MEMO

TO: Harrison Hitchins
DATE: 28 February 2022
FROM: Warren Uys
PROJECT NO.: J000402
SUBJECT: Ropata Village redevelopment - Resource Consent – 3 Waters Design Summary

BACKGROUND

Windsor Management are re-developing the 4,030m2 site at 758 - 760 High Street, Boulcott, in Lower Hutt.

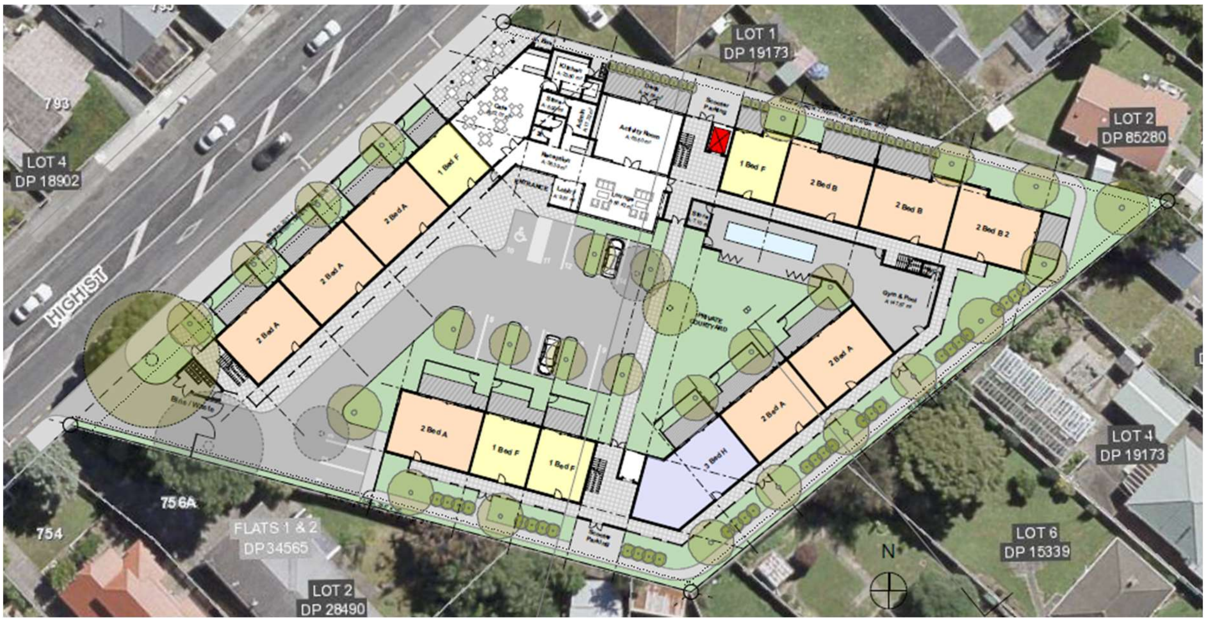

Image 1: Proposed Development

The proposed development includes a retirement village of approximately 50 apartments including 1, 2 and 3 bed typologies. The proposed build structure is 3 stories high and includes a Café, Reception/Administration, Activity Room and a Lounge.

EXISTING COUNCIL UTILITIES

Water, wastewater and stormwater pipes are all present within the High Street road corridor, in front of the proposed development (refer to Image 2 below).

The site currently discharges stormwater runoff during the primary storm event to a public manhole located near the properties' south-west corner. Downstream of the manhole a 225mm pipeline crosses High Street and discharges into a 300mm main on the north-west side of High Street. This 300mm main is shown to be approximately 1.6m deep.

A public sewer main exists to the north-west of the site (225mm dia. – 3.3m deep) within the carriageway, and another to the south-east (150mm dia.) within the rear of several Dyer Street properties.

A public water main (100mm dia.) exists outside the front of the site within the High Street carriageway.

No alterations are proposed to these existing stormwater, sewer or water mains other than the proposed new connections to these mains.

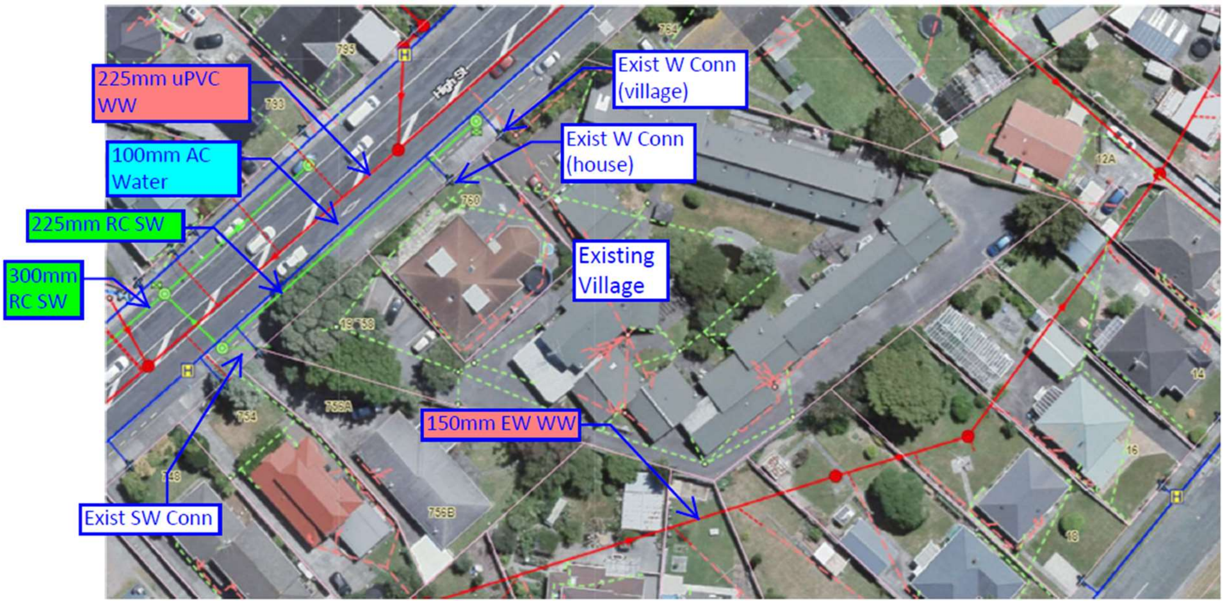

Image 2: Existing Aerial and GIS Services

EXISTING PRIVATE UTILITIES

Ropata Village currently has internal water, stormwater, wastewater and gas lines. These existing pipes will need to be removed as they will run under the proposed structures and are likely not in a condition where they can be reused. Hutt City have (via Wellington Water) stated that the existing sewer connections can be reused subject to a condition and capacity assessment, and provided that the existing buildings/structure are re-configured only (i.e. not upsized). Water laterals will require replacing if they are not of an MDPE material.

It is therefore not recommended that the existing lines are maintained for the proposed development.

COUNCIL ADVICE

Advice regarding the 3 waters was received from Hutt City Council on the 2 August 2021. The advice was via Wellington Water and is appended to this report in full (refer Appendix B). A summary of the key aspects is below:

- Stormwater
  - There is no flooding shown within the site, or in the surrounding sites, in the current Waiwhetu flood model.
  - New building floor levels are therefore to comply with the NZBC.

❖ Wastewater

- The High Street local system (225mm dia.) discharges into the trunk network (300mm dia.) There appears to be no spare capacity in this local network during a 1-year LTS design event. Furthermore, this part of the network is heavily surcharged.
- A second local network exists towards Dyer St. and appears to have at least 5 litres/sec spare capacity during a 1-year LTS design flow. This network then discharges into the same 300mm trunk network.
- The trunk network eventually discharges into the Barber Grove pump station. A significant portion of the trunk network appears to already over capacity (i.e. no spare capacity) in a 1-year design event. There are two engineered overflows at the pump station at the incoming main and the rising main. They both discharge to the Hutt River.
- Further development of this property will exacerbate this.
- It is assumed that the structure will stay as is with only possible reconfiguration. If this is the case the existing laterals could be reused however the applicant will need to reassess these and ensure they are in good condition and of capacity to convey the flow.

❖ Water

- The model shows that minimum pressure at the point of supply on the public main is expected to be about 45-50m, which meets the level of service criteria for pressure.
- The model also indicates that available fire flow capacity from the existing hydrant(s) is expected to be compliant with the NZ Fire code for residential areas (FW2).
- However, considering the building and their usage, it is recommended that a hydrant flow test is carried out and the compliance with the NZ Fire code requirements are approved by a fire engineer.
- The applicant will need to assess the water supply lateral and if not constructed of MDPE will need to change these to MDPE.

PROPOSED NEW PRIVATE UTILITIES

The proposed alignments of the internal 3-waters networks are shown in Figure 3. Further details are also provided in the attached engineering drawings.

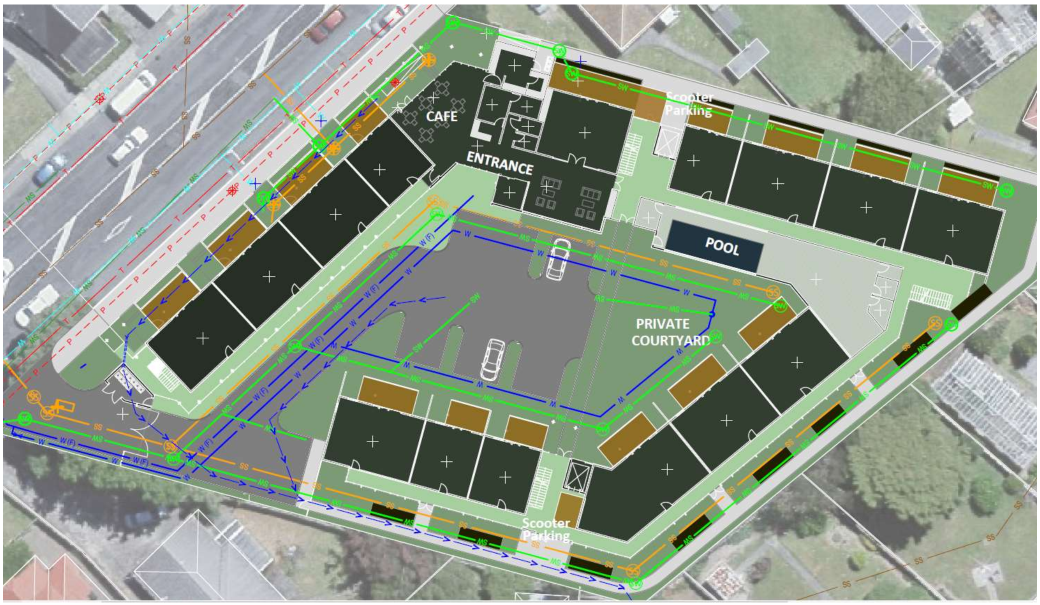

Figure 3: Proposed 3 Waters Alignments

❖ STORMWATER

➢ Proposed Design Standards

The following design standards have been adopted unless otherwise referenced.

- NZBC Clause E1 Surface Water
- Section 4 of the Regional Standard for Water Services (version 2.0: May 2019)
    - 4.4.5 Private Connections to the Public Stormwater System

➢ Design Storm Flows

Stormwater flow and volume calculations have been undertaken to assess the existing and developed scenarios. A summary of these calculations is shown in the table below. Full calculations can be found in Appendix C.

Table 1: Additional Stormwater Runoff

	10% AEP 10min (RSfWS)	**10% AEP 20min (RSfWS)**	**1% AEP 10m (RSfWS)**	**1% AEP 20m (RSfWS)**
Existing Discharge (L/s)	70.82	47.31	111.77	73.90
Developed Discharge (L/s)	71.99	48.09	113.61	75.12
Difference (L/s)	+1.17 (1.65%)	+0.78 (1.65%)	+1.84 (1.65%)	+1.22 (1.65%)
Neutrality Volume (m^3)	0.70	0.94	1.10	1.46

➢ Proposed System

The proposed stormwater lines are to run along both sides of the buildings. This will allow the roof water to be captured without having to direct the water through or under the buildings themselves. The stormwater collected by this network will drain out to High Street and into the existing public network. The existing connection to the public manhole will be replaced and upsized to a 225mm dia. pipe.

