

Fast Track Application – Stormwater Management for 531 & 535 Mill Road, Ōhoka

✦ Prepared for

Carter Group Ltd

✦ September 2026



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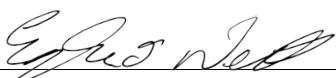
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Limitations:

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Executive Summary

This report presents the proposed stormwater management approach for the Fast Track Application for 531 and 535 Mill Road, Ōhoka. The approximately 155 ha site is currently rural land and is proposed to be developed for a mix of residential housing, rural-residential housing, business/commercial activity, a polo field, and a potential retirement village area. The assessment considers the site's existing drainage network, shallow groundwater conditions, spring-fed waterways, upstream contributing catchments, and downstream receiving environments. The site is located within a low-gradient part of the Waimakariri District where stormwater and groundwater interactions are important. Existing drainage is largely provided by land drains, springs, natural streams and culverts beneath Bradleys Road and Whites Road. Flood modelling undertaken separately for the wider proposal has informed this assessment, including the need to maintain hydraulic continuity through the site and manage flows between the upstream and downstream catchments.

This report supersedes the previous version issued in June 2026 and has been substantially updated in response to feedback from Environment Canterbury and Waimakariri District Council. The key revisions include adoption of the Waimakariri District Council Engineering Code of Practice 2025, revised primary, secondary and attenuation design criteria, incorporation of the updated development layout and catchment extents, recalculation of pre- and post-development runoff and attenuation performance across a wider range of storm durations and design events, and improved integration with the supporting flood modelling, including assessment of hydraulic continuity, tailwater effects and the proposed conveyance network. The stormwater treatment strategy, groundwater constraints, tables, figures, appendices and executive summary have also been updated to reflect the revised design and supporting analyses.

The proposed stormwater management strategy has been developed to manage both stormwater quantity and stormwater quality. Quantity control is to be achieved by retaining and formalising key flow paths through the site, maintaining connectivity between upstream and downstream drainage networks, and providing attenuation through stormwater management areas, basins, wetlands and conveyance corridors. The system has been assessed to attenuate post-development peak flows so that they do not exceed pre-development flows for the relevant design events, including climate change allowance.

Stormwater quality treatment will be provided using a treatment train tailored to the proposed land uses and available stormwater management areas. Rural-residential areas are generally proposed to be treated by swales before discharge to attenuation basins. Residential areas will be treated by a combination of first flush wet ponds, wetlands, swales and rain gardens, depending on available

space and depth to groundwater. Business/commercial runoff is proposed to be treated using proprietary filtration devices.

The stormwater design has been informed by relevant regional and district guidance, including the Canterbury Land and Water Regional Plan, the Waimakariri District Council Engineering Code of Practice, and the Christchurch City Council Waterways, Wetlands and Drainage Guide. Shallow groundwater has been recognised as a key design constraint and has influenced the selection and sizing of proposed treatment and attenuation devices.

Overall, the proposed stormwater management approach is considered appropriate for the scale and nature of the development. It provides for treatment of runoff from proposed impervious areas, manages peak discharge rates to the downstream environment, maintains hydraulic continuity through the site, and supports integration with the wider flood management strategy for the development.

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1.0 Introduction

Pattle Delamore Partners Ltd (PDP) has been engaged by Carter Group Limited (CGL) (the Client) to report on the stormwater management for the proposed development at 531 & 535 Mill Road, Ōhoka (the site).

The site is located to the southwest of Ōhoka township, within the Waimakariri District, and is bordered by Bradleys Road and Whites Road. The current site consists of approximately 155 ha of rural land. The proposed development will consist of residential housing, rural-residential housing, business/commercial areas, a polo field, and a potential retirement village area.

The site location is indicated in Figure 1 below.

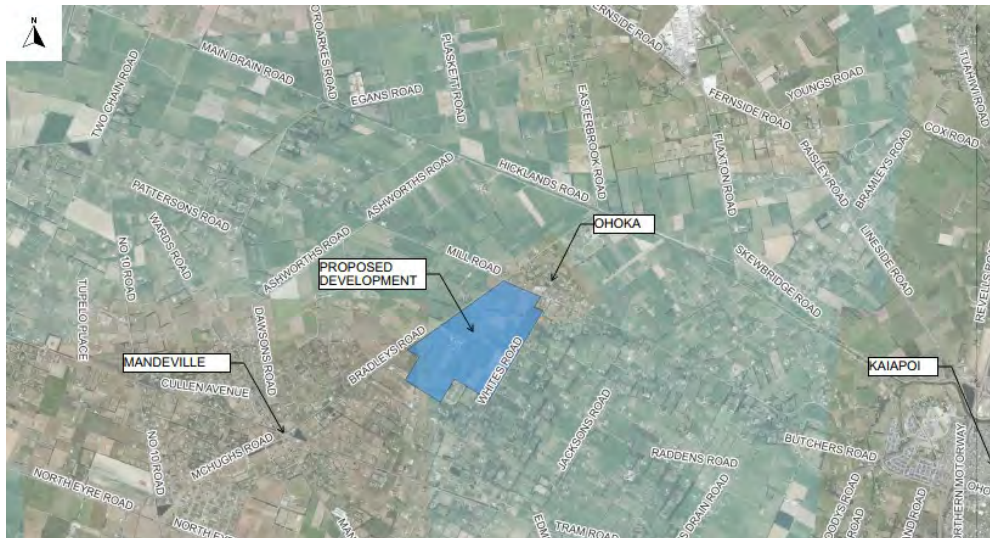


Figure 1: Location of proposed development.

This report has been updated since the previous issue in June 2026, in response to technical feedback from Environment Canterbury and Waimakariri District Council. This version of the report supersedes the previous version, and is intended to be a complete document summarising the modelling done to date as part of the Fast Track application.

The purpose of this report is to:

- ✧ An assessment of the anticipated stormwater effects of the proposed development at the site, including:
 - Stormwater runoff quality.
 - Stormwater runoff quantity.

- ✧ Summarise the stormwater management, consisting of stormwater treatment and flood attenuation systems, for the proposed development at 531 & 535 Mill Road, Ōhoka.

2.0 Existing Site

2.1 Overview of site

The proposed site is zoned as rural and is approximately 155 ha in area. The existing land use on-site consists of:

- ✧ Large undeveloped paddock areas.
- ✧ Land drainage, springs, and natural streams (e.g., Ōhoka Stream).

Impervious areas are limited to:

- ✧ Unsealed roads; and,
- ✧ Buildings and sealed areas (< 1%).

The general fall across the site is west to east and elevation ranges between RL 29 m to RL 20 m (NZVD2016). The average slope across the site is approximately 0.5% (1V:200H).

2.2 Site Geology & Soils

The site is located on the northern Canterbury Plains (i.e., the Waimakariri–Ashley Plains). The GNS geological map of the area describes the near-surface geology of the site as late Pleistocene brownish-grey river alluvium (Q2a). Previous geotechnical investigations undertaken at the site had encountered silt and clayey silt to a depth of 0.6 to 1.5 m below ground level (bgl), and sandy gravel below this.

A review of soil information available on S-map (Landcare Research) indicates that the soil types underlying the site consist of Ayreburn moderately deep clay. A small section on the northern and southern part of the site consists of Ayreburn deep clay and Leeston shallow clay, respectively.

The soils determine the rate at which rainfall and associated stormwater infiltrate into the ground. S-maps indicate that the underlying Ayreburn (moderately deep and deep) clay and the Leeston shallow clay soils are poorly drained.

2.3 Hydrogeology and Groundwater

On the Waimakariri-Ashley Plains, groundwater is dominantly sourced from infiltrating rainwater (i.e., land surface recharge) across the inland plains (to the north-west and upgradient of the site), together with some seepage losses from the Ashley River and Waimakariri River. The groundwater flows to the southeast,

towards the coast. Groundwater discharges into spring-fed streams, including the Ōhoka Stream and the Cam River/Ruataniwha.

PDP has also completed a hydrology assessment, separate from this report. Refer to *PDP, Fast Track Application – 531 & 535 Mill Road, Ōhoka – Hydrology Assessment (June 2026)* report for specific details.

A summary is provided below:

2.3.1 Groundwater levels

Groundwater at the site varies spatially and seasonally but is generally shallow (< 1 m). This is as informed by the recent installation of piezometers at the site:

- ✧ 37 piezometers were installed by Tetra Tech Coffey in 2023. These showed groundwater level to vary from approximately 0.05 – 1 m bgl.
- ✧ 23 new piezometers were installed by McMillan Drilling in March 2025. These reported groundwater levels vary from approximately 0.01 – 2.28 m bgl.

Long-term groundwater table fluctuations were obtained from an existing bore (M35/0596) which is located in the western-central part of the site. The record from M35/0596 showed that the groundwater levels at this site are generally shallow, with an average value of 0.73 m bgl. The highest recorded groundwater level was 0.22 m bgl (March 2023) and the lower recorded levels were 1.5 m bgl in July 1999 and 1.48 m bgl in February 2017. Seasonal fluctuations are relatively small, commonly being 0.5 – 0.8 m. As expected, groundwater levels are generally highest in winter/spring and lowest in summer/autumn. It is noted that bore M35/0596 is close to spring M35/7485 (mapped location is 20 m away), and so may be in an area of the site that has particularly high groundwater levels.

A general observation was that piezometers near springs or streams consistently had the shallowest groundwater levels.

2.3.2 Springs

Two documented springs (ECan reference M35/7485 and M35/7487) are located on the site.

- ✧ M35/7485 is near the centre of the western site boundary and is described in the ECan database as artesian, permanent, and having a channel or linear morphology.
- ✧ M35/7487 is near the centre of the site and is described in the ECan database as of undetermined type, permanent, and having a channel or linear morphology.

The occurrence of heavy soils and spring-fed waterways indicates that this site is located within a groundwater discharge zone, i.e. groundwater is discharging into surface waterways.

2.4 Existing Drainage & Stormwater

The existing site drainage and stormwater infrastructure consists of the following:

✧ *Upstream Catchments (Bradleys Road):*

The upstream contributing catchments have been described and assessed as part of the flood risk assessment undertaken by PDP. Refer to the report "Flood Effects - 531 & 535 Mill Road, Ōhoka, September 2026" (report C045180001R001) for further details regarding upstream catchment characteristics and extents.

Two culverts under Bradleys Road convey the majority of the upstream contributing catchment flows into the site. During significant storm events, runoff from the upstream contributing catchment breaches over Bradleys Road and discharges onto the proposed development area.

A portion of the upstream contributing catchment flows enters the site via overland flow from areas to the south of the site.

✧ *Site*

The site has limited formal stormwater infrastructure. Existing land drains collect stormwater and high groundwater from the site as well as convey upstream contributing catchment flows, directing flow downstream towards Whites Road.

✧ *Downstream Catchments (Whites Road):*

Three culverts under Whites Road convey the majority of the site flow through to the downstream environment (entering either Ōhoka tributary, South Ōhoka, or unnamed drain 1). A portion of the site flow enters into the Mill Road drain at the north of the site.

Figure 2 below shows indicatively the existing drainage and stormwater infrastructure at the site.



Figure 2: Existing Stormwater infrastructure.

2.5 Discharge Environment

2.5.1 Surface Water

The downstream catchment has comparable properties to the pre-development site. The downstream catchment is a mix of undeveloped rural land (e.g., paddocks) and lifestyle blocks with land drains collecting runoff. Again, it is expected that these likely intersect the shallow groundwater table. Downstream flow drains towards the Ōhoka Stream and eventually to the Kaiapoi River.

2.5.2 Groundwater

The site is in the Eyre Groundwater Allocation Zone, which has an allocation limit of 99,070,000 m³ per year specified in Section 8.7.2 of the LWRP.

As of a 13 April 2026 letter from ECAN, this Allocation Zone is currently 95.95% allocated. The boundary with the Cust Groundwater Allocation Zone is adjacent to the site, along Mill Road. The Cust Allocation Zone is currently 82% allocated.

2.6 Flood risk

Flood modelling has been described in a separate report¹ to assess the pre-and post- development flooding for the 2% AEP, 1% AEP and 0.5% AEP events. The 1% AEP existing flood depths are shown in Figure 3 below, which has been sourced from the PDP flood model. The 1% AEP figure is shown because it aligns with the stormwater design/secondary flow criterion, while the 0.5% AEP event is addressed in the separate flood modelling report.

The site and surrounding area are subject to widespread flooding with peaks at relatively shallow depth. There are several notable “watercourses” where runoff concentrates in flowing generally from west to east. The flood modelling results and conclusions have been used to support the stormwater calculations and analysis, particularly around the attenuation performance, and is referenced several times in this report.

¹ Flood Effects – 531 and 535 Mill Road, Ohoka, prepared for Carter Group Limited dated 6 September 2026.

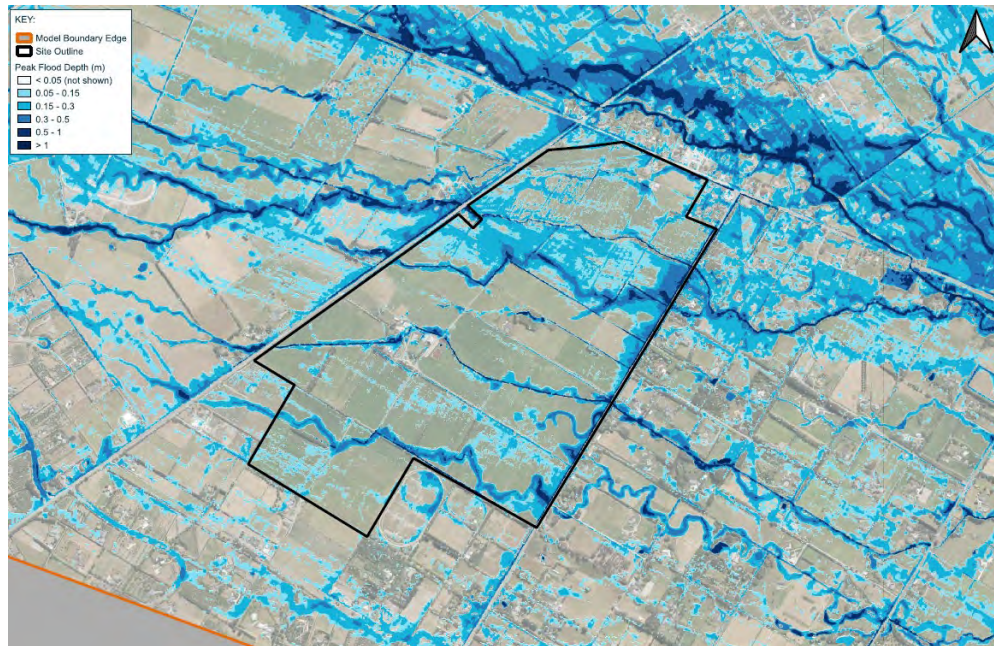


Figure 3: 100-year flood event localised flood depth.

3.0 Proposed Development

The development will convert the existing area into a mix of residential housing, rural-residential housing, business/commercial areas, a polo field, and a potential retirement village area. The proposed mix of land use types are detailed in Table 1, refer to Appendix A for the current proposed development layout.

Table 1: Proposed land-use types and areas	
Description	Total area (ha)
Residential	77.6
Rural-residential	38.7
Retirement village	7.4
Business/Commercial	1.3
Drainage corridors and stormwater management areas	21
Polo Field	8.7
Total	154.7

4.0 Design Considerations

4.1 Design Standards

The following documents and guidelines were used to inform the site stormwater design:

- ✧ Waimakariri District Council Engineering Code of Practice (WDC, July 2025) (WDC CoP).
- ✧ Christchurch City Council Waterways, Wetlands and Drainage Guide (CCC, February 2013) (WWDG).
- ✧ Environment Canterbury Regional Council, Canterbury Land and Water Regional Plan (ECan, December 2018) (CLWRP).
- ✧ Auckland City Council, Stormwater Management devices in the Auckland Region (AC, December 2017) (AC GD01)

Table 2 below states the key design criteria used as the basis for the assessment of the stormwater effects.

Table 2: Design Criteria		
Item	Design Criteria	References
Rainfall	✧ HIRDS V4 RCP 8.5 (2081 – 2100)	WDC CoP 2025
Primary flows	✧ 10-year return event (10% AEP)	WDC CoP 2025
Secondary flows	✧ 100-year return event (1% AEP)	WDC CoP 2025
Attenuation requirement	✧ Post-development peak flows for all intensities to be equal to or less than pre-development flows for up to a 100-year return event (1% AEP)	WDC CoP 2025
Runoff coefficients	✧ As per Table 5.2 & Table 5.3 of WDC Engineering Code of Practice 2025.	WDC CoP 2025
First Flush ¹ Depth	✧ 25 mm for greenfield developments	CCC WWDG Section 6
Water Quality Flow ²	✧ 5 mm/hr	CCC Onsite Stormwater Mitigation Guide
Notes: 1. First flush refers to the phenomenon where the initial runoff from a rainstorm contains a disproportionately high concentration of pollutants compared to the rest of the storm event. The capture of this depth for treatment is a common approach for stormwater design. 2. Water quality flow is the equivalent rainfall intensity to the first flush depth, used to design flow-based devices.		

4.2 Regional Plan Requirements

The following requirements apply to the stormwater management for the proposed development, as per the Canterbury Land and Water Regional Plan (ECan, Dec 2018).

Rule 5.93 The discharge of stormwater or construction-phase stormwater from a reticulated stormwater system onto or into land or into or onto land in circumstances where a contaminant may enter the water, or into groundwater or a surface waterbody is a restricted discretionary activity, provided the following conditions are met:

1. *For a discharge that existed at 11 August 2012, an application for a discharge permit is lodged prior to 30 June 2018, or at a later date as agreed between the reticulated stormwater system operator and the CRC; and*
2. *A stormwater management plan has been prepared to address the management of stormwater in the catchment and is lodged with the application; and*
3. *The discharge will not cause a limit in Schedule 8 to be exceeded.*

The exercise of discretion is restricted to the following matters:

1. *The quality of, compliance with and monitoring of the stormwater management plan prepared to address the management of stormwater in the catchment and matters set out in guidance documents prepared by the CRC; and*
2. *The rate and volume of discharge and the changes to the flow regime of a river or artificial watercourse, flood frequency, including flooding of land or dwellings, erosion of river bank and channels; and*
3. *The concentration of contaminants and resulting actual and potential adverse environmental effects, including cumulative effects on the receiving water quality of surface and groundwater, aquatic ecosystems, Ngāi Tahu cultural values and other existing uses and users of the water, including takes and discharges; and*
4. *Measures to:*
 - (a) reduce the volume and concentration of contaminants in the discharge; and*
 - (b) ensure the volume and rate of discharge do not exceed:*
 - (i) the capability of the soil and subsoil layers at the site to reduce contaminant concentrations in the discharge; and*

- (ii) the infiltration capacity of the soil and subsoil layers at the site; and
- (c) avoid the accumulation of toxic or persistent contaminants in the soil or subsoil layers; and
- (d) minimise suspended sediment in stormwater from activities involving earthworks; and
- 5. The potential benefits of the activity to the applicant, the community and the environment; and
- 6. The need for measures to protect any human or animal drinking-water sources.

