

ENGINEERING ASSESSMENT PROPOSED RETIREMENT VILLAGE 578 SPRINGS ROAD, PREBBLETON

Engineers and Geologists



# **ENGINEERING ASSESSMENT** PPOPOSED RETIREMENT VILLAGE **578 SPRINGS ROAD, PREBBLETON**

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# ENGINEERING ASSESSMENT PROPOSED RETIREMENT VILLAGE 578 SPRINGS ROAD, PREBBLETON

### 1.0 Introduction

The following report has been prepared by Riley Consultants Ltd (RILEY) at the request of Summerset Villages (Prebbleton) Ltd (Summerset). It presents the results of a civil engineering assessment for a proposed comprehensive care retirement village at the above site and is intended to support a resource consent application to Selwyn District Council (SDC) and Canterbury Regional Council (ECan). The civil engineering assessment specifically addresses finished ground profiles, as well as the provision of stormwater, wastewater, and water supply services for the proposed retirement village.

Our preliminary assessment has been based on the layout prepared by Summerset, as shown on the appended plan (Prebbleton Village Master Plan Rev 8). See Appendix A.

This report should be read in conjunction with our Earthworks and Sediment Control report (RILEY Ref: 190417-M), Geotechnical Assessment report (RILEY Ref: 190417-J) and Preliminary and Detailed Site Investigation Contaminated Land (RILEY Ref: 190417-H).

# 2.0 Site Description and Proposed Village

## 2.1 Background and Site Description

The site is located in Prebbleton, a small town approximately 12km south-west from Christchurch city centre. The site measures approximately 9Ha and is surrounded by residential areas to the north, east, and south. A mushroom factory currently occupies the northern half of the site and comprises several large warehouses and factory buildings. Most of the factory area is paved with concrete or smaller areas of gravel fill. It is understood that the southern half of the site was used as a horse training facility, and currently comprises grassed fields with several bunds up to 2m high separating the fields. A number of mature trees are present towards the south-east of the site, and a residential dwelling is currently located in the southern corner. A 40m wide strip of grassed land with a 2.5m high bund borders the northern boundary of the site. Figure 1 shows an oblique birds-eye view of the site.

Figure 1: Oblique birds-eye view of the site (image courtesy of Google Earth)







The site slopes down at a general grade of 1 in 300 from the north-west to south-east. Existing levels across the site generally range between RL 22m to RL 21m (NZ Vertical Datum 2016).

### 2.2 Proposed Retirement Village

The proposal is for the construction and operation of a comprehensive care retirement village which comprises a two and three-storey central Main Building, including assisted living suites, memory care suites and care beds with a of approximately 4000m2. The Main Building is surrounded by 224 single-storey villas and cottages, several ancillary buildings, roads, car parks, rain gardens and landscaping.

### 2.3 Geology and Soil Conditions

The GNS Science (GNS) geological map of the area (Map 16, Christchurch, 1:250,000, 2008) indicates that the site is underlain by Holocene grey river alluvium several hundred metres thick.

In general, subsurface site investigations indicated topsoil was encountered from 0.0m to 0.2m depth. This was underlain by sandy silt to an average depth of 2.0m (minimum 1.0m, maximum 3.5m), underlain by sandy gravel. Sandy gravel was observed to a maximum depth of 15.2m and is expected to be several hundred metres thick in total.

The bunds in the southern part of the site and the bund to the north of the mushroom factory were of variable composition. The northern bund comprised silt and gravel of an apparently similar nature to the natural subsurface materials and the southern bund comprised dry compacted organic topsoil.

Groundwater was measured after being left to stand overnight upon completion of Borehole (BH) BH201 and BH203 at 6.3m and 7.0m depth, respectively. It was measured again after a week of heavy rain on 3 July 2020 in piezometers installed in BH202 and BH203 at 6.2m and 6.1m depth, respectively.

Reference should be made to our Geotechnical Assessment report (RILEY Ref: 190417-J) for further detail.

# 3.0 Proposed Engineering Works

The following sections outline the proposed engineering works to be undertaken in relation to the retirement village and include consideration of the following:

- i) Earthworks.
- ii) Roading.
- iii) Retaining Walls.
- iv) Water Management.
  - a) Stormwater.
  - b) Wastewater.
  - c) Water Supply.

Engineering works have been designed to be in accordance with the Selwyn District Council, Engineering Code of Practice (SDC CoP).

### 3.1 Earthwork Activities

Earthworks are required to improve the site contours in order to satisfy the design and layout requirements for the proposed retirement village (such as site gradients and building platforms). As part of the bulk earthworks activities, the existing buildings and structures on the property will be demolished and removed.

Some contamination of soils has been confirmed in the PSI/DSI report following extensive testing across the site. The areas where soil contamination is present are localised ('hotspot' areas) which will be remediated in accordance with the Remedial Action Plan and Site Management Plan report (RILEY Ref: 190417-K)

The aforementioned Earthworks and Sediment Control report (RILEY Ref: 190417-M) provides a comprehensive analysis of the earthworks proposed, and the measures to be implemented in order to reduce the impact on the receiving environment.

### 3.2 Roading

The village main site entrance and exit will be from Springs Road. A secondary connection to Glenary Drive is also proposed from the western side of the site. Internal private roads will be constructed to form the village roading network and will vary in width dependant on hierarchy and vehicle movements. Typically, the main thoroughfare roads are 6.5m to 5.5m wide, and minor access roads are 5.0m to 4.5m wide. Driveways and off-street parking spaces will be provided for each villa. Visitor parking is proposed at various locations throughout the site (refer to Summerset's Masterplan in Appendix A).

The minimum longitudinal gradient of internal roads is greater than 0.33% (1 in 300). Roads will be formed to a flexible pavement in accordance with Part 8 of Selwyn District Council's, Engineering Code of Practice.

### 3.3 Retaining Walls

Due to the relatively flat nature of the site, it is anticipated that no significant retaining structures will be necessary. Minor retaining structures (i.e. maximum 0.5m high) are proposed along between villa blocks to provide for level building platforms, and to manage and maintain overland flows within the adjoining road network.

# 4.0 Stormwater Assessment and Management

Existing site contours indicate the site has a shallow general grade from north west to south east. Existing site stormwater runoff is generated from the factory buildings, associated hardstand area and adjacent landscaped areas. The impervious cover is extensive totalling an approximate area of 30,000m2 across the northern portion of the site. The runoff from this extensive impervious area is captured via roof and catch pits and conveyed through gravity pipe networks to a combination of soakage to ground and existing public gravity networks (via connections within Springs Road). The general site grade indicates that any excess runoff from the impervious area and the landscaped areas, flows overland and onto Springs Road.

Pre-consent meetings with Selwyn District Council confirmed that stormwater runoff is shall be discharged into the ground via soakage. The management system proposed for the retirement village will consist of multiple treatment and soakage to ground devices as demonstrated in Figure 2.

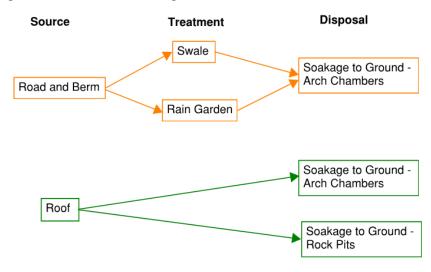


Figure 2: Stormwater Management Overview

The swales and rain gardens will provide stormwater quality mitigation for an urbanised catchment removing contaminants prior to disposing to ground. Given runoff from roof catchments does not encounter potential contaminants, discharge will be direct to ground.

Part 5.96 of the Canterbury Land and Water Plan specifies the soakage system is a permitted activity if it does not:

- a) cause stormwater from up to and including a 24-hour duration 10% Annual Exceedance Probability rainfall event to enter any other property.
- b) does not result in the ponding of stormwater on the ground for more than 48-hours, unless the pond is part of the stormwater treatment system.

In addition, Part 5.12 of SDC CoP specifies for soakage systems:

 Where the discharge of stormwater is to ground, the system should be designed for "total detention" (combination of storage and overflow soakage) of the 2% AEP critical duration storm event.

Therefore, based on the above and following discussions with SDC and ECan, the stormwater system to service the proposed retirement village has been designed for:

- Road reticulation (10% AEP event, including climate change).
- Roof/landscape area reticulation (2% AEP event, including climate change).
- Soakage system (2% AEP event, including climate change).
- Secondary overland flow (2% AEP event, including climate change).

Discussions have been held with the SDC Asset Manager (Mr Murray England), in order to develop the stormwater management concepts, and to obtain approvals in principal. The assessment of the stormwater concepts that were discussed with SDC are outlined below.

### 4.1 Assessment of Proposed Site Activities

The composition of site coverages based on the master plan for the proposed retirement village are as follows in Table 1:

Table 1: Retirement Village Site Coverage

Zoning	Area (m²)
Buildings	28,572
Paved (Roads and Patios)	26,216
Landscape	36,354
Total	91,142

Based on the total proposed site coverage, the average ratio of total impervious area to site area for the retirement village will be 60.4%.

### 4.2 Assessment of Stormwater Runoff

Methodologies used to assess stormwater runoff from the site for the primary and secondary events are based on the SDC CoP. Runoff and attenuation assessments have been undertaken to confirm sizing of the treatment, reticulation, and disposal network. The calculation methodology and results of these assessments are appended to this report (Appendix B to Appendix D).

### 4.2.1 Runoff Coefficients

A site-specific composite runoff coefficient for the Summerset Village of 0.61 has been determined based on the impervious and pervious coverage of the proposed retirement village. Runoff coefficients utilised for the stormwater assessments are presented in Table 2 and are in accordance with NZBC E1/VM1 which is referred to in the SDC CoP..

**Table 2: Runoff Coefficients** 

	Area (m²)	Runoff Coefficients	CA
Buildings	28,572	0.9*	25,954
Roads, car parks, driveways, footpaths	23,173	0.9*	20,231
Patios	3043	0.85*	3222
Landscape	36,354	0.2**	7211
Total	91,160		56,618
Site Comp	0.62		

<sup>\*</sup> For swale design this value was increased to 0.95, in accordance with GD01.

### 4.2.2 Intensity

Design rainfall intensities were determined using the method and data prescribed in SDC CoP Part 5.12. This method accounts for climate change specific to Prebbleton, utilising a scale factor to modify measured rainfall depths (at Christchurch Aero Monitoring Station) to make the data applicable for the site.

Design rainfall intensities are shown in Table 3.

<sup>\*\*</sup> For swale design this value was increased to 0.5, in accordance with GD01.

**Table 3: Rainfall Intensities** 

Average Recurrence Interval (ARI) (Year)	Design Rainfall Intensity - Climate Change Adjusted (10 min duration, mm/hr)
2.33	31.5
5	44.4
10	55.2
20	65.2
50	78.1
100	88.2

For the purposes of this assessment, a time of concentration of 10 minutes has been assumed for all catchments within the site.

### 4.3 Primary Reticulation Assessment

Interception, collection, and conveyance of stormwater runoff generated from the village will be largely achieved by conventional piped reticulation methods. Piped runoff from roofs and landscaped areas will be directed to soakage devices located throughout the village, or for the road catchments to one of six stormwater treatment devices (raingardens/swales). Refer to RILEY Dwg: 190417-18 for the layout of the proposed stormwater reticulation.

The stormwater pipe reticulation proposed to direct stormwater runoff from roof and landscape areas to ground soakage areas will be sized to convey the 50-year design flows. The stormwater pipe reticulation proposed to direct stormwater runoff from road catchments will be sized to convey the primary 10-year (10% AEP) design flows. In events up to and including the 50-year storm event (2% AEP), overland flow from the road catchments will be conveyed along the internal roads to the swales and rain gardens, to be discharged to designated ground soakage areas.

Sumps have been spaced to maintain the maximum spacing of 100m between sumps and to limit the in-flow at each sump to a maximum of 28L/s as per SDC CoP 5.10.10.

New reticulation will be designed and constructed in accordance with the recommendations outlined in this report, SDC CoP and the New Zealand Building Code (NZBC).

### 4.4 Treatment Systems

Stormwater runoff from the site consists of:

- First flush runoff from paved areas i.e. water quality volume
- Runoff from paved surfaces following the first flush
- Runoff from non-paved areas

On-site treatment solutions for first flush stormwater runoff from roading (paved) surfaces shall consist of a number of treatment swales and rain gardens. RILEY Dwg: 190417-17 shows the proposed locations and sizes of these devices and their contributing catchments. Detailed rain garden and swale sizing calculations are presented in Appendix B and Appendix C.

It is proposed that the 10-year design flows will be piped to the inlet of each treatment device, while the flows in excess of this and up to the 50-year design flows will be directed along overland flow paths (internal roading) to the inlets. The rain gardens and swales will provide a certain amount of detention storage during higher-intensity storms. However, for the purposes of this assessment, this additional storage/detention has not been considered and soakage devices within the swales and rain gardens have been designed to discharge all runoff up to the 2% AEP event. This is a conservative approach.

Rain gardens have been sized in accordance with the CCC Rain Garden Design, Construction and Maintenance Manual. In general, the sizing is governed by the requirement to provide a minimum live storage (above ground) of 40% of the first flush (water quality) volume at a maximum ponding depth of 300mm. RILEY Dwg: 190417-21 illustrates a typical rain garden cross-section.

Rain gardens treat the first flush volume, defined as runoff from the first 15-25mm of rainfall (SDC CoP 5.12.9) regardless of storm event. In total, four rain gardens are proposed for the site. Each rain garden has been designed to treat the water quality volume (based on 25mm first flush depth) from the combined area of roads, driveways and car parks in its contributing catchment. Runoff is treated as it percolates through the 600mm layer of filter material at the base of each rain garden. It is then collected by perforated underdrains that direct flow to buried storage chambers to be discharged to ground by soakage (refer Section 1.6.5).

Stormwater treatment swales have been sized utilizing the rational method and Manning's equation for open channel flow, in accordance with SDC CoP Part 5 and Auckland Council GD01 C6.2.1. Buried storage chambers will be installed at the downstream end of each swale so that clean stormwater can be discharged to ground via soakage. Two swales are proposed for the site. The swales have been designed and sized to provide the following:

- adequate treatment for runoff flows from a peak water quality rainfall intensity of 10mm/hr (as per SDC CoP 5.12.9) applied over the combined area of roads, driveways and car parks in the contributing catchment.
- conveyance of flows from storm events up to the 2% AEP 10-minute event and with minimum 150mm freeboard to adjacent structures.

Swales have been designed to treat a rainfall intensity of 10mm/hr (SDC Co 5.12.9) applied over the catchment, resulting in a runoff flowrate that when passed through the treatment swale meets certain criteria (maximum velocity, depth etc.) for adequate treatment. Flows in excess of this in events up to the 2% AEP are simply conveyed to the downstream end of the swale to be discharged directly to ground.

### 4.5 Soakage to Ground

Stormwater runoff generated from the site during all storm events up to the 50-year storm event will be directed, detained, and discharged by on-site soakage devices. Overflow from the soakage areas from events in excess of the 2% AEP, will be directed back to the private roading network which forms the overland flow path network.

Eight soakage tests were carried out across the site by Geotechnics within boreholes augered during the geotechnical subsurface investigations. Testing was concentrated on the gravel layer encountered within the boreholes which were all above the groundwater levels. Falling head tests, in accordance with the Christchurch City Council (CCC) Waterways, Wetlands and Drainage Guide, App 6 - Soakage and Permeability Field Test Methods, were undertaken to confirm the soakage potential. Whilst good soakage rates were achieved at the site, we understand from review of technical reports and documentation regarding soakage devices, these rates can reduce over the life of the device. As a result, the lower end soakage rate of 10,000mm/hour has been factored by 0.5 and a design soakage rate of 5,000mm/hour applied.

Preliminary assessment has been carried out to confirm the extent of stormwater soakage systems required to service the site, and to satisfy design principles and SDC requirements. The assessment was carried out in accordance with SDC CoP and E1/VM1 of the NZBC assuming a combination of soakage systems. A combination of storm durations were assumed to confirm that the soakage systems were sized adequately to detain and discharge all runoff for the 2% AEP storm event (including climate change).

Two types of soakage systems were assessed and consist of the following:

- Type 1: Concrete manholes to a depth range of 2.5m to 4.5m, with 200mm diameter perforated (slotted) pipes extending from the sides (just above the bottom) of the manholes into a layer of porous rock 1m thick. These would be located within landscaped areas with the intention of managing 'clean' runoff from roofed and pervious surfaces.
- Type 2: Buried storage chambers (Cirtex Triton Chambers). These devices will provide
  a localised soakage area and a significant storage volume (for detention during higher
  intensity events) and will generally be placed beneath swales and rain gardens.

Typical details of the above types of soakage systems is shown on RILEY Dwg: 190417-24 and 190417-25. The maximum proposed depth of the soakage system is 4.5m below existing ground level. Given that ground water on the site has been measured at 6.3m below existing ground level, more than 1m separation between the proposed soakage devise and site ground water is available.

Preliminary assessment of available areas for stormwater soakage systems shows adequate capacity to service the retirement village within the confines of the site. Please refer to the appended calculations and results sheet for the soakage device sizing (Appendix D).

Soakage systems will be designed and constructed in accordance with the SDC CoP and the NZBC. Specific soakage testing will be carried out post soakage system installation to confirm adequate soakage is achieved.