Two (type 2) sumps are proposed to collect the stormwater in the driveway and carpark areas. These sumps will be connected to the proposed internal stormwater pipes collecting water from the roofs. These pipes have been designed for the 10-year rainfall event. However, in the case that these sumps are overwhelmed or blocked, the carpark will temporarily flood and then flow via a secondary flow path around the southern building to the existing release points from the site (the secondary flow paths are shown as blue arrows).

All internal pipes have been sized for the 10-year rainfall event, at a minimum gradient of 0.5m/s. Resulting pipe sizes range 150mm – 225mm dia.

➢ Site Neutrality

Although there was no requirement in Wellington Water's initial advice for attenuation, it is good practice to consider the impact of any increase in runoff due to the proposed re-development of the site. An analysis of the runoff increase due to the proposed development has therefore been completed. This was done by comparing the roof and hardstand areas between the existing and proposed site. As shown in the above table, it was found that the increase in runoff was only 1.5 m^3 for the 20-min 100-year storm (Wellington Water generally consider the 20min duration to be critical). This is because there was very little increase in the impervious area in the proposed development.

If there are any existing stormwater detention structures, such as tanks with a controlled outlet or temporary ponding, then the development will need to provide the same detention capacity plus the additional capacity as required to mitigate the additional runoff from the development. If this is required, then the proposed carpark design provides sufficient temporary attenuation within the carpark and existing pond areas of approximately 17 m^3 (to the spill point at RL 11.370m (WLG 1953 datum)).

- Overland Flow Path Impact

Based on an analysis of the existing topographical data provided, it appears that secondary runoff currently generated from the site flows towards three locations along the south and south-east boundaries of the site, being the rear of No.16, 20 and 26 Dyer Street. Three existing sumps are located in these low points, and although bounded by vertical kerbs would overflow in a concentrated fashion into these properties when their capacities are exceeded.

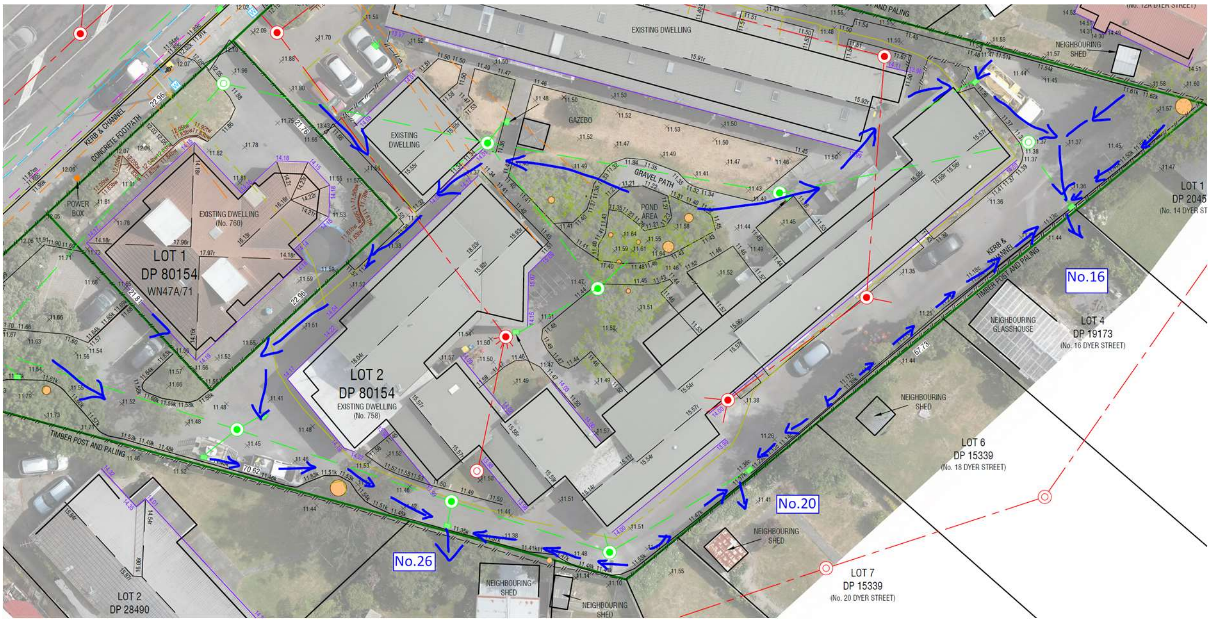

Figure 4: Existing Overland Flow Paths

Runoff from the existing central courtyard and pond area would first temporarily flood to an RL of 11.500m (WLG 1953 datum).

The new design will generally imitate the above temporary ponding, and the existing secondary flow paths. These aspects are to be finalized during the detailed design phase. It is therefore considered that the current overland flow system will remain generally unaffected during the 1% AEP peak storm due to the re-development of the village.

- Required Floor Level

The proposed maximum temporary water level within the central carpark/landscape area is 11.370m (WLG 1953 datum). This is the designed 'spill-over' level to the proposed southern secondary flow path. A minimum floor level of RL 11.700m has also been provided by the architect. This allows for 330mm of freeboard above the maximum flood level. Although the proposed temporary ponding is >100mm in depth, it will not extend from the carpark/accessway directly to any building. Therefore, the proposed freeboard is in excess of the required 150mm (NZBC E1 Clause 4.3.1).

- Stormwater Quality

It is not known whether Hutt City require any stormwater quality measures to be incorporated into the redevelopment of such sites as this Village.

However, the Regional Standard states that *"Where practicable, and unless directed otherwise by the council, sustainable stormwater techniques as outlined in section 4.2.12 should be employed to minimise the potential adverse effects of development."*. The Standard further states that *"Consideration shall be given to pre-treatment of stormwater discharges to aquatic receiving environments, including harbours and inlets, to minimise potential adverse effects."*.

While there is no direct discharge to any immediate water body that could be considered to be of ecological value, an opportunity exists within the proposed carpark and landscape area to capture and treat runoff via rain gardens. Calculations in Appendix C confirm that for 100m^2 of impervious area, a Water Quality Volume (WQV) of approximately 3m^3 is required to be collected and treated by a minimum rain garden area of approximately 6m^2. This is based on capturing a rainfall depth of up to 30mm. Final rain garden locations and contributing impervious areas are to be confirmed based on the above requirements.

❖ WASTEWATER

➢ Proposed Design Standards

The following design standards have been adopted unless otherwise referenced.

- NZBC Clause G13 Foul Water
- Section 5 of the Regional Standard for Water Services (version 2.0: May 2019)

➢ Proposed System

The re-development of the village will result in an increase in sewer flow. This is due to the increase in resident beds, and the addition of a Café. It is expected that the Average Dry Weather Flow (ADWF) will increase from 0.169 L/sec to 0.407 L/sec (refer to attached full calculations in Appendix C). As described above, Wellington Water have indicated that the sewer network in High Street is over capacity, and that the Dyer Street network contributes to downstream overflows at the pump station. As such, sewer mitigation will be required for the development.

Table 2: Increase in Sewer Flow

HEADING	PREDEVELOPMENT	POST DEVELOPMENT
ADWF	0.169 L/sec	0.407 L/sec
PDWF	1.47 L/sec	3.5 L/sec
PWWF	1.55 L/sec	3.58 L/sec

Due to the above, it is proposed to collect and store 100% of the sewer flow generated by the village within a pump station chamber (plus emergency storage), and pump this into the High Street network during off peak periods. The allowable peak pump rate should be no greater than the predevelopment rate shown above. Pump selection and storage volumes are to be finalised during the detailed design phase.

Internal pipe reticulation (gravity) has been designed to ensure minimum self-cleansing velocities are achieved. Pipe mains are to be 150mm dia. with gradients ranging 1.5 - 1.6%.

❖ WATER

➢ Proposed Design Standards

The following design standards have been adopted unless otherwise referenced.

- NZBC Clause G12 Water Supplies
- Section 6 of the Regional Standard for Water Services (version 2.0: May 2019)

➢ Potable Water System

The expected peak potable water demand for the proposed development is Q_{peak}=2.92 L/sec. This has been calculated using the Wellington Water Standards (refer to attached calculations in Appendix C). Modelling by Wellington Water indicates that there is sufficient flow and pressure (40 – 50m) to supply the development.

Calculations confirm that for the entire ring main, a DN 63 mm 12.5 PE100 water pipe will suit the requirements of the site, allowing for an expected total head loss of 6.16m.

A dedicated DN 63mm potable water line to the Village is proposed, with a flow meter and non-return RPZ.

➢ Fire Flow

The required fire flow can be calculated using the NZ Fire Standard SNZPAS 4509-2008. A fire engineering assessment will be required to confirm the Hazard Category and the largest fire cell. For this assessment, it is assumed that the development has a hazard class of FHC 1 and a fire water classification FW4. This requires 50 L/s within 135m and an additional 50 L/s within 270 m. This can be supplied from a maximum of 4 Hydrants. A further demand may be necessary to be taken into account for sprinkler flow (generally 400-500 L/min).

The hydrant flow testing undertaken by AON in September 2021 on a hydrant outside 2 Lincoln Ave/776 High Street confirmed that the main can provide a flow of 44 L/sec at a pressure of 540 kPa. This hydrant is some 88 m from the site. There are three further hydrants within 70 m of the site.

It is therefore likely that the above fire requirements can be met by the network, however this will need to be confirmed. Likewise, the size of the proposed fire supply line to the apartment complex will need to be determined based on the final fire demand and any fire sprinkler requirements of the village. Assuming a sprinkler demand of 500 L/min (8.33 L/sec) and an internal hydrant demand of 12.5 L/sec, then a dedicated 90mm ID fire main will be sufficient to provide fire flows to the development (at a head loss of 10.2m accounting for static losses).

SUMMARY:

The proposed re-development of the retirement village at No. 758 - 760 High Street, Boulcott, Lower Hutt, can be serviced with wastewater, stormwater, and water connections as described above, and as shown on the attached Awa Drawings. The design is subject to confirmation of details, in particular the following:

1) Existing site storage volume (temporary ponding or underground tanks).
2) Final rain garden locations and contributing impervious areas are to be confirmed.
3) Allowable peak sewer discharge into the High Street local network (to be confirmed by Wellington Water).
4) Sewer pump selection and storage volume.
5) Final total fire flow demand by a qualified Fire Engineer.

LIMITATIONS:

This memorandum has been prepared to support the Resource Consent application only, for the development at 758 – 760 High Street in Boulcott, Lower Hutt. Use of this document and any document or drawing referred to herein, may not be used for any other purpose without written permission from Awa.

ATTACHMENTS:

- Appendix A – Awa Drawings
- Appendix B – Wellington Water Initial Advice
- Appendix C – Awa Design Calculations

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APPENDIX A: AWA DRAWINGS

NOTES:

1. MINIMUM STORMWATER PIPE GRADIENTS ARE 0.5%.
2. SEWER PIPES GRADIENTS RANGE FROM 1.5-1.6%.
3. PIPE SIZES ARE DESIGNED FOR THE 10% AEP PEAK STORM
4. SEWER DESIGN ASSUMES MITIGATION IS REQUIRED BY WELLINGTON WATER.

