THREE WATERS STRATEGY REPORT

Sunfield – Fast-Track Approvals Application

Ardmore, Auckland

PROJECT INFORMATION

CLIENT	Winton Land Limited
PROJECT	215010

DOCUMENT CONTROL

DATE OF ISSUE	07/02/2025
REVISION	E
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1.0 INTRODUCTION

1.1 EXECUTIVE SUMMARY OVERVIEW

Sunfield Developments Limited (**SDL**) is proposing to consent a contiguous 244.5-hectare (ha) site to allow the development of a master planned community to be known as "Sunfield", (the **Site**).

This report outlines the strategy for the provision of stormwater, wastewater and water supply (the **Three Waters Strategy**) for the Site and has been prepared to support the Fast-track Approvals Act Application (**FTA**) and subsequent development of the Site. This report is to be read in conjunction with the Stormwater Management Plan and Infrastructure report prepared by Maven.

The scope of this report includes the identification of key design strategies, developing design solutions for stormwater and wastewater disposal and water supply, and articulating the designs in accordance with the Three Waters Strategy.

The assessments included in this report are formed from a desktop analysis based on information available at time of issue. Acceptable engineering solutions have been determined and details of which are contained within the main body of the report. The proposed consent conditions for this consent application require the detailed design of these solutions to be subject to review through an Engineering Plan Approval (EPA) process by the local authority.

The engineering solutions for the Site outlined within this report have been prepared based on the standards and requirement of each of Auckland Council, Watercare and Healthy Waters, and are in line with best practice options.

1.2 SITE DESCRIPTION

The Site is located over several land tiles and is indicatively shown on the aerial photo below. The Site is bounded by Old Wairoa Road to the south, Cosgrave Road to the west and Airfield Road to the north.

Figure 1: Aerial Photo (indicative extent of Sunfield Master plan shown in red outline)

The current land zoning for the Site comprises approximately 57ha of land identified as Future Urban Zone (**FUZ**) and 187ha as Mixed Rural Zone (**MRZ**) under the Auckland Unitary Plan (**AUP(OP)**).

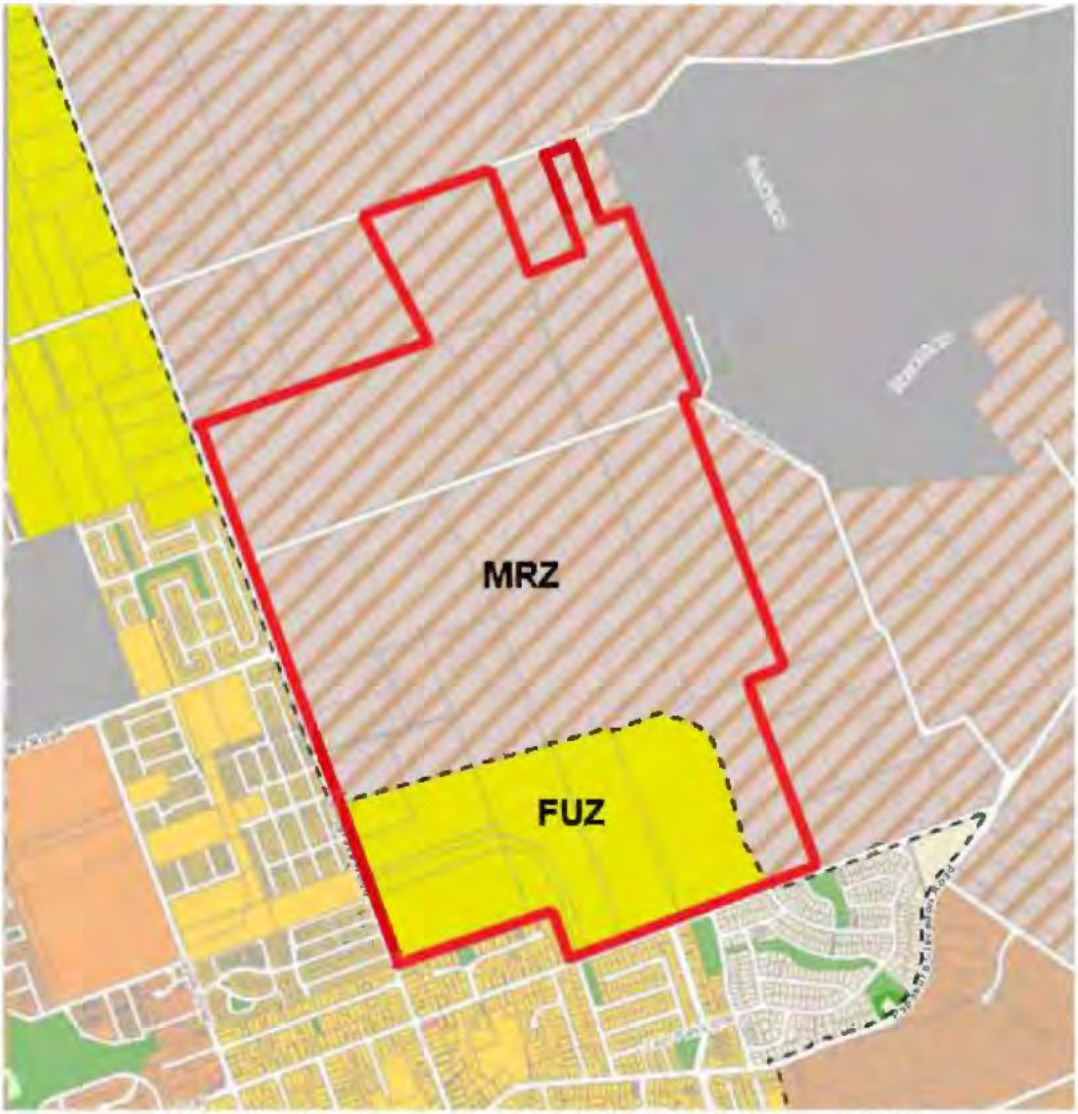

Figure 2: Current Zoning Plan (extent of FUZ land shown in yellow)

Auckland Council's Framework Plan for the adjacent Awakeri Wetland Development provides a possible density for the existing FUZ land, and when a similar density is interpolated through the full FUZ land, approximately 1,550 lots can be realised. The interpolated density is the maximum probable development (**MPD**) of the land.

The MPD of 1,550 lots sets the development baseline for the FUZ land, from which stormwater and wastewater discharges are calculated and analysed within this report.

Further interpolating the density guidance for the adjacent MRZ land results in a total MPD for the Site of 4,000 residential lots.

1.3 LEGAL DESCRIPTION

The legal description and underlying zoning of the existing land parcels within the Site are shown below.

Address	Legal Description	Record of Title	Area (ha)	Underlying Zoning
55 Cosgrave Road, Papakura	Section 3-4 Survey Office Plan 495342	828127	9.2433	Future Urban
Old Wairoa Road, Papakura	Section 5-6 Survey Office Plan 495342	828128	11.8128	Future Urban
Old Wairoa Road, Papakura	Lot 1 Deposited Plan 55480	NA6C/1128	5.8014	Future Urban
Old Wairoa Road, Papakura	Lot 4 Deposited Plan 55480	NA6C/1131	10.3587	Future Urban
508 Old Wairoa Road, Ardmore	Deposited Plan 10383	NA258/245	23.6336	Future Urban & Rural
85 Hamlin Road, Ardmore	Lot 8 Deeds Plan Whau 38	NA778/296	22.5233	Rural
80 Hamlin Road, Ardmore	Part Lot 2 Deposited Plan 22141	NA1B/856	18.9937	Rural
80 Hamlin Road, Ardmore	Lot 2 Deposited Plan 21397	NA477/291	10.1171	Rural
80 Hamlin Road, Ardmore	Lot 1 Deposited Plan 21397	NA477/75	30.7192	Rural
80 Hamlin Road, Ardmore	Lot 5 Deposited Plan 12961	NA631/77	35.9057	Rural
80 Hamlin Road, Ardmore	Part Lot 4 Deposited Plan 12961	NA636/171	21.8505	Rural
279 Airfields Road, Ardmore	Lot 2 Deposited Plan 199521	NA128A/553	14.4224	Rural
92 Hamlin Road, Ardmore	Lot 1 Deposited Plan 46615	NA1666/17	0.0911	Rural

143 Cosgrave Road, Papakura	Lot 1 Deposited Plan 103787	NA57A/1149	3.0400	Rural
131 Cosgrave Road, Papakura	Lot 2 Deposited Plan 103787	NA57A/1150	3.0370	Rural
121A Cosgrave Road, Papakura	Lot 3 Deposited Plan 103787 and 1/3 Share in Lot 7 Deposited Plan 103787	NA57A/1151	3.0400	Rural
123 Cosgrave Road, Papakura	Lot 4 Deposited Plan 103787 and 1/3 Share in Lot 7 Deposited Plan 103787	NA57A/1152	8.6325	Rural
119A Cosgrave Road, Papakura	Lot 5 Deposited Plan 103787 and 1/3 Share in Lot 7 Deposited Plan 103787	NA61A/530	3.0370	Rural
119A, 121A and 123 Cosgrave Road, Papakura	Lot 7 Deposited Plan 103787		0.2417	Rural
119 Cosgrave Road, Papakura	Lot 6 Deposited Plan 103787	NA57A/1154	3.0360	Rural
101 Cosgrave Road, Papakura	Part Lot 1 Deposited Plan 45156	NA24C/216	1.9425	Future Urban
103 Cosgrave Road, Papakura	Lot 1 Deposited Plan 62629	NA18B/646	0.0809	Future Urban
55A Cosgrave Road, Papakura	Section 1-2 Survey Office Plan 495342	828126	2.9343	Future Urban
Total			244.4947	

Table 1- Legal Description & Existing Zoning Summary

1.4 PROPOSAL

The proposed development of Sunfield is a large-scale master-planned community, consisting of approximately 4,000 residential lots, and approximately 56.5ha of industrial/employment land. In addition to residential and industrial use, other uses to support a new community of this size are proposed, such as, a town centre, health care, aged care, local hub, a school, parks/open space, stormwater reserves and green connections/shared pathways. The Sunfield development concept plan is shown in Figure 3 below.

Figure 3: Sunfield Concept Masterplan

2.0 THREE WATERS STRATEGY

2.1 DESIGN STRATEGY

A key issue in developing and consenting a master-planned community is ensuring the Site can be serviced by necessary infrastructure. In particular, appropriate infrastructure for stormwater, wastewater and water supply is critical.

This report helps to shape the masterplan for the Site by incorporating the infrastructure solutions, while also providing a basis for consultation with third party stakeholders. These stakeholders include Veolia, Watercare, Auckland Council, Auckland Transport, Healthy Waters, and Mana Whenua.

The overarching design principle driving the Three Waters Strategy for the Sunfield development is the incorporation of Water Sensitive Urban Design (**WSUD**).

WSUD is a land planning and engineering design approach that integrates the urban water cycle, including stormwater, groundwater and wastewater management and water supply, to minimise environmental degradation and improve aesthetic and recreational outcomes.

This Three Waters Strategy incorporates WSUD engineering design principles to create a low impact, sustainable development which appropriately manages stormwater and wastewater discharge from the Site.

The proposed Three Waters infrastructure will be subject to conditions of consent whereby the implementation and ongoing management of the infrastructure will be regulated by consent conditions.

Further detail on each of the pillars of the Three Waters Strategy are set out in Sections 2.2 to 2.4 below, with the key outcomes sought by the strategy being as follows:

Stormwater

- Enable development by delivering stormwater servicing to a catchment where stormwater servicing is not currently available.
- A full Stormwater Management Plan (**SMP**).
- Emphasise a water-sensitive design approach that:
  - minimises or mitigates the adverse effects on water quality, freshwater systems, stream health, and ecological values of the receiving environment through the implementation of stormwater management devices; and
  - protects and enhances stream systems and natural hydrology while mitigating hydrological changes and managing flooding effects.
- Minimise the generation and discharge of contaminants/sediments into the sensitive receiving environment of the Manukau Harbour.
- Manage the 100-year Average Recurrence Interval (**ARI**) floodplain to ensure there are no adverse effects on proposed development.
- Ensure that the flood risk upstream or downstream for events up to the 100-year ARI is not increased.
- Allow for the effects of climate change by including a climate change factor of 3.8°C degrees in accordance with Auckland Council's latest Stormwater Code of Practice (**SWCoP**) Version 4.

Wastewater & Water supply

- Wastewater networks, including new and existing private connections to the networks, allow the minimum practicable seepage into and out of the networks.
- Sewage entering the networks is controlled to avoid or minimise adverse effects on physical assets, wastewater treatment processes and the environment.
- Overflows from the networks during both dry and wet weather are minimised as far as practicable.
- Infrastructure that is created is of good quality, meets health requirements and minimises ongoing maintenance costs.
- Future demands on maintainability and access are met, as infrastructure ages and the natural environment changes.
- Water is used efficiently, and wastage is minimised as best practicable.

2.2 STORMWATER STRATEGY (SUMMARY)

2.2.1 APPROACH

The stormwater approach adopted as part of this Three Waters Strategy has been developed in accordance with the following:

1. Auckland Council's policies and plans.,
2. Best practice stormwater management techniques to meet AUP(OP) regulatory policies and provisions.
3. Auckland Council's stormwater-specific guidelines and Network Discharge Consent (**NDC**) requirements.
4. National Policy Statement on Urban Development (**NPS-UD**).
5. Consultation with Mana Whenua.

Due to the presence of peat soils onsite and their recharge requirements best practice alternatives are recommended.

Details of these requirements and guidelines have been included Section 3.2 of this report.

An Integrated Stormwater Management Approach has been adopted in the design and in accordance with the policies in the AUP(OP)- Sections E1.3, B7 and B8.

2.2.2 CATCHMENT AREAS – BOTH CURRENT AND POST DEVELOPMENT

Section 3 below sets out the details of the current (existing) catchment areas of the Site, comprising the Papakura Stream (known as the Eastern Catchment) and the Pahurehure Inlet (known as the Western Catchment). For analysis purposes, where this report refers to the post development proposal, the Site and upstream stormwater catchments have been divided into four main catchments, being Catchments A, B, C and D. Further detail of these areas is included in Section 3.1 of this report and the areas are summarised in Table 2 of Section 3.2. The catchments have different strategies tailored to their specific stormwater management requirements.

To achieve the stormwater outcomes sought by this report (refer Section 2.1 above), the following stormwater management principles are proposed for the Site:

- Flood Management:
  - Using flood management devices, manage the 100-year ARI floodplain to ensure there are no negative effects on proposed development.
    - For Catchment A:
      - Enable stormwater servicing and reduce the catchment draining to the Papakura Stream catchment by utilising existing flood management infrastructure; and
      - Attenuate peak flows for the 10 and 100-year ARI storms to achieve peak flow rate and peak water level design criteria as provided by Healthy Waters for Stages 2 and 3 of the Awakeri Wetland (based on the MPD of the FUZ land).
    - For Catchments B and D:
      - Mitigate downstream flooding by attenuating peak flows up to the 100-year ARI storms to match pre-development peaks flows. Attenuation will be provided via stormwater management devices within the Site.
    - For Catchment C:
      - Direct peak flows from the undeveloped upstream catchment around the eastern perimeter of the proposed development site via a swale to allow for passing flows forward (no attenuation) for flows up to the 100-year ARI storm.
- Conveyance:
  - Primary flows up to 10-year ARI event will be conveyed by swales and piped network.
  - Storm events from 10 to 100-year ARI events will be conveyed along public roads and swales within drainage reserves and green spaces as overland flow paths.
  - Extension of the Awakeri Wetland from the already completed and commissioned Stage 1 will provide conveyance of storm events up to 100-year ARI from Catchment A of the Site.
- Hydrological Mitigation:
  - Hydrological mitigation to minimum of AUP SMAF 1 hydrological mitigation standard will be provided for the Site to minimise the change in hydrology (maintain pre-development).
  - SMAF 1 requirements are as follows:
    - Retention (volume reduction) of at least 5 mm of runoff depth from impervious surfaces; and
    - Detention (temporary storage) and a drain down period of 24 hours for the difference between the pre-development and post-development runoff volumes from a 95th percentile, 24-hour rainfall event minus the achieved retention volume, over the impervious area for which hydrological mitigation is required.
- Water Quality:
  - Provide stormwater quality treatment through stormwater treatment devices such as a stormwater conveyance channels and wetlands.

- Ground Water Recharge:
  - Ground water recharge of the peat soil via soakage pit/recharge pit to ensure the retention of existing groundwater levels. Recharge pits will be designed to retain the stormwater runoff from all impervious areas from the first 15mm of any rainfall event. The retention provided by the recharge pits will also provide hydrological mitigation.

2.3 WASTEWATER STRATEGY (SUMMARY)

The wastewater strategy for the Site is to restrict wastewater discharge to an acceptable level to avoid any capacity issues with downstream wastewater infrastructure.

Wastewater discharge from the developed Site will be limited to the discharge anticipated from the MPD of the FUZ land.

The proposed wastewater strategy outlined in this report entails the utilisation of a Low-Pressure Sewer (**LPS**) wastewater system. LPS systems eliminate peak wet weather flows by utilising a sealed network which eliminates inflow and infiltration.

Due to the flat topography, poor ground conditions and high-water table of the Site, a LPS system is considered an acceptable alternative solution to a standard gravity option for wastewater servicing.

2.4 WATER SUPPLY STRATEGY (SUMMARY)

The proposed water supply approach outlined in this Three Waters Strategy report is to reticulate the development with a new water supply network for potable water and firefighting services.

Preliminary investigations with Veolia have indicated that a connection to the nearest Bulk Pressure Supply Point (**BSP**) will be necessary to provide the minimum firefighting water supply classification for the development of the Site. A new public water supply reticulation will need to be extended from the BSP to service the development.

There are two existing BSP points on the existing 450mmØ transmission line located in the near vicinity of the Site. The closest BSP point is in the front berm of 393 Porchester Road, with another at the intersection of Porchester Road and Airfield Road.

The closest existing BSP may need to be upgraded as part of provisioning for the Site. If that BSP point does not have sufficient capacity, a new BSP point may need to be constructed on the transmission line. Consultation with Veolia and Watercare will be required to confirm the preferred connection point and capacity.

3.0 STORMWATER

3.1 EXISTING STORMWATER CATCHMENTS

As part of the Integrated Stormwater Catchment Management Plan (**ICMP**) study undertaken by Papakura District Council (**PDC**) in 2007, and later adopted and edited by Auckland Council (2016), the Papakura District has been divided into five major stormwater management areas, mainly based on their respective receiving environments. The five major catchment areas are:

- Papakura Stream;
- Pahurehure Inlet;
- Slippery Creek;
- Drury Creek; and
- Upper Taitaia.

The Site is located within both the Pahurehure Inlet and Papakura Stream catchments. As demonstrated in Figure 4 below, the Site is located in the upper half of the Pahurehure inlet catchment and midway of the Papakura stream catchment.

Most of the Site is identified as draining to the Papakura Stream catchment to the north. However, it must be noted that Auckland Council's stormwater catchment boundaries may not accurately define the exact borders between the two catchments due to the difficulties in delineating those boundaries from the very flat topography.

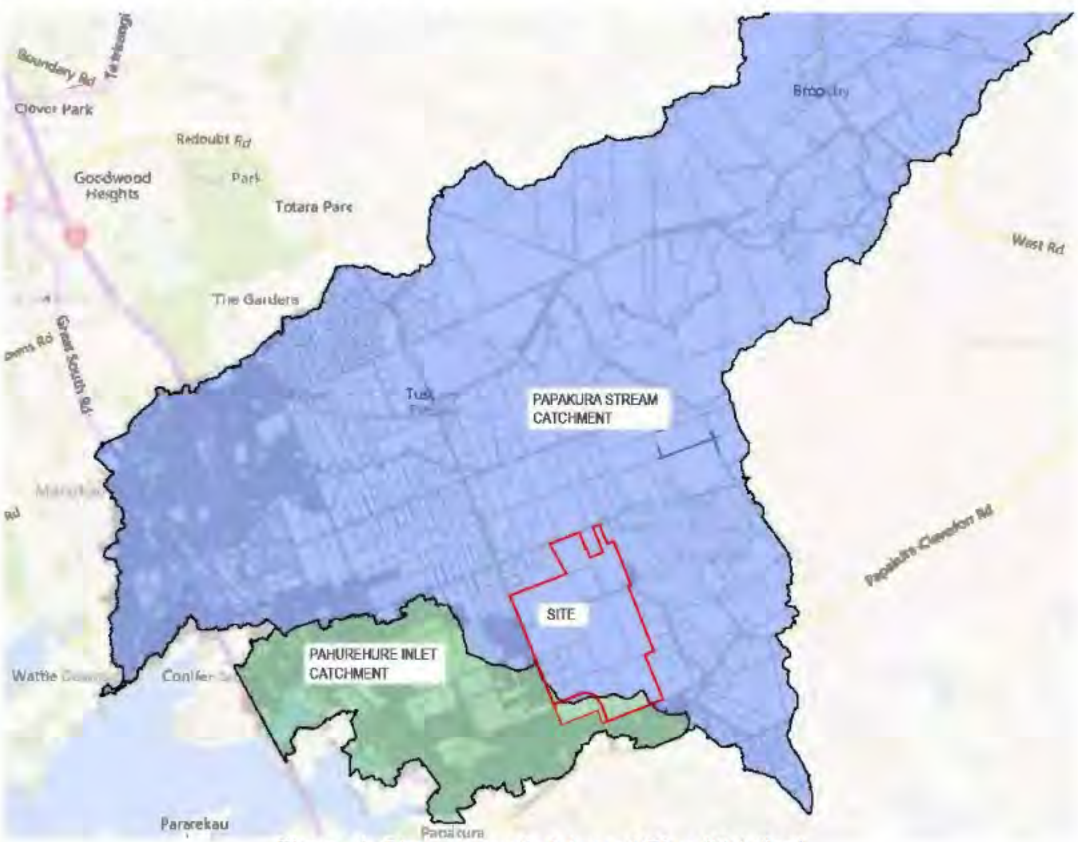

Figure 4: Stormwater Catchment Plan (Existing)

For the purpose of addressing future stormwater management for the Site it has been divided into four catchments – Catchments A, B, C and D. The pre-development and post-development extents for these catchments have been identified in Figures 8 and 9 in Section 3.3 below.

Catchment A is within with the Pahurehure Inlet catchment and Catchments B, C, and D are within the Papakura Stream catchment.

The current land zoning for the Site comprises approximately 56.5ha of FUZ land and 188ha as MRZ land under the AUP(OP).

The existing land use within the Site is rural with large pastoral lots. Existing buildings in the area are predominately farm related buildings or single dwellings. The western and southern side of the Site is bordered by residential land, and the eastern and northern is bordered by rural land.

The topography of the Site is generally flat, rising steeply on the south-eastern side to a ridge with a further flat land and existing stream corridor on the eastern side of the ridge.

3.1.1 WESTERN STORMWATER CATCHMENT (A1 and A2)

The catchment downstream of existing western catchment discharges to Pahurehure Inlet and ultimately the Manukau Harbour. The Pahurehure Inlet is a large shallow sheltered inlet, with the Southern Motorway traversing through the middle and dividing the inlet into two distinctive portions. To the east of the motorway, the inlet is further sheltered by the motorway embankment.

Downstream Existing Land Use:

Existing land uses downstream of the existing western catchment in the Central Papakura Area are dominated by residential activities. There is also commercial land use in the Central Business District in the south-western corner of the Pahurehure Inlet catchment.

Existing Infrastructure:

The major existing stormwater infrastructure (catering for flows up to 100-year ARI event) servicing the existing developed residential areas downstream, and including the western catchment, are man-made. The infrastructure includes Stage 1 of Awakeri Wetlands, a box culvert (from Grove Road to McLennan Wetland) under Battalion Drive, McLennan Wetland and the Artillery Drive Tunnel.

The Awakeri Wetlands/TSWCC:

The Awakeri Wetlands (also known as Takanini Stormwater Conveyance Channel (**TSWCC**)) forms part of a greater scheme to provide stormwater servicing for the Takanini south-east area.

The purpose of the Awakeri Wetlands was to provide Areas 2A, 2B and part of Area 4 (2B4) of the Takanini Structure Plan (Areas shown in Figure 5 below) with the following:

- Provide for the full 100-year ARI event flows, effectively removing the floodplain from surrounding land.
- Deliver stormwater servicing for development within the catchment area that is not currently available.

In addition to the above, the Awakeri Wetlands also offer a corridor (both terrestrial and aquatic) that would otherwise not be provided and affords an open space with significant amenity value and the provision for pedestrian linkages and cycleways.

Figure 5 below shows the catchment areas that the current Awakeri Wetlands are intended to service.

The area is approximately 155 hectares (ha) and consists of areas 2A (50.3 ha), 'Wallace' (9.1 ha), 2B4 (57.3 ha), 2B (38.0 ha) as shown as a dotted purple line in Figure 5 below.

This catchment is within the Central Papakura ICMP area. The sub-catchments are similar to those in the ICMP and Old Wairoa Road Catchment Management Plan, with the exception of area 2B4, which, in the ICMP and Old Wairoa Road Catchment Management Plan, excludes a small triangular shaped area at the end of Pukeroa Place. The size of this area is approximately 1 hectare and is included in the catchment area of the balance of the Awakeri Wetlands.

Figure 5: Existing Awakeri Wetlands Catchment Boundary

The Awakeri Wetlands convey flows from Old Wairoa Road, Cosgrave Road, Walters Road, and Grove Road. The Awakeri Wetlands have been designed to convey the 100-year ARI storm event flows.

The development of the Awakeri Wetlands has been staged. As construction of the Wetlands is primarily required to facilitate development, timing of the relevant staging has been dependent on the timing of adjacent development.

The construction of Awakeri Wetlands has been staged as follows:

- Stage 1 – Grove Road to Cosgrave Road, including the northern branch channel.
- Stage 2 – Cosgrave Road Culvert Crossing.
- Stage 3 – Cosgrave Road to Old Wairoa Road.

Stage 1 of the Awakeri Wetlands was recently commissioned by Healthy Waters and provides stormwater servicing and flood management to land parcels west of Cosgrave Road (Area 2A and Wallace).

SDL has reached an agreement with Auckland Council to undertake the design, consenting and construction of Stages 2 and 3 of the Awakeri Wetlands.

Grove Road Box Culvert:

The existing Grove Road concrete box culvert (2.5m(H) x 3.5m(W)) conveys flows (up to 100-year ARI) from the Awakeri Wetlands catchment to the McLennan Wetland. The culvert starts at Grove Road and runs under Battalion Drive and ends at the McLennan Wetland.

McLennan Wetland:

The McLennan Wetland was constructed in 2002. This wetland already receives stormwater from the nearby Housing New Zealand development and Papakura Military Camp through to Bruce Pulman Park in the north; and Willis Road and Clevedon Road to the south. The wetland provides attenuation and treatment for of the Old Wairoa Road catchment, which includes the Awakeri Wetlands catchment), as per Figure 6 below.

Figure 6: McLennan Wetland Upper Pond Treatment Sub-Catchment (As plotted in the Takanini Stormwater Conveyance Channel – Volume two – Appendix A – Stormwater Report prepared by Hill Young Cooper limited – April 2016)

The McLennan Wetland was included in a hydrological model held by Auckland Council, which confirmed that there is enough storage to attenuate flows to an acceptable level in the Artillery Drive Tunnel. The McLennan Wetland has an upper wetland pond and a lower wetland pond. The upper wetland pond is designed to attenuate peak flood flows up to the 100-year ARI event from the upstream catchment. The lower wetland pond provides further polishing to the treated runoff from the upper pond.

In accordance with the Central Papakura ICMP, the estimated efficiency of the upper McLennan wetland is estimated at 72% (Estimated for sediment removal based on ARC TP10 on long term average basis).

The Artillery Drive Tunnel:

The Artillery Drive Tunnel is 2.5m diameter that extends over approximately 1.1km from the upper pond at McLennan Wetland and ends at the Pahurehure Inlet near the junction of Coles Crescent and Gill Avenue. The pipeline has been designed to discharge the attenuated peak flow from the McLennan Wetland in a 100-year ARI event directly to the Pahurehure Inlet.

3.1.2 EASTERN STORMWATER CATCHMENT (A3, A4, B, C AND D)

The stormwater runoff from the existing Eastern Catchment discharges north to rural land and into a tributary of the Papakura Stream, before discharging into the Papakura Stream and ultimately into the Pahurehure Inlet. As noted above, the existing Eastern Catchment is located inside the wider Papakura Stream catchment.

The Papakura Stream catchment covers an area of approximately 5,326ha, with a total stream length of approximately 63 kilometres. The land-use within the catchment is predominantly rural, with the urban area being in the lower catchment. Commercial and native forests are in the upper catchment, along with the Brookby Quarry operation. The Site is contained with the sub-catchment known as the Ardmore sub-catchment.

The Papakura Stream is a fourth order significant open watercourse draining the catchment. The stream channels are mostly natural, except for a section of engineered channels between Porchester Road and Great South Road, and otherwise where interrupted by road and railway crossings.

3.1.3 EXISTING FLOOD HAZARDS

3.1.3.1 EXISTING FLOOD PLAIN

Due to the topography of the Site being generally flat, there is doubt about the location and extent of the existing overland flow paths. The existing overland flow paths form a widespread sheet flow over a large area with shallow surface ponding in localised depressions. The existing flood plain that encompasses the Site is identified as a 100-year ARI storm event flood plain in both the Operative Auckland District Plan (Papakura Section), referred to as the "Papakura District Plan" and the Proposed Auckland Unitary Plan. The flood plain is also recorded on the Auckland Council GeoMaps.

The flood plain is approximately 15km^2 and encompasses a majority of the Takanini/Papakura area. Approximately 1.8km^2 and 430,000m^3 (12% of the overall flood plain) of this flood plain is contained within the Site. Currently in a 100-year ARI storm event, the Site is predicted to be inundated to a depth of 200 to 800mm. The floodplain is primarily a result of ineffective stormwater drainage but also flat topography, high groundwater tables and limited soakage capacity of the underlying peat fields.

There is currently no stormwater servicing of the MRZ land. This land would naturally discharge via localised ponding/flooding with overflow (via sheet flow) occurring during larger storm events to Papakura Stream, north of the Site.

Refer to Figure 7 below which shows the extent of flooding within the site in a 100-year ARI storm event.

3.1.3.2 EXISTING FLOOD PRONE AREAS

As identified within Auckland Council GeoMaps, the Site also contains localised flood prone areas scattered throughout. These are present due to localised depressions within the Site and as a result of existing roads being higher than the adjacent land.

3.1.3.3 EXISTING OVERLAND FLOW PATHS

Auckland Council's GeoMaps indicate several existing major overland flow paths (OLFPs) which traverse through the Site and generally flow from south-east to north-west direction, as mentioned previously there is doubt as to the scale and location of the OLFPs due to the very flat nature of the topography . The OLFPs originate within the Site and upstream. The catchments of the OLFPs are identified in Section 3.3.1 below. The 100-year ARI event flow rate of the respective OLFPs/catchments are discussed later in this report.

Figure 7: Existing OLFPs and Floodplain as plotted on Auckland Council's GeoMaps

3.2 STORMWATER REGULATORY AND DESIGN REQUIREMENTS

Stormwater management requirements for the Site are set out Table 2 below, with a summary of each of the listed requirements presented in **Appendix A**.

Table 2: Regulatory requirements and design guidelines relevant to the application

Requirement	Relevant Regulatory / Design to Follow
Natural resources of the Regional Policy Statement	AUP Chapter B7
Significant ecological areas	AUP Chapter D9
Water quality and integrated management	AUP Chapter E1
Stormwater management devices design	GD01
Application of principles of water sensitive design	GD04
Discharge and diversion	AUP Chapter E8
High contaminant generating areas	AUP Chapter E9
Hydrological mitigation	AUP Chapter E10
Natural hazards and flooding	AUP Chapter E36
Auckland Council Regionwide Network Discharge Consent (NDC)	NDC Schedule 4
National Policy Statement on Freshwater Management	
National Policy Statement on Urban Development	

3.3 PROPOSED STORMWATER MANAGEMENT

3.3.1 PROPOSED (POST DEVELOPMENT) STORMWATER CATCHMENTS

The stormwater analysis for the Site has been completed based on four post development catchments – Catchments A, B, C and D. In the Stormwater Modelling Report (**Appendix B**), Catchment A is identified as the exsiting Western Catchment and Catchments B, C and D are identified as the existing Eastern Catchment.

Refer to Figures 8 and 9 below for the pre and post-development stormwater catchment plan for the 100-year ARI storm event.

CATCHMENTS

Catchment A has been developed to include diverting a part of the existing Papakura Stream Catchment into the Pahurehure Inlet Catchment and incorporates all land south of the proposed Hamlin Road realignment. The diversion enables stormwater servicing of Catchment A and reduces the catchment area draining to Papakura Stream Catchment by utilising the capacity of the existing downstream flood management infrastructure, being Stage 1 of the Awakeri Wetlands and further devices downstream. Catchment A discharges to the Awakeri Wetlands and ultimately to the Manukau Harbour.

The catchment diversion reduces the area draining to Papakura Stream Catchment in the post-development scenario. The reduction in catchment size along with the proposed attenuation reduces the overall peak discharge flow rate (for up to 100-year ARI event) to less than pre-development peak discharge flow rate. This is a positive effect on the Papakura Stream Catchment which is known to have downstream flooding issues.

Catchment B discharges to the Papakura Stream Catchment and features a portion of land north of the Hamlin Road realignment. Post-development Catchment B discharges north to 526 Mill Road & 237 Airfield Road, this discharge point will be referred to as "**Northern Outflow 1".**

Catchment C discharges to the Papakura Stream Catchment and diverts the existing upstream catchment from overland flow traversing the site to a post-development engineered channel around the eastern and northern perimeter of the Site. Post-development Catchment C discharges to **Northern Outflow 1**.

Catchment D1 discharges to the Papakura Stream Catchment and encompasses land between Airfield Road and Catchment B. Post-development Catchment D1 discharges to Airfield Road, this discharge point will be referred as "**Northern Outflow 2**".

Catchment D2 discharges to the Papakura Stream Catchment and encompasses land between Airfield Road and Catchment B in the north-eastern portion of the Site. Post-development Catchment D2 discharges to Airfield Road, this discharge point will be referred as "**Northern Outflow 3".**

The proposed Hamlin Road realignment is considered the best location for stormwater catchment delineation. Hamlin Road will become a key collector road linking the Site, and the proposed industrial land to the east, to the existing urban area to the west. It is preferable not to have stormwater flows crossing a key collector road. The proposed road level will be raised above the floodplain to provide safe vehicle egress and help direct flood flows away from Hamlin Road during storm events (to the north and south discharge points).

Pre Development Catchments Overview

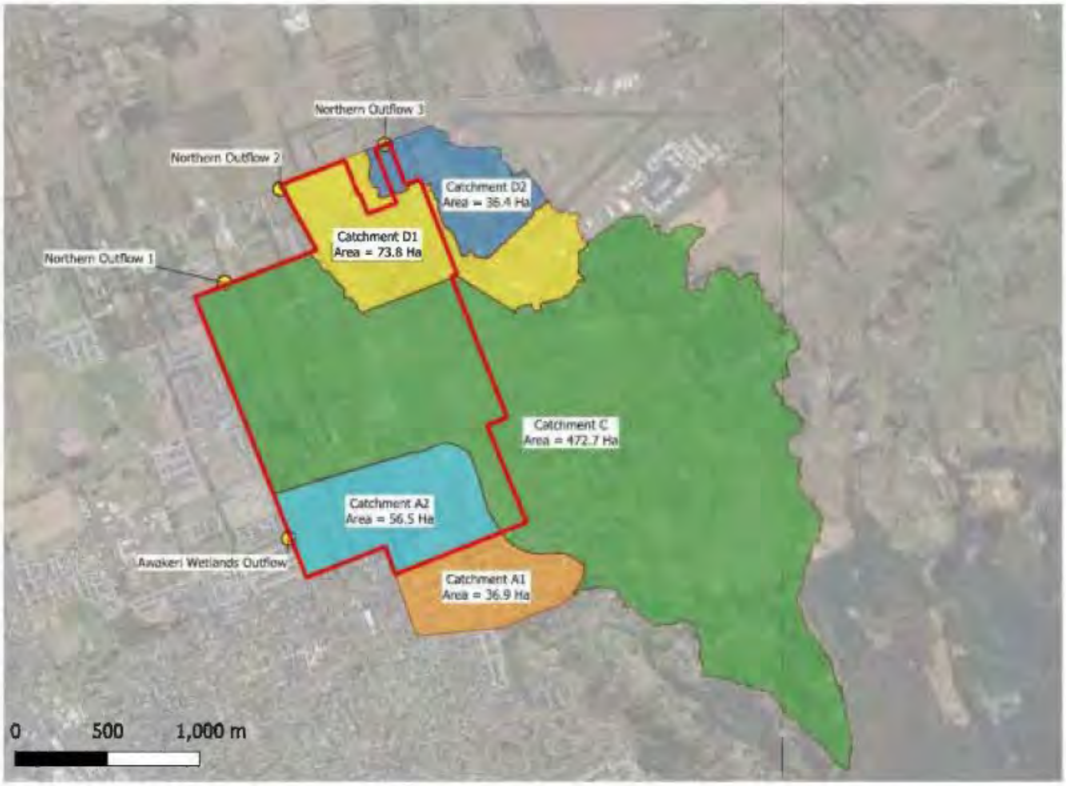

Figure 8: Pre – Development Stormwater Catchment Plan for 1% AEP storm event

Post Development Catchments

Figure 9: Post – Development Stormwater Catchment Plan for 1% AEP storm event

3.3.2 SUMMARY OF PROPOSED STORMWATER MANAGEMENT

Table 3 below provides a summary of the proposed stormwater management for the respective catchments. The strategy for each catchment is detailed in the subsequent sections of this report.

Table 3: Stormwater management approach for the Site.

<table>
<tr><th>Proposed Catchment</th><th>Catchment A</th><th>Catchment B</th><th>Catchment C</th><th>Catchment D1</th><th>Catchment D2</th></tr>
<tr><td>Discharge to</td><td>Pahurehure Inlet Catchment

Immediate discharge point: Awakeri Wetlands

Final Discharge: Manukau Harbour</td><td colspan="4">Papakura Stream Catchment

Immediate discharge point: land adjacent to site (maintain existing discharge position)

Final Discharge: Manukau Harbour</td></tr>
<tr><td>Pre-Development Discharge</td><td>93.4 ha (Of which 50ha is in FUZ Zone)</td><td>-</td><td>472.7 ha undeveloped</td><td>73.8 ha undeveloped</td><td>36.4 ha undeveloped</td></tr>
<tr><td>Post-Development Discharge</td><td>148.3 ha (Total Catchment) developed</td><td>89.2 ha developed</td><td>382.2 ha undeveloped</td><td>19.8 ha developed</td><td>36.8 ha developed</td></tr>
<tr><td rowspan="3">SW Management</td><td colspan="5">Manage the 100-year ARI floodplain to be clear of areas of the proposed development site that will be habitable to buildings.</td></tr>
<tr><td>-Attenuate 10 to 100-year flows to achieve peak flow rate and peak water level design criteria provided by Healthy Waters for Stage 2 & 3 Awakeri Wetlands (based on MPD of FUZ land).</td><td>-Attenuate 10-to-100-year flows to maintain peak flow rate to pre-development.</td><td rowspan="2">-Conveyance of flows up to 100-year ARI storm for upstream catchment.</td><td colspan="2" rowspan="2">-Attenuate 10-to-100-year flows to maintain peak flow rate to pre-development.

-Conveyance of flows up to 100-year ARI storm.

-Hydrological mitigation to minimum SMAF 1 standard.
-Water quality in accordance with GD01.

-Ground water recharge, provide retention of 15mm runoff depth for all impervious area.</td></tr>
<tr><td colspan="2">-Conveyance of flows up to 100-year ARI storm.
-Hydrological mitigation to minimum SMAF 1 standard.
-Water quality in accordance with GD01.
-Ground water recharge, provide retention of 15mm runoff depth for all impervious area.</td></tr>
<tr><td>Proposed Catchment</td><td>Catchment A</td><td>Catchment B</td><td>Catchment C</td><td>Catchment D1</td><td>Catchment D2</td></tr>
</table>

Proposed Devices to Achieve Performance Standards				
Flood Management: Alter the extent of existing floodplain	Public Roads, Swales & Stages 2-4 of Awakeri Wetlands.	Stormwater attenuation basins, swales and roads.	Swale	Stormwater attenuation basins, swales and roads.
Flood Management: –Attenuation for 10-to-100-year flow	Communal attenuation basins.	Communal attenuation basin in the form of wetland	N/A	Communal attenuation basin in the form of wetland
Conveyance up to 10-year flow	Piped reticulation, Swales & Stages 2-4 of Awakeri Wetlands.	Piped reticulation, Swales	Swale	Piped reticulation, Swales
Conveyance up to 100-year flow	Public Roads, Swales & Stages 2-4 of Awakeri Wetlands.	Public Roads & Swales	Swale	Public Roads & Swales
SMAF 1 Requirements	Recharges pits and Communal attenuation basins.		N/A	Recharges pits and Communal attenuation basins.
Water quality – Primary Treatment	Use of non-contaminating building materials, grated catchpits and inlets to stormwater, gross pollutant filters within catchpits.		N/A	Use of non-contaminating building materials, grated catchpits and inlets to stormwater, gross pollutant filters within catchpits.
Water quality – Secondary Primary Treatment	Swales		N/A	Swales
Water quality – Tertiary Treatment	Awakeri Wetlands and Existing McLennan Wetland.	Wetland	N/A	Wetland
Ground Water Recharge & Retention	Soakage/Recharge Pits		N/A	Soakage/Recharge Pits

3.3.3 PROPOSED FLOOD MANAGEMENT

Flood management is required to achieve the following Three Waters Strategy:

- Manage the 100-year ARI floodplain to ensure there are no effects on proposed development.
- Not worsen flood risk upstream or downstream for events up to the 100-year ARI.
- Allowance for the effects of climate change by allowing a climate change factor of 3.8°c degree in accordance with Auckland Council's latest SWCoP, Version 4.

A summary of the proposed flood mitigation is shown in Table 4 below. The flood management strategy for each catchment is detailed in the subsequent sections of this report.

Stormwater modelling of the proposed stormwater management strategy has been undertaken, refer to Section 3.3.5 for a summary of the results and **Appendix B** for the detailed stormwater modelling report.