### 4.6 Secondary Overland Flow Assessment and Flooding

Consideration has been given to maintaining secondary overland flow to cater for higher intensity rainfall events in the event of blockages or exceedances of the reticulation and soakage system. Secondary overland flow from the site will be directed via the internal road network to the rain gardens adjacent to Springs Road. The secondary overland flow path alignment is indicated on RILEY Dwg: 190417-17.

### 4.7 Operation and Maintenance

The stormwater reticulation and soakage systems will be located within the site and privately owned, operated and maintained by Summerset. A detailed stormwater operation and maintenance manual will be prepared as part of the village design and construction and will be followed by the Summerset maintenance team.

### 5.0 Wastewater

SDC has confirmed that there is capacity in their wastewater system to accept the wastewater from the village at maximum occupation. No upgrading of the existing public network is required.

Discussion with SDC also confirmed the wastewater network connection point for the site is at the recently constructed pump station adjacent the south-west corner of the site. The wastewater pump station manhole invert is RL 17.25m based on the design plans provided by SDC. A connection invert to the pump station manhole of RL 17.4m has been confirmed with SDC.

In accordance with the request from SDC, a separate wastewater reticulation network is proposed to service the main building.

### 5.1 Design Wastewater Production

### **Site Occupancy**

Based on information provided by Summerset, the following approach has been used to design the wastewater system:

- The independent living units have an average design occupancy of 1.3 person per unit.
- All assisted living and memory care suites have an occupancy of 1 person per unit/bed.
- One staff per 10 occupancy.

### **Per Capita Flow Allowances**

Design per capita allowances for the site are based on the SDC CoP. These are as follows:

- Design per capita allowances for the site of 220L/p/d
- Peak dry wastewater flow factor (P/A ratio) of 2.5.
- Storm peak factor (MF) of 2.

### **Wastewater Volumes**

The design wastewater volumes are based on anticipated occupancy for the completed retirement village are as in Table 4:

**Table 4: Design Wastewater Volumes** 

	Number of Units	Occupancy	Per Capita Flows (L/p/d)	Total WW Production (m³/d)
Villas and Cottages	224	292	220	64.2
Care/Assisted living apartments and beds	119	119	220	26.2
Staff		42	50	2.1
		Total Daily W	W Flow (m3/day)	92.5

### 5.2 Wastewater Discharge Flows

The proposed wastewater flows for the retirement village has been calculated in accordance with the SDC CoP. Unit Average Wastewater Flow (ASF) and Maximum Flow (MF) from site discharge point are shown in Table 5.

**Table 5: Wastewater Discharge Flows** 

Discharge Points	ASF (L/sec)	MF (L/sec)	
South-Eastern Corner of Site	1.07	5.35	

### 5.3 Proposed Reticulation

New private reticulation will be designed and constructed in accordance with the SDC CoP and the NZBC to provide each unit with a connection. One connection point, the existing pump station adjacent the southern corner of the site, has been agreed with in principle with SDC. Two connections will be made to this connection point – one for the general village wastewater network, and one for a dedicated main building wastewater network. Given the available depth at the proposed connection point, the site will be serviced by gravity reticulation. The gravity reticulation will consist of 150mm diameter mains and will be constructed within the internal roading network.

Pipe reticulation minimum grades for reticulation servicing:

- Up to 15 villa unit connections = 1 in 160.
- More than 15 villa unit connections = 1 in 200.
- Main building = 1 in 180.

Connection of the new private reticulation servicing the site will be made to the existing public manhole adjacent to the public pump station. New 150mm diameter pipes will be installed from this manhole to the new private manholes inside the site. The proposed layout of the wastewater reticulation is shown on RILEY Dwg: 170743-18.

The wastewater reticulation within the site will be privately owned and managed by Summerset. Calculations and results of the wastewater servicing assessment are appended for reference (Appendix E).

### 6.0 Water Demand

The potable water usage requirements for the village has been calculated from AS/NZS 3500.1:2003, although this demand has been factored to account for the reduced occupancy of the village units where standard units have been factored by 0.75 (1.3 residents on average per unit) and hospital and assisted living care units factored by 0.5 (1.0 residents per unit).

**Table 6: Village Demands** 

	Number Units	Reduction Factor	Equivale nt Units	Total Demand (L/s)	Demand per Unit (L/s)
Villa	224	0.75	168	9.903	0.0442
Assisted Living Suites	56	0.75	42	2.476	0.0442
Memory Care Suites	20	0.75	15	0.884	0.0442
Rest Home and Hospital Beds	43	0.5	21.5	1.267	0.0295
Maintenance Shed	1	0.5	0.5	0.029	0.0295
Bowling Green	1	0.5	0.5	0.029	0.0295
Total	345		247.5	14.589	

Two connection points, as discussed with SDC, will be off the existing 150mm diameter watermain located within Springs Road and the 150mm diameter watermain located within Glanary Drive. Correspondence with Council has indicated that there are no known capacity constraints in terms of supplying potable water to the retirement village even at maximum occupancy.

Connection pressures from the 150mm diameter mains have been confirmed by SDC and are shown in Table 7.

**Table 7: Connection Supply Pressure** 

Scenario	Pressure
Peak Supply	310kPa
Fire Supply	310kPa

We have carried out a preliminary hydraulic analysis of the village watermain system using EPANET software, based on two connections to the 150mm diameter public supply and connection pressure provided.

The two scenarios that have been modelled are for a, peak domestic supply, and FW2 fire flow with 67% village peak domestic demand. For the purposes of the model, the peak domestic demand for the village has been calculated based on simultaneous demand calculation from AS/NZS 3500.1:2003. Results of the modelling are presented in Table 8, and calculations are appended for reference (Appendix F).

**Table 8: Water Flow and Pressure** 

Scenario	Peak Flow	Minimum Villa Pressure	Minimum Main Building Pressure
Peak Domestic	14.67 L/s	214.0kPa	213.1kPa
FW2 Flow, 67% Peak Domestic (1)	34.79L/s	192.9kPa	4191.6kPa

<sup>(1)</sup> Excludes Main Building Sprinkler Supply

Based on the preliminary results, the available boundary pressure is sufficient to service both the expected peak and fire supply demands while maintaining the minimum required residual pressures across the site of 200kPa and 100kPa respectively.

Notwithstanding this, RILEY recommends that a booster pump be installed to service the main building due to its multi-storey nature.

RILEY has also proposed an additional second connection to the existing public main located within Glenary Drive (at the north-western boundary of the site). Modelling of the proposed network has indicated that the additional second connection would ensure better distribution and security of supply, particularly in the event of the primary connection not being in operation. SDC have agreed in principle to this additional connection point in Glenary Drive.

Please note that the minimum residual pressures are largely governed by the head loss through the boundary backflow devices – where an 80kPa loss has been allowed for (based on data collected from existing village sites).

### 6.1 Fire-Fighting Supply

RILEY has undertaken a preliminary investigation into the fire-fighting requirements for the proposed retirement village. The required fire-fighting flows have been determined in accordance with SNZ PAS 4509:2008 (Tables 1, 2, and C1), and based on a fire classification of FW2, the required fire-fighting flow is 12.5L/sec from a hydrant within 135m with another 12.5L/sec from a hydrant within 270m, residual pressure greater than 100kPa.

The proposed main building will require sprinkler fire systems. An independent fire main will be reticulated from Springs Road to a connection point to the main building to service the sprinkler system.

Preliminary results from the fire-fighting supply model indicate that residual pressures across the site exceed the required minimum of 100kPa. However, the residual pressure at the main building is still considered low, due to the anticipated demand and multi-storey nature of the building. Therefore, a booster pump will need to be designed to service the main building, with specific consideration to the firefighting requirements. This will be worked through as part of the detailed design of the fire-fighting requirements, undertaken by a suitably qualified fire engineer and in consultation with the New Zealand Fire Service.

### 6.2 Proposed Reticulation

The proposed layout of the internal potable water reticulation is shown on RILEY Dwg: 190417-20, appended. The site will be serviced by a 180mm diameter (OD) ring main system that will be located within the outer village roads, with a combination of 125mm diameter and 63mm diameter sub-mains servicing secondary and dead-end roads.

Fire hydrants will provide the fire-fighting requirements for the retirement village and these will be fitted to the 180mm and 125mm diameter water mains. Hydrants will provide a minimum 12.5L/sec of flow within 135m of any unit, plus an additional 12.5L/sec within 270m (as per SNZ PAS 4509:2008 in Table 2).

## 6.3 Irrigation Supply

The landscaped areas of the site will require an irrigation supply. Demands during peak periods typically require 120m³/day, with an associated rate of approximately 11.2L/s (flow rate provided for similar sized existing villages) to allow for effective irrigation over a 6-hour period. Irrigation will be undertaken during the night and off-peak periods.

Irrigation supply is proposed to be serviced from the public water supply. Connections to the public network for irrigation will be provided with separate backflow prevention. Location and arrangement for irrigation supply will follow a similar alignment to the 180mm diameter watermain. The design will be provided by others at the time of lodgement for building consent.

## 7.0 Summary

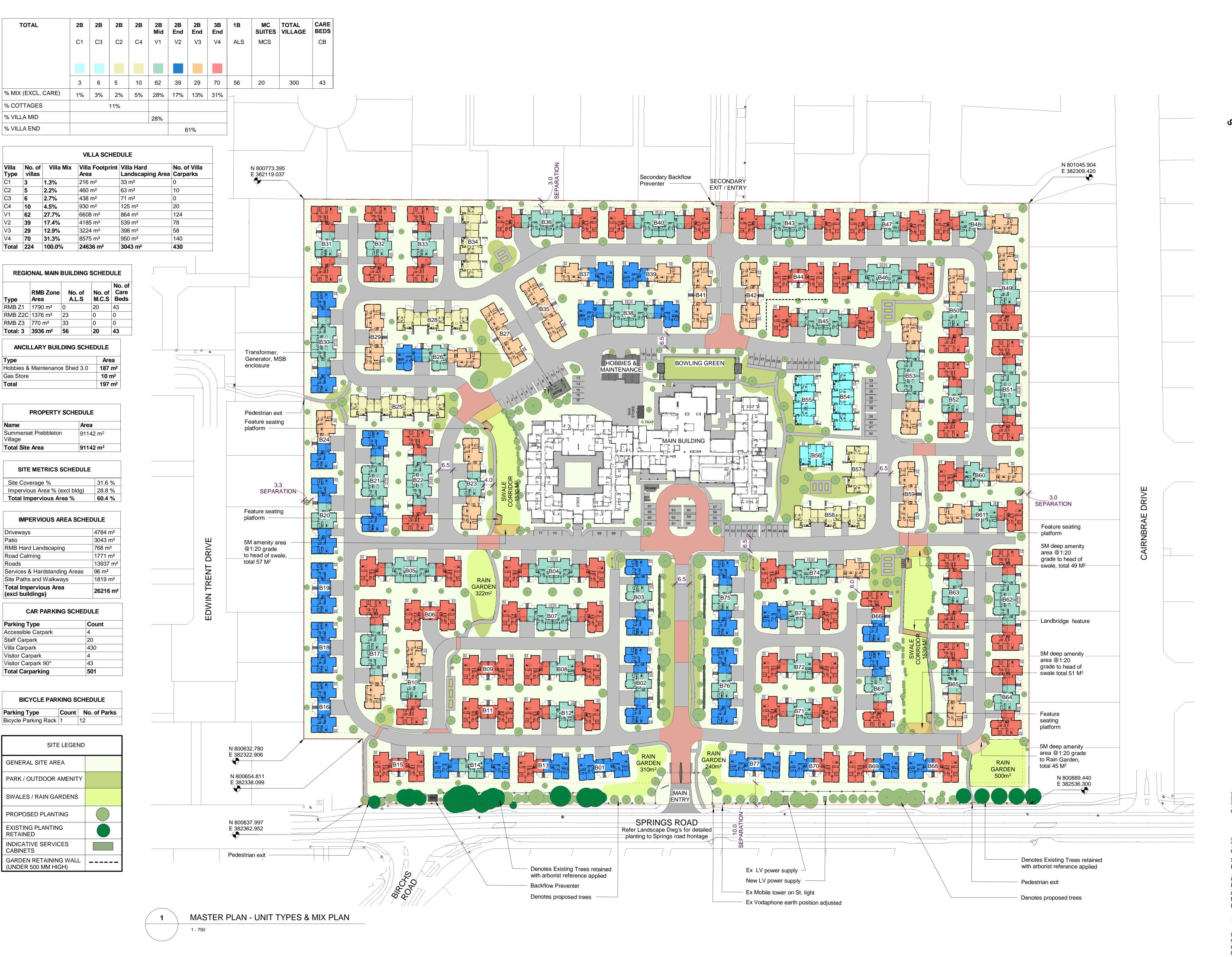
- Stormwater from the site is to be reticulated and discharged to soakage systems via treatment devices (swales and rain gardens) if required (i.e. road runoff).
- Overland flows in excess of the primary design event will be directed along internal roading to swales and rain garden basins within the site. Stormwater runoff at these points will be discharged to ground soakage specifically designed to detain and dispose of the 2% AEP storm event.
- Wastewater reticulation servicing the site will be directed to an existing manhole immediately adjacent to the SDC public pump station off Springs Road, near the south-eastern corner of the site
- Two water supply connections are currently proposed to service the site from the existing 150mm diameter public water main in Springs Road and the 150mm diameter water main in Glenary Drive.

### 8.0 Limitation

This report has been prepared solely for the benefit of Summerset Villages Prebbleton Ltd as our client with respect to the brief and Selwyn District Council and ECan in processing the consent(s). The reliance by other parties on the information or opinions contained in the report shall, without our prior review and agreement in writing, be at such parties' sole risk.

Opinions and judgements expressed herein are based on our understanding and interpretation of current regulatory standards and should not be construed as legal or planning opinions. Where opinions or judgements are to be relied on, they should be independently verified with appropriate advice.

# APPENDIX A **Prebbleton Village Master Plan Rev 8**



A 21.09.20 RC Application Issue
Rev Date Description

Client.

Summerset

Summerset Head Office Ph. 04 894 7320 Fax. 04 894 7319 headoffice@summerset.co.nz www.summerset.co.nz

Project Name.

SUMMERSET
PREBBLETON VILLAGE

Project Stage.

RESOURCE CONSENT

Drawing Title.

MASTER PLAN - UNIT TYPES & MIX

DO NOT SCALE DRAWINGS. DATE
Original Sheet Size: A1 21/09/20
Project Number. Drawing Number. Rev

# **APPENDIX B** Calculation Sheet -Rain Gardens



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Description:

Raingarden Design

### PART A: SIZE RAIN GARDENS FOR WATER QUALITY

### Purpose:

Size rain gardens to treat the water quality volume from specific catchments of the site.

### **Overview:**

Runoff from storm events up to the 10-minute duration 2% AEP event in excess of the water quality volume will overflow from the rain gardens via a scruffy dome manhole and be discharged directly to ground. Four rain gardens shall be provided as shown in Figure 1. Each rain garden is designed to treat the water quality volume from the combined area of roads, driveways and carparks in its respective catchment. Discharge of secondary flows shall include all runoff from the catchment it is serving (i.e. runoff from roads, driveways and carparks and additional runoff from a portion of the roof, patio and landscaped (pervious) areas of each catchment). Refer Table below.

Table 1. Catchment Areas - all units m<sup>2</sup>

Table 1. Catchment Areas –		I <del>-</del>		
Rain Garden	Α	В	C	D
Catchment	4	5	2	6
Treatment Area				
(roads, carparks,	2649	3562	2002	2440
driveways, footpaths)				
2% AEP Area	Includes direct	Includes direct	Includes direct	
additional	soakage	soakage	soakage	
	catchments	catchments DS	catchment DS 17	
	DS 20, 21	27, 28		
Roof	1005	1714	916	0
Patios	124	206	110	0
Pervious	3990	5912	2090	1002
Total	7768	11395	5119	3442

The water quality volume for the catchments upstream (Catchments 1 and 3 as per Figure 1 below) will be treated by stormwater treatment swales (addressed in separate calculation sheet).

### Method:

- Waterways, Wetlands and Drainage Guide (WWDG), Chapter 6
- CCC Rain Garden Design, Construction and Maintenance Manual (RGDCMM)

### Water Quality Volume Input Parameters:

The First Flush water volume was calculated in accordance with WWDG 6.4.1. Refer to Figure 1 on the next page showing a plan view of the catchments contributing to each of rain gardens A, B, C and D.