LEGEND

EX	EXISTING LEVELS
PR	PROPOSED LEVELS
	PROPOSED CROSS FALL
	SEWER MITIGATION TANK
	NEW SUMP
SS	NEW SEWER MH
SW	NEW STORMWATER MH
	NEW VALVE
P	POWER
T	TELECOMMUNICATIONS
SS	EXIST SEWER
SS	NEW SEWER
SW	EXIST STORMWATER
SW	NEW STORMWATER
W	EXIST WATER
W	NEW WATER
W (F)	FIRE MAIN
	OVERLAND FLOW PATH
	CULVERT/DRAIN
	MINOR CONT 0.1 m
	MAJOR CONT 0.5 m

Rev	Detail	App.	Date
B	Including Contours	PR	FEB 2022
A	For Review		FEB 2022

Revisions

Status

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Client

WINDSOR MANAGEMENT

Project

ROPATA VILLAGE UTILITIES ASSESSMENT 758 - 768 HIGH ST LOWER HUTT

Drawing Title

PROPOSED SPOT HEIGHTS

Scale 0 mm 10 mm 20 mm 30 mm 1: 400

Project Number	Drawing Number
J000502	2901

Designed	Checked	Approved	Revision
WDU		PR	B
Signed	Signed	Signed	

NOTES:
1. MINIMUM STORMWATER PIPE GRADIENTS ARE 0.5%.
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LEGEND

EX	EXISTING LEVELS
PR	PROPOSED LEVELS
	PROPOSED CROSS FALL
	SEWER MITIGATION TANK
	NEW SUMP
SS	NEW SEWER MH
SW	NEW STORMWATER MH
	NEW VALVE
P	POWER
T	TELECOMMUNICATIONS
SS	EXIST SEWER
SS	NEW SEWER
SW	EXIST STORMWATER
SW	NEW STORMWATER
W	EXIST WATER
W	NEW WATER
W (F)	FIRE MAIN
	OVERLAND FLOW PATH
	CULVERT/DRAIN
	MINOR CONT 0.1 m
	MAJOR CONT 0.5 m

Rev	Detail	App.	Date
B	Added SW Outlet Pipe Size	PR	FEB 2022
A	For Review	PR	FEB 2022

Revisions

Status

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Project
ROPATA VILLAGE UTILITIES ASSESSMENT 758 - 768 HIGH ST LOWER HUTT

Drawing Title
PROPOSED 3 WATERS UTILITIES

Scale
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1: 400

Project Number	Drawing Number
J000502	6000

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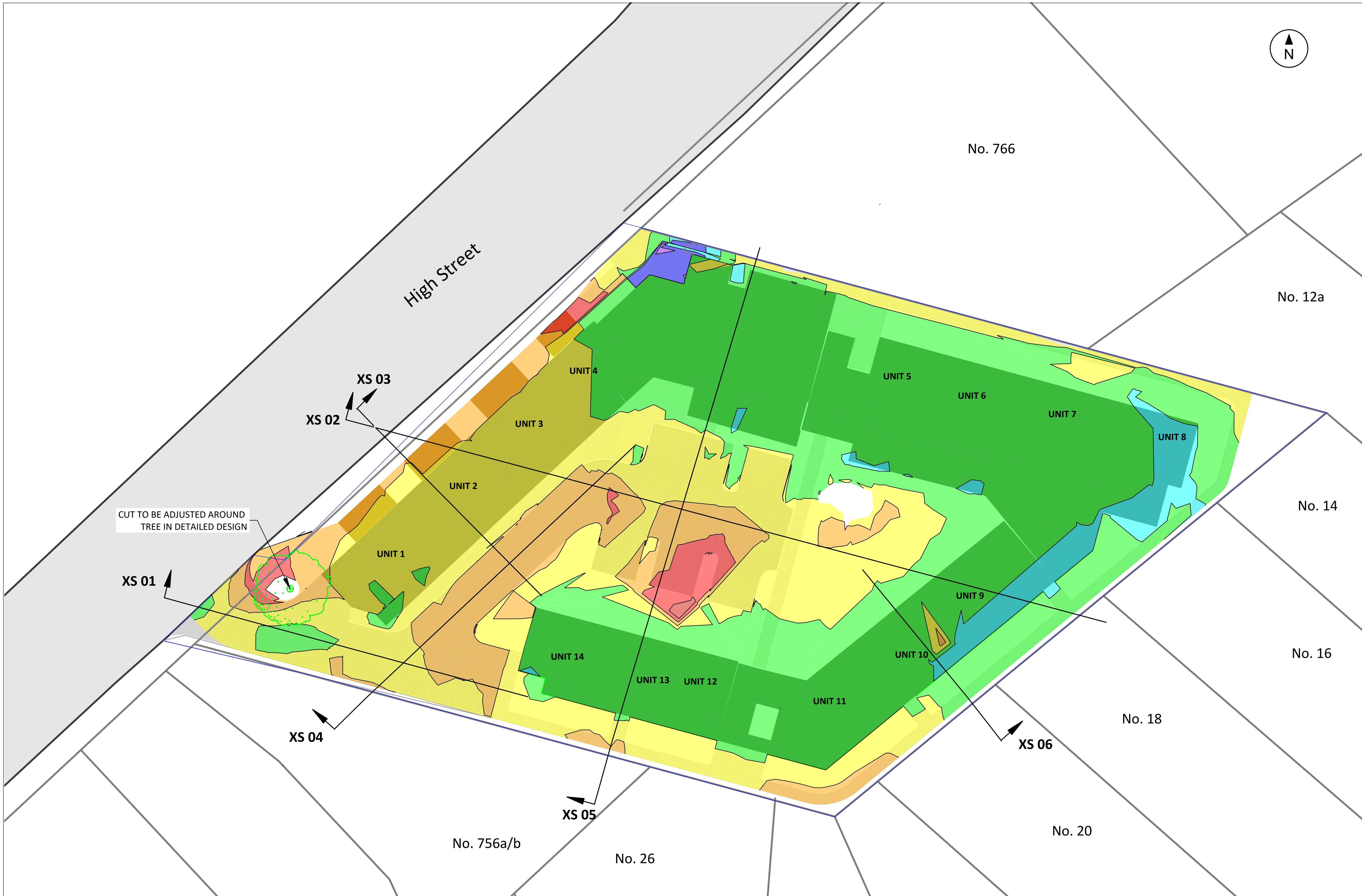

EARTHWORK VOLUMES

CUT (STRIP)	1,015 m³
CUT TO FILL	325 m³
TOTAL CUT	1,340 m³
FILL (BALANCE)	1,160 m³
AREA (CUT)	4,060 m²

VOLUME NOTES:

1) STRIP VOLUME ALLOWS FOR 250 mm BELOW EXISTING GROUND LEVEL, AND INCLUDES PAVEMENT, EXISTING SLABS, EXISTING GROUND COVER VEGETATION/TOPSOIL ETC.
2) THE FILL BALANCE WILL IN PART BE MADE UP FROM NEW ROAD PAVEMENT, TOPSOIL, LANDSCAPE MATERIAL AND NEW FLOOR CONSTRUCTION. SO IN REALITY THE IMPORTED FILL MATERIAL MAY BE MUCH LESS.

NOTES:

1. EARTHWORK VOLUMES COMPARE DESIGN SURFACE (MINUS 300 MM) TO EXISTING SURFACE (MINUS 250 MM).
2. DESIGN SURFACE INCLUDES BUILDING FFL
3. THE 3D DESIGN SURFACE IS PRELIMINARY ONLY AND SUBJECT TO DETAILED DESIGN. VOLUMES ARE THEREFORE SUBJECT TO CHANGE.

LEGEND

- FILL 0.75 - 1.0 m
- FILL 0.5 - 0.75 m
- FILL 0.25 - 0.5 m
- FILL 0.0 - 0.25 m
- CUT 0.25 - 0.0 m
- CUT 0.5 - 0.25 m
- CUT 0.75 - 0.5 m
- CUT 1.0 - 0.75 m
- NEW SUMP
- SS NEW SEWER MH
- SW NEW STORMWATER MH
- SS EXIST SEWER
- SS NEW SEWER
- SW EXIST STORMWATER
- SW NEW STORMWATER
- W EXIST WATER
- W NEW WATER
- W (F) FIRE MAIN

Rev	Detail	App.	Date
B	MAINTAIN TREE AND POND	PR	MAR 2022
A	CUT/FILL VOLUMES		FEB 2022

Revisions

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Client: WINDSOR MANAGEMENT

Project: ROPATA VILLAGE UTILITIES ASSESSMENT 758 - 768 HIGH ST LOWER HUTT

Drawing Title: EARTHWORKS PLAN

Scale: 0 mm 10 mm 20 mm 30 mm 1: 400

Project Number	Drawing Number
J000502	2200

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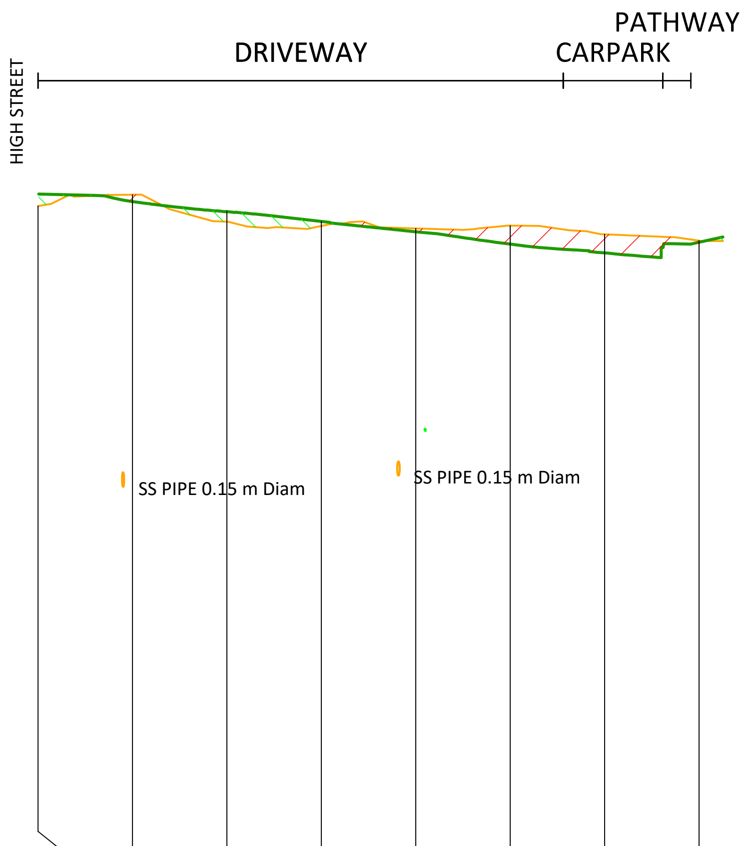

DATUM 5

FINISHED SURFACE		11.82	11.72	11.61	11.50	11.37	11.28	11.39
EXISTING GROUND	11.78	11.90	11.61	11.57	11.54	11.57	11.48	11.41
CUT/FILL		0.12	-0.06	0.00	0.09	0.24	0.25	0.07
OFFSET	0.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00

CROSS SECTION 1

Rev	Detail	App.	Date
A	CUT/FILL VOLUMES		FEB 2022

Revisions

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Project
ROPATA VILLAGE UTILITIES
758 - 768 HIGH ST
LOWER HUTT

Drawing Title
EARTHWORKS CROSS SECTIONS

Scale
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Project Number	Drawing Number
J000502	2210

