Rongotea Private Plan Change

Three Waters Assessment

Te Kapiti Trust

9 November 2022

→ **The Power of Commitment**

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The Power of Commitment

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1. Introduction

1.1 Background

GHD has been engaged by Te Kapiti Trust (through The Property Group, TPG) to provide a Three Waters Assessment (i.e., potable water, wastewater, stormwater/flooding) in support of a proposed Private Plan Change at 14 Banks Road, Rongotea. The plan change seeks to rezone approximately 21 hectares of land owned by Te Kapiti Trust for residential development. The site is currently zoned as “rural” under the Manawatū District Council (MDC) District Plan and the proposal is to rezone the area to a “Village Zone”. An adjacent 10 hectare lot, also owned by Te Kapiti Trust, is available to be utilised for stormwater management in support of the primary development site. A draft structure plan provided by TPG was used for this investigation as shown in Figure 1 below.

Figure 1 Draft Structure Plan from The Property Group

1.2 Purpose of this report

The purpose of this report is to:

– Document the investigations undertaken as part of the high-level three waters serviceability assessment to determine if growth within the proposed plan change area can be accommodated with the existing infrastructure or with feasible upgrades.
– Assess the 0.5% AEP flood risk to the site for assessment under the One Plan, including floodplain modelling as required by Horizons Regional Council.
– Provide concept-level sizing of core three waters infrastructure for the plan change area, including water trunk mains, sewer trunk mains, stormwater mains and stormwater treatment and attenuation area(s), to inform the structure plan development and servicing feasibility assessment.

This report also documents the key assumptions made and methodology used for the various assessments.

1.3 Scope and limitations

This report: has been prepared by GHD for Te Kapiti Trust and may only be used and relied on by Te Kapiti Trust for the purpose agreed between GHD and Te Kapiti Trust as set out above.

GHD otherwise disclaims responsibility to any person other than Te Kapiti Trust arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report (refer section 1.4 of this report). GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Te Kapiti Trust, Manawatu District Council, and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

GHD has not been involved in the preparation of the Urban Design Framework or any other plan change documents developed for the Rongotea Private Plan Change, and has had no contribution to, or review of, these documents other than in this Three Waters Assessment. GHD shall not be liable to any person for any error in, omission from, or false or misleading statement in, any other part of the Urban Design Framework and other plan change documents for the Rongotea Private Plan Change.

1.4 Assumptions

The assessment carried out as part of this report are based on the following key assumptions:

General

– All information provided by MDC and other data sources are correct and current. No attempts were made to validate the data.
– The investigations done as part of this report are high-level only and are not suitable for detailed design purposes.

Water Supply

– Fully developed, Rongotea has an average household occupancy rate of 2.8 people per property (as per MDC Engineering Standards for Land Development 2017).
– Average Daily Demand = 290 l/h/d (as per MDC Engineering Standards for Land Development 2017).
– Peaking Factor (PF) – peak day demand = 2 for populations below 2,000.
– Peaking Factor (PF) – peak hourly demand = 5 for populations below 2,000.
– Reticulation pipe location and sizes of the existing infrastructure are as per the as-built drawings provided by MDC.
– All properties are currently connected to the reticulation network.
– The fire pump cannot run concurrently with the duty pump as their shutoff head and relative speed varies.

Wastewater

– Fully developed, Rongotea will have an average occupancy rate of 2.9 people per property (as per MDC Engineering Standards for Land Development 2017).

- Average dry weather flow (ADWF) from Rongotea is 290 l/h/d (as per MDC Engineering Standards for Land Development 2017).
- Dry weather peaking factor = 2.5 x ADWF (design parameter given in NZS 4404:2010).
- Wet weather peaking factor = 2 x PDWF (design parameter given in NZS 4404:2010).
- All properties are currently connected to the reticulation network.
- All properties are residential type wastewater users, except for the Rongotea School. A wastewater production of 30 litres/child/school day have been assumed based on previous investigations completed by GHD. The number of students was obtained from the Rongotea School website.

Flooding

- Culverts and drainage structures upstream of the development site are adequately sized and do not restrict flow.
- Upstream culverts are blockage and sediment free.
- The Digital Elevation Model (DEM) is representative of existing terrain. Fences or walls that may constrict or divert flow paths are not resolved in the DEM and thus not resolved in the flood model.
- The size and depth of existing drainage channels is adequately represented in the DEM.
- The flood model has not been validated or calibrated.

Stormwater

- Except for filling in gullies on the site, the finished ground will generally maintain existing elevations and falls. Final grades of the road reserves will allow drainage of overland flow paths to a centralised wetland and attenuation facilities.
- The soils are suitable for drainage swales.
- The soils are suitable for the stormwater wetland and attenuation pond design.
- The extent of stormwater management required to meet the Resource Consent for the discharge of stormwater includes stormwater treatment and ‘hydraulic neutrality’. Hydraulic neutrality has been applied as limiting post-development peak run-off from the site to pre-development peak run-off.
- The impervious area percentages provided by the Property Group are appropriate and representative for the development.
- Horizons will agree to recontouring of the floodplain and realignment of part of the Campbells Drain (if required) on the adjacent property for stormwater use.

1.5 Site Visit

As part of this high-level three waters assessment, a site visit was carried out by GHD staff on 3 June 2022. The purpose the site visit was for site familiarisation and to collect basic input data and measurements to support the three waters assessment. GHD staff also visited MDC’s water treatment and pump station to collect information on the water distribution network and pump configuration. To support the flooding and stormwater assessment, key culverts were measured on Kellow Road, Rongotea Road and on the farm road running along the west perimeter of the private plan change area.

2. Stormwater

2.1 Catchment Description

The plan change area is located in the upstream reaches of Horizon's Te Kawau drainage scheme, as shown in Figure 2. The drainage scheme has a total area of 14,024 ha, and the 20.7 ha private plan change area site represents approximately 0.15% of this total area. The scheme ultimately discharges to the Oroua river approximately 15 km downstream of the private plan change area. The private plan change area is shown in relation to the Te Kawau drainage channels in Figure 3. The confluence of the Ruivaldts and Walsh drains is located on the north boundary of the site. This channel drains south through the plan change site to its confluence with the Campbells drain on the southern boundary of the plan change site. The Campbells drain continues south through the adjacent property that has been identified for stormwater use and toward Banks Rd.

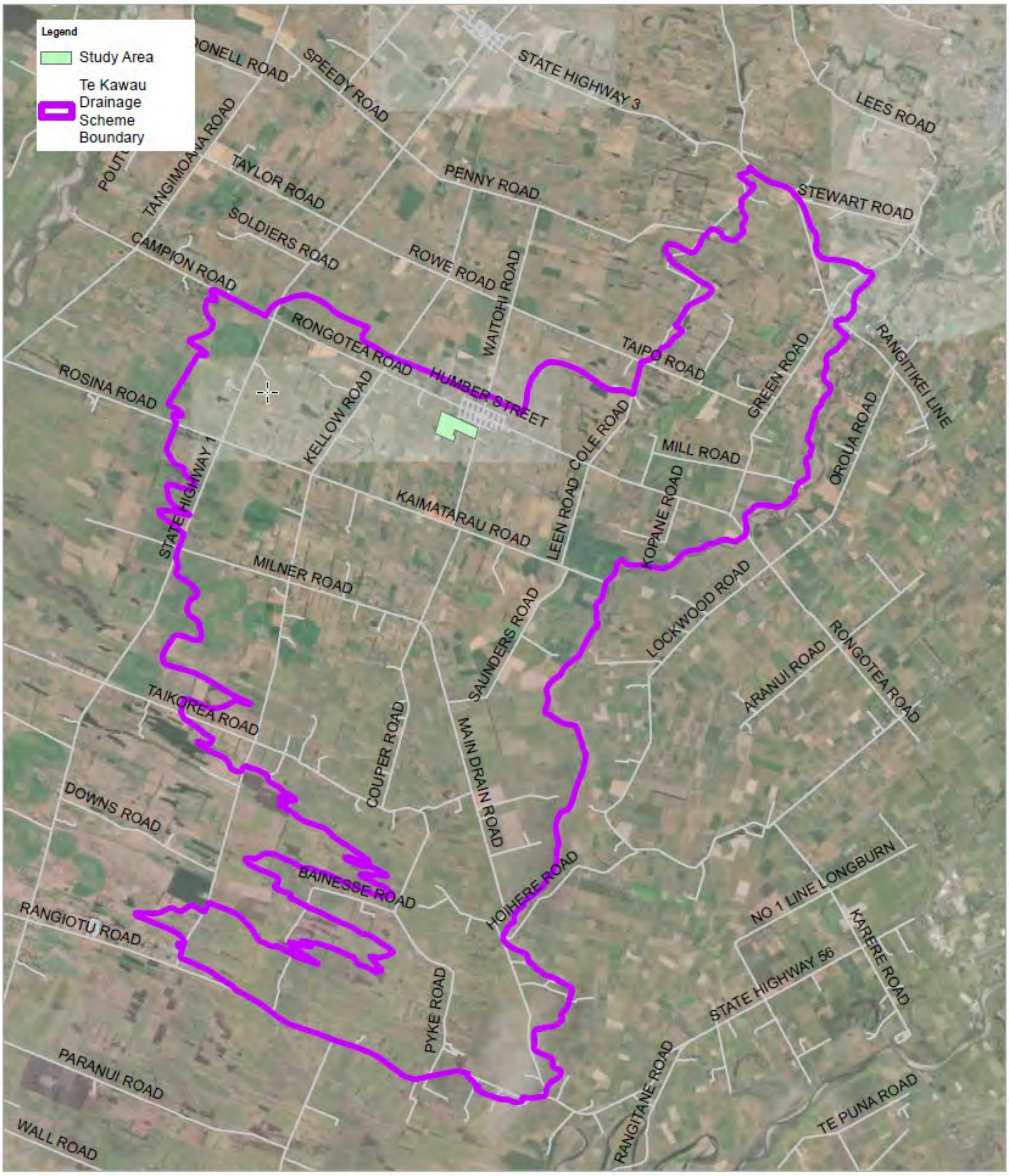


Figure 2 Te Kawau Drainage Scheme

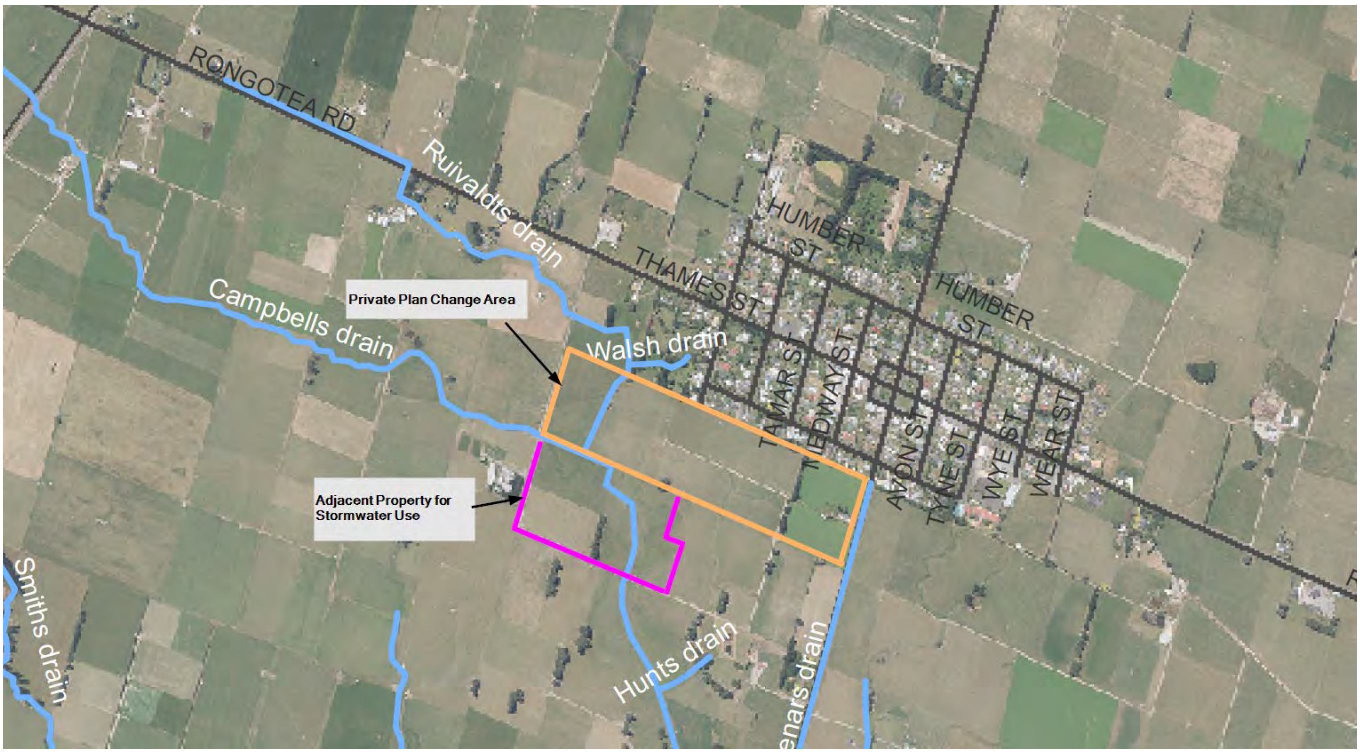


Figure 3 ***Private plan change area in relation to Te Kawau Drainage Scheme drains***

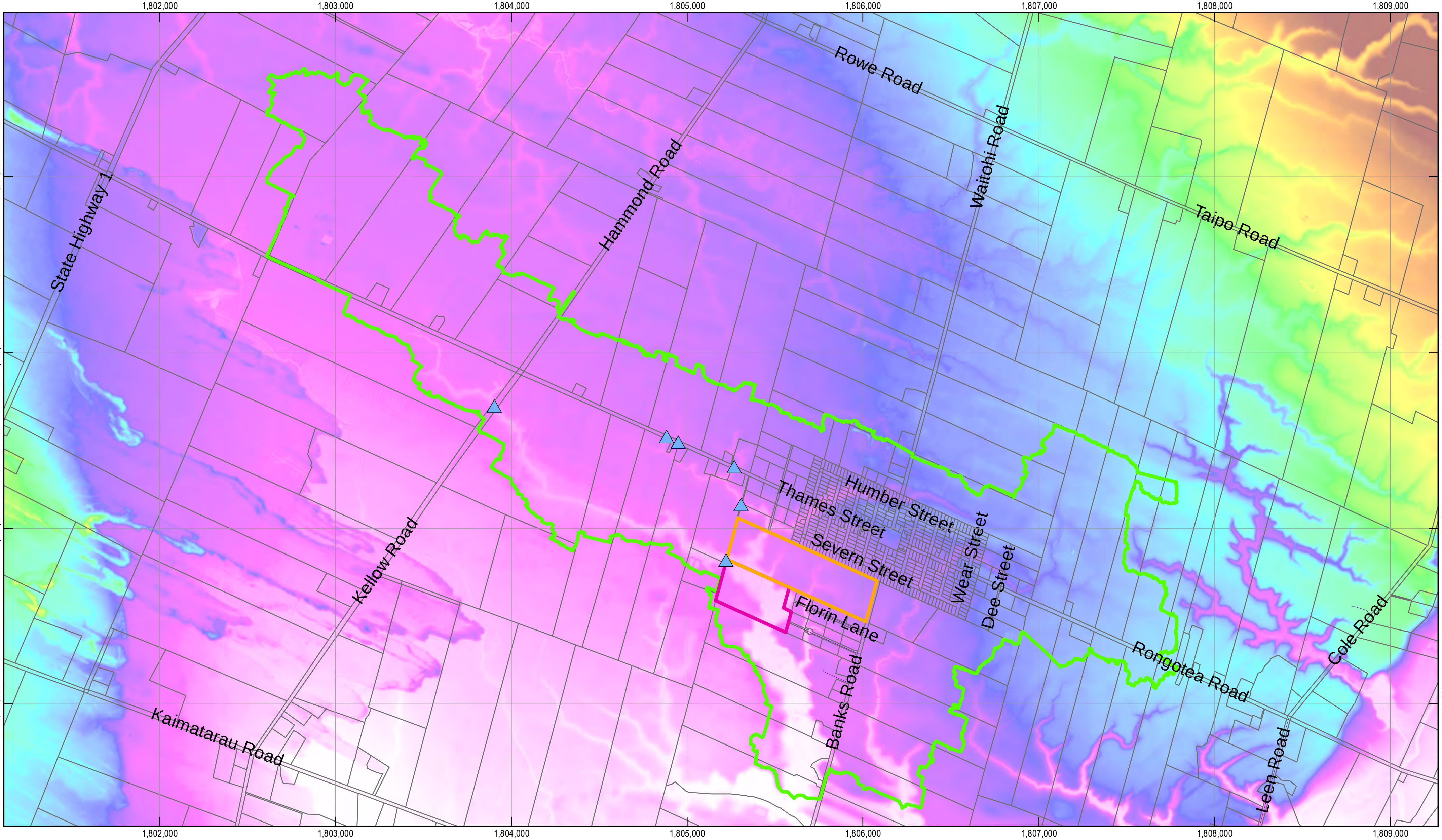
2.2 Flood Assessment

2.2.1 Flood Modelling

Flood mapping in the vicinity of the plan change area has not yet been undertaken by Horizons. To support the private plan change, a flood model was developed in HEC-RAS, a modelling tool developed at the Hydrologic Engineering Center (HEC), which is a division of the U.S. Army Corps of Engineers. The River Analysis System is used to perform one-dimensional steady flow hydraulics and one and two-dimension unsteady flow river hydraulics calculations, amongst other things, and is used to perform floodplain encroachment analyses.

The flood model included 2D surface flows for the catchment to the north of and draining into the plan change area, and extended approximately 1 km south of the plan change area. The extent of the flood model is shown in Figure 4. A grid size of 20 m^2 was used for the wider model area, with refined areas of 5 m^2 on the plan change site, adjacent site for stormwater use and for the channels draining into the site.

Except for key culverts, the model included 2D surface flows only; the existing reticulation network in the urban area of Rongotea was not included. Culverts upstream of the site were measured during the site visit (see section 1.5) and included in the model. These included the culverts at Kellow Road, Rongotea Road and the farm road running along the west boundary of the site and are shown in Figure 4. In other areas, such as through the urban area in Rongotea and downstream of the site, basic terrain editing was carried out to burn channels into the DEM and to ensure continuous flow representative of the actual site conditions.



Paper Size A4

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Metres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

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LEGEND
Primary Plan Change Area
Adjacent Property for Stormwater Use
Model Boundary
Culverts

Ground Elevation (m NZVD2016)
High : 63.748
Low : 15.149

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Three Waters Assessment

Flood Model Extent

Job Number	12581369
Revision	A
Date	14 Jun 2022

Figure 4

\\ghdnet\ghd\NZ\Palmerston North\Projects\51\12581369\GIS\Maps\Working\Report Figures\Figure X - Flood Model.mxd

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Data source: Data Custodian, Data Set Name/Title, Version/Date. Created by:cmurray2

The SCS curve number method was used to model hydrology in the catchment. Type D soils were assumed in the catchment even though the surrounding area is predominantly silty loams (based on soil mapping available from LINZ). This is conservative but likely represents the behaviour of wet saturated soils typically observed in the winter months. Land use in the catchment was delineated based on aerial photography and preliminary results for the overland flow paths. Manning's N and the SCS curve number was assigned based on land use and the values in Table 1.

Table 1 Manning's N roughness and SCS curve number for Rongotea flood model

Land Use Type	Manning's N	SCS Curve Number
Urban	0.06	87
Rural	0.1	80
Open Channels/Overland Flow Paths	0.035	80

In accordance with Horizons Regional Council's One Plan, flooding modelling of the site was determined for the 1-in-200-year annual recurrence interval (ARI) rainfall event. Rainfall hyetographs were generated using rainfall intensities obtained from NIWA's High Intensity Rainfall Design System (HIRDS V4.0) extracted in the centre of Rongotea. Several 200-year synthetic rainfall events were constructed and simulated to determine the event that produced the most conservative flood extents. These included the TP106 24-hour nested storm and NIWA's temporal storms for East of NI of varying durations (refer Figure 5). Ultimately the 6-hour NIWA storm produced the most conservative flood extents and has been carried forward.