### Table 2. WQ Parameters

Water quality volume						
Input parameters						
Impervious % contribution	$C_{eff}$	100	%	WWDG Table 6-10		
Discharge coefficient	C <sub>D</sub>	0.9		WWDG Table 6-10		
Composite first flush coefficient	Cff	0.9		WWDG Eqn 6-3. Assumes all roof runoff managed separately.		
First flush depth	$d_{ff}$	25	mm	First flush depth as per WWDG Eqn 6-2		



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Description:

Raingarden Design

### **Water Quality Volume Calculations and Output Parameters:**

**Table 3. First Flush Volume Calculation** 

Water quality volume							
Rain Garden		Α	В	С	D		
Intermediate calcu	lations						
Contributing catchment	А	2649	3562	2002	2440	m <sup>2</sup>	
Water quality area	A <sub>wq</sub>	2384.1	3205.8	1801.8	2196	m²	(A <sub>eff</sub> x C <sub>ff</sub> ) WWDG Eqn 6-2
Outputs							
Volume of first flush	Vff	59.6	80.1	45.0	54.9	m³	(d <sub>ff</sub> x A <sub>wq</sub> ) WWDG Eqn 6-2

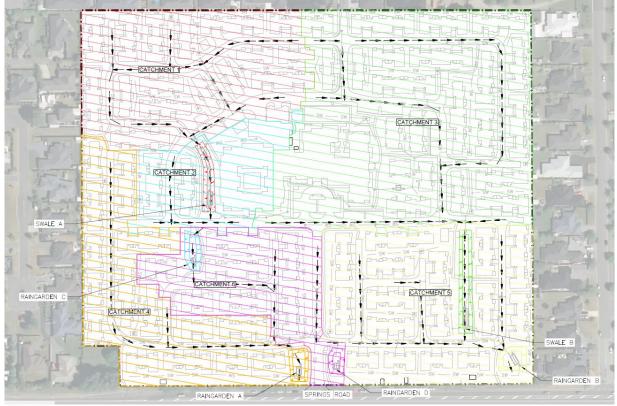


Figure 1: Catchment Areas

### **Rain Garden Area Input Parameters:**

Standard parameters for rain garden filtration were based on the recommended values in RGDCMM Chapter 3.

### **Table 4. Filtration Parameters**

Filtration area of rain garden						
Input parameters						
Planting soil depth	drg	0.6	m	RGDCMM 3.2.1		



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Description: Raingarden Design

Coefficient of permeability	k	30	mm/ hr	(0.72m/day) RGDCMM 3.2.1
Average height of water (half maximum depth)	h	0.15	m	RGDCMM 3.2.1
Time to pass Vff through soil bed	trg	1.0	day	RGDCMM 3.2.1

### **Rain Garden Area Output Parameters:**

A minimum live storage of 40% of the total first flush volume was assumed as recommended by RGDCMM. The total required rain garden footprint area for each rain garden was calculated as follows:

- Rain Garden A 91.3m<sup>2</sup>
- Rain Garden B 125.3m<sup>2</sup>
- Rain Garden C 60.1m<sup>2</sup>
- Rain Garden D 42.9m<sup>2</sup>

Table 5. Rain Garden Sizing

Area of Rain Garden							
Rain Garden		Α	В	С	D		
Outputs							
Minimum live							
storage	LS	23.8	32.1	18.0	22.0	m³	(40% x V <sub>ff</sub> ) RGDCMM Table 1
Filtration area							RGDCMM Equation (1) A <sub>rg</sub> =
of rain garden	Arg	66.2	89.1	50.1	61.0	m <sup>2</sup>	$41.67 * (V_{ff}*d_{rg})/(k*(h+d_{rg})*t_{rg})$
Storage							
provided by this		19.869	26.717	15.016	18.301		
surface area	S	09	14	2	46	m³	(2h x A <sub>rg</sub> ) RGDCMM
Extended							
detention							
(storage) area	AE						
of rain garden	DD	79.5	106.9	60.1	73.2	m²	(LS/2h) RGDCMM Equation (2)

At the resource consent stage RILEY is recommending a reserve allowance for the rain gardens as follows:

• Rain garden A: 300m²

• Rain garden B: 300m<sup>2</sup>

• Rain garden C: 200m<sup>2</sup>

• Rain garden D: 220m²

The above areas make allowance for 1:4 earthworks slope batters. Furthermore these areas as positioned are at the lowest points on site and present a good location to store excess stormwater for discharge to ground during large storm events (in excess of 2% AEP). The shape of the raingarden should follow normal design parameters of a pond for the most efficient use of area with a length to width ratio of no more than 5:1. By increasing this ratio the device would likely become very slender and the base would be dominated by batters.

### PART B: SOAKAGE DESIGN FOR DISCHARGE OF 2% AEP FLOWS

### **Purpose**

Design soakage devices for the discharge of treated and clean runoff (up to the 2% AEP storm runoff) to ground. The soakage device to be used is the Triton double-stack arch system or similar, where the



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**Description:** 

Raingarden Design

Tel: 03 379 4402

design length of the system modifies the base area (discharge surface area) to achieve the required volumetric discharge rate.

### **Method**

- Selywn District Council Code of Practice Part 5: Stormwater and Land Drainage (SDC CoP)
- Rational Method
- NZ Building Code E1/VM1 Surface Water Section 9 Disposal to Soak Pit

### **Input Parameters**

Design soakage rate was determined in accordance with E1/VM1 Section 9. Table 6 presents a summary of the on-site soakage tests that were carried out by Geotechnics. A design soakage rate of 10,000mm/hr was selected on the basis of this information, and a reduction factor of 0.5 was applied to this rate (see Calculation Procedure section below).

Table 6. Soakage Testing

Test	Soakage rate (mm/hr)	Comment
1	10,000	Below average rate over the test duration
2*	3,000	Lowest rate at end of test
3 or 6**	30,000	Steady state rate at end of test
4,5,8***	80,000	Lowest rate at end of test
7*	216	Lowest rate at end of test

<sup>\*</sup>Considered low outliers so ignored

Refer Table 1 for the areas of various surface types tributary to each rain garden. Table 7 below shows the runoff coefficients adopted for the different surface types as suggested in E1/VM1 Table 1, and Table 8 shows rainfall intensities specific to Prebbleton, as determined in accordance with SDC CoP 5.1.2. Rainfall intensities have been adjusted for climate change using a factor of 1.16 for the Selwyn District as recommended by SDC CoP 5.12.2.

**Table 7. Runoff Coefficients** 

Surface Type	Runoff Coefficient
Roof	0.9
Road, driveway, carpark	0.9
Patios	0.85
Landscaped (pervious)	0.2

### **Table 8. Rainfall Intensities**

<b>Duration (min)</b>	Rainfall Intensities (mm/hr)					
	10% AEP	2% AEP				
10	55.2	78.1				
20	42.7	60.6				
30	33.9	47.6				
60	22.7	30.9				
120	16.2	23.2				
360	8.8	13.3				
720	6.1	8.9				
1440	3.9	78.1				

<sup>\*\*</sup>Identical data provided by Geotechnics for these tests

<sup>\*\*\*</sup>Tests 4 and 8 – rate of discharge greater than supplied flowrate from submersible pump, therefore assume equal to highest rate measured from tests



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Table 9. Triton Double-Stack Arch System Dimensions/ Properties

Property Value						
Value						
1.49m						
2.67m						
2.3m						
3.85m						
0.65						

### **Calculation Procedure**

The following is an example calculation for rain garden A, for the 10-minute duration 2% AEP storm

1) Composite runoff coefficient

$$C = \frac{2649 * 0.9 + 1005 * 0.9 + 124 * 0.85 + 3990 * 0.2}{7768} = 0.540$$

Peak flowrate by Rational Method

$$Q = 0.540 * 7768m^2 * 78.1 \frac{mm}{hr} * \frac{0.278}{1000} = 91.1 l/s$$

$$V_{in} = 91.1 \frac{l}{s} * 10min * \frac{60s}{min} * \frac{1}{1000} = 54.7m^3$$

Total volume to be discharged/detained  $V_{in} = 91.1 \frac{l}{s} * 10min * \frac{60s}{min} * \frac{1}{1000} = 54.7m^3$  Storage volume -try triton arch system 2 units wide, 2 units deep, 4m long (conservatively assume 0.65 void ratio applies throughout the volume)

$$V_{\text{stor}} = 2.67m * 3.85m * 4m * 0.65 = 26.7m^3$$

$$V_{stor} = 2.67m * 3.85m * 4m * 0.65 = 26.7m^3$$
  
Volumetric discharge rate to soakage (3.85m x 4m soakage area)  
 $Q_{out} = 0.5 * 10000 \frac{mm}{hr} * 4m * 3.85m \frac{1}{3600} = 21.4 \ l/s$ 

6) Volume discharged over storm duration

$$V_{out} = 21.4 \frac{l}{s} * \frac{1}{1000} * \frac{60s}{min} * 10min = 12.8m^3$$

7) Is the runoff contained?

$$Overflow = 54.7 - 12.8 - 26.7 = 20m^3$$

Rain garden A has an additional 27m3 of storage within its extended detention depth, so the 10minute duration 2% AEP event runoff is contained. Repeat calculation process for longer duration storm events and increase the triton arch system length as necessary.

### **Outputs**

Table 10, Triton Soakage System Sizing

Rain Garden	Dimension (m)		(m)	Comment
	Width Depth Length		Length	
		3.85 2.67 6		2 units wide, 2 units deep. Maximum 6.12m <sup>3</sup> overflow contained within rain garden A's
А	3.85			27m³ storage
				2 units wide, 2 units deep. Maximum 18.61m <sup>3</sup> overflow contained within rain garden B's
В	3.85	2.67	8	37m³ storage



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С	3.85	2.67	4	2 units wide, 2 units deep. Maximum 11.02m³ overflow contained within rain garden C's 18m³ storage
				2 units wide, 2 units deep. No overflow in 2%
D	3.85	2.67	4	AEP, 10 min duration storm

# **APPENDIX C** Calculation Sheet -**Swales**



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Description:

**Stormwater Treatment Swale Design** 

### PART A: SIZE STORMWATER SWALES FOR TREATMENT AND CONVEYANCE **Purpose:**

Size stormwater swales for treatment of Water Quality Storm flows and conveyance of the 2% AEP 10 minute event flows.

### **Overview:**

Two stormwater swales shall be provided, as summarised in the table below.

Table 1. Catchment Areas - all units m2

Swale	A	В
Catchment	1	3
Treatment Area (roads, carparks, driveways, footpaths)	4092	7734
2% AEP Area additional		
Roof	0	0
Patios	0	0
Pervious (25% catchment pervious)	1376	2476
Total	5468	10210

The water quality volume for the areas downstream of the swales (Catchments 2, 4 and 5) will be treated by rain gardens.

Note, 3m exclusion zone to structures has been allowed for within the clients masterplan design.

### Method:

- Selwyn District Council Code of Practice Part 5: Stormwater and Land Drainage (SDC CoP)
- Rational Method
- Auckland Council GD01 C6.2.1

Estimate runoff flowrate into each swale using the Rational Method and subsequently design the swale using the procedure set out in GD01 C6.2.1. Finally, check that the swale is capable of conveying runoff from the 10-minute duration 2% AEP storm event.



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**Description:** Stormwater Treatment Swale Design

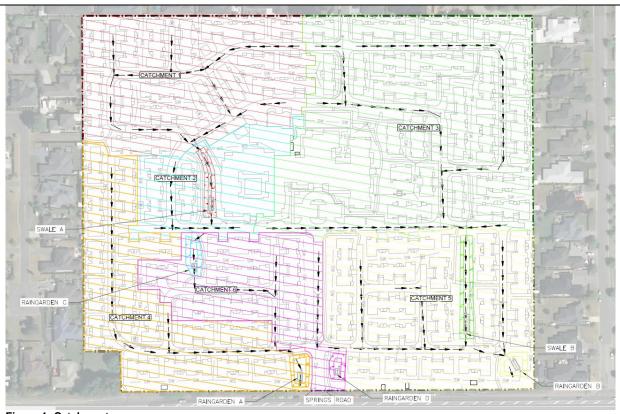


Figure 1: Catchment areas

Figure 1 above shows the catchments upstream of swales A and B (catchments 1 and 3 respectively). Flow is assumed to enter at the head of each swale. Each swale is designed to treat the water quality volume from the combined area of roads, driveways and carparks in its respective catchment. Refer Table 1 above. Runoff coefficients were applied in accordance with the GD01 C6.2.1 method.

Table 2. Runoff Coefficients

Table 2. Runon Coemcients					
Surface Type	Runoff Coefficient				
Road, carpark, driveway, patio	0.95				
Landscaped	0.5				

### **Runoff Flowrate for Water Quality:**

A rainfall intensity of 10mm/hr for the water quality storm was utilsed as per the recommendation in SDC CoP Part 5. Runoff flowrate was determined via the Rational Method.

### Table 3. WQ Flowrate

Runoff for Water Quality Storm									
Swale		А	В						
Input Parameters									
Rainfall intensity	i	10	10	mm/hr	As per SDC CoP part 5				
Outputs									
Peak runoff from impervious surfaces	Q <sub>imp</sub>	0.0108	0.0204	m³/s	(C <sub>imp</sub> x I x A <sub>imp</sub> /360)				



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### **Runoff Flowrate for Conveyance:**

Table 4 shows rainfall intensities specific to Prebbleton, as determined in accordance with SDC CoP 5.12.2. Rainfall intensities have been adjusted for climate change using a factor of 1.16 for the Selwyn District as recommended by SDC CoP 5.12.2. Runoff flowrate was determined via the Rational Method.

**Table 4. Rainfall Intensities** 

<b>Duration (min)</b>	Rainfall Intensities (mm/hr)			
	10% AEP	2% AEP		
10	55.2	78.1		
20	42.7	60.6		
30	33.9	47.6		
60	22.7	30.9		
120	16.2	23.2		
360	8.8	13.3		
720	6.1	8.9		
1440	3.9	78.1		

Table 5. 2% AEP, 10 min Duration Flowrates

Runoff for Conveyance (10 min, 2% AEP Event)									
Swale		Α	В						
Input Parameters									
Rainfall intensity	i	78.1	78.1	mm/hr	SDC CoP 5.1.2.				
Outputs									
Total runoff	Q	0.0993	0.1864	m³/s					

### **Design Swale Cross-Section:**

Manning's equation for open channel flow was used to determine the design flow velocities and depths. The main design inputs/requirements for the treatment swales were as follows:

- Longitudinal slope of swales 0.3%
- Swale A 50m long, Swale B 65m long
- Maximum flow velocity of 0.8m/s and minimum hydraulic residence time of 9 minutes under water quality storm flows
- Capable of conveying 50-year flows with adequate 150mm freeboard within swale whilst satisfying maximum flow velocity requirement of 1.5m/s

Figures 2 and 3 below show the resulting minimum sized cross-sections for swales A and B respectively. Note, the proposed depth of the swales is greater than 0.6m (providing additional free board for 50-year conveyance).



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**Stormwater Treatment Swale Design** 

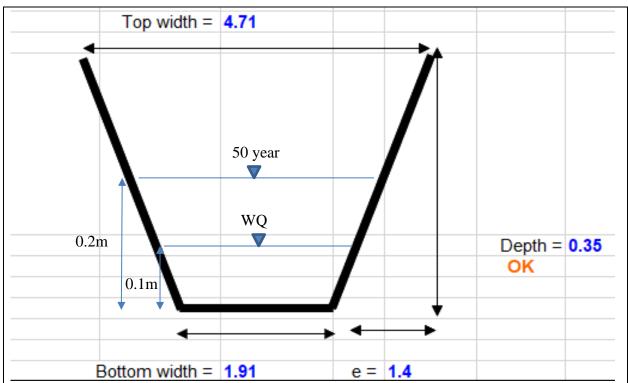


Figure 2: Swale A cross-section

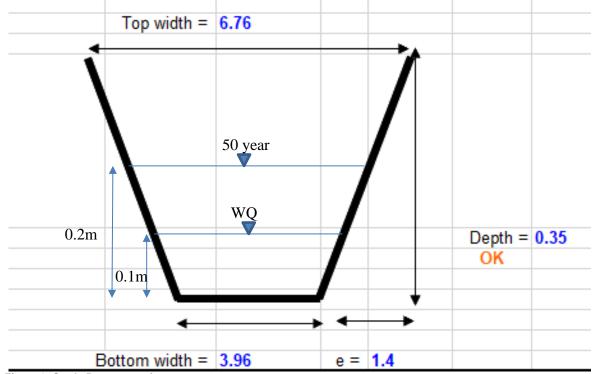


Figure 3: Swale B cross-section



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**Stormwater Treatment Swale Design** 

### **5m Amenity Areas**

Description:

Based on Summerset's request, allowance has been made for 5m long amenity areas at both ends of each swale. This has been assessed to have no effect on the typical swale cross-sections.

The amenity areas themselves shall have the following cross-section, with 1:20 side slopes, and have been designed to convey overland flow form the 50 year, 10 minute storm event.

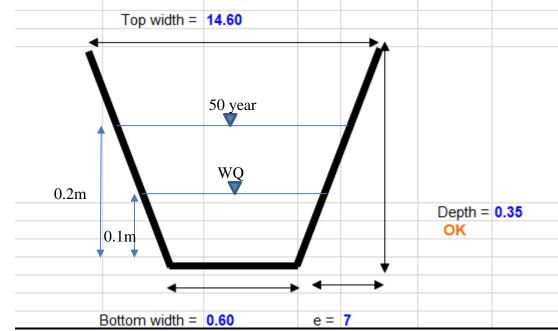
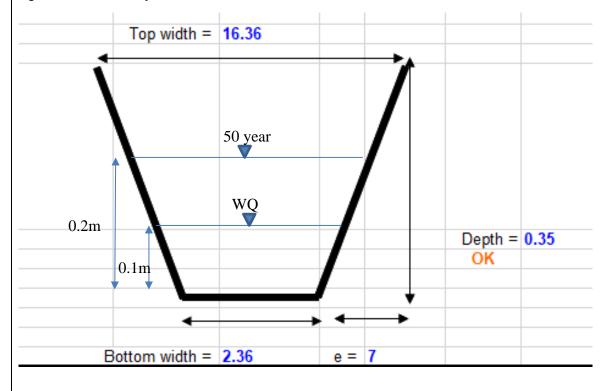


Figure 4: Swale A amenity cross-section





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**Description:** Stormwater Treatment Swale Design

### PART B: SOAKAGE DESIGN FOR DISCHARGE OF 2% AEP FLOWS

### **Purpose**

Design soakage devices for the discharge of treated and clean runoff (up to the 2% AEP storm runoff) to ground. The soakage device to be used is the Triton double-stack arch system or similar, where the design length of the system modifies the base area (discharge surface area) to achieve the required volumetric discharge rate.

