Stormwater Management Strategy

Plan Change G – Aokautere

Palmerston North City Council

23 May 2022

→ **The Power of Commitment**

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Contents

1.	**Introduction**	**1**
1.1	Background	1
1.2	Purpose of this report	1
1.3	Scope and limitations	2
1.4	Assumptions	2
2.	**Existing Conditions**	**3**
3.	**Hydrologic and Hydraulic Modelling**	**7**
3.1	Pre-Development Model	7
3.1.1	Subcatchment Parameters	7
3.1.2	Infiltration Model	9
3.1.3	Rainfall Input	11
3.1.4	Drainage System	12
3.2	Post-Development Uncontrolled Model	12
3.2.1	Subcatchment Parameters	13
3.2.2	Infiltration Model	13
3.2.3	Rainfall Input	13
3.2.4	Drainage System	13
3.3	Sensitivity Analysis	15
4.	**Stormwater Runoff Assessment**	**17**
4.1	Flood Assessment	17
4.2	Erosion Assessment	18
4.2.1	Determination of Erosion Thresholds	19
4.2.2	Erosion Threshold Exceedance	21
4.2.3	Gully Slope Erosion Impacts	25
4.3	Water Quality Assessment	25
5.	**Stormwater Concept Design**	**29**
5.1	Design Criteria	29
5.2	Stormwater Constraints	30
5.3	Stormwater Concept Design	33
5.4	Review of Draft Plan Change Provisions	36
5.5	Horizons Regional Council One Plan Considerations	36
6.	**Conclusions and Recommendations**	**37**
7.	**References**	**38**

Table index

Table 3.1	Manning's 'n' and depression storage values based on land cover for pre-development conditions	9
Table 3.2	Curve number lookup table based on land cover and hydrologic soil group	11
Table 3.3	Summary of 24-hour ARI rainfall intensities for the historical and RCP 6.0 climate change scenario for 2081 – 2100	11

Table 3.4	24-hour nested design storm rainfall intensities for pre- and post-development conditions	12
Table 3.5	Sensitivity of Modelled Peak Runoff to Flow Length, Subcatchment Slope, and Manning's 'n'	15
Table 3.6	Sensitivity of Modelled Peak Runoff to Depression Storage, Curve Number, and Manning's 'n' for Channelized Flow	15
Table 3.7	Sensitivity of Modelled Runoff Volume to Flow Length, Subcatchment Slope, and Manning's 'n'	15
Table 3.8	Sensitivity of Modelled Runoff Volume to Depression Storage and Curve Number	16
Table 4.1	Flood assessment results and criteria	17
Table 4.2	Summary of stream characteristics for erosion threshold assessment	21
Table 4.3	Summary of erosion threshold exceedance analysis – uncontrolled post-development (lower threshold)	23
Table 4.4	Summary of erosion threshold exceedance analysis – uncontrolled post-development (higher threshold)	23
Table 4.5	Summary of erosion threshold exceedance analysis – controlled post-development – Aokautere Church Stream	24
Table 4.6	Summary of erosion threshold exceedance analysis – controlled post-development – Moonshine Valley Reserve Stream	24
Table 4.7	Summary of erosion threshold exceedance analysis – controlled post-development – Tutukiwi Reserve Stream	25
Table 4.8	Summary of WQF and WQV values by discharge location	27
Table 5.1	Required detention storage, footprints and bioretention footprints by discharge location	29
Table 5.2	Stormwater wetland footprints for flood control and water quality treatment on plateaus (alternative option)	33

Figure index

Figure 1.1	Aokautere Plan Change stormwater assessment study area	1
Figure 2.1	Aokautere stormwater assessment catchment area	4
Figure 2.2	Preliminary development layout (Oct 2019) and existing stormwater reticulation	5
Figure 2.3	Major and minor stormwater discharge locations	6
Figure 3.1	Pre-development stormwater catchments	8
Figure 3.2	Study area soil types	10
Figure 3.3	Post-development stormwater catchments	14
Figure 4.1	PCSWMM storage calculator tool output (sample)	18
Figure 4.2	Summary of Effectiveness of Stormwater Management Devices in terms of Water Quantity Control and Water Quality Treatment (GD2017/01, Table 15)	26
Figure 4.3	Ranked Daily Rainfall Depth at Palmerston North AWS (1991-2019)	27
Figure 5.1	Study area stormwater constraints	32
Figure 5.2	Stormwater concept design overview	35

Appendices

Appendix A	Aokautere Structure Plan Development Layout (McIndoe Urban, October 2019, July 2021 and March 2022)
Appendix B	Summary of Flood Assessment Results and Criteria
Appendix C	Erosion Threshold Exceedance Analysis Results Graphs

1. Introduction

1.1 Background

GHD Limited (GHD) has been retained by Palmerston North City Council (PNCC) to undertake a stormwater management servicing assessment and strategy for the Aokautere Plan Change (Plan Change G). PNCC is undertaking the Aokautere Plan Change in response to ongoing existing and anticipated future development in the Aokautere area, in order to ensure that development proceeds in a coordinated, thoughtful, and effective manner that protects existing development and sensitive ecological features.

Aokautere is a rural community area located immediately southeast of the Manawatū River across from the Hokowhitu suburb of Palmerston North. Perched among the foothills of the Tararua Range, Aokautere exhibits a natural hilly terrain with a network of gullies and plateaus supporting significant recreational and ecological amenities. The location of the Aokautere Plan Change area (referred to as the site, study area) is presented on Figure 1.1.

Although the site is largely used for agricultural purposes, there are several areas of existing residential developments dating from the 1980s and onwards, and several other residential developments have recently begun to be constructed along the study area's plateaus. Several small gullies have also been filled during construction of the existing developments, and development pressure in this area is expected to increase in the future.

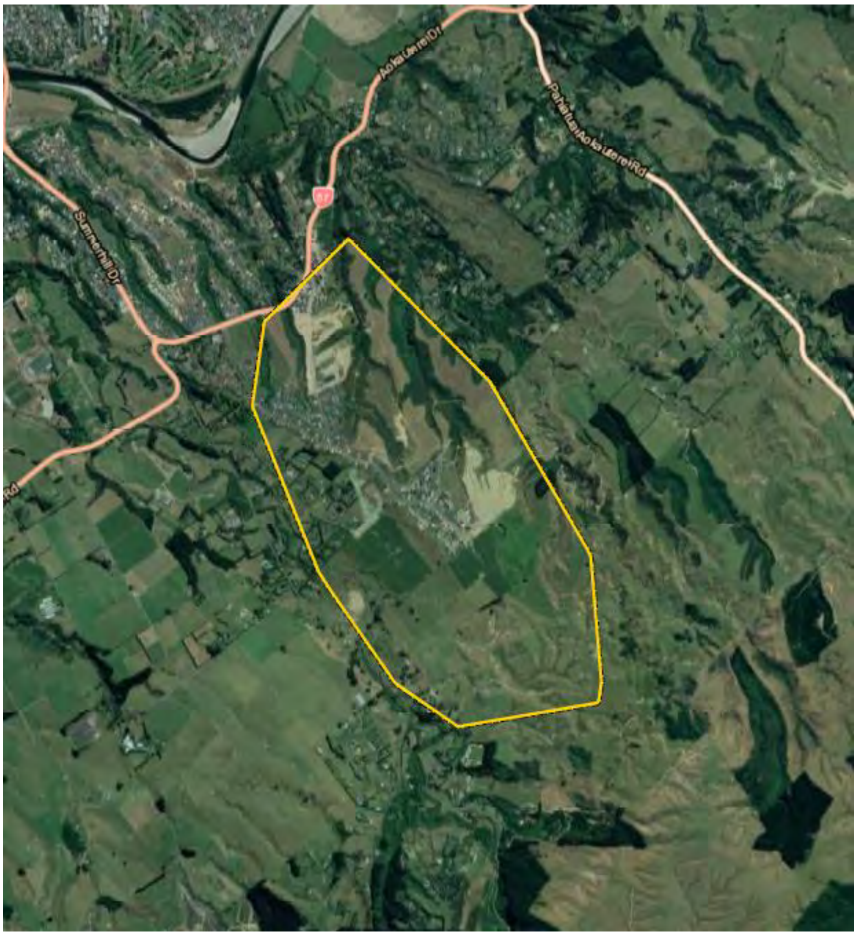

Figure 1.1 Aokautere Plan Change stormwater assessment study area

1.2 Purpose of this report

The purpose of this report is to present the outcomes of the stormwater management analysis and recommend a stormwater management strategy for the Aokautere Plan Change. This report will contribute to and form part of the Plan Change structure plan for implementation during detailed design, resource consenting and construction of the proposed developments.

1.3 Scope and limitations

The scope of the stormwater management analysis is to estimate and assess the flooding, erosion and water quality impacts of the development on the receiving watercourses and downstream areas. The analysis completed for this project includes hydrologic and hydraulic modelling of the study area (detailed in section 3) and quantitative assessment of the stormwater impacts based on the modelling results and available industry standards and guidelines (described in section 4). There are three components of the stormwater management analysis, including:

– Flood assessment to predict the impact of the development on peak flow rates and flood potential of the receiving environment.
– Erosion assessment to predict the impact of the development on the frequency and duration of flow events that exceed the erosion threshold of the receiving gully system.
– Water quality assessment to identify contaminant profiles typically associated with runoff from residential development and identify potential impacts to the receiving environment.

The stormwater management strategy (described in section 5) proposes design criteria and conceptual design alternatives for stormwater controls to mitigate the assessed impacts. The existing developments that are already in place are assessed in conjunction with the proposed development areas in order to recommend stormwater management controls that collectively achieve the established design criteria for the entire Structure Plan area.

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1.4 Assumptions

The analysis is based on the publicly available GIS data for the existing site conditions and the preliminary subdivision layout of the proposed development provided by McIndoe Urban (received 16 October, 2019, Appendix A) and subsequent revisions (final dated 11 May 2022, Appendix A). All grades and elevations for proposed conditions are assumed to generally follow the existing topography, noting that some modification of these grades is likely to occur during development. Additional modelling assumptions are outlined in section 3 of the report.

2. Existing Conditions

The development area is primarily located on a series of plateaus that form the headwaters of smaller tributaries of the natural drainage system, located in a series of gullies that separate the plateaus into distinct areas. The gullies within the study area ultimately discharge into larger streams that originate in the Tararua Range, specifically the Moonshine Valley Stream and the Turitea Stream, which flow around the development area to the Manawatū River.

A map of the total catchment area studied in the stormwater management analysis is presented on Figure 2.1, and includes the contributory drainage areas of the Moonshine Valley and Turitea Streams as well as the catchments within the study area itself. The preliminary proposed subdivision layout and the existing storm main reticulation are presented on Figure 2.2 (as of October 2019). The data for the existing storm main reticulation was obtained from PNCC Open Data. It is GHD's understanding that as of late 2019 there is currently only one identified stormwater management device (a gross pollutant trap) in the study area, located at the end of Silicon Way. However, development around Atlantic Drive within the Plan Change area is currently ongoing and will include installation of rain gardens and attenuation ponds. These facilities have not been incorporated into this stormwater assessment.

Potential discharge locations from the development area were identified based on a review of the existing storm main reticulation, the preliminary subdivision layout, and the existing site topography (contours). Site contours were generated from a 2018 Digital Elevation Model (DEM) obtained from PNCC Open Data, the most recent elevation data available at the time of this analysis.

Major discharge locations were identified where stormwater runoff ultimately exits the study area to the natural drainage system. Minor discharge locations were identified where stormwater is expected to discharge from the engineered stormwater collection and treatment system to the gullies within the study area. Figure 2.3 shows the major and minor discharge locations and their respective contributing drainage areas, which form the basis for the stormwater modelling and analysis described in the remainder of this report. Actual minor stormwater discharge locations from developed areas into the receiving environment will ultimately depend on site-specific design to be completed at the subdivision consent stage.

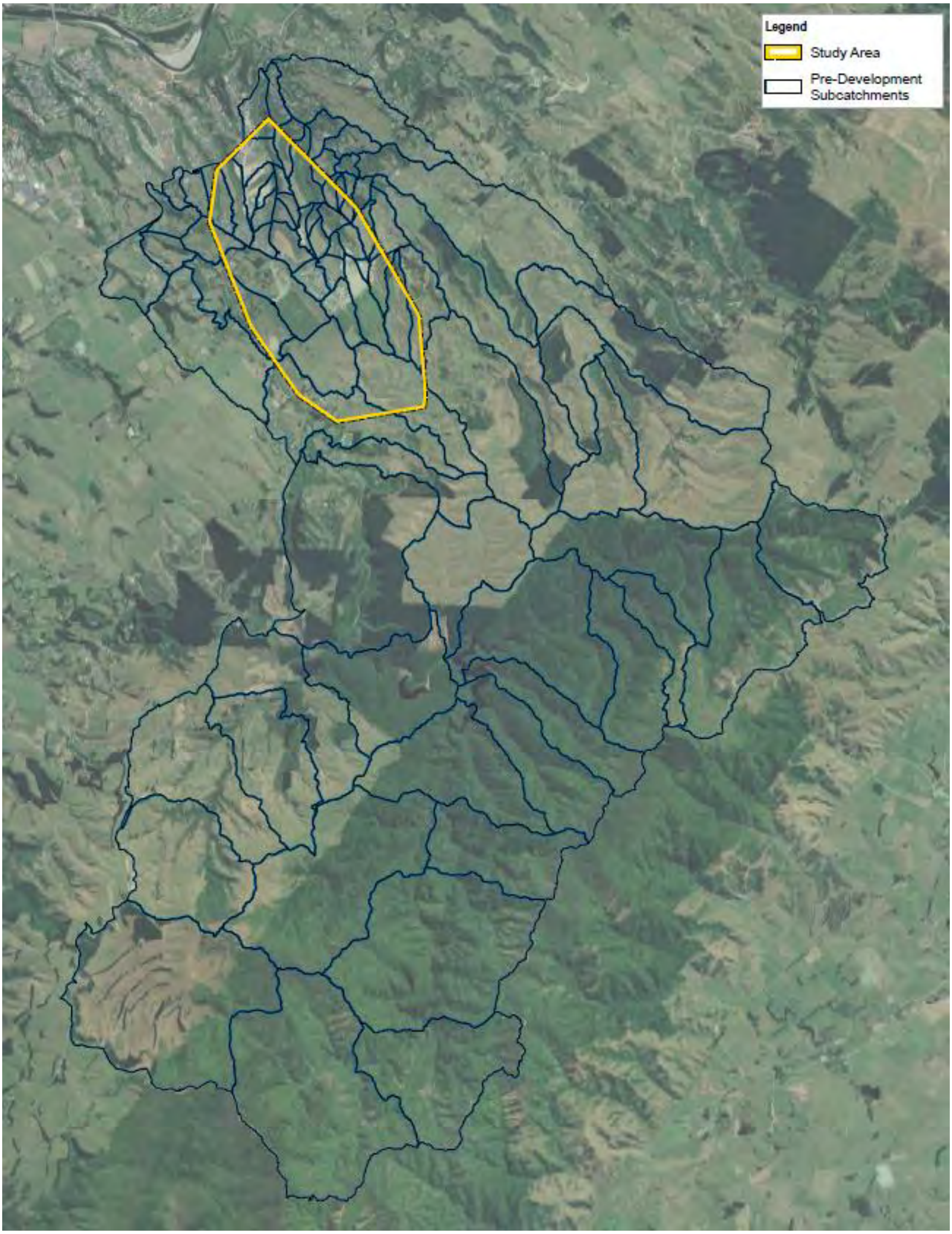


Figure 2.1 *Aokautere stormwater assessment catchment area*

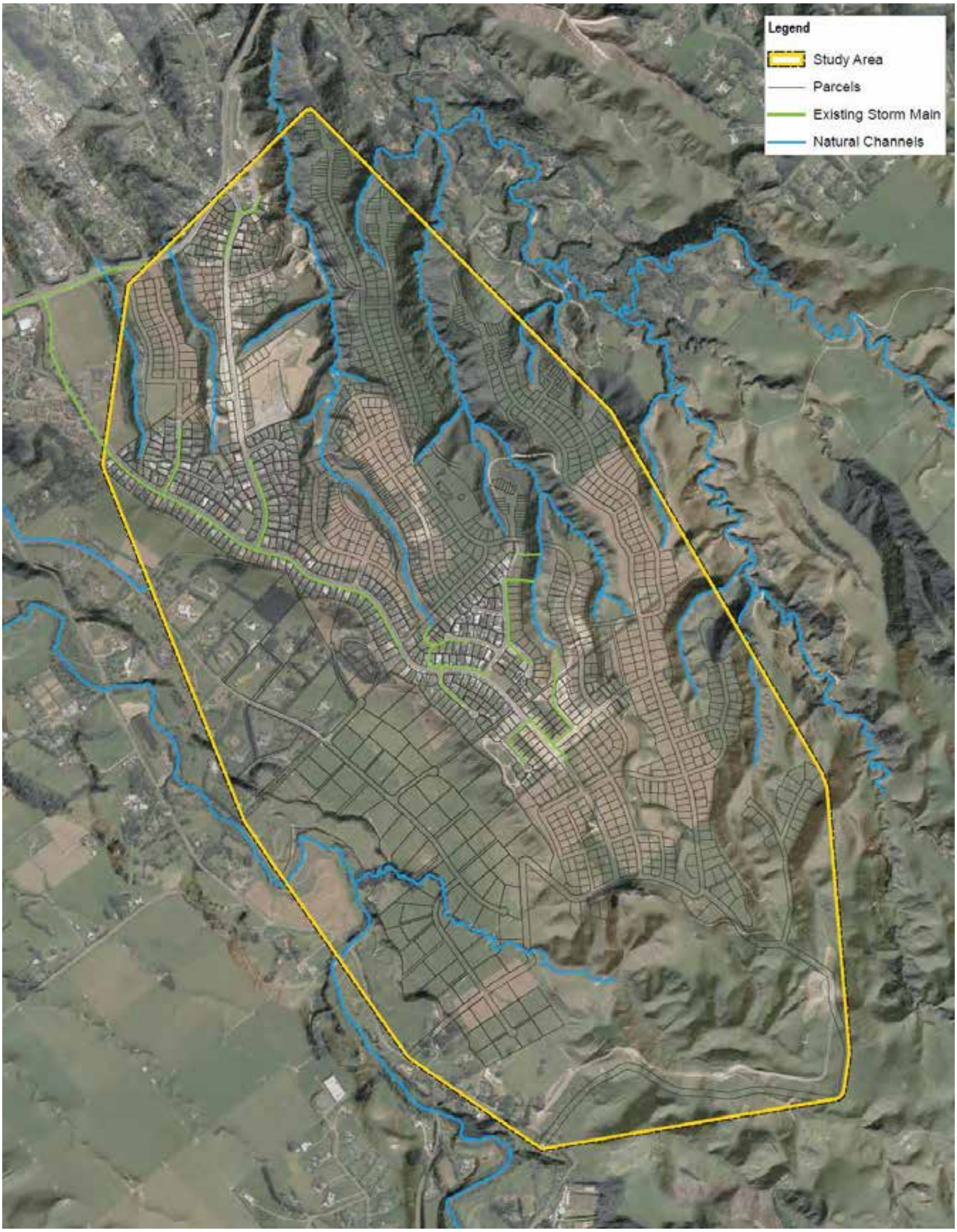


Figure 2.2 Preliminary development layout (Oct 2019) and existing stormwater reticulation

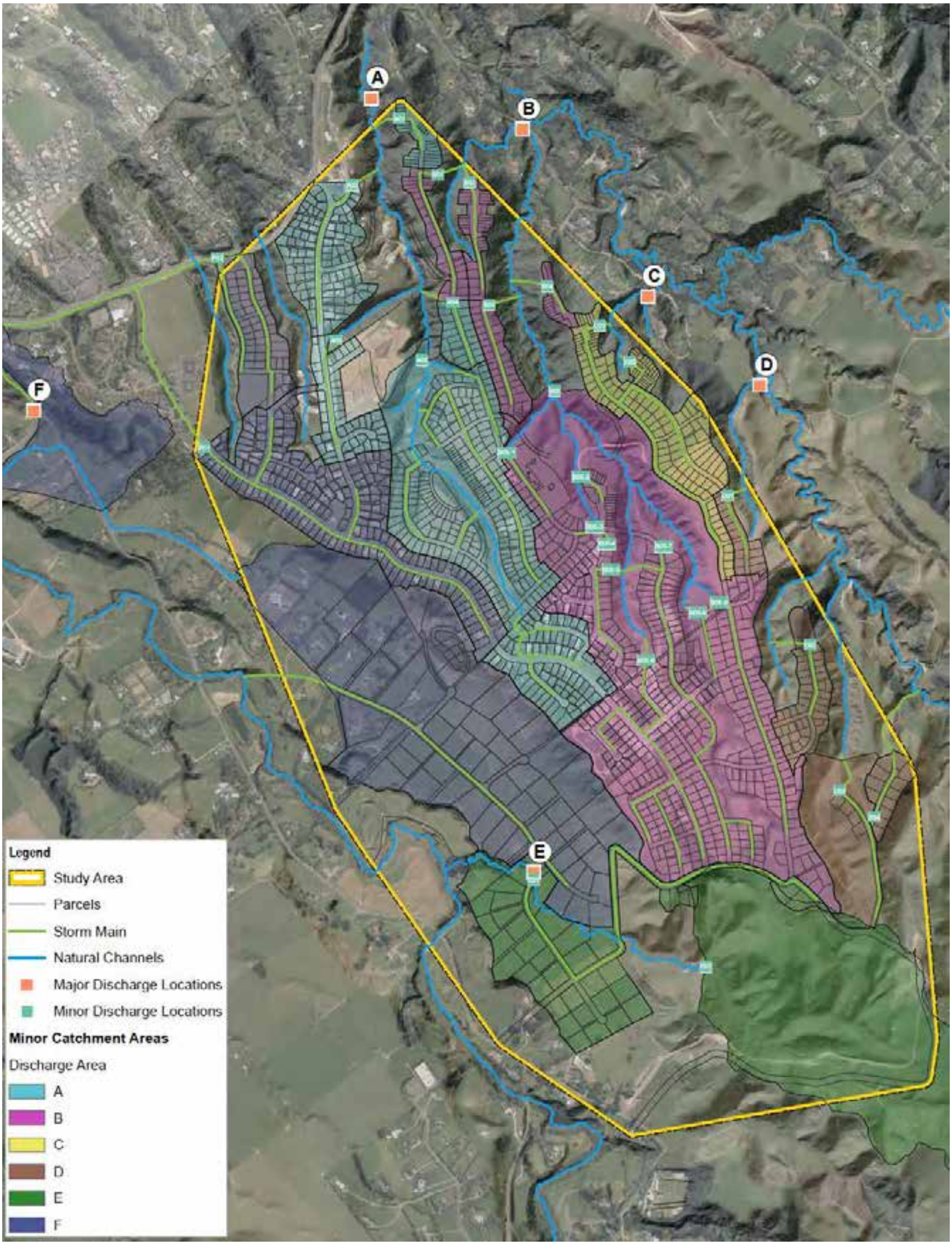


Figure 2.3 Major and minor stormwater discharge locations

3. Hydrologic and Hydraulic Modelling

Hydrologic and hydraulic modelling of the study area was undertaken to characterise stormwater runoff conditions in support of the flood and erosion impact assessments, including the quantification of pre-development and post-development (uncontrolled) runoff flows and volumes at the site discharge locations. The models described in this section are further employed in the impact assessment and conceptual design of stormwater management controls, described in sections 4 and 5.

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

The setup of the pre- and post-development stormwater models are described in sections 3.1 and 3.2, respectively.

3.1 Pre-Development Model

For the purpose of this analysis, pre-development conditions were established as the land conditions prior to any residential development within the study area, including those limited areas of development constructed over the previous 20 to 30 years. This reflects the intention of the stormwater strategy to effectively address all stormwater runoff in the study area and avoid the “grandfathering” of existing areas which would then incur a disproportionately high impact to the receiving environment.

3.1.1 Subcatchment Parameters

Pre-development subcatchments were delineated using the Watershed Delineation Tool (WDT) in PCSWMM. The WDT employs flow direction, flow accumulation, stream definition, and watershed delineation calculations to create subcatchments from a DEM. The 2018 DEM was used to delineate subcatchments in the development area, and a 2015 DEM was used to delineate subcatchments in the upstream drainage area, as the more recent 2018 DEM did not extend to the full catchment limits. The 2015 DEM was obtained from the Land Information New Zealand (LINZ) Data Service. A map of the pre-development subcatchments is presented on Figure 3.1.

Flow lengths and subcatchment slopes were calculated manually using the site contours and measuring tool in PCSWMM. Flow lengths of large subcatchments (primarily located upstream of the study area) were calculated as the longest flow path to main channel, which include long sections of concentrated channel flow. Flow lengths of small subcatchments were calculated as the maximum overland flow path to the main channel, assuming that only limited concentrated flow occurs. Higher Manning’s ‘n’ values were assigned to the small subcatchments to represent the higher frictional forces associated with overland flow compared to concentrated flow.