Table 4: Proposed Flood Management Devices

Proposed Catchment	Catchment A	Catchment B	Catchment C	Catchment D1	Catchment D2
Approach	Attenuation	Attenuation	Diversion of upstream catchment around development site.	Attenuation	Attenuation
Pre-Development Discharge	93.4 ha (Of which 50ha is in FUZ Zone)	-	472.7 ha undeveloped	73.8 ha undeveloped	36.4 ha undeveloped
Post-Development Discharge	148.3 ha (Total Catchment) developed	89.2 ha developed	382.2 ha undeveloped	19.8 ha developed	36.8 ha developed
Outcomes Sought By Flood Management	- Manage the 100-year ARI floodplain to be clear of areas of the proposed development site that will be habitable to buildings. -Provide 100-year stormwater servicing to a catchment where stormwater servicing is not currently present. - Not worsen flood risk upstream or downstream for events up to the 100-year ARI.				
	-Attenuate 10 to 100-year flows to achieve peak flow rate and peak water level design criteria provided by Healthy Waters for Stage 2 & 3 Awakeri Wetlands (based on MPD of FUZ land).	-Attenuate 10 to 100-year flows to maintain peak flow rate to pre-development.	-Conveyance of flows up to 100-year ARI storm for upstream catchment.	-Attenuate 10 to 100-year flows to maintain peak flow rate to pre-development.	-Attenuate 10 to 100-year flows to maintain peak flow rate to pre-development.
Proposed Devices to Achieve Performance Standards					
Conveyance up to 100-year flow	Public roads, swales & extension of Awakeri Wetlands Stages 2, 3 & 4 referred to as "**Awakeri Wetlands Stage 4**".	Public roads & swales	Perimeter swale to divert the upstream catchment to the existing discharge point referred to as "**Northern Outflow 1**"	Public roads & swales to convey OLFP to discharge point referred to as "**Northern Outflow 2**"	Public roads & swales to convey OLFP to discharge point referred to as "**Northern Outflow 3**"
Overland flow & Flood plain Management for 10-to-100-year flow.	Attenuation pond referred to as "**Stormwater Pond 4**".	-Reduction in catchment (Catchment diverted to Catchment A). -Attenuation pond referred to as "**Stormwater Pond 1**".		Attenuation pond referred to as "**Stormwater Pond 2**"	Attenuation pond referred to as "**Stormwater Pond 3**"

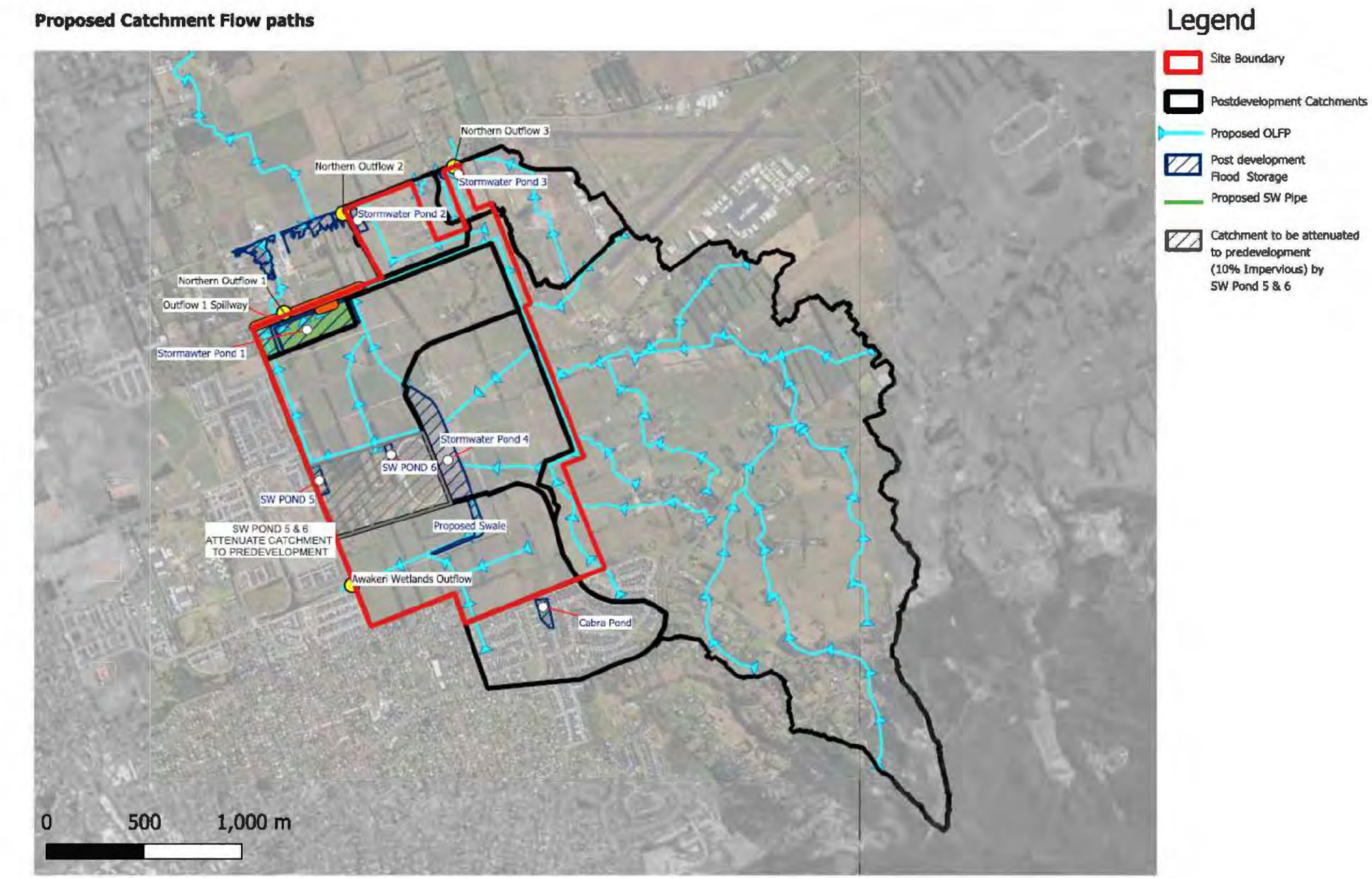

Figure 10: Eastern Catchment - Stormwater Management Devices for 100-year ARI storm event

3.3.3.1 PROPOSED FLOOD MANAGEMENT CATCHMENT A

The post-development Catchment A (148.3ha) consists of the following components:

- Existing FUZ area (56.5ha) that flows to the Awakeri Wetlands – pre-development Catchment A.
- The MRZ area south of Hamlin Road (54.9ha) – part of pre-development Catchment C (Papakura Stream catchment).
- Existing developed residential area south of Old Wairoa Road (36.9ha).

The key flood management strategy for Catchment A is the diversion of a portion of catchment draining north to Papakura Stream Catchment, this diversion into the Pahurehure Catchment is seen as beneficial to utilise the recently implemented highly engineered existing downstream flood management infrastructure.

The proposed catchment diversion reduces the area draining to Papakura Stream Catchment in post-development scenario. The reduction in catchment size along with the proposed attenuation reduces the overall peak discharge flow rate (for up to 100-year ARI event) to less than pre-development peak discharge flow rate. The 100-year peak discharge draining to the Papakura Stream catchment has reduced from 57.07m^3/s to 56.81m^3/s This has a possitive effect on the Papakura Stream Catchment which is known to have downstream flooding issues.

Due to the increase in catchment area for the post-development scenario flowing to the Awakeri Wetlands, the stormwater strategy proposes that Catchment A peak flows be attenuated to achieve downstream peak flow rate and peak water level design criteria provided by Healthy Waters for Stages 2 and 3 of the Awakeri Wetlands. These parameters have been set by Healthy Waters based on the capacity of Stage 1 of the Awakeri Wetlands, which was designed to take flows from the MPD of the FUZ land. The MPD of the FUZ land is the permitted development baseline.

3.3.3.1.1 DESIGN PARAMETERS FOR CATCHMENT A

Overall, the total stormwater flow from Catchment A (148.3ha) post-development will be attenuated to pre-development flow expected from the development of the FUZ land (56.5ha).

Detailed design has been undertaken by Healthy Waters into the development of the Awakeri Wetlands. Consented design and completed works for Stage 1 of the Awakeri Wetlands by Healthy Waters, Hill Young and Cooper and GHD have set the parameters and constraints to consider for the upstream development of the Awakeri Wetlands).

The proposed development of the Site will increase stormwater runoff due to an increase of impervious area. Overall, the stormwater management strategy for Catchment A aims to manage this increase in stormwater runoff within the Site and eliminate any flood hazards or adverse effects either within the Site, or downstream of the Site, which could result from development. Peak flows, water levels and entry and exit locations of OLFPs shall be maintained to ensure upstream and downstream conditions are not adversely affected by the development of the Site.

Specific Requirement for site discharge into the existing Awakeri Wetlands:

Through consultation, Healthy Waters has provided the design parameters for Stages 2 and 3 of the Awakeri Wetlands (included in **Appendix 10** of stormwater modelling report (**Appendix B)**). The proposed stormwater management for Catchment A proposes to maintain these parameters.

The design requirements were prepared based on the SWCoP version 3 climate change factors. As detailed in the stormwater modelling report, to account to the updated climate change factors a baseline scenario model was developed for three storm events (50%, 10% and 1% AEP) showing the flows and water levels in the Awakeri Wetlands and downstream with the updated climate change factor outlined in AC SWCoP version 4.

Topographical survey was undertaken to confirm the existing elevation of the Upper McLennan wetland. This was surveyed to be generally 14.86 mRL (NZVD2016).

3.3.3.1.2 PROPOSED EXTENSION OF AWAKERI WETLANDS (STAGES 2, 3 and 4)

The detailed design undertaken by Healthy Waters for the Awakeri Wetlands proposes to control the 100-year ARI event flood flows within the design Wetland corridor, effectively removing the floodplain from the surrounding area.

Stage 1 of the Awakeri Wetlands was recently commissioned by Healthy Waters, and the Wetlands now provide stormwater servicing to land parcels west of Cosgrave Road. The recent extreme storm events in Auckland, which generated rainfall in excess of a 100-year ARI event (200-year ARI event recorded in central Auckland), provided a means of testing the performance of Stage 1 of the Awakeri Wetlands. Overland flows were conveyed within the existing road network and discharged into the existing Wetlands. The Wetlands conveyance channel performed as designed, with the flood hazards being contained and conveyed within the channel. The performance of the Awakeri Wetlands conveyance channel provides validation that the stormwater management approach adopted for the extension of the conveyance channel can perform in real life scenarios and perform to withstand rarer storm events. The proposed extension (Stages 2, 3 and 4) of the Awakeri Wetlands will service the Catchment A area and will be designed to the same standards as the existing section of the Awakeri Wetlands.

SDL has reached an agreement with Auckland Council to undertake the design, consenting and construction of Stages 2 and 3 of the Awakeri Wetlands.

Stages 2 and 3 will provide stormwater servicing for the FUZ land and a portion of the MRZ land.

Detailed engineering plans have been prepared for both Stages 2 and 3. Stage 2 design is for the box culverts under Cosgrave Road and the Stage 3 design is a 40m wide channel up to 3m in depth with a low-level permanent stream and side batters ranging from 1:3 to 1:5 up to ground level.

To provide stormwater servicing of the remainder of Catchment A (part MRZ land) an extension to the Awakeri Wetlands (Stage 4) where a pond is proposed (refer to Figure 10 above for location). The proposal is for a 40m wide channel extension to the northern border of the FUZ land, and a 100m wide extension through the MRZ land.

The extension (Stage 4) will be 2m deep and have 1:3 side batters on each side up to ground level. It is envisaged that weirs will be incorporated between the 40m and 100m sections to ensure that flows are adequately restricted through each stage and to mitigate downstream flooding effects.

As required by the proposed consent conditions, the detailed design of the proposed engineering solution will be subject to EPA process by the local authority.

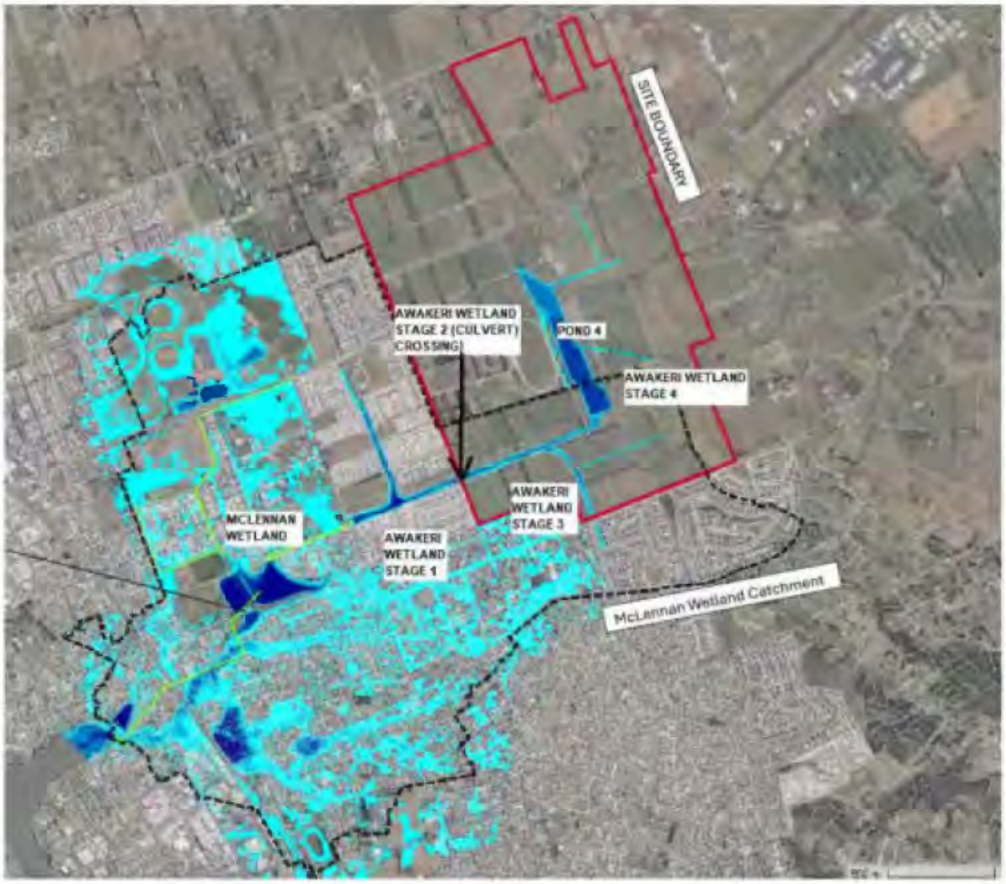

Figure 12- Post development 100-year ARI extent in Awakeri Wetlands Stages 2-4

3.3.4 ADDITIONAL STORMWATER SWALES

Secondary stormwater swales have been incorporated into the overall masterplan to convey stormwater runoff from rain fall events up to the 10-year event and also provide additional storage for the 100-year flood flows.

The stormwater swales will convey flows from the development area to the centralised main conveyance channel (Awakeri Wetlands). Each stormwater swale will be 1.0 – 2.0m deep with base widths 2 -10m. Side slopes will be 1:4 batters with overall channel widths ranging from - 22m.

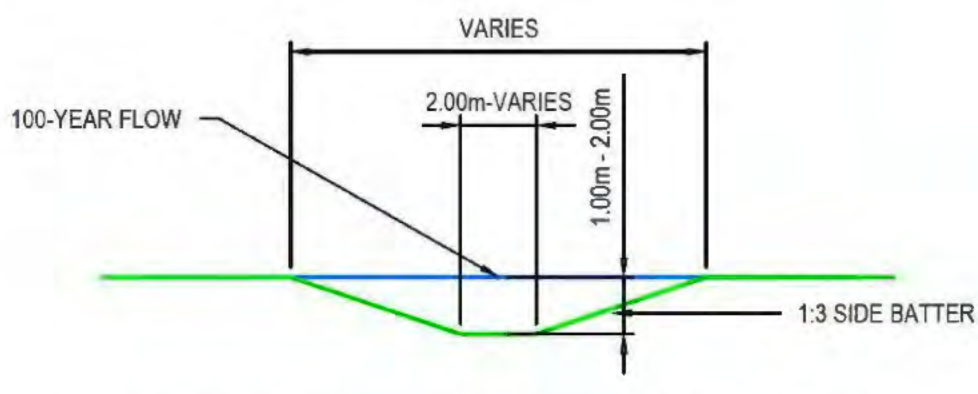

Figure 13: Typical Section detail of Secondary Stormwater Swales

3.3.4.1 PROPOSED FLOOD MANAGEMENT STRATEGY FOR CATCHMENT B

Catchment B is approximately 89.2ha (area north of Hamlin Road) and is entirely zoned MRZ. This catchment will continue to discharge stormwater to the north into the Papakura Stream via Northern Outflow 1.

Overall stormwater flow from Catchment B post-development will be attenuated to discharge at existing pre-development levels.

Stormwater attenuation will limit stormwater runoff to the pre-development conditions (flow rate) by utilising stormwater storage during rain fall events up to and including the 100-year ARI event.

Stormwater attenuation will prevent any increase in flows resulting from future land use changes/increase in impervious surfaces associated with the development of the Site.

There will be no increased risk of flooding from displaced flood storage as:

1. compensatory flood storage will be provided within the proposed wetland pond (Stormwater Pond 1) and stormwater swales; and
2. the wetland pond and stormwater swales have been designed to contain 100-year ARI event flood flows.

Post-development 100-year ARI event flows are proposed to be attenuated to existing pre-development levels via a proposed wetland pond (Stormwater Pond 1) and secondary stormwater swales, which will convey flows from the catchment to the wetland pond.

A summary of pre to post peak flows rates and attenuation volumes for Catchment B are provided below in Section 3.3.5.

3.2.1 PROPOSED FLOOD MANAGEMENT STRATEGY FOR CATCHMENT C

Stormwater runoff from Catchment C1 (382.2ha) currently flows through the development site via overland flow during larger rain events. Auckland Council Geomaps aerial image identifies two permanent watercourses which traverse the site from south-east to north-west direction. Onsite these watercourses are in the form of artificial farm drains.

It is proposed to redirect the flows from the two permanent upstream watercourses along the eastern perimeter of the Site via an engineered swale. Refer to Figure 14: Overland Flowpath Diversion Swale.

Stormwater modelling (**Appendix B**) has determined that the proposed swale will require a surface area ranging from 20-40m wide and will require a trapezoid shape up to 1.5m deep. This engineered swale will be formed with a low flow channel representing a natural stream during final design.

Once the swale reaches the northern boundary, it is proposed to continue the swale west along the northern boundary of the Site at a flat grade to form a basin with a weir or level spreader outlet structure. Stormwater would pond within the proposed swale before overflowing to the north via controlled sheet flow over the proposed level spreader at the current pre-development flows.

Figure 14: Perimeter overland Flowpath Diversion Swale

The interface from the eastern boundary to northern boundary will incorporate specific erosion control with a raised turnout area to ensure stormwater flows do not overtop the swale during larger storm events.

The proposed swale alignment intercepts all stormwater runoff from the adjacent upstream land, with the run-off conveyed through an engineered channel along the outer boundary. It is likely that the low-flow swale will form a permanent natural steam shape, with large riparian banks either side to cater for larger storm events.

The 100-year ARI event flood flows within the proposed swale are designed to have a minimum 500mm freeboard to finished floor level of any habitable buildings in the adjacent neighbouring properties.

Figure 15: Extent of flooding within Perimeter Swale and Proposed Pond 1 for the diversion of upstream catchment in Eastern Catchment

3.2.2 PROPOSED FLOOD MANAGEMENT STRATEGY FOR CATCHMENT D

Catchment D comprises two sub-catchments, known as Catchment D1 and Catchment D2.

Catchment D1:

Post development Catchment D1 is approximately 19.8ha and is proposed to be entirely zoned Industrial/Employment. Catchment D1 will continue to discharge stormwater to the north via an existing discharge point ("Northern Outflow 2") and onwards to the Papakura Stream.

Catchment D2:

Post development Catchment D2 is approximately 36.8ha and is proposed to be zoned a mix of rural, residential, commercial and Industrial/Employment. Catchment D2 will continue to discharge stormwater to the north via an existing discharge point ("Northern Outflow 3") and onwards to the Papakura Stream.

Attenuation for 10 & 100-year ARI events will be provided for catchment D1 and D2 via the proposed storm water ponds (Stormwater Pond 2 for Catchment D1 and Stormwater Pond 3 for Catchment D2).

A summary of pre to post peak flows rates and attenuation volumes for Catchment D are provided below in Section 3.3.5.

3.3.5 SUMMARY OF STORMWATER MODELLING RESULTS

Stormwater modelling has been undertaken by Maven to provide preliminary sizing of stormwater management devices. A detailed Stormwater Model is contained in **Appendix B,** this includes details of all parameters used in the modelling.

Below are summary of the proposed stormwater management devices.

Eastern catchment:

Element	2yr Storage Volume (m3)	10yr Storage Volume (m3)	100yr Storage Volume (m3)	Outlet
Stormwater Pond 1 (Outflow 1)	68,000	77,000	141,000	700m weir
Stormwater Pond 2 (Outflow 2)	8,390	13,580	22,290	180mm SMAF outlet 1350mm Scruffy dome
Stormwater Pond 3 (Outflow 3)	1,030	1,510	1,820	68mm SMAF outlet 700mm weir cutout

Table 5.1: Eastern catchment attenuation device configuration summary

Element	10yr Peak flow Pre-development(m^3/s)	10yr Peak flow Post development(m^3/s)
Northern Outfow 1	22.0	21.6
Northern Outfow 2*	2.35	0.64
Northern Outfow 3*	0.50	0.49

Table 5.2: 10year Eastern catchment site discharge predevelopment versus post development flow summary

Element	100yr Peak flow Pre development(m^3/s)	100yr Peak flow Post development(m^3/s)
Northern Outfow 1	52.0	51.8
Northern Outfow 2*	4.17	4.14
Northern Outfow 3*	0.90	0.87

Table 5.3: 100year Eastern catchment site discharge predevelopment versus post development flow summary

Flood levels and peak flow post development were compared to the predevelopment flood levels and peak flows. No negative effects were highlighted in any of the modelling results.

Western Catchment:

Element	2yr Storage Volume (m3)	10yr Storage Volume (m3)	100yr Storage Volume (m3)	Outlet
Stormwater Pond 4	23,280	51,170	94,000	Box Culvert 1.0m x 1.0m

Table 7.4: Western catchment attenuation device configuration summary

Storm Event	Baseline Modelled Scenario Peak SWCoPv4 Climate Change factors flow (m^3/s)	Post development modelled Peak SWCoPv4 Climate Change factors flow(m^3/s)
50% AEP (2yr ARI)	6.9	5.8
10% AEP (10yr ARI)	15.9	13.9
1% AEP (100yr ARI)	28.0	25.3

Table 5.5: Awakeri Wetlands Stage 2 peak flows

Storm Event	Baseline Modelled Scenario Peak SWCoPv4 Climate Change factors flow (m^3/s)	Post development modelled Peak SWCoPv4 Climate Change factors flow(m^3/s)
50% AEP (2yr ARI)	11.6	10.6
10% AEP (10yr ARI)	26.3	24.6
1% AEP (100yr ARI)	45.6	44.1

Table 5.6: Grove Road Culvert peak flow difference from post development site discharge

Results from the modelling analysis conclude the proposed development will not adversely impact the upstream and downstream properties. Modelled peak flow levels within the TSWCC either remain unchanged or are reduced as a result of the development.

Flood storage in the post development scenario is shown to be contained within the Upper McLennan wetland. Peak flows spilling out of the Upper McLennan Spillway during a 1%AEP storm are shown to be slightly reduced in the post development scenario.

3.3.6 CONVEYANCE

3.3.6.1 STORMWATER NETWORKS

Within the proposed development primary flows generated by flows up to the 10 year ARI storm event will be conveyed by swales and piped network to the downstream receiving environment. Stormwater infrastructure will be designed to accommodate the 10-year ARI storm event for the MPD, including climate change, and in accordance with requirements of Chapter 4 of the Auckland Council Code of Practice for Land Development and Subdivision (Auckland Council SWCOP v4)

Within both the FUZ land and MRZ land of the Site, proposed secondary stormwater swales have been incorporated into the design to convey stormwater flows from rainfall events to the proposed stormwater devices (Awakeri Wetlands extension and proposed Stormwater Ponds 1-4).

Each proposed stormwater swale will be 1.0 – 2.0m deep with base widths of 2 – 10m, side slopes will be 1:4 batters with overall channel widths ranging from 8 - 22m.

The primary 10-year ARI event pipe reticulation will be designed to outfall to either the proposed stormwater swales, or directly to the main conveyance channel where possible. The pipe network will be installed to the invert of the swales with ground levels raising away at gradients similar to design pipe gradients to maintain pipe cover.

This design principle has been used on sites adjacent to the existing Awakeri Wetlands Stage 1 where there was limited cover and fall to maintain minimum ground cover as per the SWCOP This resulted in some surcharge in the pipe networks, which was considered acceptable.

The 10-year ARI event flood levels will be modelled at detailed design stage and will be a proportion of the 100-year ARI event flow depth. It is envisaged the 10-year ARI flows will be restricted to the lower portions of the Awakeri Wetlands extension and proposed Stormwater Pond 1, with limited surcharge into the secondary stormwater swales and/or the stormwater pipe reticulation.

As required by the proposed consent conditions, the detailed design of the proposed stormwater network will be subject to EPA process by the local authority.

For stormwater modelling purposes, although initial stormwater runoff recharges directly to ground before overflowing to the public network, no initial abstraction (decrease in initial runoff) has been used for stormwater modelling purposes (assume full saturation during 100-year event).

It is likely however that a portion of stormwater discharge during rainfall events up to and including the 10-year ARI event would recharge to ground via the recharge pits prior to discharge to the reticulation network and public stormwater devices.

3.3.6.2 OVERLAND FLOW PATHS

For events greater than a 10-year ARI storm event and up to a 100-year ARI storm, secondary flows (i.e. the flows which exceed the primary network) will be conveyed along road and swales/drainage reserves and green spaces as overland flow paths.

The overland flow paths will adhere to the following design criteria:

- Overland flow paths will be designed with sufficient capacity to accommodate the 100-year ARI storm event for the MPD, including climate change, in accordance with the Auckland Council SWCOP.
- They will be unobstructed, with capacity to safely convey runoff through the development.
- Overland flows to follow either road reserves or dedicated green areas. All flow paths are proposed to be located within public areas (roads/parks) where practicable and not over private properties without easement or other approval by Auckland Council.

3.3.7 HYDROLOGICAL MITIGATION

Hydrological mitigation seeks to minimise the change in hydrology, namely runoff volumes and flow rate, as a result of development.

Chapter E10 of the AUP(OP) sets out a hydrological mitigation framework for brownfield sites that discharge to sensitive or high-value stream environments and have been identified as particularly susceptible to the effects of development. This framework must be applied to developments within the AUP(OP) management Stormwater Management Area Control – Flow 1 and Flow 2 (SMAF) overlay.

The Site is a greenfield development and therefore does not fall within the AUP(OP) SMAF overlay. However, Schedule 4 of the NDC specifies that all greenfield sites located outside a SMAF zone that discharge to a stream via public stormwater network should "achieve equivalent hydrology (infiltration, runoff volume, peak flow) to pre-development (grassed state) levels. A method of achieving equivalent hydrology to pre-development (grassed state) is to" provide retention (volume reduction) and detention (temporary storage) for all impervious areas equivalent to SMAF 1.

Catchments B & D discharge to the Papakura Stream catchment and therefore to achieve NDC Schedule 4 equivalent hydrology requirements, it is proposed to provide the equivalent of SMAF 1 framework for these two catchments.

Catchment A does not discharge downstream to a stream and therefore Schedule 4 of the NDC does not require the stormwater network to achieve equivalent hydrology. Although NDC does not require this catchment to provide equivalent hydrology, to provide positive effects to the receiving environment, Catchment A will also provide the equivalent of SMAF 1 framework.

The SMAF 1 hydrological mitigation requirements in the Table E10.6.3.1.1 of AUP(OP) are:

- **Retention** (volume reduction) of at least 5mm runoff depth for the impervious area for which hydrology mitigation is required; and
- **Detention** (temporary storage) and a drain down period of 24 hours for the difference between the pre-development and post-development runoff volumes from a 95th percentile, 24-hour rainfall event minus the achieved retention volume, over the impervious area for which hydrology mitigation is required.

Exceptions for providing retention can be made in cases where soil infiltration rates preclude disposal to ground, and rainwater reuse is not possible. The retention volume may be taken up by detention if a suitably qualified person has confirmed that soil infiltration rates are less than 2 mm/hr or there is no area on the site of sufficient size to accommodate all required infiltration that is free of geotechnical limitations (including slope, setback from infrastructure, building structures or boundaries and water table depth).

Proposal to Achieve SMAF Retention Requirement:

The requirement to provide 5mm of run off depth retention will be achieved by providing ground water recharge pits, which will provide the infiltration equivalent to 15mm of runoff depth from impervious area.

Proposal to Achieve SMAF Detention Requirement:

The detention required will be provided within the proposed stormwater ponds and recharge pits within each catchment. It is noted that the detention volume required to achieve SMAF 1 involves subtracting the retention. Therefore, the additional retention provided beyond the minimal required by SMAF 1 will result in less detention volume being required within the ponds.

3.3.8 PROPOSED GROUND WATER RECHARGE

A geotechnical review of the Site has indicated that peat soils are present throughout the majority of the site and therefore stormwater recharge of the ground will be required wherever impervious area is proposed.

Geotechnical investigations recorded groundwater depths ranging from 1.5m to 3m below ground level. In order to maintain the groundwater levels as close to their current state as possible, recharge pits will be installed to allow recharge of the peat soils. Recharge pits will be installed wherever impervious surfaces are proposed to capture runoff and infiltrate into the peat to recharge the localised groundwater table.

Recharge pits will be designed to retain the stormwater runoff from all impervious areas from the first 15mm of any rainfall event. Recharge pits will be designed as per the Auckland Council guidance document GD07 - SWCoP and Stormwater Soakage and Groundwater Recharge in the Auckland Region.

The retention provided by the recharge pits will provide hydrological mitigation via supporting stream baseflows and reducing erosive flows during small storm events.

Recharge pits will be installed as part of each lot specific development.

Preliminary calculations have indicated that onsite recharge pits for each lot would entail a 4.7m² below ground recharge pit for every house, based on 150m² impervious area per lot. (100m² roof captured and 50m² pavement) Refer **Appendix C.**

Recharge pits will also be installed as part of the public road design located adjacent to proposed catchpits and sizing will be based on impervious areas typically ranging from 100m² up to 1,000m².

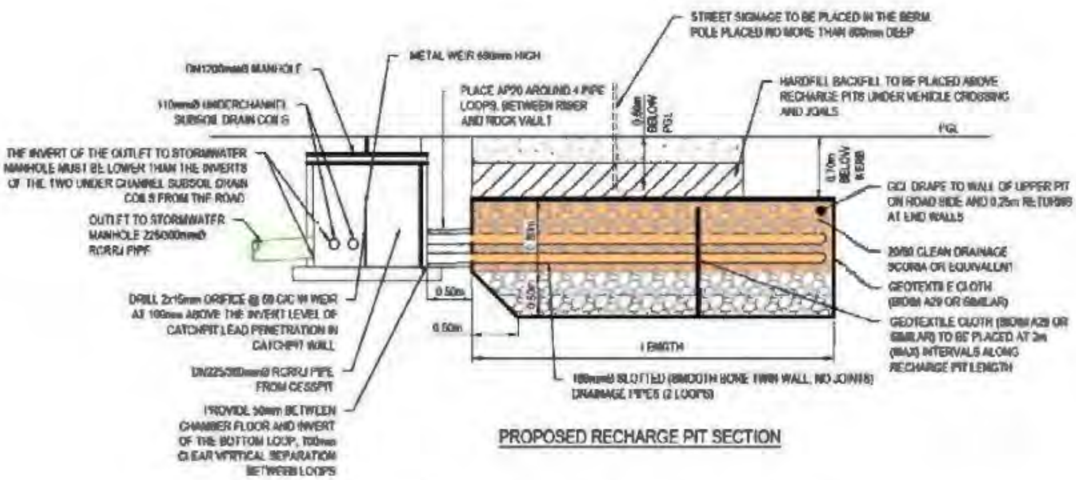

Figure 16: Typical Road Reserve Recharge Pit Detail

3.3.9 WATER QUALITY

Stormwater quality requirements will be provided via stormwater management devices consistent with the requirements of Auckland Council guidance document GD01 - Stormwater Management Devices.

Stormwater quality will be achieved via a stormwater treatment train and will include:

1. Primary treatment of stormwater will occur at the source, via use of non-contaminating building materials, grated catchpits and inlets to stormwater, gross pollutant filters, such as tetra traps within catchpits to ensure a high quality of stormwater recharge into the underlying peat soils (via recharge pits).
2. Runoff from public roads will be captured by a catchpit fitted with a 'tetra trap' or similar over the outlet pipe before overflow to the reticulated pipe network. This will help prevent coarse sediment and other gross pollutants entering the recharge pits. Although tetra

traps do not provide GD01 level of treatment as per the NDC requirements, their use is currently standard practice in peat land areas and is considered the best practical option.

3. Secondary treatment will be provided via stormwater swales. The stormwater swales will direct and slow stormwater across vegetation. Swales filter sediments, nutrients and contaminants. The stormwater swales will convey flows for up to 100-year ARI event from within the site to discharge into the Awakeri Wetlands (for Catchment A) and proposed Wetlands (for Catchments B and D).
4. Tertiary treatment for Catchment A will be provided by proposed Wetland 1, Awakeri Wetlands and the existing McLennan Wetland. Wetlands uses biological processes to provide sediment removal through enhanced sedimentation and biological uptake.

Catchment A drains to the existing McLennan Wetland. As detailed in Section 3.1.1, the McLennan Wetland provides stormwater quality treatment for the zoned upstream land (57Ha FUZ) before ultimately discharging stormwater to Pahurehure Inlet. The remainder of catchment A will be treated by the proposed Stormwater Pond 1.

Tertiary treatment for Catchments B and D will be provided by the proposed Stormwater Ponds 1, 2, 3 & 4. This will provide a high level of stormwater quality treatment before ultimately discharging to the Papakura Stream.

3.3.10 DOWNSTREAM EFFECTS

3.3.10.1 CATCHMENT A

As described in Section 3.1 above, Catchment A discharges to Pahurehure Inlet and ultimately the Manukau Harbour. Existing infrastructure that conveys Catchment A to the Pahurehure Inlet is all man-made and includes:

1. Stage 1 of Awakeri Wetlands;
2. Box Culvert (2.5m(H) x 3.5m(W) from Grove Road to McLennan Wetland) under Battalion Drive;
3. McLennan Wetland; and
4. Artillery Drive Tunnel

3.3.10.1.1 FLOW EFFECTS

Within Catchment A, stormwater attenuation will limit peak flows resulting from future land use changes/increase in impervious surfaces to acceptable levels by restricting the peak flows (up to 100-year ARI) and water levels to that set by Healthy Waters for Stage 1 of the Awakeri Wetlands. The existing Awakeri Wetlands and the other downstream infrastructure have sufficient capacity to service the peak flows from Catchment A without creating any additional surcharge of the existing infrastructure downstream as a result of increased flows and water levels.

Attenuation is typically avoided in the lower portion of a stormwater catchment and encouraged in the upper half, as it is likely to create coincidence of flood peaks that would worsen the downstream flooding and increase flood risk upstream. There will be no increase in flood levels in the downstream sections of the Awakeri Wetlands or on the existing adjacent local stormwater network.

The location of the Site is in the upper half of the catchments and will therefore not create coincident peak flows as demonstrated by stormwater modelling provided in **Appendix B**.

There will be no increased risk of flooding from displaced flood storage as compensatory flood storage will be provided within the proposed Awakeri Wetlands Stage 4 extension and additional stormwater swales proposed as part of this stormwater management strategy, having been designed to have capacity to contain 100-year ARI flood flows.

3.3.10.1.2 VOLUME EFFECTS

Although stormwater management devices are typically used on greenfield developments to manage the post-development stormwater hydrology, it is common for the management device not to retain stormwater runoff volume to pre-development level and only manage flows to pre-development level. This is because the size of the management device required to achieve volume retention typically makes it unfeasible, and the volume effects are considered to have less adverse effect on the environment than flow.

In Auckland, SMAF regulations have been placed to protect and enhance rivers, streams and aquatic biodiversity within Auckland's urban areas, whilst also allowing developments to continue. SMAF retention requires a portion of flow to be kept out of the stormwater network to reduce the risks associated with flash flows in regular small events. For the Site, the proposed hydrological mitigation to flash flows is provided through the retention of the proposed ground water recharge pits being 15mm runoff depth for all impervious area. This is a greater level of retention than the

5mm run off depth retention that is set out by AUP(OP) framework (Chapter E10) SMAF hydrological requirements.

The catchment size for Catchment A will increase pre to post-development and therefore the stormwater runoff volume will exceed the runoff volume anticipated from the current catchment. As a result, the effects of increased stormwater volume need to be considered on the existing downstream infrastructure servicing Catchment A.

As described above, the existing infrastructure that conveys Catchment A is man-made. The increase in stormwater runoff volume on the existing concrete infrastructure (Grove Road Box culvert and Artillery Drive Tunnel) has negligible effect as concrete structures servicing stormwater are typically designed to always be saturated for the intended design life.

It is assumed that the increase in stormwater runoff volume will have negligible effect on Stage 1 of the Awakeri Wetlands in terms of erosion as the wetland channel is designed with permanent retentions pools of water to manage erosion. The increase in stormwater runoff volume will not change the function of these pools in terms of erosion.

The increase in stormwater runoff volume may result in increased sediment deposition into the existing wetland channel and therefore the frequency of maintenance could increase. Consultation with Healthy Waters as part of this application will need to be required to confirm whether current maintenance frequency will be sufficient or whether a new maintenance regime will need to be adopted.

The treatment catchment (zoned land) of the existing McLennan remains the same as remainder of Catchment A is being treated by proposed Wetland 1, therefore the development will not have any adverse effects in terms of the McLennan Wetlands treatment capacity. The existing wetland has an overflow system for any larger flows and therefore any effects from increase in volume can be considered negligible.

3.3.10.2 CATCHMENTS B, C AND D

The stormwater runoff from Catchments B, C and D discharges north to rural land and then into minor tributary streams before discharging into the Papakura Stream and finally out into the Pahurehure Inlet.

Unless carefully managed, urbanisation can lead to adverse stream bank erosion effects due to the increased runoff rate and volume. Mitigation measures (such as increased detention, flood plain management or in-stream works) may be required to manage any potential effects when there are already bank erosion and stream stability issues in the downstream watercourses.

The scale and severity of this requires more detailed geomorphological assessment as a part of engineering design, and so should be addressed at EPA stage.

3.3.10.2.1 FLOW EFFECTS

Stormwater mitigation for Catchments B and D involves attenuating peak flows up to the 100-year ARI storms to match pre-development peak flows.

As a result, the effects on the downstream environment will not worsen from peak flows for up to the 100-year ARI storm.

With the combination of reduction in catchment and attenuation, the overall peak discharge flow for up to 100 ARI-event has reduced. This is a positive effect for the Papakura stream catchment which is known to have flooding issues downstream.

AUP(OP) SMAF requirements were created to minimise the adverse effects of stormwater runoff on rivers and streams to retain, and where possible enhance, stream naturalness, biodiversity, bank stability and other values. SMAF 1 hydrological mitigation is being provided for Catchments B and D, which includes providing detention (temporary storage) and a drain down period of 24 hours for the difference between the pre-development and post-development runoff volumes from a 95th percentile, 24-hour rainfall event minus the achieved retention volume, over the impervious area.

3.3.10.2.2 VOLUME EFFECTS

Erosion susceptibility is typically mitigated through retention of post-development stormwater flows. Retention requires a portion of flows to be kept out of the stormwater network to reduce the risks associated with flash flows in regular small events.

The hydrological mitigation for flash flows from stormwater runoff from Catchments B and D of the Site is provided by SMAF 1 hydrological mitigation. SMAF 1 requires retention (volume reduction) of at least 5 mm of runoff depth from impervious surfaces. For the Site, a higher level of retention will be provided than that required by SMAF 1 through the retention of ground water recharge to 15mm runoff depth for all impervious area. In addition, the proposal to include riparian plantings in stormwater management devices is also expected to reduce erosion vulnerability.

4.0 WASTEWATER AND WATER SUPPLY

4.1 CURRENT WASTEWATER CONTEXT

Watercare Services are tasked with servicing the greater Auckland region with both wastewater and potable water supply. The only area of Auckland to which this applies to a lesser extent is Papakura. Within this area, Watercare are responsible for the overall network and trunk mains, whilst Veolia operates the local network.

Wastewater generated by the existing activities within the development area is treated through septic tanks.

The surrounding developed residential areas dispose of wastewater via LPS systems and gravity reticulation to the existing 525mmØ Takanini Branch Sewer line located on Walters Road on the eastern boundary of Bruce Pullman Park. The transmission line traverses north-west and discharges into the transmission pump station located at the Wattle Farm Ponds Reserve in Manurewa. From there, the transmission network continues to traverse north-west and ultimately discharges into the Māngere Wastewater Treatment Plant.

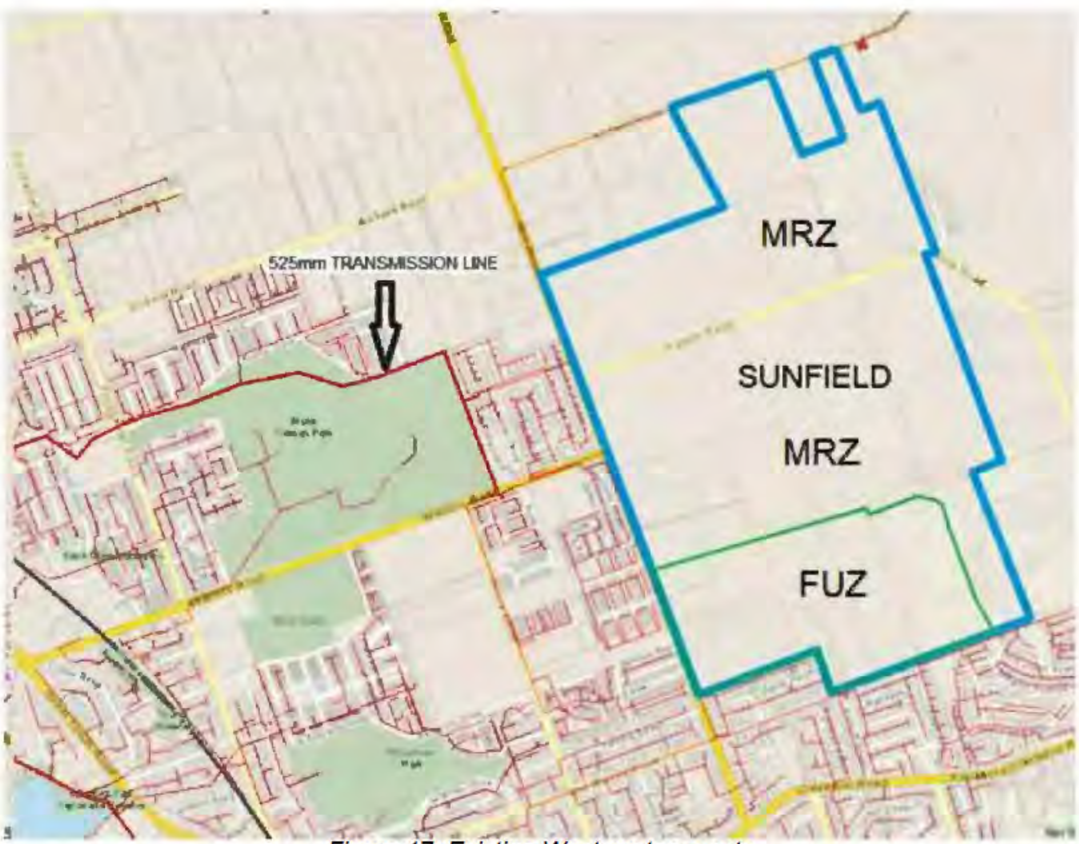

Figure 17: Existing Wastewater assets

4.2 PROPOSED WASTEWATER STRATEGY

Wastewater discharge from the developed site will be restricted to the allowable discharge anticipated under the development of the FUZ land to avoid adverse downstream effects.

