`

STORMWATER MANAGEMENT PLAN

ASHBOURNE DEVELOPMENTS

CIVIL ENGINEERING ▼ SURVEYING ▼ LAND DEVELOPMENT

PROJECT INFORMATION

CLIENT:	Matamata Development Ltd
PROJECT:	289001

DOCUMENT CONTROL

DATE OF ISSUE:	18TH NOVEMBER 2025
REVISION	Rev B
AUTHOR	Raatite Kanimako Engineer
REVIEWED BY	Mitchell Smith Associate
APPROVED BY	Dean Morris Regional Director

Level 1, 5 Owens Road
Epsom, 1023
Auckland
New Zealand
Phone 09 571 0050
www.maven.co.nz

TABLE OF CONTENTS

1 OVERVIEW

This stormwater management plan ('SMP') outlines the proposed management of stormwater for the Ashbourne developments, located west of the Matamata town centre.

This report has been updated following further winter groundwater level investigations during October and November 2025. The proposed stormwater solution for the residential development catchments C and D have been updated for the higher than initially expected peak groundwater levels. The proposed stormwater solution for the Retirement Village has been updated to address the challenges posed by near surface peak groundwater levels. The stormwater design for areas of high groundwater are no longer reliant on soakage. Updates for revision B of this report are marked in blue for ease of review.

The overall development is illustrated in Figure 1 below. The project spans approximately 100 hectares of land, encompassing areas both north and south of Station Road, as shown below. The southern areas extend westward toward the Waitoa River.

The SMP outlines the overarching stormwater management principles that will form the basis of stormwater design to support future development of the proposed sites.

Discharge consent is being sought to develop the system to enable future stormwater discharge from the proposed sites. This SMP is prepared to support this discharge consent application.

The project is split into 4 sites.

1. Residential Development
2. Northern Solar Farm Development
3. Southern Solar Farm Development
4. Retirement Village

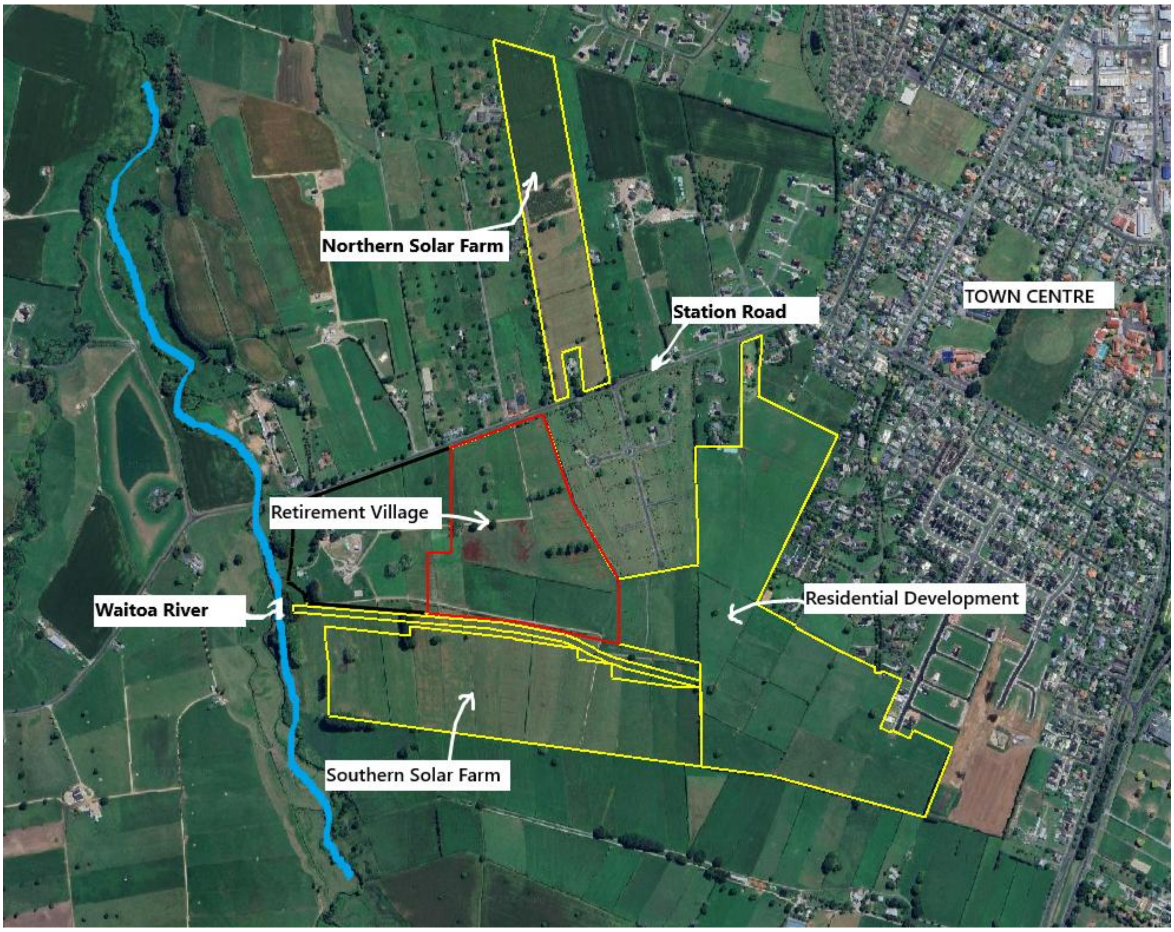


Figure 1: Site locality plan.

SITE	AREA (Ha)
Northern Solar Farm	13.00
Southern solar Farm	24.82
Retirement village	20.00
Residential	45.00

Table 1: Site Areas

There are total of 8 number of legal parcels distributed within the 4 project sites. List of these is provided is table 2 below.

PARCEL ID	AREA (ha)	OWNERS
Northern Solar Farm		
Lot 2 DP 567678	13.00	R.A Hemmings Limited
Southern Solar Farm and Retirement village		
Lot 1 DP 21055	33.23	R.A Hemmings Limited
Lot 2 DP 21055	27.38	R.A Hemmings Limited
Residential		

Lot 5 DP 365568	3.29	P & M Equipment Hire Limited
Lot 1 DPS 65481	4.20	CAT Limited, RM Craig, WJ Perry
Lot 5 DP 384886	8.10	Eldonwood Limited
Lot 3 DP 14362	13.71	R.A Hemmings Limited
Lot 204 DP 535395	24.14	Eldonwood Limited

Table 2: Legal Parcels, associated area, and Owners

The areas are currently zoned within rural, rural residential and future designated residential areas. The development has been master planned by the client in collaboration with Matamata-Piako District Council ('MPDC'), Waikato Regional Council ('WRC') and a design consultant specialists. As a result, this SMP is built upon previous discussions around stormwater management for the overall site with WRC and MPDC.

A discharge consent is required to enable the future stormwater discharge from these developments, which necessitate the importance of this SMP. The new discharge consent will be transferred to Council and ultimately form part of the Comprehensive Discharge Consent upon the vesting of the public network.

Stormwater is proposed to be discharged via the following methods:

- Soakage within the site using stormwater devices such as raingardens, soakage trenches and soakage basins.
- A proposed Greenway running east to west which discharges into Waitoa River at 80% of pre-development flows. The proposed greenway will also aid in diverting existing flows heading north from lands south of the project.
- Existing drains or overland flow paths leaving the project site at less than 80% of pre-development flows.

The overall Development area is divided into four distinct developments as listed and summarised below.

1.1 ASHBOURNE NORTHERN SOLAR FARM

This development spans approximately 13 Ha and will serve as the first solar farm constructed within the Ashbourne development. The site will feature 14,642 solar panels, generating required power to meet regional renewable energy requirements. The farm incorporates permeable ground coverage and minimal impervious surfaces to ensure effective stormwater infiltration and flow dispersion. A network of grass swales and drains will manage runoff from solar panels and ancillary infrastructure, maintaining water quality prior to discharging into the existing channel/farm drain along the northern boundary.

1.2 ASHBOURNE SOUTHERN SOLAR FARM

The Ashbourne southern solar farm spans approximately 24 ha and is the second solar farm planned for the Ashbourne development. The site will include 33,946 solar panels, generating required power to meet regional renewable energy requirements. The farm incorporates permeable ground coverage and minimal impervious surfaces to ensure effective stormwater infiltration and flow dispersion. A network of grass swales and drains will manage runoff from solar panels and ancillary infrastructure, maintaining water quality prior to discharging into the proposed greenway before discharging into Waitoa River.

1.3 RETIREMENT VILLAGE

The Retirement Village spans approximately 16 ha and is designed to accommodate 218 villas, one Aged Care Hospital, and other required facilities tailored to meet the needs of an aging population. The development aims to provide a high quality, connected environment for retirees, emphasizing on community living and accessibility.

Stormwater is proposed to be managed by lot connections, catchpits, pipe networks and swales which convey flows to 4 centralised raingardens and 2 artificial wetlands for stormwater quality treatment and flood storage and attenuation. Stormwater is managed to discharge from the site at 80% of pre-development flows.

1.4 RESIDENTIAL DEVELOPMENT

Spanning approximately 45 hectares, the residential development includes 518 Lots, designed with mix of housing typologies and densities to meet client and local authorities' requirements. Key feature of this development is the formation of the proposed greenway which serves as a stormwater management system for Catchment B of the residential areas. The Greenway conveys and attenuate overflows from the road and lot areas releasing at 80% of pre-development flows into the Waitoa River. Lot Areas and Road carriageways will be treated and discharged at source via the use of soakage systems where viable. The northern catchment is proposed to be serviced by pipe networks and wetlands.

1.5 STAGING, TIMING, RESPONSIBILITY AND FUNDING

The proposed development includes four distinct projects: the Residential Area, the Retirement Village, the northern solar farm, and the southern solar farm. Construction of these projects is not anticipated to occur simultaneously.

Key infrastructure elements, such as the greenway, provide shared downstream conveyance capacity; however, not all projects will depend on its availability. The Retirement Village will be proposing to install its own conveyance channel to discharge flows into the Waito River, and only Catchment B of the Residential area is set to discharge into the Greenway

The southern solar farms will utilise existing natural channels or, where necessary, create new discharge pathways to the Waitoa River prior to the establishment of the Greenway, while the northern solar farm will operate independently of the greenway system.

The Residential and Retirement Village developments will be carried out in staged phases, with each stage designed to comply with the principles and requirements outlined in this Stormwater Management Plan (SMP). Interim measures, such as swales and temporary conveyance routes, will be implemented to ensure ongoing mitigation throughout the construction period.

1.6 COSTS, FUNDING AND VESTING ASSETS

- The construction of the proposed stormwater management devices will be undertaken by the consent holder. The proposed stormwater infrastructure includes a greenway swale, public piped networks, soakage trenches, detention basins, raingardens, and private propriety devices.
- The public assets will be vested to council at the appropriate time as the development progresses. Private assets will remain in private ownership where appropriate legal instruments will be set up to ensure ongoing operation and maintenance responsibilities are with the owners. Discussions will be undertaken with council(s) as to the design of the infrastructure, location, and purpose, with all public infrastructure subject to the relevant Engineering Approval process.

1.7 OPERATION, MAINTENANCE AND MONITORING PLAN

Operation and maintenance plans will be provided for all stormwater management devices that will be vested to Council(s), which will be required as a condition of any approved Resource Consent.

2 HYDRAULIC CONNECTIVITY

2.1 WATER SENSITIVE DESIGN

- Stormwater management for the developments will include several devices which are considered as water sensitive design elements. These include the following:
- Dry basins and a greenway with landscaping.
- Soakage Devices
- Wetlands
- On-lot stormwater Retention through Soakage
- Protection of existing bush and covenant features.
- Planting of riparian areas if applicable.

- The Water Quality Volume (WQV) is treated at source for both Road and Lot areas where soakage is viable. Overflows above the 10-year ARI cc event are directed to Dry basins and Wetlands for both RV and Residential Developments (Refer to RV and Residential section for more detail). With both solar farms, existing runoff route is to be maintained with minimal impervious runoff being added. The proposed stormwater solution ensures stormwater is being treated before discharging into the receiving environment.

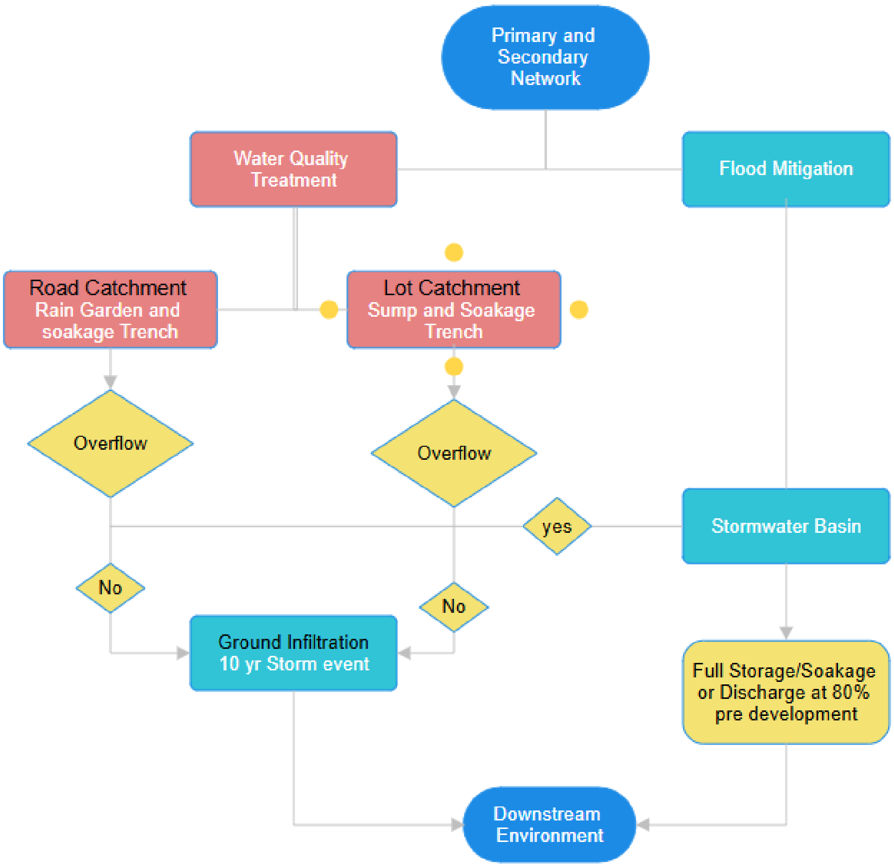


Figure 2: Hydraulics Connectivity Chart

3 EXISTING CATCHMENT CONTEXT

The Ashbourne development covers approximately 100 Ha, including areas, north and south of Station Road. The site features a flat to gently sloping topography shaped by its historical use as pastoral farmland. Gound elevations across the site range from RL 67m around the north area with RL 72m towards west and south, with the site generally draining in a northwest direction towards Waitoa River. The site is bounded to the west by Waitoa River, to the east by Existing Residential developments, and to the south by additional pastoral land, with Station Road dividing the northern and southern areas.

The existing land is predominantly grass covered, used for grazing livestock, characterised by grass covered paddocks, sporadic mature trees, and agricultural infrastructure such as farm buildings and swales. A network of farm drains across the site facilitates surface water conveyance, diverting water in line with the natural drainage pattern. Historical aerial photographs confirm the landform and drainage network have remained largely unchanged for the course of 50 years.

3.1 EXISTING CALCULATION PARAMETERS

The design and calculation assumptions used for pre-development are outlined in the table below. These values were established using the available GIS/aerial view, topography information, geotechnical report, and NIWA.

Pre-Development CN	
Pervious	**Impervious**
61	98
Pre-Development 24hr Rainfall Depth (mm)	
10yr	**100 yr**
128	200

Table 3: Design Parameters

Rainfall data. Two predevelopment scenarios have been adopted in this stormwater Management Plan, depending on the purpose of the assessment;

1. **Design and sizing of stormwater mitigation devices** (Section 6). Pre-development data used when Designing and sizing of proposed devices uses historical rainfall event data without climate change. Refer to table 3 above.

2. **Flood sensitivity Analysis** (Section 8) for assessing impact of the surrounding environment adopts RCP8.5 for Existing scenarios consistent with the proposed scenario. This would allow assessment of impacts using same events to quantify its relative impacts.

3.2 EXISTING OVERALL CATCHMENT

A catchment analysis was undertaken to establish the extent of the Existing catchment within the proposed development areas. TR20/07 and HEC HMS were used for this analysis, with results shown below and further detailed in plans 410 series. Refer to Table 4 below for catchment areas. Approximately 2-5% of existing impervious coverage is likely present within the existing catchment; however, it is assumed in our calculation that there is none. This assumption is conservative.

Figure 3 Existing catchments & Flow direction (Refer to Plans 410 Series)

The peak flow rates for both the 10- and 100-year events were calculated using HEC HMS, using the catchment analysis findings in addition to calculation parameters provided under section 3.1 and are provided in the table below. A few inflows from upstream areas were established from this catchment analysis. Either diversion away from development or accommodating it/them within catchment design for post development will be required (Proposal regarding these inflows are provided under Post development Section of this SMP).

			Pre-Development (historical no cc) Flow	
SITE	**Catchment**	**Area (Ha)**	**10-year flow (m^3/s)**	**100-year flow (m^3/s)**
RESIDENTIAL	A1	2.39	0.18	0.37
	A2	8.78	0.47	0.96
	A3	0.52	0.04	0.09
	A4	3.33	0.21	0.43
	A5	0.47	0.04	0.08
	A6	9.33	0.52	1.06
	A7	28.51	1.64	2.97
	A8	0.54	0.05	0.1
SOUTH SOLAR	A9	121	4.22	8.63
RETIREMENT VILLAGE	A10	10.1	0.39	0.81
	A11	4.12	0.03	0.07
	A12	17	0.87	1.78
NORTH SOLAR	A13	0.26	0.05	0.11
	A14	2.33	0.15	0.28
	A15	17.89	0.83	1.71
	A16	0.66	0.05	0.11

Table 4: Overall Existing Catchment Flow table (refer to plan 410)

The above table shows pre-development catchment areas and their respective flowrates, prior to development and serve as a baseline for checking overall flowrate pre vs post for the residential. This is linked to the 410 plans (Appendix A of infrastructure Report) and later referenced in tables 18 and 19.

3.3 WAITOA RIVER

The Waitoa River is one of the primary receiving environments for stormwater discharges from the Ashbourne development. It runs along the western boundary of the southern site and acts as the discharge point for both the residential and southern solar farm stormwater systems. The river plays a critical role in regional drainage and is known to experience periodic flood events, especially under prolonged rainfall conditions.

All stormwater outfalls from the development that discharge into the Waitoa River have been designed to include attenuation and treatment upstream, with flows directed either through dry basins, the greenway, or existing conveyance features. This discharge strategy ensures that post-development flows into the Waitoa River are maintained at or reduced from pre-development conditions, and energy dissipation structures are proposed to minimize erosion risk at outfall locations.

Future assessments of the river's bank stability, riparian condition, and erosion susceptibility near discharge points will be conducted during the detailed design stage to ensure the integration of ecological and hydraulic objectives. Monitoring may also be proposed as a consent condition to assess any long-term impacts on the river system.

3.4 NORTHERN SOLAR FARM

The northern solar farm site, legally described as Lot 2 DP567678, encompasses an area of approximately 13hectares and is situated on the northern side of station road. The site comprises predominantly open pasture with sparse tree cover with no signs of existing dwellings. the topography shows a gradual slope across the site, generally falling from the southwest towards the northeast corner. The surrounding landscape features slightly elevated land to the west and south, contributing to the general eastward and northerly flow pattern across the site.

3.5 RETIREMENT VILLAGE

The southern Solar farm and the Retirement village comprise two adjoining rural land parcels located south of station road. Retirement Village site covers the northern portion of the two adjoining areas and Southern Solar farm to the South.

The area is predominantly open pastureland with generally flat typology with minimal existing infrastructures. A farm track currently crosses the site. The entrance is located on Station Road and provides access across the farmland, including the existing dwelling, which is situated to the west of the Retirement Village development boundary. Notably, the existing dwelling has its primary vehicle entrance located further west along Station Road.

The Retirement village is located adjacent to station road and occupies northern areas of the two adjoining parcel lands. General typology reveals majority of the site slopes towards east, with portion slopes south and westward towards the Waitoa river. This channel conveys flow to an existing culvert beneath station road, where it continues downstream.

Existing Inflows into the site are mainly from the southern solar farm areas and the residential area. Table 4 above summarises each sub catchment flows with plans 410 series (Appendix A) providing further details.

3.6 SOUTHERN SOLAR FARM

The southern Solar farm and the Retirement village comprise two adjoining rural land parcels located south of station road. Retirement Village site covers the northern portion of the two adjoining areas and Southern Solar farm to the South.

The area is predominantly open pastureland with generally flat typology with minimal existing infrastructures

The solar farm occupies the southern areas of the two parcel lands and receives additional inflows from the upstream catchment located to the south of the site. The general landform from the survey data reveals a general fall north until the flows splits, one drains down into Waitoa and rest discharges further north to the retirement village through the existing drain along the Eastern boundary, traverses over station Road, through further farmlands and finally will get discharge into the Waitoa River.

3.7 Existing Culvert – Station Road

Existing culverts and drains are located within and around the development sites, primarily to convey surface runoff, around the site, beneath station road and through ex accessways or farm tracks. The culverts form part of the existing drainage network conveying stormwater within the site and from upstream catchment. these culverts remain in service; however, their capacity and condition vary across the site.

Figure 4: Existing Culvert with blockage Issue

One critical culvert is located along the eastern boundary corner of the retirement village area. This is where an open drain collects runoff and dischargers under station road through this culvert. This culvert serves as a key outlet for surface water from the Retirement village site and further upstream catchment.

Visual inspection indicates that this culvert is in bad condition, with vegetation overgrowth and sediment accumulation presents at both the inlet and outlet locations. These conditions will restrict flow, capacity, particularly during higher rainfall events, and poses risk of upstream ponding.

3.8 RESIDENTIAL DEVELOPMENT

The Residential development site comprises of multiple existing land parcels, refer to overview section of the report. The land is currently used for rural and pastoral purposes with only one existing dwelling adjacent to station road.

The site lies to the south and west of established residential areas and is bounded by station road to the north, with rural properties including the solar farm site covering the western and southern areas. The site has a gentle and flat areas, sloping towards low lying areas in the central and northern parts of the site. Surface runoff generally conveyed through naturally formed low lying areas including existing farm drains etc.

The residential development existing catchment area has been subdivided into sub catchments, each draining towards the identified discharge points as outlined in the table below and stormwater plans 410 series.

3.9 ECOLOGICAL FEATURES

An Ecological assessment was conducted for the site covering the areas below. this assessment identified the western portion of the southern parcel, bordering the Waitoa River, as an area of heightened ecological sensitivity. This area contains a network of aquatic features, including oxbows, minor ponds, and a second order stream Channel, alongside existing riparian plantings of native species. While most of the site is highly modified and has low ecological value, this floodable corridor was recommended as a "no-development zone" due to its potential for ecological restoration and its contribution to habitat diversity and freshwater functions.

Figure 5: Existing Culvert with blockage Issue

While the proposed development areas do not fall within this "no development zone," the proposed greenway outlet does encroach into this area. As covered in section 6.2, the proposed greenway provides essential flood conveyance and will also enhance the ecological value of the site through planting and other measures; therefore, works within this area will further improve its ecological value.

Further value Engineering and design at detailed design stage will be undertaken to ensure full compliance with the ecological report recommendations and referenced standards.

3.10 GEOTECHNICAL REPORT

Two preliminary geotechnical investigations were undertaken to provide full coverage of the Ashbourne's development area. The investigations cover assessed ground water levels, soil soakage capacity, liquefaction potential, and lateral spreading risks, ensuring comprehensive data for stormwater management, earthworks, Roading and foundation design. The two referenced Geotechnical report in this SMP are listed below (Appendix Of the Infrastructure Report).

1. HAM2023-0124AB Rev 1 (dated 12th December 2023)
2. HAM2023-0124AE Rev 0 (dated 5 July 2024)

The site is characterised by a predominantly, flat to gently sloping topography, underlain by Hinuera formation soils, with ground water levels varying seasonally and across different areas.

The findings provide crucial insights into the site's Geotechnical behaviour, have informed the design completed to date and will inform future design considerations. The key findings are summarised below.

<u>Ground Water Levels</u>

- Depths recorded between 1.5m and 4.2m across the sites.
- Seasonal Fluctuations of up to 1m observed.

<u>Soil Soakage Capacity</u>

- 2 x 10^-6m/s to 5x10^-6m/s for Silty Soils
- 7 x 10^-6m/s to 6x10^-6m/s for Sandier Soils

Soakage test Area	Results	Soakage rate adopted for Design purposes (mm/hr)	Soakage Rate factored by 0.5 as per RITS for conservatism (mm/hr)
Basin D Area (SOA24-15&16)	31-78 mm/hr	54.5	27.25
Basin C Area (SOA24-13&14)	2-51 mm/hr	26.5	13.25
Basin A Area (SOA24-23&24)	178-345 mm/hr	261.5	130.75
Northern Residential Catchment (SOA23-01) *	171mm/hr	171	171

Table 5: Soakage tests Results

Soakage Test Area	Results (L/min?m^2)	Soakage Rate Adopted (L/min/m^2)	Soakage Rate factored by 0.5 as per RITS for Design (L/min/m^2)
SOA23-01	Varies Refer Calc Appendices	3.3	1.7
SOA23-02	Varies Refer Calc Appendices	11.3	5.7

Table 5.1: Soakage Tests Result

Infiltration testing across the site revealed significant variability in soakage performance, with notably low rates, such as 2 mm/hr, recorded within the North-Western Residential Catchment. According to the Geotech report, these low rates are attributed to perched groundwater, where infiltrating water is impeded by underlying less permeable layers, resulting in temporary saturation near the surface. Observations from soakage testing and variable groundwater depths indicate that perched layers are likely present in parts of the site. As such, the adopted soakage rates for design have incorporated conservative assumptions, and further investigation at the detailed design stage is recommended to confirm underlying groundwater conditions beyond any perched layers.

The results in the Geotechnical report, shows soils at this site are generally suitable to provide soakage through use of SW Devices such as raingarden/ basins etc. This soakage rate has been derived from permeability test results, the results are averaged to adopt it for design purposes and factored by 0.5 to reflect long term reduction in soakage over time.

Topsoil and Sub-surface profile

- Topsoil depth: 0.1m to 0.5m
- Subsurface Soilis: Hinuera Formation sands, silts, and gravels. Walton Subgroup soils at greater depths.

Retirement Village

Per WGA's latest memo, November 2025, results of the further winter groundwater level investigations indicated that groundwater levels are already very close to the ground surface in the northern areas of the planned Retirement Village.

Based on the above, we have adjusted our proposed stormwater design accordingly as detailed in updated section 5 such that it is not reliant on low groundwater levels.

However, it is worth noting that whilst existing peak groundwater levels are near the existing surface, the proposed development includes subsoil drainage beneath each of the roadways as well as a new stormwater pipe network, artificial wetlands, raingardens and a greenway to the south as shown in the engineering plans. The combination of the infrastructure described above will lower the peak groundwater elevation preventing areas of ponding due to groundwater.

3.11 CN - VALUES

Soil Class Group B has been adopted for pre-development modelling based on the soil types encountered during geotechnical investigations (HAM2023-0124AE), which identified dense sands and stiff silts consistent with sandy loam and loam textures. These are typically associated with Group B under TP108 and USDA-NRCS classification systems. Additionally, the observed infiltration performance during site testing supports this classification.

Pre Development CN		Post Development CN	
Pervious	**Impervious**	**Pervious**	**Impervious**
61	98	74	98

Table 6: Design CN Values

For post-development conditions, a higher Curve Number of 74 has been used to account for reduced permeability due to compaction from earthworks, which may result in soil behaviour more consistent with Group C. This approach ensures a conservative assessment of stormwater runoff. The classification may be refined at the detailed design stage if additional testing is undertaken.

3.12 WGA MOUNDING ASSESSEMENT – SOAKAGE RATE

WGA conducted a mounding assessment to determine the long-term infiltration capacity of Basins A, C, and D within the Residential development. This assessment builds on the geotechnical soakage test data by incorporating additional site-specific factors such as aquifer thickness, existing groundwater levels, and the potential for mounding beneath each basin. While the geotechnical tests provide essential input on the soil's infiltration characteristics, the mounding assessment enhances this by evaluating how infiltration behaves over time, particularly under a sustained 24-hour storm event. As a result, the average 1-day hydraulic conductivity values derived from this assessment have been adopted for design based on table below:

	mm/hr Rate
Basin A	36.03
Basin C	20.69
Basin D	20.74

Table 7: Soakage tests Results

These values reflect a realistic and technically robust approach to stormwater management for the site.

3.13 CONTAMINATION

Contamination Investigation will be carried out. Once this is completed, we will incorporate the results and information into this SMP

3.14 ARCHAEOLOGICAL ASSESSMENT

Archaeological Assessment will be carried out. Once this is completed, we will incorporate the results and information into this SMP

3.15 IWI CONSULTATION

Iwi consultation is currently ongoing, and we will incorporate the applicable information into this SMP once completed.

4 EXISTING DEVELOPMENT FLOOD MODEL

4.1 EXISTING FLOODING, MODELLING & PARAMETERS

A catchment Analysis was undertaken to assess the runoff and flow conditions dynamically during the 100year event, within and around the four development sites. This undertaking establishes existing conditions within and around the sites, and support and informs the design proposal within these areas.

A predevelopment flood analysis was undertaken using a calibrated base model and geometry. The parameters used specific to the site are detailed below.

A 100-year (24-hour) rainfall event was analysed using various points along the catchment, as shown in the figure below. Rainfall depth values were obtained at multiple locations throughout the nominated 2D area and were then averaged to determine the representative depth to be used. HIRDS RCP8.5 rainfall data was applied for both Pre-Development and Post-Development scenarios

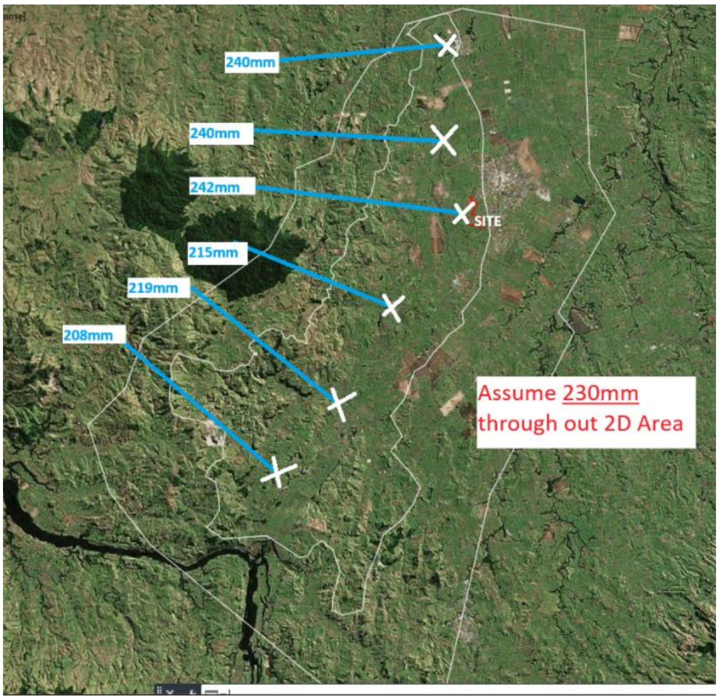


Figure 6 Rainfall depth across the 2D Area

Using HEC HMS software rainfall distribution within the shown 2D was created which has been inserted as a hyetograph onto the 2D area in HEC RAS.

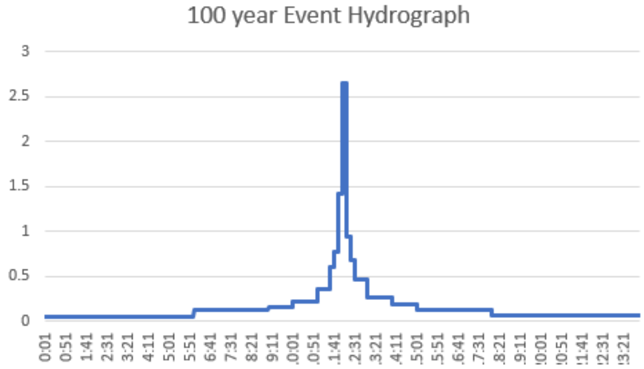


Figure 7 Hyetograph used in the Model (Data included in Appendix B)

CN Values used as shown below.

- CN numbers
    - Pre-development CN=61 for pervious Areas
    - Pre-development CN=98 for Impervious Areas
- Rainfall data from National Institute of Water and Atmospheric Research (NIWA) rainfall pattern and depth (Refer to Figure 6)

As part of the flood risk assessment for the site, an initial review was undertaken to identify any existing flood data from both the Waikato Regional Council (WRC) and the Matamata Piao District Council (MDPC). WRC confirmed that the wider area is within a flood hazard zone, however the available data was based on historical records dating back to 1996 and was not sufficiently detailed for direct use in assessing the site.

WRC did, however, provide access to gauge data from the Waitoa River at Sahara station and rainfall data from the Matamata Aerodrome Station. This formed the basis for the development and calibration of the hydraulic model, established as part of the overall flood Assessment.

The 2D flood model was built in HEC RAS version 6.6 and calibrated using observed flood events from the 2017 storm. The model simulates a 100year rainfall event (24hrs) duration and is simulated for over 3 days to ensure peak conditions are captured in the results. The calibration model/geometry was used therein to assess the existing and post development flows.

Additionally, this model will form base of the Sensitivity Analysis under section 7 of this report, the model assumes all primary system are 100% blocked.

4.2 FLOOD MODEL CALIBRATION

The Model was calibrated using observed rainfall data from the Tamhane (Matamata Aerodrome) rain gauge and river level data from the Waitoa River (Waharoa Control) Station, covering the period from April 4 to April 5, 2017. These stations are located within 6-9km downstream from the site. These data can be referred to in Appendix A of this memo.

The 2D area of the flood model was extended further downstream for calibration purposes to capture both data set available for this calibration exercise.

The datasets selected for calibration falls within the flood events that were caused by cyclone Debbie followed by Cyclone Cook in 2017. These two cyclones caused widespread of flooding across the Matamata region.

Figure 8 Calibration data Source Locations

Due to the 3km separation between the two stations, natural variability in flow response was anticipated in the model result.

The initial model result (Blue line as shown in graph below) indicated a quicker peak response than the observed data (Black Line as shown in graph below) with the modelled hydrograph rising more steeply and reaching its peak earlier than the observed data measurements.

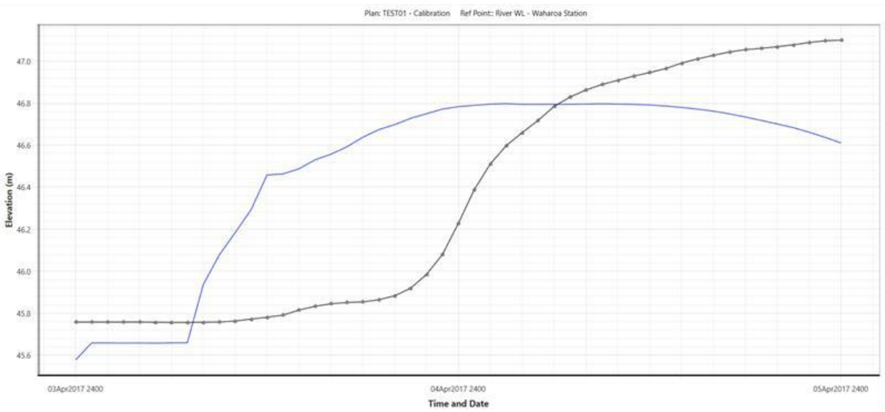


Figure 9 Elevation vs Time graph for both Model and Observed Data

Discrepancies in water levels at the beginning and end of simulation suggested differences in initial conditions, as model started with an empty channel while observed data included existing flow.

	Observed Data	Model Result	Difference
Initial Water RL	45.76	45.58	0.18
Max RL	47.1	46.8	0.3

Table 8 *First Run Results*

Area type	Road	Grass /farm	Stream Area	Rest of Area
Mannings n	0.02	0.045	0.05	0.04

Table 9 Mannings – Pre-Calibration of Model

The first calibration results presented above in first table, showing level differences between the observed data and the model. If we consider the model to have the same Water level as the observed data, the level difference is **120mm**. Mannings numbers used in the model is shown in the above second table.

Multiple runs were further undertaken of the calibration model to achieve a better alignment of the two sets. Mannings were adjusted including cell sizes. Final calibration results shown below table, including Manning’s finalised values.

The second graph shows results of one of the adjustments which has improved the difference between the two sets, demonstrating a better representation of flood behaviour within the Waitoa River.

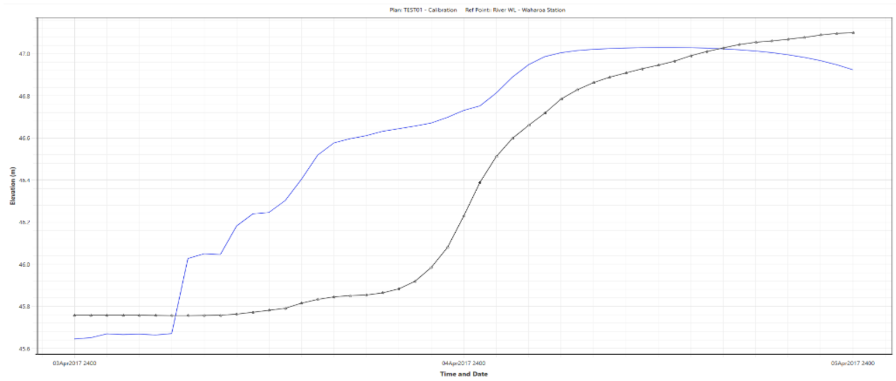


Figure 10 Elevation vs Time graph for both Model and Observed Data

	Observed Data	Model Result	Difference
Starting RL	45.76	45.58	0.18
Max RL	47.1	46.86	0.24

Table 10 Final Run Results

Area type	Road	Grass /farm	Stream Area	Rest of Area
Manning; s n	0.02	0.04	0.06	0.04

Table 11 Mannings – Final Run

If model is considered to have the same Water level as the observed data, the level difference is now **60mm**. Mannings numbers used in the model is shown in the above table. Further refinement of the model is not possible as adjusted parameters will be unrealistic.

Due to the nature of this model and data available it is not practical nor necessary to do further refinement until both models are the same. Theres is variances through rainfall distribution, land use, un modelled hydraulic devices. Instead, the focus was on ensuring that water level in the model is as realistically close as practically possible to the observed data, as this will establish a robust model base to use for flood analysis on the proposed site. Based on the above, this calibration achieves a good overall balance between real world conditions and modelling.

As mentioned in previous sections, The RCP8.5 climate change adjustment has been applied to both scenarios (pre- and post-development) to enable comparison between the two events when assessing compliance.

4.3 EXISTING MODEL RESULTS

The flood map results below display the extent of flooding with a tolerance of 50mm depth minimum. The results show the there are multiple interconnected overland flow paths traversing through sites. The general route for these flows is either a direct discharge into the Waitoa river or initially traversing ponding on land prior to discharging into the Waitoa River.

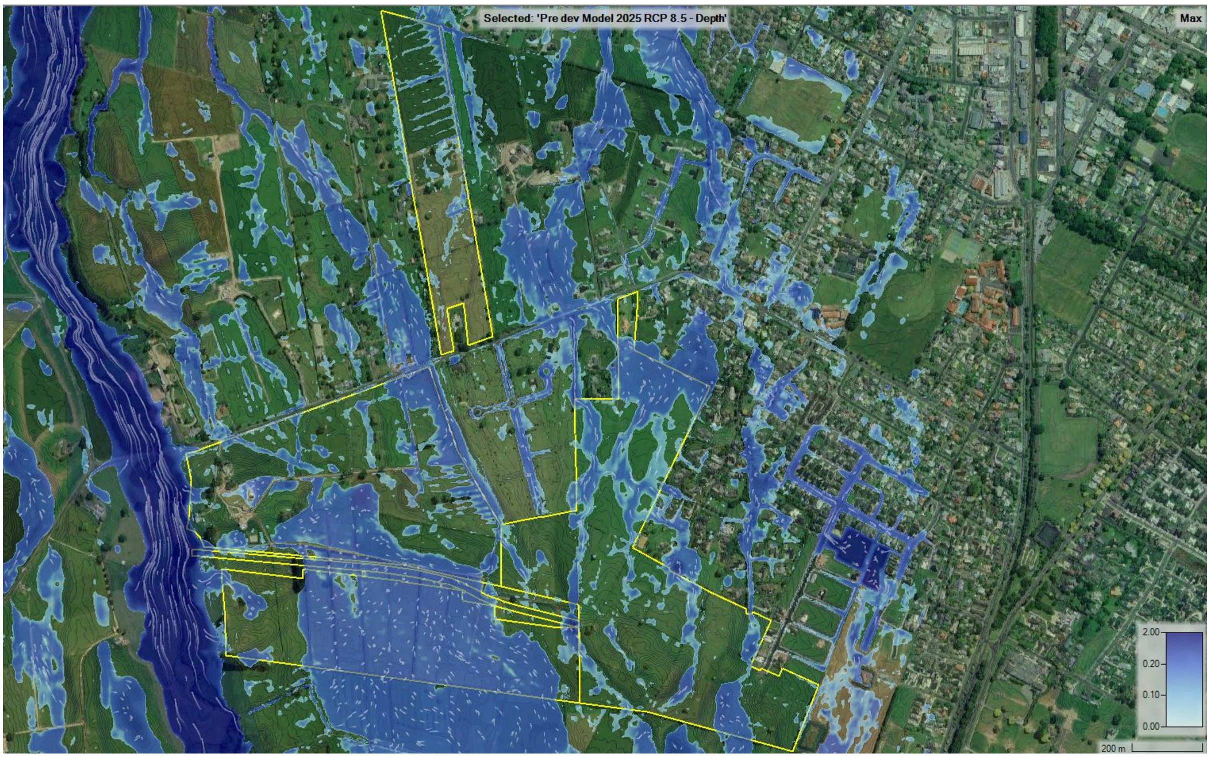


Figure 11: Overall Pre-Development Flood Map RCP 8.50

4.4 RESIDENTIAL AREA

As mentioned earlier in the report, there are several inflows into the residential area, originating from the upstream regions. These flows move northward into the site, following existing natural landforms and farm drains, before exiting along the lower northern boundary.

Notably, one of the discharge points from the residential catchment serves as an inflow into the retirement village catchment, as depicted in the screenshot, the flood model report, and the results below.

4.5 NORTHERN SOLAR FARM

inflow from the west enters the site and is conveyed through a network of existing farm drains. Most of the flows continues northward, with southern lower portion of the site discharges further east and west which is due to a crest line through the site within that portion of the site.

4.6 RETIREMENT VILLAGE

The retirement village catchment receives inflow from both southern solar farm and contributing portion (North-western Areas) of the residential catchments. There a few service tracks, farm drains and an existing conveyance channel running along the eastern boundary that leads to the existing culvert as mentioned in section 2 of this report

Due to blockage of the culvert beneath station road, as assumed in the flood model, floodwaters upstream of the culvert fills up the low land, flat areas of the RV site, flooded the area before it overtops station road.

4.7 SOUTHERN SOLAR FARM

This area receives upstream external catchment to the south. Runoff through the site is interfered by an existing farm track acting as a barrier, causing temporary ponding upstream of the farm track. The flow eventually overtops where the flow splits, north into the retirement village through the existing open channel while remainder continues westward, ponding within the localised depression before discharging into the Waitoa river. An isolated but much smaller area within the solar farm, northwest of the site, discharges directly into the Waitoa river

5 OPERATIVE CATCHMENT AND DEVELOPMENT PLANS

The development comprises both general residential and medium density residential zones as per the Operative Matamata-Piako District Plan (MPDP). This section provides a high-level summary of the planning and regulatory requirements.

5.1 REGULATORY AND DESIGN REQUIREMENTS

The relevant regulatory, technical and design requirements that the stormwater management for the development must meet are summarised in Table 12.

Table 12: Summary of regulatory and design requirements.

Requirements	Relevant regulatory/design to follow
Matamata-Piako District Plan (MPDP) Section 5.9.1 Infrastructure and servicing performance standards	• The subdivision and development of land shall be carried out so as to provide for effective stormwater management, in compliance with the Development Manual. • Be adequate to meet the maximum potential demand on site arising from the development and use of the land as permitted under the District Plan and taking into account the actual and potential land uses up-gradient of the site. • In the first instance be managed and disposed of on-site. Only where onsite disposal is unable to be achieved will Council consider new connections to public drainage, where available. • The design capacity of any piped stormwater facilities should be sufficient to accommodate the surface water flows resulting from a 10-year storm event in the case of Residential, Industrial and Business zones, and to avoid flood damage to the existing or potential principal buildings on the site, resulting from a 100-year storm event . • Overland flow paths must be able to cater for a minimum of a 1 in 100 year return period storm. Flood paths within the subject site must be protected by an easement registered against the titles affected throughout their length. Where flood paths are not feasible, the piped system must cater for a minimum of a 1 in 100 year return period storm. Where disposal is to ground soakage with no flood path, the soakage must cater for a 1 in 100 year return period storm. • Secondary flows exiting the subject site following development must not exceed pre-development overland flows. • Stormwater proposals must take into account the requirements of the Council's current stormwater discharge consents from the WRC. All proposals must be consistent with the conditions of this consent including requirements for low impact design principles, stormwater management devices and best practicable options as set out in the consent. • Stormwater works should be provided in a manner which avoids excessive modification of natural drainage systems and minimises any detriment to the environment particularly through potential contamination of natural water.

	• The integrity of the stormwater system should be maintained, and its safe and efficient operation facilitated, while ensuring an adequate level of safety to the public and those operating and maintaining the facilities. • Any detriment to the enjoyment and development of individual allotments arising from the provision and operation of the stormwater system needs to be minimised. • The known or predicted effects of climate change on a proposal, based on best available scientific knowledge, shall be taken into account. • In terms of operation and maintenance, the stormwater system shall be in line with community expectations regarding anticipated performance. • The lifecycle and maintenance costs meet community expectations. • Any disposal or treatment areas located off-site, other than to Council owned systems, shall be protected by easements as appropriate. • Council may require a detailed stormwater plan to ensure that there are no adverse stormwater effects off-site. • Any necessary consents shall be obtained from the WRC.
Comprehensive Network Discharge Consent 105063	Condition 28 of the comprehensive stormwater discharge consent (#105063) states: "The consent holder shall prepare a Stormwater Management Plan for the Matamata-Piako District Township and municipal stormwater diversions and discharge activities, which shall be submitted to the Waikato Regional Council within 12 months of the commencement of this consent". The Plan shall be developed in consultation with interested parties and shall detail the procedures, initiatives and stormwater management systems that will be implemented to operate in accordance with the conditions of this resource consent. As a minimum the Stormwater Management Plan shall describe the following: a) The relationship and integration of the Stormwater Management Plan with other Matamata-Piako District Council planning instruments and regulatory systems, including existing and proposed planning and regulatory controls that will be utilised to assist the control of routine and non-routine contaminant discharges to the stormwater system. b) Contributing catchments and the existing land uses, catchment receiving waters (including physical and biological characteristics, riparian vegetation, existing uses and values) and the municipal stormwater system characteristics (a diagram showing locations of the reticulation system, designated overland flow paths, treatment and disposal systems should be included). c) Potential risks to stormwater quality in the Matamata-Piako urban areas (i.e., resulting from routine and non-routine contaminant discharges to the municipal stormwater system;

	d) Stormwater management systems and implementation methods to avoid, remedy or mitigate routine contaminant discharges to the municipal stormwater system. e) Contingency measures and reporting systems to be implemented in the event of non-routine contaminant discharges from the municipal stormwater system. f) Methods which will be used to manage risks to stormwater quality, streambed scouring and erosion, and adverse flooding; g) Methods to identify and provide for stormwater overland flow paths in urban areas; h) Strategies for identifying municipal stormwater system structures that are impeding the upstream or downstream movement of fish, and system upgrades, implementation methods and timeframes to address these; i) Initiatives and implementation methods for improving the aesthetic appearance of drainage structures and stormwater detention areas; j) Street cleaning operations; k) All stormwater management devices and methods for ensuring that stormwater management devices are constructed and maintained in accordance with the Auckland Regional Council Technical Publication No 10 "Stormwater Management Devices; Design Guidelines Manual" (ARC, 2003) or any other technical publication approved in advance by the Waikato Regional Council acting in a technical certification capacity; l) Management and operational procedures to avoid contaminants discharging from the municipal stormwater system as a result of the various municipal operation and maintenance activities; m) Investigations and remediation programmes to discontinue informal wastewater system connections to the municipal stormwater system; n) Operation and maintenance programmes to minimise the discharge of any municipal wastewater system contaminants to the municipal stormwater system; o) Methods which will be encouraged by the Matamata-Piako District Council to minimise the effects of stormwater discharges from new subdivisions; p) Methods for ensuring consideration of Low Impact Design principles (as contained in the Auckland Regional Council Technical Publication No. 124 "Low Impact Design Manual for the Auckland Region" (ARC 2000)) for proposed greenfield development sites; q) Methods for identifying and implementing Best Practicable Options to manage the municipal stormwater system and prevent or mitigate adverse effects on aquatic ecosystems; r) Methods for implementing stormwater management education initiatives; s) A prioritised schedule for implementing the procedures, initiatives and stormwater management systems that are identified in the Stormwater management plan".

6 PROPOSED DEVELOPMENT & STORMWATER MANAGMENT

The overall development comprises of four distinct projects, as summarised below

1. Residential Development
2. Northern Solar Farm Development
3. Southern Solar Farm Development
4. Retirement Village

The residential development incorporates wetlands and roadside raingarden for treatment for high contaminant generating areas within the road carriageway area. Where areas allow, Roadside soakage integrated with raingarden combines to fully store and soak incoming flows for event up to the 10year cc storm event. Areas with no soakage to be discharged into primary pipe networks and discharged into proposed wetlands downstream for treatment and attenuation purposes. Proposed Basins and Wetlands downstream will provide for attenuation up to 100-year cc events.

Flows exceeding this are conveyed via overland flow paths within road carriageway to downstream basins for attenuation and/or soakage.

Following discussions with WRC for on-lot stormwater management, they accept in most cases it is not practical for on-lot stormwater systems to have multiple devices providing treatment. Therefore, a driveway catchpit or strip drain is proposed which discharges up to the 10-year cc event into a soakage trench/soak pit. A lot connection into the roadside soakage trench will also be provided for larger events and provides redundancy should on-lot soakage fail. Flows exceeding this are conveyed via overland flow paths within the road carriageway to downstream basins for attenuation and/or soakage.

The two solar farm projects, located north and south of Station Road, introduces minimal land disturbance. The solar panels are elevated on stilts and placed on existing pastoral land, with only limited earthworks required for the two developments. Stormwater management approach for the two solar farm sites, is utilising the existing farm swales for conveyance, treatment, and attenuation.

The retirement village incorporates 4 raingardens, 2 located north and 2 located southeast, and 2 wetlands for attenuation and treatment for 2, 10 and 100 year rainfall events. A new proposed stormwater network has been designed to convey flows via catchpits, stormwater pipes and swales to its relevant discharge points.

As mentioned in previous sections, The RCP8.5 climate change adjustment has been applied to both scenarios (pre- and post-development)

6.1 POST DEVELOPMENT CATCHMENTS

The post development catchment layout for both the residential and retirement village areas has been defined based on the proposed stormwater basins, which also serve as the primary discharge points for each sub catchment. the placement of these basins has been carefully considered to align with existing discharge locations ensuring that no new discharge points are introduced as part of the development.

Each post development catchment incorporates not only the development catchment but also includes upstream inflows that have been identified and considered in the design. These inflow areas have been accounted for in the post-development analysis to ensure ongoing conveyance and to avoid any adverse effects both upstream and downstream resulting from the proposed development

The two solar farm projects are not expected to alter the existing catchment flow patterns, as existing flow routes are not altered within the sites. As the solar panels will be mounted on steel frames with minimal ground disturbance with only access tracks introduced offsetting existing ones likely to be removed due to the development, most of the land will remain as it was. Therefore, post development flow paths in these areas closely follow existing conditions, retaining predevelopment conditions.

