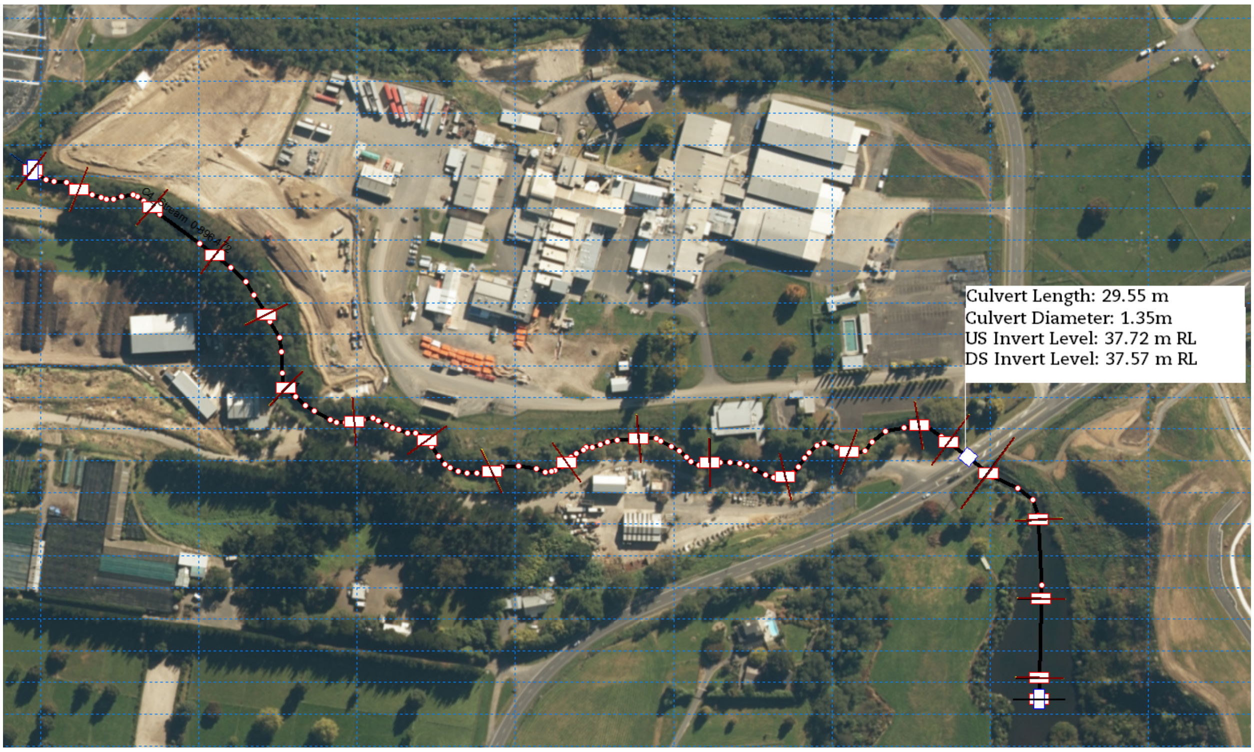


Fig 6. MIKE 11 1D model

LiDAR 2018 was used to generate the 2x2m grid in the MIKE 21 model. The surface roughness was assumed to be 0.05 all across the catchment, while manning roughness for the river reach was 0.03.

Figure 7 presents the coupled 1D/2D model extent in the MIKE FLOOD.

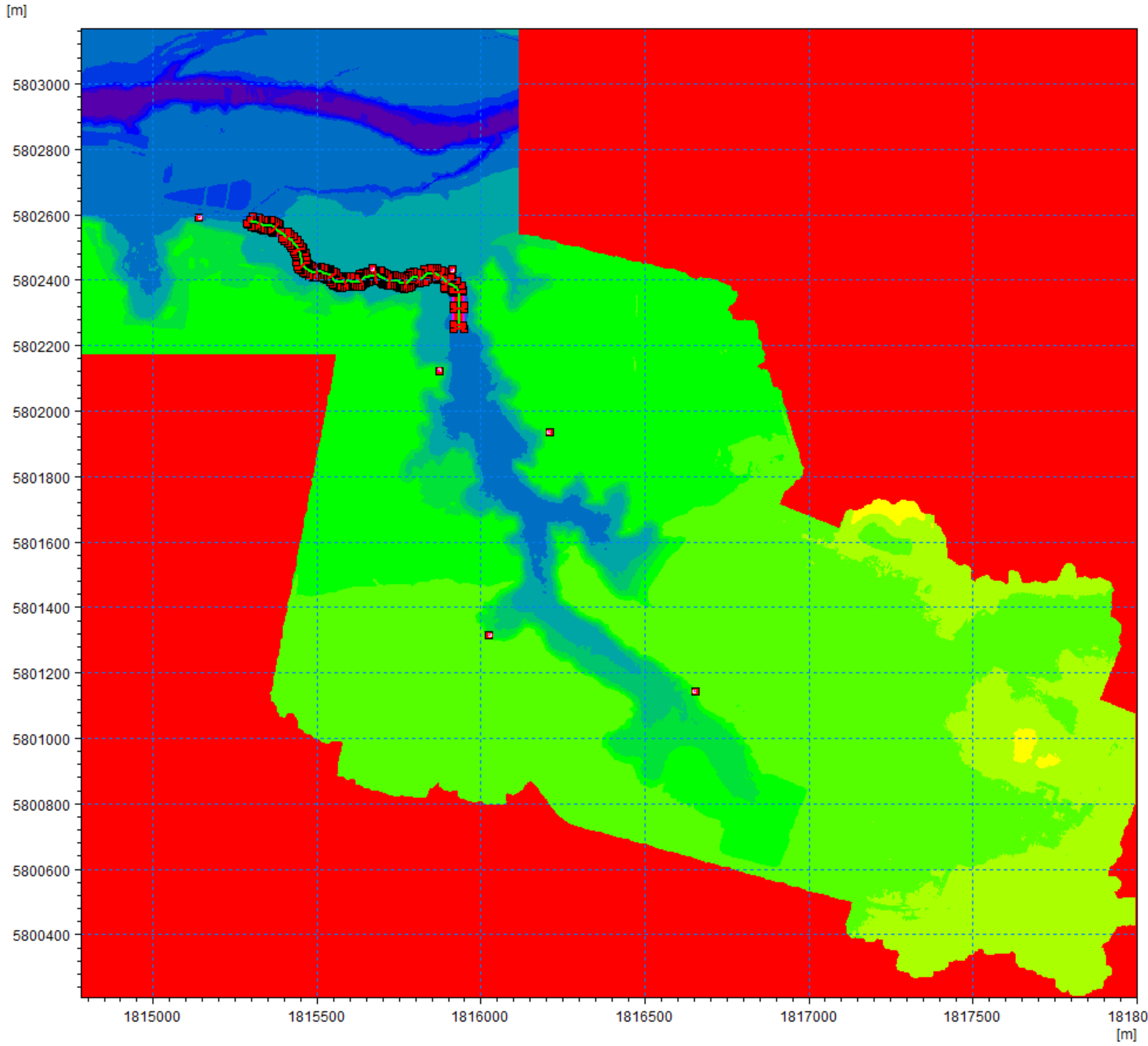


Fig 7. MIKE FLOOD model

3.0 MODELLING RESULTS

The model results are presented in appendix 1.

Map 146182-00-001 presents the comparison of maximum water level and maximum flow across the catchment for the pre-development and post-development scenarios, while Map 146182-00-002 compares the flood extent through the pre- and post-development scenarios.

Comparing the results of pre-development model with the post-development models shows that:

- The development in C4 growth cell may not make any significant adverse impact on the upstream or downstream maximum water level and flood extent, while the maximum water level changes are within the range of 100 mm of pre-development levels both at the upstream and downstream of the site. This is because the expansive gully system adjacent to the growth area provides a large natural storage area.
- Since the post-development max flow and max water level is not significantly higher than the pre-development, there is no need for any flow attenuation through the site.

4.0 CONCLUSIONS

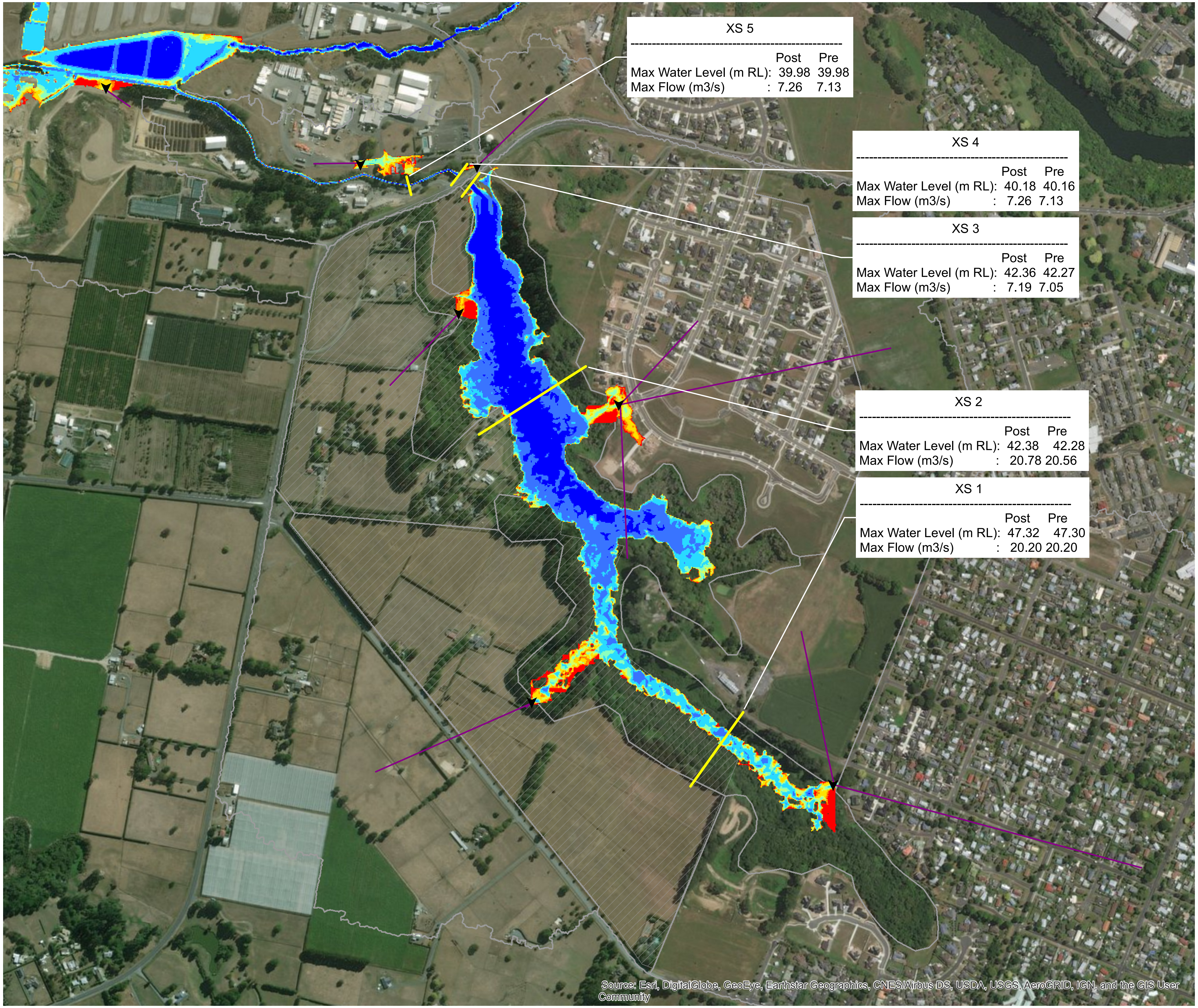
In order to evaluate the flood impact of residential development at the C4 growth cell at Cambridge, a coupled 1D/2D MIKE FLOOD model was developed.

The pre-development and post development models were re-run for the 100yr ARI event. Comparing the post developments results with the pre development showed that

- The residential development at C4 growth cell may not have any major adverse impact in terms of flood level and flood extent on the upstream and downstream of the site. Thus, there is no need to undertake post development flow attenuation.

APPENDICES

APPENDIX 1
MAPS



XS 5

	Post	Pre
Max Water Level (m RL):	39.98	39.98
Max Flow (m3/s) :	7.26	7.13

XS 4

	Post	Pre
Max Water Level (m RL):	40.18	40.16
Max Flow (m3/s) :	7.26	7.13

XS 3

	Post	Pre
Max Water Level (m RL):	42.36	42.27
Max Flow (m3/s) :	7.19	7.05

XS 2

	Post	Pre
Max Water Level (m RL):	42.38	42.28
Max Flow (m3/s) :	20.78	20.56

XS 1

	Post	Pre
Max Water Level (m RL):	47.32	47.30
Max Flow (m3/s) :	20.20	20.20

Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

HARRISON GRIERSON

Legend

- ▼ Outlets
- — Catchment Connection
- — Cross Section

Flood depth (m)

- 🟥 < 0.05
- 🟧 0.05 - 0.1
- 🟨 0.1 - 0.2
- 🟩 0.2 - 0.3
- 🟩 0.3 - 0.5
- 🟦 0.5 - 1
- 🟦 1 - 2
- 🟦 2 - 100
- ☐ Subcatchments
- ▨ C4 Growth Area

EDv9	DRAFT FOR INFORMATION	191212
REV	**ISSUE STATUS**	**DATE**

DESIGNED:	SXG	**DATE:**	12/12/2019
DRAWN:	SXG	**DATE:**	12/12/2019
CHECKED:	MYL	**DATE:**	12/12/2019
APPROVED:	MYL	**DATE:**	12/12/2019

N

Scale: 1:7,500 (A3 Size)

0 0.05 0.1 0.2 0.3 Kilometers

C4 - MASTER PLANNING, CAMBRIDGE
Flood Risk Assessment

Post-Development 100yr ARI Flood Map
CC HIRDSv4 RCP6 (only Site)

MAP NO:

146182_00_001

File Path:\\hgc-trgfp01.harrisongrierson.com\Jobs\1610\146182_01 Cambridge C4\400 Tech\460 Water Resources\02GIS\02MAPS\HG_CambridgeC4_20190930_F3_FloodMapV9.mxd

Last saved by: SXG 2019-12-13 Last Plotted: 2015-11-27


Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

HARRISON GRIERSON

Legend

- ▼ Outlets
- — Catchment Connection
- ▨ Flood Extent, Post-Development
- ■ Flood Extent, Pre-Development
- ☐ Subcatchments
- ▨ C4 Growth Area

ASSOCIATION OF CONSULTING
ENGINEERS NEW ZEALAND

ISO 9001
QUALITY ASSURED

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REV	ISSUE STATUS	DATE
PDv9	DRAFT FOR INFORMATION	191212

DESIGNED:	SXG	**DATE:**	12/12/2019
DRAWN:	SXG	**DATE:**	12/12/2019
CHECKED:	MYL	**DATE:**	12/12/2019
APPROVED:	MYL	**DATE:**	12/12/2019

N

Scale: 1:7,500 (A3 Size)

0 0.05 0.1 0.2 0.3 Kilometers

C4 - MASTER PLANNING, CAMBRIDGE
Flood Risk Assessment

100yr ARI Flood Extent
Post-development Vs. Pre-Development

MAP NO:

146182_00_002

esri

File Path:\\hgc-trgfp01.harrisongrierson.com\Jobs\1610\146182_01 Cambridge C4\400 Tech\460 Water Resources\02GIS\02MAPS\HG_CambridgeC4_20190930_F3_FloodExtentV9.mxd

Last saved by: SXG 2019-12-13 Last Plotted: 2015-11-27

APPENDIX 7 SOAKAGE TESTING

Mark T Mitchell Ltd

Consulting Geotechnical Engineers

1150 Victoria Street
P O Box 9123
Hamilton, New Zealand
Facsimile 07 839 3125
Telephone 07 838 3119
email: mtm@geocon.co.nz

Ref: W – 16064.6
1 November, 2019

Waipa District Council
c/- Mitchell Daysh
Planning Consultants
PO Box 1307
Hamilton 3240

Attention: Abbie Fowler

Dear Madam

Re: Supplementary Site Investigation for Stormwater Disposal Testing
Proposed C4 Growth Cell – Cambridge Road and Lamb Street, Cambridge

In accordance with your request, we have carried out a supplementary site investigation and Stormwater Disposal Testing the above referenced development area. The purpose of our studies was to determine and evaluate the subsurface conditions within the site and assess the feasibility for on-site stormwater disposal within the proposed C4 Growth Cell.