Veolia and Watercare have confirmed that the existing Watercare Transmission network has capacity to service the peak wet weather flow ("**PWWF**") from the FUZ upon its development into 1,550 dwellings, entailing a PWWF of 64.91L/s. This has been calculated by taking 1550 household units (as shown in Councils Awakeri Stage 3 Framework Plan) and calculating wastewater disposal as per Watercare standards, using a PWWF factor of 6.7.

The proposed wastewater servicing strategy for Site is to design and construct an LPS system. The final design will be detailed at engineering approval stage.

The wastewater network will provide wastewater reticulation within the development and will discharge flows to the downstream Takanini Branch Sewer (being the existing 525mmØ transmission line) via a new rising main along Cosgrave Road, Walters Road and Mill Road. Refer to attached engineering plans for the proposed wastewater network.

LPS systems are considered an acceptable alternative to the typical gravity wastewater disposal systems in areas that have:

- flat low-lying terrain;
- poor underlying soil quality; and
- a high water table.

The Site includes each of these components. Its underlying low strength peat soils and high water table (which varies from 1m to 3m below the ground surface) have historically led to gravity wastewater networks 'dipping' and holding wastewater overtime and increases the risk of inflow and infiltration.

This is supported by evidence during the construction of the downstream Takanini Branch Sewer, whereby it was noted significant baseflows were entering the system due to the high ground water table.

LPS systems have been successfully implemented, and adopted by Watercare, in residential developments throughout Auckland including at the Kuaka Drive development of 210 lots and the Mill Road development of 330 lots, both of which are located in close proximity to Sunfield.

Both of these developments utilise LPS due to the reasons outlined above (being flat ground structure, underlying ground conditions and high water table) and have been successfully operational for a number of years.

The incorporation of an LPS system greatly reduces the ultimate peak discharge. Without inflow and infiltration, the Watercare standards indicate that ADWF ("**Average Dry Weather Flow**") with an added capacity safety factor of 1.2 per dwelling unit can be used for discharge instead of the PWWF. Through the inclusion of an LPS system, the preliminary calculations for the demand for the Site entails a flow of 57.63L/s, which is less than the 64.91L/s of capacity of network anticipated from the FUZ land.

Relevant wastewater demand calculations are contained within Figure 16.

This option would therefore provide wastewater servicing for the Site, keeping discharge below the existing downstream capacity.

This ensures that subject to the network extensions proposed, no downstream infrastructure upgrades are required to service the intended development on the Site.

4.3 LOW PRESSURE SEWER OWNERSHIP MODEL

For LPS systems, Watercare has adopted the private pump ownership model. As such, all on-site installation responsibilities fall onto the property owner.

Under the private ownership model, the property owner is responsible for selecting and purchasing the grinder pump and associated on-property equipment usually from a list of pre assessed pumps defined by the system designer and approved by Auckland Council.

Under this option, the property owner (or their representative, such as a residential builder or building company) is primarily responsible for the installation. The public reticulation from the point of supply, including the boundary kit is designed, installed, and vested in Auckland Council by the developer.

The publicly vested pressure reticulation network will be located in the public road reserve parallel to the property boundaries. Where a subdivision does not provide a dwelling with direct public road frontage, a multi-kit box shall be provided.

A multi-kit box shall not house more than six individual boundary kits. Where more than six individual boundary kits are required for dwellings not fronted by a public road, a bulk point installation shall be used with individual private boundary kits located inside the property.

For industrial and commercial lots, 'custom' storage tanks with multiple pumps can be installed. Detailed design will be required for the design and use of the custom units.

4.3.1 MONITORING AND MAINTENANCE

The development will set up a residents' society to monitor and maintain the LPS system. The monitoring and maintenance of the system will be controlled by a reputable supplier similar to Ecoflow and will use a OneBox/smart controller on each pump to control the pumps.

The smart controller allows the private pumps to 'talk' to each other and allows pumps to activate at different times. This allows the morning and evening peak flows to be decreased, therefore decreasing the chance of any overflow.

Each smart controller will have an alarm to alert potential overflow and allow emptying as required. An uncommon issue is an extended power cut. The developments solar power energy supply would help prevent this issue. Monitoring and maintenance from a reputable supplier though a residents' society would ensure potential overflow would be unlikely to occur. A sucker truck can also be dispatched to empty private pump systems to prevent overflow, if necessary.

4.3.2 FLUSHING

Flushing will be provided for the LPS system at the subdivision staged occupancy rates of 30%, 50%, 80%, and greater. The developer will provide the expected development occupancy fill rate. Based on the expected speed of development and flushing requirements, the developer will be responsible for the flushing costs until an occupancy rate is achieved that will provide adequate self-cleansing flowrates in the pressure main.

The developer will fund these costs before connecting to the Watercare system and will also control the residents' society. A flushing programme with fresh water and/or injection of special chemicals will prevent any potential for Hydrogen Sulphide build up.

4.4 LOW PRESSURE SEWER SYSTEMS

As per Watercare's Code of Practice, the use of an LPS system will require approval from Veolia and Watercare. The LPS must be demonstrated to provide:

(a) Equivalent or lower life cycle cost to Watercare than other options.

The public reticulation will consist of shallow pressure mains and boundary kits located in the public road reserve. The overall network reticulation will be less than a standard gravity wastewater model, which would incorporate pump stations and deep gravity lines requiring increased maintenance overtime.

(b) Costs passed onto homeowners are reasonable.

LPS systems place instalment and maintenance costs onto the property owners. Installation costs of the LPS system will be factored into sale prices. Ongoing maintenance costs will be minimal and be covered by the residents' society, which will levy owners.

(c) A reliable service in accordance with Watercare's customer charter so that failure of a component does not cause total system failure.

The development will set up a residents' society to monitor and maintain the LPS system. As noted in clause [4.3.1] above, the monitoring and maintenance of the system will be controlled by a reputable supplier using a OneBox/smart controller on each pump to control the pumps to prevent system failure.

(d) Which site-specific problems it will overcome and how.

The Site contains low strength peat soils, which have historically led to gravity wastewater networks 'dipping' and holding wastewater overtime. The risk of inflow and infiltration is high in peat soils, LPS systems create a sealed network eliminating inflow and infiltration.

(e) How the system will impact on the environment from events arising from system failures such as spills, power outage or pipe breaks and how the system mitigates these issues.

The residents' society will ensure that the maintenance and operation of the LPS system will be ongoing and prevent owners from failing to maintain or replace their private wastewater infrastructure.

(f) A discharge point that can be integrated into the existing wastewater network.

The site is located less than 500m away from the 525mmØ transmission line located on Walters Road on the eastern boundary of Bruce Pullman Park, identified as the Takanini Branch Sewer. A connection to this line along Walter Road is feasible.

4.4.1 PEAK FLOWS

LPS systems create a sealed network eliminating inflow and infiltration reducing peak flow discharge. Without inflow and infiltration, the Watercare standards determine that ADWF be used for discharge instead of PWWF, an LPS safety factor of 1.2 is specified.

FUZ using Gravity/ Pumpstation			
Population	Dwellings	People	Occupancy
Permitted Discharge	1550	3	4650
Permitted Discharge	Persons	Rate l/p/day	Flow l/s
ADWF	4650	180	9.69
PDWDF	4650	540	29.06
PWWF	4650	1206	64.91 l/s
Development Site using LPS			
Residential/ Retirement	Dwellings	People	
	4000	3	
Discharges	Persons	Rate l/p/day	Flow l/s
ADWF	12000	180	25.00
Light Industrial	Ha		
	55.9		
Assume 55% building coverage	30.8		
Discharges	Ha	Rate l/m2/day	Flow l/s
ADWF	30.8	4.5	16.04
Retail, Town Centre & Health Care	Ha		
5.3ha + 4.9ha + 3.3ha	13.5		
Assume 55% building coverage	7.4		
Assume 80% net area	5.9		
Discharges	Ha	Rate l/ha/s	Flow l/s
ADWF	5.94	1	5.94
Schools	Students	Rate l/person/day	
	2000	45	
Discharges	Persons	Rate l/person/day	Flow l/s
ADWF	2000	45	1.04
ADWF			48.02
PWWF(1.2 LPS Peaking Factor)	48.02	1.2	57.63 l/s
Total Discharge			57.63 l/s

Figure 18: Allowable and Predicted Wastewater Discharge

**The table above indicates using an LPS system will generate similar wastewater discharge as a typical gravity feed system servicing the FUZ area – based on using a 1.2 PWWF factor of safety component on all ADWF flows.*

Utilising an LPS system decreases calculated discharge volumes. Consequently, the number of equivalent household units able to discharge into the downstream Watercare network can be increased (refer to the Figure 18 Wastewater discharge). This option would therefore provide wastewater servicing for the entire Site, whilst keeping discharge to the permitted development baseline (development of the FUZ land).

Detailed design and acceptance of the LPS system is to be confirmed with Watercare and Veolia as part of the s.224(c) approval.

4.5 PROPOSED WATER SUPPLY STRATEGY

The proposed strategy outlined in this Three Waters Strategy report is to reticulate the development with a new water supply network for potable water and firefighting services, to be supplied from the existing water supply network.

The proposed Sunfield development is located fully within the old Papakura District Council area and is partly included in Watercare's identified Takanini Water Supply Zone. Responsibility for the operation, maintenance and connections to the public water supply networks are with Veolia.

Future development of the Site will require a network water supply for potable water and firefighting servicing designed to Watercare's Code of Practice requirements and subject to approval from Veolia.

Water supply demand calculations for the Site have been completed and are attached in **Appendix E**. Water demand is calculated at approximately 70.56 l/s for average daily demand, 85.84 l/s peak day demand and 154.59 l/s for peak hourly demand.

4.5.1 WATER SUPPLY/ FIRE FIGHTING REQUIREMENTS

The Watercare Code of Practice for Land Development and Subdivision sets out the design principles for water supply and requires assessment against SNZPAS 4509:2008 NZ Fire Service Fire Fighting Water Supply Code of Practice.

The minimum firefighting water supply classification for residential development is FW2. Therefore, any future residential development must meet the following water supply requirements:

- A primary water flow of 12.5 litres/sec within a laid distance of 135m.
- An additional secondary flow of 12.5 litres/sec within a radial distance of 270m.
- The required flow must be achieved from a maximum of one or two hydrants operating simultaneously.
- A minimum running pressure of 100kPa.

For the industrial and commercial areas, specific design will be required to identify the FW classification as per SNZPAS 4509:2008 NZ Fire Service Fire Fighting Water Supply Code of Practice.

4.5.2 BULK SUPPLY POINT

Preliminary Investigations with Veolia have indicated that a connection to the nearest BSP will be necessary to provide the minimum firefighting water supply classification for the development of the site. A public water main will need to be extended from the BSP point to the Site.

Figure 19 – Wastewater and Water supply transmission lines below indicates the closest BSP points located on Airfield Road. The two closest BSPs identified from Watercare's BSP GIS file are the Airfield #1 and Porchester Road BSPs.

To provide sufficient water supply for future development of the Site, a new public water main will connect the site to the Airfield #1 BSP located on the 450mmØ transmission line on Airfield. The BSP may need to be upgraded as part of these works.

If this BSP point does not have sufficient capacity, a new BSP point may need to be constructed on the transmission line closer to the Cosgrave Road intersection. Consultation with Veolia and Watercare will be required to confirm the preferred connection point and capacity.

As the majority of water supply for Auckland originates in the south and the close proximity of transmission line to the site, an engineering solution for either an upgraded BSP or new BSP can be developed to supply the Site's water demand.

Water supply hydraulic modelling (Contained within **Appendix F**) has been undertaken to preliminary size the bulk water supply network for the site. For design purposes, an 800 kPa pressure is assumed at the connection points to the transmission network. Given its primary role as a water transmission line and its proximity to the Drury Water Pump Station and the Hunua Water Reservoir, the pressure within this transmission network is expected to be relatively high, ranging between 250 kPa and 1200 kPa, in accordance with the Design Principles for Transmission Water and Wastewater Pipeline Systems (DP-07, Version 1.1).

The pressure drop caused by water draw from the proposed development is anticipated to be minimal due to the constant regulated pressure maintained by the Hunua 1 and Waikato 1 transmission mains. This pressure level has been adopted as the baseline for the proposed water main loop connections.

Figure 19: Water supply Transmission lines

5.0 CONCLUSIONS

The Three Waters Strategy for the Site is to incorporate a WSUD approach to create a low impact, sustainable development that minimises stormwater and wastewater discharge from the site.

The overarching principle of the Sunfield Three Waters Strategy is to implement an integrated management approach, which:

Stormwater:

- Enables development by delivering stormwater servicing to catchments where stormwater servicing is not currently present.
- Emphasises a water-sensitive design approach that:
  - minimises or mitigates the adverse effects on water quality, freshwater systems, stream health, and ecological values of the receiving environment through the implementation of stormwater management devices; and
  - protects and enhances stream systems and natural hydrology, while mitigating hydrological changes and managing flooding effects.
- Minimises the generation and discharge of contaminants/sediments into the sensitive receiving environment of the Manukau Harbour.
- Manages the 100-year ARI floodplain to ensure there are no adverse effects on proposed development.
- Ensures that the flood risk upstream or downstream for events up to the 100-year ARI is not increased.
- Allows for the effects of climate change by allowing a climate change factor of 3.8°c degrees in accordance with SWCoP Version 4.

The proposed stormwater management strategy adopts integrated best-practice approach across the Site to:

- Flood Management:
  - Use flood management devices and manage the 100-year ARI floodplain to ensure there are no adverse effects on proposed development.
  - Enable stormwater servicing of Catchment A and reduce the catchment area draining to the Papakura Stream Catchment by utilising existing flood management infrastructure.
  - For Catchment A, attenuate peak flows up to the 100-year ARI storms to mitigate downstream flooding by attenuating peak flows up to the 100-year ARI events to match pre-development peaks flows.

    Results from the modelling analysis of Catchment A conclude the proposed development will not adversely impact the upstream and downstream properties. Modelled peak flow levels within the TSWCC either remain unchanged (meets requirements set out by Healthy Waters for TSWCC) or are reduced as a result of the development.

    Flood storage in the post development scenario is shown to be contained within the Upper McLennan wetland. Peak flows spilling out of the Upper McLennan Spillway during a 1%AEP storm are shown to be slightly reduced in the post development scenario.

- For Catchments B and D, mitigate downstream flooding by attenuating peak flows for the 2, 10 & 100-year ARI events to match pre-development peaks flows. Attenuation will be provided via proposed on-site stormwater management devices.

  Flood levels and peak flow post development were compared to the predevelopment flood levels and peak flows. No negative effects were highlighted in any of the modelling results.

- For Catchment C, direct peak flows from the undeveloped upstream catchment around the eastern perimeter of the proposed development site via an engineered swale to allow flows to bypass (no attenuation) for flows up to the 100-year ARI event.

An Auckland Unitary Plan E36 flood risk assessment has been undertaken to demonstrate how the proposed stormwater management mitigates flood risks. This is contained within **Appendix 14** of the Stormwater Modelling Report (**Appendix B**).

- Conveyance:
  - Ensure primary flows of up to 10-year ARI event be conveyed by swales and a piped network.
  - Ensure that storm events generating 10 to 100-year ARI stormwater will be conveyed along public roads and swales within drainage reserves and green spaces as dedicated overland flow paths.
  - Extend Awakeri Wetlands from the already completed and commissioned Stage 1 to provide conveyance of storm flows up to 100-year ARI event from Catchment A of the Site.
- Hydrological Mitigation:
  - Hydrological mitigation to minimum of AUP(OP) SMAF 1 hydrological mitigation standard will be provided to minimise the change in hydrology (maintain predevelopment) for post-development catchments draining to Papakura Stream Catchment (Catchments B and D).
  - SMAF 1 requirements are as follows:
    - Retention (volume reduction) of at least 5mm of runoff depth from impervious surfaces.
    - Detention (temporary storage) and a drain down period of 24 hours for the difference between the pre-development and post-development runoff volumes from a 95th percentile, 24-hour rainfall event minus the achieved retention volume, over the impervious area for which hydrology mitigation is required.
- Water Quality:
  - Provide stormwater quality treatment through treatment devices such as Tetra Traps, stormwater swales and stormwater/wetland ponds.
- Ground Water Recharge:
  - Ground water recharge of the peat soil via soakage/recharge pit to ensure the retention of existing groundwater levels. Recharge pits will retain the stormwater

runoff from all impervious areas from the first 15mm of any rainfall event The retention provided by the recharge pits will also provide hydrological mitigation.

Catchment A – Pahurehure Inlet Catchment (148ha), which includes the FUZ area (50ha) and MRZ south of Hamlin Road (98ha), is to drain to the proposed Awakeri Wetlands being a proposed extension to Stage 1 of the Wetlands (Stages 2, 3 and 4).

To mitigate downstream flooding stormwater attenuation (for up to the 100-year ARI events) will be provided to Catchment A to attenuate flows. This will be achieve via extension to the Awakeri Wetlands Stage 4, Stormwater Pond 4 and secondary stormwater swales within FUZ land).

Catchment B – Papakura Stream Catchment (89ha) will continue to discharge stormwater to the north. Post-development flow will be attenuated to pre-development conditions via a combination of a proposed stormwater wetland pond (via Stormwater Pond 1) and swales.

Catchment C – Papakura Stream Catchment (382ha) stormwater runoff from upstream currently flows through the Site. It is proposed to redirect this flow via an engineered swale along the eastern boundary of the Site and discharge north via a newly constructed level spreader outlet structure (upgrade of the existing discharge point).

Catchment D (comprising both Catchments D1 and D2) – Papakura Stream Catchment (20ha & 36ha respectively) - will continue to discharge stormwater to the north. Post-development flow in Both Catchments D1 and D2 will be attenuated to pre-development flows (existing pre-development catchment discharging north) via proposed stormwater ponds.

Wastewater & Water supply

- Incorporates wastewater networks, including new and existing private connections, to the network allows the minimum practicable seepage into and out of the networks.
- Ensures waste materials entering the networks are controlled to avoid or minimise adverse effects on physical assets, wastewater treatment processes and the environment.
- Minimises overflows from the networks during both dry and wet weather as far as practicable.
- Creation of infrastructure that is of good quality, meets health requirements and minimises ongoing maintenance costs.
- Meets future demands on maintainability and access as infrastructure ages and the natural environment changes.
- Uses water efficiently and minimises wastage as best practicable.

5.1 RECOMMENDATIONS

From an engineering perspective, proposed infrastructure servicing can be achieved via methods consistent with current relevant AUP(OP) requirements and relevant Engineering Standards. Subject to detailed design and approval from the local authorities, we believe there are no infrastructure issues that would preclude the Site being developed for the proposed land use.

We recommend final solutions are developed after consultation with third party stakeholders including Veolia, Watercare, Auckland Council, Auckland Transport and Healthy Waters.

6.0 GLOSSARY

Definition	Meaning
SDL	Sunfield Developments Limited
FTA	Fast-track Approvals Act
Site	The contiguous 244.5ha site to be developed as a master planned community and to be known as "Sunfield"
FUZ	Future Urban zone
MRZ	Mixed Rural zone
AUP(OP)	Auckland Unitary Plan (Operative)
MPD	Maximum probable development
SMP	Stormwater Management Plan
WSUD	Water Sensitive Urban Design
ARI	Average Recurrence Interval
SWCoP	Auckland Council's Stormwater Code of Practice
NDC	Network Discharge Consent
NPS-UD	National Policy Statement on Urban Development
Eastern Catchment	Current – Papakura Stream
Western Catchment	Current – Pahurehure Inlet
Catchment A	post development catchment
Catchment B	post development catchment
Catchment C	post development catchment
Catchment D	post development catchment
LPS	Low-pressure sewer
BSP	Bulk pressure supply point
ICMP	Integrated Stormwater Catchment Management Plan
Awakeri Wetlands OR TSWCC	Takanini Stormwater Conveyance Channel
OLFP	Overland flow path
PWWF	Peak wet weather flow
AEP	Annual Exceedance Probability
ADWF	Average dry weather flow

APPENDIX A – A SUMMARY OF REGULATORY REQUIREMENTS

The relevant regulatory guidelines and stormwater specific are listed in Table 2 of Section 3.2 of this report and a summary on each of the listed requirements is presented below.

Natural resources of the Regional Policy Statement

Chapter B7 of the AUP sets out policies for degraded coastal water, freshwater and geothermal water areas requiring an integrated management, the minimisation of the generation and discharge of contaminants in stormwater and the adoption of the BPO for every stormwater diversion and discharge. These are:

- Policy 4, 6 and 7 (B7.4.2.4, B7.4.2.6 and B7.4.2.7):

  Identify areas of coastal water and freshwater bodies that have been degraded by human activities and progressively improve water quality in areas identified as having degraded water quality through managing subdivision, use, development and discharges to avoid where practicable, and otherwise minimise, all of the following:

  - *significant bacterial contamination of freshwater and coastal water.*
  - *adverse effects on the quality of freshwater and coastal water.*
  - *adverse effects from contaminants, including nutrients generated on or applied to land, and the potential for these to enter freshwater and coastal water from both point and non-point sources.*
  - *adverse effects on Mana Whenua values associated with coastal water, freshwater and*
    geothermal water, including wāhi tapu, wāhi taonga and mahinga kai.
  - *adverse effects on the water quality of catchments and aquifers that provide water for domestic and municipal supply.*

- Policy 8 (B7.4.2.8):

  Minimise the loss of sediment from subdivision, use and development, and manage the discharge of sediment into freshwater and coastal water, by:

  - *promoting the use of soil conservation and management measures to retain soil and sediment on land.*
  - *requiring land disturbing activities to use industry best practice and standards appropriate to the nature and scale of the land disturbing activity and the sensitivity of the receiving environment.*

- Policy 9 (B7.4.2.9):

  Manage stormwater by all of the following:

  - *requiring subdivision, use and development to:*
    - *minimise the generation and discharge of contaminants.*
    - *minimise adverse effects on freshwater and coastal water and the capacity of the stormwater network.*
  - *adopting the BPO for every stormwater diversion and discharge.*
  - *controlling the diversion and discharge of stormwater outside of areas serviced by a public stormwater network.*

Significant ecological areas

Chapter D9 of the AUP sets out policies regarding the management of stormwater runoff to receiving environments within a Significant ecological Areas. The relevant stormwater policy is summarised below:

- Policy 2 (D9.3.2)

Adverse effects on indigenous biodiversity values in significant ecological areas that are required to be avoided, remedied, mitigated or offset may include, but are not limited to, downstream effects on wetlands, rivers, streams, and lakes from hydrological changes further up the catchment.

Water quality and integrated management requirements

Chapter E1 of the AUP contains the following relevant stormwater management policies:

- Policy 2a and 2b (E1.3.2a and E1.3.2b)

Manage discharges, subdivision, use and development that affect freshwater systems to:

- *Maintain or enhance water quality, flows, stream channels and their margins and other freshwater values where the current condition is above the relevant thresholds (refer Table E1.3.1 of the AUP).*

  OR

- *Enhance water quality, flows, stream channels and their margins and other freshwater values where the current condition is below the relevant thresholds (refer Table E1.3.1 of the AUP).*

- Policy 3 (E1.3.3)

Require freshwater systems to be enhanced unless existing intensive land use and development has irreversibly modified them such that it practicably precludes enhancement.

- Policy 4 (E1.3.4)

Discharges must *avoid contamination that will have an adverse effect on the life supporting capacity of freshwater.*

- Policy 5 (E1.3.5)

Discharges must *avoid contamination that will have an adverse effect on health of people and communities.*

- Policy 8 (E1.3.8)

Avoid as far as practicable, or otherwise minimise or mitigate, adverse effects of stormwater runoff from greenfield development on freshwater systems, freshwater and coastal water by:

- *Taking an integrated stormwater management approach (refer to Policy E1.3.10)*
- *Minimising the generation and discharge of contaminants, particularly from high contaminant generating car parks and high use roads and into sensitive receiving environments*
- *Minimising or mitigating changes in hydrology, including loss of infiltration, to:*
  - *Minimise erosion and associated effects on stream health and values*
  - *Maintain stream baseflows*
  - *Support groundwater recharge*
- *Where practicable, minimising or mitigating the effects on freshwater systems arising from changes in water temperature caused by stormwater discharges*
- *Providing for the management of gross stormwater pollutants, such as litter, in areas where the generation of these may be an issue.*

- Policy 10 (E1.3.10)

An integrated stormwater management approach must have regard to all of the following:

- *The nature and scale of the development and practical and cost considerations*
- *The location and design of site and infrastructure to protect significant site features and minimise effects on receiving environments*
- *The nature and sensitivity of receiving environments*
- *Reducing stormwater flows and contaminants at source*
- *The use and enhancement of natural hydrological features and green infrastructure where practicable.*

Policy 11 (E1.3.11)

Avoid, minimise or mitigate adverse effects of stormwater diversions and discharges.

Policy 12 (E1.3.12)

Manage contaminants in stormwater runoff from high contaminant generating car parks (> 50 cars*) and high use roads* (>5000 vehicles per day) *to minimise new adverse effects and progressively reduce existing adverse effects on water and sediment quality in freshwater systems and coastal waters.*

Policy 13 (E1.3.13)

Require Stormwater quality or flow management to be achieved on-site unless there is a downstream communal device.

Policy 14 (E1.3.14)

Adopt the best practicable option to minimise the adverse effects of stormwater discharges.

Policy 15 (E1.3.15)

Utilise stormwater discharge to ground soakage where it is possible to do so in a safe and effective manner.

Water sensitive design

Water-sensitive design is a philosophy that is integral to achieving integrated stormwater management, required by Policy 8 (E1.3.8). Water-sensitive design is defined as:

"An approach to freshwater management, it is applied to land use planning and development at complementary scales including region, catchment, development and site. Water sensitive design seeks to protect and enhance natural freshwater systems, sustainably manage water resources, and mimic natural processes to achieve enhanced outcomes for ecosystems and our communities."[11]

Water-sensitive design principles are further detailed in GD04. The key principles for water sensitive design are summarised as follows:

- Promoting inter-disciplinary planning and design
- Protecting and enhancing the values and functions of natural ecosystems

- Addressing stormwater effects as close to source as possible
- Mimicking natural systems and processes for stormwater management.

Discharge and diversion

Chapter E8 of the AUP sets out policies which regulate the diversion and discharge of stormwater runoff from impervious areas into or onto land, or into water, or into the coastal marine area. The objectives are consistent with Chapter E1 and E2 of the AUP. The general standards (E8.6.1) are summarised below:

- The design of the proposed stormwater management device(s) must have consistent with any relevant precinct plan that addresses or addressed stormwater matters.
- The diversion and discharge must not cause or increase scouring or erosion at the point if discharge or downstream.
- The diversion and discharge must not result in or increase the following:
  - Flooding of other properties in rainfall events up to the 10 Year ARI; or
  - Inundation of buildings on other properties in events up to the 100 Year ARI.
- The diversion and discharge must not cause or increase nuisance or damage to other properties
- The diversion and discharge of stormwater runoff must not give rise to the following in any surface water:
  - The production of conspicuous oil or grease films, scums or foams, or floatable or suspended materials
  - Any conspicuous change in colour or visual clarity
  - Any emissions of objectionable odour
  - The rendering of fresh water unsuitable for consumption by farm animals; or
  - Any significant adverse effects on aquatic life
- Any existing requirements for ground soakage, including devices to manage discharges and soakage, must be complied with.

For diversion and discharge of stormwater runoff from lawfully established impervious areas as at 30 September 2013 not directed to a stormwater network or combined sewer network (E8.6.2.2) the following policies also apply:

- As a result of a new land activity, a change in land use or the removal of existing stormwater management measures, stormwater flows and volumes and the concentration and load of contaminants in stormwater flows from the existing impervious areas must not be increased above those that would result from lawfully established impervious areas existing as of 30 September 2013
- Any existing stormwater management devices must not be reduced, and the location of discharge must not change.

High contaminant generating areas

Chapter E9 of the AUP outlines the regional land use rules for managing stormwater runoff quality from high contaminant generating areas (HCGAs). Treatment of runoff is required for HCGAs (as defined in the AUP) including:

- High use roads (with greater than 5,000 vehicle movements per day)
- Car park areas with greater than 50 vehicles per day
- High contaminant yielding building and roofing materials
- Industrial/Trade sites listed as high risk in Schedule 3 will require assessment

under the ITA rules which may result in treatment being provided

- Treatment of discharges to the CMA will be required due to the receiving environment being identified as a SEA.

Stormwater runoff from the HCGAs is to be treated by stormwater management device(s) which is sized and design in accordance with Guidance Document 2017/001 - Stormwater Management Devices in the Auckland Region (GD01) or where alternative devices are proposed, the device must demonstrate it is designed to achieve an equivalent level of contaminant or sediment removal performance to that in GD01.

Hydrological mitigation

Hydrological mitigation seeks to minimise the change in hydrology, namely runoff volumes and flow rate, as a result of development. Chapter E10 of the AUP sets out a hydrological mitigation framework for brownfield sites which discharge to sensitive or high-value stream environments that have been identified as particularly susceptible to the effects of development. This framework must be applied to developments within the AUP management Stormwater Management Area Control – Flow 1 and Flow 2 (SMAF) overlay.

The subject development is a greenfield development and therefore does not fall within the AUP SMAF overlay. However, Schedule 4 of the NDC specifies that all greenfield sites located outside a SMAF zone which discharge to a stream via public stormwater network should "achieve equivalent hydrology (infiltration, runoff volume, peak flow) to pre-development (grassed state) levels. A method of achieving equivalent hydrology to pre-development (grassed state) is to" provide retention (volume reduction) and detention (temporary storage) for all impervious areas equivalent to SMAF 1. The proposed stormwater management principles for the subject development is to provide a minimum of the SMAF 1 framework to provide hydrological mitigation for all impervious surfaces within Catchment B,C & D (as these catchments ultimately discharge to Papakura Stream downstream). The SMAF 1 hydrological mitigation requirements in the AUP are:

- Retention (volume reduction) of at least 5 mm of runoff depth from impervious surfaces where possible with limitations set out in Table E10.6.3.1.1.

- Detention (temporary storage) and a drain down period of 24 hours for the difference between the pre-development and post-development runoff volumes from a 95th percentile, 24-hour rainfall event minus the achieved retention volume, over the impervious area for which hydrology mitigation is required.

Exceptions for providing retention can be made in cases where soil infiltration rates preclude disposal to ground and rainwater reuse is not possible. The retention volume may be taken up by detention if:

- a suitably qualified person has confirmed that soil infiltration rates are less than 2 mm/hr or there is no area on the site of sufficient size to accommodate all required infiltration that is free of geotechnical limitations (including slope, setback from infrastructure, building structures or boundaries and water table depth)

Natural Hazards and flooding

Chapter E36 of the AUP sets out the policies relating to the management of natural hazards and flooding. The relevant policies are summarised briefly below.

- Policy 1 (E36.3.1)

Identify land subject to natural hazards, taking into account the likely effects of climate change.

- Policy 5 (E36.3.5)

Avoid development in greenfield areas which would result in an increased risk of adverse effects from coastal hazards, taking account of a longer-term rise in sea level in areas subject to coastal hazard

- Policy 17 (E36.3.17)

Avoid locating buildings in the 100 year ARI flood plain unless it can be designed to be resilient to flood related damage.

- Policy 20 (E36.3.20)

Earthworks within the 100 year ARI flood plain should not permanently reduce floodplain conveyance or exacerbate flooding experienced by other sites upstream or downstream.

- Policy 21 (E36.3.21)

Ensure all development in the 100 year flood plain does not increase adverse effects or increased flood depths or velocities to other properties upstream or downstream of the site.

- Policy 29 and 30 (E36.3.29 and E36.3.30)

Maintain the function and capacity of overland flow paths to convey stormwater runoff safely and without damage to the receiving environment.

Network Discharge Consent

The Auckland region-wide NDC came into effect in October 2019. The NDC allows for the stormwater diversion and discharges from developments to be incorporated under Auckland Council's consent, and for assets to be vested to Auckland Council, provided they comply with the NDC conditions.

The revised requirements and template for an SMP under the NDC are quite different to previous SMP formats and identify either a compliant approach or a BPO approach. The NDC requirements for greenfield developments, relevant to the development site, and as stipulated in the NDC Schedule 4, are:

- Treatment of 100% of impervious areas by a water quality device designed in accordance with GD01/TP10 for the relevant contaminants
- Achieve equivalent hydrology (infiltration, runoff volume, peak flow) to pre-development (grassed state) levels. A method of achieving equivalent hydrology to pre-development (grassed state) is to provide retention (volume reduction) and detention (temporary storage) for all impervious areas equivalent to SMAF 1
- Ensure that there is sufficient capacity within the pipe network downstream of the connection point to cater for the stormwater associated with the development in the 10 year ARI event, including incorporating flows from contributing catchment at MPD
- Buildings must not be flooded in the 100 year ARI event
- All new assets that are intended to become part of the public stormwater network are to be designed and constructed to be durable and perform to the required level of service for the life of the asset, subject to reasonable asset maintenance
- Stormwater management assets in the road corridor require approval from Auckland Transport prior to vesting

The requirement to provide water quality and hydrological mitigation to all impervious surfaces is more stringent than the regulations outlined in AUP, which only require treatment for high contaminant generating car parks and high use roads. It is common practice on greenfield developments with sensitive receiving environments to have treatment for all impervious areas (at least those generating contaminants, so if inert building materials are adopted it is expected that roofs can be excluded).

National Policy Statement on Freshwater Management

The NPS-FM came into effect in September 2020 and provides local authorities with direction on how to manage freshwater (including streams and wetlands) under the Resource Management Act 1991. The NPS-FM is a first step to improve freshwater management at a national level. The previous NPS-FM released in 2014 and revised in 2017, set out some keys requirements to manage freshwater resources. These include, but not limited to, the following:

- 'Consider and recognise' Te Mana o te Wai in freshwater management.
- Identify and reflect tāngata whenua values and interests in the management of freshwater and in decision-making around freshwater planning.
- Safeguard freshwater's life-supporting capacity, ecosystem processes, and indigenous species.
- Maintain or improve the overall quality of freshwater within a freshwater management unit but improve it where people recreate so that it is suitable for primary contact more often.
- Follow a specific process - the national objectives framework - for identifying the values that tāngata whenua and communities have for water.
  - Use a specified set of water quality measures (called attributes) to set freshwater objectives to achieve those values
  - Set water quality and quantity limits on resource use (e.g. how much water can be taken or how much of a contaminant can be discharged) to meet the freshwater objectives over time and ensure they continue to be met.
- Protect the significant values of wetlands and outstanding freshwater bodies.

  Take an integrated approach to managing land use, freshwater and coastal water. The 2020 version expanded on these requirements to include the following:
- Manage freshwater in a way that 'gives effect' to Te Mana o te Wai:
  - through involving tāngata whenua.
  - working with tāngata whenua and communities to set out long-term visions in the regional policy statement.
  - prioritising the health and wellbeing of water bodies, then the essential needs of people, followed by other uses.
- Improve degraded water bodies and maintain or improve all others using bottom lines defined in the NPS-FM.
- An expanded national objectives framework:
  - Two additional values - threatened species and mahinga kai - join ecosystem health and human health for recreation, as compulsory values.
  - Councils must develop plan objectives that describe the environmental outcome sought for all values (including an objective for each of the five individual components of ecosystem health).
  - New attributes, aimed specifically at providing for ecosystem health, include fish index of biotic integrity, sediment, macroinvertebrates, dissolved oxygen, ecosystem metabolism and submerged plants in lakes; councils will have to develop action plans and/or set limits on resource use to achieve these attributes.
  - Tougher national bottom lines for the ammonia and nitrate toxicity attributes to protect 95% of species from toxic effects (up from 80%).

  - No national bottom lines for dissolved inorganic nitrogen or dissolved reactive phosphorus (as consulted on) but there is a requirement to manage these attributes as they relate to periphyton and other ecosystem health attributes, and to provide for the health of downstream ecosystems.
- Avoid any further loss or degradation of wetlands and streams, map existing wetlands and encourage their restoration.
- Identify and work towards target outcomes for fish abundance, diversity and passage and address in-stream barriers to fish passage over time.
- Set an aquatic life objective for fish and address in-stream barriers to fish passage over time.
- Monitor and report annually on freshwater (including the data used); publish a synthesis report every five years containing a single ecosystem health score and respond to any deterioration.

The policies of the NPS-FM are supported by NES Freshwater which set out rules related to works within, discharges to and diversion of water, as it relates to streams and wetlands.

National Policy Statement on Urban Development

The NPSUD developed by the Ministry for the Environment and Ministry of Housing and Urban Development came into effect on 20 August 2020. The policy recognises the national significance of:

- Having well-functioning urban environments that enable all people and communities to provide for their social, economic, and cultural wellbeing, and for their health and safety, now and into the future.
- Providing sufficient development capacity to meet the different needs of people and communities.

The NPS-UD requires councils to plan well for growth and ensure a well-functioning urban environment for all people, communities and future generations. The policy states that this includes the following:

- Ensuring urban development occurs in a way that takes into account the principles of the Treaty of Waitangi (te Tiriti o Waitangi).
- Ensuring that plans make room for growth both 'up' and 'out', and that rules are not unnecessarily constraining growth.
- Developing, monitoring and maintaining an evidence base about demand, supply and prices for housing and land to inform planning decisions
- Aligning and coordinating planning across urban areas.

APPENDIX B – STORMWATER MODELLING REPORT

FLOOD MODELLING REPORT

Sunfield – Fast-Track Approvals Application

Ardmore, Auckland

CIVIL ENGINEERING ▼ SURVEYING ▼ LAND DEVELOPMENT

PROJECT INFORMATION

CLIENT	Winton Land Limited
PROJECT	215010

DOCUMENT CONTROL

DATE OF ISSUE	07/02/2025
REVISION	G
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CONTENTS

APPENDIX

Executive Summary

For the proposed development area both the western and eastern catchments had the flood effects modelled for the 50%, 10%, and 1% Annual Exceedance Probability (AEP) storm events.

- All modelling considered the Auckland Council SWCoP version 4 climate change factors.
- A comparison was completed of the pre-development and post development peak flows and flood levels.
- The analysis focused on managing stormwater flows and flood impacts through strategic attenuation design for the development across the different storm event scenarios.
- No negative effects were highlighted in any of the modelling results.
- An Auckland Unitary Plan E36 Assessment has been carried out and may be found in Appendix 14.

1 INTRODUCTION

1.1 PROJECT

This report outlines stormwater modelling that was undertaken by Maven Associates to support Sunfield Developments Limited's proposed Sunfield Fast-track Approvals Act (FAA) application.

The modelling outlines the proposed overall stormwater mitigation strategy for the site in terms of incoming flows and mitigation through conveyance channels. The latest Masterplan has been incorporated as shown in the image below.

Figure 1.1 – Masterplan

1.2 BACKGROUND

The subject site is located in Ardmore, Auckland with a total site area of 244.5 Ha. The site is located within two stormwater catchments as shown in Figure 1.2 below. The northern portion of the site, with an area of 188.0 Ha, is located within the Papakura Stream catchment and the southern portion, with an area of 56.5 Ha, within Pahurehure Inlet Catchment. Both catchments discharge into the Manukau Harbour. For the purposes of this report the portion of site within the Papakura Stream Catchment shall be referred to as the Eastern Catchment and portion within the Pahurehure Inlet Catchment the Western Catchment.

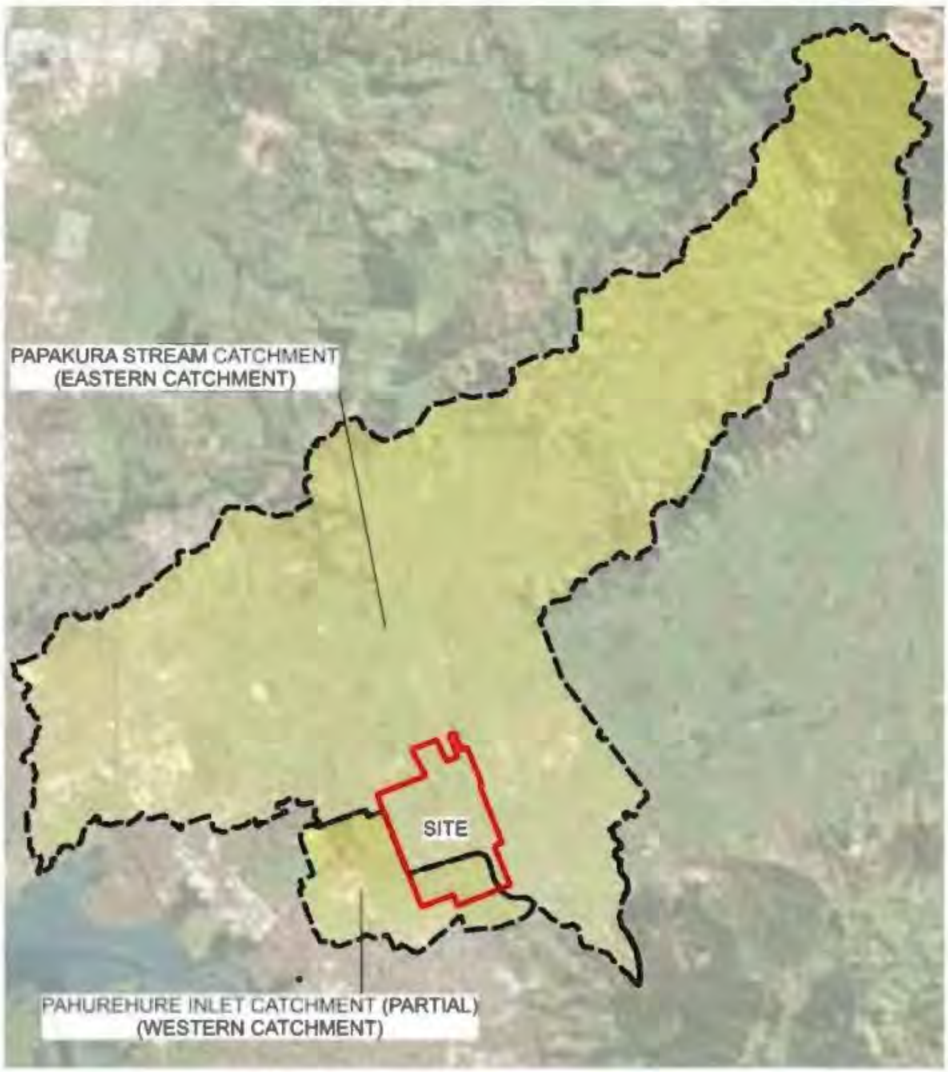


Figure 1.2 Stormwater Catchments

As shown in Figure 1.3 below, Auckland Council Geomaps shows a large portion of the site to be located within a 1% AEP floodplain (3.8°C climate change factor applied). It should be noted that the floodplain within the Western Catchment is located within the catchment area of the Takanini Stormwater

Conveyance Channel (TSWCC). The final stages of the TSWCC is part of a separate resource consent application and once completed shall provide stormwater management for the site's western catchment and significantly reduce the flood plain shown.