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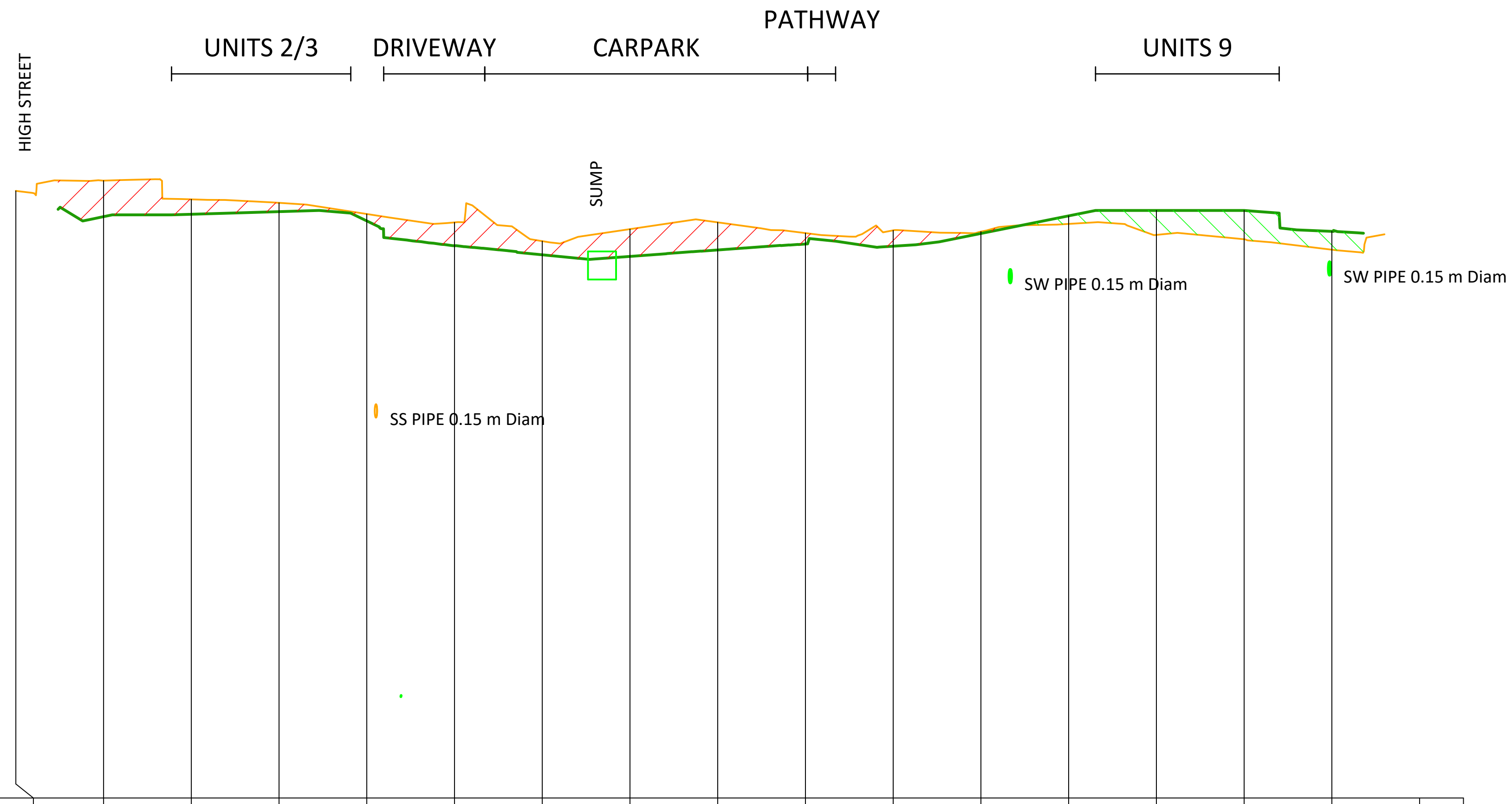

DATUM 5

FINISHED SURFACE		11.63	11.66	11.69	11.58	11.30	11.19	11.18	11.25	11.32	11.29	11.44	11.64	11.70	11.70	11.46	
EXISTING GROUND	11.92	12.04	11.83	11.79	11.66	11.56	11.35	11.49	11.56	11.44	11.47	11.46	11.55	11.42	11.37	11.25	
CUT/FILL		0.46	0.22	0.15	0.13	0.32	0.20	0.36	0.36	0.17	0.23	0.07	-0.04	-0.23	-0.28	-0.16	
OFFSET	0.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00	45.00	50.00	55.00	60.00	65.00	70.00	75.00	80.00

CROSS SECTION 2

Rev	Detail	App.	Date
A	CUT/FILL VOLUMES		FEB 2022

Revisions

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Project
ROPATA VILLAGE UTILITIES
758 - 768 HIGH ST
LOWER HUTT

Drawing Title
EARTHWORKS CROSS SECTIONS

Scale
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Drawing Number: 2211

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Approved: PR
Revision: A
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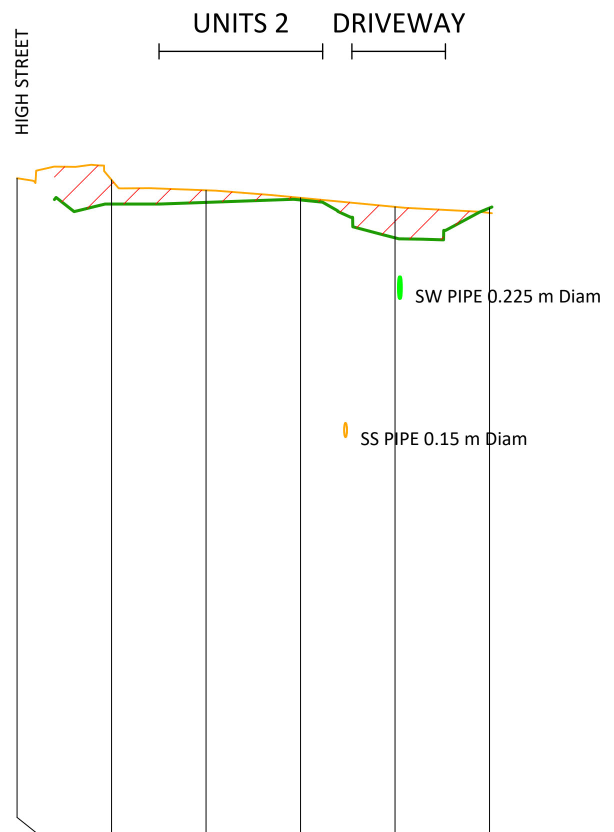

DATUM 5

FINISHED SURFACE		11.65	11.67	11.69	11.29	11.61
EXISTING GROUND	11.92	11.91	11.80	11.72	11.62	11.56
CUT/FILL		0.31	0.18	0.07	0.38	-0.00
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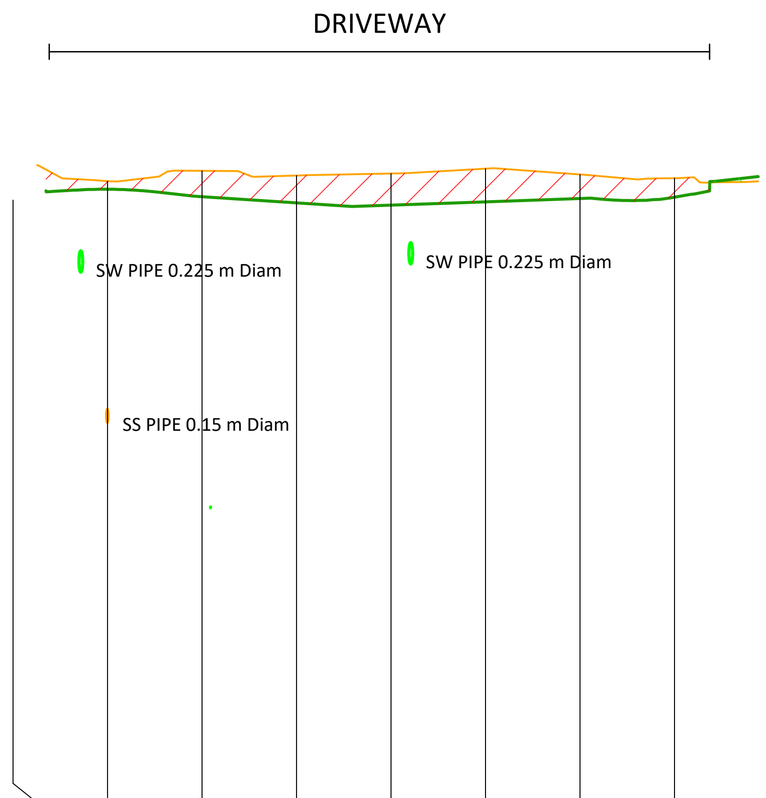
CROSS SECTION 3



DATUM 5

FINISHED SURFACE		11.44	11.36	11.30	11.27	11.31	11.34	11.36
EXISTING GROUND		11.53	11.64	11.59	11.61	11.66	11.60	11.56
CUT/FILL		0.14	0.32	0.34	0.38	0.40	0.30	0.25
OFFSET	0.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00

CROSS SECTION 4

Rev	Detail	App.	Date
A	CUT/FILL VOLUMES		FEB 2022

Revisions

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Project
ROPATA VILLAGE UTILITIES
758 - 768 HIGH ST
LOWER HUTT

Drawing Title
EARTHWORKS CROSS SECTIONS

Scale
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Project Number	Drawing Number
J000502	2212

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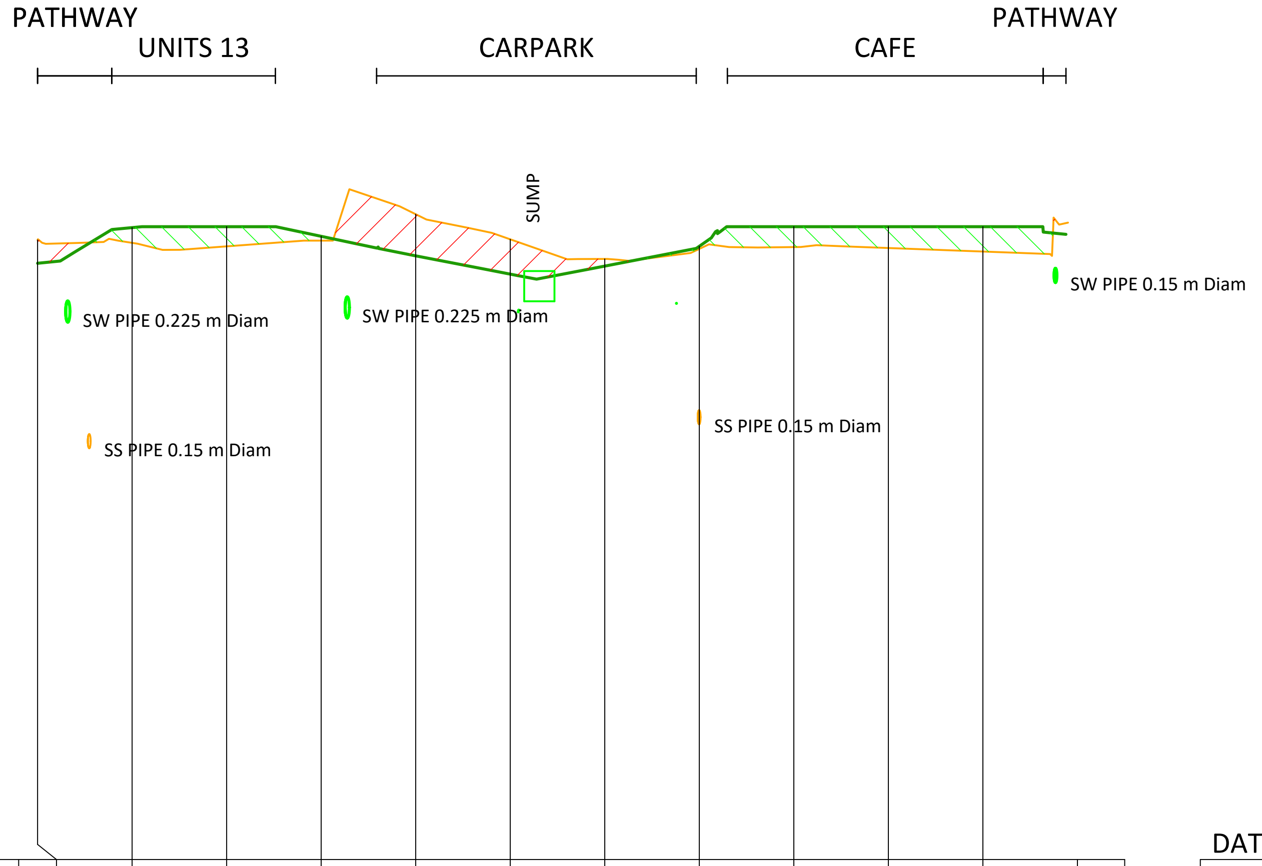
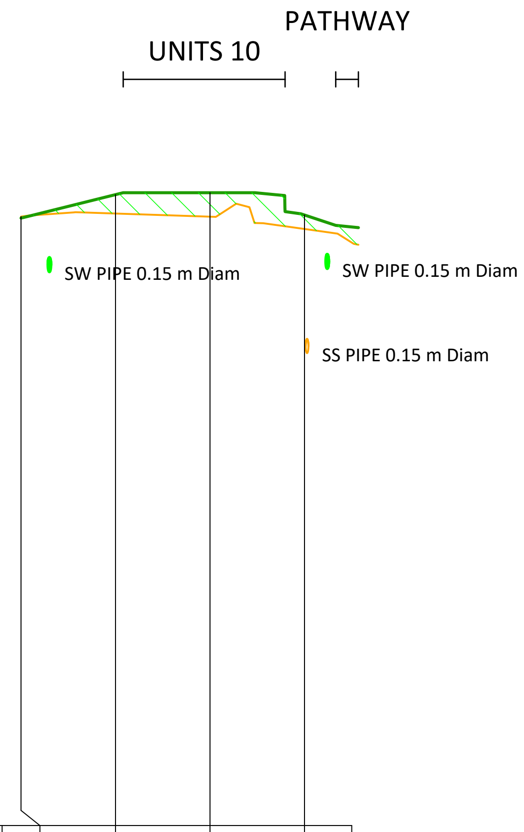