Subcatchment parameters including percent impervious area, Manning’s ‘n’ values, and depression storage values were assigned based on land cover type, spatially averaged over the subcatchments. Land cover data was obtained from the Land Cover Database (LCDB) v40 layer of the Landcare Research Portal, added June 27, 2014.

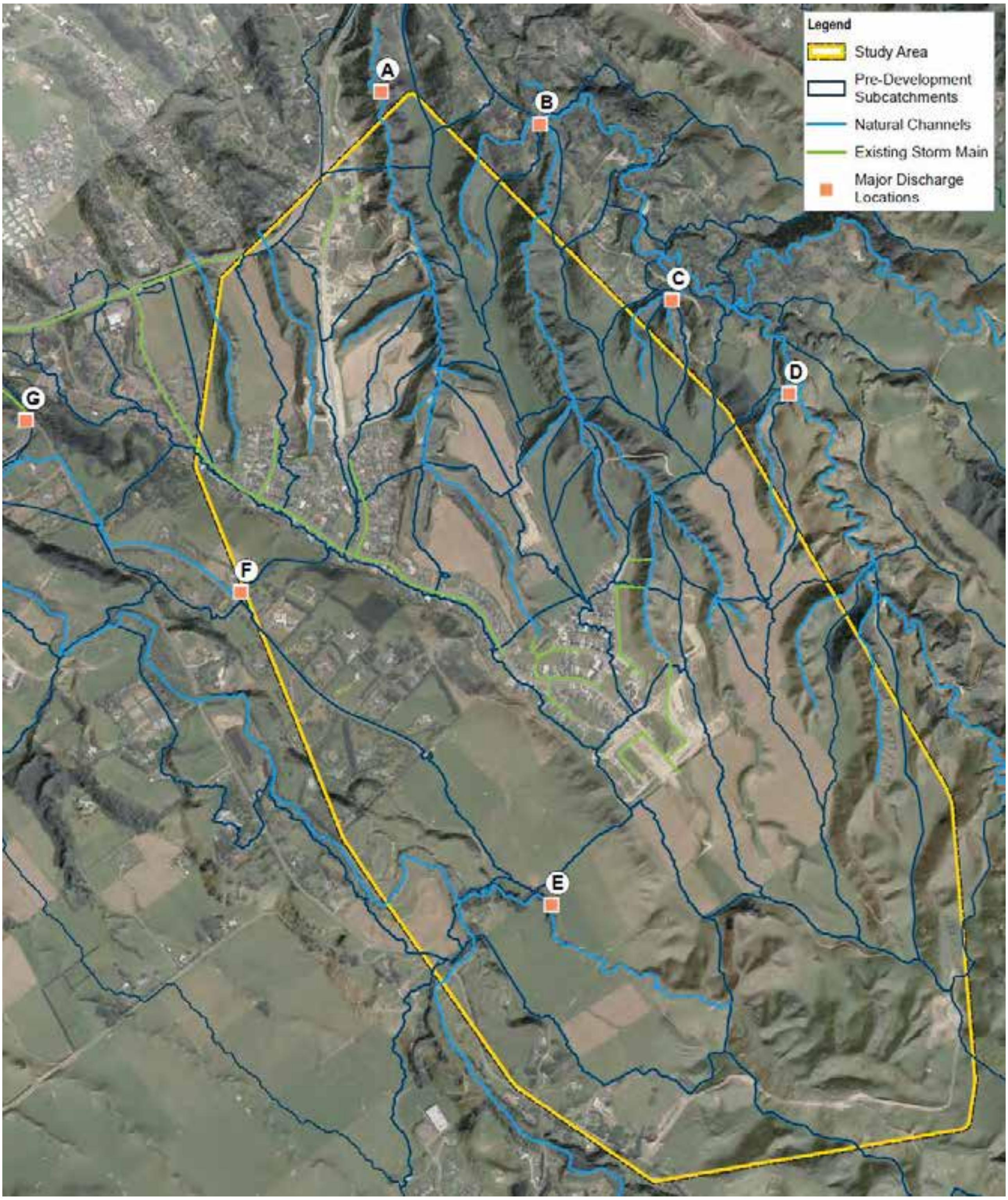


Figure 3.1 Pre-development stormwater catchments

Table 3.1 presents a summary of the Manning's 'n' values and depression storage values based on land cover type (Chow, 1959; US Environmental Protection Agency, 2015).

Table 3.1 Manning's 'n' and depression storage values based on land cover for pre-development conditions

No.	Land Cover Classification	Manning's 'n' (Channelised Flow)	Manning's 'n' (Overland Flow)	Depression Storage (mm)
1	Built-up Area	0.013	0.013	2
2	Urban Parkland/Open Space	0.030	0.030	5
5	Transport Infrastructure	0.013	0.013	2
6	Surface Mine/Dump	0.024	0.024	2
16	Gravel/Rock	0.024	0.024	2
20	Lake/Pond	0.040	0.040	0
21	River	0.040	0.040	0
30	Short-rotation Cropland	0.045	0.170	5
33	Orchard/Vineyard/Other Perennial	0.050	0.170	5
40	High Producing Exotic Grassland	0.050	0.130	5
41	Low Producing Grassland	0.050	0.130	5
45	Herbaceous Freshwater Vegetation	0.070	0.400	8
51	Gorse/Broom	0.160	0.400	8
52	Manuka/Kanuka	0.160	0.400	8
54	Broadleaved Indigenous Hardwoods	0.160	0.400	8
64	Mixed Exotic Shrubland	0.160	0.400	8
68	Forest-Harvested	0.160	0.400	8
69	Deciduous Hardwoods	0.160	0.400	8
71	Indigenous Forest	0.160	0.400	8

3.1.2 Infiltration Model

The Soil Conservation Service (SCS) curve number method was used to calculate infiltration and other hydrologic losses in the flood and erosion assessments. Curve numbers represent the average antecedent runoff/infiltration conditions of the subcatchments, and were selected from the "Guidelines for Stormwater Runoff Modelling in Auckland Region (TP 108)" and "Urban Hydrology for Small Watersheds (TR-55)" based on the hydrologic soil group and the land cover type of the subcatchments.

The hydrologic soil group was assumed based on the particle size distribution from the surficial soil map (FSL Particle Size Classification layer from the Land Resource Information System (LRIS) portal, Landcare Research, added on June 7, 2010). Sand, silt, loam, and clay designations from the LRIS data were assigned to hydrologic soil groups A, B, C, and D, respectively. The predominant soil type within the study area is classified as loam and was assigned to the C soil group. The soil types used for the hydrologic assessment are shown in Figure 3.2.

A curve number polygon shapefile was created by performing a spatial intersection of the surficial soil map (that provides information on the particle size distribution and hydrologic soil group), and the land cover map. The resultant polygons were assigned curve numbers using a site-specific look up table that was created for the development area. Curve numbers were then spatially averaged over the subcatchments. Table 3.2 presents the site's curve number lookup table.

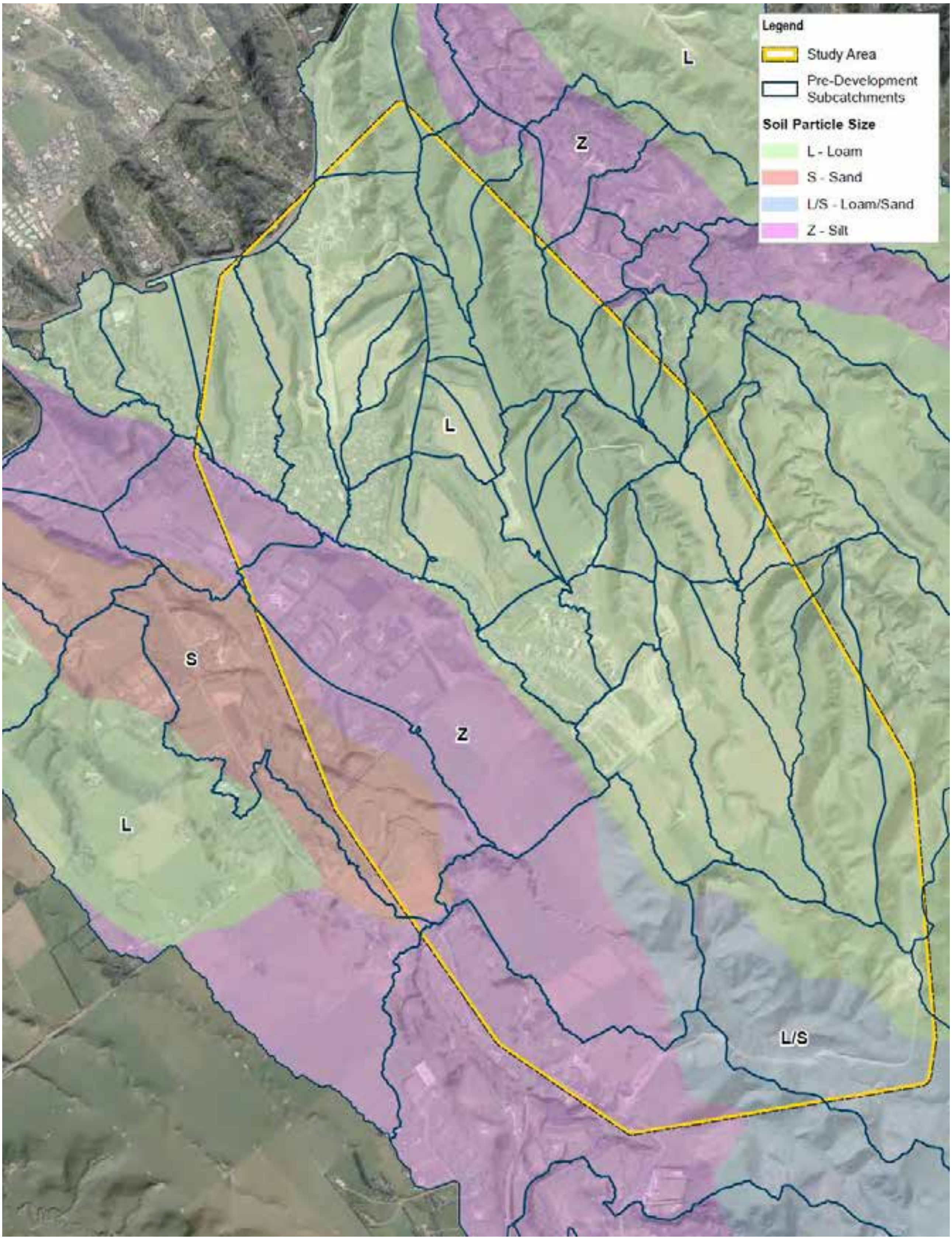


Figure 3.2 Study area soil types

Table 3.2 Curve number lookup table based on land cover and hydrologic soil group

Land Cover Data		Hydrologic Soil Group			
		A	B	C	D
1	Built-up Area	61	75	83	87
2	Urban Parkland/Open Space	39	61	74	80
5	Transport Infrastructure	98	98	98	98
6	Surface Mine/Dump	98	98	98	98
16	Gravel/Rock	77	86	91	94
20	Lake/Pond	98	98	98	98
21	River	98	98	98	98
30	Short-rotation Cropland	72	81	88	91
33	Orchard/Vineyard/Other Perennial	32	58	72	79
40	High Producing Exotic Grassland	39	61	74	80
41	Low Producing Grassland	39	61	74	80
45	Herbaceous Freshwater Vegetation	68	79	86	89
51	Gorse/Broom	48	67	77	83
52	Manuka/Kanuka	48	67	77	83
54	Broadleaved Indigenous Hardwoods	48	67	77	83
64	Mixed Exotic Shrubland	48	67	77	83
68	Forest-Harvested	30	55	70	77
69	Deciduous Hardwoods	30	55	70	77
71	Indigenous Forest	30	55	70	77
-	Proposed Residential (1/4 Acre Lots)	61	75	83	87
-	Proposed Residential (1 Acre Lots)	51	68	79	84
-	Proposed Residential (2 Acre Lots)	46	65	77	82

3.1.3 Rainfall Input

Rainfall was modelled using a design storm approach for the flood assessment and was based on the requirements outlined in the PNCC Engineering Standards for Land Development (ESLD) (2019). Rainfall hyetographs were created using a Normalised 24-hour Design Storm distribution, and rainfall intensities for the 2-year, 10-year, 50-year and 100-year 24-hour return period events. Historical rainfall was used to form the hyetographs for the pre-development model, and the climate change projected rainfall was used to form hyetographs for the post-development model using the RCP 6.0 climate change scenario for the 2081 to 2100 time period (as per PNCC ESLD). The return period rainfall intensities used in the analysis are summarized in Table 3.3. Rainfall intensities were obtained from the National Institute of Water and Atmospheric Research (NIWA) High Intensity Rainfall Design System (HIRDS). The Normalised 24-hour design storm distribution was acquired from TP 108 and is summarised in Table 3.4. The RCP 6.0 2081-2100 rainfall intensities are generally 11% to 14% larger than historical values.

Table 3.3 Summary of 24-hour ARI rainfall intensities for the historical and RCP 6.0 climate change scenario for 2081 – 2100

Scenario	Rainfall Intensity (mm/hour)			
	2-year ARI	10-year ARI	50-year ARI	100-year ARI
Historical	2.34	3.50	4.70	5.25
Projected (RCP 6.0)	2.62	3.96	5.35	5.98

Table 3.4 24-hour nested design storm rainfall intensities for pre- and post-development conditions

Begin Time	End Time	I/I24[1]	Historical (Pre-Development) [mm/hr]				RCP 6.0 2081-2100 (Post-Development) [mm/hr]			
			2-year	10-year	50-year	100-year	2-year	10-year	50-year	100-year
00:00	06:00	0.34	0.80	1.19	1.60	1.79	0.89	1.35	1.82	2.03
06:00	09:00	0.74	1.73	2.59	3.48	3.89	1.94	2.93	3.96	4.43
09:00	10:00	0.96	2.25	3.36	4.51	5.04	2.52	3.80	5.14	5.74
10:00	11:00	1.4	3.28	4.90	6.58	7.35	3.67	5.54	7.49	8.37
11:00	11:30	2.2	5.15	7.70	10.34	11.55	5.76	8.71	11.77	13.16
11:30	11:40	3.8	8.89	13.30	17.86	19.95	9.96	15.05	20.33	22.72
11:40	11:50	4.8	11.23	16.80	22.56	25.20	12.58	19.01	25.68	28.70
11:50	12:00	8.7	20.36	30.45	40.89	45.68	22.79	34.45	46.55	52.03
12:00	12:10	16.2	37.91	56.70	76.14	85.05	42.44	64.15	86.67	96.88
12:10	12:20	5.9	13.81	20.65	27.73	30.98	15.46	23.36	31.57	35.28
12:20	12:30	4.2	9.83	14.70	19.74	22.05	11.00	16.63	22.47	25.12
12:30	13:00	2.9	6.79	10.15	13.63	15.23	7.60	11.48	15.52	17.34
13:00	14:00	1.7	3.98	5.95	7.99	8.93	4.45	6.73	9.10	10.17
14:00	15:00	1.2	2.81	4.20	5.64	6.30	3.14	4.75	6.42	7.18
15:00	18:00	0.75	1.76	2.63	3.53	3.94	1.97	2.97	4.01	4.49
18:00	00:00	0.4	0.94	1.40	1.88	2.10	1.05	1.58	2.14	2.39

1. Note, I/I24 represents the nested design storm ratio of intensity for the specific time step to the 24-hour intensity for the appropriate ARI.

A continuous modelling approach was used to perform the erosion assessment in order to assess the total erosive forces imposed on the receiving watercourses across a representative range of flow events. This modelling approach requires continuous hourly rainfall data as input. The hourly rainfall time series was obtained from the National Climate Database (NIWA) for the Palmerston North Ews gauge for the 2012 to 2016 time period.

3.1.4 Drainage System

The natural drainage system was generated by the WDT in PCSWMM using the DEM. Figure 3.1 shows the delineated drainage system. The natural channels were represented by a stepped cross-section estimated based on the available elevation contours and visual observations. The channel portion of the cross-section consists of a trapezoidal geometry with a 1 m depth, 1 m bottom width, and 2H:1V side slopes.

Channel slopes were calculated as the change in elevation over the channel lengths, where elevations of the upstream/downstream ends of the channel were assigned based on the DEM. Manning's 'n' values were selected for the channels according to a review of the aerial imagery of the catchment area. A Manning's 'n' value of 0.030 was assigned to the channel portions of the cross-section. Manning's 'n' values of 0.100 and 0.035 were assigned to floodplain areas that were more representative of forest and grassland cover, respectively (Chow, 1959).

3.2 Post-Development Uncontrolled Model

An intermediary but critical step in the development of an effective stormwater management strategy for the study area is the quantification of runoff impacts related to the proposed development. To accomplish this, a post-development model was developed that included the full extent of residential development within the study area but excluded stormwater management controls. The setup of this uncontrolled model is described in the following sections.

3.2.1 Subcatchment Parameters

The post-development model was based primarily on the pre-development model to ensure that the results could be considered comparable. Post-development subcatchment areas were modified from the pre-development catchments in the study area to represent the preliminary subdivision layout. The lots were assumed to be graded toward the roads, where minor flows are captured and conveyed in a reticulated storm sewer system and major flows are conveyed along the road. Upstream subcatchments that are outside of the development area remain unchanged from pre-development conditions. Figure 3.3 presents a map of the post-development catchments, which show a marked increase in the number of catchments within the study area in order to represent the complexity of urban development drainage pathways.

Post-development subcatchments were characterized in a similar manner to the pre-development subcatchments. The land cover shapefile was modified to include the roads and lots of the preliminary subdivision layout. Roads were included in the Transportation Infrastructure land cover classification, which has a Manning’s ‘n’ value of 0.013, and a depression storage value of 2 mm. Lots were assigned a land cover classification called Proposed Residential (developed by GHD for the purpose of this study). The majority of the Proposed Residential land cover is assumed to be 70 percent impervious (based on typical PNCC practice for recent structure plans), with a weighted Manning’s ‘n’ value of 0.018, and depression storage value of 3 mm for impervious/pervious areas. Some residential lot sizes are notably larger in the south and west extents of the study area. In these areas the land cover is assumed to be 30 percent impervious, with a weighted Manning’s ‘n’ value of 0.025, and a depression storage value of 4 mm for the impervious/pervious areas.

3.2.2 Infiltration Model

The curve number shapefile for the flood assessment was modified to reflect proposed conditions. The roads were included in the Transportation Infrastructure land cover classification that has a curve number of 98. The lots were assigned to the GHD-defined Proposed Residential land cover classification and assigned a curve number based on the Residential District land use type with an average lot size of one-quarter acre (approximately 1,000 m^2), 1 acre (4,000 m^2), or 2 acres (8,000 m^2) from the TR-55 document. Although the average lot size has not been confirmed, the selected curve number represents an impervious cover percentage that is expected to be appropriate to the development, regardless of lot size.

3.2.3 Rainfall Input

Similar to the pre-development model, rainfall was modelled using a design storm approach for the flood assessment. Rainfall hyetographs were created using a Normalised 24-hour Design Storm distribution, and rainfall intensities for the 2-year, 10-year, 50-year and 100-year 24-hour return period events. Climate change projected rainfall was used to form hyetographs for the post-development model using the RCP 6.0 climate change scenario for the 2081 to 2100 time period. The return period rainfall intensities used in the analysis are summarized in Table 3.3 in section 3.1.3. Rainfall intensities were obtained from the National Institute of Water and Atmospheric Research (NIWA) High Intensity Rainfall Design System (HIRDS). The Normalised 24-hour design storm distribution was acquired from TP 108.

3.2.4 Drainage System

The natural drainage channels were modelled using stepped cross sections as in the pre-development model. In the development area, the minor and major drainage networks were represented by stormwater mains and road cross-sections, respectively. The stormwater mains were sized to convey the 10-year peak flow rate under climate change conditions using the Manning’s equation, as per the *PNCC ESLD* (2019). Surface elevations of catch basins and manholes were determined from existing topography. Invert elevations were assumed and set to ensure a minimum pipe slope of 0.3 to 0.5% (depending on the pipe size, as per PNCC standards), a minimum depth of cover of 1.0 m and maximum depth to invert of 2.9 m below grade. The road cross-sections were classified as local, local-collector, or collector based on the preliminary subdivision layout provided by McIndoe Urban. The travelled portions of the local, local-collector, and collector roads were assumed to be 5.5 m, 6.5 m, and 9.5 m wide, respectively, with 0.15 m high curbs. The remainder of the right-of-way was modelled as a grassed surface with 4 percent slope toward the road, on either side of the roadway, to ensure that all flow is contained within this corridor.

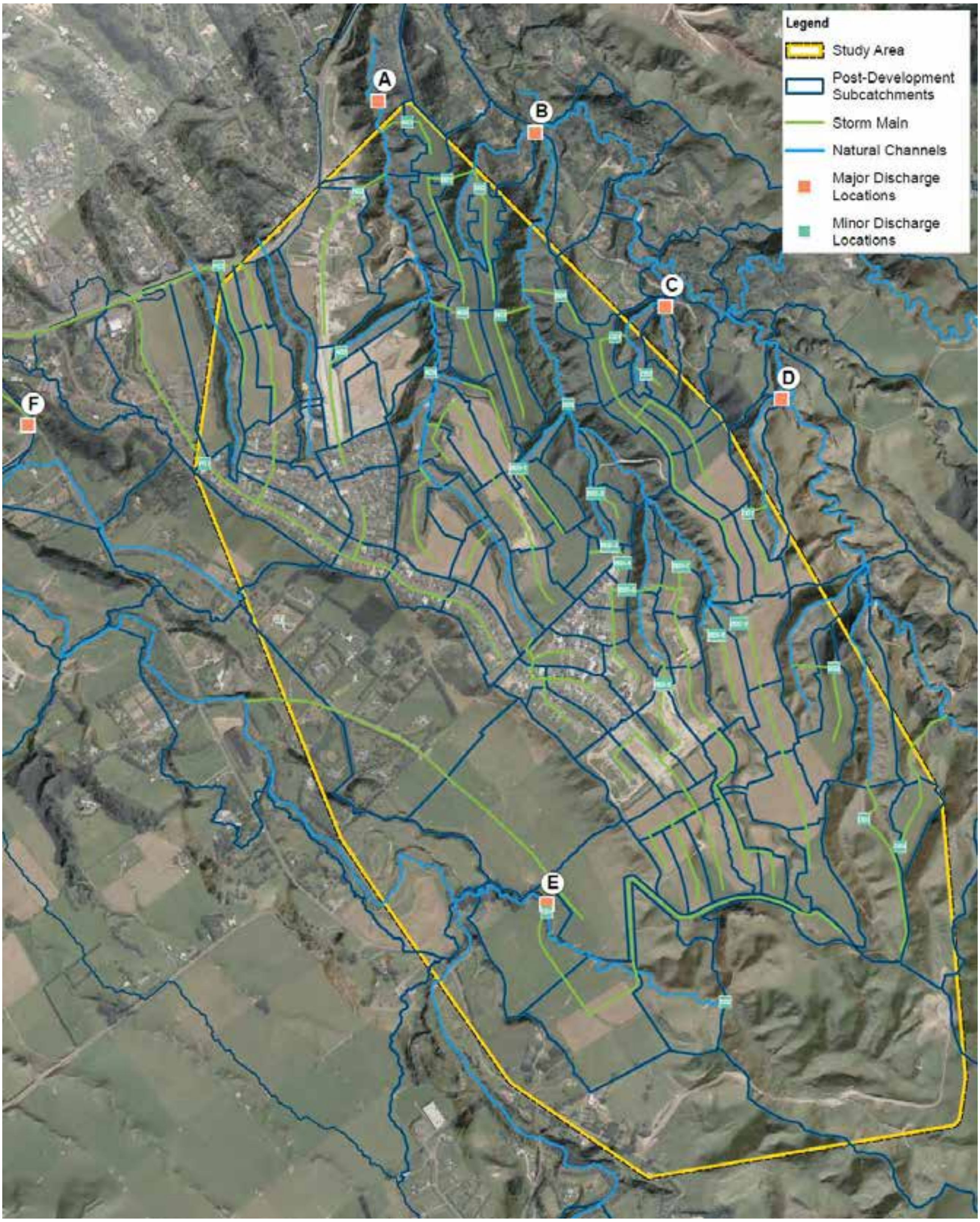


Figure 3.3 Post-development stormwater catchments

The drainage system employed in the post-development model is conceptual only for the purpose of this study. The network configuration and pipe diameters must be confirmed during detailed design for each development area to ensure the appropriate engineering standards and design criteria are met.

3.3 Sensitivity Analysis

Calibration is an important component of the hydrologic modelling process to ensure the model provides meaningful results. Calibration involves tuning model parameters to match modelled output to measured streamflow records. However, since streamflow records are not available for any of the site discharge locations, model calibration could not be performed. As such, an analysis was performed to assess the sensitivity of the model to the selected hydrologic parameters in order to understand the level of uncertainty associated with the model output. The results of the sensitivity analysis are important to consider when using the model output to support decision making processes such as for the number/size/type of stormwater management controls required for the development area.