### Method

- Selywn District Council Code of Practice Part 5: Stormwater and Land Drainage (SDC CoP)
- Rational Method
- NZ Building Code E1/VM1 Surface Water Section 9 Disposal to Soak Pit

### **Input Parameters**

Design soakage rate was determined in accordance with E1/VM1 Section 9. Table 6 presents a summary of the on-site soakage tests that were carried out by Geotechnics. A design soakage rate of 10,000mm/hr was selected on the basis of this information, and a reduction factor of 0.5 was applied to this rate (see Calculation Procedure section below).

Table 6. Soakage Testing

Test	Soakage rate (mm/hr)	Comment
1	10,000	Below average rate over
		the test duration
2*	3,000	Lowest rate at end of test
3 or 6**	30,000	Steady state rate at end of
		test
4,5,8***	80,000	Lowest rate at end of test
7*	216	Lowest rate at end of test

<sup>\*</sup>Considered low outliers so ignored

Refer Table 7 for the areas of various surface types tributary to the soakage at each swale (note that these differ from the areas for swale conveyance, as some soakage areas are piped directly into the soakage device). Table 8 below shows the runoff coefficients adopted for the different surface types as suggested in E1/VM1 Table 1, and Table 9 shows rainfall intensities specific to Prebbleton, as determined in accordance with SDC CoP 5.1.2. Rainfall intensities have been adjusted for climate change using a factor of 1.16 for the Selwyn District as recommended by SDC CoP 5.12.2.

Table 7. Catchments areas for swale soakage

Swale	A	В
Catchment	1	3
Treatment Area (roads, carparks, driveways, footpaths)	4092	7734
2% AEP Area additional	Includes direct soakage catchments DS9, DS29, DS30	Includes direct soakage catchments DS25, DS26
Roof	2150	1260
Patios	415	151
Pervious	3786	4830

<sup>\*\*</sup>Identical data provided by Geotechnics for these tests

<sup>\*\*\*</sup>Tests 4 and 8 – rate of discharge greater than supplied flowrate from submersible pump, therefore assume equal to highest rate measured from tests



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**Stormwater Treatment Swale Design** 

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Total	10,443	13975

### **Table 8. Runoff Coefficients**

Surface Type	Runoff Coefficient
Roof	0.9
Road, driveway, carpark	0.9
Patios	0.85
Landscaped (pervious)	0.2

### **Table 9. Rainfall Intensities**

<b>Duration (min)</b>	Rainfall Intensities (mm/hr)			
	10% AEP	2% AEP		
10	55.2	78.1		
20	42.7	60.6		
30	33.9	47.6		
60	22.7	30.9		
120	16.2	23.2		
360	8.8	13.3		
720	6.1	8.9		
1440	3.9	78.1		

Table 10. Triton Double-Stack Arch System Dimensions/ Properties

Table 10: The Double Stack Arch Cystem Dimensions, 1 Toperties			
Property	Value		
Depth			
1 unit deep	1.49m		
2 units deep	2.67m		
Width			
1 unit wide	2.3m		
2 units wide	3.85m		
Void Ratio	0.65		

### **Calculation Procedure**

The following is an example calculation for Swale A, for the 10-minute duration 2% AEP storm event.

1) Composite runoff coefficient

$$C = \frac{4092 * 0.9 + 2150 * 0.9 + 415 * 0.85 + 3786 * 0.2}{10443} = 0.644$$

2) Peak flowrate by Rational Method

$$Q = 0.644 * 10443m^{2} * 78.1 \frac{mm}{hr} * \frac{0.278}{1000} = 146.0 \ l/s$$

Total volume to be discharged/detained

$$V_{in} = 146.0 \frac{l}{s} * 10min * \frac{60s}{min} * \frac{1}{1000} = 87.6m^3$$

 $V_{in}=146.0\frac{l}{s}*10min*\frac{60s}{min}*\frac{1}{1000}=87.6m^3$  Storage volume -try triton arch system 2 units wide, 2 units deep, 8m long (conservatively assume 0.65 void ratio applies throughout the volume)

$$V_{stor} = 2.67m * 3.85m * 8m * 0.65 = 53.4m^3$$

Volumetric discharge rate to soakage (3.85m x 8m soakage area) 
$$Q_{out} = 0.5*10000 \frac{mm}{hr} * 8m*3.85m \frac{1}{3600} = 42.8 \ l/s$$

6) Volume discharged over storm duration



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SJB 02/09/2020 Check: Date:

Description:

**Stormwater Treatment Swale Design** 

$$V_{out} = 42.8 \frac{l}{s} * \frac{1}{1000} * \frac{60s}{min} * 10min = 25.7m^3$$

### 7) Is the runoff contained?

$$Overflow = 87.6 - 53.4 - 25.7 = 8.5$$

Swale A has an estimated additional 13m<sup>3</sup> of storage within its length, so the 10-minute duration 2% AEP event runoff is contained. Repeat calculation process for longer duration storm events and increase the triton arch system length as necessary.

### **Outputs**

Table 11. Triton Soakage System Sizing

Swale	Dimension (m)		Dimension (m) Comment	
	Width	Depth	Length	
				2 units wide, 2 units deep. Maximum 4.98m <sup>3</sup> overflow contained within swale A's
Α	3.85	2.67	10	estimated 13m³ storage
				2 units wide, 2 units deep. Maximum 15.4m <sup>3</sup> overflow contained within swale B's
В	3.85	2.67	13	estimated 29m³ storage

# APPENDIX D Calculation Sheet -Soakage Chambers



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Description:

Soakage Chamber Calculations, Reticulation Sizing

### SOAKAGE MANHOLE DESIGN FOR DISCHARGE OF 2% AEP FLOWS **Purpose**

Design soakage manholes for the discharge of clean runoff (up to the 2% AEP storm runoff) to gravel.

The soakage devices to be used shall be concrete manholes (between 1050mm-1800mm diameter) to a minimum depth of 2.5m, with 200mm diameter slotted novaflo pipes extending from the sides (just above the bottom) of the manholes into a layer of porous rock 1m thick. The base surface area of the layer of porous rock minus the base area of the manhole is assumed to be the area over which the design soakage rate is applied. This requires sizing of both the manholes (storage volume) and the dimensions of the porous rock layer (discharge rate) around the manhole.

Note: catchments DS20, DS21, DS27, DS28, DS17, DS9, DS29, DS30, DS25 and DS26 discharge via Triton arch chambers (instead of manhole with porous rock base) either positioned to capture runoff from multiple DS catchments, or at the ends of swales/rain gardens. Similar calculation procedure for soakage system design/sizing followed - refer rain garden and swale calculation sheets.

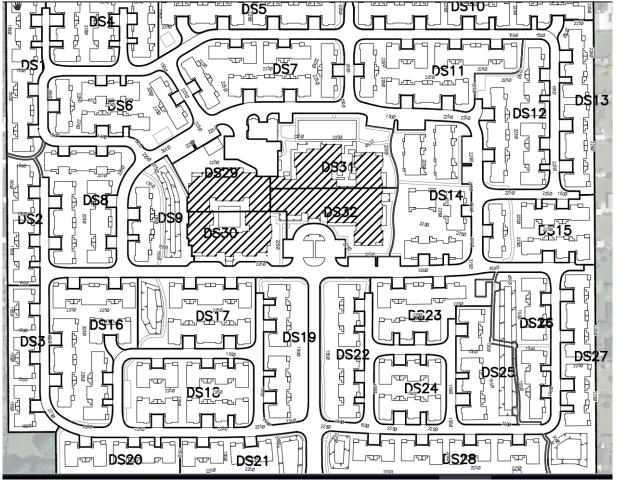


Figure 1. Direct Soakage Catchments

### Method

- Selywn District Council Code of Practice Part 5: Stormwater and Land Drainage (SDC CoP)
- Rational Method
- NZ Building Code E1/VM1 Surface Water Section 9 Disposal to Soak Pit



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Soakage Chamber Calculations, Reticulation Sizing

### **Input Parameters**

Design soakage rate was determined in accordance with E1/VM1 Section 9. Table 1 presents a summary of the on-site soakage tests that were carried out by Geotechnics. A design soakage rate of 10,000mm/hr was selected on the basis of this information, and a reduction factor of 0.5 was applied to this rate (see Calculation Procedure section below).

Table 1. Soakage Testing

Test	Soakage rate (mm/hr)	Comment
1	10,000	Below average rate over the test duration
2*	3,000	Lowest rate at end of test
3 or 6**	30,000	Steady state rate at end of test
4,5,8***	80,000	Lowest rate at end of test
7*	216	Lowest rate at end of test

<sup>\*</sup>Considered low outliers so ignored

Refer Table 2 for the areas of various surface types tributary to the soakage at each manhole. Table 3 below shows the runoff coefficients adopted for the different surface types as suggested in E1/VM1 Table 1, and Table 4 shows rainfall intensities specific to Prebbleton, as determined in accordance with SDC CoP 5.1.2. Rainfall intensities have been adjusted for climate change using a factor of 1.16 for the Selwyn District as recommended by SDC CoP 5.12.2.

Table 2. Catchments areas for swale soakage

Catchment	Total	Roof	Patios	Landscaped (75%)
DS1	1449.1	775.0	96.6	577.6
DS2	1058.3	536.7	69.1	452.6
DS3	1279.1	640.1	82.9	556.1
DS4	1875.2	983.3	119.7	772.2
DS5	1803.4	922.0	110.0	771.4
DS6	2218.4	1050.4	133.9	1034.1
DS7	2617.8	1308.5	165.5	1143.8
DS8	2319.3	1255.4	160.5	903.4
DS9	1422.1	331.5	41.4	1049.2
DS10	2084.8	1158.5	137.3	789.0
DS11	2558.4	1383.4	164.6	1010.4
DS12	2301.5	1126.6	137.6	1037.3
DS13	1513.7	804.2	96.2	613.3
DS14	2722.8	1116.3	167.0	1439.5
DS15	1852.6	905.1	109.9	837.5
DS16	2802.8	1489.9	178.6	1134.4
DS17	2123.4	916.2	110.0	1097.2
DS18	2325.1	1195.2	136.4	993.5
DS19	1659.6	648.2	83.2	928.3
DS20	1530.2	563.5	68.7	898.0
DS21	1510.5	441.8	55.0	1013.7
DS22	1644.2	639.5	83.2	921.5
DS23	1635.2	783.1	96.5	755.6

<sup>\*\*</sup>Identical data provided by Geotechnics for these tests

<sup>\*\*\*</sup>Tests 4 and 8 – rate of discharge greater than supplied flowrate from submersible pump, therefore assume equal to highest rate measured from tests



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Soakage Chamber Calculations, Reticulation Sizing **Description:** 

DS24	1280.9	704.8	82.2	493.9
DS25	1512.1	570.8	68.7	872.6
DS26	1664.4	688.8	82.3	893.4
DS27	2041.2	815.6	96.1	1129.5
DS28	2745.5	898.8	109.8	1736.8
DS29	1490.0	935.6	186.8	367.7
DS30	1460.3	883.1	186.8	390.4
DS31	2473.0	1111.6	186.8	1174.6
DS32	1860.9	1254.8	186.8	419.4

### **Table 3. Runoff Coefficients**

Surface Type	Runoff Coefficient
Roof	0.9
Road, driveway, carpark	0.9
Patios	0.85
Landscaped (pervious)	0.2

### Table 4. Rainfall Intensities

<b>Duration (min)</b>	Rainfall Intensities (mm/hr)		
	10% AEP	2% AEP	
10	55.2	78.1	
20	42.7	60.6	
30	33.9	47.6	
60	22.7	30.9	
120	16.2	23.2	
360	8.8	13.3	
720	6.1	8.9	
1440	3.9	78.1	

### **Calculation Procedure**

Following is an example calculation for Direct Soakage catchment 1 (DS1), for the 10-minute duration 2% AEP storm event.

1) Composite runoff coefficient

$$C = \frac{775 * 0.9 + 96.6 * 0.85 + 577.6 * 0.2}{1449.1} = 0.618$$

2) Peak flowrate by Rational Method

$$Q = 0.618 * 1449.1m^2 * 78.1 \frac{mm}{hr} * \frac{0.278}{1000} = 19.4 l/s$$

Total volume to be discharged/detained

$$V_{in} = 19.4 \frac{l}{s} * 10min * \frac{60s}{min} * \frac{1}{1000} = 11.6m^3$$

$$V_{stor} = \pi * \frac{\left(\frac{1050}{1000}\right)^2}{4} * 2.5m = 2.16m^3$$

Total volume to be discharged/detained  $V_{in} = 19.4 \frac{l}{s} * 10min * \frac{60s}{min} * \frac{1}{1000} = 11.6m^3$  Storage volume -try 1050mm manhole 2.5m deep  $V_{stor} = \pi * \frac{\left(\frac{1050}{1000}\right)^2}{4} * 2.5m = 2.16m^3$  Volumetric discharge rate to soakage (assume the porous rock layer covers a 3m by 3m base soakage area)

$$Q_{out} = 0.5 * 10000 \frac{mm}{hr} * 3m * 3m * \frac{1}{3600} = 12.5 l/s$$



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**Description:** | Soakage Chamber Calculations, Reticulation Sizing

6) Volume discharged over storm duration

$$V_{out} = 12.5 \frac{l}{s} * \frac{1}{1000} * \frac{60s}{min} * 10min = 7.5m^3$$

7) Is the runoff contained?

$$Overflow = 11.6 - 2.16 - 7.5 = 1.94m^3$$

Allowing a maximum of 10m<sup>3</sup> of overflow (drainage of overflow from critical duration 50 year event will drain down in less than 1 hour) per catchment, a 1050 manhole to 2.5m depth with 3m x 3m porous rock base is sufficient for the 10-minute duration 2% AEP event runoff. Repeat calculation process for longer duration storm events and increase manhole size or the length/width of the porous rock base as necessary

### **Outputs**

Table 5. Manhole / Porous Rock Layer sizing

Catchment	Manhole diameter (mm)	Area of porous rock base (m²)
DS1	1050	8
DS2	1050	8
DS3	1050	8
DS4	1500	7
DS5	1050	8
DS6	1050	14
DS7	1500	13
DS8	1500	13
DS9	1050	8
DS10	1050	14
DS11	1500	13
DS12	1200	14
DS13	1050	8
DS14	1200	14
DS15	1050	8
DS16	1800	12
DS17	1500	7
DS18	1050	14
DS19	1050	8
DS20	1050	8
DS21	1050	8
DS22	1050	8
DS23	1050	8
DS24	1050	8
DS25	1050	8
DS26	1050	8
DS27	1050	8
DS28	1050	14
DS29	1200	8
DS30	1050	8
DS31	1050	14
DS32	1050	14



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Soakage Chamber Calculations, Reticulation Sizing

### **RETICULATION SIZING**

Reticulation for the direct-to-soakage catchments has been sized using the Colebrook-White equation for piped flow, assuming a minimum pipe grade of 1:200 and roughness coefficient of 0.015mm. Pipes have been sized for the 50 year flows to be consistent with the design for the soakage chambers. Minimum pipe sizes for each DS catchment are presented in the table below.

Table 6. Pipe Sizing	
50 year flow (I/s)	Pipe diameter (mm)
19.44	225
13.73	150
16.46	225
24.79	225
23.41	225
27.50	225
33.61	225
31.43	225
11.80	150
28.61	225
34.48	225
29.07	225
20.16	225
31.16	225
23.36	225
37.35	300
24.71	225
30.20	225
18.24	225
16.19	225
14.06	225
18.04	225
20.37	225
17.44	225
16.22	225
18.87	225
22.63	225
27.15	225
23.34	225
22.41	225
30.28	225
29.80	225

# **APPENDIX E** Calculation Sheet -Wastewater



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**Description:** Wastewater Supply Demand Calculations

### Aim:

Determine wastewater demands and design reticulation to service the proposed retirement village.

### Method:

In general accordance with Selwyn District Council (SDC), Code of Practice (CoP) and NZBC G13/AS2.

### Demand:

The wastewater demands for development such as a retirement village have been calculated based on SDC CoP part 6, using the following parameters:

- All independent units have an average design occupancy of 1.3 people per unit.
- All assisted living units and care beds have an occupancy of 1 person per unit/ bed.
- 1 staff member per 10 village occupants.