Rule 5.95 The discharge of stormwater, other than into or from a reticulated stormwater system, into a river, lake, wetland or artificial watercourse or onto or into land in circumstances where a contaminant may enter a river, lake, wetland, or artificial watercourse is a permitted activity, provided the following conditions are met:

- 1. The discharge is not from, into or onto contaminated or potentially contaminated land; and
- 2. The discharge is not into: (a) a water race, as defined in Section 5 of the Local Government Act 2002; and (b) a wetland, unless the wetland is part of a lawfully established stormwater or wastewater treatment system; and (c) a waterbody that is Natural State, unless the discharge was lawfully established before 1 November 2013; and
- 3. The discharge does not result in an increase in the flow in the receiving waterbody at the point of discharge of more than 1% of a flood event with an Annual Exceedance Probability of 20% (one in five year event); and
- 4. The discharge meets the water quality standards in Schedule 5 after reasonable mixing with the receiving waters, in accordance with Schedule 5; and
- 5. The concentration of total suspended solids in the discharge shall not exceed: (a) 50 g/m³, where the discharge is to any spring-fed river, Banks Peninsula river, or to a lake except when the background total suspended solids in the waterbody is greater than 50 g/m³ in which case the Schedule 5 visual clarity standards shall apply; or Canterbury Land and Water Regional Plan Page | 122 (b) 100 g/m³ where the discharge is to any other river or to an artificial watercourse except when the background total suspended solids in the waterbody is greater than 100 g/m³ in which case the Schedule 5 visual clarity standards shall apply; and

6. *The discharge to water is not within a Community Drinking-water Protection Zone as set out in Schedule 1; and*
7. *The discharge does not occur where there is an available reticulated stormwater system.*

5.0 Stormwater Catchments

5.1 Upstream Contributing Catchments

The upstream contributing catchment extends to the north of the site and consists of rural land and paddocks. The upstream contributing area discharges flow towards Bradleys Road with flow entering the site via two culverts underneath Bradleys Road. During significant storm events, runoff from the upstream contributing catchment breaches over Bradleys Road and discharges onto the proposed development area. The upstream contributing catchments are described and assessed as part of the flood assessment².

The runoff from the upstream catchments remains unchanged for the site's pre- and post-development scenarios.

5.2 Site Catchments Characteristics & Runoff

This section reports on the catchment characteristics and runoff (where applicable) for the existing and proposed site development. The following information was used to delineate catchments:

- ✧ Available surface information (LiDAR and survey).
- ✧ Observations from site investigation and aerial images.
- ✧ Existing stormwater infrastructure as per the Waimakariri District Council (WDC) GIS.
- ✧ Proposed site development as developed by others (e.g., Inovo) as described in Section 3 with consideration given to proposed roads and associated infrastructure.

Refer to Appendix B for the pre- and post-development catchment areas.

5.2.1 Pre-development

The existing land use for the pre-development areas is rural with a general slope of less than 0.5% (Refer to Section 2.1). The pre-development runoff was calculated for the 100-year ARI (1% AEP) 6-hour³ rainfall event, inclusive of

² Flood Effects – 531 and 535 Mill Road, Ohoka, prepared for Carter Group Limited dated 6 September 2026.

³ Multiple flood durations were assessed, with the 6-hour event being critical in terms of peak flows in the existing scenario. Differences in pre- and post-development flows were assessed for multiple events.

climate change based on HIRDS V4 RCP 8.5 up to period 2100, and is reported in Table 3. The pre-development catchments have only been used to determine the attenuation requirements and are further discussed in Section 6.3.

The pre-development time of concentration was determined using the longest contributing flow path and using the WWDG nomograph for overland sheet flows (refer WWDG Figure 21-1) and an assumed stream velocity of 0.6 m/s for catchments with flat slopes (refer to WWDG Table 21-4).

Table 3: Pre-development Catchment Characteristics					
Catchment	Downstream Receiving Point	Land-use	Area (ha)	Time of Concentration (min)	1% AEP ² critical 6-hour event flow (m ³ /s)
C1	Unnamed Drain 1	Rural	30.6	57	1.30
C2	South Ohoka Stream	Rural	54.3	63	2.27
C3	Ohoka Stream Tributary	Rural	51.3	58	2.17
C4	Mill Road Drain	Rural	18.5	50	0.80
Notes: - Pre-development areas exclude drains and buffers. - Inclusive of climate change based on HIRDS V4 RCP 8.5 rainfall for the period up to 2100.					

5.2.2 Post-development

The post-development catchments were delineated based on the proposed site development with fourteen (14) post-development catchments delineated.

The post-development pervious and impervious areas were identified based on the proposed development plan (Refer to Section 3). The runoff coefficients, including allowance for slope adjustment, in Tables 5.2 and 5.3 of the WDC CoP 2025 were used to determine the post-development runoff coefficient. The runoff coefficients used are summarised in Table 4.

Table 4: Post-development runoff coefficients			
Development Type	Surface Type	C-Value	Notes
Small Lots Residential	Residential, roads, SMAs ¹	0.65	Considered for total sub-catchment area and as per WDC CoP Table 5.3.
Polo Field	Park, playground, reserve	0.30	As per WDC CoP with no adjustment for slope consider improved drainage.
Large Lots Residential	Large lots , roads, greenspace, SMAs ¹	0.47	Developed based on WDC CoP Table 5.2 values and max on lot impervious percentage of 20%.
Business	Business and commercial areas	0.80	As per WDC CoP Table 5.3.
Notes: 1. Stormwater Management Areas			

The post-development catchment area characteristics are summarised in Table 5 below. The post-development time of concentration was determined allowing for flow within roads based on the available development layout (refer Section 3.0) and assumed pipe velocities of 0.8 m/s (refer WWDG Table 21-3).

Table 5: Post-development catchment characteristics				
Pre-development Catchment	Post-development Catchment	Land-use	Area (ha)	Time of Concentration (min)
C1	2A	Residential	24.6	53
	2B	Residential	16.2	51
C2	1	Residential – small lots	10.3	33
	3	Residential – small lots	10.7	21

Table 5: Post-development catchment characteristics				
Pre-development Catchment	Post-development Catchment	Land-use	Area (ha)	Time of Concentration (min)
	4	Residential – small lots	9.5	15
	Drains	Conveyance channels	12.5	30
C3	5A	Residential – small lots	14.6	50
	5B	Residential – small lots	1.7	43
	6	Residential – small lots/retirement village	17.9	27
	9A	Residential – small lots	3.4	21
	9B	Commercial	1.6	12
	9C	Residential – small lots	9.8	25
	10	Polo fields	8.7	52
	Drains	Conveyance channels	13.6	35
C4	8	Residential – small lots	4.1	24
	11	Residential – small lots	1.8	15

Pre- and post-development flows are shown in Table 6 below for the 1 hour, 6 hour, 12 hour and 24 hour events. The post-development runoff was calculated for the 100-year (1% AEP) rainfall event, inclusive of climate change based on HIRDS V4 RCP 8.5 up to period 2100. Note that the table also presents the corresponding pre-development catchment flow. Critical peak flow values have been bolded.

Table 6: Calculated peak flows for each of the post-development catchments				
Event	Catchment	Pre- development flow (m³/s)	Post-development flow (m³/s)	Change (m³/s)
10% AEP - 1-hour	C1	0.6	0.12	-0.48
	C2	1.05	0.6	-0.45
	C3	1	0.45	-0.55
	C4	0.37	0.18	-0.19
10% AEP - 6-hour	C1	0.7	0.57	-0.13
	C2	1.22	1.16	-0.06
	C3	1.17	1.01	-0.16
	C4	0.43	0.24	-0.19
10% AEP - 12-hour	C1	0.60	0.41	-0.19
	C2	1.05	0.86	-0.21
	C3	1.00	0.98	-0.02
	C4	0.37	0.36	-0.01
10% AEP - 24-hour	C1	0.46	0.42	-0.04
	C2	0.82	0.75	-0.07
	C3	0.78	0.77	-0.01
	C4	0.28	0.22	-0.06
1% AEP – 1 hour	C1	1.61	0.46	-1.15
	C2	2.81	1.41	-1.40
	C3	2.69	1.69	-1.00
	C4	0.99	0.57	-0.42
1% AEP – 6 hour	C1	1.94	1.10	-0.84
	C2	3.40	2.82	-0.58
	C3	3.24	2.46	-0.78
	C4	1.19	0.49	-0.70
1% AEP - 12-hour	C1	1.27	1.13	-0.14
	C2	2.24	2.11	-0.13

Table 6: Calculated peak flows for each of the post-development catchments				
Event	Catchment	Pre- development flow (m³/s)	Post-development flow (m³/s)	Change (m³/s)
1% AEP - 24-hour	C3	2.13	2.11	-0.02
	C4	0.78	0.35	-0.43
	C1	0.91	0.90	-0.01
	C2	1.61	1.35	-0.26
	C3	1.52	1.46	-0.06
	C4	0.55	0.23	-0.32

6.0 Proposed Stormwater Management Plan

The stormwater management for the site has been developed in collaboration with the following assessments and/or designs:

- ✧ *Inovo, Infrastructure Report – 531 & 535 Mill Road, Ōhoka, (Inovo Infrastructure Report)*

The report provides details of the site's primary and secondary stormwater infrastructure discharging towards the proposed stormwater management areas.

- ✧ *PDP, Flood Effects – 531 and 535 Mill Road, Ohoka, September 2026*

The report provides an assessment of the flood effects of the proposed development for events larger than the 2% AEP event (up to and including the 0.5% AEP event).

The proposed stormwater management for the site has been designed to achieve the following:

- ✧ Hydraulic Continuity

Hydraulic continuity between upstream and downstream catchments of the proposed development will be provided by the retention of existing Ōhoka tributary and Ōhoka South waterways (refer to Figure 2) through the proposed development, creating formalised flow path corridors. These flow path corridors will collect upstream flows and site runoff, directing water towards the existing downstream culverts underneath Whites Road. Upstream flows breaching Bradleys Road will be intercepted by swales / cut-off drains and direct flow toward the flow path corridors. All existing roadside drains are to be retained. Many existing drains internally within the site are proposed to be filled and/or rerouted within the site. Refer to the *Inovo Infrastructure Report* for

locations of these drains. Refer to the *Ecology Report* and the *Hydrology Report* for impacts and recommendations relating to the on-site drains.

Connectivity between the onsite springheads and downstream waterways will be maintained and will be separated from new stormwater runoff collection and treatment infrastructure.

The proposed flow paths will provide the following functions:

- Maintain hydraulic connectivity between the upstream and downstream catchments, including baseflow.
- Collect and convey runoff from the proposed development towards the downstream environment.
- Provide attenuation for up to the 1% AEP runoff event (inclusive of climate change) for runoff from the proposed site development not to exceed the pre-development runoff.
- Allow for storage during events exceeding the 1% AEP inclusive of climate change through:
 - ✧ Instream attenuation volume to be retained and enhanced.
 - Compensatory storage along the flow path corridor by flooding of low-lying and depression areas.
 - Backflow into attenuation facilities during storm events exceeding the 1% AEP critical storm event.

The flow paths will be designed to capture and convey up to the 0.5% AEP event (200-year event) to reduce the impact of flooding on proposed flood levels within the development. The flow paths will be designed not to scour up to and including the 1% AEP event flows.

✧ Primary flows

Primary flows will be collected along kerbed road sections and/or swales. Where present, a pipe network will direct flows toward the stormwater management areas where treatment and/or attenuation will be provided. Infrastructure will be sized to accommodate flow for the 10% AEP inclusive of climate change.

Refer to the *Inovo Infrastructure Report*.

✧ Secondary flows

Secondary flows will be directed towards the formalised flow paths via roads and dedicated easements where there are no roads available to convey secondary flow. The secondary flow design will be based on managing flows from the 1% AEP inclusive of climate change.

Refer to the *Inovo Infrastructure Report*.

6.1 Stormwater Management Strategy

The stormwater management for the proposed development applied the following general strategy:

- ✧ Manage stormwater runoff to not exceed the pre-development runoff through attenuation to limit the flow at each key downstream discharge point from the site (Ōhoka tributary culvert, Ōhoka South culvert, unnamed drain 1 and the Mill Road drain) to pre-development conditions.
- ✧ Treat stormwater runoff for the first flush volume and/or flow via a range of treatment options.

The type and sizing of the stormwater management within a particular sub-catchment is influenced by the depth to groundwater (i.e., maximum recorded groundwater level). As reported in Section 2.3, the proposed site has a shallow depth to groundwater. Based on the best information available (refer to *PDP, Fast Track Application – 531 & 535 Mill Road, Ōhoka – Hydrology Assessment (June 2026)*), the adopted depth to maximum groundwater within the proposed SMA's (i.e., lower end) for a sub-catchment is presented in Table 7.

Table 7: Depth to maximum groundwater	
Catchment	Depth to Max. Groundwater range (m)
1	0.11 to 0.39
2A	0.51 to 0.57
2B	0.39
3	0.23 to 0.38
4	0.12 to 0.30
5A	0.60 to 0.85 away from the spring 0.15 to 0.25 closer to the spring
5B	0.40
6	0.30
8	0.20 to 0.35
9A	0.41 to 0.46
9B	0.41
9C	0.51 to 0.91
10	Not Tested
11	Not Tested

The proposed stormwater management for each post-development catchment is summarised in Table 8 below and was developed in collaboration with the Inovo Infrastructure Report to balance proposed land use and density with the required stormwater management area. Further detail with regards to proposed treatment and attenuation is provided in the subsequent sections.

Table 8: Proposed stormwater management per catchment

Catchment	Land-use	Total Catchment Area (ha)	Primary Infrastructure ¹	Stormwater Treatment	Attenuation
1	Rural-residential housing	10.3	Piped infrastructure	First flush basin & wetland	Above the first flush basin & wetland
2A	Rural-residential housing	24.6	Swales	Swales	Attenuation basin (wet pond)
2B	Residential housing	16.2	Swales	Swales	Attenuation basin (wet pond)
3	Residential housing	10.7	Piped infrastructure	First flush basin & wetland.	Above the first flush basin & wetland
4	Residential housing	9.5	Piped infrastructure	First flush basin & wetland. Raingardens.	Above the first flush basin & wetland In the portion of Catchment 4 treated by raingardens attenuation will be provided by an attenuation basin
5A	Residential housing	14.6	Piped infrastructure	First flush basin & wetland. Raingardens.	Above the first flush basin & wetland In the portion of Catchment 5A treated by raingardens attenuation will be provided by an attenuation basin
5B	Residential housing	1.7	Swales	Swales	Attenuation basin (wet pond)

Table 8: Proposed stormwater management per catchment

Catchment	Land-use	Total Catchment Area (ha)	Primary Infrastructure ¹	Stormwater Treatment	Attenuation
6	Residential housing Retirement village ²	17.9	Swales Retirement village servicing is left open for future operator.	Swales Retirement village will have internal SMA or will use proprietary devices.	Offset by storage within Catchment 10. Treatment/reticulation to be confirmed by the future operator, but stormwater effects have been conservatively allowed for through offset attenuation within the same receiving catchment
8	Residential housing	4.1	Piped infrastructure	First flush basin & wetland.	Above the first flush basin & wetland
9A	Residential housing	3.4	Piped infrastructure	First flush basin & wetland.	Above the first flush basin & wetland
9B	Business / Commercial Property	1.6	Piped infrastructure	Proprietary devices (flow-based)	Within the detention basin
9C	Residential housing	9.8	Piped infrastructure	First flush basin & wetland.	Above the first flush basin & wetland
10	Polo field/park	8.7	Under drainage	N/A	Storage within the polo field
11	Residential housing	1.8	Piped infrastructure	Raingardens	Off-set storage within catchment 8

Notes:

1. For the proposed primary infrastructure design, refer to Inovo, Infrastructure Report – 531 & 535 Mills Road, Ōhoka.
2. Retirement Village will be developed by a private provider. Reticulation and treatment options are therefore left open, however attenuation has been accounted for through over-attenuation of other subcatchments with discharge to the same pre-development catchment receiving location.

6.2 Stormwater Treatment

6.2.1 Stormwater Quality

PDP has prepared a separate water quality memo (*PDP, 531 & 535 Mill Road – Water Quality Assessment, June 2026*), attached as Appendix C, which identifies and quantifies the expected water quality from the site post-development.

The water quality memo does not report on the expected water quality volume to be treated, this is set out below in Table 9. This reports on the expected first flush volume and/or flow to be treated at each of the catchments based on the proposed development within the catchment and the proposed stormwater treatment device type for each catchment. It is important to note that the ‘area to be treated’ is defined as the total catchment area minus the total area of the treatment devices, therefore the catchment sizes are slightly smaller than Table 8. The following parameters were used to determine the first flush in accordance with the method in WWDG:

- ✧ Parks, drain areas, and buffer strips were excluded from the first flush calculations as these areas would not require treatment.
- ✧ A discharge coefficient of 0.9 (i.e., CD) was applied to the runoff coefficient (refer to Table 4), as per the WWDG method for estimating first flush volume/flow.
- ✧ A first flush rainfall depth of 25 mm was used as per the WWDG for green field developments for treatment by means of first flush ponds and wetlands
- ✧ The Water Quality flow rate has been calculated using a rate of 5 mm/hr as per the WWDG for treatment by means of swales, raingardens or proprietary devices

Table 9: Water Quality –Catchment First Flush Requirements

Catchment	Area Requiring Treatment (ha) ¹	Composite First Flush Coefficient	Stormwater Treatment Infrastructure	First Flush Volume (m ³)	Water Quality Flow (L/s)
1	9.4	0.59	First flush basin & wetland	1,387	N/A
2A	23.4	0.42	Swales	N/A	136
2B	15.3	0.42	Swales	N/A	90
3	9.7	0.59	First flush basin & wetland.	1,436	N/A
4	8.3	0.59	First flush basin & wetland. Raingardens.	1,218	67
5A	13.6	0.59	First flush basin & wetland Raingardens	1,989	110
5B	1.4	0.59	Swales (flow based)	N/A	12

Table 9: Water Quality –Catchment First Flush Requirements

Catchment	Area Requiring Treatment (ha) ¹	Composite First Flush Coefficient	Stormwater Treatment Infrastructure	First Flush Volume (m ³)	Water Quality Flow (L/s)
6	10.42 ²	0.59	Swales (flow based) Retirement village servicing is left open for future operator.	N/A	85
8	3.6	0.59	First flush basin & wetland.	528	N/A
9A	3.0	0.59	First flush basin & wetland.	449	N/A
9B	1.3	0.72	Proprietary devices (flow-based)	N/A	13
9C	8.8	0.59	First flush basin & wetland.	1,287	N/A
10	8.1	N/A as park area (polo field)			
11	1.9	0.59	Raingardens	277	15

Notes:

1. Area required to be treated excludes treatment areas, drains/buffers, park areas and other areas that do not generate runoff requiring water quality treatment.
2. For Catchment 6, the reduced treatment area excludes the retirement village area. Treatment and reticulation for that area will be confirmed by the future operator.

6.2.2 Stormwater Treatment Infrastructure

Stormwater treatment varies by sub-catchment, with the type of treatment infrastructure (Refer to Table 9) developed in collaboration with the infrastructure design report (*Inovo, Infrastructure Report – 531 & 535 Mills Road, Ōhoka*). The proposed stormwater management design is for predominantly “green infrastructure”, which provides additional benefits to the community and environment via the creation of green spaces and wildlife habitats.