The sensitivity analysis was conducted using the pre-development model. The analysis was performed by varying subcatchment and routing parameters by 75% and 125% of the initial value as per the *Technical Guidelines for Flood Hazard Mapping* (Environmental Water Resources Group Ltd., 2017). The analysis was performed for subcatchment parameters including flow length, subcatchment slope, Manning's 'n' (impervious and pervious), depression storage (impervious and pervious), and curve number. The analysis was also performed for the Manning's 'n' values of the natural channels.

Table 3.5 and Table 3.6 present the sensitivity of modelled peak runoff to a 25% change in subcatchment and routing parameter values. The resultant percent change in peak runoff was calculated for each subcatchment (and conduit for the Manning's 'n' of the channels) for the full range of ARI design flows included in the model, and the statistics (maximum and average) of percent change values were reported.

Similarly, Table 3.7 and Table 3.8 present the sensitivity of the modelled runoff volume to a 25% change in subcatchment parameter values. The percent change in runoff volume was calculated for each subcatchment area for the full range of ARI design flows included in the model, and the maximum and average percent change values were reported.

Table 3.5 Sensitivity of Modelled Peak Runoff to Flow Length, Subcatchment Slope, and Manning's 'n'

Statistic	Flow Length		Subcatchment Slope		Manning's 'n' Pervious	
	-25%	+25%	-25%	+25%	-25%	+25%
Max. Change in Peak Runoff (%)	14.7%	-15.0%	-10.0%	8.6%	15.3%	-15.0%
Avg. Change in Peak Runoff (%)	11.2%	-10.7%	-5.5%	5.5%	11.2%	-10.7%

Table 3.6 Sensitivity of Modelled Peak Runoff to Depression Storage, Curve Number, and Manning's 'n' for Channelized Flow

Statistics	Depression Storage Pervious		Curve Number		Manning's 'n' Channel	
	-25%	+25%	-25%	+25%	-25%	+25%
Max. Change in Peak Runoff (%)	7.0%	-10.0%	-45.0%	83.9%	6.2%	-8.1%
Avg. Change in Peak Runoff (%)	1.8%	-2.8%	-40.6%	62.6%	1.4%	-1.5%

Table 3.7 Sensitivity of Modelled Runoff Volume to Flow Length, Subcatchment Slope, and Manning's 'n'

Statistics	Flow Length		Subcatchment Slope		Manning's 'n' Pervious	
	-25%	+25%	-25%	+25%	-25%	+25%
Max. Change in Runoff Volume (%)	1.5%	-1.7%	-0.8%	0.8%	1.5%	-1.7%
Avg. Change in Runoff Volume (%)	0.5%	-0.6%	-0.3%	0.3%	0.5%	-0.6%

Table 3.8 Sensitivity of Modelled Runoff Volume to Depression Storage and Curve Number

Statistics	Depression Storage Pervious		Curve Number	
	-25%	+25%	-25%	+25%
Max. Change in Runoff Volume (%)	3.2	-4.0	-33.4	53.2
Avg. Change in Runoff Volume (%)	1.7	-2.1	-30.2	44.6

The sensitivity analysis results demonstrate that peak runoff flow is generally more sensitive than runoff volumes to the selected model parameters. Peak runoff is most sensitive to curve number with a maximum percent increase in peak runoff of approximately 84% due to a 25% increase in the curve number. This indicates that the model results are highly sensitive to the overall level of impervious cover that will ultimately be constructed in the study area; any modifications to the development plan (i.e., densification) need to be considered in the context of their potential stormwater impacts.

4. Stormwater Runoff Assessment

This section provides the results of the flood assessment, erosion assessment and water quality assessment which were undertaken to define the targets for the design of stormwater management controls in the development area. Development of the stormwater concept design to meet these design criteria is discussed in section 5.

4.1 Flood Assessment

The purpose of the flood assessment is to understand the impact of the proposed development on the peak runoff flows where the site discharges to the receiving environment. Higher peak flow rates create higher water levels in channels, which increases the risk of flooding for adjacent people, property, infrastructure, and natural habitat. Generally, paving of roads and construction of residential areas results in an increase in impervious area, which causes an increase in peak flows and runoff volumes as less water infiltrates into the soil or evaporates.

The proposed stormwater management design criterion is to control the post-development peak flow rates to pre-development levels. It is recommended to provide flood control for the full range of ARI events from the 2-year to the 100-year events to ensure a robust level of runoff management.

Pre-development and uncontrolled post-development peak flow rates were established at all site discharge locations using the PCSWMM models described in section 3. Stormwater management controls must be designed to reduce the pre-development flow rates such that they are equal to or less than post-development levels at each of these locations.

Typically, stormwater management measures are used to attenuate post-development peak flow rates through the provision of storage. Wetlands, dry ponds, and wet ponds are examples of stormwater management measures that may be considered to attenuate peak flow rates for flood control. The selection and conceptual design of these controls is further assessed in section 5 of the report.

Table 4.1 presents the pre-development and uncontrolled post-development peak flow rates for the 50-year and 100-year return period events for each discharge location, previously identified in Figure 2.3. It also presents the estimated minimum storage volume that is required to attenuate the post-development flow rates to pre-development levels for the maximum design storm event, based on analysis using the Storage Calculator Tool in PCSWMM (which provides a useful but conceptual indication of required storage volume). A full summary of the results for all ARI events is included in Appendix B. In all cases the 100-year ARI storage volume is the largest value of the ARI events assessed.

Table 4.1 Flood assessment results and criteria

Discharge Point[1]	50-year ARI			100-year ARI		
	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required
	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)
A01	0.09	0.23	288	0.11	0.27	323
A02	0.49	1.49	2,129	0.61	1.71	2,288
A03	0.27	0.86	1,159	0.33	0.99	1,267
A04	0.12	0.40	506	0.15	0.46	546
A05	1.88	5.65	7,948	2.33	6.45	8,604
B01	0.14	0.37	630	0.17	0.42	696
B02	0.12	0.36	480	0.15	0.42	521
B03	0.16	0.55	741	0.20	0.63	793
B04	0.05	0.13	147	0.06	0.15	168
B05-1	0.14	0.48	654	0.17	0.55	715
B05-2-3-4[2]	0.24	0.66	871	0.30	0.77	947
B05-5-6-7[2]	1.36	4.51	6,220	1.69	5.02	6,732
B05-8	0.81	2.68	3,649	1.00	2.99	3,986

Discharge Point[1]	50-year ARI			100-year ARI		
	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required
	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)
B05-9	0.68	1.97	2,917	0.84	2.18	3,175
C01	0.59	1.77	2,519	0.76	1.95	2,639
C02	0.05	0.12	131	0.06	0.15	149
D01	0.05	0.33	699	0.06	0.38	783
D02	0.17	0.91	1,949	0.22	1.07	2,141
D03	0.09	0.52	1,113	0.11	0.61	1,252
D04	0.06	0.43	851	0.08	0.50	912
E01	0.58	2.47	4,392	0.78	2.95	4,725
E02	1.84	2.94	5,826	2.45	3.73	6,362
F01	0.55	2.87	6,348	0.73	3.25	6,599
F02	0.24	1.41	2,572	0.31	1.53	2,728

1. The discharge point may include smaller upstream discharge points, as indicated in the naming of the catchment.

2. Discharge points B05-2, B05-3 and B05-4 as well as B05-5, B05-6, B05-7 have the potential to be combined into centralized storage areas; as such, the required storage volumes for these groups of outlets have been lumped together.

A screen capture of the storage calculator tool is provided in Figure 4.1. The post-development uncontrolled hydrograph is presented in yellow, and the controlled hydrograph defined by the target peak flow rate is presented in blue. The storage volume required to attenuate the post to pre-development peak flow rates is estimated as the blue shaded area between the two hydrographs.

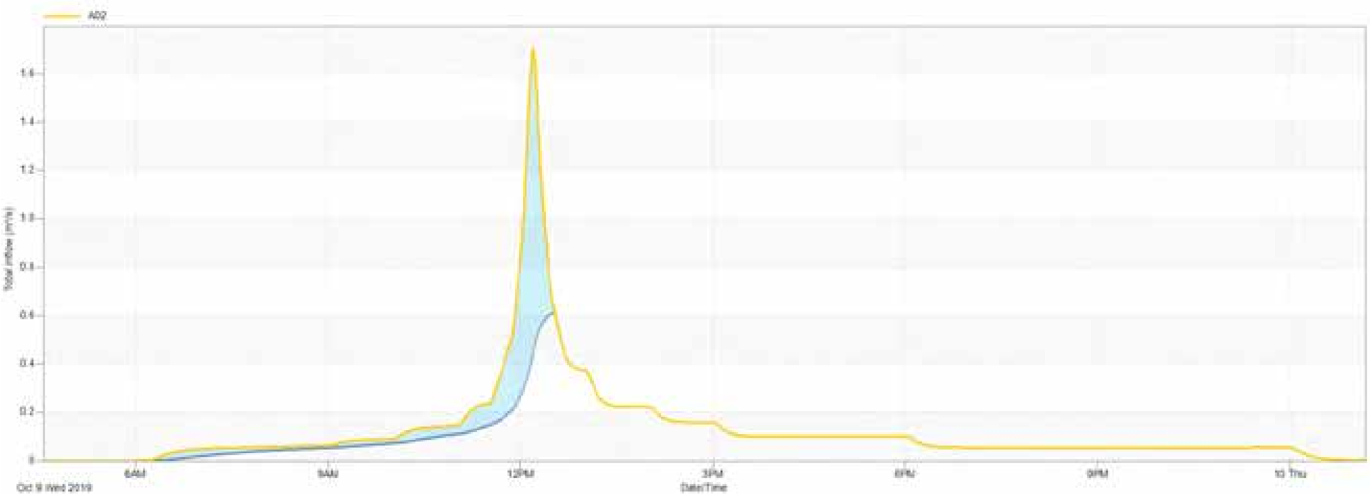


Figure 4.1 PCSWMM storage calculator tool output (sample)

4.2 Erosion Assessment

The purpose of the erosion assessment is to understand the impact of the proposed development on the degree and rate of erosion in the receiving watercourses, which is determined both by the erosivity of the watercourse bed and bank soils, and the magnitude and duration of flows within the watercourse.

The first factor, the erosivity of the watercourse bed and bank soils, is quantified by an erosion threshold, representing the flow level at which the soil particles will erode. The determination of erosion thresholds for the selected Aokautere receiving watercourses is further discussed in section 4.2.1.

The second factor, the magnitude and duration of flows within the watercourse, is assessed through a continuous hydrologic and hydraulic model using a long-term rainfall time series. The resulting time series of watercourse flows is then used to calculate the frequency and duration of erosion threshold exceedance, as well as the cumulative effective work exerted on the watercourse by the flows. Pre- and post-development modelled flow time series can be compared in this way to assess the expected erosion impacts related to the development. The evaluation of erosion threshold exceedance is described in section 4.2.2.

4.2.1 Determination of Erosion Thresholds

An understanding of the potential geomorphic response of a receiving watercourse to changes in flow regime allows the sensitivity of the channels to be assessed. It also allows quantification of their potential to assimilate flows without exacerbating or increasing instream erosion beyond natural rates. In support of this approach a target flow is usually defined for comparison between pre- and post-development conditions. This target flow is usually defined as an erosion threshold, which is the flow that theoretically can entrain bed or bank sediments within the most sensitive reach (i.e., section) of the watercourse.

An erosion threshold is assessed by identifying the most sensitive reaches along the network of channels that could potentially be impacted. From this reach, a target critical velocity, or critical shear stress for the bed or bank materials, is defined. Once the critical shear stress or velocity is known, the equivalent discharge can be determined from detailed measurements of the watercourse geometry.

For the Aokautere Structure Plan area, the watercourses located in the gullies upstream of discharge points A and B were assessed as a significant portion of the proposed development would discharge stormwater runoff into these gullies. As well, there is a desire to minimise the need for additional watercourse stabilisation or restoration in response to stormwater flows, as the gullies have been identified as having significant terrestrial and aquatic features. The stream located downstream of discharge points C and D was assessed as an indication of a larger watercourse in similar conditions as the gully streams. For the purpose of this study, these three watercourses were designated as Aokautere Church Stream (upstream of discharge point A, close to the Aokautere Community Church), the Moonshine Valley Reserve Stream (upstream of discharge point B, located in the Moonshine Valley Reserve) and the Tutukiwi Reserve Stream (downstream of discharge points C and D, located in the Tutukiwi Reserve).

The Aokautere Church Stream in the study reach was observed to be a narrow, grass-lined watercourse with a bed largely composed of fine silt and clay (Photo 1 below). However, in one location at the downstream end of the study reach, a large scour pool had formed (Photo 2 below), exposing the layer of sand, gravel and cobbles underlying the thick silt and clay layer. The stream was observed to have very little active flow, consisting mostly of remnant pools from the previous rainfall event.

↑ Photo 1: Aokautere Church Stream (typical)

↑ Photo 2: Scour pool in Aokautere Church Stream

The Moonshine Valley Reserve Stream flows through the densely vegetated Reserve in two distinct reaches, located upstream and downstream of a significant scour pool and “knick point” in the stream bed (Photo 3 below). Upstream of the scour pool, the stream is roughly rectangular in section and relatively narrow (Photo 4 below), with a bed composed of soft silt and clay similar to the Aokautere Church Stream. Downstream of the scour pool, significant erosion has widened and deepened the stream, exposing a layer of sand, gravel and cobbles underlying the silt and clay.

↑ Photo 3: Scour pool in Moonshine Valley Reserve Stream

↑ Photo 4: Moonshine Valley Reserve Stream upstream of scour pool (typical)

The Tutukiwi Reserve Stream receives drainage from several of the eastern slopes of the Structure Plan area as well as rural lands southeast of the study area, and is significantly larger than both the Aokautere Church Stream and Moonshine Valley Reserve Stream. Although some reaches of the stream are exhibiting signs of erosion and adjustment (Photo 5 below), the stream appears to be generally in stable condition where it flows through the established bush (Photo 6 below).

↑ Photo 5: Eroded meander bend in Tutukiwi Reserve Stream

↑ Photo 6: Tutukiwi Reserve Stream (typical)

The threshold discharge in the study streams is limited by the shear stress resistance of the bed and bank material. Critical shear stress is the shear acting on the bed that can potentially entrain a given characteristic sediment size. To compare the sensitivity of the bed and banks and examine initial entrainment and full bed mobilization, shear stress thresholds for bank materials, the median bed materials (D50) and larger bed materials (defined by the D84) are provided. The critical shear stress for the median bed materials (D50) and larger bed materials (D84) are based on critical shear for non-cohesive sediments from the Miller et al. (1977) model. The calculation of critical shear stress for bank materials was based on cohesive sediments from tables and plots from Chow (1959). The shear stress on the bank material was estimated as 75% of the average bed shear stress since bank materials are not subject to the same level of shear as the bed (Chow, 1959). The resulting erosion thresholds were determined based on the sensitivity of both channel bed and banks. The determination was aided by field observations at the time of survey.

Table 4.2 summarises the key stream characteristics used in the assessment of erosion thresholds, as observed and/or measured on site.

Table 4.2 Summary of stream characteristics for erosion threshold assessment

Parameter	Aokautere Church Stream	Moonshine Valley Reserve Stream	Tutukiwi Reserve Stream
Bankfull Width (m)	2.2	2.5	4.1
Bankfull Depth (m)	0.4	0.6	0.6
Channel Gradient (%)	3.24	1.87	0.94
Bed Material D50 (mm)	1	1	15
Bed Material D84 (mm)	2	2	32
Manning's n[1]	0.045	0.04	0.04
Bankfull Velocity (m/s)	2.02	2.14	1.72
Bankfull Discharge (m^3/s)	1.90	3.01	3.99
Flow competence[2] (m/s) for D50	0.2	0.2	0.7
Flow competence[2] (m/s) for D84	0.3	0.3	1.0
Critical Shear (Nm-2)[3] for D50	0.73	0.73	10.93
Critical Shear (Nm-2)[3] for D84	1.46	1.46	23.31
Critical Shear (Nm-2)[4] for bank material	5	5	5
Parameters for Typical Channel Cross Section at the Erosion Threshold			
Max Critical depth (m) for entrainment	0.024	0.029	0.083
Critical discharge (m^3/s) for entrainment	0.001	0.003	0.038
Critical velocity (m/s) for entrainment	0.20	0.23	0.37

1. *based on visual estimate (2019) and checked using technique outlined in Chow (1959)*
2. *according to Komar (1987)*
3. *according to Miller et al. (1977)*
4. *from tables in Chow (1959)*

For both the Aokautere Church Stream and the Moonshine Valley Reserve Stream, the erosion threshold was determined to be very low due to the sensitivity of the existing fine silt and clay bed and bank materials. This is compared to the more mature/established Tutukiwi Stream which has a coarser bed material and thus higher thresholds for erosion. It is likely that these streams erode easily in response to flow, which is considered reasonable given the location of the streams within gullies that have been historically shaped by erosive processes. This also makes the streams highly sensitive to changes in flow and presents a significant challenge to stormwater management in mitigating erosion impacts from development. To assess the sensitivity of the streams to changes in this threshold, a higher threshold of 0.05 m^3/s was also analysed, representing a potential higher end threshold for more cohesive fine-grained materials.

An assessment of the erosion threshold exceedance at both streams is presented in the following section.

4.2.2 Erosion Threshold Exceedance

Erosion thresholds are regularly exceeded in natural systems with defined watercourses; in fact, it is erosion that forms the watercourse out of the surrounding soils. In order to minimize potential impacts and maintain the natural channel function, post-development erosive flows should match as closely as possible the pre-development erosive flows in magnitude and duration. Comparisons of exceedance of the erosion threshold by the pre- and post-development flows can be determined using several criteria including frequency of exceedance and cumulative effective work.

Exceedance criteria are determined through the use of a continuous hydrologic model to compare time series of discharge over several years. The frequency or cumulative time of exceedance provides a simple comparison of the amount of time the discharge is above the erosion threshold, and thus the amount of time in which channel erosion is more likely to occur. It does not however account for the excess work above the erosion threshold, which represents the accumulation of active force applied against the stream bed and banks from the flows. The excess work is dependent on the magnitude and duration of the flow exceedance. It can be represented by the

Cumulative Effective Work Index or the total amount of stream power above the erosion threshold as defined by the threshold shear stress. It is calculated following the method described in Rowney and MacRae (1992):

$$PWR = \sum (\tau_o - \tau_{thr}) V \Delta t$$

Where PWR is the cumulative stream energy expended above a threshold value (J/m^2)

τ_o is the instantaneous shear stress at the stream reach (N/m^2)

τ_{thr} is the threshold shear at the stream reach (N/m^2)

Δt time step

V velocity (m/s)

The Cumulative Effective Work Index is typically a better indicator of potential impacts to watercourse geomorphology than the time of exceedance, since it incorporates both the frequency and magnitude of the exceedance.

The Cumulative Effective Work Index was determined for the uncontrolled post-development conditions in Aokautere using a custom MATLAB program developed for this analysis and compared to the pre-development conditions to determine potential impacts on the local watercourse geomorphology. Table 4.3 and Table 4.4 summarise the results of the erosion threshold exceedance analysis for the lower and higher thresholds, respectively, for both the pre-development and uncontrolled post-development scenarios. Graphs of the exceedance analysis results are included in Appendix C.

The parameters summarised in Table 4.3 and Table 4.4 demonstrate the complex interaction between the magnitude and duration of stream flows and the erosive potential of those flows. An overview of the significance of each parameter reported in the tables is included below:

– Total Exceedance Time represents the total number of hours over the five-year simulation period (total of approximately 43,848 hours) when the modelled stream flow exceeds the erosion threshold.
– % Exceedance Time is the total exceedance time represented as a percentage of the total simulation time, to illustrate the significance of the total values.
– Number of Exceedance Events shows the number of discrete occasions where the flow exceeded the erosion threshold and then returned to a level below the threshold, which could indicate a higher potential for erosion.
– Cumulative Effective Discharge sums the total modelled flow volume that was above the erosion threshold for the stream, indicating the raw volume of erosive flow that will act on the stream.
– Cumulative Effective Work Index sums the total amount of “work” exerted on the stream above the erosion threshold, indicating the total amount of erosive energy over the simulation period.

Table 4.3 Summary of erosion threshold exceedance analysis – uncontrolled post-development (lower threshold)

Parameter	Aokautere Church Stream (0.001 m^3/s threshold)			Moonshine Valley Reserve Stream (0.003 m^3/s threshold)			Tutukiwi Reserve Stream (0.038 m^3/s threshold)		
	Pre-Dev.	Post-Dev. Uncontrolled	% Change	Pre-Dev.	Post-Dev. Uncontrolled	% Change	Pre-Dev.	Post-Dev. Uncontrolled	% Change
Total Exceedance Time (hrs)	5,131	3,924	-24%	3,336	3,140	-6%	3,466	3,432	-1%
% Exceedance Time	11.7	8.9	-24%	7.6	7.2	-5%	7.9	7.8	-1%
Number of Exceedance Events	169	263	56%	184	287	56%	100	149	49%
Cumulative Effective Discharge (m^3)	890,800	918,800	3%	738,700	982,400	33%	606,700	585,000	-4%
Cumulative Effective Work Index (J/m^2)	173,700,000	176,100,000	1%	84,360,000	109,900,000	30%	149,400,000	143,900,000	-4%

Note that the total simulation time extends over 5 years, totalling approximately 43,848 hours.

Table 4.4 Summary of erosion threshold exceedance analysis – uncontrolled post-development (higher threshold)

Parameter	Aokautere Church Stream (0.05 m^3/s threshold)			Moonshine Valley Reserve Stream (0.05 m^3/s threshold)			Tutukiwi Reserve Stream (0.05 m^3/s threshold)		
	Pre-Dev.	Post-Dev. Uncontrolled	% Change	Pre-Dev.	Post-Dev. Uncontrolled	% Change	Pre-Dev.	Post-Dev. Uncontrolled	% Change
Total Exceedance Time (hrs)	1,207	1,112	-8%	997	1,131	13%	3,171	3,110	-2%
% Exceedance Time	2.8	2.5	-11%	2.3	2.6	13%	7.2	7.1	-1%
Number of Exceedance Events	188	292	55%	177	286	62%	102	144	41%
Cumulative. Effective Discharge (m^3)	531,000	621,300	17%	468,300	710,900	52%	592,400	570,900	-4%
Cumulative Effective Work Index (J/m^2)	83,250,000	97,010,000	17%	44,400,000	66,940,000	51%	143,600,000	138,300,000	-4%

Note that the total simulation time extends over 5 years, totalling approximately 43,848 hours.

The results shown in Table 4.3 and Table 4.4 indicate that the uncontrolled post-development stormwater runoff flows have the potential to incur a significant amount of additional “work” on the smaller streams in the study area, representing a similarly significant increase in the potential for erosion and stream degradation. Although post-development conditions in the Tutukiwi Reserve Stream show a significant increase in the number of exceedance events, this does not translate into a direct increase in erosive potential. This is due to the much larger catchment of this stream compared to the Aokautere Church Stream and Moonshine Valley Reserve Stream, as well as the more mature and stable channel configuration observed in the Tutukiwi Reserve Stream where it has accessed the granular/cobble glacial till that underlies the finer deposits in the study area.

To mitigate erosion impacts from urban runoff, stormwater controls need to mitigate this additional work through reducing the shear stress, velocity, and duration of flow events in the stream that exceed the erosion threshold. This is accomplished in similar ways to the mitigation of flood risk: detention to reduce peak flows and infiltration to reduce total flow volume.

For the Aokautere Structure Plan area, infiltration is not considered an effective means of stormwater management due to the widespread presence of cohesive soils with low hydraulic conductivity, as well as to the high levels of maintenance and pre-treatment required to sustain large-scale infiltration systems. Therefore, stormwater detention was investigated for erosion risk mitigation. The continuous hydrologic model used to assess the uncontrolled post-development erosion impacts was revised to incorporate the recommended stormwater detention volumes established in Section 4.1 to mitigate flood risk. The erosion threshold exceedance analysis was then completed for these controlled post-development runoff results.

For the purpose of evaluating the effectiveness of proposed stormwater management controls, the higher erosion threshold was deemed more appropriate as it better accommodates flow changes that result from urbanisation and stormwater management. As well, the higher threshold appears to provide a more conservative assessment of stream erosion impact due to stormwater runoff, leading to more robust and resilient recommendations for the Structure Plan.

Erosion threshold analysis results for the preliminary controlled post-development conditions are summarised in Table 4.5, Table 4.6 and Table 4.7.