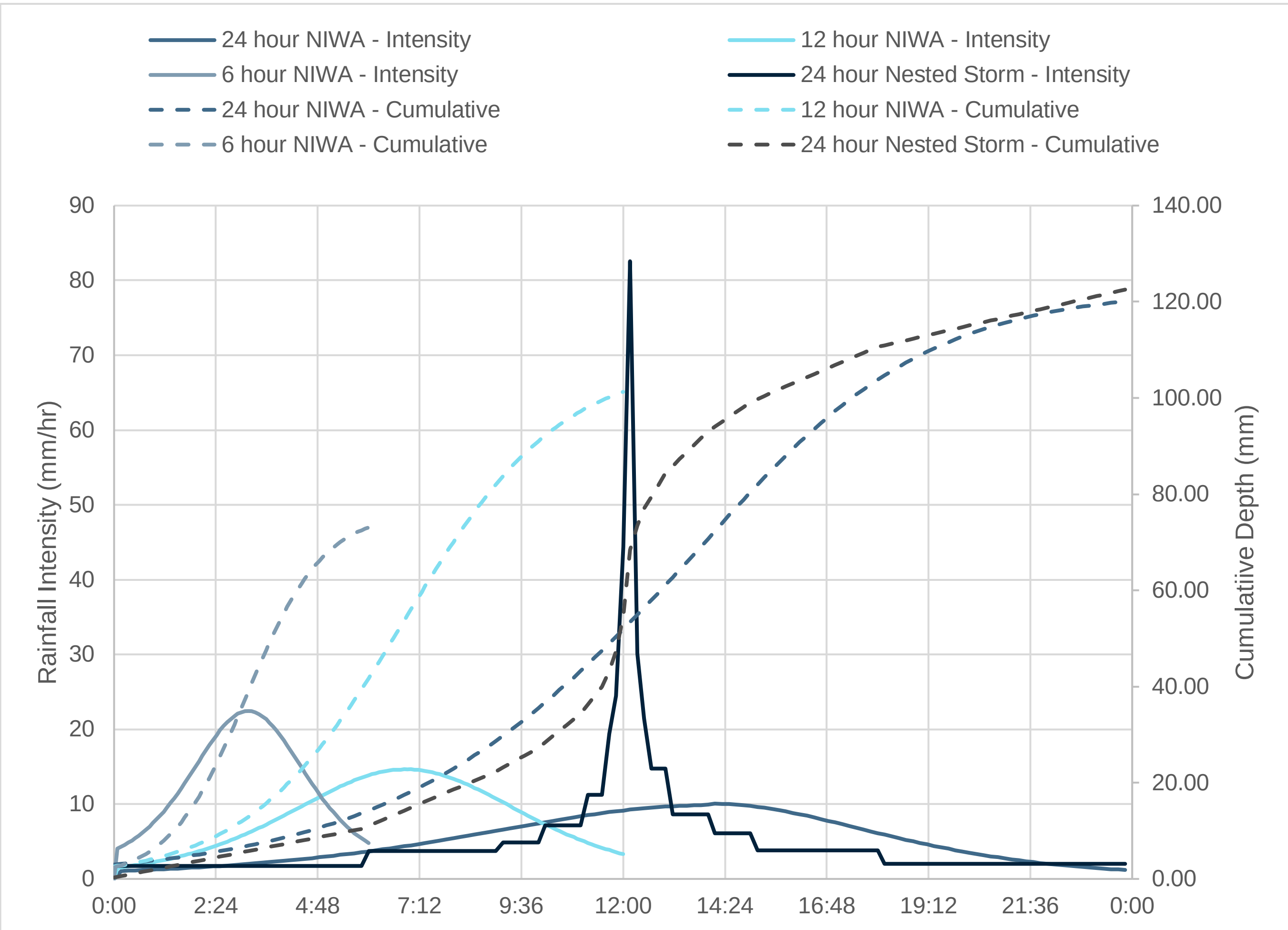
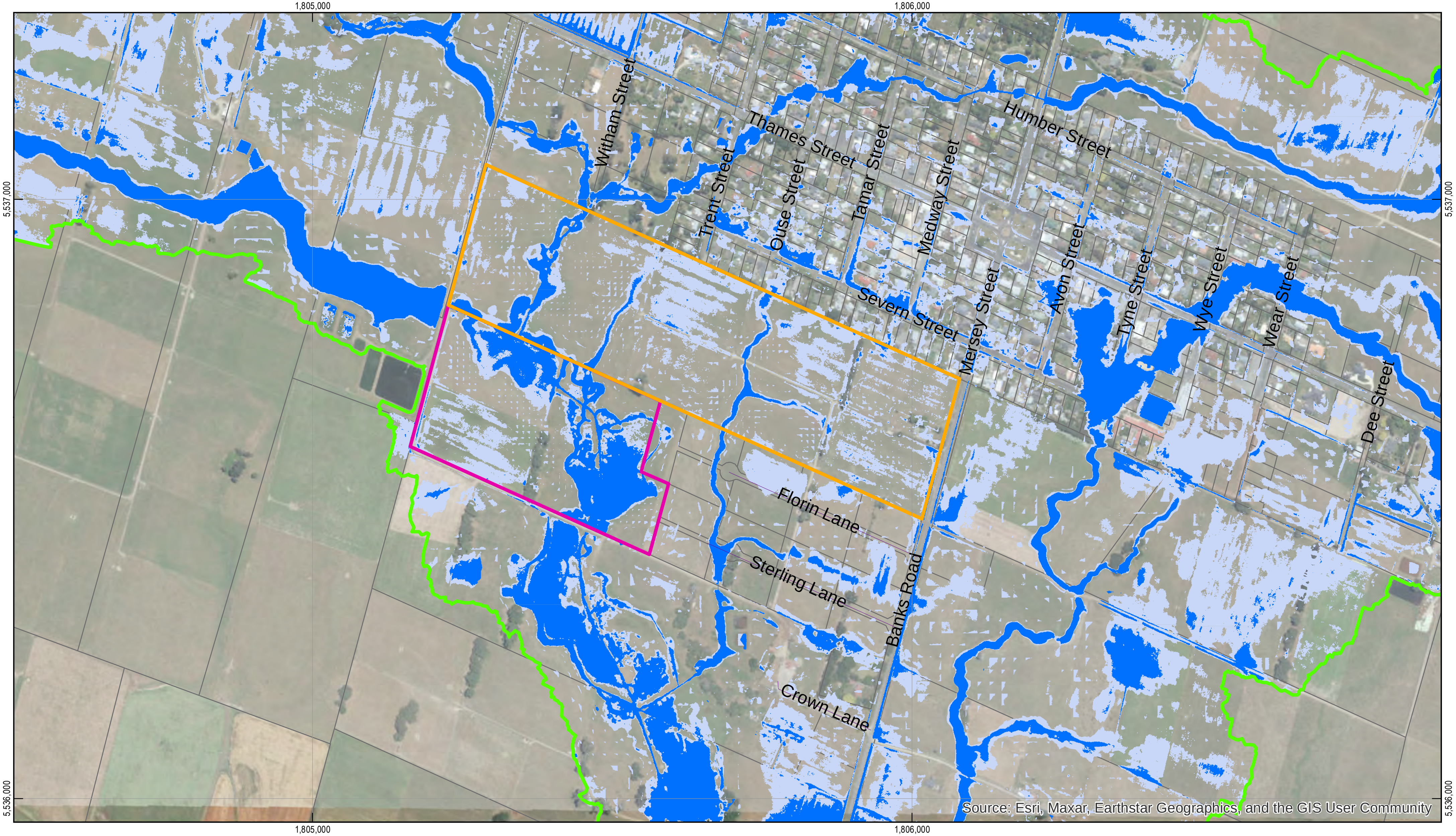


Figure 5 Comparison of rainfall events / hyetographs for flood assessment

2.2.2 Flood Risk

The 200-year flood extent results are shown in Figure 6. The darker blue areas indicate areas with inundation depths of 0.1 m or greater; the light blue areas are inundated areas with depths between 0 and 0.1 m. For the purposes of the conceptual design of the stormwater servicing, the lighter blue areas are considered nuisance surface ponding that would be addressed through site grading. The darker blue areas are indicative of drainage channels or overland flow paths that need to be addressed in the design.

In general, flooding is confined to the open drains that traverse the development site. There are three darker blue areas, or overland flow paths, predicted in the flood model. The most western is the Ruivaldts and Campbells drain. The two overland flow paths to the east of the drain (herein called “West” and “East” overland flow paths) are overland flow paths carrying run-off from the urban area of Rongotea. In order for development to occur these overland flow paths are to be managed in the stormwater design. The area surrounding the Ruivaldts and Campbells drain should also be identified as a flood prone area to prevent residential development or loss of floodplain storage.



Paper Size A4

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Metres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

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LEGEND

- Primary Plan Change Area
- Adjacent Property for Stormwater Use
- Model Boundary
- Inundation Extent - 0.1 m
- Inundation Extent - 0 m

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Job Number	12581369
Revision	A
Date	10 Jun 2022

200 year Inundation Extents

Figure 6

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2.3 Stormwater Assessment

The general approach and objectives for stormwater management at the site includes the following:

- Manage overland flow paths onto the site from the catchment upstream;
- Collect and convey all run-off generated on the site to centralised treatment and attenuation facilities; and
- Discharge treated and attenuated flow to the Campbells drain.

The Ruivaldts drain forms a divide through the site and means collecting all run-off to a single offline central location is not feasible. The area has therefore been delineated into two separate catchments: one to the west of the Ruivaldts drain, approximately 5.0 ha in size; the other to the east of the drain, approximately 15.6 ha in size. The east and west sub-catchments are shown in Figure 7 below.

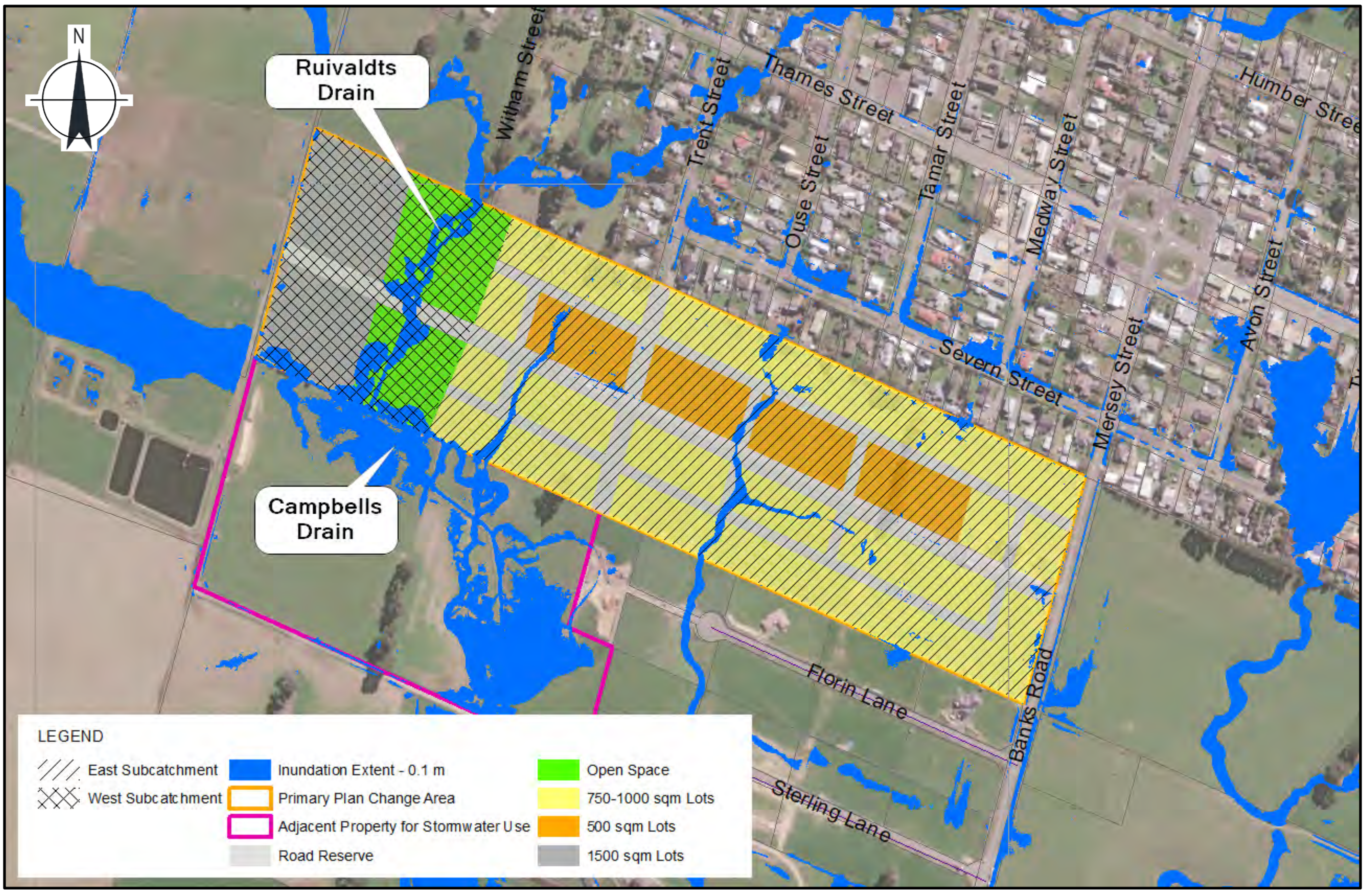


Figure 7 ***Structure plan showing west and east sub-catchments***

2.3.1 Management of Overland Flow Paths

The stormwater network has been sized to convey the runoff generated from the development site only. That is to ensure the piped infrastructure and stormwater treatment facilities are not overly sized. In order to do this while still maintaining overland flowpaths so as not to cause flooding upstream or downstream, runoff onto the development site from the urban area of Rongotea is proposed to be intercepted using cut-off drains along the northern boundary. A swale and bypass pipe have been sized to convey the predicted peak flow from the HEC-RAS model for the 200-year event (not including climate change). The size of the swale has been sized assuming a simple V-shaped drain with 3H:1V side slopes. The Ruivaldts drain will need to be culverted under the main east-west road running through the development. A twin 1050 mm diameter culvert has been sized to convey the 200-year event.

The concept for this overland flow bypass management is outlined in the table below and shown in Figure 8.

Table 2 Peak flow in drainage channels and overland flow paths through site

Overland Flow Path	200-year Peak Flow (m^3/s)	Details of Bypass/Management
Ruivaldts drain	3.57	Twin 1050 mm diameter culvert under new main road
West overland flow path	0.47	0.4 m deep V-shaped swale, with 3H:1V side slopes (2.4 m total width)
East overland flow path	0.58	0.5 m deep V-shaped swale, with 3H:1V side slopes (3 m total width) draining to scruffy dome. 600 mm diameter pipe.

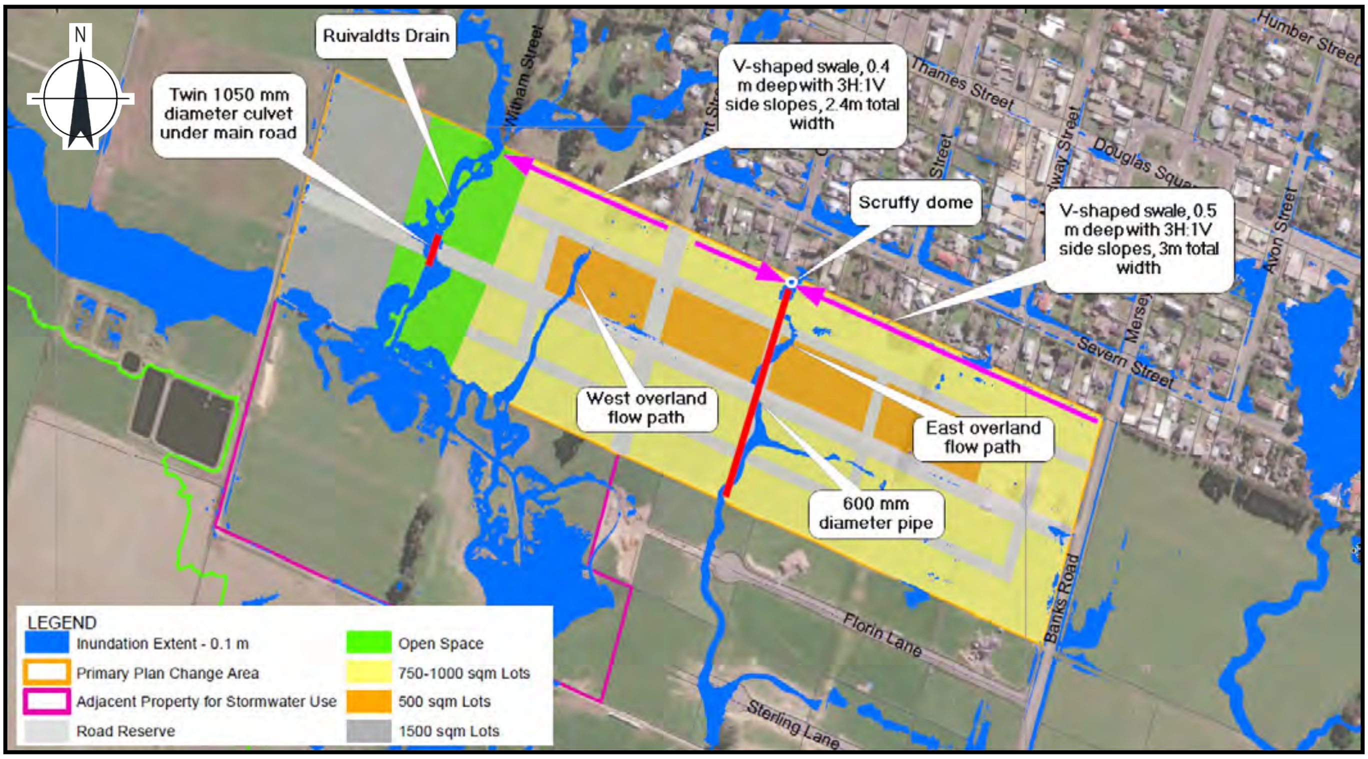


Figure 8 Management of overland flow paths through the plan change area

The east overland flow path currently flows through the private plan change area and through channels and culverts in the Florin and Sterling Lanes lifestyle block development. To maintain this feature and character, the baseflow through the development site to Florin and Sterling Lanes will need to be maintained. This can be accommodated with a DN600 pipe.

2.3.2 Stormwater Model Development

Model Development and Assumptions

A stormwater model was developed in PCSWMM to provide indicative sizing of the stormwater network and attenuation for the development site. PCSWMM software (Computational Hydraulics International, 2017) PCSWMM is a spatial decision support system for the U.S. Environmental Protection Agency SWMM 5 software. The model requires input of topographical features (catchment area, flow length, slope, hydraulic roughness), ground cover conditions (land use, vegetation cover), infiltration parameters (infiltration capacity, drainage time), rainfall (hyetograph), and drainage paths (channels, channel lengths, roughness) in order to effectively simulate the stormwater runoff conditions of a subject site.

Sub-catchments were delineated based on the structure plan in Figure 1. Catchment hydrology was based on the SCS curve method and the curve numbers for each land use type shown in Table 3. The impervious percentages for the three lot sizes (500 sq.m., 750-1000 sq.m. and 1500 sq.m.) were based on indicative planning control information provided by The Property Group.

Table 3 *Manning's N roughness and SCS curve number for conceptual stormwater design*

Land Use Type	Percentage Impervious	Manning's N	SCS Curve Number
Pre-development			
Pasture/Open Space	0%	0.1	74
Post-development			
500 sqm lots	50%	0.05	86
750 – 1000 sqm lots	50%	0.05	86
1500 sqm lot	35%	0.08	82
Open Space / Reserve Area	0%	0.1	74
Road Reserves	75%	0.02	92

Rainfall Events

Rainfall intensities from HIRDS v4.0 for RCP 6.0 (2081-2100) for Rongotea were used to generate synthetic rainfall events for the conceptual sizing of stormwater conveyance and attenuation facilities. Two storm profiles were considered: the AP106 nested storm event and NIWA temporal design storm profile. For the 10-year ARI event, only the AP106 nested storm profile was used. For the 100-year ARI events the nested storm profile was considered in addition to the 1, 6, 12 and 24-hour NIWA storm profiles. The nested storm event is a synthetic hyetograph which embeds numerous duration rainfall intensities, whereas the NIWA profiles are based on actual rainfall events analysed in the development of HIRDS v4.0. Design rainfall events used in the stormwater analysis are shown in Figure 9.

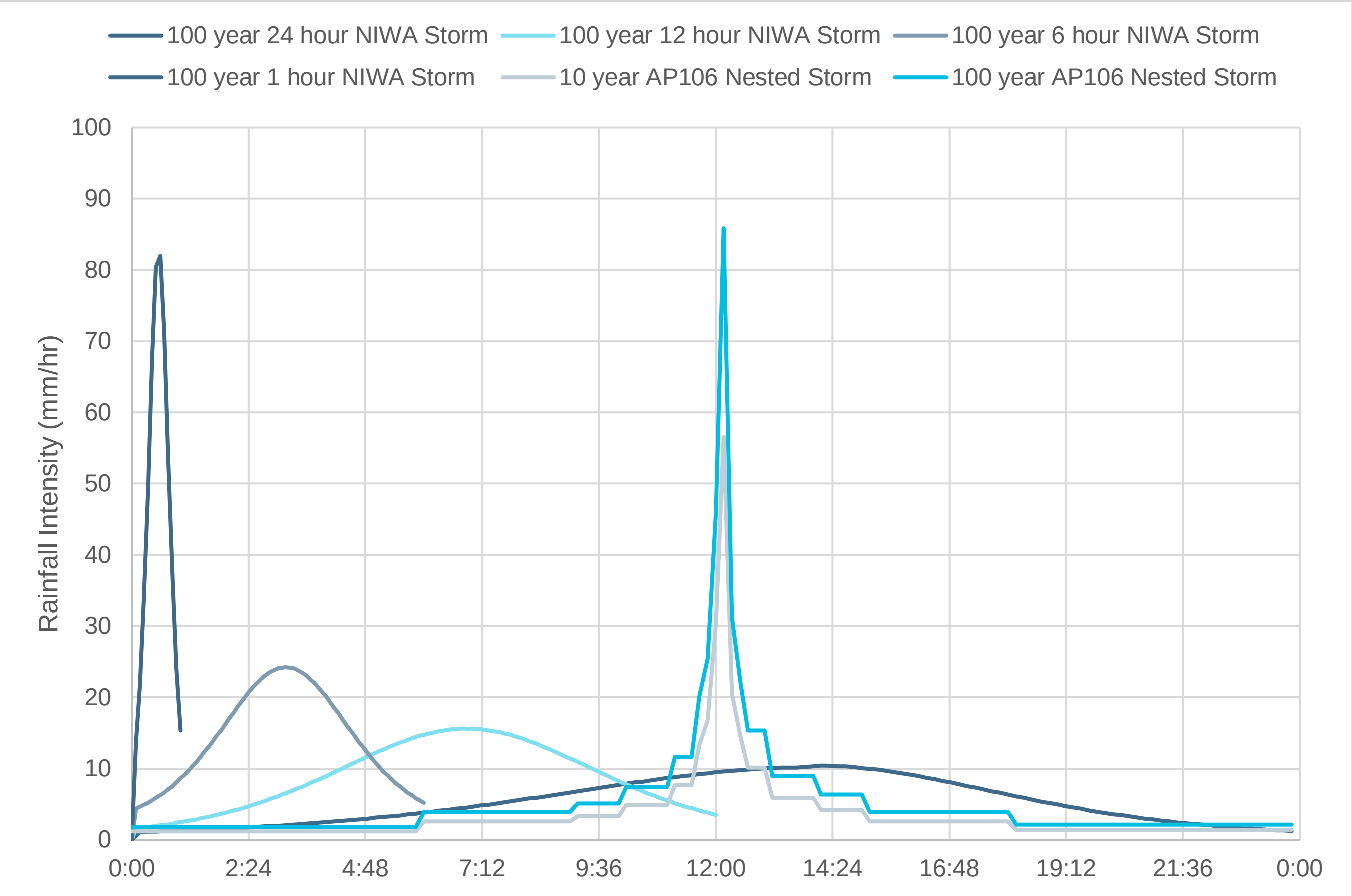


Figure 9 *Comparison rainfall hyetographs for 10 and 100-year ARI events*

2.3.3 Stormwater Analysis

Peak Runoff Flows

The PCSWMM model was used to determine the increase in run-off generated from the development. Table 4 presents the peak run-off flows for the pre- and post-development scenarios for the 10-year event and the various 100-year storm profiles (refer to Figure 9).