Our associate company, Geocon Geotechnical Ltd, has carried out field testing and calculations which are set out in the attached report, dated 31 October, 2019. This report is a supplement to Section 9: Stormwater Disposal within the Waipa District Council - C4 Growth Cell, Geotechnical Report dated 27 September, 2019 (Ref: W-16064.7).

Yours faithfully

Mark T Mitchell Ltd

Mark T Mitchell
Director

cc. Mike Chapman (Te Miro Water Consultants Ltd)

SUPPLEMENTARY SITE INVESTIGATION
STORMWATER DISPOSAL TESTING

WAIPA DISTRICT COUNCIL C4 GROWTH CELL
CAMBRDIGE ROAD AND LAMB STREET, LEAMINGTON, CAMBRIDGE

The following report is a supplement to Section 9: Stormwater Disposal, within the Waipa District Council C4 Growth Cell, Geotechnical Report dated 27 September, 2019 (Ref: W-16064-7).

1. Test Locations

The following report is based on site conditions as observed during site investigations carried out by our **geologists on 14 and 15 October, 2019. Testing was carried out within the Upper Terrace (Zone 1) in the locations** shown on the attached Site Plan, Drawing No. 16064-20.

The site was investigated by drilling two each, machine auger borings at four locations (eight tests) as shown on the attached Site Plan. The Bore Holes are designated Stormwater Tests A1 to D2, with the Bore Hole log presented on Fig. A-100 to -103.

The purpose of the boring and associated test was to provide guidance as to the general subsurface soil profile within the building site area. Actual ground conditions may vary across the site however, and may differ slightly from those as described below.

2. Field Procedures

The capacity of the site soils to receive concentrated stormwater flows was determined by conducting *insitu* falling head permeability testing within the pre-drilled investigation holes.

Falling Head testing was conducted in accordance with the following general procedure:

1. Pre-drill 85mm-diameter bore hole to design or test depth;
2. Ream out and scarify the bore hole using a 95mm-diameter hand auger so that the sides of the hole are not smeared;
3. Insert and push 65mm-internal diameter, open-ended and slotted PVC pipe to the base of pre-drilled test hole;
4. Pre-soak soils within the test hole by filling the PVC casing and allowing a single cycle of water drainage from the test hole;
5. Refill the test hole and monitor the rate of water level drop over time.

3. Subsurface Conditions

The near surface soils encountered within the stormwater test bore holes revealed 200mm of TOPSOIL overlying SILT (Loam) to between 0.4 to 0.8 metres depth.

The Silt was underlain by gravelly fine to coarse grained SAND to at least the base of the 1.5 to 3.0 metre deep bore holes.

Groundwater was not encountered within the bore holes during the spring site investigation.

4. Test Permeability Test Results

The results of the falling head permeability testing are presented on the attached Tables 1A to 1H with an analysis of the falling head test presented on Figs. E-100 to E-107. The test results are presented on the table below.

Stormwater Test Location	Stormwater Test Depth (Metres)	Hydraulic Conductivity (metres per second)	Hydraulic Conductivity (metres per day)
A1	1.5	2.6×10^{-5}	2.2
A2	2.8	4.5×10^{-6}	0.4
B1	1.5	2.7×10^{-5}	2.4
B2	3.0	1.4×10^{-5}	1.2
C1	1.5	1.3×10^{-5}	1.1
C2	3.0	1.3×10^{-5}	1.1
D1	1.5	2.7×10^{-5}	2.3
D2	3.0	3.3×10^{-5}	2.8

The testing was conducted as per New Zealand standard with the calculation procedure followed in general accordance with widely accepted methods following Hvorslev. The results represent the theoretical soil hydraulic conductivity or ability of that soil medium to transmit water flows under a simulated water level head.

An alternative procedure to determine design soakage rate is presented in the New Zealand Building Code Verification Method E1/VM1 (MBIE, 1992) which involves the selection of a particular gradient on the draw down curve. This procedure is discussed further by Trigger MD (2017) as it generally results in less conservative test soakage test rates and thus substantially smaller systems are designed.

5. Conclusions – Review of Test Results

The results of the testing indicate:

- Five of the six tests revealed consistent hydraulic conductivity (k) with values between 1.1 to 2.8 metres per day.
- The other test (A2 - 3.0 metre deep test) provided inconsistent results. This is likely to be on account of:
  - Heavy rainfall encountered in the days prior to testing
  - Perched water above Silt lenses which are exposed in the gully branch located south of the test site.
  - The possibility of some of the deeper sands being very dense which limited pore space availability.

The results may not be fully representative of the full capacity of the soils and further testing is to be carried out such as with a ring permeater in the base of the proposed stormwater trenches.

Prepared by:

Timothy Dunton
Engineering Geologist

Report Approved by:

Geraint Walters
Operations Manager

Notes:

1. This aerial image is sourced from Google Earth.
2. Contours reproduced from Waikato Regional Council LIDAR data.

Geocon Geotechnical Ltd
Geotechnical Engineers
1150 Victoria Street, P.O. Box 9123, Hamilton

WAIPA DISTRICT COUNCIL

Stormwater Investigation for C4 Growth Cell
Cambridge Road and Lamb Street, Cambridge

STORMWATER SITE PLAN

DRAWING No. 16064-20
DATE October 2019
ISSUE DATE

STORMWATER TEST A-1

SOIL DESCRIPTION

GRAPHIC LOG	SOIL DESCRIPTION	DEPTH (metres)	GEOLOGICAL FORMATION	VANE SHEAR STRENGTH - kPa (In-situ/Remoulded)	SCALA PENETROMETER (blows/100mm) 1 2 3 4 5 6 7 8 9 10	PIEZOMETER / WATER LEVEL
▨	TOPSOIL.		TS			No Groundwater
× × ×	SILT with some fine to coarse sand and minor fine gravel. Moist to wet, light yellowish brown.		Loam			
░	gravelly fine to coarse SAND with trace silt. Wet, light brownish grey.		Hinuera Formation			
░	fine to medium gravelly fine to coarse SAND with trace silt. Wet, grey.	1				
	Bottom of Bore Hole completed 15/10/19					
		2				
		3				

STORMWATER TEST A-2

NOTES - The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

SOIL DESCRIPTION

GRAPHIC LOG	SOIL DESCRIPTION	DEPTH (metres)	GEOLOGICAL FORMATION	VANE SHEAR STRENGTH - kPa (In-situ/Remoulded)	SCALA PENETROMETER (blows/100mm) 1 2 3 4 5 6 7 8 9 10	PIEZOMETER / WATER LEVEL
▨	TOPSOIL.		TS			No Groundwater
× × ×	SILT with some fine to coarse sand and minor fine gravel. Moist to wet, light yellowish brown.		Loam			
░	silty fine to coarse SAND with fine to coarse gravel. Moist to wet, light greyish brown.		Recent Alluvium			
░	fine to coarse gravelly fine to coarse SAND with trace silt. Wet, brownish grey.	1 2	Hinuera Formation			
	Bottom of Bore Hole completed 15/10/19	3				

JOB NAME: WAIPA DISTRICT COUNCIL
JOB LOCATION: Cambridge Road and Lamb Street, Cambridge
JOB NUMBER: W-16064

DRILL METHOD: Machine Auger
RIG: HILUX VANE No.
DRILLER: PS

LOGGED: HZ PLOTTED: PS
DATE LOGGED: 15/10/19
CHECKED:

Geocon Geotechnical Ltd
Geotechnical Engineers
1150 Victoria Street, P.O. Box 9123, Hamilton

BORE HOLE LOG

STORMWATER TEST A-1 & A-2

LOCATION: refer Site Plan RL (m):
SHEET: 1 OF 1 Fig. No. A-100

STORMWATER TEST B-1

SOIL DESCRIPTION

GRAPHIC LOG	SOIL DESCRIPTION	DEPTH (metres)	GEOLOGICAL FORMATION	VANE SHEAR STRENGTH - kPa (In-situ/Remoulded)	SCALA PENETROMETER (blows/100mm) 1 2 3 4 5 6 7 8 9 10	PIEZOMETER / WATER LEVEL
▨	TOPSOIL.		TS			No Groundwater
× × ×	SILT with minor fine to medium sand. Moist, yellowish brown.		Loam			
⠿	fine to medium gravelly fine to coarse SAND with minor silt. Moist, light greyish brown.		Recent Alluvium			
⠿	fine to medium gravelly fine to coarse SAND with minor silt. Moist, brownish grey. Becoming fine to coarse gravelly fine to coarse SAND @ 1.4 metres.	1	Hinuera Formation			
	Bottom of Bore Hole completed 15/10/19	2 3				

STORMWATER TEST B-2

NOTES - The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

SOIL DESCRIPTION

GRAPHIC LOG	SOIL DESCRIPTION	DEPTH (metres)	GEOLOGICAL FORMATION	VANE SHEAR STRENGTH - kPa (In-situ/Remoulded)	SCALA PENETROMETER (blows/100mm) 1 2 3 4 5 6 7 8 9 10	PIEZOMETER / WATER LEVEL
▨	TOPSOIL.		TS			No Groundwater
× × ×	SILT with minor fine to medium sand. Moist, yellowish brown.		Loam			
⠿	silty fine to coarse SAND with some fine gravel. Wet, light greyish brown.		RA			
⠿	fine to medium gravelly fine to coarse SAND with some silt. Wet, light greyish brown. Becoming brownish grey @ 1.4 metres. Containing trace silt @ 1.9 metres.	1 2	Hinuera Formation			
	Bottom of Bore Hole completed 15/10/19	3				

JOB NAME: WAIPA DISTRICT COUNCIL
JOB LOCATION: Cambridge Road and Lamb Street, Cambridge
JOB NUMBER: W-16064

DRILL METHOD: Machine Auger
RIG: HILUX VANE No.
DRILLER: PS

LOGGED: HZ PLOTTED: PS
DATE LOGGED: 15/10/19
CHECKED:

Geocon Geotechnical Ltd
Geotechnical Engineers
1150 Victoria Street, P.O. Box 9123, Hamilton

BORE HOLE LOG

STORMWATER TEST B-1 & B-2

LOCATION: refer Site Plan RL (m):
SHEET: 1 OF 1 Fig. No. A-101

STORMWATER TEST C-1

SOIL DESCRIPTION

GRAPHIC LOG	SOIL DESCRIPTION	DEPTH (metres)	GEOLOGICAL FORMATION	VANE SHEAR STRENGTH - kPa (In-situ/Remoulded)	SCALA PENETROMETER (blows/100mm) 1 2 3 4 5 6 7 8 9 10	PIEZOMETER / WATER LEVEL
	TOPSOIL.		TS			No Groundwater
	SILT with minor fine to medium sand and trace fine gravel. Moist, yellowish brown.		Loam			
	silty fine to medium SAND with trace fine gravel. Moist, yellowish brown.		RA			
	fine to medium gravelly fine to coarse SAND. Moist, greyish brown. Becoming brownish grey @ 1.4 metres.	1	Hinuera Formation			
	Bottom of Bore Hole completed 14/10/19	2 3				

NOTES - The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

STORMWATER TEST C-2

SOIL DESCRIPTION

GRAPHIC LOG	SOIL DESCRIPTION	DEPTH (metres)	GEOLOGICAL FORMATION	VANE SHEAR STRENGTH - kPa (In-situ/Remoulded)	SCALA PENETROMETER (blows/100mm) 1 2 3 4 5 6 7 8 9 10	PIEZOMETER / WATER LEVEL
	TOPSOIL.		TS			No Groundwater
	SILT with minor fine to medium sand and trace fine gravel. Moist, yellowish brown.		Loam			
	silty fine to medium SAND with minor fine gravel. Moist, yellowish brown.		RA			
	fine to medium gravelly fine to coarse SAND with minor silt. Moist to wet, light greyish brown. Becoming wet @ 1.5 metres. " light brownish grey@ 2.0 metres.	1 2	Hinuera Formation			
	Bottom of Bore Hole completed 14/10/19	3				

JOB NAME: WAIPA DISTRICT COUNCIL
JOB LOCATION: Cambridge Road and Lamb Street, Cambridge
JOB NUMBER: W-16064

DRILL METHOD: Machine Auger
RIG: HILUX VANE No.
DRILLER: PS

LOGGED: HZ PLOTTED: PS
DATE LOGGED: 14/10/19
CHECKED:

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BORE HOLE LOG

STORMWATER TEST C-1 & C-2
LOCATION: refer Site Plan RL (m):
SHEET: 1 OF 1 Fig. No. A-102

STORMWATER TEST D-1

SOIL DESCRIPTION

GRAPHIC LOG	SOIL DESCRIPTION	DEPTH (metres)	GEOLOGICAL FORMATION	VANE SHEAR STRENGTH - kPa (In-situ/Remoulded)	SCALA PENETROMETER (blows/100mm) 1 2 3 4 5 6 7 8 9 10	PIEZOMETER / WATER LEVEL
	TOPSOIL.		TS			No Groundwater
	SILT with some fine to coarse sand. Moist, yellowish brown.		Loam			
	fine to coarse SAND with some silt and minor gravel. Moist, light greyish brown.		RA			
	gravelly fine to medium SAND. Moist to wet, light brownish grey.	1	Hinuera Formation			
	Bottom of Bore Hole completed 14/10/19	2 3				

NOTES - The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

STORMWATER TEST D-2

SOIL DESCRIPTION

GRAPHIC LOG	SOIL DESCRIPTION	DEPTH (metres)	GEOLOGICAL FORMATION	VANE SHEAR STRENGTH - kPa (In-situ/Remoulded)	SCALA PENETROMETER (blows/100mm) 1 2 3 4 5 6 7 8 9 10	PIEZOMETER / WATER LEVEL
	TOPSOIL.		TS			No Groundwater
	SILT with minor fine to coarse sand and trace fine gravel. Moist, yellowish brown.		Loam			
	fine to medium gravelly fine to coarse SAND with trace silt. Moist, greyish brown.		RA			
	fine to medium gravelly fine to coarse SAND with trace silt. Moist, light brownish grey. Containing minor silt @ 1.8 metres. Becoming moist to wet @ 2.0 metres. " wet @ 2.6 metres	1 2	Hinuera Formation			
	Bottom of Bore Hole completed 14/10/19	3				

JOB NAME: WAIPA DISTRICT COUNCIL
JOB LOCATION: Cambridge Road and Lamb Street, Cambridge
JOB NUMBER: W-16064

DRILL METHOD: Machine Auger
RIG: HILUX VANE No.
DRILLER: PS

LOGGED: HZ PLOTTED: TD
DATE LOGGED: 14/10/19
CHECKED:

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BORE HOLE LOG

STORMWATER TEST D-1 & D-2

LOCATION: refer Site Plan RL (m):
SHEET: 1 OF 1 Fig. No. A-103

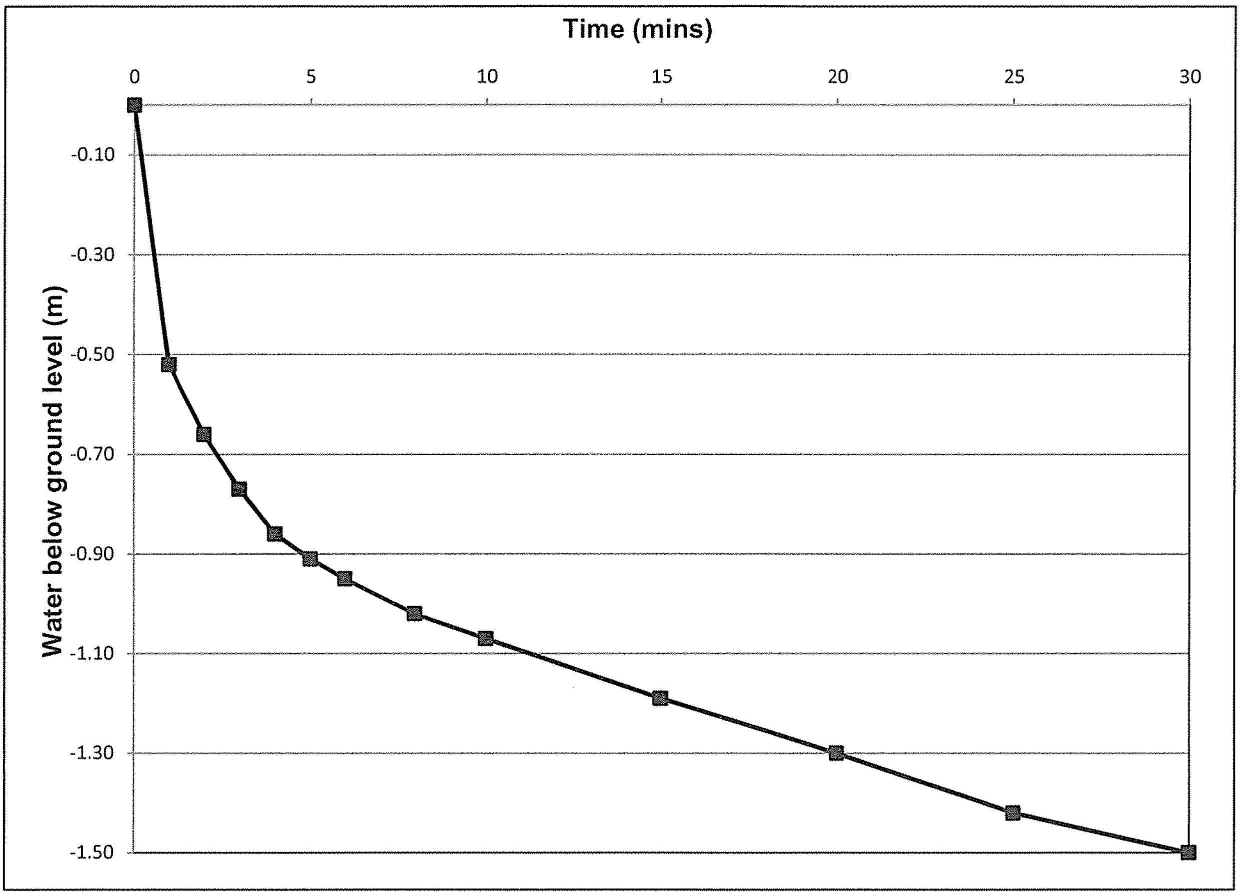
FALLING HEAD SOAKAGE TEST

JOB NO. W-16064

PROJECT: WAIPA DISTRICT COUNCIL

LOCATION: C4 Growth Cell

SOAKAGE TEST A1



Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0	0.62	0.00	0.00	1.50
1	1.14	-0.52	0.52	0.98
2	1.28	-0.66	0.14	0.84
3	1.39	-0.77	0.11	0.73
4	1.48	-0.86	0.09	0.64
5	1.53	-0.91	0.05	0.59
6	1.57	-0.95	0.04	0.55
8	1.64	-1.02	0.07	0.48
10	1.69	-1.07	0.05	0.43
15	1.81	-1.19	0.12	0.31
20	1.92	-1.30	0.11	0.20
25	2.04	-1.42	0.12	0.08
30	2.12	-1.5	0.16	0.00

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FALLING HEAD SOAKAGE TEST RESULTS

Figure No. E-100

DATE: October 2019

CHECKED:

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WAIPA DISTRICT COUNCIL — W-16064

Stormwater Assessment for C4 Growth Cell — Date of test: 15 October, 2019

Field Soakage Test Data

TABLE 1A: FALLING HEAD SOAKAGE TEST RESUL STORMWATER TEST A1

Length of PVC Casing (m)	2.12
Length of PVC Above Ground (m)	0.62
Depth of Soakhole (m)	1.50
Groundwater Level (m)	0.00
Groundwater Level (height above base of Soakhole) (m)	na
Test Hole Diameter (m)	0.095

Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0.0	0.62	0.00	0.00	1.50
1.0	1.14	-0.52	0.52	0.98
2.0	1.28	-0.66	0.14	0.84
3.0	1.39	-0.77	0.11	0.73
4.0	1.48	-0.86	0.09	0.64
5.0	1.53	-0.91	0.05	0.59
6.0	1.57	-0.95	0.04	0.55
8.0	1.64	-1.02	0.07	0.48
10.0	1.69	-1.07	0.05	0.43
15.0	1.81	-1.19	0.12	0.31
20.0	1.92	-1.30	0.11	0.20
25.0	2.04	-1.42	0.12	0.08
30.0	2.12	-1.50	0.08	0.00

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COEFFICIENT OF PERMEABILITY DERIVATION

Use Hvorslev Case 7 (from Kortegast NZGS Vol 16 Issue 1) - hole extended in uniform soil
ie. soakage occurs out the side and base of test hole (slotted) with overlying restrictive layer

PERMEABILITY CALCULATIONS — STORMWATER TEST A1

Shape Factor F = 2 x PI x L / ln{(L/R) + [1 + (L/R)^2]^0.5}

where L = soakage (sand) length (m)
R = test hole radius (m)

Perm coeff. k = A / F x (t2 - t1) x ln h1/h2

where A = test hole flow area
h1 = initial water level
h2 = final water level
t1 = time at h1
t2 = time at h2

Soil Parameters

0.6 m impermeable materal depth
0.2 m permeable materal depth

Elapsed Time (mins)	Water Level head (m)	Av Water Level Head (m) (=H/2)	L (m)	Av. L (m)	F	k (m/sec)
0.0	1.50	1.40	0.20			
1.0	0.98	0.88	0.20	0.20	0.59	9.4E-05
2.0	0.84	0.74	0.20	0.20	0.59	3.5E-05
3.0	0.73	0.63	0.20	0.20	0.59	3.2E-05
4.0	0.64	0.54	0.20	0.20	0.59	3.1E-05
5.0	0.59	0.49	0.20	0.20	0.59	2.0E-05
6.0	0.55	0.45	0.20	0.20	0.59	1.7E-05
8.0	0.48	0.38	0.20	0.20	0.59	1.7E-05
10.0	0.43	0.33	0.20	0.20	0.59	1.4E-05
15.0	0.31	0.21	0.20	0.20	0.59	1.8E-05
20.0	0.20	0.10	0.20	0.20	0.59	3.0E-05
25.0	0.08	0.04	0.08	0.14	0.49	4.4E-05
30.0	0.00	0.00	0.00	0.04	0.33	ERR

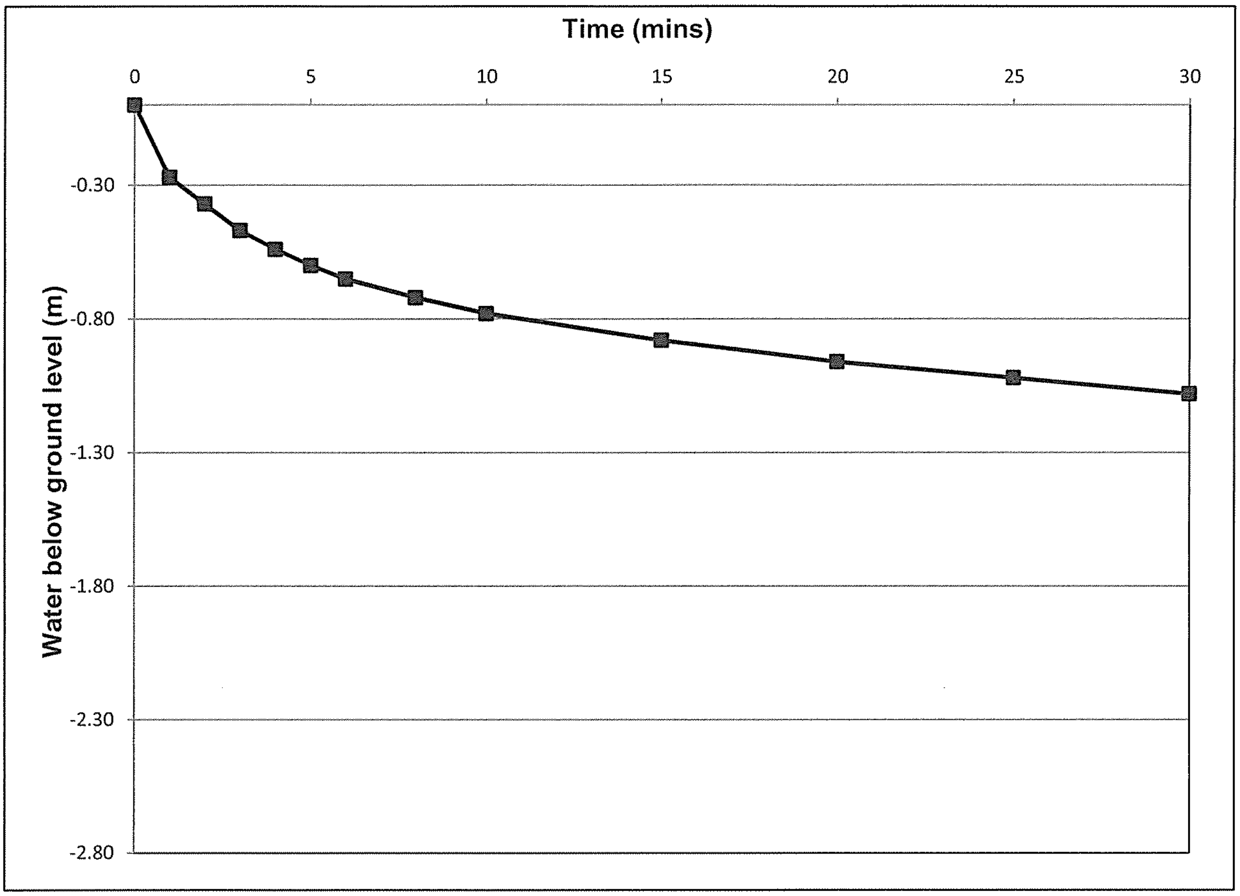
COMPUTED ADJUSTED AVERAGE: 2.6E-05 m/sec
2.2 m/day

Checked:

FALLING HEAD SOAKAGE TEST

JOB NO. W-16064
PROJECT: WAIPA DISTRICT COUNCIL
LOCATION: C4 Growth Cell

SOAKAGE TEST A2



Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0	0.45	0.00	0.00	2.80
1	0.72	-0.27	0.27	2.53
2	0.82	-0.37	0.10	2.43
3	0.92	-0.47	0.10	2.33
4	0.99	-0.54	0.07	2.26
5	1.05	-0.60	0.06	2.20
6	1.10	-0.65	0.05	2.15
8	1.17	-0.72	0.07	2.08
10	1.23	-0.78	0.06	2.02
15	1.33	-0.88	0.10	1.92
20	1.41	-0.96	0.08	1.84
25	1.47	-1.02	0.06	1.78
30	1.53	-1.08	0.06	1.72

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FALLING HEAD SOAKAGE TEST RESULTS

Figure No. E-101
DATE: October 2019
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WAIPA DISTRICT COUNCIL W-16064
Stormwater Assessment for C4 Growth Cell Date of test: 15 October, 2019
Field Soakage Test Data

TABLE 1B: FALLING HEAD SOAKAGE TEST RESUL STORMWATER TEST A2

Length of PVC Casing (m)	3.25
Length of PVC Above Ground (m)	0.45
Depth of Soakhole (m)	2.80
Groundwater Level (m)	0.00
Groundwater Level (height above base of Soakhole) (m)	na
Test Hole Diameter (m)	0.095

Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0.0	0.45	0.00	0.00	2.80
1.0	0.72	-0.27	0.27	2.53
2.0	0.82	-0.37	0.10	2.43
3.0	0.92	-0.47	0.10	2.33
4.0	0.99	-0.54	0.07	2.26
5.0	1.05	-0.60	0.06	2.20
6.0	1.10	-0.65	0.05	2.15
8.0	1.17	-0.72	0.07	2.08
10.0	1.23	-0.78	0.06	2.02
15.0	1.33	-0.88	0.10	1.92
20.0	1.41	-0.96	0.08	1.84
25.0	1.47	-1.02	0.06	1.78
30.0	1.53	-1.08	0.06	1.72

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COEFFICIENT OF PERMEABILITY DERIVATION

Use Hvorslev Case 7 (from Kortegast NZGS Vol 16 Issue 1) - hole extended in uniform soil
ie. soakage occurs out the side and base of test hole (slotted) with overlying restrictive layer

PERMEABILITY CALCULATIONS STORMWATER TEST A2

Shape Factor F = 2 x PI x L / ln{(L/R) + [1 + (L/R)^2]^0.5}

where
L = soakage (sand) length (m)
R = test hole radius (m)

Perm coeff. k = A / F x (t2 - t1) x ln h1 / h2

where
A = test hole flow area
h1 = initial water level
h2 = final water level
t1 = time at h1
t2 = time at h2

Soil Parameters
0.6 m impermeable materal depth
0.2 m permeable materal depth

Elapsed Time (mins)	Water Level head (m)	Av Water Level Head (m) (=H/2)	L (m)	Av. L (m)	F	k (m/sec)
0.0	2.80	2.70	0.20			
1.0	2.53	2.43	0.20	0.20	0.59	2.1E-05
2.0	2.43	2.33	0.20	0.20	0.59	8.5E-06
3.0	2.33	2.23	0.20	0.20	0.59	8.8E-06
4.0	2.26	2.16	0.20	0.20	0.59	6.4E-06
5.0	2.20	2.10	0.20	0.20	0.59	5.7E-06
6.0	2.15	2.05	0.20	0.20	0.59	4.9E-06
8.0	2.08	1.98	0.20	0.20	0.59	3.5E-06
10.0	2.02	1.92	0.20	0.20	0.59	3.1E-06
15.0	1.92	1.82	0.20	0.20	0.59	2.2E-06
20.0	1.84	1.74	0.20	0.20	0.59	1.8E-06
25.0	1.78	1.68	0.20	0.20	0.59	1.4E-06
30.0	1.72	1.62	0.20	0.20	0.59	1.5E-06

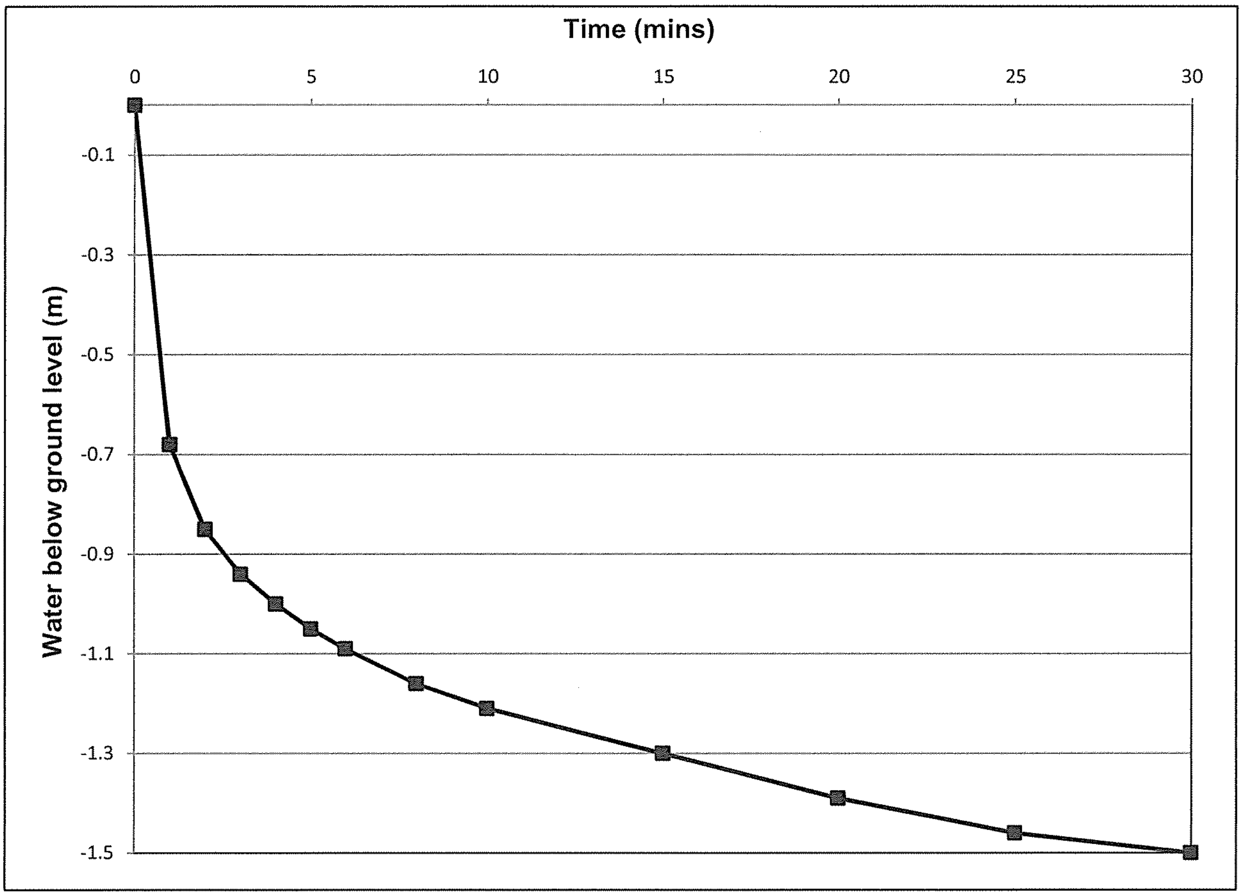
COMPUTED ADJUSTED AVERAGE: 4.5E-06 m/sec
0.4 m/day