Table 4.5 Summary of erosion threshold exceedance analysis – controlled post-development – Aokautere Church Stream

Parameter	Aokautere Church Stream (0.05 m^3/s threshold)				
	Pre-Dev.	Post-Dev. Uncontrolled	% Change[1]	Post-Dev. Controlled	% Change[1]
Total Exceedance Time (hrs)	1,207	1,112	-8%	1,382	14%
% Exceedance Time	2.8	2.5	-11%	3.2	14%
Number of Exceedance Events	188	292	55%	179	-5%
Cumul. Effective Discharge (m^3)	531,000	621,300	17%	466,100	-12%
Cumul. Effective Work Index (J/m^2)	83,250,000	97,010,000	17%	72,390,000	-13%

1. *The % Change figure is calculated for uncontrolled and controlled post-development conditions relative to pre-development conditions.*

Table 4.6 Summary of erosion threshold exceedance analysis – controlled post-development – Moonshine Valley Reserve Stream

Parameter	Moonshine Valley Reserve Stream (0.05 m^3/s threshold)				
	Pre-Dev.	Post-Dev. Uncontrolled	% Change[1]	Post-Dev. Controlled	% Change[1]
Total Exceedance Time (hrs)	997	1,131	13%	1,605	61%
% Exceedance Time	2.3	2.6	13%	3.7	61%
Number of Exceedance Events	177	286	62%	147	-17%
Cumul. Effective Discharge (m^3)	468,300	710,900	52%	489,700	5%
Cumul. Effective Work Index (J/m^2)	44,400,000	66,940,000	51%	45,930,000	3%

1. *The % Change figure is calculated for uncontrolled and controlled post-development conditions relative to pre-development conditions.*

Table 4.7 Summary of erosion threshold exceedance analysis – controlled post-development – Tutukiwi Reserve Stream

Parameter	Tutukiwi Reserve Stream (0.05 m^3/s threshold)				
	Pre-Dev.	Post-Dev. Uncontrolled	% Change[1]	Post-Dev. Controlled	% Change[1]
Total Exceedance Time (hrs)	3,171	3,110	-2%	3,065	-3%
% Exceedance Time	7.2	7.1	-1%	7.0	-3%
Number of Exceedance Events	102	144	41%	103	1%
Cumul. Effective Discharge (m^3)	592,400	570,900	-4%	568,100	-4%
Cumul. Effective Work Index (J/m^2)	143,600,000	138,300,000	-4%	137,800,000	-4%

1. *The % Change figure is calculated for uncontrolled and controlled post-development conditions relative to pre-development conditions.*

The erosion threshold assessment of the preliminary controlled post-development conditions indicate that the proposed stormwater detention volumes required for flood risk mitigation are largely adequate to mitigate erosion risk in the study area streams. However, this is in comparison only to the current pre-development conditions. The Aokautere Church Stream and Moonshine Valley Reserve Stream will remain highly sensitive to erosion in the future regardless of upstream development and will continue to erode and degrade in a manner that may create slope stability risk or water quality impacts.

4.2.3 Gully Slope Erosion Impacts

In addition to the potential for erosion impacts to the receiving watercourses of the study area, there are potential impacts to the gully slopes from uncontrolled overland flows. There are numerous existing rills and channels that have formed along the gully slopes due to overland flow from the plateaus, which are equally as or more sensitive than the primary gully watercourses and represent a potential slope stability and erosion risk if stormwater flows from development are permitted uncontrolled access to these areas. As well, encroachment of development on these critical slopes can cause further disturbance and impact to vegetation and slope stability. Formalised stormwater collection could be considered along the tops of the gully slopes (e.g., swales) with controlled slope drains to safely discharge the runoff to the gully floor. Further, a buffer/setback from the top of slope, particularly along the rear of residential properties, could be considered to minimise encroachment and provide a corridor for the proposed stormwater collection system. This is further evaluated in section 5. It should be noted that this assessment does not consider any geotechnical risks that may require further setback.

4.3 Water Quality Assessment

Stormwater quality parameters of concern due to urbanization include suspended solids, metals (i.e., copper, zinc, lead, etc.), petroleum hydrocarbons, nutrients, and other organic compounds. Urban catchments can also increase runoff temperature, which may impact habitat for sensitive cold-water species. These contaminants have the potential to impact aquatic life and habitat in receiving streams. Impacts can be particularly high during construction of the proposed development if not managed through effective erosion and sediment controls.

Enhanced approaches to water quality control include provision for retention and disposal of the “first flush” runoff from urban catchments, i.e., the initial runoff from a previously dry catchment. The first flush typically carries the highest proportion of contaminants as particles, oils, and other urban contaminants are washed from the impervious surfaces. A typical first flush management target would be to retain and dispose of the first 10 to 25 mm of a rainfall event on site through infiltration, detention, evaporation, etc. This equates to approximately the 90th percentile rainfall depth over a 24-hour period, capturing the vast majority of smaller, more frequent events during a typical year. A treatment train approach may be required, where multiple water quality devices are implemented throughout the drainage system to achieve the water quality criteria.

Figure 4.2 (from GD2017/01, Table 15) summarizes the effectiveness of various stormwater management devices in terms of their water quantity control and water quality treatment performance. The table shows that bioretention devices, including swales, rain gardens, tree pits, and planter boxes, provide effective water quality treatment for all listed contaminants. Constructed wetlands provide effective water quality control for most contaminants with the exception of indicator bacteria and temperature, for which they achieve partially effective control. Wet ponds provide effective water quality control for sediments and gross pollutants, and partially effective control for all other parameters, with the exception of temperature. Constructed wetlands and wet ponds also provide the benefit of effective water quantity control.

	Quantity control					Quality control								
Key ● Effective ○ Partially effective - Not effective	1% AEP	Detention of 50% and 10% AEP	90th & 95th percentile detention	Groundwater recharge	Retention	Sediment	Gross pollutants	Heavy metals	Oils and grease	Nutrients	Organics	Hydrocarbons	Indicator bacteria	Temperature
Pervious pavement - unlined	-	-	●	○	●	●	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]
Pervious pavement - lined	-	-	●	-	-	●	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]	-[b]
Living roof	-	-	●[a]	-	●	○	NA	○	NA	○	○	NA	○	●
Rainwater tank (no reuse)	-	○	●	-	-	●	NA	○	NA	○	○	NA	○	○
Rainwater tank (with reuse)	-	○	●	-	●	●	NA	○	NA	○	○	NA	○	○
Infiltration device	-	○	●[a]	●	●	-	-	-	-	-	-	-	-	●
Swale (lined)	-	-	-	-	-	●	○	○	○	○	○	○	○	●
Bioretention swale (unlined)	-	-	●	●	●	●	●	●	●	●	●	●	●	●
Rain garden	-	-	●	●	●	●	●	●	●	●	●	●	●	●
Stormwater tree pit[c]	-	-	○	○	●	●	●	●	●	●	●	●	●	●
Planter box	-	-	○	○	●	●	●	●	●	●	●	●	●	●
Constructed wetland	-[d]	●	●	-	○	●	●	●	●	●	●	●	○	○
Wet pond	●	●	●	-	-	●	●	○	○	○	○	○	○	-
Dry pond (detention basin)	●	●	●	-	-	-	-	-	-	-	-	-	-	●

Notes:

NB: Assumes sizing, construction and maintenance are compliant with this guideline's requirements

NA: Not applicable, does not treat this pollutant because it is generally not present in the drainage area

●[a]: Assumes retention of up to the 90th and 95th percentile events

-[b]: Assumes limited water quality treatment for active pervious paving systems. Passive pervious paving is assumed to have some treatment effectiveness if maintained correctly

[c]: Stormwater tree pits are different to street tree pits in that they are specifically designed for stormwater management and must be sized accordingly.

-[d] Wetlands designs should bypass large storm events to protect vegetation and ensure sediments are not resuspended

Figure 4.2 Summary of Effectiveness of Stormwater Management Devices in terms of Water Quantity Control and Water Quality Treatment (GD2017/01, Table 15)

Bioretention devices can temporarily store, treat, and infiltrate runoff at the source. A number of bioretention devices may be implemented as part of a multi-device treatment approach. The Water Quality Flow (WQF) event is used to size bioretention devices for water quality treatment purposes, where WQF is calculated with a rainfall intensity of 10 mm/hour (an approximate 90th percentile annual rainfall intensity for the Auckland area) for all impervious areas. The minimum area for a bioretention bed is calculated as the WQF divided by the infiltration rate of the engineered filter media, which is typically 1000 mm/hr or less. A safety factor of '0.5' is applied to the infiltration rate to account for clogging of the filter media, similar to the sizing of soakage devices which are commonly used in certain parts of Palmerston North.

Wetlands and wet ponds are designed to provide water quality treatment through the provision of a permanent water quality volume and/or minimum 24-hour detention of the water quality volume. The permanent water volume (or water quality volume, WQV), is equivalent to the runoff volume generated by the 90th percentile storm event from all impervious areas. Wetlands and wet ponds are typically located at the base of a catchment at the end of a treatment train. Wetlands and wet ponds should have a minimum catchment area of 5 ha and a preferred catchment area of 10 ha in order to maintain volumetric turnover of the permanent pool volume (Ontario Ministry of the Environment, 2003).

Figure 4.3 shows that the 90th percentile rainfall depth over a 24-hour duration is approximately 15 mm based on the Palmerston North AWS record over the 1991 to 2019 period. It may also be desired to design water quality controls for a 25 mm daily rainfall depth, a typical “first flush” target in other jurisdictions, as a more conservative approach.

A summary of WQF and WQV calculations for the Aokautere discharge locations is shown in Table 4.8.

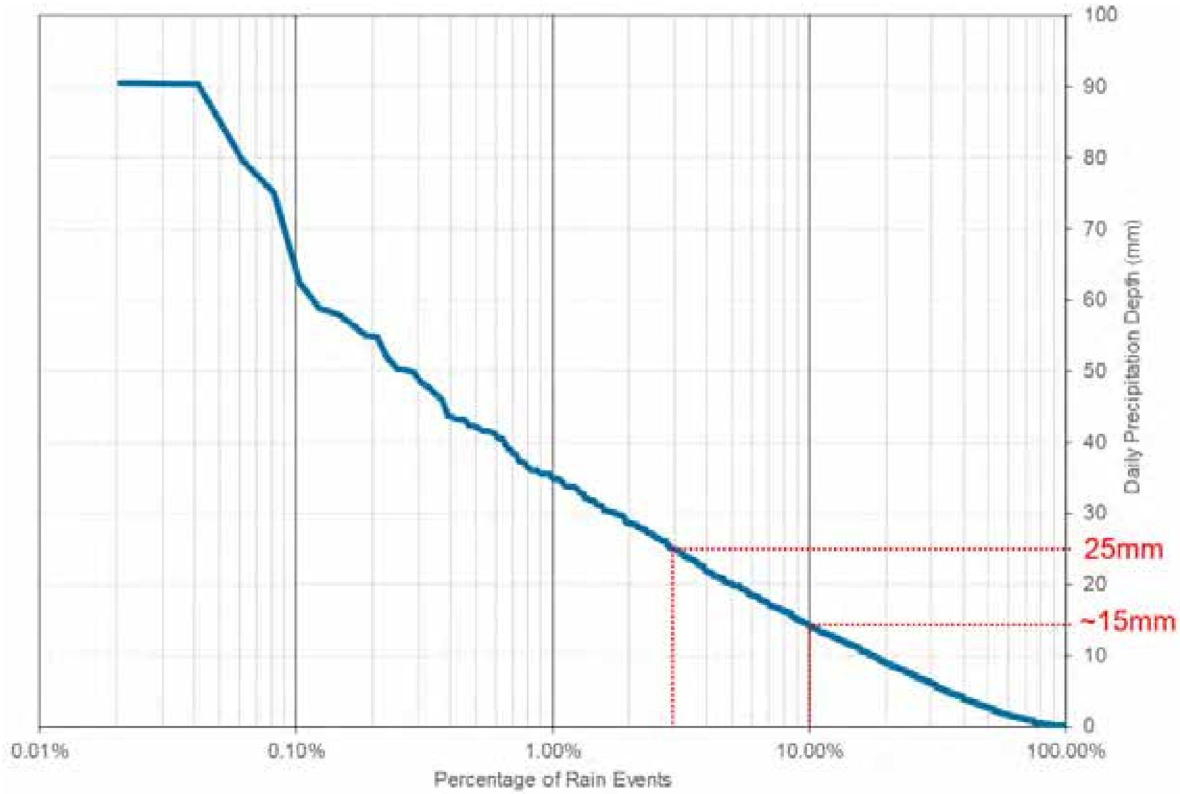


Figure 4.3 Ranked Daily Rainfall Depth at Palmerston North AWS (1991-2019)

Table 4.8 Summary of WQF and WQV values by discharge location

Discharge Location	Area (ha)	Impervious Area (ha)	Proportion of Impervious Area	Runoff Coefficient[1]	WQF (m^3/hr)	WQV (m^3)[2]
A01	1.6	0.7	0.4	0.4	67.3	99.2
A02	8.5	6.1	0.7	0.7	592.6	916.8
A03	4.6	3.6	0.8	0.7	342.7	532.7
A04	2.0	1.5	0.8	0.7	149.2	231.8
A05	32.6	22.4	0.7	0.7	2,178.0	3,358.7
B01	2.4	1.8	0.7	0.7	174.0	269.9
B02	2.2	1.2	0.6	0.5	118.3	179.3
B03	2.8	2.2	0.8	0.8	210.3	327.2
B04	0.8	0.3	0.4	0.4	34.5	50.4
B05-1	2.4	1.9	0.8	0.8	187.0	291.3
B05-2-3-4	4.3	1.9	0.4	0.5	193.9	287.4
B05-5-6-7	24.1	18.5	0.8	0.7	1,783.1	2,771.0

Discharge Location	Area (ha)	Impervious Area (ha)	Proportion of Impervious Area	Runoff Coefficient[1]	WQF (m^3/hr)	WQV (m^3)[2]
B05-8	14.3	10.6	0.7	0.7	1,029.2	1,596.2
B05-9	12.0	8.7	0.7	0.7	845.9	1,309.7
C01	9.8	7.5	0.8	0.7	726.7	1,129.7
C02	0.8	0.4	0.5	0.5	36.9	55.1
D01	1.7	1.3	0.8	0.8	129.7	202.2
D02	6.0	2.9	0.5	0.5	288.4	430.7
D03	3.1	1.2	0.4	0.4	120.1	174.3
D04	2.2	1.6	0.7	0.7	155.2	240.6
E01	19.6	7.6	0.4	0.4	778.9	1,134.9
E02	61.8	6.2	0.1	0.1	865.2	927.0
F01	18.8	14.5	0.8	0.7	1,395.1	2,168.6
F02	8.0	5.9	0.7	0.7	570.9	884.9

1. *The runoff coefficient (c) is derived from the proportion of impervious area (i) using the equation $c = 0.05 + 0.9i$, based on typical industry guidance*

2. *The water quality volume is determined using the 90th percentile rainfall depth of 15 mm, as shown in Figure 4.3*

Oil grit separator (OGS) units are another type of water quality control (not listed in Table 15 of GD2017/01) that are used to collect oils and sediment from urban runoff. OGS units may be used to provide pre-treatment in a multi-device approach, or they may provide adequate treatment at the base of a small catchment area that is typically less than 2 ha in size (Ontario Ministry of the Environment, 2003). Suppliers typically assist in the selection of an appropriate OGS model, based on site rainfall characteristics, impervious area, and a representative sediment particle size distribution.

A single device (wetland, wet pond) or multi-device treatment approach may be employed to achieve the desired water quality treatment performance at each outfall location. Opportunities to integrate stormwater management features such as rain gardens or wetlands into public amenity spaces should be identified; these opportunities may in turn suggest alternative methods of stormwater treatment that will better integrate into these public spaces. For example, community gardens could be located in rain garden areas, or a stormwater wetland could be the focus of a recreational park or pathway.

5. Stormwater Concept Design

The concept design for the Aokautere stormwater management system has been developed to mitigate the flood, erosion and water quality impacts identified and quantified in section 4. A variety of stormwater controls can be considered to develop the overall treatment and mitigation approach for the study area, assuming that the selected controls satisfy the design criteria based on the analyses established in section 4. The selected controls will need to be evaluated for their constructability, capital costs, maintenance requirements, and impact on the surrounding development (in terms of space and amenity).

The concept design has been based on a residential land use throughout the Plan Change area. A proposal was put forward in early 2022 from one of the major landowners (Aokautere Land Holdings) to place a retirement village close to the proposed Town Centre, just off Pacific Drive and within the catchment of Pond A05, replacing an area previously identified for smaller-lot, medium density residential use. A proposed layout for the retirement village is included in Appendix A. The stormwater impact of the retirement village is considered to be comparable to the previously identified medium-density residential use, and so no specific additional controls are considered to be required; the retirement village design will need to incorporate bioretention/rain garden treatment of stormwater prior to discharge to external areas in accordance with the overall stormwater management strategy.

5.1 Design Criteria

For the Aokautere Plan Change the following design criteria are recommended to be adopted:

- Control of runoff peak flows to pre-development levels for the 2-year, 5-year, 10-year, 20-year, 50-year and 100-year ARI flows, to control flood risk.
- Further control of peak flows as needed to match the pre-development erosion threshold exceedance cumulative effective work index in the Aokautere Church Stream, Moonshine Valley Reserve Stream, and Tutukiwi Reserve Stream. The Erosion Assessment included in Section 4.2 indicates that erosion risk can be largely mitigated through the recommended detention volumes for flood control.
- Treatment of the 90th percentile rainfall volume (i.e., 15 mm) from impervious developed areas through a stormwater treatment device or multi-device system.

Table 5.1 summarises the requirements for flood and erosion control and water quality treatment for the study area discharge locations, based on the proposed location of stormwater detention (i.e., on the plateau or in the gullies).

Table 5.1 Required detention storage, footprints and bioretention footprints by discharge location

Discharge Location	Total Area (ha)	Impervious Area (ha)	Bioretention Footprint (m^2) [1, 2]	100-Year ARI Storage Footprint for Flood Storage on Plateaus (m^2) [3]	100-Year ARI Storage Volumes for Flood Storage in Gullies (m^2)
A01	1.6	0.7	140	500	
A02	8.5	6.1	1,190	1,980	
A03	4.6	3.6	690	1,220	
A04	2.0	1.5	300	650	
A05 [4]	32.6	22.4	4,360		8,610
B01	2.4	1.8	350	770	
B02	2.2	1.2	240	630	
B03	2.8	2.2	430	850	
B04	0.8	0.3	70	320	
B05-1	2.4	1.9	380	790	
B05-2-3-4	4.3	1.9	390		950
B05-5-6-7	24.1	18.5	3,570		6,740
B05-8	14.3	10.6	2,060		3,990

Discharge Location	Total Area (ha)	Impervious Area (ha)	Bioretention Footprint (m²) [1, 2]	100-Year ARI Storage Footprint for Flood Storage on Plateaus (m²) [3]	100-Year ARI Storage Volumes for Flood Storage in Gullies (m²)
B05-9	12.0	8.7	1,700	2,600	
C01	9.8	7.5	1,460	2,230	
C02	0.8	0.4	80	290	
D01	1.7	1.3	260	850	
D02	6.0	2.9	580		2,150
D03	3.1	1.2	250	1,210	
D04	2.2	1.6	320	950	
E01	19.6	7.6	1,560	3,670	
E02	61.8	6.2	1,740	4,760	
F01	18.8	14.5	2,800	4,920	
F02	8.0	5.9	1,150	2,290	
TOTAL	**246**	**130**	**26,070**	**31,480**	**22,440**

Note: bioretention, and flood storage footprint and volume values are rounded up to the nearest 10.

1. Bioretention footprint and permanent pool volumes reflect alternate approaches to achieving water quality treatment targets and can be considered mutually exclusive.

2. Bioretention footprint was determined using an assumed infiltration rate for engineered filter media of 1,000 mm/hr and a safety factor of 0.5 to account for potential clogging.

3. 100-Year ARI storage footprint was determined assuming a 2 metre maximum storage depth for the 100-year ARI flow, including a 0.3 m freeboard, and accounts for flood control only, not water quality treatment. Required volumes have been listed previously in Section 4.1.

4. Total values do not include the minor discharge locations that contribute to downstream discharge points (i.e., A05-1, A05-2, etc. are counted in A05).

5.2 Stormwater Constraints

The location and design of stormwater management controls needs to be considered in the context of a number of factors, including (but not limited to):

– the proposed development layout;
– sensitive environmental areas (i.e., areas of high vegetation or ecological value);
– the topography of the site;
– constructability; and
– ongoing maintenance requirements.

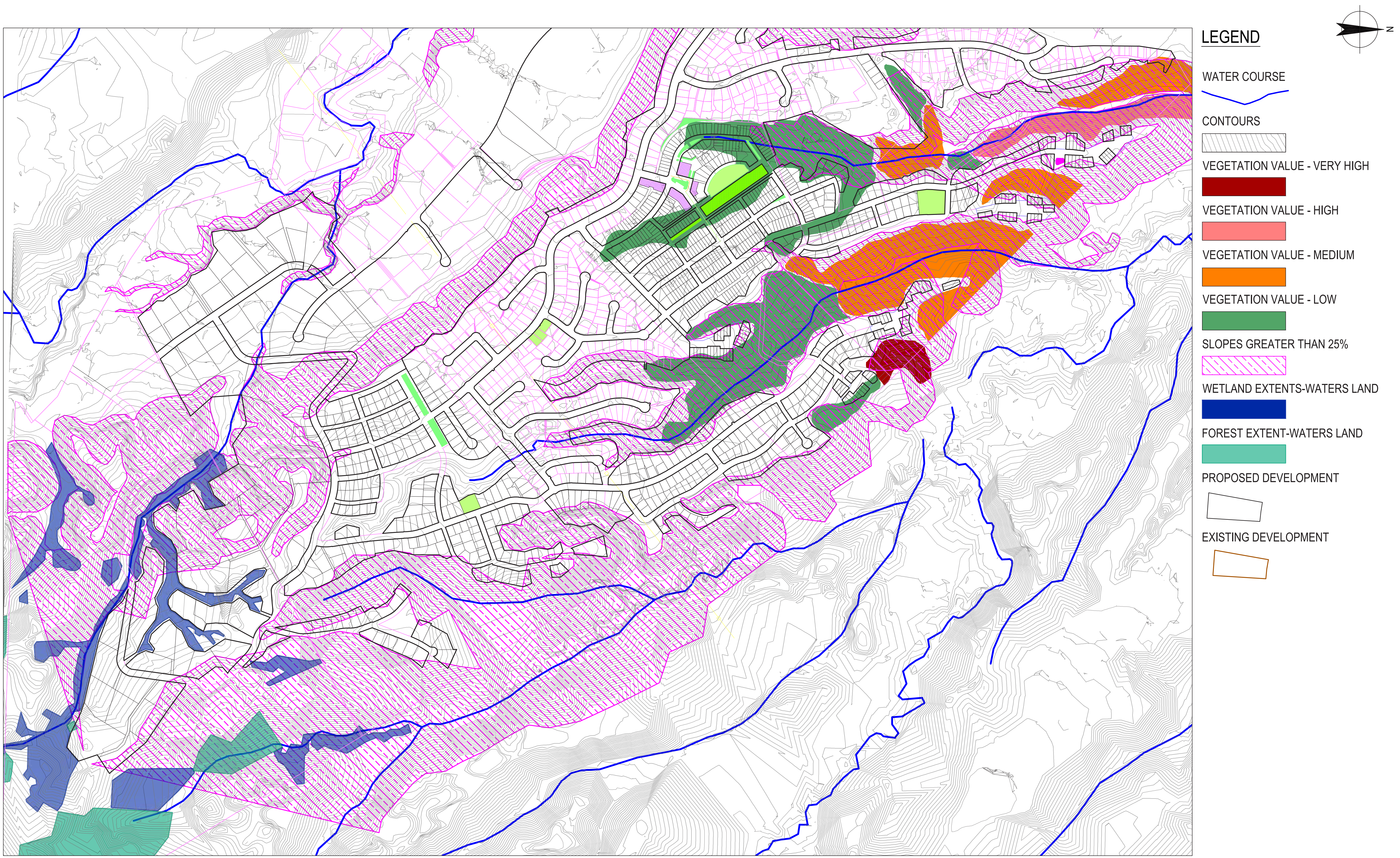
The incorporation of the recommended stormwater controls into the development layout will be completed by McIndoe Urban as part of the Structure Plan, and for the purpose of this concept design no modifications to the development layout were made. Where possible (i.e., when not constrained by other factors), stormwater controls were located in the gully system to minimise impact on developable area. As well, lots that have already been subdivided within the study area were avoided when placing stormwater detention features.

An environmental constraint study for the Plan Change area was completed by Forbes Ecology in parallel with this stormwater assessment, which identified areas of terrestrial/vegetation and aquatic ecological value; these areas are shown on Figure 5.1. Note that the aquatic potential of the Aokautere Church Stream and Moonshine Valley Reserve Stream were assessed to be higher downstream of the "low" value vegetation areas, where the streams transition from intermittent to permanent. As well, several wetland and forest areas have been identified by Forbes Ecology in the "Waters Land" property of the Structure Plan area (catchment E02); these areas are also shown on Figure 5.1 and are avoided in the placement of conceptual stormwater facilities.