The post development model and calculations are built upon the following input and assumptions:

- CN numbers
    - Post-development CN=74 for pervious Areas
    - Post-development CN=98 for Impervious Areas
- Rainfall data from National Institute of Water and Atmospheric Research (NIWA) rainfall pattern and depth:

Post Development CN		
Pervious	**Impervious**	
74	98	
Pre Development 24hr Rainfall Depth (mm)		
10yr RCP8.5	100 yr RCP8.5	100yr-10yr
167	265	98

Table 13 Design Parameters table

6.2 RESIDENTIAL DEVELOPMENT – CATCHMENT A-D

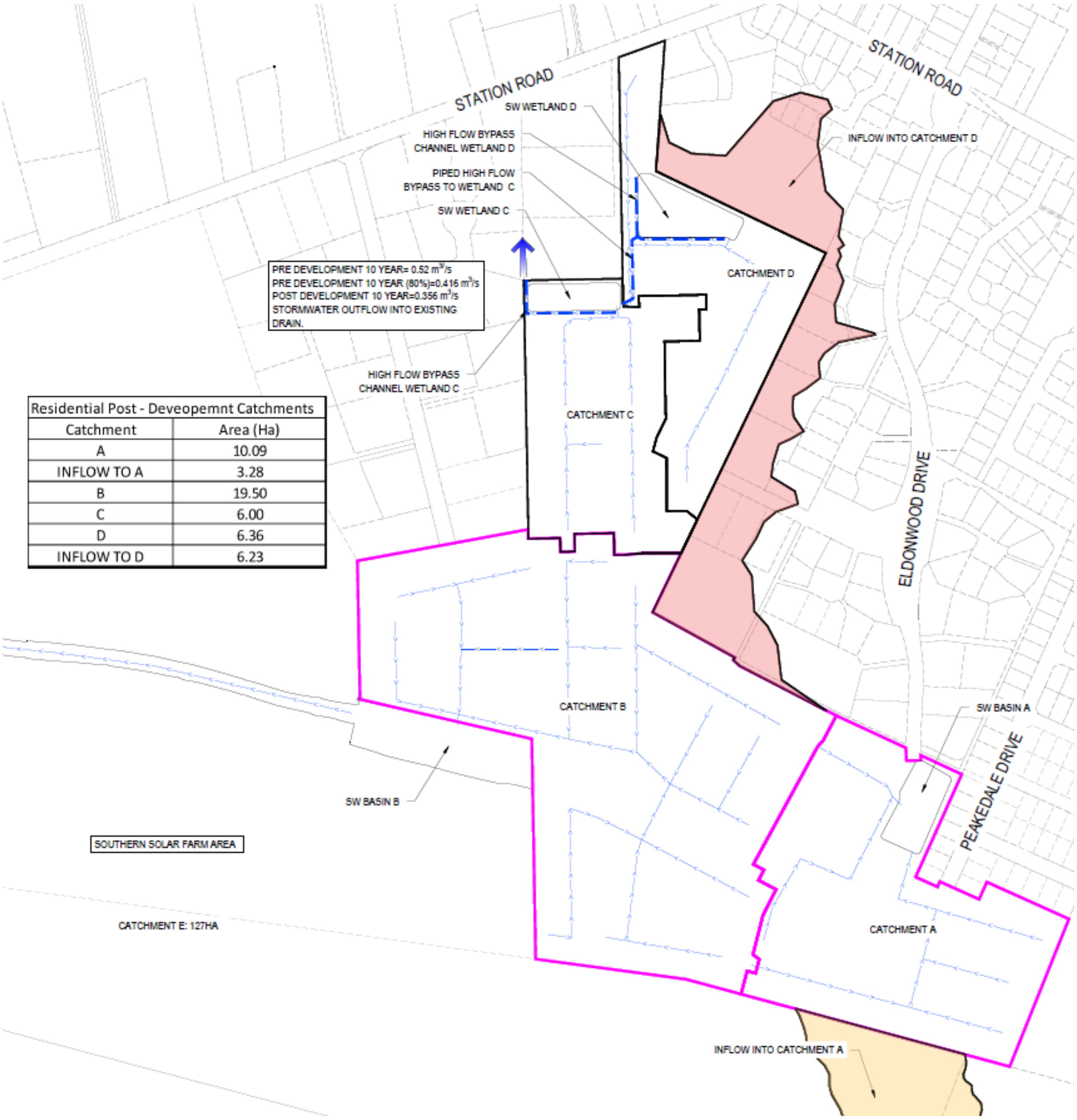


Figure 13: Overall Residential Development Catchment Plan

Catchment A is the south-eastern portion of the site. Soakage will be provided within the lots and the road to manage the stormwater flows for up to 10-year cc storm events. Storm events more than the 10-year cc storm events, will be conveyed as overland flows to a new designated stormwater basin downstream of each catchment to provide further mitigation up to the 100year cc event. upstream inflow into catchment A has been accounted for in the HEC HMS model. Refer to Appendix B.

Catchment B is the south-central portion of the site that connects to the proposed greenway due to high ground water relative to the invert of the basin, soakage was not incorporated into the design for mitigation of flow within catchment B in line with the overall strategy, the flow up to the 10-year ARI cc will be accommodated in roadside raingarden/soakage and private soakage for lot areas. Flow above this event will be drained to a newly formed Basin + greenway. The greenway will start from the western boundary, and it will continue west to connect into the Waitoa River. At the end of the greenway an orifice of 1.25m ins dimeter is to be set at base of the greenway to allow for attenuation of the post-development 100year flow to 80 % predevelopment level.

Catchment C and D are both located in the northern part of the site.

Roof runoff for these catchments will be directed to individual on-lot detention tanks sized to attenuate up to the 10-year ARI (including climate change) discharging at 80% of the pre-development peak flow.

Road runoffs within the Catchments will be divided between sections of the corridor with soakage systems (providing retention up to the 10-year ARI cc event) and sections without soakage that drain directly to the downstream wetlands. For storm events exceeding the 10-year ARI cc event, overflows from the on-lot detention tanks and from the road soakage systems, along with runoff from the non-soakage road areas, will be conveyed via the road drainage system to the proposed stormwater wetland.

The wetland provides water-quality volume treatment, extended Detention, and peak flow mitigation for the 10 and 100yr cc events.

Residential Staging

The residential portion of the development is intended to proceed in stages. As each stage progresses, interim swales will be implemented to ensure that stormwater mitigation measures remain consistent with the intent of this Stormwater Management Plan (SMP). During early stages, undeveloped areas will retain their natural flow patterns, and any new works will be designed to avoid adverse impacts on these areas. The strategic use of swales will provide effective flow management and treatment during construction and staging, ensuring that runoff is appropriately directed and controlled until full development and permanent infrastructure are completed.

6.3 RETIREMENT VILLAGE

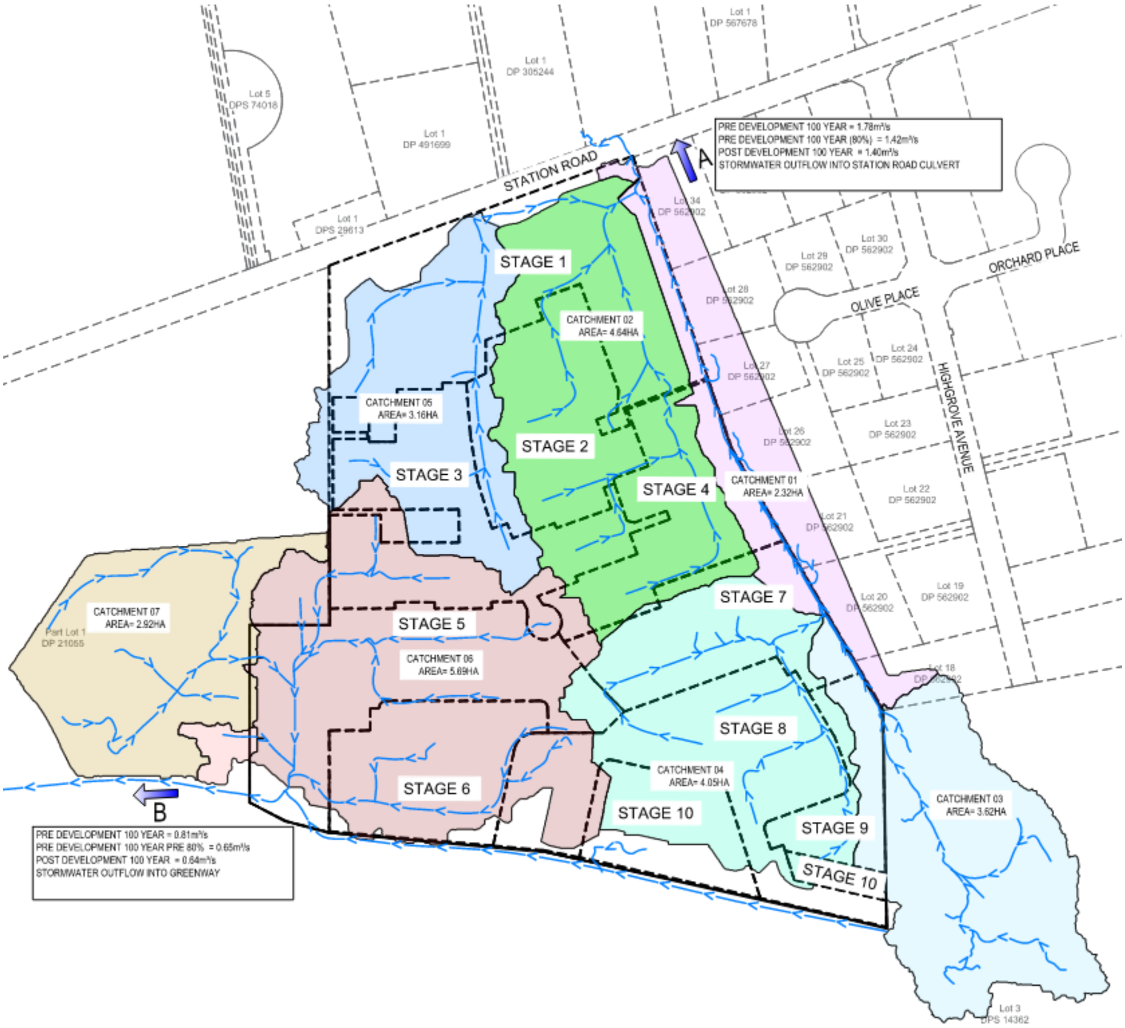


Figure 13: Overall Retirement Village Development Catchment Plan

Ashbourne Retirement Village will be serviced by a proposed stormwater network which will collect stormwater runoff from the buildings and road corridors via lot connections and catchpits respectively and discharge up to the 10-year ARI storm event to 4 raingardens and 2 proposed artificial wetlands. Secondary flows up to the 100-year ARI will be conveyed via overland flow contained within the road corridors and swales to the proposed wetlands. Raingardens and artificial wetlands will provide water quality treatment and extended detention. The wetlands will also provide attenuation and flood storage to ensure discharge from the proposed development is at 80% pre-development flows for the 10-year and 100-year ARI events.

6.4 SOLAR FARMS – NORTH AND SOUTH

As the solar panels themselves are built on steel frames, only approximately 5% of the solar farm will be changed to impervious surface. The remainder of the site remains in pasture suitable for sheep grazing. Therefore, assuming adherence to best practice stormwater management, the effects of increased stormwater runoff is considered to be ‘Low’.

Southern solar will have combination of treatment methods where it will be initially treated using existing features such as drains etc, and eventually allow to flow to its natural flow path and existing streams. Farm site has 4m wide that runs from west to east centrally dividing the site. To manage stormwater flow from the southern side of the farm, open channels are proposed run in such a way that the flow is directed towards culverts. Culverts are placed at natural low points/existing streams allow discharge from southern side of the farm flow naturally to the other side discharging to proposed greenway and eventually flowing to Waitoa river. Culverts are designed to flow at 0.5% slope and average of 1m cover to comply with WRC standards. It will be sized to accommodate the discharge from receiving environment at detail design stages.

Northern solar farm site gently slopes from south to North. Existing farm drains will be incorporated around the site to facilitate pre-treatment of stormwater which will ultimately discharge into the Waitoa River.

6.5 CATCHMENT FLOW ANALYSIS

A hydrological model was created to assess the peak flows from the post development catchments. The model is based on the following input and assumptions:

- 10-year cc and 100-year cc rainfall from the post development catchments.
- Catchment extents as per C410 & 420 Series. refer to Appendix A
- Catchment characteristics as outlined in Appendix B (SW Calculations)
- Full soakage of up to the 10year cc event for Catchments for all lot and road catchments.
- Basin A, C and D full storage and Soakage for Excess flow from upstream up to 100year cc event. Attenuation to 80% predevelopment flow for Catchment B.
- For all Catchments: road corridors and accessways to convey secondary overland flow paths to the proposed Dry basins.
- For the Solar farm catchments: road corridors and accessways to convey secondary overland flow paths to the watercourses directly.

6.6 PRINCIPLES OF STORMWATER MANAGEMENT

The proposed stormwater management for the 4 developments incorporates a number of stormwater management principles, which are focused on:

- Enhancing ecological value of the wider catchment.
- Preserving existing and waterways within the development sites.
- Mitigating flooding impacts.
- Treating stormwater runoff from the proposed impervious areas.

The principles set out in this SMP aligns with the previous projects SMP approved by WRC (Lockerbie Estate Subdivision AUTH141393.02.01) located in the same district of the proposed sites. The key components of the Ashbourne Developments SMP are as follows:

- Stormwater conveyance for up to 10year cc ARI rainfall event.
- Overland flow paths for 100-year cc ARI rainfall event to be accommodated within the site and conveyed by the road and green corridors.
- Downstream mitigation through attenuation of 100-year cc ARI rainfall event within the site:
    - Discharge limited to 80% pre-development levels (Maximum).
- Downstream mitigation through detention of 10-year cc ARI rainfall event within the site:
    - Discharge limited to pre-development levels (Maximum).

- Treatment of runoff prior to discharge into receiving environment in accordance with TP10 / GD01 / Waikato Stormwater Management Guidelines (WRC Technical Report 2020/07).
- Retention of initial abstraction runoff volume using raingardens and soakage devices for the development.

6.7 GREENWAY AND BASIN B-Residential

Basin B is connected and located upstream of the proposed Greenway. Both forms part of the overall greenway/Basin B stormwater Mitigation device designed to cater for attenuation of flow from catchment B of the Residential development area and the diversion of the existing flows south and north of the Greenway.

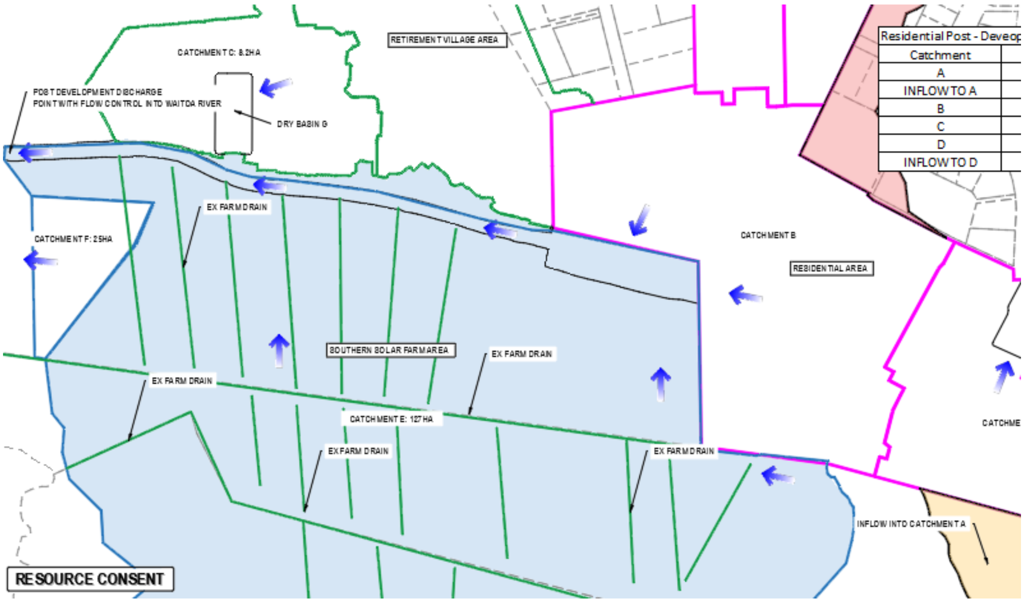


Figure 13A: Overall Greenway and Basin B Catchment Plan

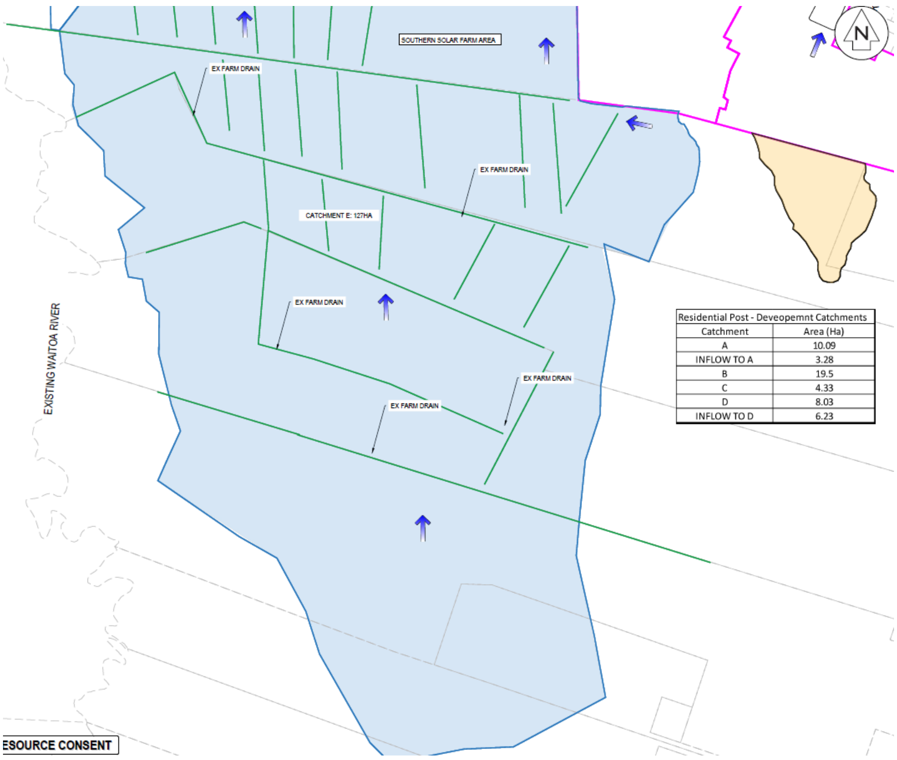


Figure 13B: Extent of Southern Catchment Plan for Greenway

Catchment	Area (Ha)
RES B	19.5
Inflow RV	8.2
Inflow South	127

Table 14 Greenway & Basin B Catchments

The 127-hectare inflow south catchment includes the neighbouring lands located south of the proposed solar farm and residential subdivision. A review of LiDAR data indicates that these lands generally slope gently from south to north.

There are a number of farm drains located throughout the area which appear to convey and direct flows into the Waitoa River. This is illustrated in the figures above. However, given the limited accuracy of the LiDAR data, and given these existing drains may become overwhelmed or blocked we have taken a conservative approach and designed the greenway to accommodate the full 127-hectare catchment, along with RES B and inflow RV, bringing the total to 155 hectares under an RCP8.5 1:100-year storm event.

The greenway will deliver significant benefits to this catchment as well as other surrounding catchments by acting as a large conveyance channel to the Waitoa River and helping to reduce localised flooding

The proposed Greenway corridor interconnects infrastructure, ecological wellbeing, connectivity, and amenity to support a place-based identity. Several uses are proposed along this corridor to encourage future residents to interact with the greenway, such as sheltered rest areas for relaxation and socialisation, active mode pathways, and play areas.

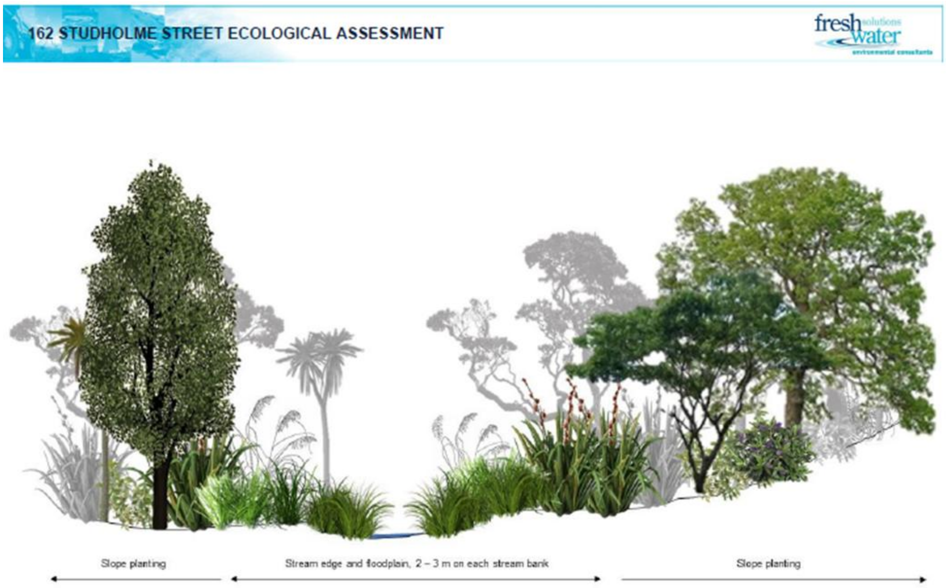


Figure 14: Greenway Cross Section (Plan 490-17)

The proposed greenway is sized to accommodate the 100-year ARI cc stormwater event flows less the 10-year ARI cc event from the Resident Area B. The 10 year cc ARI are proposed to be discharged via soakage in the road carriageway and the in the lot areas upstream of the proposed Greenway. Additionally, there is an inflow from the RV site through proposed South Easter RV Basin as shown in the above catchment diagram. This basin provides stormwater mitigation by capturing and discharging flows at 80% predevelopment flow into the greenway.

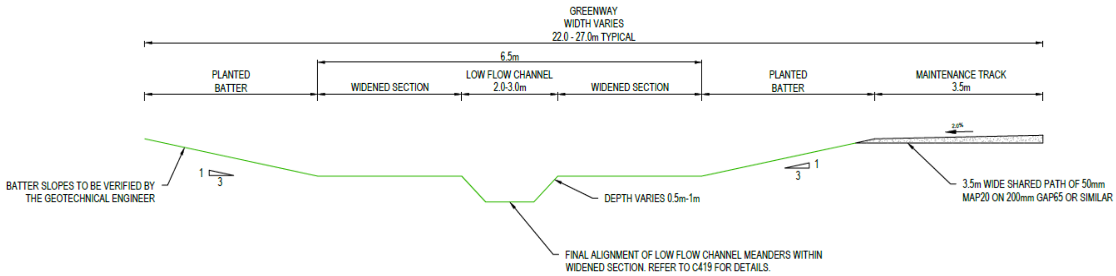


Figure 15: Greenway Cross Section (Plan 490-17)

A low-flow channel is incorporated at the base of the proposed greenway to replicate existing flow conditions and support continuous baseflows through the corridor. The channel is designed to have a width of approximately 2 to 3 meters and a depth of 0.5 to 1 meter, providing a defined conveyance path for low flows while maintaining ecological connectivity. This configuration ensures that hydraulic function is preserved during dry weather periods and provides controlled conveyance under baseflow conditions. The alignment and shape of the low-flow channel will follow the greenway's finished levels, and an impermeable liner will be considered where required to retain flow and minimize infiltration losses.

To provide for future maintenance of the greenway a 3.5m wide maintenance track will be constructed along the northern side of the greenway. The maintenance track will also provide a shared access track for pedestrians and cyclists. The greenway will have widened sections to provide some additional flood storage and to enhance the aesthetics of the greenway. The typical greenway section is shown above (figure 15).

6.8 DRY BASIN A - RESIDENTIAL

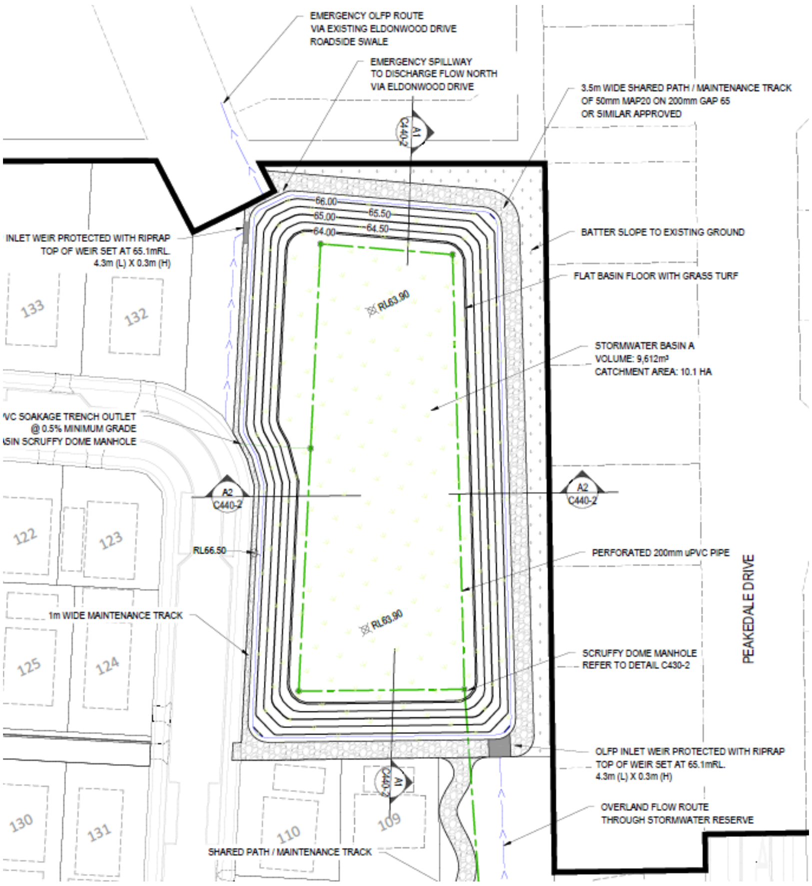


Figure 16: Dry Basin Typical

The proposed Dry basins captures excess runoff from upstream devices above the 10-year cc event. and up to the 100-year cc event. Attenuation, storage, and soakage is carried out within the dry basins with flow rate released no more than 80% Predevelopment rate.

Residential Dry basins for A have been designed conservatively allowing full storage of the from upstream catchment. there is soakage element to them that has been applied in the HEC HMS model having a constant soakage throughout the 24hr period. Further calculations were carried out to determine time to drain from full condition (peak) with max allowable 48hrs.

Refer to calculation found in appendix B of this SMP.

For this resource consent application, stormwater dry basins have been designed to conservatively store the 100-year cc less 10-year cc event and to discharge via soakage without any release of flows into existing channels or via existing OLFPs. Therefore, there is potential for further refinement of dry basin sizing to utilise existing OLFP's up to a maximum release rate of 80% pre-development

6.9 WETLANDS C AND D – RESIDENTIAL

The stormwater strategy for catchment C and D for Residential is to convey flows via pipe network or via overland flow through road network to the 2 wetlands downstream for treatment and attenuation up to 100year cc event.

Artificial wetlands C and D forms a critical part of the overall stormwater mitigation system. Wetlands will be incorporated into the stormwater system to treat the water quality volume, provide extended detention (1.2xWQV) for their corresponding catchments as well as manage peak flows up to a 100-year return period storm event (including climate change) for the two catchments. These wetlands are designed to temporarily store runoff during storm events and release it at a controlled rate, thereby reducing downstream flooding risk and protecting receiving environments.

The two wetlands are connected via a dedicated pipe system. Wetland D provides treatment and detention for runoff from Catchment D and discharges through a controlled orifice into Wetland C. Wetland C also provides treatment for its own contributing catchment. The outlet structure at Wetland C is sized so that the combined discharge from Wetland C (its own catchment) and the attenuated inflow from Wetland D does not exceed 80% of the corresponding pre-development peak flows to the existing stream

Key wetland design considerations include:

- Sizing based on achieving water quality volume within the permanent storage zone, the extended detention and attenuation of the 10 and 100-year storm events releasing at 80% of pre-development.
- Extended detention up to max 350mm depth and release over 24-hours.
- Flow splitter device upstream of each wetland to direct <2-year event through the wetland, >2-year event is directed to a highflow bypass channel to prevent scour of treatment elements and re-mobilisation of accumulated sediments.
- Highflow bypass channel outlet includes an outlet control. Flows back up and engage flood storage within the wetland.
- Permanent storage pool bathymetry per RITS to be detailed during EPA.
- Maintenance access ramp and platform adjacent the forebay will be detailed during EPA.

Freeboard and spillway design to safely pass extreme events.

6.10 WETLANDS AND RAINGARDENS – RETIREMENT VILLAGE

The stormwater management strategy within the retirement village is to convey flows via a stormwater pipe network which outlet into either 3 centralised raingardens or 2 artificial wetlands for treatment, extended detention, flood attenuation and storage before discharging to existing watercourses.

Stormwater raingardens will provide treatment of the water quality volume for their corresponding catchments. Extended detention up to 300mm depth is provided in accordance with RITS. Larger flows up to the 10-year ARI storm event will be discharged via a scruffy dome. Flood storage and attenuation will be provided for by wetland 1.

Artificial wetlands 1 and 2 forms a critical part of the overall stormwater mitigation system. Wetlands will be incorporated into the stormwater system to treat the water quality volume, provide extended detention (1.2xWQV) for their corresponding catchments as well as manage peak flows up to a 100-year return period storm event (including climate change) for the proposed retirement village. These wetlands are designed to temporarily store runoff during storm events and release it at a controlled rate, thereby reducing downstream flooding risk and protecting receiving environments. Wetland 1 discharges to an existing culvert which crosses Station Road and heads north via an existing channel eventually reaching Waitoa river. Wetland 2 discharges to the proposed greenway before discharging into the Waitoa River.

Key wetland design considerations include:

- Sizing based on achieving water quality volume within the permanent storage zone, the extended detention and attenuation of the 10 and 100-year storm events releasing at 80% of pre-development.
- Extended detention up to max 350mm depth and release over 24-hours.
- Flow splitter device upstream of each wetland to direct <2-year event through the wetland, >2-year event is directed to a highflow bypass channel to prevent scour of treatment elements and re-mobilisation of accumulated sediments.
- Highflow bypass channel outlet includes an outlet control. Flows back up and engage flood storage within the wetland.
- Permanent storage pool bathymetry per RITS to be detailed during EPA.
- Maintenance access ramp and platform adjacent the forebay will be detailed during EPA.
- Freeboard and spillway design to safely pass extreme events.

6.11 SOAKAGE DEVICES

6.11.1.1 On Lot Soakage

Subject to further field and infiltration testing, The on-lot drainage system (Soakage) for the Ashbourne Residential is proposed for catchment A and B where sufficient infiltration result are encountered. The system to consist of the following.

- Soakage Device
- Slot drain (if needed) connected to a Catchpit in the driveway
- Pipe system from the roof and catchpit (driveway) to the soakage Device
- Lot Connection pipe that connects Catchpit to the roadside soakage device.

Driveway/Impervious area runoff will flow to the slot drain directing flow to proposed catchpit with sump where settlement of courser suspended solids can occur. The flow then gets discharged into the soakage trench where it will be treated further and then soak to the ground. Pre-treatment for sediments is not required for typical residential roof loadings therefore the roof runoff with be discharged directly to Soakage trench.

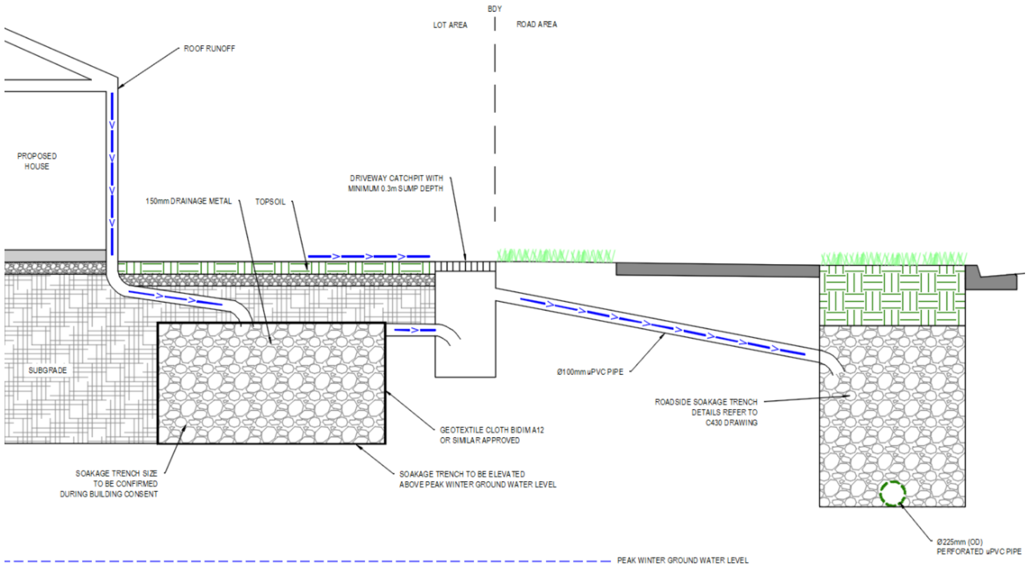


Figure 17: On Lot Drainage system/Soakage Typical Det

The proposed on lot systems have been designed to cater for up to the 10year cc rainfall event. Events above the 10year cc will be directed into the carriageway. This will be provided through a primary overflow mechanism, in the form of a connection from the proposed catchpit whin each lot to the stormwater soakage trench within the road reserve. This will form a lot connection for each of the lot areas within the Ashbourne Residential Area. Any flows above this, will be conveyed by overland flow into the road carriageway.

The sizing of soakage trenches for residential lots was based on the methodology outlined in E1/VM1. Stormwater catchment volumes were calculated for three typical impervious roof areas—150 m^2, 200 m^2, and 250 m^2. Corresponding soakage disposal volumes were determined using three representative soil infiltration rates (0.5, 1.5, and 3.0 L/min/m^2).

The required trench storage volume for each scenario was derived by subtracting the volume of water disposed by soakage from the total catchment volume. In total, nine soakage trench volumes were calculated to represent different combinations of impervious area and soakage performance, allowing flexibility in design depending on lot-specific conditions. Refer to Appendix B for full calculation. Also refer to plans which states Soakage Device Sizing requirement table.

Additional site-specific geotechnical and/or infiltration testing will be undertaken prior to construction to confirm soakage values sizing of the proposed devices

6.12 On Lot Detention TANKS (RESIDENTIAL)

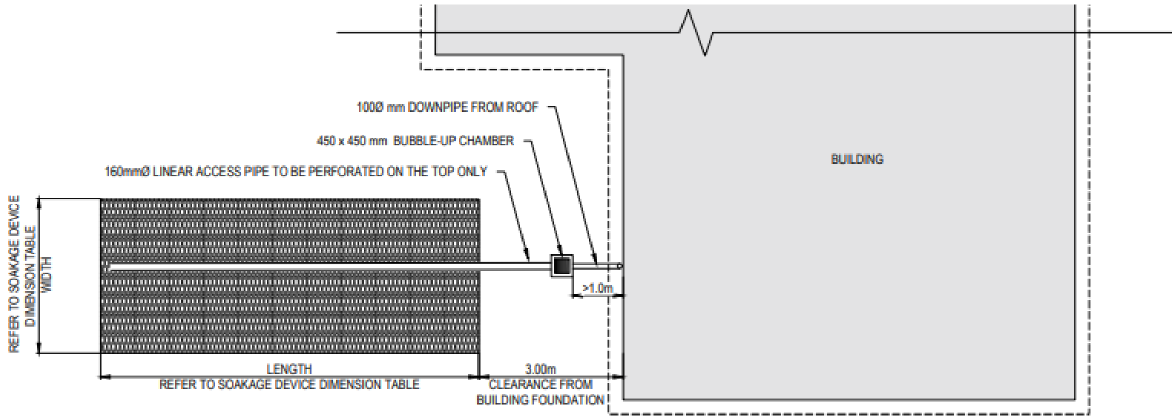


Figure 18: On Lot Detention Tank Typical Det

Onsite stormwater management for the Residential development is achieved using Detention tanks where soakage is not viable. Each lot is fitted with a 6000L tank designed to attenuate runoff for the lot for events up to the year storm event, discharging flows at no more than 80% predevelopment peak flow. Overflow from the tank is manged either by connection to the lots drain connection or via bubble up pit located within proposed vehicle crossings.

Refinement of the proposal may be occurred during detailed design stage.

6.13 Road Soakage & Raingarden – Residential

Combination of raingarden and soakage trench is used for managing impervious area flows of 10-year cc stormwater event. The raingardens will be connected via piped network to Stormwater basin.

Water from catchpits, roads and manhole will flow to raingarden where it will filter through the layers of raingarden thus removing contaminants and treating it initially. It will be placed at high point of soakage trench so it can flow naturally towards the soakage trench.

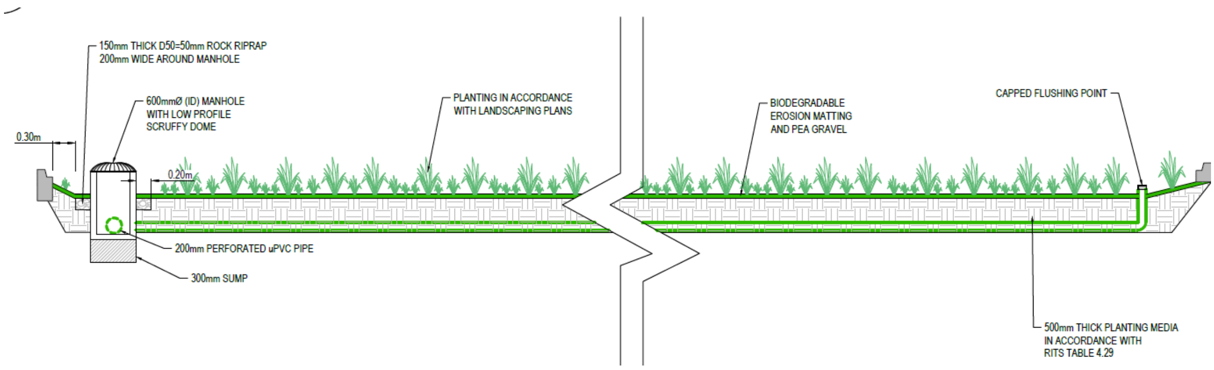


Figure 19: Typical RG (C430)

Raingardens were sized in accordance with RITS (Regional Infrastructure Technical Specification) requirements, using a volume equal to 2% of the contributing catchment area. To determine the required raingarden area, the total volume was divided by an assumed 1.0 m raingarden depth. This area was then divided by the road chainage within the relevant catchment to calculate the linear length of raingarden required per metre of road. This approach ensures adequate stormwater treatment and attenuation across road catchments, aligning with local council expectations for water quality management.

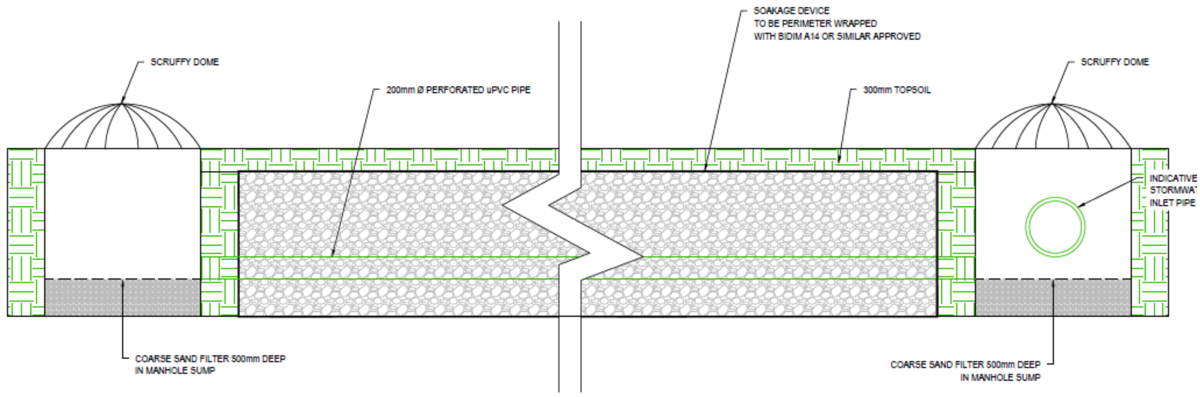


Figure 20: Typical Soakage Detail (C430-RES)

The soakage trench for the road carriageway area was designed in accordance with the Matamata-Piako District Council (MDPC) Soakage Manual, with the effective soakage area calculated using the methodology specified in the manual. The design objective is to ensure the trench can accommodate runoff up to a 10 year cc rainfall event.

Soakage rates were obtained from CMW Geotechnical testing,(allowing for 0.5 factor in accordance with the manual) using two representative rates, based on test pit results:

- 1.7 L/min/m²
- 5.7 L/min/m²

Due to the absence of a 10-year, 24-hour rainfall intensity graph in the MPDC Manual has been reasonably assumed/interpolated based on available information. Impervious area limits were estimated based on standard road widths.

For 18 m wide roads, with a trench dimension of 2.0 m wide x 1.5 m deep, the required trench length per meter of road was

- 0.55 m (0.28 m each side) for the 1.7 L/min/m² soakage rate.
- 0.27 m (0.135 m each side) for the 5.7 L/min/m² rate.

For 20-meter-wide roads, with a trench size of 1.2 m wide x 1.5 m deep, trench lengths of 1.1 m total (0.55 m per side) and 0.62 m total (0.31 m per side) were required for the lower and higher soakage rates, respectively.

- 1.1 m (0.55 m each side) for the 1.7 L/min/m² soakage rate.
- 0.62 m (0.31 m each side) for the 5.7 L/min/m² rate.

These trench sizing calculations above sets guidelines for sizing of soakage based on road typology and relevant soakage testing results onsite.

Additional site-specific geotechnical and/or infiltration testing will be undertaken prior to construction to confirm soakage values sizing of the proposed devices

6.14 INITIAL ABSTRACTION VOLUME

Initial abstraction volume of runoff from the proposed catchments will be retained by the proposed on-lot soakage Trenches and catchpit sumps. Initial abstraction volume of runoff from the proposed roads will be retained by the proposed raingardens/soakage devices. The preliminary design of these devices is summarised above and as provided in appendix B.

7 POST DEVELOPMENT FLOODING

7.1 OVERLAND FLOWPATHS (OLFPS) - CARRIAGEWAY

- Additional branches of OLFPs will be created as roading corridors are formed. The following measures will be adopted to mitigate their effects of these overland flow paths on the proposed development.
  - Identify and maintain natural overland flow/watercourse locations to convey concentrated stormwater from the site. Utilise existing culverts (where possible) to maintain the same discharge locations, post development.
  - Identify and retain any upstream OLFPs and/or watercourses to avoid any upstream flooding.
  - Ensure OLFPs are to be designed where possible within the roading network and discharge into watercourses and 100-year detention devices.
- The preliminary OLFP design is shown in Maven Associates drawings C460, **Appendix** B Detailed design of the OLFPs will be provided at future detail design stage following the approval of the resource consent.
- A Preliminary assessment of the post development overland flow paths (OLPs) has been carried out to evaluate the behaviour of surface runoff in the road carriageway under the proposed stormwater management system. The design scenario is based on the RCP8.5 climate change scenario, incorporating all proposed soakage and treatment devices and the assessment is done through Autodesk Hydraflow software. The OLFPs represents the conveyance of surface runoff as a result of the proposed system during the 100-year cc storm event.
- Flow depths and velocities were assessed at key locations throughout the development covering all the various road/Accessway typologies ensuring and confirming conveyance o the OLFP is viable through proposed carriageway.
- See below table showing results at the key locations and is linked to plans 430 For Residential Development

Section	Peak Flow (m^3/s)	Max Depth (m)	Max Velocity (m/s)	V x D (m^2/s)
A1	0.6	0.137	0.77	0.105
A2	1.1	0.158	0.97	0.153
A3	0.6	0.131	0.82	0.107
B1	0.9	0.167	0.81	0.135
B2	0.3	0.116	0.59	0.068
B3	0.2	0.091	0.61	0.056
B4	1.5	0.216	0.79	0.171
B5	0.6	0.121	0.912	0.11
C1	0.5	0.125	0.81	0.101
D1	0.3	0.116	0.59	0.068
D2	0.3	0.109	0.59	0.064

Table 15: OLFP Assessment Results (Residential Area)

Section	Peak Flow (m^3/s)	Max Depth (m)	Max Velocity (m/s)	V x D (m^2/s)
A	0.17	0.09	0.75	0.07
B	0.26	0.1	0.77	0.08
C	0.29	0.11	0.8	0.09
D	0.34	0.12	0.82	0.1
E	0.44	0.13	0.85	0.11
F	0.45	0.13	0.87	0.11
G	0.55	0.14	0.89	0.12
H	0.67	0.15	0.91	0.14
I	0.7	0.16	0.94	0.15
J	0.73	0.16	0.96	0.15
K	0.85	0.17	0.98	0.17
L	0.96	0.18	1.01	0.18
M	1.07	0.19	1.03	0.2
N	1.24	0.21	1.05	0.22
O	0.58	0.14	0.9	0.13

Table 16: OLFP Assessment Results (Retirement Village)

- Most OLFP sections comply with standard design thresholds. However, three sections in the residential development and six sections in the retirement village recorded maximum water depths above the 150mm guideline.
- Residential
- Catchment A – Section 2: Max Depth = 0.158
- Catchment B – Section 1: Max Depth = 0.167
- Catchment B – Section 4: Max Depth = 0.216
- Retirement Village
- Sections I – N: Highest Max depth is 0.21m and Lowest Max Depth: 0.16m
- Despite minor exceedances in depth, depth x velocity (m2/s) values remain well below critical safety thresholds defined in Austroads 2012 Part 5, which specify.
- < 0.4m2/s pedestrian Safety
- <0.6m2/s for vehicle safety
- The highest recorded value was $0.22m^2/s$ confirming safe flow conveyance for both pedestrians and vehicles under design conditions. Flow is primarily routed along proposed roads conveyed into roadside treatment and 10year cc event mitigation devices prior to spilling back (during event above the 10year cc) onto the road and get discharged into the proposed basins or Greenway.
- It is noted that a separate flood sensitivity analysis has been completed using HEC-RAS 2D modelling assuming all stormwater devices are fully blocked. The assessment detailed in section 7 of this report, evaluates overland flow behaviour under worst case flooding conditions within and surrounding the site.

7.1 PROPOSED BASINS

The proposed basins located downstream of each catchment are primarily intended to capture rainfall runoff from events that exceed the 10 year cc event from the subject sites (RV and Residential). Please note that the table below presents the water levels for the 10-year cc event. This relates to upstream inflows, rather than those from the subject sites.

The table below displays the expected peak water levels for the basins during the critical duration 24-hour rainfall events for both the 10-year and 100-year periods.

SITE	BASIN	Water Level - HEC HMS		Min Platform Levels adjacent units
		10 years	100year	
RESIDENTIAL	A	64.10	66.20	66.70
	B	66.80	66.9	67.4
	C	65.22	65.75	66.25
	D	65.30	65.90	66.40
RETIREMENT VILLAGE	1	65.99	66.70	67.20
	2	66.20	66.75	67.25

Table 17: Stormwater Basin Water Levels

Note Min platforms levels for both the Residential and Retirement village complies with minimum freeboard requirements. This requirement is based on the 500mm freeboard referenced from both E1 Building Code and Matamata Piako district plan 11.4.

7.2 CONVEYANCE CHANNELS – EXTERNAL INFLOWS

The overall stormwater design accounts for all inflows from upstream catchments that flow through the site under pre-development conditions. As introduced in section 2.2 of this report, these inflows will require management to ensure conveyance while minimising impact to surrounding environment is maintained. To manage these flows, the proposal includes three conveyance channels positioned around the perimeter of the residential development. These channels aim to redirect upstream overland flows downstream, thereby minimizing flood risk and preventing adverse impacts on existing neighbouring properties.

The proposed channels vary in width from 3 to 4 meters. Preliminary sizing was conducted using HEC-RAS as part of the Flood RCP 8.5 sensitivity analysis. This process involved defining the channel alignments and iteratively adjusting their positions and widths to effectively accommodate flow during the design storm event while minimizing upstream impacts.

These inflows through the conveyance channel have been incorporated in the design of the greenway and the Stormwater Basins.

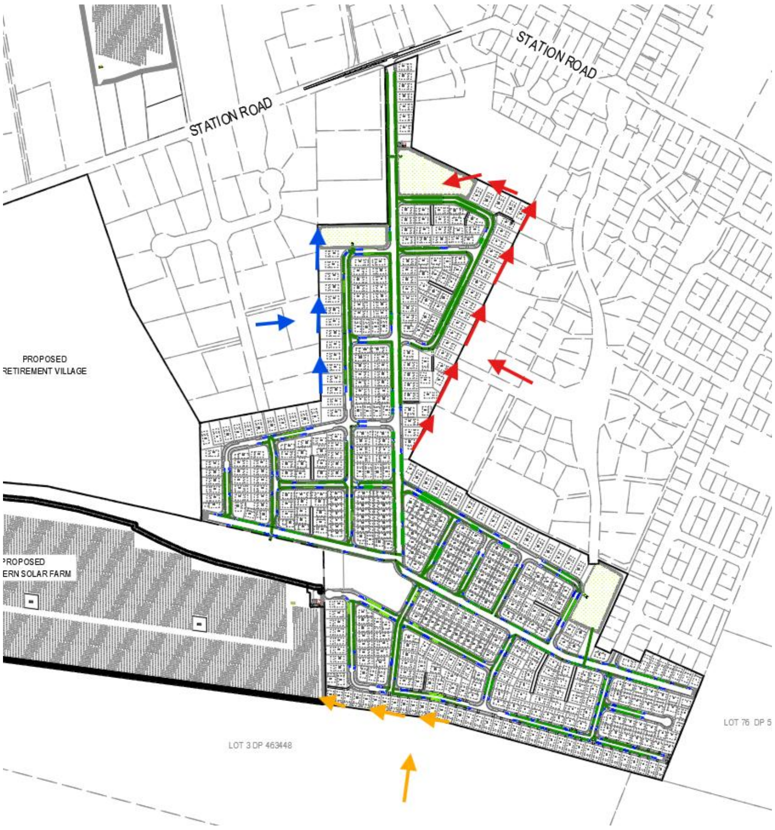


Figure 22: Greenway Cross Section (Plan 490-17)

Conveyance South of Residential Development - Orange

The proposed conveyance is required here for Runoff from the upstream portion of the post development Catchment B, located south of the residential development. This inflow will be redirected to the western edge of the residential area. From there, it will discharge into the southern solar farm zone. The redirected flow will ultimately be conveyed into the Waitoa River via the proposed greenway corridor.

Conveyance East of Residential Development – Red

Overland flow along the eastern boundary of Post Development Catchment D will be captured by this channel along the adjacent Eastern boundary. Under heavy rainfall, northern area of Basin D will pond within the neighbouring property, similar to existing pre-development conditions. Once ponding reaches elevation of RL 65.60, channel will overflow into Basin D. The basin also receives runoff from Catchment D (100-10yr cc event) and is designed with a soakage base, similar to Basins A and C, allowing for further storage for all inflows into Basin D.

Conveyance West of Residential Catchment C - Blue

A Conveyance channel is proposed along the western boundary of Post Development Catchment C to accommodate the natural overland flow from upstream areas. Under existing conditions, this flow would pass through the development area before entering the northern natural stream. The proposed channel will convey this existing flow downstream into the existing channel.

This approach ensures that upstream flows are managed efficiently, preventing backwater effects due to infilling of the downstream areas.

7.3 DESIGN FLOW RESULTS – EXISTING DISCHARGE POINTS

SITE	Catchment (See Plan C-400 Series)	Pre-Development Flow: 10-year flow (m^3/s)	Pre-Development Flow: 100-year flow (m^3/s)	Post Development Flow: 10-year flow (m^3/s)	Post Development Flow: 100-year flow (m^3/s)
RESIDENTIAL	A1	0.18	0.37	0	0
	A2	0.47	0.96	0	0
	A3	0.04	0.09	0	0
	A4	0.21	0.43	0	0
	A5	0.04	0.08	0	0
	RES-CATCH B Discharge through Greenway			0.85	1.72
	A6	0.52	1.06	0.356	0.827
	A7	1.64	2.97	0	0
	A8	0.05	0.1	0	0
SOUTH SOLAR	A9	4.22	8.63	2.8	5.75
RETIREMENT VILLAGE	A10	0.39	0.81	0.30	0.64
	A11	0.03	0.07	0.03	0.07
	A12	0.87	1.78	0.69	1.40
NORTH SOLAR	A13	0.05	0.11	0.05	0.11
	A14	0.15	0.28	0.15	0.28
	A15	0.83	1.71	0.83	1.71
	A16	0.05	0.11	0.05	0.11

Table 18: HEC HMS discharge Table

The flow comparison table above presents the pre- and post-development peak flows across the various catchments within the Ashbourne development. Pre-development flows show a natural distribution of runoff across the site with majority discharging due north of the sites and into the Waitoa River.

In the post-development scenario, many of these flows are reduced to zero, particularly within the Residential and Retirement Village areas. This is due to the proposed use of basins with integrated soakage components, both within the basins themselves and in upstream areas, designed to retain stormwater on-site. Meanwhile, some catchments still show controlled discharge where infiltration is not feasible.