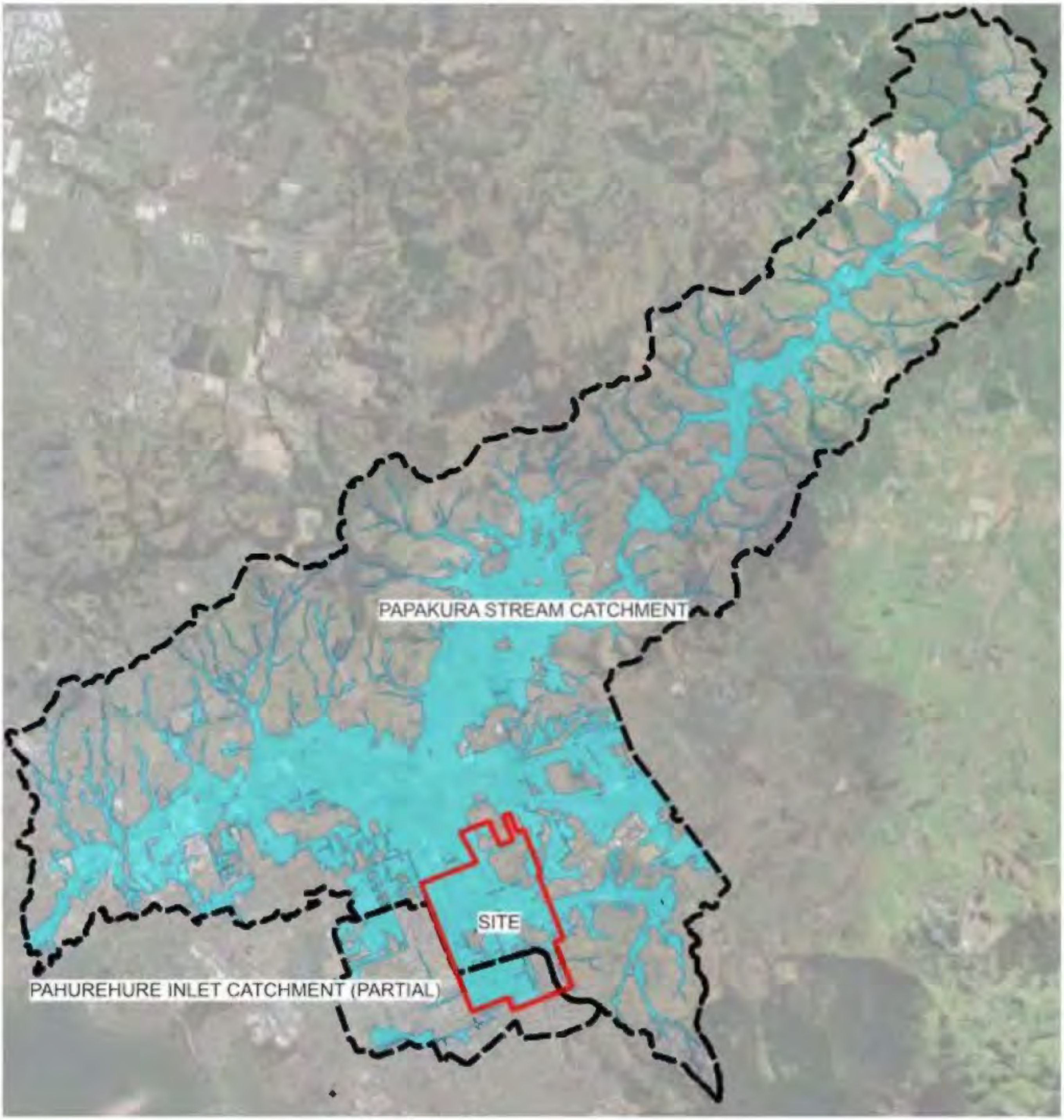


Figure 1.3 Auckland Council Geomaps Floodplain

Takanini Stormwater Conveyance Channel

Central to the strategy of the proposed stormwater management of the Western Catchment is the Awakeri Wetlands, Stages 1, 2 and 3. The Awakeri Wetlands is a part of the TSWCC, the TSWCC was proposed by Auckland Council in 2014 to provide stormwater servicing for the Takanini south-east area.

The Awakeri Wetlands is designed to pass forward flows from Old Wairoa Road, Cosgrave Road, Walters Road and Grove Road, to a box culvert at Grove Road. The Grove Road Box Culvert conveys flows to the McLennan Wetland. During large storm events, flow is attenuated in the Upper McLennan Wetland before being discharged to the Pahurehure inlet via the proposed Artillery Drive Tunnel. At the time of the writing of this report the construction of the Artillery Drive Tunnel, the Grove Road box culvert and Stage 1 of the Awakeri Wetlands have been completed (ie all the SW infrastructure to the west of Cosgrave Road). The remaining Stages 2 and 3 are proposed to be constructed separate to this application.

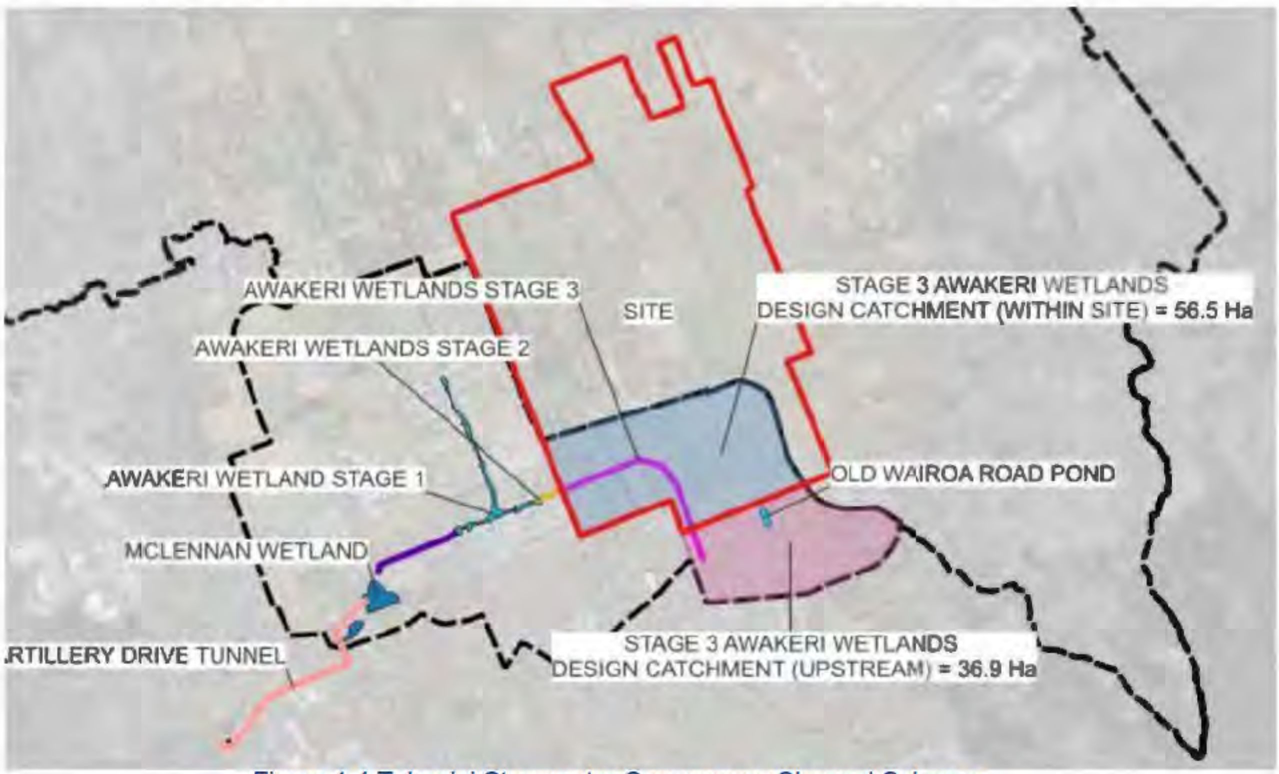


Figure 1.4 Takanini Stormwater Conveyance Channel Scheme

As shown in Figure 1.4 (and outline in the Awakeri Stage 1 design report which may be found in Appendix 11), 56.5 Ha of the site is located within the designed catchment area of the Awakeri Wetlands. An upstream catchment on the southern side of Old Wairoa Road with an area of 36.9 Ha also discharges into Stage 3 of the Awakeri Wetlands which then discharges into Stage 2. The Awakeri Wetlands have been designed to convey the upstream catchments post development flows. Details of the peak flows and conveyance capacity of Awakeri Wetlands Stages 2 and 3 may be found in Appendix 10.

Western Catchment Strategy

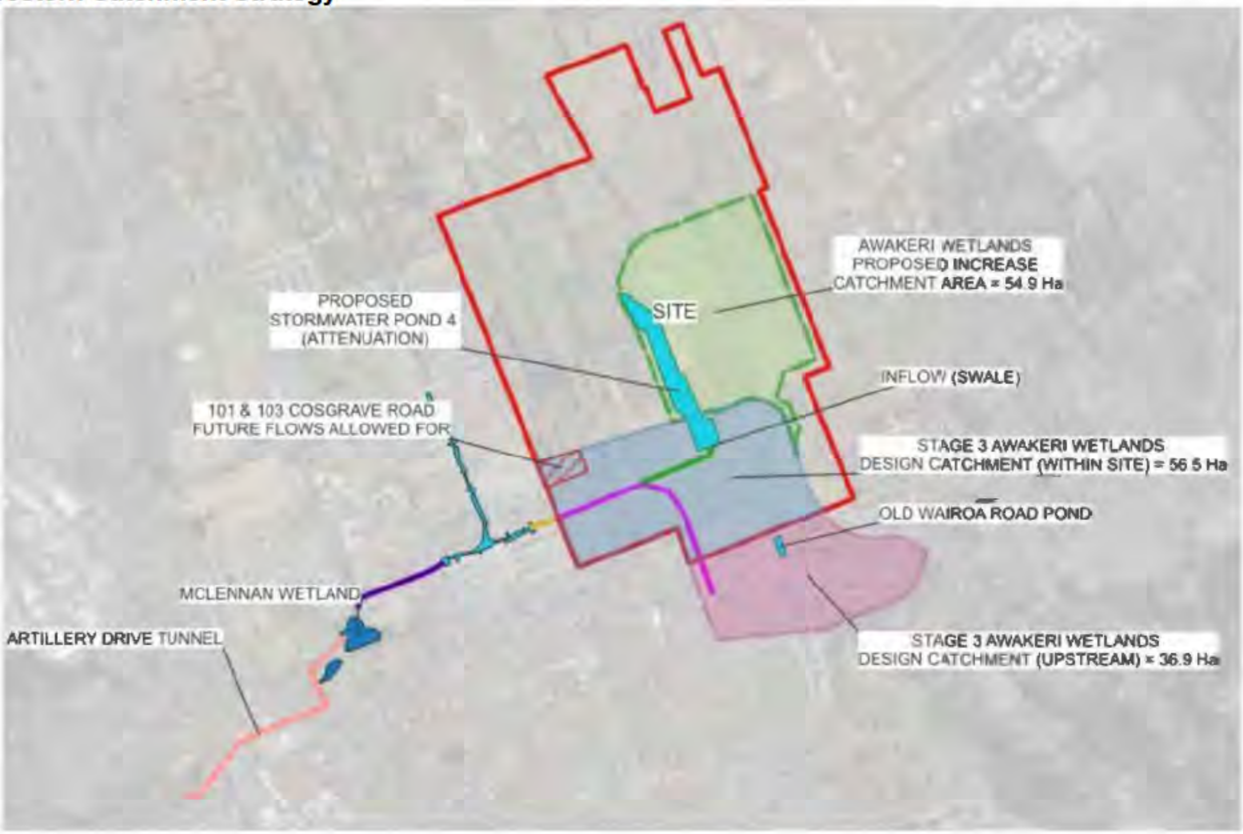


Figure 1.5 Proposed Western Catchment Strategy

The proposed stormwater management strategy for the Western Catchment aims to manage stormwater runoff and mitigate flood hazards within the site without increasing any flooding to upstream and downstream properties. The strategy will also maximise utilisation of the recently developed stormwater infrastructure adjacent to the site, particularly the Awakeri Wetlands and the McLennan Upper Wetland. The development proposes to increase the catchment area discharging to the Stage 3 channel without increasing flows or water levels within the channel upstream or downstream including within the McLennan Upper Wetland (refer to section 1.4 for more detail). An additional catchment of 54.9 Ha is proposed to convey flows to the Awakeri Wetlands as shown in figure 1.5 above (ie 54.9 Ha of the pre-development Eastern Catchment is to be diverted to the Western Catchment and into the Awakeri Wetlands). The catchment diversion is proposed to help managed flows to the Eastern Catchment where there are existing issues with the extents of flooding. Flows from the increased Western Catchment are to be attenuated via a stormwater pond before discharging into the Awakeri Wetlands.

Details of analysis of the proposed solution and assessment of the capacity and performance of the downstream infrastructure including the Stage 1 and McLennan Wetland may be found in Sections 2 and 3 of the report.

Eastern Catchment Strategy

The proposed stormwater management strategy for the Eastern Catchment of the site aims to manage flood hazards within the site without increasing any flooding to downstream properties. No formal existing stormwater infrastructure is located within the eastern portion of the site. There are existing Overland Flow Paths (OLFPs) entering the site across the eastern boundary and exiting across the northern boundary, these OLFP's include flows generated within the site boundary.

The post development strategy is to divert the upstream catchments (Catchment C and a portion of Catchment D1 as shown in figure 1.7 and 1.8) around the perimeter of the site to discharge location at Northern Outflow 1 (adjacent SW Pond 1). SW Pond 1 provides peak flow diversion storage for this upstream flow to maintain the peak flow across Northern Outflow 1. As discussed later in the report (section 4) peak flows across the northern boundary are governed by this upstream flow which arrives at the site after site discharges. The post development catchment discharging to northern outflow 1 (adjacent SW Pond 1) is proposed to be passed forward. Catchments discharging to Northern Outflow 2 and 3 area proposed to be attenuated to pre development. Details of analysis and proposed solution may be found in Section 4 and 5 of the report.

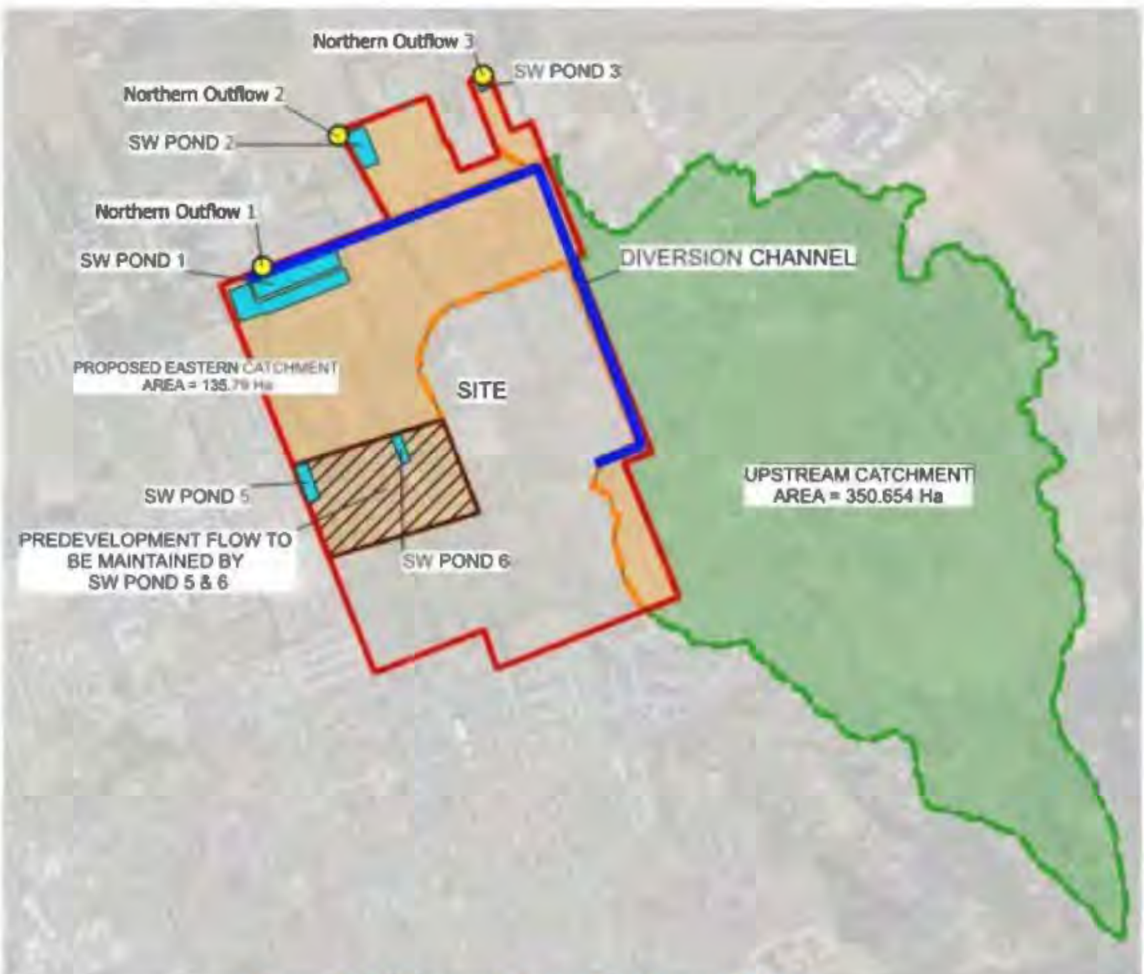


Figure 1.6 Proposed Eastern Catchment Strategy

Catchment Changes

The figures below show the overall catchments predevelopment and post development.

Figure 1.7 Predevelopment Catchments

Figure 1.8 Post development Catchments

1.3 MODELLING APPROACH

The software packages HEC HMS and HEC RAS have been used for hydrological and hydraulic assessment. All analysis has been completed in accordance with TP108 and in accordance with guidelines of the Auckland Council Stormwater Code of Practice.

TP108 has been adopted to be consistent with what stormwater modelling analysis has been undertaken in the area for recent projects, in particular – the design of Awakeri Wetlands Stages 1,2 and 3 and the McLennan Wetland Spillway Options Modelling, 2021.

Level Datum

All levels included in this modelling report are New Zealand Vertical Datum 2016.

Levels in this report can be transformed from New Zealand Vertical Datum 2016 into Auckland Vertical Datum 1946 by applying an offset value of 0.28 m.

For example:

$HAUK_{1946} = HNZVD_{2016} + \text{Offset Value}$

Western Catchment

For the Western Catchment HEC HMS was used to develop inflow hydrographs boundary conditions and HEC RAS was used to model the hydraulics and finalise the solution.

The analysis was done using the following steps:

HEC HMS (hydrological modelling)

1. Delineate the catchments and sub-basins,
2. Use TP108 to calculate parameters,
3. Compute inflow hydrographs for catchments

HEC RAS (hydraulic modelling)

4. Delineate the perimeter for the grid,
5. Create grid and sub-grid areas,
6. Input flow hydrographs and other boundaries
7. Input structures,
8. Run scenarios.

Eastern Catchment

For the Eastern Catchment a HEC RAS model was used to model pre and post development flows and finalise the solution. An existing flow gauge located within the Papakura Stream was used to calibrate the model against the January 2023 Auckland Anniversary flood event. A series of storm durations using NIWA HIRDs rainfall patterns were compared with the TP108 nested storm to confirm the critical storm of the catchment has been assessed (confirming suitability of the TP108 method used).

The analysis was done using the following steps:

HEC HMS (hydrological modelling) for Critical Storm analysis

1. Delineate the catchments and sub-basins
2. Use TP108 to calculate parameters,
3. Compute inflow hydrographs for catchments

HEC RAS (hydraulic modelling)

4. Delineate the perimeter for the grid,
5. Create grid and sub-grid areas,
6. Calibrate model against historical storm (Jan 2023 Auckland Anniversary Flood event)
7. Assess critical storm
8. Input flow hydrographs and other boundaries
9. Input structures,
10. Run scenarios.

TP108 Modelling Limitations

Areal reduction has not been applied for the subbasins. The reduction factor should be based on sub catchment size not the size of the entire catchment (Shamseldin,2008). The largest sub catchment used is Catchment C with an area of 3.7 km2.

1.4 DESIGN FLOW REQUIREMENTS

The proposed development of the site shall increase stormwater runoff generated from the site due to an increase of impervious area. Overall, the stormwater management strategy for both the Eastern and Western Catchments aim to manage this increase in stormwater runoff within the site and eliminate any flood hazard adverse effects which would result from the development of the site. Peak flows, water levels and entry and exit locations of overland flow paths shall be maintained to ensure upstream and downstream properties of the site are not adversely affected by the development.

Takanini Stormwater Conveyance Channel (TSWCC)

The western catchment is proposed to be discharged into the Awakeri Wetlands Stage 3 channel, which discharges to Stage 1 Awakeri Wetlands. Flow from the Awakeri Wetlands is then conveyed to the Upper Mclennan Wetland via a box culvert at Grove Road. The Upper Mclennan Wetland is designed to attenuate flows upto and including 1% AEP flows which are then drained by the Artillery Drive Stormwater Tunnel (ADST) to a coastal outlet at Gills Avenue. A spillway assessment way completed by Tonkin & Taylor in 2021 for Auckland Council (refer to Appendix 12).

For the 50% and 10% AEP flow event an assessment has been undertaken to demonstrate proposed development does not result in increased peak water levels within the Awakeri Wetlands. This assessment demonstrates there are no adverse impacts on the existing primary networks discharging into the Awakeri Wetlands.

For the 1% AEP flow event assessment has been undertaken to demonstrate the existing downstream infrastructure, specifically Awakeri Stage 1 and McLennan Upper Wetland no increase in loading shall be placed on the infrastructure as a result of the proposed development.

1.5 SCENARIOS MODELLED

Table 1.4 and 1.5 shows the scenarios modelled. Further details of the scenarios may be found in section 2.2.

Western Catchment

Scenario	AEP*	Land-use	Catchment	Rainfall
1	50%	Developed	Baseline	Climate change
2	10%	Developed	Baseline	Climate change
3	1%	Developed	Baseline	Climate change
4	50%	Developed	Proposed	Climate change
5	10%	Developed	Proposed	Climate change
6	1%	Developed	Proposed	Climate change

Table 1.4 – Western Catchment Scenarios modelled

**AEP (Annual Exceedance Probability)*

Eastern Catchment

Scenario	AEP*	Land-use	Catchment	Rainfall
1	50%	Developed MPD	Proposed	Climate change
2	10%	Developed MPD	Proposed	Climate change
3	1%	Developed MPD	Proposed	Climate change
4	50%	Existing MPD	Predevelopment	Climate change
5	10%	Existing MPD	Predevelopment	Climate change
6	1%	Existing MPD	Predevelopment	Climate change

Table 1.5 – Eastern Catchment Scenarios modelled

**AEP (Annual Exceedance Probability)*

1.6 SOURCES OF DATA

Attribute	Organisation
Catchment Plans	Maven Associates and Auckland Council Geomaps
Contours	GHD & Healthy Waters (previous design level / Stage 1 channel design) Maven Associates Design (Stage 2&3) LINZ LiDAR data captured between 2016 – 2018
Flow & WL data	Auckland Council's State of the Environment monitoring programme (Historic Storm January 2023 river and rain gauge and
Flood level evidence	None

Table 1.6 – Source of Data

1.7 REFERENCE TECHNICAL DOCUMENTS

- AUCKLAND COUNCIL CODE OF PRACTICE FOR LAND DEVELOPMENT AND SUBDIVISION. CHAPTER4 – STORMWATER, VERSION 4.00
- AUCKLAND COUNCIL TP108
- ACCEPTABLE SOLUTIONS AND VERIFIABLE METHODS, DOCUMENT E1 SURFACE WATER, MINISTRY OF BUSINESS, INNOVATION AND EMPLOYMENT,
- AWAKERI WETLANDS STAGE 2, COSGROVE CULVERT, HEALTHY WATERS, 1 JULY 2019
- TAKANINI STORMWATER CONVEYANCE CHANNEL, HILL YOUNG COOPER, APRIL 2016
- MCLENNAN WETLAND SPILLWAY OPTIONS MODELLING, AUCKLAND COUNCIL, JUNE 2021

2 HYDROLOGICAL MODELLING WITH HEC-HMS WESTERN CATCHMENT

2.1 METHODOLOGY

The analysis was done using the following steps:

1. Delineate the catchments,
2. Use TP108 to calculate parameters,
3. Use HEC-HMS to create a rainfall hyetograph and flow hydrographs,

2.2 RAINFALL DEPTH

TP108 gives the following rainfall depths which have then been adjusted for climate change as shown in Table 2.1. The climate change factor from the version 4 SWCoP have been used.

Rain event	TP108 24 hr rainfall (not including climate change) (mm)	CoP v3 24 hr design rainfall including climate change (mm)	CoP v4 24 hr design rainfall including climate change (mm)
1% AEP	220	257 (+16.8%)	292 (+32.7%)
10% AEP	140	159 (+13.2%)	164 (17.0+%)
50% AEP	70	76 (+9.0%)	81(+15.1%)

Table 2.1 Western Catchment rainfall depths

It is noted the TP108 rainfall depths used are conservative in comparison to that on NIWA Hirds version 4. (the total rainfall depth 24 hour for a 100year storm event for the climate change scenario RCP8.5 scenario on HIRDSv4 is 206mm, 92mm less than the modelled TP108 depth CoP v4 1%AEP depth).

2.3 RAINFALL HYETOGRAPH

The normalised 24-hour temporal rainfall intensity profiles for future climate change condition were used in accordance with Auckland Council code of practice (Version 4) section 4.2.10 Table 2.

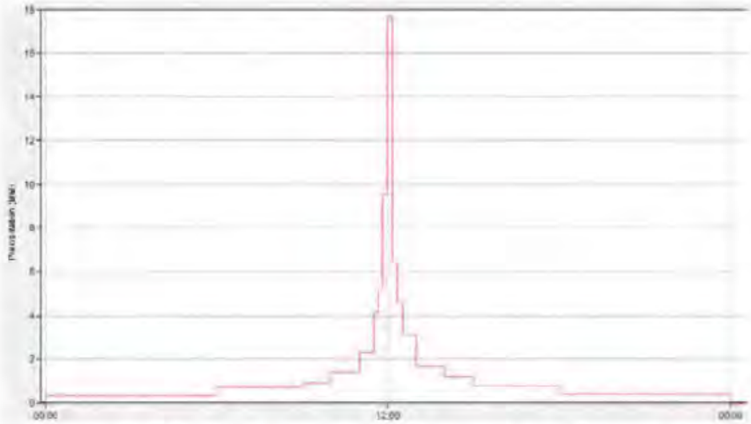


Figure 2.1 shows the 10%AEP future climate change – 2.1° TP108 normalised rainfall intensity (I/I24) from SWCoP version 4

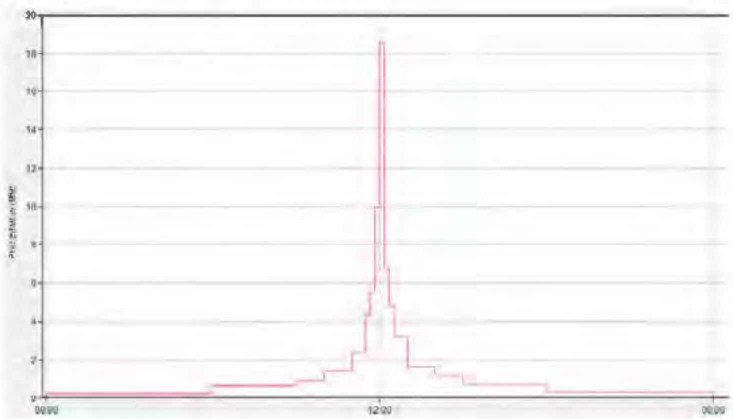


Figure 2.2 shows the 1%AEP future climate change – 3.8° TP108 normalised rainfall intensity (I/I24) from SWCoP version 4

2.4 SCENARIOS AND CATCHMENTS

For the purposes of this assessment the baseline scenario that has been adopted includes the completed Takanini Stormwater Conveyance Channel (TSWCC). The scheme design was developed by GHD in July 2016 as part of a Resource Consent process and is described in the Awakeri Wetlands Design Report and the Takanini Stormwater Conveyance Channel Stormwater Report. Review of the Awakeri design documentation (Appendix 11 and 13) show the catchments 2B4_1, 2B4_2 and 2B4_3 are accounted for the in design of the TSWCC scheme with FUZ (Future Urban Zoning) impervious coverage of 60% maximum impervious area.

It is noted that Auckland Council's assessment of the McLennan Wetland Spillway Options included Stages 2 and 3 of the Awakeri Wetlands catchments in the assessment.

Figures 2.3 and 2.4 below show the catchment areas used in the HEC HMS model to generate inflow hydrographs for the baseline scenario and proposed scenario.

The subcatchment areas and naming convention for the baseline scenario have been extracted from the existing design report. The area shown in yellow hatch indicates the 2d flow area used to model flows and water depths (refer to section 3 for more details). The post development scenario proposed subcatchments including the additional 54.9 Ha discharging from the post development Western Catchment.

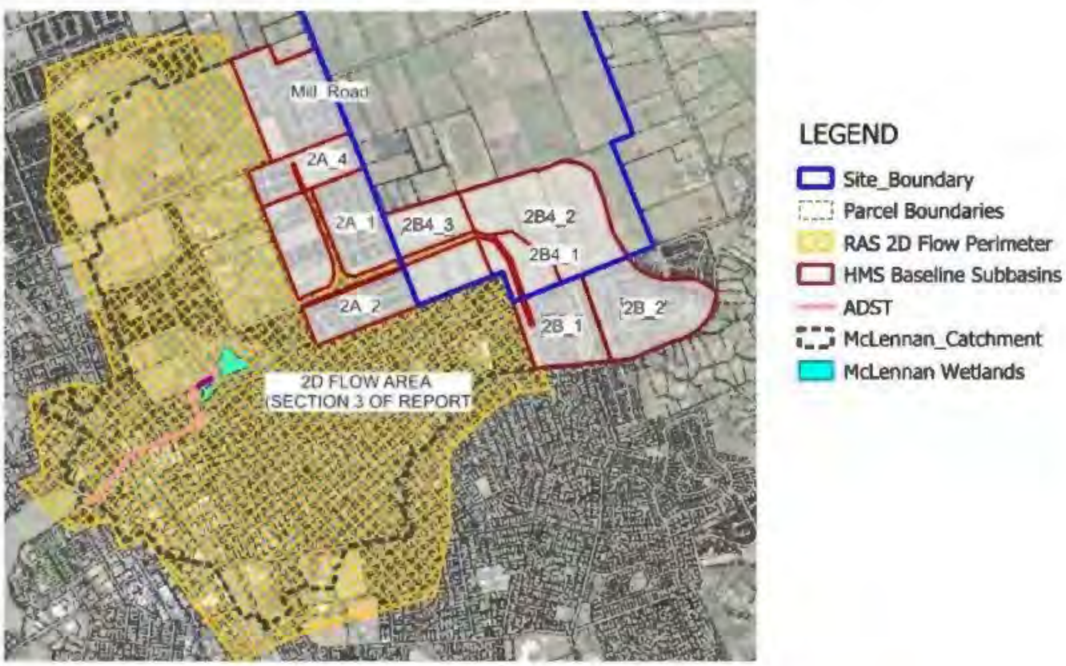


Figure 2.3 HEC HMS model extents for Western Catchment baseline scenario

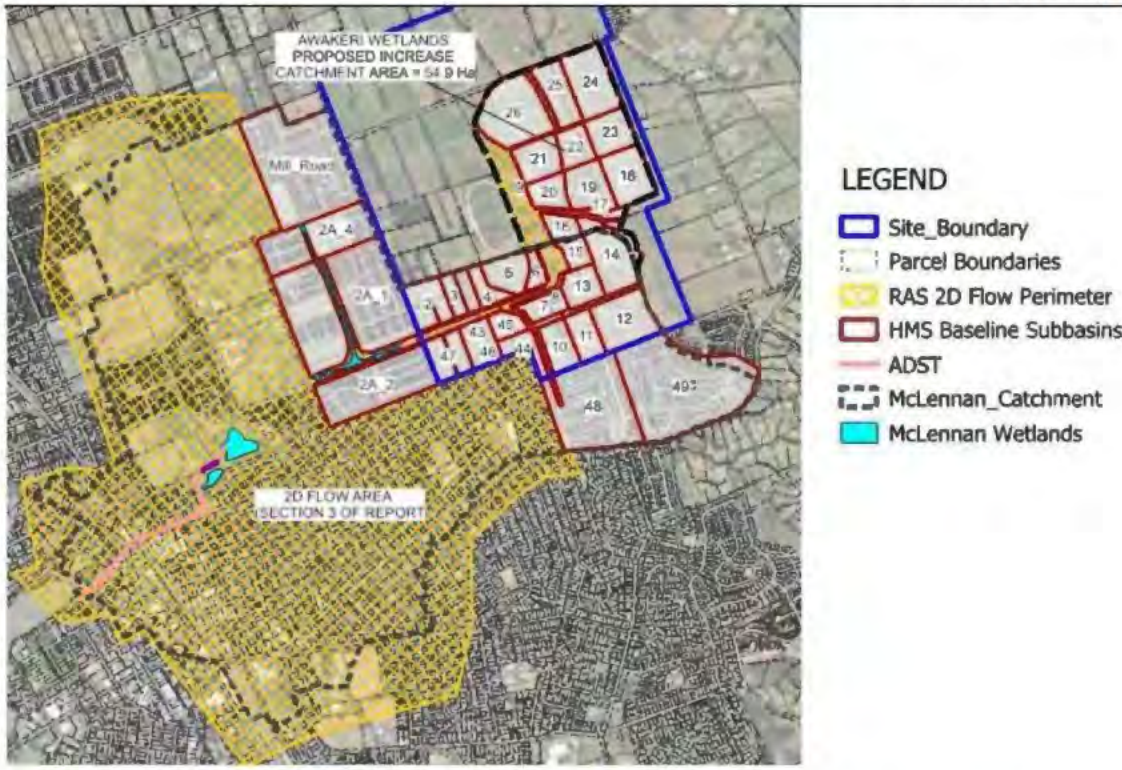


Figure 2.4 HEC HMS model extents for western catchment post development scenario

2.5 SOILS PARAMETERS

A SCS Curve Number (CN) of 74 has been used for peat soils for the predevelopment scenario as per the Papakura ICMP and TP108. Previous geotechnical observations of peat present on site indicate that the top crust of the soil can harden when exposed to oxygen and become impervious . This gives further support to using a curve number of 74. The post-developed scenario also uses a CN of 74 for pervious areas based on likely imported fill characteristics or existing peat soils as per above. For impervious areas in the catchment a CN of 98 has been used.

2.6 LAND-USE

For the purposes of analysis Table 2.2 following shows the impervious percentages used for the proposed zoning and existing zoning within the model extents. Appendix 9 shown plan of the zoning.

Zone	Impervious %
Commercial, Town Center	100
Industrial	90
Residential, retirement village	60
Road	85
Open space	10
SW channel (Awakeri Wetlands)	10

Table 2.2 – Impervious percentage for Zoning

2.7 CHANNELISATION FACTORS AND TIME OF CONCENTRATION

The channelisation factors in Table 2.3 were used for each of the storm events respectively.

For the 50% & 10%AEP storms the channelisation factors of 0.6 have been used for impervious areas. This factor reflects the piped stormwater systems. For pervious areas a factor of 0.8 has been used to reflect the use of open stormwater systems for pervious areas

For the 1%AEP storms the channelisation factors of 0.8 have been used for impervious areas. This factor reflects the swales and green corridors used for overland flow paths. For the previous areas a factor of 1.0 to reflect the sheet overland flow.

	Storm event	
Channelisation Factor	50% &10% AEP Storm	1% AEP Storm
Impervious	0.6	0.8
Pervious	0.8	1.0

Table 2.3 – Channelisation factors

Time of concentration

The values for flow length and time of peak flow have been derived from calculations based on the TP108 methodology. The slopes and catchment lengths consider the developed slopes of the catchment draining to the proposed channel.

2.8 SUBBASIN PARAMETERS

Please refer to Appendix 3 for a summary of the HEC HMS parameters.

2.9 HEC-HMS MODEL

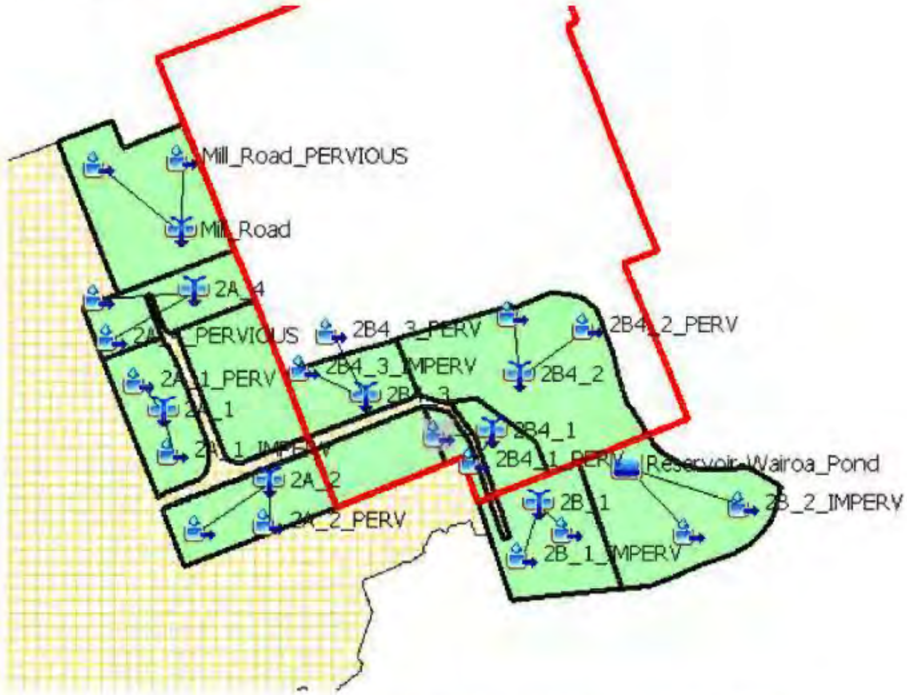


Figure 2.5 –Western Catchment HEC-HMS Model Set-Up – Baseline

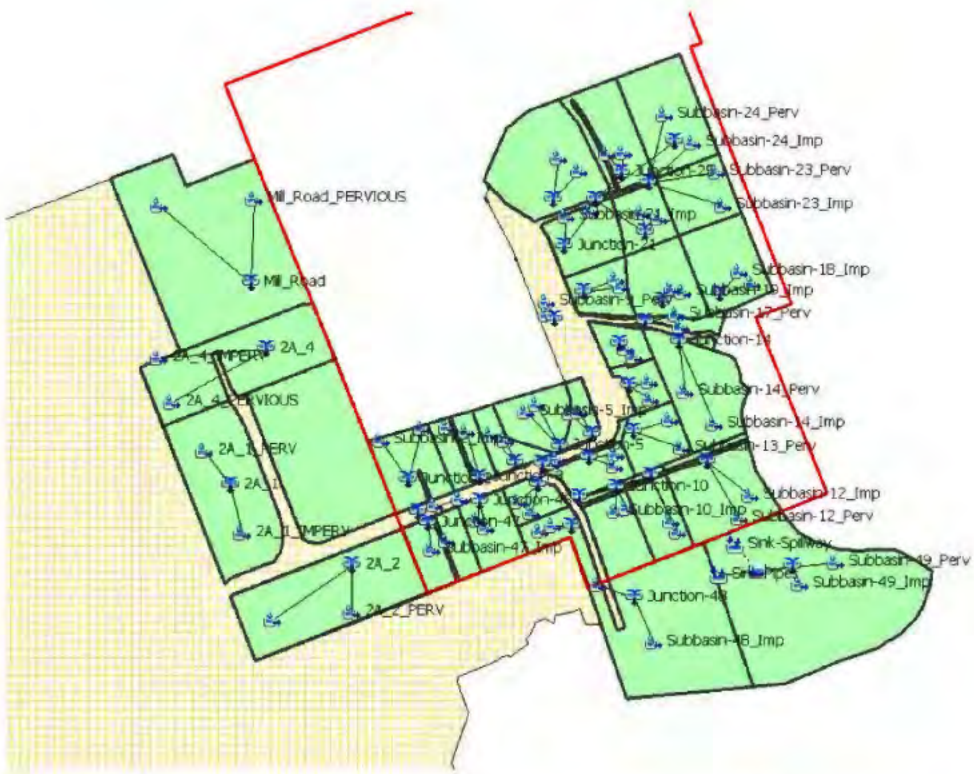


Figure 2.6 –Western Catchment HEC-HMS Model Set-Up -Post Development

2.10 CATCHMENT STORAGE AND ATTENUATION

Please refer to Appendix 3 for a summary HEC HMS pair and cross section parameters data associated with the existing attenuation reservoir.

2.1.1 Existing Upstream Old Wairoa Road Pond Attenuation (Subbasin-49)

Generally, there is limited attenuation in the existing western catchment, as noted in the Awakeri Wetlands Design report (Appendix 11), the proposed wetland channel was designed to convey post-development flows. The exception is for the sub-catchment 49 (sub catchment 2B_2 in baseline scenario). Auckland Council Geomaps shows the pond as a stormwater treatment facility named "Old Wairoa Road Pond". Geomaps shows the pond to have a volume to spill of 9,919 m3 with a 1200mm concrete pipe outlet. The pond has been modelled as a reservoir in the model, with a culvert outlet and spillway (outlet information was obtained from Geomaps and contours). Reservoir initial condition was set to outflow = inflow. Generated hydrograph discharge was used as inflow to the HEC RAS model (outlined in section 3).

2.1.2 Proposed Stormwater Pond 4 (Subbasin-9 & 14 to 26)

Runoff from 63.4 Ha of the site is proposed to drain into stormwater pond 4. Flows shall be attenuated prior to discharge into the Awakeri Wetlands. The basin shall have an outlet and swale to connect to the Awakeri wetlands channel. This pond has not been included in the HEC HMS model. To allow for any hydraulic influence of tailwater in the channel the stormwater pond shall be modelled in section 3 (using HEC-RAS software).