The following design considerations from SDC CoP section 6.4 were utilised to determine the wastewater flows:

- Design per capita allowances for the site of 220L/p/d
- Peak dry wastewater flow factor (P/A ratio) of 2.5
- Storm peak factor (SPF) of 2

From previous design experience with village developments wastewater flows for staff are typically entered at 50 L/p/d, which is in line with flows for commercial facilities as per AS/NZS 1547 table H4.

The design wastewater volumes based on anticipated occupancy for the completed development are as follows:

Table 1: Design Wastewater Volumes - Completed Development

	Number of Units	Occupancy	Per Capita Flows (L/p/d)	Total WW Production (m3/d)
Villas and Cottages	224	292	220	64.2
Care apartments and beds	119	119	220	26.2
Staff		42	50	2.1
		92.5		

With consideration to the above flows, the Unit Average Wastewater Flow (ASF) and Maximum Flow (MF) for the site as follows:

**Table 2: Wastewater Discharge Flows** 

Discharge Point	ASF (L/s)	MF (L/s)
Total	1.07	5.35



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Description:

**Wastewater Supply Demand Calculations** 

### Reticulation:

The site is to be serviced by the existing public pump station as confirmed in pre-consent meetings. Design information provided by SDC specifies the invert level on of the inlet manhole is 17.25 RLm.

Gravity pipe networks will be utilised to service the village. As instructed by SDC, two networks will be required – one servicing the villa units and the other dedicated to the regional main building. The indicative layout of the networks has been provided in the civil design plans.

In accordance with table 1 in SDC CoP section 6.5.5, the following minimum grades are proposed:

- Main building = 1 in 180
- 15 villa unit connections = 1 in 160
- More than 15 villa unit connections = 1 in 200

An assessment utilising the lengths from the proposed network, a connection invert level to the public pump station of 17.4 RLm and the proposed manhole lid levels (LL) at the upstream end of the network presented in table 3 below confirms a minimum cover of 0.8m can be achieved at the upstream ends of the wastewater network:

**Table 3: Wastewater Grade and ILs** 

		1:160	1:200	1:180	Pipe	МН	Design		
		length	length	length	Drop	Drop	FGL [LL]	MH IL	Cover
		(m)	(m)	(m)	(m)	(m)	(RLm)	(RLm)	(m)
	Line A	106	384		2.58	0.30	21.60	20.44	1.00
	Line B	72	435		2.63	0.35	21.50	20.54	0.81
	Line C	109	280		2.08	0.20	20.95	19.84	0.95
Villa	Line D	147	57		1.20	0.20	21.20	19.54	1.50
	Line E	46	105		0.81	0.05	21.60	19.94	1.50
	Line F	74	312		2.02	0.40	21.90	20.24	1.50
	Line G	103	259		1.94	0.30	22.10	20.44	1.50
	Line I								
RMB	(MH1)			454	2.84	0.30	21.90	20.70	1.04
KIVID	Line H								
	(MH2)			362	2.26	0.25	21.60	20.07	1.37

These MH levels are presented on the wastewater alignment plan (Ref 190417-19).



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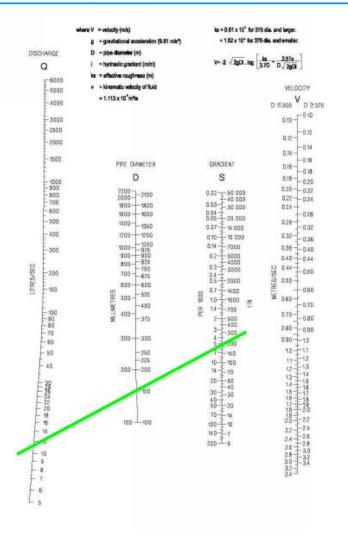
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**Wastewater Supply Demand Calculations** 

All pipes are proposed as 150mm diameter PVC. Conservatively assuming maximum flow was conveyed through one section of pipe laid at minimum grade proposed (1 in 200), the pipe flow nomograph from SDC CoP demonstrates that the pipe can convey 10.9 L/s (double the calculated maximum wastewater flow):

Figure 1: Pipe Flow

### APPENDIX I. PIPE FLOW NOMOGRAPH<sup>1</sup>



# APPENDIX F Calculation Sheet –

Water Supply EPANET



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**Potable Water EPAnet Modelling** 

### **Purpose:**

Create water network model to confirm water reticulation operating pressures for the proposed Summerset Prebbleton Retirement Village. Calculations intended to provide supporting information for Building Consent application to Selwyn District Council.

Method: EPANET Model (2.0).

### **Model Parameters:**

Water network pressures have been checked using EPANET modelling software using parameters outlined below.

### Pipe Type:

All water pipes are to be:

- PE80B SDR11 PN12.5 for watermains less than 110mmø (OD)
- PE100 SDR13.6 PN12.5 for all watermains 110mmø or greater (OD)

Pipe IDs modelled as per manufacturers' dimensions as follows:

Nominal	Internal				
Diameter	Diameter				
(mm)	(mm)				
180	152.8				
63	51.0				
20	16.1				

### 3.0 PE80B Pipe Dimensions

Standard AS/NZS 4130

Polyethylene Catalogue

		PN10 SDR 13.6				PN12.5 SDR 11				PN16 SDR 9			
Nominal Size	Mean OD	Mean Bore	T Min	T Max	Mass kg/m	Mean Bore	T Min	T Max	Mass kg/m	Mean Bore	T Min	T Max	Mass kg/m
20	20.2	16.7	1.6	1.9	0.096	16.1	1.9	2.2	0.110	15.2	2.3	2.7	0.132
25	25.2	21.1	1.9	2.2	0.142	20.2	2.3	2.7	0.168	19.2	2.8	3.2	0.199
32	32.2	27.0	2.4	2.8	0.230	26.0	2.9	3.3	0.266	24.5	3.6	4.1	0.326
40	40.2	33.8	3.0	3.4	0.353	32.3	3.7	4.2	0.423	30.6	4.5	5.1	0.507
50	50.3	42.4	3.7	4.2	0.546	40.4	4.6	5.2	0.657	38.4	5.6	6.3	0.788
63	63.3	53.3	4.7	5.3	0.870	51.0	5.8	6.5	1.038	48.2	7.1	8.0	1.256
75	75.4	63.7	5.5	6.2	1.214	61.0	6.8	7.6	1.450	57.6	8.4	9.4	1.766
90	90.5	76.5	6.6	7.4	1.744	73.1	8.2	9.2	2.102	69.1	10.1	11.3	2.548
110	110.5	93.3	8.1	9.1	2.615	89.4	10.0	11.1	3.114	84.5	12.3	13.7	3.783
125	125.6	106.1	9.2	10.3	3.371	101.5	11.4	12.7	4.041	96.1	14.0	15.5	4.880
140	140.7	118.9	10.3	11.5	4.223	113.9	12.7	14.1	5.037	107.6	15.7	17.4	6.132
160	160.8	135.9	11.8	13.1	5.512	130.0	14.6	16.2	6.612	123.1	17.9	19.8	7.986
180	180.9	152.8	13.3	14.8	6.996	146.3	16.4	18.2	8.358	138.5	20.1	22.3	10.104
200	200.9	169.9	14.7	16.3	8.577	162.5	18.2	20.2	10.302	153.7	22.4	24.8	12.488
225	226.1	191.1	16.6	18.4	10.895	182.9	20.5	22.7	13.044	173.2	25.1	27.8	15.760
250	251.2	212.4	18.4	20.4	13.421	203.4	22.7	25.1	16.043	192.5	27.9	30.8	19.433
280	281.3	237.9	20.6	22.8	16.813	227.8	25.4	28.1	20.108	215.4	31.3	34.6	24.423
315	316.5	267.6	23.2	25.7	21.311	256.3	28.6	31.6	25.458	242.4	35.2	38.9	30.901
355	356.6	301.6	26.1	28.9	27.011	288.8	32.2	35.6	32.305	273.3	39.6	43.7	39.150
400	401.8	339.9	29.4	32.5	34.256	325.4	36.3	40.1	41.017	307.8	44.7	49.3	49.769
450	452.1	382.4	33.1	36.6	43.398	366.1	40.9	45.1	51.949	346.5	50.2	55.4	62.923
500	502.3	424.9	36.8	40.6	53.546	406.8	45.4	50.1	64.096	385.0	55.8	61.5	77.657
560	562.5	475.9	41.2	45.5	67.167	455.8	50.8	56.0	80.283				
630	632.9	535.5	46.3	51.1	84.911	512.6	57.2	63.1	101.737				
710	713.2	603.4	52.2	57.6	107.862								
800	803.6	680.0	58.8	64.8	136.820								
1000	1004.5												

PE80B: SDR 13.6 - SDR 9

**Dimensions** 



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**Potable Water EPAnet Modelling** 

### PE100 Pipe Dimensions

Standard AS/NZS 4130

	20	PN10 SDR 17				PN12.5 SDR 13.6				PN16 SDR 11			
Nominal Size	Mean OD	Mean Bore	T Min	T Max	Mass kg/m	Mean Bore	T Min	T Max	Mass kg/m	Mean Bore	T Min	T Max	Mass kg/m
20	20.2	16.7	1.6	1.9	0.096	16.7	1.6	1.9	0.096	16.1	1.9	2.2	0.110
25	25.2	21.7	1.6	1.9	0.122	21.1	1.9	2.2	0.142	20.2	2.3	2.7	0.168
32	32.2	28.1	1.9	2.2	0.184	27.0	2.4	2.8	0.230	26.0	2.9	3.3	0.266
40	40.2	35.0	2.4	2.8	0.292	33.8	3.0	3.4	0.353	32.3	3.7	4.2	0.423
50	50.3	43.9	3.0	3.4	0.450	42.4	3.7	4.2	0.546	40.4	4.6	5.2	0.657
63	63.3	55.2	3.8	4.3	0.716	53.3	4.7	5.3	0.870	51.0	5.8	6.5	1.038
75	75.4	65.8	4.5	5.1	1.011	63.7	5.5	6.2	1.214	61.0	6.8	7.6	1.450
90	90.5	79.0	5.4	6.1	1.454	76.5	6.6	7.4	1.744	73.1	8.2	9.2	2.102
110	110.5	96.5	6.6	7.4	2.162	93.3	8.1	9.1	2.615	89.4	10.0	11.1	3.114
125	125.6	109.9	7.4	8.3	2.759	106.1	9.2	10.3	3.371	101.5	11.4	12.7	4.041
140	140.7	123.1	8.3	9.3	3.464	118.9	10.3	11.5	4.223	113.9	12.7	14.1	5.037
160	160.8	140.7	9.5	10.6	4.522	135.9	11.8	13.1	5.512	130.0	14.6	16.2	6.612
180	180.9	158.3	10.7	11.9	5.720	152.8	13.3	14.8	6.996	146.3	16.4	18.2	8.358
200	200.9	175.8	11.9	13.2	7.055	169.9	14.7	16.3	8.577	162.5	18.2	20.2	10.302
225	226.1	197.8	13.4	14.9	8.951	191.1	16.6	18.4	10.895	182.9	20.5	22.7	13.044
250	251.2	220.0	14.8	16.4	10.969	212.4	18.4	20.4	13.421	203.4	22.7	25.1	16.043
280	281.3	246.3	16.6	18.4	13.778	237.9	20.6	22.8	16.813	227.8	25.4	28.1	20.108
315	316.5	277.1	18.7	20.7	17.450	267.6	23.2	25.7	21.311	256.3	28.6	31.6	25.458
355	356.6	311.1	21.1	23.4	22.203	301.6	26.1	28.9	27.011	288.8	32.2	35.6	32.305
400	401.8	351.9	23.7	26.2	28.062	339.9	29.4	32.5	34.256	325.4	36.3	40.1	41.017
450	452.1	395.9	26.7	29.5	35.559	382.4	33.1	36.6	43.398	366.1	40.9	45.1	51.949
500	502.3	440.0	29.6	32.7	43.802	424.9	36.8	40.6	53.546	406.8	45.4	50.1	64.096
560	562.5	492.7	33.2	36.7	55.028	475.9	41.2	45.5	67.167	455.8	50.8	56.0	80.283
630	632.9	554.4	37.3	41.2	69.541	535.5	46.3	51.1	84.911	512.6	57.2	63.1	101.73
710	713.2	624.6	42.1	46.5	88.438	603.4	52.2	57.6	107.862				
800	803.6	703.9	47.4	52.3	112.141	680.0	58.8	64.8	136.820				
1000	1004.5	879.8	59.3	65.4	175.319								

PE100: SDR 17 - SDR 11

Dimensions

### Losses

olyethylene Catalogue

- Pipe: For PE pipes Ks (mm) = 0.015 (as per NZS4404: Table 6.1). This allows for pipe aging and fittings effects.
- RPZ backflow preventer: 8 m loss has been applied through RPZ (based on product specification sheets and data collected from other villages).
- Minor fitting loss coefficients have been applied throughout the network based on the following:
  - Valve (fully open) = 0.2
  - Tee (flow through run) = 0.6
  - Tee (branch) = 1.8
  - Reducers = 0.5
  - $-90^{\circ} \text{ bend} = 0.6$
  - $-45^{\circ}$  bend = 0.3
  - Hydrant = 2.5

### Operating Pressures

Two connections to the council's existing public mains have been proposed for the village: one to the main along Springs Road, and one to the main within Glenary Drive at the north-western boundary of the site. The connections have been modelled simultaneously with a minimum operating pressure of 310 kPa at each connection for both normal supply and fire supply as per in-person correspondence with Murray England from Selwyn District Council during pre-consent meetings.

Elevation differences throughout the site have been modelled. Note, the site has approximately 1.5m elevation difference generally grading from the north west to the south east.



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**Potable Water EPAnet Modelling** 

### Demand

The potable water usage requirements for the village have been calculated from AS/NZS 3500.1:2003, although this demand has been factored to account for the reduced occupancy of the village units where standard units have been factored by 0.75 (1.3 residents on average per unit) and hospital and dementia care units factored by 0.5 (1.0 residents per unit). Allowance has also been made to service the bowling green and maintenance shed – both have been factored by 0.5.

Based on the factored demand, the total village instantaneous potable demand is 14.6 L/s. Village unit composition is outlined in Table 1 below, in accordance with Summerset Masterplan 200721-55 Unit Mix Rev 8, received 22 July 2020.

Total demand based on AS/NZS 3500.1:2003

$$Q = 0.03n + 0.4554\sqrt{n}$$
where  $n = 345$  units
$$Q = 0.03(345) + 0.4554\sqrt{345}$$

$$= 18.81 L/s$$

Factored demand based on reduced occupancy

where n = 247.5 equivalent units  $Q = 0.03(247.5) + 0.4554\sqrt{247.5}$ = 14.59 L/s

**Table 1: Village Demands** 

Table 11 Table 2 Communication	Number Units	Reduction Factor	Equivalent Units	Total Demand (L/s)	Demand per Unit (L/s)
Villa/Townhouse	224	0.75	168	9.903	0.0442
Asssisted Living Suites*	56	0.75	42	2.476	0.0442
Memory Care Suites*	20	0.75	15	0.884	0.0442
Resthome & Hospital Beds*	43	0.5	21.5	1.267	0.0295
Maintenance Shed	1	0.5	0.5	0.029	0.0295
Bowling Green	1	0.5	0.5	0.029	0.0295
Total	345		247.5	14.589	

<sup>\*</sup>Portion of main building demand, summing to total main building demand of 4.627L/s

Given irrigation will occur at off peak times, it has been assumed no irrigation demand during peak demand.

### **Firefighting**

Firefighting water supply of FW2 in accordance with SNZ PAS 4509:2008. This requires one hydrant capable of 12.5 L/s at 10m residual head is located within 135 m of every dwelling, and a second hydrant capable of delivering a further 12.5 L/s is located within 270 m of every dwelling. Hydrants have been located in accordance with FW2 requirements.

For the purposes of calculating residual pressures, the peak firefighting design of 25 L/s has been assumed to occur simultaneously with 67% peak demands from the village villas, apartments, main building, and irrigation demand.



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**Potable Water EPAnet Modelling** 

### Layout

Internal reticulation within the site has been modelled in accordance with DWG 190417-20 with connections as follows:

- 1x connection to existing 150Ø public water main at the main entrance (along Springs Road)
- 1x connection to existing 1500 public water main within Glenary Drive

Figure 1: Proposed Water Supply Layout





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**Potable Water EPAnet Modelling** 

### Results:

**Table 2: EPANET results** 

Scenario	Minimum Main Building Pressure (m)	Location	Minimum villa pressure (m)	Location	Minimum residual pressure at FH (m)
Scenario 1					
No Fire Fighting	21.31	Junc n18	21.40	Junc n392	-
FH1 / FH2**	19.16	Junc n18	19.29	Junc n396	19.24/20.89

Scenario 1: No firefighting: Village demand as per Table 1. Glenary Drive operating pressure = 310kPa, Springs Road operating pressure = 310kPa

Scenario 2: Firefighting assumes two hydrants open concurrently (12.5L/s each) + 67% of the peak demand listed in Table 1Glenary Drive operating pressure = 310kPa, Springs Road operating pressure = 310kPa \*\*\* Block refers to numbering from Summerset Plan

The results table shows that in the peak water supply scenario (no firefighting), residual pressures are above the minimum requirement of 20m for residential zones in on-demand water supply areas (Selwyn District Council Code of Practice Part 7: Water Supply, Table 1). The lowest residual pressure of 21.31m occurs at the main building, while the lowest pressure at a villa is 21.40m.