Table 4 Peak pre-development and post-development unattenuated flows for various storm profiles

	West Sub-Catchment		East Sub-Catchment	
100 year Storm Profile	**Pre-development Peak Runoff (m^3/s)**	**Post-development Peak Runoff (Unattenuated) (m^3/s)**	**Pre-development Peak Runoff (m^3/s)**	**Post-development Peak Runoff (Unattenuated) (m^3/s)**
10 year event				
AP106 24 hour Nested Design Storm	0.15	0.20	0.36	1.28
100 year event				
AP106 24 hour Nested Design Storm	0.35	0.42	0.84	1.81
24 hour NIWA Storm	0.10	0.11	0.31	0.40
12 hour NIWA Storm	0.14	0.15	0.40	0.57
6 hour NIWA Storm	0.18	0.20	0.48	1.11
1 hour NIWA Storm	0.16	0.22	0.35	1.54

Runoff Volumes

Runoff volumes were also determined for the pre- and post-development scenarios for the 10-year ARI design event and various 100-year storm profiles. Table 5 presents the total run-off volumes for the west and east sub-catchments.

Table 5 Peak pre-development and post-development runoff volumes for various storm profiles

	West of Drainage Channel		East of Drainage Channel	
100 year Storm Profile	**Pre-development Runoff Volume (m^3)**	**Post-development Runoff Volume (Unattenuated) (m^3/s)**	**Pre-development Runoff Volume (m^3)**	**Post-development Runoff Volume (Unattenuated) (m^3/s)**
10 year event				
AP106 24 hour Nested Design Storm	1939	2279	5768	8916
100 year event				
AP106 24 hour Nested Design Storm	3635	4104	10910	15000
24 hour NIWA Storm	3634	4102	10880	15150
12 hour NIWA Storm	2844	3285	8510	12320
6 hour NIWA Storm	1992	2370	5851	9328
1 hour NIWA Storm	440	658	1069	3650

2.4 Stormwater Management for East Sub-catchment

2.4.1 Overview

The ecology assessment carried out by Forbes Ecology (12 July 2022) identified an existing natural inland wetland along the Campbells drain in the adjacent lot to the south set aside for stormwater use. Any activity in this area will need to consider the implications under the National Environmental Standard for Freshwater 2020 (NES-F), which includes treatment of stormwater discharge.

Initially, a single stormwater concept, Option A, was developed for the development to the east of the Ruivaldts/Walsh drain. In Option A, the wetland and attenuation facility were located on the adjacent lot for stormwater use and overlapping the existing natural inland wetland. Although the facility for the development would be separate from the natural wetland, it would also allow for opportunities to improve and enhance the existing natural inland wetland.

The plan change application was submitted with Option A on 29 July 2022 and Council requested further information under s92 of the RMA on 8 September 2022. Included in this was a request to consult further with Horizon's Regional Council over the stormwater aspects of the plan change application. Concern was raised on the proposed location of the constructed wetland for stormwater treatment and attenuation that is adjacent to the area identified as a 'natural inland wetland' by Forbes Ecology. That is, Council wanted to understand how Horizons Regional Council would view an application to discharge stormwater at this location, considering the NES-F.

The NES-F at Part 3, clause 54 states that the taking, use, damming, diversion, or discharge of water within, or within a 100m setback from a natural wetland is a non-complying activity. A non-complying activity under the Resource Management Act requires a more stringent test to be passed. Council would need to be satisfied that the adverse effects of the activity on the environment would be minor, and that the activity will not be contrary to the objectives and policies of the Horizons' One Plan.

The discussions on the difficultly that the NES-F poses for the discharge of stormwater at this location resulted in a request for an alternate location to be considered that would result in the area assessed as 'natural inland wetland' to remain undisturbed. This resulted in the development of an Option B. For Option B, the stormwater pond has been located at least 100 m from the natural inland wetland identified by Forbes Ecology (2022).

The general topography of the study area is shown in Figure 10, which includes the natural inland wetland extent identified by Forbes Ecology and a 100 m offset from the natural inland wetland extent.

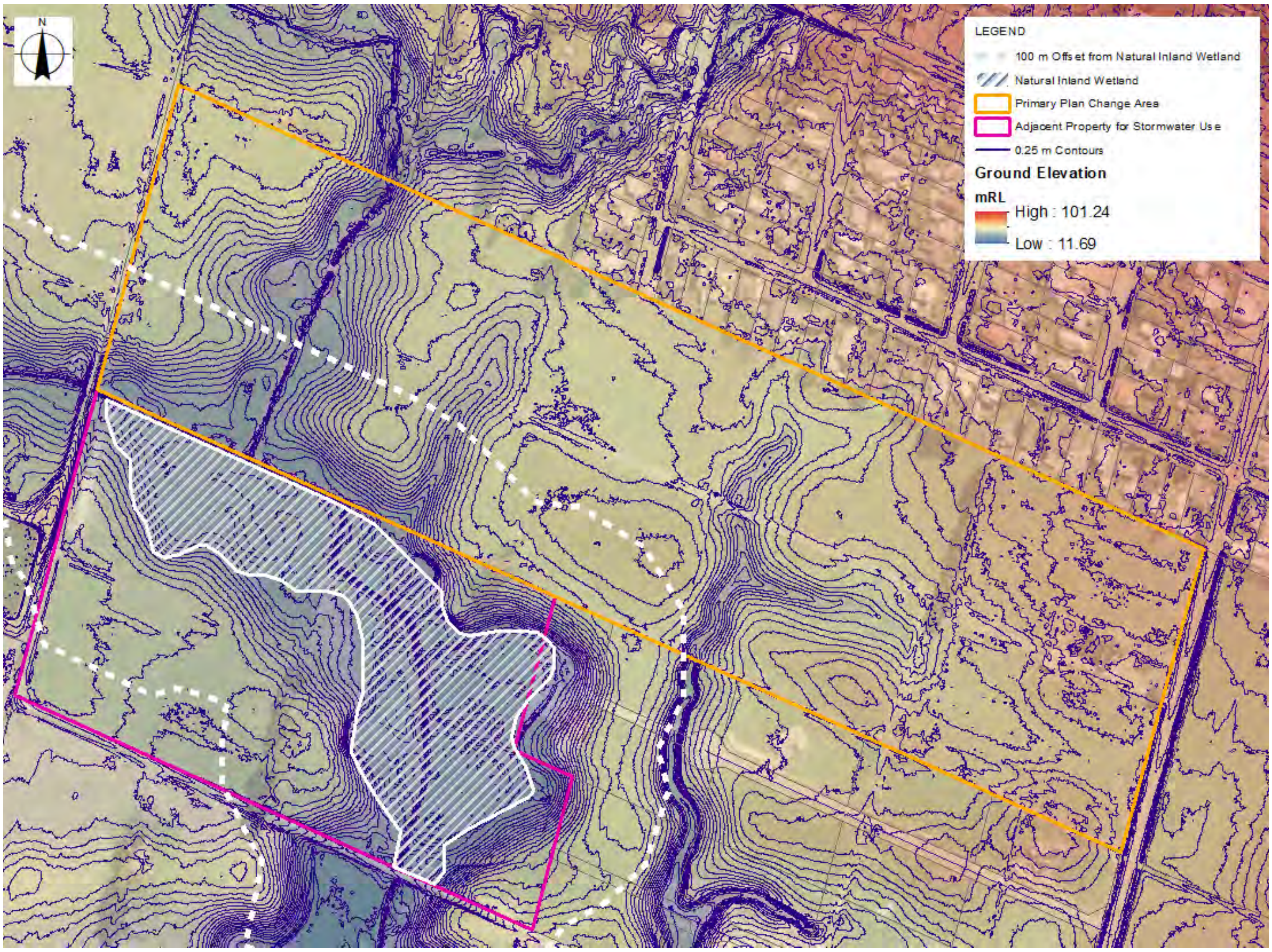


Figure 10 ***General topography of study area with natural inland wetland extent and 100 m offset from natural inland wetland***

The general approach used to develop both stormwater options is described below.

Conveyance

Initial engagement with MDC indicated a likely preference for a piped stormwater network over swales. This was mainly due to concerns over the suitability of the soil on the site for swales. A piped stormwater network has been sized for this three waters assessment, but the use of swales on some of the minor roads may be explored during detailed design.

Conveyance has been sized for the 10-minute duration 10-year ARI event with climate change for the period 2081-2100 under RCP 6.0.

Treatment and Attenuation

The proposed treatment and attenuation approach is to provide a combined wetland and attenuation facility for the entire development. Treatment of the first flush is proposed via a constructed wetland, with attenuation provided to achieve hydraulic neutrality. Flow into the wetland would be controlled via an inlet control structure. During large events flow would be diverted into an adjacent attenuation pond sized to attenuate the 10-year event so as not to damage the wetland. During the 100-year event, the combined footprint of the wetland plus the 10-year attenuation basin would be flooded to provide the attenuation volume required. This is shown in the schematic cross-section in Figure 11.

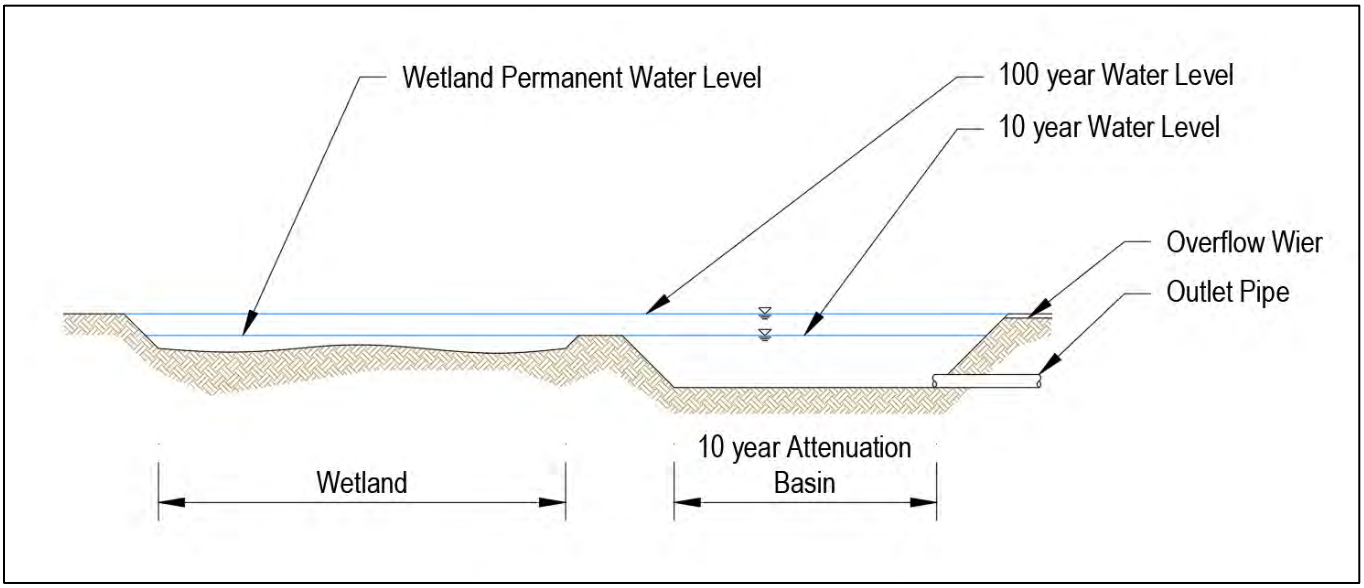


Figure 11 Schematic cross-section through the wetland and attenuation facility

The footprint of the constructed wetland is based on the impervious area. For the purposes of this study it was assumed to be the same for both Option A and Option B. The east subcatchment has a total area of 15.6 ha and a weighted average impervious percentage of 56%. The preliminary footprint of the wetland is 0.60 ha and includes a 20% allowance for side slopes, planting of riparian margins and buffers. The wetland has been sized assuming an average ponding depth of 0.3 m. Options for the wetland inlet include a flow control in a manhole or a diversion weir from an open channel. The details of the inlet, outlet, size and wetland shape will need to be considered as part of detailed design.

2.4.2 Option A

Conveyance

The conceptual pipe sizing is shown in Figure 12 for Option A. The stormwater trunk main has been located along the main roads with a maximum pipe diameter of 825 mm diameter located at the downstream end of the network. The invert at the downstream end of the network is 21.5 m RL.

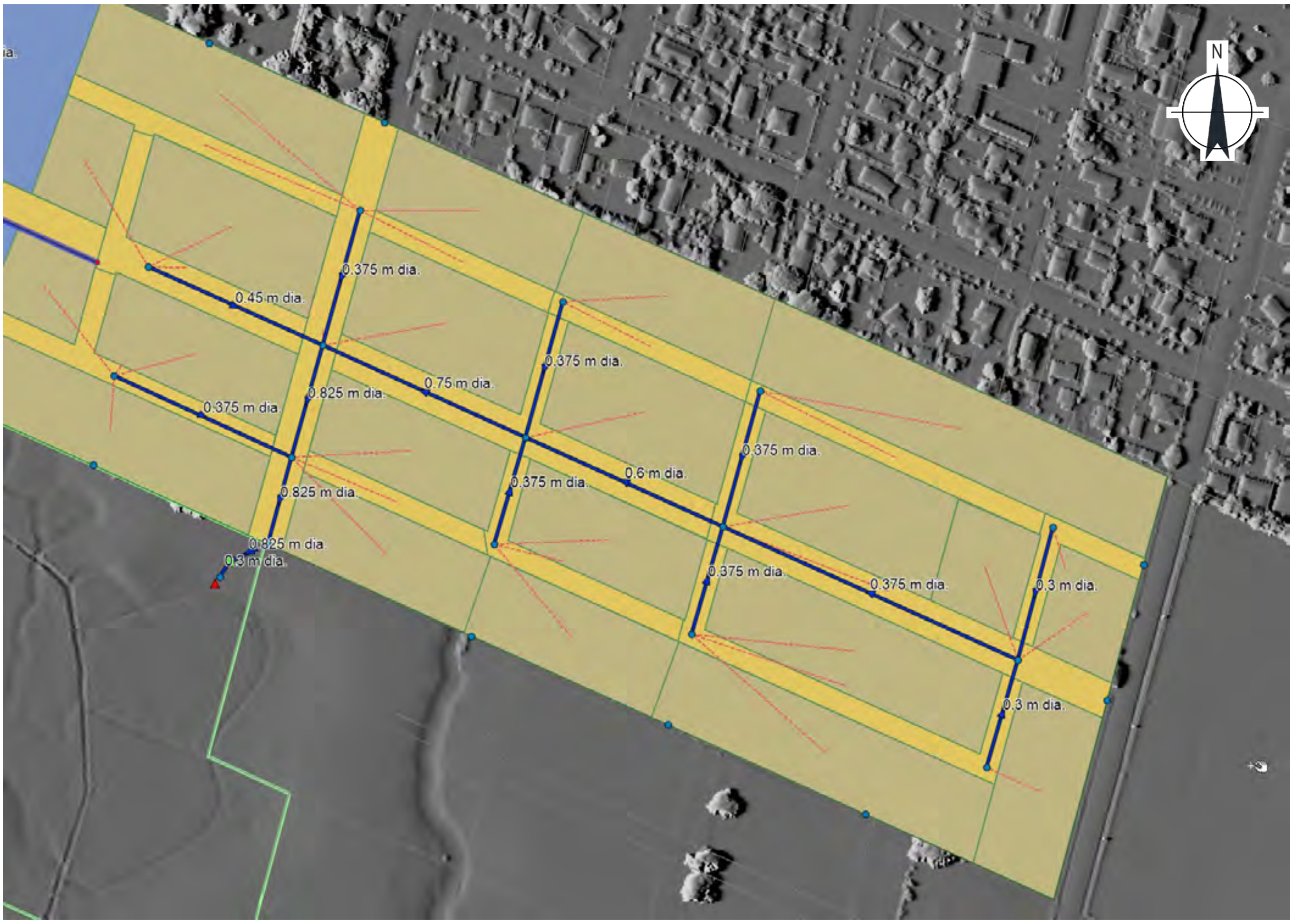


Figure 12 **Conceptual layout of Option A stormwater reticulation for east sub-catchment**

Treatment and Attenuation

The downstream end of the reticulation network on the development site is located at the north-east corner of the adjacent property identified for stormwater use. The existing ground level at the boundary is approximately 24 mRL before grading down at about 8H:1V to a relatively flat area at 20 mRL. To avoid the use of bunds to detain the required attenuation volume, the top of the wetland and attenuation facility will be located at the lower elevation of 20 mRL. The invert on the Campbells Drain at the discharge point is approximately 18.8 mRL, providing 1.2 m of available total depth for the pond and wetland facility.

The 10-year pond volume and outlet has been sized to attenuate outflow to the pre-development flow (0.36 m^3/s – refer to Table 4). The 10-year attenuation pond has a maximum depth of 1.0 m, 5H:1V side slopes and a maximum capacity of 3000 m^3. The footprint at the top of the pond is 0.37 ha. A 300 mm diameter outlet pipe, located at the base of the attenuation pond, has been sized to limit the peak discharge from the pond to the peak pre-development flow. The peak post-development attenuated discharge is 0.28 m^3/s.

For this high-level assessment, the concept design assumes the remaining 0.2 m depth from the top of the 10-year pond and wetland is used to attenuate the 100-year event. During the 100-year event, flow is discharged from the pond via the 300 mm diameter outlet pipe and an overflow weir located 0.1 m below the top of the pond, or 1.1 m above the base of the pond. An additional 2100 m^3 of storage volume is provided on top of the 10-year attenuation volume, which will limit the peak flow to pre-development levels.

The five 100-year storm profiles were run through the attenuation pond to determine the peak water level and volume for each storm. The results are summarised in Table 6. The attenuation volume needed is dependent on the storm profile used in the design. Because the guidance around pond sizing varies across councils, it is recommended that engagement with MDC is undertaken during detailed design to select and confirm the storm profile and approach to be used in the design of the attenuation facilities. Ultimately the overall volume of the attenuation base is governed by the 100-year event, and the conceptual sizing in this 3 waters assessment is likely to be conservative since it satisfies all five storm profiles.

Table 6 Peak pre-development and post-development flows for various storm profiles in east sub-catchment

Storm Profile	Pre-development Peak Runoff (m^3/s)	Post-development Peak Runoff (Unattenuated) (m^3/s)	Total Storage (m^3)	Peak Water depth (m)	Post-development Peak Discharge (Attenuated) (m^3/s)
10 year event					
AP106 24 hour Nested Design Storm	0.36	1.28	3050	1.01	0.28
100 year event					
AP106 24 hour Nested Design Storm	0.84	1.85	5100	1.21	0.66
24 hour NIWA Storm	0.31	0.40	3820	1.08	0.29
12 hour NIWA Storm	0.40	0.58	4640	1.17	0.40
6 hour NIWA Storm	0.77	1.12	4750	1.18	0.46
1 hour NIWA Storm	0.37	1.60	2670	0.91	0.26

The footprint of the 100-year attenuation is 1.01 ha. A 5 m wide allowance has been added around the perimeter of the attenuation/wetland facility to allow for maintenance access, grading, and pedestrian access. The final conceptual footprint of the attenuation/treatment facility is 1.25 ha and is summarised in Table 7. Pond levels are provided in Table 8.

Table 7 Summary of Option A treatment and attenuation facility footprint

Element	Area (ha)
Constructed wetland	0.60
10 year attenuation basin (top area)	0.37
100 year attenuation (top area)	1.01
100 year Attenuation basin footprint including 5 m maintenance access/buffer around perimeter	1.25

Table 8 Summary of Option A attenuation pond operating levels

Component	Details
Outlet pipe	300 mm diameter pipe
Overflow weir	19.9 m RL (1.1 m above bottom of pond)
Base of 10 year Attenuation Pond	18.8 m RL
Peak water level during 10 year event	19.8 m RL (1.0 m above bottom of pond)
Peak water level during 100 year event	20.0 m RL(1.2 m above bottom of pond)
Top of pond	20.0 m RL (1.2 m above bottom of pond)
Invert of Campbells Drain at discharge point	18.8 m RL

The 1.25 ha footprint can be accommodated in the north-east corner of the adjacent property that has been identified for stormwater use (see Figure 13) and will partially take up the footprint of the existing wetland. A 10 m

offset from the southern boundary of the plan change area has been assumed to allow for grading down to 20.0 mRL. The footprint overlaps approximately 0.46 ha of existing floodplain (inundation extents greater than 0.1 m deep in Figure 6). This loss of floodplain can be offset by excavating to create additional floodplain storage on the west side of the drain (see indicative areas in Figure 13). There may also be the opportunity to realign part of the Campbells Drain if required to accommodate the facility in the detailed design. However, any such works will need to consider the implications on the existing natural wetland. Since restoration of the existing wetland would be necessary under the NES-F, additional floodplain storage should be achievable as part of the enhancement works.