Treatment of water quality flows from site sub-catchments is proposed to be managed by one of the following:

- ✧ First flush basins and treatment wetlands

First-flush basins (designed as wet ponds) have been sized to capture, retain, and partially treat the first flush volume from the contributing catchment. Refer to Table 9 for the first flush volumes.

The sizing of the first flush basins is governed by the volume that is required to be stored and the depth of groundwater at the site. The depth of the basins is set at approximately 1 m to allow for the appropriate settling of suspended solids entrained in stormwater while minimising the risk of resuspension from wind effects. The available storage was determined using the maximum seasonal groundwater depth as reported in Section 6.1.

First flush basins drain via a controlled outlet to treatment wetlands which provide further treatment of the captured stormwater. The proposed wetlands have been sized using the WWDG simplistic method, including:

- The first flush basin to release water quality over 4 days.
- Wetland hydraulic residence time of 2 days.
- Wetland average depth of 0.25 m and vegetation porosity of 75%.
- A wetland will have deep (approx. 1 m) open pools and shallow (approx. 0.25 m) planted areas.
- Refer to Appendix D for typical first flush and wetland details.

The treated stormwater will exit the wetland via a controlled outlet into the proposed drainage corridors and streams.

The design intent is to ensure that there are no tailwater effects from the existing drains on the wetlands. Modelling will be completed at detailed design stage to ensure that there is adequate head within each wetland.

✧ Swales

Treatment swales will be used in place of pipe networks, capturing flow from the site sub-catchment and providing treatment of water quality flows (based on the water quality flow intensity value of 5 mm/hr).

Swales are to be sized at the detailed design phase of the project.

Swales will drain to downstream infrastructure consisting of attenuation storage via wet ponds.

✧ Rain gardens

Rain gardens will be located within the development which cannot be treated by swale or SMA. Runoff from impervious areas (mainly roads) will be directed toward rain gardens to capture and treat the first flush volume. It is proposed that high infiltration raingardens are used in place of conventional raingardens to reduce the total area required as well as reduce the total number of raingardens. A proprietary high-infiltration raingarden like the SW360 Filtterra® type is proposed which has a designed media to treat up to 4,450 mm/hr.

To manage shallow groundwater, it is proposed that the shallow depth Filtterra® rain garden be used. The shallow rain garden has a depth from the invert of the kerb to the invert of the rain garden base of approximately 700 mm. This is compared to a conventional rain garden depth of approximately 900 mm. The rain garden will be completely enclosed by the standard rain garden concrete surround. Refer to Appendix E for product details of the proposed SW360 Filtterra® rain garden including a typical drawing.

✧ Proprietary devices

For high contaminant loading areas (typically business/commercial land use) treatment will be provided by the following:

- Gross pollutants are to be captured at sumps by means of submerged outlets and/or proprietary devices within the sumps (i.e., LittaTrap™ or TetraTrap®).
- Water quality flows are to be directed to a proprietary filter system (e.g., SW360 Stormfilter®) to treat runoff by removing total suspended solids, hydrocarbons, and heavy metals. The number of cartridges within each filter can be scaled to meet water quality flow requirements. The presence of high groundwater can be mitigated by installed filters in a shallow chamber (< 1 m deep) and filters that operate under low head conditions. The exact design and number of filters will be confirmed at the detailed design phase of the project.

Proprietary filters are sized based on the water quality flow rate and typically treat approximately 1.2 L/s of first flush runoff per filter unit or pod.

Refer to Appendix F for product details of the proposed proprietary treatment devices including typical drawings.

A summary of the required treatment and sizing of first-flush basins and wetlands is included in Appendix G.

6.3 Stormwater Attenuation

Stormwater attenuation has been accounted for in the layout of the site in two ways;

- ✧ Live storage in the Stormwater Management Areas (SMAs) and wetlands to attenuate site runoff so that peak flows are the same pre- and post-development, offsetting the effects of increasing impervious area on the site. This attenuation is described in this report.
- ✧ Management of inflows coming from upstream of the site via flood attenuation devices built into the proposed landform, to retain the current overland flow conveyance and storage in the pre-development scenario, to ensure no increases in peak flood level downstream. This encompasses areas separate to the SMAs such as the polo field, and enhancement of existing waterway corridors through the site. This is the subject of the flood modelling report, however we have described elements of the flood management strategy in this section as relevant.

At this stage of the project, the two have been assessed separately, as the SMAs have not been accounted for in the flood modelling, and the additional on-site temporary flood storage measures such as the polo field (described later in this section) have not been accounted for in this report. The two systems have been developed in isolation, but both contribute to the overall attenuation strategy for the site. As such, the approach is conservative, and the analysis will be combined at later (detailed design) stages of the project. It is expected that combining the two systems will result in better stormwater outcomes than what is described in both this report and the flood modelling report.

The approach to the peak flow attenuation or live storage is described in this section.

6.3.1 Live storage

Peak flow attenuation as described in this report will be provided for up to the 1% AEP event using the following infrastructure where available:

- ✧ Storage within first flush basins (wet ponds).

- ∴ Storage above the wetland permanent water level within the wetland (PWL).

The following process was used to determine the required attenuation volume:

- ∴ Attenuation volumes were estimated using an HEC-HMS model based on curve numbers and initial abstraction. The required storage above the water quality treatment volume was determined by limiting post-development peak runoff to the pre-development maximum flow. HEC-HMS is appropriate for this assessment because the catchments exceed approximately 15 ha.
- ∴ The calculated attenuation volumes were checked against the proposed first flush volume and available storage above the proposed wetland within a catchment (where applicable). The available storage above the wetland permanent water level was limited by the depth to maximum groundwater (refer to Table 6) and SMAs reflected in the design include a 500 mm freeboard allowance.

The attenuation for each catchment has either been incorporated into the stormwater quality devices proposed (wetlands) or through attenuation basins. Descriptions of the proposed infrastructure are provided below.

- ∴ First flush basins and treatment wetlands

For catchments where treatment includes first flush basins and wetlands, attenuation will be provided practically/feasibly by the first flush volume being stored, with the remainder being provided within the storage depth between the permanent water level (PWL) within the wetland and the natural ground level.

Once the first flush volume has been reached, secondary flow from the contributing catchment will be directed towards the wetland via overland flow paths and piped bypass infrastructure.

The first flush volume will be discharged over the water quality design requirement of 4 days (refer to WWDG) while the volume attenuated within the wetland area will be discharged via an outlet structure over a shorter period (less than 24hrs). The exact type of outlet structure for each SMA will be determined at detailed design and will be designed so as to limit flow to maximum pre-development flows from the receiving catchments for the SMA. The proposal is such that, in up to a 1% AEP event, the post-development contributing flow to the waterway will be no more than the existing pre-development flow.

Each wetland will have a rock armoured spillway to allow for events exceeding the 1% AEP (inclusive of climate change) to have a controlled discharge towards the downstream environment. This will assist in managing any erosion potential in the vicinity of discharge locations.

Refer to Appendix D for typical details of the proposed first flush and wetland.

✧ Attenuation Basins / Wet ponds

In catchments that require attenuation but provide treatment via swales or other means (refer to Section 6.1.1), attenuation will be provided by means of basins at the end of pipe. The basins will be wet ponds due to the high groundwater across the site and the available storage was determined based on the maximum groundwater depth and natural ground level.

Discharge from the basin will be controlled by means of an outlet structure. The outlet structure will typically consist of a manhole with a scruffy dome and an outlet orifice sized to limit flow to maximum pre-development flows.

The basin will have a spillway to allow for events exceeding the 1% AEP (inclusive of climate change) to have a controlled discharge towards the downstream environment.

Refer to Appendix D for typical details of the proposed wet ponds.

✧ On-site storage

The proposed retirement village within post-development Catchment 6 has not been relied on to provide attenuation within this assessment. Instead, stormwater effects from that area have been conservatively accounted for through offset attenuation within other sub catchments discharging to the same pre-development receiving catchment. Final internal reticulation, treatment and any on-site attenuation for the retirement village will be confirmed by the future operator at detailed design. Management of flows from upstream of the site

In addition to the peak flow attenuation devices that have been described above, the site also contains other features that are used for flood attenuation that primarily act to convey and store flows from upstream of the site, but will also work in conjunction with the peak flow attenuation devices.

A key flood attenuation measure is the polo field, which sits within catchment 10. The proposed polo field is within post-development catchment 10 and will be utilised to provide temporary attenuation for the inflows upstream of the site. Attenuation will be provided on top of the field area with proposed under drainage (consisting of drainage layers and sub-soil pipes) connecting to an outlet structure (manhole with scruffy dome and orifice plate) located at the low point within the polo field. The outlet structure (i.e., scruffy dome) will attenuate flow from the polo field catchment (post-development catchment 10).

The hydraulic performance of the polo field has been tested in the PDP TUFLOW model and described in detail in the flood modelling report. The polo field has not been represented in the HEC-HMS or any stormwater calculations in this report, nor have the SMAs been represented in the TUFLOW model. This is due to the fact that the SMAs and polo field storage have been designed for different purposes. However they do assess the same (1% AEP) event, showing that the two systems in isolation perform as intended but will likely provide greater attenuation benefit when combined.

Figure 4 below shows 1% AEP peak flood depths from the PDP modelling report, within each stormwater catchment, demonstrating the points above. The model results show significant attenuation within Catchment 10 (the polo field) which also prevents spill into Catchment 6 and the retirement village. There is also a small amount of ponding in the retirement village catchment which would ultimately be managed in a SMA, but due to the polo field attenuation, the retirement village will only have to manage runoff from the local catchment only.

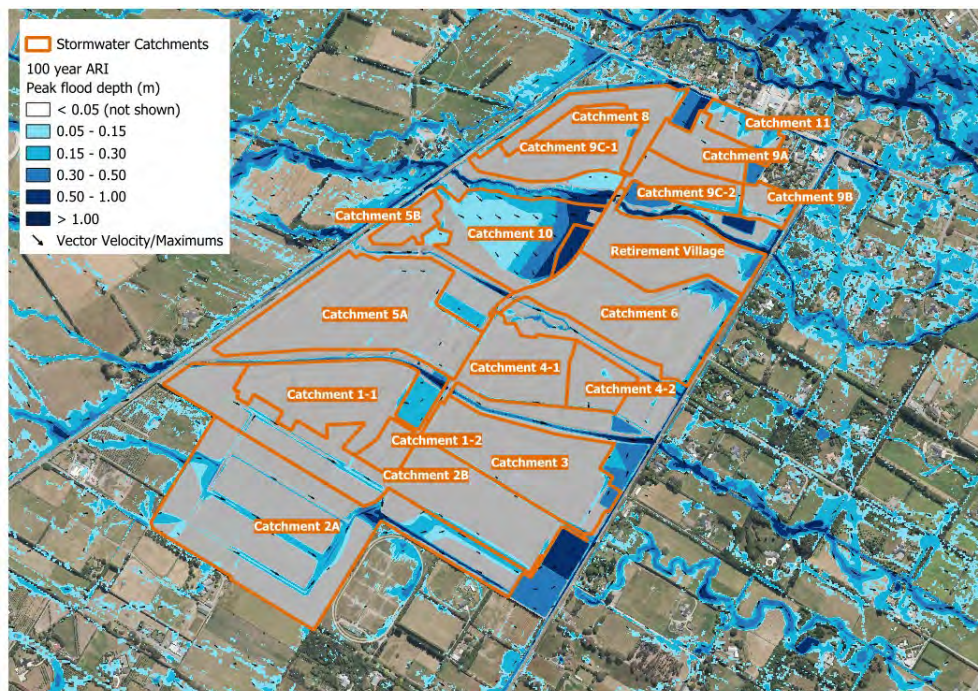


Figure 4: 1% AEP peak flood depths with the stormwater catchments

6.3.2 Overall attenuation strategy

The overall attenuation strategy is provided in Table 10 below, which provides an overview of the attenuation strategy within each of the catchments as a combination of the storage represented in the flood modelling and this report. Refer to Appendix H for a summary of the attenuation provided and required.

Table 10: Attenuation		
Pre-Development Catchment	Post-Development Catchment	Attenuation provided
C1	2A	Storage will be provided within attenuation basins downstream of the catchment. The basins will not provide treatment; however, they will be wet ponds due to the shallow groundwater.
	2B	
C2	1	Storage will be provided within the first flush and wetland areas.
	3	
	4	In the portion of Catchment 4 treated by raingardens attenuation will be provided by an attenuation basin.
C3	5A	Storage for Catchments 5A, 5B, 9A and 9C will be provided within the first flush and wetland areas.
	5B	
	6	
	9A	In the portion of Catchment 5A treated by raingardens attenuation will be provided by an attenuation basin
	9B	
	9C	Attenuation for Catchment 9B will be via detention basin.
	10	Attenuation requirements for Catchment 6 will be offset by attenuation provided within Catchment 10 (polo field). The proposed retirement village within Catchment 6 has conservatively modelled without internal attenuation, with its effects offset by over-attenuation/storage within the same receiving catchment, while final internal servicing and treatment will be confirmed by the future operator
C4	8	Storage will be provided within the first flush and wetland areas within catchment 8. Catchment 8 will provide offset storage for Catchment 11.
	11	

6.4 Tailwater impacts

As the modelling used to undertake the HEC-HMS basin sizing does not account for tailwater effects, we have used the TUFLOW (flood) model results to undertake a high-level assessment on the likely effects of tailwater on the performance of the SMAs. The model was run for the same 6-hour rainfall hyetograph as the HEC-HMS model to assess the relative timing of the peak flood level in the immediately receiving waterways vs the timing of the peak flow out of the SMAs in the HEC-HMS model. The results are shown in Table 11, noting that this TUFLOW model run only outputs time series results in 15-minute increments.

Table 11: Tailwater effect analysis using the TUFLOW model results, overlaps shown in red		
SMA	Time of peak (HEC-HMS)	Window of time where the flood level is higher than the Permanent Water Level (PWL) in the TUFLOW model
Wetland 9C-1	04:14	4:45 – 15:00
Wetland 9C-2	05:52	6:30 – 18:45
Wetland 5A-1	06:01	N/A ¹
Wetland 5A-2	04:49	N/A ¹
SMA 9A	05:56	4:00 – 16:45
SMA 5B	06:12	N/A ¹
SMA 9B	05:53	4:00 – 16:45
SMA 10	04:58	4:45 – 15:00
SMA 2A	06:04	N/A ¹
SMA 2B	05:56	N/A ¹
Wetland 1	04:51	N/A ¹
SMA3	04:41	4:00 – 8:00
Wetland 4-1	04:31	N/A ¹
Wetland 4-2	04:38	N/A ¹
Notes: 1. The flood level in the receiving waterway does not exceed the PWL		

For most SMAs, peak receiving-water levels are either below the SMA permanent water level or occur after the SMA discharge peak. The exceptions are SMA9A, SMA9B, SMA10 and SMA3. With the exception of SMA10, these SMAs are located near the eastern, downstream end of the site, where floodwater backs up upstream of Whites Road. Consequently, floodwater reaches these SMAs later than it reaches upstream SMAs and more closely coincides with peak flows from the local site catchments. SMA10 is located within the polo field, which forms part of the flood-storage area, and is therefore expected to experience high tailwater effects.

The outlet configurations for these SMAs will be reviewed during detailed design. Larger outlets may be adopted to bring forward the discharge peak and thereby reduce the required attenuation volume. The analysis completed to date is conservative with respect to SMA storage volumes used for peak-flow attenuation, but not necessarily outlet sizes.

The flood model indicates no significant surface flooding across the site during the 1% AEP event, even when the SMAs are assumed to provide no additional storage because their outlets are effectively blocked and the SMAs are pre-filled (see Figure 4). Submerged outlets would further restrict post-development discharge. As the current calculations show that post-development peak flows are attenuated to pre-development levels under free-outfall conditions, accounting for tailwater effects would be expected to reduce the reported post-development peak flows.

At a later design stage, the SMA system will be incorporated into the TUFLOW model to assess tailwater effects in greater detail. Given the methods applied in the HEC-HMS and TUFLOW models, the current assessment is considered conservative in relation to the total attenuation volume and is appropriate for the present design stage.

7.0 Stormwater Runoff Quality and Quantity Effects

7.1 Surface Water Quality

The water quality memo (*PDP, 531 & 535 Mill Road – Water Quality Assessment, June 2026*) attached as Appendix C reports on the expected discharge water quality from the proposed development.

The stormwater discharge quality from the site is expected to be unimpeded with regard to Schedule 8 limits, and will meet the LWRP total suspended solids (TSS) limit of 50 mg/L (for spring-fed waterways) along with the 95% species protection limit for total lead of 0.0034 mg/L. However, the stormwater discharge quality is not expected to directly meet the 95% species protection limits for total zinc and total copper at all points of discharge.

It is noted that the limits for which this assessment has compared the treated stormwater are the chronic values from the ANZG 2018 guidelines. This is a conservative approach for stormwater flows, which occur only during rainfall and for short periods before mixing with runoff from the wider catchment.

It is also noted that the 95% species protection limits are intended to be used as instream water quality targets, after reasonable mixing (i.e., downstream of the mixing zone). For the proposed development, there will be various discharge points into the tributaries of the Ōhoka Stream, and depending on the characteristics of each waterway, the extent of the mixing zones will vary.

The above combine to indicate that the water quality expected in the downstream environment will most likely be of significantly better quality than the stated discharge values. Therefore, any effects of the stormwater discharge quality not directly meeting the 95% species protection limits for total zinc and total copper at all points of discharge is considered to be less than minor.

7.2 Surface Water Quantity

7.2.1 Peak flows

The proposed attenuation within the first flush and wetlands or basins (e.g., wet ponds) provide sufficient attenuation for the post-development 1% AEP, inclusive of climate change, event not to exceed the pre-development peak flows as calculated along Whites Road.

The available storage volumes within the basins were calculated based on the maximum groundwater expected within the basins. This is based on monitoring data as described in Section 2.3 of this report.

The following is noted:

- ✧ Pre-development Catchment C1

The attenuation provided is sufficient to limit the post-development runoff to the pre-development runoff for catchment C1. The proposed basins may be optimised as part of the design development.

- ✧ Pre-development Catchment C2

The proposed first flush and wetlands in post-development catchments 1, 3, and 4 are sufficiently sized to meet water quality requirements to provide attenuation as well. The first flush basins alone have sufficient volume to attenuate the targeted volume for the pre-development runoff to not exceed the post-development runoff.

- ✧ Pre-development Catchment C3

The attenuation provided is sufficient to limit the post-development runoff to the pre-development runoff for catchment C3. The proposed

basins may be optimised as part of the design development. Pre-development Catchment C4

The post-development runoff from catchments 8 and 11 is less than the pre-development catchment runoff due to a large area of pre-development catchment C4 being routed towards post-development catchment C3 (consisting of catchments 5A, 5B, 6, 9A, 9B, 9C and 10).

7.2.2 Groundwater Interception

The proposed basins (first flush and wet ponds) and wetlands are likely to intercept the groundwater during certain parts of the year. Wet ponds and wetlands are expected to have a maximum depth of 1 m to 1.2 m where sediments are to be removed through settling (e.g., first flush basins as per the WWVG). The depth is likely to extend through the groundwater table (refer to 2.3.1) and the underlying silt layer, intercepting the gravel layer below in some situations.