DATUM 5

FINISHED SURFACE	11.31	11.69	11.70	11.60	11.39	11.20	11.28	11.49	11.70	11.70	11.70	
EXISTING GROUND	11.56	11.53	11.49	11.55	11.83	11.57	11.36	11.46	11.48	11.47	11.44	
CUT/FILL	0.30	-0.11	-0.16	0.00	0.49	0.42	0.13	0.02	-0.17	-0.18	-0.21	
OFFSET	0.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00	45.00	50.00	55.00

CROSS SECTION 5



DATUM 5

FINISHED SURFACE	11.43	11.68	11.70	11.46
EXISTING GROUND	11.45	11.48	11.45	11.32
CUT/FILL	0.07	-0.15	-0.20	-0.10
OFFSET	0.00	5.00	10.00	15.00

CROSS SECTION 6

Rev	Detail	App.	Date
A	CUT/FILL VOLUMES		FEB 2022

Revisions

Statis

1 Ghuznee St
Wellington
6011

4 Williamson Ave
Grey Lynn Auckland
1060

e-mail Info@awa.kiwi
web www.awa.kiwi

Client: WINDSOR MANAGEMENT

Project: ROPATA VILLAGE UTILITIES 758 - 768 HIGH ST LOWER HUTT

Drawing Title: EARTHWORKS CROSS SECTIONS

Scale: 0 mm 10 mm 20 mm 30 mm 1: $scale

Project Number: J000502

Drawing Number: 2213

Designed: WDU | Checked: | Approved: PR | Revision: A

Signed | Signed | Signed

APPENDIX B: WELLINGTON WATER INITIAL ADVICE

Warren Uys

From: Niamey Izzett <Niamey.Izzett@huttcity.govt.nz>
Sent: Monday, 2 August 2021 7:58 AM
To: Kerry Wynne
Subject: FW: [EXTERNAL] WW Pre-app Response - 758 - 760 High Street, Boulcott - PREAPP210175 - 30 July 2021

Good morning Kerry

Hope you are doing well and had a good weekend!!

Wellington Water comments for 758-760 High Street are as below.

Thanks

Niamey Izzett
Resource Consents Planner

Hutt City Council, 30 Laings Road, 5040, Lower Hutt 5040, New Zealand
T 04 570 6666, W www.huttcity.govt.nz

HUTT CITY TE AWA KAIRANGI

From: Marlene Roberts-Saidy [mailto:Marlene.Saidy@wellingtonwater.co.nz] **On Behalf Of** Land Development
Sent: Saturday, 31 July 2021 9:14 AM
To: Niamey Izzett
Cc: Peter McDonald; Sarah Zhou
Subject: [EXTERNAL] WW Pre-app Response - 758 - 760 High Street, Boulcott - PREAPP210175 - 30 July 2021

Good day Niamey, please see comments below .

My time on this was 2hrs 10mins.

With

Water Supply

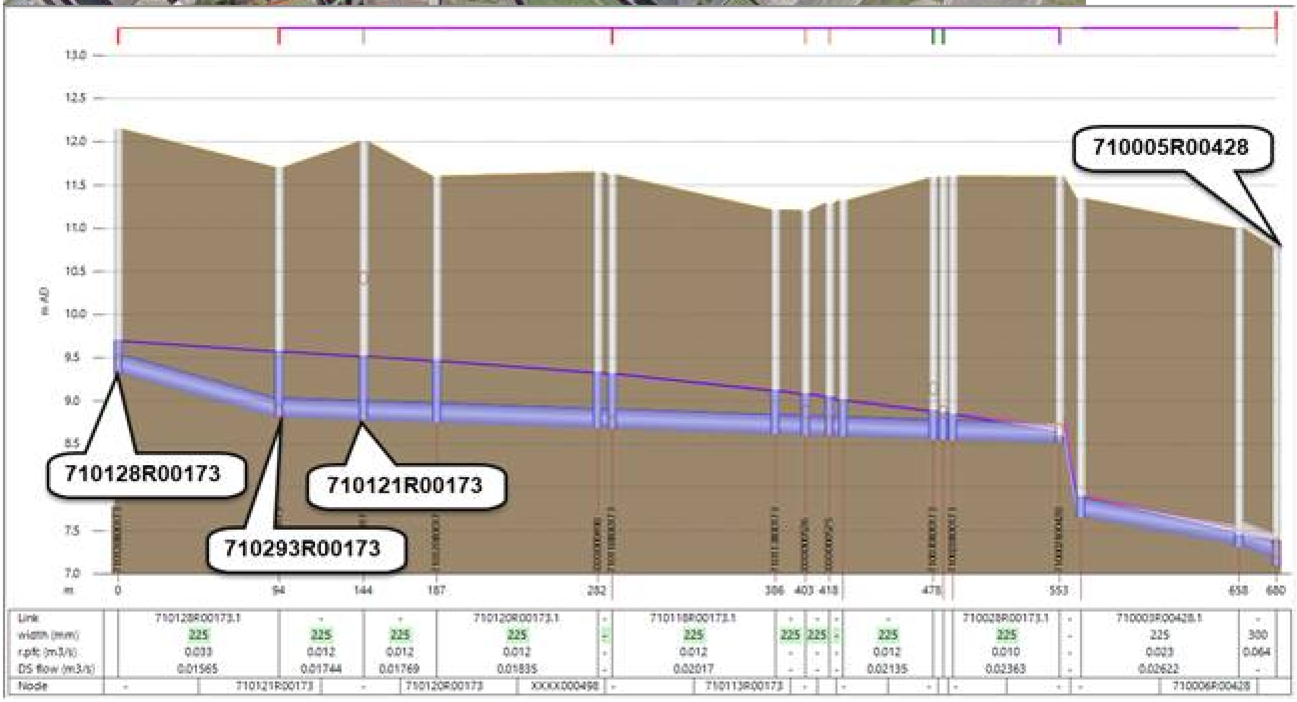
- The model shows that minimum pressure at the point of supply on the public main is expected to be about 45-50m, which meets the level of service criteria for pressure. The model also indicates that available fire flow capacity from the existing hydrant(s) is expected to be compliant with the NZ Fire code for residential areas (FW2). However, considering the building and their usage, it is recommended that a hydrant flow test is carried out and the compliance with the NZ Fire code requirements are approved by a fire engineer.

- This modelling assessment only represents the existing network based on WWL hydraulic model developed in 2019. The analysis takes no account of developments that have occurred since then, currently underway, or future developments. Non-hydraulic parameters like pipe age, conditions and likelihood of their failure have not been assessed. Please also note the above are just the result of WWL hydraulic model which could be impacted by day-to-day operational changes within the network and may need to be verified in the field through pressure logging and hydrant flow tests.

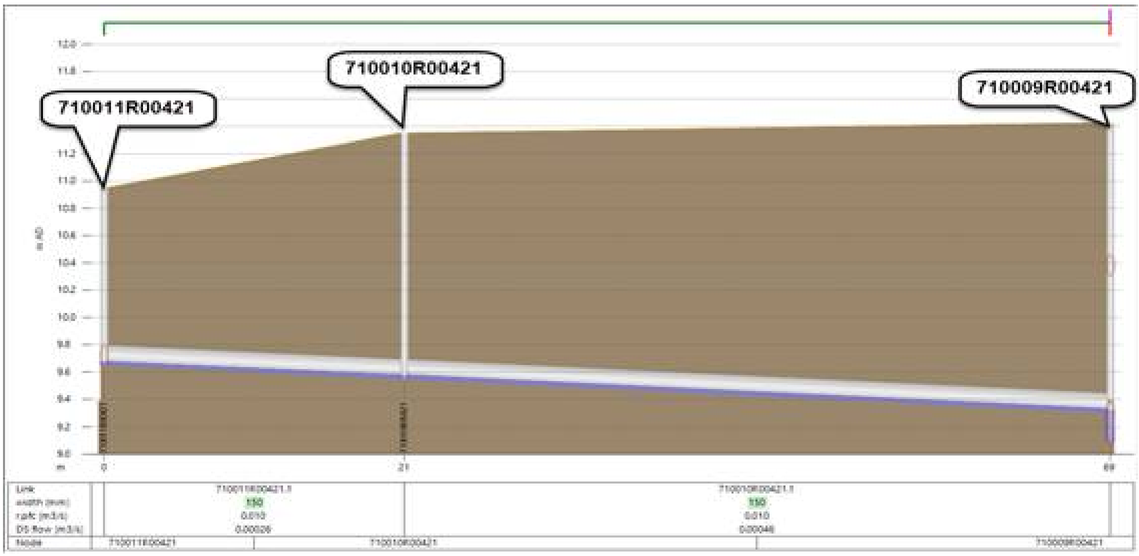
- The applicant will need to assess the water supply lateral and if not constructed of MDPE will need to change these to MDPE.

Wastewater

710128R00173
710121R00173
710293R00173
710009R00421
710010R00421
710011R00421



710005R00428
710128R00173
710121R00173
710293R00173



710010R00421
710011R00421
710009R00421

- The property could be connected to the local network at High St, between manhole 710128R00173 and 710293R00173, and alternatively towards Dyer St, between manholes 710011R00421 and 710009R00421.

LOCAL and TRUNK NETWORK (pipes 300mm dia and above) – High St.

- The property is proposed to be connected to the local network between manholes 710128R00173 and 710121R00173. This system discharges into the trunk network at manhole 710005R00428. There appears to be no spare capacity in the local network during a 1-year LTS design event. Furthermore, this part of the network is heavily surcharged.

- The trunk network downstream of manhole 710005R00428 discharges into the Barber Grove pump station. A significant portion of the trunk network appears to already over capacity (i.e. no spare capacity) in a 1-year flow. There are two engineered overflows at the pump station at the incoming main and the rising main. They both discharge to the Hutt River. At present it is not clear from the model results as to whether they operate and the frequency. Further development of this property will exacerbate this.

LOCAL and TRUNK NETWORK (pipes 300mm dia and above) – Dyer St.

- The property is alternatively proposed to be connected to the local network between manholes 710011R00421 and 710009R00421. This system discharges into the trunk network at manhole 710009R00421. There appears to be at least 5 litres/sec of spare capacity in the local network during a 1-year LTS design event

- The trunk network downstream of manhole 710009R00421 discharges into the Barber Grove pump station. A significant portion of the trunk network appears to already over capacity (i.e. no spare capacity) in a 1-year flow. There are two engineered overflows at the pump station at the incoming main and the rising main. They both discharge to the Hutt River. At present it is not clear from the model results as to whether they operate and the frequency. Further development of this property will exacerbate this.

Summary

- While only the local network towards Dyer St. appears to have at least 5 litres/sec spare capacity during a 1-year LTS design flow, there appears to be no spare capacity (with overflows occurring into the Hutt River) in the trunk network for further development of this property. The first proposal of connecting to the local network at High St, shows that the local network has no spare capacity as well during the 1-year LTS design event.