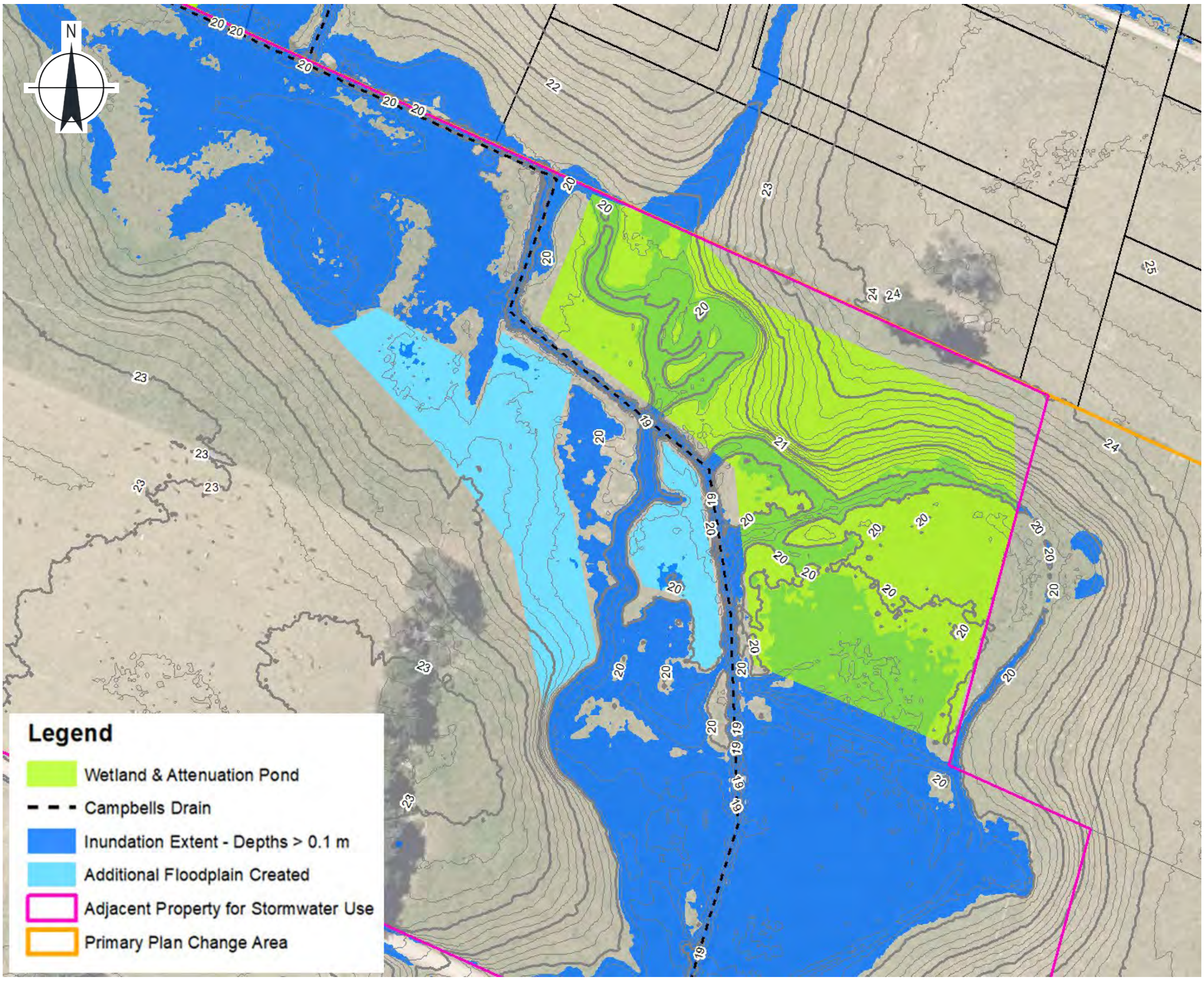


Figure 13 *Conceptual footprint of Option A wetland and attenuation pond*

2.4.3 Option B

The premise of Option B is to avoid any disturbance to the natural inland wetland and locate the stormwater treatment and attenuation facility outside the 100 m buffer. Based on the existing grades (refer to Figure 10), the most sensible location for the attenuation and wetland facility is on the east side of the Ruivaldts Drain outside of the 100 m offset from the natural inland wetland.

The conceptual reticulation, treatment and attenuation sizing for Option B assumes run-off from the full structure plan in Figure 1. This results in conservative pipe sizes and pond footprints since some of the lots would need to be removed to accommodate the wetland and attenuation pond footprint within the actual structure plan area. As with Option A, it is assumed that site earthworks would allow overland flow paths to drain to the new pond site.

Conveyance

The conceptual conveyance for Option B was developed to drain the east sub-catchment to Ruivaldts Drain. The conceptual pipe sizing is shown in Figure 14. The stormwater trunk main has been located along the main road with a maximum pipe diameter of 825 mm diameter located at the downstream end of the network. The invert at the downstream end of the network is 21.2 m RL.

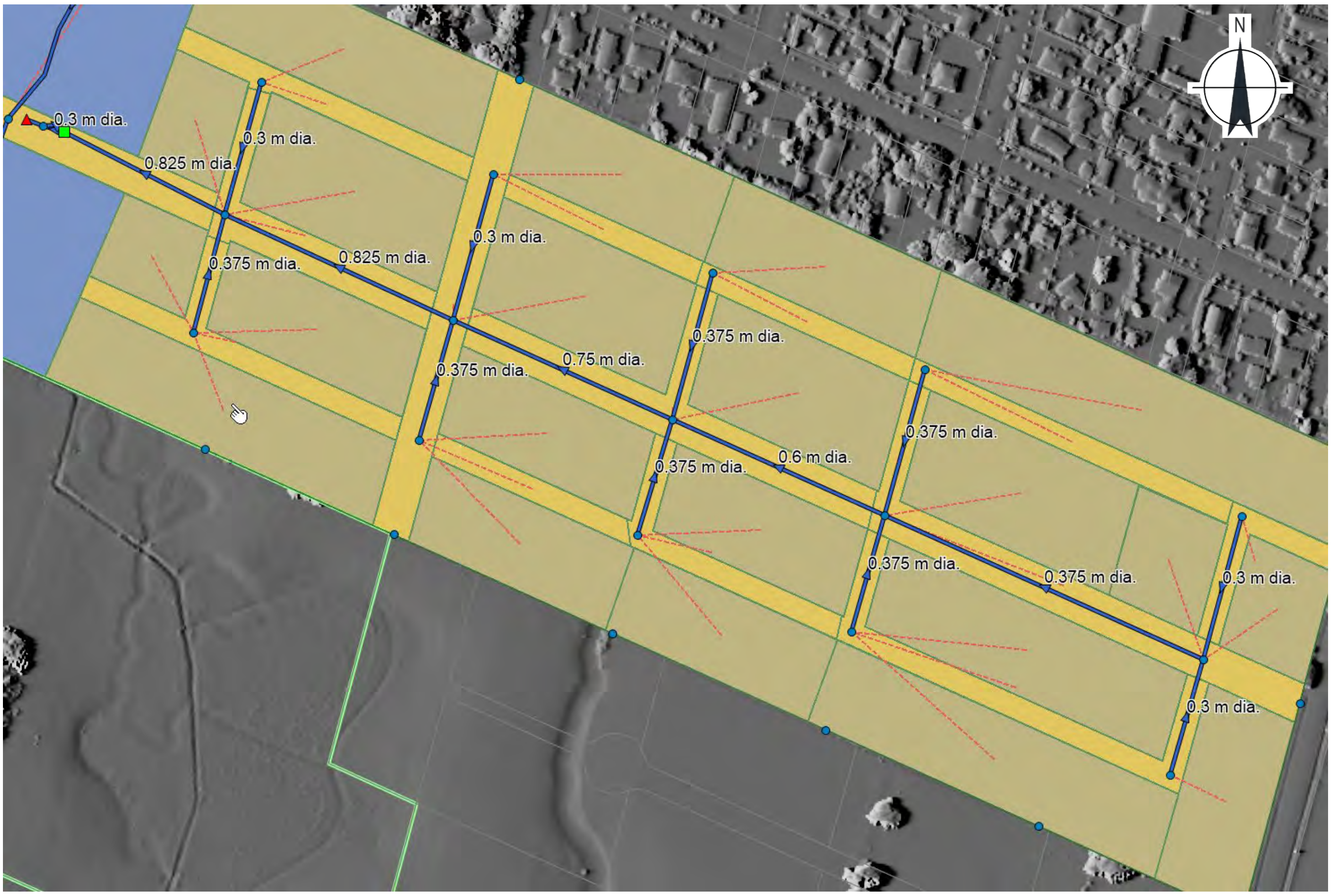


Figure 14 ***Conceptual layout of Option B stormwater reticulation for east sub-catchment***

Treatment and Attenuation

An existing cross-section near the proposed site of the wetland and attenuation pond is shown in Figure 15. The existing ground in this area slopes up from the drain at a maximum slope of about 15H:1V, to a plateau at about 24 m RL.

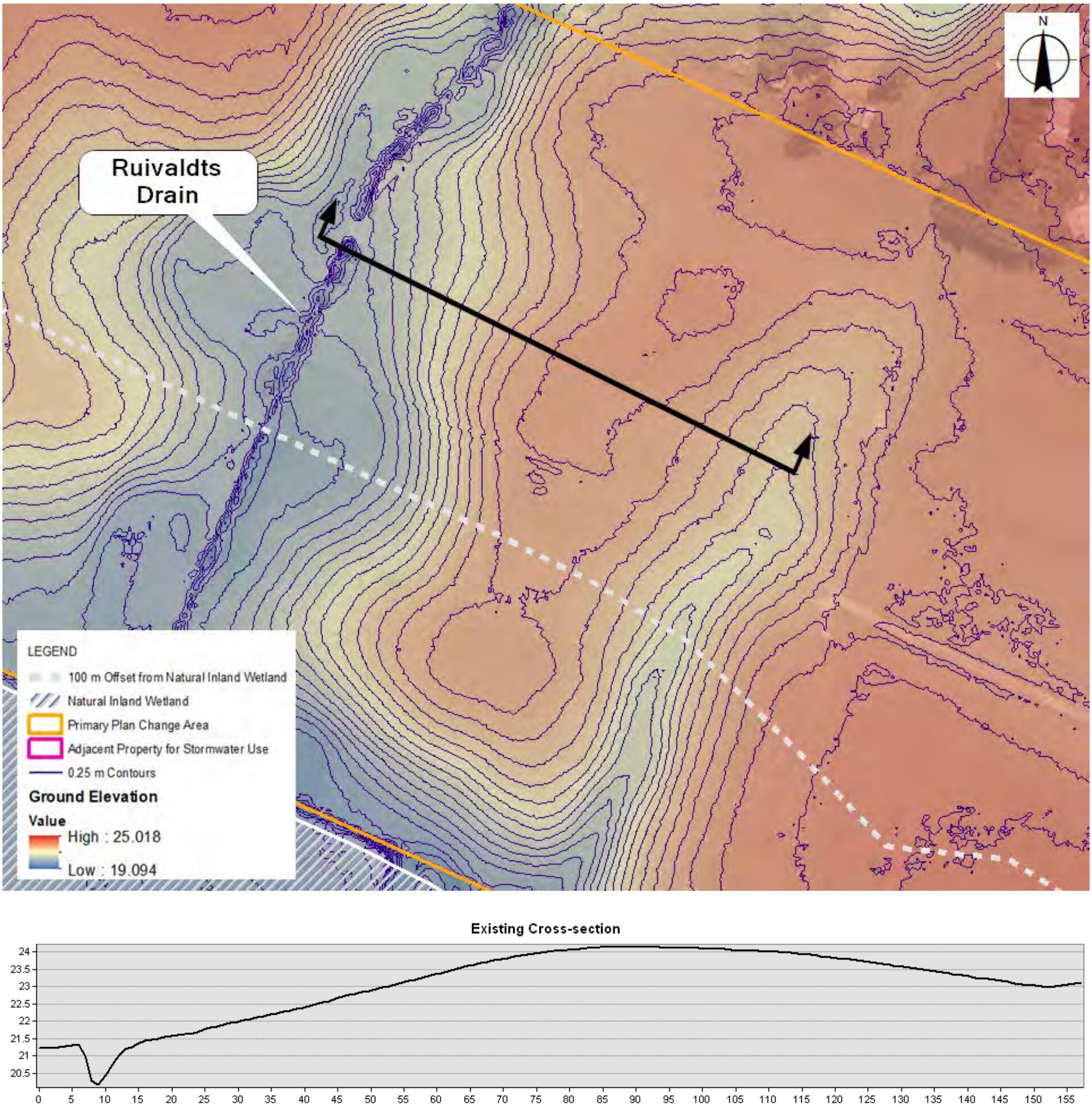


Figure 15 *Existing topography and cross-section near Option B stormwater pond location*

To assess the feasibility of this option, the same parameters (such as water depths) as Option A have been assumed. This results in the same overall footprint as Option A of 1.25 ha including maintenance access. An indicative footprint of the pond is shown in Figure 16. The conceptual section based on existing grades is shown set into the existing slope. This creates high embankments around the perimeter of the pond (particularly on the east side of the pond). These can be reduced through overall site grading or refinement in the pond design. In the cross-section, a bund is effectively created between the attenuation basin and Ruivaldts drain; a geotechnical assessment is needed to confirm soil conditions and the pond may need to be lined for stability of the bund.

There is an approximately 1 m difference between the downstream invert of the conceptual reticulation network (21.2 m RL) and the invert of Ruivaldts Drain (20.2 m RL). Depending on a geotechnical assessment, and final ground and network levels, the pond footprint, depth, and pond base invert can be refined to reduce the overall size or depth.

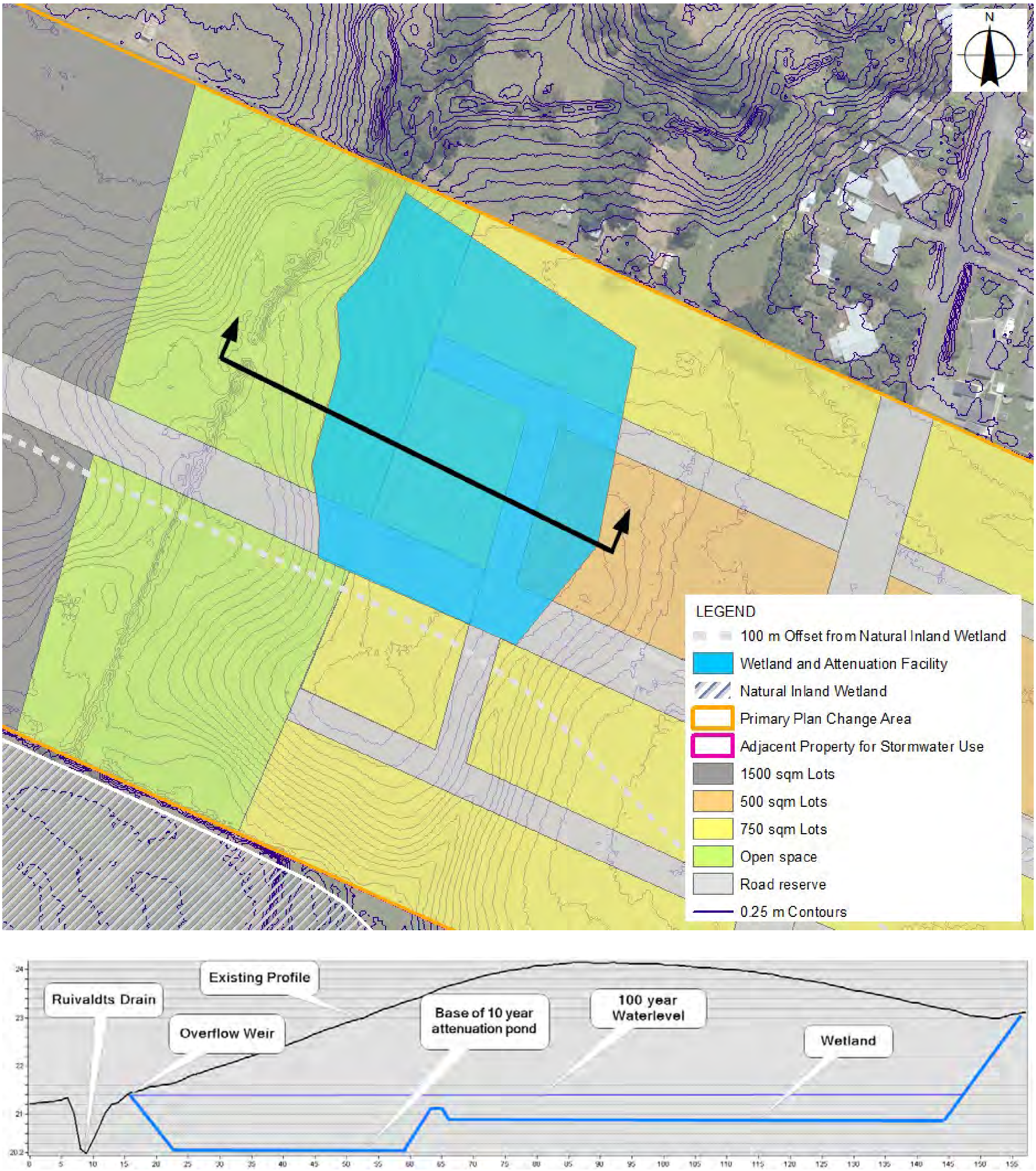


Figure 16 *Attenuation and Wetland Facility Footprint for Option B*

2.5 Stormwater Management for West Sub-catchment

As previously discussed, the Ruivaldts / Walsh drain bisects the development site into two sub-catchments, making it impossible to provide a single treatment and attenuation facility for the entire development. The general approach for the west sub-catchment is to convey runoff via kerb and channel to raingardens to treat and attenuate the flow. The treated and attenuated runoff can then be discharged to the Ruivaldts Drain.

The raingarden area required for treatment is based on the impervious area draining to the raingarden. This includes impervious area in the 1500 m^2 lots and from the road corridor in the west sub-catchment. The total impervious area for treatment is assumed to be 1.2 ha, equating to a raingarden area (measured at the bottom of the filter media) of 240 m^2 to treat the first flush. This represents the total area required and can be divided into several raingardens along the road corridor. A typical raingarden cross-section is shown in Figure 17.

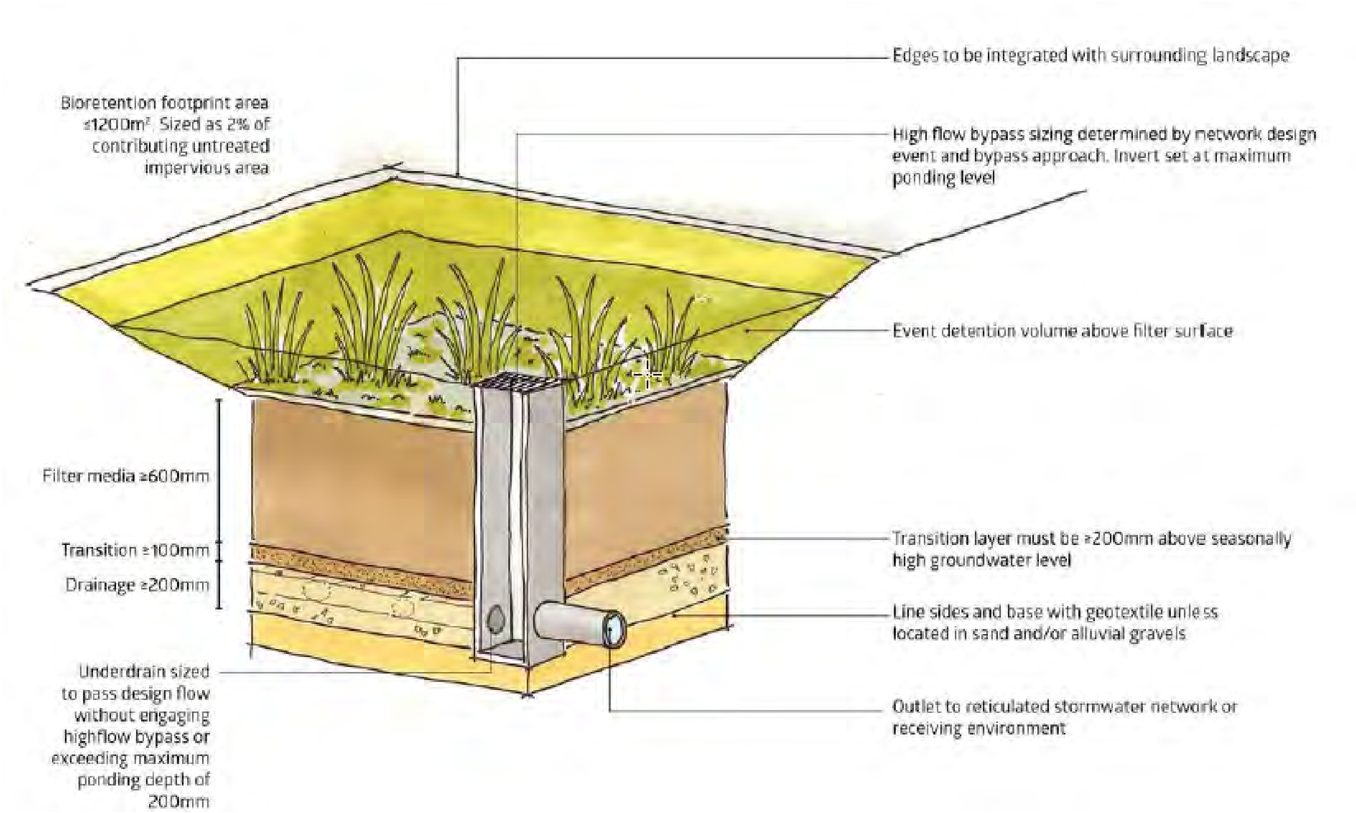


Figure 17 Typical construction of raingarden (source: Wellington Water Limited Water Sensitive Design Guidelines)

During the 100-year event, storage above the ponding depth of the raingardens (illustrated above in dark green in “event detention volume above filter surface”) and controlled outlets will provide attenuation to pre-development flows. Assuming a raingarden width of 3 m and side slope of 3H:1V above the raingarden, the storage volume and maximum ponding depth was calculated to achieve attenuation to pre-development flows for the five 100-year storms. These are summarised in Table 9. The critical event is the 24-hour NIWA storm which requires 210 m^3 of storage and reaches a maximum ponding depth of 0.46 m. This corresponds to a total area of 560 m^2 at full ponding depth. As with the attenuation pond, the design storm should be confirmed with MDC during detailed design to determine the ultimate area and ponding depth in the raingardens. Detailed design will need to consider the implications of having a deep ponding depth, and alternative attenuation measures (i.e., underground storage) may be preferable to council.