Checked:

FALLING HEAD SOAKAGE TEST

JOB NO. W-16064
PROJECT: WAIPA DISTRICT COUNCIL
LOCATION: C4 Growth Cell

SOAKAGE TEST B1



Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0	0.62	0	0	1.5
1	1.3	-0.68	0.68	0.82
2	1.47	-0.85	0.17	0.65
3	1.56	-0.94	0.09	0.56
4	1.62	-1	0.06	0.5
5	1.67	-1.05	0.05	0.45
6	1.71	-1.09	0.04	0.41
8	1.78	-1.16	0.07	0.34
10	1.83	-1.21	0.05	0.29
15	1.92	-1.3	0.09	0.2
20	2.01	-1.39	0.09	0.11
25	2.08	-1.46	0.07	0.04
30	2.12	-1.5	0.04	0

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FALLING HEAD SOAKAGE TEST RESULTS

Figure No. E-102
DATE: October 2019
CHECKED: Gr

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WAIPA DISTRICT COUNCIL W-16064
Stormwater Assessment for C4 Growth Cell
Date of test: 15 October, 2019
Field Soakage Test Data

TABLE 1C: FALLING HEAD SOAKAGE TEST RESUL STORMWATER TEST B1

Length of PVC Casing (m)	2.12
Length of PVC Above Ground (m)	0.62
Depth of Soakhole (m)	1.50
Groundwater Level (m)	0.00
Groundwater Level (height above base of Soakhole) (m)	na
Test Hole Diameter (m)	0.095

Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0.0	0.62	0.00	0.00	1.50
1.0	1.30	-0.68	0.68	0.82
2.0	1.47	-0.85	0.17	0.65
3.0	1.56	-0.94	0.09	0.56
4.0	1.62	-1.00	0.06	0.50
5.0	1.67	-1.05	0.05	0.45
6.0	1.71	-1.09	0.04	0.41
8.0	1.78	-1.16	0.07	0.34
10.0	1.83	-1.21	0.05	0.29
15.0	1.92	-1.30	0.09	0.20
20.0	2.01	-1.39	0.09	0.11
25.0	2.08	-1.46	0.07	0.04
30.0	2.12	-1.50	0.04	0.00

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COEFFICIENT OF PERMEABILITY DERIVATION

Use Hvorslev Case 7 (from Kortegast NZGS Vol 16 Issue 1) - hole extended in uniform soil
ie. soakage occurs out the side and base of test hole (slotted) with overlying restrictive layer

PERMEABILITY CALCULATIONS STORMWATER TEST B1

$$\text{Shape Factor } F = \frac{2 \times PI \times L}{\ln\{(L/R) + [1 + (L/R)^2]^{0.5}\}}$$

where
L = soakage (sand) length (m)
R = test hole radius (m)

$$\text{Perm coeff. } k = \frac{A}{F \times (t2 - t1)} \times \ln \frac{h1}{h2}$$

where
A = test hole flow area
h1 = initial water level
h2 = final water level
t1 = time at h1
t2 = time at h2

Soil Parameters
0.5 m impermeable materal depth
0.2 m permeable materal depth

Elapsed Time (mins)	Water Level head (m)	Av Water Level Head (m) (=H/2)	L (m)	Av. L (m)	F	k (m/sec)
0.0	1.50	1.40	0.20			
1.0	0.82	0.72	0.20	0.20	0.59	1.3E-04
2.0	0.65	0.55	0.20	0.20	0.59	5.4E-05
3.0	0.56	0.46	0.20	0.20	0.59	3.6E-05
4.0	0.50	0.40	0.20	0.20	0.59	2.8E-05
5.0	0.45	0.35	0.20	0.20	0.59	2.7E-05
6.0	0.41	0.31	0.20	0.20	0.59	2.4E-05
8.0	0.34	0.24	0.20	0.20	0.59	2.6E-05
10.0	0.29	0.19	0.20	0.20	0.59	2.4E-05
15.0	0.20	0.10	0.20	0.20	0.59	2.6E-05
20.0	0.11	0.05	0.11	0.16	0.51	2.8E-05
25.0	0.04	0.02	0.04	0.07	0.38	6.3E-05
30.0	0.00	0.00	0.00	0.02	0.31	ERR

COMPUTED ADJUSTED AVERAGE: 2.7E-05 m/sec
2.4 m/day

Checked:

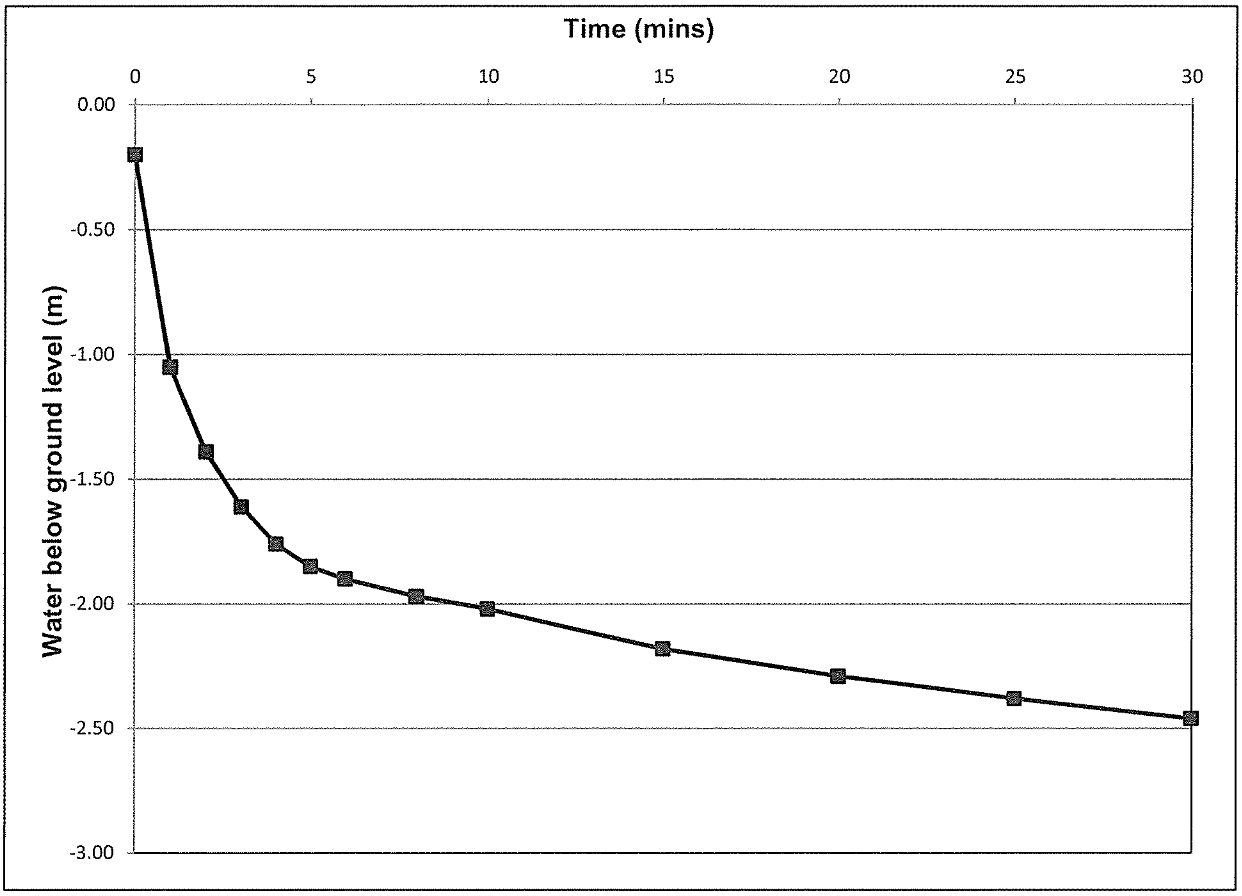
FALLING HEAD SOAKAGE TEST

JOB NO. W-16064

PROJECT: WAIPA DISTRICT COUNCIL

LOCATION: C4 Growth Cell

SOAKAGE TEST B2



Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0	0.45	-0.20	0.00	2.80
1	1.30	-1.05	0.85	1.95
2	1.64	-1.39	0.34	1.61
3	1.86	-1.61	0.22	1.39
4	2.01	-1.76	0.15	1.24
5	2.10	-1.85	0.09	1.15
6	2.15	-1.90	0.05	1.10
8	2.22	-1.97	0.07	1.03
10	2.27	-2.02	0.07	0.98
15	2.43	-2.18	0.16	0.82
20	2.54	-2.29	0.11	0.71
25	2.63	-2.38	0.09	0.62
30	2.71	-2.46	0.08	0.54

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FALLING HEAD SOAKAGE TEST RESULTS

Figure No. E-103

DATE: October 2019

CHECKED: SW

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WAIPA DISTRICT COUNCIL — W-16064

Stormwater Assessment for C4 Growth Cell — Date of test: 15 October, 2019

Field Soakage Test Data

TABLE 1D: FALLING HEAD SOAKAGE TEST RESUL STORMWATER TEST B2

Length of PVC Casing (m)	3.25
Length of PVC Above Ground (m)	0.25
Depth of Soakhole (m)	3.00
Groundwater Level (m)	0.00
Groundwater Level (height above base of Soakhole) (m)	na
Test Hole Diameter (m)	0.095

Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0.0	0.45	-0.20	0.00	2.80
1.0	1.30	-1.05	0.85	1.95
2.0	1.64	-1.39	0.34	1.61
3.0	1.86	-1.61	0.22	1.39
4.0	2.01	-1.76	0.15	1.24
5.0	2.10	-1.85	0.09	1.15
6.0	2.15	-1.90	0.05	1.10
8.0	2.22	-1.97	0.07	1.03
10.0	2.27	-2.02	0.05	0.98
15.0	2.43	-2.18	0.16	0.82
20.0	2.54	-2.29	0.11	0.71
25.0	2.63	-2.38	0.09	0.62
30.0	2.71	-2.46	0.08	0.54

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COEFFICIENT OF PERMEABILITY DERIVATION

Use Hvorslev Case 7 (from Kortegast NZGS Vol 16 Issue 1) - hole extended in uniform soil
ie. soakage occurs out the side and base of test hole (slotted) with overlying restrictive layer

PERMEABILITY CALCULATIONS — STORMWATER TEST B2

Shape Factor F = $\frac{2 \times PI \times L}{\ln\{(L/R) + [1 + (L/R)^2]^{0.5}\}}$

where
L = soakage (sand) length (m)
R = test hole radius (m)

Perm coeff. k = $\frac{A}{F \times (t2 - t1)} \times \ln \frac{h1}{h2}$

where
A = test hole flow area
h1 = initial water level
h2 = final water level
t1 = time at h1
t2 = time at h2

Soil Parameters

0.5 m impermeable materal depth
0.2 m permeable materal depth

Elapsed Time (mins)	Water Level head (m)	Av Water Level Head (m) (=H/2)	L (m)	Av. L (m)	F	k (m/sec)
0.0	2.80	2.70	0.20			
1.0	1.95	1.85	0.20	0.20	0.59	7.6E-05
2.0	1.61	1.51	0.20	0.20	0.59	4.1E-05
3.0	1.39	1.29	0.20	0.20	0.59	3.2E-05
4.0	1.24	1.14	0.20	0.20	0.59	2.5E-05
5.0	1.15	1.05	0.20	0.20	0.59	1.7E-05
6.0	1.10	1.00	0.20	0.20	0.59	9.8E-06
8.0	1.03	0.93	0.20	0.20	0.59	7.3E-06
10.0	0.98	0.88	0.20	0.20	0.59	5.6E-06
15.0	0.82	0.72	0.20	0.20	0.59	8.1E-06
20.0	0.71	0.61	0.20	0.20	0.59	6.7E-06
25.0	0.62	0.52	0.20	0.20	0.59	6.4E-06
30.0	0.54	0.44	0.20	0.20	0.59	6.7E-06

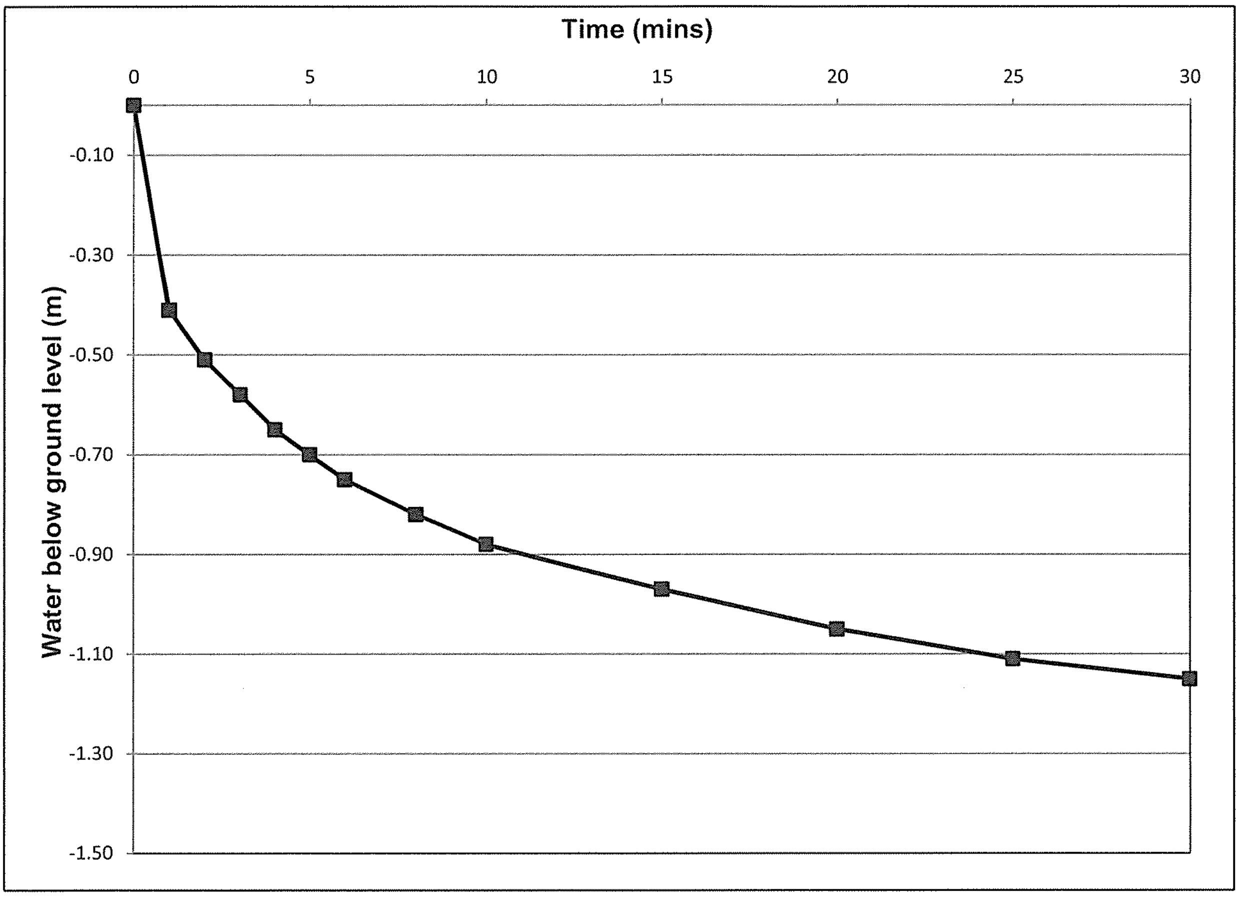
COMPUTED ADJUSTED AVERAGE: 1.4E-05 m/sec
1.2 m/day