In the firefighting scenario, the minimum residual pressure of 10m (SNZ PAS 4509, 2008 section K3) is met at every location.

Based on the preceding results, Riley recommends that the proposed connections to the public mains in Springs Road and Glenary Drive will have sufficient operating pressures to service the site. However, a booster pump is recommended to service the main building given it's design as a multi-storey building.

The additional connection to the main within Glenary Drive was agreed with SDC and provides additional pressure to the villa units and security in the supply (particularly if the main connection off Springs Road is unavailable).

The following figures present the results from the modelling assuming both proposed connections (Springs Road and Glenary Drive) are operational.



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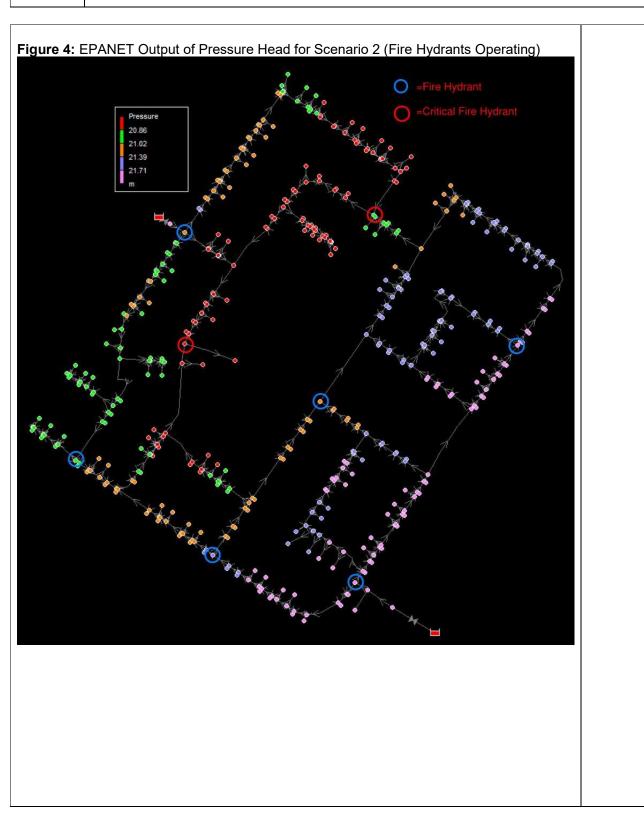
**Potable Water EPAnet Modelling** 





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**Potable Water EPAnet Modelling** 



APPENDIX G

RILEY Dwgs: 190417-10 to -25

# SUMMERSET VILLAGES (PREBBLETON) LTD

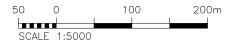
## SUMMERSET PREBBLETON RETIREMENT VILLAGE 578-606 SPRINGS ROAD, PREBBLETON

**RESOURCE CONSENT ISSUE DRAWINGS - AUGUST 2020** 

DRAWING NO.	DRAWING TITLE	REV
190417-10	DRAWING LIST AND LOCALITY PLAN	1
190417-11	EXISTING SITE PLAN	1
190417-12	PROPOSED FINISHED CONTOUR PLAN	1
190417-13	CUT / FILL PLAN	1
190417-14	EARTHWORKS AND SEDIMENT CONTROL PLAN	1
190417-15	SEDIMENT POND TYPICAL SECTION AND DETAILS	1
190417-16	EROSION AND SEDIMENT CONTROL TYPICAL DETAILS	1
190417-17	STORMWATER CATCHMENT & OVERLAND FLOW PATH PLAN	1
190417-18	STORMWATER RETICULATION PLAN	1
190417-19	WASTEWATER RETICULATION PLAN	1
190417-20	POTABLE WATER RETICULATION PLAN	1
190417-21	RAIN GARDEN TYPICAL SECTION	1
190417-22	TREATMENT SWALES TYPICAL SECTIONS	1
190417-23	TREATMENT SWALE OULET TYPICAL SECTION	1
190417-24	SWALE AMENITY AND SOAKAGE MANHOLE DETAIL	1
190417-25	ARCH SOAKAGE CHAMBER DETAILS	1



### LOCALITY PLAN SCALE 1:5000



### **GENERAL NOTES:**

- 1. ALL DRAWINGS AND SPECIFICATIONS TO BE READ IN CONJUNCTION WITH THE ARCHITECT, LANDSCAPING, SERVICES PROVIDERS, STRUCTURAL AND HYDRAULIC ENGINEER'S DOCUMENTATION.
- 2. ALL EARTHWORKS, INCLUDING TRENCHING FOR SERVICES, ARE TO BE FULLY SUPPORTED.
- 3. DO NOT SCALE OR DIMENSION FROM THESE DRAWINGS.
- 4. TOPOGRAPHICAL SURVEY PROVIDED BY GRAHAM SURVEYING LTD.
- 5. LAYOUT MASTERPLAN PROVIDED BY SUMMERSET GROUP HOLDING LTD.
- 6. LEVELS ARE IN TERMS OF CHRISTCHURCH DRAINAGE DATUM (CDD).
- 7. ALL WORK & MATERIALS TO BE IN ACCORDANCE WITH COUNCIL STANDARDS & SPECIFICATIONS.
- 8. ALL FINISHED LEVELS SHALL BE FORMED TO THE ARCHITECT'S DESIGNS
- 9. THE CONTRACTOR IS TO LOCATE & PROTECT ALL EXISTING SERVICES, INCLUDING POWER, TELECOM, GAS, WATER, STORMWATER & WASTEWATER BEFORE COMMENCING WORKS. LIAISE WITH RESPECTIVE SERVICE AUTHORITIES FOR ASSISTANCE.
- 10. ANY DISCREPANCIES BETWEEN THE DRAWINGS & SITE CONDITIONS ARE TO BE NOTIFIED TO THE ENGINEER IMMEDIATELY.
- 11. ALL NEW PIPEWORK SHALL BE PROTECTED AGAINST DAMAGE DURING CONSTRUCTION.
- 12. ALL WORKS TO BE SET OUT ON SITE PRIOR TO CONSTRUCTION. ANY DISCREPANCIES ARE TO BE REPORTED TO THE ENGINEER PRIOR TO COMMENCEMENT.
- 13. CONTRACTOR TO GIVE NOTICE TO THE ENGINEER PRIOR TO CONCEALING WORK.
- 14. ALL WORKS SHALL COMPLY WITH THE HEALTH & SAFETY PLAN SUBMITTED BY THE CONTRACTOR.

### **SERVICES NOTES:**

- 1. REGISTERED DRAINLAYERS ONLY TO UNDERTAKE THE WORKS.
- 2. ALL PRIVATE DRAINAGE TO BE IN ACCORDANCE WITH THE NZBC RELEVANT SECTIONS.
- 2. ALL PUBLIC DRAINAGE TO BE IN ACCORDANCE WITH THE SELWYN DISTRICT COUNCIL CODE OF PRACTICE RELEVANT SECTIONS.
- ALL STORMWATER AND WASTEWATER PIPES ARE TO BE PVC-U SN16 U.N.O. REFER TO STORMWATER AND WASTEWATER RETICULATION LAYOUT FOR PIPE SIZES. ALL PVC PIPE SIZES NOTED ARE MINIMUM INTERNAL DIAMETERS (ID) U.N.O.
- 4. ALL WATERMAINS ARE PE100 SDR17 (PN16) U.N.O. REFER TO WATER SUPPLY RETICULATION LAYOUT FOR WATER SUPPLY SIZES. ALL PE WATERMAIN PIPE DIMENSIONS ARE OUTSIDE DIAMETERS
- 5. PRIVATE STORMWATER LEADS FROM SUMPS TO BE 150mmø @ 1:100 MINIMUM GRADE U.N.O.
- 6. WASTEWATER HOUSE CONNECTIONS TO BE 100mmø PVC-U SN16 @ 1:60 MINIMUM GRADE U.N.O.
- 7. STORMWATER HOUSE CONNECTIONS TO BE 150mmø PVC-U SN16 @ 1:100 MINIMUM GRADE U.N.O.
- 8. ANY DISCREPANCIES BETWEEN DRAWINGS AND SITE CONDITIONS ARE TO BE REPORTED TO THE ENGINEER IMMEDIATELY

### FOR CONSENT

				DESIGN	DES CHECK	APPROVED FOR ISSUE
				SB	SJ	
				DRAWN	CAD CHECK	S. JAMES
				FY	RT	
1	23.09.20	CONSENT ISSUE	SB	DATE DRAWN		ISSUE DATE
REV	DATE	ISSUE	BY	15.06.2	2020	23 / 9 / 20





CLIENT	SUMMERSET VILLAGES (PREBBLETON) LTD
ADDRESS	578-606 SPRINGS ROAD, PREBBLETON
PROJECT	SUMMERSET PREBBLETON RETIREMENT VILLAGE
SHEET TITLE	DRAWING LIST AND LOCALITY PLAN