As the site is a redevelopment in order to understand the impact on the network we would need to understand the pre and post development flows.

This assessment is based on the results from WWL hydraulic models as defined in this memorandum. It does not take into account the impact on the spare design capacity of other developments that have occurred since then, are currently underway, or possible future developments Non-hydraulic parameters like pipe age, conditions and likelihood of their failure have not been assessed. Flow monitoring may be required to verify these results. This development may impact on the spare design capacity available for possible future developments along the downstream network.

- From the info, the site is for re-development – it is assumed that the structure will stay as is with only possible reconfiguration. If this is the case the existing laterals could be reused however the applicant will need to reassess these and ensure they are in good condition and of capacity to convey the flow.

Stormwater

DISCLAIMER
Hazard Classification and Flood Depth data is derived from Wellington Water models. Mapped flooding information may not be survey-accurate, and is bound by the model assumptions and limitations. Care should be taken that information is verified as part of any flood risk analysis, concept or detail design

FLOODING RESULTS	
Software	InfoWorks ICM
Model	Waiwhetu
Model Status	Validated
Flood Scenario	100 year ARI + Climate Change (assuming 2.1 C temperature increase)
Sea Water Level	2.1 m aMSL

FLOOD IMPACT ON THE PROPERTY

Maximum Flood Depth	<50mm
Maximum Water Level	N/A
Minimum Water Level	N/A
Overland Flow	N/A

RECOMMENDATIONS	
Minimum Floor Level (including 200 mm Freeboard)	Minimum floor level should be set based on the building code.
Overland Flow	N/A

Thanks

Marlene

From: Niamey Izzett <Niamey.Izzett@huttcity.govt.nz>
Sent: Tuesday, 20 July 2021 8:28 am
To: Land Development <Land.Development@wellingtonwater.co.nz>
Subject: 758 - 760 High Street, Boulcott - PREAPP210175

Hi there,

An applicant is proposing a redevelopment of the existing retirement village at 758-760 High Street. The proposed yield is 49 apartments.

Please provide any comment you may have on three water infrastructure and flooding issues.

Kind regards,

Niamey Izzett

Resource Consents Planner

Hutt City Council, 30 Laings Road, 5040, Lower Hutt 5040, New Zealand

T 04 570 6666, **W** www.huttcity.govt.nz

APPENDIX C: AWA DESIGN CALCULATIONS

THREE WATERS CALCULATIONS

❖ SEWER

The proposed development will result in additional sewer loading. Table 1 compares the existing sewer load of the site to the loading as per the proposed development.

The Wellington Water Standards indicate that the sewer flows are calculated based on an occupancy per dwelling basis. Following the guidance for Lower Hutt, a occupancy of 3.5 is assumed per dwelling.

The site currently has 20 units and a 4-bedroom house.

Q_{ADWF}= 21 x 3.5 x 0.0023= 0.169 L/sec

PF = 7.23 x $Area^{-0.2}$ = 8.67

PDWF = ADWF x PF = 1.47 L/sec

Proposed Development has 50 lots and a café area of 93.32 m² (0.0093 ha). (Area = Café + Kitchen + Toilets)

From Wellington Water Standards Section 5.3.1:

(Dwellings) (Café)

Q_{ADWF}= (50 x 3.5 x 0.0023) + (0.52 x 0.0093)= 0.407 L/s

PF = 7.23 x $Area^{-0.2}$= (Assuming Area is for the area of the property Area = 0.403 ha)

PF= 7.23 x $0.403^{-0.2}$= 8.67

PDWF= PF x $Q_{ADWF\ (Res)}$+ 1.56 x $A_{café}$=

PDWF= 8.67 x 50 x 3.5 x 0.0023 + 1.56 x 0.0093 = 3.5 L/sec

Assuming a sewer pipe network length of 150 m within the development for calculation of infiltration flow. Q_{Inf} =0.15 x 0.55= 0.0825 L/sec. This applies to both cases.

Table 1: Sewer Flow for development using Wellington Water Standard (Dec 2021)

HEADING	PREDEVELOPMENT	POST DEVELOPMENT
ADWF	0.169 L/sec	0.407 L/sec
PF	8.67	8.67
PDWF	1.47 L/sec	3.5 L/sec
Pipe Length	(150m Estimate)	(150 m Estimate)
PWWF	1.55 L/sec	3.58 L/sec

➢ Café Sewer Loading

Using the TP 58 Guidance on sewer design, the sewer loading from the café is defined (wet Retail) as 15 L/m². As the area of the café and dining area is 82 m² the Flow from the Café is Q= 1220.4 L/day. This corresponds to a $ADWF_{(café)}$ =0.014 L/sec with a $PDWF_{(café)}$ = 0.028 L/sec

❖ STORMWATER

The development will need to be hydraulically neutral. Comparing the areas of Hardstand and roofing in both the predevelopment and post development scenarios is necessary. See Table 2 for a comparison.

The development is expected to have a time of concentration of less then 20 min. This has been estimated using Figures 1, 2 and 3 of the New Zealand Building Code E1.

Table 2: Area of Impermeable surface post development.

HEADING	C VALUE	PREDEVELOPMENT AREA	POST DEVELOPMENT AREA
Roof Area	0.9	1303 m²	1884 m²
Concrete area	0.85	1602 m²	1177 m²
Garden	0.25	1425 m²	969 m²
Additional Runoff 1% AEP 20 min		-	1.462 m³
Additional Runoff 10% AEP 20 min		-	0.936 m³

There is minimal increase in impermeable area with the development as currently proposed. As such the required detention for the 1% and 10% AEP storms is also minimal. However, this assumes a 20 min storm duration. For the size of the site, it is recommended that a longer duration storm is considered.

Note that the additional detention volume required for the development is in addition to any existing detention and stormwater management devices.

❖ WATER

➢ Flow Requirements

The expected residential population of the development is calculated using Clause 5.3.1.2 from the Sewer section of the Wellington Water Standards (Dec 2021). The development is expected to have 50 apartments, plus the water demand from the café (fire demand is discussed in the next section).

Using the guidance in the Wellington water documentation, the residential section of the village is expected to have a population of 3.5 people per dwelling (this is likely more than what would actually be present due to the nature of the occupancy. But there is no reason not to design for this village becoming a regular apartment complex in the future)

$Q_{peak\ res}$= 3.5 x 50 x 0.0162 = 2.835 L/s.

The calculation for the water demand for the café is not clearly defined in the Wellington Water Standards. As such we have used the Auckland Standard to size the flow volumes. This can be located in cop_water_chapter.pdf (windows.net) Clause 6.3.5.6. This states a daily water use of 15 L/m². With a café and kitchen area of 82 m² This corresponds to a flow of 1230 L/day, or 0.014 L/sec.

$Q_{ave\ cafe}$= 82 x 15 /(24 x 60 x 60) = 0.0142 L/sec.

For the purpose of assessing a peak flow demand for the Café, a peaking factor of 6 has been used. This is due to the café likely operating for 8 hours a day, plus an additional factor of 2 for safety (though it is noted that due to the café and residential demand being taken from the same line, and that they both will have peak demand at different times, it is unlikely that the peaking of the café demand is important.)

$Q_{peak\ cafe}$= $Q_{ave\ cafe}$ x 6 = 0.0852 L/sec.

Therefore, a peak water demand for the development is expected to be 2.92 L/sec

Q_{peak} = $Q_{peak\ res}$ + $Q_{peak\ cafe}$ =2.92 L/sec.

Using the simple headless calculation excel sheet shows that for the entire ring main a DN 63 mm 12.5 PE100 water pipe will suit the requirements of the site. This pipe will result in a head loss of 6.16 m. As the pressure head being supplied to the site is between 40 and 50 m, the pressure provided to the apartments and café is expected to be good.

The site itself is at a lower elevation than the hydrant where the pressure test was carried out. The ground level at the boundary of the village is 11.35 m RL, which is 0.008 m lower than the ground level at the furthest part of the internal potable water ring main (11.358 m RL). This is not significant, and can hence be ignored. Hence the Net headloss from friction losses is 6.16 m (this does not account for the specific head losses due to fittings and joins, but total headloss is not expected to be more than 10 m which will still provide ample pressure to the development)

- Fire Hydrants

According to the fire code SNZ PAS 4509 the development will need to have sufficient fire fighting flow. From Table 1, it is assumed that that the development has a water supply classification FW4. Though this is deponent on the largest fire cell in the development. (Hazard category FHC 1). This requires 50 L/s flow from within 135m, and another 50L within 270 m.

Water pressure testing of the hydrant on 2 Lincoln Ave/776 High Street can provide a flow of 44 L/sec at a pressure of 540 kPa.

Title:	Winsor Management	Job No.	J000502
	Ropata Village	Page No.	1
Description:	Water Supply - Internal	Date	14/02/2022
	Ring Main	Author:	WDU
		Reviewer:	PR
		Revision:	

Using the Darcy-Weisbach calculation, utilising the Swamee-Jain method

$$R = \frac{V * D}{10^{-6}}$$

$$f = \frac{0.25}{\left[log_{10}\left(\frac{\varepsilon / D}{3.7} + \frac{5.74}{R^{0.9}}\right)\right]^2}$$

$$h_f = f \frac{L}{D} \frac{V^2}{2g}$$

			Pipe 1:		
Flow	Q	=	2.92L/s		
Pipe Diameter	D	=	53mm	DN 63 mm	12.5 PE100
Pipe Roughness	Ɛ	=	0.15		
Length of pipe	L	=	135.0m		
Velocity	V	=	1.31m/s		
Reynolds No.	R	=	69753		
Friction Factor	f	=	0.028		
Headloss	h_f	=	**6.16m**		

Change in	US	=	11.35	m
Ground level	DS	=	11.358	m
		=	-0.008	m

Table 7.3 – Colebrook-White Pipe Roughness (mm)

Material	Age of pipe (see notes below) < 10 years⁺	10-25 years⁺	> 25 years*
Asbestos Cement (AC)	0.03	0.06	0.5
PVC / Polyethylene	0.06	0.06	0.15
Clay/earthenware	0.06	0.15	0.15
Cast iron	0.3	0.6	3
Concrete lined ductile iron (DICL)	0.06	0.15	0.15
Concrete lined steel (STCL)	0.06	0.15	0.15
Copper (Cu)	0.03	0.06	0.5
Ductile iron (unlined)	0.045	0.06	3
Galvanised iron (GI)	0.3	0.6	3.0
Steel (unlined)	0.03	0.06	3
Reinforced concrete (RC)	0.15	0.6	3
Unknown	0.03	0.06	0.5

⁺These factors should only be used for the simulation and calibration of existing networks and pipelines. They shall NOT be used for the design of new pipelines.

*For the design of new/replacement pipelines, roughness factors for pipes >25 years shall be used to ensure network performance can be maintained throughout the lifespan of the pipeline/network.

Title:	Winsor Management	Job No.	J000502
	Ropata Village	Page No.	1
Description:	Water Supply - Internal	Date	14/02/2022
	Fire supply	Author:	WDU
		Reviewer:	PR
		Revision:	

Using the Darcy-Weisbach calculation, utilising the Swamee-Jain method

$$R = \frac{V * D}{10^{-6}} \qquad f = \frac{0.25}{\left[log_{10}\left(\frac{\varepsilon / D}{3.7} + \frac{5.74}{R^{0.9}}\right)\right]^2} \qquad h_f = f\,\frac{L}{D}\,\frac{V^2}{2g}$$

			Pipe 1:		
Flow	Q	=	20.83L/s		
Pipe Diameter	D	=	90mm	DN 63 mm	12.5 PE100
Pipe Roughness	ε	=	0.15		
Length of pipe	L	=	70.0m		
Velocity	V	=	3.27m/s		
Reynolds No.	R	=	294684		
Friction Factor	f	=	0.023		
Headloss	h_f	=	**9.85m**		

Sprinkler Flow 8.33 L/sec (assumed)

Fire Flow = 12.5 L/sec (internal Hydrant)

Elevation Change	US	=	11.35	m
	DS	=	11.7	m
			-0.35	m
Headloss Total	h_{Tf}	**=**	**10.20**	**m**

Table 7.3 – Colebrook-White Pipe Roughness (mm)

	Age of pipe (see notes below)		
Material	< 10 years+	10-25 years+	> 25 years*
Asbestos Cement (AC)	0.03	0.06	0.5
PVC / Polyethylene	0.06	0.06	0.15
Clay/earthenware	0.06	0.15	0.15
Cast iron	0.3	0.6	3
Concrete lined ductile iron (DICL)	0.06	0.15	0.15
Concrete lined steel (STCL)	0.06	0.15	0.15
Copper (Cu)	0.03	0.06	0.5
Ductile iron (unlined)	0.045	0.06	3
Galvanised iron (GI)	0.3	0.6	3.0
Steel (unlined)	0.03	0.06	3
Reinforced concrete (RC)	0.15	0.6	3
Unknown	0.03	0.06	0.5

+These factors should only be used for the simulation and calibration of existing networks and pipelines. They shall NOT be used for the design of new pipelines.