Figure 2.7 –Western Catchment proposed Stormwater attenuation Pond

2.11 INFLOW HYDROGRAPHS

Inflows generated from the HEC HMS model were then transferred to HEC RAS as inflow boundary conditions, the HEC RAS modelling shall incorporate stormwater hydraulics to the modelling. Please refer to section 3 for hydraulic modelling.

3 WESTERN CATCHMENT HYDRAULIC MODELLING WITH HEC-RAS

3.1 METHODOLOGY

The analysis was done using the following steps:

1. Delineate the perimeter for the grid,
2. Create a grid and sub-grid areas,
3. Input flow hydrographs and other boundaries
4. Input structures,
5. Run scenarios.

3.2 HEC-RAS MODEL LAYOUT

A 2D model was developed using design terrain of Awakeri Wetlands Stage 1 and proposed design contours of Awakeri Stages 2 and 3 (no deviations from the original Stages 2 and 3 Design). A Manning's n of 0.03 was used for the low flow areas and 0.045 for the rest of the channel. (Manning values have been used in consistency with previous modelling by Healthy Waters).

Hydraulic structures were added as outlined in section 3.4. A triangle mesh with cell size generally between 2m and 5m was used to model the 2D flow area. Figure 3.1 and 3.2 shows the grids and its boundary conditions.

HEC-RAS software was used to generate water levels within the main channels, the proposed stormwater pond 4 and the McLennan Wetland.

<u>McLennan Wetland Spillway</u>

The McLennan Wetland spillway has been topographically surveyed. The existing spillway level has a general elevation of 14.86 mRL. The surveyed terrain of the spillway has been incorporated into the model terrain for all scenarios.

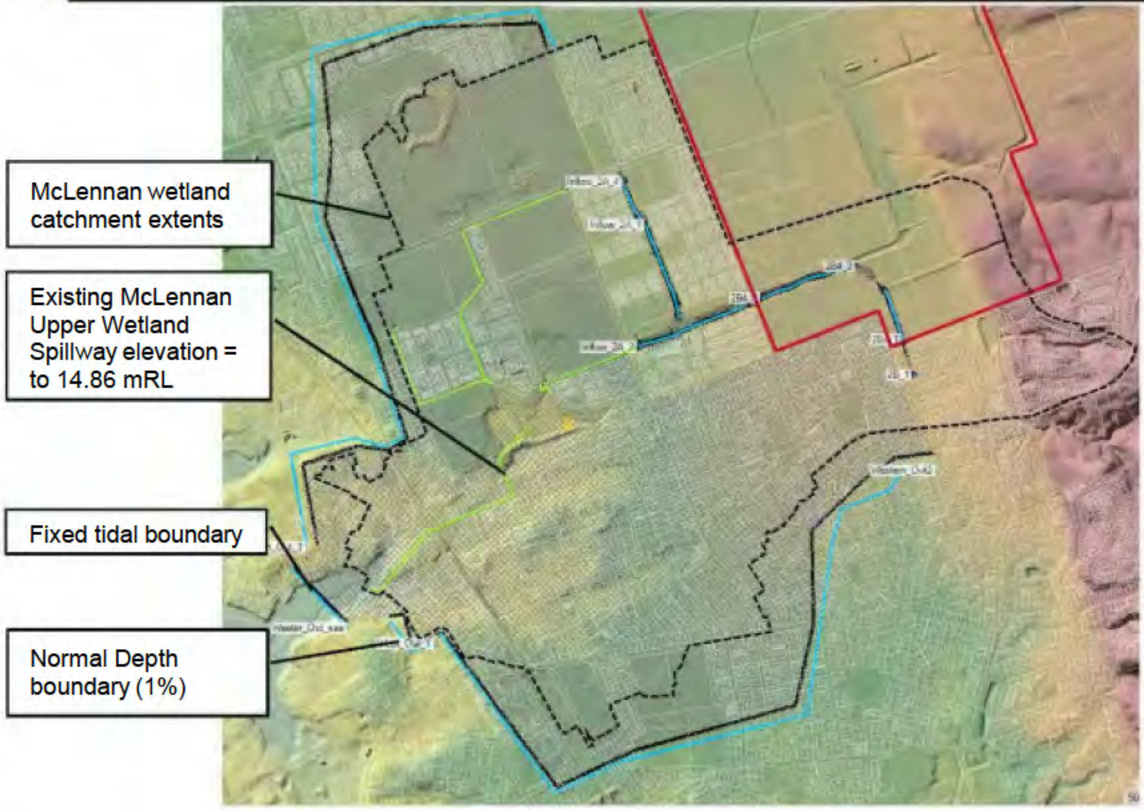


Figure 3.1 – HEC-RAS Western model set-up – Baseline

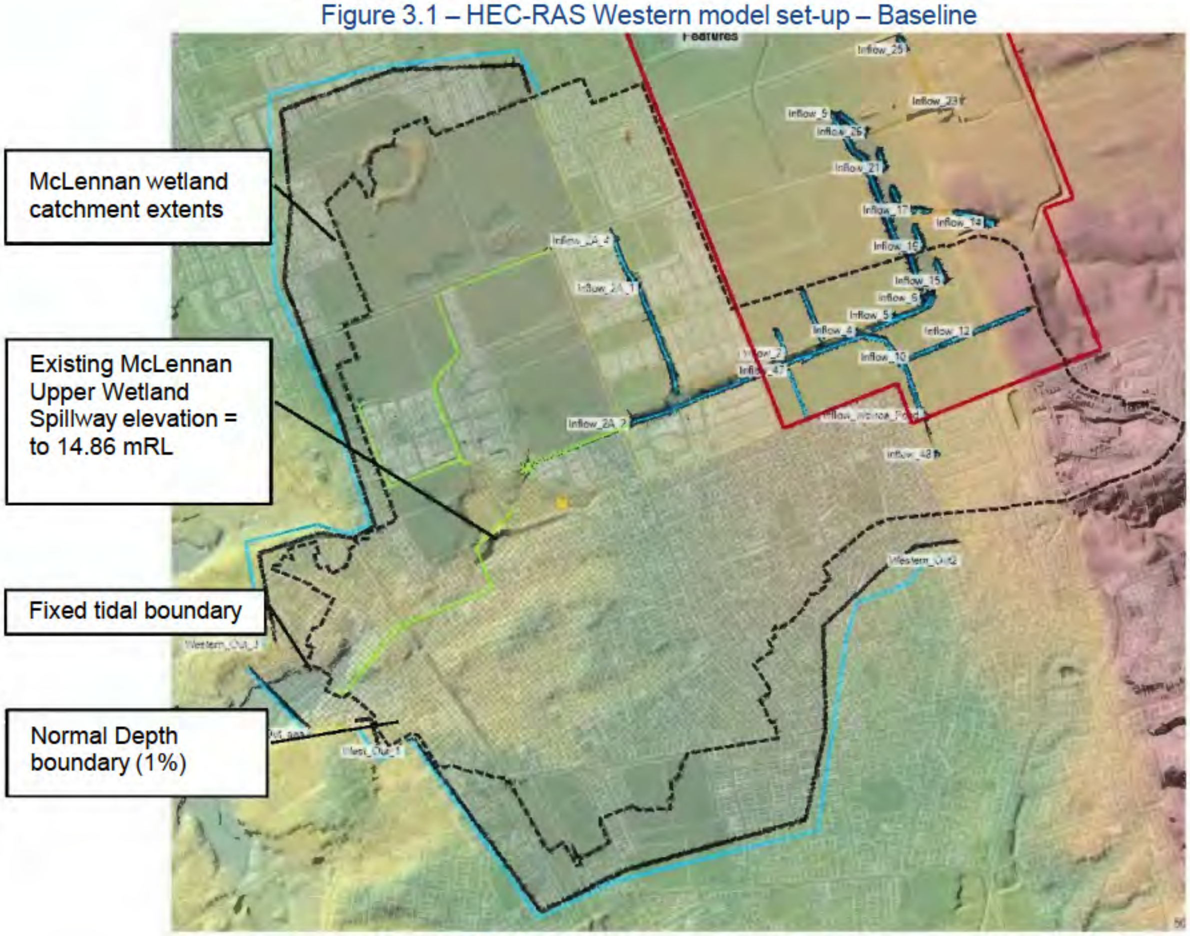


Figure 3.2 – HEC-RAS Western model set-up - Post development

3.3 HYDRAULIC STRUCTURES AND CULVERTS

Within the Awakeri Wetlands hydraulic structures have been incorporated in general accordance with the Healthy Waters design of the Awakeri Wetlands (shown in Appendix 10). Design deviations include the addition of a swale connecting stormwater pond 4 to the Awakeri Wetlands and update of the culvert at chainage 1140 to match the proposed road layout. Downstream of the site a major pipes have been incorporated in the modelling including the Artillery Drive Tunnel within the Upper McLennan wetland. Two types of structures are present, weirs and culverts. As per outlined in the Awakeri Wetlands design reports, the weirs function is to keep a permanent water level in the channel.
A total of ten culverts have been included in the model as well as weir structures. A summary of the structures is included in Table 3.1 following.

Culverts structures

Name	Chainage	Size
Stage 2 Awakeri Wetlands	550	3 x Box culvert 1.5m x 2.5m
Proposed Chainage 1140 Culvert	1140	2 x Box culvert 1.5m x 2.0m
Existing Wairoa Road Culvert	1400	2 x 1500ø
Stage 4 Attenuation Pond Culvert	-	1 x Box culvert 1.0m x 1.0m
Grove Road Culvert	0	2.5 x 3.5 Box Culvert
Artillery Drive Stormwater Tunnel	-	QH Curve from McLennan Spillway Modelling (Appendix 12)
Battalion Road Culvert (SAP ID 3000092665)	-	1.2m Circular Pipe
Battalion Road Culvert (SAP ID 3000049172)	-	1.05m Circular Pipe
Walsh Road Pipe (SAP ID 3000034935)	-	0.75m Circular Pipe
Walters Road Pipe (SAP ID 2001081576)	-	0.6m Circular Pipe

Table 3.1 – Western Catchment Culvert summary

Awakeri Wetlands Weir structures

Chainage	Height mRL (NZVD2016)
0	20.41
80A	20.62
100B	21.25
180B	21.07
260B	21.43
330A	21.52
340B	21.60
440A	21.97
480B	21.70
580A	22.31
610A	22.65
690A	22.88
800A	23.11
900A	23.34
950A	23.57
1160	23.80
1240	24.03
1300	24.26
1460A	24.49

Table 3.2 – Western Catchment weir summary

Figure 3.3 Awakeri Stage 1 Existing weirs and Stages 2 and 3 design weirs

3.4 BOUNDARY CONDITIONS

The below boundary conditions were used in the model:

- A 2d grid – as per figure 3.1 and 3.2

The grid extents include the proposed stormwater pond 4 located within the site, the Awakeri Wetlands, the McLennan Wetland and its contributing area and the outlet area of the Artillery Drive Stormwater Tunnel (ADST).

- Rain on grid – Precipitation has been applied across the 2d grid
- Inflow hydrographs imported from HEC HMS (outlined in section 2)
- Permanent water levels – Initial water elevations were set at the top of weir levels
- The downstream outflow boundary condition has been setup at the sea boundary as a constant stage elevation of mRL 2.34 mRL AUK1946 (2.06 mRL NZVD2016). This was selected for consistency with the level Auckland Council requested T&T to use in the McLennan wetland spillway options modelling, June 2021, appendix 12.
- The ADST and inlet structures have been modelled using a discharge-stage (QH) relationship extracted from Auckland Council's 2019 McLennan Spillway report (refer to appendix 12). The QH includes allowances for the tailwater condition and hydraulic losses at the inlets, outlet, pipe bends and roughness. QH curve may be found in figure below.

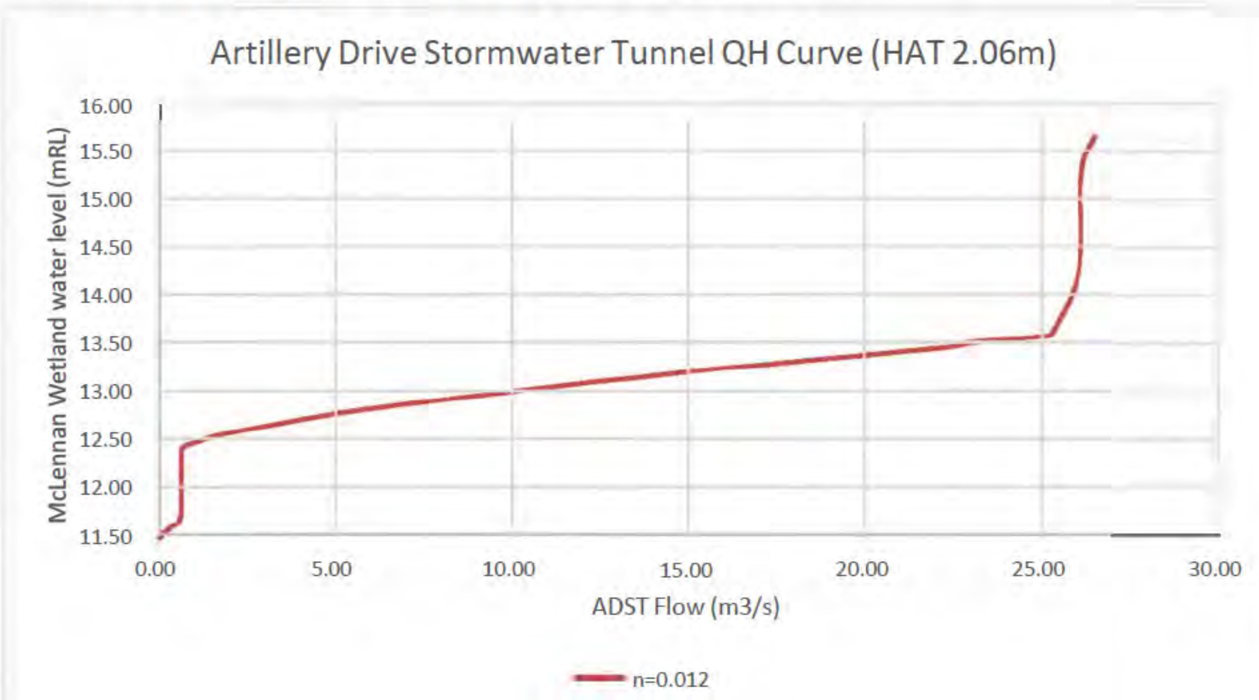


Figure 3.4 *QH curves for ADST and inlet structures.*

3.5 CLIMATE CHANGE FACTOR INCREASE – BASELINE SCENARIO

At the time of the writing of this report Auckland Council is transitioning from Auckland Council Stormwater Code of Practice (SWCoP) version 3 to version 4. One key change included in the transition is the increase in the climate change factor, where new climate change factors are incorporated. This change in design assumption increases the design rainfall depth as well as temporal rain profiles. It should be noted that the Awakeri Wetlands Design report flows assume a the SWCoP version 3 climate change factors. However, the assessment of this report assumes the updated climate change factors . It is noted that this will increase the inflows into the Awakeri Wetlands.

To account to the updated climate change factors a baseline scenario model was developed for three storm events (50%, 10% and 1% AEP) showing the flows and water levels in the Awakeri Wetlands and downstream with the updated climate change factor outlined in AC SWCoP version 4.

Topographical survey was undertaken to confirm the existing elevation of the Upper McLennan wetland. This was surveyed to be generally 14.86 mRL (NZVD2016)

3.6 RESULTS – FLOOD MAPPING

Figure 3.5 below shows the modelled flooding depth of the proposed development for a 1% AEP storm. Flood mapping for each of the modelled scenarios may be found in appendix 6.

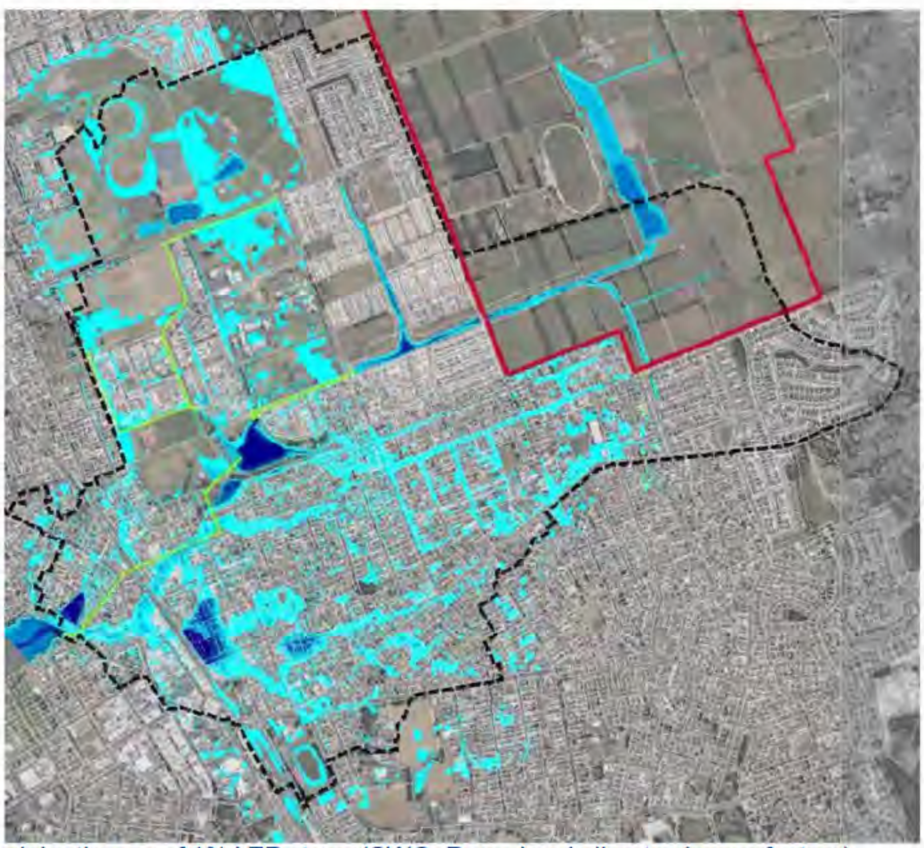


Figure 3.5 Flood depth map of 1%AEP storm (SWCoP version 4 climate change factors)

3.7 RESULTS - AWAKERI WETLANDS PEAK FLOW DEPTHS

Peak post development 1%, 10% and 50% AEP water levels within the Awakeri Wetlands for the baseline scenario are shown in figure 3.4 and for the post development scenario in figure 3.5 Review of the modelling results from the western catchment are shown below. Flood level difference maps may be found in Appendix 6. The flood level difference maps show a minor reduction in water level downstream of the site within the Awakeri Wetlands and upstream to remain unchanged.

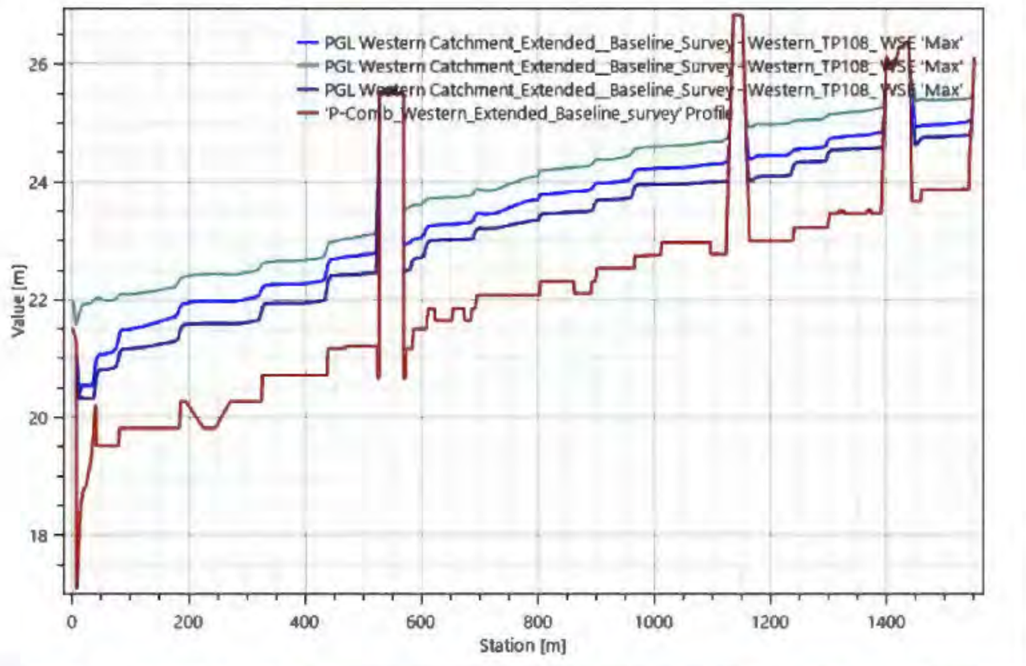


Figure 3.4 Baseline peak 1%,10% and 50% AEP water levels within Awakeri wetlands

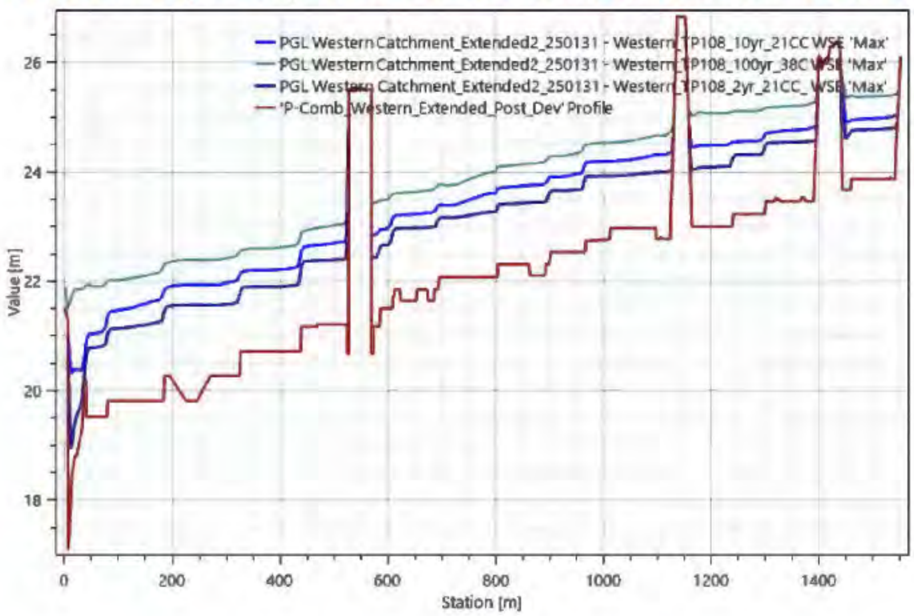


Figure 3.5 Post development peak 1%,10% and 50% AEP water levels within Awakeri wetlands

3.8 RESULTS - UPPER MCLENNAN WETLAND

The ADST was built in 2017 to facilitate growth in the catchment upstream of McLennan Wetland without increased flood risk to downstream properties. One of the design objectives of the ADST was to prevent the spillway from the upper McLennan wetland storage area being activated in a 1% Annual Exceedance Probability (AEP) rainfall event, including allowance for climate change (CC) and Maximum Probable Development (MPD). Topographical survey of the Upper McLennan spillway found the elevation to be 14.86 mRL (NZVD2016). It is noted that at the time of the ADST design and construction a smaller climate change factor was applied to the design rainfall.
Results are summarised in Figure 3.6 and Table 3.2 below.
Modelling of the baseline 1%AEP baseline scenario shows peak water levels of 15.04mRl. The peak flow exceeds and overtops the existing spillway. The peak flow across the spillway was shown to be 11.1 m3/s.

Modelling of the 1%AEP post development scenario shows peak water levels of 15.03mRl. The peak flow exceeds and overtops the existing spillway. The peak flow across the spillway was shown to be 10.3 m3/s.

In summary, modelling shows the McLennan Wetland is overtopped in both the baseline and post development scenario. In the post development scenario a minor decrease in peak flow is shown across the spillway, reducing from 11.1 m3/s to 10.3 m3/s (7% reduction).

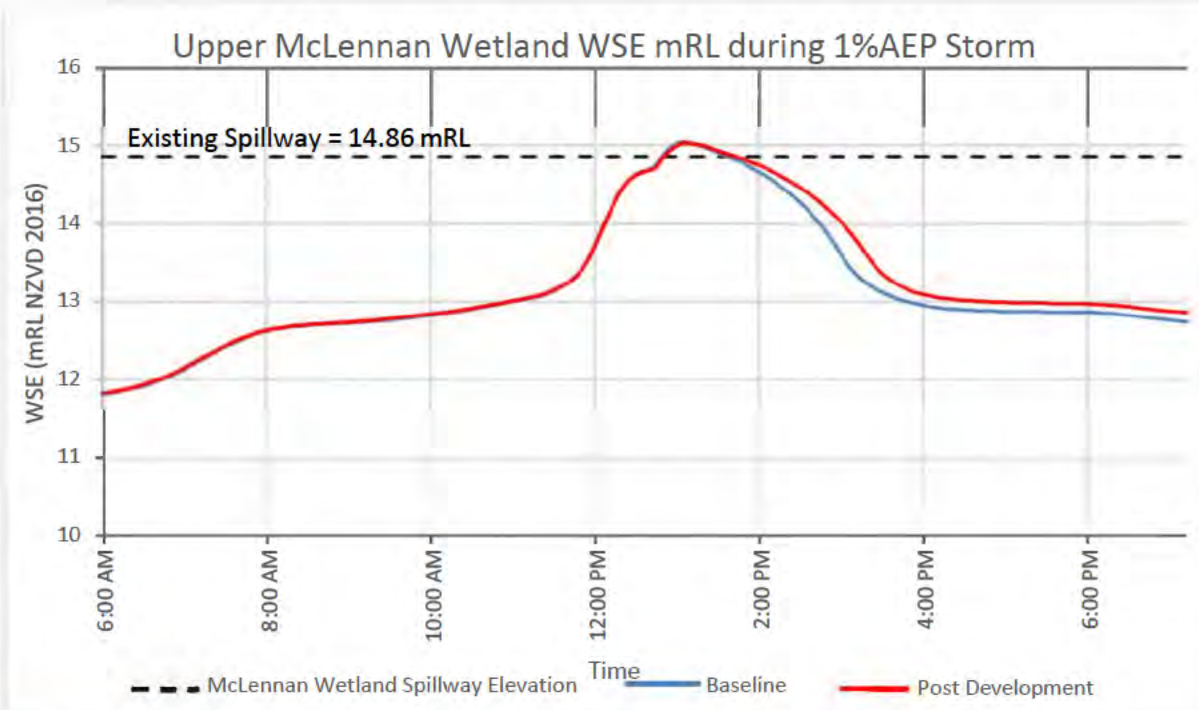


Figure 3.6 Comparison of water surface elevation in McLennan Wetland 1%AEP

Event	MPD 1%AEP (3.8cc Factor)	
Scenario	Baseline	Post Development
Peak Water Peak water level in upper McLennan Wetland (m RL)	15.04	15.03
Freeboard to current spillway level (14.86 mRL)	-0.18	-0.17
Peak flow Artillery Drive Stormwater Tunnel (m^3/s)	26.22	26.22
Peak flow over spillway (m^3/s)	11.1	10.3
Duration for water level above spillway level ((hours:minutes)	1:10	1:15

Table 3.2 – McLennan Wetlands result summary

3.9 RESULTS - WESTERN CATCHMENT PEAK FLOW

A comparison of peak flow rates between the baseline and post development scenarios shows that flow rates either remain unchanged or have a small decrease within the modelled Western Catchment for a 1%AEP storm. A decrease in peak flow rate of 3% is observed at Awakeri Stages 2 and a decrease in peak flow rate of 10% is observed at Grove Road Culvert. This is attributed to the proposed stormwater pond within the site, which is providing attenuation and decreasing peak flows.

Storm Event	Baseline Modelled Scenario Peak SWCoPv4 Climate Change factors flow (m3/s)	Post development modelled Peak SWCoPv4 Climate Change factors flow (m3/s)
50% AEP	6.9	5.8
10% AEP	15.9	13.9
1% AEP	28.0	25.3

Table 3.3 – Awakeri Wetlands Stage 2 peak flow difference from post development site discharge

Storm Event	Baseline Modelled Scenario Peak SWCoPv4 Climate Change factors flow (m3/s)	Post development modelled Peak SWCoPv4 Climate Change factors flow (m3/s)
50% AEP	11.6	10.6
10% AEP	26.3	24.6
1% AEP	45.6	44.1

Table 3.5 – Grove Road Culvert peak flow difference from post development site discharge

3.10 WESTERN CATCHMENT ATTENUATION VOLUMES

Attenuation for the post development scenario is provided by a stormwater pond (SW Pond 4). The configuration of the outlets and storage volumes are summarised in the table below.

Element	Stormwater Pond 4	Outlet
50% AEP Pond Peak storage Vol (m3)	23,280	Box Culvert 1.0m x 1.0m
10% AEP Pond Peak storage Vol (m3)	51,170	
1% AEP Pond Peak storage Vol (m3)	94,000	

Table 3.6 – Western Catchment attenuation volumes

3.11 OUTFLOW VOLUME CHECK

The HEC RAS computation volume error for each scenario is summarised in the table below;

Scenario	Volumes error m3	Error as percentage
50% AEP Baseline	1,059	0.18%
10% AEP Baseline	1,387	0.16%
1% AEP Baseline	123	0.01%
50% AEP Post development	350	0.51%
10% AEP Post development	33	0.01%
1% AEP Post development	2	0.00%

Table 3.6 – Outflow volume check for western catchment HEC RAS model

The HEC-RAS model used in this study yielded volume errors ranging between 0.03% and 0.51%. These errors are well within acceptable limits for hydraulic modelling. According to established guidelines and best practices in hydraulic modelling, volume errors below 1-2% are generally considered negligible and indicative of a high degree of model accuracy.

3.12 CONCLUSION – WESTERN CATCHMENT

A flood model has been built to assess flood effects of the proposed development of the site during 50%, 10% and 1% AEP storm events assuming the Auckland Council SWCoP version 4 climate change factors.

The post development scenario was compared to the existing Awakeri Wetlands catchment scheme (baseline scenario).

The proposed development includes an additional 54.9 ha catchment area (increase to the Western Catchment) into the Awakeri Wetlands to help manage flows and downstream flood issues in the Eastern Catchment. Post development flows from the additional catchment are attenuated in a proposed stormwater pond prior to discharge into the Awakeri Wetlands.

Results from the modelling analysis conclude the proposed development will not adversely impact the upstream and downstream properties. Modelled peak flow levels within the TSWCC either remain unchanged or are reduced as a result of the development.
Flood storage in the post development scenario is shown to be contained within the Upper McLennan wetland. Peak flows spilling out of the Upper McLennan Spillway during a 1%AEP storm are shown to be slightly reduced in the post development scenario.

An Auckland Unitary Plan E36 flood risk assessment may be found in Appendix 14.

4 HYDROLOGICAL MODELLING WITH HEC-HMS EASTERN CATCHMENT

4.1 METHODOLOGY

The analysis was done using the following steps:

1. Delineate the catchments where inflow hydrographs required
2. Use TP108 to calculate parameters
3. Use HEC-HMS to create a rainfall hyetograph and flow hydrographs
4. Size attenuation devices for stormwater pond 2 and 3

4.2 RAINFALL DEPTH

TP108 gives the following rainfall depths which have then been adjusted for climate change as shown in Table 2.1. The climate change factor from the Auckland Council version 4 SWCoP have been used.

Rain event	TP108 24 hr rainfall (not including climate change) (mm)	SWCoP v4 24 hr design rainfall including climate change (mm)
1% AEP	225	298
10% AEP	145	170
50% AEP	75	86

Table 2.1 Eastern Catchment rainfall depths

It is noted the TP108 rainfall depths used are conservative in comparison to that on NIWA Hirds version 4. (the total rainfall depth 24 hour for a 100year storm event for the climate change scenario RCP8.5 scenario on HIRDSv4 is 206mm, 92mm less than the modelled TP108 depth CoP v4 1%AEP depth).

4.3 EASTERN CATCHMENTS

Northern Outflow 1 - (Routed through Stormwater Pond 1)

The catchment area within the site discharging to the Northern outflow 1 via stormwater pond 1 is 109.1 Ha, of this area 29.5 Ha of the site is allocated to stormwater management as either swales or the Stormwater Pond 1. Flow within the stormwater management areas within the site aswell as the upstream and downstream catchment shall be modelled in a 2d flow are in HEC RAS (outlined in section 5).

Developed lot catchments within the site discharging to Stormwater Pond 1 have a total area of 64.2ha. Post development subcatchments for this area are delineated by where they discharge into the site's swale network (ie 2d flow area). Flows upstream and downstream of the site are generated from rain on grid (and are detailed in section 5). Figures below shows the HEC HMS subbasin delineations.

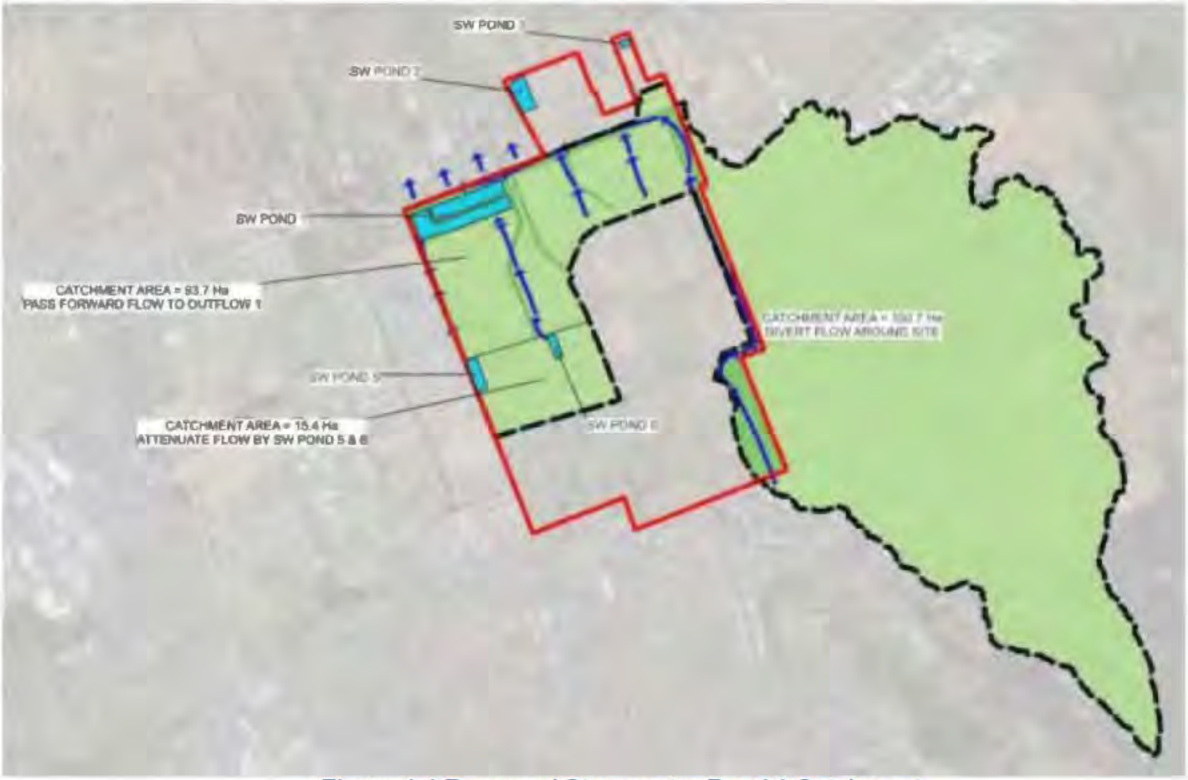


Figure 4.1 Proposed Stormwater Pond 1 Catchment

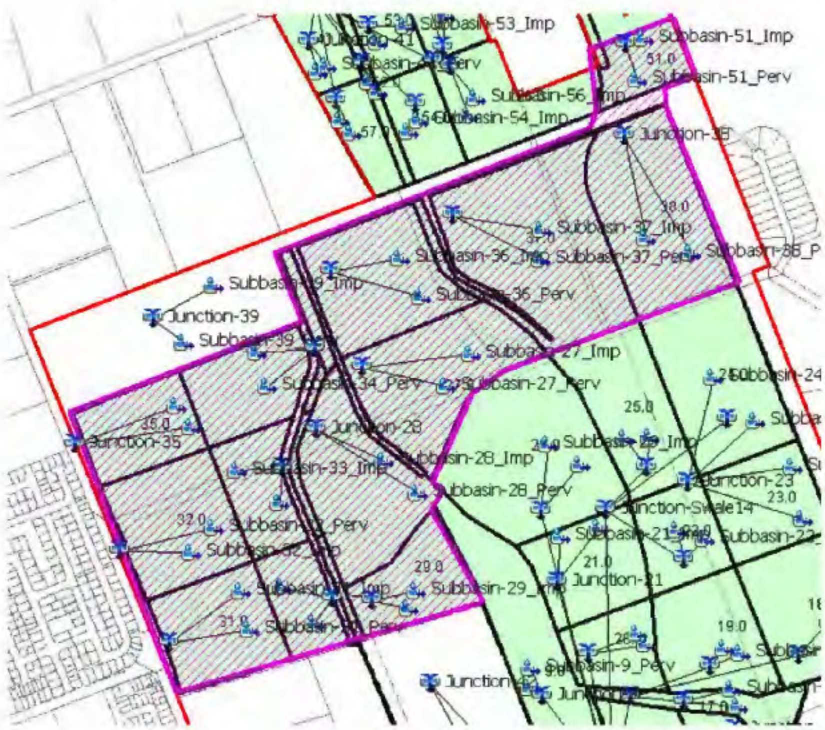
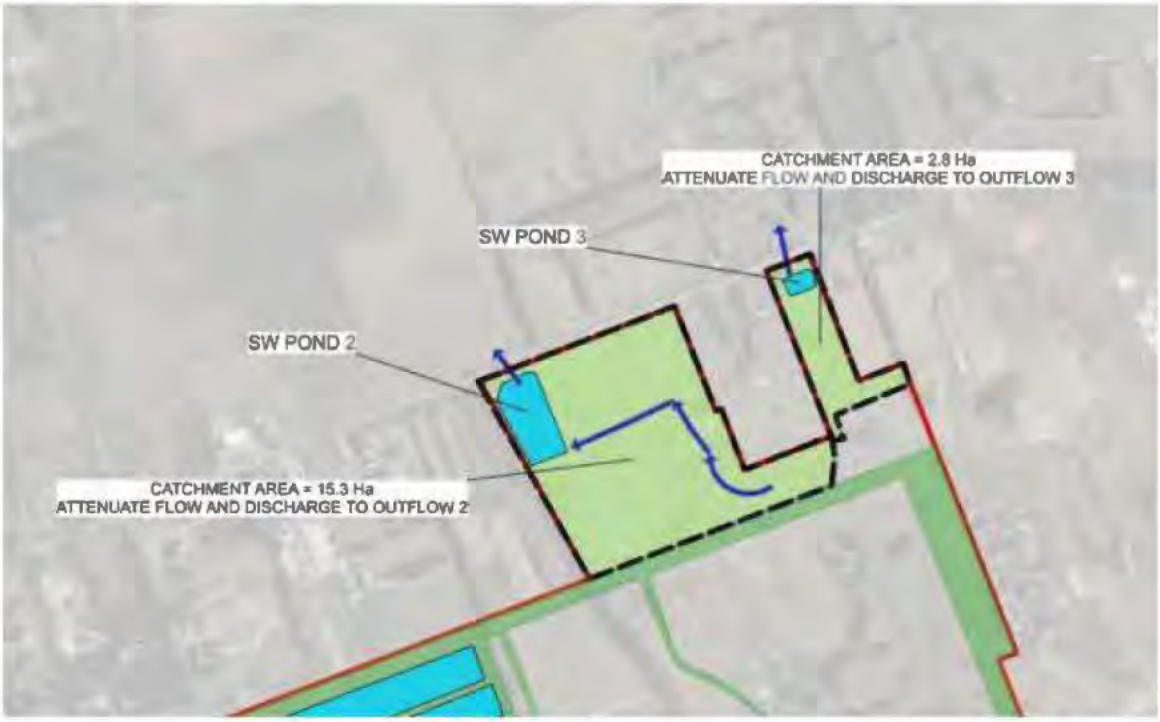


Figure 4.2 –Eastern Catchment Outflow 1 HEC-HMS Model Set-Up for inflow hydrograph

Northern Outflows 2 & 3 (with area routed through Stormwater Ponds 2 & 3)



For the site area located within Catchments D1 and D2 it is proposed to attenuate post development flows to peak pre-development flows (as shown in Appendix 5) HEC HMS has been used to size the attenuation volume required for the 2%AEP, 10% AEP and 1%AEP storm. The model setup is shown in figure 4.3 below.

Figure 4.3 Proposed Stormwater Pond 2 & 3 Catchment

Figure 4.4 –Eastern Catchment 1%AEP HEC-HMS Model Set-Up for Stormwater Pond 2&3

Eastern model

The climate change factor from the Auckland Council version 4 SWCoP has been applied for the Eastern catchment rainfall.

Rain event	TP108 24 hr rainfall (not including climate change) (mm)	CoP v4 24 hr design rainfall including climate change (mm)
1% AEP	225	298 (32.7% increase according to 3.8°c)
10% AEP	145	170 (17.0% increase according to 2.1°c)
50% AEP	75	86 (15.1% increase according to 2.1°c)

Table4.1 Eastern Catchment rainfall depths

4.4 RAINFALL HYETOGRAPH

The normalised 24-hour temporal rainfall intensity profiles for future climate change condition were used in accordance with Auckland Council code of practice (Version 3 and 4) section 4.2.10 Table 2.