This pattern reflects a conservative design approach and highlights opportunities for further optimisation in the detailed design phase to improve land use efficiency while maintaining regulatory compliance.

8 POST DEVELOPMENT FLOODING – SENSITIVITY ANALYSIS

This section presents a sensitivity analysis carried out using HE RAS 2D for the 100year cc event which assumes all soakage and pond systems are fully blocked. This assessment builds on the Section 7 results but provides a more conservative view of surface flooding behaviour across the sites.

Objective of this analysis is to observe whether the overland flow paths can convey runoff when the mentioned key stormwater components are fully blocked, and how this will impact the development and the surrounding environment.

The map results shown below shows extent of the 100year cc flood event within and neighbouring site.

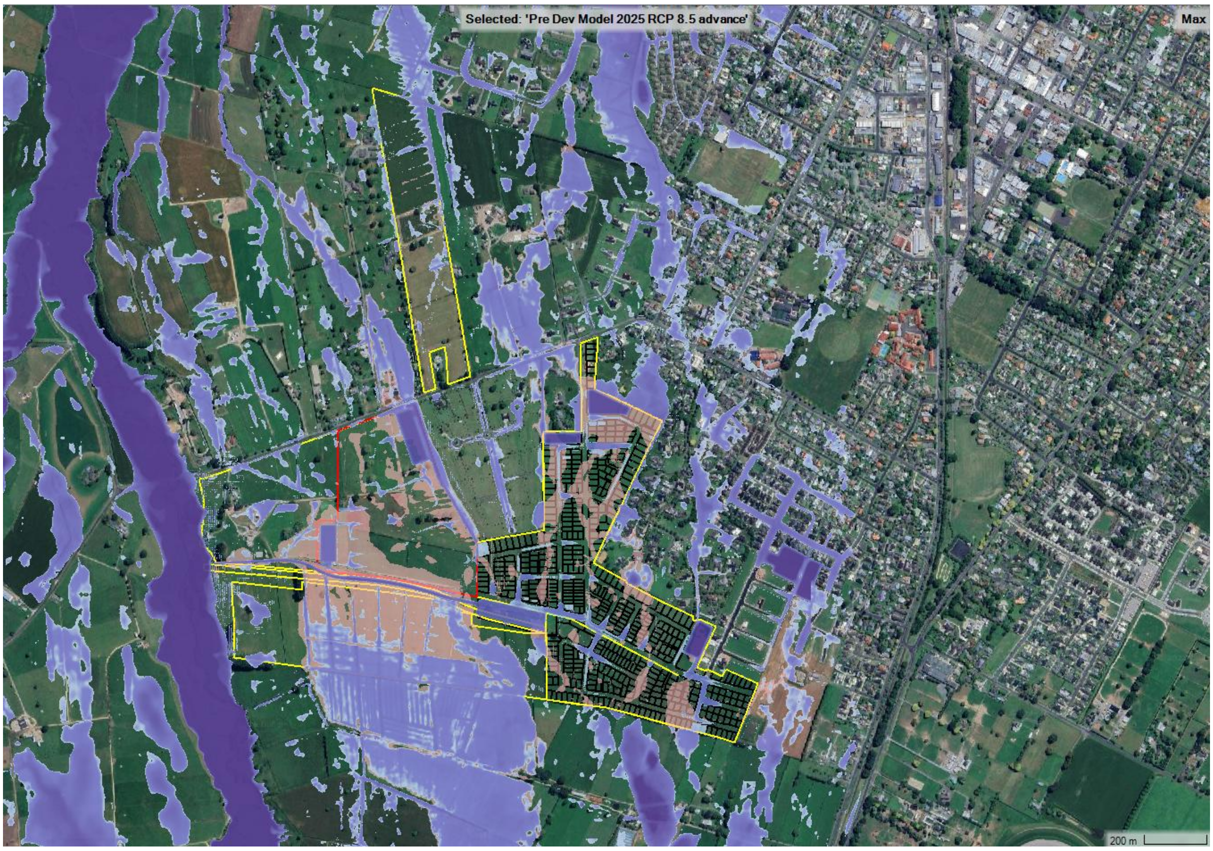


Figure 23: Overall Pre & Post Development Flood Map RCP 8.50 - Sensitivity

HEC RAS Catchment Flows (100 YEAR cc)m³/s				
SITE	**Catchment**	**Pre Development**	**HEC RAS Section**	**Post Development**
RESIDENTIAL	A1	0.85	RES SEC 11	0.2
	A2	0.96	RES SEC 10	0.96
	A3	2.05	RES SEC 09	0.05
	A4 & A5	0.85	RES SEC 03	0.08
	A6	2.3	RES SEC 06	1.74
	A7	2.6	RES SEC 04	1.2
	A8	0.13	RES SEC 12	0.05
SOUTH SOLAR	A9	1.7	GRNWAY SEC 01	9.8
RETIREMENT VILLAGE	A10		GRNWAY SEC 01	
	A11	0.84	RV SEC 01	0.9
	A12	2.99	RV SEC 02	1.03
NORTH SOLAR	A13	1.6	S.North SEC 01	1.5
	A14	0.9	S.North SEC 02	0.95
	A15		S.North SEC 01	
	A16	6.13	S.North SEC 03	6.1

Table 19: HEC RAS Sensitivity discharge Table

Referring to the table above, although the overall post-development scenario indicates reduced discharge at most locations, a few key areas require additional context.

For the Retirement Village, the increase in flow at the downstream section (RV Section 1) is attributed to preliminary surface levels, for small areas along the fringes of the RV development draining into the existing Road Swales on Station Road. These levels will be refined during the detailed design stage to ensure that all overland flows are directed appropriately into the proposed ponds, eliminating unintended bypass or overflow conditions.

8.1 Greenway and Waitoa River Interface.

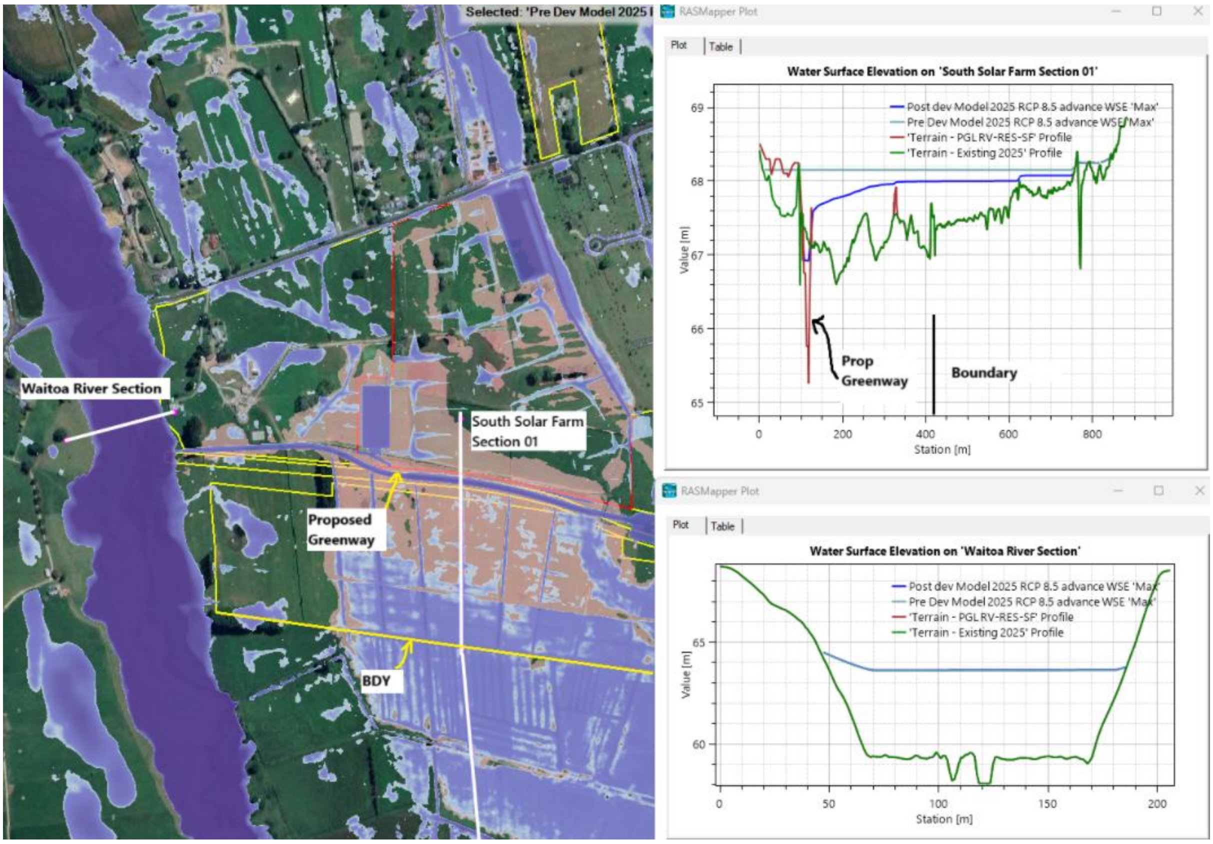


Figure 24: Overall Pre &Post Development Flood Map RCP 8.50 - Sensitivity

The proposed greenway plays a crucial role in the overall stormwater strategy for the development. One of its primary functions is to receive and attenuate flows from Catchment B of the residential area before discharging them into the Waitoa River. Another function is to convey flow from the south and north of the greenway. Refer to section 6.3, which provides more information the proposed greenway/Basin B and its function.

Under pre-development conditions, significant surface ponding was observed in the model within and upstream of the southern solar farm area, as seen in the results screenshot (blue—post dev & pink—pre dev). With the introduction of the greenway, this water is now conveyed more effectively downstream, resulting in an 80–90 mm reduction in flood depths in those upstream areas. This improvement supports the project's intent to alleviate localized flooding and enhance overland flow management.

However, the greenway's increased conveyance capability results in a higher runoff directed to the Waitoa River. This has led to a slight increase in the flood level, showing a 20 mm increase in water surface elevation observed in the flood sensitivity assessment. While this increase is acknowledged, it is considered minor and is outweighed by the substantial upstream benefits and overall reduction in localized flooding across the site.

8.2 North Of Basin D

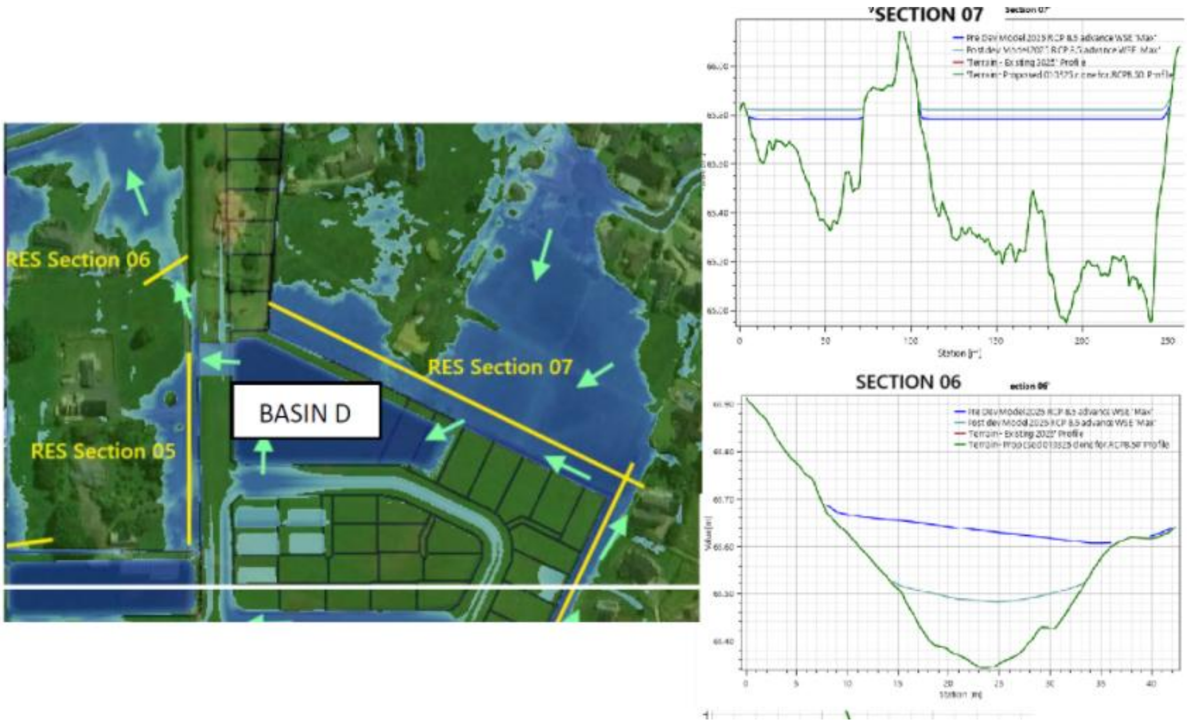


Figure 25: Basin D - RES Flood Map RCP 8.50 – Sensitivity

Basin D receives runoff from Catchment D and inflows from the northern and eastern areas upstream of Basin D, as referred to in the C410 catchment plan. As section 7.2 explains, this inflow is conveyed along the east boundary through the proposed convergence channel. Initially, the inflow will pond to the north of Basin D, similar to the pre-existing scenario. Over time, during the 24-hour event, it will eventually overtop and spill into Basin D. Once Basin D reaches its storage capacity, it will overflow westward onto the adjacent road corridor, before discharging into the neighbouring environment via the existing OLFP route.

The RES Section 07 has been placed to assess the development's impact on the neighbouring areas due to the rerouting of this inflow, as described above. The results indicate a slight increase in water depth ranging from 30 to 40 mm adjacent to the eastern boundary of Basin D, where it ponds before spilling into the Basin.

In contrast, RES Section 06 is located at the westward discharge point from the pond to assess the development's impact downstream. The results indicate a just over 100 mm reduction in post-development flood levels. This reduction underscores the effectiveness of the basin design in attenuating flows and, consequently, reducing flood risk in downstream areas.

9 SUMMARY OF PROPOSED STORMWATER MANAGEMENT

Stormwater management solutions for each post development sub-catchment is summarised in Table 20.

Table 20: Summary of stormwater management

<table>
<tr><th>Developments</th><th>Source</th><th>Water Quality</th><th>Flood management</th><th>Stream Protection</th><th>Water Sensitive Design</th></tr>
<tr>
<td rowspan="2">• Residential Sub catchment- A, B, C &D</td>
<td>• Lot Areas</td>
<td>• Inert roof materials.
• Soakage Trench.</td>
<td rowspan="2">• Stormwater pipe network and swales for 10yr ARI cc rainfall events.
• Roadside raingardens and soakage trenches for 10yr ARI cc rainfall events where soakage is viable.
• Catchment A, C & D; Overland flow paths for 100yr ARI cc events within road reserves, directed to detention and Soakage basins and Wetlands.
• Catchment B: Overland flow paths for 100yr ARI cc events within road reserves, directed to detention basins and Greenway.</td>
<td>• At source on lot soakage where soakage is viable.
• Full soakage/Mitigation of up to 10year cc event
• Extended detention within raingardens and artificial wetlands. Controlled release of 10-year and 100-year cc events at 80% pre-development flows.</td>
<td rowspan="2">• Stormwater detention basins, raingardens and wetlands with planting.
• 75% TSS as per requirements of GD01/TR2018/01 for all impervious surfaces.
• Protection of streams and riparian planting.</td>
</tr>
<tr>
<td>• Public Roads</td>
<td>• At source Raingardens, soakage trenches, stormwater basins and Wetlands
• 75% TSS as per requirements of GD01/ TR2018/01 for all impervious surfaces.</td>
<td>• Raingardens, soakage trench, dry basins and wetlands.</td>
</tr>
</table>

Retirement Village	Lot Areas	• Inert roof materials. • Raingardens and wetlands •	• Stormwater pipe network and swales for 10yr ARI cc rainfall events. • Overland flow paths for 100yr ARI cc events within road reserves, directed to wetlands 1 and 2. • Attenuation and flood storage up to 100yr ARI cc. Discharge of 10-year and 100-year ARI cc events at 80% pre-development flows.	Extended detention within raingardens and artificial wetlands. Controlled release of 10-year and 100-year cc events at 80% pre-development flows.	
	Public Roads	• **Centralised raingardens and artificial wetlands** 75% TSS as per requirements of GD01/ TR2018/01 for all impervious surfaces.			

Notes:

1. *On site refers to at source, e.g., adjacent to the road carriageway or within the lot boundary.*
2. *Catchment wide refers to outside of the road carriageway or lot boundary however within the Ashbourne Development Areas*

10 DESIGN OPTIMISATION

As part of the ongoing refinement of the stormwater management system, two key optimization strategies can be explored during the next stage of the project. These aim to maximize land availability further, improve hydraulic performance, and reduce the infrastructure footprint where appropriate:

- Reduction of Stormwater Pond Size via Controlled Discharge: The current post-development model assumes zero discharge from several catchments, which is conservative and beneficial from a flood mitigation perspective. This means the sizes of the proposed basins can be further optimized. To achieve this, we will investigate discharging post-development flows at up to 80% of the pre-development flow rate, which aligns with council standards and requirements. This approach balances flood control with land efficiency and has the potential to reduce pond size, thereby creating more developable land for additional units where possible.

- Incorporation of Base Flow Discharge to Existing Streams: To better mimic the natural hydrological conditions, base flow discharge will be reintroduced where feasible. This can be achieved by quantifying the existing base flow and assigning selected units to discharge directly into the stream at this defined rate and quantity. This strategy improves ecological outcomes and further reduces onsite storage demand, particularly for minor and frequent rainfall events.

These optimization works will be further assessed through refined modelling and consultation with the council and external consultants during the further design stage to ensure compliance with council requirements while maximizing development potential.

11 DEPARTURES FROM STANDARD

A departure from standard stormwater design has been identified in a few Overland Flow Path (OLFP) sections where the flow depth exceeds the recommended threshold of 150 mm according to RITS. These exceedances are minor and occur in isolated sections of the residential OLFP network.

Sections exceeding the 150mm limit.

Residential

- Catchment A – Section 2: Max Depth = 0.158
- Catchment B – Section 1: Max Depth = 0.167
- Catchment B – Section 4: Max Depth = 0.216

Retirement Village

- Sections I – N: Highest Max depth is 0.21m and Lowest Max Depth: 0.16m

These areas remain safely contained within their respective flow corridors and do not pose a risk to vehicles and pedestrians as VD values remain well below the threshold. Further refinement of levels, swale profiles, or berms at the detailed design stage is expected to reduce these values and bring all OLFP sections fully into compliance.

12 CONCLUSIONS

This Stormwater Management Plan (SMP) has been prepared to support a discharge consent application for the proposed Ashbourne development, which comprises four key projects:

- the Residential Development
- Northern and Southern Solar Farms
- the Retirement Village.

Each site has been considered in detail through hydrological and hydraulic modelling, including sensitivity scenarios under future climate conditions.

The proposed stormwater management system for the residential development (with the exception of catchments C and D) has been designed to provide:

- full soakage and treatment for up to the 10- year ARI cc storm event through a combination of on- lot soakage devices, raingardens, and road soakage systems.
- Conveyance of overflows above the 10- year cc event into dry detention basins and a central greenway corridor sized for up to the 100- year cc storm event. Limit post- development peak discharges to no more than 80% of pre- development flows, where applicable.
- Preserve existing flow paths and discharge locations to avoid introducing new hydrological impacts.
- Integrate water- sensitive design elements, including green corridors, baseflow retention, and modular soakage systems such as Cirtex Rain Smart
- Account for external inflows and upstream catchments to ensure downstream impacts are not worsened under post- development conditions
- Support future maintenance, accessibility, and ecological enhancement through features such as maintenance tracks and riparian planting zones.

The proposed stormwater management system for the retirement village and the residential development catchments C and D has been designed to provide:

- Primary conveyance of up to the 10-year ARI cc storm event via catchpits, lot connections into a stormwater pipe network and swales.
- Secondary conveyance of overflows up to the 100-year ARI cc storm event via road networks and swales
- Water quality treatment and extended detention via centralised raingardens and artificial wetlands.
- Artificial wetlands to provide flood attenuation and storage and release at 80% of 10- year and 100-year ARI cc storm pre-development flows.
- Preserve existing flow paths and discharge locations to avoid introducing new hydrological impacts.
- Integrate water- sensitive design elements, including green raingardens and artificial wetland areas.
- Support future maintenance, accessibility, and ecological enhancement through features such as maintenance tracks and riparian planting zones.

A flood model has been developed and calibrated using the region's observed rainfall and river data. The model has been used to test a sensitivity scenario in which all primary stormwater devices are blocked. Even under this worst-case condition, the model demonstrates that the development maintains flood immunity, with only minor exceedances expected to be mitigated through detailed design refinements.

This SMP aligns with the requirements of both the Matamata- Piako District Council and Waikato Regional Council and has been developed in accordance with relevant planning instruments, including MPDC Section 5 Infrastructure Standards and the Waikato Comprehensive Network Discharge Consent (#105063).

Several design aspects of the proposed Stormwater will require further refinement during detailed design stage to ensure compliance and for further value engineering covered under design optimisation section of this SMP.

In conclusion, the proposed stormwater management approach is robust, resilient, and environmentally sensitive. It provides a strong foundation for the ongoing development of the Ashbourne site and offers a balanced integration of flood mitigation, water quality treatment, ecological protection, and long-term sustainability.

13 APPENDICES

APPENDIX A – ENGINEERING PLANS - BOUNDED WITH INFRASTRUCTURE REPORTS

APPENDIX B – STORMWATER CALCULATIONS AND RESULTS

	Maven Associates Ltd.	Job Number 289001	Sheet 1	Rev A
Job Title Calc Title	Station Road, Matamata Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B	Open Space (Sandy Loam or Silty Loam)	61	10.10	616.10
* from Appendix B		Totals =	10.10	616.10

WQV

CN (weighted) = total product / total area = 616.10 / 10.10 = 61.0

Ia (weighted) = 5 x pervious area / total area = 5 x 10.10 / 10.10 = 5.0 mm

2. Time of Concentration

Channelisation factor C = 1 (From Table 4.2) natural channels

Catchment length L = 0.39 km (along drainage path)

Catchment Slope Sc= 0.005 m/m (by equal area method)

Runoff factor, CN / (200 - CN) = 61.0 / (200- 61.0) = 0.44

$t_c = 0.14\ C\ L^{0.66}\ (CN/200-CN)^{-0.55}\ Sc^{-0.30}$

= 0 1 0.54 1.57 4.90 = 0.580 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.388 hrs

OK
use
0.39 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd.	Job Number 289001	Sheet 2	Rev A
Job Title / Calc Title — Station Road, Matamata / Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

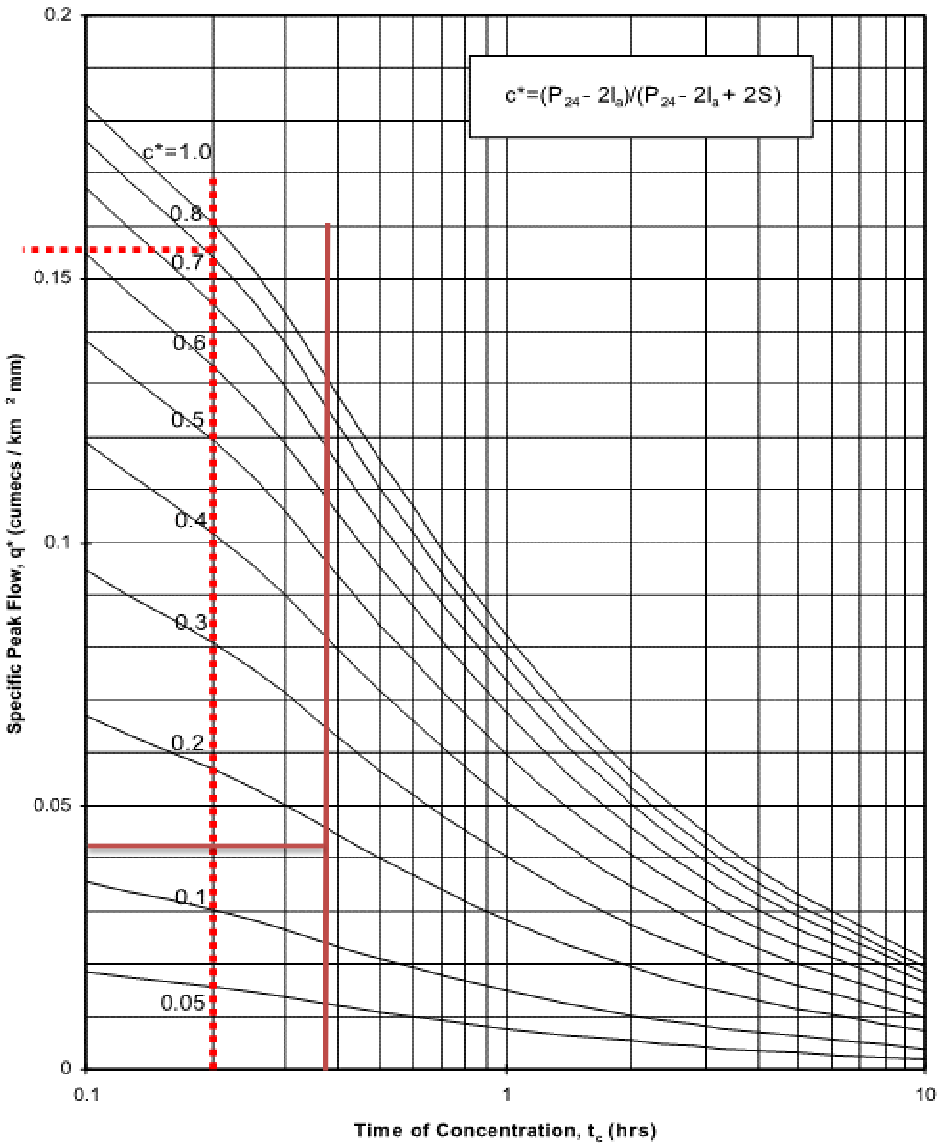
Catchment Area	A=	0.10100	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.39	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

3. Average recurrence interval, ARI	2	(yr)
4. 24 hour rainfall depth, P24	83	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.184	
6. Specific peak flow rate q*	0.042	
7. Peak flow rate, q_p=q*A*P_{24}	**0.352**	(m3/s)
8. Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	25.3	
9. Runoff volume, V_{24} = 1000xQ_{24}A	2556.16	(m3)

NO CLIMATE CHANGE

Worksheet 2: Graphical Peak Flow Rate


c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 3	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data
   Catchment Area A= 0.10100 km2(100ha =1km2)

   Runoff curve number CN= 61.0 (from worksheet 1)

   Initial abstraction Ia= 5.0 mm (from worksheet 1)

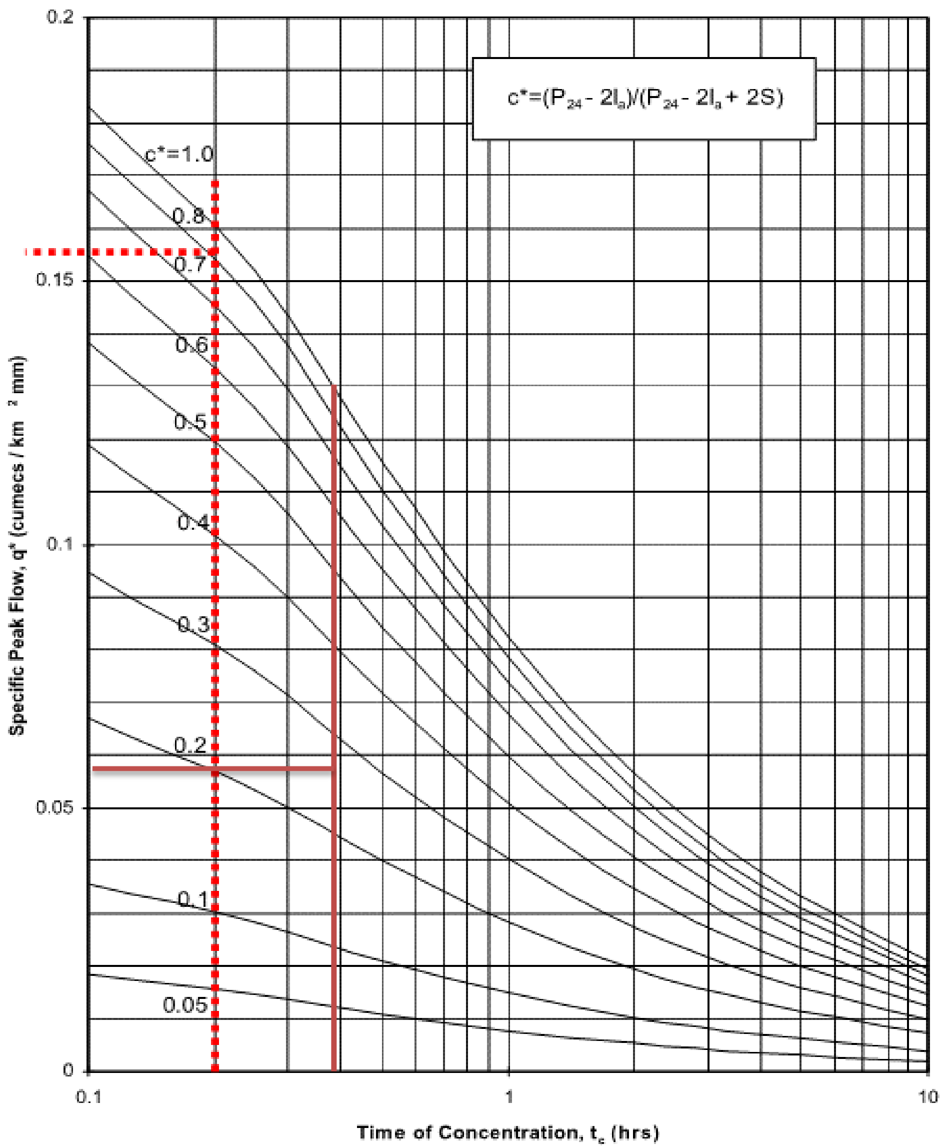
   Time of concentration tc= 0.39 hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3.	Average recurrence interval, ARI	10	(yr)
4.	24 hour rainfall depth, P24	128	(mm)
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.266	
6.	Specific peak flow rate q*	0.058	
7.	Peak flow rate, q_p=q*A*P_{24}	**0.750**	(m3/s)
8.	Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	53.0	
9.	Runoff volume, V_{24} = 1000xQ_{24}A	5354.11	(m3)

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd.		Job Number 289001	Sheet 4	Rev A
Job Title Calc Title	Station Road, Matamata Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data
   Catchment Area A= 0.10100 km2(100ha =1km2)

   Runoff curve number CN= 61.0 (from worksheet 1)

   Initial abstraction Ia= 5.0 mm (from worksheet 1)

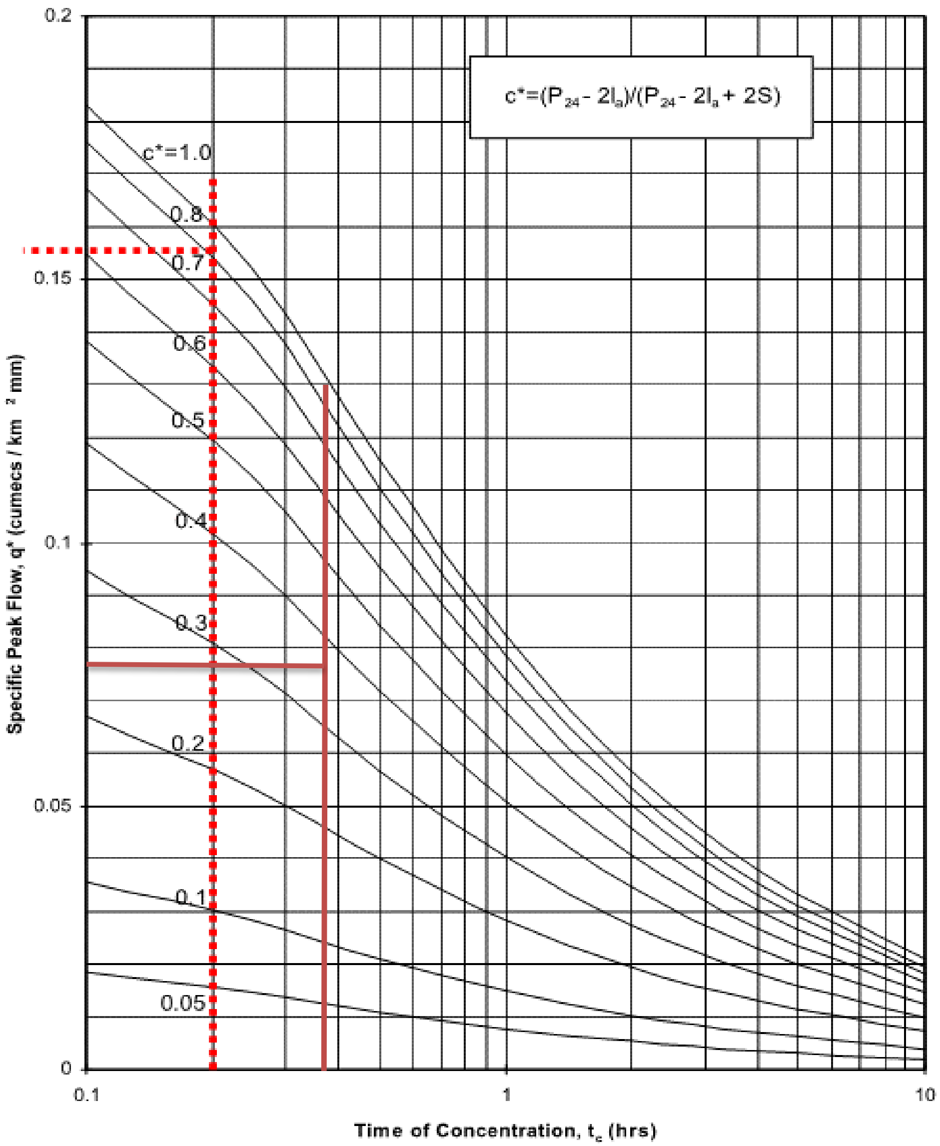
   Time of concentration tc= 0.39 hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	100	(yr)
4. 24 hour rainfall depth, P24	200	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.369	
6. Specific peak flow rate q*	0.078	
7. Peak flow rate, q_p=q*A*P_{24}	**1.576**	(m3/s)
8. Runoff depth, Q_{24} = $(P_{24}$-Ia$)^2$/(P_{24}-Ia)+S	106.4	
9. Runoff volume, V_{24} = 1000xQ_{24}A	10745.93	(m3)

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km 2 mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, tc (hrs)

Maven Associates Ltd.		Job Number 289001	Sheet 5	Rev A
Job Title Calc Title	Eldonwood, Matamata Post-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road	98	2.76	270.5
C	Residential District (30% PERVIOUS)	74	2.2	163.5
C	Residential District (70% IMPERVIOUS)	90	5.13	461.7
* from Appendix B		Totals =	10.10	895.7

CN (weighted) = total product = / total area — 895.72 / 10.100 = 88.7

Ia (weighted) = 5 x pervious area = / total area — 5 x 2.21 / 10.10 — 1.1 mm

2. Time of Concentration

Channelisation factor C = 0.6 (From Table 4.2) piped

Catchment length L = 0.39 km (along drainage path)

Catchment Slope Sc= 0.01 m/m (by equal area method)

Runoff factor, CN / 200 - CN = 88.7 / 200- 88.7 = 0.80

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0.1 0.6 0.54 1.13 3.98 = 0.204 hrs

SCS Lag for HEC-HMS…. $t_p = 2/3\ t_c$ = 0.136 hrs

NO GOOD
use
0.170 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

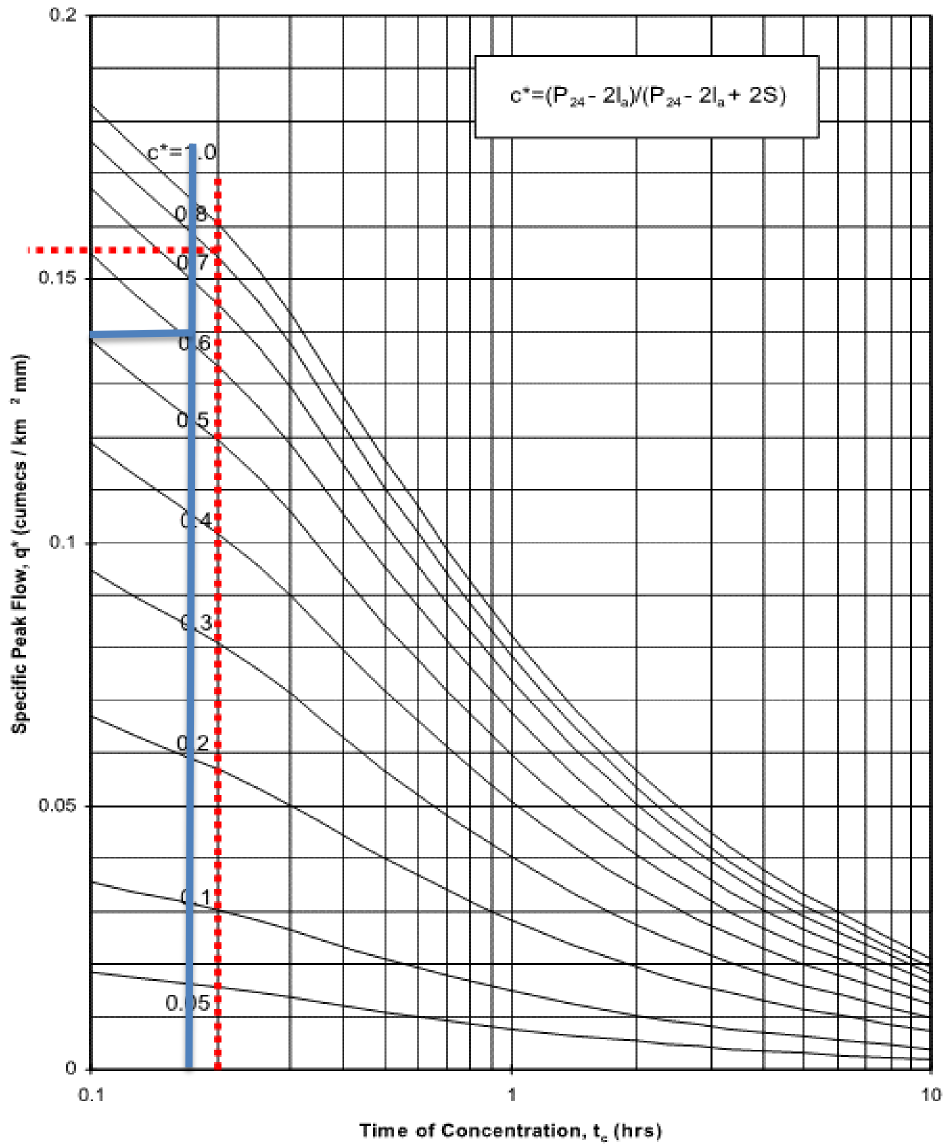
Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd	Job Number 289001	Sheet 6	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1.	Data			
	Catchment Area	A=	0.101	km2(100ha =1km2)
	Runoff curve number	CN=	88.7	(from worksheet 1)
	Initial abstraction	Ia=	1.1	mm (from worksheet 1)
	Time of concentration	tc=	0.170	hrs (from worksheet 1)
2.	Calculate storage, S =(1000/CN - 10)25.4	=	32	mm

3.	Average recurrence interval, ARI	2	(yr)	
4.	24 hour rainfall depth, P24	106	(mm)	
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.616		
6.	Specific peak flow rate q*	0.140		***HEC-HMS Check***
7.	Peak flow rate, q_p=q*A*P_{24}	**1.499**		Pre-Dev
8.	Runoff depth, $Q_{24} = (P_{24}\text{-}Ia)^2/(P_{24}\text{-}Ia)+S$	80.1		
9.	Runoff volume, V_{24} = 1000xQ_{24}A	8094.91	(m3)	
	Pre development run off volume	**2556.16**	(m3)	
	Post development run off volume	**8094.91**	(m3)	
	Pre development flow rate	**0.35**	(m3/s)	
	Post development flow rate	**1.50**	(m3/s)	

Worksheet 2: Graphical Peak Flow Rate


c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd	Job Number 289001	Sheet 7	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.101	km2(100ha =1km2)
Runoff curve number	CN=	88.7	(from worksheet 1)
Initial abstraction	Ia=	1.1	mm (from worksheet 1)
Time of concentration	tc=	0.170	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 32 mm

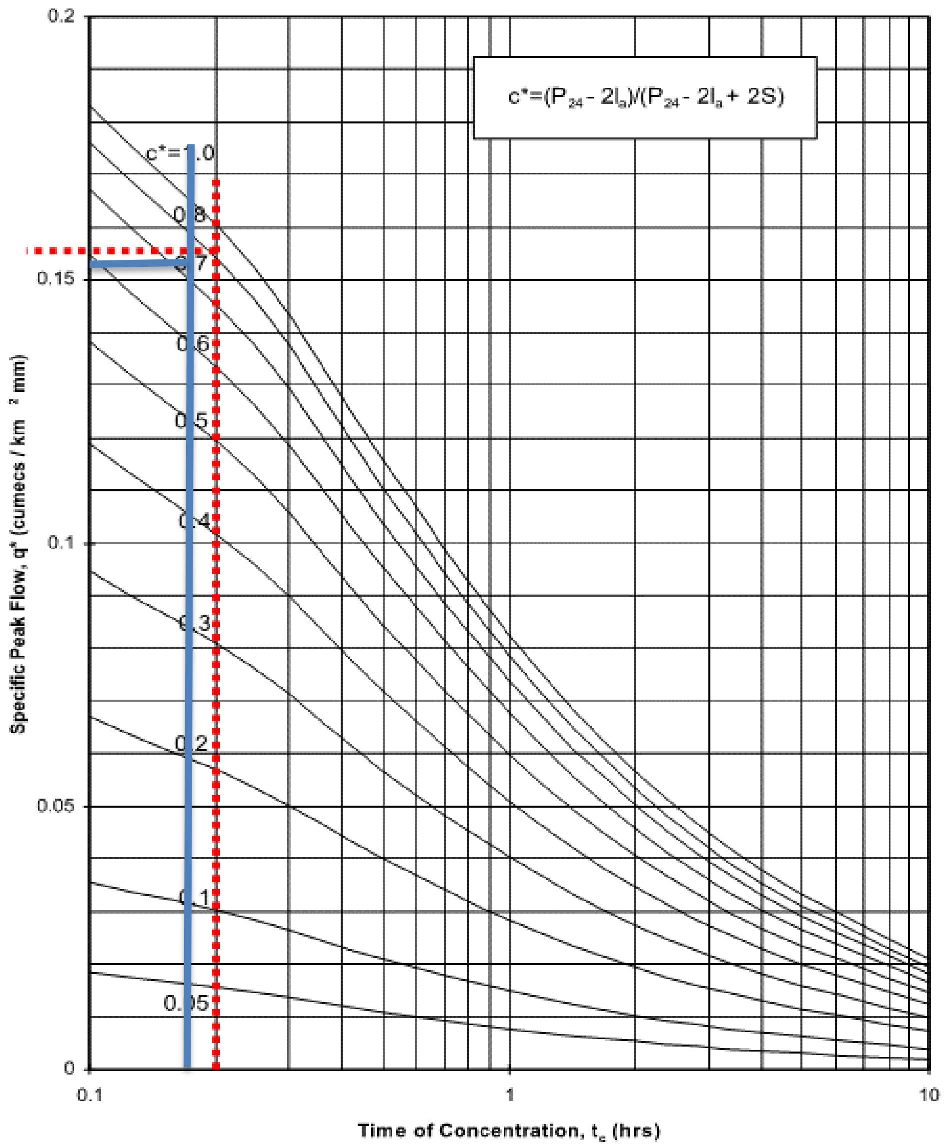
3.	Average recurrence interval, ARI	10	(yr)
4.	24 hour rainfall depth, P24	167	(mm)
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.718	
6.	Specific peak flow rate q*	0.153	
7.	Peak flow rate, q_p=q*A*P_{24}	**2.581**	
8.	Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})$+S	138.8	
9.	Runoff volume, V_{24} = 1000xQ_{24}A	14018.30	(m3)

HEC-HMS Check

Pre-Dev

Pre development run off volume	**5354.11**	(m3)
Post development run off volume	**14018.30**	(m3)
Pre development flow rate	**0.75**	(m3/s)
Post development flow rate	**2.58**	(m3/s)

Worksheet 2: Graphical Peak Flow Rate


c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd	Job Number 289001	Sheet 8	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

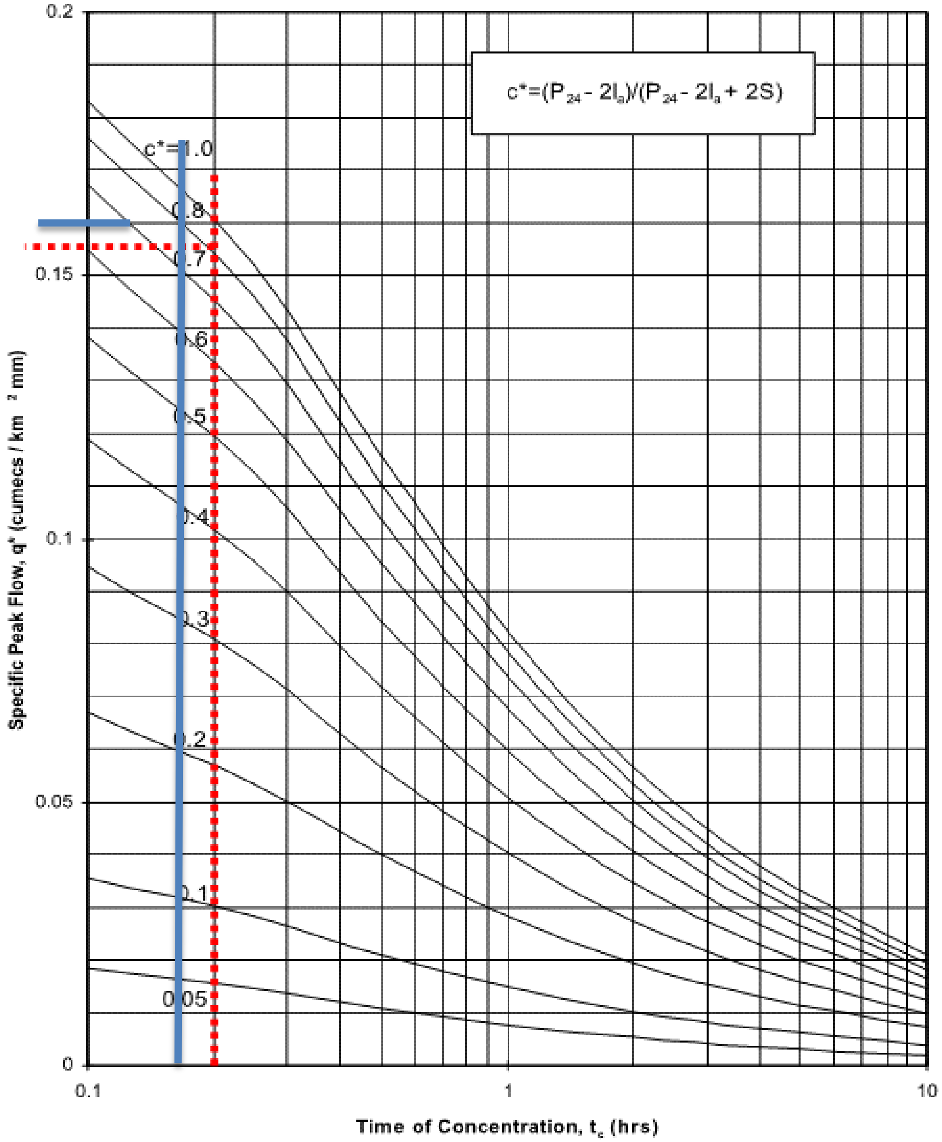
1. Data

Catchment Area	A=	0.101	km2(100ha =1km2)
Runoff curve number	CN=	88.7	(from worksheet 1)
Initial abstraction	Ia=	1.1	mm (from worksheet 1)
Time of concentration	tc=	0.170	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 32 mm

				HEC-HMS Check	
3.	Average recurrence interval, ARI	100	(yr)		
4.	24 hour rainfall depth, P24	265	(mm)		
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.802			
6.	Specific peak flow rate q*	0.160			
7.	Peak flow rate, q_p=q*A*P_{24}	**4.282**		**1.260**	80% Pre
8.	Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	235.0			
9.	Runoff volume, V_{24} = 1000xQ_{24}A	23739.41	(m3)		
	Pre development run off volume	**10745.93**	(m3)		
	Post development run off volume	**23739.41**	(m3)		
	Pre development flow rate	**1.58**	(m3/s)		
	Post development flow rate	**4.28**	(m3/s)		
	100yr - 10yr post development	**9721.11**	(m3)		

Worksheet 2: Graphical Peak Flow Rate



0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, t_c (hrs)
c*=(P_{24} - 2I_a)/(P_{24} - 2I_a + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05

Maven Associates Ltd.	Job Number 289001	Sheet 1	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B	Open Space (Sandy Loam or Silty Loam)	61	19.50	1189.50
* from Appendix B		Totals =	19.50	1189.50

WQV

CN (weighted) = total product / total area = 1189.50 / 19.50 = 61.0

Ia (weighted) = 5 x pervious area / total area = 5 x 19.50 / 19.50 = 5.0 mm

2. Time of Concentration

Channelisation factor C = 1 (From Table 4.2) natural channels

Catchment length L = 0.6 km (along drainage path)

Catchment Slope Sc= 0.005 m/m (by equal area method)

Runoff factor, CN / 200 - CN = 61.0 / 200- 61.0 = 0.44

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0 1 0.71 1.57 4.90 = 0.770 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.516 hrs

OK
use
0.52 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd.		Job Number 289001	Sheet 2	Rev A
Job Title Calc Title	Station Road, Matamata Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data
   Catchment Area A= 0.19500 km2(100ha =1km2)

   Runoff curve number CN= 61.0 (from worksheet 1)

   Initial abstraction Ia= 5.0 mm (from worksheet 1)

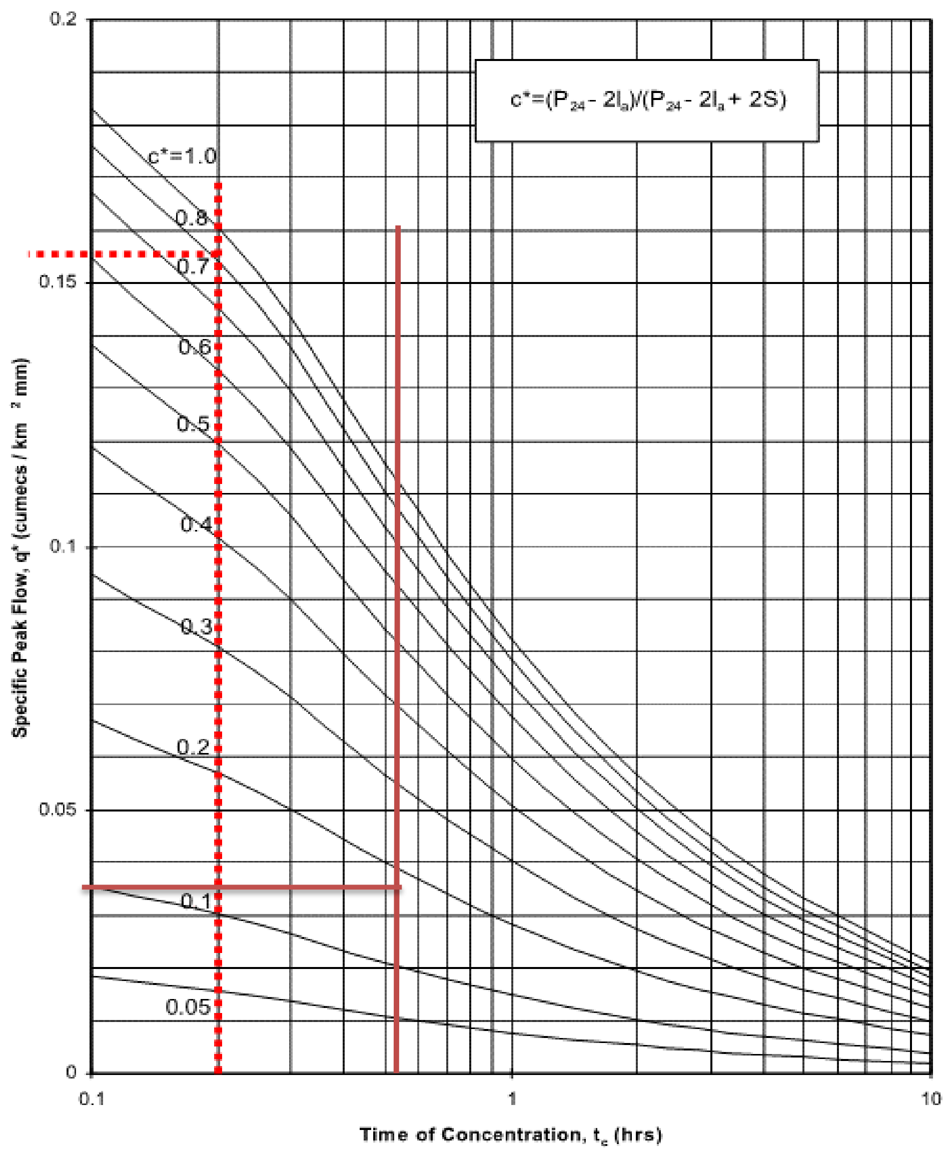
   Time of concentration tc= 0.52 hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3.	Average recurrence interval, ARI	2	(yr)
4.	24 hour rainfall depth, P24	83	(mm)
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.184	
6.	Specific peak flow rate q*	0.036	
7.	Peak flow rate, q_p=q*A*P_{24}	**0.583**	(m3/s)
8.	Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	25.3	
9.	Runoff volume, V_{24} = 1000xQ_{24}A	4935.16	(m3)