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ISSUE DATE 23 / 9 / 20

SB DATE DRAWN

BY 15.06.2020

1 23.09.20 CONSENT ISSUE

REV DATE ISSUE

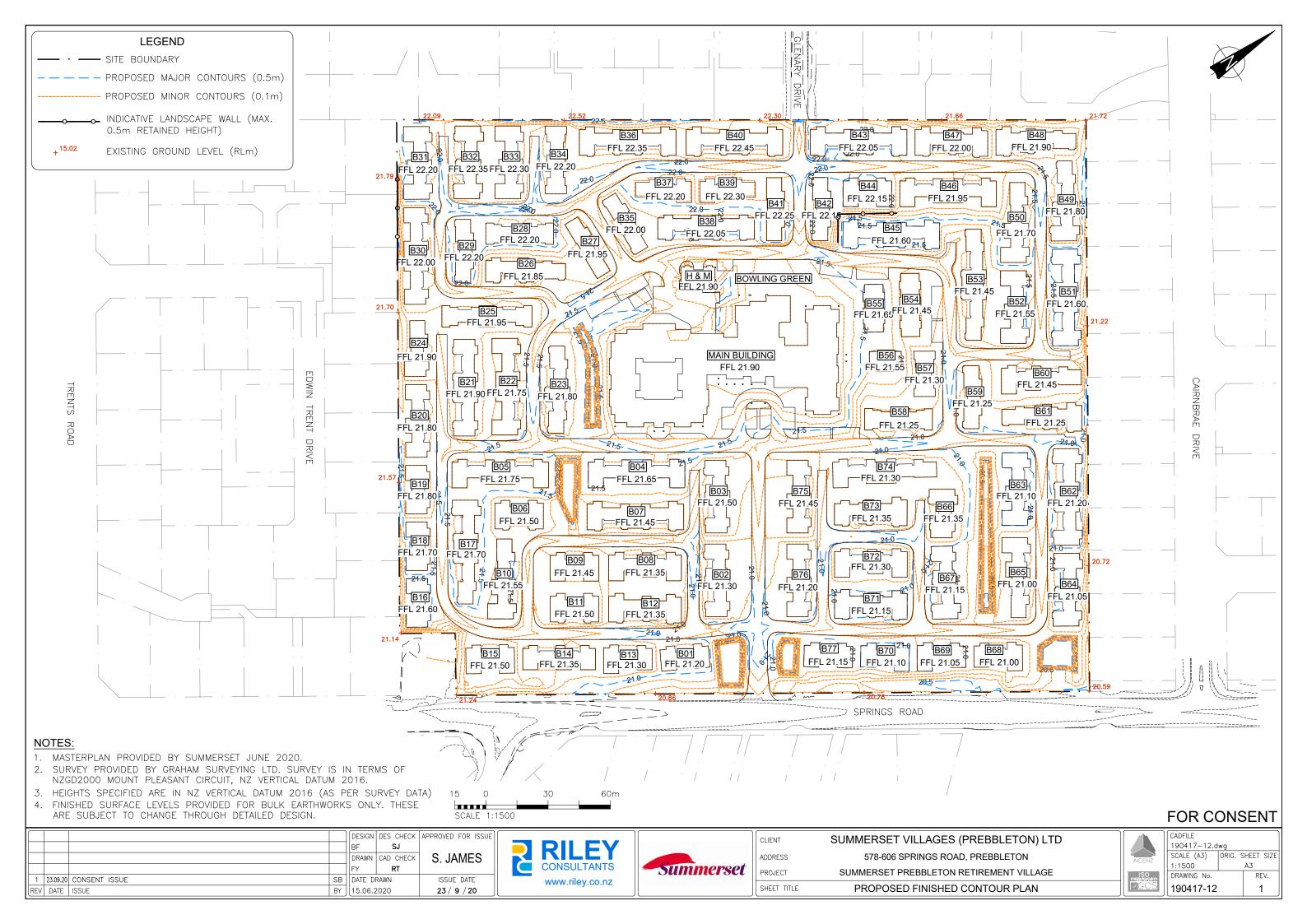
CONSULTANTS www.riley.co.nz

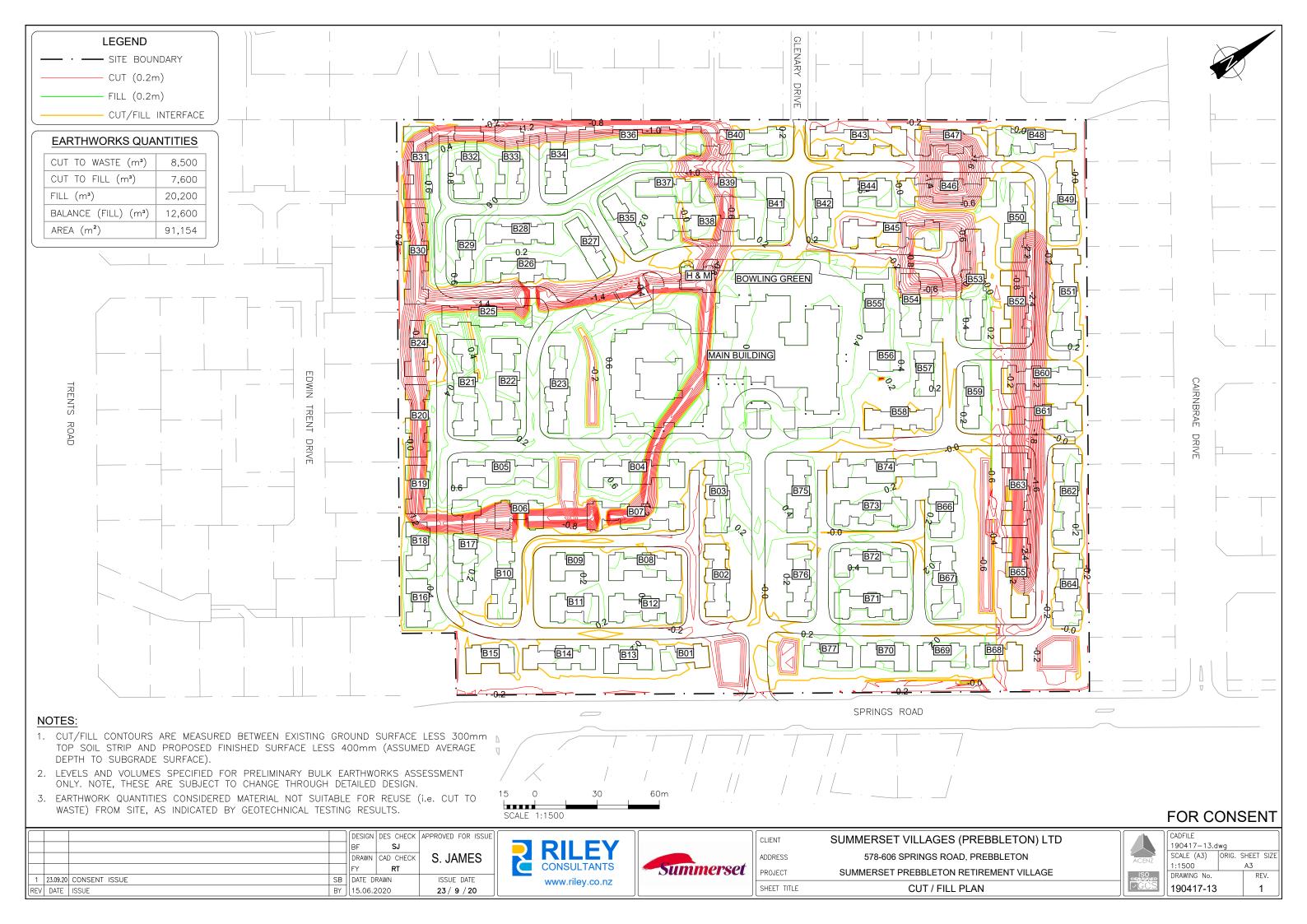


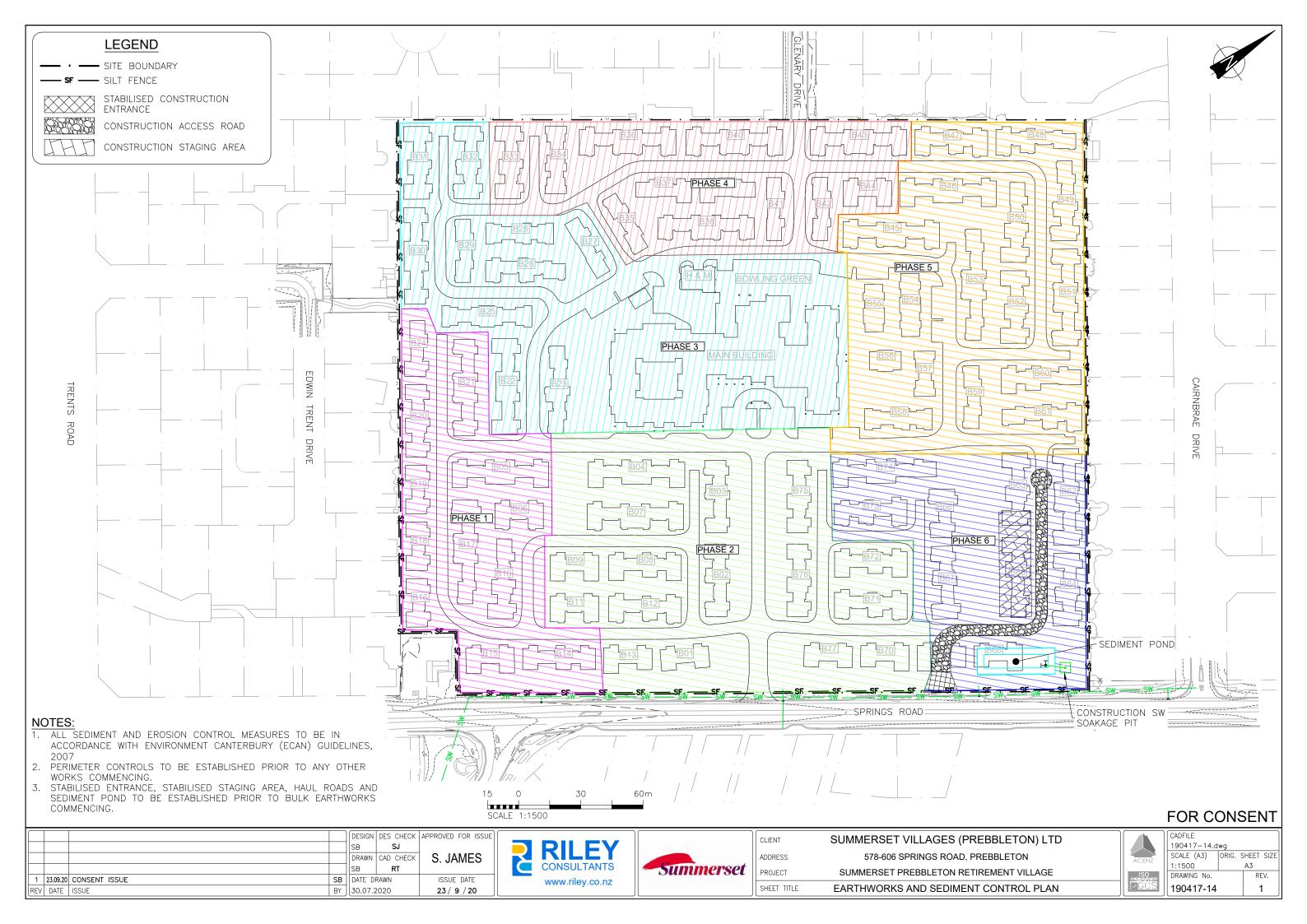
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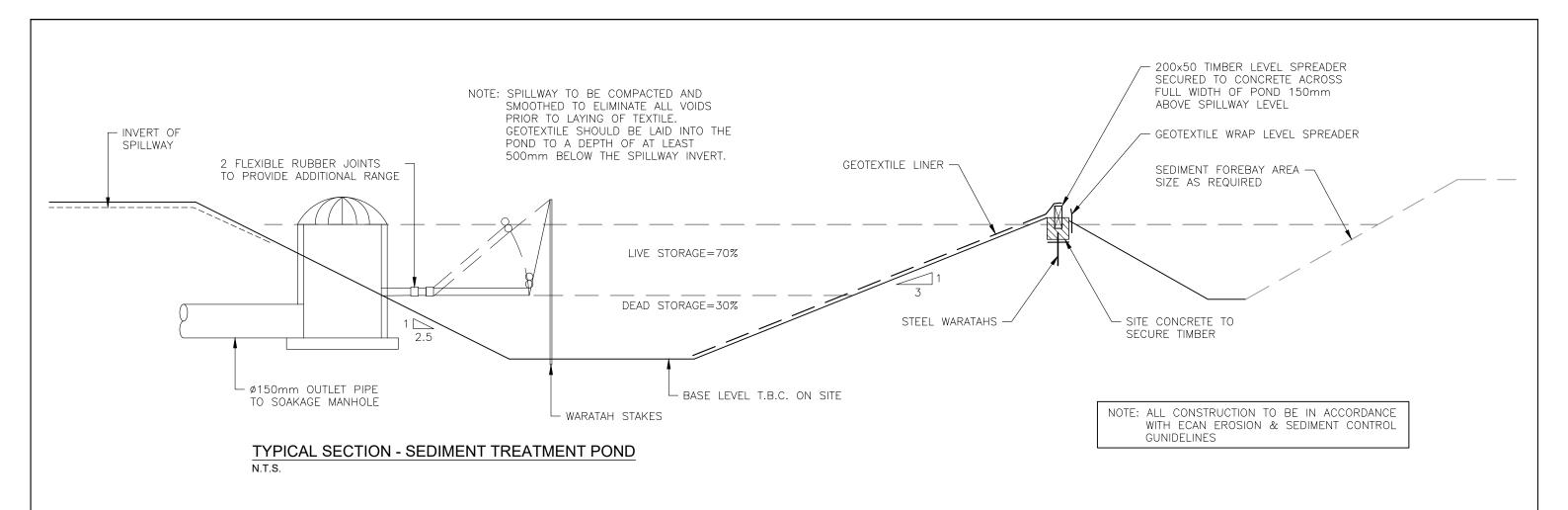
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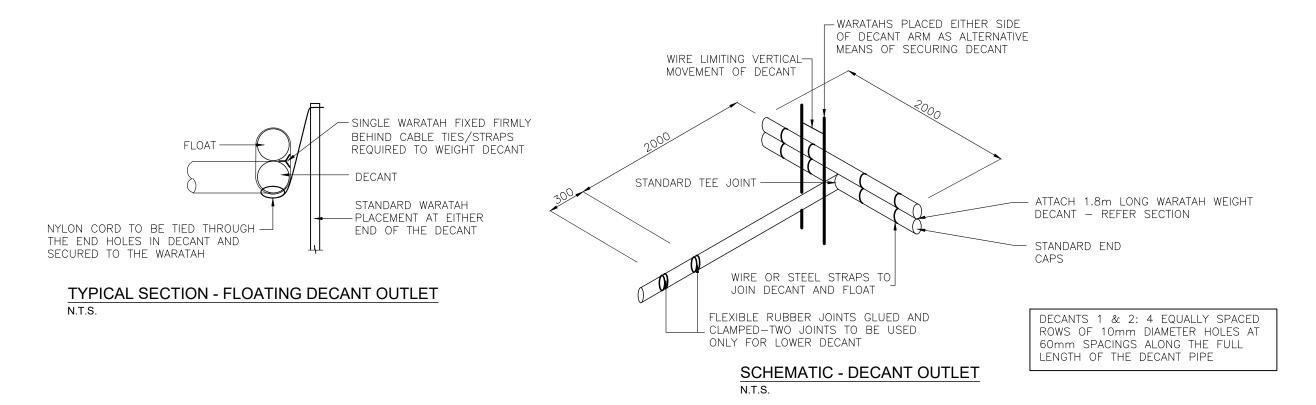
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### FOR CONSENT