The site topography constraints assessed for this study include the gully slopes and the suitability of the gullies to contain stormwater management features. For the assessment of gully slopes, an elevation analysis was completed to identify those areas where the ground slope was equal to or steeper than 25% (i.e., 4:1 horizontal to vertical), forming a perimeter around the tops of the gullies and approximately delineating a threshold beyond which some manner of runoff control is recommended. This 4:1 slope line is shown on Figure 5.1 and is intended only to evaluate the potential impact on developable land resulting from this type of control/buffer. A setback or buffer along the rear limits of residential properties roughly in alignment with the 4:1 slope line is recommended to both prevent development encroachment and serve as a stormwater collection corridor to protect the sensitive gully slopes. The specific width or location of the buffer should be modified to suit specific development needs if sufficient allowance for stormwater conveyance and future maintenance is made, and as permitted by geotechnical slope hazard concerns, using the 4:1 slope line as a baseline/reference. Figure 5.2 illustrates the Structure Plan implementation of a recommended stormwater setback along the rear property lines of residences, developed in collaboration with McIndoe Urban.

The suitability of the gullies to contain stormwater management features was assessed based on the vegetation and aquatic ecology constraints (e.g., no stormwater controls were recommended for areas of “medium” or greater vegetation value), constructability and access, as well as the topography of the gullies themselves. A conceptual assessment of gully grading was undertaken to identify if the required storage volume for stormwater detention could be accommodated on the gully floors (specifically at discharge points A05 and B05), allowing one centralised facility to manage runoff from several contributing upstream discharge points. Discharge point A05 appears to be suitable for such an installation, whereas point B05 is located in a narrow, steep portion of gully with limited opportunities for stormwater detention. However, several smaller upstream gullies have been used to locate stormwater detention for various subsets of the total B05 catchment area.

Property and topography constraints along Pacific Drive in the F01 catchment has limited the available locations for a detention pond to service this catchment. However, this catchment is already connected to a consented section of Council reticulation and is largely fully developed. There is an opportunity to install a new detention pond at the outlet of this reticulation system near the intersection of Old W Rd and Turitea Rd, but this is located outside of the Plan Change area and is therefore not included in the stormwater concept design.



Rev	Description	Checked	Approved	Date
B	LAYOUT CHANGES	DA*		22/04/22

Author C. O'DONNELL Drafting Check
Designer Design Check

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Project No. 12554906

Client PALMERSTON NORTH CITY COUNCIL

Project STUDY AREA STORMWATER CONSTRAINTS

Status FINAL

Status Code

Drawing Title FIGURE 5.1

Size A3

Drawing No. 12554906-FIGURE 5.1

Rev B

Plot Date: 30 May 2022 - 12:47 PM Plotted by: Danny Dao File Name: \\ghdnet\ghd\NZ\Palmerston North\Projects\51\12518392\CADD\Drawings\11204663-Figure 5.1.dwg

5.3 Stormwater Concept Design

The proposed stormwater concept design for the Aokautere structure plan area is recommended to be comprised of the elements identified below.

1. Detention ponds

Indicative conceptual locations and footprints for stormwater detention areas to mitigate flood and erosion risk are shown on Figure 5.2 for consideration in the development layout. The figure shows the footprints required for storage only; additional area would need to be added for treatment (as discussed further below).

2. Buffer strip

A 5 m buffer along the rear property lines of residential lots is also recommended to be incorporated into the development layout, in which no buildings would be recommended to be placed. This excludes any setback required for risk associated with landslips or other geotechnical hazard.

3. Stormwater reticulation

Stormwater reticulation is to be designed to accommodate the 10-year ARI flow and overland flow paths designed to accommodate the 100-year ARI flow, as per PNCC engineering standards. All stormwater outlets into the gully system must be designed as a piped system to safely convey the 100-year runoff to the gully floor, with appropriately designed erosion control measures at the ultimate outlet in the gully. Direct overland discharge down the gully slope is not recommended, nor is it permitted in the PNCC ESLD.

4. Water quality treatment

The incorporation of water quality controls is recommended to be completed when finalising the development layout as follows:

– Incorporate bioretention (rain gardens) into the road reserve in such a way that all discharge from impervious surfaces (roofs, driveways, road surface) is directed into the rain gardens for filtration. Underdrains located under the rain garden filter media will then direct the runoff into the reticulation network. Ensure that the minimum required bioretention footprint summarised in section 5.1 is achieved.

– If it is desired to utilise wet ponds or wetlands for stormwater quality treatment instead of (or in combination with) bioretention, the required permanent pool volume must be incorporated into the detention pond *in addition to* the required storage volume for flood control. In this case the footprints of the detention facilities would need to be increased to accommodate the shallower detention depth (assuming a maximum pond depth of 2 m is maintained, including 0.3 m of freeboard). Although the detention facilities would occupy a larger space, the centralisation of treatment has several benefits for operations and maintenance, as well as potential savings from reduced need for distributed bioretention facilities. An indicative estimate of wetland footprint for each discharge location is shown in Table 5.2, assuming an average 0.5 m deep wetland permanent pool and 1.1 m active storage detention area. Wetland footprints are indicated only for those facilities with a minimum catchment area of 5 ha to promote volumetric turnover of the permanent pool.

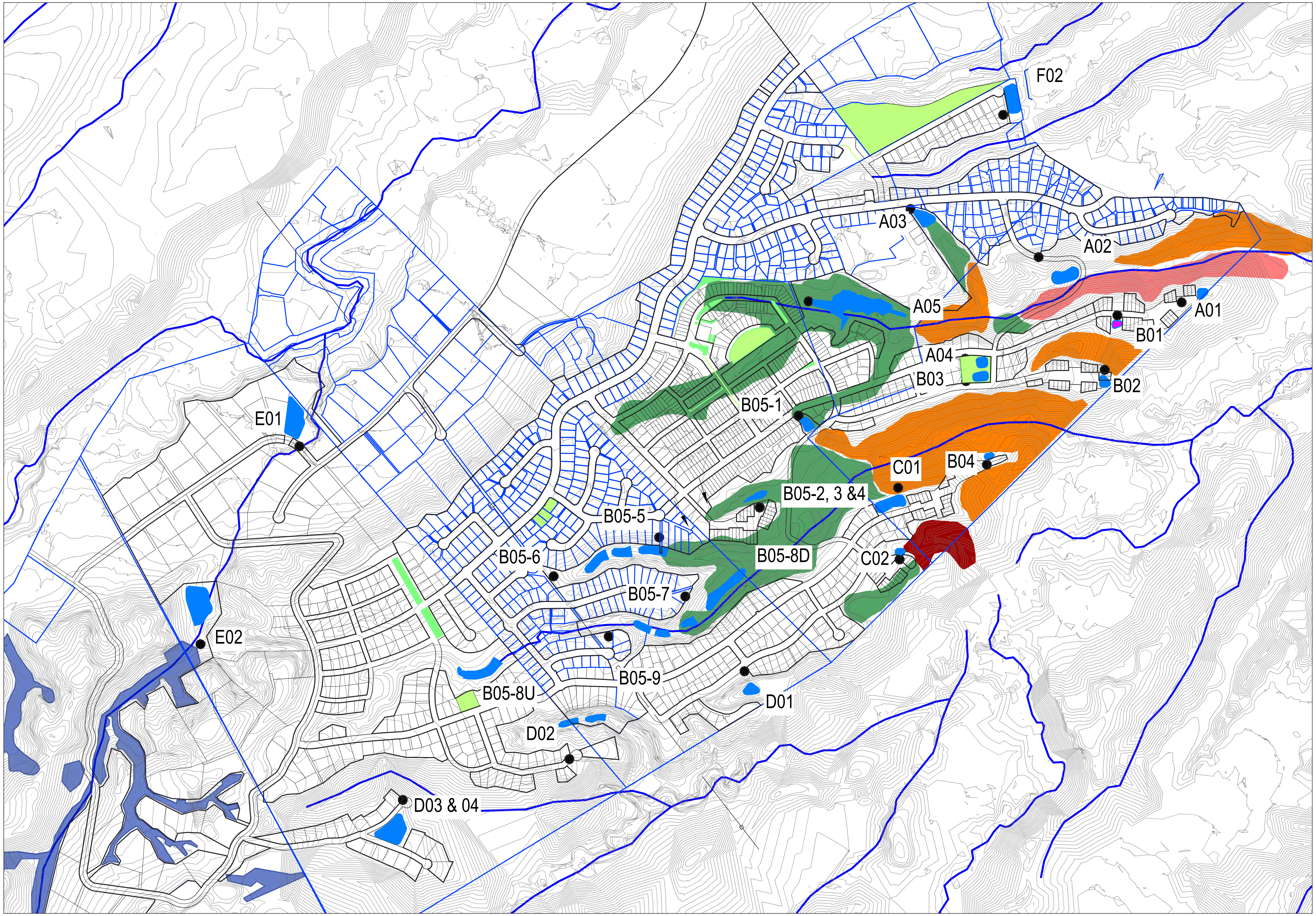
Table 5.2 Stormwater wetland footprints for flood control and water quality treatment on plateaus (alternative option)

Discharge Location[1]	Total Area (m^2)	Permanent Pool Volume (m^3)	100-Year ARI Storage Footprint with Permanent Pool (m^2)[2]
A02	8.5	920	2,770
B05-9	12.0	1,310	3,720
C01	9.8	1,130	3,290
E01	19.6	1,140	3,300
E02	61.8	930	2,790
F01	18.8	2,170	5,730
F02	8.0	890	2,690

Note: permanent pool and flood storage footprint and volume values are rounded up to the nearest 10.
1. Only discharge points with minimum 5 ha catchment area were assessed.
2. Permanent pool volume is equivalent to Water Quality Volume (WQV) summarized in Table 4.5.

5. Stream stabilisation

Based on the erosion assessment and site visits carried out, the proposed stormwater detention volumes required for flood risk mitigation are largely adequate to mitigate erosion risk in the study area streams when considering pre-development and post-development flows. However, the Aokautere Church Stream and Moonshine Valley Reserve Stream will remain highly sensitive to erosion in the future regardless of upstream development. Therefore, stream stabilisation within the gullies may be prudent to offset any perceived impacts from the development.



LEGEND

WATER COURSE

CONTOURS

VEGETATION VALUE - VERY HIGH

VEGETATION VALUE - HIGH

VEGETATION VALUE - MEDIUM

VEGETATION VALUE - LOW

DISCHARGE LOCATION

B03

STORAGE LOCATION

PROPOSED DEVELOPMENT

EXISTING DEVELOPMENT

WETLAND EXTENTS-WATERS LAND

FOREST EXTENT-WATERS LAND

Rev	Description	Checked	Approved	Date
C	LAYOUT CHANGE	DA*		22/04/21

Author C. O'DONNELL Drafting Check

Designer D. ARSENEAU Design Check

0 100 200 300m

SCALE 1:10,000 AT ORIGINAL SIZE

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Project No. 12554906

Client PALMERSTON NORTH CITY COUNCIL

Project STORMWATER CONCEPT DESIGN OVERVIEW

Status PRELIMINARY - SHARED

Status Code S4

Drawing Title FIGURE 5.2

Size A3

Drawing No. 12554906-FIGURE 5.2

Rev C

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5.4 Review of Draft Plan Change Provisions

The Aokautere Plan Change (Plan Change G) will involve updates/revisions to existing District Plan provisions across a number of current chapters/sections. Those sections with most relevance to stormwater management include: Section 7 Subdivision, Section 7A Greenfield Residential, Section 10 Residential, and Section 15 Recreation. Draft versions of proposed District Plan revisions were reviewed in March 2022 to assess whether the updates would achieve the desired stormwater management outcomes for Aokautere.

In general, the provisions establish a restricted discretionary activity status for most stormwater-relevant items related to subdivision and residential development, including the management of effects on the existing gully network, which is considered appropriate. These are reflected both in the high-level objectives/policies as well as more detailed rules for specific activities. Existing provisions are expanded/strengthened to require a wider focus for stormwater management assessments beyond the immediate site boundaries (i.e., flooding and erosion in downstream watercourses) for a more robust approach to management of effects. New provisions require a Stormwater Management Plan (as part of a Comprehensive Development Plan) prepared by a suitably qualified stormwater design consultant to demonstrate the management of flooding, erosion and water quality effects from the development. New provisions are also included to establish the 5 metre no-build setback along the backs of residential lots to protect the gully slopes, as recommended in this report, and provide for the long-protection of the gully systems through vesting of those areas to Council for conservation and amenity purposes. Vesting of any other community-scale stormwater management measures should also be included in the District Plan to ensure adequate maintenance and operation of the facilities in the future.

5.5 Horizons Regional Council One Plan Considerations

Horizons Regional Council (Horizons) provided feedback on a draft circulation of the Plan Change in March 2021 and indicated several aspects of the proposed residential development related to stormwater that may require consenting or further inspection under the One Plan provisions. A summary of the feedback received and how the Aokautere stormwater management strategy addresses the feedback is included below.

1. Flooding – Horizons notes the existing waterways that traverse the site, primarily in a north-westerly direction through the gully systems, with potential flooding hazard to any new development. The proposed structure plan does not place any residential development within the gullies, effectively avoiding this hazard. Proposed development along Valley Views appears to be outside of the floodable area from the Turitea Stream, and therefore incurs little flood risk for future housing.
2. Stormwater Management – Horizons notes several broad concerns related to the management of stormwater, including water quality issues, the potential for exacerbation of existing erosion issues, and issues related to the discharge of stormwater across sloping land. The Aokautere stormwater management strategy explicitly requires water quality treatment of runoff prior to discharge, and has presented an in-depth erosion assessment to identify potential future effects. As well, the strategy includes provision for a slope setback to manage runoff that would otherwise flow directly down the steep gully slopes.
3. Natural Hazards – The structure plan avoids the placement of residential development in areas that could potentially be impacted by flooding hazards.
4. Climate Change – Stormwater quantity and quality controls have been assessed in consideration of future climate change, using best available guidance from NIWA (via the HIRDS v4 system).
5. National Policy Statement on Freshwater Management – The stormwater strategy provides for water quality treatment and attenuation of runoff prior to discharge, and will directly lead to the vesting and protection of the natural stream systems within the gullies. Stream restoration/stabilisation work is also recommended as part of the strategy to mitigate the ongoing existing erosion issues in these areas.

In general, the Aokautere stormwater management strategy is considered to address Horizons One Plan requirements in a comprehensive manner.

6. Conclusions and Recommendations

GHD completed a stormwater management analysis for the Aokautere Structure Plan area, including assessment of flood risk, erosion risk, and water quality requirements for the proposed development with an assumed level of post-development impervious cover of 70%. Based on this analysis, a recommended stormwater management strategy was developed including a conceptual design of stormwater detention and water quality treatment facilities; note that if post-development impervious cover is higher than 70%, additional stormwater management areas may be required. This report will contribute to and form part of the Structure Plan for implementation during detailed design, resource consenting and construction of the proposed developments.

Ultimately, the following stormwater design criteria are recommended to be adopted:

- Control of runoff peak flows to pre-development levels for the 2-year, 5-year, 10-year, 20-year, 50-year and 100-year ARI flows, to control flood risk.
- Further control of peak flows as needed to match post-development erosion threshold exceedance cumulative effective work index in the Aokautere Church Stream, Moonshine Valley Reserve Stream, and Tutukiwi Reserve Stream.
- Treatment of the 90th percentile rainfall volume (i.e., 15 mm) from impervious developed areas through a stormwater treatment device or multi-device system.

The key constraints identified for the area include:

- Erosion
- Slope stability
- Perceived stream degradation
- Existing vegetation and wetland areas

The stormwater management concept design recommended to be included in the Aokautere Structure Plan includes the following components:

- Stormwater detention facilities to mitigate flood and erosion risk.
- Roadside bioretention facilities (rain gardens) to provide water quality treatment, with underdrains connecting to the stormwater reticulation.
- An indicative top-of-slope setback of 5 m from the 4:1 grade line to safely convey stormwater runoff to designated discharge points and protect gully slopes from erosion.
- Stormwater reticulation to accommodate the 10-year ARI climate change flow, with major overland flow network to accommodate the 100-year ARI climate change flow, as per PNCC ESLD.
- Discharge of stormwater runoff downstream of detention facilities into the receiving gullies through a pipe installed down the gully slopes, sized for the 100-year ARI climate change flow. No overland discharge down the gully slopes should be permitted.
- Consideration of stream stabilisation in the Aokautere Church Stream and Moonshine Valley Reserve Stream to mitigate perceived impacts from the development and enhance both aquatic habitat and community amenity.

Indicative footprints have been provided for the various detention and bioretention facilities for the structure plan area as a feasibility test; these locations and footprints are to be confirmed through subdivision design stage to assess site-specific requirements and constructability. Prior to the approval of any engineering plans for the proposed developments, the developer should be required to provide a stormwater management report and design demonstrating compliance with these design criteria and concepts.

Generally, the draft District Plan provisions developed for this Plan Change appear to be suitable to achieve the desired stormwater outcomes.

7. References

Auckland Council, Stormwater Management Devices in the Auckland Region, Guideline Document 2017/001 Version 1, 2017.

Auckland Regional Council, Guidelines for stormwater runoff modelling in the Auckland Region, Technical Publication No. 108, April 1999.

Chow, V.T., Open Channel Hydraulics, McGraw-Hill, 1959.

Environmental Water Resources Group Ltd., Technical Guidelines for Flood Hazard Mapping, 2017.

Komar, P.D. (1987). Selective gravel entrainment and the empirical evaluation of flow competence. Sedimentology, 34: 1165-1176.

MacRae, C.R. and Rowney, A.C. 1992. The Role of Moderate Flow Events and Bank Structure in the Determination of Channel Response to Urbanization. 45th Annual Conference of the Canadian Water Resources Assoc., Kingston, Ontario.

Miller, M.C., McCave, I.N., and Komar, P.D. (1977) Threshold of sediment motion under unidirectional currents. Sedimentology 24: 5 7 527.

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US Environmental Protection Agency, Storm Water Management Model User’s Manual Version 5.1, 2015.

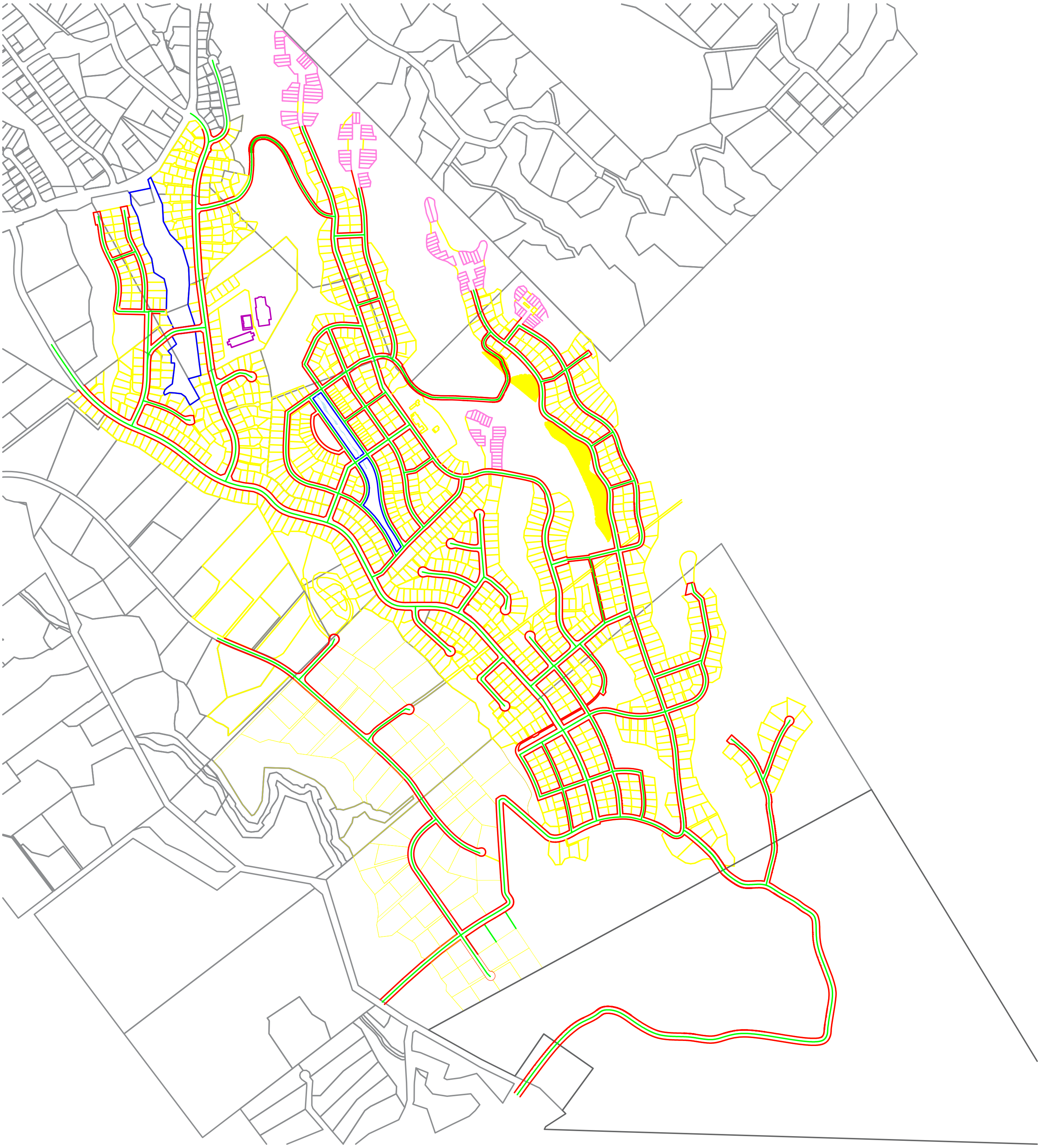
Appendices

Appendix A

Aokautere Structure Plan Development Layout (McIndoe Urban, October 2019, July 2021 and March 2022)

Draft Aokautere Structure Plan Development Layout

Received from McIndoe Urban
16 October 2019

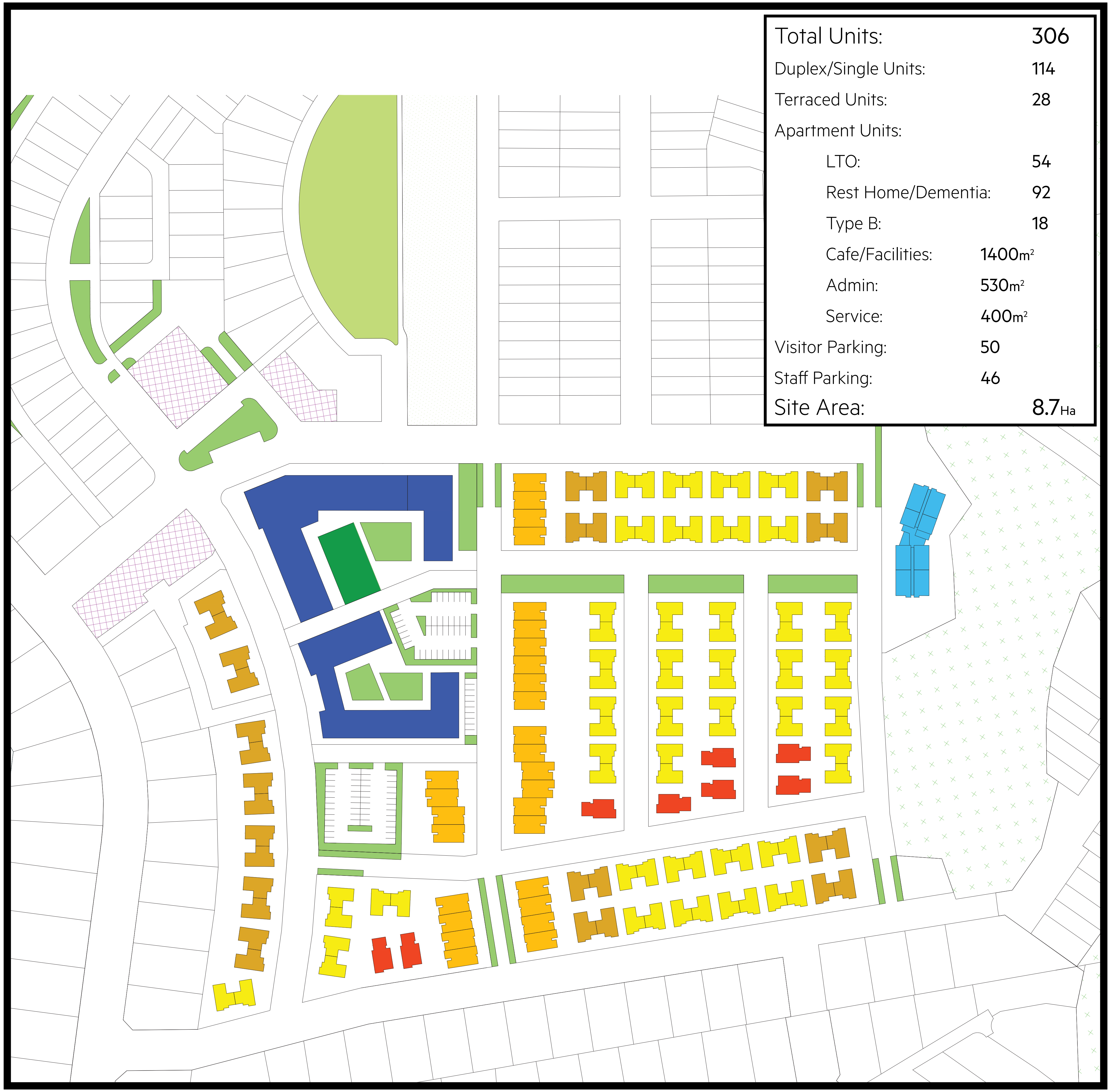


Aokautere Structure Plan: Final Structure Plan

McIndoeURBAN Ltd\Projects - Documents\08 Manawatu-Whanganui (2000-2499)\2038c_Aokautere Structure Plan