Checked:

FALLING HEAD SOAKAGE TEST

JOB NO. W-16064
PROJECT: WAIPA DISTRICT COUNCIL
LOCATION: C4 Growth Cell

SOAKAGE TEST C1



Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0	0.62	0.00	0.00	1.50
1	1.03	-0.41	0.41	1.09
2	1.13	-0.51	0.10	0.99
3	1.20	-0.58	0.07	0.92
4	1.27	-0.65	0.07	0.85
5	1.32	-0.70	0.05	0.80
6	1.37	-0.75	0.05	0.75
8	1.44	-0.82	0.07	0.68
10	1.50	-0.88	0.06	0.62
15	1.59	-0.97	0.09	0.53
20	1.67	-1.05	0.08	0.45
25	1.73	-1.11	0.06	0.39
30	1.77	-1.15	0.04	0.35

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FALLING HEAD SOAKAGE TEST RESULTS

Figure No. E-104
DATE: October 2019
CHECKED: SW

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WAIPA DISTRICT COUNCIL — W-16064

Stormwater Assessment for C4 Growth Cell — Date of test: 14 October, 2019

Field Soakage Test Data

TABLE 1E: FALLING HEAD SOAKAGE TEST RESUL STORMWATER TEST C1

Length of PVC Casing (m)	2.12
Length of PVC Above Ground (m)	0.62
Depth of Soakhole (m)	1.50
Groundwater Level (m)	0.00
Groundwater Level (height above base of Soakhole) (m)	na
Test Hole Diameter (m)	0.095

Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0.0	0.62	0.00	0.00	1.50
1.0	1.03	-0.41	0.41	1.09
2.0	1.13	-0.51	0.10	0.99
3.0	1.20	-0.58	0.07	0.92
4.0	1.27	-0.65	0.07	0.85
5.0	1.32	-0.70	0.05	0.80
6.0	1.37	-0.75	0.05	0.75
8.0	1.44	-0.82	0.07	0.68
10.0	1.50	-0.88	0.06	0.62
15.0	1.59	-0.97	0.09	0.53
20.0	1.67	-1.05	0.08	0.45
25.0	1.73	-1.11	0.06	0.39
30.0	1.77	-1.15	0.04	0.35

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COEFFICIENT OF PERMEABILITY DERIVATION

Use Hvorslev Case 7 (from Kortegast NZGS Vol 16 Issue 1) - hole extended in uniform soil
ie. soakage occurs out the side and base of test hole (slotted) with overlying restrictive layer

PERMEABILITY CALCULATIONS STORMWATER TEST C1

Shape Factor F = 2 x PI x L / ln{(L/R) + [1 + (L/R)^2]^0.5}

where L = soakage (sand) length (m)
R = test hole radius (m)

Perm coeff. k = A / F x (t2 - t1) x ln h1/h2

where A = test hole flow area
h1 = initial water level
h2 = final water level
t1 = time at h1
t2 = time at h2

Soil Parameters

0.6 m impermeable materal depth
0.2 m permeable materal depth

Elapsed Time (mins)	Water Level head (m)	Av Water Level Head (m) (=H/2)	L (m)	Av. L (m)	F	k (m/sec)
0.0	1.50	1.40	0.20			
1.0	1.09	0.99	0.20	0.20	0.59	7.0E-05
2.0	0.99	0.89	0.20	0.20	0.59	2.1E-05
3.0	0.92	0.82	0.20	0.20	0.59	1.7E-05
4.0	0.85	0.75	0.20	0.20	0.59	1.8E-05
5.0	0.80	0.70	0.20	0.20	0.59	1.4E-05
6.0	0.75	0.65	0.20	0.20	0.59	1.5E-05
8.0	0.68	0.58	0.20	0.20	0.59	1.1E-05
10.0	0.62	0.52	0.20	0.20	0.59	1.1E-05
15.0	0.53	0.43	0.20	0.20	0.59	7.7E-06
20.0	0.45	0.35	0.20	0.20	0.59	8.3E-06
25.0	0.39	0.29	0.20	0.20	0.59	7.6E-06
30.0	0.35	0.25	0.20	0.20	0.59	6.0E-06

COMPUTED ADJUSTED AVERAGE: 1.3E-05 m/sec
1.1 m/day

Checked:

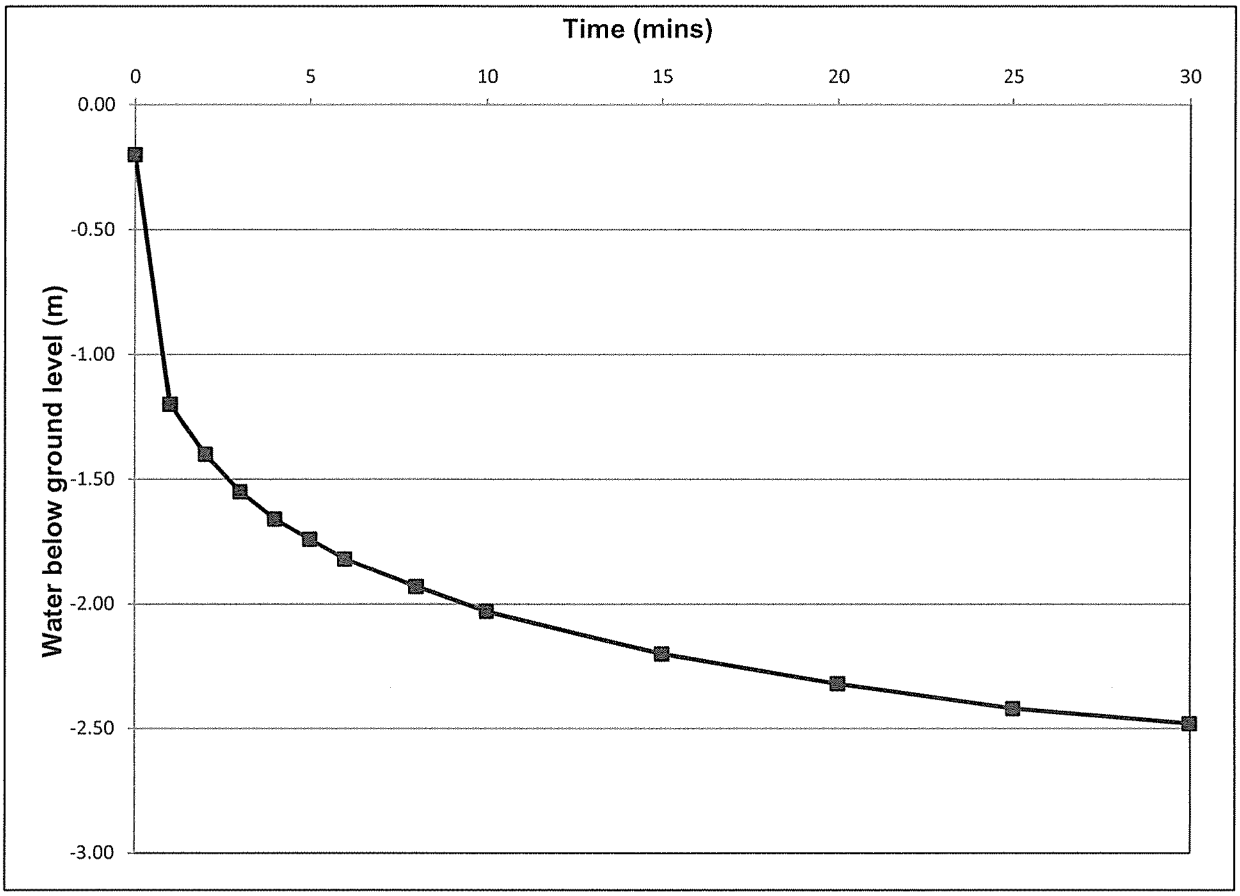
FALLING HEAD SOAKAGE TEST

JOB NO. W-16064

PROJECT: WAIPA DISTRICT COUNCIL

LOCATION: C4 Growth Cell

SOAKAGE TEST C2



Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0	0.45	-0.20	0.00	2.80
1	1.45	-1.20	1.00	1.80
2	1.65	-1.40	0.20	1.60
3	1.80	-1.55	0.15	1.45
4	1.91	-1.66	0.11	1.34
5	1.99	-1.74	0.08	1.26
6	2.07	-1.82	0.08	1.18
8	2.18	-1.93	0.11	1.07
10	2.28	-2.03	1.10	0.97
15	2.45	-2.20	0.17	0.80
20	2.57	-2.32	0.12	0.68
25	2.67	-2.42	0.10	0.58
30	2.73	-2.48	0.06	0.52

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FALLING HEAD SOAKAGE TEST RESULTS

Figure No. E-105

DATE: October 2019

CHECKED:

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WAIPA DISTRICT COUNCIL W-16064
Stormwater Assessment for C4 Growth Cell
Date of test: 14 October, 2019
Field Soakage Test Data

TABLE 1F: FALLING HEAD SOAKAGE TEST RESUL STORMWATER TEST C2

Length of PVC Casing (m)	3.25
Length of PVC Above Ground (m)	0.25
Depth of Soakhole (m)	3.00
Groundwater Level (m)	0.00
Groundwater Level (height above base of Soakhole) (m)	na
Test Hole Diameter (m)	0.095

Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0.0	0.45	-0.20	0.00	2.80
1.0	1.45	-1.20	1.00	1.80
2.0	1.65	-1.40	0.20	1.60
3.0	1.80	-1.55	0.15	1.45
4.0	1.91	-1.66	0.11	1.34
5.0	1.99	-1.74	0.08	1.26
6.0	2.07	-1.82	0.08	1.18
8.0	2.18	-1.93	0.11	1.07
10.0	2.28	-2.03	0.10	0.97
15.0	2.45	-2.20	0.17	0.80
20.0	2.57	-2.32	0.12	0.68
25.0	2.67	-2.42	0.10	0.58
30.0	2.73	-2.48	0.06	0.52

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COEFFICIENT OF PERMEABILITY DERIVATION

Use Hvorslev Case 7 (from Kortegast NZGS Vol 16 Issue 1) - hole extended in uniform soil
ie. soakage occurs out the side and base of test hole (slotted) with overlying restrictive layer

PERMEABILITY CALCULATIONS STORMWATER TEST C2

Shape Factor F = 2 x PI x L / ln{(L/R) + [1 + (L/R)^2]^0.5}

where
L = soakage (sand) length (m)
R = test hole radius (m)

Perm coeff. k = A / F x (t2 - t1) x ln h1 / h2

where
A = test hole flow area
h1 = initial water level
h2 = final water level
t1 = time at h1
t2 = time at h2

Soil Parameters

0.7 m impermeable materal depth
0.2 m permeable materal depth

Elapsed Time (mins)	Water Level head (m)	Av Water Level Head (m) (=H/2)	L (m)	Av. L (m)	F	k (m/sec)
0.0	2.80	2.70	0.20			
1.0	1.80	1.70	0.20	0.20	0.59	9.3E-05
2.0	1.60	1.50	0.20	0.20	0.59	2.5E-05
3.0	1.45	1.35	0.20	0.20	0.59	2.1E-05
4.0	1.34	1.24	0.20	0.20	0.59	1.7E-05
5.0	1.26	1.16	0.20	0.20	0.59	1.3E-05
6.0	1.18	1.08	0.20	0.20	0.59	1.4E-05
8.0	1.07	0.97	0.20	0.20	0.59	1.1E-05
10.0	0.97	0.87	0.20	0.20	0.59	1.1E-05
15.0	0.80	0.70	0.20	0.20	0.59	8.8E-06
20.0	0.68	0.58	0.20	0.20	0.59	7.6E-06
25.0	0.58	0.48	0.20	0.20	0.59	7.6E-06
30.0	0.52	0.42	0.20	0.20	0.59	5.4E-06

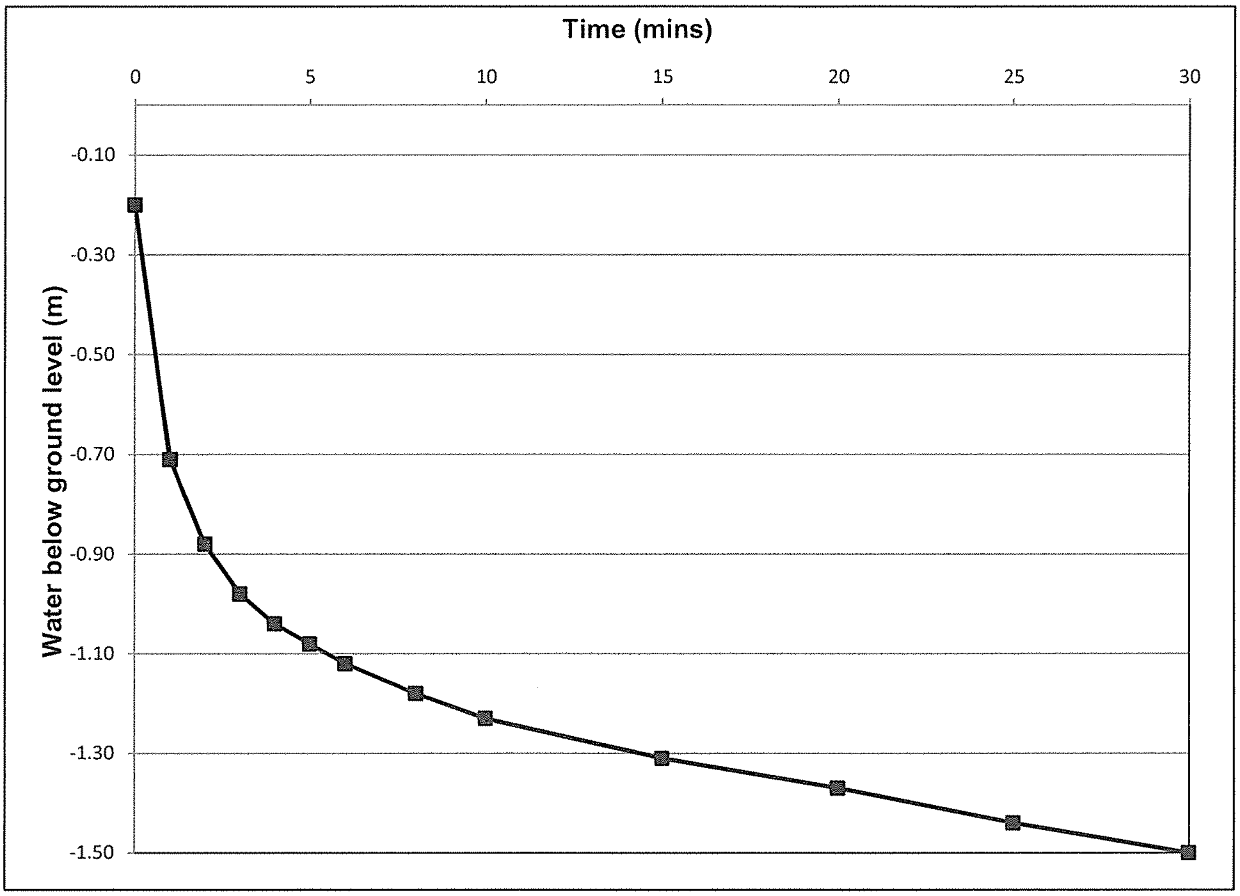
COMPUTED ADJUSTED AVERAGE: 1.3E-05 m/sec
1.1 m/day

Checked:

FALLING HEAD SOAKAGE TEST

JOB NO. W-16064
PROJECT: WAIPA DISTRICT COUNCIL
LOCATION: C4 Growth Cell

SOAKAGE TEST D1



Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0	0.82	-0.20	0.00	1.30
1	1.33	-0.71	0.51	0.79
2	1.50	-0.88	0.17	0.62
3	1.60	-0.98	0.10	0.52
4	1.66	-1.04	0.06	0.46
5	1.70	-1.08	0.04	0.42
6	1.74	-1.12	0.04	0.38
8	1.80	-1.18	0.06	0.32
10	1.85	-1.23	0.05	0.27
15	1.93	-1.31	0.08	0.19
20	1.99	-1.37	0.06	0.13
25	2.06	-1.44	0.07	0.06
30	2.12	-1.5	0.06	0.00

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FALLING HEAD SOAKAGE TEST RESULTS