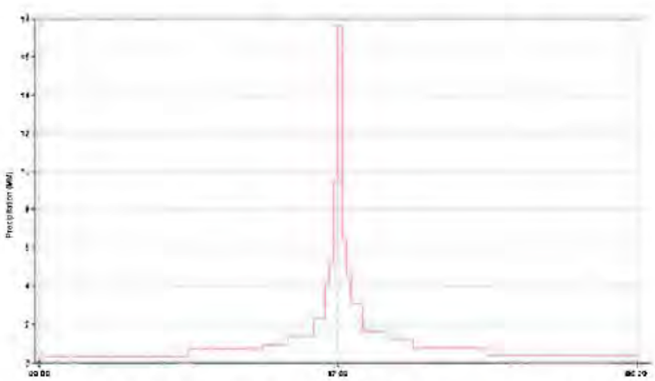


Figure 4.4 shows the 10%AEP future climate change – 2.1° TP108 normalised rainfall intensity (I/I24) from SWCoP version 4

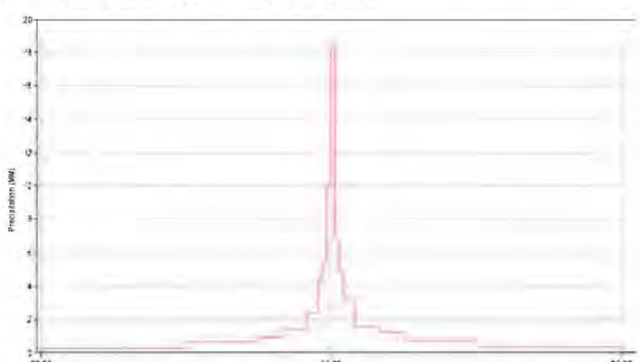


Figure 4.5 shows the 1%AEP future climate change – 3.8° TP108 normalised rainfall intensity (I/I24) from SWCoP version 4

4.5 SOILS PARAMETERS

A SCS Curve Number (CN) of 74 has been used for peat soils for the predevelopment scenario as per the Papakura ICMP and as per TP108. Previous geotechnical observations of peat present on site indicate that the top crust of the soil can harden when exposed to oxygen and sheds water. This gives further support to using a curve number of 74. The post-developed scenario also uses a CN of 74 for pervious areas based on likely imported fill characteristics or existing peat soils as per above. For impervious areas in the catchment a CN of 98 has been used.

4.6 LAND-USE

For the purposes of this analysis the table below shows the impervious percentages of land used for the proposed zoning and existing zoning within the model extents. Appendix 9 shown plan of the zoning.

Zone	Impervious %
Commercial, Town Center	100
Industrial	90
Residential, retirement village	60
Road	85
Open space	10
SW channel (Awakeri Wetlands)	10

Table 4.2 – Impervious percentage for Zoning

4.7 CHANNELISATION FACTORS AND TIME OF CONCENTRATION

The channelisation factors in Table 6 were used for each of the storm events respectively.

	Storm event	
Channelisation Factor	**10% AEP Storm**	**1% AEP Storm**
Impervious	0.6	0.8
Pervious	0.8	1.0

Table 4.3 – Channelisation factors

Time of concentration

The values for flow length and time of peak flow have been derived from calculations based on the TP108 methodology.

4.8 SUBBASIN PARAMETERS

Please refer to Appendix 8 for a summary of the HEC HMS parameters.

4.9 STORAGE AND ATTENUATION

Calculation for the sizing of the stormwater pond 2 for subbasin 41, 52 to 56 and sizing of the stormwater pond 3 for subbasin 40,50 and 58 are shown in Appendix 8. The ponds have been sized to attenuate 50%, 10% and 1% AEP to pre-development conditions.

Element	50%AEP Storage Volume (m3)	10%AEP Storage Volume (m3)	1%AEP Storage Volume (m3)	Outlet
Stormwater Pond (Outflow 2)	8,390	13,580	22,290	180mm SMAF outlet 2m Scruffy dome cutout
Stormwater Pond (Outflow 3)	1,030	1,510	1,820	68mm SMAF outlet 700mm weir cutout

Table 4.4 Eastern Catchment attenuation device peak flow summary

Element	50%AEP Peak flow Pre development(m3/s)	50%AEP Peak flow Post development(m3/s)
Northern Outflow 2	0.82	0.06
Northern Outflow 3	0.18	0.07

Table 4.5 50%AEP Eastern Catchment site discharge pre-development versus post development flow

Element	10%AEP Peak flow Pre development(m3/s)	10%AEP Peak flow Post development(m3/s)
Northern Outflow 2	2.35	0.64
Northern Outflow 3	0.50	0.49

Table 4.5 10%AEP Eastern Catchment site discharge predevelopment versus post development flow summary

Element	1%AEP Peak flow Pre development (m3/s)	1%AEP Peak flow Post development (m3/s)
Northern Outflow 2*	4.17	4.14
Northern Outflow 3*	0.90	0.87

Table 4.6 1%AEP Eastern Catchment site discharge predevelopment versus post development flow summary

4.10 INFLOW FOR HEC RAS

Upstream inflows generated from the HEC HMS model were then transferred to HEC RAS as inflow boundary conditions, the HEC RAS modelling shall incorporate stormwater hydraulics to the modelling. Please refer to section 5 for hydraulic modelling.

5 EASTERN CATCHMENT HYDRAULIC MODELLING WITH HEC-RAS

5.1 METHODOLOGY

The analysis was done using the following steps:

6. Delineate the perimeter for the grid,
7. Create a grid and sub-grid areas,
8. Input flow hydrographs and other boundaries
9. Input structures,
10. Run scenarios.

5.2 HEC-RAS MODEL LAYOUT

HEC-RAS software was used to generate water levels within the diversion channel, stormwater dry pond, wetland, upstream and downstream of the site. A 2D model was developed using a proposed design contour, LINZ Terrain data and site-specific LiDAR and topographical survey. Review of difference in LINZ terrain and topographical survey showed minor levels differences, especially at critical points, no adjustments were required for the import.

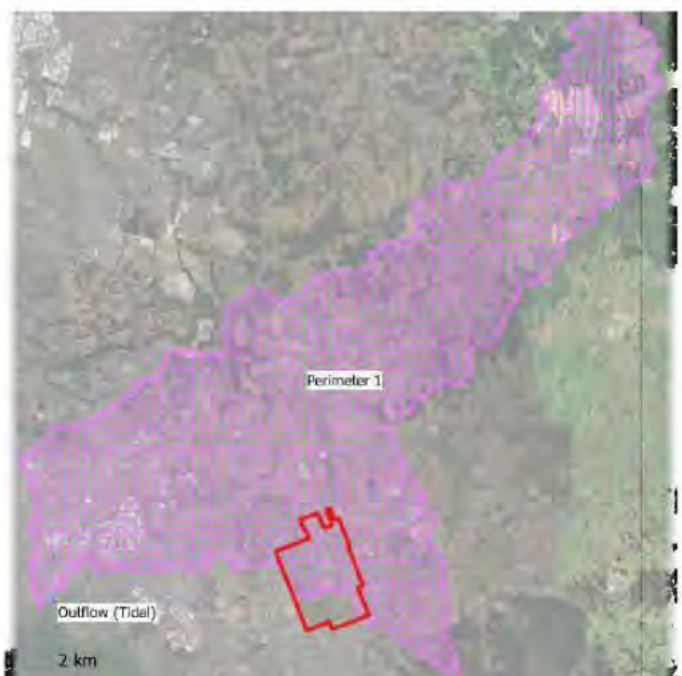


Figure 5.1 – HEC-RAS Predevelopment Eastern model set-up

Surface roughness values adopted in the model were based on land use as categorised in Landcare Research's Land Cover Database version 5 (LCDBv5). This database was released in January 2020 and considers land use classification up until the end of 2018. Details of specific roughness values applied to the different land uses are summarised in Table 5.1. In addition to the above, all road centrelines and major watercourse centrelines were buffered to widths shown in aerial The resulting areas were overlaid with a Manning's n roughness of 0.02 and 0.06. Manning roughness values calibration was undertaken against an existing flow gauge in the Papakura Stream as outlined in Section 5.3. A triangular mesh was used for modelled 2D grid with cell sizes ranging between 2m and 5m for refinement regions and 20m grids for floodplains. Break lines were drawn along critical channels and crests within the terrain. Figure 5.1 shows the grid and its boundary conditions. A predevelopment and post development SCS curve number infiltration layer number was used based on the zoning. Appendix 7 shows the model layout.

Description	Manning's n
Broadleaved Indigenous Hardwoods	0.1
Built-up Area (settlement)	0.2
Deciduous Hardwoods	0.15
Estuarine Open Water	0.022
Exotic Forest	0.1
Forest - Harvested	0.16
Gorse and or Broom	0.08
High Producing Exotic Grassland	0.25
Herbaceous Freshwater Vegetation	0.05
Indigenous Forest	0.15
Lake or Pond	0.04
Low Producing Grassland	0.125
Mangrove	0.02
Manuka and or Kanuka	0.016
Mixed Exotic Shrubland	0.028
Orchard, Vineyard or Other Perennial Crop	0.06
River	0.06
Road	0.02
Short-rotation Cropland	0.1
Surface Mine or Dump	0.09
Transport Infrastructure	0.125
Urban Parkland Open Space	0.035

Table 5.1 Manning Roughness values

5.3 MODEL CALIBRATION

An existing flow gauge was identified downstream of the site, located in the Papakura Stream. Data sets were obtained from the Auckland Council Environmental Data Portal which included the flow gauge data from the hydrology station "Papakura @ Great South Road Bridge" and rainfall data from rainfall located within the modelled catchment. River discharge and rainfall data was obtained from the following rainfall gauges for the 2023 Auckland Anniversary flood event, between the dates of 27th and 29th January 2023. The rainfall gauge measured a total rainfall depth of 229.5mm over 72 hours.

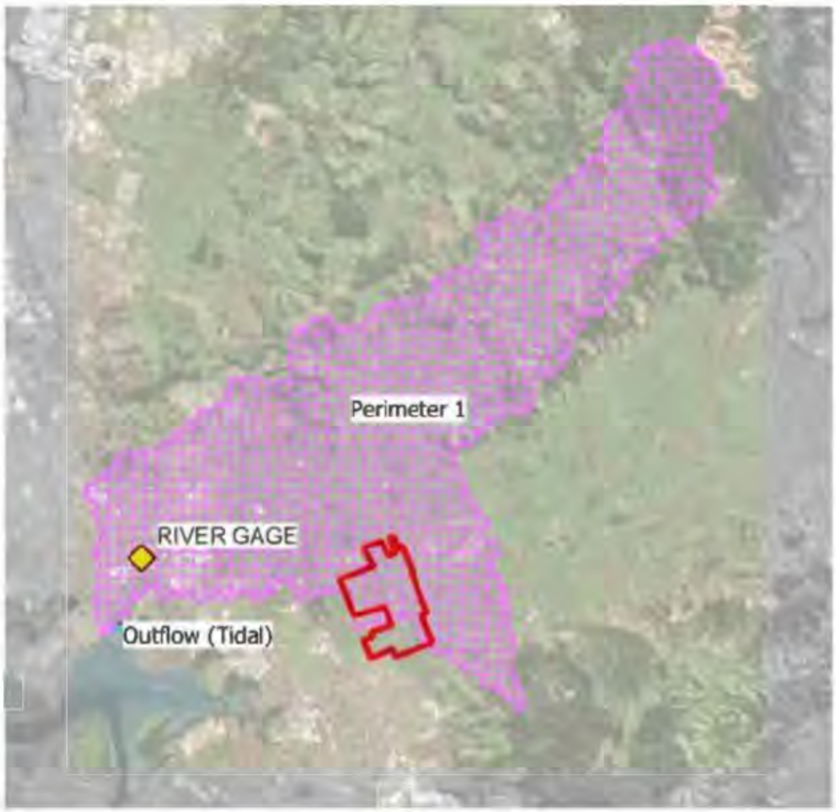


Figure 5.2 – HEC-RAS Papakura Stream Gage calibration

Gauge ID	Gauge Name
43803	Papakura @ Great South Road Bridge
740945	Puhinui at Botanical Gardens

Table 5.2 River and Rain gauges for calibration

Graphical and statistical comparison between the calibration event and model may be found below. The calibration achieved a Nash-Sutcliffe value 0.765 which is considered a very good performance rating per the HEC HMS technical reference manual.

Project: FAB_Eastern_Catchment Simulation Run: Jan2023_HECRAS_Check
Sink: Gage flow comparison

Start of Run:	27Jan2023, 00:10	Basin Model:	Jan2023_HEC_RAS
End of Run:	29Jan2023, 11:00	Meteorologic Model:	TP108_100yr_298mm_CoPv
Compute Time:	24Oct2024, 11:24:15	Control Specifications:	27Jan-31Jan_2023

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Discharge:	56.80800 (M3/S)	Date/Time of Peak Discharge	28Jan2023, 02:10
Volume:	4857.94650 (1000 M3)		

Observed Flow Gage River_Jan2023

Peak Discharge:	59.03875 (M3/S)	Date/Time of Peak Discharge:	27Jan2023, 19:10
Volume:	4667.77290 (1000 M3)		
RMSE Std Dev:	0.48434	Nash-Sutcliffe:	0.765
Percent Bias:	0.24 %		

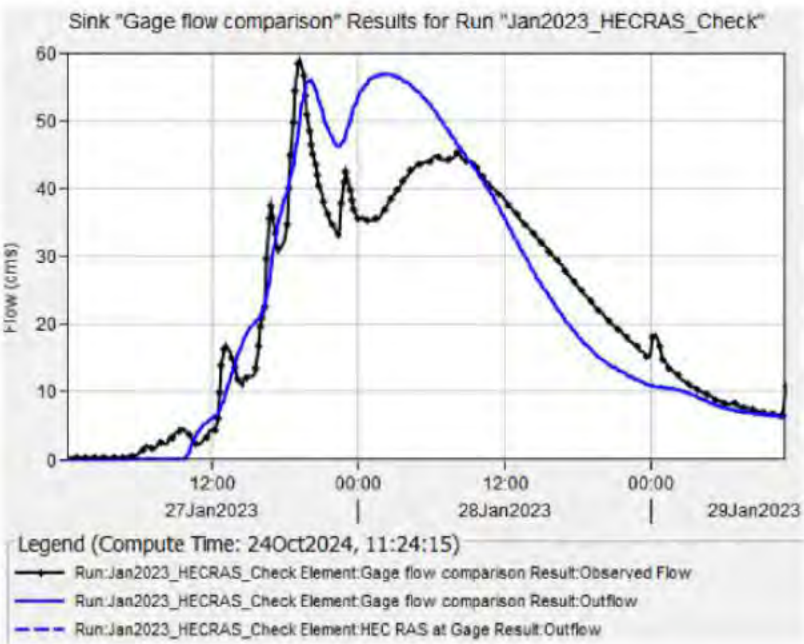


Figure 5.3 – HEC-RAS Papakura Stream Gage calibration statistics

5.4 BOUNDARIES

There are three boundaries. These are:

- Rain on grid – as per figure 5.1.
- Inflow hydrographs imported from HEC HMS (outlined in section 2)
  HEC HMS subbasins have been used as inflows (please refer to appendix 8 for plan)
- Outflow boundary - Tidal Boundary

Runoff from the eastern catchment eventually discharges to Manukau Harbour.
The downstream boundary was constructed using a fixed stage for the tidal boundary condition at 2.34 mRL (AUK1946) or 2.06 NZVD2016. This level has been used for consistency with the Western Catchment. However, it is noted the tidal boundary is located 7km downstream of the site with an elevation 19m below the site and therefore will not have any effect on this assessment.

5.5 CRITICAL STORM DURATION ANALYSIS

It is noted that the TP108 approach used in this modelling assessment used a nested storm, created from a range of durations up to 24 hours. A critical storm duration analysis was undertaken to verify the suitability of the TP108 storm. Rainfall patterns for the north of the north island from NIWA HIRDSv4 were used for the storm durations 1hr, 6hr, 12hr , 24hr, 48hr and 72hr. Rainfall depths for each storm were obtained from the NIWA HIRDSv4 for the 10%AEP and 1%AEP events, using the most conservative available climate change assumption of representative concentration pathways 8.5 (RCP 8.5, 2081-2100).
A critical storm check was completed at five locations within the catchment. All checked locations show the critical storm to be the nested TP108 24hr storm. This verifies the TP108 critical storm to be applicable to the site analysis. Hydrographs for each of the checks may be found in Appendix 2

5.6 HYDRAULIC STRUCTURES

At the end of the eastern main diversion channel a lateral weir of length 700m is proposed across the northern site boundary at an elevation ranging between mRL 22.42 to 22.60 to control flow exiting the northern site boundary (Northern Outflow 1). A stormwater pond (Stormwater Pond 1) is located adjacent the channel with proposed invert level 20.70 and mRL has two storage basins to manage the 50%, 10% and 1%AEP storm peak flows. During 50% and 10% AEP peak flows a 340m weir of elevation mRL22.52 diverts the low flow to a box culvert (0.4m x 1.2m) to the 10%AEP storage basin. During 1% AEP peak flows a 410m weir of elevation mRL22.59 diverts flow to the 1%AEP storage basin. Figure 5.4 below shows the proposed configuration (weirs are shown in yellow). Stormwater pipes with check valves shall be installed between storage basins and the diversion swale to allow draindown of storage basins post storm events.

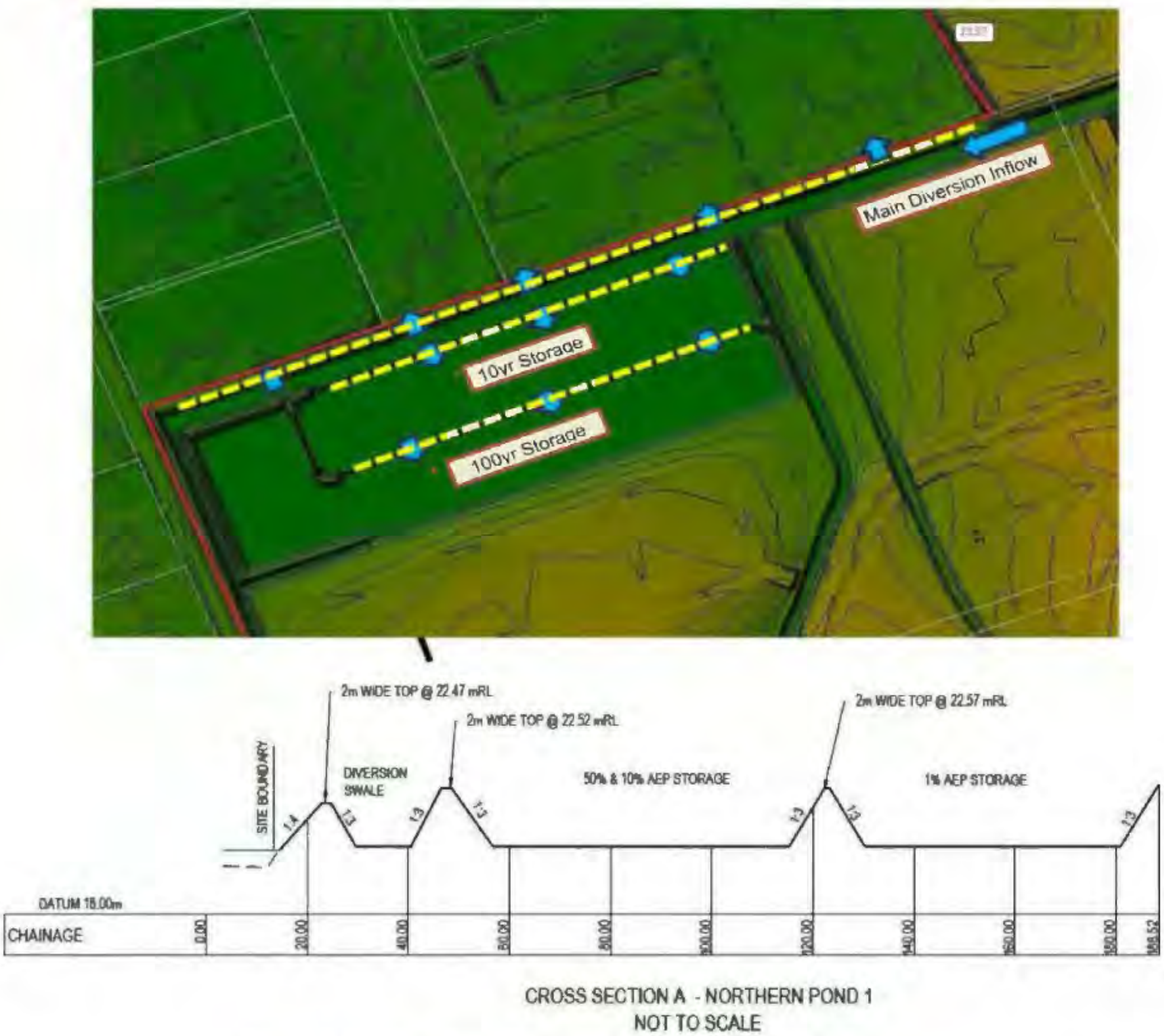


Figure 5.4 – HEC-RAS Post development Outflow 1 Configuration

5.7 OUTFLOW VOLUME CHECK

The HEC RAS computation volume error for each scenario is summarised in the table below;

Scenario	Volumes error m3	Error as percentage
50%AEP Predevelopment	516	0.02%
50% AEP Post development	1,625	0.06%
10%AEP Predevelopment	920	0.01%
10% AEP Post development	1,885	0.03%
1%AEP Predevelopment	1,727	0.01%
1% AEP Post development	2,512	0.02%

Table 5.3 Outflow volume check for Eastern Catchment HEC RAS model

5.8 STORMWATER POND 5 & 6

The properties 119, 119A, 121A, 123, 131 and 143 Cosgrave Road has an area of 24.1Ha. This area is likely to be developed at a later date to the rest of the site. For the purposes of this assessment flows in the post development scenario of this catchment have been modelled with the existing terrain in this area with infiltration based on the existing MPD impervious percentage of 10%. Flows generated from the site are discharged to the site swale network and conveyed to Northern Outflow 1. Stormwater ponds 5 and 6 have been indicatively shown as future development of this catchment shall require stormwater ponds to attenuate flows from the catchment to a pre-development condition.

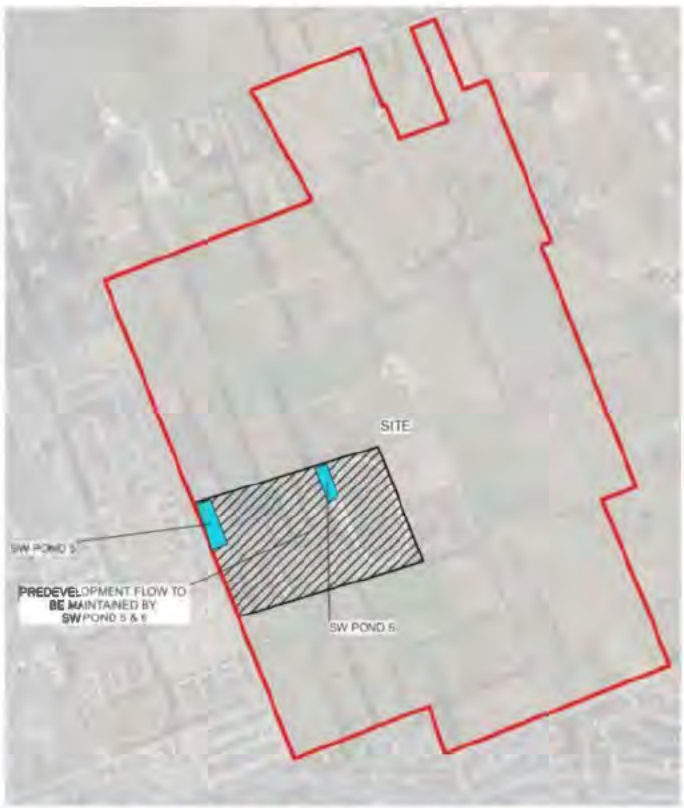


Figure 5.4 – HEC-RAS Post development Outflow 1 Configuration

5.9 EASTERN CATCHMENT PEAK FLOW RESULTS

Peak flow results for the Eastern Catchment can be found in the Appendix 7.

Review of the modelling results (at the northern outflow 1), show a predevelopment a peak flow for the 10%AEP and 1%AEP peak flows to remain effectively unchanged post development.

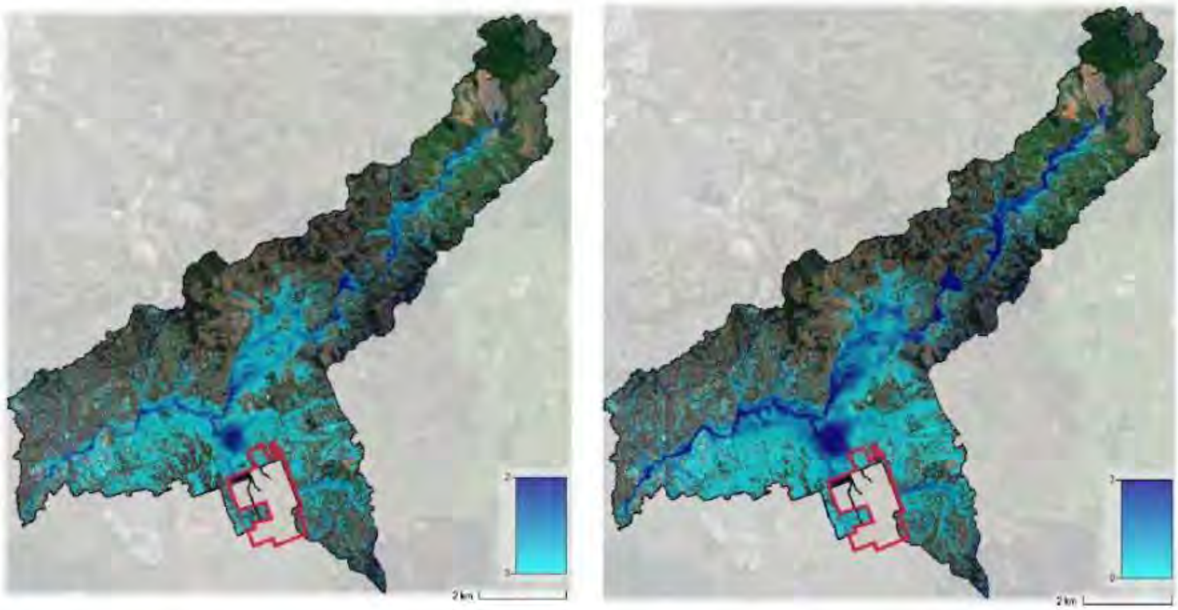


Figure 5.5 10%AEP (left) and 1%AEP (right) modelled post development flood depths

Element	Outflow 1 Peak flow Pre development(m3/s)	Outflow 1 Peak Peak flow Post development(m3/s)
50% AEP	6.1	6.1
10% AEP	22.0	21.6
1% AEP	52.0	51.8

Table 4.5 Outflow 1 site discharge pre-development versus post development flow

Post development flows show a minor reduction or negligible change in comparison to pre-development flows. The 1%AEP flow of 52.0 m3/s has a reduction to 51.8 m3/s, the 10%AEP flow shows a minor decrease in flow post development from 22.0 to 21.6 m3/s. The 50% AEP flow remains unchanged at 6.1 m3/s. It is concluded that the proposed development has no adverse effects on downstream properties during the modelled 50%, 10% and 1% AEP storm event.

Plans in Appendix 7 show a comparison in flood levels and hydrographs exiting the northern boundary.

5.10 RESULTS - EASTERN CATCHMENT DOWNSTREAM PEAK FLOW LEVEL AT OUTFLOW 1

The modelling results from the eastern catchment are shown on plans in Appendix 7.

The weir outlet along the northern boundary has been iteratively designed to simulate the predevelopment flow exiting the site as much as possible no notable increase in downstream flood levels was observed in the post development model.

5.11 RESULTS - PAPAKURA STREAM EFFECTS

A comparison of peak flow rates between the existing and post development scenarios shows that flow rates and peak flows in the Papakura stream either remain unchanged or have a small decrease. Peak water levels for the 1%AEP storm are reduced by approximately 70mm and peak flows reduced by approximately 5% in the Papakura Stream. This is attributed to the reduced time of concentration of Catchment C. This finding supports the proposed pass-forward strategy for outflow 1 of the site. The modelling results from the eastern catchment are shown on plans in Appendix 7.

Figure 5.6 shows a decrease in peak water levels for both the 10%AEP and 1%AEP storm events

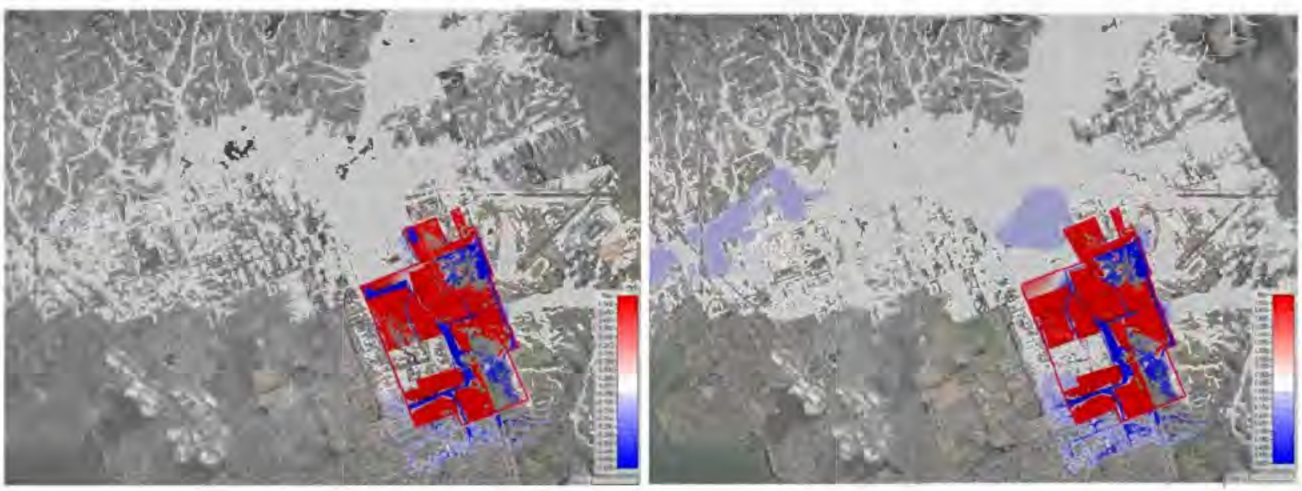


Figure 5.6 10%AEP (left) and 1%AEP (right) modelled post development downstream effects (red is reduction in peak water levels)

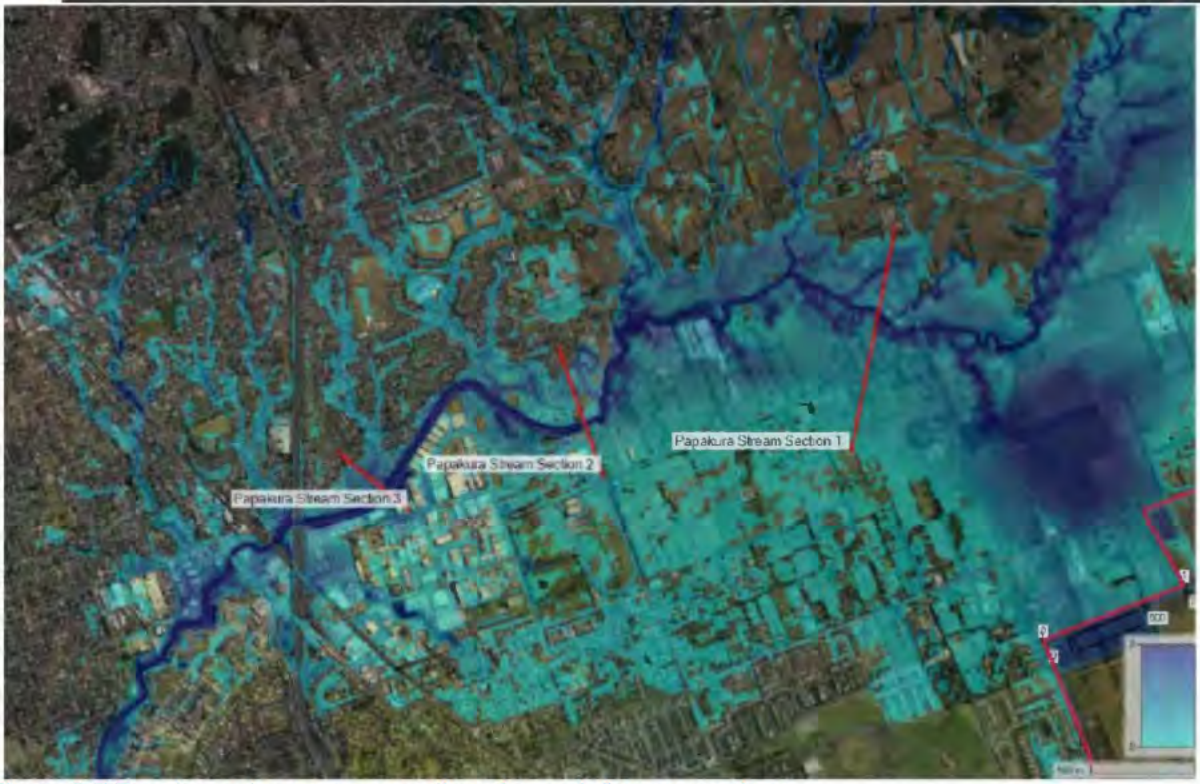


Figure 5.7 Papakura stream cross sections for peak flow comparison

Element	10%AEP peak flow existing (m3 /s)	10%AEP peak flow post development (m3 /s)	Change
Cross section 1	70.3	66.2	-4.1 (-6%)
Cross section 2	73.0	69.1	-3.9 (-5%)
Cross section 3	74.3	70.4	-3.9 (-5%)

Element	1%AEP peak flow existing (m3 /s)	1%AEP peak flow post development (m3 /s)	Change
Cross section 1	192.2	181.9	-10.3 (-5%)
Cross section 2	199.1	188.7	-10.4 (-5%)
Cross section 3	195.3	187.2	-8.1 (-4%)

5.12 EASTERN CATCHMENT ATTENUATION DEVICES

Table 5.4 summarises the proposed (post development) Eastern Catchment stormwater pond storage and attenuation devices.

Element	50% AEP Storage Volume (m3)	10% AEP Storage Volume (m3)	1% AEP Storage Volume (m3)	Outlet
Stormwater Pond 1 (Outflow 1)	68,000	77,000	141,000	700m weir
Stormwater Pond 2 (Outflow 2)	8,390	13,580	22,290	180mm SMAF outlet 1350mm Scruffy dome
Stormwater Pond 3 (Outflow 3)	1,030	1,510	1,820	68mm SMAF outlet 700mm weir cutout

Table 5.4 Eastern Catchment attenuation device configuration summary

Element	10% AEP Peak flow Pre development(m3/s)	10% AEP Peak flow Post development(m3/s)
Northern Outflow 1	22.0	21.6
Northern Outflow 2*	2.35	0.64
Northern Outflow 3*	0.50	0.49

**Refer to HMS in section 2 for calculations*

Table 5.5 10%AEP Eastern Catchment site discharge predevelopment versus post development flow summary

Element	1% AEP Peak flow Pre development(m3/s)	1% AEP Peak flow Post development(m3/s)
Northern Outfow 1	52.0	51.8
Northern Outfow 2*	4.17	4.14
Northern Outfow 3*	0.90	0.87

**Refer to HMS in section 2 for flows*

Table 5.6 1%AEP Eastern Catchment site discharge predevelopment versus post development flow summary

5.13 EASTERN CATCHMENT OUTFLOW 1 PASS FORWARD FLOW

A pass-forward flow strategy is proposed for the Northern Out flow 1. This has been assessed to be the best practical option for the large 350.7 Ha of Catchment C due to the smaller time of concentration of site discharges in comparison to the flow from the large upstream catchment.The upstream catchment (350.7 Ha) generates a substantial 1% AEP peak flow of 54 m³/s, which enters the site's eastern boundary at 13:20 (with a time to peak of approximately 80 minutes). Flows generated from the site have an average time of concentration of 20minutes, the combined peak of the site discharge in the swales has a peak 1%AEP flow of 26 m3/s. Figure 5.8 shows a comparison of the hydrographs. Pass-forward flow shall allow flow from the site, which have a smaller peak flow to that of the upstream, to exit the site before arrival the upstream catchment peak flow reaches the site. It is noted that if an alternative strategy such as peak flow attenuation was applied to the catchment the attenuated from the site exiting via outflow 1 would coincide with the upstream peak flow and result in a larger resultant peak flow. Section 5.11 of the report shows assessment of the effect further downstream of the site in Papakura Stream. No increases in peak flow or water levels were observed as a results of the pass-forward flow of norther outflow 1.

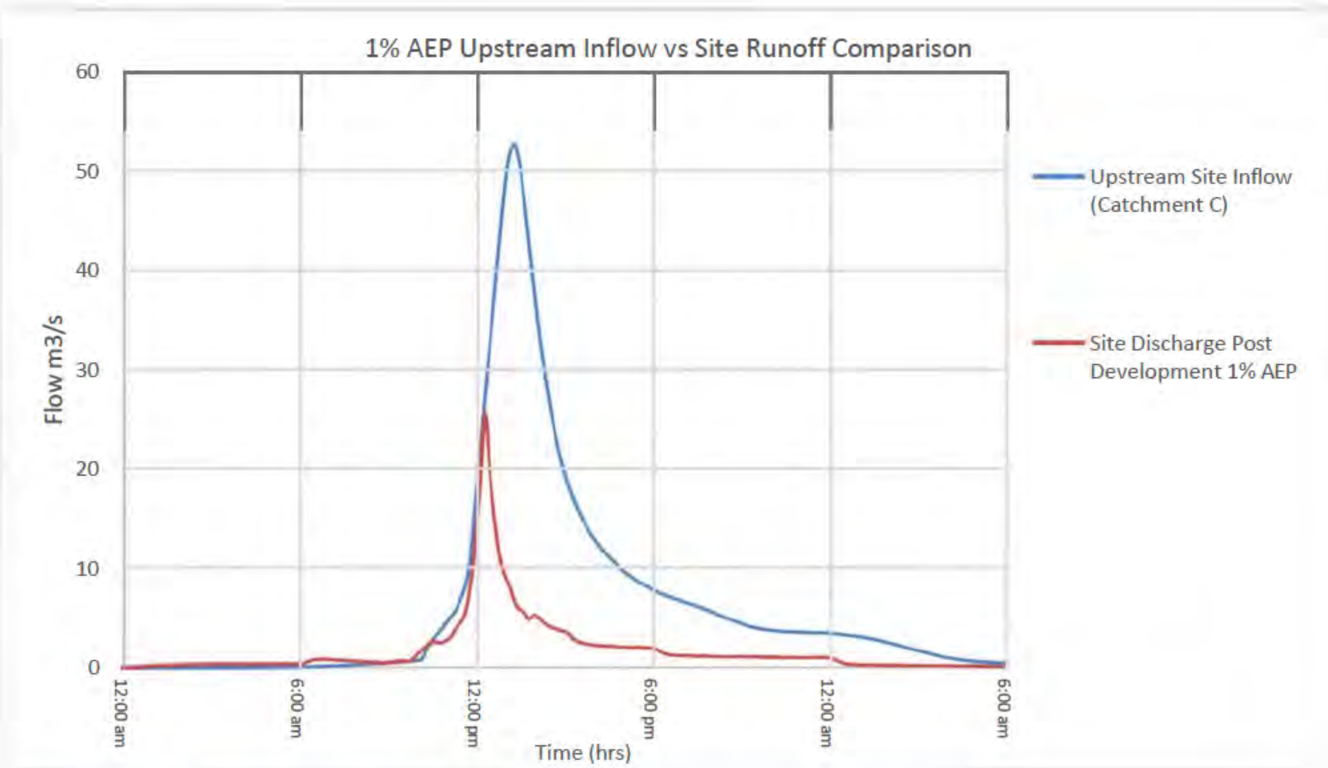


Figure 5.8 Upstream versus site discharge 1%AEP flow hydrograph comparison

5.14 CONCLUSION – EASTERN CATCHMENT

A flood model has been built to assess flood effects of the proposed post development from the Eastern site catchment during the 50%, 10% and 1% AEP storm events assuming the Auckland Council SWCoP version 4 climate change factors.

Flood levels and peak flow post development were compared to the predevelopment flood levels and peak flows. No negative effects were highlighted in any of the modelling results

Site area within the post development catchment D1 (15.3 Ha) and D2 (2.8 Ha) discharge to Outflows 2 and 3 respectively. Flows from these catchments are proposed to be attenuated via stormwater ponds to pre-development flows for the 50%, 10% and 1%AEP storms.

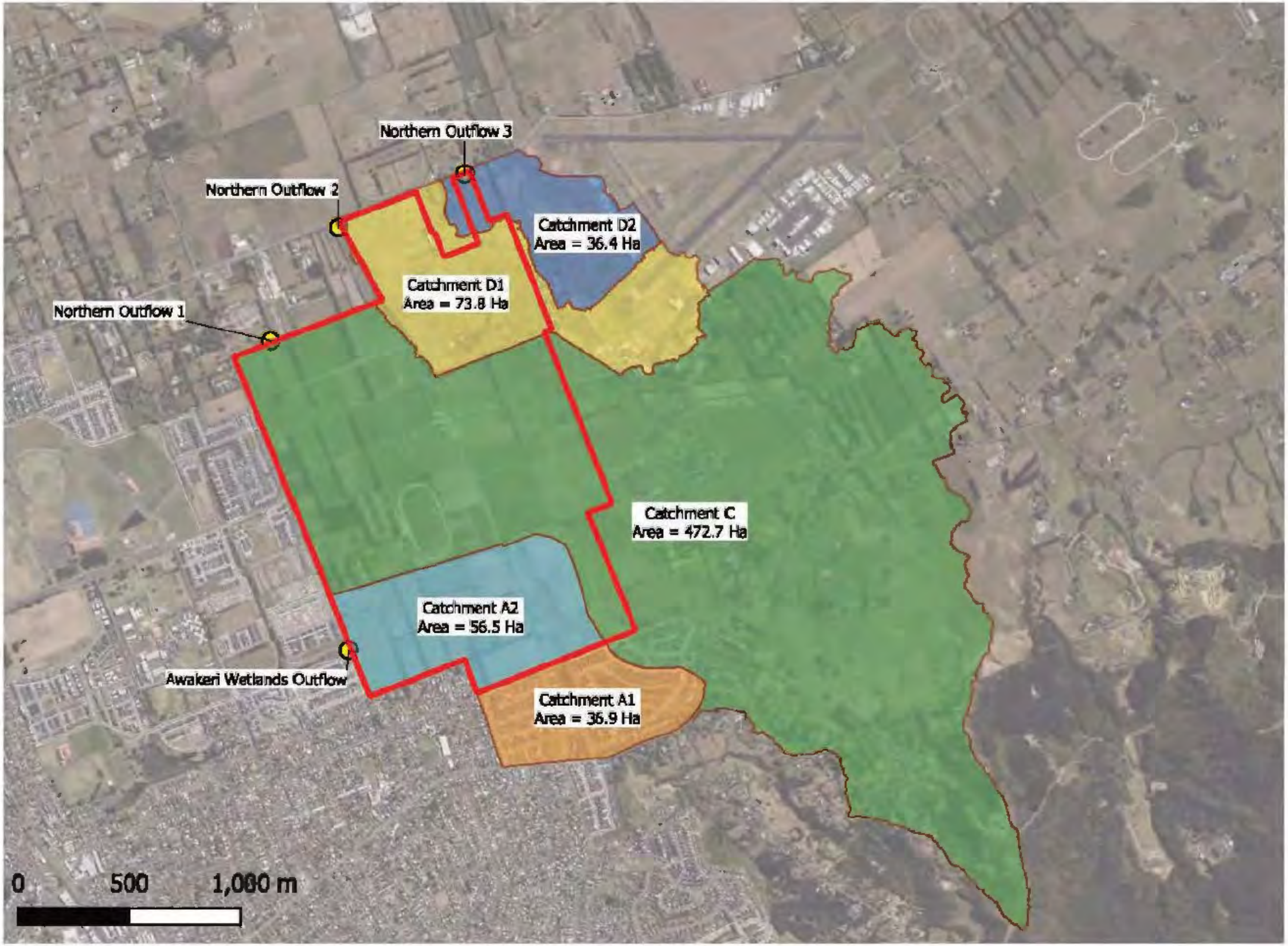
The catchment area within the site discharging to the Northern Outflow 1 via Stormwater Pond 1 is 109.1 Ha, of this area 29.5 Ha of the site is allocated to stormwater management as either swales or Stormwater Pond 1. Peak flow across Northern Outflow 1 is governed by the large upstream catchment to the east of the site. Site discharge across northern outflow 1 is proposed to be passed forward while maintaining the existing peak flows.