Table 9 Peak pre-development and post-development flows for various storm profiles in west sub-catchment

Storm Profile	Pre-development Peak Runoff (m^3/s)	Post-development Peak Runoff (Unattenuated) (m^3/s)	Total Storage (m^3)	Peak Water depth (m)	Post-development Peak Discharge (Attenuated) (m^3/s)
100 year event					
AP106 24 hour Nested Design Storm	0.35	0.42	160	0.38	0.35

Storm Profile	Pre-development Peak Runoff (m^3/s)	Post-development Peak Runoff (Unattenuated) (m^3/s)	Total Storage (m^3)	Peak Water depth (m)	Post-development Peak Discharge (Attenuated) (m^3/s)
24 hour NIWA Storm	0.10	0.11	210	0.46	0.10
12 hour NIWA Storm	0.14	0.15	170	0.39	0.14
6 hour NIWA Storm	0.18	0.20	170	0.39	0.18
1 hour NIWA Storm	0.16	0.22	150	0.36	0.16

Indicative footprints of the raingardens are shown in Figure 18. The northern block of 1500 m^2 lots and the road reserve can likely be treated using raingardens in the road reserve. A separate raingarden will likely be required to treat and attenuate runoff from the southern block of 1500 m^2 lots based on existing ground contours falling to the south. The size, number and siting of raingardens will need to be refined in conjunction with site grading and design of the road corridor. Alternative treatment methods such as planted swales or proprietary treatment devices for the west sub-catchment can also be considered in consultation with MDC as part of detailed design.

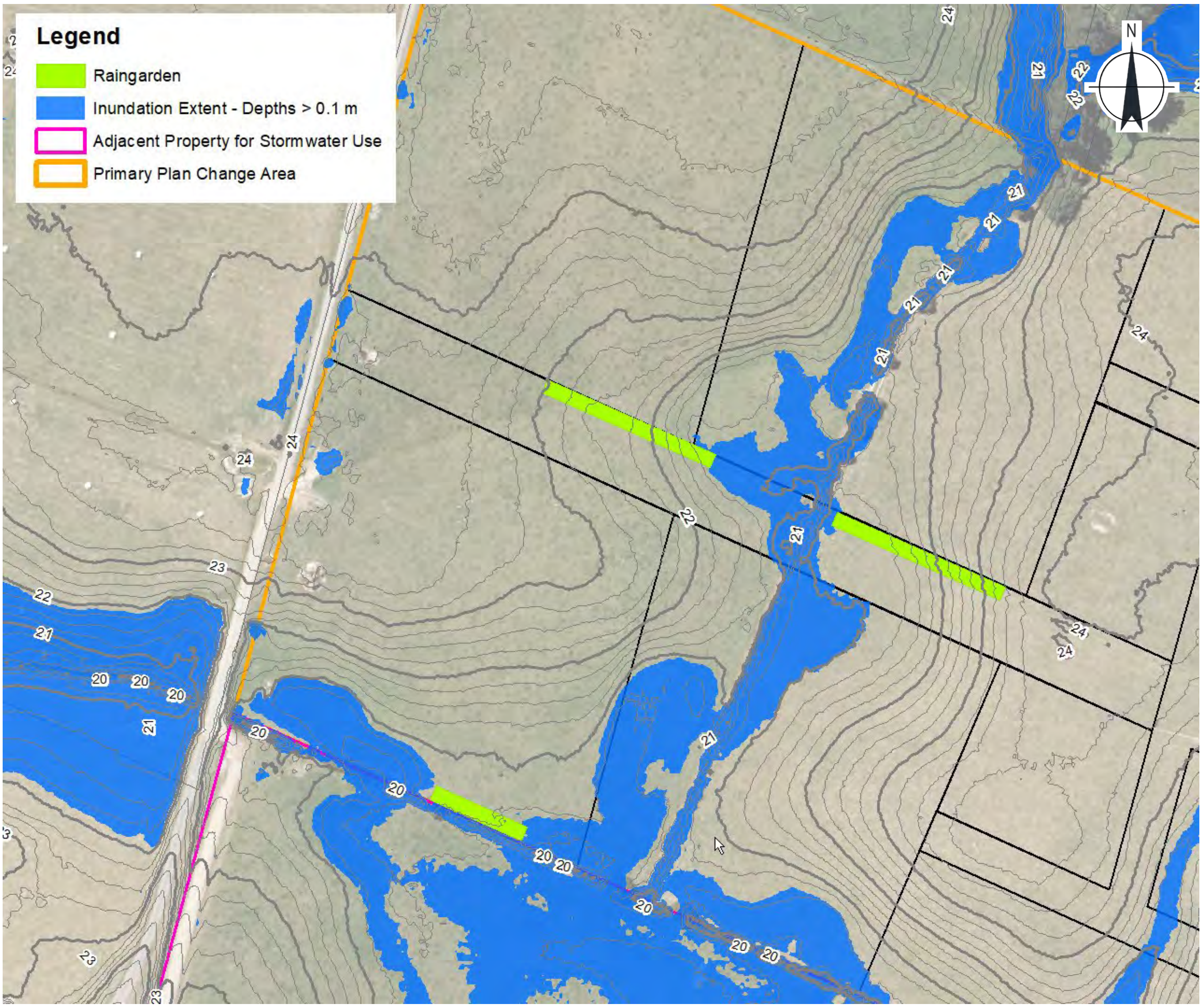


Figure 18 Indicative locations of raingardens for west sub-catchment

2.6 Summary

From the investigations carried out as part of this report, it has been concluded that:

- With the exception of the Ruivaldts / Walsh / Campbells drain area, the plan change area is not subject to inundation.
- The Ruivaldts / Walsh / Campbells drain should be identified as a flood prone area.
- Due to the flooding further downstream of the Te Kawau drainage scheme, hydraulic neutrality must be achieved with respect to peak flows.
- Overland flow paths through the plan change area will need to be accommodated.
- The existence of a natural inland wetland (identified by Forbes Ecology 12 July 2022) requires stormwater runoff from the development to be treated.
- The Ruivaldts / Walsh drain cuts the plan change area into two catchments, requiring separate treatment and attenuation for the east and west sub-catchments.
- The sizing of the attenuation required is heavily dependent on the storm profile used, which will need to be confirmed with MDC as part of detailed design.
- The wetland and attenuation facility can be feasibly located on either the adjacent lot identified for stormwater use or adjacent to the Ruivaldts Drain with a 100 m offset from the natural inland wetland.
- Earthworks in the floodplain will be required to offset the loss of area taken by the Option A wetland and attenuation pond, but also take into consideration the sensitivity of the existing inland wetland. This option also provides the opportunity to enhance the existing natural inland wetland.
- The footprint of the attenuation and wetland facility in Option B will result in a loss of lots and reconfiguration of the structure plan.

In order to service the plan change area, it is recommended that:

- A cut-off drain along the northern boundary be included to divert runoff from the urban area around the development site.
- The east sub-catchment be designed with the following stormwater reticulation:
  - Piped or combined open drain system to convey the 10-year ARI event to a centralised wetland and attenuation facility.
  - A secondary overland flow path be provided to convey the 100-year ARI to the main drain.
  - A wetland approximately 0.6 ha in size to treat the first flush.
  - A 10-year attenuation pond approximately 0.37 ha in size to reduce peak flows from the development to pre-development levels.
  - A total footprint of approximately 1.25 ha be set aside for the 100-year attenuation and required maintenance access to reduce peak flows from the development to pre-development levels.
- The west sub-catchment be designed with the following stormwater reticulation:
  - Approximately 240 m^2 of raingardens in the roading corridor to treat the first flush.
  - Approximately 210 m^3 of storage be provided for attenuation to reduce peak flows from the development to pre-development levels.
- Earthworks in the floodplain area be undertaken to offset the loss from the east catchment (for Option A), whilst improving and enhancing the existing natural inland wetland.

3. Potable Water

3.1 Existing Water Supply System

3.1.1 Water Source

Rongotea's potable water is sourced from a bore located on the eastern side of the Village next to the Te Kawau Memorial Recreation Centre and Sportsfields at the end of Wye Street. Currently the bore has a consented groundwater take limit of 800 m^3/day. Water from the bore is being chlorinated at the source prior to being stored in a concrete storage tank. The storage tank has a capacity of 800 m^3 consisting of 620 m^3 or 40 hours storage for normal operating conditions and 180 m^3 or 1 hr of firefighting storage at 50 l/s (commercial fire flow). From the storage tank water is supplied to the reticulation system via a booster pump system located in the adjacent pump station building. The booster pump system consists of three 7.5 kW vertical multistage duty pumps and one 30 kW vertical inline firefighting pump. The location of the bore, storage tank, and pump station building is shown in Figure 19 below.

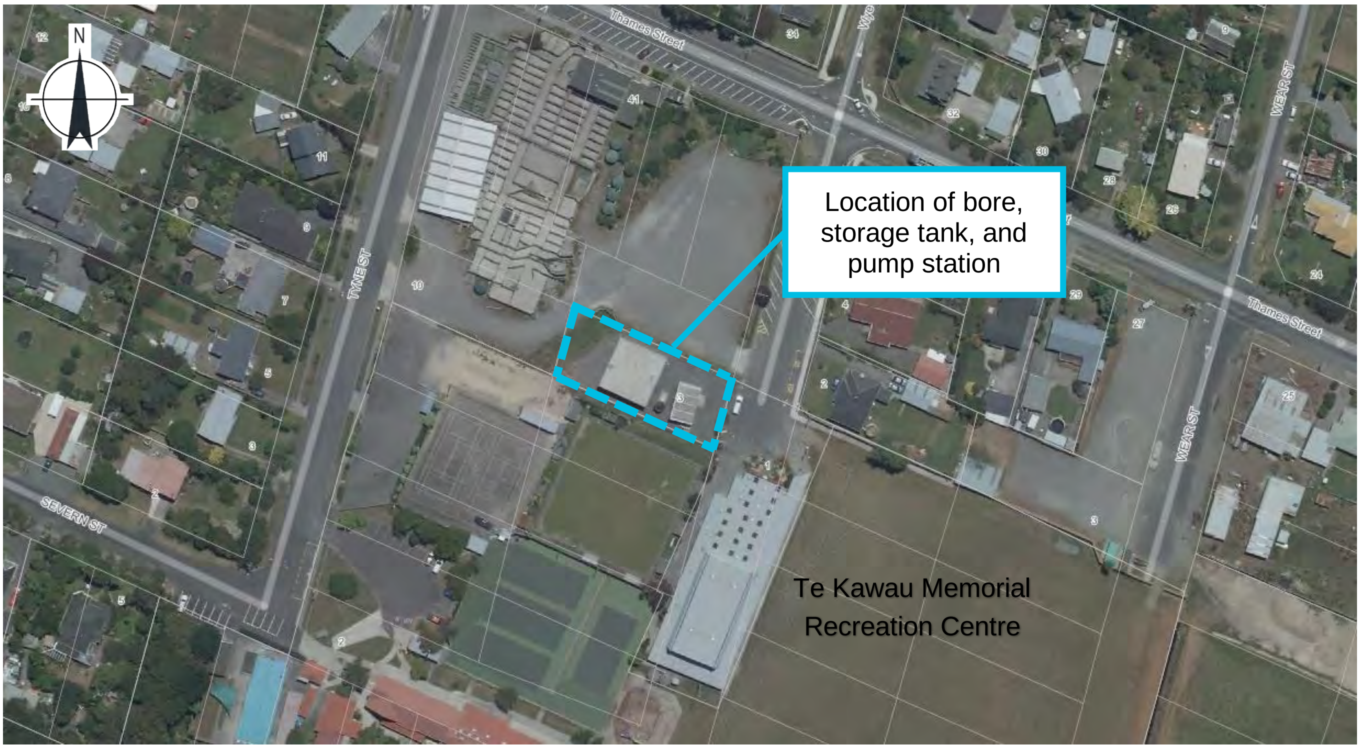


Figure 19 Location of bore, storage tank, and booster pump station

3.1.2 Reticulation System

Rongotea's potable water reticulation system was renewed in 2014 and predominantly consists of DN125 PE80 PN12.5 principal mains and DN63 PE80 PN12.5 rider mains. There are sections of DN200 PVC-O PN12.5 pipe along Tyne Street and between the pump station and Tyne Street. The network layout along the majority of the roads does not follow the typical MDC Engineering Standards for Land Development (ESLD) reticulation layout (i.e., a principal main one side of the road and rider main opposite side of the road), as most of the roads have DN63 rider mains on both sides of the carriageway. The system consists of multiple ring mains and is well interconnected providing good network resilience. Fire hydrants are provided at 135 m intervals as required by SNZ PAS 4509:2008 and isolation valves are sufficiently spaced ensuring that all properties can be sufficiently isolated. Rongotea's existing water reticulation layout indicating the various pipe sizes is shown in Figure 20 below.

Figure 20 Rongotea existing water reticulation layout

3.2 Water Model Development

3.2.1 General

A steady state hydraulic model of Rongotea's water reticulation system was created with Bently's WaterGEMS software to evaluate the capacity of the existing system and identify possible upgrades required to accommodate future growth within the proposed plan change area. For the purpose of this high-level assessment an extended period simulation (EPS) was not required. No pressure logging or hydrant flow tests were carried out to confirm the pressures within the existing network. Consequently, no attempts were made to calibrate the model.

3.2.2 Pipes

Rongotea's existing pipe network was created with as-built information provided by MDC. Nominal diameters (DN) were used as the inside diameter of each pipe. A roughness value (ε) of 0.1 mm was used for all pipes in the model. The Darcy-Weisbach equation in conjunction with the Swamee-Jain equation were used to calculate head losses in the system. Service connection pipes were excluded from the model for simplicity.

3.2.3 Nodes

Nodes were generally created at tees, crosses, change of pipe diameter, fire hydrants, or distributed along longer sections of straight pipe. LiDAR data sourced from Linz Data Service was used to assign ground elevation data to each node. Water demand as discussed in Section 3.3 below was distributed equally across the nodes as a base demand. Additional firefighting demand was added on top of the base demand of each node that also represents a fire hydrant. Due to the size of the concrete reservoir (800 m^3) being sufficiently sized for storage, it was not required to assess the water level in the reservoir with an EPS analysis, therefore, the reservoir has been modelled to have unlimited capacity. This will not affect any other results obtained from the model.

3.2.4 Pumps

Actual pump performance curves have been used in the model. MDC indicated that currently the pumps are set to maintain a pressure in the network of 450 kPa. The duty pumps are set up in parallel which means that all three pumps can run concurrently covering a large flow range. The additional firefighting pump has a lower shutoff head and a different relative speed than the duty pumps, therefore it has been assumed that the firefighting pump is unable to run concurrently with the duty pumps, and has consequently been modelled to operate separately from the duty pumps. Pump performance curves used in the model are shown in Figure 21 and Figure 22 below.

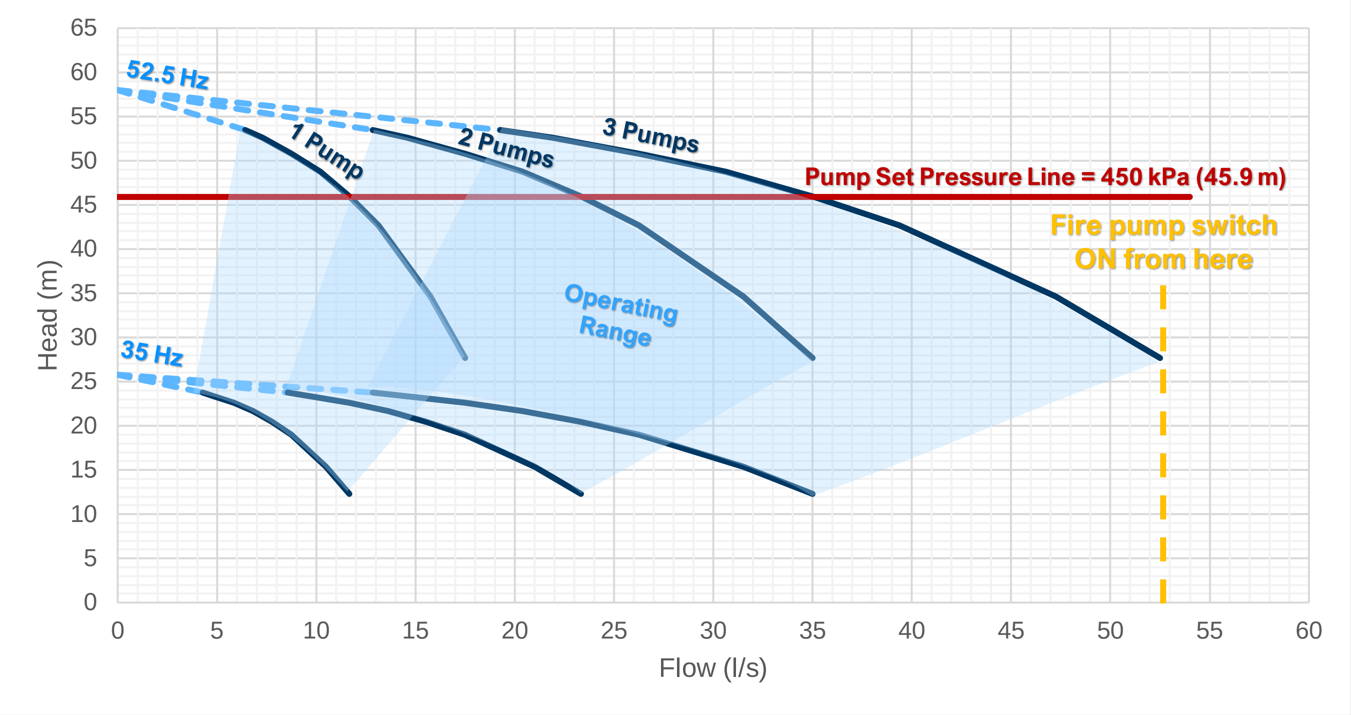


Figure 21 Duty pump performance curves

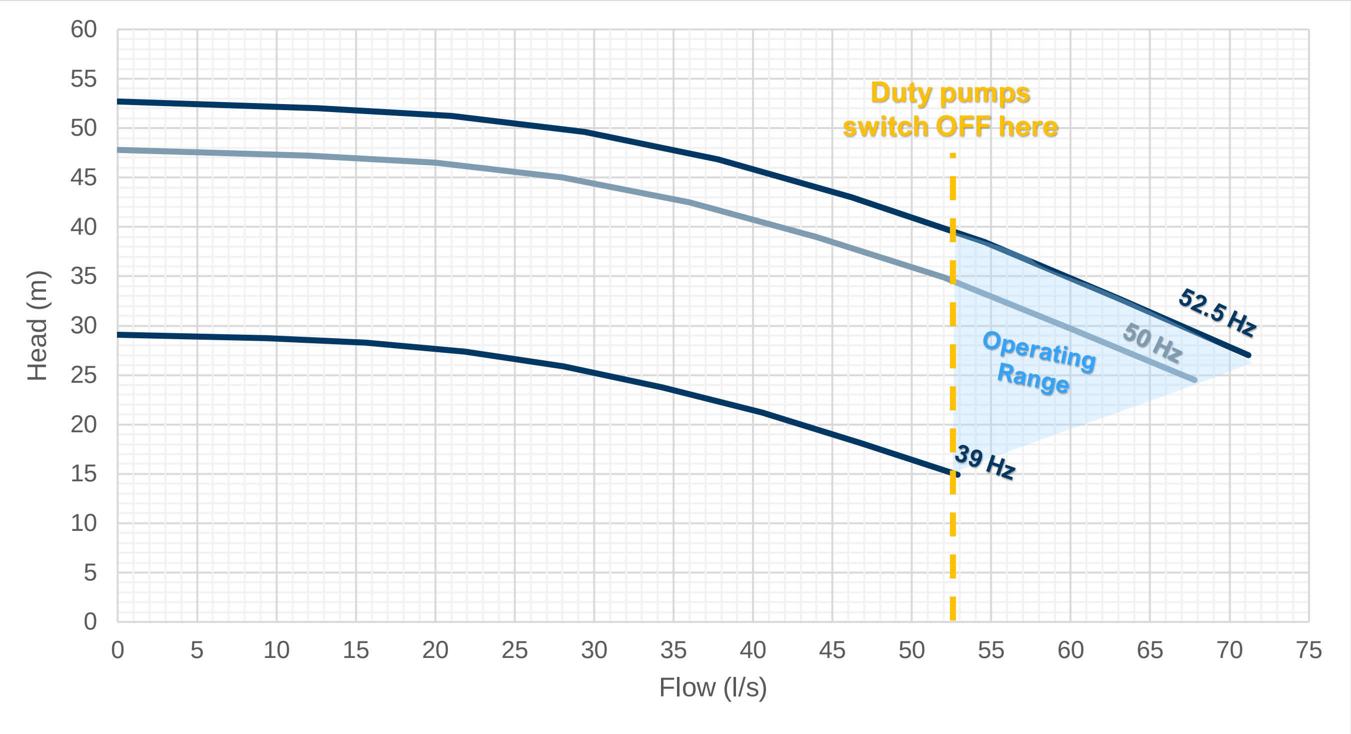


Figure 22 Fire pump performance curves

The three duty pumps have a combined ultimate flow rate of 52.5 l/s at 28 m head and 52.5 Hz as shown in Figure 21. The fire pump will start up when the required flow exceeds 52.5 l/s, i.e., during commercial fire flow scenarios. The fire pump has an ultimate capacity of 71 l/s at 27 m head and 52.5 Hz as shown in Figure 22.

3.3 Water Demand

3.3.1 Peak Flows

Peak flows were calculated using design parameters, as per the latest version of MDC's Engineering Standards, (refer Section 0 of this report). For this investigation these peak flows were used as the basis to evaluate Rongotea's existing and proposed future water supply infrastructure. It should be noted that in the absence of actual reliable water consumption data, these peak flow rates may be significantly higher than the actual water usage in Rongotea and should be viewed as being conservative. As a comparison, preliminary water consumption data provided by MDC for 2021 indicated that the peak hourly demand is approximately half of the peak hourly demand calculated with the design parameters from MDC's Engineering Standards.

A summary of Rongotea's existing and future peak hourly demand is shown in Table 10 below. The results indicate that the peak hourly demand of Rongotea is 21.6 l/s currently and Rongotea's future peak hourly demand is 36.59 l/s, which is nearly double that of the current peak flows.

Table 10 Summary of Rongotea's peak hourly water demand

Description	No. of Properties	Ave. Day Demand (l/s)	Peak Day Demand (l/s)	Peak Hourly Demand (l/s)
Existing Rongotea	229	2.155	4.31	21.55
New Subdivision	160	1.504	3.01	15.04
Total (Future Rongotea)	**389**	**3.659**	**7.32**	**36.59**

Table Notes:
1. A population of 2.8 people per property were used to calculate total population.
2. An average daily consumption of 290 l/p/d were used to calculate the average day demand.
3. A daily peaking factor of 2 for populations below 2,000 were used.
4. An hourly peaking factor of 5 for populations below 2,000 were used.