Figure No. E-106
DATE: October 2019
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WAIPA DISTRICT COUNCIL — W-16064

Stormwater Assessment for C4 Growth Cell — Date of test: 14 October, 2019

Field Soakage Test Data

TABLE 1G: FALLING HEAD SOAKAGE TEST RESUL STORMWATER TEST D1

Length of PVC Casing (m)	2.12
Length of PVC Above Ground (m)	0.62
Depth of Soakhole (m)	1.50
Groundwater Level (m)	0.00
Groundwater Level (height above base of Soakhole) (m)	na
Test Hole Diameter (m)	0.095

Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0.0	0.82	-0.20	0.00	1.30
1.0	1.33	-0.71	0.51	0.79
2.0	1.50	-0.88	0.17	0.62
3.0	1.60	-0.98	0.10	0.52
4.0	1.66	-1.04	0.06	0.46
5.0	1.70	-1.08	0.04	0.42
6.0	1.74	-1.12	0.04	0.38
8.0	1.80	-1.18	0.06	0.32
10.0	1.85	-1.23	0.05	0.27
15.0	1.93	-1.31	0.08	0.19
20.0	1.99	-1.37	0.06	0.13
25.0	2.06	-1.44	0.07	0.06
30.0	2.12	-1.50	0.06	0.00

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COEFFICIENT OF PERMEABILITY DERIVATION

Use Hvorslev Case 7 (from Kortegast NZGS Vol 16 Issue 1) - hole extended in uniform soil
ie. soakage occurs out the side and base of test hole (slotted) with overlying restrictive layer

PERMEABILITY CALCULATIONS — STORMWATER TEST D1

Shape Factor F = 2 x PI x L / ln{(L/R) + [1 + (L/R)^2]^0.5}

where
L = soakage (sand) length (m)
R = test hole radius (m)

Perm coeff. k = A / F x (t2 - t1) x ln h1/h2

where
A = test hole flow area
h1 = initial water level
h2 = final water level
t1 = time at h1
t2 = time at h2

Soil Parameters

0.3 m impermeable materal depth
0.2 m permeable materal depth

Elapsed Time (mins)	Water Level head (m)	Av Water Level Head (m) (=H/2)	L (m)	Av. L (m)	F	k (m/sec)
0.0	1.30	1.20	0.20			
1.0	0.79	0.69	0.20	0.20	0.59	1.1E-04
2.0	0.62	0.52	0.20	0.20	0.59	5.7E-05
3.0	0.52	0.42	0.20	0.20	0.59	4.3E-05
4.0	0.46	0.36	0.20	0.20	0.59	3.1E-05
5.0	0.42	0.32	0.20	0.20	0.59	2.4E-05
6.0	0.38	0.28	0.20	0.20	0.59	2.7E-05
8.0	0.32	0.22	0.20	0.20	0.59	2.4E-05
10.0	0.27	0.17	0.20	0.20	0.59	2.6E-05
15.0	0.19	0.10	0.19	0.20	0.58	2.4E-05
20.0	0.13	0.07	0.13	0.16	0.52	1.7E-05
25.0	0.06	0.03	0.06	0.10	0.41	4.4E-05
30.0	0.00	0.00	0.00	0.03	0.32	ERR

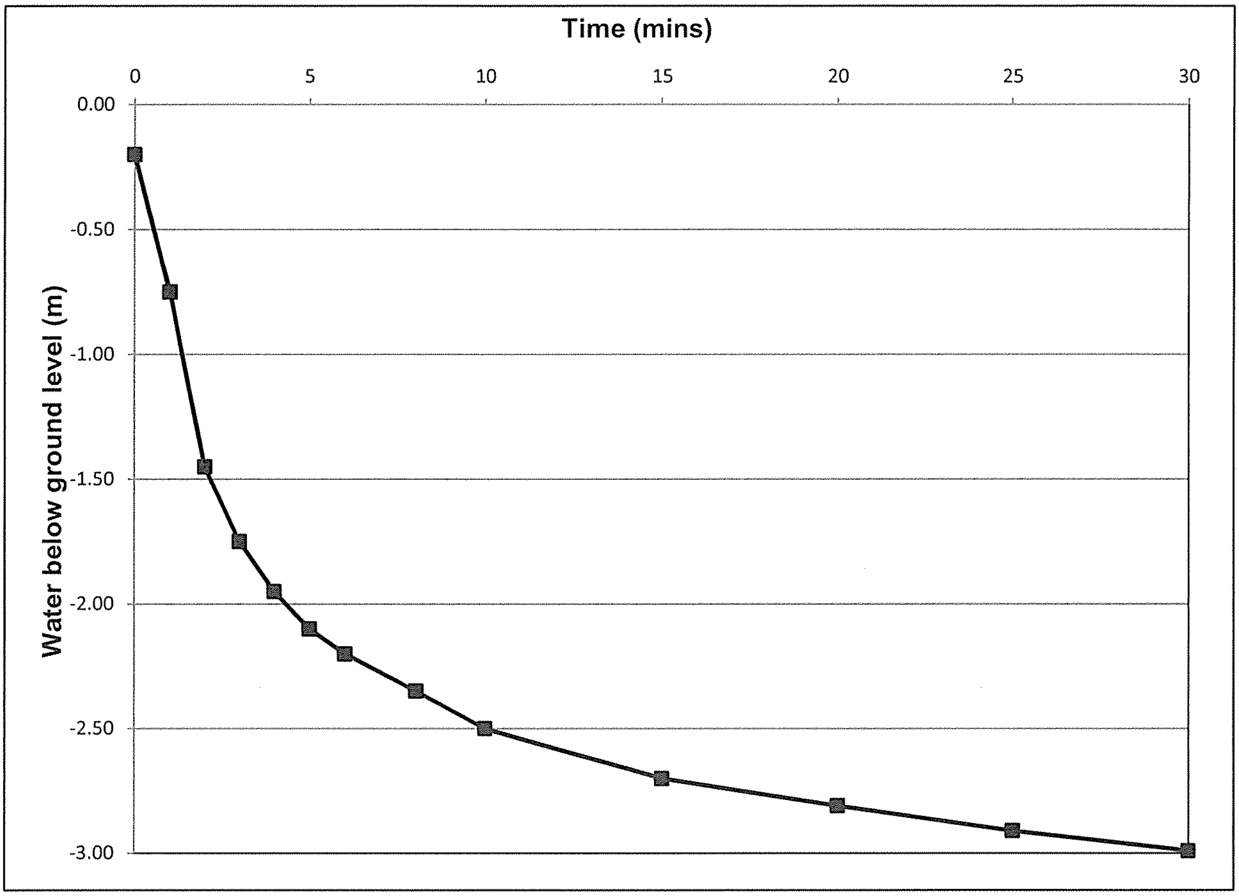
COMPUTED ADJUSTED AVERAGE: 2.7E-05 m/sec
2.3 m/day

Checked:

FALLING HEAD SOAKAGE TEST

JOB NO. W-16064
PROJECT: WAIPA DISTRICT COUNCIL
LOCATION: C4 Growth Cell

SOAKAGE TEST D2



Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0	0.45	-0.20	0.00	2.80
1	1.00	-0.75	0.55	2.25
2	1.70	-1.45	0.70	1.55
3	2.00	-1.75	0.30	1.25
4	2.20	-1.95	0.20	1.05
5	2.35	-2.10	0.15	0.90
6	2.45	-2.20	0.10	0.80
8	2.60	-2.35	0.15	0.65
10	2.75	-2.50	0.15	0.50
15	2.95	-2.70	0.20	0.30
20	3.06	-2.81	0.11	0.19
25	3.16	-2.91	0.10	0.09
30	3.24	-2.99	0.08	0.01

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FALLING HEAD SOAKAGE TEST RESULTS

Figure No. E-107
DATE: October 2019
CHECKED:

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WAIPA DISTRICT COUNCIL — W-16064

Stormwater Assessment for C4 Growth Cell — Date of test: 14 October, 2019

Field Soakage Test Data

TABLE 1H: FALLING HEAD SOAKAGE TEST RESUL STORMWATER TEST D2

Length of PVC Casing (m)	3.25
Length of PVC Above Ground (m)	0.25
Depth of Soakhole (m)	3.00
Groundwater Level (m)	0.00
Groundwater Level (height above base of Soakhole) (m)	na
Test Hole Diameter (m)	0.095

Time (mins)	Water Level below top of PVC (m)	Water Level Relative to Ground Level (m)	Change in Water Level (m)	Water Level head (m)
0.0	0.45	-0.20	0.00	2.80
1.0	1.00	-0.75	0.55	2.25
2.0	1.70	-1.45	0.70	1.55
3.0	2.00	-1.75	0.30	1.25
4.0	2.20	-1.95	0.20	1.05
5.0	2.35	-2.10	0.15	0.90
6.0	2.45	-2.20	0.10	0.80
8.0	2.60	-2.35	0.15	0.65
10.0	2.75	-2.50	0.15	0.50
15.0	2.95	-2.70	0.20	0.30
20.0	3.06	-2.81	0.11	0.19
25.0	3.16	-2.91	0.10	0.09
30.0	3.24	-2.99	0.08	0.01

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COEFFICIENT OF PERMEABILITY DERIVATION

Use Hvorslev Case 7 (from Kortegast NZGS Vol 16 Issue 1) - hole extended in uniform soil
ie. soakage occurs out the side and base of test hole (slotted) with overlying restrictive layer

PERMEABILITY CALCULATIONS — STORMWATER TEST D2

Shape Factor F = 2 x PI x L / ln{(L/R) + [1 + (L/R)^2]^0.5}

where
L = soakage (sand) length (m)
R = test hole radius (m)

Perm coeff. k = A / F x (t2 - t1) x ln h1/h2

where
A = test hole flow area
h1 = initial water level
h2 = final water level
t1 = time at h1
t2 = time at h2

Soil Parameters

0.6 m impermeable materal depth
0.2 m permeable materal depth

Elapsed Time (mins)	Water Level head (m)	Av Water Level Head (m) (=H/2)	L (m)	Av. L (m)	F	k (m/sec)
0.0	2.80	2.70	0.20			
1.0	2.25	2.15	0.20	0.20	0.59	4.6E-05
2.0	1.55	1.45	0.20	0.20	0.59	7.9E-05
3.0	1.25	1.15	0.20	0.20	0.59	4.7E-05
4.0	1.05	0.95	0.20	0.20	0.59	3.9E-05
5.0	0.90	0.80	0.20	0.20	0.59	3.5E-05
6.0	0.80	0.70	0.20	0.20	0.59	2.7E-05
8.0	0.65	0.55	0.20	0.20	0.59	2.4E-05
10.0	0.50	0.40	0.20	0.20	0.59	3.2E-05
15.0	0.30	0.20	0.20	0.20	0.59	2.8E-05
20.0	0.19	0.09	0.19	0.20	0.58	3.0E-05
25.0	0.09	0.04	0.09	0.14	0.49	3.6E-05
30.0	0.01	0.00	0.01	0.05	0.34	1.5E-04

COMPUTED ADJUSTED AVERAGE: 3.3E-05 m/sec
2.8 m/day

Checked:

APPENDIX 8 LID MATRIX SCORING

APPENDIX 8 SOURCE AND LIDS CONTROL CALCULATIONS

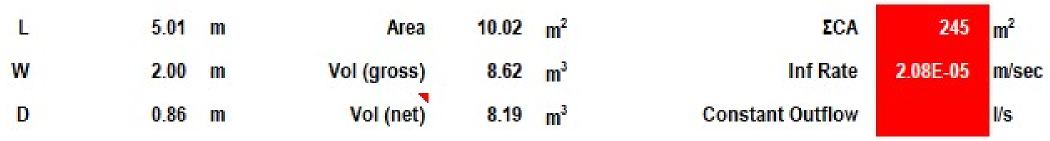
CATCHMENT 1

Catchment 1 Source Control Estimates

Catchment 1 – Proposed Layout under Integrated SW Design Principles	Quantity	Units
Residential development area (assume 85% of total residential area of lots)	9.1	ha
Road and access way area (assume roads and foot paths is 15% of total residential area)	1.6	ha
Open space/park land area	3.0	ha
Native bush area	0.0	ha
Total area	13.7	Ha
Assumed number of lots dwelling count (assume an average lot size of 600 sq.m)	151	No.
Assumed area of impervious per lot (250 sq.m with 100 sq.m for patio/driveway)	0.035	Ha
Total Impervious lot area for residential development area	5.3	Ha
Percentage lot impervious surface	58	%
Road imperious area (assume 80% of road reserve)	1.3	ha
Total impervious area for Catchment 1	6.6	ha
Total fraction impervious for Catchment 1	50	%
Catchment 1 – Comparison from Traditional Development FI	**Quantity**	**Units**
Conventional housing impervious values	50% (as per district plan)	
Number of houses if allow 600sq metre lots	151	No.
Include houses in drainage reserve area	42	No.
Include road in drainage reserve area	0.4	Ha
Total houses in conventional build	194	no.
Total impervious area if conventional build	60	%
% reduction area FI from conventional development	10	%
Site disturbance reduced from a conventional development approach		
Catchment 1 – Comparison of Disturbed Area	**Quantity**	**Units**
Proposed disturbed area	10.7	ha
Conventional disturbed area	13.7	ha
Reduction disturbed area	10.7	ha
% reduction disturbed area	20	%

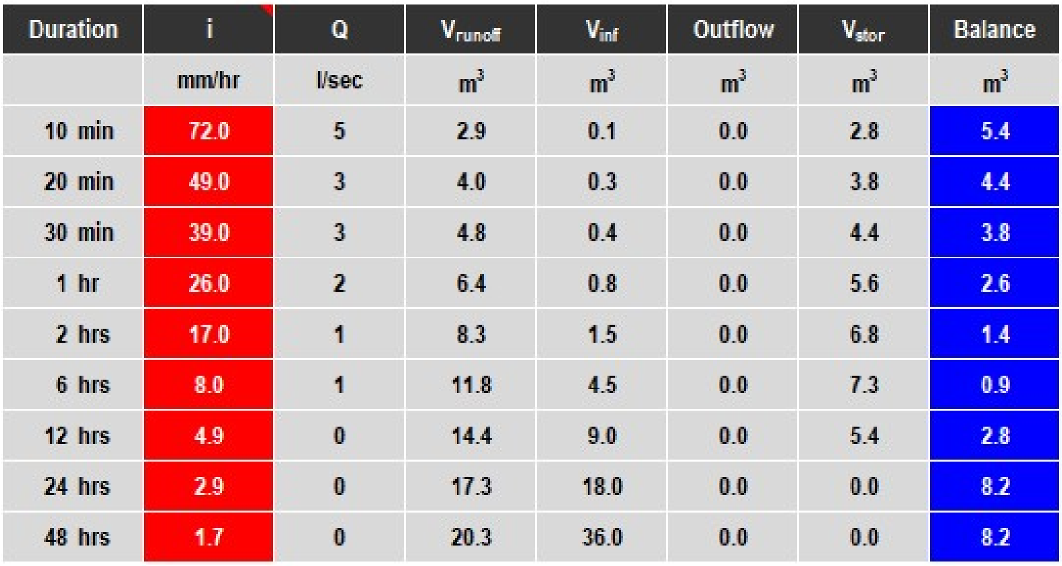
On lot device sizing – 2 year ARI – 70 mm/hr

Dimensions



L	5.01	m	Area	10.02	m^2	ΣCA	245	m^2
W	2.00	m	Vol (gross)	8.62	m^3	Inf Rate	2.08E-05	m/sec
D	0.86	m	Vol (net)	8.19	m^3	Constant Outflow		l/s

Intensity and Critical Storm



Duration	i	Q	V_{runoff}	V_{inf}	Outflow	V_{stor}	Balance
	mm/hr	l/sec	m^3	m^3	m^3	m^3	m^3
10 min	72.0	5	2.9	0.1	0.0	2.8	5.4
20 min	49.0	3	4.0	0.3	0.0	3.8	4.4
30 min	39.0	3	4.8	0.4	0.0	4.4	3.8
1 hr	26.0	2	6.4	0.8	0.0	5.6	2.6
2 hrs	17.0	1	8.3	1.5	0.0	6.8	1.4
6 hrs	8.0	1	11.8	4.5	0.0	7.3	0.9
12 hrs	4.9	0	14.4	9.0	0.0	5.4	2.8
24 hrs	2.9	0	17.3	18.0	0.0	0.0	8.2
48 hrs	1.7	0	20.3	36.0	0.0	0.0	8.2