Aokautere - Structure Plan

McIndoe Urban
11.05.2022



Total Units:	306
Duplex/Single Units:	114
Terraced Units:	28
Apartment Units:	
LTO:	54
Rest Home/Dementia:	92
Type B:	18
Cafe/Facilities:	1400m²
Admin:	530m²
Service:	400m²
Visitor Parking:	50
Staff Parking:	46
Site Area:	8.7Ha

OPTION 2 (revised 2):

Metlifecare Gulf Rise Retirement Homes / Warren and Mahoney

DRAFT

project name
Aokautere Retirement Village Options

project no.
2038c

client
PNCC

scale
1:2000@A3

N

date
10.05.2022

McIndoe Urban
t +64 4 385 9006
e admin@mcindoeurban.co.nz
w www.mcindoeurban.co.nz
po box 11908
Wellington 6142
New Zealand

Appendix B

Summary of Flood Assessment Results and Criteria

Appendix B - Flood assessment results and criteria

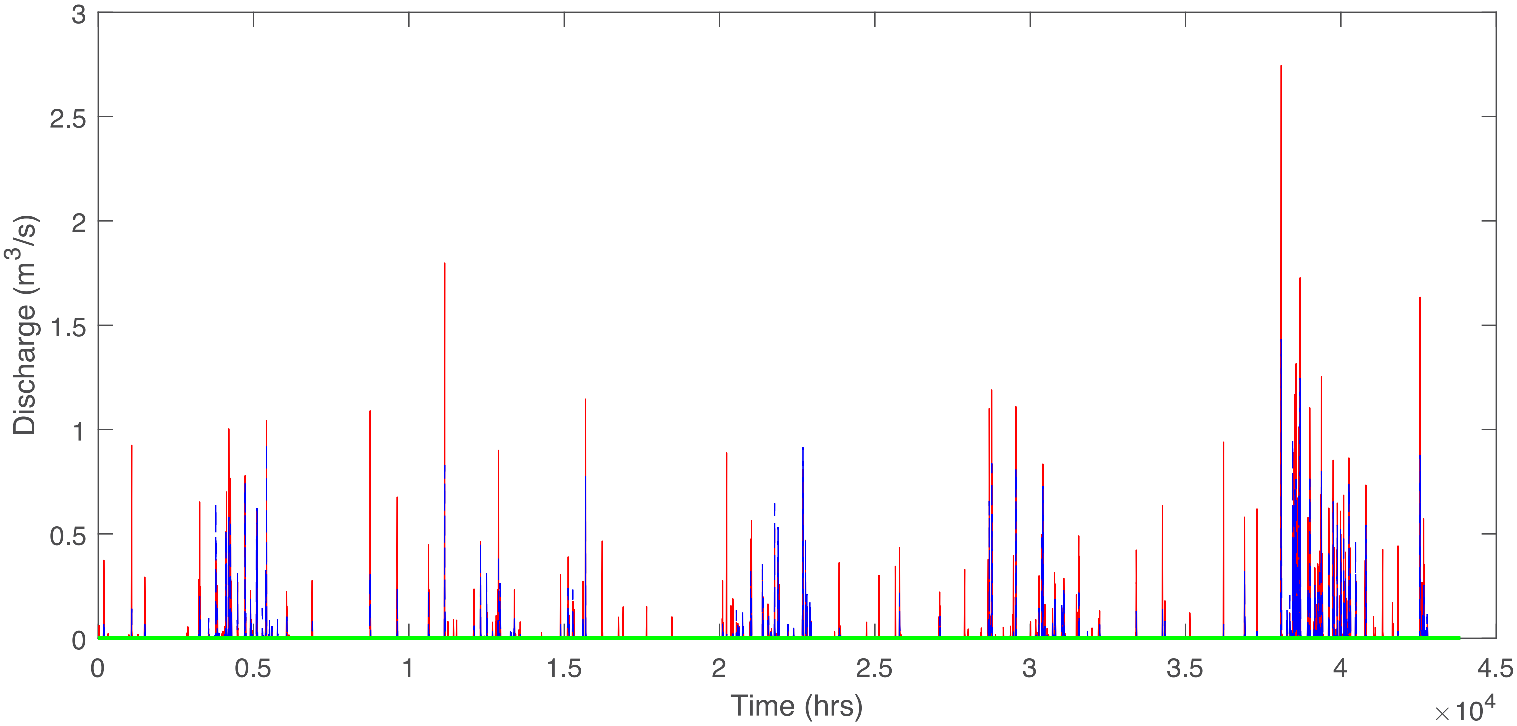
Discharge Point	2-year ARI			5-year ARI			10-year ARI			20-year ARI			50-year ARI			100-year ARI		
	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required	Pre-Development	Post-Development	Storage Required
	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)	(m^3/s)	(m^3/s)	(m^3)
A01	0.02	0.07	117	0.03	0.11	195	0.05	0.15	208	0.07	0.18	230	0.09	0.23	288	0.11	0.27	323
A02	0.09	0.56	1,188	0.18	0.81	1,498	0.27	1.00	1,652	0.36	1.20	1,849	0.49	1.49	2,129	0.61	1.71	2,288
A03	0.05	0.32	656	0.10	0.47	819	0.15	0.58	896	0.20	0.70	1,003	0.27	0.86	1,159	0.33	0.99	1,267
A04	0.02	0.15	297	0.04	0.22	379	0.06	0.28	417	0.09	0.33	434	0.12	0.40	506	0.15	0.46	546
A05	0.33	2.00	4,527	0.70	3.02	5,563	1.03	3.78	6,173	1.38	4.57	6,900	1.88	5.65	7,948	2.33	6.45	8,604
B01	0.02	0.19	391	0.04	0.28	506	0.07	0.30	513	0.10	0.32	549	0.14	0.37	630	0.17	0.42	696
B02	0.02	0.12	253	0.04	0.18	340	0.06	0.23	383	0.09	0.29	403	0.12	0.36	480	0.15	0.42	521
B03	0.02	0.21	505	0.05	0.30	580	0.08	0.37	613	0.11	0.44	676	0.16	0.55	741	0.20	0.63	793
B04	0.01	0.04	64	0.02	0.06	88	0.02	0.08	136	0.03	0.10	146	0.05	0.13	147	0.06	0.15	168
B05-1	0.02	0.18	433	0.04	0.26	541	0.07	0.32	547	0.10	0.39	584	0.14	0.48	654	0.17	0.55	715
B05-2-3-4	0.03	0.20	542	0.08	0.32	608	0.12	0.42	695	0.17	0.52	756	0.24	0.66	871	0.30	0.77	947
B05-5-6-7	0.19	1.69	3,991	0.44	2.46	4,723	0.68	3.06	5,104	0.96	3.69	5,543	1.36	4.51	6,220	1.69	5.02	6,732
B05-8	0.11	1.02	2,351	0.26	1.50	2,772	0.41	1.86	2,966	0.57	2.23	3,256	0.81	2.68	3,649	1.00	2.99	3,986
B05-9	0.09	0.69	1,937	0.22	1.04	2,240	0.34	1.30	2,424	0.48	1.58	2,606	0.68	1.97	2,918	0.84	2.18	3,175
C01	0.06	0.73	1,914	0.16	1.05	2,126	0.27	1.30	2,202	0.39	1.54	2,351	0.59	1.77	2,519	0.76	1.95	2,639
C02	0.00	0.04	91	0.01	0.06	125	0.02	0.08	124	0.03	0.10	132	0.05	0.12	131	0.06	0.15	149
D01	0.01	0.13	354	0.01	0.18	576	0.02	0.22	598	0.03	0.27	651	0.05	0.33	699	0.06	0.38	783
D02	0.02	0.27	1,055	0.05	0.44	1,300	0.08	0.57	1,481	0.11	0.71	1,702	0.17	0.91	1,949	0.22	1.07	2,141
D03	0.01	0.18	634	0.03	0.27	715	0.04	0.35	879	0.06	0.42	963	0.09	0.52	1,113	0.11	0.61	1,252
D04	0.01	0.17	423	0.02	0.24	558	0.03	0.30	646	0.04	0.35	736	0.06	0.43	851	0.08	0.50	912
E01	0.06	0.65	2,179	0.15	1.14	2,837	0.24	1.51	3,345	0.37	1.89	3,790	0.58	2.47	4,392	0.78	2.93	4,725
E02	0.19	0.36	2,783	0.47	0.84	3,747	0.76	1.36	4,477	1.17	1.97	4,920	1.84	2.94	5,826	2.45	3.73	6,362
F01	0.06	1.12	4,069	0.14	1.62	4,955	0.23	2.00	5,453	0.36	2.37	5,721	0.55	2.87	6,348	0.73	3.25	6,599
F02	0.02	0.52	1,648	0.06	0.76	1,899	0.10	0.94	2,111	0.15	1.12	2,330	0.24	1.41	2,572	0.31	1.53	2,728

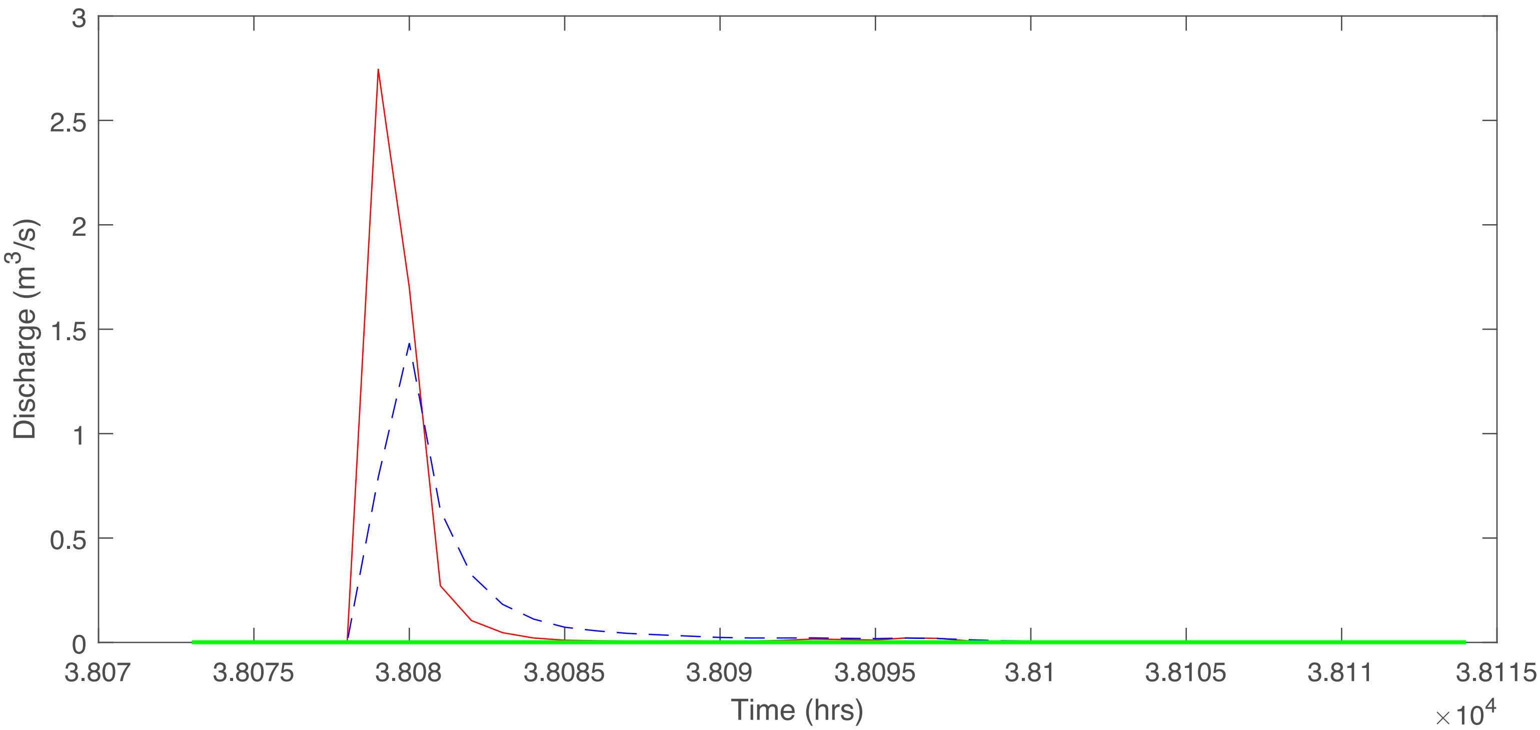
Appendix C

Erosion Threshold Exceedance Analysis

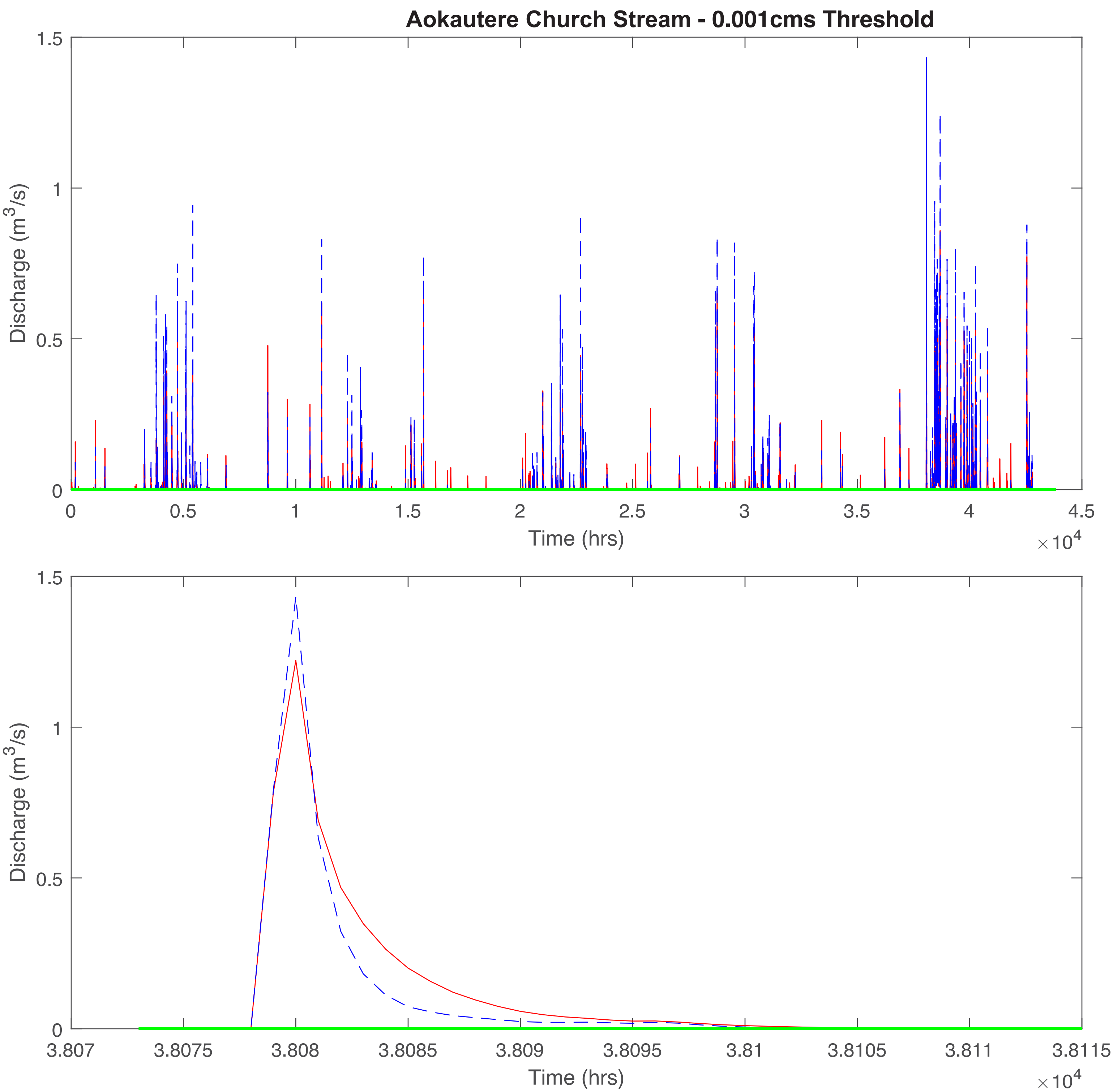
Results Graphs

Aokautere Church Stream - 0.001cms Threshold

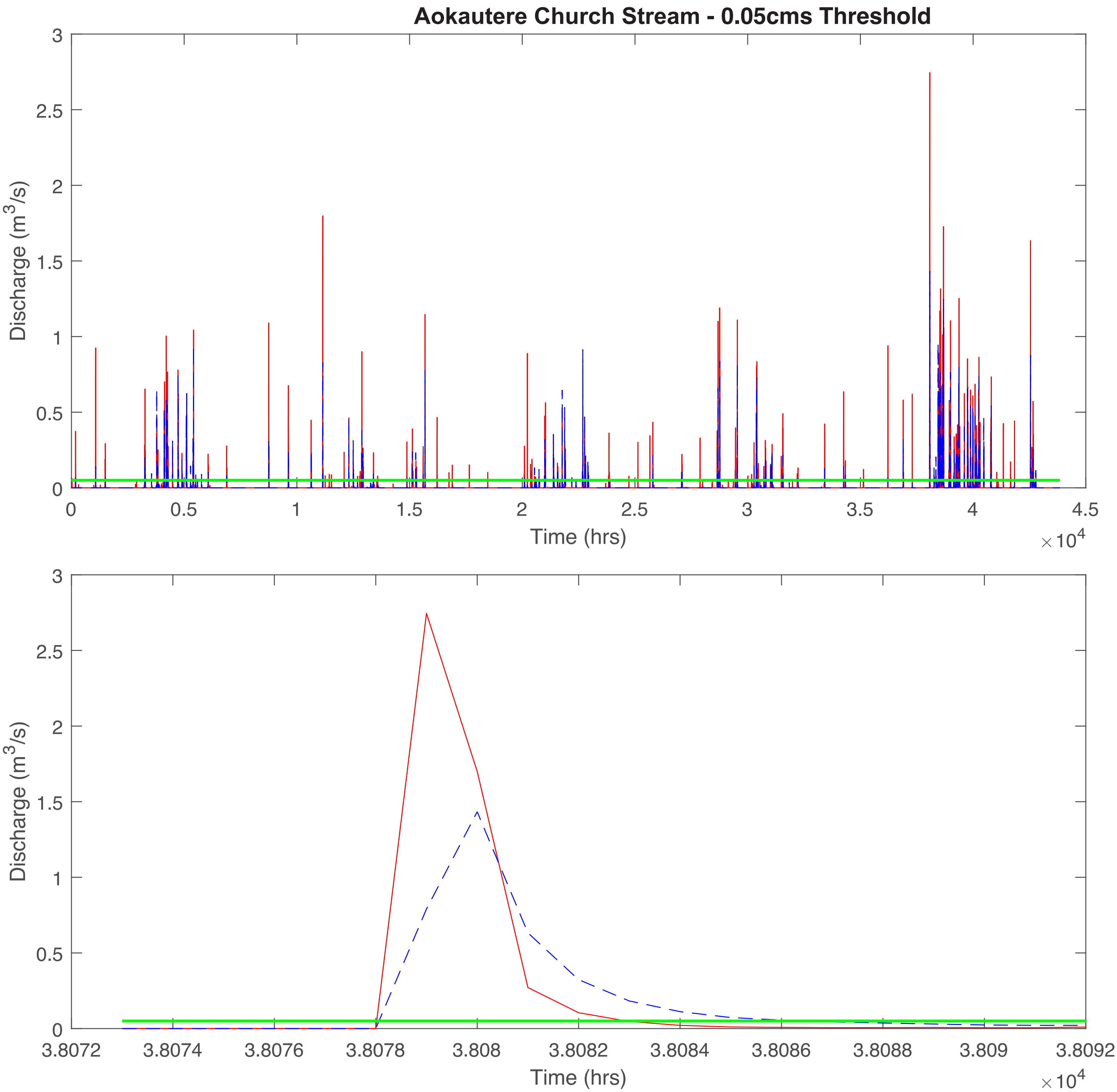




Pre-Development (Blue)	**Post-Development (Red)**
Total Exceedance Time = 5131 hrs	Total Exceedance Time = 3924 hrs
% Exceedance Time = 11.7	% Exceedance Time = 8.9
Number of Exceedance Events = 169	Number of Exceedance Events = 263
Cumul. Effective Discharge(m^3) = 8.908E+05	Cumul. Effective Discharge(m^3) = 9.188E+05
Cumul. Effective Work Index(J/m^2) = 1.737E+08	Cumul. Effective Work Index(J/m^2) = 1.761E+08



Pre-Development (Blue)	**Post-Development - with SWM Control (Red)**
Total Exceedance Time = 5131 hrs	Total Exceedance Time = 5557 hrs
% Exceedance Time = 11.7	% Exceedance Time = 12.7
Number of Exceedance Events = 169	Number of Exceedance Events = 187
Cumul. Effective Discharge(m^3) = 8.908E+05	Cumul. Effective Discharge(m^3) = 8.907E+05 Cumul.
Cumul. Effective Work Index(J/m^2) = 1.737E+08	Effective Work Index(J/m^2) = 1.755E+08



Pre-Development (Blue)

Total Exceedance Time = 1207 hrs

% Exceedance Time = 2.8

Number of Exceedance Events = 188

Cumul. Effective Discharge(m^3) = 5.310E+05

Cumul. Effective Work Index(J/m^2) = 8.325E+07

Post-Development (Red)