During a workshop held on 30 June 2022, MDC indicated that currently one third of Rongotea's properties are not connected to the reticulation network. However due to upcoming 3 waters reforms this is expected to increase. As such, this assessment assumes that all properties within Rongotea are connected to the water network.

3.3.2 Fire Flows

MDC requires all lots to have fire-protection in accordance with SNZ PAS 4509:2008 – Code of Practice for Firefighting Water Supplies.

MDC indicated that Rongotea's existing lots are zoned predominately as residential type lots with only a few commercial lots along Thames Street. Therefore, according to SNZ PAS 4509, Rongotea requires a reticulated water supply system that can provide firefighting flows corresponding to FW 2 (25 l/s) for residential areas and FW 3 (50 l/s) for the commercial area.

3.3.3 Design Flows

SNZ PAS 4509 recommends that water supply systems be designed to provide two thirds of annual peak demand in addition to the fire flow requirements while maintaining residual pressures throughout the network above 100 kPa.

Table 11 below summarises Rongotea's existing and future design flow requirements.

Table 11 Summary of Rongotea's water design flow summary

Description	Peak Hourly Demand (l/s)	2/3 Peak Flow (l/s)	Required Fire Flow (l/s)	Design Flow (l/s)
Existing Rongotea (FW2)	21.55	14.37	25	**39.37**
Existing Rongotea (FW3)	21.55	14.37	50	**64.37**
Future Rongotea (FW2)	36.59	24.39	25	**49.39**
Future Rongotea (FW3)	36.59	24.39	50	**74.39**

These flows were used in the hydraulic analysis discussed below to evaluate the water supply capacity of the network during various peak and fire flow scenarios.

3.4 Water Supply Capacity Evaluation

The capability of the existing network to provide current and future demand is defined as follows:

– MDC ESLD requires delivery pressures at each node to be between 250 kPa and 800 kPa during peak flow
– SNZ PAS 4509 requires a minimum residual pressure of 100 kPa at each node during fire flow

The following scenarios were investigated and used to size the proposed network within the development site and identify any upgrades to the existing network required due to the development.

– Scenario 1 – Rongotea existing peak flow (21.6 l/s)
– Scenario 2 – Rongotea existing 2/3 peak + residential fire flow (39.4 l/s)
– Scenario 3 – Rongotea existing 2/3 peak + commercial fire flow (64.4 l/s)
– Scenario 4 – Rongotea future peak flow (36.6 l/s)
– Scenario 5 – Rongotea future 2/3 peak + residential fire flow (49.4 l/s)
– Scenario 6 – Rongotea future 2/3 peak + commercial fire flow (74.4 l/s)

3.4.1 Existing System Evaluation

Peak flow

For scenario 1, two of the three duty pumps are running to achieve the required peak flow rate. The pumps can maintain a pressure of 450 kPa at the start of the reticulation network. All nodes within the network have pressures above the required 250 kPa threshold.

Fire flow

For scenario 2, all three duty pumps are running simultaneously to achieve the required residential fire flow rate. The pumps are only able to produce a pressure of 403 kPa (< 450 kPa) at the start of the reticulation network. Fortunately, all nodes within the network have residual pressures above the required 100 kPa.

For scenario 3, the duty pumps are switched off with only the fire pump running. The fire pump can only provide the required commercial fire flow at a reduced pressure of 261 kPa (< 450 kPa) at the start of the reticulation network. Consequently, approximately half of the nodes on the western side of Rongotea have residual pressures below the required 100 kPa limit.

Based on the high-level modelling undertaken and assumptions around the operation of the duty pumps and fire pump, the required residual pressure during commercial fire flow is not met. The pumps would have to produce an additional pressure of at least 280 kPa at the start of the reticulation system to be compliant.

Storage

The Water Supply Code of Australia (WSA 03) requires a useable reservoir capacity to be equal to a minimum of 8 to 24 hours consumption at peak day demand.

SNZ PAS 4509 requires firefighting storage of at least 1 hour at 50 l/s for a commercial fire which is equal to 180 m^3.

The existing storage tank has a total capacity of 800 m^3. The storage volume is currently separated as follows:

- 620 m^3 is reserved for normal operating purposes which is equivalent to 40 hours at peak day demand.
- 180 m^3 is reserved for firefighting purposes which is equivalent to 1 hour at 50 l/s.

The existing storage tank therefore has sufficient storage capacity as required by WSA 03 and SNZ PAS 4509.

3.4.2 Proposed System Evaluation

The reticulation within the plan change area has been sized to minimise pipe size while maintaining a sufficient level of service as required by MDC and SNZ PAS 4509. The reticulation layout follows the roading layout provided in the structure plan (refer Figure 1). The proposed reticulation network for the plan change area is shown in Figure 23 below.

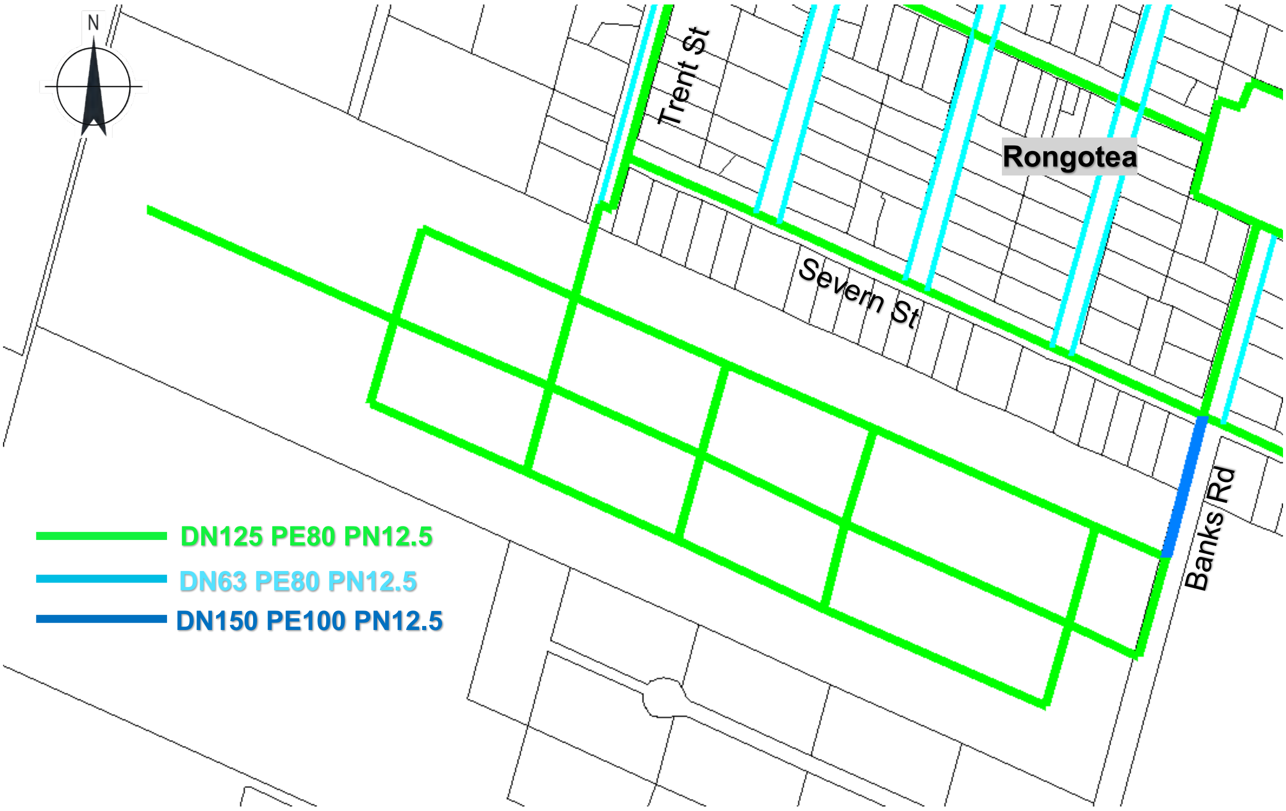


Figure 23 Proposed water reticulation network

As shown in Figure 23 above, all the reticulation pipes are sized as DN100 mains. There are two connections to the existing reticulation network: one being a new DN150 connection at the intersection of Banks Road and Severn Street and the other a DN100 connection at the intersection of Trent Street and Severn Street.

Peak flow

For scenario 4, all three duty pumps are running simultaneously to achieve the required future peak flow rate. The pumps can only produce a pressure of 428 kPa (< 450 kPa) at the start of the reticulation network. Fortunately, all nodes within the network have pressures above the required 250 kPa threshold.

A minor upgrade of the existing Trent Street water main from DN50 to DN100 is required to service the plan change area.

Fire flow

For scenario 5, all three duty pumps are running simultaneously to achieve the required residential fire flow rates. The pumps are only able to produce a pressure of 296 kPa (< 450 kPa) at the start of the reticulation network. Consequently, the properties on the western side of the proposed plan change have residual pressures less than the required 100 kPa. The pumps would have to produce an additional pressure of at least 175 kPa at the start of the reticulation system to be compliant.

For scenario 6, the duty pumps are switched off with only the fire pump running. The fire pump can only provide the required commercial fire flow at a reduced pressure of 173 kPa (< 450 kPa) at the start of the reticulation network. Consequently, most of the nodes within Rongotea have residual pressures below the required 100 kPa limit.

Based on the high-level modelling undertaken and assumptions around the operation of the duty pumps and fire pump, the required residual pressure during both residential and commercial fire flow is not met. The pumps would have to produce an additional pressure of at least 380 kPa at the start of the reticulation system to be compliant under the commercial fire flow scenario.

Storage

Due the increase in peak day demand the existing storage for normal operating purposes has been reduced from 40 hours to 23.5 hours, which is still within WSA 03's requirements for a minimum storage of 8 to 24 hours.

There is no change in the storage amount required for firefighting purposes.

3.5 Summary

From the investigations carried out as part of this report, and based on the assumptions made around the operation of the duty and fire pumps, it has been concluded that:

- The existing reticulation network is not able to meet commercial fire flow requirements and an upgrade to the existing system would be required.
- Additional demand from the plan change area will further reduce the level of service of the existing reticulation during commercial fire flow events.
- In order to meet residential fire flow requirements within the plan change area itself, an upgrade to the water pump station will be required assuming the whole town is connected. This will likely be met with the upgrade to meet existing commercial fire flow requirements.
- No immediate upgrades to the existing reticulation network are required for peak day demand.

In order to service the plan change area, it is recommended that:

- The plan change area be connected to the existing reticulation network at two locations as follows:
  - A DN150 connection to the existing DN100 main at the intersection of Banks Rd and Severn Street.
  - A DN100 connection to the existing DN100 main at the intersection of Trent Street and Severn Street. The existing DN50 rider main at the southern end of Trent Street needs to be upgraded to at least a DN100 main.
- The existing pumps be upgraded to produce an additional pressure of at least 380 kPa at the start of the reticulation system. This will ensure that all current and future fire flow requirements are met.
- The upgrades to the pumps be investigated during the next replacement cycle which is due within the next 5 to 10 years.

4. Wastewater

4.1 Existing Wastewater System

4.1.1 Gravity System

Rongotea's wastewater generally drains via gravity in a western direction. The reticulation system is primarily separated into three distinct catchments, as shown in Figure 24 below. Catchment 3 drains towards an existing pump station next to the Rongotea School along Severn Street. From this pump station wastewater is conveyed to Catchment 1 via a short rising main along Severn Street. Catchments 1 and 2 discharge to an existing pump station along Trent Street. From this pump station wastewater is being pumped via two rising mains to the existing wastewater treatment facility (facultative ponds) where the wastewater is treated and discharged into the environment. The proposed plan change area is located on the southern side of Rongotea next to Catchment 1.

The reticulation network consists mostly of DN150 asbestos cement (AC) pipes and several DN200 AC mains on the southern side of Catchment 1 along Trent Street and Severn Street. Most of the reticulation network was installed in 1977. The location of the contributing catchments, reticulation layout, pump stations, wastewater treatment facility, and the proposed plan change area are shown in Figure 24.

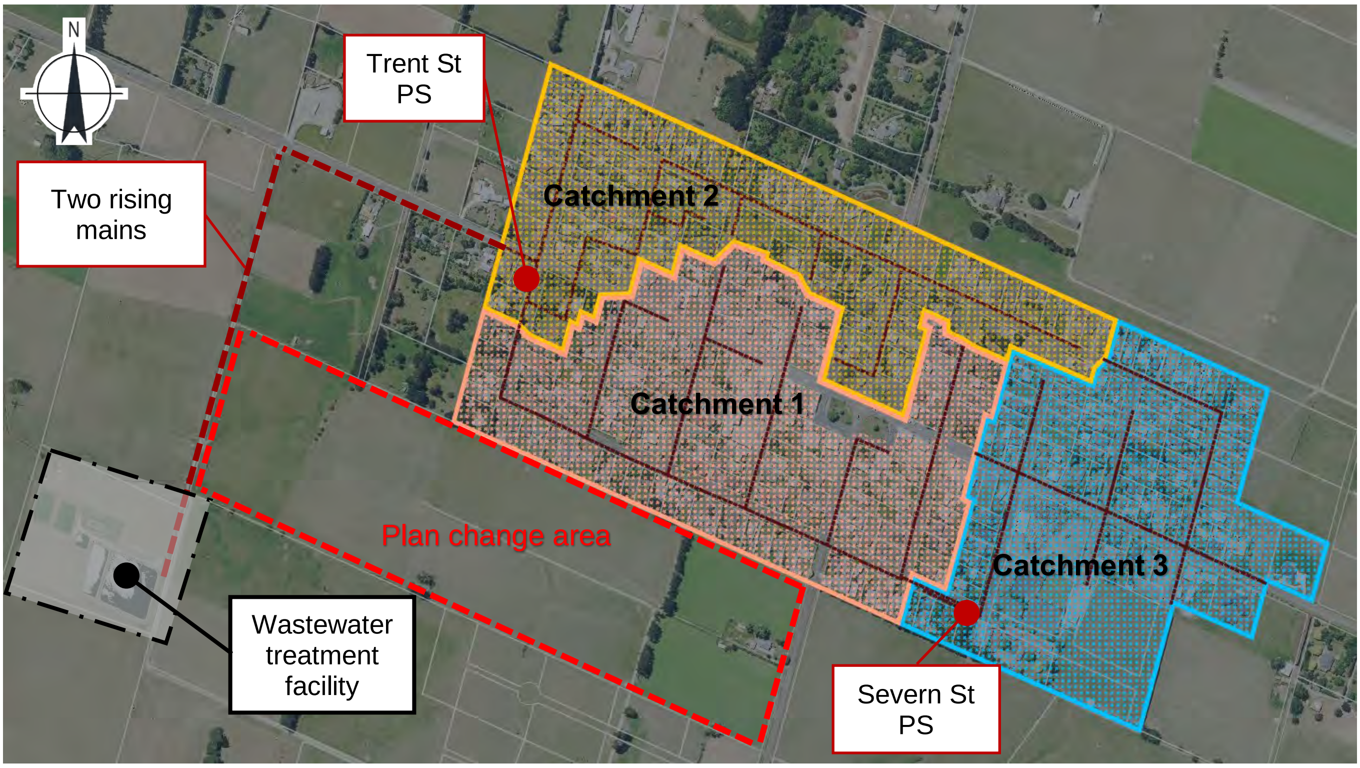


Figure 24 Existing wastewater reticulation system

4.1.2 Pump Stations

Currently two pump stations are required to convey Rongotea's wastewater to the wastewater treatment facility. One pump station is along Severn Street on the southern side of Rongotea that serves catchment 3, and the other pump station is along Trent Street on the western side of Rongotea that serves the entire Village.

The operational philosophy of the pump stations is unknown, and the pump capacity of the pump stations could not be ascertained. For the purposes of this high-level assessment the capacity of the Trent St pump station (to which the development site will discharge to) has been determined based on the capacity of the rising mains. It has been assumed that only one rising main from Trent St is being utilised during a normal pump cycle but that both rising mains can be utilised during high inflow periods. The approximate capacity of a single rising main is approximately 11.5 l/s; therefore, the pump station has a maximum capacity of 23 l/s when both rising mains are operating.

There is currently approximately 15 m^3 emergency storage provided at the pump station. A high-level overflow discharging to the environment is also provided at the pump station.

4.1.3 Wastewater Treatment Facility

MDC have indicated that the existing facultative ponds at the wastewater treatment facility have a design capacity equal to a population of approximately 550 people, or 137.5 m^3/day. The 2018 census population data indicates that Rongotea has a population of 642 people; therefore, the existing ponds are undersized for the current population.

MDC is currently undertaking a wastewater centralisation project that involves conveying wastewater from various rural villages to Feilding's wastewater treatment plant (WWTP). Consequently, MDC is planning to decommission Rongotea's existing facultative ponds and pump Rongotea's wastewater to Feilding from the Trent Street pump station. An additional pump station on the eastern side of Rongotea may be required as part of this work, as well as pump station upgrades. MDC have indicated that the centralisation project will accommodate projected growth, which includes this plan change area.

4.2 Wastewater Model Development

4.2.1 General

A hydraulic model of Rongotea's wastewater reticulation system was created with Bently's SewerGEMS software to evaluate the capacity of the existing system and identify possible upgrades due to future growth within the proposed plan change area. The model utilises the dynamic wave routing method which takes any backwater, flow reversal, and pressurised flow effects into account and produces the most theoretically accurate results. For this investigation no flow monitoring was carried out to confirm actual wastewater demands.

4.2.2 Pipes

Rongotea's existing pipe network was created with GIS data provided by MDC. Nominal diameters (DN) were used as the inside diameter of each pipe. The manning's equation was used to determine the capacity of any gravity pipes and the Darcy-Weisbach equation in conjunction with the Swamee-Jain equation were used to determine the capacity of pressure pipes. A Manning's n value of 0.012 and 0.011 were used for existing AC pipes and new PVC pipes respectively as per NZS 4404:2010 – Table 5.2. Lateral connections were excluded from the model.

4.2.3 Nodes

Nodes were generally created at manholes within the network. A manhole survey provided by MDC was used to assign lid levels to the existing manholes. Invert levels of the existing manholes were calculated from depth data from MDC's GIS data. No on-site measurements were taken to validate any lid or invert levels.

4.2.4 Catchments

Contributing catchments were delineated from MDC's sewer main GIS data and parcel layout data sourced from LINZ. The catchments were further divided into sub-catchments to distribute flow appropriately across the network. The number of properties counted within each catchment were used to calculate the wastewater demands in Section 4.3 below. For the purposes of this assessment the internal network within catchment 2 was excluded from the model since the network capacity in that catchment was irrelevant. Flow from that catchment was loaded as a baseflow.

It should be noted that not all lots included in the catchments are developed and connected to the network, therefore, the wastewater demand calculated from the catchments are considered conservate but suitable for this investigation.

4.2.5 Pumps

Severn St Pump Station

MDC indicated that there are currently two submersible pumps installed at the Severn Street pump station. Actual pump performance curves were not available, therefore the capacity of the pump station has been estimated as 10 l/s from pump curves sourced from the supplier. However, for the purpose of this investigation it has been assumed that a constant flow of 4.2 l/s, as per Table 12, is being pumped to the gravity network of catchment 1. Invert levels were estimated from drawings and GIS data provided by MDC.

Trent St Pump Station

MDC indicated that there are currently two submersible pumps installed at the Trent Street pump station. Actual pump performance curves and flow meter data were not available. Pump performance curves sourced from the supplier however indicated that the capacity of the pump station is in the order of 23 l/s assuming both rising mains are operating concurrently. Invert and operational levels were obtained from drawings and GIS data provided by MDC. The levels provided by MDC indicated that the pump station is roughly 6 m deep.

4.3 Wastewater Demand

Peak flows were calculated using design parameters, as per the latest version of MDC's Engineering Standards and NZS 4404:2010 (refer Section 1.4 of this report). For the purpose of this assessment, these peak flows were used as the basis to evaluate Rongotea's existing and proposed future wastewater infrastructure.

There are no significant commercial/industrial wastewater users in Rongotea. Therefore, it has been assumed that all properties are residential type users, except for the Rongotea School. Based on previous investigations completed by GHD a wastewater demand of 30 litres/child/school day have been assumed.

A summary of Rongotea's existing and future peak wet weather flows are shown in Table 12 below. The table indicates that currently Rongotea has a PWWF of 16.53 l/s and that future PWWF is estimated to be 24.31 l/s.

Table 12 Summary of Rongotea's wastewater demand

Catchment	No of Properties	ADWF (l/s)	PDWF (l/s)	PWWF (l/s)
Catchment 1	150	1.460	3.650	7.300
Catchment 2	103	1.003	2.506	5.013
Catchment 3	82	0.842	2.106	4.211
Existing Rongotea	**335**	**3.305**	**8.262**	**16.525**
New Subdivision	160	1.557	3.894	7.787
Total (Future Rongotea)	**495**	**4.862**	**12.156**	**24.312**

Table Notes:
1. A population of 2.9 people per property were used to calculate total population
2. An average daily consumption of 290 l/p/d were used to calculate ADWF
3. Dry weather peaking factor of 2.5 were used
4. Inflow and infiltration factor for wet weather of 2 were used
5. Catchment 3 includes Rongotea School

The above flows were used in the hydraulic analysis discussed below to evaluate the capacity of the existing wastewater network during PWWF.

4.4 Wastewater Capacity Evaluation

The design criteria used to evaluate the capacity of the existing reticulation network and sizing of the proposed reticulation network includes the following:

- MDC ESLD does not allow any surcharging within the network during PWWF scenarios
- Minimum grades for self-cleansing as per Table 4.1 of MDC ESLD
- Minimum self-cleansing velocities at full flow of 0.7 m/s and maximum velocities of 3 m/s
- A minimum of 4 hours of storage capacity equivalent to 2 x ADWF above the high-level alarm

The following scenarios were investigated to evaluate the capacity of the existing network and sizing of the proposed reticulation network:

- Scenario 1 – Rongotea existing PWWF (16.53 l/s)
- Scenario 2 – Rongotea future PWWF (24.31 l/s) – Gravity and rising main
- Scenario 3 – Rongotea future PWWF (24.31 l/s) – Gravity, LPS, and rising main

A description of the above scenarios and the results are further discussed below.