WETLAND DESIGN

Curve Number and Ia

Soil	Cover description	Curve Number	Area (ha)		Product of
classification		CN	impervious	pervious	CN * area
A	Road and Driveway	98	3.38		331
Total area (ha)			3.38	Total area (km^2)	0.0338
Weighted CN			98.0		
I_a (weighted) (mm)			0.26		
S (mm)			5		

Time of Concentration

Time of Concentration			
Catchment length along main channel (m)			700 m
pipe flow			2 m/s
Time of Concentration		t_c (minutes)	10.000

Wetland Design

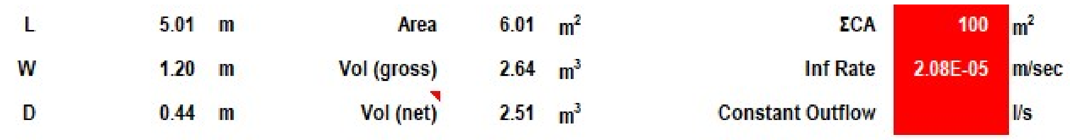
Select A R I (years) or A E P (%)	WQ	EDV	Forebay Volume	Total (50% WQ +EDV)	Surface Area (4% of Contributing Catchment)	Width (NWL) (m)	Length (NWL) (m)	Additional 20% for batters and maintenance (sq.m)
Read 24 hour rainfall depth for that recurrence interval (mm)	24.167							
c*	0.695							
Read q* from chart	0.1670							
Peak Flow rate (m^3/s)	0.136							
Runoff depth (mm)	20							
Runoff volume (V)	664	797	100	1129				
Device Area					1352			1662
Device Dimensions						20	70	

Catchment 2 Source Control Estimates

Catchment 1 – Proposed Layout under Integrated SW Design Principles	Quantity	Units
Residential development area (assume 85% of total residential area of lots)	0.6	ha
Road and access way area (assume roads and foot paths is 15% of total residential area)	0.1	ha
Open space/park land area	0.0	ha
Native bush area	0.0	ha
Total area	0.0	Ha
Assumed number of lots dwelling count (assume an average lot size of 600 sq.m)	0.8	No.
Assumed area of impervious per lot (250 sq.m with 100 sq.m for patio/driveway)	10.8	Ha
Total Impervious lot area for residential development area	0.0	Ha
Percentage lot impervious surface	40	%
Road imperious area (assume 80% of road reserve)	0.6	ha
Total impervious area for Catchment 1	0.1	ha
Total fraction impervious for Catchment 1	60	%
Catchment 1 – Comparison from Traditional Development FI	Quantity	Units
Conventional housing impervious values	50% (as per district plan)	
Number of houses if allow 600sq metre lots	11	No.
Include houses in drainage reserve area	0	No.
Include road in drainage reserve area	0	Ha
Total houses in conventional build	11	no.
Total impervious area if conventional build	60	%
% reduction area FI from conventional development	0	%
Site disturbance reduced from a conventional development approach		
Catchment 1 – Comparison of Disturbed Area	Quantity	Units
Proposed disturbed area	0.8	ha
Conventional disturbed area	0.8	ha
Reduction disturbed area	0	ha
% reduction disturbed area	0	%

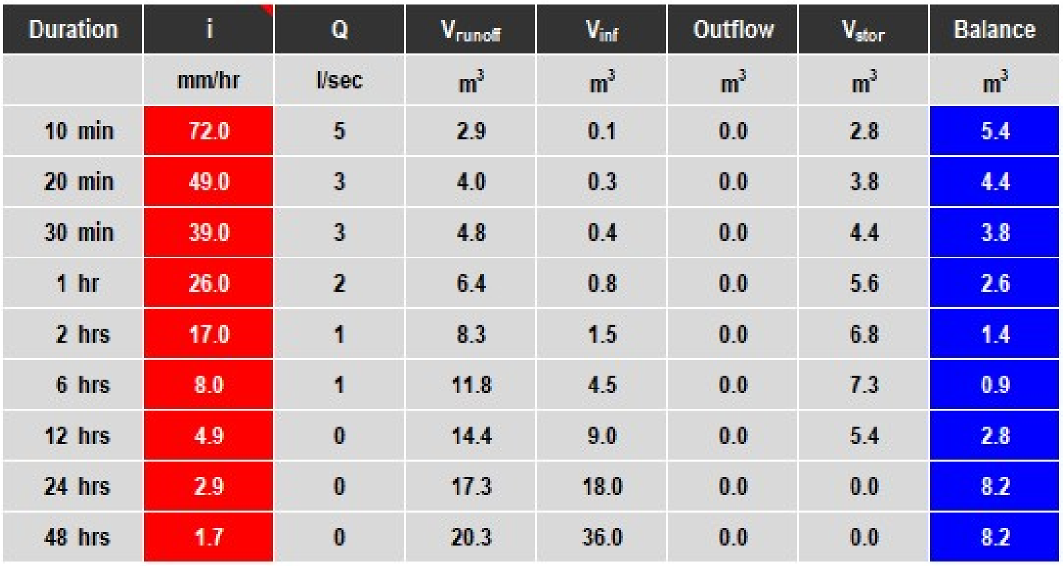
On lot device sizing – 2 year ARI – 70 mm/hr

Dimensions



L	5.01	m	Area	6.01	m^2	ΣCA	100	m^2
W	1.20	m	Vol (gross)	2.64	m^3	Inf Rate	2.08E-05	m/sec
D	0.44	m	Vol (net)	2.51	m^3	Constant Outflow		l/s

<u>Intensity and Critical Storm</u>



Duration	i	Q	V_{runoff}	V_{inf}	Outflow	V_{stor}	Balance
	mm/hr	l/sec	m^3	m^3	m^3	m^3	m^3
10 min	72.0	5	2.9	0.1	0.0	2.8	5.4
20 min	49.0	3	4.0	0.3	0.0	3.8	4.4
30 min	39.0	3	4.8	0.4	0.0	4.4	3.8
1 hr	26.0	2	6.4	0.8	0.0	5.6	2.6
2 hrs	17.0	1	8.3	1.5	0.0	6.8	1.4
6 hrs	8.0	1	11.8	4.5	0.0	7.3	0.9
12 hrs	4.9	0	14.4	9.0	0.0	5.4	2.8
24 hrs	2.9	0	17.3	18.0	0.0	0.0	8.2
48 hrs	1.7	0	20.3	36.0	0.0	0.0	8.2

CATCHMENT 3:

Catchment 3 Source Control Estimates

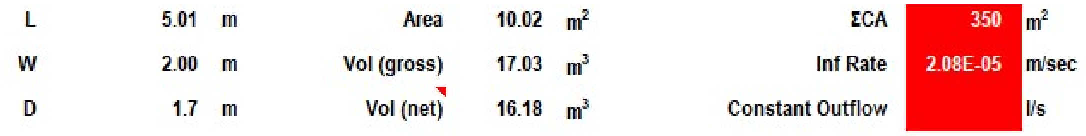
Catchment 1 – Proposed Layout under Integrated SW Design Principles	Quantity	Units
Residential development area (assume 85% of total residential area of lots)	10.7	ha
Road and access way area (assume roads and foot paths is 15% of total residential area)	1.9	ha
Open space/park land area	1.3	ha
Native bush area	5.0	ha
Total area	18.8	Ha
Assumed number of lots dwelling count (assume an average lot size of 600 sq.m)	178	No.
Assumed area of impervious per lot (250 sq.m with 100 sq.m for patio/driveway)	0.0	Ha
Total Impervious lot area for residential development area	6.2	Ha
Percentage lot impervious surface	60	%
Road imperious area (assume 80% of road reserve)	1.5	ha
Total impervious area for Catchment 1	7.7	ha
Total fraction impervious for Catchment 1	40	%
Catchment 1 – Comparison from Traditional Development FI	Quantity	Units
Conventional housing impervious values	50% (as per district plan)	
Number of houses if allow 600sq metre lots	178	No.
Include houses in drainage reserve area	53	No.
Include road in drainage reserve area	1.68	Ha
Total houses in conventional build	231	no.
Total impervious area if conventional build	60	%
% reduction area FI from conventional development	19	%
Site disturbance reduced from a conventional development approach		

Catchment 1 – Comparison of Disturbed Area	Quantity	Units
Proposed disturbed area	18.8	ha
Conventional disturbed area	12.6	ha
Reduction disturbed area	6.3	ha
% reduction disturbed area	30	%

Catchment 3 LIDS Estimates

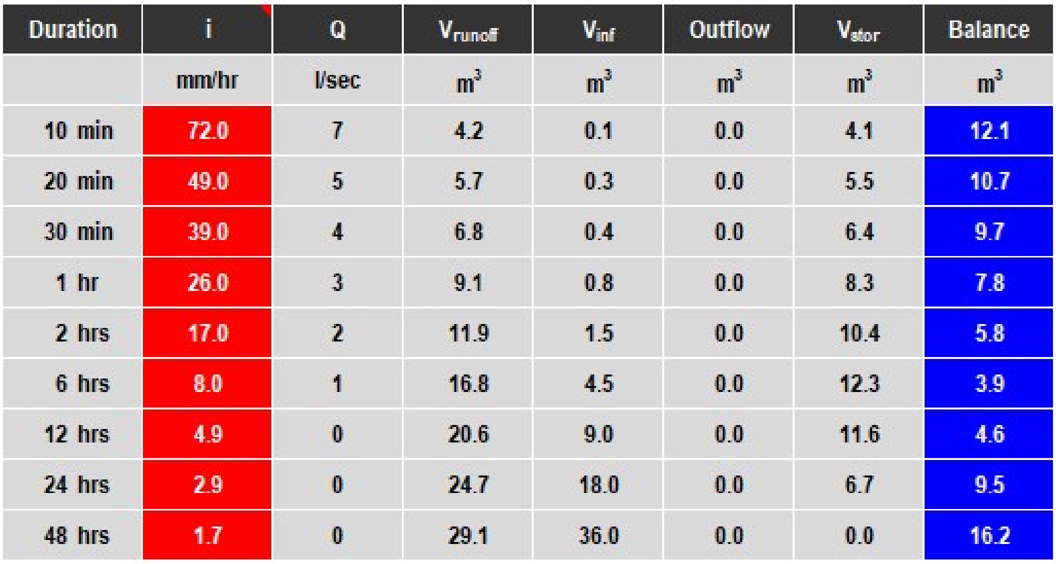
Onlot device sizing – 2 year ARI – 70 mm/hr

Dimensions



L	5.01	m	Area	10.02	m^2	ΣCA	350	m^2
W	2.00	m	Vol (gross)	17.03	m^3	Inf Rate	2.08E-05	m/sec
D	1.7	m	Vol (net)	16.18	m^3	Constant Outflow		l/s

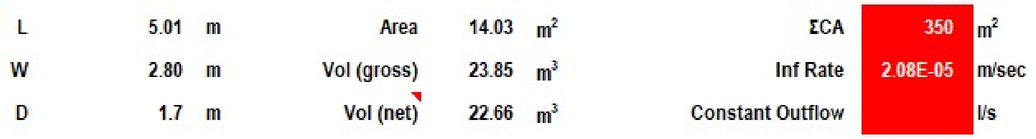
Intensity and Critical Storm



Duration	i	Q	V_{runoff}	V_{inf}	Outflow	V_{stor}	Balance
	mm/hr	l/sec	m^3	m^3	m^3	m^3	m^3
10 min	72.0	7	4.2	0.1	0.0	4.1	12.1
20 min	49.0	5	5.7	0.3	0.0	5.5	10.7
30 min	39.0	4	6.8	0.4	0.0	6.4	9.7
1 hr	26.0	3	9.1	0.8	0.0	8.3	7.8
2 hrs	17.0	2	11.9	1.5	0.0	10.4	5.8
6 hrs	8.0	1	16.8	4.5	0.0	12.3	3.9
12 hrs	4.9	0	20.6	9.0	0.0	11.6	4.6
24 hrs	2.9	0	24.7	18.0	0.0	6.7	9.5
48 hrs	1.7	0	29.1	36.0	0.0	0.0	16.2

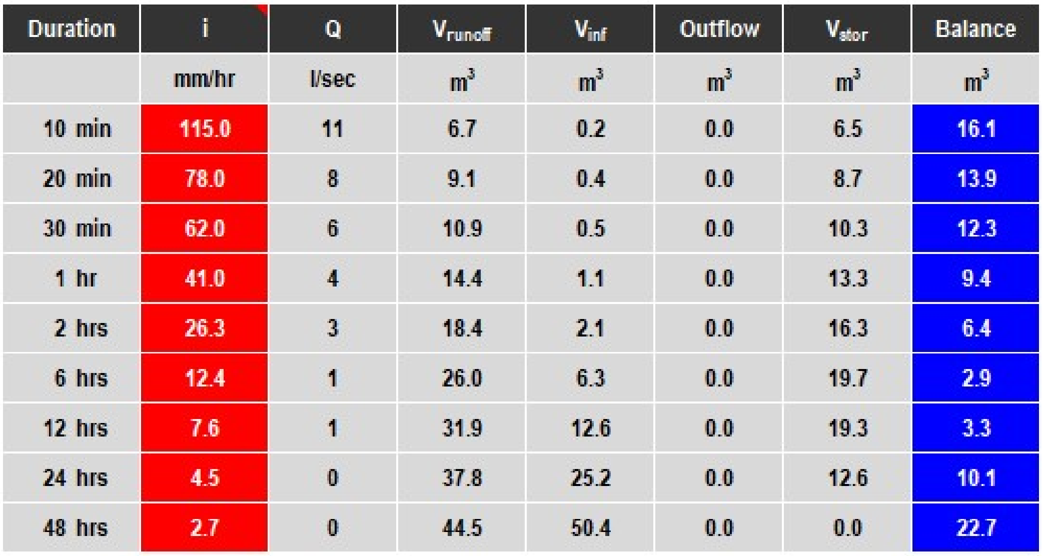
On lot device sizing – 10 year ARI – 70 mm/hr

Dimensions



L	5.01	m	Area	14.03	m^2	ΣCA	350	m^2
W	2.80	m	Vol (gross)	23.85	m^3	Inf Rate	2.08E-05	m/sec
D	1.7	m	Vol (net)	22.66	m^3	Constant Outflow		l/s

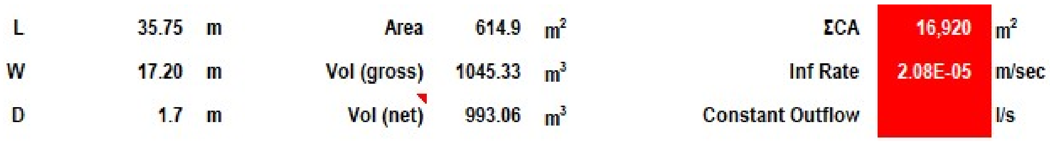
Intensity and Critical Storm



Duration	i	Q	V_{runoff}	V_{inf}	Outflow	V_{stor}	Balance
	mm/hr	l/sec	m^3	m^3	m^3	m^3	m^3
10 min	115.0	11	6.7	0.2	0.0	6.5	16.1
20 min	78.0	8	9.1	0.4	0.0	8.7	13.9
30 min	62.0	6	10.9	0.5	0.0	10.3	12.3
1 hr	41.0	4	14.4	1.1	0.0	13.3	9.4
2 hrs	26.3	3	18.4	2.1	0.0	16.3	6.4
6 hrs	12.4	1	26.0	6.3	0.0	19.7	2.9
12 hrs	7.6	1	31.9	12.6	0.0	19.3	3.3
24 hrs	4.5	0	37.8	25.2	0.0	12.6	10.1
48 hrs	2.7	0	44.5	50.4	0.0	0.0	22.7