An Auckland Unitary Plan E36 flood risk assessment may be found in Appendix 14.

APPENDIX 1 – CATCHMENT PLANS

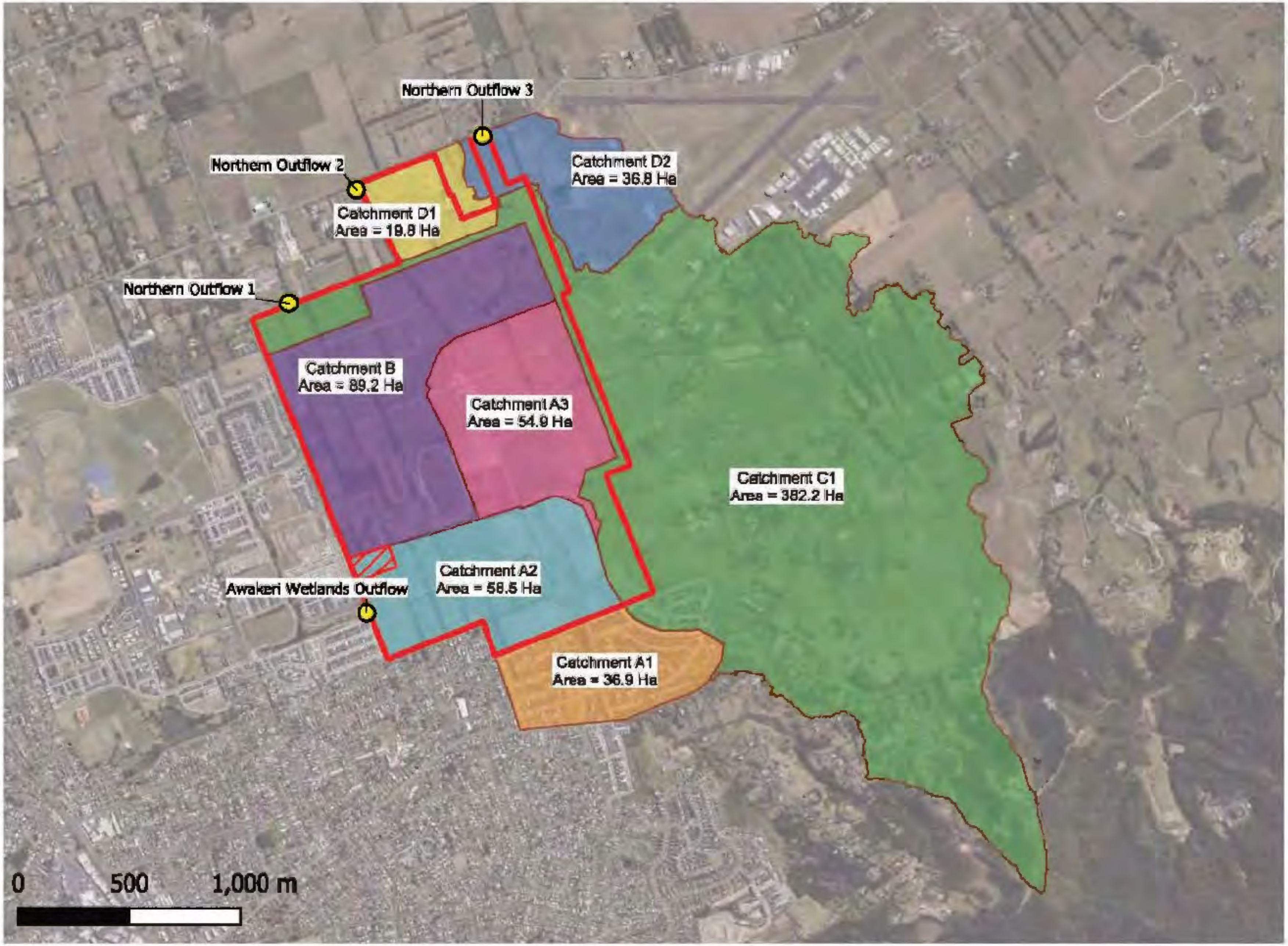
Pre Development Catchments Overview

Legend



SK001
REV 002

Post Development Catchments



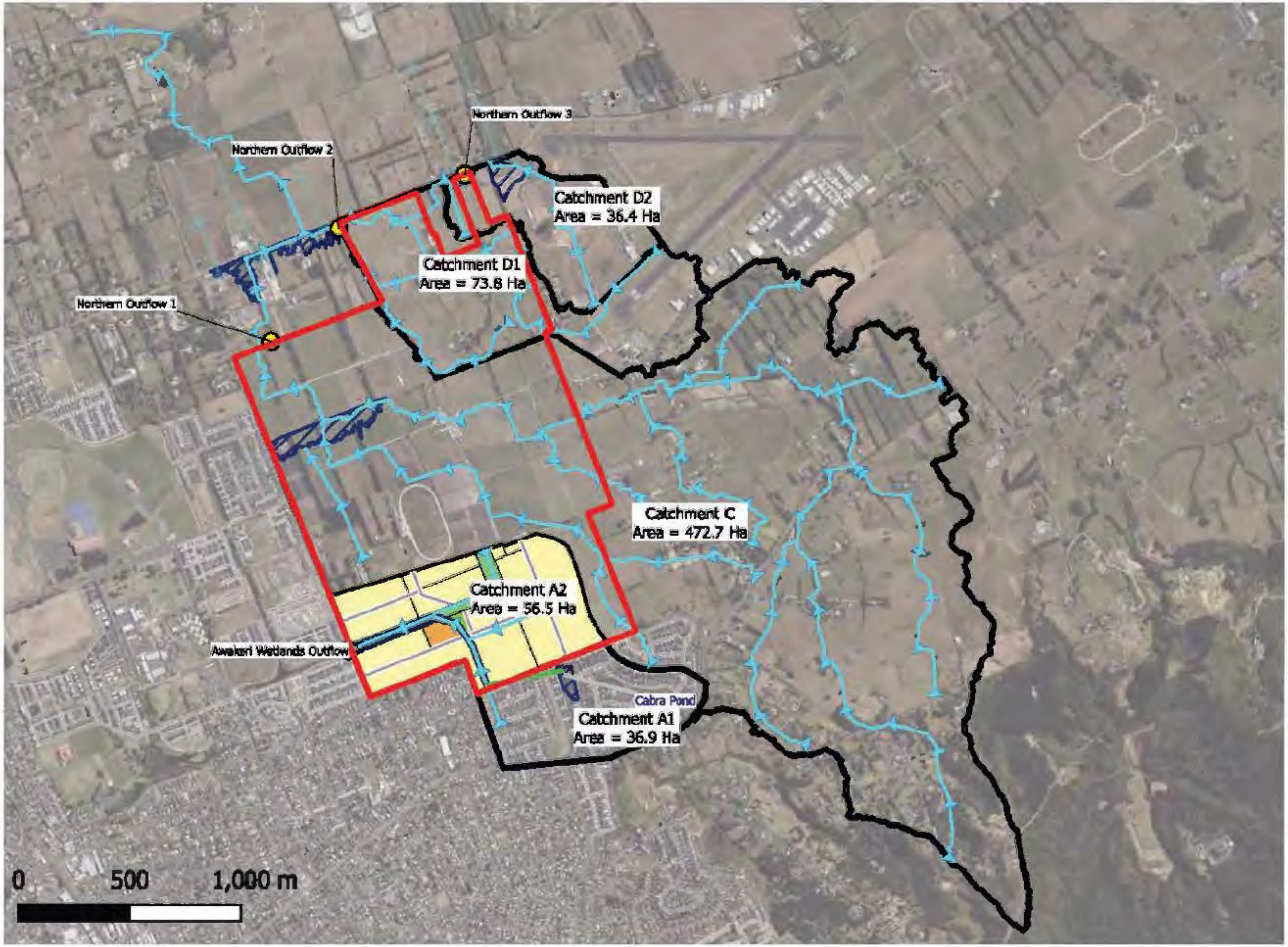
Legend

Site Boundary

101CosgraveRoad Flow Allowance made for discharge to Awakeri wetland

SK005
REV 001

Existing Catchment Flow paths



Legend

Site Boundary

Predevelopment Catchments

Catchments

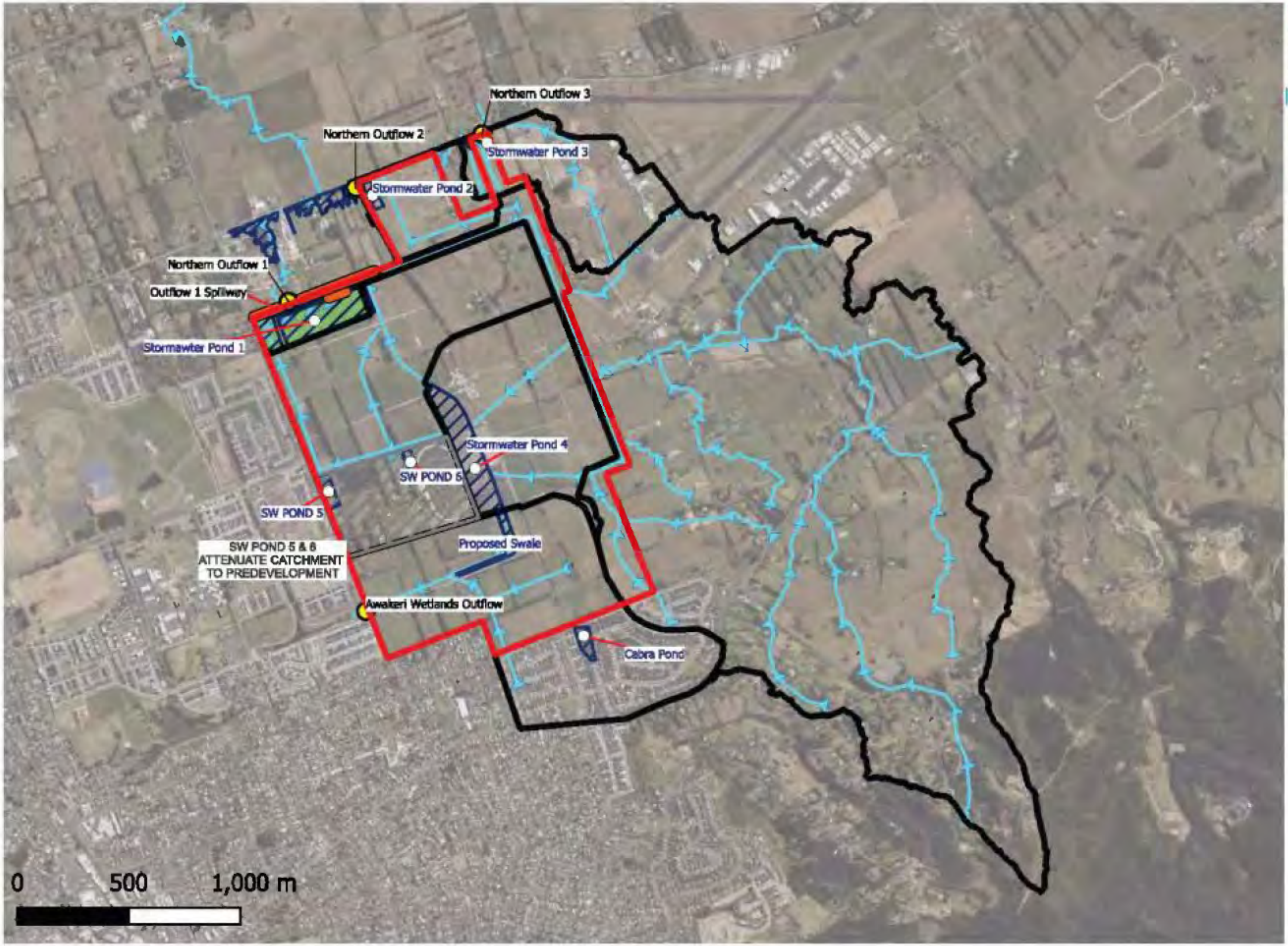
Existing OLFP

Existing Flood Prone Storage

Site Outflow

SK003
REV 003

Proposed Catchment Flow paths



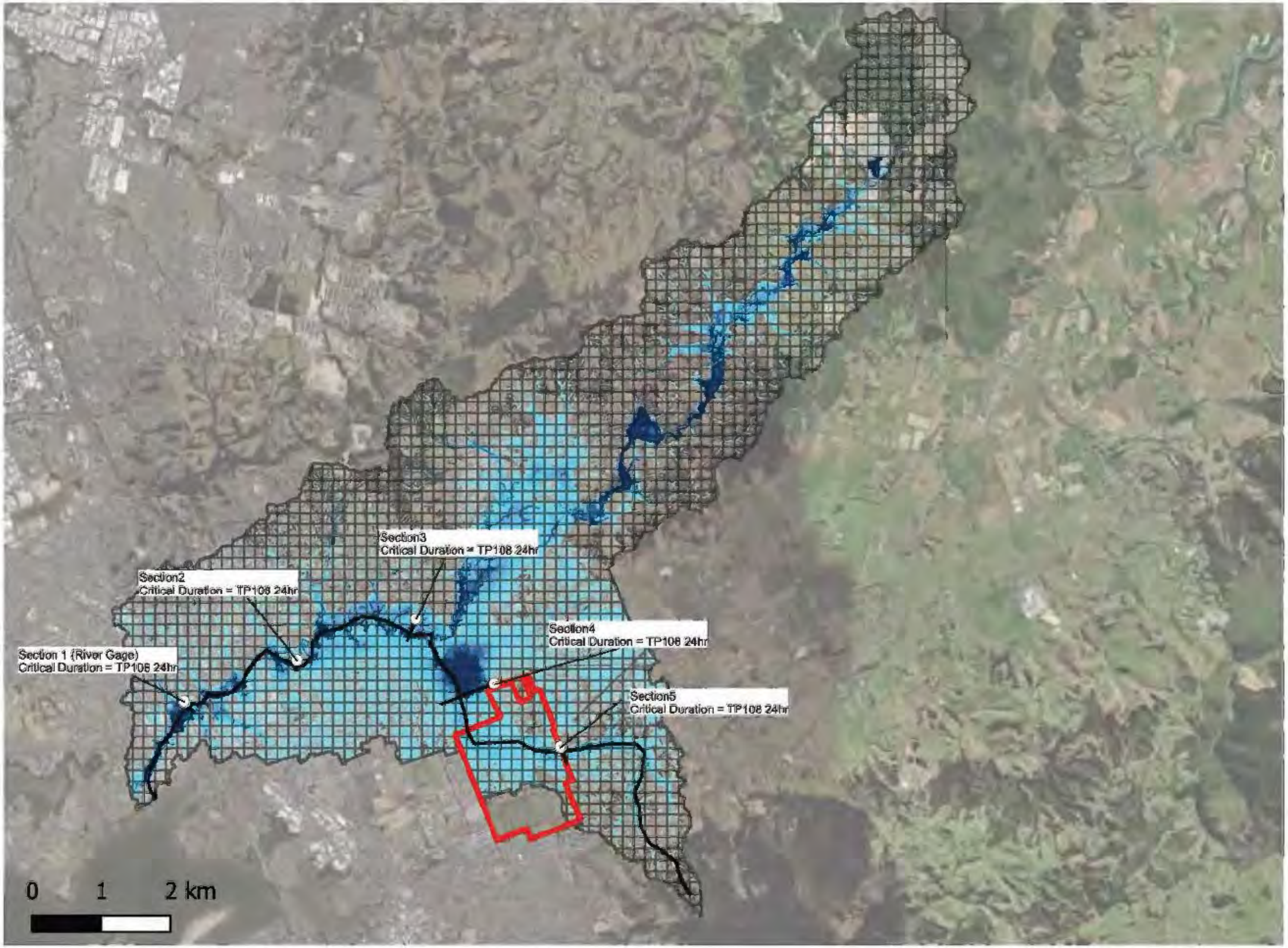
Legend

- Site Boundary
- Postdevelopment Catchments
- Proposed OLFP
- Post development Flood Storage
- Proposed SW Pipe
- Catchment to be attenuated to predevelopment (10% Impervious) by SW Pond 5 & 6

SK006
REV 003

APPENDIX 2 – Critical Storm Check

Eastern Catchment Critical Storm Check



Legend

Pr-Bdy

Outflow Boundary Conditions

Perimeter

Eastern catchment
Pre development
1% AEP peak flow depths

<= 0.50000

0.50000 - 1.00000

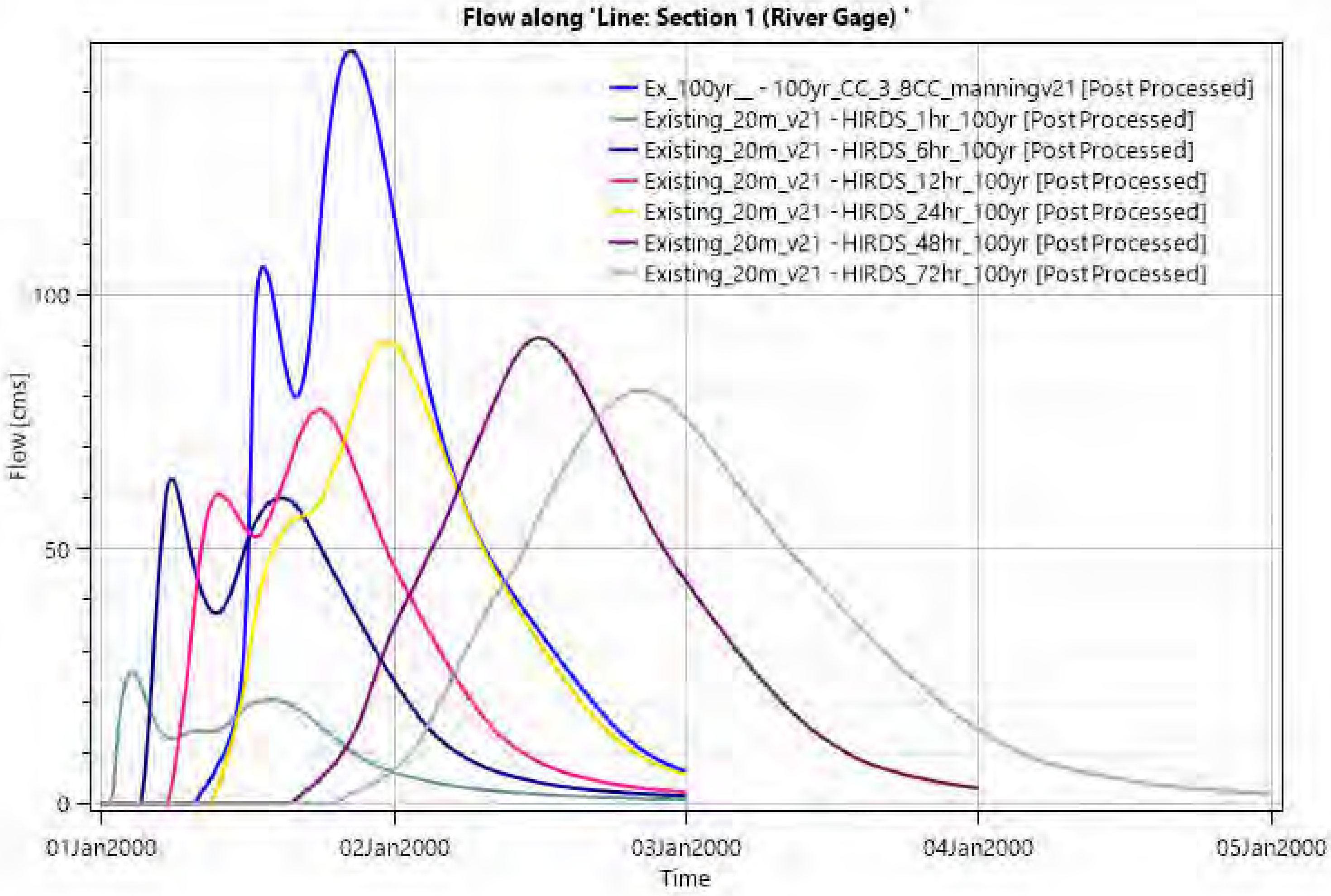
1.00000 - 1.50000

1.50000 - 2.00000

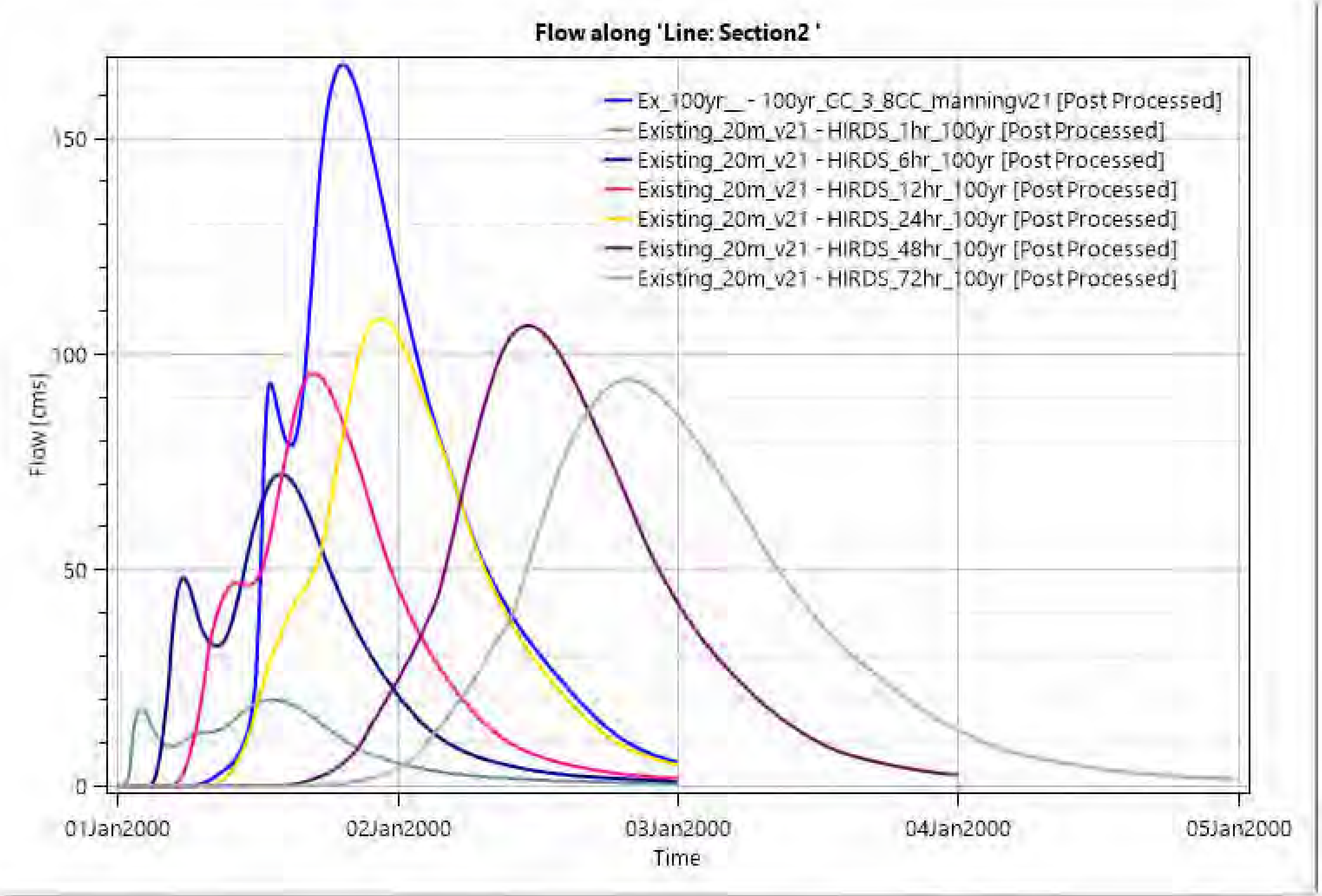
> 2.00000

SK035
REV001

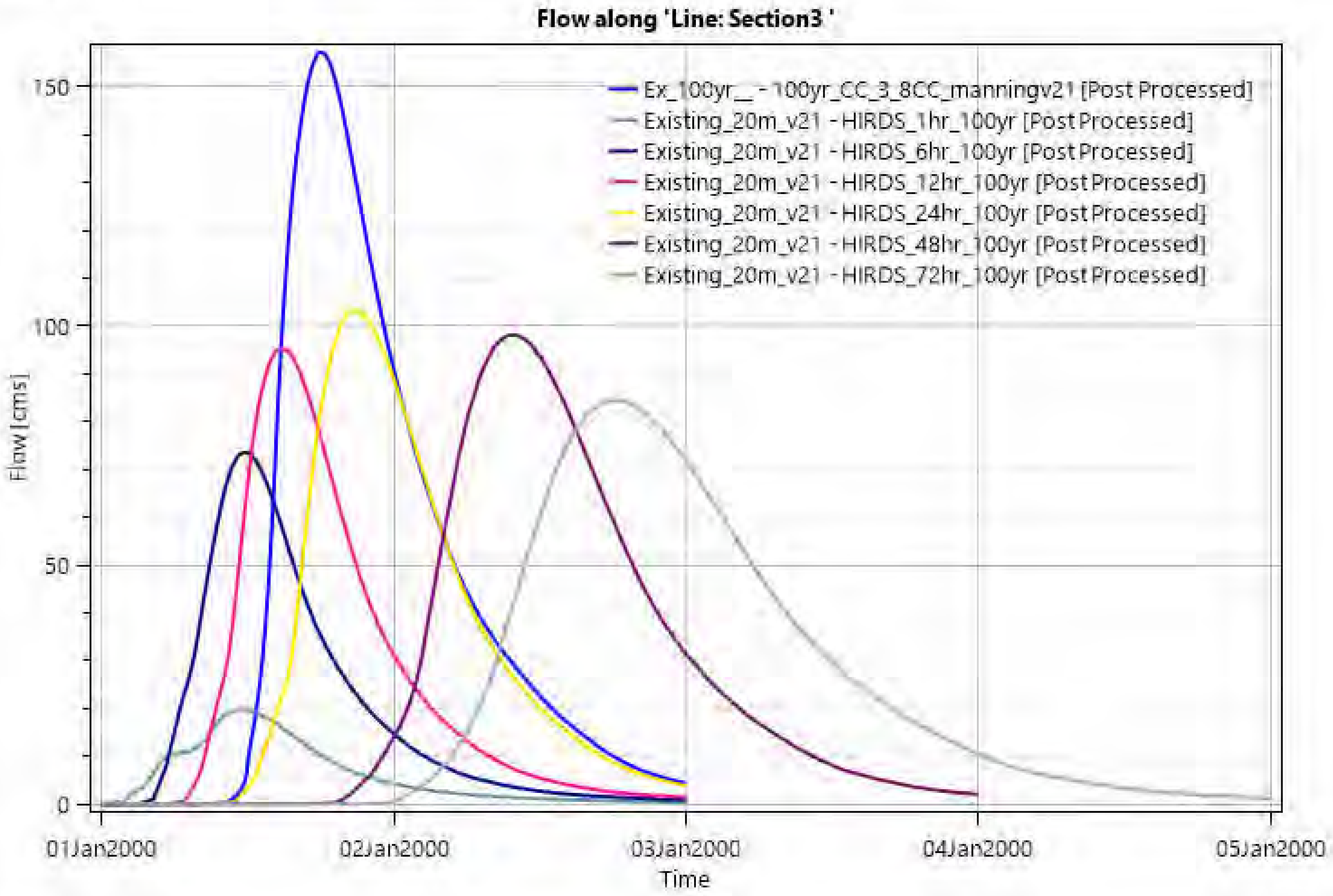
Critical Storm Check – 1%AEP Cross section 1



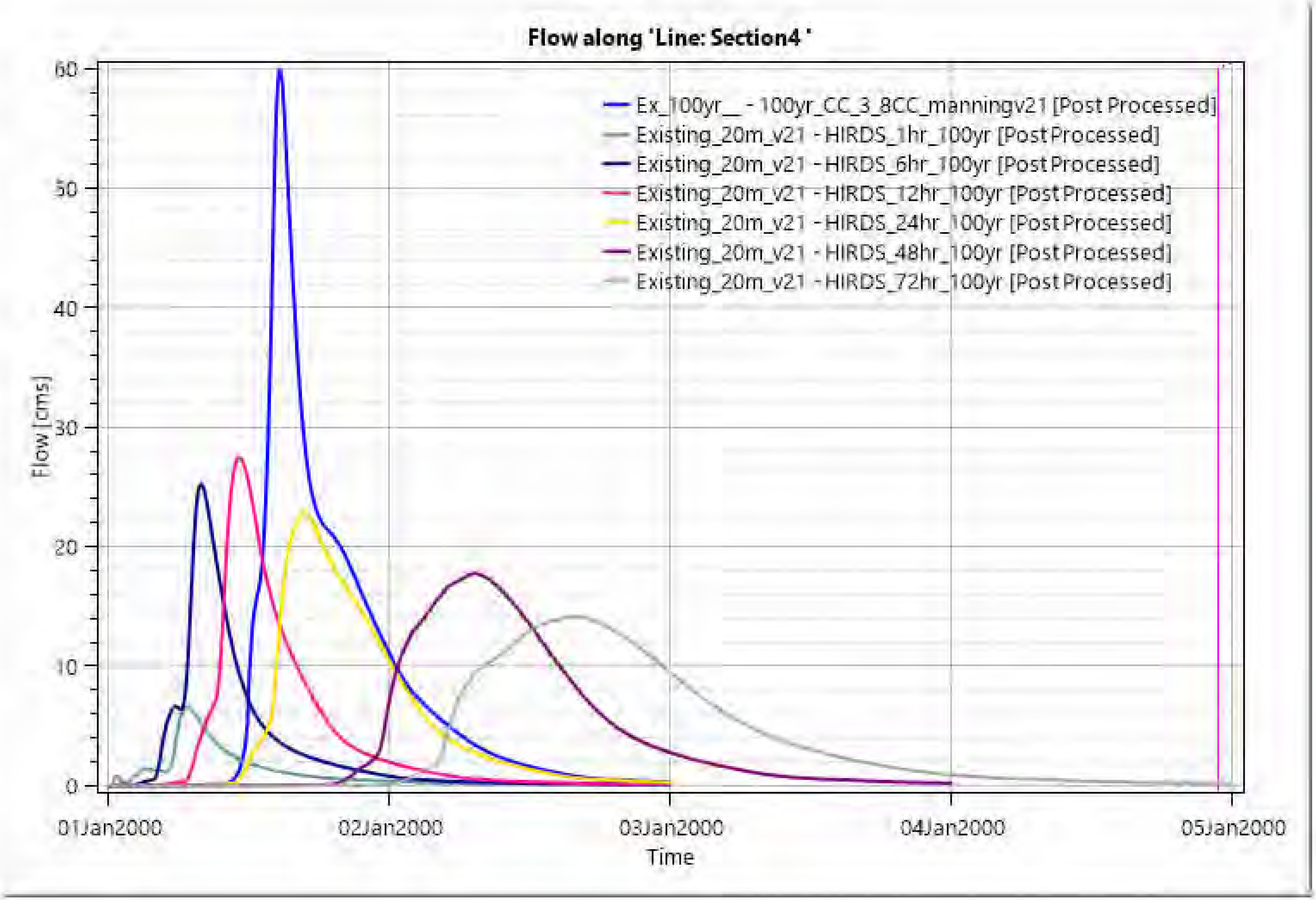
Critical Storm Check – 1%AEP Cross section 2



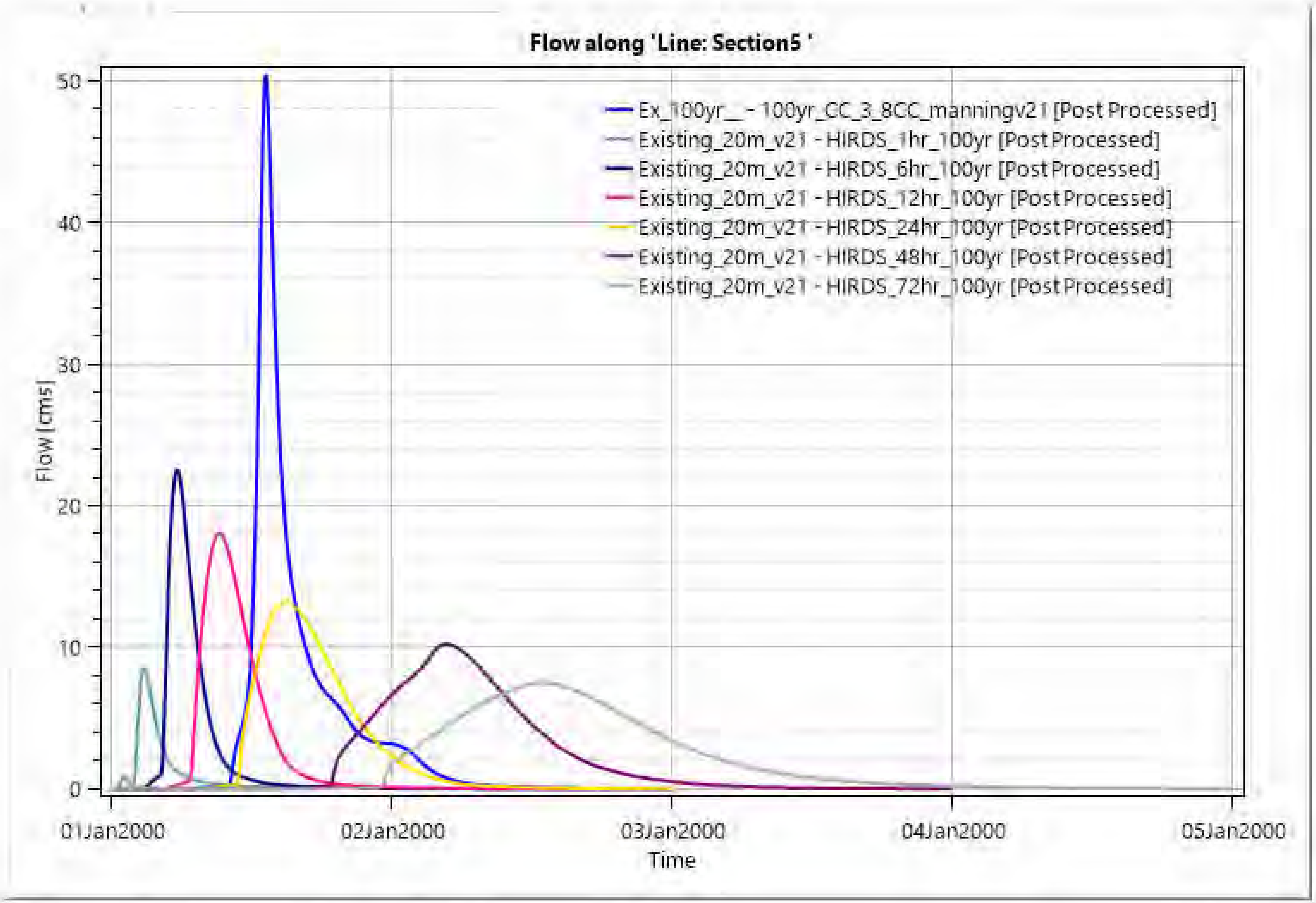
Critical Storm Check – 1%AEP Cross section 3



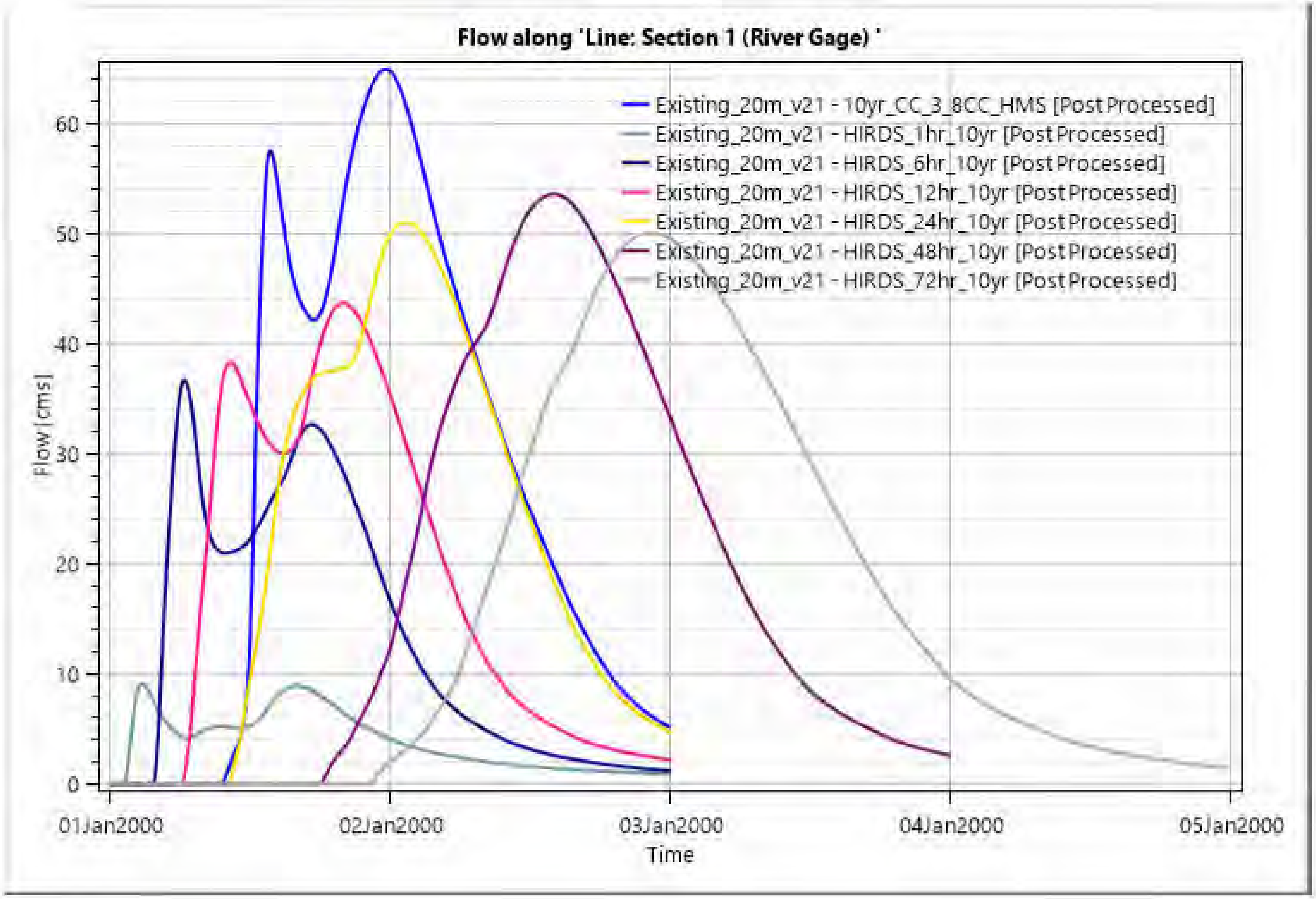
Critical Storm Check – 1%AEP Cross section 4



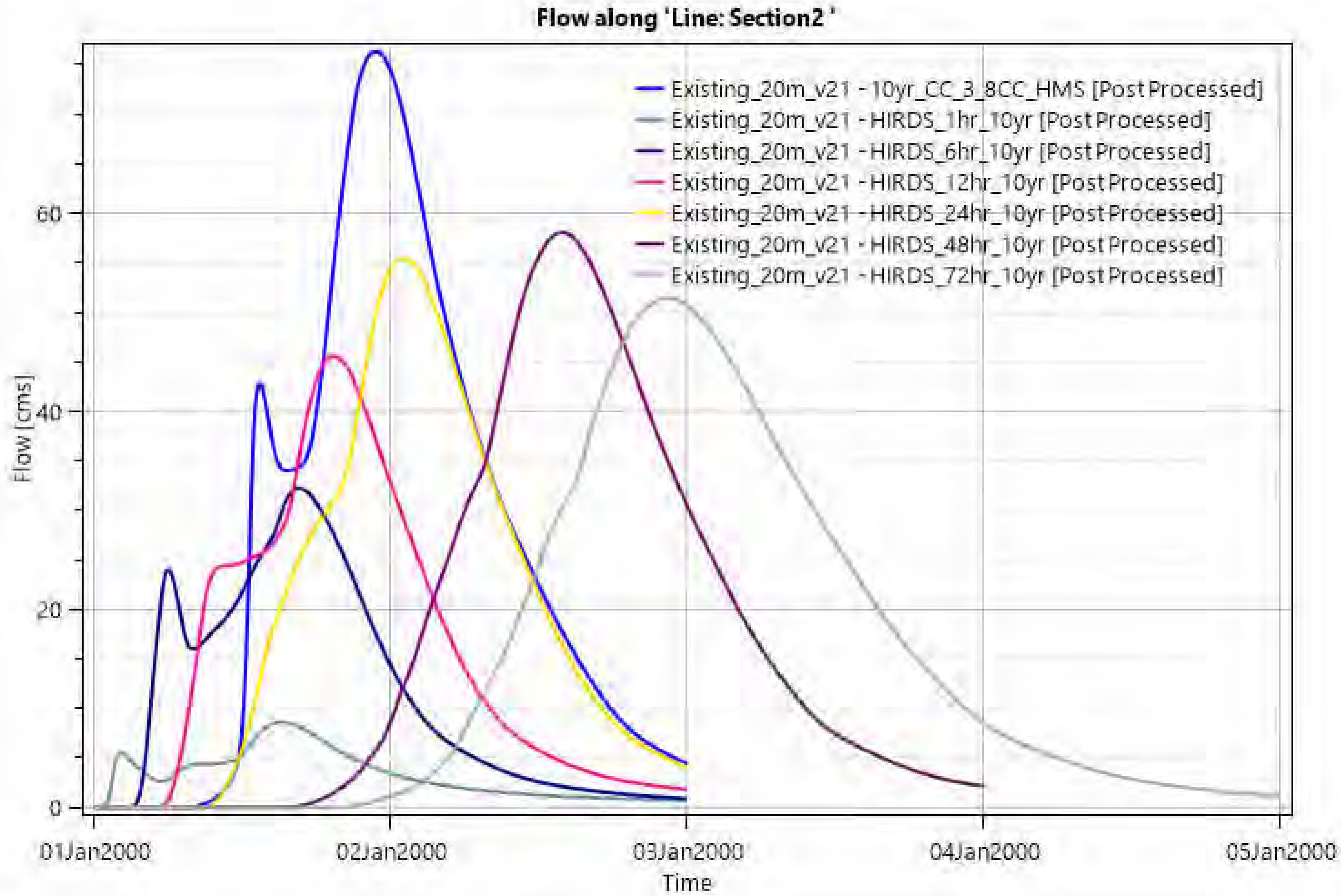
Critical Storm Check – 1%AEP Cross section 5