4.4.1 Existing System Evaluation

Gravity network

The proposed plan change area is adjacent to existing Catchment 1 as shown in Figure 24. Due to the nature of the topography of the area falling predominantly to the west, the most suitable location to connect the proposed plan change area to the existing reticulation network is at the intersection of Trent Street and Severn Street as shown in Figure 25 below.

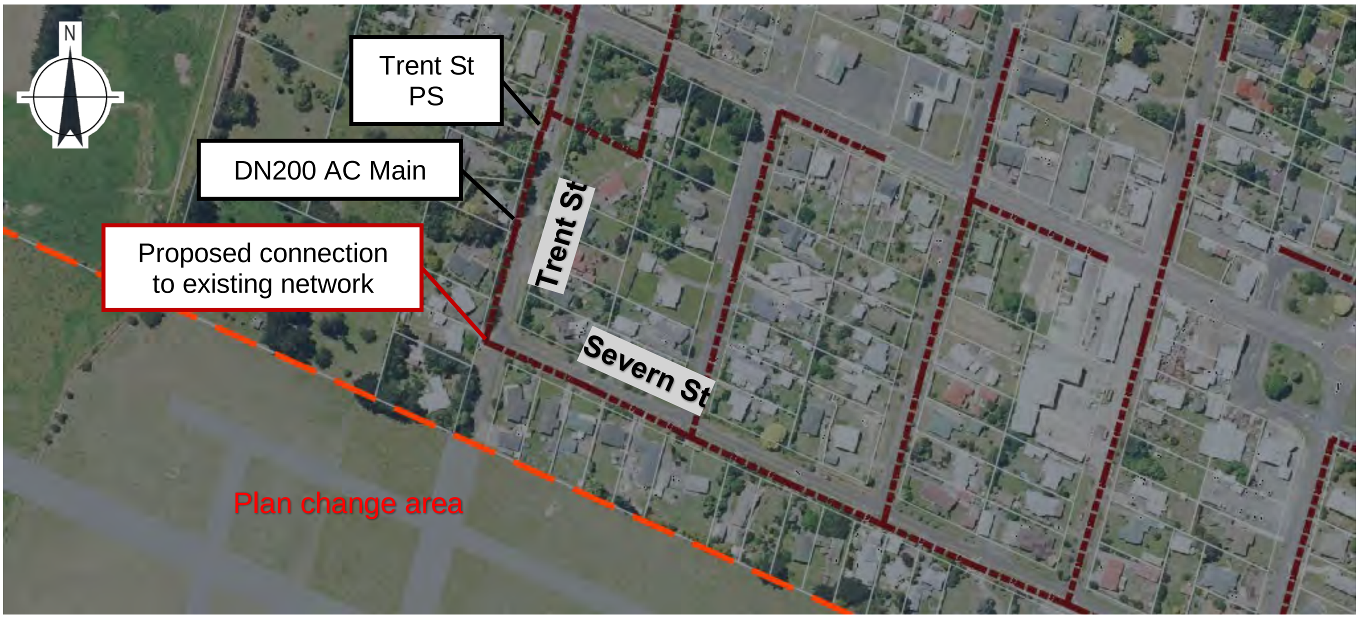


Figure 25 Proposed connection to existing wastewater network – Plan view

The existing reticulation network at this location is a DN200 AC main. The slope of the main directly downstream of the proposed connection point is approximately 1:250 or 0.4 % and has a capacity of 24 l/s. Flow from the majority of Catchment 1 and Catchment 3 are flowing through the pipe at the proposed connection point. For scenario 1, the total PWWF flowing through this point is 11.5 l/s which is less than the capacity of 24 l/s. A profile indicating the HGL and capacity of the existing system upstream and downstream of the proposed connection point is shown in Figure 26 below.

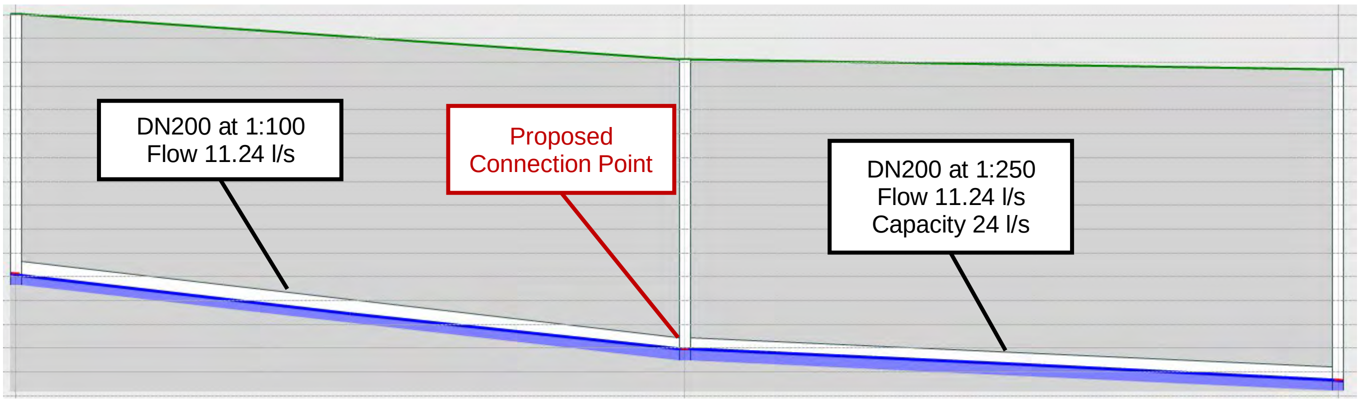


Figure 26 Existing wastewater network – Profile view

Trent Street Pump Station

Currently there is approximately 15 m^3 emergency storage at the Trent Street pump station. The provided storage is currently not compliant with MDC's ESLD which requires an emergency storage of approximately 95 m^3 at ADWF as per Table 12. However, MDC is planning to provide an additional 80 m^3 of emergency storage as part of the wastewater centralisation project.

The pump station has an assumed capacity of approximately 23 l/s. For scenario 1, as indicated in Table 12, Rongotea has a PWWF of 16 l/s which is less than the approximate capacity of the pump station of 23 l/s. This indicates that the Trent St pump station currently has sufficient capacity.

4.4.2 Proposed System Evaluation

Generally, the wastewater reticulation within the plan change area has been sized to minimise the pipe size while maintaining minimum grades and self-cleansing velocities. The reticulation layout follows the roading layout and any changes to the roading layout will affect the layout of the reticulation network and may affect the levels, gradients, and therefore pipe sizing. Due to the topography of the area generally being lower than the existing developed area, a new pump station and rising main will be required to service the plan change area.

The following two options to service the proposed plan change area have been considered:

– Scenario 2 – A gravity network on either side of the Ruivaldts / Walsh drain discharging to a new pump station
– Scenario 3 – A gravity network on the eastern side of the Ruivaldts / Walsh drain and a low-pressure sewer (LPS) system on the western side of the drain

The proposed reticulation network for scenarios 2 and 3 of the plan change area is shown in Figure 27 below.

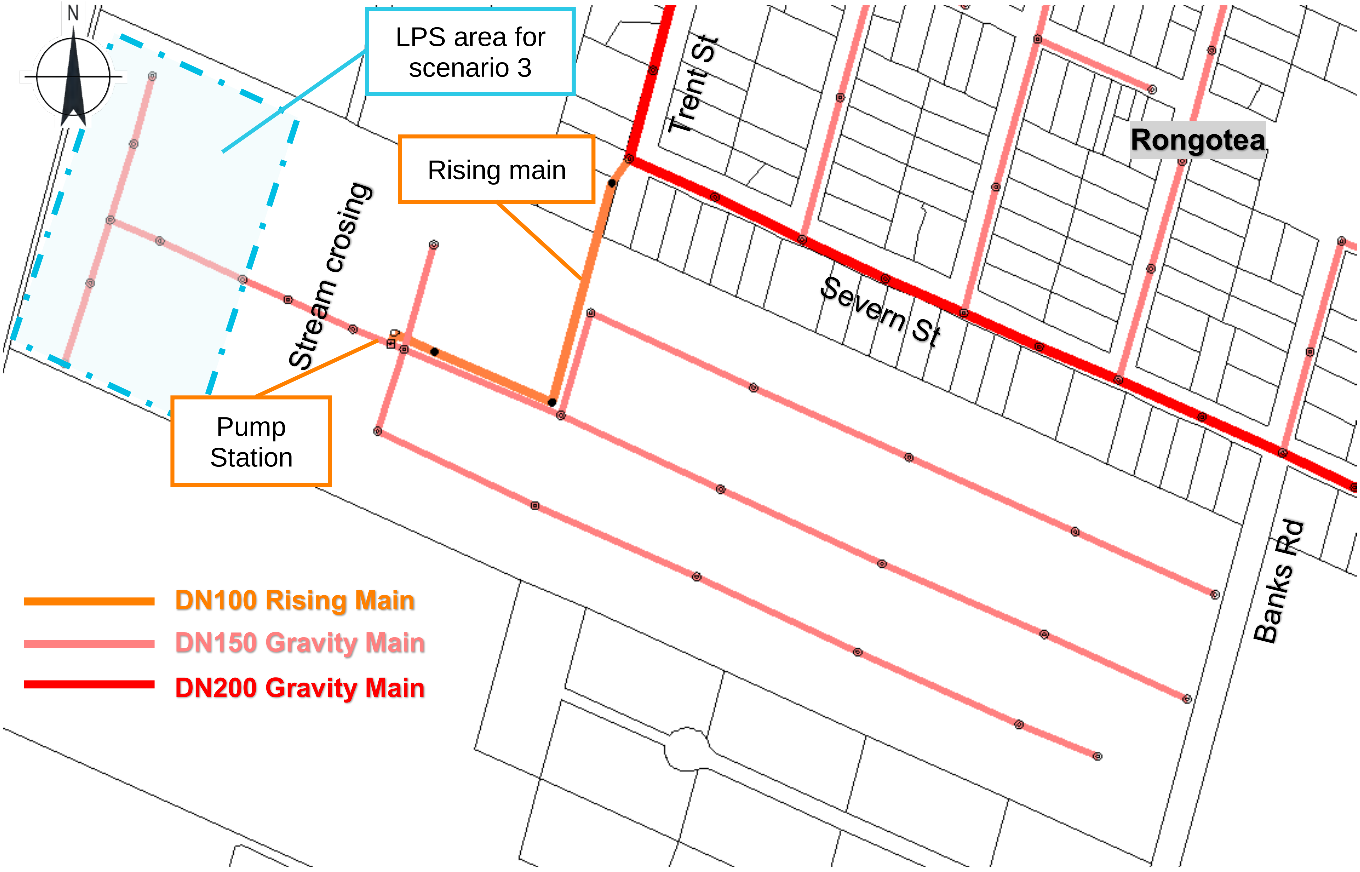


Figure 27 Proposed wastewater reticulation network

As shown in Figure 27 above, the gravity mains are all DN150 mains laid at the minimum grade of 1:150. As discussed with MDC during the 30 June workshop, the gravity main does not allow for further expansion of the residential zone to the south. That is, the depth and size of the gravity main with the plan change area is based on the plan change area only.

The proposed location of the new pump station is on the eastern side of the Ruivaldts / Walsh drain. From the pump station a new DN100 rising main is required up to the proposed connection point at the intersection of Trent Street and Severn Street.

For scenario 2, properties on the eastern and western side of the stream crossing are serviced via DN150 gravity mains. The gravity network from the west need to cross underneath the Ruivaldts / Walsh drain, consequently resulting in a deep network and a new 8-9 m deep pump station. A profile along the western gravity line including the pump station is shown in Figure 28 below. This assumes a 1 m minimum separation below the drain.

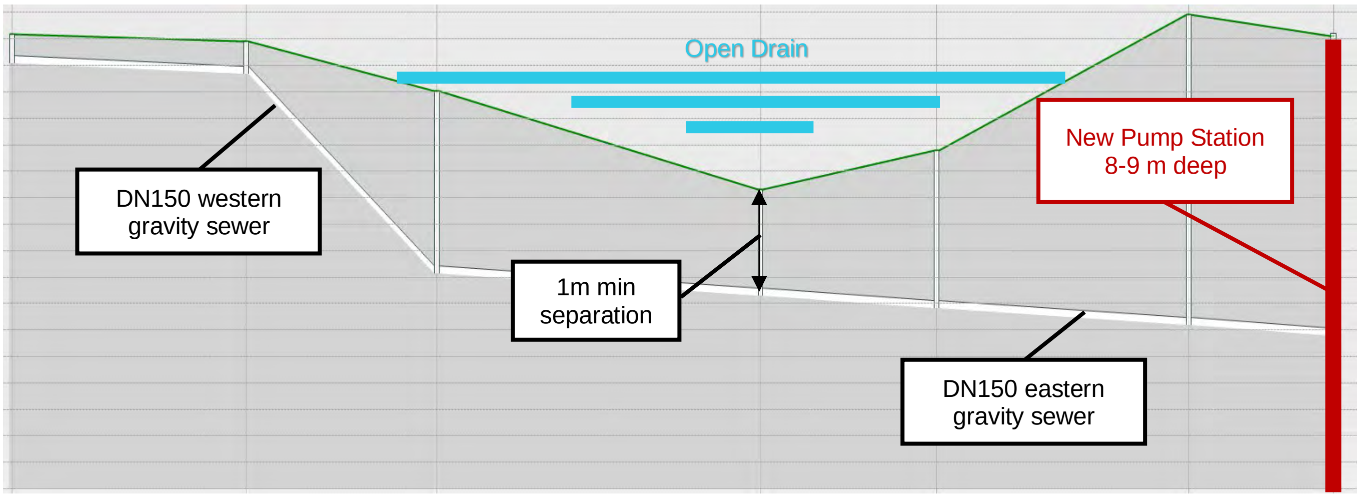


Figure 28 Western gravity sewer and pump station long section (scenario 2)

For scenario 3, it is proposed that the properties on the western side of the stream be serviced via a low-pressure sewer (LPS) system. LPS systems are alternatives to conventional gravity systems and have advantages in areas with technical constraints such as liquifiable land, rolling terrain, or downstream network capacity constraints.

A LPS system typically includes the following elements:

- Grinder pumping units and chambers (installed on each property)
- Boundary kits located at the legal boundary
- Property discharge lines that connect the pumps to the boundary kits
- Control/alarm panels that controls the operation of the pumps
- A pressure pipe network typically consisting of small bore mains, isolation valves, flushing pits, and air release valves

Palmerston North City Council requires that the boundary kit and pressure network be owned by the Council while the property owner is responsible for the property discharge lines and all parts of the system upstream of the boundary kit. It is expected that MDC would also adopt this ownership approach.

Since the LPS system does not rely on gravity to dictate invert levels, the new pump station will be shallower (approximately 5-6 m deep) than the pump station of scenario 2. A profile along the eastern gravity line including the pump station is shown Figure 29 below.

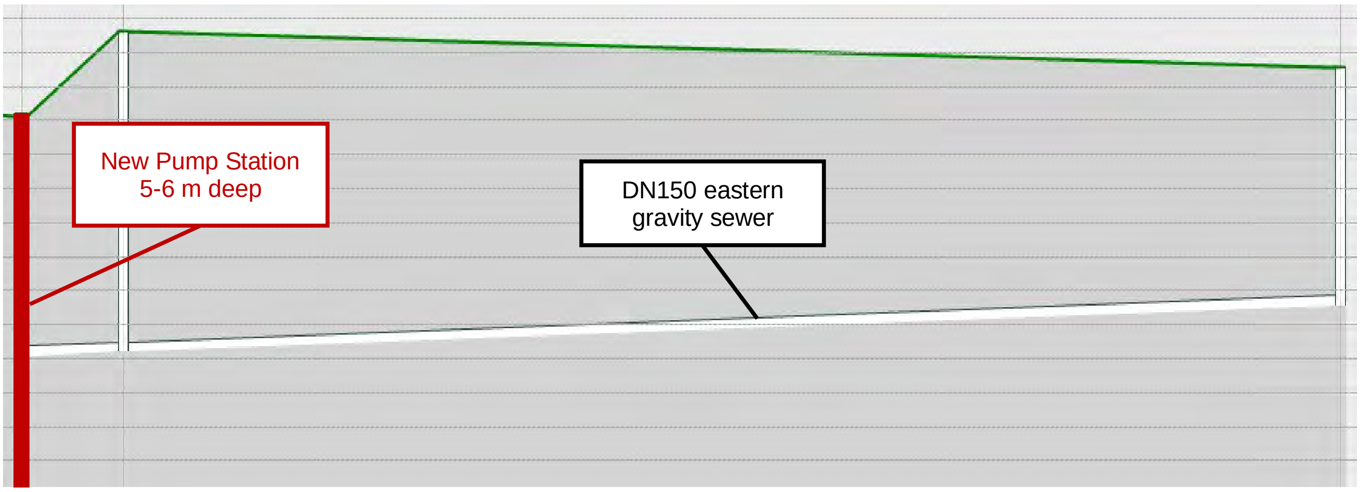


Figure 29 *Eastern gravity sewer and pump station long section (scenario 3)*

For both scenarios, the emergency storage required at the new pump station is 45 m^3 based on a future ADWF of 1.6 l/s as per Table 12. Various layouts are possible to configure the pump station and emergency storage structure. As an example, a DN 2050 storage tank 14 m long has a capacity of approximately 45 m^3 and may fit into the road reserve. Alternatively two DN 2050 storage tanks 7 m long can be used, which would require a portion of a proposed lot to be used.

Existing Network

Based on a total of 160 lots, the PWWF from the proposed plan change area for scenario 2 and scenario 3 is 7.8 l/s. Therefore the total flow at the connection point (intersection of Trent St and Severn St) to the existing gravity network is 19.3 l/s, which is less than the capacity of the existing system of 24 l/s. This means that the existing receiving wastewater system has sufficient capacity to convey PWWF from the proposed new development to the Trent Street pump station. A profile indicating the HGL and capacity of the existing system upstream and downstream of the proposed connection point is shown in Figure 30 below.

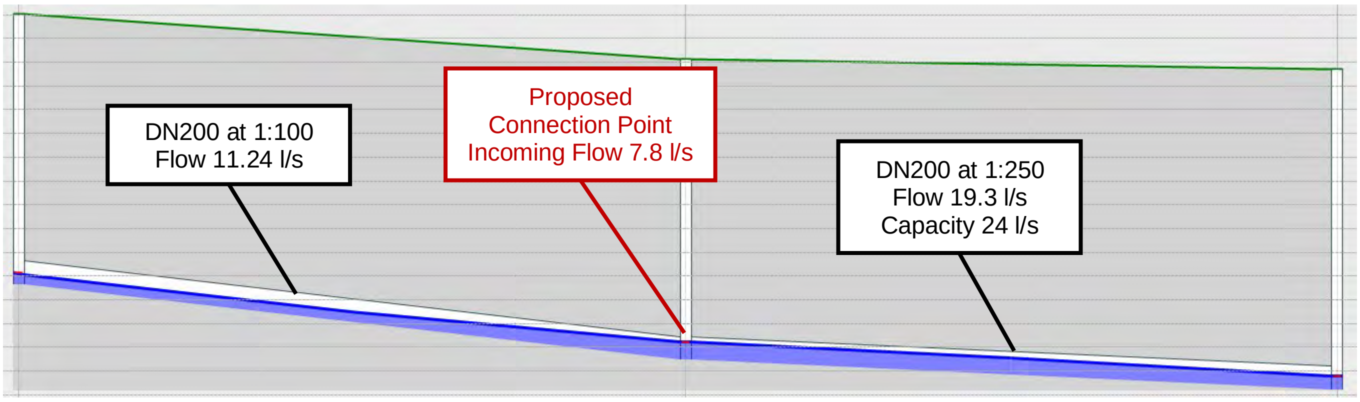


Figure 30 Existing wastewater network with future flows – Profile view

Trent Street Pump Station

As indicated in Table 12, the proposed future PWWF of Rongotea for scenario 2 is 24.3 l/s which exceeds the capacity of the existing pump station of 23 l/s by 1.3 l/s assuming both rising mains are operating. Although peak flows under Scenario 3 would likely be reduced due to the lower peaking factors in LPS systems, due to the small number of properties the peak flow would only be expected to decrease by 0.3 l/s. The peak flow under both scenarios has therefore been assumed gravity to be conservative.

By assuming a future storage volume of 95 m^3, it will take approximately 20 hrs to fill up the storage during PWWF. PWWF typically only lasts a couple of hours, therefore it is not anticipated that any additional storage or upgrades to the pump station will required due to future flows from the plan change area.

4.5 Summary

From the investigations carried out as part of this report, it has been concluded that:

– The existing wastewater treatment facility does not have sufficient capacity for the existing population. However, MDC is planning to pump Rongotea's wastewater to Feilding, therefore eliminating the need to upgrade the existing wastewater treatment facility.
– The provided storage of 15 m^3 at the Trent Street pump station is not compliant with MDC's ESLD which requires an emergency storage of 95 m^3. However, MDC is already planning to supplement the existing storage with an additional 80 m^3 as part of the centralisation project.
– The receiving wastewater gravity network has sufficient capacity for the additional demand from the proposed plan change area.
– Due to the topography of the area generally being lower than the existing residential area, a new pump station and rising main will be required to service the plan change area in addition to a conventional gravity system.
– The plan change area can be serviced via the following two options:
  - Option 1 – A gravity system on either side of the Ruivaldts / Walsh drain. The bottom of the drain is the catchment low point which makes servicing of the properties on the western side of the drain via gravity expensive due to deep trenches. This option will also require a deep pump station (approximately 8-9 m deep).
  - Option 2 – A gravity system on the east of the drain and a LPS system on the west of the drain. A LPS system is an alternative to conventional gravity system and will allow for a shallower network, but requires the property owner to take ownership of some of the components of the system.

In order to service the plan change area it is recommended that:

– The plan change area be connected to the existing reticulation network at the intersection of Trent Street and Severn Street.
– Due to technical constraints of the area the properties on the western side of the Ruivaldts / Walsh drain be connected to the new pump station via a LPS system instead of a conventional gravity system.

– A conventional gravity system be provided to service the properties on the eastern side of the stream crossing.

5. Conclusions and Recommendations

5.1 Conclusions

The high-level three waters servicing assessment shows that servicing the private plan change area is feasible. Conceptual three waters network layouts are provided in Appendix A.