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km 2 mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, tc (hrs)

Maven Associates Ltd.		Job Number 289001	Sheet 3	Rev A
Job Title Calc Title	Station Road, Matamata Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

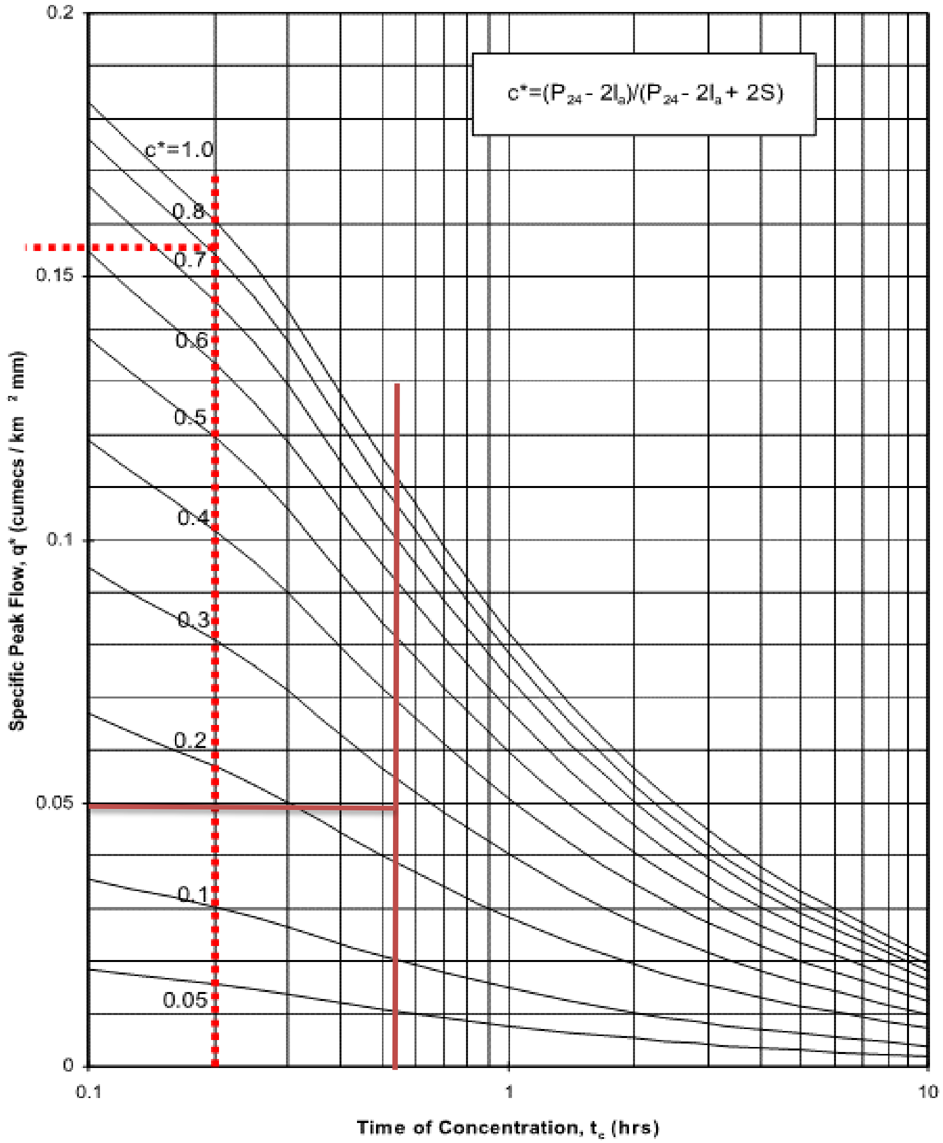
Catchment Area	A=	0.19500	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.52	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

3. Average recurrence interval, ARI	10	(yr)
4. 24 hour rainfall depth, P24	128	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.266	
6. Specific peak flow rate q*	0.049	
7. Peak flow rate, q_p=q*A*P_{24}	**1.223**	(m3/s)
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	53.0	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	10337.15	(m3)

NO CLIMATE CHANGE

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
Specific Peak Flow, q* (cumecs / km 2 mm)
0.1
1
10
Time of Concentration, t_c (hrs)

Maven Associates Ltd.		Job Number 289001	Sheet 4	Rev A
Job Title Calc Title	Station Road, Matamata Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

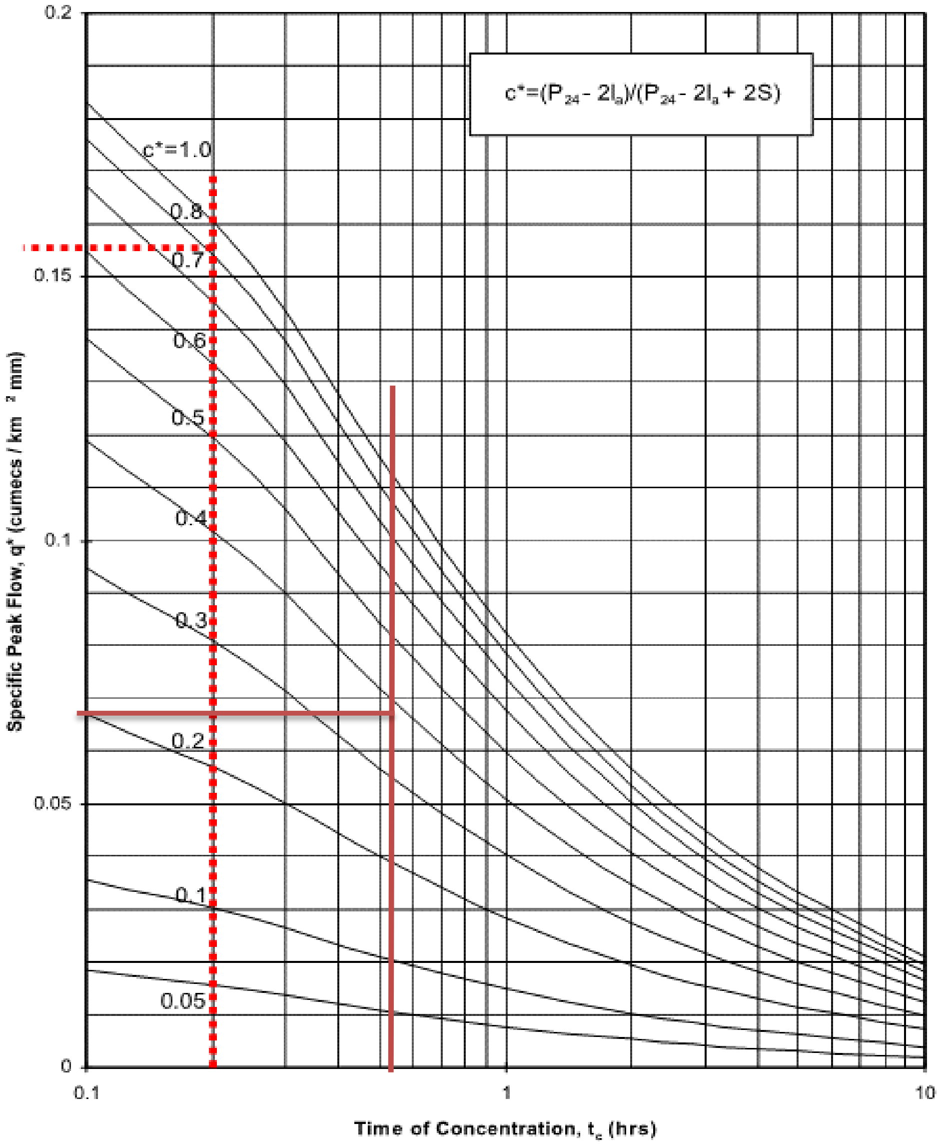
Catchment Area	A=	0.19500	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.52	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	100	(yr)
4. 24 hour rainfall depth, P24	200	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.369	
6. Specific peak flow rate q*	0.068	
7. Peak flow rate, q_p=q*A*P_{24}	**2.652**	(m3/s)
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	106.4	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	20747.09	(m3)

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km 2 mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, tc (hrs)

Maven Associates Ltd.		Job Number 289001	Sheet 5	Rev A
Job Title Calc Title	Eldonwood, Matamata Post-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road	98	5.96	583.6
C	Residential District (30% PERVIOUS)	74	4.1	300.8
C	Residential District (70% IMPERVIOUS)	90	9.48	853.6
* from Appendix B		Totals =	19.50	1738.0

CN (weighted) = total product / total area = 1738.02 / 19.505 = 89.1

Ia (weighted) = 5 x pervious area / total area = 5 x 4.06 / 19.50 = 1.0 mm

2. Time of Concentration

Channelisation factor C = 0.6 (From Table 4.2) piped

Catchment length L = 0.6 km (along drainage path)

Catchment Slope Sc= 0.01 m/m (by equal area method)

Runoff factor, CN / 200 - CN = 89.1 / 200- 89.1 = 0.80

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0.1 0.6 0.71 1.13 3.98 = 0.269 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.180 hrs

OK
use
0.180 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd	Job Number 289001	Sheet 6	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.195	km2(100ha =1km2)
Runoff curve number	CN=	89.1	(from worksheet 1)
Initial abstraction	Ia=	1.0	mm (from worksheet 1)
Time of concentration	tc=	0.180	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 31 mm

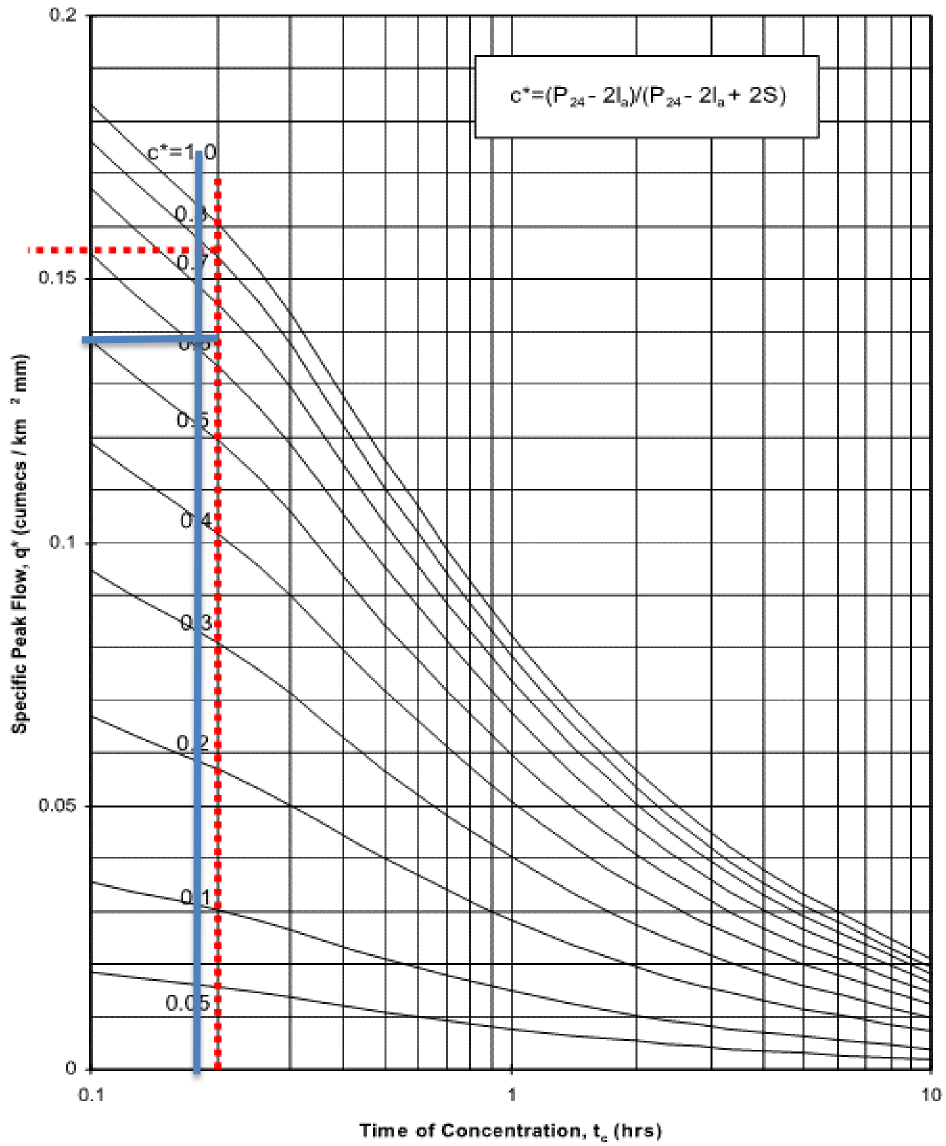
3.	Average recurrence interval, ARI	2	(yr)
4.	24 hour rainfall depth, P24	106	(mm)
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.626	
6.	Specific peak flow rate q*	0.139	
7.	Peak flow rate, q_p=q*A*P_{24}	**2.874**	
8.	Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	81.0	
9.	Runoff volume, V_{24} = 1000xQ_{24}A	15798.35	(m3)

HEC-HMS Check

Pre-Dev

Pre development run off volume	**4935.16**	(m3)
Post development run off volume	**15798.35**	(m3)
Pre development flow rate	**0.58**	(m3/s)
Post development flow rate	**2.87**	(m3/s)

Worksheet 2: Graphical Peak Flow Rate


c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.7
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km 2 mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, tc (hrs)

Maven Associates Ltd	Job Number 289001	Sheet 7	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

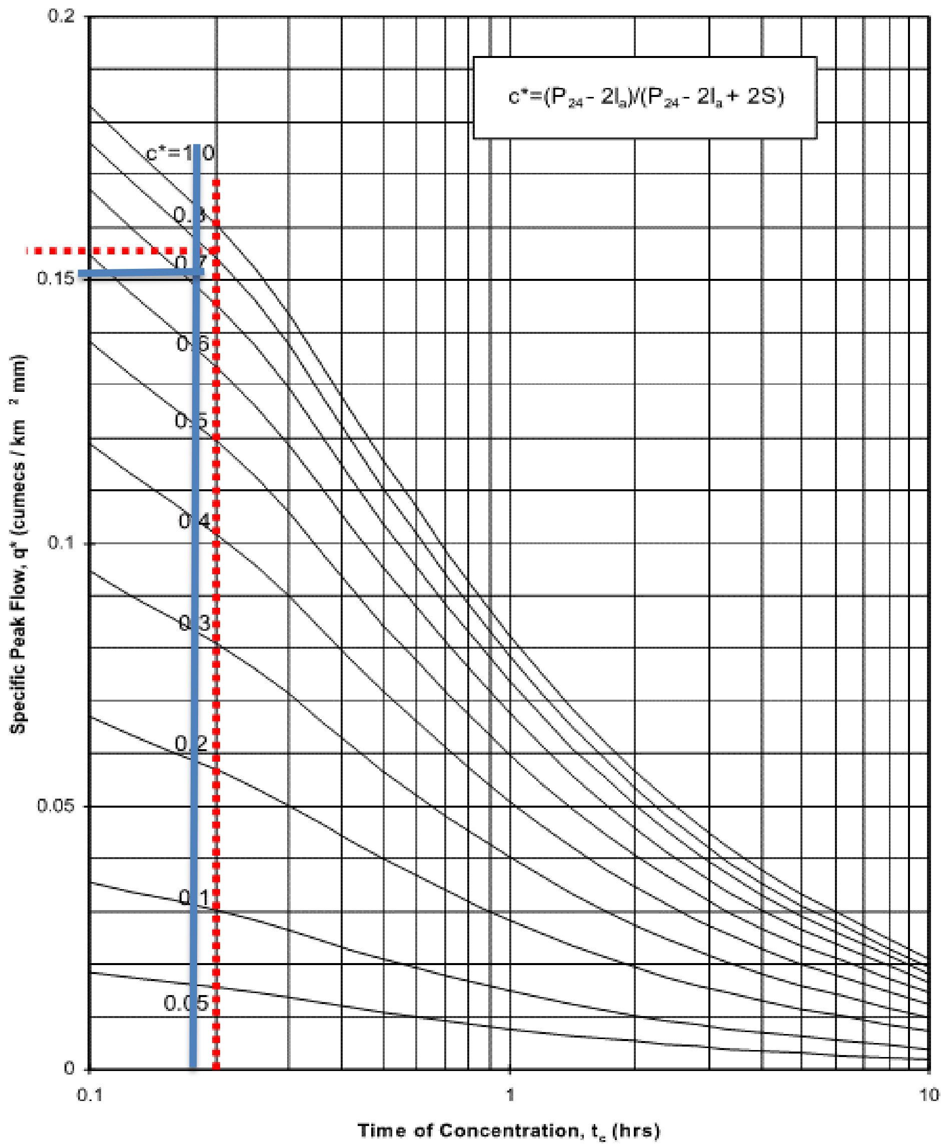
1. Data

Catchment Area	A=	0.195	km2(100ha =1km2)
Runoff curve number	CN=	89.1	(from worksheet 1)
Initial abstraction	Ia=	1.0	mm (from worksheet 1)
Time of concentration	tc=	0.180	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 31 mm

				HEC-HMS Check
3.	Average recurrence interval, ARI	10	(yr)	
4.	24 hour rainfall depth, P24	167	(mm)	
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.726		
6.	Specific peak flow rate q*	0.151		
7.	Peak flow rate, q_p=q*A*P_{24}	**4.918**		Pre-Dev
8.	Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	139.8		
9.	Runoff volume, $V_{24} = 1000xQ_{24}A$	27268.12	(m3)	
	Pre development run off volume	**10337.15**	(m3)	
	Post development run off volume	**27268.12**	(m3)	
	Pre development flow rate	**1.22**	(m3/s)	
	Post development flow rate	**4.92**	(m3/s)	

Worksheet 2: Graphical Peak Flow Rate


c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd	**Job Number** 289001	**Sheet** 8	**Rev** A
Job Title Station Road, Matamata **Calc Title** Post-development SW Demand	**Author** MKS	**Date** 13/03/2025	**Checked** DJM

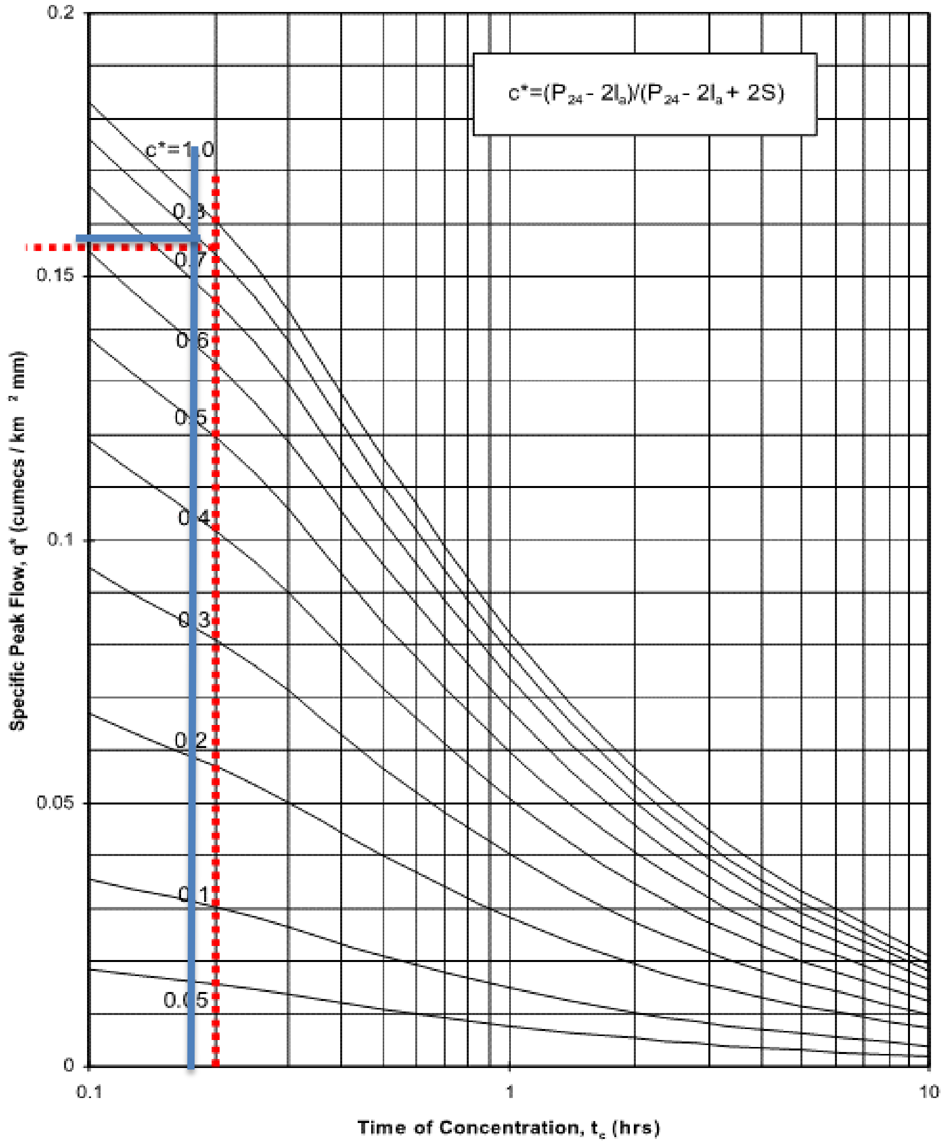
1. Data

Catchment Area	A=	0.195	km2(100ha =1km2)
Runoff curve number	CN=	89.1	(from worksheet 1)
Initial abstraction	Ia=	1.0	mm (from worksheet 1)
Time of concentration	tc=	0.180	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 31 mm

3.	Average recurrence interval, ARI	100	(yr)		
4.	24 hour rainfall depth, P24	265	(mm)		
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.809			
6.	Specific peak flow rate q*	0.158		*HEC-HMS Check*	
7.	Peak flow rate, q_p=q*A*P_{24}	**8.167**		**2.122**	80% Pre
8.	Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	236.2			
9.	Runoff volume, V_{24} = 1000xQ_{24}A	46065.59	(m3)		
	Pre development run off volume	**20747.09**	(m3)		
	Post development run off volume	**46065.59**	(m3)		
	Pre development flow rate	**2.65**	(m3/s)		
	Post development flow rate	**8.17**	(m3/s)		
	100yr - 10yr post development	**18797.47**	(m3)		

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24} - 2I_a)/(P_{24} - 2I_a + 2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km² mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, t_c (hrs)

289001 – Ashbourne Soakage Calculation

1. The total effective area (per metre length of road)

= **Impervious Area + (0.3 * Pervious Area)**

Based on MPDC definition, we can calculate 2 effective areas for our design.

a. 20m wide road = 16.3 + (0.3*3.7) = **17.41m²**
b. 18m wide road = 14.6 + (0.3*3.4) = **15.62m²**

2. Design Soakage Rate

DSR of 4 Soakage holes from CMW Geotech report were calculated which are:
SO-01: 1.7 litres/min/m²
SO-02: 5.7 litres/min/m²
SO-03: 0.5 litres/min/m²
SO-04: 0.2 litres/min/m² -> But we go with 0.5 litres/min/m²

->Considering Test Pit Results, only 1.7 and 5.7 litres/min/m² will be used.

3. Maximum Impervious Area per Soakage Trench

There is no graph available for 10y 24h soakage trench with 1.2m width and 2.0m width, so we need a fair assumption for Maximum Impervious Area per soakage trench.

For 1.7 Design Soakage Rate (litres/min/m²)

0.4m wide	1m deep = 6m²
	2m deep = 12m²
0.8m wide	1m deep = 8m²
	2m deep = 16m²
1.2m wide	1m deep = 10.6m² (33.3% Increased per 0.4m)
	1.5m deep = 16m² (33.3% Increased per 0.4m)
	1.8m deep = 19.1m² (33.3% Increased per 0.4m)
	2m deep = 21.3m² (33.3% Increased per 0.4m)
2.0m wide	1m deep = 18.9m² (33.3% Increased per 0.4m)
	1.5m deep = 28.4m² (33.3% Increased per 0.4m)
	2m deep = 37.8m² (33.3% Increased per 0.4m)

For 5.7 Design Soakage Rate (litres/min/m²)

0.4m wide	1m deep = $9m^2$
	2m deep = $18m^2$
0.8m wide	1m deep = $13m^2$
	2m deep = $26m^2$
1.2m wide	1m deep = $18.7m^2$ (44.4% Increased per 0.4m)
	1.5m deep = $28.2m^2$ (44.4% Increased per 0.4m)
	2m deep = $37.5m^2$ (44.4% Increased per 0.4m)
2.0m wide	1m deep = $39m^2$ (44.4% Increased per 0.4m)
	1.5m deep = $58.7m^2$ (44.4% Increased per 0.4m)
	2m deep = $78.2m^2$ (44.4% Increased per 0.4m)

4. Comparison between Road and Soakage Trench

1.7 Design Soakage Rate (litres/min/m²) Zone:

2.0m wide and 1.0m deep soakage trench is recommended for 18m wide road, this will give us:

- 15.62 / 18.9 = 0.83 -> 1:0.83 ratio (18m wide road)

-> Every 1m long road construction, we need 0.83m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.415m long soakage trench in each side.

2.0m wide and 1.5m deep soakage trench is recommended for 18m wide road, this will give us:

- 15.62 / 28.4 = 0.55 -> 1:0.55 ratio (18m wide road)

-> Every 1m long road construction, we need 0.55m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.28m long soakage trench in each side.

1.2m wide and 1.5m deep soakage trench is recommended for 20m wide road, this will give us:

- 17.41 / 16 = 1.1 -> 1:1.1 ratio (20m wide road)

-> Every 1m long road construction, we need 1.1m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.55m long soakage trench in each side.

1.2m wide and 1.8m deep soakage trench is recommended for 20m wide road, this will give us:

- 17.41 / 19.1 = 0.91 -> 1:0.91 ratio (20m wide road)

-> Every 1m long road construction, we need 0.91m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.455m long soakage trench in each side.

5.7 Design Soakage Rate (litres/min/m²) Zone:

2.0m wide and 1.5m deep soakage trench is recommended for 18m wide road, this will give us:

- 15.62 / 58.7 = 0.27 -> 1:0.27 ratio (18m wide road)

-> Every 1m long road construction, we need 0.27m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.135m long soakage trench in each side.

1.2m wide and 1.5m deep soakage trench is recommended for 20m wide road, this will give us:

- 17.41 / 28.2 = 0.62 -> 1:0.62 ratio (20m wide road)

-> Every 1m long road construction, we need 0.62m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.31m long soakage trench in each side.

289001 – Ashbourne Soakage Calculation (E1/VM1)

1. Soakage Rate from the Percolation Test

- ➔ Soakage Rate 0.5l/min/m² -> 21mm/hr from CMW Geotech Report HAM2023-0124AB Rev1
- ➔ Soakage Rate 1.5l/min/m² -> 85mm/hr (half of 171mm/hr)
- ➔ Soakage Rate 3.0l/min/m² -> 171mm/hr from CMW Geotech Report HAM2023-0124AB Rev1

2. Stormwater Catchment Volume (Rc)

Formula: Rc = 10 * CIA

1. Assume: Impervious area: 150m²
2. Assume: Impervious area: 200m²
3. Assume: Impervious area: 250m

Run-off coefficient (C) = 0.9 from E1/VM1 Table 1 (Hard Surfaces)

Rain Intensity (I) = 48.8mm/hour (10-year, RCP 8.5)

1. Rc (10-year) = (10 * 0.9 * 48.8 * 0.015) = 6.588m³
2. Rc (10-year) = (10 * 0.9 * 48.8 * 0.02) = 8.784m³
3. Rc (10-year) = (10 * 0.9 * 48.8 * 0.025) = 10.98m³

3. Volume of Water Disposed by Soakage: V(soak)

Formula: V(soak) = Soakage Trench Area * Soakage Rate / 1000

Impervious area (150m²)

0.5 Soakage Rate (21mm/hr): Soakage Trench Size = 7m (L) * 2.44m (W) * 1m (H)

Soakage Trench Volume =

7 * 2.44 * 1.0 * 0.38 (void ratio) = 6.49m³

This gives V(soak): 7 * 2.44 * 21 /1000 = 0.36m³

1.5 Soakage Rate (85mm/hr): Soakage Trench Size = 6m (L) * 2.44m (W) * 1m (H)

Soakage Trench Volume =

6 * 2.44 * 1.0 * 0.38 (void ratio) = 5.56m³

This gives V(soak): 6 * 2.44 * 85 /1000 = 1.24m³

3.0 Soakage Rate (171mm/hr): Soakage Trench Size = 5m (L) * 2.44m (W) * 1m (H)

Soakage Trench Volume =

5 * 2.44 * 1.0 * 0.38 (void ratio) = 4.64m^3

This gives V(soak): 5 * 2.44 * 171 /1000 = 2.086m^3

Impervious area (200m^2)

0.5 Soakage Rate (21mm/hr): Soakage Trench Size = 13.5m (L) * 1.65m (W) * 1m (H)

Soakage Trench Volume =

13.5 * 1.65 * 1.0 * 0.38 (void ratio) = 8.46m^3

This gives V(soak): 13.5 * 1.65 * 21 /1000 = 0.47m^3

1.5 Soakage Rate (85mm/hr): Soakage Trench Size = 11.5m (L) * 1.65m (W) * 1m (H)

Soakage Trench Volume =

11.5 * 1.65 * 1.0 * 0.38 (void ratio) = 7.21m^3

This gives V(soak): 11.5 * 1.65 * 85 /1000 = 1.613m^3

3.0 Soakage Rate (171mm/hr): Soakage Trench Size = 10m (L) * 1.65m (W) * 1m (H)

Soakage Trench Volume =

10 * 1.65 * 1.0 * 0.38 (void ratio) = 6.27m^3

This gives V(soak): 10 * 1.65 * 171 /1000 = 2.82m^3

Impervious area (250m^2)

0.5 Soakage Rate (21mm/hr): Soakage Trench Size = 11.74m (L) * 1.6m (W) * 1.5m (H)

Soakage Trench Volume =

11.74 * 1.6 * 1.5 * 0.38 (void ratio) = 10.71m^3

This gives V(soak): 11.74 * 1.6 * 21 /1000 = 0.39m^3

1.5 Soakage Rate (85mm/hr): Soakage Trench Size = 11.74m (L) * 1.6m (W) * 1.4m (H)

Soakage Trench Volume =

11.74 * 1.6 * 1.4 * 0.38 (void ratio) = 9.99m^3

This gives V(soak): 11.74 * 1.6 * 85 /1000 = 1.60m^3

3.0 Soakage Rate (171mm/hr): Soakage Trench Size = 11.74m (L) * 1.6m (W) * 1.1m (H)

Soakage Trench Volume =

11.74 * 1.6 * 1.1 * 0.38 (void ratio) = 7.85m^3

This gives V(soak): 11.74 * 1.6 * 171 /1000 = 3.21m^3

4. Required Storage Volume: V(storage)

Formula: V(storage) = Rc – V(soak)

Impervious area (150m^2)

1. 21mm/hr
- ➔ Required 10-year V(storage) = 6.588 – 0.36 = 6.23m^3
- ➔ Proposed Soakage Trench Volume (6.49m^3) is larger than Required Volume (6.23m^3),
- ➔ Soakage trench sizing is OK

2. 85mm/hr
- ➔ Required 10-year V(storage) = 6.588 – 1.24 = 5.35m^3
- ➔ Proposed Soakage Trench Volume (5.56m^3) is larger than Required Volume (5.35m^3)
- ➔ Soakage trench sizing is OK

3. 171mm/hr
- ➔ Required 10-year V(storage) = 6.588 – 2.086 = 4.5m^3
- ➔ Proposed Soakage Trench Volume (4.64m^3) is larger than Required Volume (4.5m^3)
- ➔ Soakage trench sizing is OK

Impervious area (200m^2)

4. 21mm/hr
- ➔ Required 10-year V(storage) = 8.78 – 0.47 = 8.31m^3
- ➔ Proposed Soakage Trench Volume (8.46m^3) is larger than Required Volume (8.31m^3),
- ➔ Soakage trench sizing is OK

5. 85mm/hr
- ➔ Required 10-year V(storage) = 8.78 – 1.61 = 7.17m^3

➔ Proposed Soakage Trench Volume (7.21m^3) is larger than Required Volume (7.17m^3)

➔ Soakage trench sizing is OK

6. 171mm/hr

➔ Required 10-year V(storage) = 8.78 – 2.82 = 5.96m^3

➔ Proposed Soakage Trench Volume (6.27m^3) is larger than Required Volume (5.96m^3)

➔ Soakage trench sizing is OK

Impervious area (250m^2)

7. 21mm/hr

➔ Required 10-year V(storage) = 10.98 – 0.39 = 10.59m^3

➔ Proposed Soakage Trench Volume (10.71m^3) is larger than Required Volume (10.59m^3),

➔ Soakage trench sizing is OK

8. 85mm/hr

➔ Required 10-year V(storage) = 10.98 – 1.6 = 9.38m^3

➔ Proposed Soakage Trench Volume (9.99m^3) is larger than Required Volume (9.38m^3)

➔ Soakage trench sizing is OK

9. 171mm/hr

➔ Required 10-year V(storage) = 10.98 – 3.21 = 7.77m^3

➔ Proposed Soakage Trench Volume (7.85m^3) is larger than Required Volume (7.77m^3)

➔ Soakage trench sizing is OK

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title Calc Title	STATION ROAD - RESIDENTIAL Catchment Areas	Author LP	Date 10/11/2025	Checked MHS

Wetland C Catchment	ha	Area (m2)	% impervious
Residential catchment	1.8059	18059	70%

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title: STATION ROAD - RESIDENTIAL Calc Title: Pre-development		Author LP	Date 10/11/2025	Checked MHS

Wetland C
Total Catchment 18059

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (100%)	61	1.81	110.16
C		Impervious (0%)	98	0.00	0.00
					0.00
			Totals =	1.81	110.16

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{110.16}{1.806}$ = **61.0**

S= $\left(\frac{1000}{CN} - 10\right) * 25.4$ $\left(\frac{1000}{61.0} - 10\right) * 25.4$ = **162.4** mm

Ia= 0.05*S 0.05x 162.4 = **8.1** mm

2. Time of Concentration

Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001.00	Sheet 2	Rev: A
Job Title: STATION ROAD - RESIDENTIAL Calc Title: Pre-development	Author LP	Date 10/11/2025	Checked MSH

1. Data

Catchment Area	A=	0.018059	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	8.1	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24 *as per HIRDS Historical data	27.9	(mm)	83.6	129	202	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.035		0.172	0.258	0.364	
6. Specific peak flow rate q*	0.011		0.052	0.074	0.099	
7. Peak flow rate, q_p=q*A*P_{24}	**0.006**	m^3/s	**0.078**	**0.173**	**0.360**	m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	2.1		24.0	51.6	105.5	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	39	m^3	**432.53**	**931.53**	**1905.36**	m^3

Maven Associates		Job Number 289001.00	Sheet 1	Rev: A
Job Title STATION ROAD - RESIDENTIAL Calc Title Post development (Pervious)		Author LP	Date 10/11/2025	Checked MHS

Wetland C
Total Catchmer 18059

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (30%)	74	0.54	40.09
C		Impervious (70%)	98		0.00
					0.00
			Totals =	0.54	40.09

CN (weighted) = total product = / total area = 40.09 / 0.542 = **74.0**

S= (1000 / CN -10) * 25.4 = (1000 / 74.0 - 10) *25.4 = **89.2** mm

Ia= 0.05*S = 0.05x 89.2 = **4.5** mm

2. Time of Concentration

Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001.00	Sheet 2		Rev: A
Job Title: STATION ROAD - RESIDENTIAL Calc Title: Post development (Pervious)	Author LP	Date 10/11/2025		Checked MHS

1. Data

Catchment Area	A=	0.0054177	km2(100ha =1km2)
Runoff curve number	CN=	74.0	(from worksheet 1)
Initial abstraction	Ia=	4.5	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 89 mm

	WQV						
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	100	
4. 24 hour rainfall depth, P24 *as per HIRDS RITS 6.0 2081-2100	32.3	(mm)	97	152	240	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations				
6. Specific peak flow rate q*			Unnecessary for volume calculations				
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations				m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	6.6		47.1	91.9	170.8	170.8	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	36	m^3	**255.21**	**498.05**	**925.43**	**925.43**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title STATION ROAD - RESIDENTIAL Calc Title Post development (Impervious)		Author LP	Date 10/11/2025	Checked MHS

Wetland C
Total Catchmer 18059

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (30%)	74		0.00
C		Impervious (70%)	98	1.26	123.88
					0.00
			Totals =	1.26	123.88

CN (weighted) = total product = / total area = 123.88 / 1.264 = **98.0**

S= $\left(\frac{1000}{CN} - 10\right) * 25.4$ = $\left(\frac{1000}{98.0} - 10\right) * 25.4$ = **5.2** mm

Ia= 0.05*S 0.05x 5.2 = **0.3** mm

2. Time of Concentration

Unnecessary for volume calculations

Maven Associates	Job Number 289001.00	Sheet 2		Rev: A
Job Title STATION ROAD - RESIDENTIAL Calc Title Post development (Impervious)	Author LP	Date 10/11/2025		Checked MHS

1. Data

Catchment Area	A=	0.0126413	km2(100ha =1km2)
Runoff curve number	CN=	98.0	(from worksheet 1)
Initial abstraction	Ia=	0.3	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 5 mm

	WQV						
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	100	yr
4. 24 hour rainfall depth, P24 *as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations				
6. Specific peak flow rate q*			Unnecessary for volume calculations				
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations				m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	27.6		91.8	146.7	234.7	234.7	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	349	m^3	**1160.73**	**1854.84**	**2966.49**	**2966.49**	m^3

Worksheet 2: Graphical Peak Flow Rate

	Maven Associates	Job Number 289001	Sheet 1	Rev: A
Job Title Calc Title	STATION ROAD - RESIDENTIAL Post development (whole site)	Author LP	Date 10/11/2025	Checked MHS

Wetland **C**
Total Catchmer 18059

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (30%)	74	0.54	40.09
C		Impervious (70%)	98	1.26	123.88
					0.00
			Totals =	1.81	163.98

CN (weighted) = total product = / total area = 163.98 / 1.806 = **90.8**

S= (1000 / CN -10) * 25.4 (1000 - 10) *25.4 / 90.8 = **25.7** mm

Ia= 0.05*S 0.05x 25.7 = **1.3** mm

2. Time of Concentration
Unnecessary for volume calculations

Maven Associates	Job Number 289001.00	Sheet 2		Rev: A
Job Title / Calc Title — STATION ROAD - RESIDENTIAL / Post development (whole site)	Author LP	Date 10/11/2025		Checked MHS

1. Data
   Catchment Area A= 0.018059 km2(100ha =1km2)
   Runoff curve number CN= 90.8 (from worksheet 1)
   Initial abstraction Ia= 1.3 mm (from worksheet 1)
   Time of concentration tc= 0.17 hrs (from worksheet 1)
2. Calculate storage, S =(1000/CN - 10)25.4 = 26 mm

		WQV					
3. Average recurrence interval, ARI	1/4 of 2yr	1/3 of 2yr	2	10	100	100	yr
4. 24 hour rainfall depth, P24 *as per HIRDS RCP 6.0 2081-2100 data	24.25	32.3	97	152		240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.296	0.366	0.647	0.744	-0.053	0.822	
6. Specific peak flow rate q*	0.084	0.099	0.145	0.155	-0.018	0.161	
7. Peak flow rate, q_p=q*A*P_{24}	**0.037**	**0.058**	**0.254**	**0.426**	**0.000**	**0.697**	m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$							
9. Runoff volume, $V_{24} = 1000xQ_{24}A$							m^3

Pre development run off volume	432.53	931.53	1905.36	1905.36	m^3
Post development run off volume - Pervious	255.21	498.05	925.43	925.43	m^3
Post development run off volume - Impervious	1160.73	1854.84	2966.49	2966.49	m^3
Post development run off volume - total	1415.95	2352.89	3891.93	3891.93	m^3

Pre development flow rate	0.08	0.17	0.36	0.36	m^3/s
Post development flow rate	0.25	0.43	0.00	0.70	m^3/s

Worksheet 2: Graphical Peak Flow Rate

Maven Associates	Job Number 289001.00	Sheet 2	Rev: A
Job Title: STATION ROAD - RESIDENTIAL Calc Title: WQV and ED	Author LP	Date 10/11/2025	Checked MHS

1. Data

Runoff volume (pervious)	V_p=	36	m^3
Runoff volume (impervious)	V_{ip}=	349	m^3
Combined volume	V=	385.0	m^3
Pre-development initial abstration	I_{a1}=	8.1	mm
Post-development compacted pervious area CN		74	Class C
Post-development initial abstration of pervious area	I_{a2}=	4.5	mm
Post development Impervious area	A_{ip}=	1.26	ha.
Post development compacted pervious areas	A_{pp}=	0.54	ha.

2. Retention reduction

Impervious surface retention	Vrip=	102.6	m^3
Pervious surface retention	Vrp=	19.8	m^3

3. Water Quality Volume	WQV=	262.5	m^3
4. Extended Detention Volume	ED=	315	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title Calc Title	STATION ROAD - RESIDENTIAL Catchment Areas	Author LP	Date 10/11/2025	Checked MHS

Wetland D Catchment	ha	Area (m2)	% impervious
Residential catchment	2.9415	29415	70%

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title / Calc Title	STATION ROAD - RESIDENTIAL Pre-development	Author LP	Date 10/11/2025	Checked MHS

Wetland D
Total Catchment 29415

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (100%)	61	2.94	179.43
C		Impervious (0%)	98	0.00	0.00
					0.00
			Totals =	2.94	179.43

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{179.43}{2.942} = \boxed{61.0}$

S= $\left(\frac{1000}{CN} - 10\right) * 25.4$ $\left(\frac{1000}{61.0} - 10\right) *25.4 = \boxed{162.4}$ mm

Ia= 0.05*S 0.05x 162.4 = $\boxed{8.1}$ mm

2. Time of Concentration
Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 2	Rev: A
Job Title: STATION ROAD - RESIDENTIAL Calc Title: Pre-development	Author LP	Date 10/11/2025	Checked MSH

1. Data
   - Catchment Area A= 0.029415 km2(100ha =1km2)
   - Runoff curve number CN= 61.0 (from worksheet 1)
   - Initial abstraction Ia= 8.1 mm (from worksheet 1)
   - Time of concentration tc= 0.00 hrs (from worksheet 1)
2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24 *as per HIRDS Historical data	27.9	(mm)	83.6	129	202	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.035		0.172	0.258	0.364	
6. Specific peak flow rate q*	0.011		0.052	0.074	0.099	
7. Peak flow rate, q_p=q*A*P_{24}	**0.009**	m^3/s	**0.128**	**0.282**	**0.586**	m^3/s
8. Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	2.1		24.0	51.6	105.5	
9. Runoff volume, V_{24} = 1000xQ_{24}A	63	m^3	**704.51**	**1517.31**	**3103.51**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title STATION ROAD - RESIDENTIAL Calc Title Post development (Pervious)		Author LP	Date 10/11/2025	Checked MHS

Wetland D
Total Catchmer 29415

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (30%)	74	0.88	65.30
C		Impervious (70%)	98		0.00
					0.00
			Totals =	0.88	65.30

CN (weighted) = total product = / total area = 65.30 / 0.882 = **74.0**

S= (1000 / CN -10) * 25.4 (1000 - 10) *25.4 / 74.0 = **89.2** mm

Ia= 0.05*S 0.05x 89.2 = **4.5** mm

2. Time of Concentration

Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates		**Job Number** **289001**	**Sheet** **2**		**Rev: B**
Job Title **Calc Title**	**STATION ROAD - RESIDENTIAL** **Post development (Pervious)**	**Author** **LP**	**Date** **10/11/2025**		**Checked** **MHS**

1. Data

Catchment Area A= 0.0088245 km2(100ha =1km2)

Runoff curve number CN= 74.0 (from worksheet 1)

Initial abstraction Ia= 4.5 mm (from worksheet 1)

Time of concentration tc= 0.00 hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 89 mm

	WQV						
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	100	
4. 24 hour rainfall depth, P24 *as per HIRDS RITS 6.0 2081-2100	32.3	(mm)	97	152	240	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations				
6. Specific peak flow rate q*			Unnecessary for volume calculations				
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations				m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	6.6		47.1	91.9	170.8	170.8	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	59	m^3	**415.70**	**811.24**	**1507.37**	**1507.37**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title STATION ROAD - RESIDENTIAL Calc Title Post development (Impervious)		Author LP	Date 10/11/2025	Checked MHS

Wetland D
Total Catchmer 29415

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (30%)	74		0.00
C		Impervious (70%)	98	2.06	201.79
					0.00
			Totals =	2.06	201.79

CN (weighted) = total product = / total area = 201.79 / 2.059 = **98.0**

S= (1000 / CN -10) * 25.4 (1000 - 10) *25.4 / 98.0 = **5.2** mm

Ia= 0.05*S 0.05x 5.2 = **0.3** mm

2. Time of Concentration

Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001.00	Sheet 2		Rev: A
Job Title: STATION ROAD - RESIDENTIAL Calc Title: Post development (Impervious)	Author LP	Date 10/11/2025		Checked MHS

1. Data

Catchment Area	A=	0.0205905	km2(100ha =1km2)
Runoff curve number	CN=	98.0	(from worksheet 1)
Initial abstraction	Ia=	0.3	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 5 mm

	WQV						
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	100	yr
4. 24 hour rainfall depth, P24 *as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations				
6. Specific peak flow rate q*			Unnecessary for volume calculations				
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations				m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	27.6		91.8	146.7	234.7	234.7	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	569	m^3	**1890.64**	**3021.21**	**4831.91**	**4831.91**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates	Job Number 289001	Sheet 1	Rev: A
Job Title: STATION ROAD - RESIDENTIAL Calc Title: Post development (whole site)	Author LP	Date 10/11/2025	Checked MHS

Wetland D
Total Catchmer 29415

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (30%)	74	0.88	65.30
C		Impervious (70%)	98	2.06	201.79
					0.00
			Totals =	2.94	267.09

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{267.09}{2.942} =$ **90.8**

S= $\left(\frac{1000}{CN} - 10\right) * 25.4$ $\left(\frac{1000}{90.8} - 10\right) *25.4$ = **25.7** mm

Ia= 0.05*S 0.05x 25.7 = **1.3** mm

2. Time of Concentration

Unnecessary for volume calculations

Maven Associates	**Job Number** 289001.00	**Sheet** 2		**Rev: A**
Job Title STATION ROAD - RESIDENTIAL **Calc Title** Post development (whole site)	**Author** LP	**Date** 10/11/2025		**Checked** MHS

1. Data
   Catchment Area A= 0.029415 km2(100ha =1km2)

   Runoff curve number CN= 90.8 (from worksheet 1)

   Initial abstraction Ia= 1.3 mm (from worksheet 1)

   Time of concentration tc= 0.17 hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 26 mm

		WQV					
3. Average recurrence interval, ARI	1/4 of 2yr	1/3 of 2yr	2	10	100	100	yr
4. 24 hour rainfall depth, P24 *as per HIRDS RCP 6.0 2081-2100 data	24.25	32.3	97	152	240	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.296	0.366	0.647	0.744	0.822	0.822	
6. Specific peak flow rate q*	0.084	0.099	0.145	0.155	0.161	0.161	
7. Peak flow rate, q_p=q*A*P_{24}	**0.060**	**0.094**	**0.414**	**0.694**	**1.135**	**1.135**	m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$							
9. Runoff volume, $V_{24} = 1000xQ_{24}A$							m^3

Pre development run off volume	704.51	1517.31	3103.51	3103.51	m^3
Post development run off volume - Pervious	415.70	811.24	1507.37	1507.37	m^3
Post development run off volume - Impervious	1890.64	3021.21	4831.91	4831.91	m^3
Post development run off volume - total	2306.34	3832.45	6339.28	6339.28	m^3

Pre development flow rate	0.13	0.28	0.59	0.59	m^3/s
Post development flow rate	0.41	0.69	1.14	1.14	m^3/s

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001.00	Sheet 2	Rev: A
Job Title Calc Title	STATION ROAD - RESIDENTIAL WQV and ED	Author LP	Date 10/11/2025	Checked MHS

1. Data

Runoff volume (pervious)	V_p=	59	m^3	
Runoff volume (impervious)	V_{ip}=	569	m^3	
Combined volume	V=	627.1	m^3	
Pre-development initial abstration	I_{a1}=	8.1	mm	
Post-development compacted pervious area CN		74		Class C
Post-development initial abstration of pervious area	I_{a2}=	4.5	mm	
Post development Impervious area	A_{ip}=	2.06	ha.	
Post development compacted pervious areas	A_{pp}=	0.88	ha.	

2. Retention reduction

Impervious surface retention	Vrip=	167.2	m^3
Pervious surface retention	Vrp=	32.3	m^3

3. Water Quality Volume	WQV=	427.6	m^3
4. Extended Detention Volume	ED=	513	m^3

Worksheet 2: Graphical Peak Flow Rate

MAVEN	**MAVEN ASSOCIATES**	**Job Number** 289001	**Sheet** 1	**Rev** A
Job Title / **Calc Title**	**STATION ROAD -RESIDENTIAL** / **Wetland C Outlet design**	**Author** LP	**Date** 10-Nov	**Checked** MHS

Wetland C

Refer TR202006 Wetland WQV 2yr 100yr for volume and flow rate calculations.
Refer Hec report for modelling and Flow rates

SUMMARY TABLE - VOLUMES

Actual WQV	263	m3	Taking permanent water depth x wetland area
Actual ED	315	m^3	1.2WQV
Designed ED	594	m^3	From model; OK

LEVELS

PWL.	64.6	m	
EDV el.	64.95	m	Set to 350mm above PWL per ICMP
2yr el.	65.25	m	Set 300mm above EDV
Q2 post	0.25	m^3/s	See "TR202006 Wetland WQV 2yr 100yr" for calculations

Calc. 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

Qed = 315 per 24 hours
0.0036 m3/s

At full EDV elev., maximum release rate is assumed to be Qmax = 2Qed

Qmax = 0.007 m3/s

Vol. of storage savings can be realised if edv is determined by routing flow through the pond

Qi = 0.62A(2gh)&0.5 where A = area of ED orifice

orifice dia	0.078	m	
A =	0.0047399	m	
h =	0.3111574	m	
Qi =	0.007	m3/s	OK

Use 80 mm orifice

	orifice dia	0.078	m	
	A =	0.0047399	m	
	h =	0.61	m	
Outflow from ED orifice	Qi =	0.0101761	m3/s	OK

Outpool 2YR spillway design

(this is more representative for spillway to a channel)

Q $0.57\,(2g)^{1/2}\,(2/3Lh^{3/2} + 8/15Zh^{5/2})$

Where

Q=	discharge through the spillway	m3/s
L =	horizontal bottom width of the spillway	m
h =	depth of design flow	m
z=	horizontal/vertical side slope (recommended	

Q=	0.24	m3/s	(Q2 post - Qi 2yr)	
Q=	0.3235943	m3/s		
h=	0.3	m		
z=	3		1V:3H slope	
L=	0.45	m	trial and error	OK

Min. Weir width = **L= 0.45 m**

MAVEN	MAVEN ASSOCIATES	Job Number 289001	Sheet 1	Rev A
Job Title Calc Title	STATION ROAD -RESIDENTIAL Wetland C Outlet design	Author LP	Date 10-Nov	Checked MHS

Wetland C

Refer TR202006 Wetland WQV 2yr 100yr for volume and flow rate calculations.
Refer Hec report for modelling and Flow rates

SUMMARY TABLE - VOLUMES

Actual WQV	428	m3	Taking permanent water depth x wetland area
Actual ED	513	m^3	1.2WQV
Designed ED	1813	m^3	From model; OK

LEVELS

PWL.	64.95	m	
EDV el.	65.3	m	Set to 350mm above PWL per ICMP
2yr el.	65.6	m	Set 300mm above EDV
Q2 post	0.41	m^3/s	See "TR202006 Wetland WQV 2yr 100yr" for calculations

Calc. 1: Extended Detention Orifice

Outlet to be sized to release the EDV over a 24-hour period

Qed = 513 per 24 hours
0.0059 m3/s

At full EDV elev., maximum release rate is assumed to be Qmax = 2Qed

Qmax = 0.012 m3/s

Vol. of storage savings can be realised if edv is determined by routing flow through the pond

Qi = 0.62A(2gh)&0.5 where A = area of ED orifice

orifice dia 0.100 m
A = 0.007854 m
h = 0.3 m

Qi = 0.012 m3/s OK

Use 100 mm orifice

orifice dia 0.100 m
A = 0.007854 m
h = 0.60 m

Outflow from ED orifice Qi = 0.0167073 m3/s OK

Outpool 2YR spillway design

(this is more representative for spillway to a channel)

Q $0.57\ (2g)^{1/2}\ (2/3Lh^{3/2} + 8/15Zh^{5/2})$

Where

Q=	discharge through the spillway	m3/s
L =	horizontal bottom width of the spillway	m
h =	depth of design flow	m
z=	horizontal/vertical side slope (recommended	

Q= 0.40 m3/s (Q2 post - Qi 2yr)

Q= 0.572513 m3/s
h= 0.3 m
z= 3 1V:3H slope

L= 1.35 m trial and error OK

Min. Weir width = **L= 1.35 m**

Maven Associates		Job Number 180006	Sheet 1	Rev: A
Job Title STATION ROAD - -RESIDENTIAL **Calc Title** Wetland Sizing and dimensions **Catchment** Catchment and Wetland C		Author LP	Date 16/11/2025	Checked MHS

References:
WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITS V2.5 October 2024 - 4.2.18 Constructed Wetlands

Key: input

Wetland / Pond Sizing and dimensions

Catchment Area	1.81	ha	
Imperviousness <70%?	no		
Percentage of catchment area	4%		
Wetland surface area	722	m2	per WRC 2020/06 8.5.7 Govened by EDV volume
Info Table			
	PostDev		
Q2	0.25	m^3/s	See "TR202006 Wetland WQV 2.10.100yr" for calculations
V2	1415.95	m^3	
Q10	0.43	m^3/s	
V10	2352.89	m^3	
Q100 (RCP6.0)	0.70	m^3/s	
V100 (RCP6.0)	3891.93	m^3	
Min. WQV	262.52	m^3	1/3 of the 2-year storm 24hr ARI rainfall event.