To reduce loss of groundwater, all basins (wet ponds and wetlands) will have a liner (silt or similar) installed on the base to reinstate the natural imperviousness of the silt layer.

The sizing of the SMA's, and area allowed for their construction, is based on the assumption that all outfalls from the stormwater management areas will be located above the seasonal high groundwater level to prevent groundwater discharge from these systems. Discharge through outfalls will therefore only be volumes of rainfall captured during a storm event. This is to ensure that no groundwater is lost from the site via discharges from the basins.

At detailed design stage, the anticipated wetland water level regime for each SMA location will be assessed having regard to seasonal groundwater fluctuations, seepage through the underlying low-permeability silt layer, stormwater inflows, rainfall, evaporation and evapotranspiration. The assessment will be used to confirm an appropriate permanent water level and to refine the wetland bathymetry and planting-zone elevations so that the proposed vegetation communities are suited to the anticipated range, frequency and duration of inundation and seasonal drawdown. The outlet structure will incorporate an adjustable weir, or equivalent level-control mechanism, to provide operational flexibility to modify the wetland water level following construction and during the vegetation establishment period. This will allow the water regime to be adjusted in response to observed site conditions where necessary to avoid excessive drying or prolonged inundation of planting zones, while maintaining the intended permanent pool, hydraulic performance and stormwater treatment function.

8.0 Conclusion

The stormwater management for the proposed development has been developed to attenuate runoff from the site to not exceed the pre-development flow rates, during the 1% AEP event and inclusive of climate change, along Whites Road.

The proposed attenuation basins consisting of either first flush and wetlands or attenuation basins (e.g., wet ponds) will be optimised during further design development. This will include incorporating the attenuation basins in the TUFLOW model to assess the effects of tailwater and to check the combined response of the additional attenuation measures such as the polo field with the additional attenuation from the stormwater management areas. At present both systems have been proven to work in isolation for their intended purposes, and will likely perform better as a system when combined.

The proposed treatment is considered appropriate to manage and treat runoff from a subdivision and is in accordance with guideline documents (e.g., WWDC and WDC CoP).

The water quality from the development at the point of discharge is expected to meet the LWRP targets for TSS and lead. Stormwater discharge quality is not expected to directly meet the “in-stream” 95% species protection limits for total zinc and total copper at points of discharge, but any effects are considered less than minor. The proposed treatment utilises typical best practice approaches for the type of proposed development (e.g., rural-residential, residential, and commercial) with the estimated untreated water quality parameters and proposed treatment removal rates based on typical and published data.



- NOTES:**
- 1. THESE DRAWINGS ARE TO BE READ AS A COMPLETE SET. ANY DISCREPANCIES MUST BE REFERRED TO THE ENGINEER FOR CLARIFICATION.
 - 2. ALL WORKS ARE TO BE CARRIED OUT IN ACCORDANCE WITH THE WAIMAKARIRI DISTRICT COUNCIL ENGINEERING CODE OF PRACTICE.
 - 3. ANY INSTRUCTIONS ISSUED BY COUNCIL OR OTHER AUTHORITIES MUST BE CONFIRMED WITH THE ENGINEER.
 - 4. THE CONTRACTOR IS RESPONSIBLE FOR LOCATING, IDENTIFYING, AND PROTECTING ALL EXISTING SERVICES PRIOR TO COMMENCING ANY EARTHWORKS.
 - 5. THE CONTRACTOR MUST REMOVE ALL UNSUITABLE MATERIAL (E.G., VEGETATION, UNSUITABLE TOPSOIL, ETC.), INCLUDING ANY METALLED AREAS OTHER THAN AT THE SITE ENTRANCE.
 - 6. LOT AREAS AND BOUNDARY DIMENSIONS ARE PROVISIONAL AND SUBJECT TO CONFIRMATION UPON COMPLETION OF THE FINAL LAND TRANSFER SURVEY.

- LEGEND:**
- STAGE BOUNDARY
 - ABUTTAL BOUNDARY
 - PROPOSED LOT BOUNDARY
 - PROPOSED ROAD CENTERLINE
 - RESIDENTIAL LOTS
 - COMMERCIAL AREA
 - PROPOSED ROAD
 - PROPOSED RIGHT OF WAY
 - PROPOSED WATER SUPPLY
 - BOREHOLE LOT
 - PROPOSED UPGRADED WATER TREATMENT PLANT
 - PROPOSED WASTEWATER PUMP STATION LOT
 - PROPOSED STORMWATER MANAGEMENT AREA
 - RESERVE / WALKWAYS
 - PROPOSED TOP OF BANK
 - STORMWATER SECONDARY FLOW PATH
 - GROUND DISTURBANCE / IMPERVIOUS SURFACE SETBACK
 - MINIMUM RIPARIAN ENHANCEMENT SETBACK
 - DRAIN SETBACK
 - PROPOSED SWALE
 - EXISTING TELECOM. TOWER
 - EXISTING OVERHEAD POWER LINE
 - EXISTING TRANSPOWER NATIONAL GRID YARD - 19m
 - 15m LANDSCAPING STRIP SETBACK

B	ISSUED FOR FAST TRACK APPLICATION	LM	10/09/2026
A	ISSUED FOR FAST TRACK APPLICATION	SS	02/07/2025
REVISION DETAILS		BY	DATE

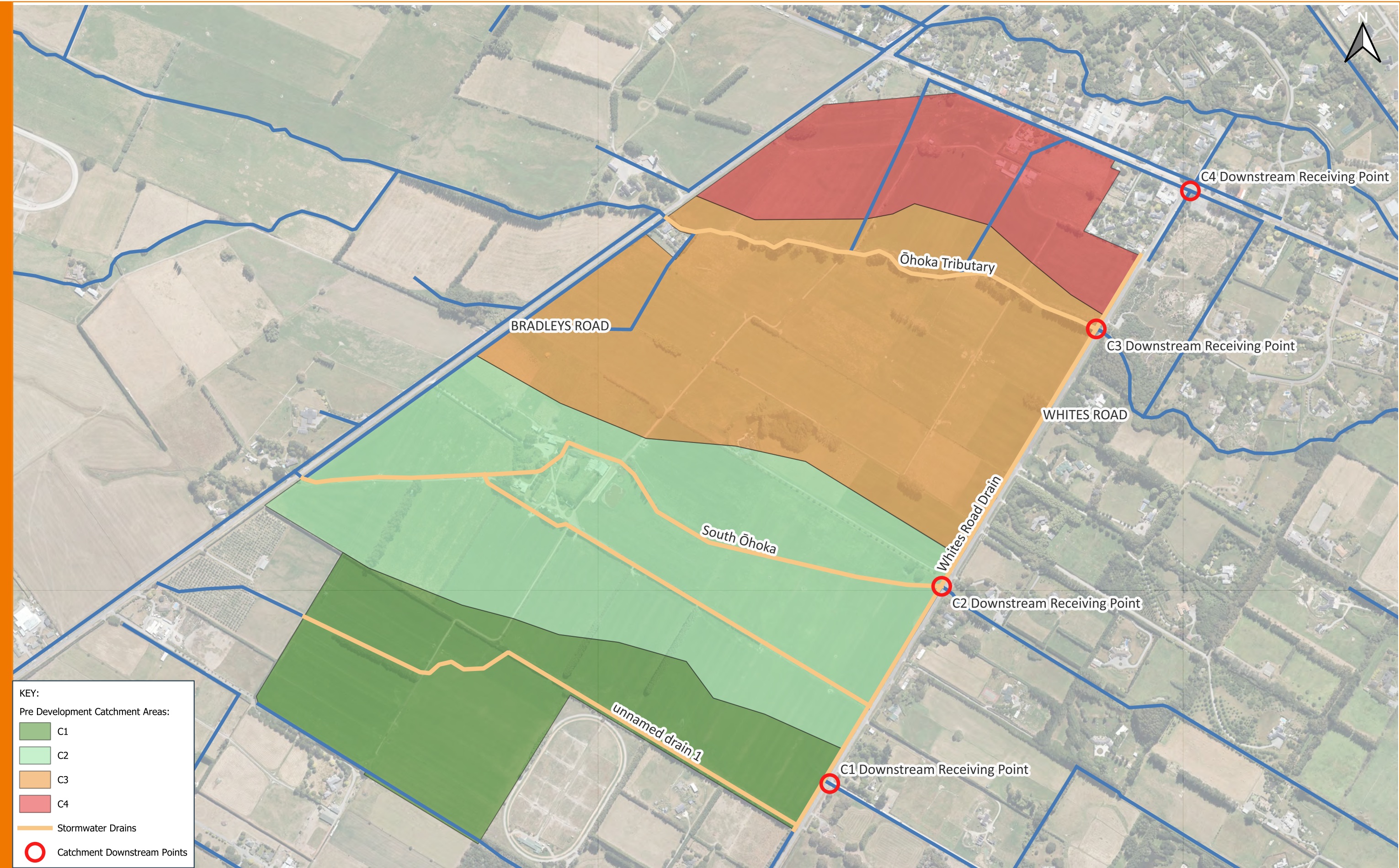
PROJECT:
**MILL ROAD, OHOKA
WAIMAKARIRI
CANTERBURY**

DESCRIPTION:
SCHEME OVERVIEW PLAN

	SURVEYED	-	-
	DESIGNED	PE	10/09/2026
	DRAWN	PE	10/09/2026
	CHECKED	LM	10/09/2026
	APPROVED	MP	10/09/2026
SCALE	1:4,000 @A1	1:8,000 @A3	REVISION
STATUS	FOR CONSENT		B
PROJECT	16013		
DWG NO	16013-00-RC-1010		

0 80 200 400 SCALE (m) 1:4000 @A1 1:8000 @A3

Appendix B: Pre- and Post-Development Catchment Areas



0 200 400 m

METRES (A3)
SCALE : 1:7,000

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-	DRAFT	22/05/26	JA
NO.	REVISION	DATE	BY



memorandum



TO	Tim Carter	FROM	Philip Claassens
	Carter Group Limited	DATE	4 June 2026
RE	531 & 535 Mill Road – Water Quality Assessment		

1.0 Introduction

Carter Group Limited (CGL) is proposing a new residential development at 531 and 535 Mill Road, Ōhoka, Canterbury. The proposed development is located between Bradleys Road and Whites Road, directly south of Mill Road, comprising a total area of approximately 155 ha.

Pattle Delamore Partners Limited (PDP) has been engaged by CGL to assess the stormwater effects of the proposed land use change and development to support the fast-track application. This memorandum has been prepared to advise on the expected contaminants entrained in the stormwater runoff from the site, the expected removal of contaminants through the proposed stormwater management and treatment system, and the eventual contaminant concentrations expected after reasonable mixing with background flows.

This memorandum should be read in conjunction with the PDP report for stormwater management, titled “Fast Track Application – Stormwater Management for 531 and 535 Mill Road, Ōhoka” (report C045180001R002) (PDP, June 2026).

2.0 Proposed Stormwater Treatment System

The proposed development is for mixed land use, primarily consisting of new residential property with varying lot sizes. A breakdown of this land use is presented in Table 1 (separated by individual stormwater catchments). The associated treatment proposed is also included in this table.

Table 1: Post-development Catchment Characteristics and Proposed Treatment

Catchment	Land Use	Area (ha)	Proposed Treatment
1	Residential housing	9.4	First flush basin + wetland
2A	Rural-residential housing	23.4	Swales + detention basin
2B	Rural-residential housing	15.3	Swales + detention basin
3	Residential housing	9.7	First flush basin + wetland
4	Residential housing	9.8	First flush basin + wetland, rain gardens
5A	Residential housing	14.3	First flush basin + wetland, rain gardens
5B	Residential housing	1.4	Swales + detention basin
6	Residential housing Retirement village ¹	12.0	Swales + attenuation via offset with Catchment 10 ²
8	Residential housing	3.6	First flush basin + wetland
9A	Residential housing	3.0	First flush basin + wetland
9B	Business / commercial Property	1.2	Proprietary device ³ + detention basin
9C	Residential housing	9.4	First flush basin + wetland
10	Polo fields	8.9	Detention only ⁴
11	Residential housing	1.9	Rain gardens
Stormwater Management Areas and Drainage Corridors		23.6	N/A
Total		154.7	
Notes: - Stormwater from the dedicated retirement village area is to be managed within its internal footprint (future development, by others). - Detention for catchment 6 will be located in catchment 10. - For the treatment of stormwater runoff from commercial areas, treatment will be provided by a proprietary filter system. - No treatment infrastructure is expected to be required for this area; however, appropriate attenuation will be located here.			

All stormwater treatment has been designed in line with:

- ✧ Acceptable practice as per local engineering standards and guidelines, and
- ✧ Local authority requirements.

For further information on the design of the stormwater systems and their location within the development, please refer to “Fast Track Application – Stormwater Management for 531 and 535 Mill Road, Ōhoka” (report C045180001R002) (PDP, June 2026).

3.0 Expected Contaminants

The expected stormwater contaminants from a residential subdivision development are:

- ✧ Sediment (measured as Total Suspended Solids (TSS) - from atmospheric deposition and vehicle traffic).
- ✧ Heavy metals such as zinc (from tyres), copper (from brake linings and tyres), and lead (from tyre balance weights). Other heavy metal forms (such as cadmium, chromium, mercury, or nickel) are considered unlikely to be present due to the absence of industrial activity.

In addition to the aforementioned, stormwater runoff may also contain:

- ✧ Nutrients (nitrogen and phosphorus) from various activities.
- ✧ Hydrocarbons (typically measured as Total Petroleum Hydrocarbon (TPH)) from engine leaks, vehicle transmissions, and exhaust fumes.
- ✧ *E. coli* from natural animal activity.

To characterise stormwater from such a catchment, previous data from other similar developments must be relied upon to inform expected contaminant concentrations.

Contaminant concentrations have been inferred based on a local (Canterbury-specific) study of stormwater quality from modern subdivisions. This was primarily from a study completed by PDP, Selwyn District Council (SDC), and Christchurch City Council (CCC) and is summarised as a paper presented to the Water New Zealand Stormwater Conference in 2012 (PDP, SDC, CCC, 2012). The results of this study (shown in Table 2 below) are expected to best characterise the stormwater from this development.

Table 2 does not include recommendations for hydrocarbons. There is limited data available in Canterbury for Total Petroleum Hydrocarbons (TPHs) and Polycyclic Aromatic Hydrocarbons (PAHs) in urban stormwater runoff; however, Pasco (2005) reported TPH concentrations were <0.2 mg/L in stormwater entering a basin in new residential subdivisions in the Upper Heathcote (Kennedy, Allen, & Wilson, 2016). Analysis of stormwater sampling of a University of Canterbury carpark, undertaken by Adams *et al.* (2007), identified no detectable PAHs (Kennedy, Allen, & Wilson, 2016). Based on the above, it is expected that the concentration of hydrocarbons entrained in stormwater runoff from the proposed development will be low/non-detectable.

Where contaminant values have not been stated, concentrations have been stated in the CCC Waterways and Wetlands Drainage Guide (WWDG), table 6-3, have been used (CCC, 2012).

Table 2: Expected Contaminant Concentrations in Development Stormwater Runoff.

Parameter	Concentration
Total Suspended Solids (TSS) ¹ (mg/L)	70
Total Copper ¹ (mg/L)	0.007
Total Lead ¹ (mg/L)	0.004
Total Zinc ¹ (mg/L)	0.08
Total Nitrogen ¹ (mg/L)	1.0
Total Phosphorus ² (mg/L)	0.26
Hydrocarbons ³ (mg/L)	-
<i>E. coli</i> ⁴ (MPN)	-
Notes: - As per a study by PDP, SDC, and CCC. - As per the CCC WWDG table 6-3. - The concentration of hydrocarbons entrained in stormwater runoff from the proposed development is expected to be low/non-detectable. - Not stated in any of the above.	

4.0 Expected Treatment Removal Efficiency

The contaminant loadings detailed above will be reduced via the proposed stormwater treatment within each catchment. The WWDG guides potential urban pollutant removal capabilities of some of the proposed treatment systems. These are presented in Table 3.

Table 3: WWDG - Representative Removal Capability of Treatment Systems for a Number of Urban Pollutants of Concern. Adapted from Auckland (Hartwell & Silyn-Roberts, 2002) and International (Schueler, 1987) Data.

Treatment System	Pollutant Removal Efficiency (%)						Comment
	Solids	Phosphorus	Nitrogen	BOD	Trace Metals	Bacteria	
Grassed Swale	20-60	20-40	20-40	20-40	20-60	20-40	High potential for re-suspension of sediment with any storm flow.
Soakage Basin	60-100	40-80	40-80	20-60	40-100	60-100	Dependent on extent of overflow permitted.
Dry Detention Basin	40-80	40-60	20-40	20-40	20-60	0-40	Efficiency in trace metal reduction is reduced for more soluble elements.
Extended Detention Wet Pond	60-80	40-80	40-60	20-60	40-80	40-80	Sizing relative to runoff is volume dependent. Bacteria removal dependent on bird population for the system.
Wetlands	60-80	40-80	20-60	20-40	40-80	60-100	Removal includes soluble trace metals. Bacteria removal dependent on bird population attached to the system.

Note: The level of pollutant removal will be subject to the level of provision of treatment system volume or surface areas relative to catchment runoff. As a general rule, the higher the concentration of in-flowing pollutants, the greater the degree of removal.

Treatment performances for other systems proposed but not included in the above are included in Table 4 below.

Table 4: Potential Removal Efficiencies of Stormwater Treatment Devices

Treatment Device	Pollutant Removal Efficiency (%)			
	TSS	Phosphorus	Nitrogen	Trace Metals
Proprietary Device ¹	80 - 85	50 - 70	20 - 40	75-95 ²
Rain Gardens ³	90	70 - 89	47	80 - 90

Notes:

1. Stormwater360 StormFilter[®] device has been used for comparison purposes. Proprietary device removal efficiencies will vary widely, depending on their designed target contaminant.
2. Varies depending on the metal; however, the lower and upper range of removal efficiencies for total copper, total lead, and total zinc are presented. Negative percent removals are common for treatment devices that are not designed to remove the dissolved fraction of constituents such as nutrients and metals (NJCAT, 2012).
3. Filterra[®] has been used for comparison purposes (Stormwater360, 2025).

The removal efficiencies of TPHs and PAHs through these systems are not well documented. However, hydrocarbons entrained in the stormwater are expected to be captured in the upstream stormwater infrastructure (e.g., via the use of roadside sumps with submerged outlets).

5.0 Expected Discharge Quality

Based on the average (mean) removal efficiencies of the proposed treatment devices for the proposed development, the expected discharge quality from the stormwater catchments is presented in Table 5 below. It is noted that removal efficiencies can be influenced by a variety of factors such as the design and construction treatment device, the type and condition of vegetation, the climatic conditions, and pollutant loading.

The expected discharge quality from the proposed catchments has been assessed against Environment Canterbury's Land and Water Regional Plan (LWRP) contaminant limits for discharges to surface water.