ORIG. SHEET SIZE

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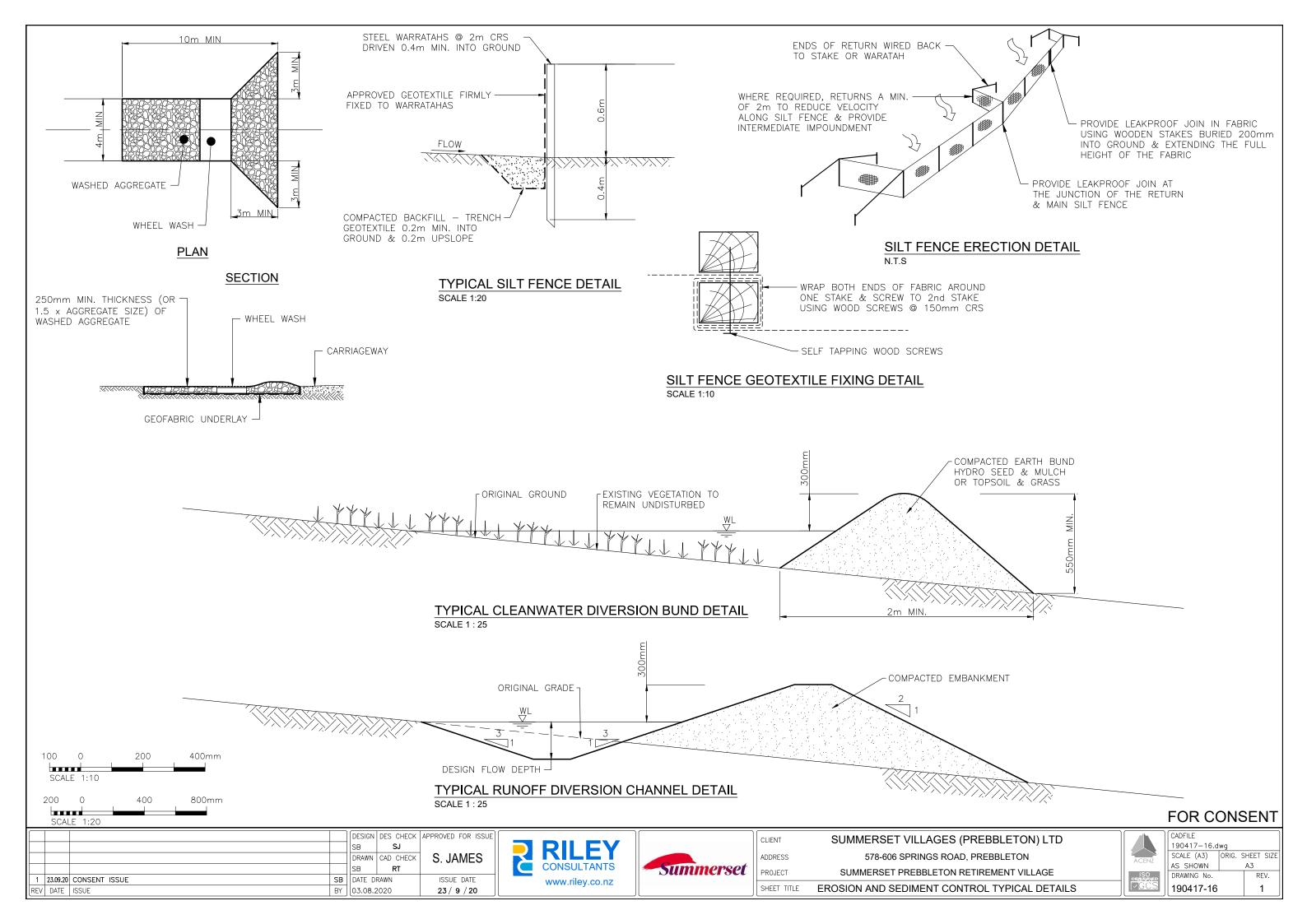
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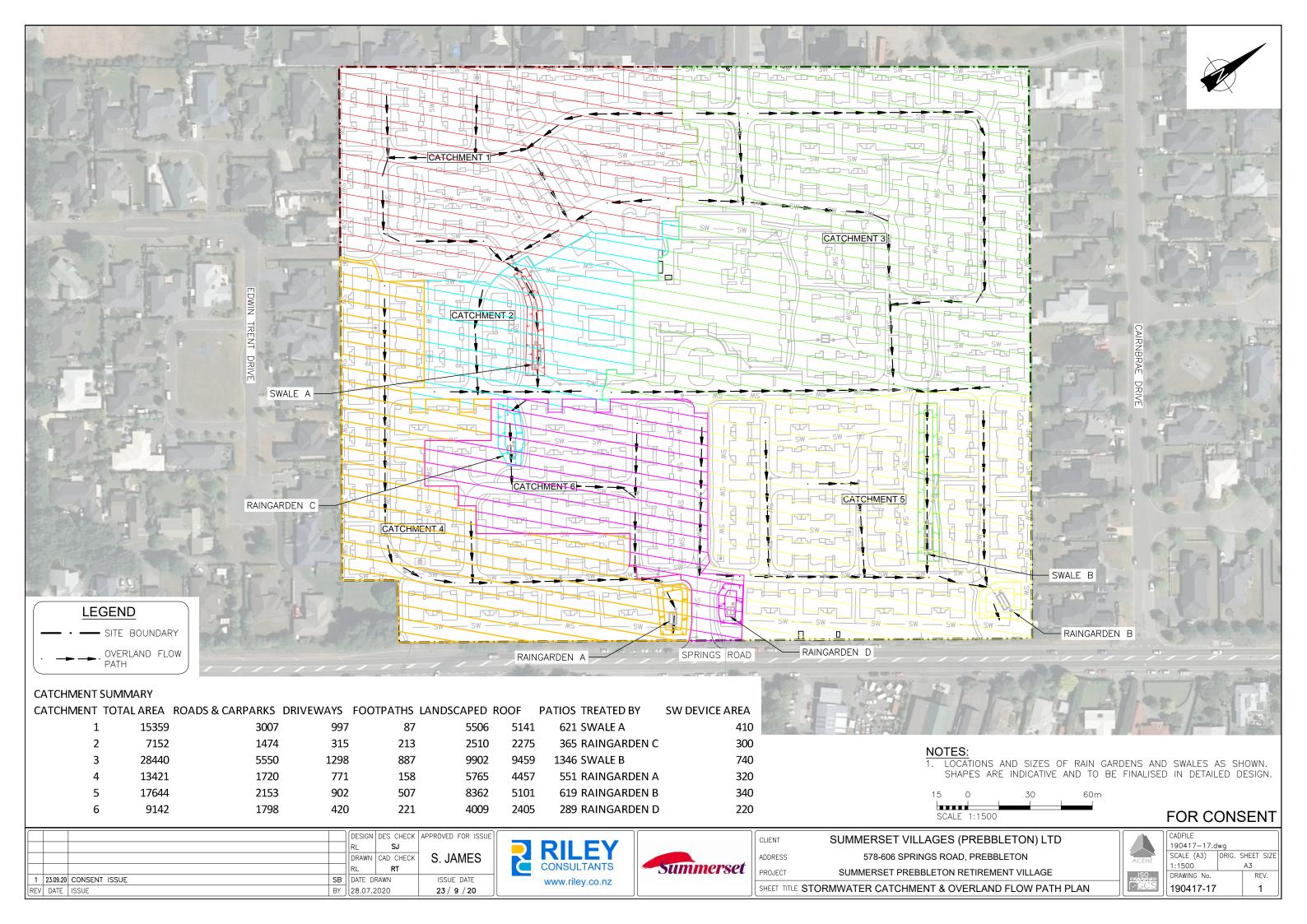
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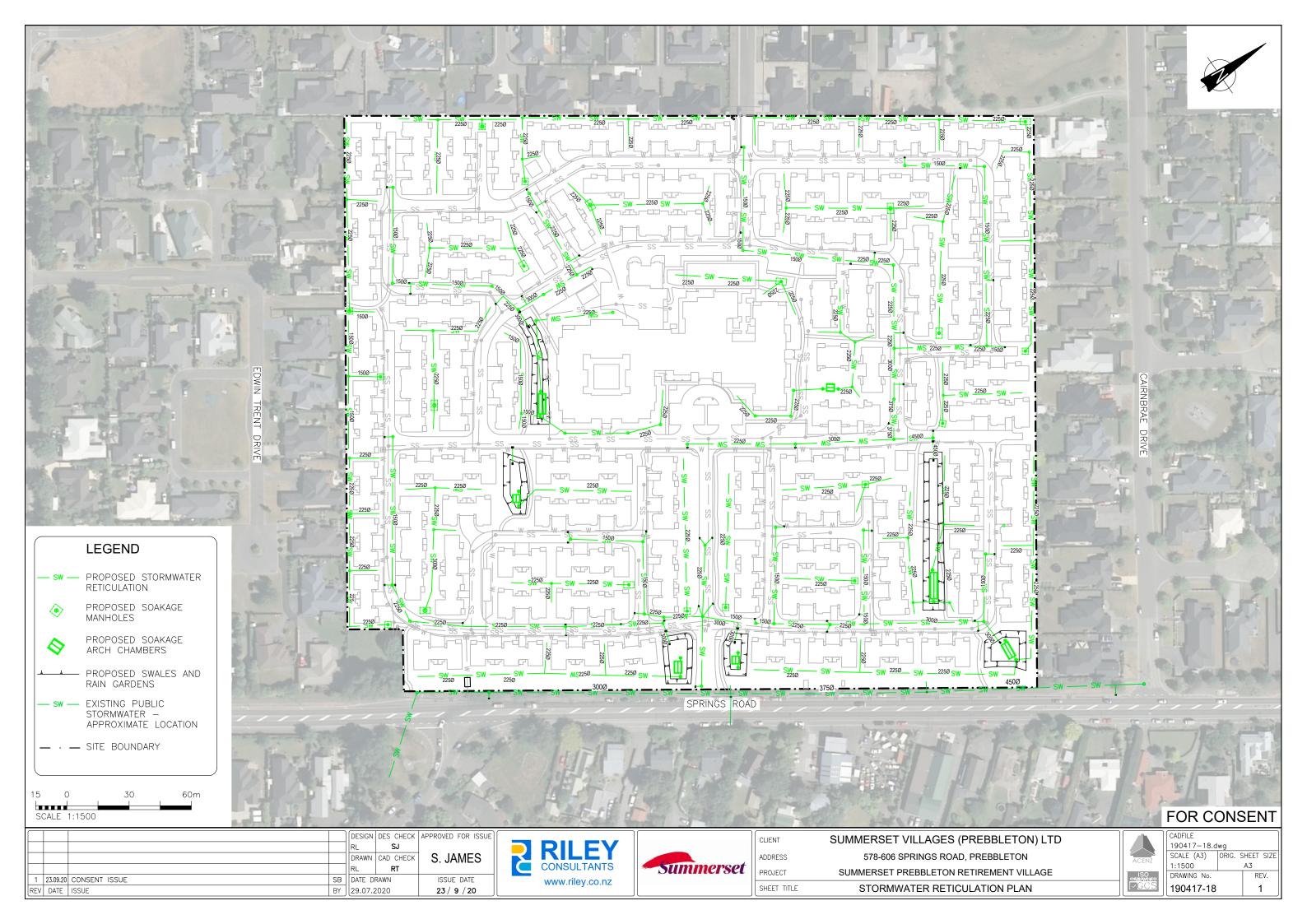
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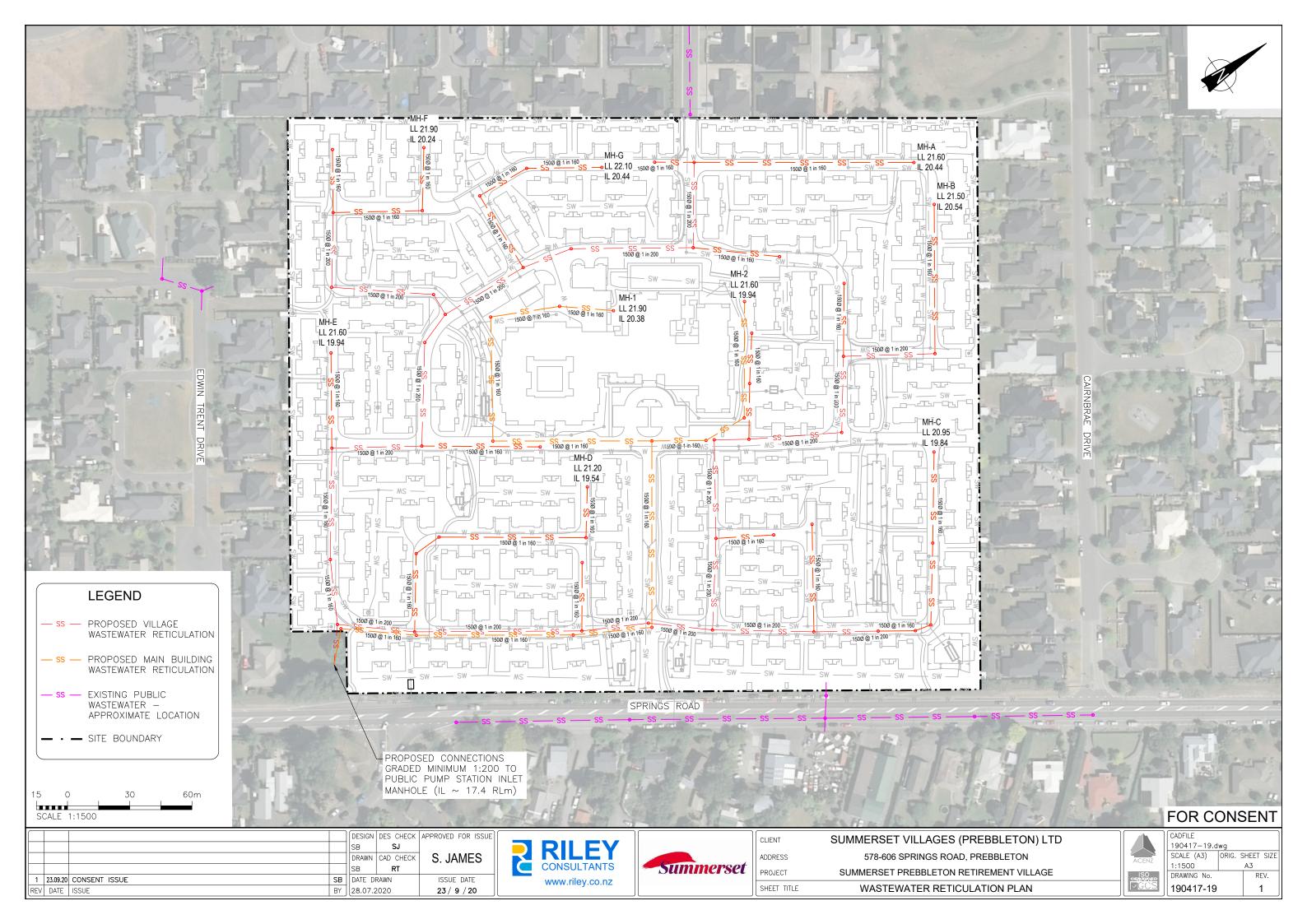
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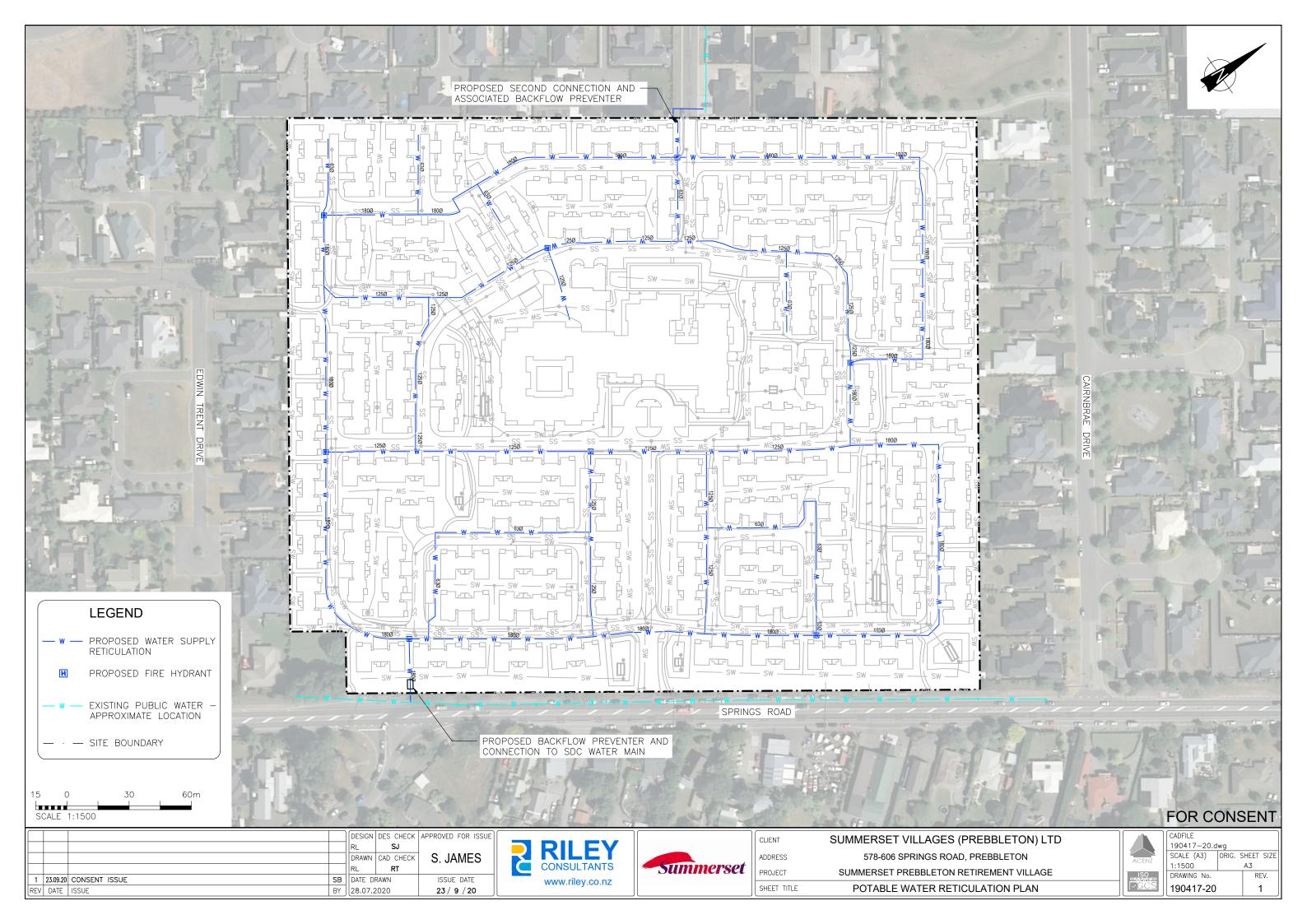
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1 27.00.20	CONSENT ISSUE	CD	SB DATE D	RT	ISSUE DATE	CONSULT	CONSULTANTS	Summerset	PROJECT	SUMMERSET PREBBLETON RETIREMENT VILLAGE
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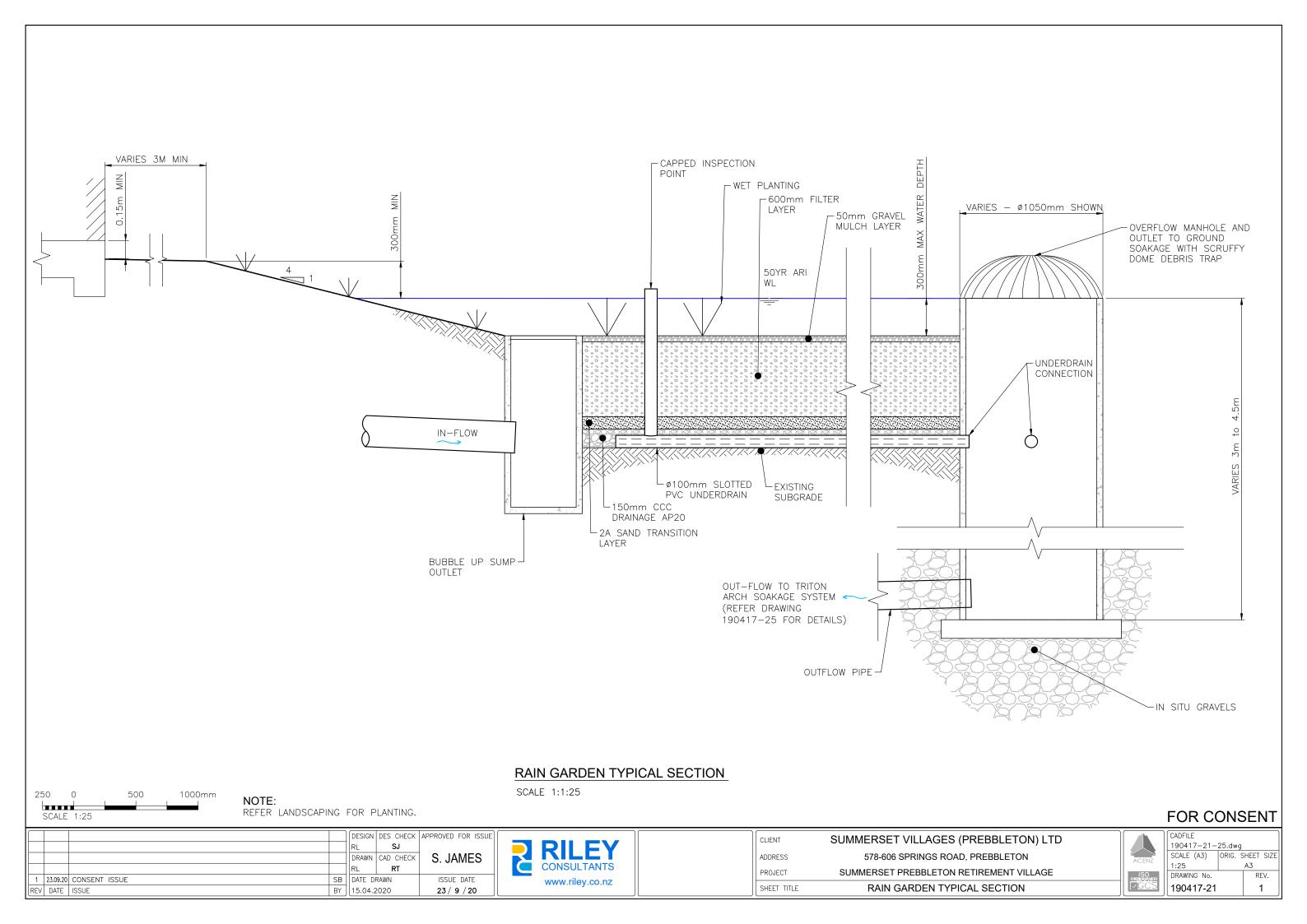


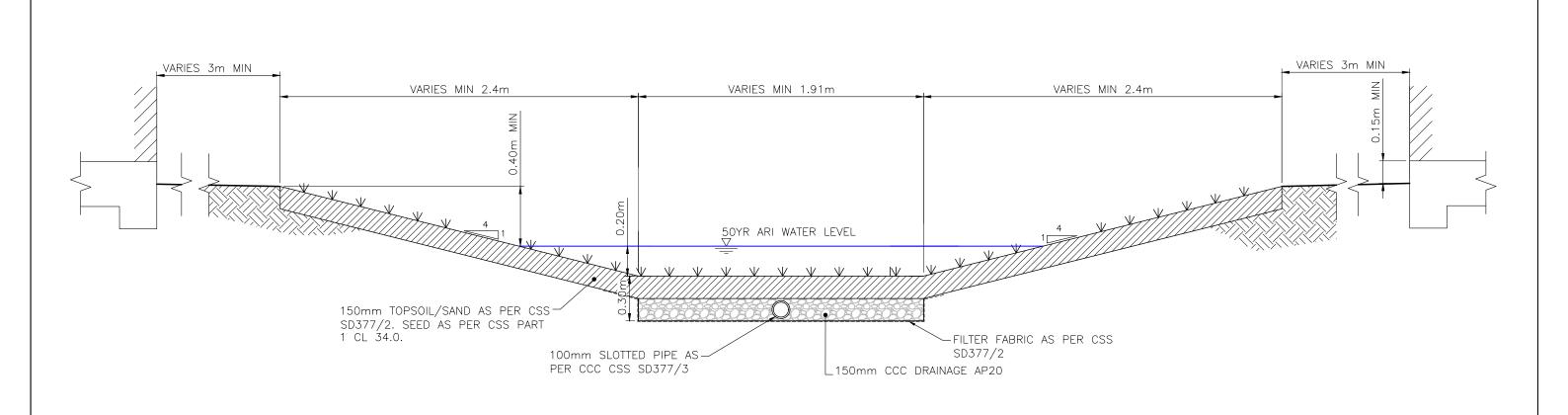






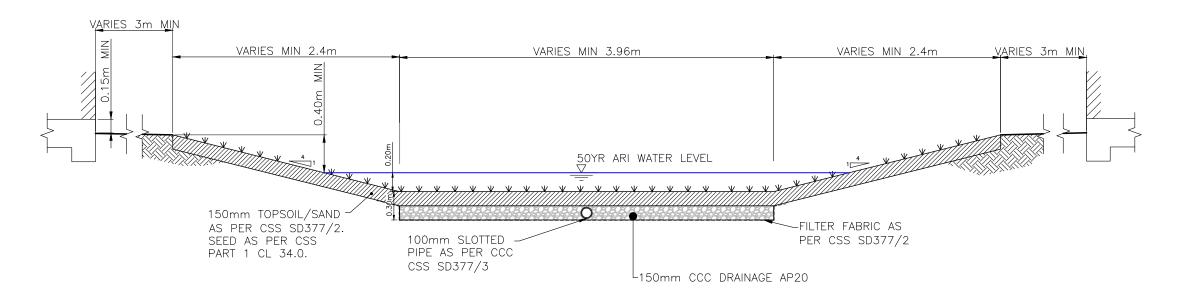






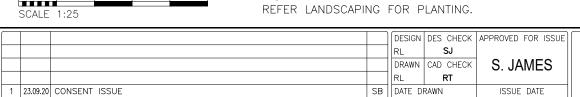
### TREATMENT SWALE A TYPICAL SECTION

SCALE 1:1:25



### TREATMENT SWALE B TYPICAL SECTION

SCALE 1:1:25



BY 15.04.2020

23 / 9 / 20

NOTE:

1000mm

250

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REV DATE ISSUE

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PROJECT	SUMMERSET PREBBLETON RETIREMENT VILLAGE	Ш
SHEET TITLE	TREATMENT SWALES TYPICAL SECTIONS	

FOR CONSENT					
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