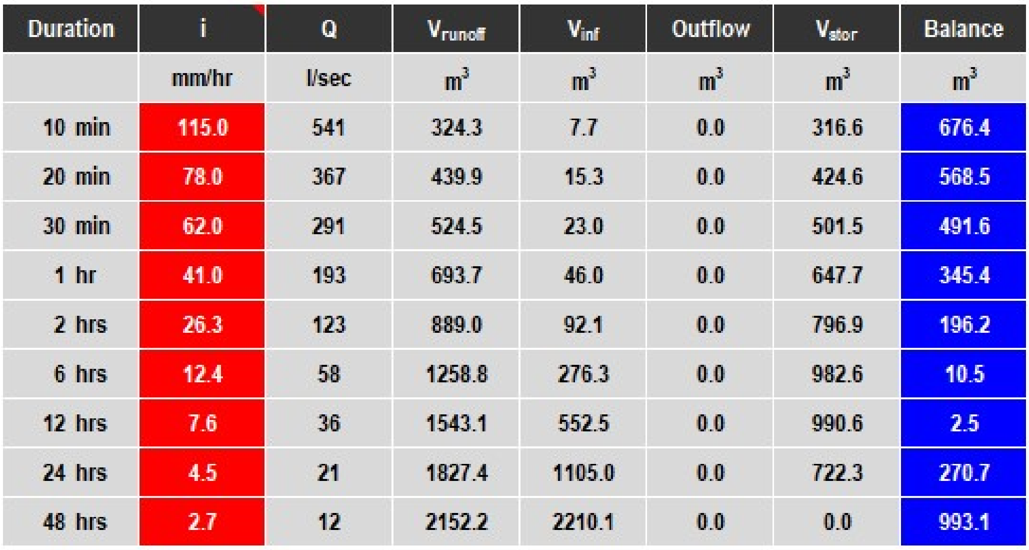
Public device sizing – 10 year ARI (roads) – 70 mm/hr

Dimensions



L	35.75	m	Area	614.9	m^2	ΣCA	16,920	m^2
W	17.20	m	Vol (gross)	1045.33	m^3	Inf Rate	2.08E-05	m/sec
D	1.7	m	Vol (net)	993.06	m^3	Constant Outflow		l/s

Intensity and Critical Storm



Duration	i	Q	V_{runoff}	V_{inf}	Outflow	V_{stor}	Balance
	mm/hr	l/sec	m^3	m^3	m^3	m^3	m^3
10 min	115.0	541	324.3	7.7	0.0	316.6	676.4
20 min	78.0	367	439.9	15.3	0.0	424.6	568.5
30 min	62.0	291	524.5	23.0	0.0	501.5	491.6
1 hr	41.0	193	693.7	46.0	0.0	647.7	345.4
2 hrs	26.3	123	889.0	92.1	0.0	796.9	196.2
6 hrs	12.4	58	1258.8	276.3	0.0	982.6	10.5
12 hrs	7.6	36	1543.1	552.5	0.0	990.6	2.5
24 hrs	4.5	21	1827.4	1105.0	0.0	722.3	270.7
48 hrs	2.7	12	2152.2	2210.1	0.0	0.0	993.1

Notes.

- 10 year is eq, to double 2 year flow – therefore can assume lot runoff is the 2 year.
- Assume swale volume - gross

Additional Volume Required for Public System (10 year -2 year ARI)

Catchment	Additional Volume Per Lot (m^3)	Number of Lots	Additional Public Storage (m^3)
Catchment 3 (lot overflow)	6.48	178	1153

Estimate of Soakage Trenches Volume

Catchment	Swale Length (m)	Base Width (m)	Average Depth (m)	Volume (m^3)
Catchment 3	250	1.2	0.5	150

Total Volume for Public System (Roads + Lot Excess)

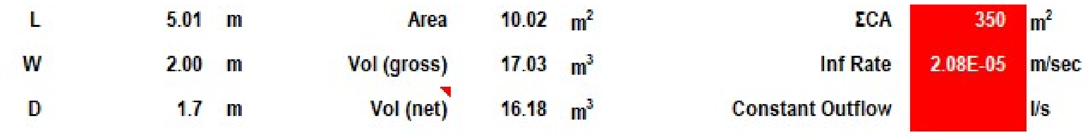
Catchment	Additional Volume From Lots (m^3)	Volume Required for Roads (m^3)	Swale Volume (m^3)	Additional Public Storage (m^3)
Catchment 3	1153	993	150	**1996**

Catchment 4 Source Control Estimates

Catchment 1 – Proposed Layout under Integrated SW Design Principles	Quantity	Units
Residential development area (assume 85% of total residential area of lots)	17.4	ha
Road and access way area (assume roads and foot paths is 15% of total residential area)	3.1	ha
Open space/park land area	2.8	ha
Native bush area	10.0	ha
Total area	33.2	Ha
Assumed number of lots dwelling count (assume an average lot size of 600 sq.m)	289	No.
Assumed area of impervious per lot (250 sq.m with 100 sq.m for patio/driveway)	0.035	Ha
Total Impervious lot area for residential development area	10.1	Ha
Percentage lot impervious surface	58	%
Road imperious area (assume 80% of road reserve)	2.5	ha
Total impervious area for Catchment 1	12.6	ha
Total fraction impervious for Catchment 1	38	%
Catchment 1 – Comparison from Traditional Development FI	**Quantity**	**Units**
Conventional housing impervious values	50% (as per district plan)	
Number of houses if allow 600sq metre lots	289	No.
Include houses in drainage reserve area	110	No.
Include road in drainage reserve area	3.53	Ha
Total houses in conventional build	399	no.
Total impervious area if conventional build	60	%
% reduction area FI from conventional development	22	%
Site disturbance reduced from a conventional development approach		
Catchment 1 – Comparison of Disturbed Area	**Quantity**	**Units**
Proposed disturbed area	20.4	ha
Conventional disturbed area	33.17	ha
Reduction disturbed area	12.75	ha
% reduction disturbed area	38	%

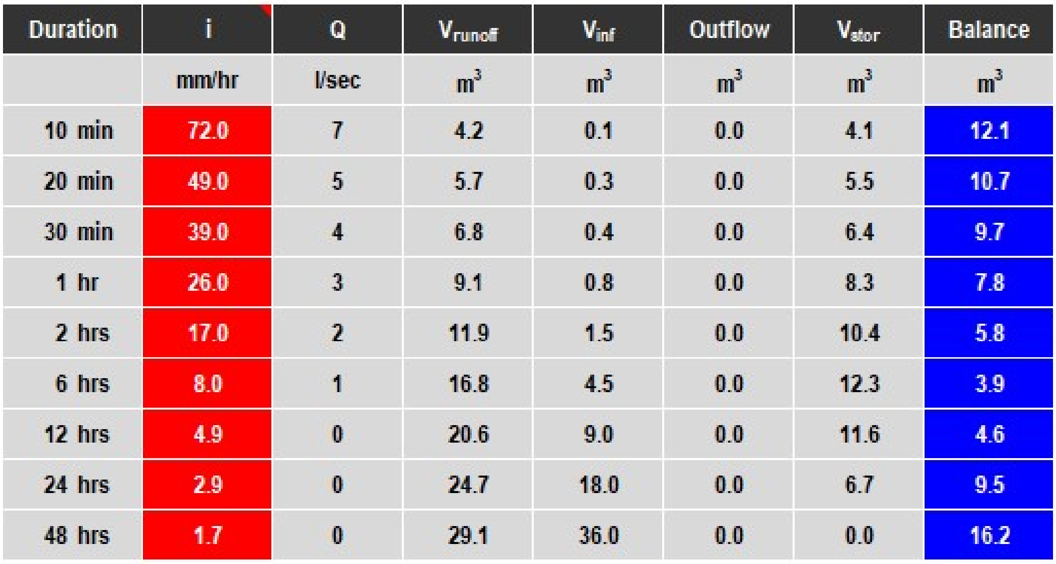
On lot device sizing – 2 year ARI – 70 mm/hr

Dimensions



L	5.01	m	Area	10.02	m^2	ΣCA	350	m^2
W	2.00	m	Vol (gross)	17.03	m^3	Inf Rate	2.08E-05	m/sec
D	1.7	m	Vol (net)	16.18	m^3	Constant Outflow		l/s

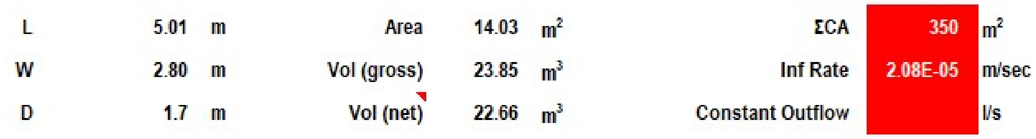
Intensity and Critical Storm



Duration	i	Q	V_{runoff}	V_{inf}	Outflow	V_{stor}	Balance
	mm/hr	l/sec	m^3	m^3	m^3	m^3	m^3
10 min	72.0	7	4.2	0.1	0.0	4.1	12.1
20 min	49.0	5	5.7	0.3	0.0	5.5	10.7
30 min	39.0	4	6.8	0.4	0.0	6.4	9.7
1 hr	26.0	3	9.1	0.8	0.0	8.3	7.8
2 hrs	17.0	2	11.9	1.5	0.0	10.4	5.8
6 hrs	8.0	1	16.8	4.5	0.0	12.3	3.9
12 hrs	4.9	0	20.6	9.0	0.0	11.6	4.6
24 hrs	2.9	0	24.7	18.0	0.0	6.7	9.5
48 hrs	1.7	0	29.1	36.0	0.0	0.0	16.2

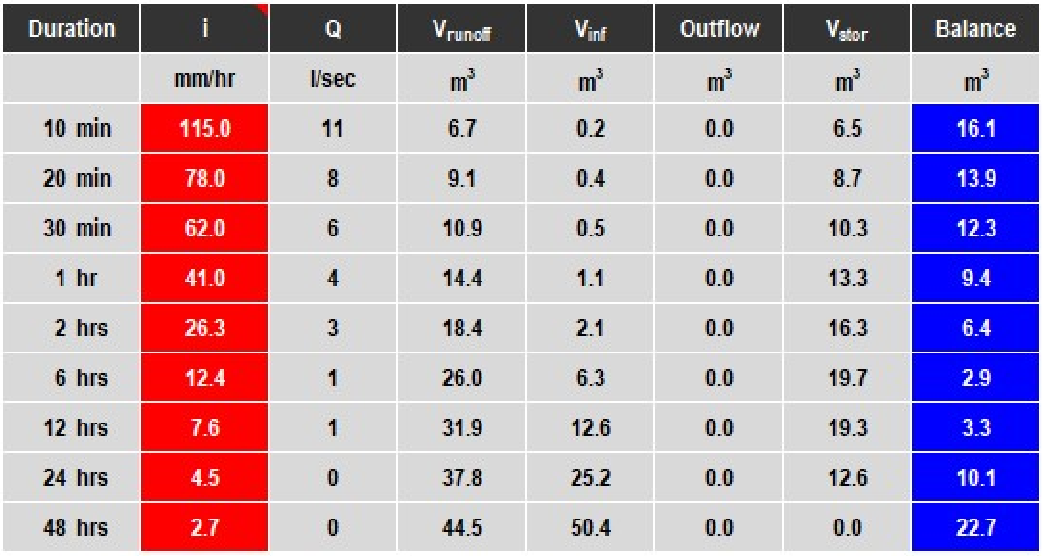
On lot device sizing – 10 year ARI – 70 mm/hr

Dimensions



L	5.01	m	Area	14.03	m^2	ΣCA	350	m^2
W	2.80	m	Vol (gross)	23.85	m^3	Inf Rate	2.08E-05	m/sec
D	1.7	m	Vol (net)	22.66	m^3	Constant Outflow		l/s

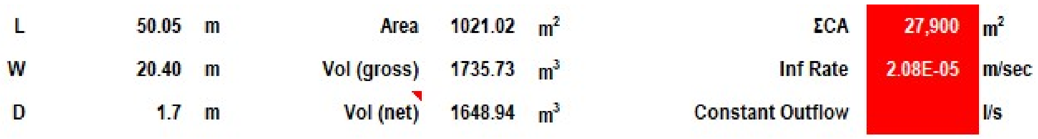
Intensity and Critical Storm



Duration	i	Q	V_{runoff}	V_{inf}	Outflow	V_{stor}	Balance
	mm/hr	l/sec	m^3	m^3	m^3	m^3	m^3
10 min	115.0	11	6.7	0.2	0.0	6.5	16.1
20 min	78.0	8	9.1	0.4	0.0	8.7	13.9
30 min	62.0	6	10.9	0.5	0.0	10.3	12.3
1 hr	41.0	4	14.4	1.1	0.0	13.3	9.4
2 hrs	26.3	3	18.4	2.1	0.0	16.3	6.4
6 hrs	12.4	1	26.0	6.3	0.0	19.7	2.9
12 hrs	7.6	1	31.9	12.6	0.0	19.3	3.3
24 hrs	4.5	0	37.8	25.2	0.0	12.6	10.1
48 hrs	2.7	0	44.5	50.4	0.0	0.0	22.7

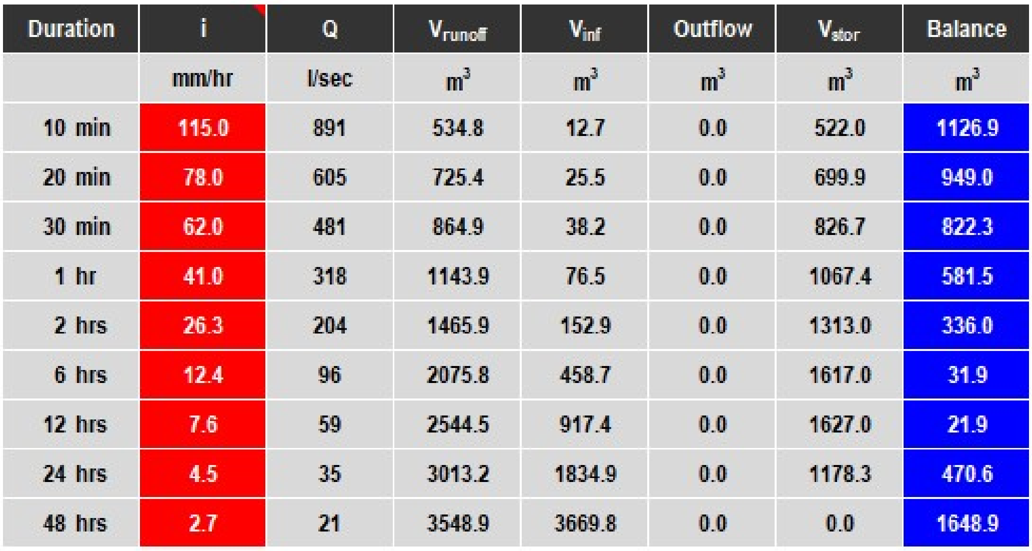
Public device sizing – 10 year ARI (roads) – 70 mm/hr

Dimensions



L	50.05	m	Area	1021.02	m^2	ΣCA	27,900	m^2
W	20.40	m	Vol (gross)	1735.73	m^3	Inf Rate	2.08E-05	m/sec
D	1.7	m	Vol (net)	1648.94	m^3	Constant Outflow		l/s

Intensity and Critical Storm



Duration	i	Q	V_{runoff}	V_{inf}	Outflow	V_{stor}	Balance
	mm/hr	l/sec	m^3	m^3	m^3	m^3	m^3
10 min	115.0	891	534.8	12.7	0.0	522.0	1126.9
20 min	78.0	605	725.4	25.5	0.0	699.9	949.0
30 min	62.0	481	864.9	38.2	0.0	826.7	822.3
1 hr	41.0	318	1143.9	76.5	0.0	1067.4	581.5
2 hrs	26.3	204	1465.9	152.9	0.0	1313.0	336.0
6 hrs	12.4	96	2075.8	458.7	0.0	1617.0	31.9
12 hrs	7.6	59	2544.5	917.4	0.0	1627.0	21.9
24 hrs	4.5	35	3013.2	1834.9	0.0	1178.3	470.6
48 hrs	2.7	21	3548.9	3669.8	0.0	0.0	1648.9

Notes:

- 10 year is eq, to double 2 year flow – therefore can assume lot runoff is the 2 year.
- Assume swale volume - gross

Additional Volume Required for Public System (10 year -2 year ARI)

Catchment	Additional Volume Per Lot (m^3)	Number of Lots	Additional Public Storage (m^3)
Catchment 4 (lot overflow)	6.48	289	1872

Estimate of Soakage Trenches Volume

Catchment	Swale Length (m)	Base Width (m)	Average Depth (m)	Volume (m^3)
Catchment 4	550	1.2	0.5	330

Total Volume for Public System (Roads + Lot Excess)

Catchment	Additional Volume From Lots (m^3)	Volume Required for Roads (m^3)	Swale Volume (m^3)	Additional Public Storage (m^3)
Catchment 4	1872	1648	330	**3190**