Stormwater

The 200-year flood modelling shows areas of inundation on the site are primarily located around Horizons' Ruivaldts and Campbells Drains. This likely provides much needed floodplain storage for Horizons' Te Kawau drainage scheme. Two overland flow paths were also predicted in the plan change area, but these can be managed through the stormwater servicing of the site.

The general stormwater approach includes intercepting and diverting overland flows generated upstream of the plan change area and conveying stormwater runoff from the site to centralised treatment and attenuation facilities before discharging to the Campbells or Ruivaldts Drains. On the east side of the Ruivaldts Drain, runoff is conveyed via kerb and channel and a piped reticulation network to the adjacent site that has been identified for stormwater use. A combined wetland and attenuation pond facility can provide treatment of the first flush and attenuation during larger storm events, before discharging to the Campbells Drain. West of Ruivaldts Drain, runoff is directed to raingardens which provide treatment and attenuation, before discharging to the Ruivaldts or Campbells Drain. Loss of floodplain storage due to the combined wetland and attenuation facility will require additional earthworks in the land identified for stormwater use, whilst considering the implications on the existing natural inland wetland along the Campbells drain. A second option involves relocating the combined wetland and attenuation facility at least 100 m away from the existing natural inland wetland. Although this second option removes the non-complying activity, it also does not provide the same opportunity as Option A does to enhance and restore the existing wetland.

Potable Water

The general water servicing approach includes providing DN100 mains with rider mains along the proposed road reserves. The roading layout provides a ring main layout, allowing for sufficient interconnectivity. New connections to the existing reticulation network are required at the intersection of Trent Street and Severn Street and at the intersection of Banks Road and Severn Street, with a minor upgrade required at Trent Street. The existing reticulation system is not compliant with SNZ PAS 4509 for a commercial (FW 3) fire classification, and an upgrade to the pump system will be required.

Wastewater

The general wastewater servicing approach includes provision of a DN150 gravity system on the eastern side of the stream crossing and either a gravity or LPS system on the western side of the drain. The area on the western side is going to be challenging to service with the conventional gravity system due to deep excavations. A new pump station and rising main is required due to the general topography of the area being lower than the existing residential area. The rising main can connect to the existing reticulation at the intersection of Trent Street and Severn Street.

5.2 Recommendations

Recommendations for 3 waters servicing of the plan change area are presented in Sections 2.6, 3.5 and 4.5 of this report.

The following next steps are recommended to support the design of the three waters infrastructure in the plan change area:

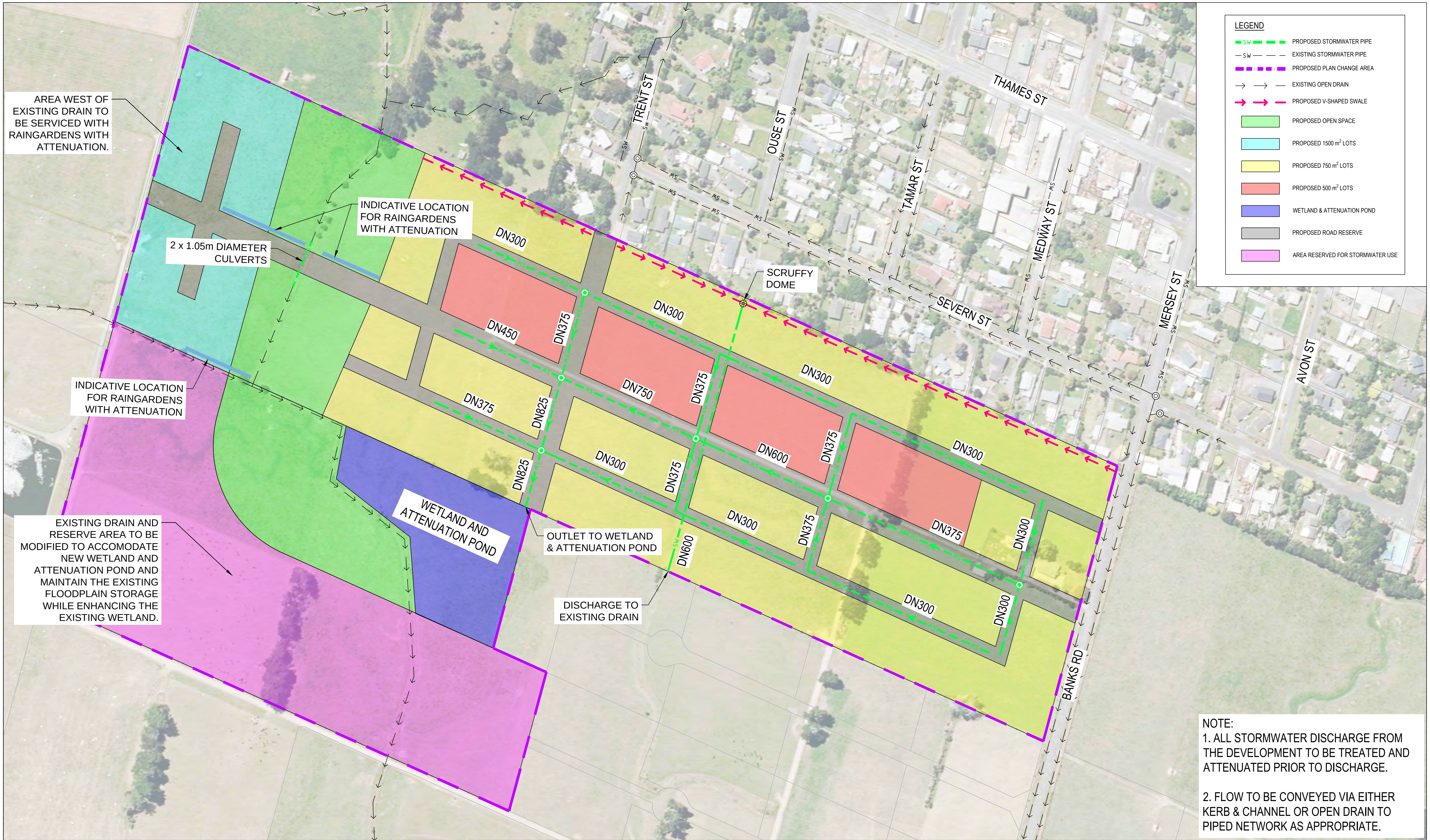
– The concept design presented herein be revisited and further developed based on the future scheme plan and associated earthworks, lot yield, impervious area assumptions, etc.

- Geotechnical investigation be carried out to support design of wetland and attenuation facility and 3 waters infrastructure for the development.
- Engagement with MDC to confirm design storm profile to be used for the attenuation pond.
- Engagement with Horizons to confirm floodplain recontouring and potential realignment of part of the Campbells Drain (if required).
- The existing water pumps be upgraded to produce an additional pressure of at least 380 kPa at the start of the reticulation system. This will ensure that all current and future fire flow requirements are met.
- The upgrades to the water pumps be investigated during the next replacement cycle, which is due within the next 5 to 10 years.
- A LPS system be used to service the properties on the western side of the stream crossing instead of using a conventional gravity system to decrease the depth of the gravity mains.
- The Rongotea to Feilding centralisation scheme be completed prior to connection of any properties to the existing network.

Appendices

Appendix A

Conceptual Three Waters Layout Drawings



PLAN
SCALE 1:1500

Rev	Description	Checked	Approved	Date
0	CONCEPT	CM	ARB	21/07/22

Author: D DAO — Drafting Check: W WAGNER
Designer: C MURRAY — Design Check: AR BAUGHAM

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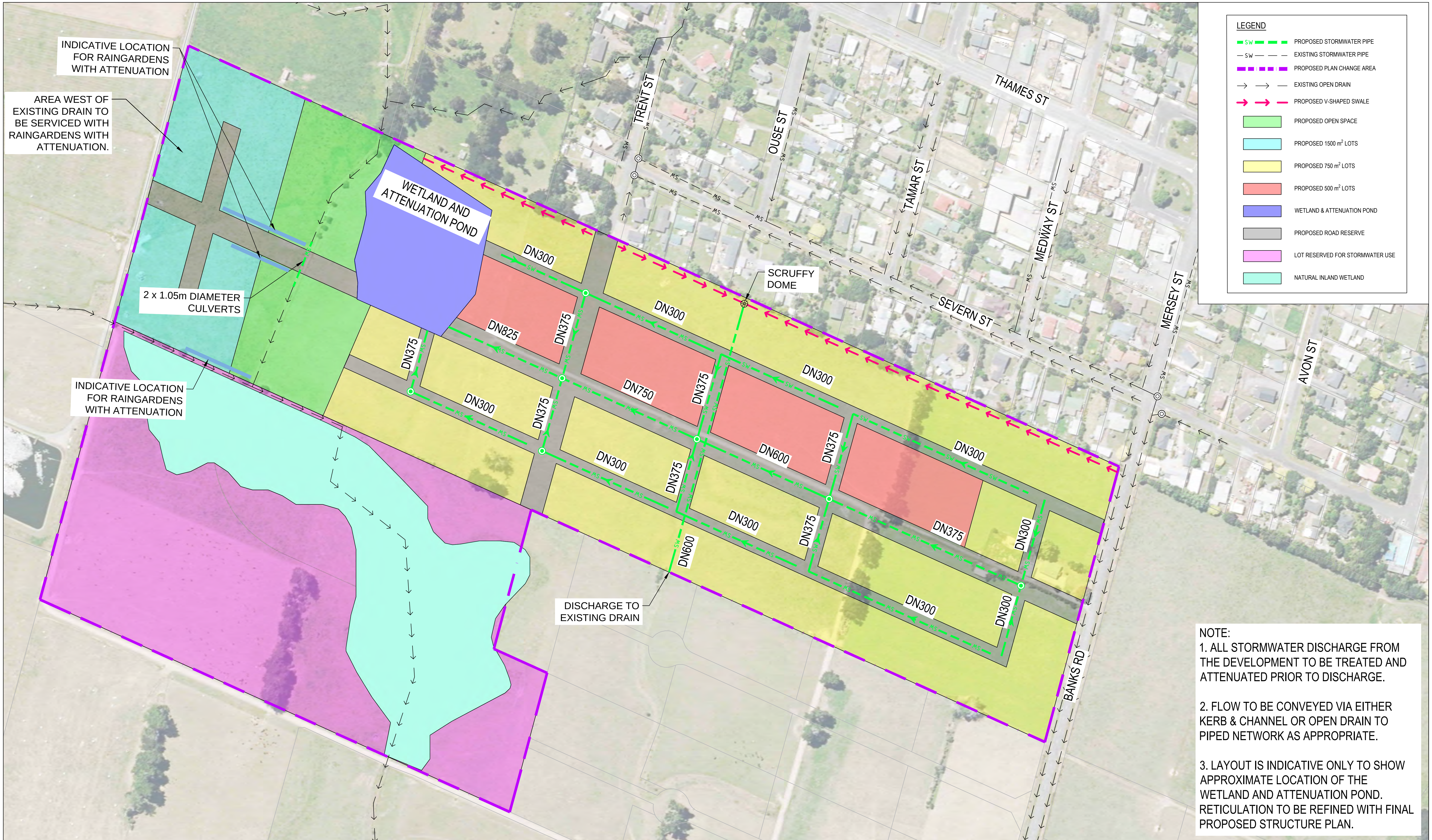
Project No. 12581369

Client: THE PROPERTY GROUP
Project: THREE WATERS SERVICES FOR RONGOTEA PRIVATE PLAN CHANGE
Status: CONCEPT — Status Code: S4
Drawing Title: CONCEPT DESIGN - STORMWATER PLAN OPTION A
Size: A1
Drawing No. 12581369 STRM-A
Rev 0

Plot Date: 9 November 2022 - 11:51 AM   Plotted by: Claire Murray   File Name: \\ghdnet\ghd\NZ\Palmerston North\Projects\51\12581369\CADD\Drawings\12581369_Rongotea_SWLayout-OptA.dwg



NOTE:
1. ALL STORMWATER DISCHARGE FROM THE DEVELOPMENT TO BE TREATED AND ATTENUATED PRIOR TO DISCHARGE.

2. FLOW TO BE CONVEYED VIA EITHER KERB & CHANNEL OR OPEN DRAIN TO PIPED NETWORK AS APPROPRIATE.

3. LAYOUT IS INDICATIVE ONLY TO SHOW APPROXIMATE LOCATION OF THE WETLAND AND ATTENUATION POND. RETICULATION TO BE REFINED WITH FINAL PROPOSED STRUCTURE PLAN.

PLAN
SCALE 1:1500

Rev	Description	Checked	Approved	Date
0	CONCEPT	CM	ARB	21/07/22

Author D DAO — Drafting Check W WAGNER
Designer C MURRAY — Design Check AR BAUGHAM

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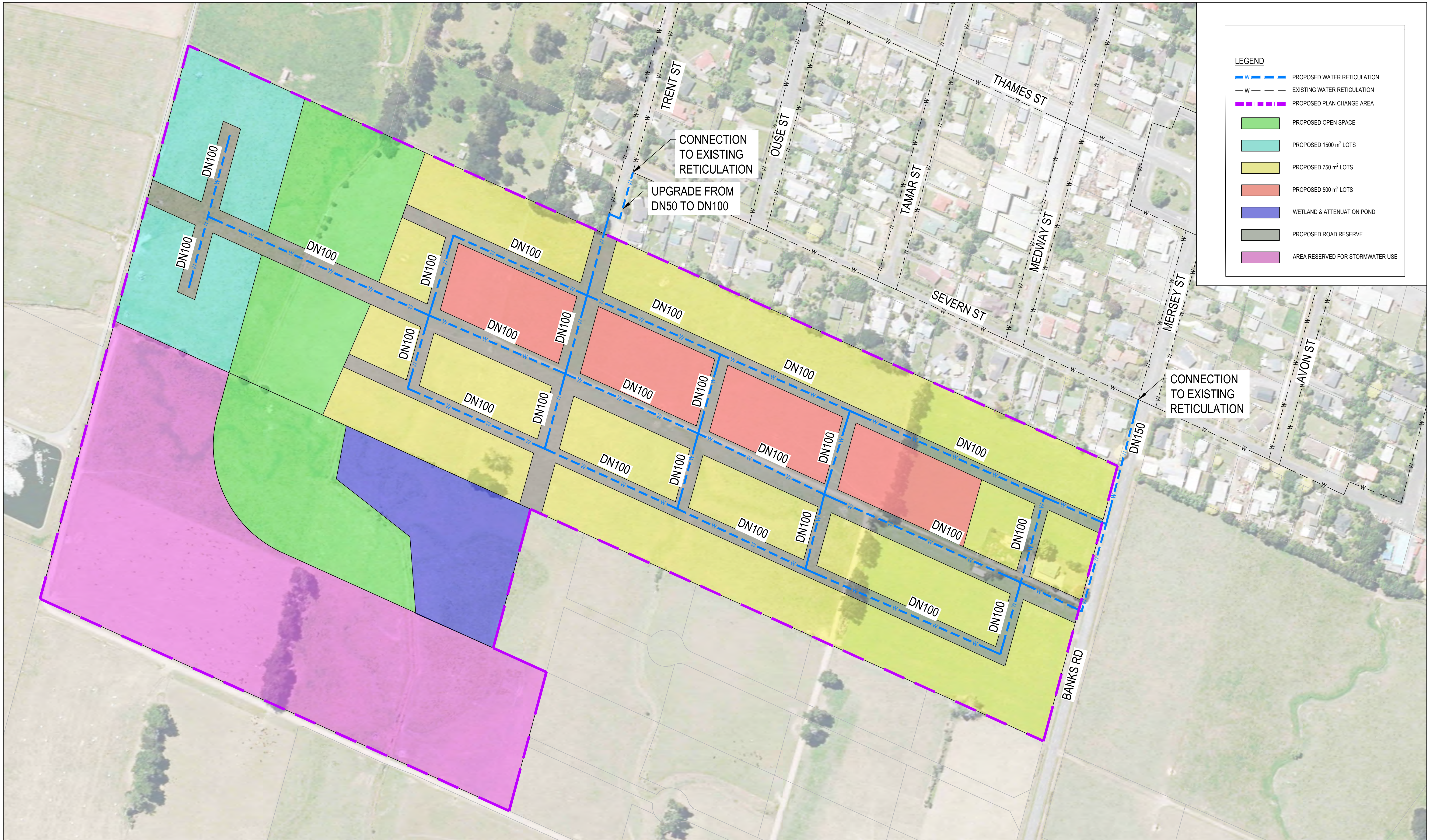
Project No. 12581369

Client: THE PROPERTY GROUP
Project: THREE WATERS SERVICES FOR RONGOTEA PRIVATE PLAN CHANGE
Status: CONCEPT — Status Code S4
Drawing Title: CONCEPT DESIGN - STORMWATER PLAN OPTION B
Size A1
Drawing No. 12581369 STRM-B
Rev 0

Plot Date: 9 November 2022 - 12:04 PM Plotted by: Claire Murray File Name: \\ghdnet\ghd\NZ\Palmerston North\Projects\51\12581369\CADD\Drawings\12581369_Rongotea_SWLayout-OptB.dwg



PLAN
SCALE 1:1500

Rev	Description	Checked	Approved	Date
0	CONCEPT	WW	ARB	21/07/22

Author D DAO — Drafting Check W WAGNER
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Project THREE WATERS SERVICES FOR RONGOTEA PRIVATE PLAN CHANGE

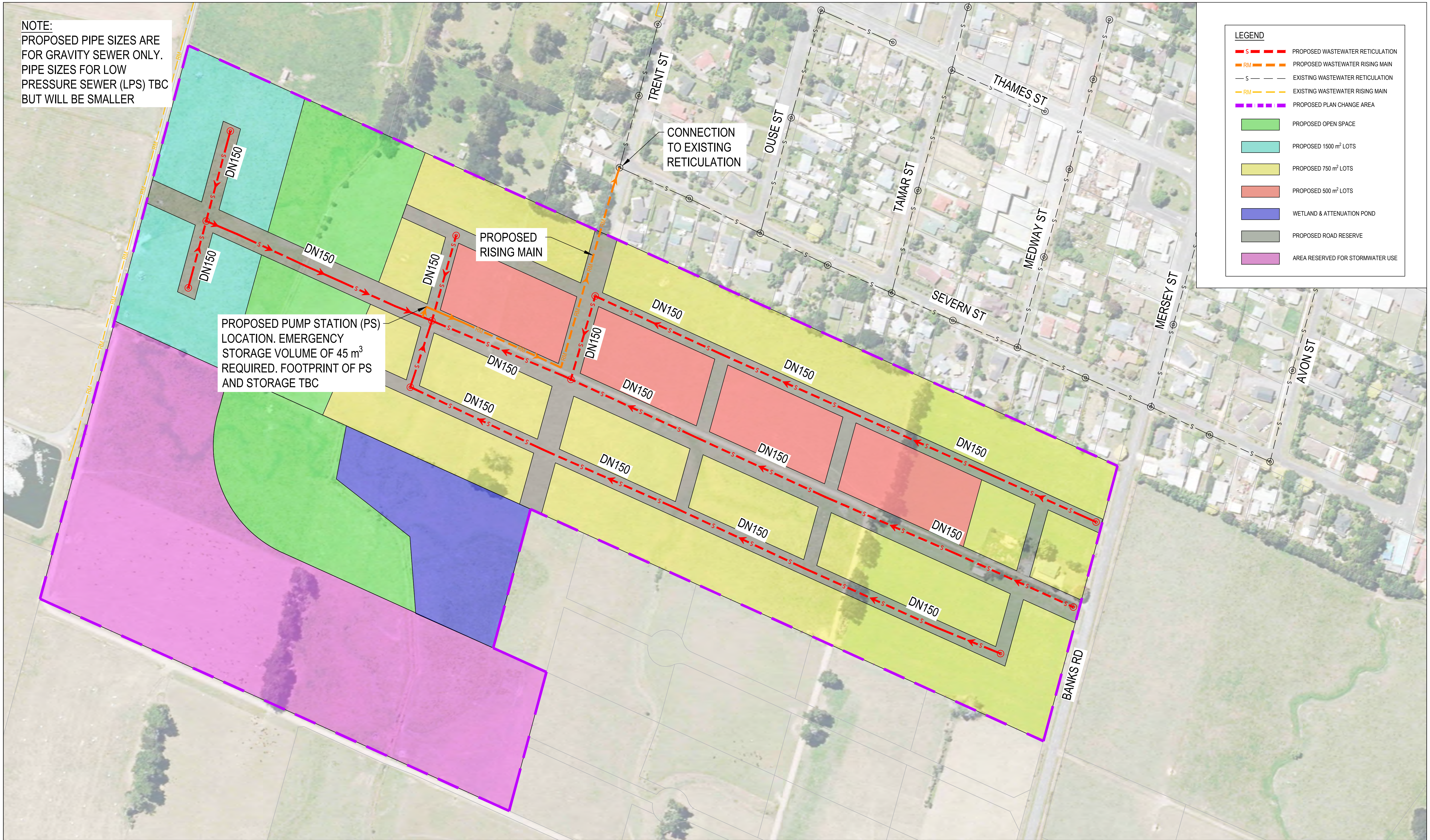
Status CONCEPT — Status Code S4

Drawing Title CONCEPT DESIGN - WATER SUPPLY LAYOUT

Size A1

Drawing No. 12581369 WAT — Rev 0

Plot Date: 21 July 2022 - 2:29 pm Plotted by: Clay O'Donnell File Name: N:\NZ\Palmerston North\Projects\51\12581369\CADD\Drawings\12581369_Rongotea_WSLayout.dwg



PLAN

SCALE 1:1500

Rev	Description	Checked	Approved	Date
0	CONCEPT	WW	ARB	21/07/22

Author D DAO — Drafting Check W WAGNER

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Project No. 12581369

Client: THE PROPERTY GROUP

Project: THREE WATERS SERVICES FOR RONGOTEA PRIVATE PLAN CHANGE

Status: CONCEPT — Status Code S4

Drawing Title: CONCEPT DESIGN - GRAVITY WASTEWATER LAYOUT

Size A1

Drawing No. 12581369 SEW

Rev 0

Plot Date: 21 July 2022 - 2:30 pm — Plotted by: Clay O'Donnell — File Name: N:\NZ\Palmerston North\Projects\51\12581369\CADD\Drawings\12581369_Rongotea_WWLayout.dwg ghd.com

The Power of Commitment