Table 5: Expected Discharge Quality from Proposed Treatment Systems

Treatment System	TSS mg/L	Total Nitrogen mg/L	Total Phosphorus mg/L	Total Zinc mg/L	Total Copper mg/L	Total Lead mg/L
Swales + detention basin Applies to catchments: 2A/2B/5B/6/10	12.6	0.35	0.07	0.0192	0.0017	0.0010
First flush basin + wetland Applies to catchments: 1/3/4/5A/8/9A/9C	6.3	0.30	0.04	0.0128	0.0011	0.0006
Proprietary device + detention basin Applies to catchments: 9B (Commercial area)	2.1	0.31	0.05	0.012	0.0011	0.0010
Rain gardens Applies to catchments: 4/5/11	7.0	0.53	0.05	0.012	0.0011	0.0006
LWRP Limits¹	50	-	-	0.0080	0.0014	0.0034

Notes:

- Limits for heavy metals as per the LWRP Schedule 5, 95% species protection level for 'Spring-Fed – Plains'. TSS limit as per Rule 5.95 Condition 5(a) of the LWRP, which applies to the proposed stormwater discharge activity.
Bold values and red boxes indicate an exceedance of LWRP limits.

Commentary on the LWRP limits and applicability to the site is provided below:

- ✧ Schedule 8 limits apply for the discharge of stormwater from a reticulated stormwater system onto or into land or into or onto land in circumstances where a contaminant may enter water. However, these do not state limits for key contaminants of concern. Schedule 5 limits have therefore been used for this work.
- ✧ The Ōhoka Stream and its tributaries are spring-fed waterways; therefore, the catchment is classified as 'spring-fed-plains' under the LWRP. For stormwater discharges to spring-fed waterways, a TSS limit of 50 mg/L applies under the LWRP. Schedule 5 limits apply to other water quality parameters.
- ✧ In accordance with Table S5A of the LWRP Schedule 5 (Mixing Zones and Receiving Water Standards), a 95% species protection level is required for toxicants.

The LWRP Schedule 5 95% species protection limits are as per the Australian and New Zealand Guidelines for Fresh and Marine Water Quality (ANZG) (2018). These are chronic limits and are intended to impact species that are exposed to contaminants for long periods or on a frequent basis (refer to point 2 below). Applying these limits is a conservative approach to assessing water quality impacts for the treated stormwater as:

- The period where water quality is at the levels stated within Table 5 will be minimal, as flows will quickly be replaced with flow from upstream catchments (at a quality reflecting current conditions).
- The frequency at which water quality is at the levels stated within Table 5 will be limited to days with rain.
- ✧ The LWRP Schedule 5 95% species protection limits apply to water quality in the receiving environment beyond the mixing zone (i.e., after reasonable mixing with receiving environment background flows).

The LWRP states that *"the area (and underlying volume) of a receiving water where the water quality standards specified for rivers, artificial watercourses and lakes do not have to be met is referred to as the Mixing Zone"*.

The mixing zone, as a result of a point source discharge of a contaminant, is for river and artificial watercourse locations with flowing water present at all times;

a) no longer than 200 m along the longest axis of the zone, and

b) occupies no greater than two-thirds of the wetted channel width at the estimated 7DMALF for that location; and

c) no longer than 10 times the wetted channel width at the estimated 7DMALF for that location.

It is assumed that the above applies to all channels at discharge locations from the subdivision, assumed to have flowing water present at all times. It is therefore considered reasonable that significant mixing of the treated stormwater will occur, further diluting contaminant loads.

6.0 Receiving Environment Quality

An important consideration for assessing the impact of development runoff after a reasonable mixing is the quality of the receiving environment's background flows. Available water quality data is limited, with currently only one round of water quality tests completed for the site by Instream Consulting Ltd. Tests were completed under baseflow conditions in the Ōhoka Stream during fine weather, reflecting the condition of no stormwater runoff from the existing catchment.

A results summary of this sampling is shown below in Table 6. For more information on this sampling, please refer to the report “Fast Track Application – 531 & 535 Mill Road - Aquatic Ecology Assessment” by Instream Consulting Ltd.

Table 6: Summary Surface Water Quality Results (as provided to PDP by Instream Consulting Ltd, 01/05/2025).

		Ōhoka Stream Tributary		Southern Groundwater Seep	Northern Spring Channel		Central Spring Channel	South Branch Ōhoka
Parameter	Unit	Upstream	Downstream	Downstream	Upstream	Downstream	Downstream	Downstream
Total Suspended Solids	mg/L	<3	<3	<3	23	<3	<3	<3
Total Nitrogen	mg/L	5.4	5.1	5.1	6.2	3.0	1.75	4.0
Nitrate Nitrogen	mg/L	<u>5.2</u>	<u>4.8</u>	<u>4.8</u>	<u>5.1</u>	2.4	1.22	<u>3.4</u>
Nitrite Nitrogen	mg/L	0.009	0.049	0.030	0.011	0.017	0.040	0.025
Total Ammoniacal Nitrogen	mg/L	<0.010	0.049	0.053	<0.010	0.034	0.040	0.018
Dissolved Inorganic Nitrogen*	mg/L	5.214	4.898	4.883	5.116	2.451	1.3	3.443
Total Phosphorus	mg/L	0.004	0.052	0.084	0.003*	0.020	0.061	0.074
Dissolved Reactive Phosphorus	mg/L	<0.004	<u>0.041</u>	<u>0.063</u>	<0.004	<u>0.020</u>	<u>0.047</u>	<u>0.060</u>
Dissolved Copper	mg/L	<0.0005	<0.0005	0.0006	<0.0005	<0.0005	<0.0005	0.0005
Dissolved Lead	mg/L	<0.00010	<0.00010	<0.00010	<0.00010	<0.00010	<0.00010	<0.00010
Dissolved Zinc	mg/L	<0.0010	<0.0010	<0.0010	<0.0010	0.0014	<0.0010	0.0010
Biochemical Oxygen Demand	mg/L	<2	<2	<2	<2	<2	<2	<2
Chemical Oxygen Demand	mg/L	<6	<6	12	30	10	8	8
<i>Escherichia coli</i>	MPN/100 ml	350	>1,600	170	5,400	130	170	350*

Notes:

Underlined represents concentration above the NSPFM national bottom line or lowest Attribute Band. Shaded cells represent exceedance of LWRP Schedule 8 limits, noting these are for annual median not one-off samples. * represents laboratory error in processing time, therefore the result is inconclusive.

The following conclusions have been drawn by PDP based on the above water quality results for baseflow conditions:

- ✧ Baseflows have very low suspended solids content, with all samples returning TSS values below 3 mg/L. Water is expected to have very high clarity as a result. Downstream users will likely notice any changes in clarity from an aesthetic viewpoint, making this a key parameter to be met.
- ✧ Baseflows have moderate levels of nitrogen forms already present. Levels of nitrate upstream and downstream of the site are likely indicative of a high level of nitrate in the shallow groundwater discharging into the streams via springs and the gravel base of the stream channels. An increase in Ammoniacal Nitrogen from upstream sites to downstream sites is likely indicative of the agricultural activities on the site and the presence of livestock in the vicinity of the stream channels.
- ✧ Baseflows show some phosphorous being present. Levels seen are approximately the same as the concentration expected to be discharged from the development (refer to Table 5).
- ✧ Baseflows have very low copper amounts present, with all samples registering as either below 0.5 micrograms (assumed to be below laboratory detection limits) or just above. All samples are below the above-stated LWRP limit. This would indicate that baseflows will provide good dilution of stormwater runoff for copper effects.
- ✧ Baseflows have very low lead amounts present, with all samples registering as below 0.1 micrograms (assumed to be below laboratory detection limits) and below the above-stated LWRP limit.
- ✧ Baseflows have very low zinc amounts present, with all samples registering as either below 1 microgram (assumed to be below laboratory detection limits) or just above. All samples are below the above-stated LWRP limit. This would indicate that baseflows will provide good dilution of stormwater runoff for zinc effects.

The above results are indicative of baseflow conditions. During storm events, when flows in the streams on site will be significantly elevated, it is expected that all of the above parameters will also be elevated to varying degrees.

A toxicant mixing assessment has been undertaken to assess the water quality beyond the mixing zone.

7.0 Mixing Assessment

7.1 Overview

A mixing assessment has been completed based on the available information, as described in the aforementioned section, by PDP to advise the water quality within the Ōhoka Stream Tributary after reasonable mixing has taken place.

The proposed development area that discharges to the Ōhoka Stream Tributary has been used as an example for discussing and assessing reasonable mixing. The Ōhoka Stream Tributary was chosen to be assessed as the water body receives the largest area of the subdivision treated stormwater and includes all types of proposed treatment presented in section 2.0. Discharge to this waterway is therefore viewed as being representative of expected discharges (i.e., stormwater quality) from the site.

Reasonable mixing with the Ōhoka Stream Tributary is influenced by the following:

- ✧ **Treated Stormwater:** The expected post-treatment water quality and quantity from the proposed development discharging into the Ōhoka Stream Tributary.

Of the contributing subdivision area (42.8 ha) that drains to the Ōhoka Stream Tributary:

- 26.7 ha is managed and treated by first flush basins and wetlands. These retain water-quality flows, discharging over four days. The slow and steady discharge will result in high levels of mixing with background flows.
- 14.9 ha is managed by treatment swales and rain gardens. Discharge from this area will have less mixing available as there is no restriction in flow (i.e., no detention within the device). Mixing will be governed by the relative times of concentration for each point of discharge relative to the expected flow entering the Ōhoka Stream Tributary from the upstream contributing catchment and the drainage corridor along the tributary.
- 1.2 ha is managed by proprietary devices. As with swales and raingardens above, there is no restriction on flow. Mixing will be governed by the relative times of concentration for each point of discharge relative to the expected flow entering the Ōhoka Stream Tributary from the upstream contributing catchment and the drainage corridor along the tributary.

Additional contributing areas include stormwater management areas, drainage corridors, and Catchment 10 (the polo field area), which drain directly to the Ōhoka Stream Tributary and are assumed to have clean runoff.

- ✧ **Baseflow:** The baseflow (i.e., flow rate) present in the receiving water body (i.e., Ōhoka Stream).

Based on gauging of the Okoka Stream, undertaken at the site between February and November 2025, a range of baseflows in the stream have been identified. As would be expected for a spring fed stream, baseflows in winter are significantly higher than baseflows in summer. Based on the gauging undertaken in February and November only, an average baseflow of 59.5 L/s has been applied to this assessment.

- ✧ **Receiving water quality:** The quality of the receiving water body at the time of discharge.

The water quality of background flows was based on the water quality analysis completed by Instream Consulting Limited (Refer to Section 6.0). As most samples for zinc, copper, and lead were below detection, the background flow water quality was assumed to be clean, allowing for full dilution.

- ✧ **Time of entry:** The time of entry at which the runoff from the subdivision enters the receiving waterway (i.e., the time of concentration) relative to the wider contributing catchment.

7.2 Assessment

The treated water quality runoff from the proposed development and the receiving water body quality and flows are described in Sections 5 and 6. To assess the impact of the proposed development, treated runoff discharge into the Ōhoka Stream Tributary, the time of entry (time of concentration) for treated runoff from the proposed development, and the coinciding flows within the Ōhoka Stream Tributary are required.

The effect of the various contributing catchments' time of concentration is best viewed by comparing the runoff expected from the development with the runoff expected from upstream and clean area flows. An example storm runoff hydrograph is shown in Figure 1. For more detailed information on the various catchment time of concentrations within the development, refer to Fast Track Application – Stormwater Management for 531 and 535 Mill Road, Ōhoka" (report C045180001R002) (PDP, June 2026).

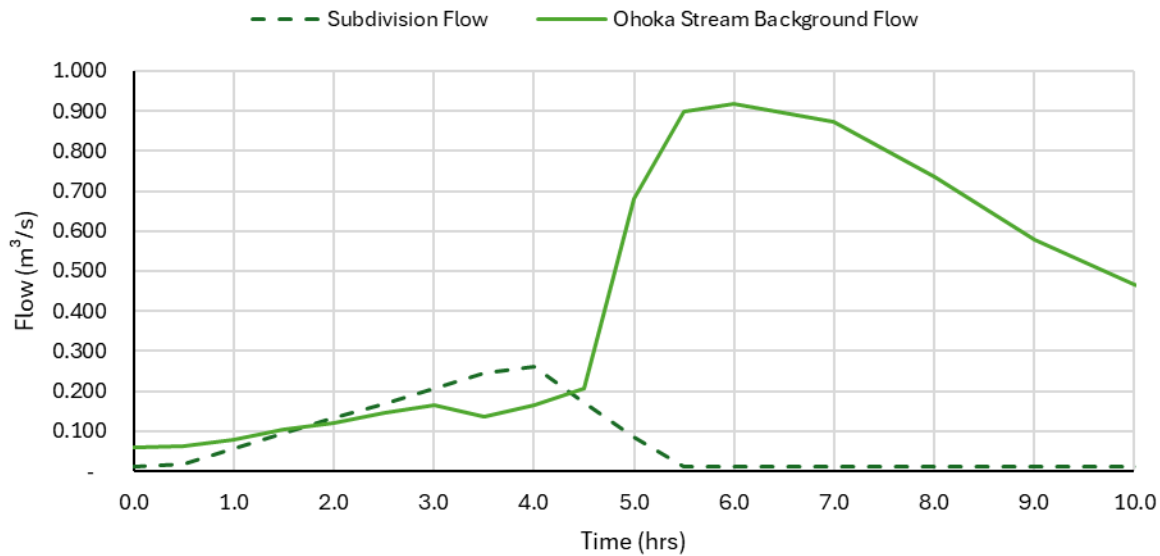


Figure 1: Example storm hydrograph.

The following was used to develop the hydrographs:

Stream Flow Hydrograph

- ✧ Based on gauging of the Ōhoka Stream by PDP, undertaken at the site between February and November 2025, a range of baseflows in the stream have been identified over various seasons. As would be expected for a spring fed stream, baseflows in winter are significantly higher than baseflows in summer. Based on the gauging undertaken in February and November only, an average baseflow in the Ōhoka Stream of 59.5 L/s has been applied to this assessment.
- ✧ A design storm (in line with CCC WWDG guidance – 25 mm depth, 5 mm/hr average intensity and a triangular hyetograph profile with a peak occurring at 0.7 times duration) was used as an intended “worse-case” scenario from a dilution perspective. The above design storm was entered into the PDP flood model (further described as "Flood Effects - 535 Mill Road, Ōhoka" (report C045180001R001)) to estimate the additional upstream contributing flows draining to the Ōhoka Stream Tributary during a storm event.

Subdivision Flow Hydrograph

- ✧ The above design storm was used to estimate flows from the respective development catchments draining to the Ōhoka Stream Tributary.
 - Catchments with wetlands were assumed to discharge over 4 days (as per design). The discharge rate into the Ōhoka Stream Tributary was thus based on the first flush from the catchment, discharging at a constant rate over 4 days.
 - Catchments with no flow restrictions (treated via swales, raingardens, and proprietary devices) will have a peak discharge associated with the time of concentration for the contributing catchments. This runoff is expected to be proportionately larger than runoff from a catchment area treated via a wetland.

Note that it was assumed for the mixing assessment that no flows are attenuated within attenuation basins. This is a very conservative assumption as the estimated peak stormwater discharge rates for this type of event are expected to be significantly buffered by passage through the first flush basins, wetlands and attenuation basins.

7.3 Findings

As shown in Table 5, only copper and zinc exceed the LWRP limits and therefore require mixing with the Ōhoka Stream Tributary. The minimum mixing needed to meet all limits is approximately 1.4 parts background flow per 1 part of catchment runoff, based on the highest zinc concentration (0.0192 mg/L) from swale + detention basin treatment system runoff.

This ratio was calculated using a mass balance approach, with the LWRP zinc limit (0.008 mg/L) as the target. Solving for the required dilution shows that approximately 1.4 parts of stream flow are needed to reduce zinc concentrations in the treated runoff to compliant levels.

It can be observed in Figure 1 that this would result in a brief period of approximately 4-hour period over which adequate mixing to meet the LWRP limits is not expected to be achieved within the Ōhoka Stream Tributary. After 4 hours, it would be expected that the upstream contributing catchments would provide sufficient mixing to dilute the contaminant load from the proposed development to within LWRP limits. As noted above, this type of event has been deliberately chosen as a worst-case scenario in terms of lower summertime baseflows and intensity of runoff into the receiving environment to contribute peak load in a relatively short event. For most typical rainfall events, the discharge would be expected to meet the LWRP limits as:

- ✧ The design storm chosen was considered to be quite intense for a storm with a sub-yearly annual return interval.
- ✧ Shorter duration storm events are unlikely to generate a lesser result for the Ōhoka Stream Tributary water quality, as not all contaminant loads will be mobilised.
- ✧ A longer duration storm event will result in less influence of different times of concentrations between the development and the wider catchment (i.e., flows from the subdivision and upper catchment will be more likely to coincide).

8.0 Conclusion

The stormwater discharge quality from the site is expected to meet the LWRP TSS limit of 50 mg/L (for spring-fed waterways) and the 95% species protection limit for total lead of 0.0034 mg/L.

However, the stormwater discharge quality is not expected to directly meet the 95% species protection limits for total zinc and total copper at all points of discharge. It is noted that the 95% species protection limits, on which the LWRP limits are based, are intended to be used as instream water quality targets, after reasonable mixing (i.e., downstream of the mixing zone). For the proposed development, there will be various discharge points into the tributaries of the Ōhoka Stream, and depending on the characteristics of each waterway, the extent of the mixing zones will vary.

It is also noted that the LWRP limits for which this assessment has compared the treated stormwater are the chronic values from the ANZG 2018 guidelines. This is a conservative approach for stormwater flows, which occur only during rainfall and for short periods before mixing with runoff from the wider catchment. Therefore, the brief and very occasional periods when the LWRP limits may be exceeded due to discharge, would not be expected to significantly impact on the health of the receiving environment. For further text on the potential effects, please see below an excerpt from the report "Fast Track Application – 531 & 535 Mill Road - Aquatic Ecology Assessment" by Instream Consulting Ltd, Section 7 - Discussion:

"The expected discharge quality has been reported to exceed the LWRP receiving water standards at the point of discharge for total zinc and copper. While copper concentrations are close to the standards and no adverse effects are expected, zinc has been assessed to not meet the standards for the faster released stormwater (from swale and proprietary devices), with a modelled time of up to 4 hours in a conservative assessment before adequate dilution will occur. To determine if adverse

effects were expected from this temporary discharge of stormwater, concentrations at the point of discharge were compared to the Interim Tier 1 acute GVs (Gadd et al 2024). These GVs are for short term effects (up to 96-hour exposure). When compared to these, total zinc falls within the 95% level of species protection for all treatment types prior to any mixing, therefore no adverse effects are expected to occur. Total copper concentration is below the acute GV for first flush basins and wetlands, but above the acute GV for the more immediate discharges. It is noted that this is prior to any mixing with the receiving environment and is a comparison to total concentration, not dissolved. When considering mixing with the receiving environment, and the conservative nature of the assessment, adverse effects are not expected."

Background water quality is high (i.e., low contaminants except for nutrients) under baseflow conditions. While this does not reflect the quality of receiving water during a storm event, it does indicate that in periods with higher baseflow, sufficient reductions of stormwater contaminant concentrations will be achieved after reasonable mixing.