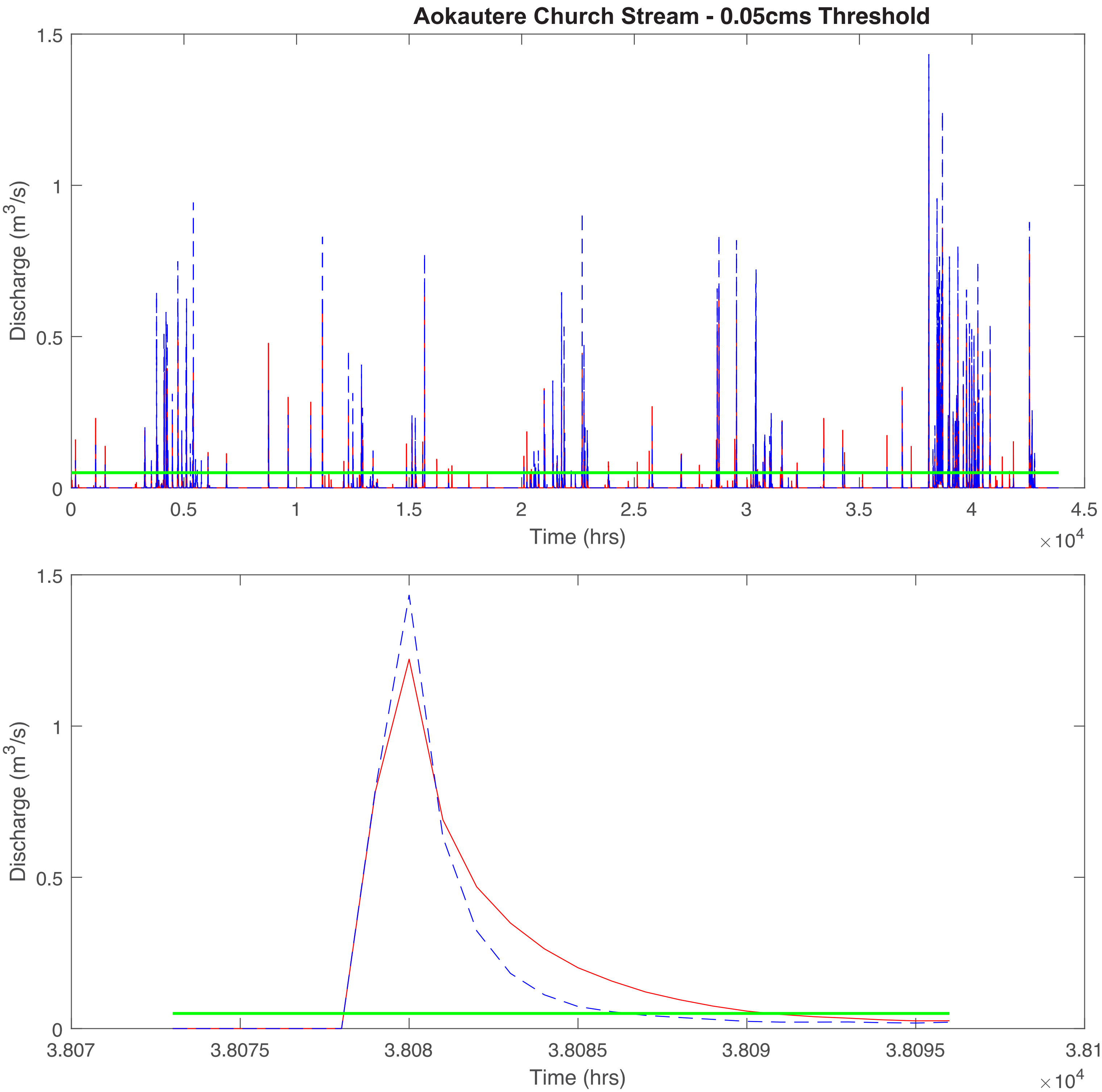
Total Exceedance Time = 1112 hrs

% Exceedance Time = 2.5

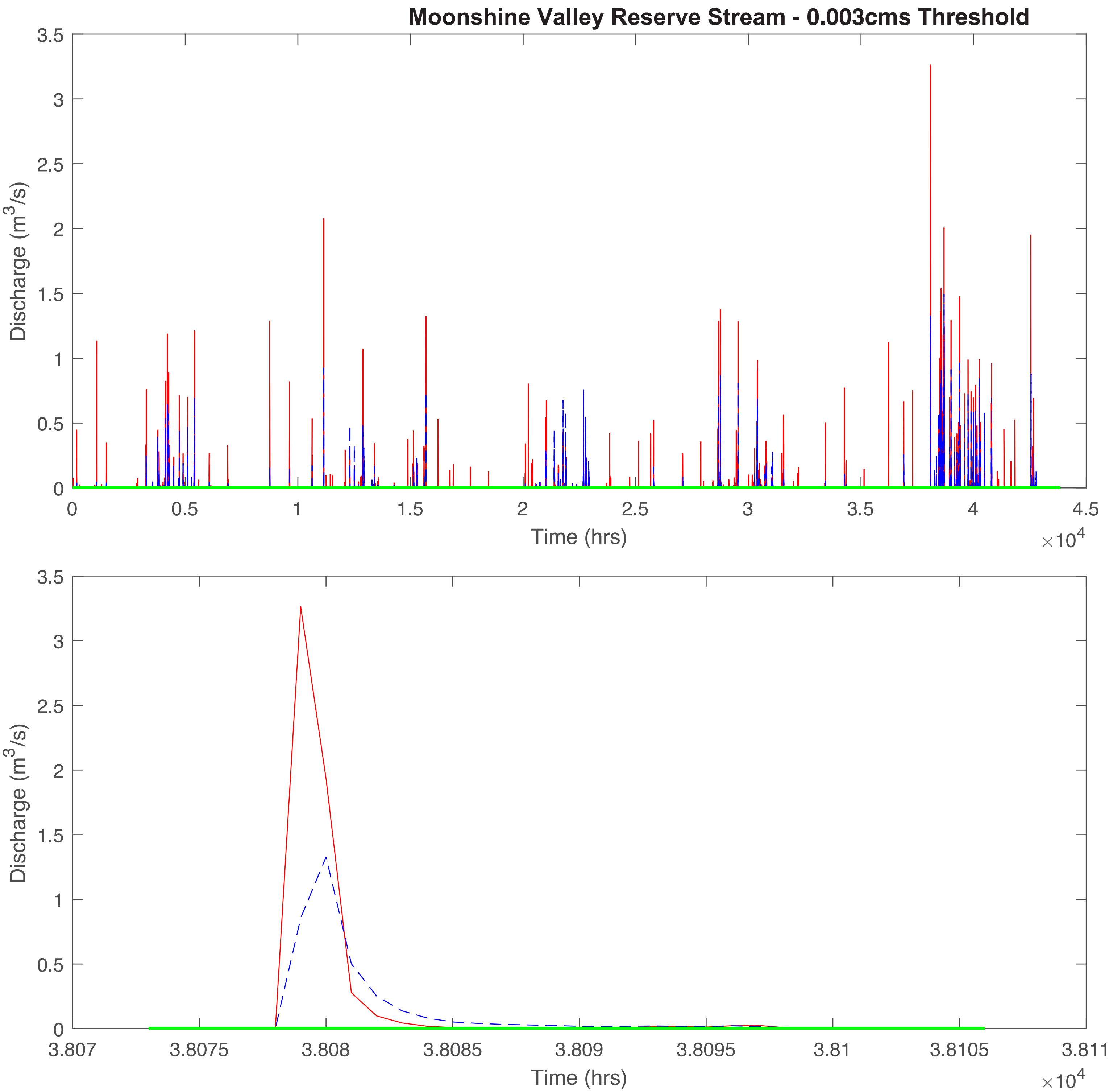
Number of Exceedance Events = 292

Cumul. Effective Discharge(m^3) = 6.213E+05

Cumul. Effective Work Index(J/m^2) = 9.701E+07



Pre-Development (Blue)	**Post-Development - with SWM Control (Red)**
Total Exceedance Time = 1207 hrs	Total Exceedance Time = 1382 hrs
% Exceedance Time = 2.8	% Exceedance Time = 3.2
Number of Exceedance Events = 188	Number of Exceedance Events = 179
Cumul. Effective Discharge(m^3) = 5.310E+05	Cumul. Effective Discharge(m^3) = 4.661E+05
Cumul. Effective Work Index(J/m^2) = 8.325E+07	Cumul. Effective Work Index(J/m^2) = 7.239E+07



Pre-Development (Blue)

Total Exceedance Time = 3336 hrs

% Exceedance Time = 7.6

Number of Exceedance Events = 184

Cumul. Effective Discharge(m^3) = 7.387E+05

Cumul. Effective Work Index(J/m^2) = 8.436E+07

Post-Development (Red)

Total Exceedance Time = 3140 hrs

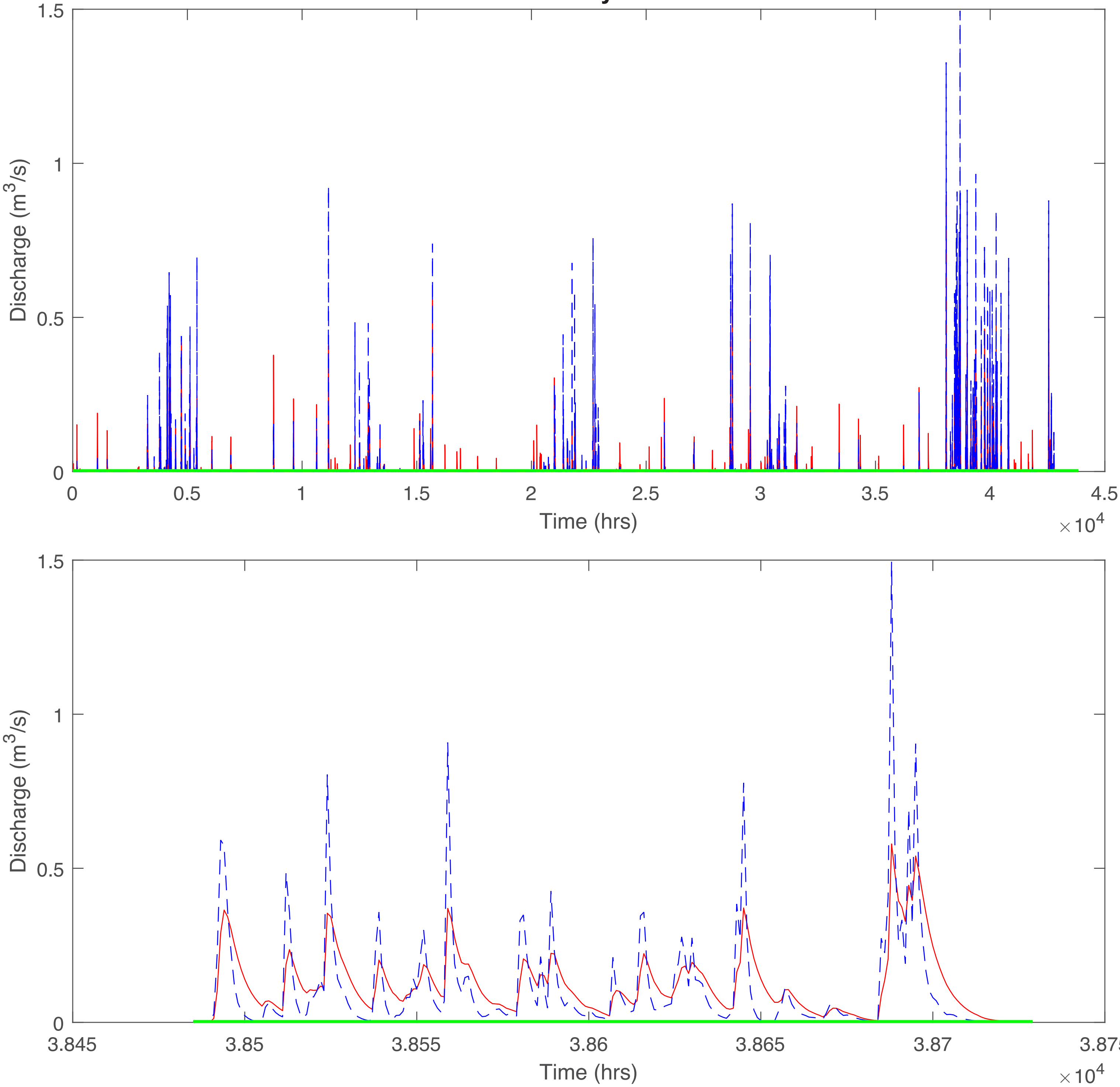
% Exceedance Time = 7.2

Number of Exceedance Events = 287

Cumul. Effective Discharge(m^3) = 9.824E+05

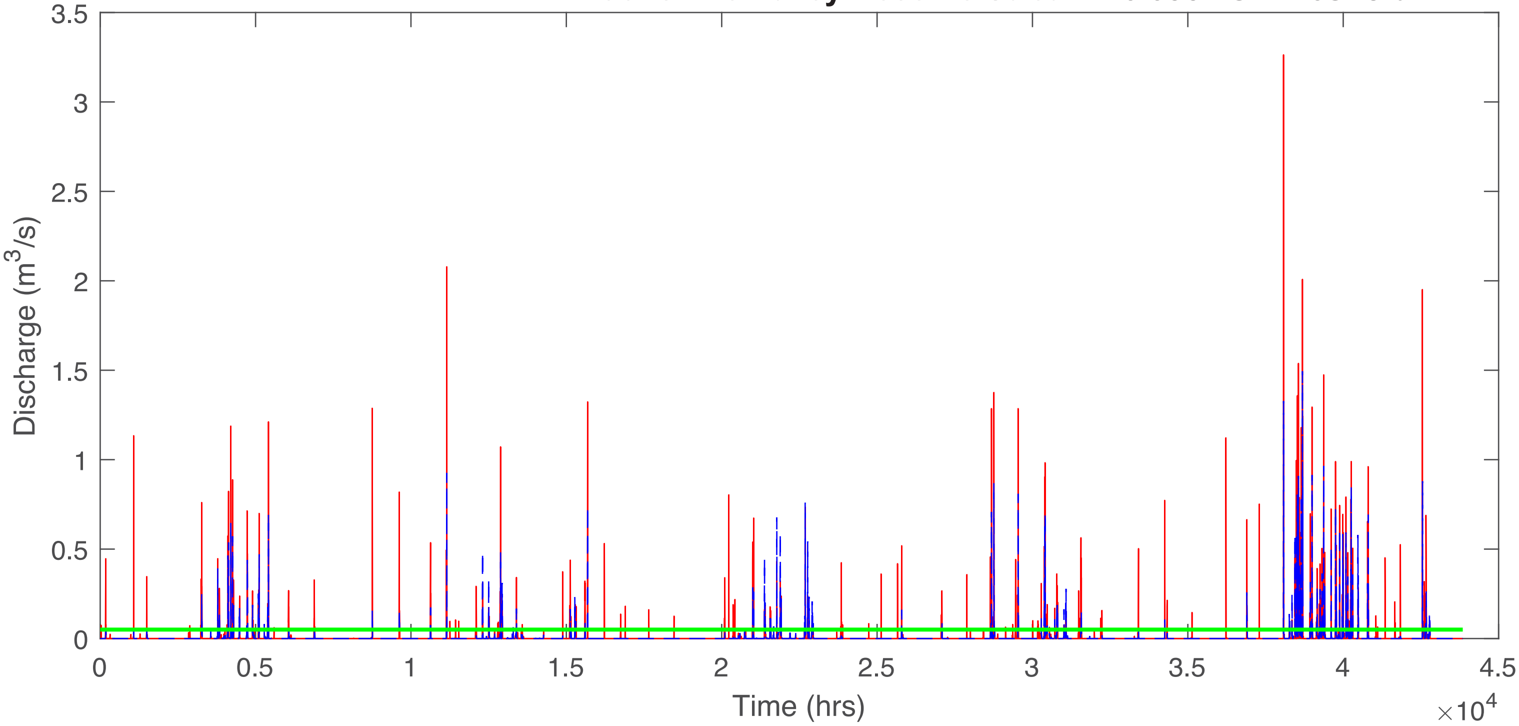
Cumul. Effective Work Index(J/m^2) = 1.099E+08

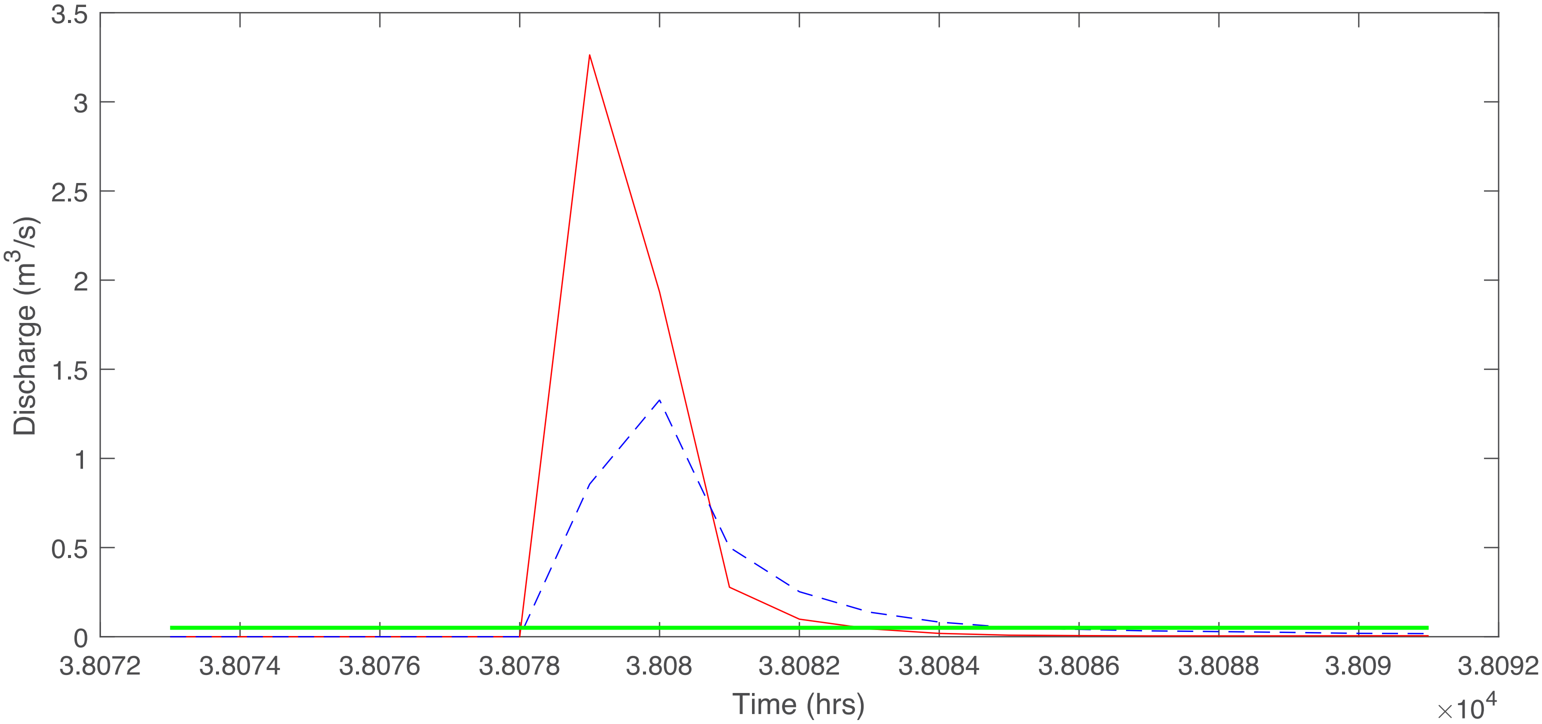
Moonshine Valley Reserve Stream - 0.003cms Threshold



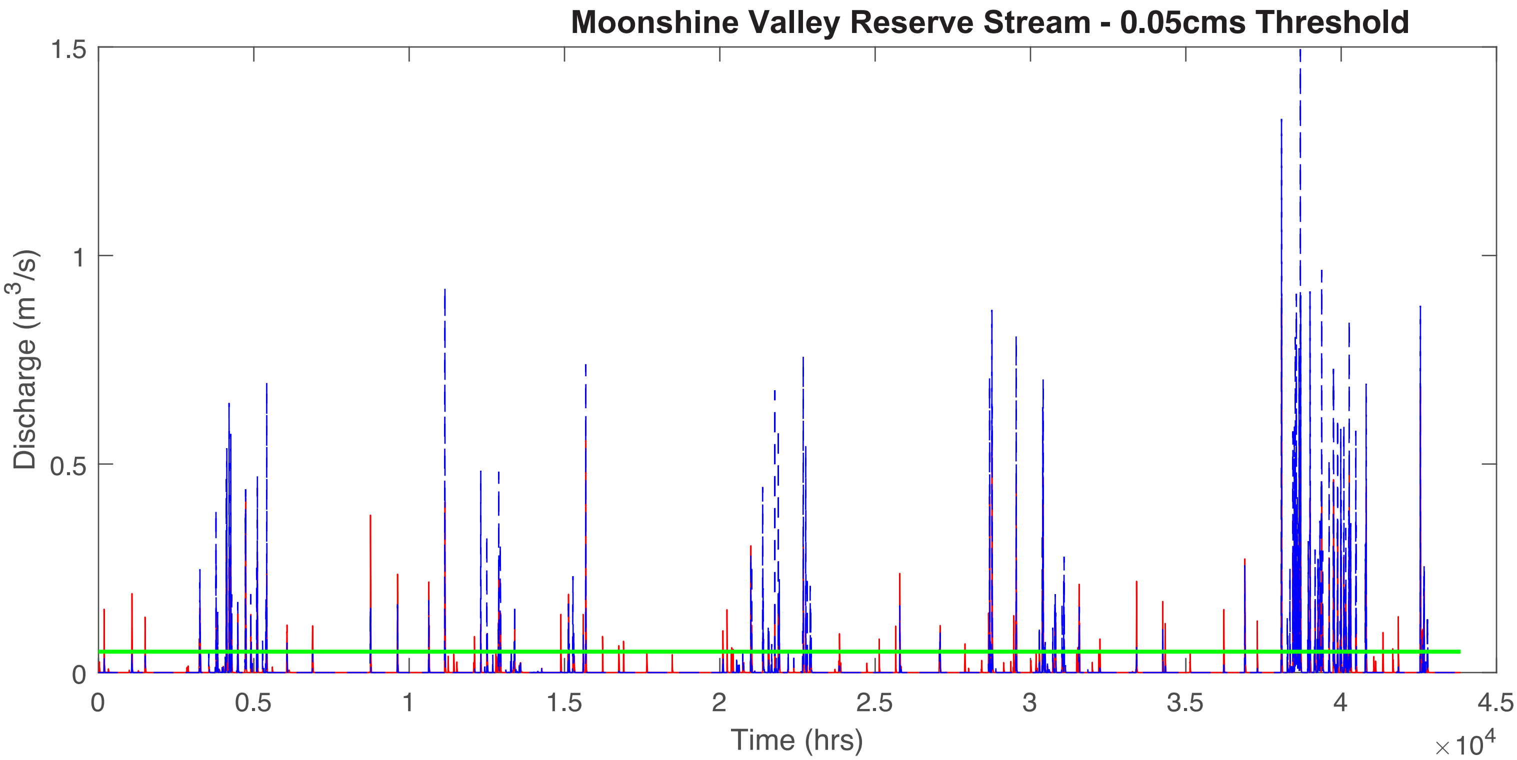
Pre-Development (Blue)	**Post-Development - with SWM Control (Red)**
Total Exceedance Time = 3336 hrs	Total Exceedance Time = 5136 hrs
% Exceedance Time = 7.6	% Exceedance Time = 11.7
Number of Exceedance Events = 184	Number of Exceedance Events = 181
Cumul. Effective Discharge(m^3) = 7.387E+05	Cumul. Effective Discharge(m^3) = 9.277E+05
Cumul. Effective Work Index(J/m^2) = 8.436E+07	Cumul. Effective Work Index(J/m^2) = 1.076E+08

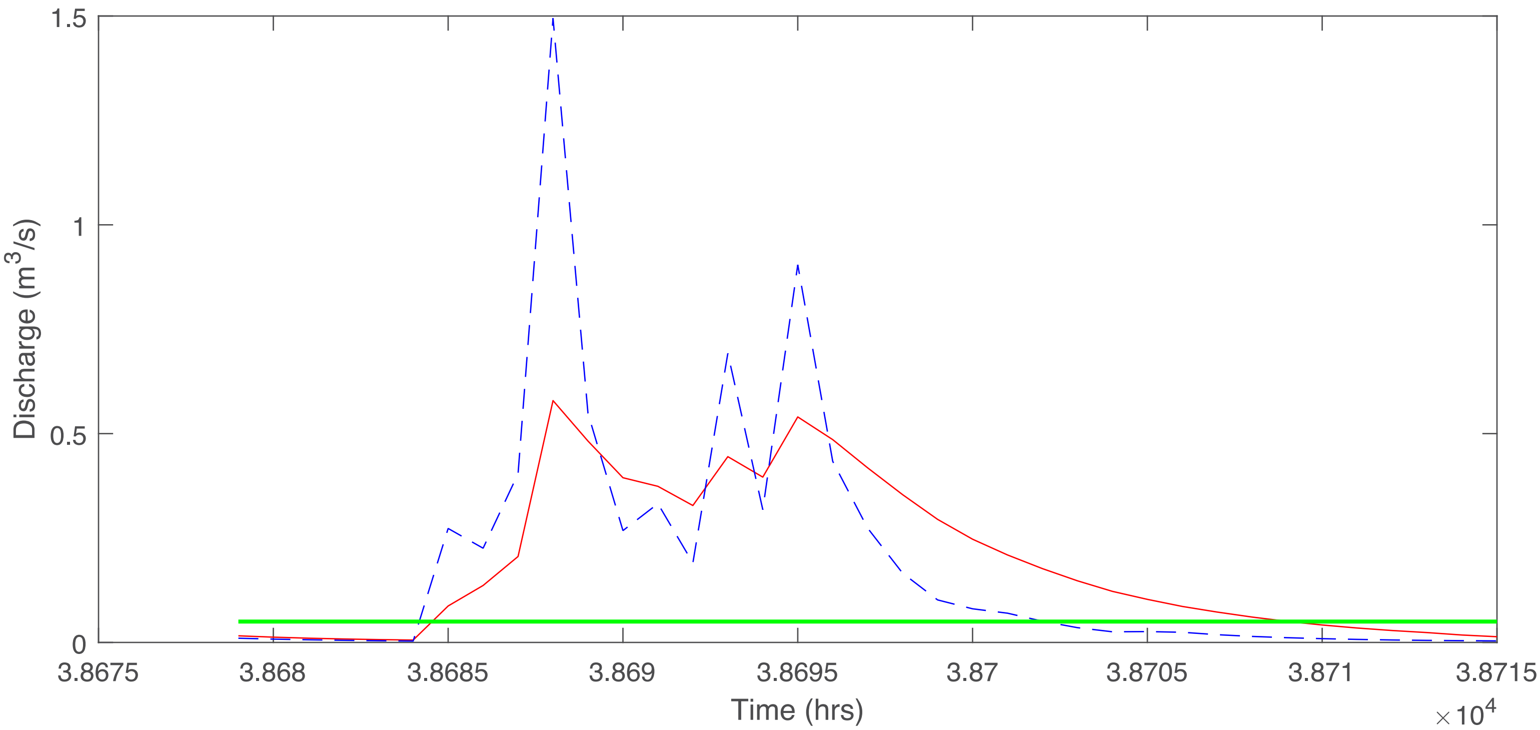
Moonshine Valley Reserve Stream - 0.05cms Threshold