SUMMARY TABLE

Min. wetland area (4% catchment)	**722**	m2	
Designed pwl wetland area	**1579**	m2	To achieve ED volume at avg. depth of 0.35m
Min. WQV	**263**	m3	
Min. ED	315	m^3	1.2WQV
Designed ED	594	m3	OK

Fore Bay Volume	(15% of WQV)	=	39.38	m^3	Per WRC
Min Fore Bay Area	(10% of Wetland Area)	=	157.90	m^2	per RITS
Designed ForeBay Area			160	m^2	from CAD

Maven Associates		Job Number 180006	Sheet 1	Rev: A
Job Title: STATION ROAD - RESIDENTIAL Calc Title: Wetland Sizing and dimensions Catchment: Catchment and Wetland D		Author LP	Date 23/10/2025	Checked MHS

References:
WRC 2020/07 8.5.6 Stormwater pond
WRC 2020/07 8.5.7 Wetlands
RITS V2.5 October 2024 - 4.2.18 Constructed Wetlands

Key: input

Wetland / Pond Sizing and dimensions

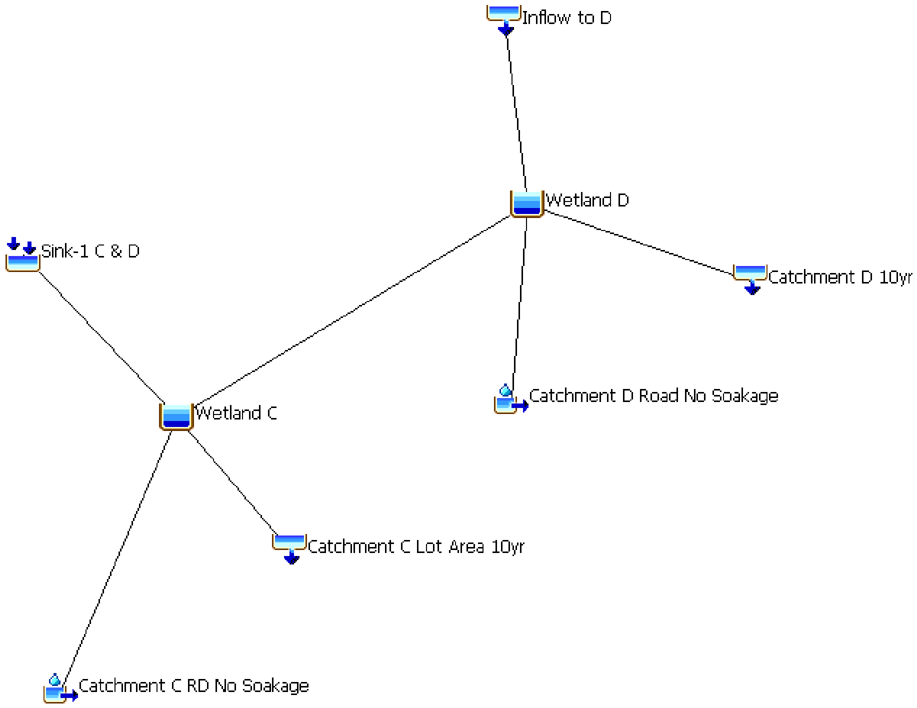
Catchment Area	2.94	ha	
Imperviousness <70%?	no		
Percentage of catchment area	4%		
Wetland surface area	1177	m2	per WRC 2020/06 8.5.7 Govened by EDV volume
Info Table			
	PostDev		
Q2	0.41	m^3/s	See "TR202006 Wetland WQV 2.10.100yr" for calculations
V2	2306.34	m^3	
Q10	0.69	m^3/s	
V10	3832.45	m^3	
Q100 (RC6.0)	1.14	m^3/s	
V100 (RCP6.0)	6339.28	m^3	
Min. WQV	427.61	m^3	1/3 of the 2-year storm 24hr ARI rainfall event.

SUMMARY TABLE

Min. wetland area (4% catchment)	**1177**	m2	
Designed wetland area	**4980**	m	From CAD
Min. WQV	**428**	m3	Maximum of calculated WQV vs water depth x wetland area
Min. ED	513	m^3	1.2WQV
Designed ED	1813	m^3	**OK**

Fore Bay Volume	(15% of WQV)	=	64.14	m^3	Per WRC
Fore Bay Area	(10% of Wetland Area)	=	498.00	m^2	per RITS
Designed ForeBay Area		=	510	m^2	From CAD

Wetland C and D 10year Results



10 year Basin Model – Post Development

Project: Basin A to D   Simulation Run: Catchment CD 10year Post -

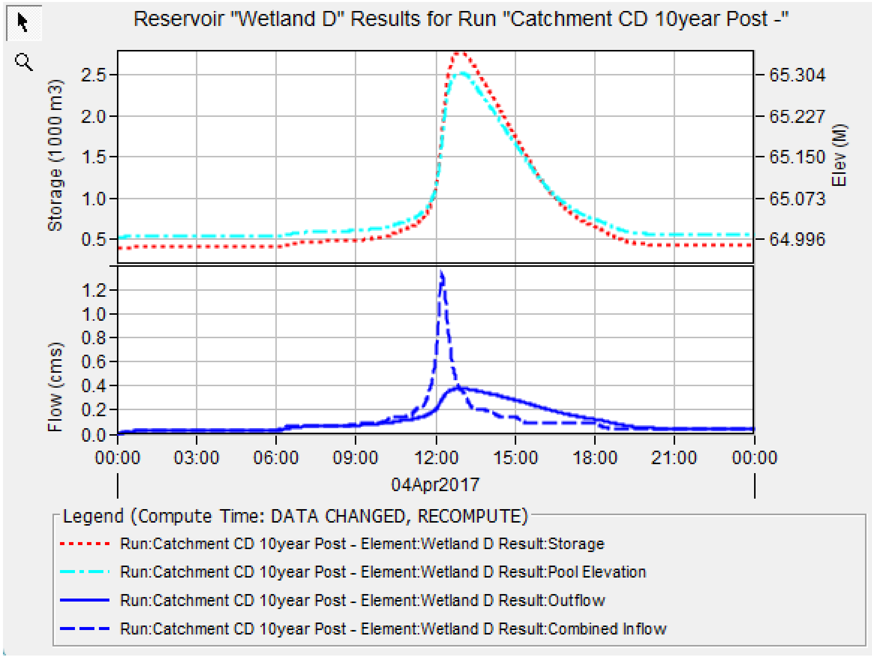
Start of Run: 04Apr2017, 00:00   Basin Model: Post Dev C & D SW MIT
End of Run: 05Apr2017, 00:00   Meteorologic Model: 10 year post
Compute Time: DATA CHANGED, RECOMPUTE   Control Specifications: Control 1

Show Elements: All Elements   Volume Units: MM 1000 M3   Sorting: Watershed Explorer

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Catchment D Road No Soakage	0.01000	0.2751	4 April 2017, 12:12	1.5569
Inflow to D		0.9050	4 April 2017, 12:19	6.5528
Catchment D 10yr		0.2376	4 April 2017, 12:12	1.3592
Wetland D		0.3757	4 April 2017, 12:56	9.4327
Catchment C No Soakage	0.00535	0.1492	4 April 2017, 12:12	0.8524
Catchment C 10yr		0.2376	4 April 2017, 12:12	1.3592
Wetland C		0.3559	4 April 2017, 14:17	10.5515
Catchment D Lot Areas	0.05860	1.2097	4 April 2017, 12:19	8.7339
Catchmend C Lot Areas	0.03260	0.8591	4 April 2017, 12:12	4.8727
Sink-1 C & D		0.3559	4 April 2017, 14:17	10.5515
Catchment C inflow Road Soak		0.2376	4 April 2017, 12:12	1.3592
Catchment D Inflow Road Soak		0.4441	4 April 2017, 12:12	2.5393

10year Post - Summary Result

Wetland D – 10yr



Wetland D Graph

Project: Basin A to D Simulation Run: Catchment CD 10year Post -
Reservoir: Wetland D

Start of Run:	04Apr2017, 00:00	Basin Model:	Post Dev C & D SW MIT
End of Run:	05Apr2017, 00:00	Meteorologic Model:	10 year post
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

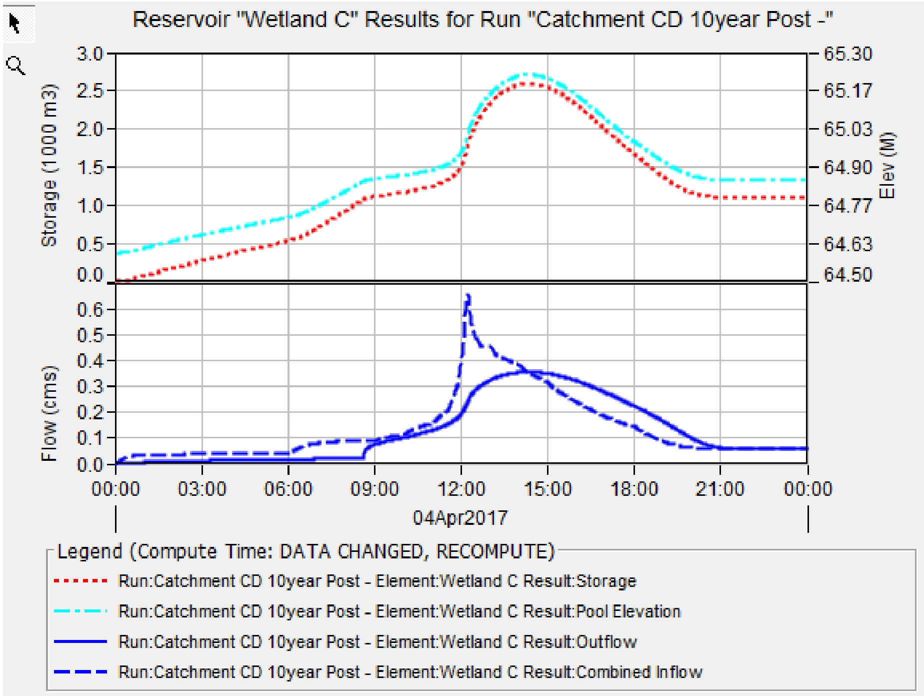
Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Inflow:	1.3193 (M3/S)	Date/Time of Peak Inflow:	04Apr2017, 12:15
Peak Discharge:	0.3757 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:56
Inflow Volume:	9.4689 (1000 M3)	Peak Storage:	2.7611 (1000 M3)
Discharge Volume:	9.4327 (1000 M3)	Peak Elevation:	65.3069 (M)

Summary Result – Wetland D 10yr

Wetland C – 10yr



Wetland C – 10yr Graph

Project: Basin A to D   Simulation Run: Catchment CD 10year Post -
Reservoir: Wetland C

Start of Run:	04Apr2017, 00:00	Basin Model:	Post Dev C & D SW MIT
End of Run:	05Apr2017, 00:00	Meteorologic Model:	10 year post
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

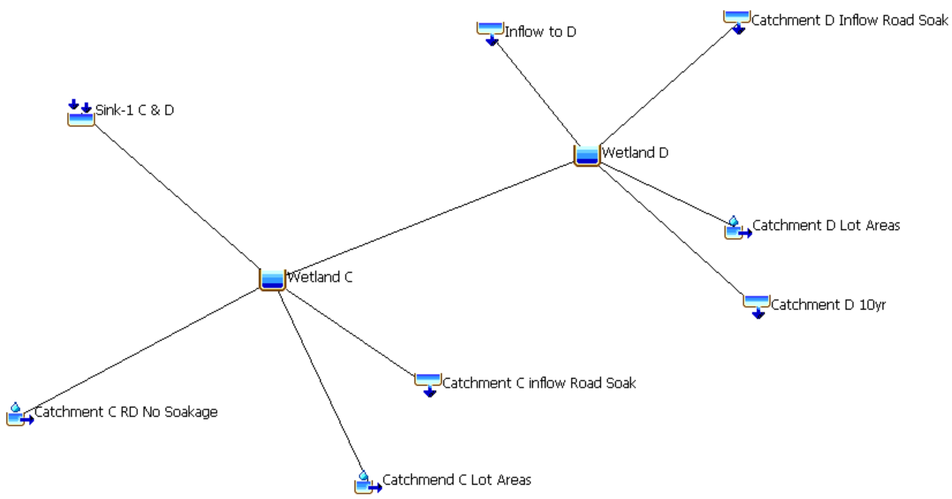
Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Inflow:	0.6596 (M3/S)	Date/Time of Peak Inflow:	04Apr2017, 12:13
Peak Discharge:	0.3559 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 14:17
Inflow Volume:	11.6443 (1000 M3)	Peak Storage:	2.5920 (1000 M3)
Discharge Volume:	10.5515 (1000 M3)	Peak Elevation:	65.2235 (M)

Summary Result – Wetland C 10yr

Wetland C and D 100year Results



100 year Basin Model – Post Development

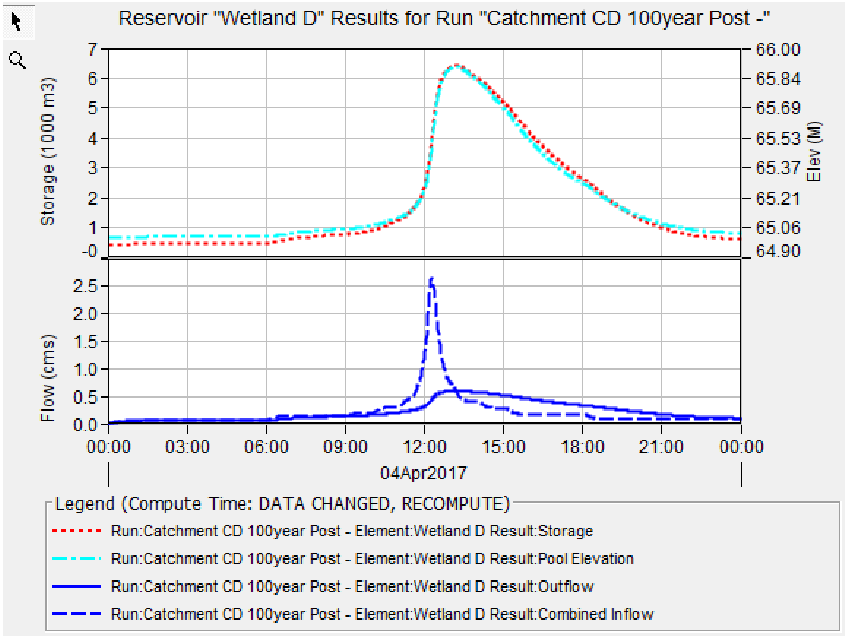
Project: Basin A to D   Simulation Run: Catchment CD 100year Post -

Start of Run: 04Apr2017, 00:00   Basin Model: Post Dev C & D SW MIT
End of Run: 05Apr2017, 00:00   Meteorologic Model: 100 year post
Compute Time:14Nov2025, 17:52:43   Control Specifications:Control 1

Show Elements: All Elements   Volume Units: MM 1000 M3   Sorting: Watershed Explorer

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Catchment D Lot Areas	0.05860	1.2097	4 April 2017, 12:19	8.7339
Catchment D Road N...	0.01000	0.2751	4 April 2017, 12:12	1.5569
Inflow to D		0.9050	4 April 2017, 12:19	6.5528
Catchment D Inflow ...		0.4441	4 April 2017, 12:12	2.5393
Wetland D		0.6009	4 April 2017, 13:12	19.1720
Catchmend C Lot Areas	0.03260	0.8591	4 April 2017, 12:12	4.8727
Catchment C RD No ...	0.00535	0.1492	4 April 2017, 12:12	0.8524
Catchment C inflow R...		0.2376	4 April 2017, 12:12	1.3592
Wetland C		0.8271	4 April 2017, 13:05	24.8027
Sink-1 C & D		0.8271	4 April 2017, 13:05	24.8027
Catchment C Lot Are...		0.2376	4 April 2017, 12:12	1.3592
Catchment D 10yr		0.2376	4 April 2017, 12:12	1.3592

100year Post - Summary Result



Wetland D – 100yr Graph

Project: Basin A to D   Simulation Run: Catchment CD 100year Post -
Reservoir: Wetland D

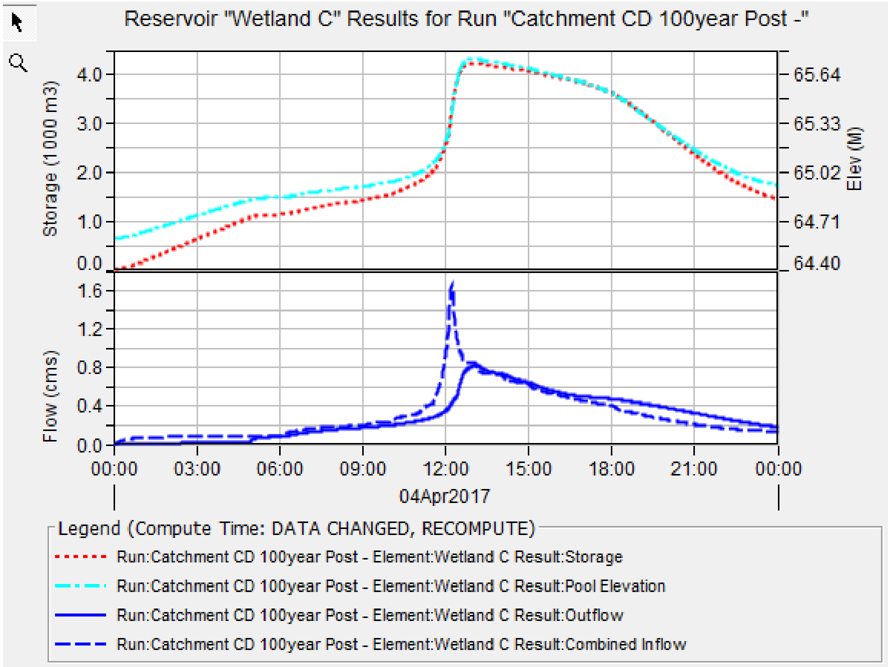
Start of Run:	04Apr2017, 00:00	Basin Model:	Post Dev C & D SW MIT
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year post
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ◯ MM ◉ 1000 M3

Computed Results

Peak Inflow:	2.6558 (M3/S)	Date/Time of Peak Inflow:	04Apr2017, 12:16
Peak Discharge:	0.6009 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 13:12
Inflow Volume:	19.3829 (1000 M3)	Peak Storage:	6.4391 (1000 M3)
Discharge Volume:	19.1720 (1000 M3)	Peak Elevation:	65.9058 (M)

Summary Result – Wetland D 100yr



Wetland C – 100yr Graph

Project: Basin A to D Simulation Run: Catchment CD 100year Post -
Reservoir: Wetland C

Start of Run:	04Apr2017, 00:00	Basin Model:	Post Dev C & D SW MIT
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year post
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ◯ MM ◉ 1000 M3

Computed Results

Peak Inflow:	1.6524 (M3/S)	Date/Time of Peak Inflow:	04Apr2017, 12:13
Peak Discharge:	0.8271 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 13:05
Inflow Volume:	26.2562 (1000 M3)	Peak Storage:	4.2481 (1000 M3)
Discharge Volume:	24.8027 (1000 M3)	Peak Elevation:	65.7509 (M)

Summary Result – Wetland C 100yr

Summary Proposal Wetland C and D

Wetland D Orifice

- 550mm @ outlet IL: 65.00m

Wetland C Orifices and Spillway

- 150mm lower Orifice IL: 64.60m
- 500mm Orifice IL: 64.85m
- Spillway IL 65.60m and Length: 3m

10 year event Check:

Pre Development Flow 10yr (A6, refer to SMP table 4, and Catchment Plan)

= 0.52m^3/s, 80% is 0.416 m^3/s

Post Development Flow based on above proposal discharging from Wetland C

= 0.356 m^3/s less than target therefore ok

100 year event Check:

Pre Development Flow 100yr (A6, refer to SMP table 4, and Catchment Plan)

= 1.06m^3/s, 80% is 0.848 m^3/s

Post Development Flow based on above proposal discharging from Wetland C

= 0.827 m^3/s less than target therefore ok

This image currently shows Basin C, which has been converted into a wetland. I considered replacing it with an image of Basin A, as it remains a dry basin, but the existing image still serves its intended purpose.

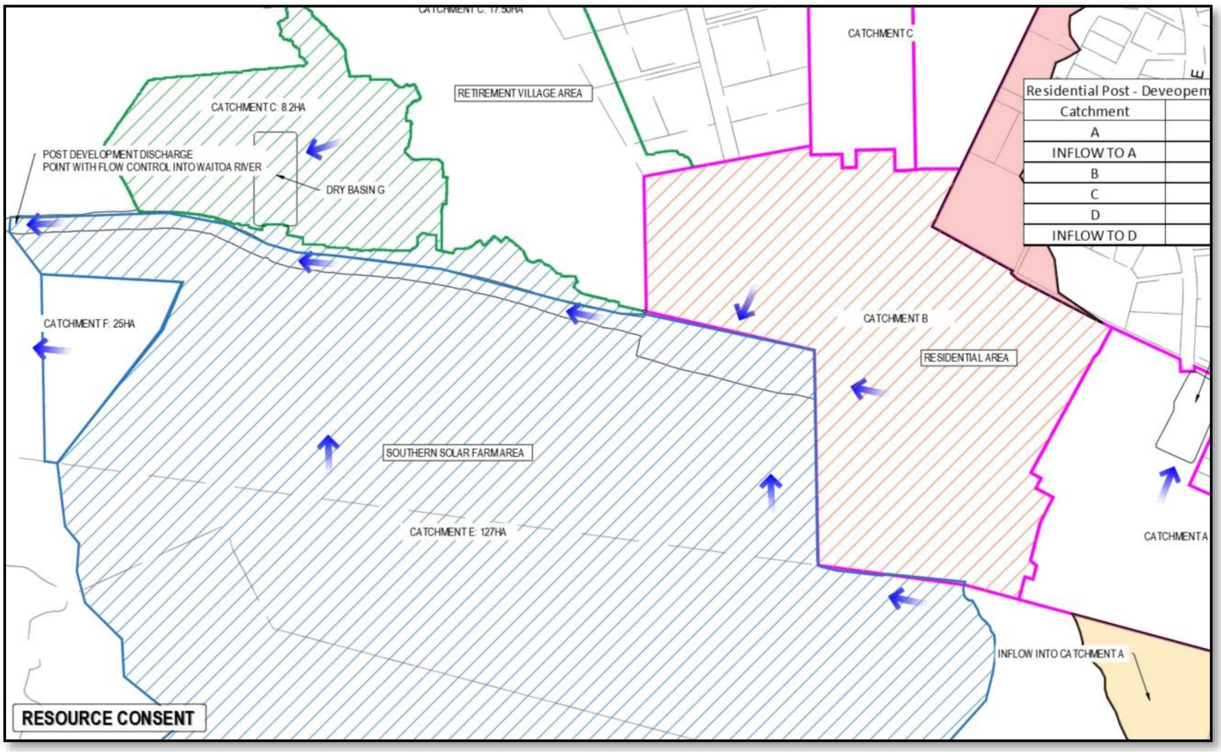
Catchments C and D are both located in the northern part of the site. Roof runoff for these catchments will be directed to individual on-lot detention tanks sized to attenuate up to the 10-year ARI (including climate change) and discharge at approximately 80% of the pre-development peak flow. Road runoffs within the Catchments will be divided between sections of the corridor with soakage systems (providing retention up to the 10-year ARI cc event) and sections without soakage that drain directly to the downstream wetlands. For storm events exceeding the 10-year ARI cc event, overflows from the on-lot detention tanks and from the road soakage systems, along with runoff from the non-soakage road areas, will be conveyed via the road drainage system to the proposed stormwater wetland.

The wetland provides water-quality volume treatment, extended Detention, and peak flow mitigation for the 10 and 100yr cc events.

GREENWAY DESIGN SUMMARY

Catchment Characteristics

The Greenway connects to Basin B and serves as an overall attenuation device for Basin B, as well as a diversion for the attenuated flow from RV and for inflow from the solar farm and upstream catchments. The catchment plan below, along with the tables further down, provides information regarding the catchment entering this stormwater device.Design



Catchment	Area (Ha)
RES B	19.5
Inflow RV	8.2
Inflow South	127

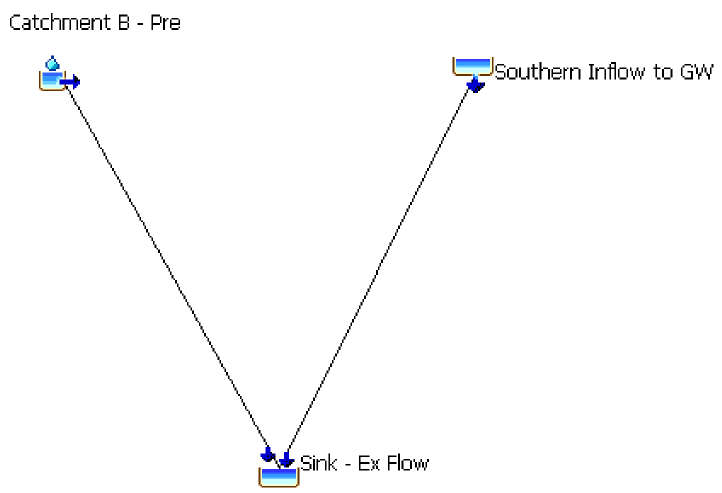
Pre-Development CN		Post Development CN	
Pervious	**Impervious**	**Pervious**	**Impervious**
61	98	74	98

	Pre Development		Post-Development (RCP 8.5)		
	10yr	100 yr	10yr	100yr	100yr-10yr
24 hour rainfall depth (mm)	128	200	167	265	98

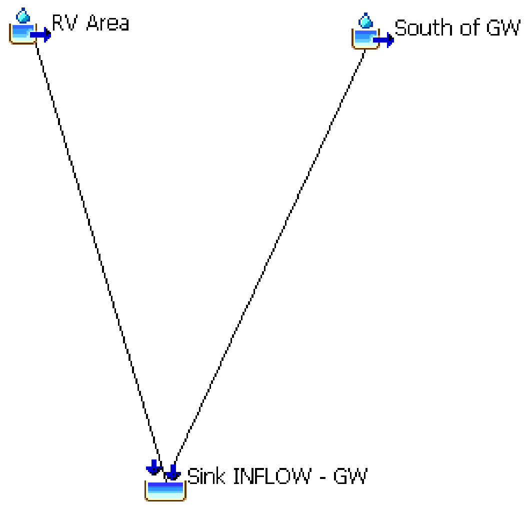
An HEC-HMS model has been prepared using the site design parameters mentioned above, along with the site hydrology data and TR20/07 data attached to this Design Summary.

Pre Development HEC HMS RESUTLS

Pre Development Basin Model



Southern Inflow to GW is separately modelled in the Basin Model shown below. Its hydrograph is inserted into the above main model representing the overall inflows outside of the Catchment B area. The same hydrograph is also incorporated into the post-development assessment of the Greenway + Basin model.



Pre Development Summary Results

Project: Basin A to D   Simulation Run: Inflow 01 - GW
Sink: Sink INFLOW - GW

Start of Run:	04Apr2017, 00:00	Basin Model:	Inflow Basin Models
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Discharge: 7.4724 (M3/S)   Date/Time of Peak Discharge 04Apr2017, 13:58
Volume: 136.6029 (1000 M3)

Inflow Results

Project: Basin A to D   Simulation Run: Catchment B 100year Pre

Start of Run:	04Apr2017, 00:00	Basin Model:	Pre Dev B
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Show Elements: All Elements   Volume Units: ○ MM ◉ 1000 M3   Sorting: Watershed Explorer

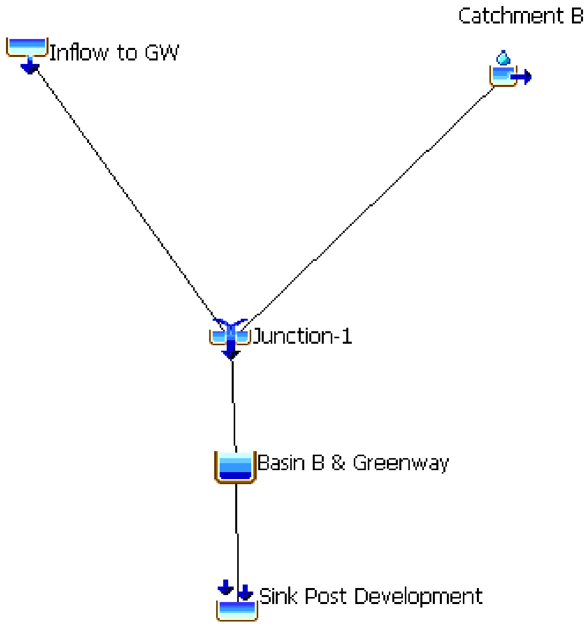
Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Catchment B - Pre	0.19500	2.1469	4 April 2017, 12:38	20.4084
Southern Inflow t...		7.4724	4 April 2017, 13:58	136.6028
Sink - Ex Flow		8.1179	4 April 2017, 13:55	157.0113

Combined Greenway + Catchment B Results

- Pre Development Peak Flow Rate Catchment B = 2.15m³/s
- 80% Pre Development Peak Flow Rate Catchment B = **1.72m³/s**
- Pre Development Peak Flow Rate Inflow Catchments = 7.47m³/s
- Maximum Combined Flow Rate Required for Post Development = **9.19m³/s**

Post Development HEC HMS RESUTLS

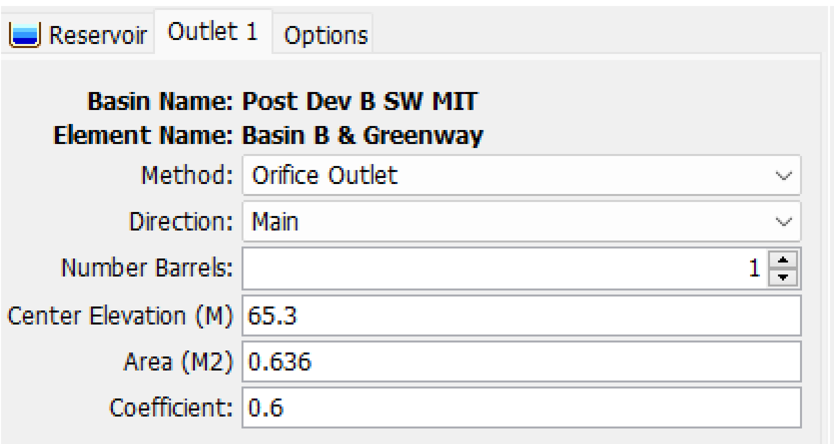
Post Development Basin Model



Basin B – Greenway – Set up

Outlet Assumptions;

- 1x Orifice 900mm @ Elevation 65.3m (note this is Center of Orifice)
- Spillway of 15m @ Elevation of 66.55m


Reservoir | Outlet 1 | Options

Basin Name: Post Dev B SW MIT
Element Name: Basin B & Greenway

Method:	Orifice Outlet
Direction:	Main
Number Barrels:	1
Center Elevation (M)	65.3
Area (M2)	0.636
Coefficient:	0.6

Basin Name: **Post Dev B SW MIT**
Element Name: **Basin B & Greenway**

Method: Broad-Crested Spillway

Direction: Main

*Elevation (M) 66.55

*Length (M) 15

Coefficient (M^0.5/S) 1.6

Gates: 0

Greenway/Basin B Storage Function

Elevation	Volume (m^3)
67.00	19369.62
66.90	16927.66
66.80	14652.14
66.70	12520.01
66.60	10492.37
66.50	8536.09
66.40	6648.73
66.30	4832.60
66.20	3318.14
66.10	2359.96
66.00	1788.82
65.90	1338.32
65.80	997.34
65.70	757.29
65.60	580.37
65.50	432.76
65.40	310.75
65.30	212.29
65.20	135.32
65.10	77.79
65.00	37.66
64.90	12.88
64.80	1.41
64.75	0.03

Post Development Summary Results – Basin B

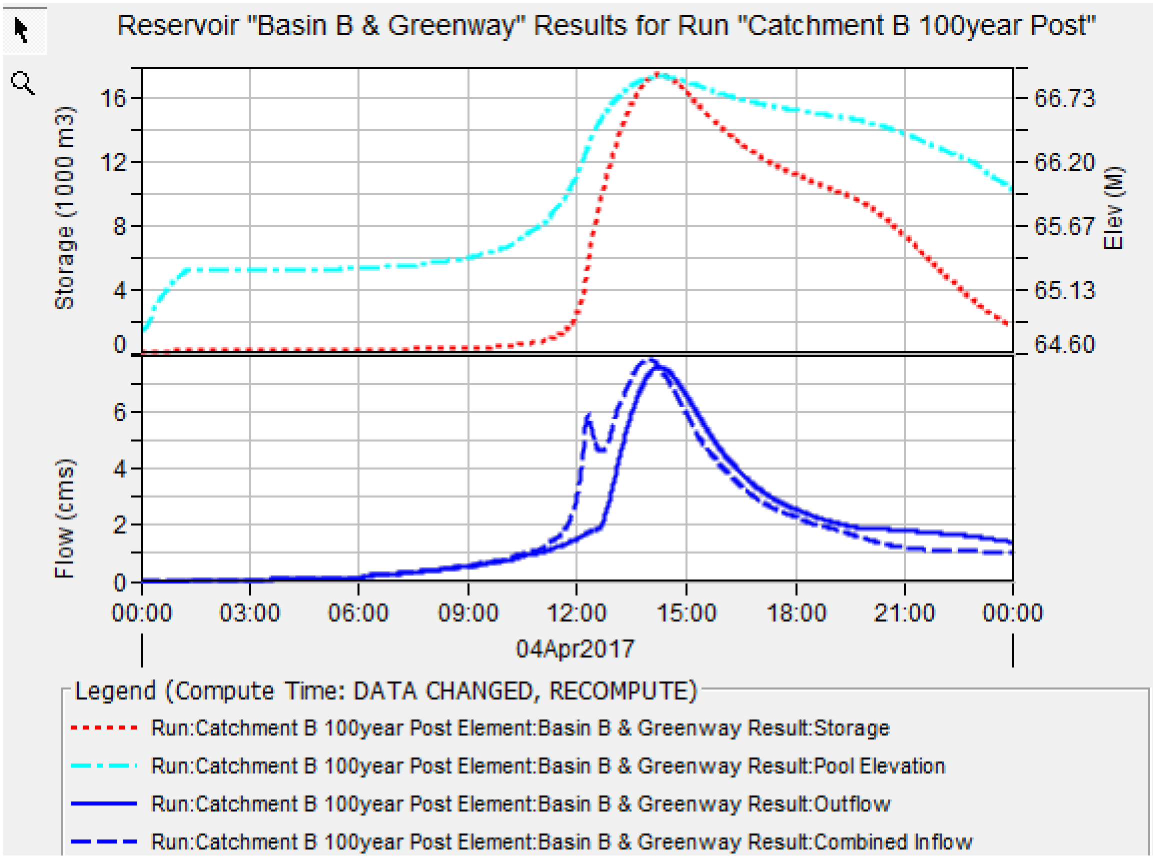
Project: Basin A to D   Simulation Run: Catchment B 100year Post
Reservoir: Basin B & Greenway

Start of Run:	04Apr2017, 00:00	Basin Model:	Post Dev B SW MIT
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100-10yr
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: MM 1000 M3

Computed Results

Peak Inflow:	7.8421 (M3/S)	Date/Time of Peak Inflow:	04Apr2017, 13:58
Peak Discharge:	7.5728 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 14:15
Inflow Volume:	n/a	Peak Storage:	17.5033 (1000 M3)
Discharge Volume:	n/a	Peak Elevation:	66.9209 (M)



- Peak Water level @ RL: 66.92m
- Peak Discharge is 7.8m^3/s
- Max Peak required is 9.19m^3/s therefore OK

Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

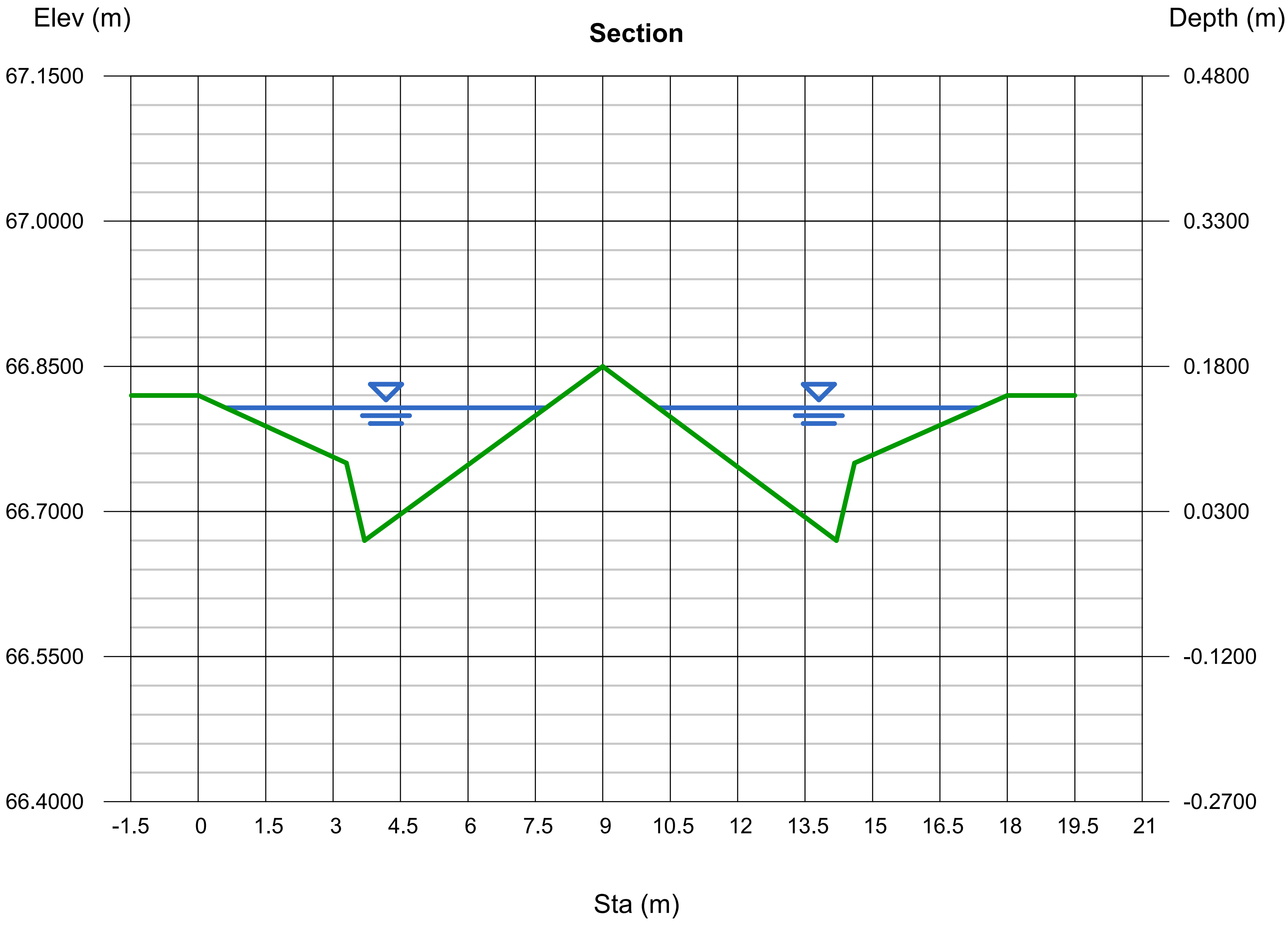
Catchemnt A Section 1

User-defined
Invert Elev (m) = 66.6700
Slope (%) = 0.7000
N-Value = 0.015

Calculations
Compute by: Known Q
Known Q (cms) = 0.6000

Highlighted
Depth (m) = 0.1372
Q (cms) = 0.6000
Area (sqm) = 0.7825
Velocity (m/s) = 0.7668
Wetted Perim (m) = 14.2849
Crit Depth, Yc (m) = 0.1402
Top Width (m) = 14.2631
EGL (m) = 0.1671

(Sta, El, n)-(Sta, El, n)...
(0.0100, 66.8200)-(3.3000, 66.7500, 0.015)-(3.7000, 66.6700, 0.015)-(9.0000, 66.8500, 0.015)-(14.2000, 66.6700, 0.015)-(14.6000, 66.7500, 0.015)-(18.0000, 66.82



Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt A Section 2

User-defined

Invert Elev (m)	=	66.5600
Slope (%)	=	0.9000
N-Value	=	0.015

Calculations

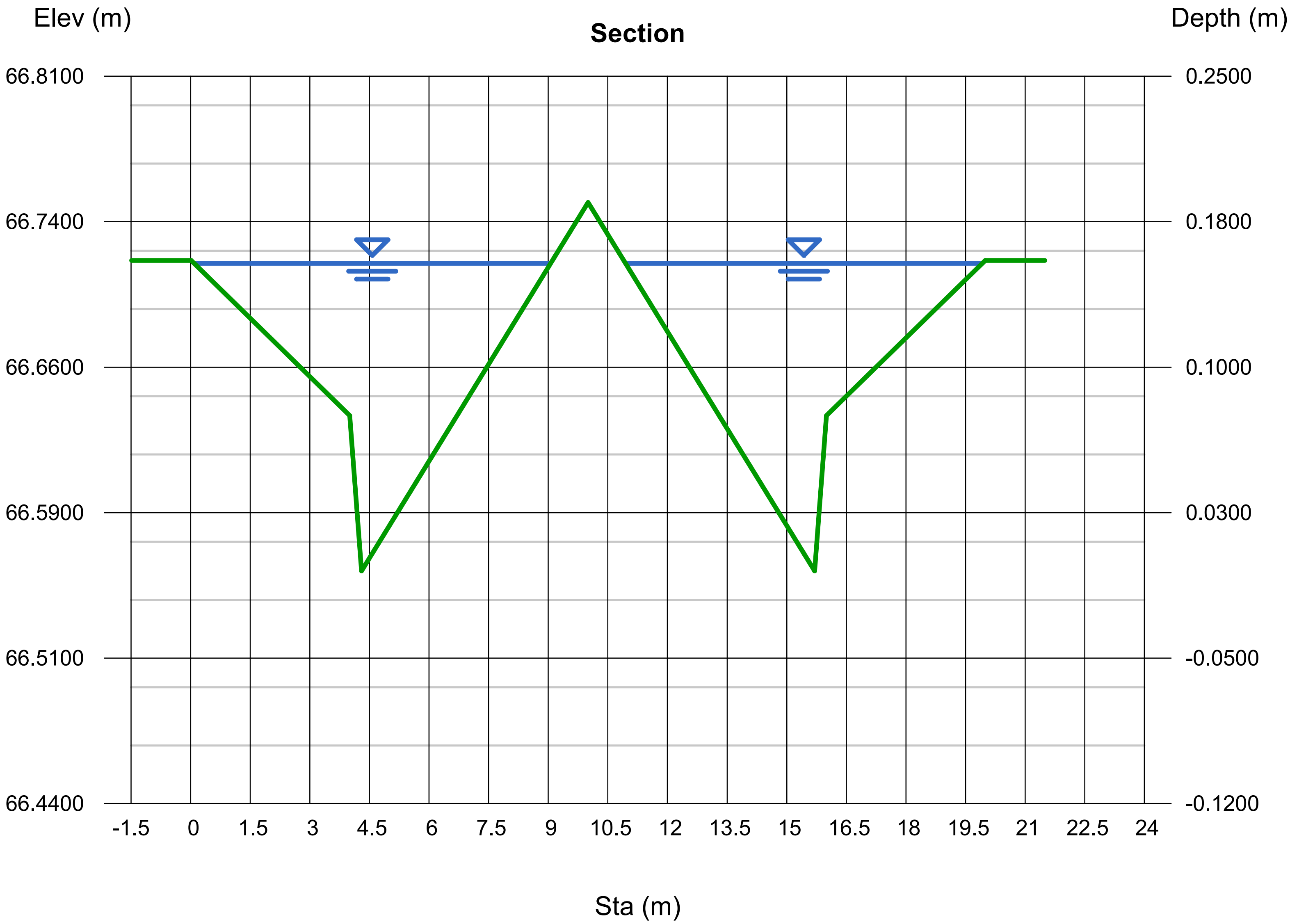
Compute by:		Known Q
Known Q (cms)	=	1.1000

Highlighted

Depth (m)	=	0.1585
Q (cms)	=	1.1000
Area (sqm)	=	1.1324
Velocity (m/s)	=	0.9714
Wetted Perim (m)	=	17.9771
Crit Depth, Yc (m)	=	0.1707
Top Width (m)	=	17.9492
EGL (m)	=	0.2066

(Sta, El, n)-(Sta, El, n)...

(0.0100, 66.7200)-(4.0000, 66.6400, 0.015)-(4.3000, 66.5600, 0.015)-(10.0000, 66.7500, 0.015)-(15.7000, 66.5600, 0.015)-(16.0000, 66.6400, 0.015)-(20.0000, 66.7



Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

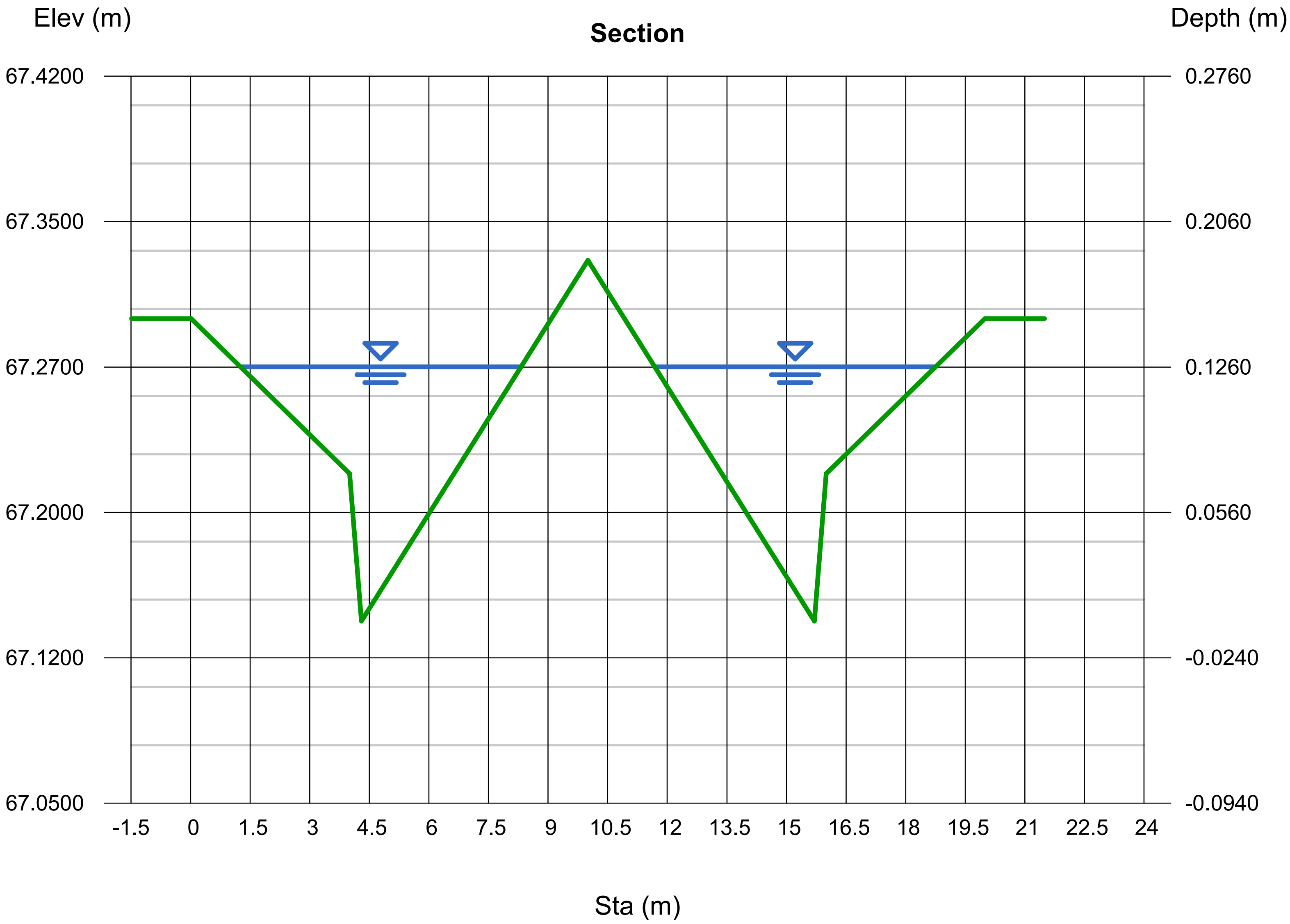
Catchemnt A Section 3

User-defined
Invert Elev (m) = 67.1440
Slope (%) = 0.9000
N-Value = 0.015

Calculations
Compute by: Known Q
Known Q (cms) = 0.6000

Highlighted
Depth (m) = 0.1311
Q (cms) = 0.6000
Area (sqm) = 0.7336
Velocity (m/s) = 0.8179
Wetted Perim (m) = 14.1554
Crit Depth, Yc (m) = 0.1372
Top Width (m) = 14.1310
EGL (m) = 0.1652

(Sta, El, n)-(Sta, El, n)...
(0.0100, 67.3000)-(4.0000, 67.2200, 0.015)-(4.3000, 67.1440, 0.015)-(10.0000, 67.3300, 0.015)-(15.7000, 67.1440, 0.015)-(16.0000, 67.2200, 0.015)-(20.0000, 67.3



Channel Report

Catchemnt B Section 1

User-defined

Invert Elev (m)	=	68.0800
Slope (%)	=	0.5000
N-Value	=	0.015

Calculations

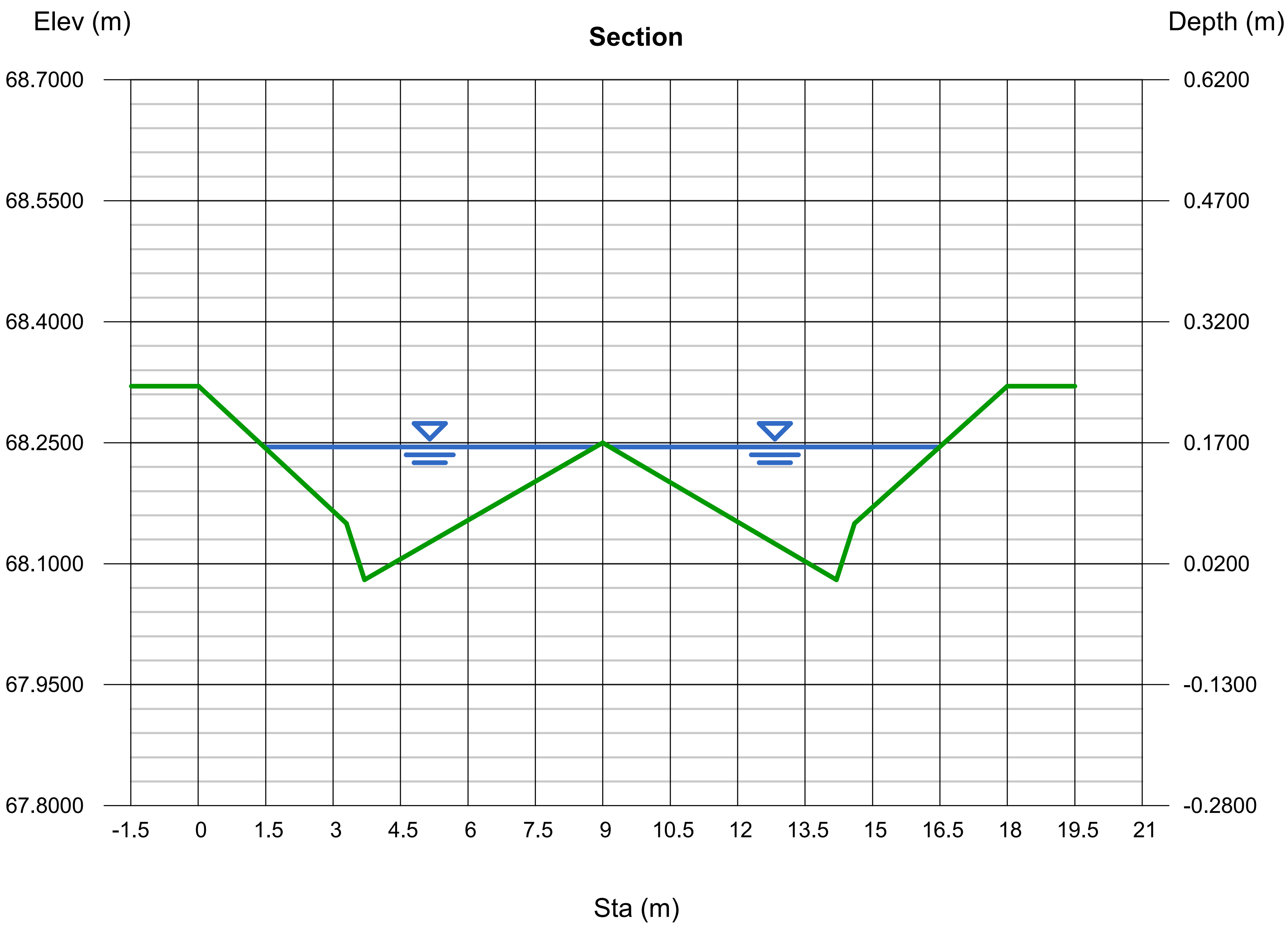
Compute by:	Known Q
Known Q (cms)	= 0.9000

Highlighted

Depth (m)	=	0.1646
Q (cms)	=	0.9000
Area (sqm)	=	1.1164
Velocity (m/s)	=	0.8062
Wetted Perim (m)	=	14.7110
Crit Depth, Yc (m)	=	0.1615
Top Width (m)	=	14.6887
EGL (m)	=	0.1977

(Sta, El, n)-(Sta, El, n)...