9.0 References

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- PDP, SDC, CCC. (2012). *Stormwater Quality - An Analysis of Runoff from Modern Subdivisions and the Implications for Stormwater Treatment*. (p. 14). Water New Zealand.
- Stormwater360. (2025). *Filtterra® Bioscape Brochure*. Retrieved from <https://45193578.fs1.hubspotusercontent-na1.net/hubfs/45193578/Brochures/Filtterra%C2%AE%20Bioscape%20Brochure.pdf>

10.0 Limitations

This memorandum has been prepared by Pattle Delamore Partners Limited (PDP) on the basis of information provided by Carter Group Limited, Inovo Group, and Instream Consulting Limited. PDP has not independently verified the provided information and has relied upon it being accurate and sufficient for use by PDP in preparing the memorandum. PDP accepts no responsibility for errors or omissions in, or the currency or sufficiency of, the provided information.

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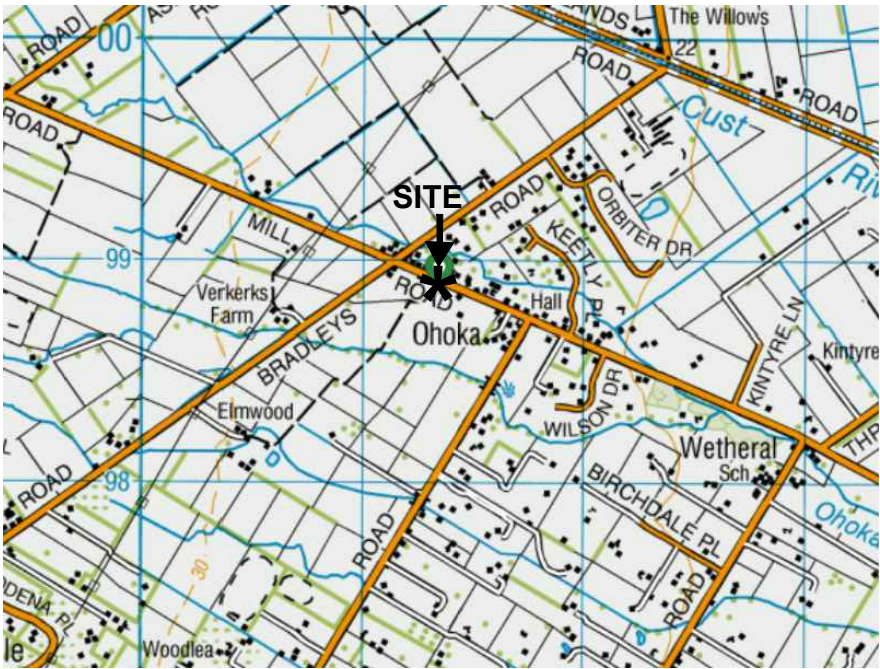
Eoghan O'Neill

Technical Director - Water Infrastructure



CARTER GROUP LTD

OHOKA WETLAND CONCEPT DESIGN



DRAWING REGISTER			
DRAWING No.	DRAWING TITLE	REV	ISSUE DATE
C04518-GN-001	DRAWING INDEX AND LOCALITY PLAN	A	APR 26
C04518-GN-002	GENERAL ARRANGEMENT PLAN	A	APR 26
C04518-GN-003	POND SMA 9C	A	APR 26
C04518-GN-004	OUTLET DETAILS	A	APR 26
C04518-GN-005	CROSS SECTION THROUGH WETLAND	A	APR 26
C04518-GN-006	TYPICAL DRAINAGE AND WETLAND DETAILS	A	APR 26

DRAWING ISSUE :

FOR REVIEW

DRAWING TITLE :

DRAWING INDEX AND LOCALITY PLAN

DRAWING NO. : REV :

C045180001-GN-001 A



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Telephone : +64 9 523 6900
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NO.	REVISION	DATE	APP.	



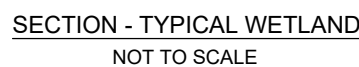
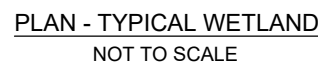
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GENERAL ARRANGEMENT PLAN				
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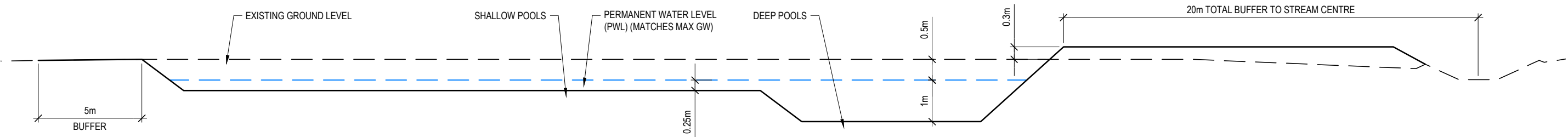
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2. ALL DIMENSIONS IN MILLIMETRE UNLESS SPECIFIED.
3. INTERNAL SIDE SLOPES BELOW THE PWL WILL BE LESS THAN OR EQUAL TO 1V:4H.
4. INTERNAL SIDE SLOPES ABOVE THE PWL WILL BE LESS THAN OR EQUAL TO 1V:3H.
5. FIRST FLUSH/WET POND SIDE SLOPE WILL BE LESS THAN OR EQUAL TO 1V:3H
6. EXTERNAL SIDE SLOPES WILL BE LESS THAN OR EQUAL TO 1V:3H.
7. ANY SLOPES THAT REQUIRE MOWING WILL BE LESS STEEP THAN 1V:3H.
8. A MINIMUM WIDTH OF 3.5m WIDE AND MAXIMUM SLOPE OF 1V:3H WILL BE PROVIDED AROUND THE WETLAND FOR MAINTENANCE REQUIREMENTS.
9. WETLAND OVERFLOW INVERT WILL BE 100mm ABOVE THE MAXIMUM WATER LEVEL IN THE WETLAND.

FOR REVIEW

[illegible]



SECTION A
GN-003
TYPICAL WETLAND CROSS SECTION
NOT TO SCALE

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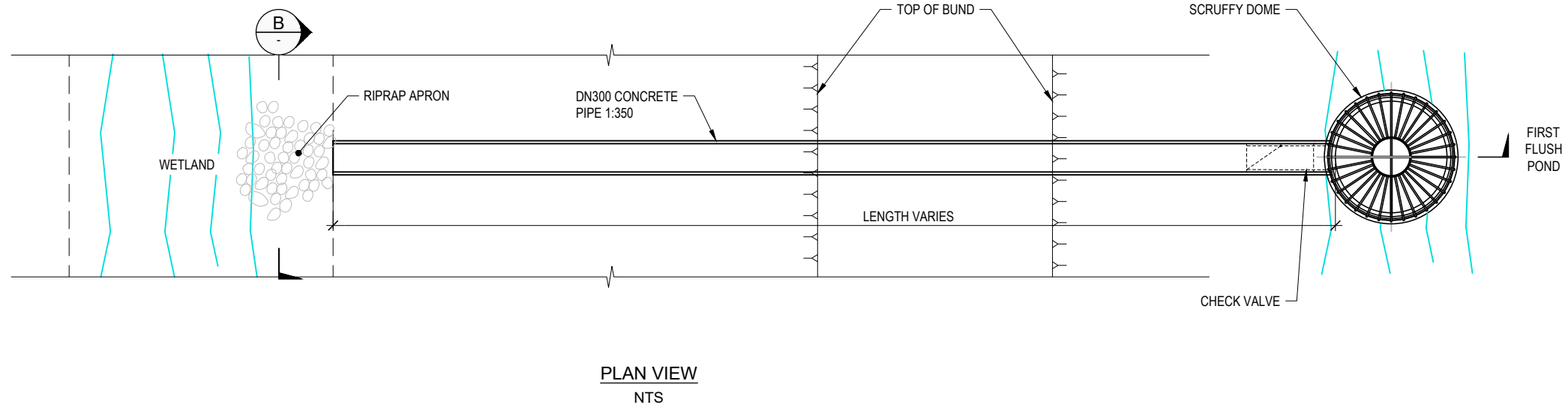
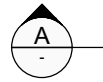
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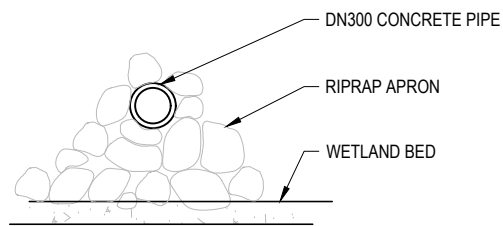
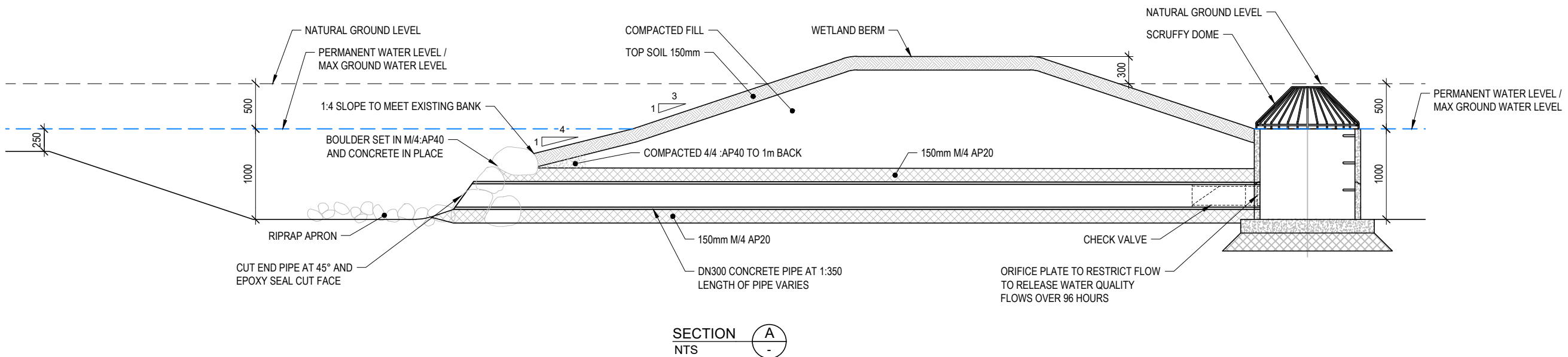
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OUTLET DETAILS		
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NOTES

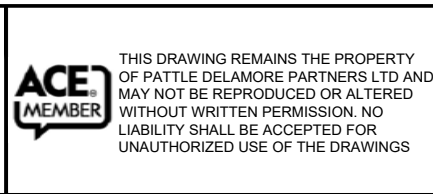
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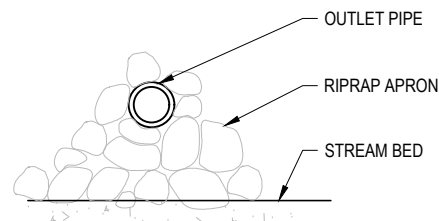
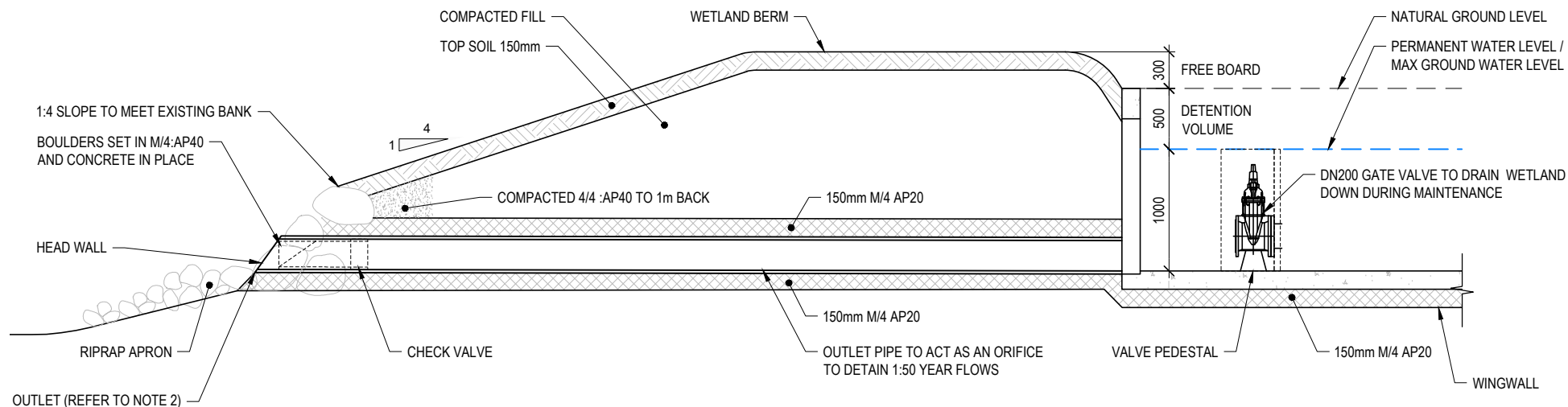
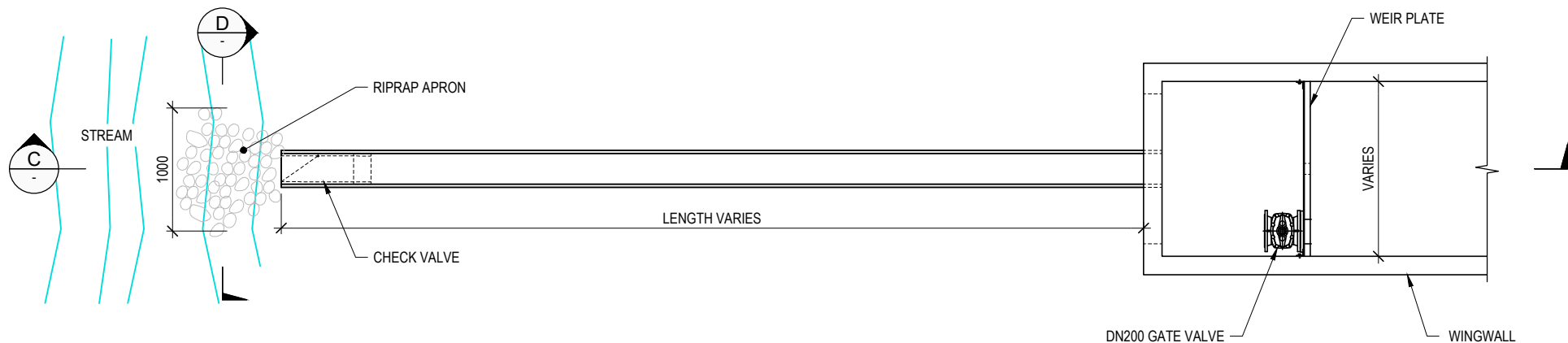
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CROSS SECTION THROUGH WETLAND		
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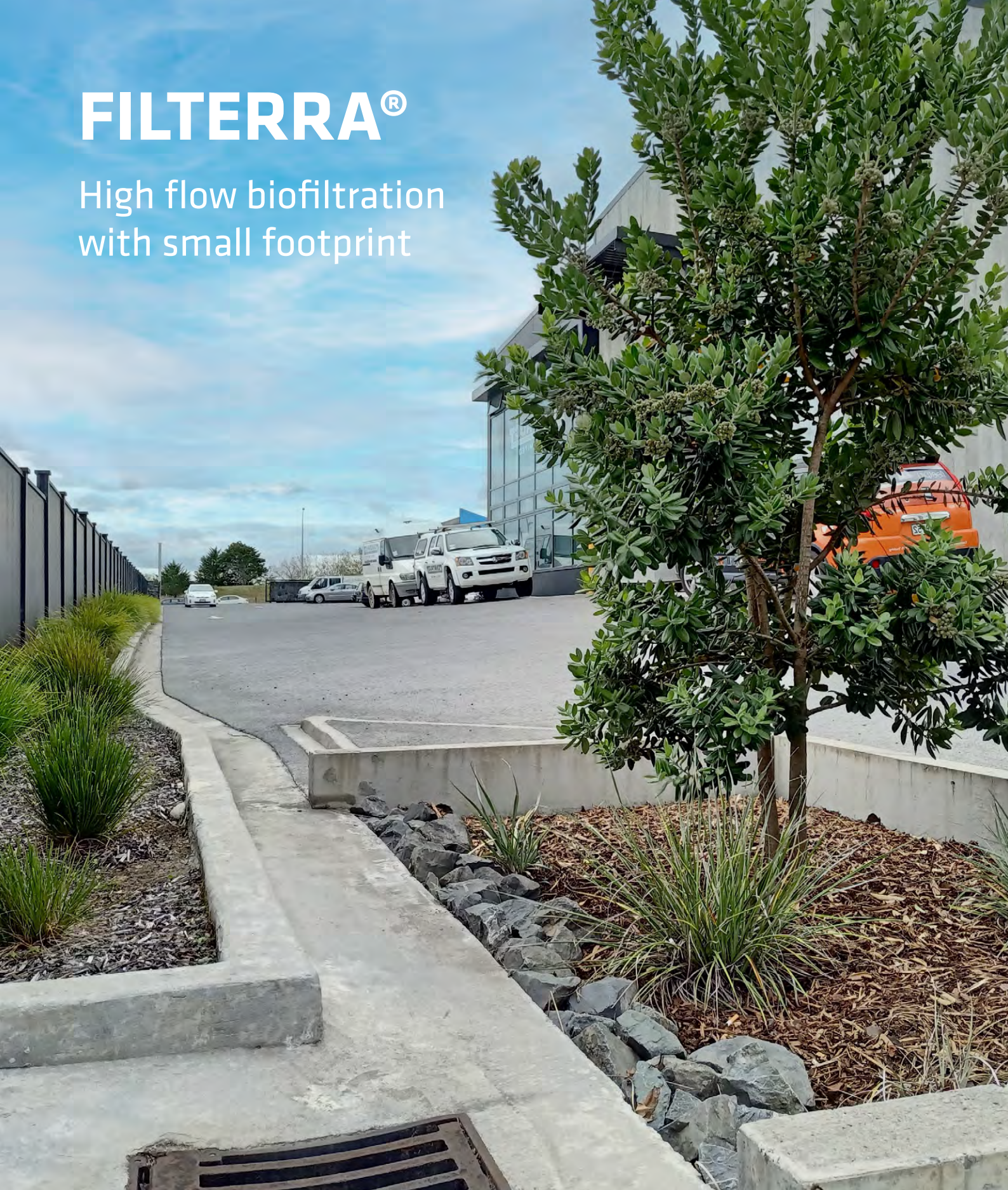
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PROJECT: OHOKA WETLAND CONCEPT DESIGN				
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AS SHOWN (A3)	C045180001-GN-006	A		

FILTERRA®

High flow biofiltration
with small footprint





WHAT IS FILTERRA®?

Green infrastructure uses natural processes to treat stormwater and provides environmental benefits. Developed after decades of research, Filterra® is a next-generation biofiltration solution, designed to provide enhanced treatment outcomes while requiring as little as 10% of the footprint of traditional rain gardens.

Filterra® is a pre-engineered and manufactured plug-and-play micro biofiltration system that removes TSS, metals and nutrients using natural, biological processes. Designed to treat up to 4450mm/hr and with a maximum treatable flow rate of 7000mm/hr the Filterra® media has been optimised to operate under high flow rates while maintaining high pollutant removal performance. The Filterra® engineered media is designed for low maintenance and does not need replacing under normal conditions, making it one of the most cost-effective treatment devices to maintain.