*For the design of new/replacement pipelines, roughness factors for pipes >25 years shall be used to ensure network performance can be maintained throughout the lifespan of the pipeline/network.

Title:	Windsor Management	Job No.	J000502
	Ropata Village	Page No.	1
Description:		Date	
	Stormwater Detention	Author:	WDU
		Reviewer:	PR
		Revision:	

Surface Run Off

Pre-development	Catchment Area:	Runoff Coefficient:	Effective Area:	Post-development	Catchment Area:	Runoff Coefficient:	Effective Area:
Roof	1,303m²	0.90	1,173m²	Roof	1,884m²	0.90	1,696m²
Paved Areas	1,602m²	0.85	1,362m²	Paved Areas	1,177m²	0.85	1,000m²
Berm / Garden	1,425m²	0.25	356m²	Berm / Garden	969m²	0.25	242m²
Deck area	0m²	0.3	0m²	Deck	0m²	0.3	0m²
Permeable Pavement	0m²	0.5	0m²	Perm Pavement	0m²	0.5	0m²
		Total:	2,891m²			Total:	2,938m²

Exclusions

Soakage Trench:		Soak Holes:	
Trench Width =	0.00m	Number of Soakholes =	0
Trench Depth =	0.00m	Ring Diameter =	0mm
Trench Length =	0.00m	Depth =	0.00m
Net Void Ratio =	0	Net Void Ratio =	0.00
Trench Volume =	0.0m³	Soakhole Volume	0.0m³
Void Volume =	0.0m³	Void Volume	0.0m³
Infiltration Rate per m² =	0.00m/h	Infiltration Rate per m² =	0.00m/h
Soakage Through Base:	Yes	Soakage Through Base:	Yes
Soakage Through Sides:	Yes	Soakage Through Sides:	Yes
Side Soak Factor =	0.50	Side Soak Factor =	0.00
Total Infiltration Rate =	0.0l/s	Total Infiltration Rate =	0.0l/s

Subtract Pre-Development? Yes

Return Period: Match Post-Develop. ~~20 year~~

Duration: Match Post-Develop. ~~60~~

Additional Volume Exclusion:

Volume = 0.0m³

Description

Additional Flow Exclusion:

Flow = 0.0l/s

Description

Rainfall Intensity Data:

Storm Return Period = 100 Years

Use Critical Duration? No

Constant Duration: 20min

Climate Change Factor: 20%

Zone Factor: 1.0

Source : HIRDS

Units : mm/h

Return Period Years	10min 0.2hr	20min 0.3hr	30min 0.5hr	60min 1.0hr	120min 2.0hr	360min 6.0hr	720min 12.0hr	1440min 24.0hr	2880min 48.0hr
2	56.64	38.04	30.48	21.12	14.76	8.32	5.68	3.79	2.45
5	74.52	49.92	39.96	27.60	19.20	10.75	7.33	4.87	3.13
10	88.20	58.92	47.04	32.40	22.56	12.60	8.54	5.68	3.64
20	102.48	68.40	54.48	37.56	26.04	14.52	9.82	6.49	4.16
50	122.40	81.60	64.92	44.52	30.84	17.04	11.54	7.63	4.87
100	139.20	92.04	73.20	50.16	34.56	19.08	12.96	8.51	5.42

Pre-development Flows:

Units : l/s

	10min	20min	30min	60min	120min	360min	720min	1440min	2880min
2	45.48	30.54	24.47	16.96	11.85	6.68	4.56	3.04	1.97
5	59.84	40.08	32.09	22.16	15.42	8.63	5.89	3.91	2.51
10	70.82	47.31	37.77	26.02	18.11	10.12	6.86	4.56	2.92
20	82.29	54.92	43.75	30.16	20.91	11.66	7.88	5.21	3.34
50	98.28	65.52	52.13	35.75	24.76	13.68	9.27	6.13	3.91
100	111.77	73.90	58.78	40.28	27.75	15.32	10.41	6.83	4.36

Post-development Flows:

Units : l/s

	10min	20min	30min	60min	120min	360min	720min	1440min	2880min
2	46.23	31.05	24.88	17.24	12.05	6.79	4.63	3.10	2.00
5	60.82	40.74	32.62	22.53	15.67	8.78	5.98	3.98	2.56
10	71.99	48.09	38.39	26.44	18.41	10.28	6.97	4.63	2.97
20	83.64	55.83	44.47	30.66	21.25	11.85	8.01	5.30	3.40
50	99.90	66.60	52.99	36.34	25.17	13.91	9.42	6.23	3.98
100	113.61	75.12	59.75	40.94	28.21	15.57	10.58	6.94	4.43

Additional SW Volume Generated by Development:

Units : m³

	10min	20min	30min	60min	120min	360min	720min	1440min	2880min
2	0.450	0.604	0.726	1.006	1.407	2.378	3.246	4.337	5.599
5	0.592	0.793	0.952	1.315	1.830	3.074	4.192	5.572	7.164
10	0.700	0.936	1.121	1.544	2.150	3.602	4.885	6.491	8.316
20	0.814	1.086	1.298	1.790	2.482	4.151	5.613	7.424	9.524
50	0.972	1.296	1.547	2.121	2.939	4.872	6.601	8.728	11.143
100	1.105	1.462	1.744	2.390	3.294	5.455	7.411	9.730	12.406

Positive numbers refer to additional volume generated, negative numbers indicate sufficient mitigation. Target a small negative number.

Output:	Return Period:	100 Years	Duration:	20min	Sufficient?	Warning	Additional Storage Required:	1.462m³

Title	Winsor Management	Job No. J000502
	Ropata Village	Page No. 1
Description:		Date
	Stormwater Pipes	Author:
		Reviewer:
		Revision:

Rainfall Intensity Data: Units : mm/h
Source: Regional Code of Practice 2012

Zone Multiplier Adjustment: 1.0
Climage Change Adjustment: 16%

Return Period	Duration 10min 0.2hr	20min 0.3hr	30min 0.5hr	60min 1.0hr	120min 2.0hr	360min 6.0hr	720min 12.0hr	1440min 24.0hr	2880min 48.0hr
2	54.75	36.77	29.46	20.42	14.27	8.04	5.49	3.67	2.37
5	72.04	48.26	38.63	26.68	18.56	10.39	7.09	4.71	3.03
10	85.26	56.96	45.47	31.32	21.81	12.18	8.26	5.49	3.51
20	99.06	66.12	52.66	36.31	25.17	14.04	9.49	6.28	4.03
50	118.32	78.88	62.76	43.04	29.81	16.47	11.16	7.38	4.71
100	134.56	88.97	70.76	48.49	33.41	18.44	12.53	8.22	5.24

Pipe Flow Method: 1

1 - Mannings with Constant n
2 - Mannings with Variable n
3 - Mannings with Escritt's Hydraulic Radius

Calculations taken from 'Simple Formulae for Velocity, Depth of Flow, and Slope Calculations in Partially Filled Circular Pipes', Omer Akgiray, Published November 3 2004 in Environmental Engineering Science

Velocity Criterea:

0.75m/s	Min. Velocity
3.00m/s	Max. Velocity
2 years	Min. Velocity Return Period:
0.5	Min. Velocity Intensity Multiplication Factor:

Catchment	Upper Node	Lower Node
Sump1-Internal	Sump1	ernal South line
Slope	Diameter	Manning's n
0.50%	150mm	0.013
Return Period	Time of concentration	
10 years	10min	

Catchment Sump1-Internal South line

Description	Area	Runoff Coefficient	Effective Area
Berm	0m²	0.25	0m²
Road	183m²	0.85	156m²
			0m²
			0m²

Upstream Contributions: Catchment	Effective Area

Effective Area
Catchment: 156m²
Upstream: 0m²
Total: 156m²

	Rainfall Intensity	Flow	Velocity	Area of Flow	Water Depth	Air/Water Ratio	Flow (with Air Entrap.)
Min Vel. Flow	27.38mm/h	1.18L/s	0.41m/s	0.00m²	35mm	-	1.18L/s
Design Flow	85.26mm/h	3.68L/s	0.55m/s	0.01m²	60mm	-	3.68L/s
Max. Capac.	-	11.98L/s	0.69m/s	0.02m²	142mm	0.005	11.92L/s

Percent Full:	**31%**
Capacity?	Okay
Velocity?	Warning

Catchment	Upper Node	Lower Node
SWMH 15-SWMH 12	SWMH 15	SWMH 12
Slope	Diameter	Manning's n
0.50%	150mm	0.013
Return Period	Time of concentration	
10 years	10min	

Catchment SWMH 15-SWMH 12

Description	Area	Runoff Coefficient	Effective Area
Garden	73m²	0.25	18m²
Pavement	173m²	0.85	147m²
Roof	274m²	0.95	260m²
deck	89m²	0.25	22m²

Upstream Contributions: Catchment	Effective Area

Effective Area
Catchment: 448m²
Upstream: 0m²
Total: 448m²

	Rainfall Intensity	Flow	Velocity	Area of Flow	Water Depth	Air/Water Ratio	Flow (with Air Entrap.)
Min Vel. Flow	27.38mm/h	3.41L/s	0.54m/s	0.01m²	58mm	-	3.41L/s
Design Flow	85.26mm/h	10.61L/s	0.69m/s	0.02m²	120mm	-	10.61L/s
Max. Capac.	-	11.98L/s	0.69m/s	0.02m²	142mm	0.005	11.92L/s

Percent Full:	**89%**
Capacity?	Okay
Velocity?	Warning

Catchment	Upper Node	Lower Node
SWMH 04-SWMH 02	SWMH 04	SWMH 02
Slope	Diameter	Manning's n
0.50%	225mm	0.013
Return Period	Time of concentration	
10 years	10min	

Catchment SWMH 04-SWMH 02

Description	Area	Runoff Coefficient	Effective Area
Berm	290m^2	0.25	73m^2
Pavement	132m^2	0.85	112m^2
Roof	456m^2	0.95	433m^2
Driveway	232m^2	0.85	197m^2

Upstream Contributions

Catchment	Effective Area

Effective Area

Catchment: 815m^2
Upstream: 0m^2
Total: 815m^2

	Rainfall Intensity	Flow	Velocity	Area of Flow	Water Depth	Air/Water Ratio	Flow (with Air Entrap.)
Min Vel. Flow	27.38mm/h	6.20L/s	0.63m/s	0.01m^2	68mm	-	6.20L/s
Design Flow	85.26mm/h	19.30L/s	0.83m/s	0.02m^2	125mm	-	19.30L/s
Max. Capac.	-	35.31L/s	0.91m/s	0.04m^2	212mm	0.005	35.13L/s

Percent Full:	**55%**
Capacity?	Okay
Velocity?	Warning

Catchment	Upper Node	Lower Node
SWMH 02-SWMH 03	SWMH 02	SWMH 03
Slope	Diameter	Manning's n
0.50%	150mm	0.013
Return Period	Time of concentration	
10 years	10min	