(0.0100, 68.3200)-(3.3000, 68.1500, 0.015)-(3.7000, 68.0800, 0.015)-(9.0000, 68.2500, 0.015)-(14.2000, 68.0800, 0.015)-(14.6000, 68.1500, 0.015)-(18.0000, 68.32



Channel Report

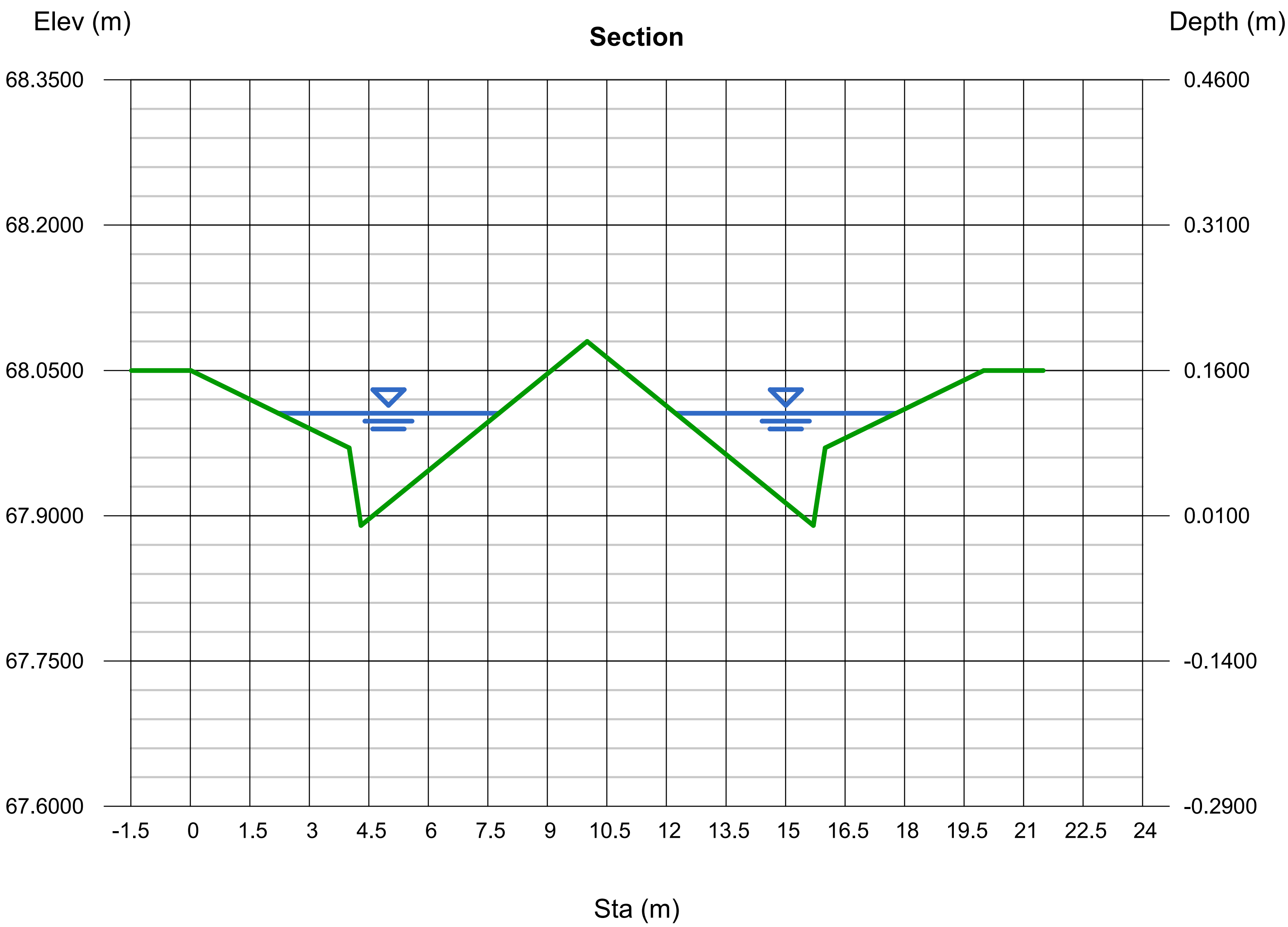
Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt B Section 2

User-defined		**Highlighted**	
Invert Elev (m)	= 67.8900	Depth (m)	= 0.1158
Slope (%)	= 0.5000	Q (cms)	= 0.3000
N-Value	= 0.015	Area (sqm)	= 0.5120
		Velocity (m/s)	= 0.5859
Calculations		Wetted Perim (m)	= 11.1529
Compute by:	Known Q	Crit Depth, Yc (m)	= 0.1128
Known Q (cms)	= 0.3000	Top Width (m)	= 11.1273
		EGL (m)	= 0.1333

(Sta, El, n)-(Sta, El, n)...

(0.0100, 68.0500)-(4.0000, 67.9700, 0.015)-(4.3000, 67.8900, 0.015)-(10.0000, 68.0800, 0.015)-(15.7000, 67.8900, 0.015)-(16.0000, 67.9700, 0.015)-(20.0000, 68.0



Channel Report

Catchemnt B Section 3

User-defined

Invert Elev (m)	=	66.3200
Slope (%)	=	0.7000
N-Value	=	0.015

Calculations

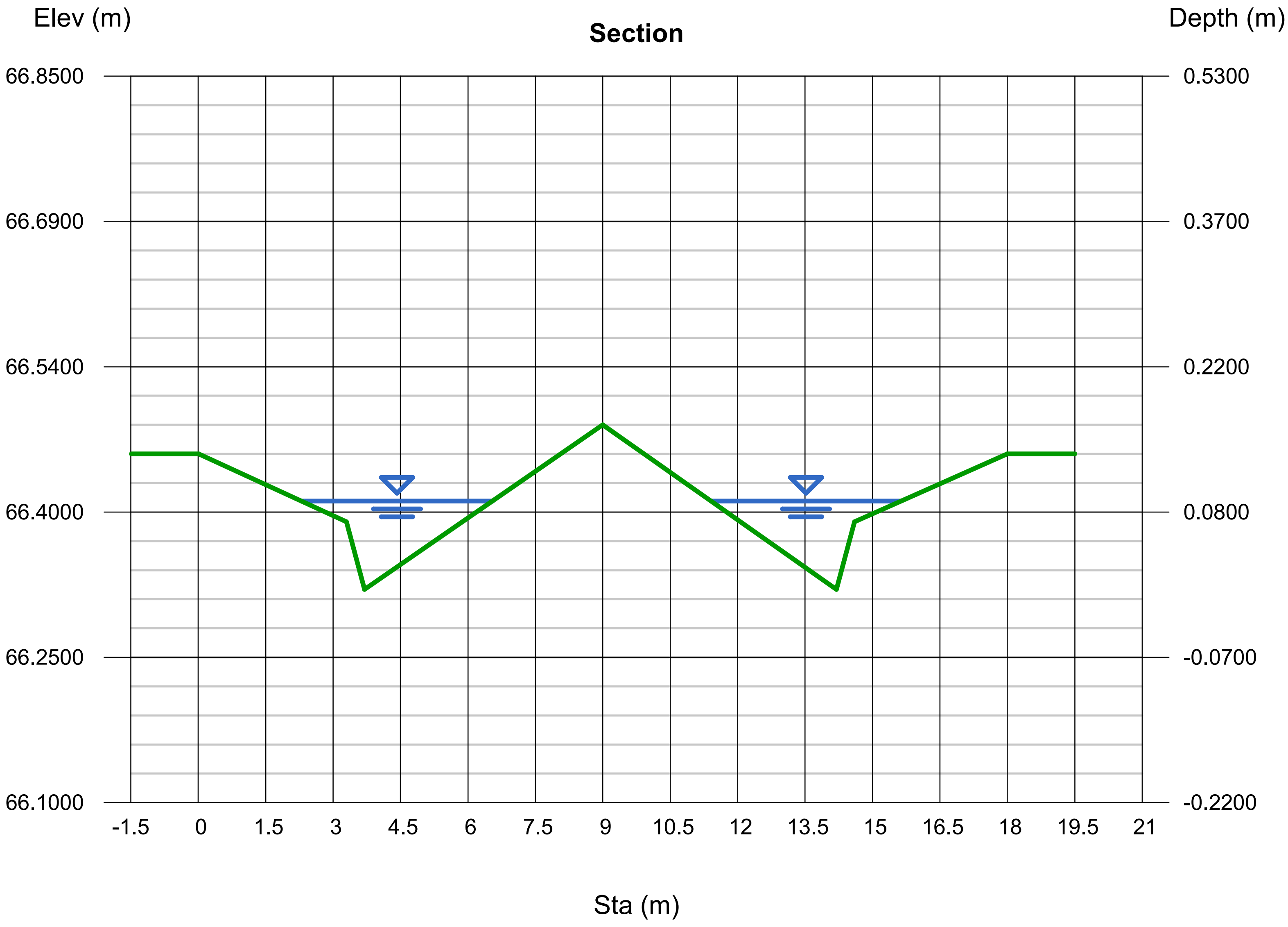
Compute by:		Known Q
Known Q (cms)	=	0.2000

Highlighted

Depth (m)	=	0.0914
Q (cms)	=	0.200
Area (sqm)	=	0.3253
Velocity (m/s)	=	0.6147
Wetted Perim (m)	=	8.5125
Crit Depth, Yc (m)	=	0.0945
Top Width (m)	=	8.4970
EGL (m)	=	0.1107

(Sta, El, n)-(Sta, El, n)...

(0.0100, 66.4600)-(3.3000, 66.3900, 0.015)-(3.7000, 66.3200, 0.015)-(9.0000, 66.4900, 0.015)-(14.2000, 66.3200, 0.015)-(14.6000, 66.3900, 0.015)-(18.0000, 66.46



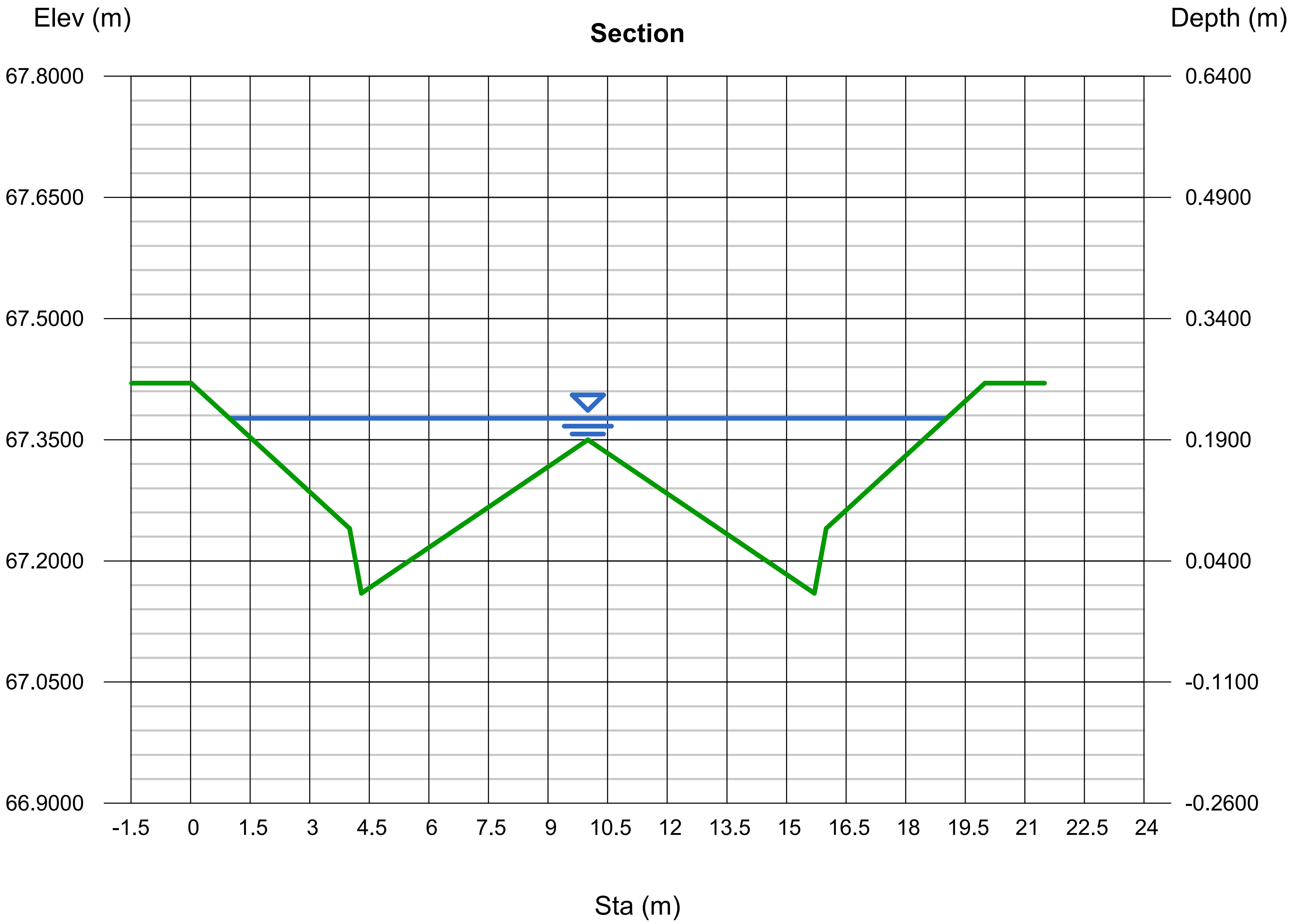
Channel Report

Catchemnt D Section 4

User-defined		**Highlighted**	
Invert Elev (m)	= 67.1600	Depth (m)	= 0.2164
Slope (%)	= 0.3000	Q (cms)	= 1.5000
N-Value	= 0.015	Area (sqm)	= 1.9030
		Velocity (m/s)	= 0.7882
Calculations		Wetted Perim (m)	= 18.0888
Compute by:	Known Q	Crit Depth, Yc (m)	= 0.2012
Known Q (cms)	= 1.5000	Top Width (m)	= 18.0554
		EGL (m)	= 0.2481

(Sta, El, n)-(Sta, El, n)...

(0.0100, 67.4200)-(4.0000, 67.2400, 0.015)-(4.3000, 67.1600, 0.015)-(10.0000, 67.3500, 0.015)-(15.7000, 67.1600, 0.015)-(16.0000, 67.2400, 0.015)-(20.0000, 67.4



Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

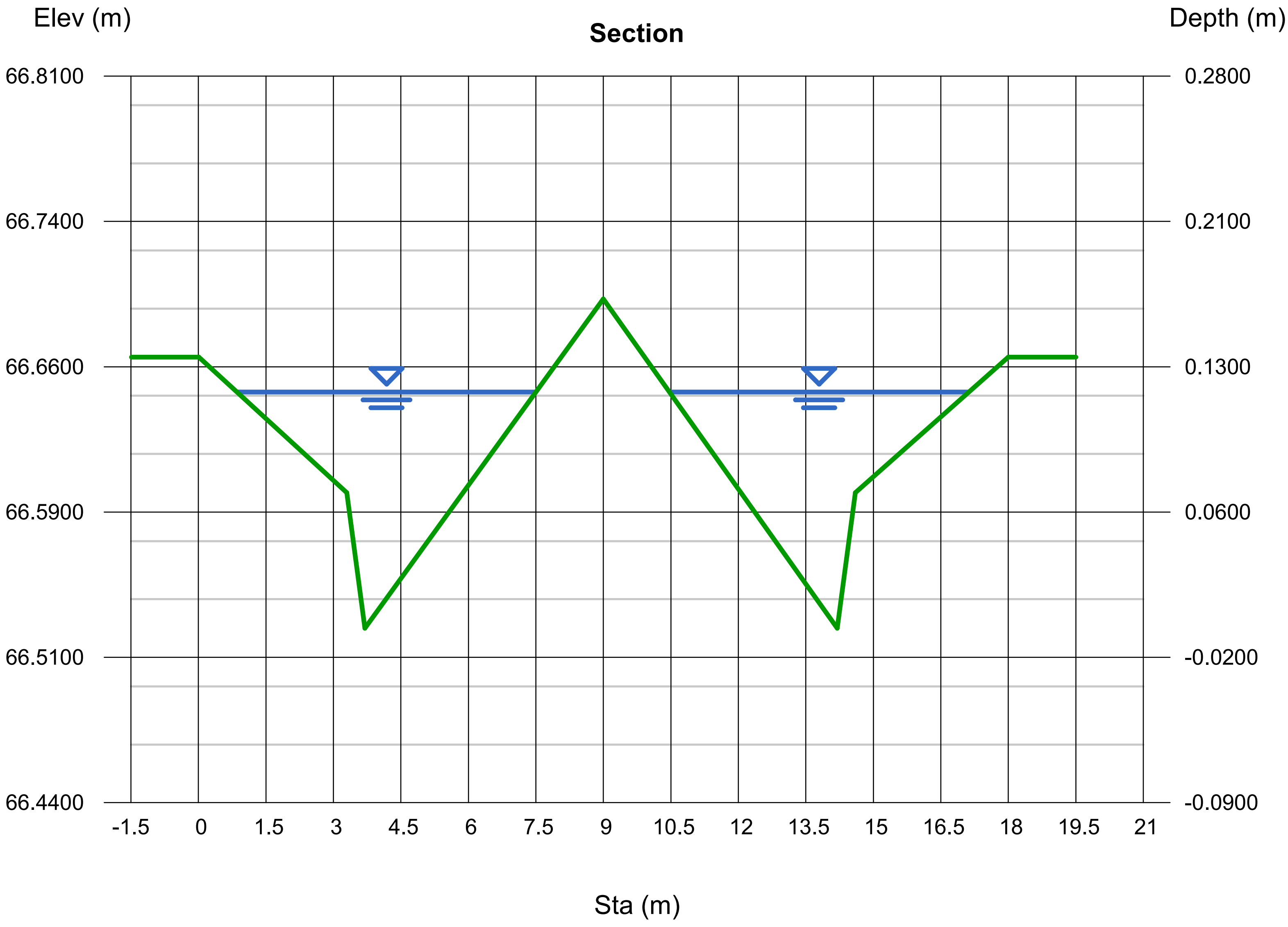
Catchemnt B Section 5

User-defined
Invert Elev (m) = 66.5300
Slope (%) = 1.1000
N-Value = 0.015

Calculations
Compute by: Known Q
Known Q (cms) = 0.6000

Highlighted
Depth (m) = 0.1219
Q (cms) = 0.6000
Area (sqm) = 0.6574
Velocity (m/s) = 0.9127
Wetted Perim (m) = 13.3098
Crit Depth, Yc (m) = 0.1341
Top Width (m) = 13.2927
EGL (m) = 0.1644

(Sta, El, n)-(Sta, El, n)...
(0.0100, 66.6700)-(3.3000, 66.6000, 0.015)-(3.7000, 66.5300, 0.015)-(9.0000, 66.7000, 0.015)-(14.2000, 66.5300, 0.015)-(14.6000, 66.6000, 0.015)-(18.0000, 66.67



Channel Report

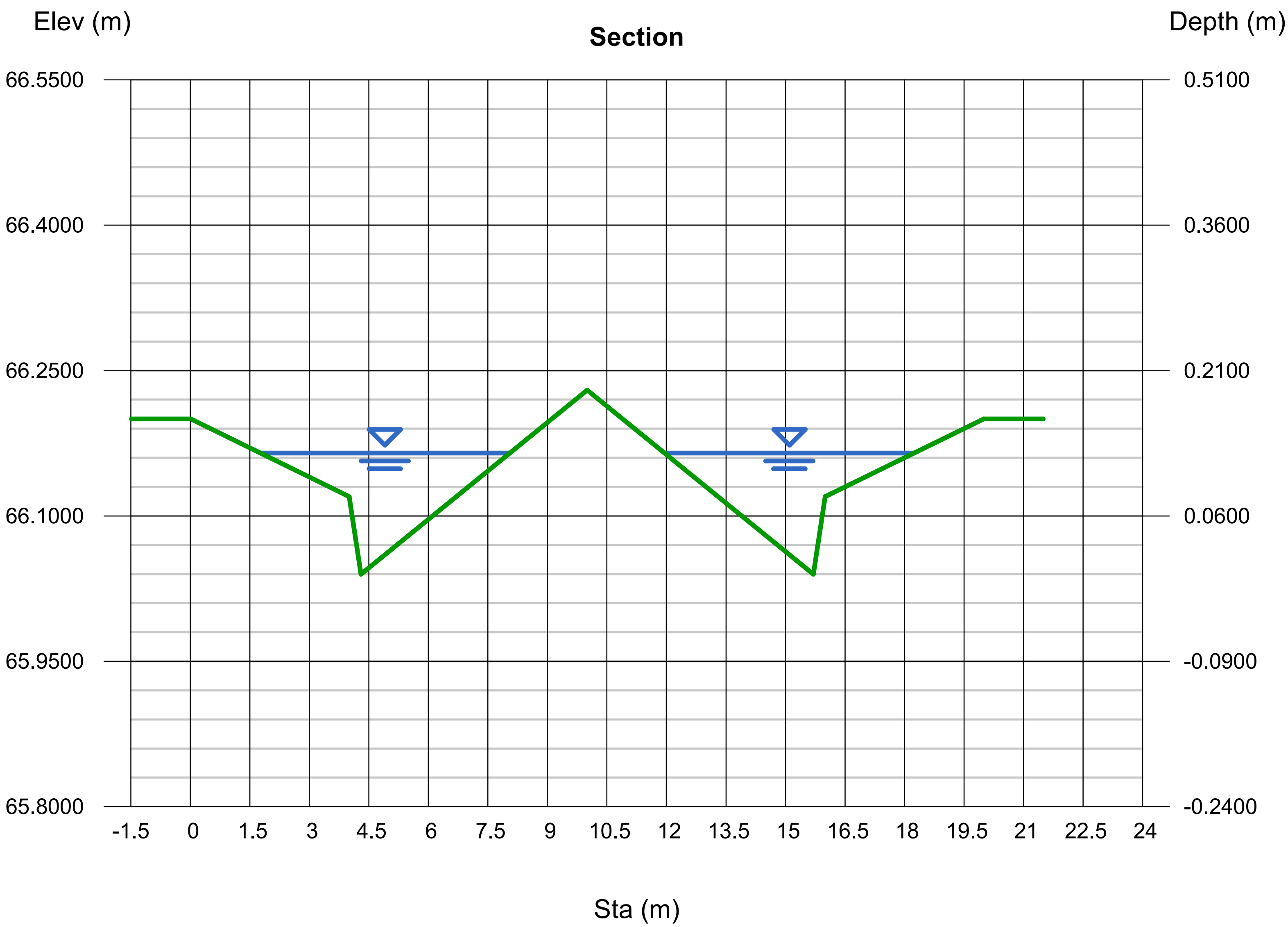
Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt C Section 1

User-defined		**Highlighted**	
Invert Elev (m)	= 66.0400	Depth (m)	= 0.1250
Slope (%)	= 0.9000	Q (cms)	= 0.5000
N-Value	= 0.015	Area (sqm)	= 0.6205
		Velocity (m/s)	= 0.8058
Calculations		Wetted Perim (m)	= 12.6157
Compute by:	Known Q	Crit Depth, Yc (m)	= 0.1341
Known Q (cms)	= 0.5000	Top Width (m)	= 12.5896
		EGL (m)	= 0.1581

(Sta, El, n)-(Sta, El, n)...

(0.0100, 66.2000)-(4.0000, 66.1200, 0.015)-(4.3000, 66.0400, 0.015)-(10.0000, 66.2300, 0.015)-(15.7000, 66.0400, 0.015)-(16.0000, 66.1200, 0.015)-(20.0000, 66.2



Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. | Monday, Mar 17 2025

Catchemnt D Section 1

User-defined

Invert Elev (m)	=	66.3000
Slope (%)	=	0.5000
N-Value	=	0.015

Calculations

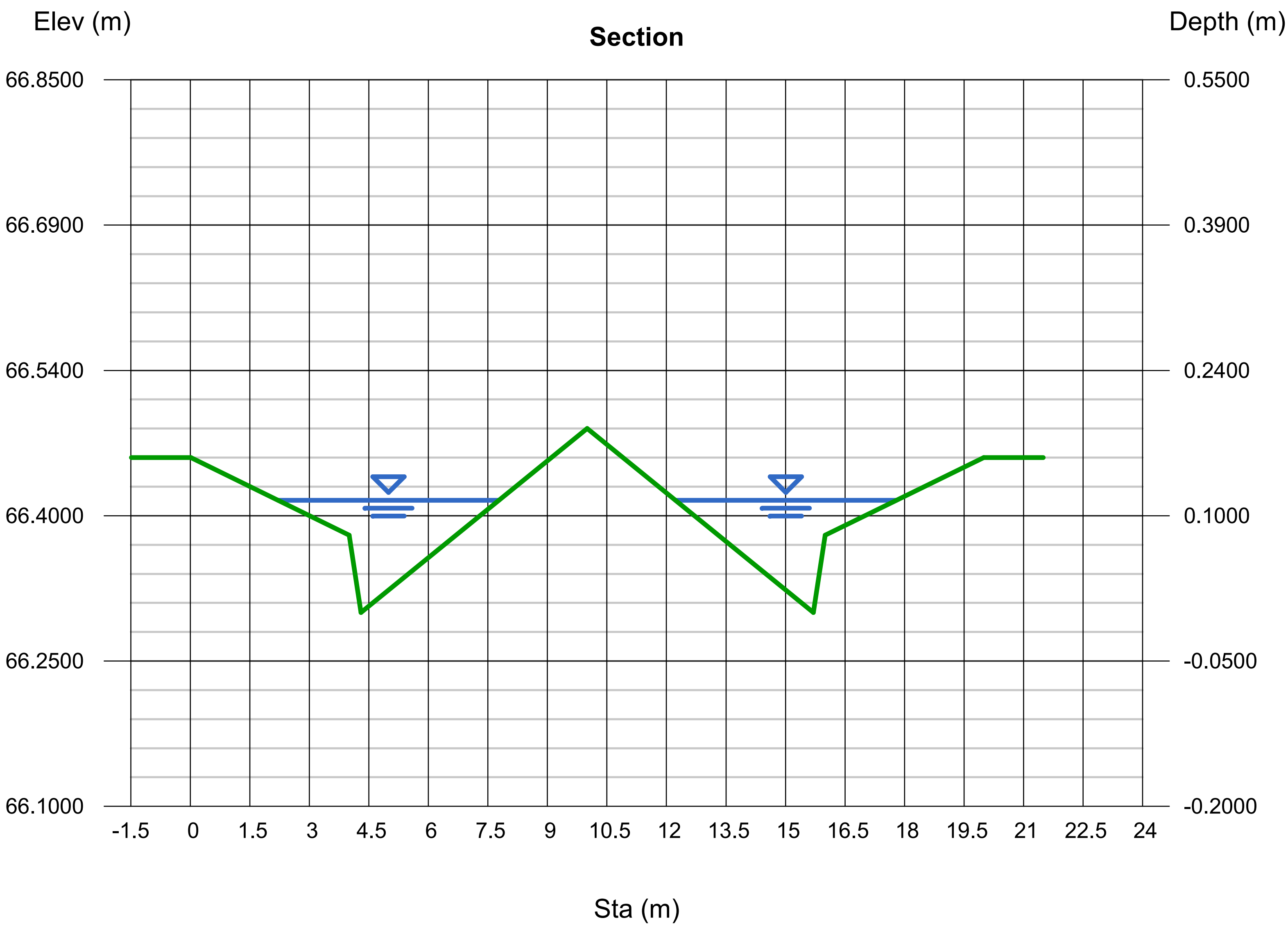
Compute by:		Known Q
Known Q (cms)	=	0.3000

Highlighted

Depth (m)	=	0.1158
Q (cms)	=	0.3000
Area (sqm)	=	0.5121
Velocity (m/s)	=	0.5858
Wetted Perim (m)	=	11.1542
Crit Depth, Yc (m)	=	0.1128
Top Width (m)	=	11.1286
EGL (m)	=	0.1333

(Sta, El, n)-(Sta, El, n)...

(0.0100, 66.4600)-(4.0000, 66.3800, 0.015)-(4.3000, 66.3000, 0.015)-(10.0000, 66.4900, 0.015)-(15.7000, 66.3000, 0.015)-(16.0000, 66.3800, 0.015)-(20.0000, 66.4



Channel Report

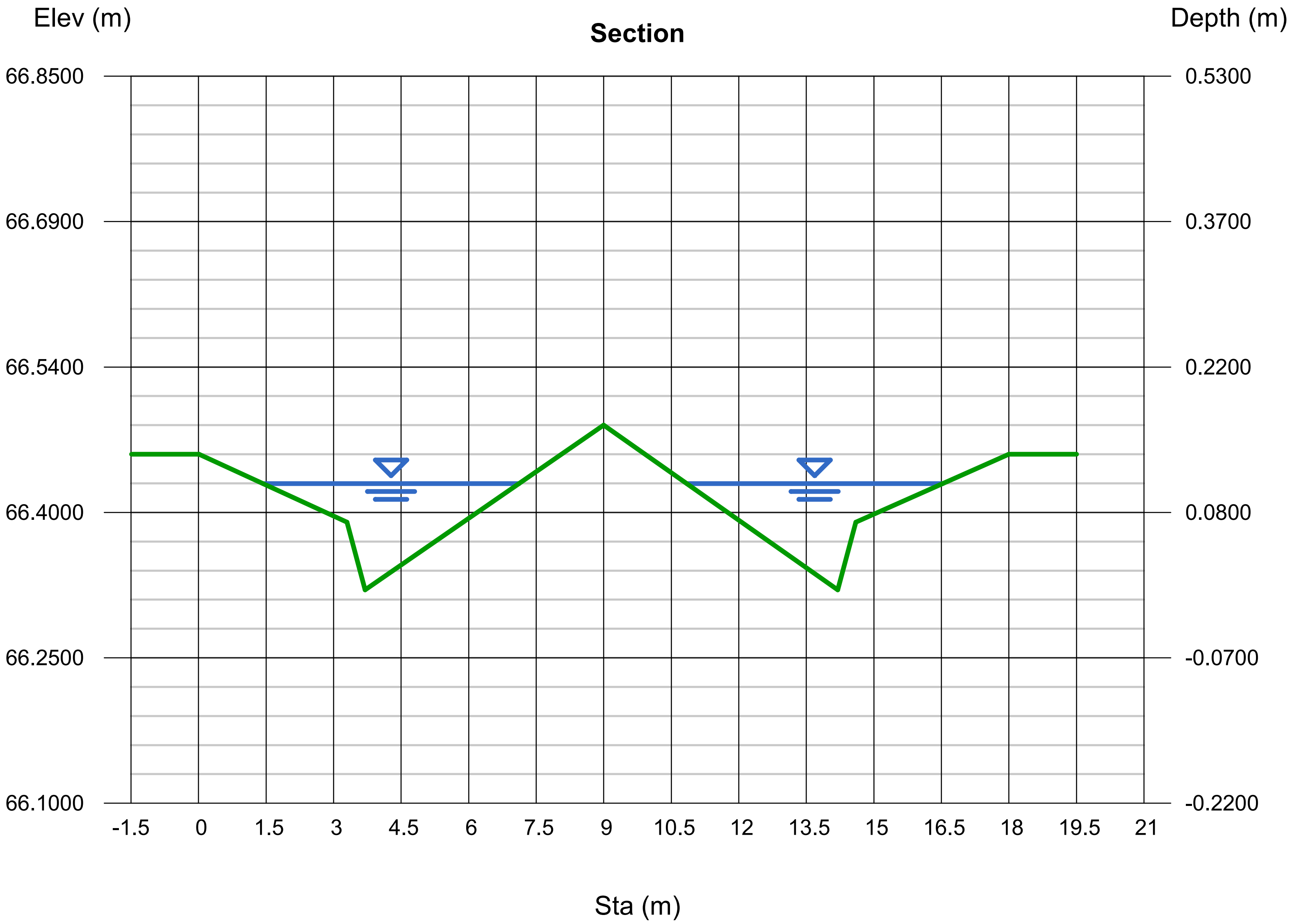
Catchemnt D Section 2

User-defined
Invert Elev (m) = 66.3200
Slope (%) = 0.5000
N-Value = 0.015

Calculations
Compute by: Known Q
Known Q (cms) = 0.3000

Highlighted
Depth (m) = 0.1097
Q (cms) = 0.3000
Area (sqm) = 0.5070
Velocity (m/s) = 0.5917
Wetted Perim (m) = 11.3907
Crit Depth, Yc (m) = 0.1067
Top Width (m) = 11.3742
EGL (m) = 0.1276

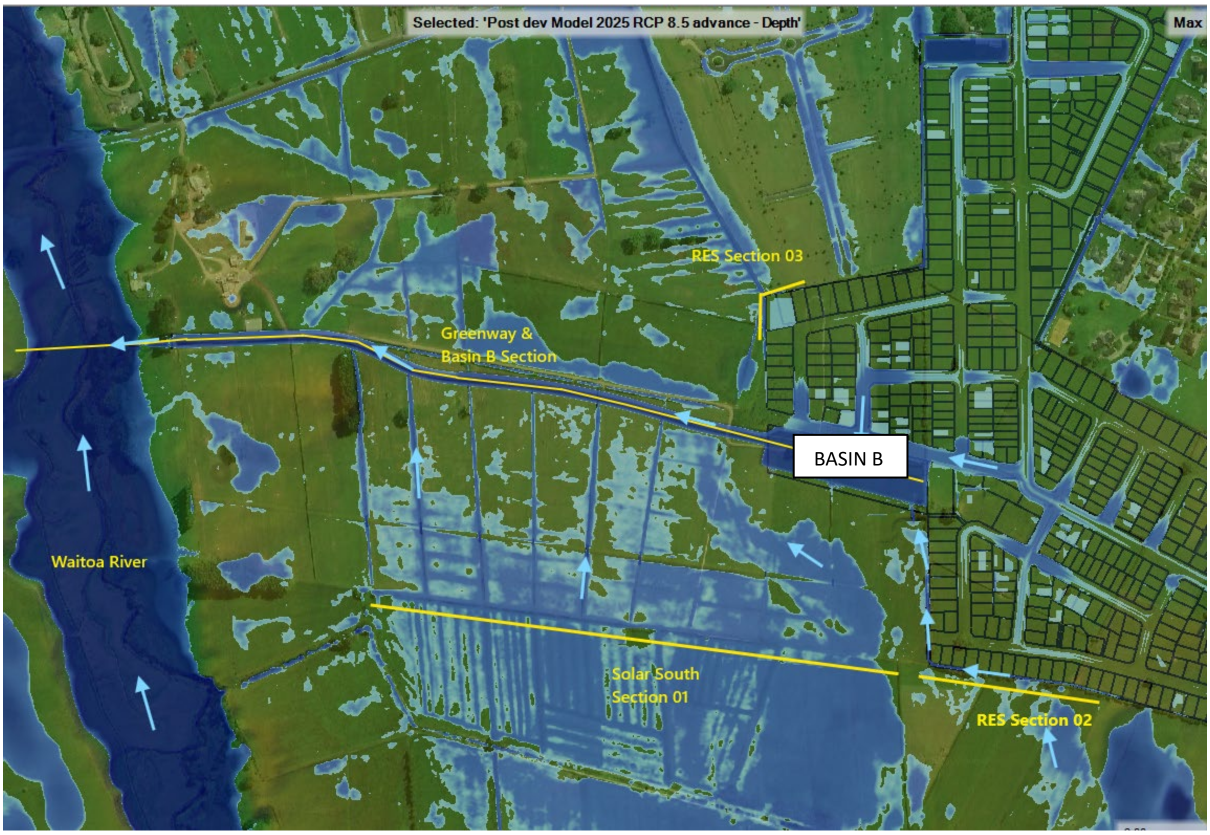
(Sta, El, n)-(Sta, El, n)...
(0.0100, 66.4600)-(3.3000, 66.3900, 0.015)-(3.7000, 66.3200, 0.015)-(9.0000, 66.4900, 0.015)-(14.2000, 66.3200, 0.015)-(14.6000, 66.3900, 0.015)-(18.0000, 66.46



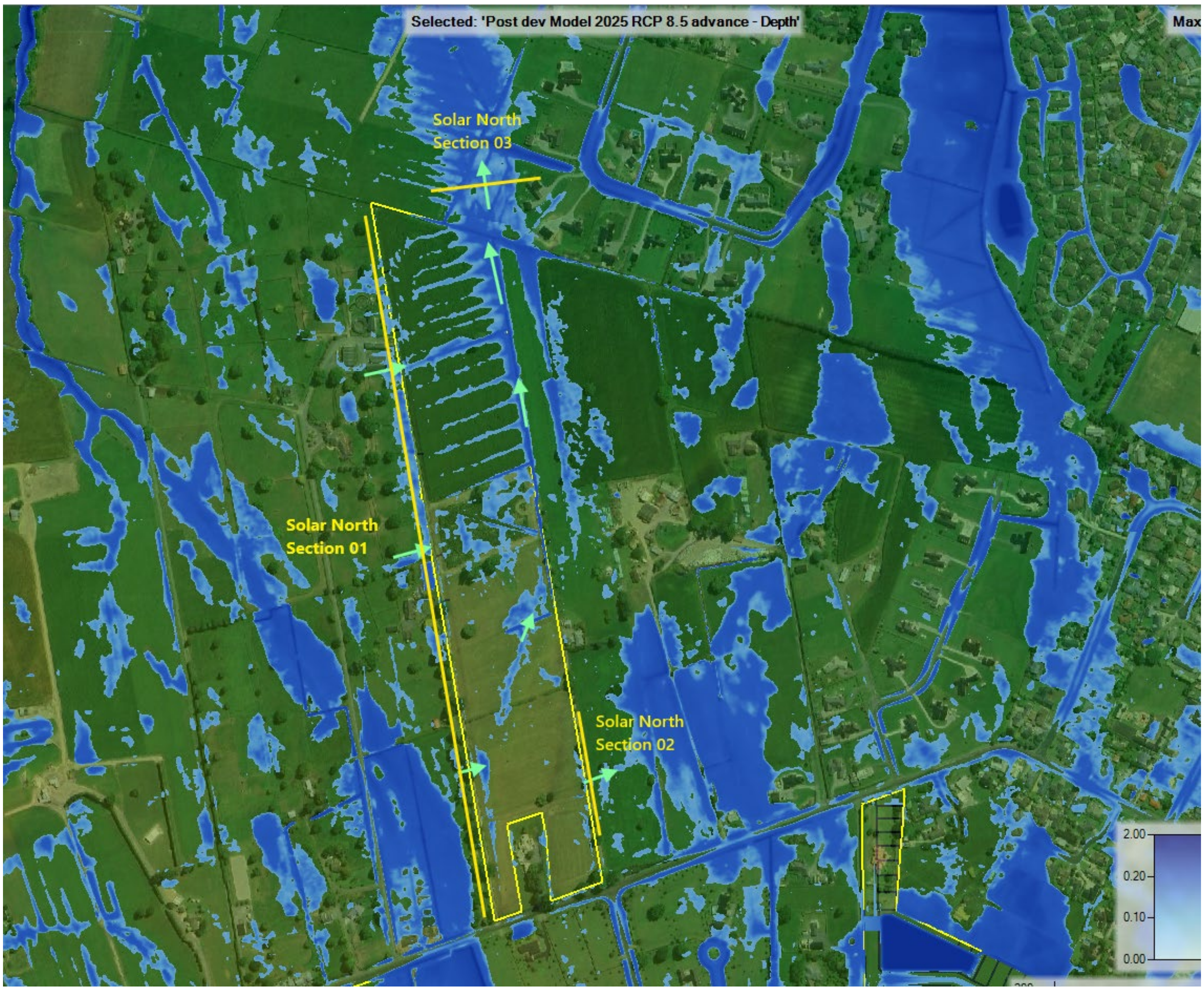
APPENDIX C - CMW GEOTECH REPORT- BOUNDED WITH INFRASTRUCTURE REPORTS

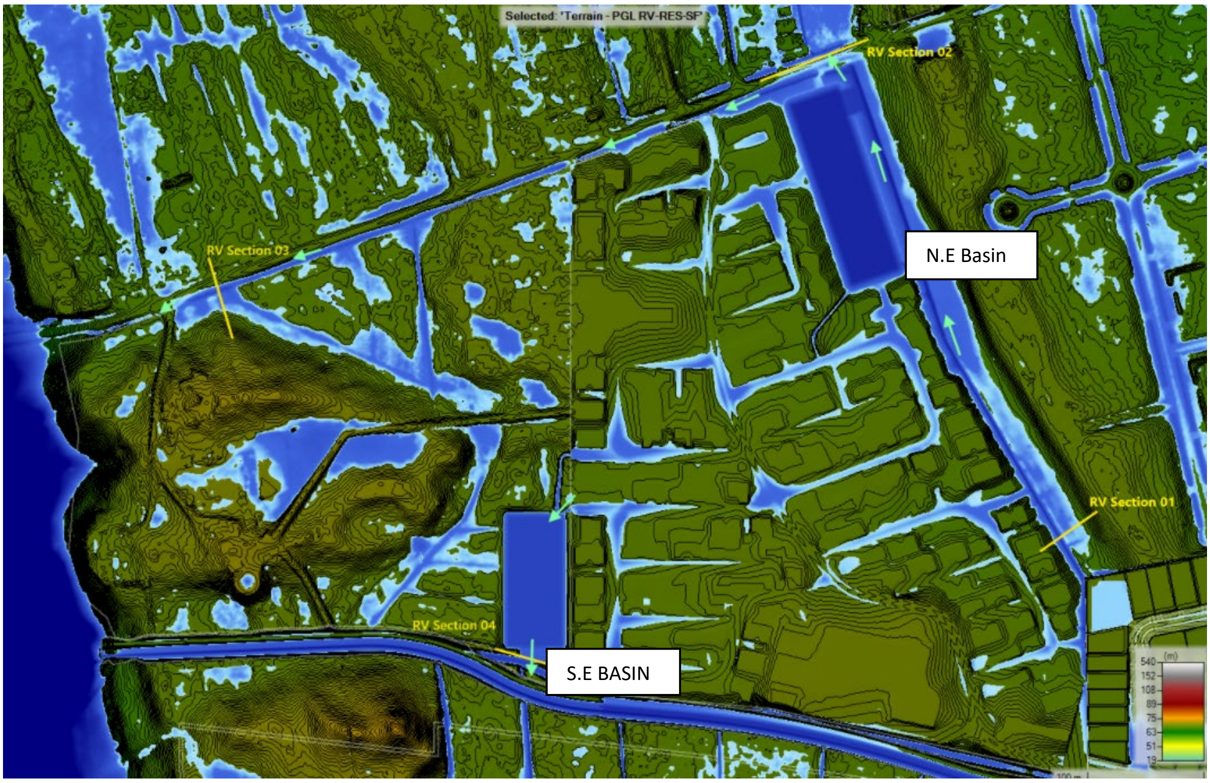
APPENDIX D – STORMWATER DEVICE OPERATIONS MAINTENANCE PLAN

<u>**POST DEVELOMENT RESULTS RCP 8.5**</u>

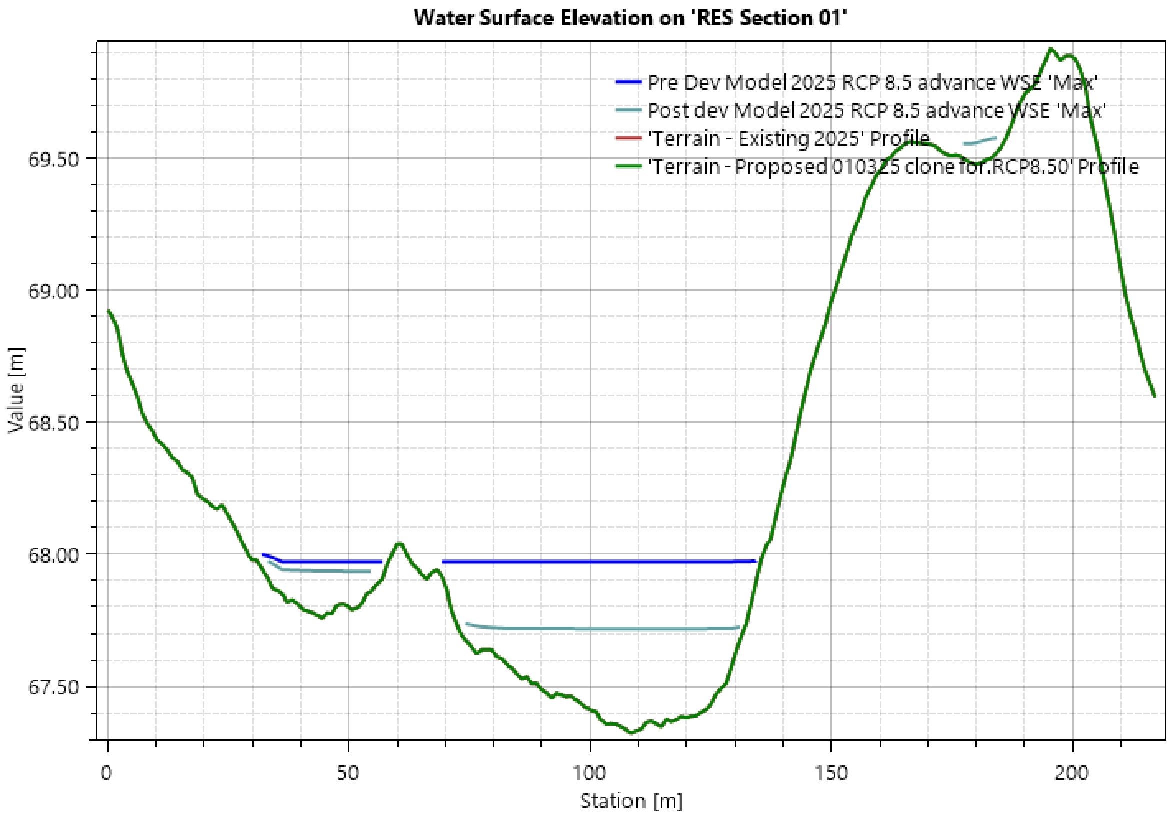

Selected: 'Post dev Model 2025 RCP 8.5 advance - Depth'
Max
RES Section 03
Greenway &
Basin B Section
BASIN B
Waitoa River
Solar South
Section 01
RES Section 02

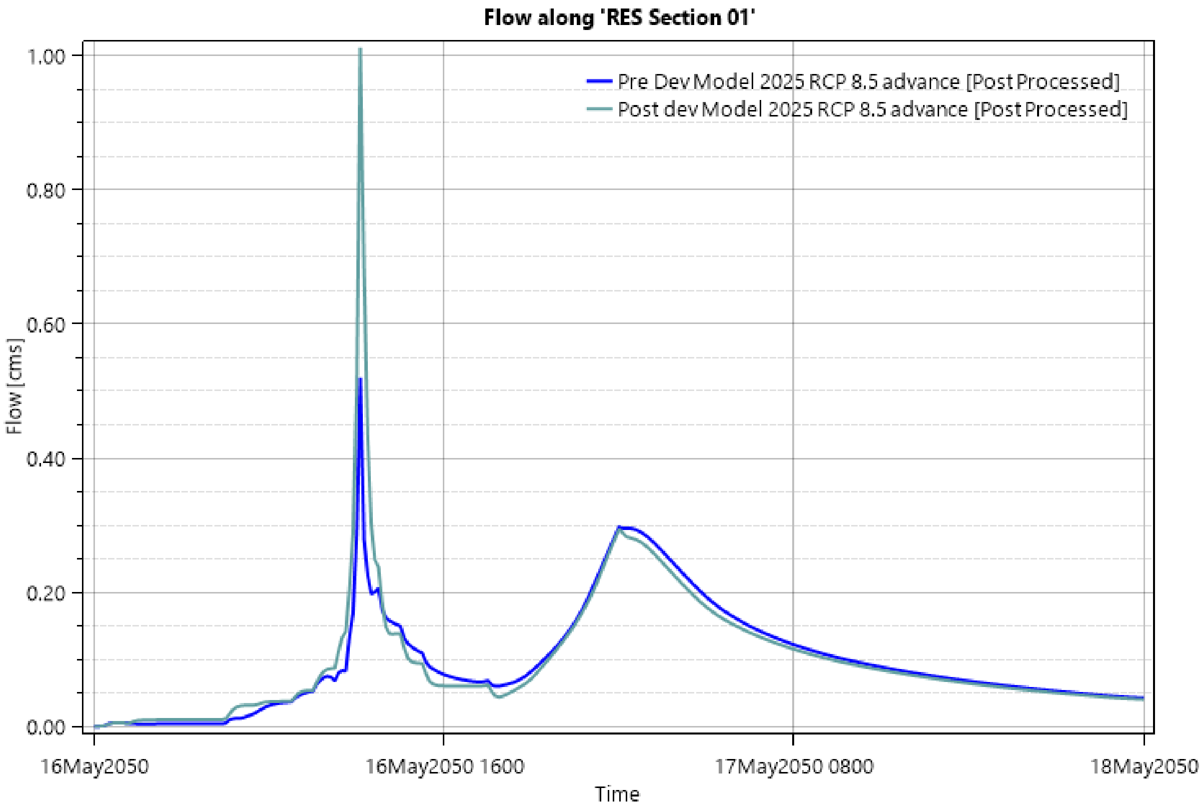

Selected: 'Post dev Model 2025 RCP 8.5 advance - Depth'
Max
RES Section 06
RES Section 07
BASIN D
RES Section 05
RES Section 04
BASIN C
RES Section 08
2.00