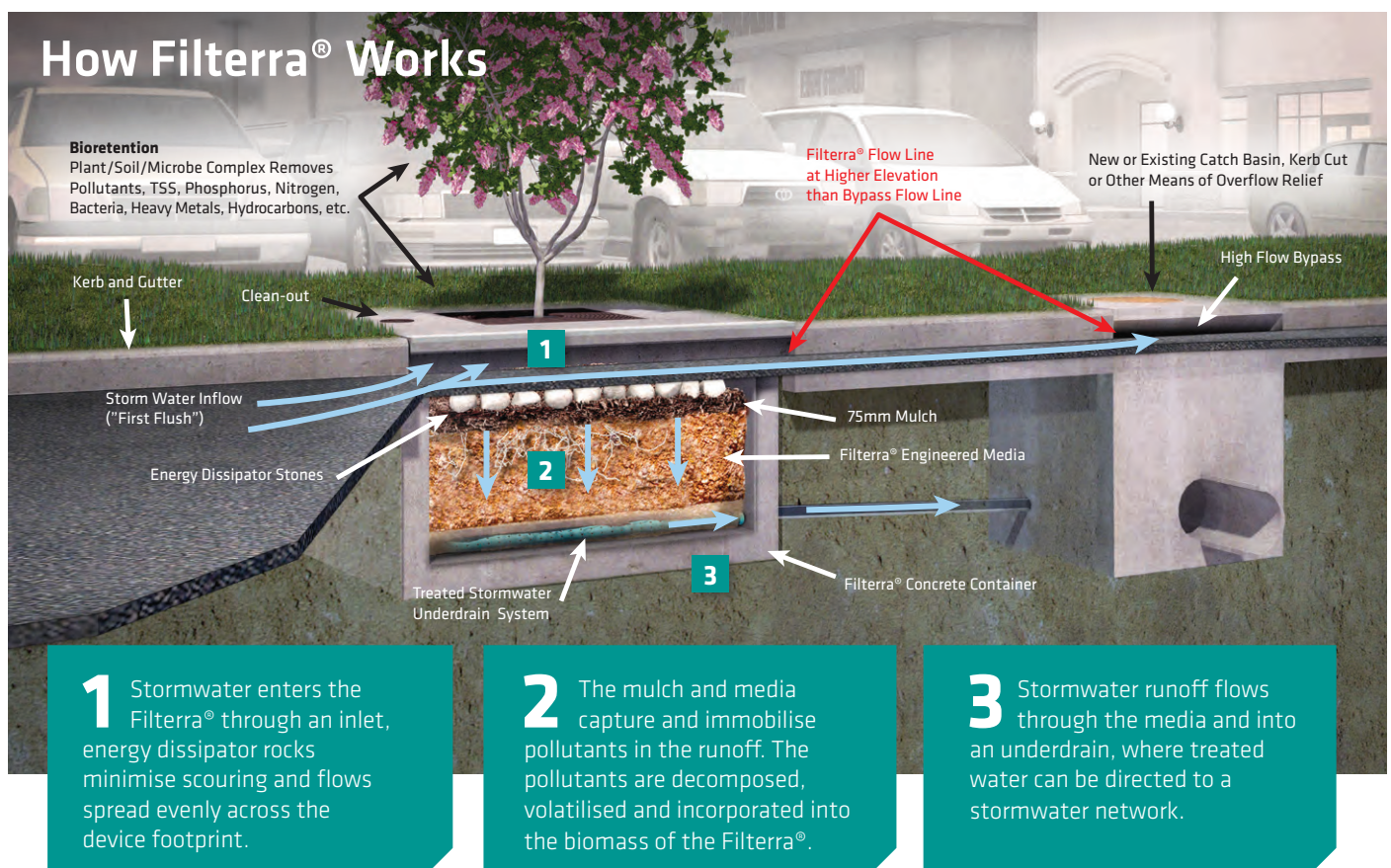
APPROVALS:

- ✓ Auckland ✓ Christchurch
- ✓ Accepted throughout New Zealand

Application

Ideally suited to land development projects, where economical treatment of large catchment areas is the main priority, Filterra® can provide treatment in a range of applications:

- » Subdivision roading
- » Local roads
- » JOAL/accessway treatment
- » Car parks for schools, commercial and retail
- » Industrial sites with elevated metals
(specialist design may be required)





BENEFITS OF FILTERRA®

Smaller Footprint

Stormwater360 work with the consultants and other project members to develop a preliminary design and functional green infrastructure design.

Increased Biodiversity

Manufactured in New Zealand from locally sourced materials, the Filterra® complements Low Impact Design (LID) principles. New Zealand native plant species promote biodiversity, community and cultural outcomes.

Easy Maintenance

Only the mulch layer needs to be replaced every 6-12 months, sourced from an approved third-party supplier. Under normal conditions the media and plants should not need replacing. Planting, commissioning and 12 months of maintenance are included in the supply of the Filterra®.

Cost Savings

A plug-and-play device with fewer, smaller units Filterra® can offer a substantial capital cost saving when compared to bespoke rain garden designs. Additionally, with no confined space requirements and off road solutions, there are further cost savings for stakeholders.

Independently Verified Life Cycle Cost data available on request.

Proven Performance

Filterra® engineered media undergoes at least 25 quality assurance assessments to ensure pollutant removal, hydraulic conductivity and performance over time. Each batch takes over a month to produce, and is guaranteed to not leach or transform contaminants such as phosphorous or copper. Plant trials have established successful plants for long term health of the system, and rigorous third-party tests in New Zealand, Canada, USA and Australia are available to demonstrate its TSS, metals and nutrient removal capabilities.



Based on a catchment area of 2000m² in Auckland, a 2% rain garden is at least 50m². The Filterra® required to treat all flows is 8.4m²: 4m x 2.1m



DESIGN

Filterra® shifts the boundaries of what was previously possible with green infrastructure. Stormwater360 can provide a customised solution for your project considering the design infiltration rate and safety factor for your land-use.

Our service includes all approvals, calculation sheets and drawings, and a range of tools are available to assist with your Filterra® design, contact us today for further information.

Planting, Operation and Maintenance

High flow bioretention plants are sourced and supplied with a 12-month warranty ensuring healthy and robust vegetation. Stormwater360 operate and maintain the Filterra® for the first 12 months.

UNITS ARE AVAILABLE IN A RANGE OF STANDARD SIZES AND CONFIGURATIONS:



MANHOLE FILTERRA®

The proven Filterra® technology is combined with the simplicity and ease of installing a manhole to create a versatile treatment solution that can easily be integrated into wider landscaping. Available in DN1050 and DN1200 options, Manhole Filterra® can be used where the supply of a precast vault is impractical or not required.



VAULT FILTERRA®

Standard Filterra® supply ranges from 1.2m x 1.2m, up to 4m x 2.1m. The treatable catchment area is up to 2,500m² in most locations, or 5,000m² in Christchurch. Vaults are supplied to the site pre-filled with media, the contractor simply needs to place the unit and connect to the network.

PLANTING PALETTE

Plant health is essential for the long-term performance and sustainability of biofiltration systems, and high infiltration rate soils can restrict plants' ability to grow.

With over 50 plants comprehensively tested by Stormwater360, we are confident that the New Zealand native species proposed by us will be successful in Filterra® installations. Our plant guide features a range of grasses and trees that align with local authority bioretention plant lists.

0800 STORMWATER (0800 786769)
www.stormwater360.co.nz

TREATMENT OUTCOMES

Stormwater treatment device efficiency is highly variable from site to site, and storm to storm. Particle size distribution, speciation and contaminant mix all influence pollutant removal efficiency and subsequent percentage removal.

Stormwater360's approach is to provide transparent independent test reports from New Zealand, Canada, USA and Australia, with third party verification, certification and approvals over a range of land uses and applications. Full test reports are available from Stormwater360.

Some field study highlights are as follows:

Washington Department of Ecology: Bellingham Study

90% TSS and 70% TP removal

Canterbury University Filterra® Roof Study

99% Total and Dissolved Zinc Removal

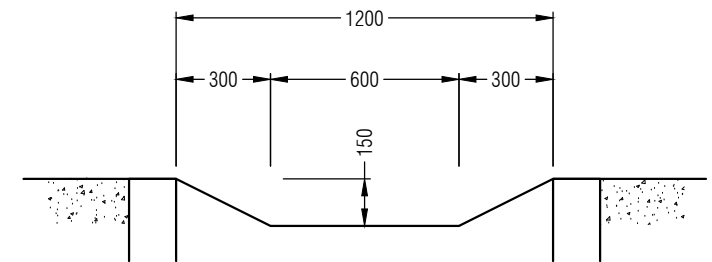
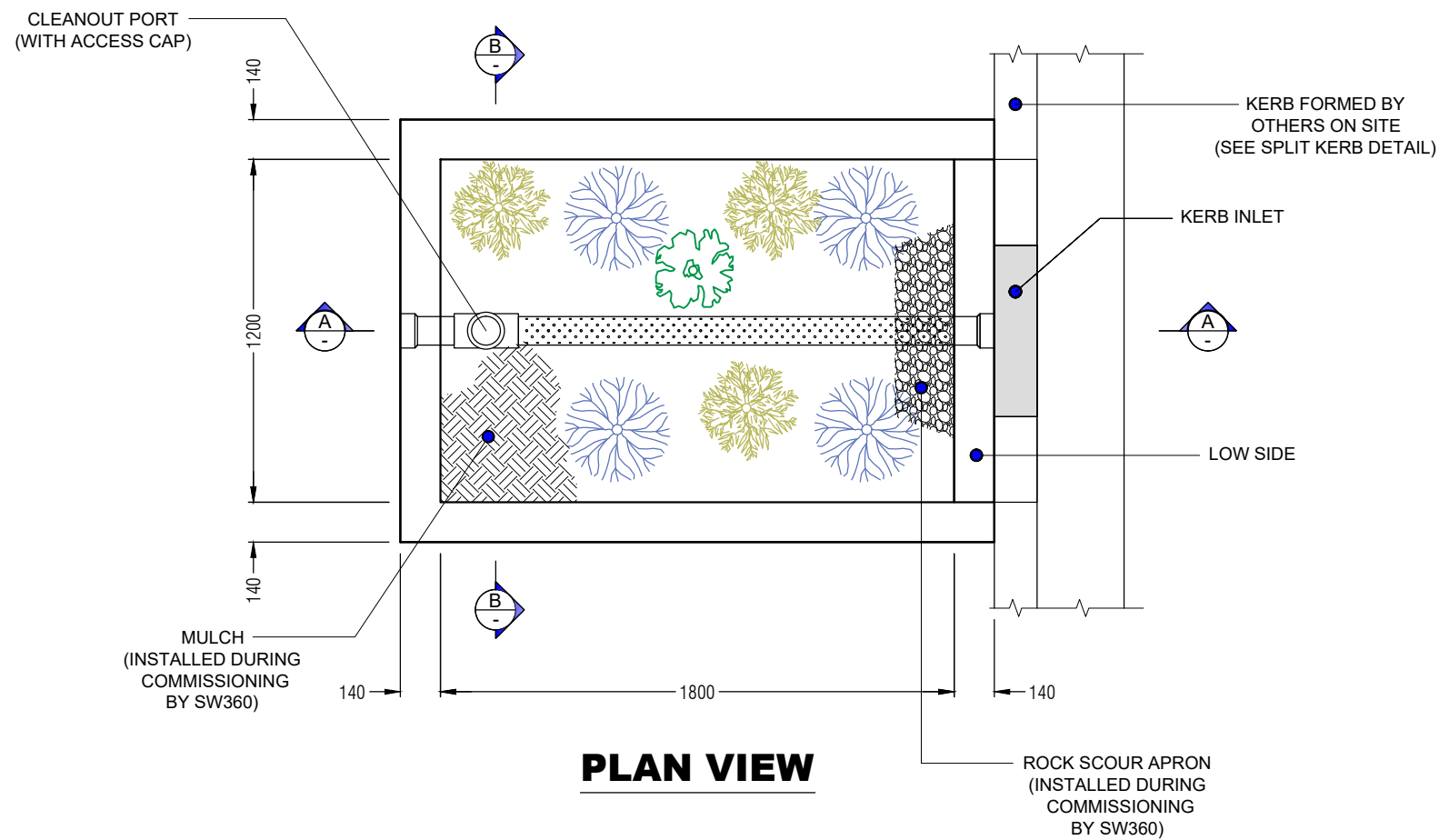
Washington Department of Ecology - ARMCO study

Total Copper 88.3%, Dissolved Copper 48.7%,

University of Western Sydney Study

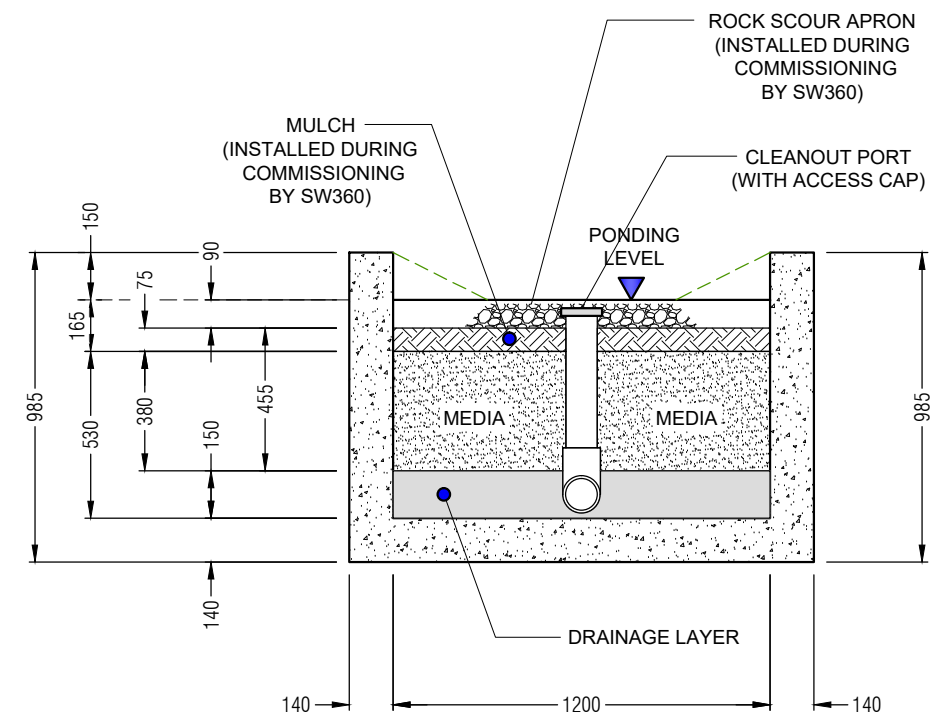
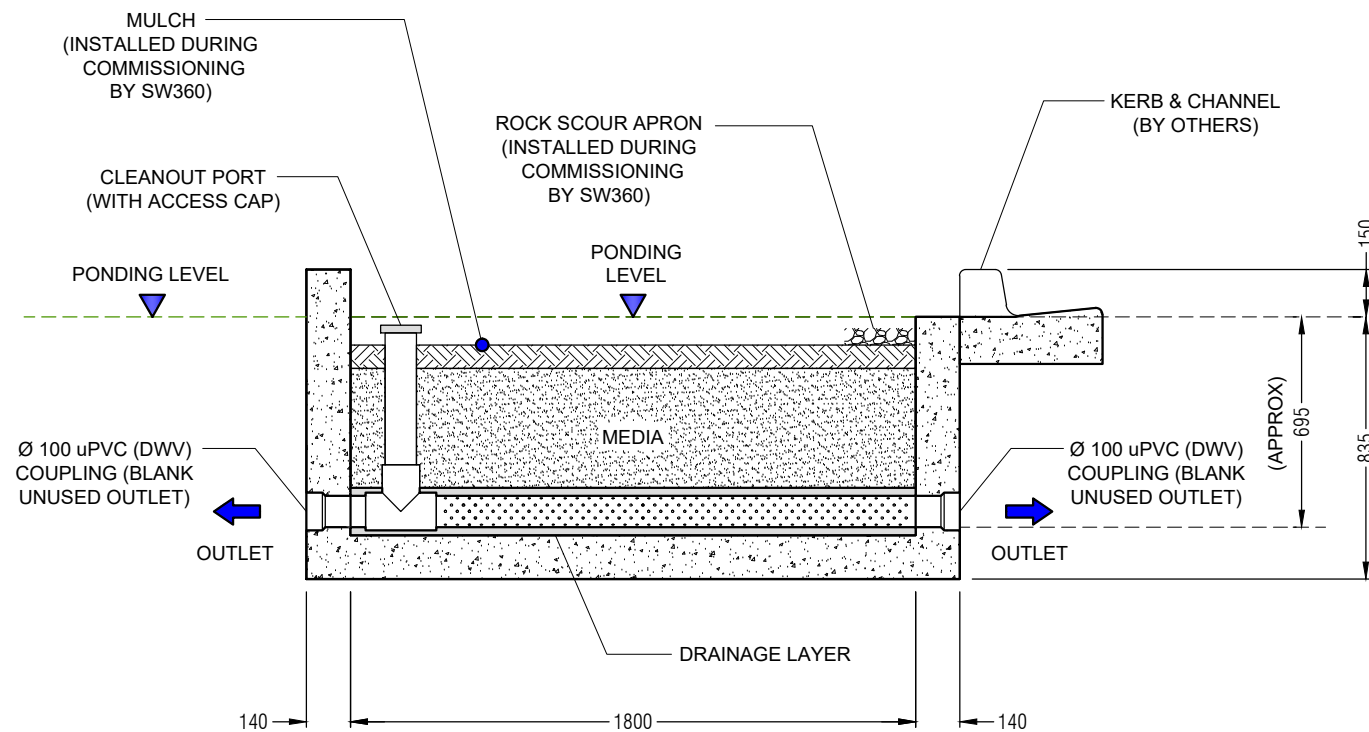
89% Total phosphorus and 47% Total Nitrogen Removal





SPLIT KERB DETAIL

RECOMMENDATION ONLY
(BY OTHERS)



ESTIMATED WEIGHT :
CONCRETE VAULT + MEDIA
+ DRAINAGE LAYER
MASS = 5400 Kg



THIS PRODUCT MAY BE PROTECTED BY ONE OR MORE OF THE FOLLOWING U.S. PATENTS: 6,277,274; 6,569,321; 7,625,485; 7,425,261; 7,833,412; RELATED FOREIGN PATENTS.