Catchment SWMH 02-SWMH 03

Description	Area	Runoff Coefficient	Effective Area
Berm	176m^2	0.25	44m^2
Pavement	73m^2	0.85	62m^2
Roof	289m^2	0.95	275m^2
Driveway		0.85	0m^2

Upstream Contributions

Catchment	Effective Area

Effective Area

Catchment: 381m^2
Upstream: 0m^2
Total: 381m^2

	Rainfall Intensity	Flow	Velocity	Area of Flow	Water Depth	Air/Water Ratio	Flow (with Air Entrap.)
Min Vel. Flow	27.38mm/h	2.89L/s	0.52m/s	0.01m^2	53mm	-	2.89L/s
Design Flow	85.26mm/h	9.01L/s	0.67m/s	0.01m^2	104mm	-	9.01L/s
Max. Capac.	-	11.98L/s	0.69m/s	0.02m^2	142mm	0.005	11.92L/s

Percent Full:	**76%**
Capacity?	Okay
Velocity?	Warning

Catchment	Upper Node	Lower Node
SWMH 07-SWMH 05	SWMH 07	SWMH 05
Slope	Diameter	Manning's n
0.50%	225mm	0.013
Return Period	Time of concentration	
10 years	10min	

Catchment SWMH 07-SWMH 05

Description	Area	Runoff Coefficient	Effective Area
Berm	414m^2	0.25	104m^2
Pavement	412m^2	0.85	350m^2
Roof	210m^2	0.95	200m^2
		0.85	0m^2

Upstream Contributions

Catchment	Effective Area

Effective Area

Catchment: 653m^2
Upstream: 0m^2
Total: 653m^2

	Rainfall Intensity	Flow	Velocity	Area of Flow	Water Depth	Air/Water Ratio	Flow (with Air Entrap.)
Min Vel. Flow	27.38mm/h	4.97L/s	0.59m/s	0.01m^2	61mm	-	4.97L/s
Design Flow	85.26mm/h	15.47L/s	0.79m/s	0.02m^2	109mm	-	15.47L/s
Max. Capac.	-	35.31L/s	0.91m/s	0.04m^2	212mm	0.005	35.13L/s

Percent Full:	**44%**
Capacity?	Okay
Velocity?	Warning

Catchment	Upper Node	Lower Node
SWMH 09-SWMH 05	SWMH 09	SWMH 05
Slope	Diameter	Manning's n
0.50%	225mm	0.013
Return Period	Time of concentration	
10 years	10min	

Catchment SWMH 09-SWMH 05

Description	Area	Runoff Coefficient	Effective Area
Berm		0.25	0m^2
Pavement	112m^2	0.85	95m^2
Roof	626m^2	0.95	595m^2
		0.85	0m^2

Upstream Contributions

Catchment	Effective Area

Effective Area

Catchment: 690m^2
Upstream: 0m^2
Total: 690m^2

	Rainfall Intensity	Flow	Velocity	Area of Flow	Water Depth	Air/Water Ratio	Flow (with Air Entrap.)
Min Vel. Flow	27.38mm/h	5.25L/s	0.60m/s	0.01m^2	63mm	-	5.25L/s
Design Flow	85.26mm/h	16.34L/s	0.80m/s	0.02m^2	113mm	-	16.34L/s
Max. Capac.	-	35.31L/s	0.91m/s	0.04m^2	212mm	0.005	35.13L/s

Percent Full:	**47%**
Capacity?	Okay
Velocity?	Warning

Catchment	Upper Node	Lower Node
SWMH 02-EX SWMH	SWMH 02	EX SWMH
Slope	Diameter	Manning's n
0.50%	300mm	0.013
Return Period	Time of concentration	
10 years	20min	

Catchment SWMH 02-EX SWMH			
Description	Area	Runoff Coefficient	Effective Area
Roof	1,566m²	0.95	1,488m²
Paved Areas	1,244m²	0.85	1,057m²
Berm / Gard	777m²	0.25	194m²
Deck	89m²	0.3	27m²
			0m²

Upstream Contributions	
Catchment	Effective Area

Effective Area	
Catchment:	2,766m²
Upstream:	0m²
Total:	2,766m²

	Rainfall Intensity	Flow	Velocity	Area of Flow	Water Depth	Air/Water Ratio	Flow (with Air Entrap.)		
Min Vel. Flow	18.39mm/h	14.13L/s	0.77m/s	0.02m²	94mm	-	14.13L/s	**Percent Full:**	**58%**
Design Flow	56.96mm/h	43.76L/s	1.01m/s	0.04m²	172mm	-	43.76L/s	Capacity?	Okay
Max. Capac.	-	76.05L/s	1.10m/s	0.07m²	283mm	0.006	75.61L/s	Velocity?	Okay

Title:	Winsor Management	Job No.	J000502
	Ropata Village	Page No.	1
Description:	Wastewater Pipes	Date	14/02/2022
		Author:	Warren
		Reviewer:	
		Revision:	A

Pipe Flow Method: 1

1 - Mannings with Constant n

2 - Mannings with Variable n

3 - Mannings with Escritt's Hydraulic Radius

Calculations taken from 'Simple Formulae for Velocity, Depth of Flow, and Slope Calculations in Partially Filled Circular Pipes', Omer Akgiray, Published November 3 2004 in Environmental Engineering Science

Min. Velocity : 0.75m/s using PDWF

Max. Velocity : 3.00m/s using PWWF

Catchment Units = Houses

Persons per House = 3.5

Use Flow per Persons/Houses? Yes

ADWF = 0.00230 L/s/person

~~+ 0 *Area (ha)~~

Use Flow per Area? No

PDWF = 7.23 * ADWF

* Area (Ha) ^ -0.2

~~+ 0 * Area (ha)~~

Use Peaking Factor per Area? Yes

Use Groundwater Infiltration per Pipe Length? Yes

Low Groundwater	0.06	L/s/km
High Groundwater	0.43	L/s/km
Unknown Groundwater	0.25	L/s/km

PWWF = 1 * PDWF

\+ Pipe Length (km) * Infiltration rate (L/s/km)

~~+ 0.55L/s/km * Length upstream of analysis (km)~~

~~+ 0 * Area (ha)~~

Use Pipe Length Inflow Upstream of Analysis? No

Lateral Length (m) = 20 m * Catchment Units

Automatically add laterals to pipe main length? Yes

LINE 101

Pipe	Upper Node	Diameter	Water Table	Main Length		No. Houses	Area	Infilt. / Inflow
	SWMH 08	150 mm	Unknown	85 m	This catch:	20	2,024 m²	0.121 L/s
SWMH 08-SWMH 04	Lower Node	Slope	Manning n	~~Upstream of Analysis~~	Upstream Catch:			
	SWMH 04	1.60%	0.011		Total =	20	2,024 m²	0.121 L/s

	Flow	Hydraulic Radius	Velocity	Area of Flow	Air/Water Ratio	Flow (with air entrap.)		
							Percent Full	**7%**
ADWF	0.16 L/s	0.006 m	0.37 m/s	0.000 m²	-	0.16 L/s		
PDWF	1.60 L/s	0.017 m	0.75 m/s	0.002 m²	-	1.60 L/s		
PWWF	1.72 L/s	0.018 m	0.77 m/s	0.002 m²	-	1.72 L/s	Capacity?	Okay
Capacity	25.32 L/s	0.043 m	1.47 m/s	0.017 m²	0.020	24.82 L/s	Velocity?	Okay

LINE 102

Pipe	Upper Node	Diameter	Water Table	Main Length		No. Houses	Area	Infilt. / Inflow
	SWMH 06	150 mm	Unknown	85 m	This catch:	28	2,110 m²	0.161 L/s
SWMH 06-SWMH 04	Lower Node	Slope	Manning n	~~Upstream of Analysis~~	Upstream Catch:			
	SWMH 04	1.50%	0.011		Total =	28	2,110 m²	0.161 L/s

	Flow	Hydraulic Radius	Velocity	Area of Flow	Air/Water Ratio	Flow (with air entrap.)		
							Percent Full	**10%**
ADWF	0.23 L/s	0.007 m	0.41 m/s	0.001 m²	-	0.23 L/s		
PDWF	2.22 L/s	0.020 m	0.81 m/s	0.003 m²	-	2.22 L/s		
PWWF	2.39 L/s	0.020 m	0.83 m/s	0.003 m²	-	2.39 L/s	Capacity?	Okay
Capacity	24.52 L/s	0.043 m	1.42 m/s	0.017 m²	0.019	24.06 L/s	Velocity?	Okay

LINE 103

Pipe	Upper Node	Diameter	Water Table	Main Length	Upstream Catch:		No. Houses	Area	Infilt. / Inflow
						This catch:	48	4,134 m²	0.261 L/s
SWMH 04-SSMH 03	SWMH 04	150 mm	Unknown	85 m					
	Lower Node	Slope	Manning n	~~Upstream of Analysis~~					
	SSMH 03	1.50%	0.011			Total =	48	4,134 m²	0.261 L/s

	Flow	Hydraulic Radius	Velocity	Area of Flow	Air/Water Ratio	Flow (with air entrap.)		
							Percent Full	**15%**
ADWF	0.39 L/s	0.009 m	0.48 m/s	0.001 m²	-	0.39 L/s		
PDWF	3.33 L/s	0.023 m	0.91 m/s	0.004 m²	-	3.33 L/s		
PWWF	3.59 L/s	0.024 m	0.93 m/s	0.004 m²	-	3.59 L/s	Capacity?	Okay
Capacity	24.52 L/s	0.043 m	1.42 m/s	0.017 m²	0.019	24.06 L/s	Velocity?	Okay

Site	Ropata Village - Lower Hutt
Job No.	J000502
Date	22/02/2022
Analysis By:	Phil Read

Rain Garden Design Approach

Inputs

Rainfall (mm) (2-year 24 Hour) (from Regional Std)	Rainfall (50 year) (from HIRDS)	Climate effect	Impervious	area (m2)	Q50 (L/s)	coefficient of permeability (m/day)	Average height of water	Planting soil depth	Time to pass through soil
75.12	0	0	0.98	150	0	0.3	0.11	1	1.5

Note: ARC TP10 states a maximum water depth of 220mm

Note: Wellington Water's Chief Advisor for Stormwater will accept using a rainfall depth of 10mm/hr, as future Wellington guidelines will be similar to Auckland's Unitary Plan. **USE**

~~1. Determine water quality storage volume – use 1/3 of the 2 year-24 hour rainfall for site location per TP 108.~~ The calculations shall be done considering the pervious and impervious catchment areas separately and their totals then summed.

Rainfall (dff) = [ ] x ~~0.333333~~ = 30

1a. Determine effective area (based on TP108)

$A_{eff}(ha) = \%imp \times Area$ = 0.98 x 0.015 = 0.015

1b. Determine first flush volume (based on TP108)

First Flush Volume (WQV) = 10 x A_{eff} x d_{ff} = WQV

= 10 x 0.015 x 30 = 4.41 m^3

2. Minimum live storage volume shall be 40% of the WQV.

V_{live} = 0.4 x WQV

= 0.4 x 4.41 = 1.8

3. Calculate the required rain garden surface area.
Af = surface area (m2)
WQV = treatment volume (m3)
df = planting soil depth (m) - normally 1m
k = coefficient of permeability (m/day) = 0.3 m/day
h = average height of water (m) = ½ max. depth

tf = time to pass WQV through soil bed (TP108 states 1 day for residential and up to 1.5 for non-residential)

$$Af = \frac{WQV \times d_f}{k\,(h + d_f) \times tf}$$

$$= \frac{4.41 \times 1}{0.3\,(0.11 + 1) \times 1.5}$$

Af = 8.8 m^2

4. Check the live storage requirement is met (Min theoretical > Live storage)

= Af x Max depth(h)

= 8.8 x 0.22 = 1.9 m^3 *Minimum theoretical storage*

1.8 m^3 *Minimum live storage required for WQV*

Minimum theoretical storage> Live storage - therefore design is ok

26 m^2 *Available plan area*

Comment 1: At a rainfall depth of 30mm, the catchment area of 150m2 requires 8.8m2 of rain-garden footprint equalling 5.8%