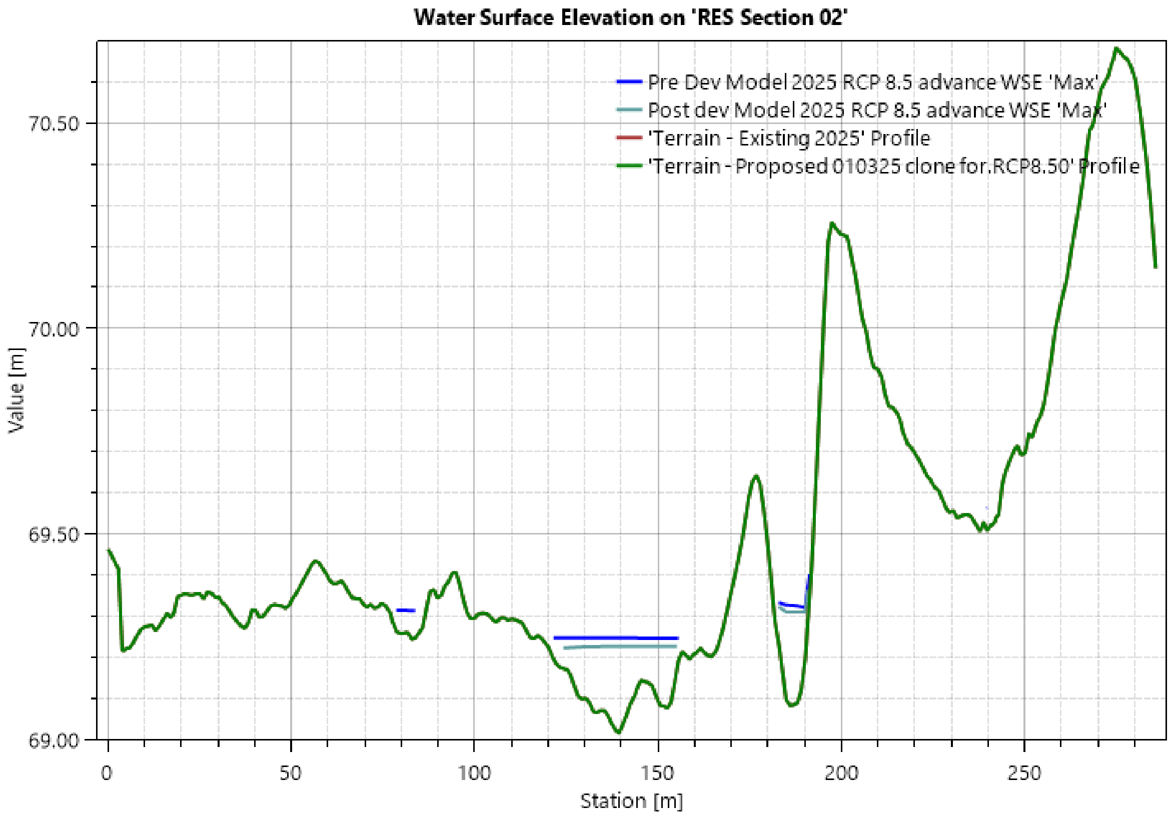

Selected: 'Post dev Model 2025 RCP 8.5 advance - Depth'
Max
Solar North
Section 03
Solar North
Section 01
Solar North
Section 02
2.00
0.20
0.10
0.00

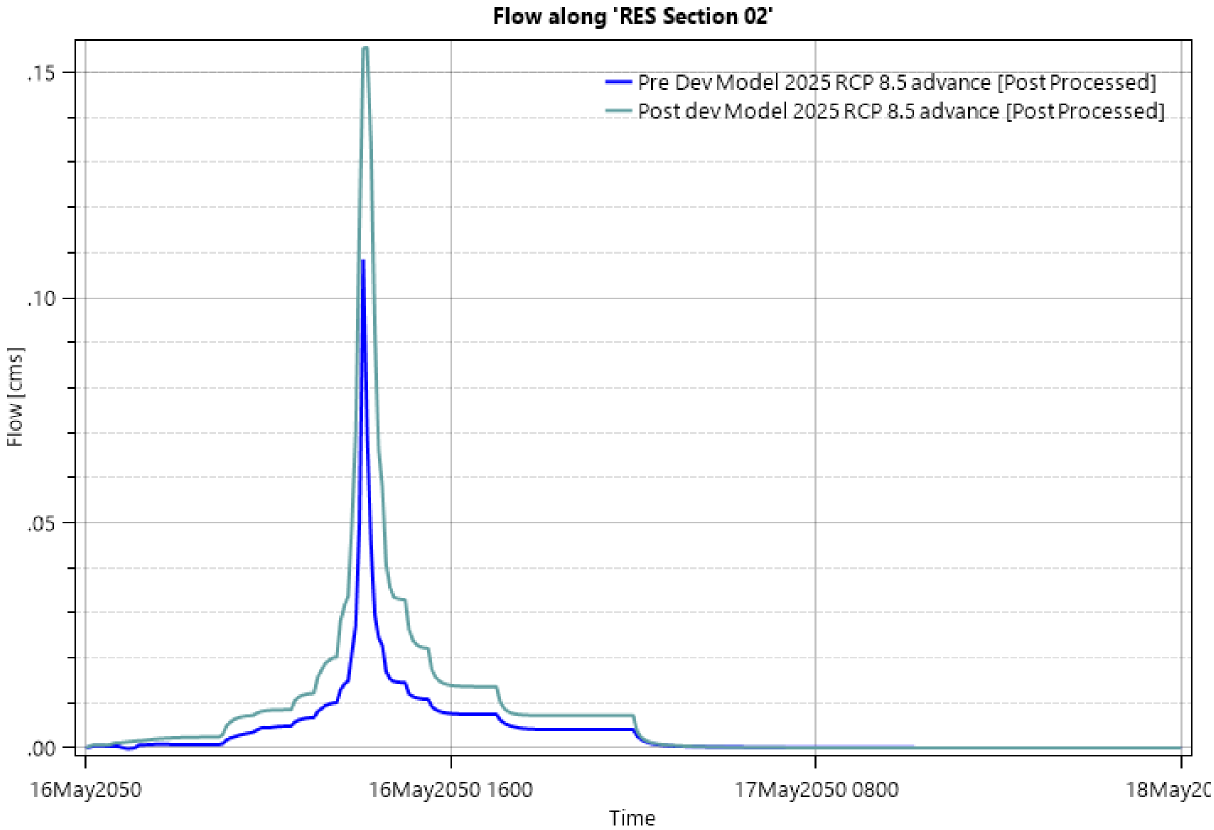

Selected: 'Terrain - PGL RV-RES-SF'
RV Section 02
RV Section 03
N.E Basin
RV Section 01
RV Section 04
S.E BASIN

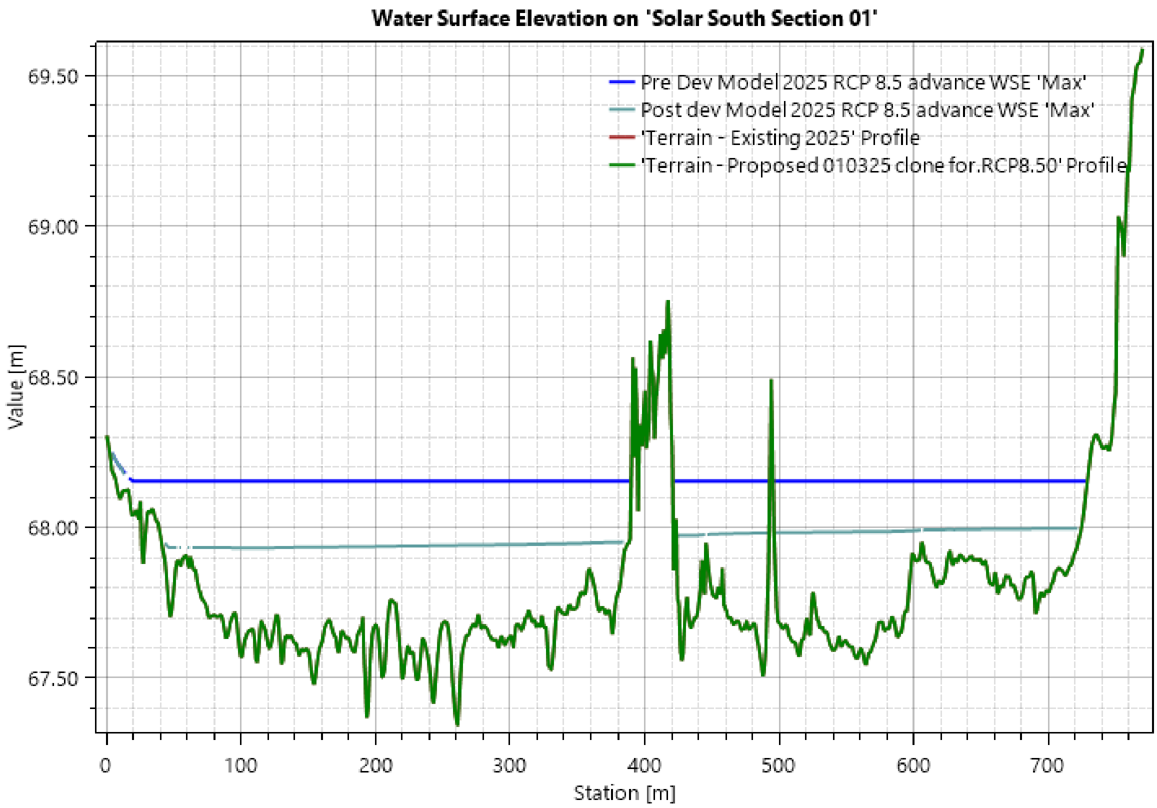
HEC RAS RESULTS – SECTIONS PRE & POST

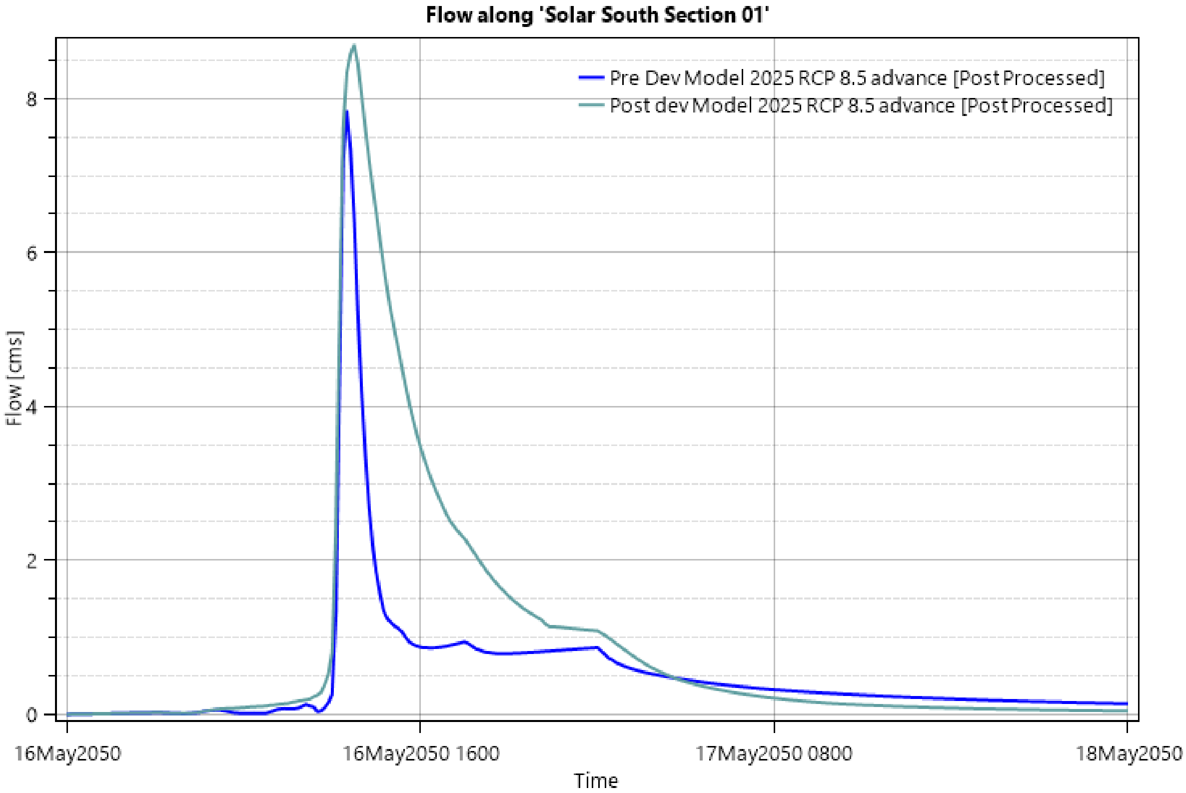



Flow along 'RES Section 01'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
1.00
0.80
0.60
0.40
0.20
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

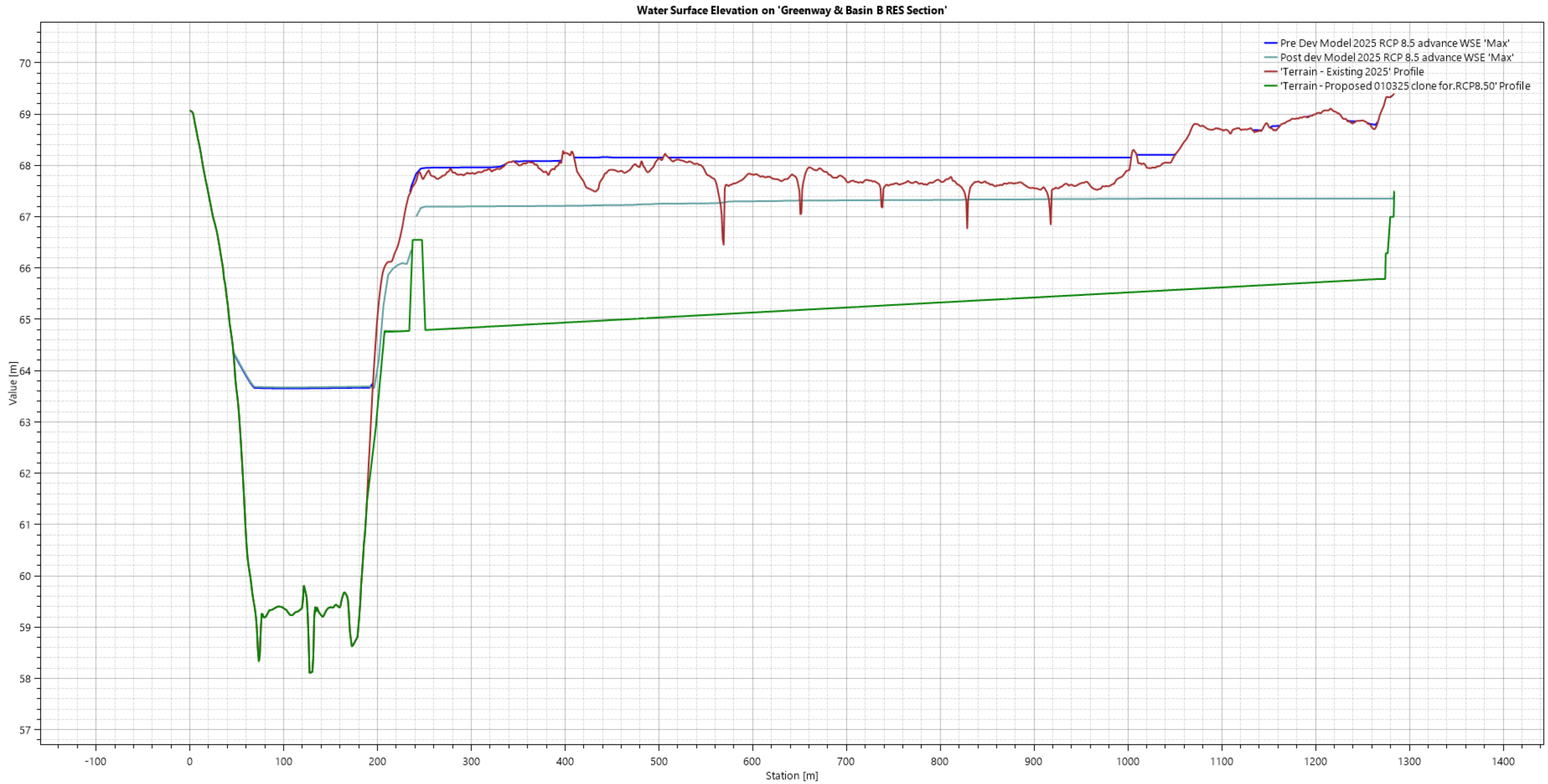

Water Surface Elevation on 'RES Section 02'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
70.50
70.00
69.50
69.00
0
50
100
150
200
250
Station [m]

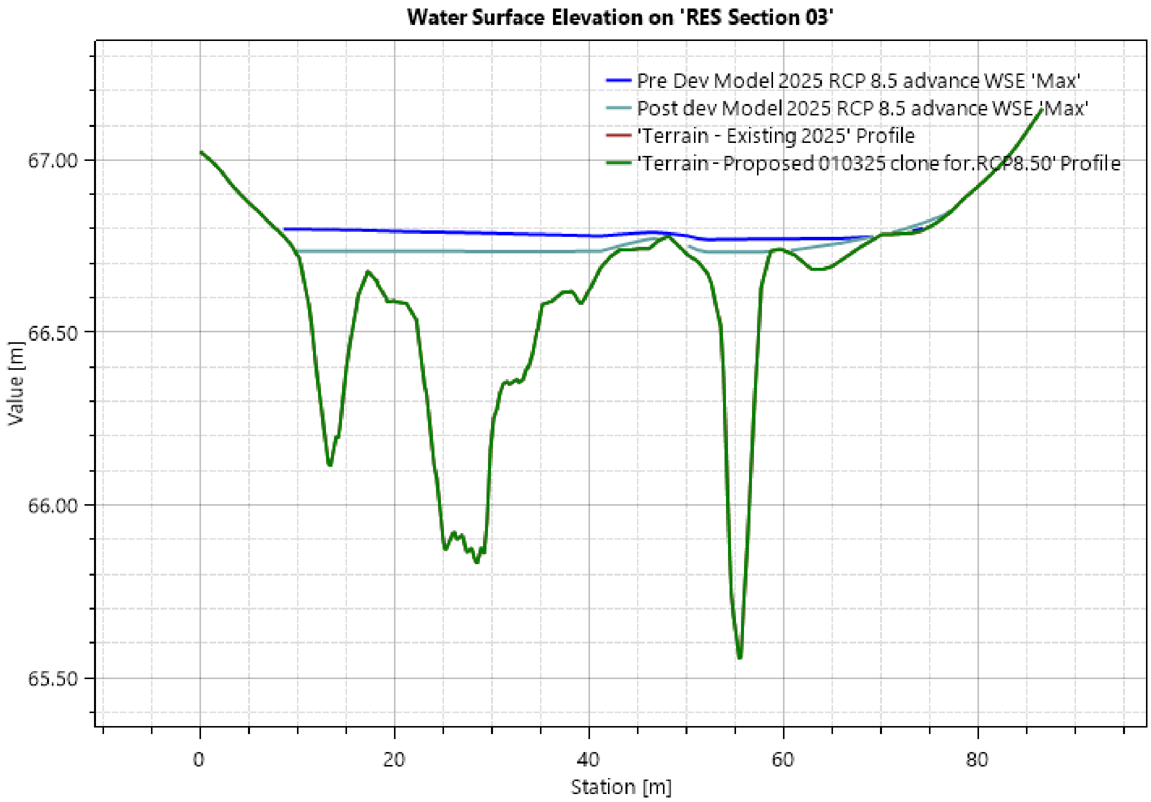

Flow along 'RES Section 02'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
.15
.10
.05
.00
16May2050
16May2050 1600
17May2050 0800
18May20
Time

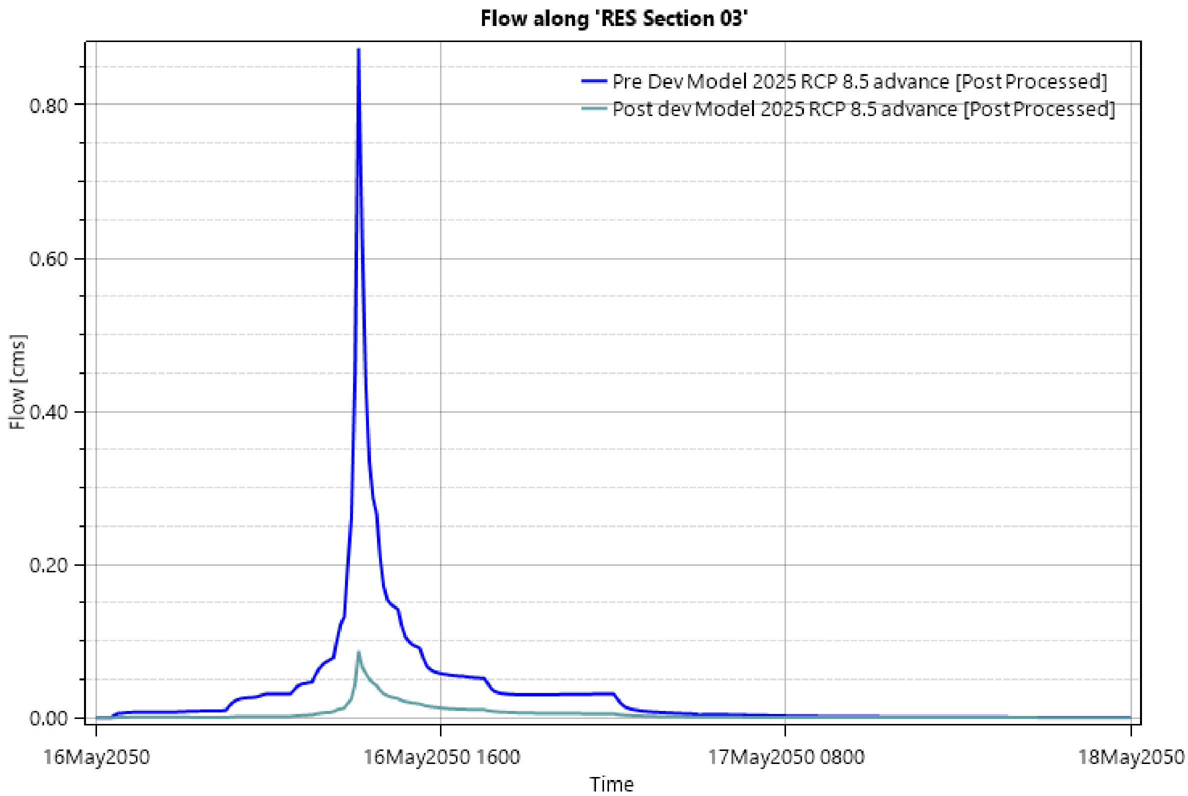

Water Surface Elevation on 'Solar South Section 01'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
69.50
69.00
68.50
68.00
67.50
0
100
200
300
400
500
600
700
Station [m]

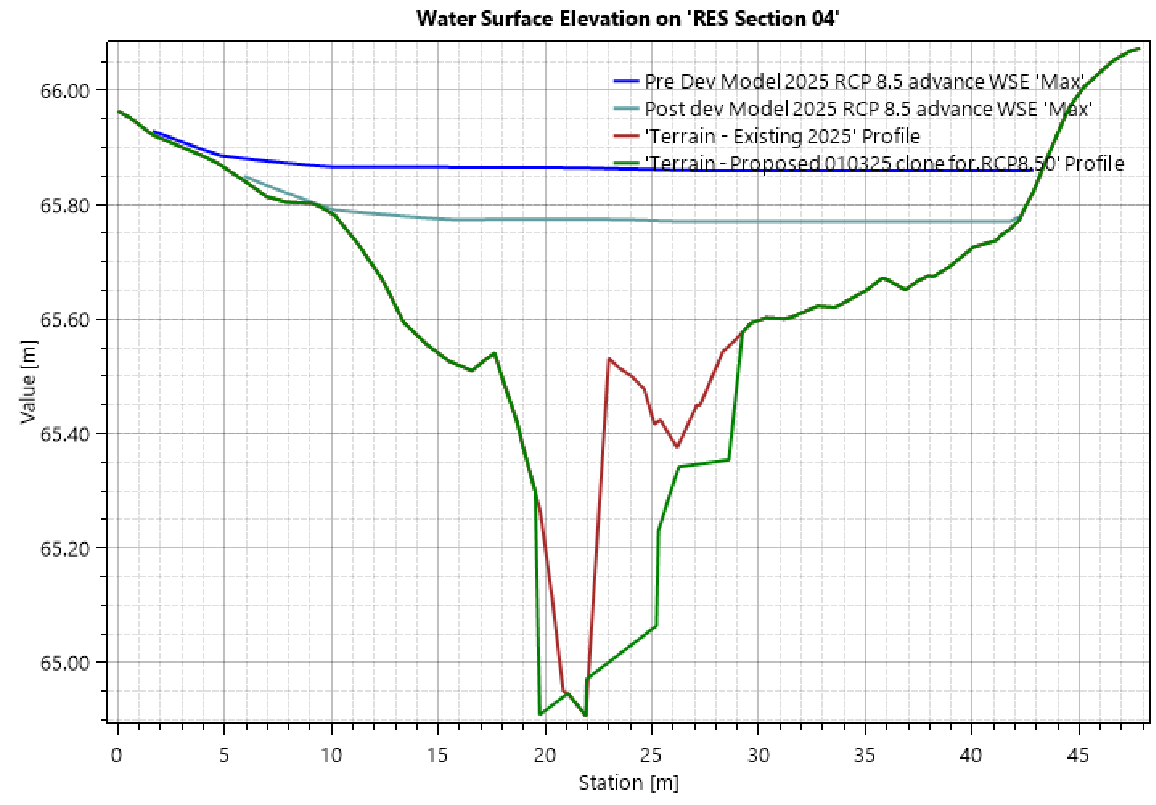


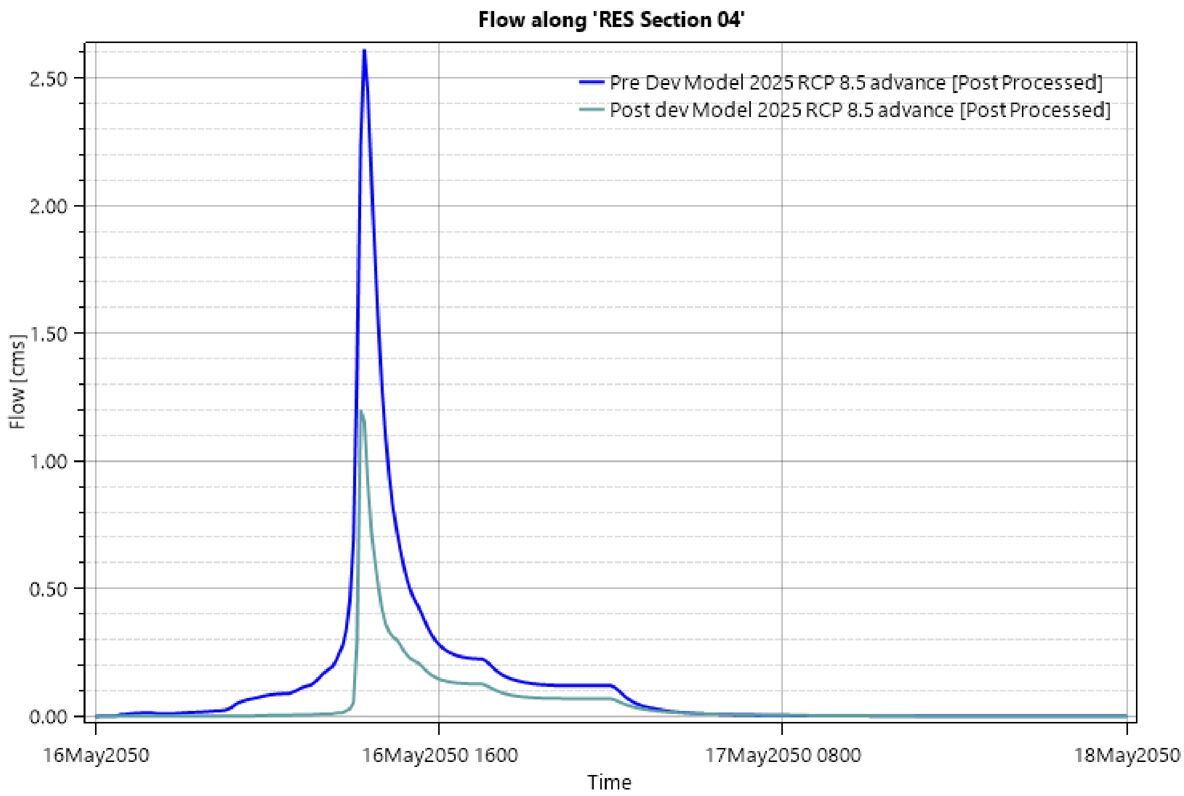
Flow along 'Solar South Section 01'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
0
2
4
6
8
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

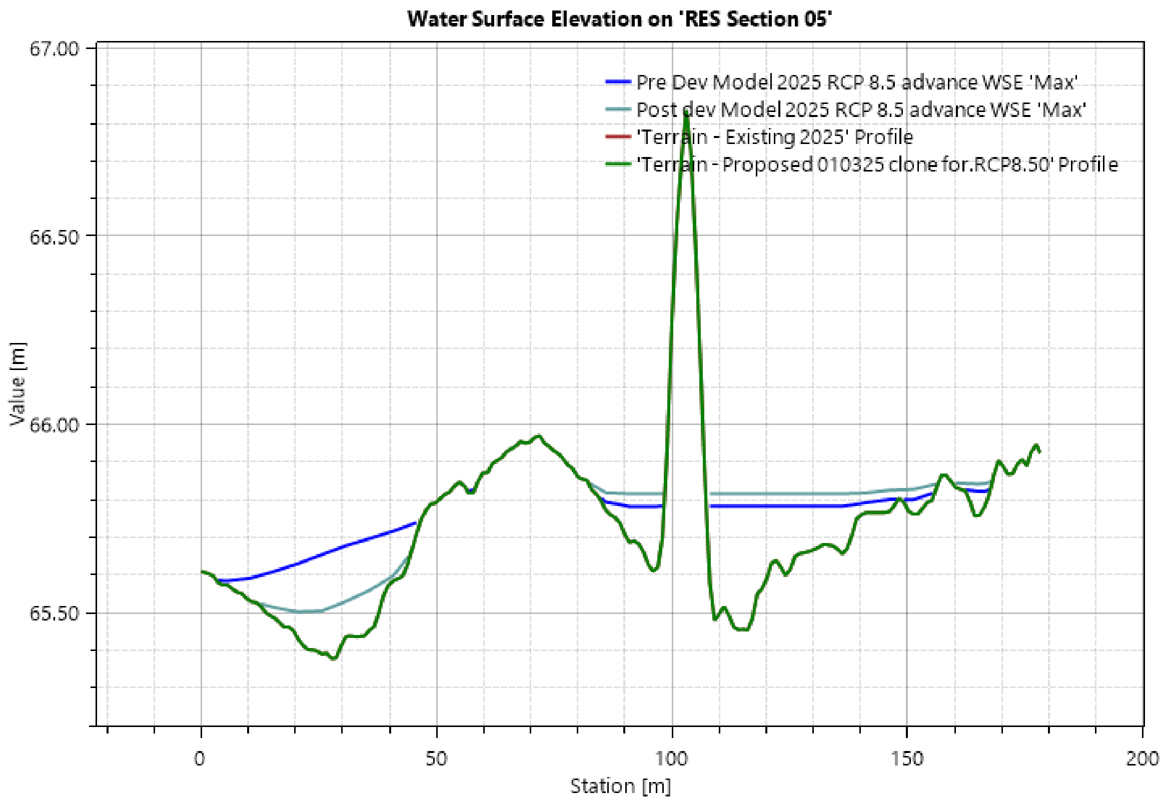

Water Surface Elevation on 'Greenway & Basin B RES Section'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
Station [m]
70
69
68
67
66
65
64
63
62
61
60
59
58
57
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400

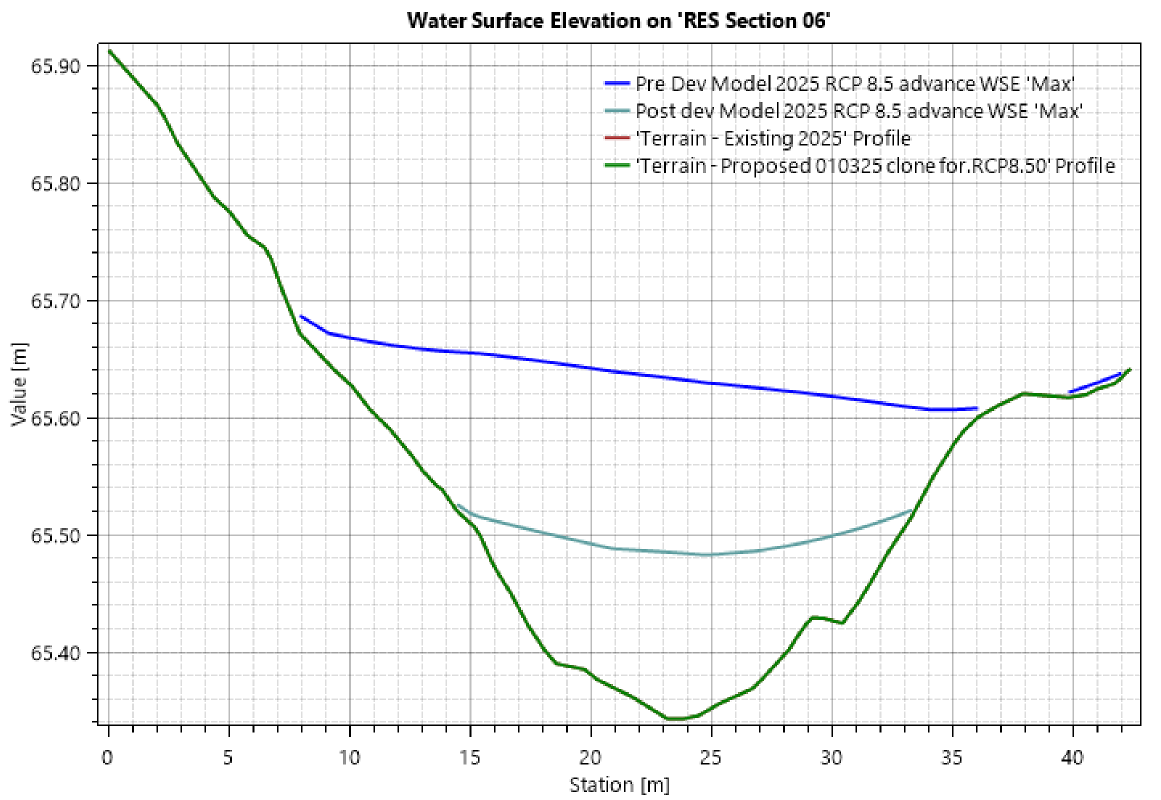

Water Surface Elevation on 'RES Section 03'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
67.00
66.50
66.00
65.50
Value [m]
0
20
40
60
80
Station [m]

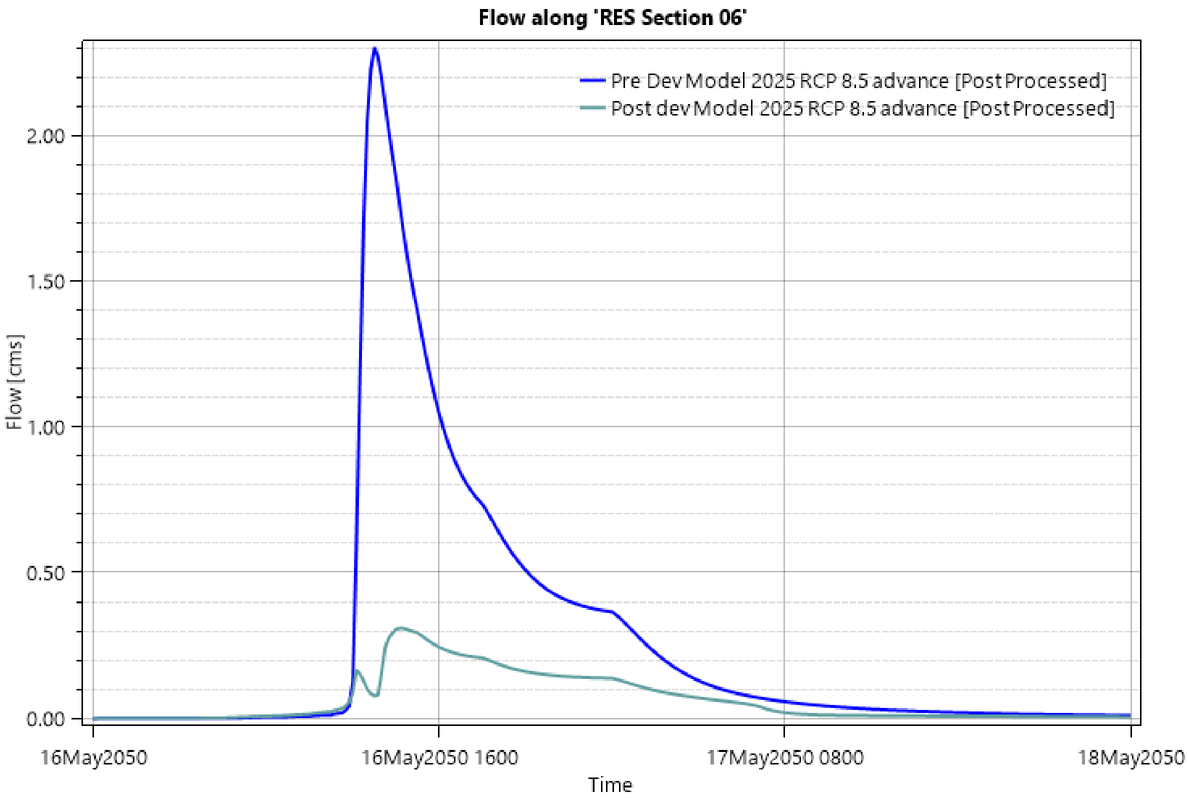

Flow along 'RES Section 03'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
0.80
0.60
0.40
0.20
0.00
Flow [cms]
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

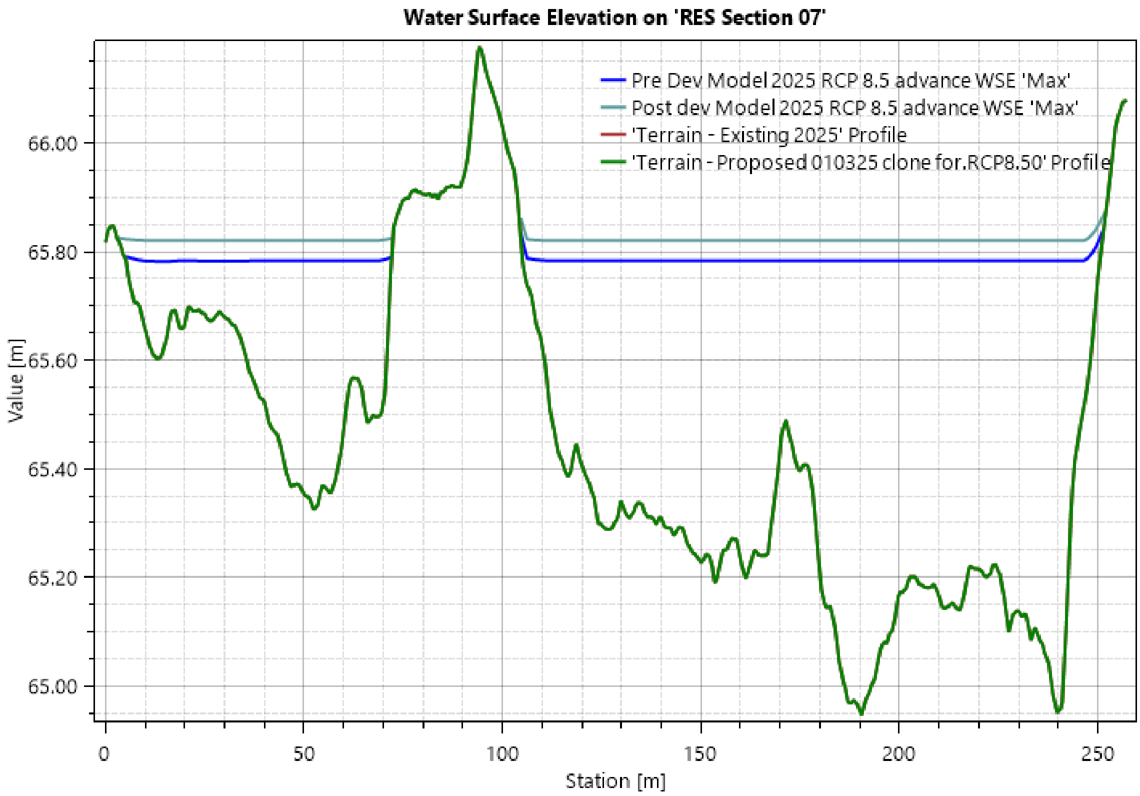

Water Surface Elevation on 'RES Section 04'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for RCP8.50' Profile
Value [m]
Station [m]
66.00
65.80
65.60
65.40
65.20
65.00
0
5
10
15
20
25
30
35
40
45

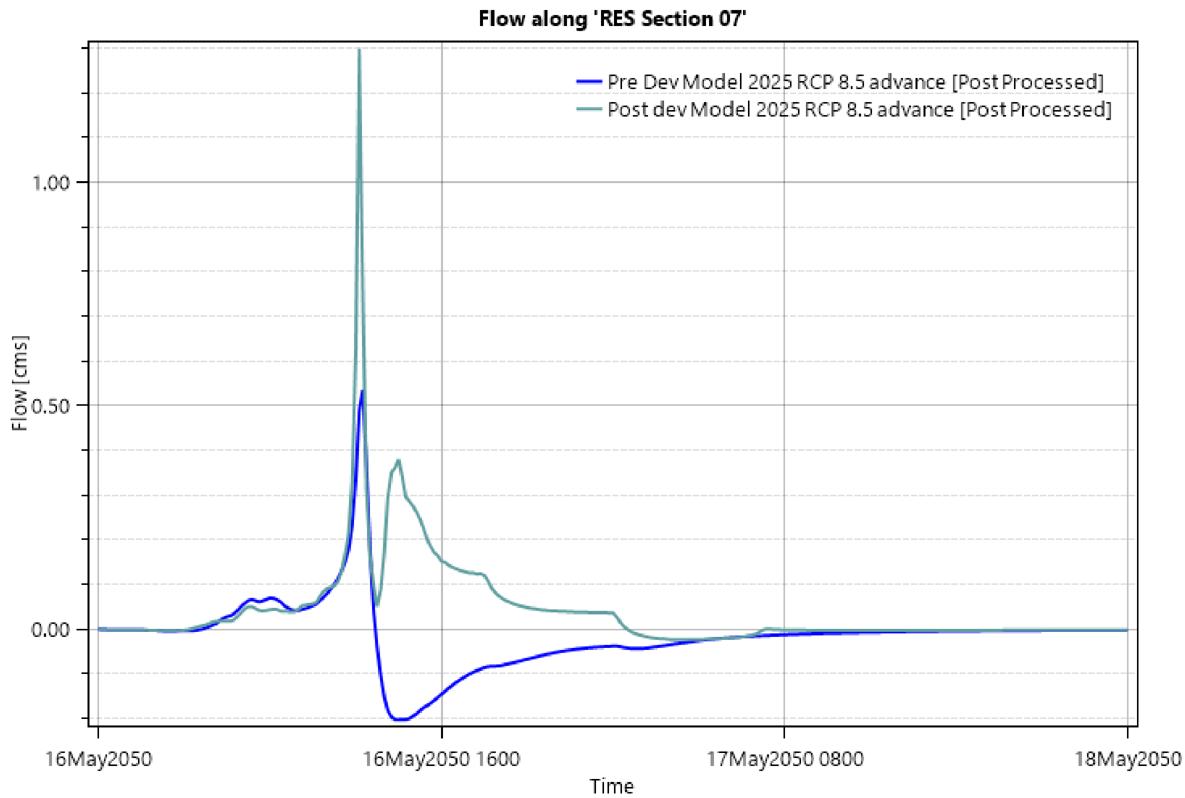

Flow along 'RES Section 04'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
Time
2.50
2.00
1.50
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050

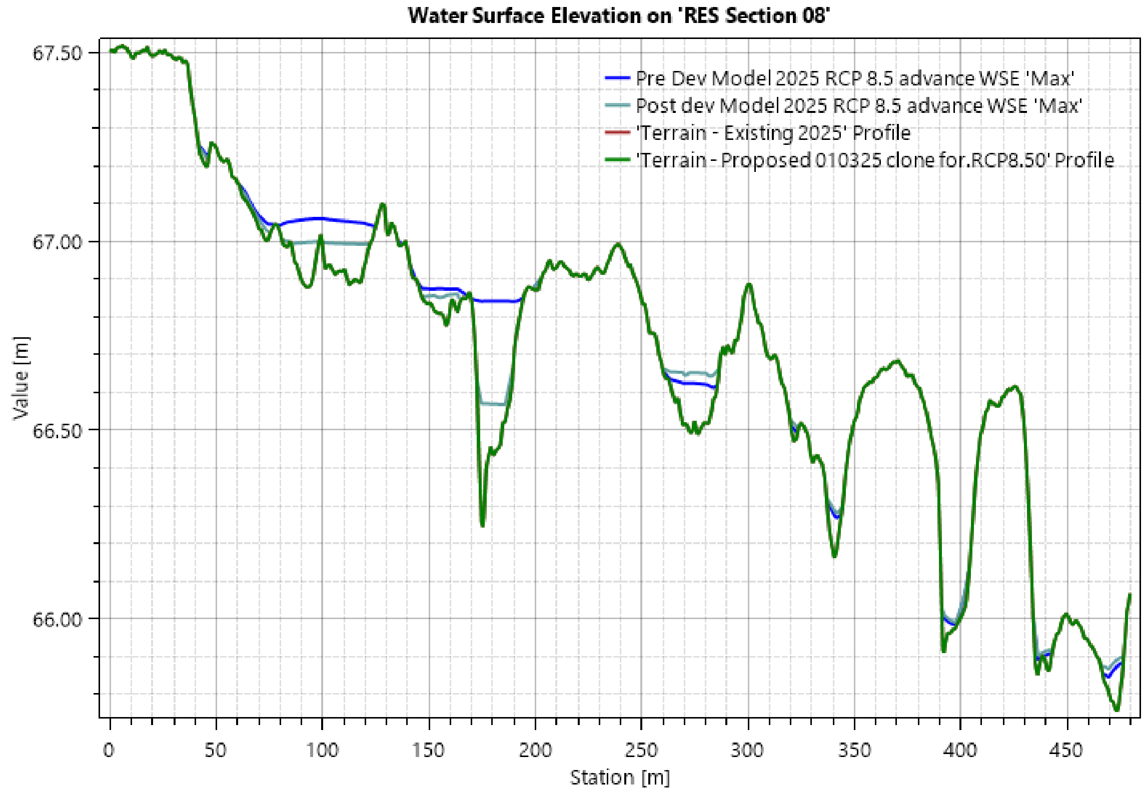

Water Surface Elevation on 'RES Section 05'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
Station [m]
67.00
66.50
66.00
65.50
0
50
100
150
200

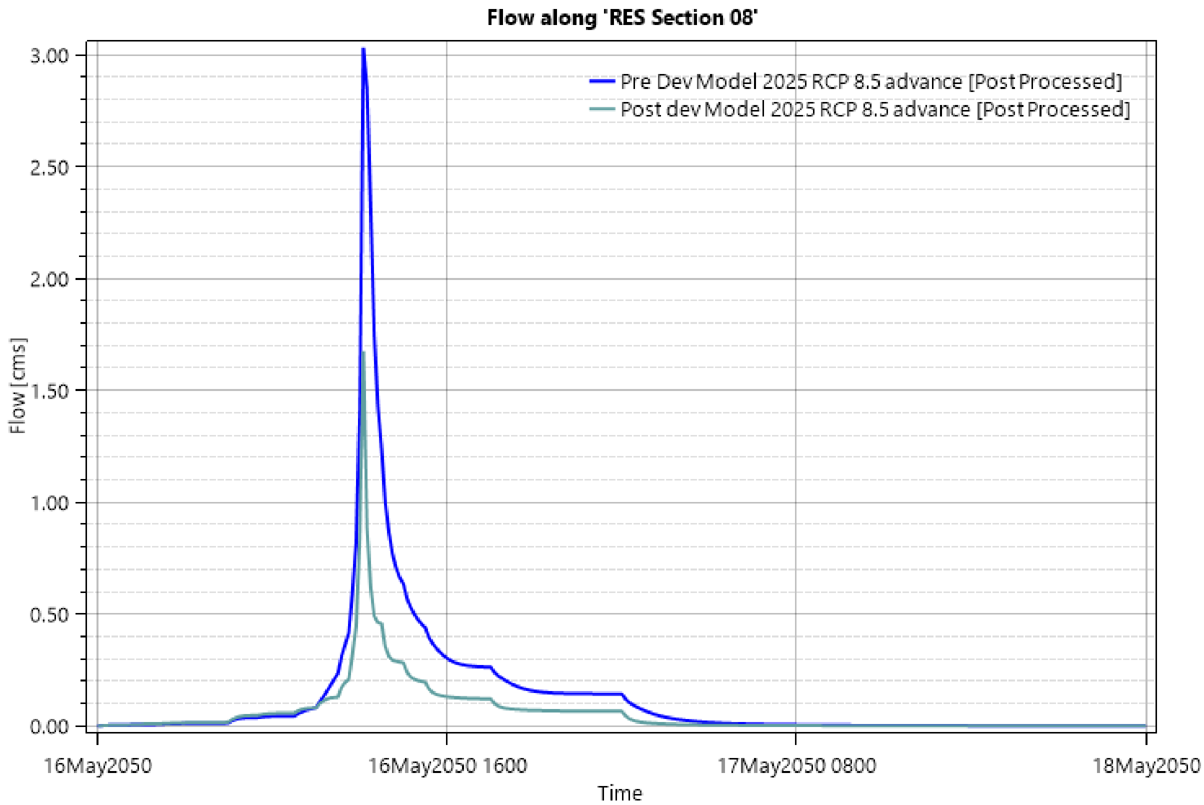

Water Surface Elevation on 'RES Section 06'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
Station [m]
65.90
65.80
65.70
65.60
65.50
65.40
0
5
10
15
20
25
30
35
40

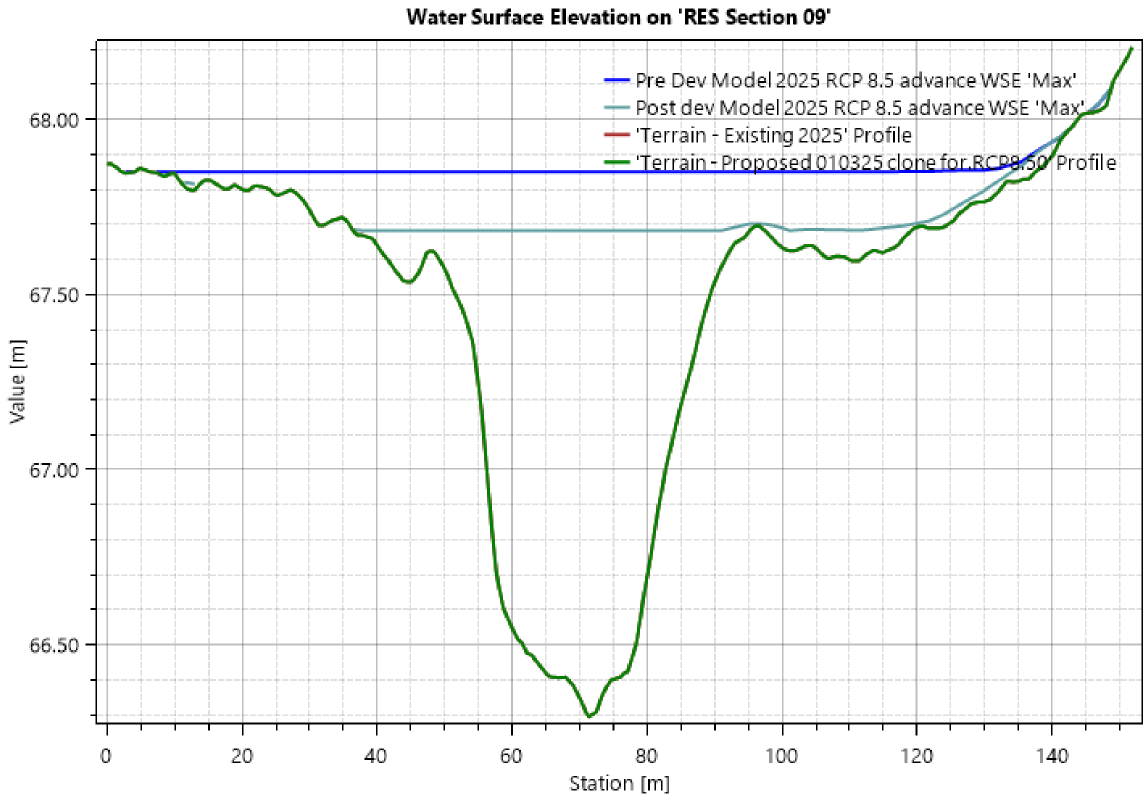

Flow along 'RES Section 06'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
2.00
1.50
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