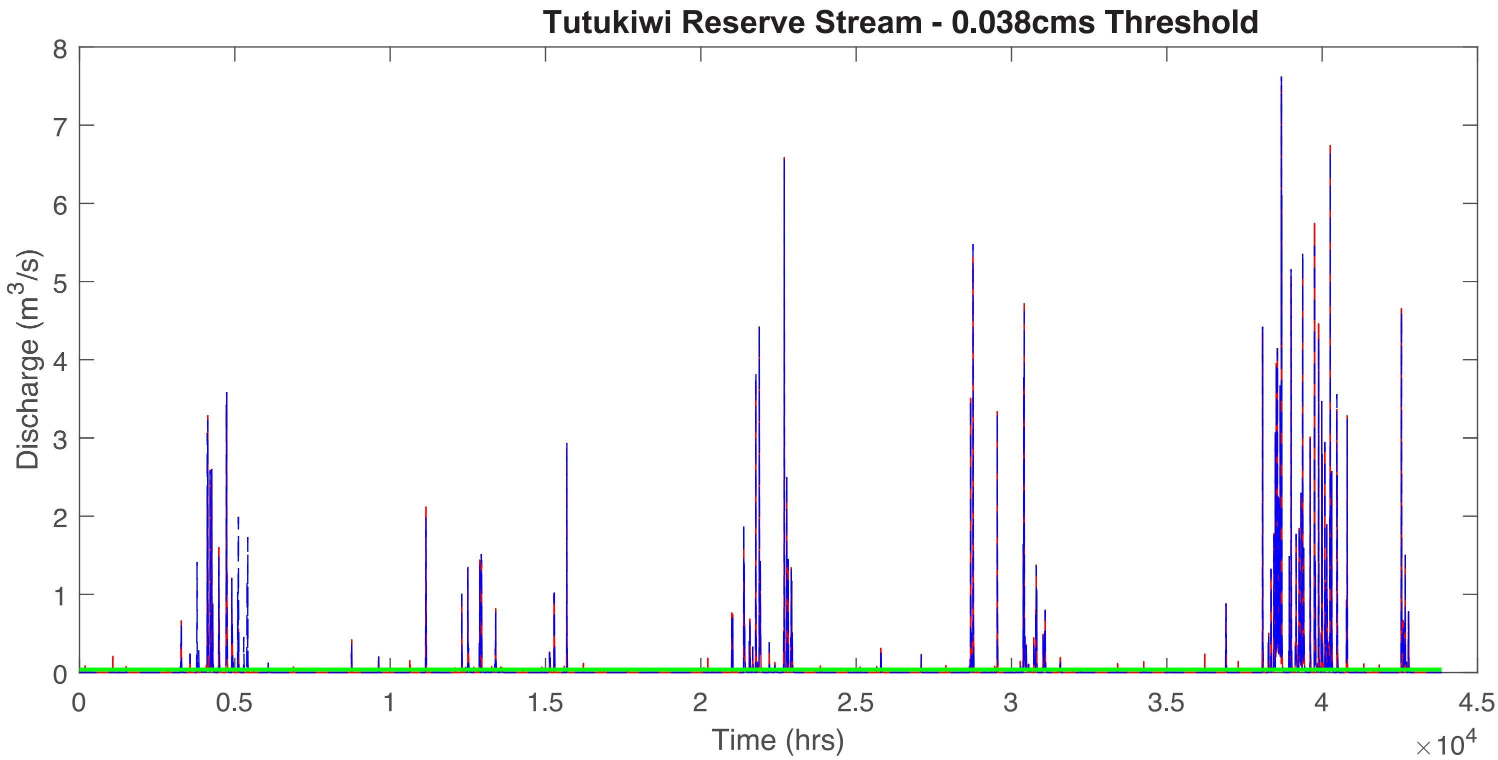


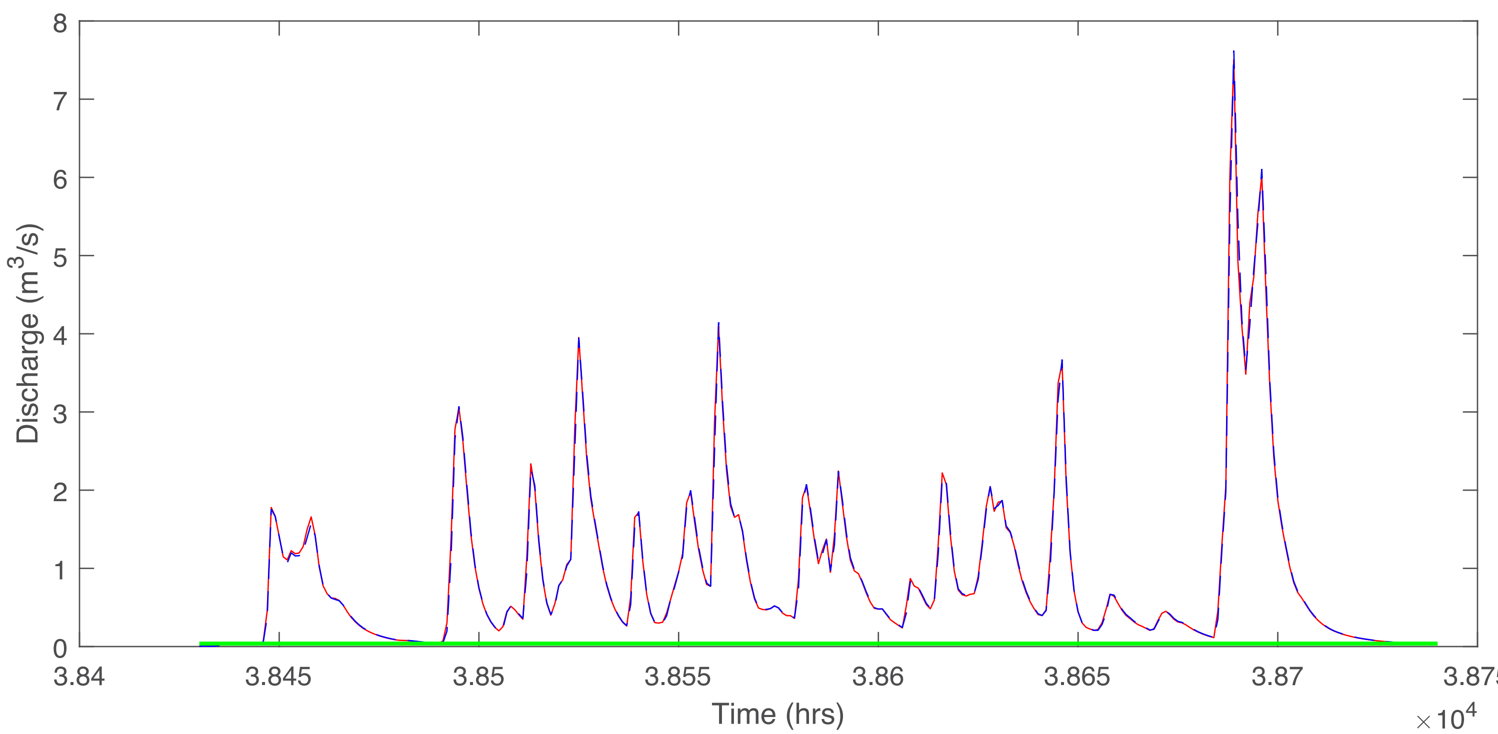
Pre-Development (Blue)	**Post-Development (Red)**
Total Exceedance Time = 997 hrs	Total Exceedance Time = 1131 hrs
% Exceedance Time = 2.3	% Exceedance Time = 2.6
Number of Exceedance Events = 177	Number of Exceedance Events = 286
Cumul. Effective Discharge(m^3) = 4.683E+05	Cumul. Effective Discharge(m^3) = 7.109E+05
Cumul. Effective Work Index(J/m^2) = 4.440E+07	Cumul. Effective Work Index(J/m^2) = 6.694E+07





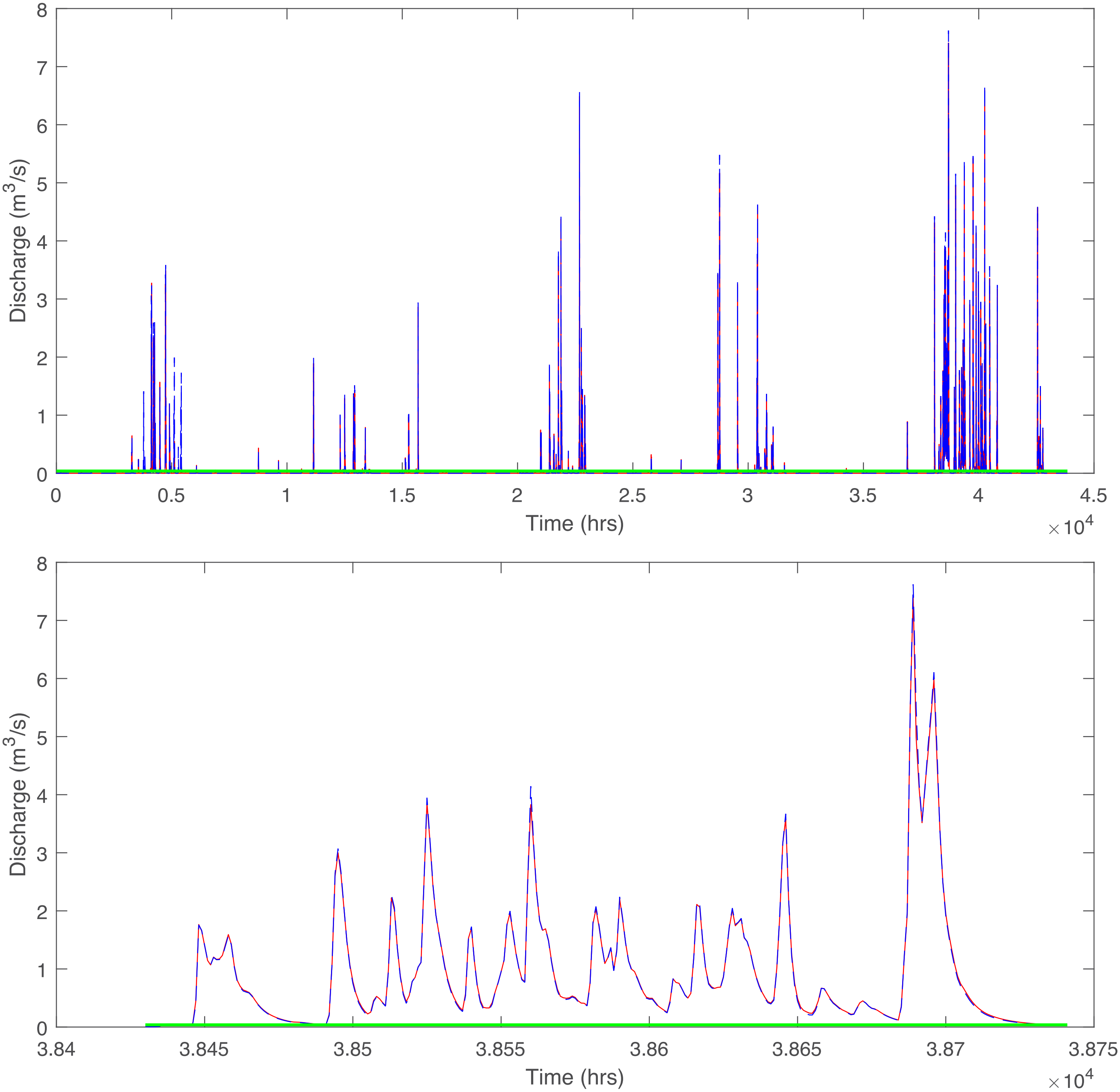
Pre-Development (Blue)	**Post-Development - with SWM Control (Red)**
Total Exceedance Time = 997 hrs	Total Exceedance Time = 1605 hrs
% Exceedance Time = 2.3	% Exceedance Time = 3.7
Number of Exceedance Events = 177	Number of Exceedance Events = 147
Cumul. Effective Discharge(m^3) = 4.683E+05	Cumul. Effective Discharge(m^3) = 4.897E+05
Cumul. Effective Work Index(J/m^2) = 4.440E+07	Cumul. Effective Work Index(J/m^2) = 4.593E+07



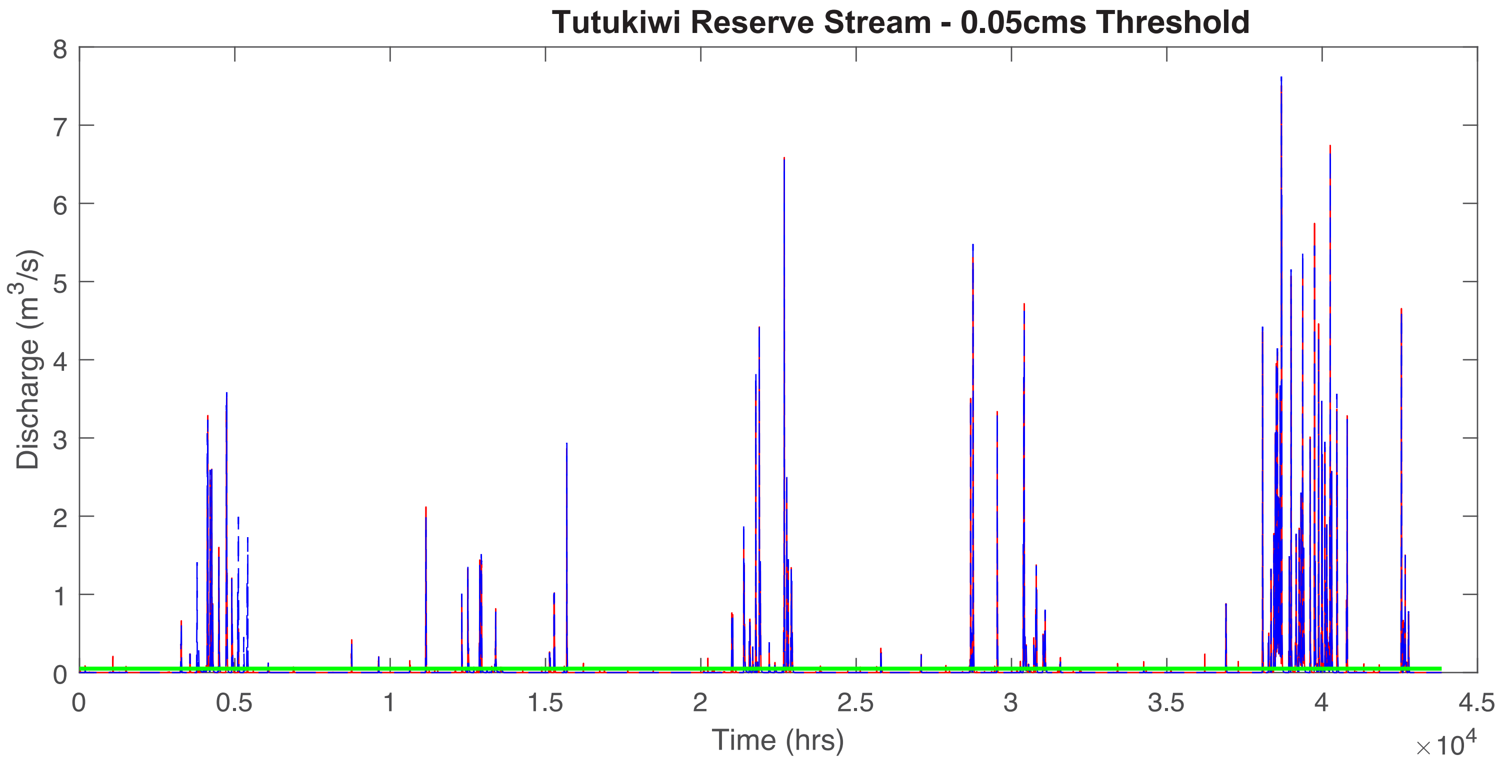


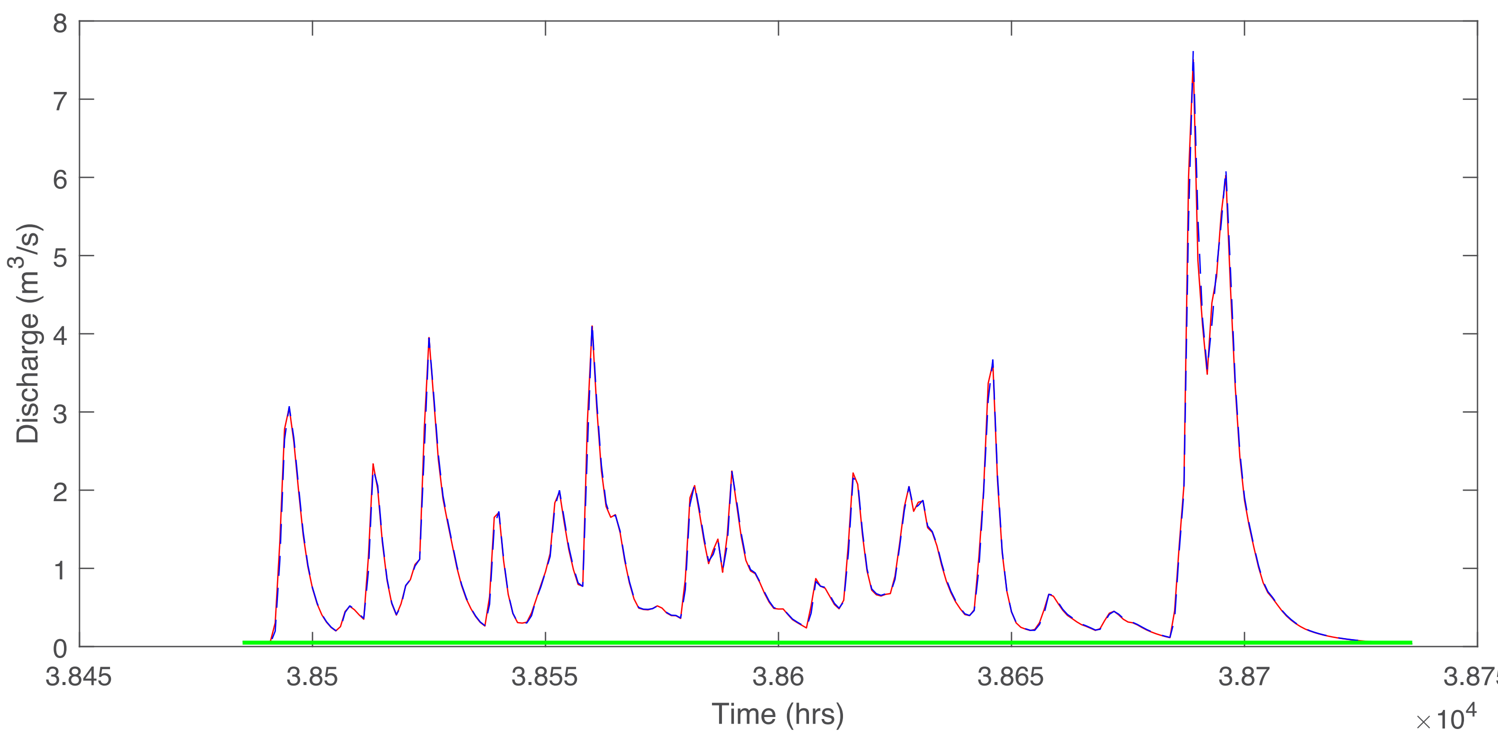
Pre-Development (Blue)	**Post-Development (Red)**
Total Exceedance Time = 3466 hrs	Total Exceedance Time = 3432 hrs
% Exceedance Time = 7.9	% Exceedance Time = 7.8
Number of Exceedance Events = 100	Number of Exceedance Events = 149
Cumul. Effective Discharge(m^3) = 6.067E+06	Cumul. Effective Discharge(m^3) = 5.850E+06
Cumul. Effective Work Index(J/m^2) = 1.494E+08	Cumul. Effective Work Index(J/m^2) = 1.439E+08

Tutukiwi Reserve Stream - 0.038cms Threshold

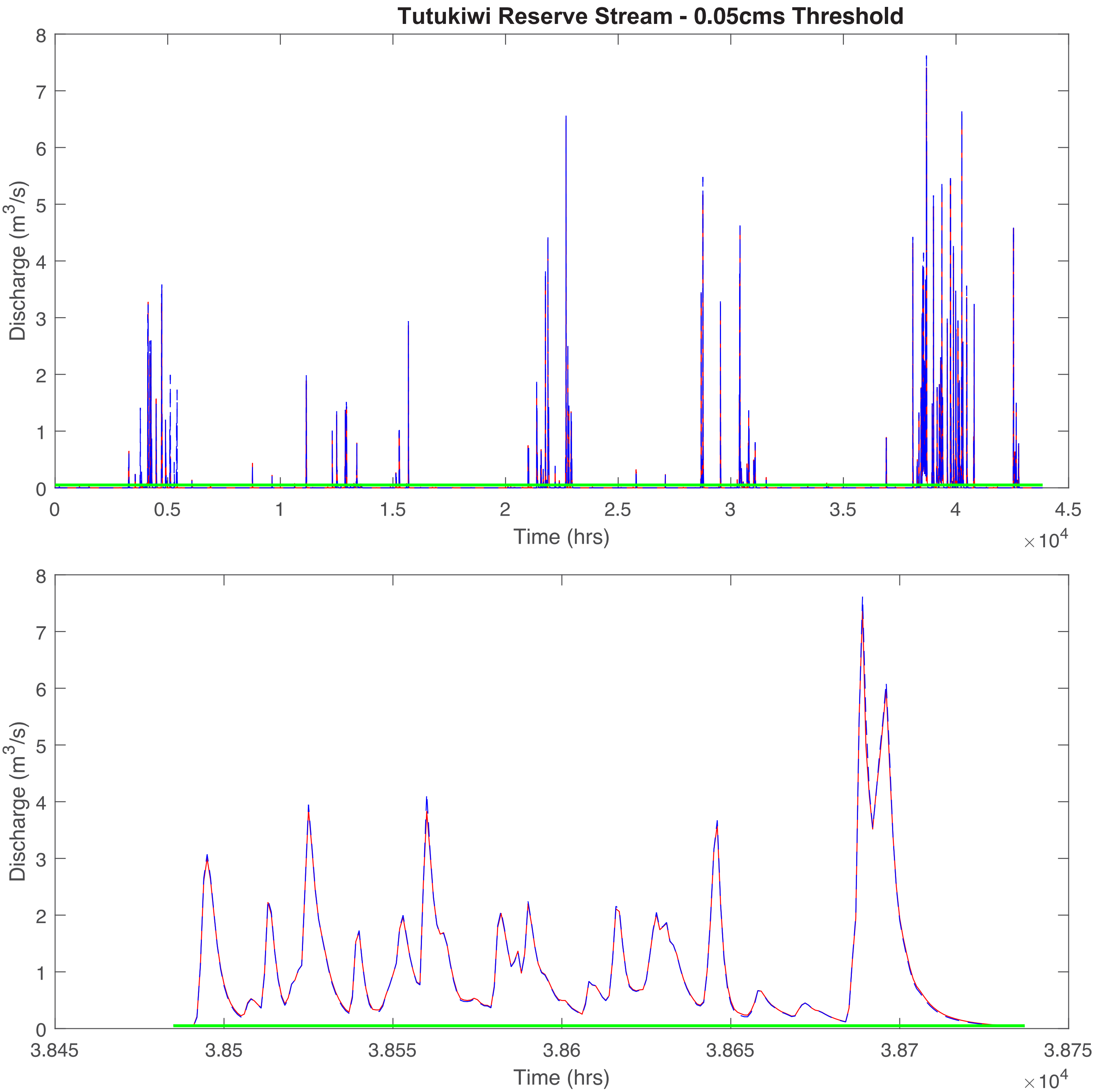


Pre-Development (Blue)	**Post-Development - with SWM Control (Red)**
Total Exceedance Time = 3466 hrs	Total Exceedance Time = 3373 hrs
% Exceedance Time = 7.9	% Exceedance Time = 7.7
Number of Exceedance Events = 100	Number of Exceedance Events = 98
Cumul. Effective Discharge(m^3) = 6.067E+06	Cumul. Effective Discharge(m^3) = 5.819E+06 Cumul.
Cumul. Effective Work Index(J/m^2) = 1.494E+08	Effective Work Index(J/m^2) = 1.434E+08





Pre-Development (Blue)	**Post-Development (Red)**
Total Exceedance Time = 3171 hrs	Total Exceedance Time = 3110 hrs
% Exceedance Time = 7.2	% Exceedance Time = 7.1
Number of Exceedance Events = 102	Number of Exceedance Events = 144
Cumul. Effective Discharge(m^3) = 5.924E+06	Cumul. Effective Discharge(m^3) = 5.709E+06
Cumul. Effective Work Index(J/m^2) = 1.436E+08	Cumul. Effective Work Index(J/m^2) = 1.383E+08



Pre-Development (Blue)

Total Exceedance Time = 3171 hrs

% Exceedance Time = 7.2

Number of Exceedance Events = 102

Cumul. Effective Discharge(m^3) = 5.924E+06

Cumul. Effective Work Index(J/m^2) = 1.436E+08

Post-Development - with SWM Control (Red)

Total Exceedance Time = 3065 hrs

% Exceedance Time = 7.0

Number of Exceedance Events = 103

Cumul. Effective Discharge(m^3) = 5.681E+06

Cumul. Effective Work Index(J/m^2) = 1.378E+08

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The Power of Commitment