0800 STORMWATER

sales@stormwater360.co.nz

www.stormwater360.co.nz

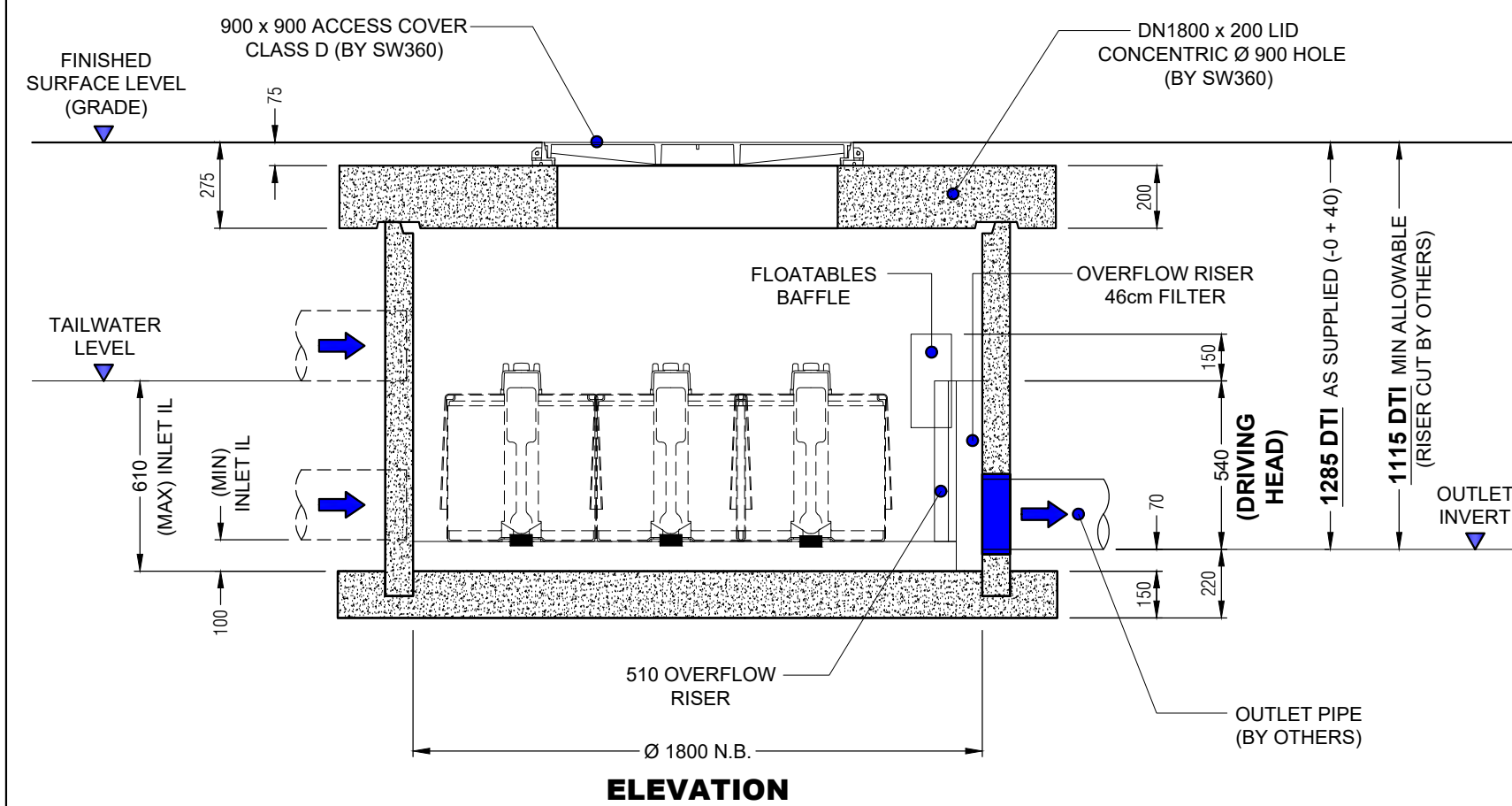
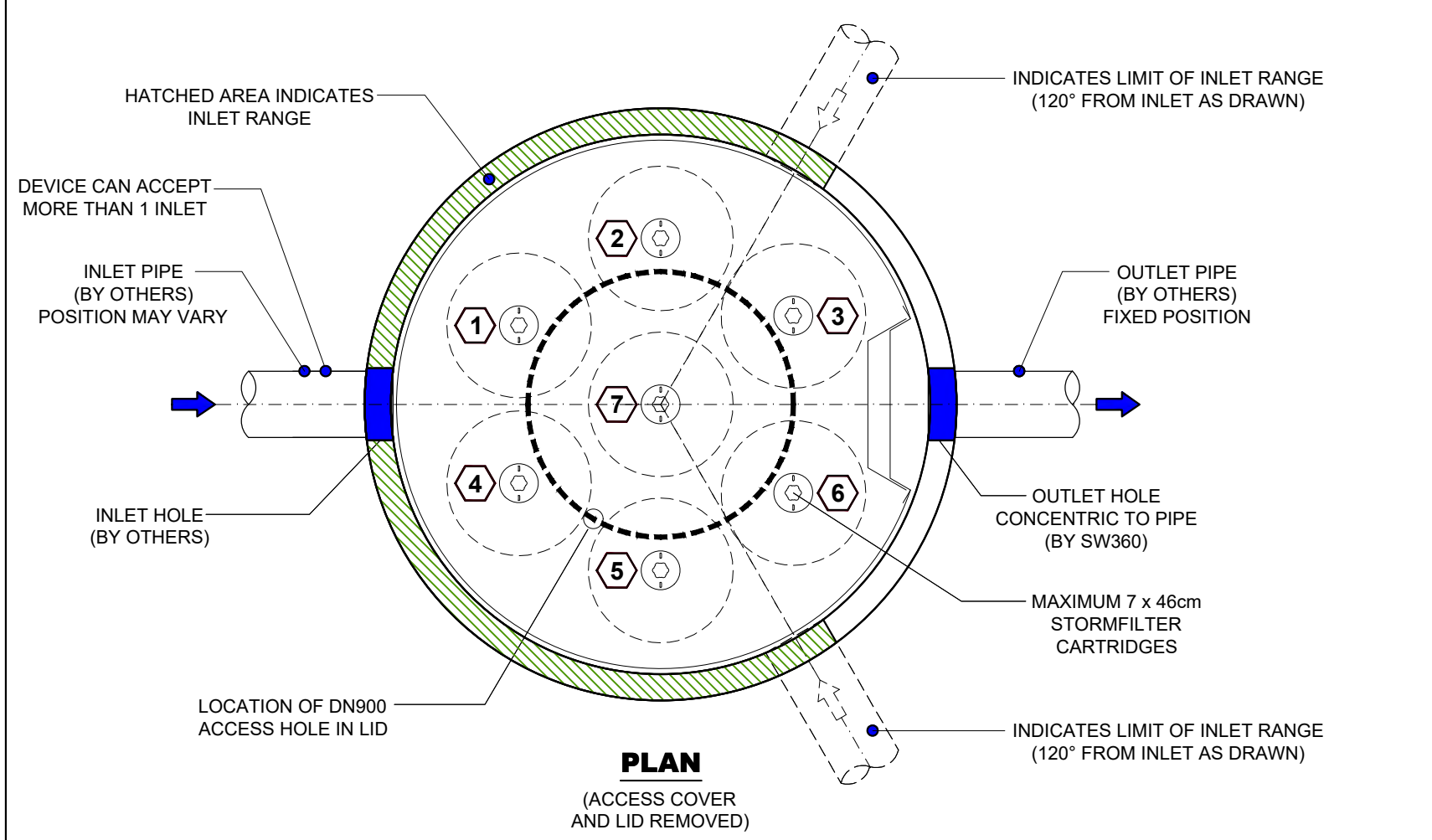
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FILTERRA®
STORMWATER BIOFILTRATION
OPEN TOP / SHALLOW DEPTH 1200 x 1800 (INSIDE)
GENERAL ARRANGEMENT

SCALE : NTS DRG No : FT-OT-SH-1218-20 REV : 1 DATE : 04.02.19

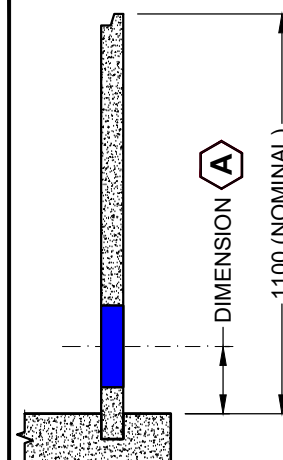
JOB NO :		REV	REVISION DETAIL	DATE
PROJECT :		0	FIRST ISSUE	04.02.19
DRN :	R.P.	04.02.19		
CKD :	G.S.	G.Y.	04.02.19	

**TO BE COMPLETED BY CUSTOMER / CONTRACTOR**

COMPANY :	P.O. NUMBER :					
SITE ADDRESS :						
SITE CONTACT & PHONE :						
PREFERRED DELIVERY DATE (TBC SW360) :						
STORMFILTER REFERENCE (IF APPLICABLE) :						
OUTLET HOLE Ø (CIRCLE) : (TYPICAL PIPE MATERIAL)	Ø 275 (DN225 PVC)	Ø 300 (DN225 RRJ)	Ø 350 (DN300 PVC)	Ø 400 (DN300 RRJ)	Ø 500 (DN375 RRJ)	Ø 600 (DN450 RRJ)
LID LEVEL (RL) :	OUTLET PIPE (IL) :			DTI :		
COMPLETED BY :	SIGNED :			DATE :		

TO BE COMPLETED BY SW360

SW360 PRODUCT CODE :				
MEDIA TYPE (CIRCLE ONE) :	PERLITE	ZPG	OTHER :	
CARTRIDGE QTY (STATE) :				PRE-INSTALLATION (Y/N) :
SP FLOW RATE (CIRCLE ONE) :	FULL (Ø 25.0 ID) RED	3 QTR (Ø 21.8 ID) GREY	HALF (Ø 17.9 ID) ORANGE	OTHER :
ACCESS COVER (CIRCLE ONE) :	900 x 900 WEB-FORGE / CLASS D			OTHER :
COMPLETED BY :	SIGNED :			DATE :

SW360 PRODUCTION DATA**OUTLET HOLE DETAILS :**
(CIRCLE REQUIREMENT)

OUTLET HOLE Ø	DIM A
275	190
300	180
350	220
400	225
500	260
600	315

DWG ISSUE No. :**REQUIRED DATE :****NOTES**

- MANHOLE UNIT FITTED WITH SWIFTLIFT ANCHOR POINTS (QTY 2). DO NOT EXCEED 60 DEGREE LIFT ANGLE. CONCRETE LID FITTED WITH SWIFTLIFT ANCHOR POINTS (QTY 4).
- UNIT SUPPLIED WITH OUTLET CORE DRILLED. INLET/S CORE DRILLED ON SITE BY OTHERS WITHIN RANGE SHOWN ON DRAWING.
- SEALING / GROUTING OF MANHOLE COMPONENTS AND PIPES BY CONTRACTOR. ENSURING LOCAL CODES AND REGULATIONS ARE COMPLIED WITH.
- ANY RISERS REQUIRED TO INCREASE THE DEPTH TO INVERT (DTI) FROM THAT AS DRAWN TO BE SUPPLIED BY THE CONTRACTOR.
- FOR A DTI EXCEEDING 5m PLEASE CONTACT **0800STORMWATER**.
- CONTRACTOR TO TAKE APPROPRIATE MEASURES TO PROTECT CARTRIDGES FROM CONSTRUCTION RELATED EROSION RUNOFF. BACKFILL, BEDDING AND BUOYANCY DESIGN BY ENGINEER OF RECORD
- QTY OF CARTRIDGES BY ENGINEER OF RECORD.
- CONCRETE MANHOLE RISERS ARE DESIGNED AND MANUFACTURED IN ACCORDANCE WITH AS/NZS 4058 : 2007
- CONCRETE MANHOLE BASES ARE DESIGNED AND MANUFACTURED IN ACCORDANCE WITH NZS 3101 : 2006 & NZS 3109 : 1997
- CONCRETE LID DESIGNED AND MANUFACTURED TO HN-HO-72
- FOR REQUIREMENTS OUTSIDE OF DRAWING SPECIFICATIONS PLEASE CONTACT **0800STORMWATER**.

APPROX WEIGHTS

MANHOLE SECTION INCLUDING CARTRIDGES : **3500 Kg**
(AS DELIVERED, BASED ON QTY 7 ZPG CARTS)
LID WEIGHT : **1850 Kg**

CONDITION OF USE

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STORMFILTER®
CARTRIDGE FILTRATION SYSTEM
46cm CART / DN1800 x 1200 MH - TRAFFICABLE
GENERAL ARRANGEMENT

SCALE : NTS DRG No : SF-MH-46-1800-T-20 REV : 2 DATE : 01.03.19

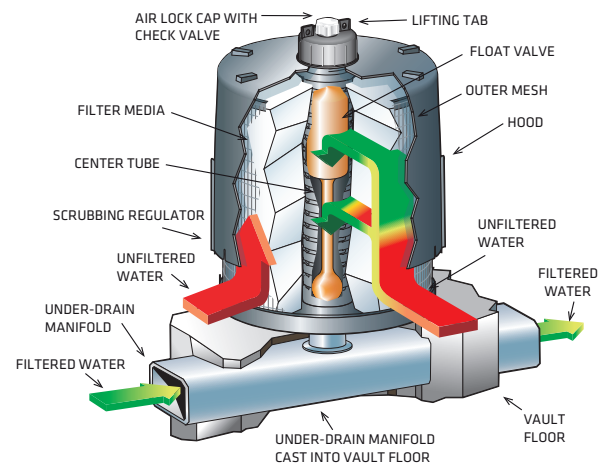
JOB NO :		REV	REVISION DETAIL	DATE
PROJECT :		0	APPROVED	01.03.19
DRN :	R.P.	01.03.19	1	MARK-UPS
CKD :	G.S.	G.Y.	2	CORRECTION
		01.03.19		17.03.20

StormFilter™

Siphon-actuated
filtration.

Our stormwater treatment filtration devices have been developed to remove an array of contaminants, including sediment, heavy metals and nutrients to meet the most stringent regulatory authority requirements.

The Stormwater Management StormFilter™ cleans stormwater through a patented passive filtration system, effectively removing pollutants to meet the most stringent regulatory requirements. Easy to install and maintain, with proven performance over time, the StormFilter™ contains rechargeable, media-filled cartridges that remove a variety of pollutants, such as sediments, oil and grease, metals, organics, and nutrients. These systems come in variable configurations to match local conditions and come with prolonged maintenance periods to ensure long-term performance and reduce operating costs.



Key Benefits

Easy maintenance and low life cycle costs.

Can be configured in any drainage structure.

Customised media options including ZPG, Perlite, Zeolite and GAC target TSS, phosphorus, heavy metals, and hydrocarbons.

Multiple cartridge heights give design solutions for site restraints.



High Sediment Removal



Maintenance

- Every 1-3 years on average.
- Cartridge Exchange Service.
- Lifetime warranty on StormFilter™ cartridges (conditions apply) .

Approvals

- ✓ ARC TP10 approval for removal of greater than 75 percent TSS.
- ✓ Auckland Council Approved for use on Public and Private Sites.
- ✓ Christchurch City Council Approved for use on Public and Private Sites.
- ✓ Washington approval and NJDEP.

Superior Hydraulics

Multiple cartridge heights give design solutions for site restraints.

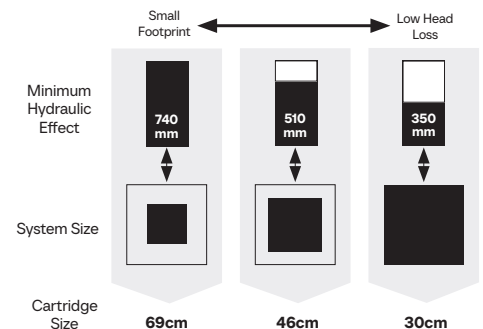


Table G1: Summary of proposed stormwater treatment								
Catchment	Area to be Treated (m ²)	First Flush Volume (m ³)	Water Quality Flow (L/s)	Stormwater Treatment	Rain Garden (m ²)	Proprietary Filters (no.)	First Flush Area (m ²)	Wetland Area (m ²)
1	9.4	1,387	76	First flush basin & wetland	N/A	N/A	3,856	4,320
2A	23.4	2,472	137	Swales	N/A	N/A	N/A	N/A
2B	15.3	1,623	90	Swales	N/A	N/A	N/A	N/A
3	9.7	1,436	79	First flush basin & wetland.	N/A	N/A	6,465	3,936
4	8.3	1,218	68	First flush basin & wetland. Raingardens.	TBC as part of design development.	N/A	6,196	4,774
5A ¹	13.6	1,989	116	First flush basin & wetland. Raingardens.	TBC as part of design development.	N/A	4,075	3,763
5B	1.4	211	12	Swales	N/A	N/A	N/A	N/A
6	10.4 ²	1,748	97	Swales *Retirement village has internal SMA	N/A	N/A	N/A	N/A
8	3.6	528	29	First flush basin & wetland.	N/A	N/A	2,723	1,406
9A	3.0	449	25	First flush basin & wetland.	N/A	N/A	1,136	1,194
9B	1.3	209	12	Proprietary devices (flow-based)	N/A	9	N/A	N/A
9C	8.8	1,287	78	First flush basin & wetland.	N/A	N/A	2,902	4,912
10	8.1	N/A as park area (polo field)						
11	1.9	277	15	Rain gardens	TBC as part of design development.	N/A	N/A	N/A

Table H1: Summary of attenuation

Pre-Development Catchment	Post-Development Catchment	Available storage (m³)				Required Storage (m³) ²	Storage required vs storage available	Notes
		First Flush ¹ (m³)	Wetland above PWL ¹ (m³)	Wet Pond ¹ (m³)	Total (m³)			
C1	2A	0	0	5,675	5,635	5,615	Sufficient	The attenuation provided is sufficient to limit the post-development runoff to the pre-development runoff for catchment C1. The proposed basins may be optimised as part of the design development.
	2B	0	0	3,723	3,723	3,622	Sufficient	
C2	1	1,387	464	0	1,851	1,790	Sufficient	The proposed first flush and wetlands in post-development catchments 1, 3, and 4 are sufficiently sized to meet water quality requirements to provide attenuation as well. The first flush basins alone have sufficient volume to attenuate the targeted volume for the pre-development runoff to not exceed the post-development runoff.
	3	1,436	1,495	0	2,931	2,825	Sufficient	
	4	869	1,432	0	2,301	2,299	Sufficient	
C3	5A	1,989	5,089	0	7,078	6,882	Sufficient	The attenuation provided is sufficient to limit the post-development runoff to the pre-development runoff for catchment C3. The proposed basins may be optimised as part of the design development.
	5B	0	0	661	661	660	Sufficient	
	6	0	0	0	0	0	Sufficient	
	9A	449	1,359	0	1,808	1,738	Sufficient	
	9B	0	0	276	276	272	Sufficient	
	9C	1,287	3,826	0	5,113	4,662	Sufficient	
	10	0	0	3,210	3,210	3,034	Sufficient	
C4	8	528	759	0	1,287	1,037	Sufficient	The post-development runoff from catchments 8 and 11 is less than the pre-development catchment runoff due to a large area of pre-development catchment C4 being routed towards post-development catchment C3 (consisting of catchments 5A, 5B, 6, 9A, 9B, 9C and 10).
	11	0	0	0	0	0	Sufficient	
Notes: 1. The attenuation volume was calculated using the maximum groundwater level as the assumed “base” level for determining the attenuation volume. 2. The required storage is based on the maximum live storage volume calculated for the 1% AEP 1 hour, 6 hour, 12 hour, and 24 hour events. The reported value is the maximum total storage below the available attenuation depth that is reached in any of the modelled events.								