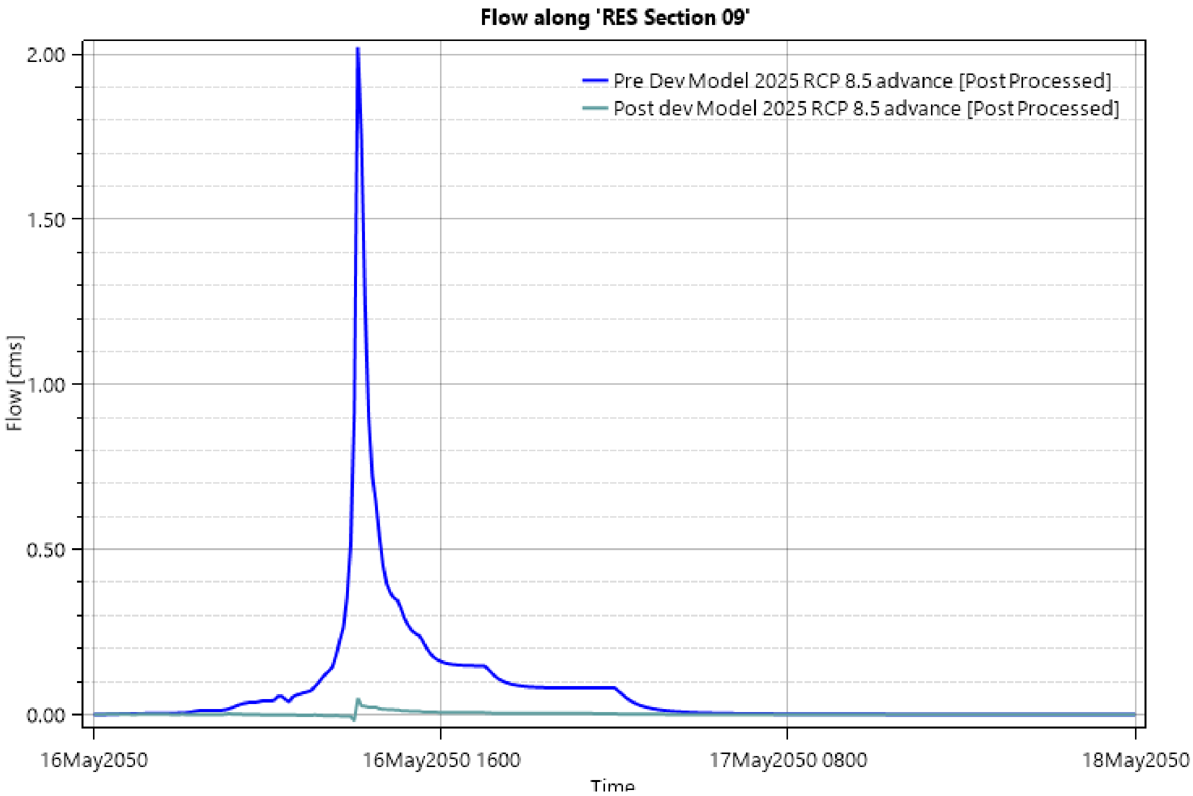

Water Surface Elevation on 'RES Section 07'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
66.00
65.80
65.60
65.40
65.20
65.00
0
50
100
150
200
250
Station [m]

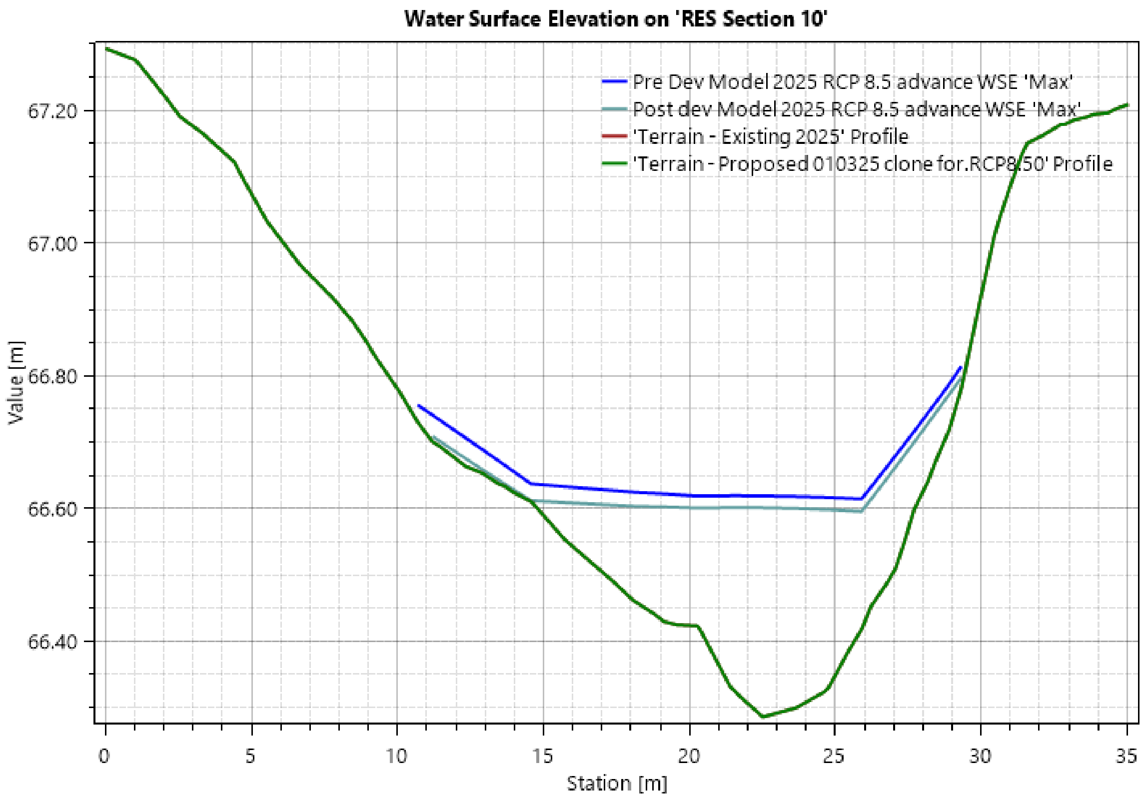

Flow along 'RES Section 07'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

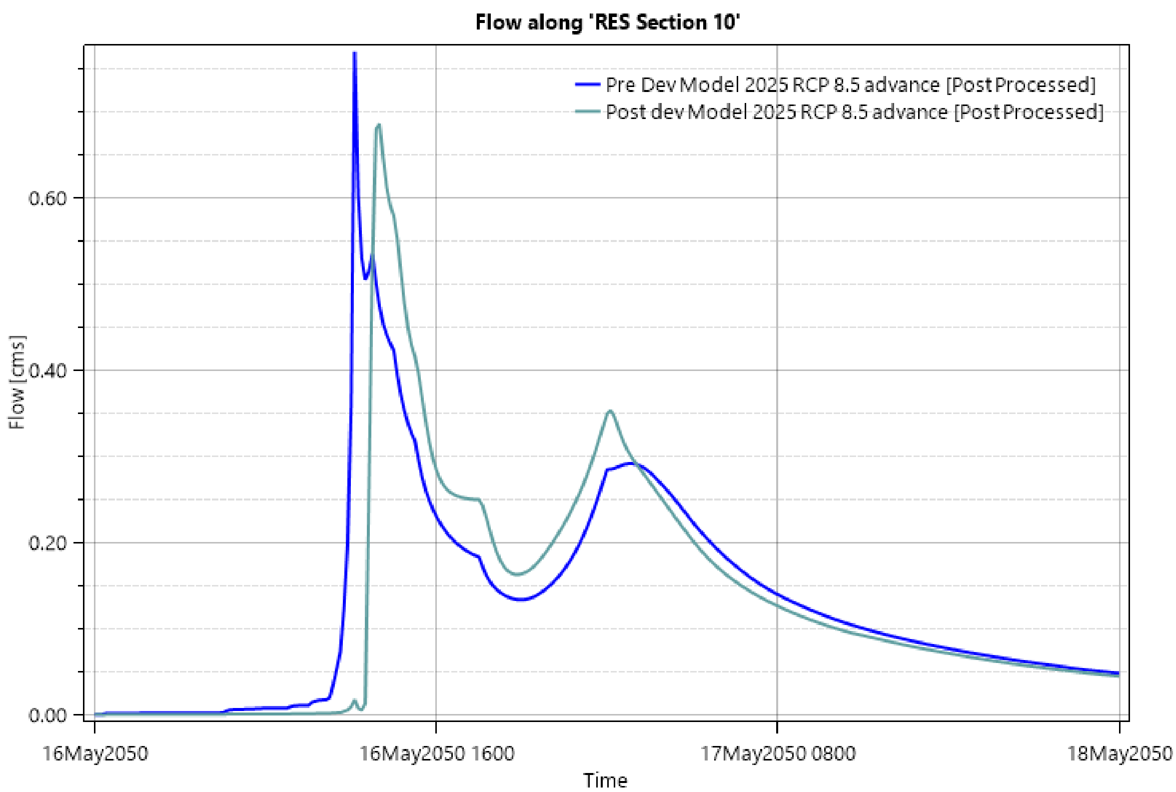

Water Surface Elevation on 'RES Section 08'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
67.50
67.00
66.50
66.00
Station [m]
0
50
100
150
200
250
300
350
400
450

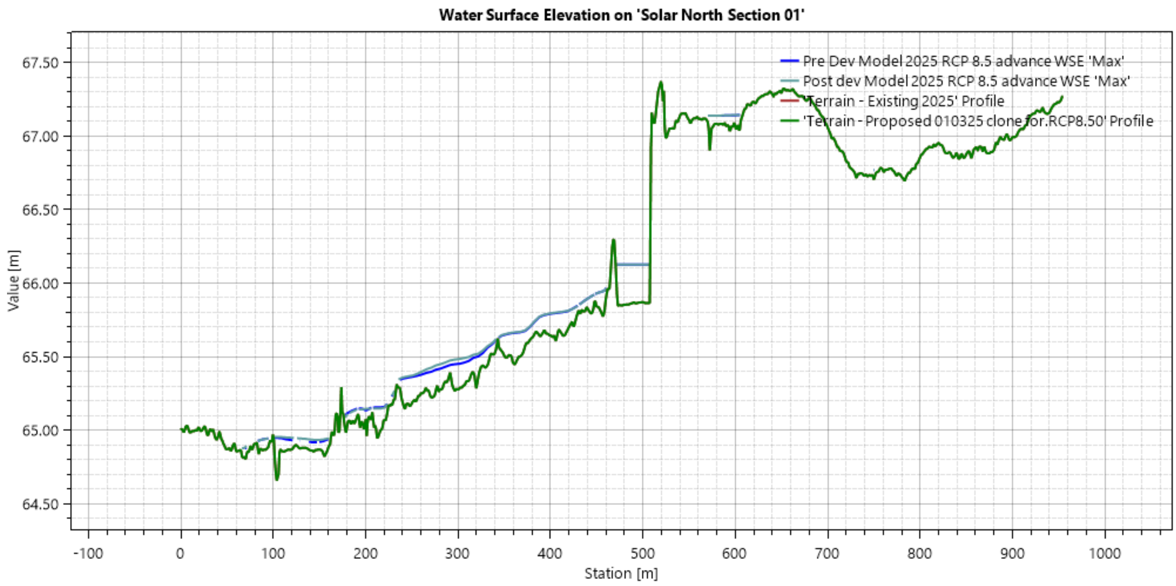

Flow along 'RES Section 08'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
3.00
2.50
2.00
1.50
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

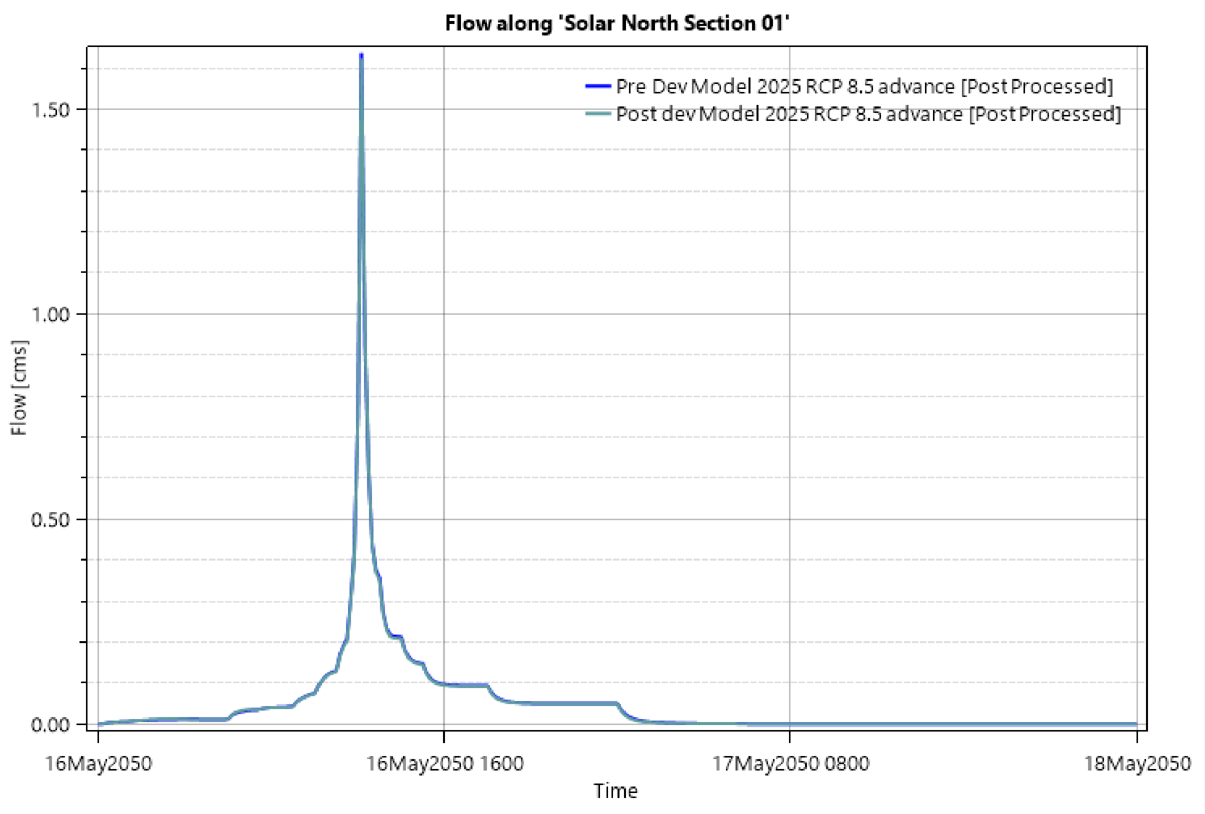

Water Surface Elevation on 'RES Section 09'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for RCP8.50' Profile
Value [m]
68.00
67.50
67.00
66.50
0
20
40
60
80
100
120
140
Station [m]

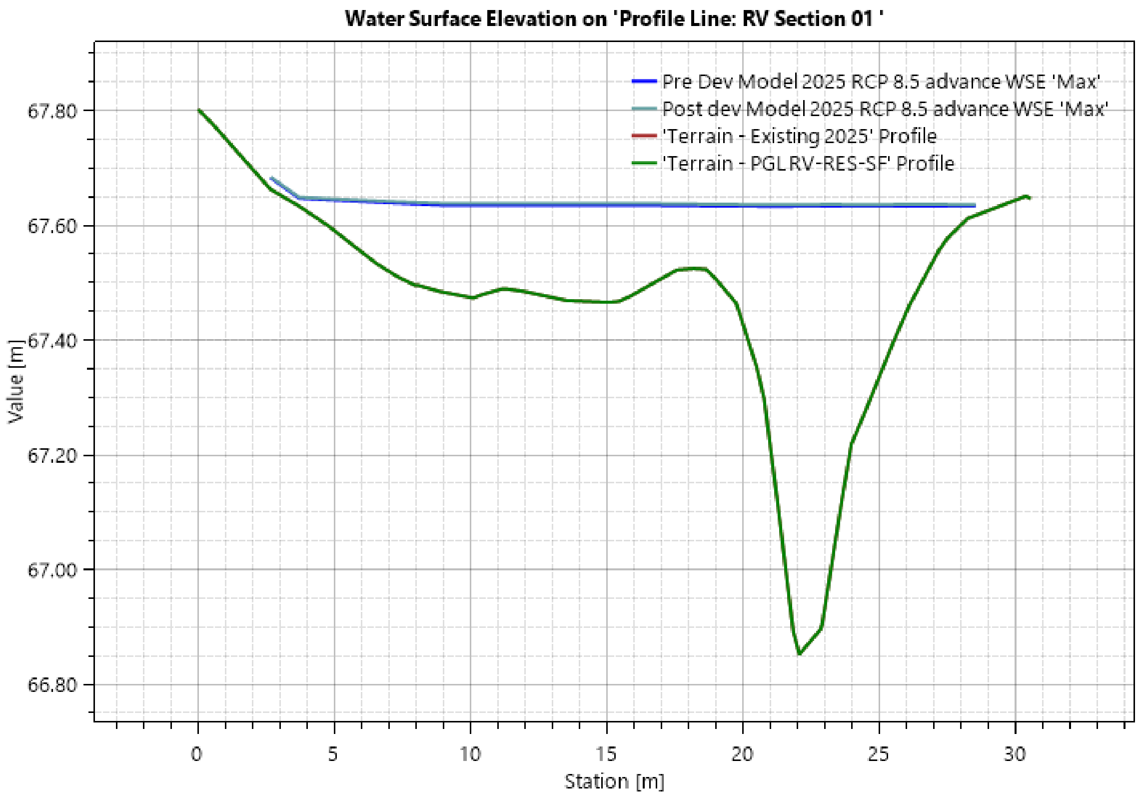

Flow along 'RES Section 09'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
2.00
1.50
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

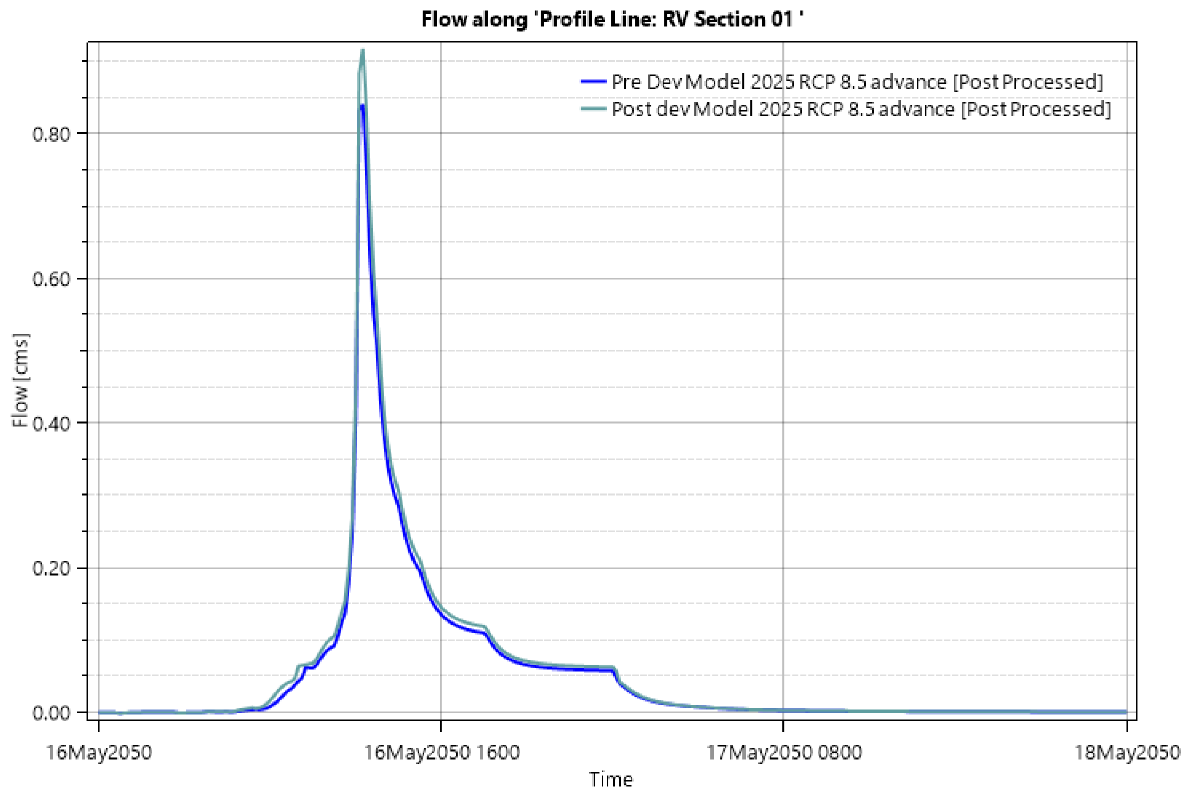

Water Surface Elevation on 'RES Section 10'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
67.20
67.00
66.80
66.60
66.40
0
5
10
15
20
25
30
35
Station [m]

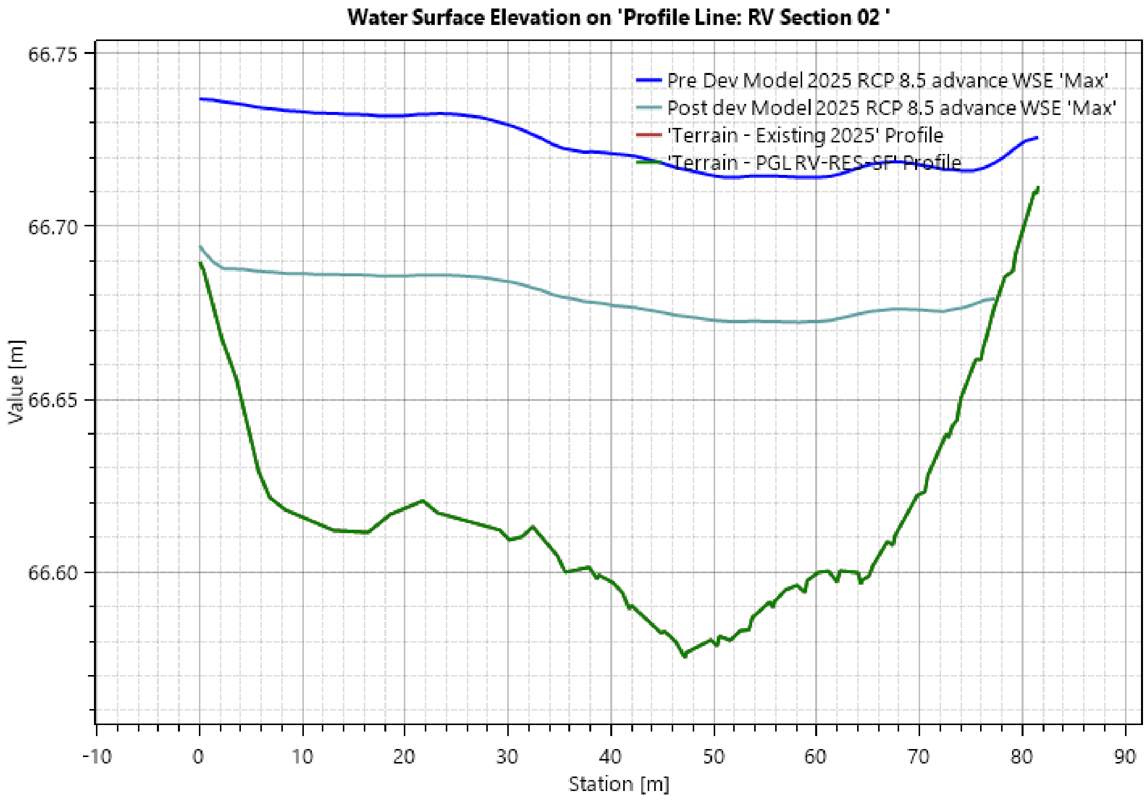

Flow along 'RES Section 10'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
0.60
0.40
0.20
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

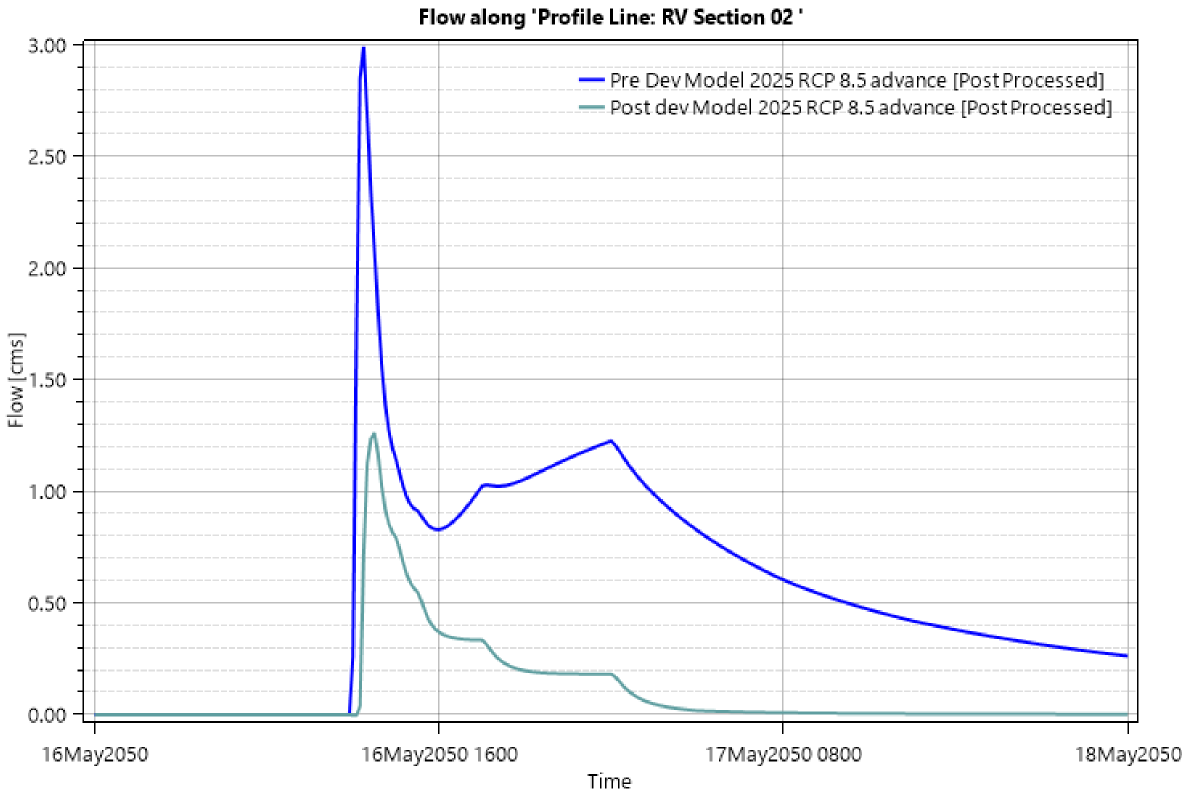

Water Surface Elevation on 'Solar North Section 01'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
Station [m]
67.50
67.00
66.50
66.00
65.50
65.00
64.50
-100
0
100
200
300
400
500
600
700
800
900
1000

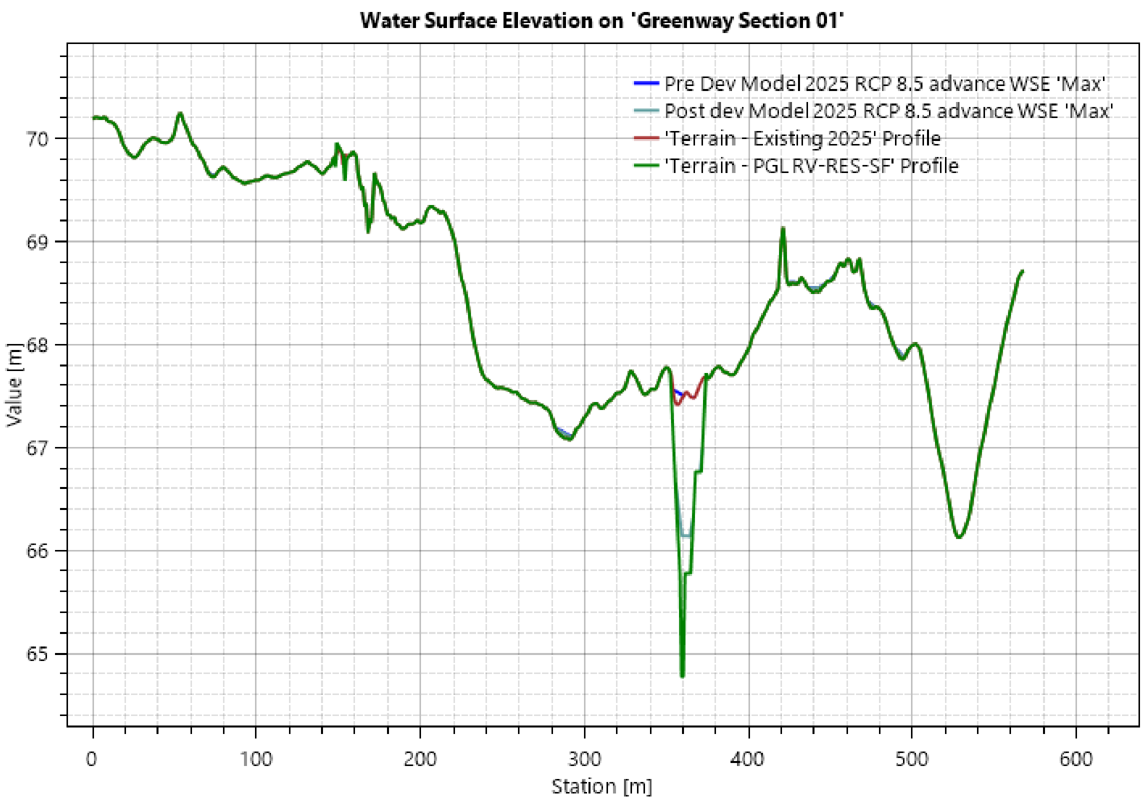

Flow along 'Solar North Section 01'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
Time
1.50
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050


Water Surface Elevation on 'Profile Line: RV Section 01 '
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - PGL RV-RES-SF' Profile
Value [m]
67.80
67.60
67.40
67.20
67.00
66.80
0
5
10
15
20
25
30
Station [m]

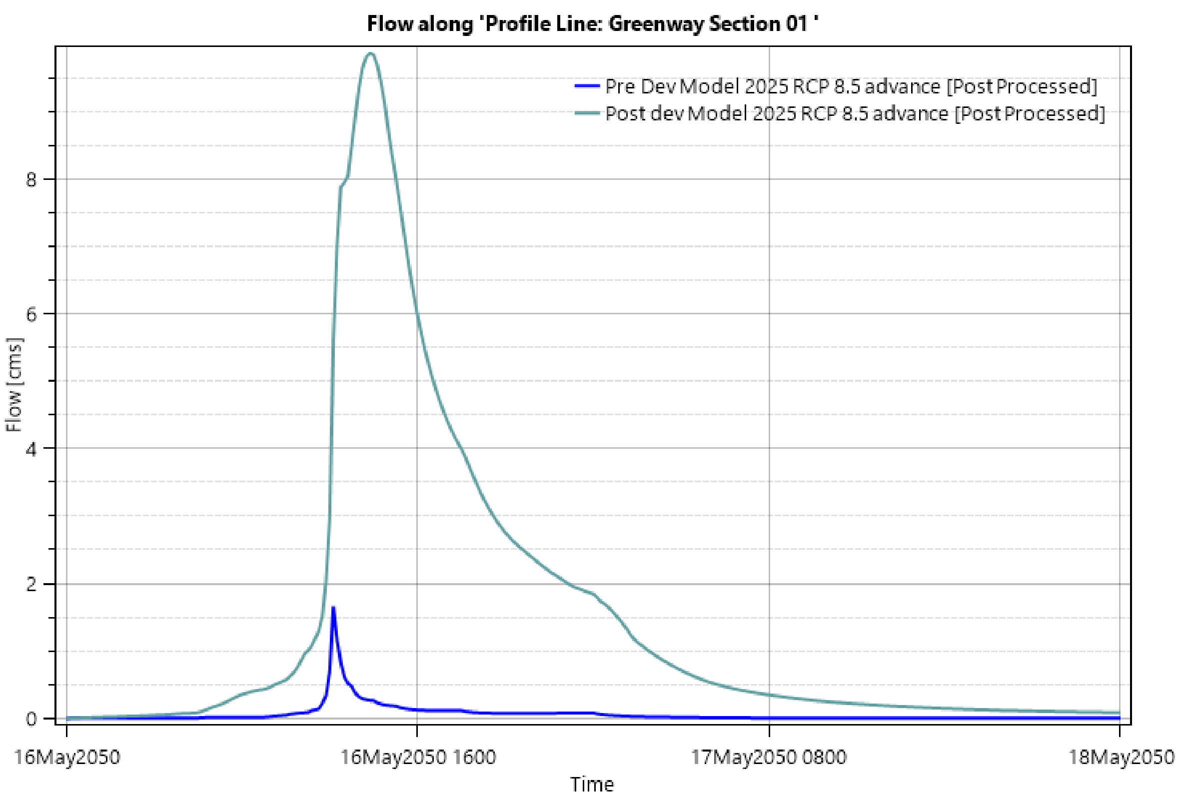

Flow along 'Profile Line: RV Section 01 '
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
0.80
0.60
0.40
0.20
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time


Water Surface Elevation on 'Profile Line: RV Section 02 '
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - PGL RV-RES-SF' Profile
66.75
66.70
66.65
66.60
Value [m]
-10
0
10
20
30
40
50
60
70
80
90
Station [m]

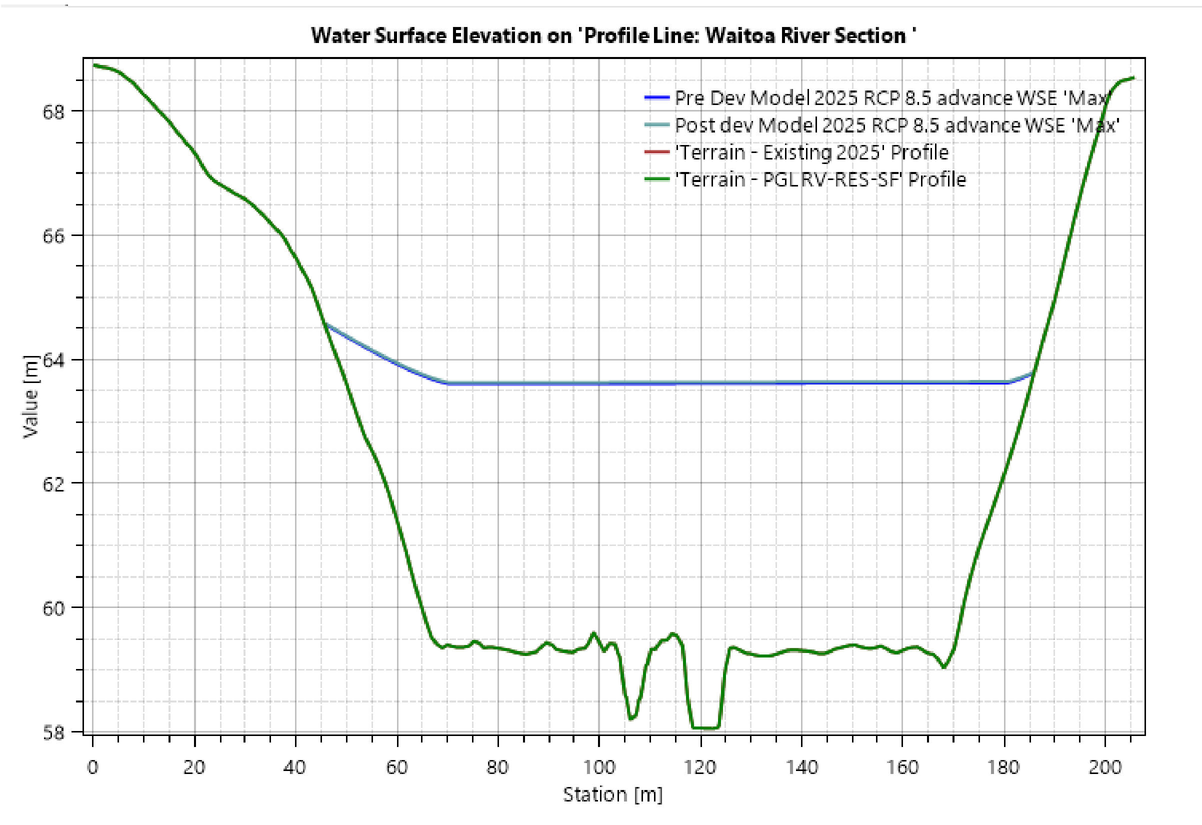

Flow along 'Profile Line: RV Section 02 '
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Flow [cms]
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

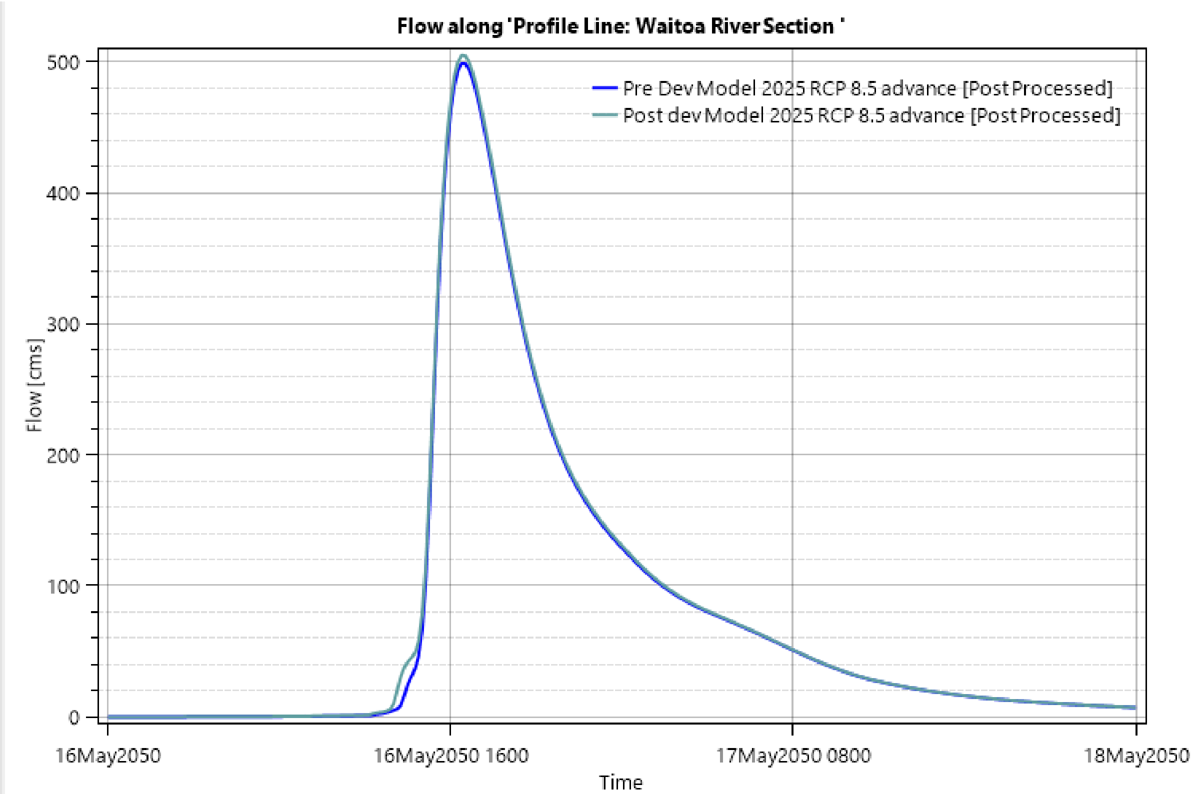


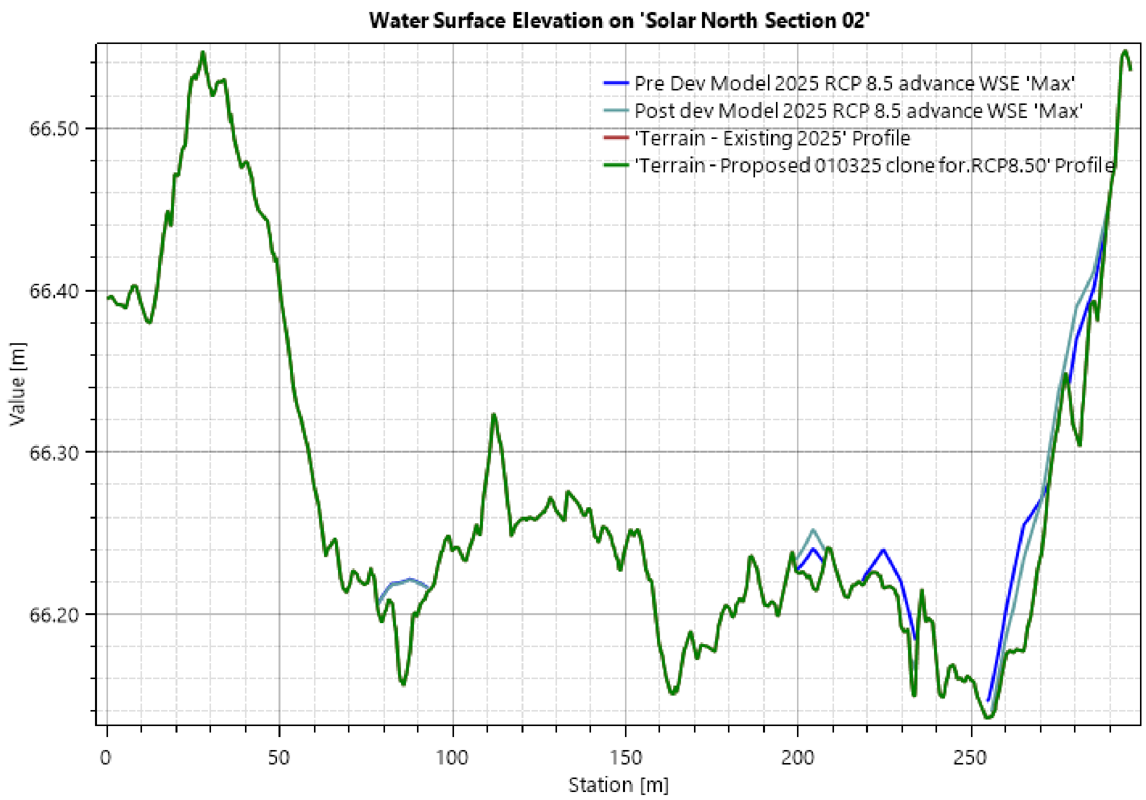
Water Surface Elevation on 'Greenway Section 01'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - PGL RV-RES-SF' Profile
Value [m]
70
69
68
67
66
65
0
100
200
300
400
500
600
Station [m]

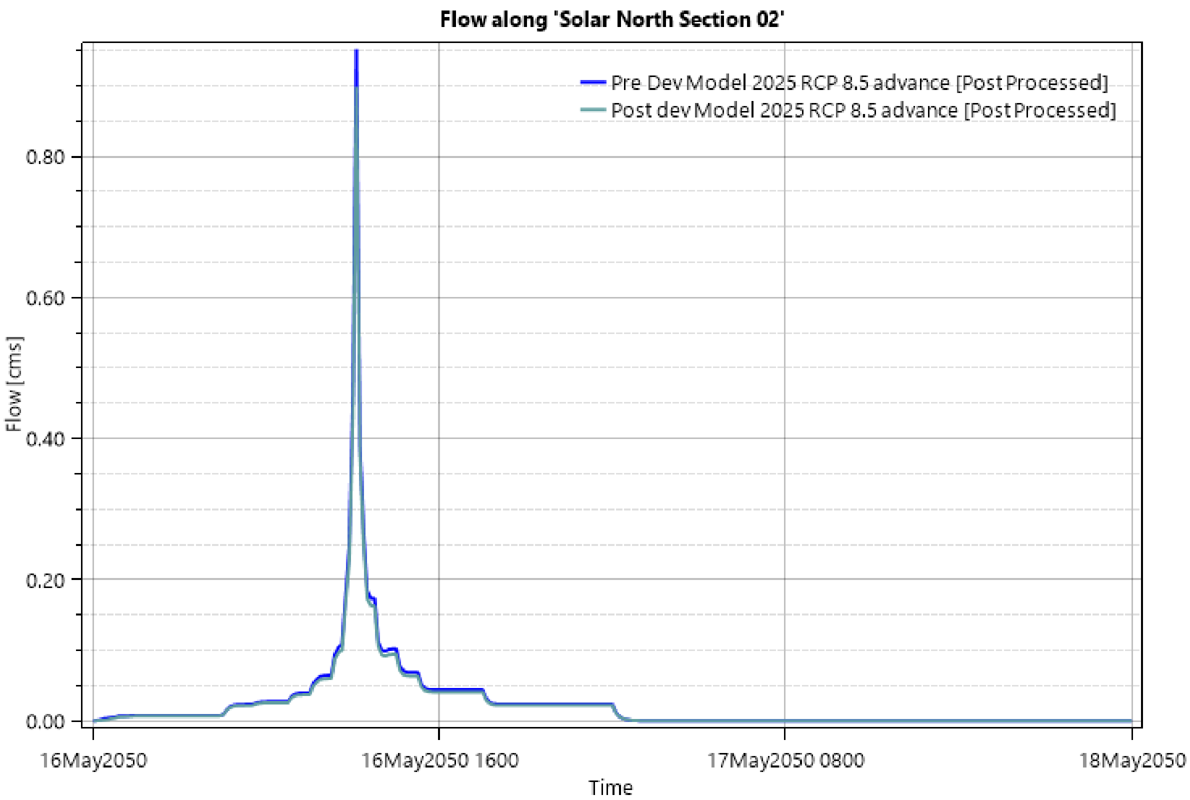


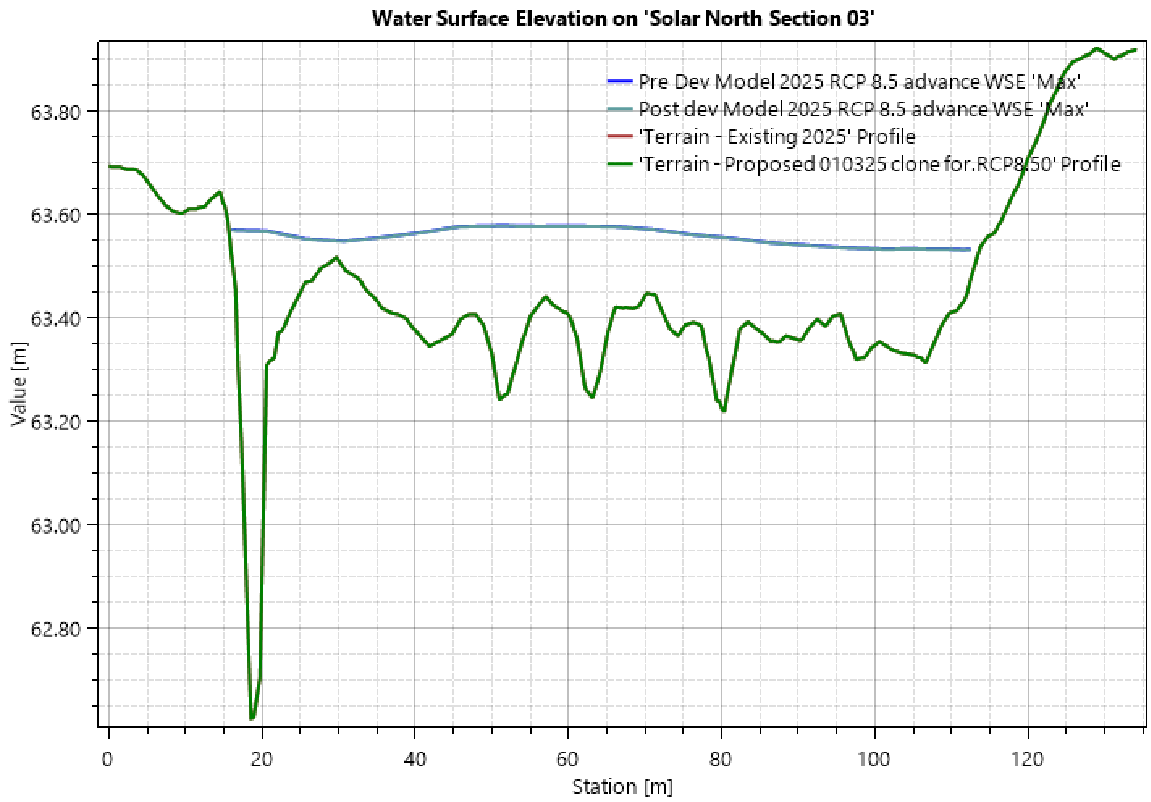
Flow along 'Profile Line: Greenway Section 01 '
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
8
6
4
2
0
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

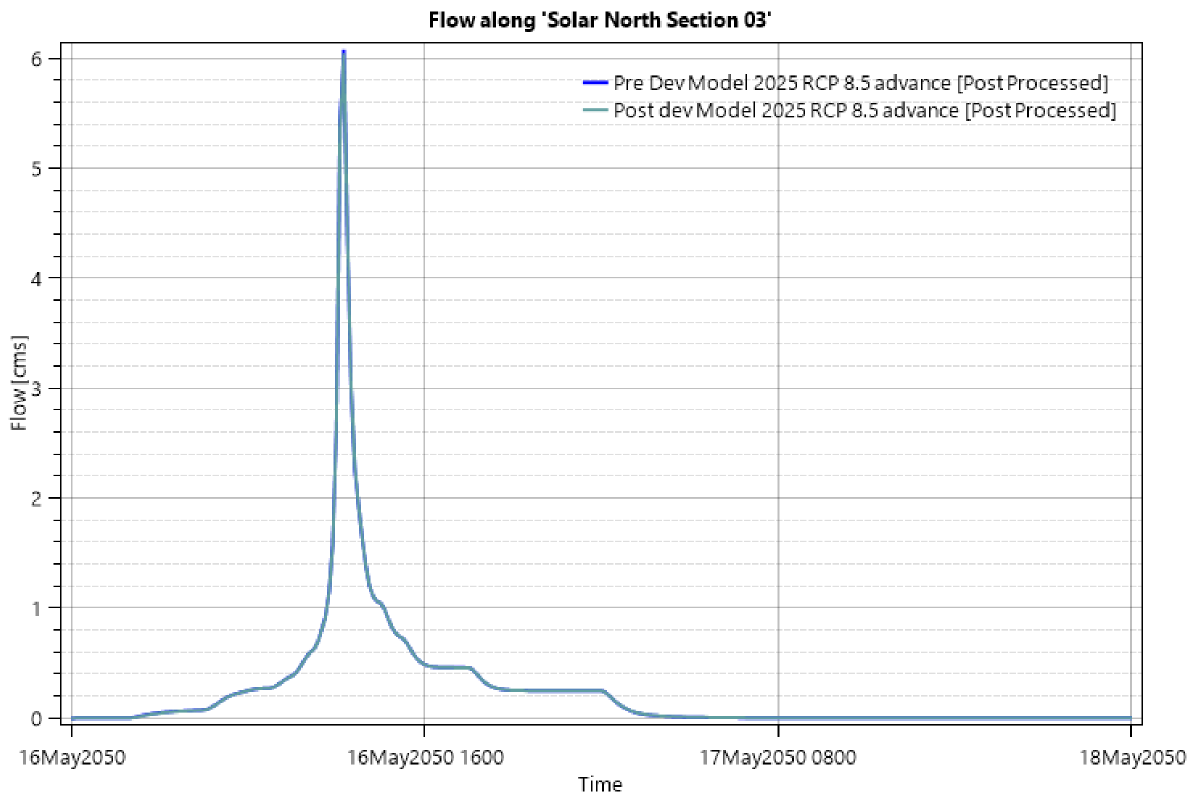

Water Surface Elevation on 'Profile Line: Waitoa River Section '
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - PGLRV-RES-SF' Profile
Value [m]
58
60
62
64
66
68
0
20
40
60
80
100
120
140
160
180
200
Station [m]


Flow along 'Profile Line: Waitoa River Section '
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
0
100
200
300
400
500
16May2050
16May2050 1600
17May2050 0800
18May2050
Time


Water Surface Elevation on 'Solar North Section 02'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
66.50
66.40
66.30
66.20
0
50
100
150
200
250
Station [m]


Flow along 'Solar North Section 02'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
0.80
0.60
0.40
0.20
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time


Water Surface Elevation on 'Solar North Section 03'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
63.80
63.60
63.40
63.20
63.00
62.80
0
20
40
60
80
100
120
Station [m]


Flow along 'Solar North Section 03'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
6
5
4
3
2
1
0
16May2050
16May2050 1600
17May2050 0800
18May2050
Time