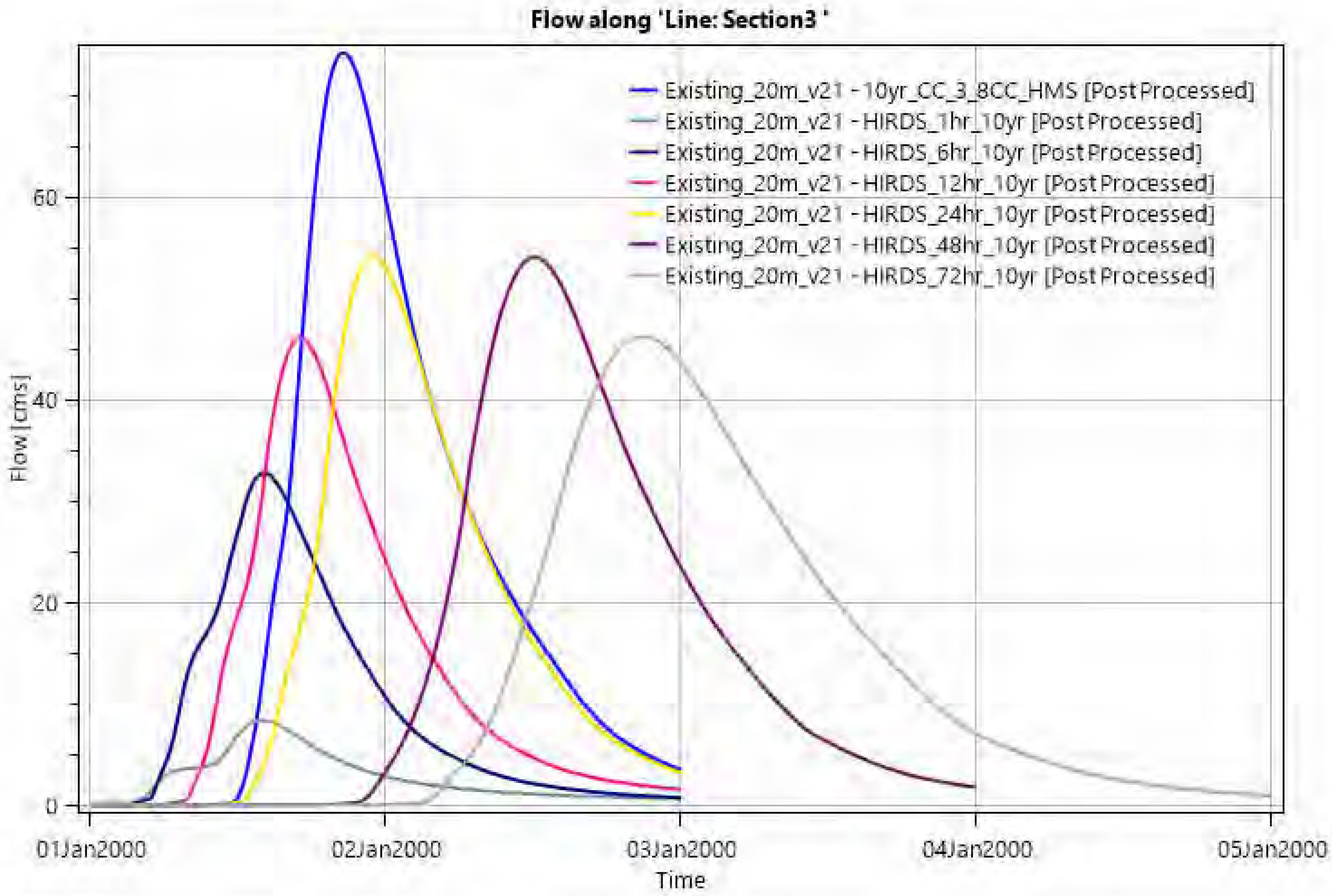
Critical Storm Check – 10%AEP Cross section 1



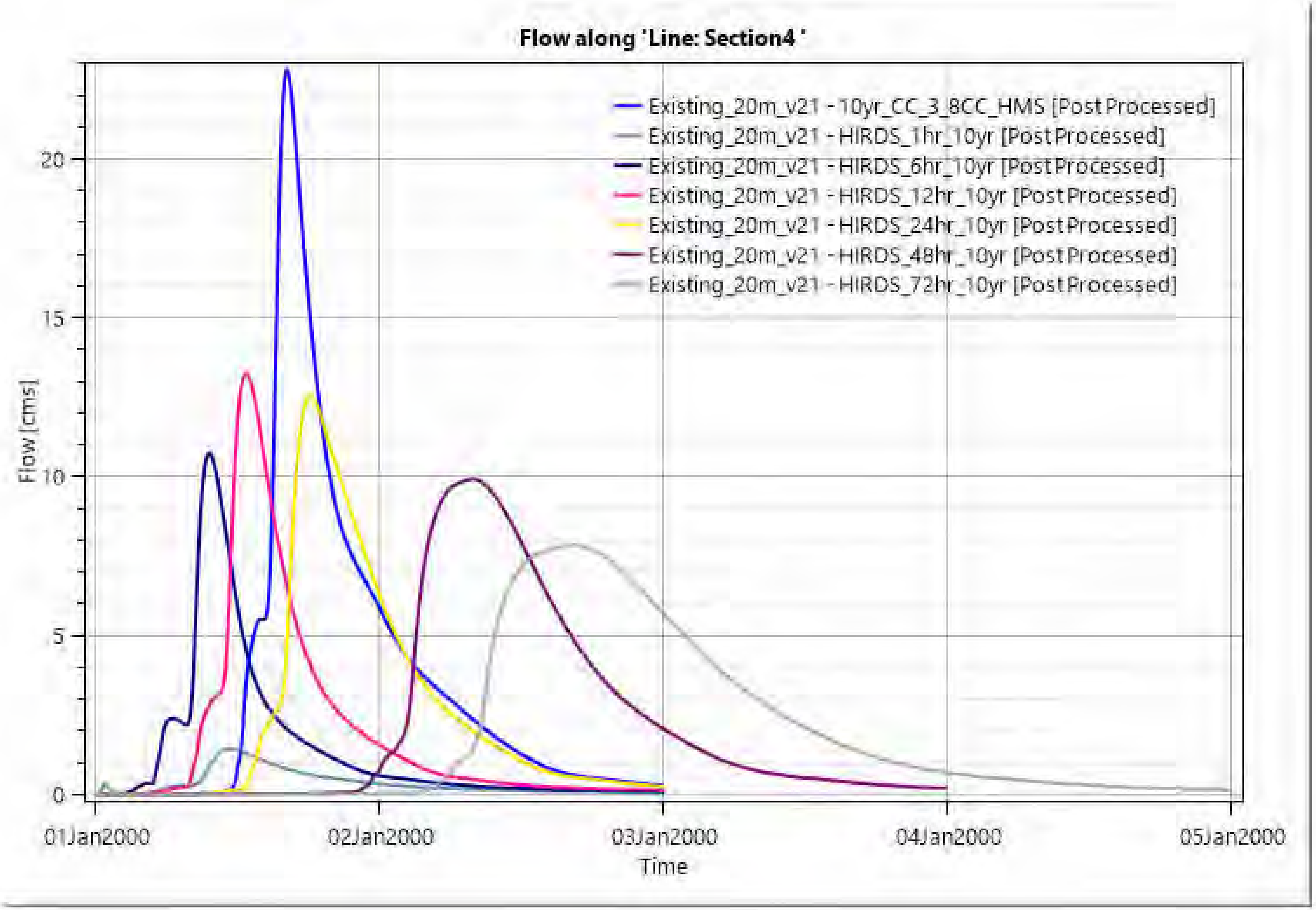
Critical Storm Check – 10%AEP Cross section 2



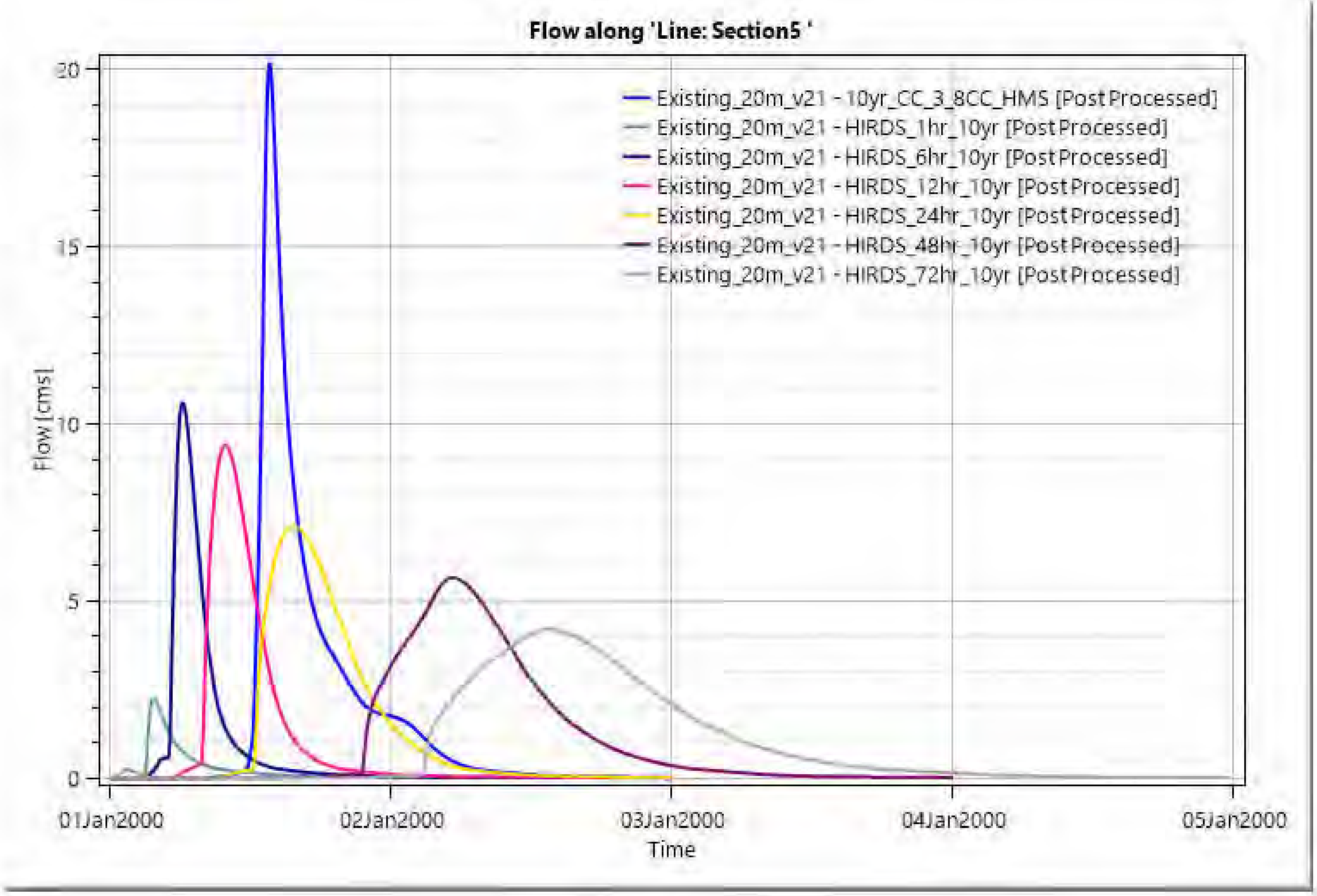
Critical Storm Check – 10%AEP Cross section 3



Critical Storm Check – 10%AEP Cross section 4



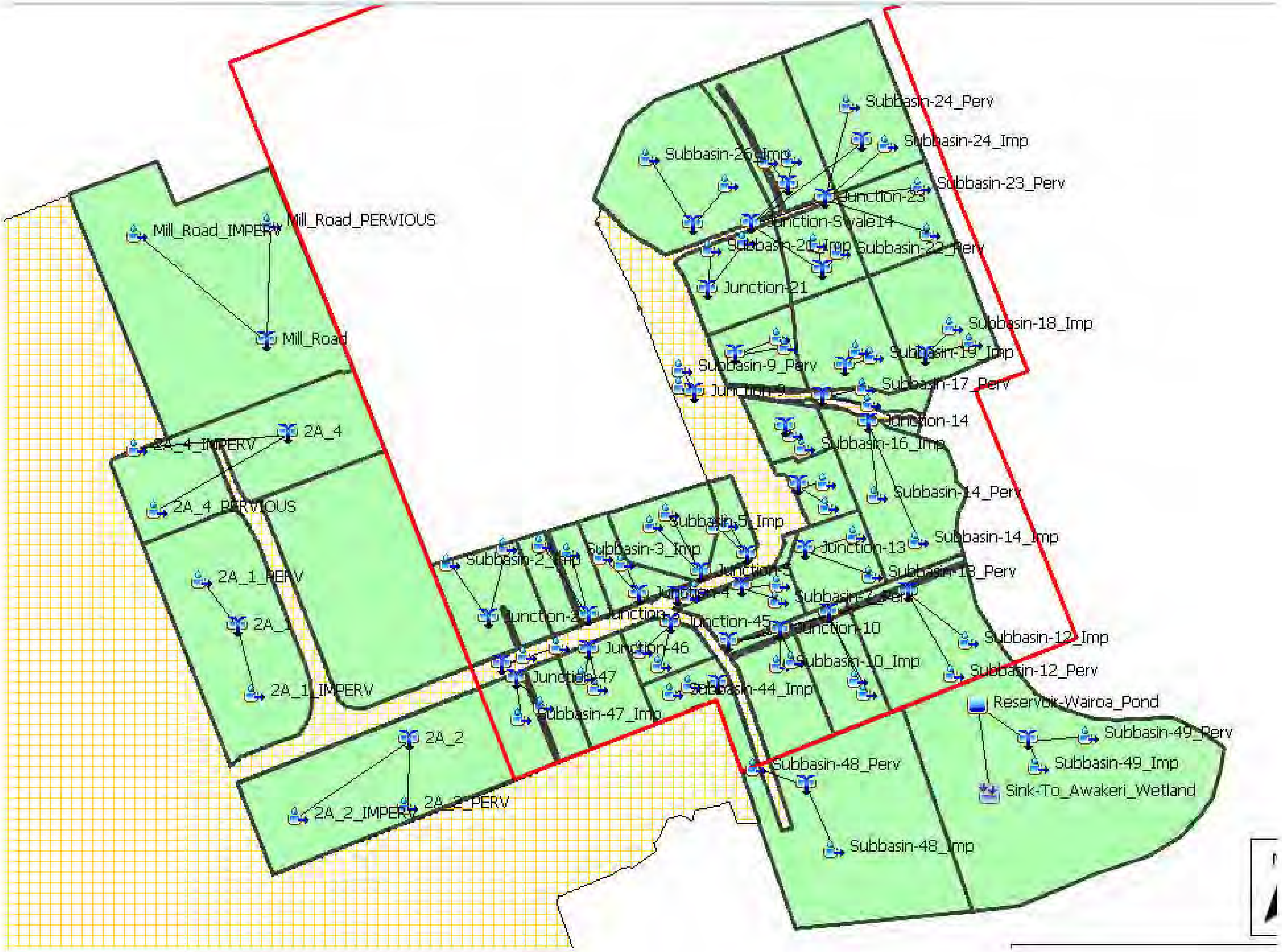
Critical Storm Check – 10%AEP Cross section 5



APPENDIX 3 – HMS Western Model Setup

Western Catchment HEC HMS Model (Baseline)

Western Catchment HEC HMS Model (Post Development)



Western Post Development HEC HMS Paired Data

Cabra Pond / Old Wairoa Road Pond

Wairoa Pond

Components Compute Results

Paired Data Table Graph

Elevation (M)	Storage (1000 M3)
26.16	0.000
28.14	9.919
29.00	15.700

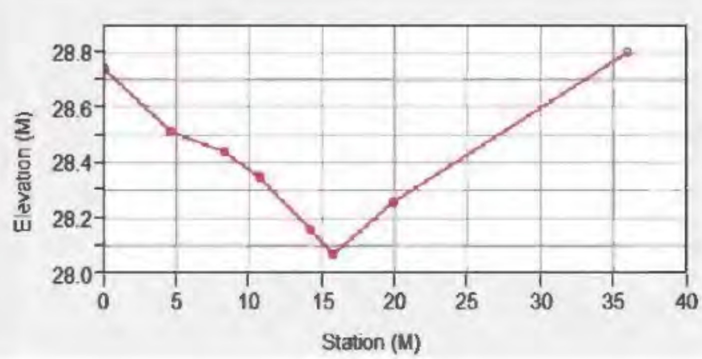
Pond Outlet (derived from Geomaps pipes)

Basin Name: Western_Catchment_Report
Element Name: Reservoir-Wairoa_Pond

Method:	Culvert Outlet
Direction:	Main
Number Barrels:	1
Solution Method:	Automatic
Shape:	Circular
Chart:	1: Concrete Pipe Culvert
Scale:	1: Square edge entrance with headwall
*Length (M)	17
*Diameter (M)	1.2
*Inlet Elevation (M)	26.16
*Entrance Coefficient:	0.3
*Outlet Elevation (M)	25.93
*Exit Coefficient:	0.5
*Mannings n:	0.013

Pond spillway surveys

Select Table Graph



Basin Name: Western_Catchment_Report
Element Name: Reservoir-Wairoa_Pond

Method:	Non-Level Overflow
Direction:	Auxiliary
*Cross Section:	Wairoa Rd Pond Spillway
*Coefficient: (M^0.5/S)	1.66

APPENDIX 4 – HMS Western Model Results

Western Catchment Baseline Scenario– HMS Inflow hydrograph summary 2yr Storm

Global Summary Results for Run "FAB_2yr_GHD 2.1CC"

Project: FAB_Swale_Sizing Simulation Run: FAB_2yr_GHD 2.1CC

Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

Basin Model: FAB_GHD_Model 10yr
Meteorologic Model: TP108_2yr_86mm
Control Specifications: 48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Box culvert entry	1.58733	11.62078	1 January 2000, 12:26	94.12139
CH0-160	1.58733	11.62078	1 January 2000, 12:26	94.12139
CH0-300A	0.56741	4.80572	1 January 2000, 12:24	33.05414
CH1400-CH1540	0.15052	1.62753	1 January 2000, 12:17	9.11195
CH160-550	0.90522	6.23702	1 January 2000, 12:27	54.02930
CH300A-550A	0.31504	2.53231	1 January 2000, 12:23	17.05075
CH550-950	0.73381	4.91369	1 January 2000, 12:23	43.65282
CH950-1400	0.36897	2.36067	1 January 2000, 12:27	21.65764
Junction-1	0.31504	2.53231	1 January 2000, 12:19	17.05075
Junction-2	0.56741	4.80572	1 January 2000, 12:18	33.05414
Junction-3	0.90522	6.23702	1 January 2000, 12:21	54.02982
Junction-4	0.73381	4.91369	1 January 2000, 12:17	43.65334
Junction-5	0.36897	2.36067	1 January 2000, 12:18	21.65841
Junction-6	0.15052	1.62753	1 January 2000, 12:13	9.11195
Main_Branch_Junction	1.58733	11.62078	1 January 2000, 12:25	94.12148
Mill_Road	0.21322	1.96337	1 January 2000, 12:19	13.52009
Mill_Road_IMPERV	0.14924	1.74389	1 January 2000, 12:18	11.30188
Mill_Road_PERVIOUS	0.06398	0.27944	1 January 2000, 12:30	2.21821
Reservoir-Wairoa_Pond	0.21846	0.93895	1 January 2000, 12:34	12.54647
2A_1	0.25237	2.67835	1 January 2000, 12:15	16.00339
2A_1_IMPERV	0.17666	2.34818	1 January 2000, 12:15	13.37838
2A_1_PERV	0.07571	0.39046	1 January 2000, 12:22	2.62501
2A_2	0.11470	1.32992	1 January 2000, 12:12	7.03804
2A_2_IMPERV	0.07456	1.10953	1 January 2000, 12:12	5.64612
2A_2_PERV	0.04015	0.24798	1 January 2000, 12:16	1.39191
2A_4	0.10182	0.56893	1 January 2000, 12:19	3.53066
2A_4_IMPERV	0.00001	0.00014	1 January 2000, 12:13	0.00076
2A_4_PERVIOUS	0.10181	0.56883	1 January 2000, 12:19	3.52990
2B_1	0.15052	1.62753	1 January 2000, 12:13	9.11195
2B_1_IMPERV	0.09483	1.35121	1 January 2000, 12:13	7.18106
2B_1_PERV	0.05569	0.31900	1 January 2000, 12:18	1.93089
2B_2_IMPERV	0.12450	1.70871	1 January 2000, 12:14	9.42833
2B_2_PERV	0.09395	0.50632	1 January 2000, 12:20	3.25756
2B4_1	0.07383	0.79344	1 January 2000, 12:13	4.37837
2B4_1_IMPERV	0.04430	0.64129	1 January 2000, 12:12	3.35450
2B4_1_PERV	0.02953	0.17299	1 January 2000, 12:18	1.02387
2B4_2	0.29101	2.83900	1 January 2000, 12:16	17.61734
2B4_2_IMPERV	0.18334	2.39260	1 January 2000, 12:15	13.88426
2B4_2_PERV	0.10767	0.54096	1 January 2000, 12:23	3.73308
2B4_3	0.17141	1.78671	1 January 2000, 12:14	10.37700
2B4_3_IMPERV	0.10799	1.49037	1 January 2000, 12:14	8.17804
2B4_3_PERV	0.06342	0.34492	1 January 2000, 12:20	2.19897

Western Catchment Baseline Scenario– HMS Inflow hydrograph summary 10yr Storm

Global Summary Results for Run "FAB_10yr_GHD 2.1CC"

Project: FAB_Swale_Sizing Simulation Run: FAB_10yr_GHD 2.1CC

Start of Run: 01Jan2000, 00:00 Basin Model: FAB_GHD_Model 10yr
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_10yr_164mm_CoPv4
Compute Time: DATA CHANGED, RECOMPUTE Control Specifications: 48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Box culvert entry	1.58733	26.54177	1 January 2000, 12:26	216.09257
CH0-160	1.58733	26.54177	1 January 2000, 12:26	216.09257
CH0-300A	0.56741	10.95746	1 January 2000, 12:25	76.40574
CH1400-CH1540	0.15052	3.62576	1 January 2000, 12:17	20.74637
CH160-550	0.90522	14.30345	1 January 2000, 12:27	123.74588
CH300A-550A	0.31504	5.98571	1 January 2000, 12:23	40.61083
CH550-950	0.73381	11.42769	1 January 2000, 12:28	100.11977
CH950-1400	0.36897	5.78616	1 January 2000, 12:27	49.95947
Junction-1	0.31504	5.98571	1 January 2000, 12:19	40.61083
Junction-2	0.56741	10.95746	1 January 2000, 12:19	76.40574
Junction-3	0.90522	14.30345	1 January 2000, 12:21	123.74646
Junction-4	0.73381	11.42769	1 January 2000, 12:22	100.12035
Junction-5	0.36897	5.78616	1 January 2000, 12:18	49.96033
Junction-6	0.15052	3.62576	1 January 2000, 12:13	20.74637
Main_Branch_Junction	1.58733	26.54177	1 January 2000, 12:25	216.09267
Mill_Road	0.21322	4.27459	1 January 2000, 12:19	30.24095
Mill_Road_IMPERV	0.14924	3.58658	1 January 2000, 12:18	23.72545
Mill_Road_PERVIOUS	0.06398	0.84338	1 January 2000, 12:29	6.51550
Reservoir-Wairoa_Pond	0.21846	2.52027	1 January 2000, 12:31	29.21396
2A_1	0.25237	5.84658	1 January 2000, 12:15	35.79491
2A_1_IMPERV	0.17666	4.82680	1 January 2000, 12:15	28.08455
2A_1_PERV	0.07571	1.17566	1 January 2000, 12:21	7.71036
2A_2	0.11470	2.95334	1 January 2000, 12:12	15.94105
2A_2_IMPERV	0.07456	2.27986	1 January 2000, 12:12	11.85261
2A_2_PERV	0.04015	0.74435	1 January 2000, 12:16	4.08843
2A_4	0.10182	1.71112	1 January 2000, 12:19	10.36988
2A_4_IMPERV	0.00001	0.00029	1 January 2000, 12:13	0.00159
2A_4_PERVIOUS	0.10181	1.71091	1 January 2000, 12:19	10.36829
2B_1	0.15052	3.62576	1 January 2000, 12:13	20.74637
2B_1_IMPERV	0.09483	2.77702	1 January 2000, 12:13	15.07482
2B_1_PERV	0.05569	0.95965	1 January 2000, 12:18	5.67155
2B_2_IMPERV	0.12450	3.51219	1 January 2000, 12:14	19.79241
2B_2_PERV	0.09395	1.52491	1 January 2000, 12:20	9.56834
2B4_1	0.07383	1.78512	1 January 2000, 12:13	10.04931
2B4_1_IMPERV	0.04430	1.31814	1 January 2000, 12:12	7.04193
2B4_1_PERV	0.02953	0.52026	1 January 2000, 12:17	3.00738
2B4_2	0.29101	6.31356	1 January 2000, 12:16	40.11158
2B4_2_IMPERV	0.18334	4.91907	1 January 2000, 12:15	29.14650
2B4_2_PERV	0.10767	1.62959	1 January 2000, 12:22	10.96507
2B4_3	0.17141	3.97569	1 January 2000, 12:14	23.62669
2B4_3_IMPERV	0.10799	3.06318	1 January 2000, 12:14	17.16773
2B4_3_PERV	0.06342	1.03764	1 January 2000, 12:20	6.45896

Western Catchment Baseline Scenario– HMS Inflow hydrograph summary 100yr Storm

Global Summary Results for Run "FAB_100yr_GHD 3.8CC"

Project: FAB_Swale_Sizing Simulation Run: FAB_100yr_GHD 3.8CC

Start of Run: 01Jan2000, 00:00 Basin Model: FAB_GHD_Model
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_100yr_292mm_CoPv4
Compute Time: 22Jan2025, 19:29:58 Control Specifications: Control 1

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Box culvert entry	1.58733	49.54181	1 January 2000, 12:30	412.30481
CH0-160	1.58733	49.54181	1 January 2000, 12:30	412.30481
CH0-300A	0.56741	19.98902	1 January 2000, 12:28	146.37587
CH1400-CH1540	0.15052	6.20300	1 January 2000, 12:21	39.39808
CH160-550	0.90522	26.69835	1 January 2000, 12:31	235.74940
CH300A-550A	0.31504	10.90382	1 January 2000, 12:27	79.11595
CH550-950	0.73381	21.75444	1 January 2000, 12:36	190.88209
CH950-1400	0.36897	11.98051	1 January 2000, 12:34	95.53585
Junction-1	0.31504	10.90382	1 January 2000, 12:23	79.11595
Junction-2	0.56741	19.98902	1 January 2000, 12:22	146.37587
Junction-3	0.90522	26.69835	1 January 2000, 12:25	235.75000
Junction-4	0.73381	21.75444	1 January 2000, 12:30	190.88270
Junction-5	0.36897	11.98051	1 January 2000, 12:25	95.53674
Junction-6	0.15052	6.20300	1 January 2000, 12:17	39.39808
Main_Branch_Junction	1.58733	49.54181	1 January 2000, 12:29	412.30491
Mill_Road	0.21322	7.41910	1 January 2000, 12:24	56.82434
Mill_Road_IMPERV	0.14924	5.98128	1 January 2000, 12:22	42.81796
Mill_Road_PERVIOUS	0.06398	1.70759	1 January 2000, 12:35	14.00637
Reservoir-Wairoa_Pond	0.21846	6.35935	1 January 2000, 12:28	56.13867
2A_1	0.25237	10.29439	1 January 2000, 12:18	67.25992
2A_1_IMPERV	0.17666	8.19521	1 January 2000, 12:17	50.68495
2A_1_PERV	0.07571	2.40340	1 January 2000, 12:25	16.57498
2A_2	0.11470	5.20410	1 January 2000, 12:16	30.17965
2A_2_IMPERV	0.07456	3.71799	1 January 2000, 12:15	21.39073
2A_2_PERV	0.04015	1.53469	1 January 2000, 12:18	8.78891
2A_4	0.10182	3.50912	1 January 2000, 12:22	22.29161
2A_4_IMPERV	0.00001	0.00046	1 January 2000, 12:18	0.00287
2A_4_PERVIOUS	0.10181	3.50870	1 January 2000, 12:22	22.28874
2B_1	0.15052	6.20300	1 January 2000, 12:17	39.39808
2B_1_IMPERV	0.09483	4.56484	1 January 2000, 12:16	27.20593
2B_1_PERV	0.05569	1.84820	1 January 2000, 12:23	12.19214
2B_2_IMPERV	0.12450	6.20859	1 January 2000, 12:15	35.71989
2B_2_PERV	0.09395	3.31235	1 January 2000, 12:21	20.56909
2B4_1	0.07383	3.25448	1 January 2000, 12:15	19.17372
2B4_1_IMPERV	0.04430	2.28353	1 January 2000, 12:14	12.70876
2B4_1_PERV	0.02953	1.07100	1 January 2000, 12:20	6.46496
2B4_2	0.29101	11.22664	1 January 2000, 12:19	76.17312
2B4_2_IMPERV	0.18334	8.31998	1 January 2000, 12:18	52.60148
2B4_2_PERV	0.10767	3.32846	1 January 2000, 12:26	23.57164
2B4_3	0.17141	7.11826	1 January 2000, 12:17	44.86791
2B4_3_IMPERV	0.10799	5.22982	1 January 2000, 12:16	30.98306
2B4_3_PERV	0.06342	2.12316	1 January 2000, 12:23	13.88485

Western Catchment Proposed Scenario– HMS Inflow hydrograph summary 2yr Storm

Global Summary Results for Run "2yr_FAB v2"

Project: FAB_Swale_Sizing   Simulation Run: 2yr_FAB v2

Start of Run: 01Jan2000, 00:00   Basin Model: 2yr_Pr v2
End of Run: 03Jan2000, 00:00   Meteorologic Model: TP108_2yr_86mm
Compute Time: 31Jan2025, 15:23:04   Control Specifications: 48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Junction-Swale 13	0.108	1.326	1 January 2000, 12:13	7.299
Junction-Swale14	0.177	2.342	1 January 2000, 12:13	12.682
Junction-Swale8	0.136	1.363	1 January 2000, 12:15	8.154
Junction-10	0.036	0.374	1 January 2000, 12:14	2.193
Junction-11	0.032	0.338	1 January 2000, 12:14	1.943
Junction-12	0.067	0.657	1 January 2000, 12:15	4.019
Junction-13	0.030	0.336	1 January 2000, 12:12	1.786
Junction-14	0.067	0.697	1 January 2000, 12:14	4.049
Junction-15	0.017	0.200	1 January 2000, 12:11	1.009
Junction-16	0.018	0.211	1 January 2000, 12:11	1.066
Junction-17	0.012	0.071	1 January 2000, 12:20	0.482
Junction-18	0.051	0.672	1 January 2000, 12:13	3.680
Junction-19	0.044	0.595	1 January 2000, 12:12	3.137
Junction-2	0.041	0.441	1 January 2000, 12:13	2.477
Junction-20	0.026	0.384	1 January 2000, 12:12	1.938
Junction-21	0.037	0.555	1 January 2000, 12:12	2.835
Junction-22	0.028	0.388	1 January 2000, 12:12	1.977
Junction-23	0.103	1.361	1 January 2000, 12:13	7.376
Junction-24	0.000	0.000	31 December 1999, 24:00	0.000
Junction-25	0.046	0.601	1 January 2000, 12:14	3.328
Junction-26	0.085	1.206	1 January 2000, 12:13	6.474
Junction-3	0.027	0.300	1 January 2000, 12:13	1.652
Junction-4	0.020	0.222	1 January 2000, 12:12	1.174
Junction-43	0.001	0.007	1 January 2000, 12:24	0.056
Junction-44	0.022	0.250	1 January 2000, 12:12	1.295
Junction-45	0.017	0.268	1 January 2000, 12:11	1.311
Junction-46	0.031	0.338	1 January 2000, 12:13	1.842
Junction-47	0.035	0.390	1 January 2000, 12:13	2.125
Junction-48	0.149	1.666	1 January 2000, 12:13	9.269
Junction-49	0.220	2.207	1 January 2000, 12:14	13.008
Junction-5	0.033	0.364	1 January 2000, 12:13	2.000
Junction-6	0.014	0.158	1 January 2000, 12:12	0.814
Junction-7	0.024	0.269	1 January 2000, 12:12	1.435
Junction-8	0.004	0.032	1 January 2000, 12:15	0.194
Junction-9	0.000	0.002	1 January 2000, 12:25	0.017
Reservoir-Wairoa_Pond	0.220	0.973	1 January 2000, 12:34	12.869
Sink-To_Awakeri_Wetland	0.220	0.973	1 January 2000, 12:34	12.869
Subbasin-10_Imp	0.023	0.310	1 January 2000, 12:14	1.712
Subbasin-10_Perv	0.014	0.075	1 January 2000, 12:20	0.481
Subbasin-11_Imp	0.020	0.280	1 January 2000, 12:13	1.517
Subbasin-11_Perv	0.012	0.068	1 January 2000, 12:19	0.426
Subbasin-12_Imp	0.042	0.549	1 January 2000, 12:15	3.145

Project: FAB_Swale_Sizing Simulation Run: 2yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 2yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_2yr_86mm
Compute Time:31Jan2025, 15:23:04 Control Specifications:48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-12_Imp	0.042	0.549	1 January 2000, 12:15	3.145
Subbasin-12_Perv	0.025	0.129	1 January 2000, 12:22	0.874
Subbasin-13_Imp	0.018	0.273	1 January 2000, 12:12	1.389
Subbasin-13_Perv	0.011	0.071	1 January 2000, 12:16	0.397
Subbasin-14_Imp	0.042	0.576	1 January 2000, 12:14	3.162
Subbasin-14_Perv	0.026	0.140	1 January 2000, 12:20	0.887
Subbasin-15_Imp	0.010	0.161	1 January 2000, 12:11	0.788
Subbasin-15_Perv	0.006	0.043	1 January 2000, 12:14	0.221
Subbasin-16_Imp	0.011	0.170	1 January 2000, 12:11	0.832
Subbasin-16_Perv	0.007	0.045	1 January 2000, 12:14	0.234
Subbasin-17_Imp	0.001	0.017	1 January 2000, 12:15	0.094
Subbasin-17_Perv	0.011	0.058	1 January 2000, 12:22	0.388
Subbasin-18_Imp	0.046	0.648	1 January 2000, 12:13	3.502
Subbasin-18_Perv	0.005	0.029	1 January 2000, 12:19	0.178
Subbasin-19_Imp	0.039	0.573	1 January 2000, 12:12	2.986
Subbasin-19_Perv	0.004	0.026	1 January 2000, 12:17	0.152
Subbasin-2_Imp	0.026	0.364	1 January 2000, 12:13	1.934
Subbasin-2_Perv	0.016	0.089	1 January 2000, 12:18	0.543
Subbasin-20_Imp	0.026	0.384	1 January 2000, 12:12	1.937
Subbasin-20_Perv	0.000	0.000	1 January 2000, 12:16	0.000
Subbasin-21_Imp	0.037	0.555	1 January 2000, 12:12	2.834
Subbasin-21_Perv	0.000	0.000	1 January 2000, 12:16	0.000
Subbasin-22_Imp	0.025	0.373	1 January 2000, 12:12	1.882
Subbasin-22_Perv	0.003	0.017	1 January 2000, 12:16	0.096
Subbasin-23_Imp	0.034	0.508	1 January 2000, 12:12	2.585
Subbasin-23_Perv	0.004	0.023	1 January 2000, 12:16	0.132
Subbasin-24_Imp	0.059	0.811	1 January 2000, 12:13	4.434
Subbasin-24_Perv	0.007	0.036	1 January 2000, 12:20	0.226
Subbasin-25_Imp	0.042	0.579	1 January 2000, 12:13	3.167
Subbasin-25_Perv	0.005	0.025	1 January 2000, 12:20	0.161
Subbasin-26_Imp	0.085	1.206	1 January 2000, 12:13	6.474
Subbasin-26_Perv	0.000	0.000	1 January 2000, 12:19	0.000
Subbasin-3_Imp	0.017	0.247	1 January 2000, 12:12	1.290
Subbasin-3_Perv	0.010	0.061	1 January 2000, 12:18	0.362
Subbasin-4_Imp	0.012	0.181	1 January 2000, 12:12	0.917
Subbasin-4_Perv	0.007	0.046	1 January 2000, 12:16	0.257
Subbasin-43_Imp	0.000	0.004	1 January 2000, 12:20	0.027
Subbasin-43_Perv	0.001	0.003	1 January 2000, 12:33	0.029
Subbasin-44_Imp	0.013	0.203	1 January 2000, 12:11	1.011
Subbasin-44_Perv	0.008	0.053	1 January 2000, 12:15	0.284
Subbasin-45_Imp	0.017	0.268	1 January 2000, 12:11	1.310
Subbasin-45_Perv	0.000	0.000	1 January 2000, 12:14	0.000

Global Summary Results for Run "2yr_FAB v2"

Project: FAB_Swale_Sizing Simulation Run: 2yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 2yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_2yr_86mm
Compute Time:31Jan2025, 15:23:04 Control Specifications:48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-44_Perv	0.008	0.053	1 January 2000, 12:15	0.284
Subbasin-45_Imp	0.017	0.268	1 January 2000, 12:11	1.310
Subbasin-45_Perv	0.000	0.000	1 January 2000, 12:14	0.000
Subbasin-46_Imp	0.019	0.277	1 January 2000, 12:12	1.438
Subbasin-46_Perv	0.012	0.069	1 January 2000, 12:17	0.404
Subbasin-47_Imp	0.022	0.320	1 January 2000, 12:12	1.659
Subbasin-47_Perv	0.013	0.080	1 January 2000, 12:17	0.466
Subbasin-48_Imp	0.100	1.419	1 January 2000, 12:13	7.543
Subbasin-48_Perv	0.050	0.285	1 January 2000, 12:18	1.727
Subbasin-49_Imp	0.131	1.802	1 January 2000, 12:14	9.943
Subbasin-49_Perv	0.088	0.476	1 January 2000, 12:20	3.065
Subbasin-5_Imp	0.021	0.299	1 January 2000, 12:12	1.562
Subbasin-5_Perv	0.013	0.074	1 January 2000, 12:17	0.438
Subbasin-6_Imp	0.008	0.128	1 January 2000, 12:11	0.636
Subbasin-6_Perv	0.005	0.033	1 January 2000, 12:15	0.178
Subbasin-7_Imp	0.015	0.219	1 January 2000, 12:12	1.120
Subbasin-7_Perv	0.009	0.056	1 January 2000, 12:16	0.314
Subbasin-8_Imp	0.001	0.017	1 January 2000, 12:13	0.094
Subbasin-8_Perv	0.003	0.016	1 January 2000, 12:19	0.100
Subbasin-9_Imp	0.000	0.001	1 January 2000, 12:19	0.006
Subbasin-9_Perv	0.000	0.001	1 January 2000, 12:31	0.011

Western Catchment Proposed Scenario – HMS Inflow hydrograph summary 10yr Storm

Global Summary Results for Run "10yr_FAB v2"

Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 10yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_10yr_164mm_CoPv4
Compute Time: 31Jan2025, 16:29:51 Control Specifications: 48hr

Show Elements: All Elements Volume Units: MM 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Central_Pond	0.519	1.704	1 January 2000, 13:19	73.241
Junction-Swale 13	0.108	2.820	1 January 2000, 12:13	15.923
Junction-Swale14	0.177	4.902	1 January 2000, 12:13	27.137
Junction-Swale8	0.136	3.043	1 January 2000, 12:15	18.610
Junction-10	0.036	0.834	1 January 2000, 12:14	5.006
Junction-11	0.032	0.755	1 January 2000, 12:14	4.435
Junction-12	0.067	1.462	1 January 2000, 12:15	9.168
Junction-13	0.030	0.754	1 January 2000, 12:12	4.081
Junction-14	0.067	1.555	1 January 2000, 12:14	9.244
Junction-15	0.017	0.450	1 January 2000, 12:11	2.303
Junction-16	0.018	0.473	1 January 2000, 12:11	2.433
Junction-17	0.012	0.201	1 January 2000, 12:20	1.336
Junction-18	0.051	1.406	1 January 2000, 12:13	7.874
Junction-19	0.044	1.245	1 January 2000, 12:12	6.714
Junction-2	0.041	0.984	1 January 2000, 12:13	5.654
Junction-20	0.026	0.789	1 January 2000, 12:12	4.068
Junction-21	0.037	1.141	1 January 2000, 12:12	5.951
Junction-22	0.028	0.814	1 January 2000, 12:12	4.231
Junction-23	0.103	2.850	1 January 2000, 12:13	15.784
Junction-24	0.000	0.000	31 December 1999, 24:00	0.000
Junction-25	0.046	1.257	1 January 2000, 12:14	7.122
Junction-26	0.085	2.480	1 January 2000, 12:13	13.591
Junction-3	0.027	0.672	1 January 2000, 12:13	3.771
Junction-4	0.020	0.497	1 January 2000, 12:12	2.680
Junction-43	0.001	0.017	1 January 2000, 12:25	0.142
Junction-44	0.022	0.562	1 January 2000, 12:12	2.957
Junction-45	0.017	0.550	1 January 2000, 12:11	2.752
Junction-46	0.031	0.756	1 January 2000, 12:13	4.204
Junction-47	0.035	0.872	1 January 2000, 12:13	4.851
Junction-48	0.149	3.676	1 January 2000, 12:13	20.906
Junction-49	0.220	4.953	1 January 2000, 12:14	29.876
Junction-5	0.033	0.815	1 January 2000, 12:13	4.566
Junction-6	0.014	0.355	1 January 2000, 12:12	1.859
Junction-7	0.024	0.602	1 January 2000, 12:12	3.275
Junction-8	0.004	0.080	1 January 2000, 12:15	0.490
Junction-9	0.000	0.006	1 January 2000, 12:26	0.045
Reservoir-Wairoa_Pond	0.220	2.562	1 January 2000, 12:31	29.729
Sink-To_Awakeri_Wetland	0.220	2.562	1 January 2000, 12:31	29.729
Sink-1	0.519	1.704	1 January 2000, 13:19	73.241
Subbasin-10_Imp	0.023	0.638	1 January 2000, 12:14	3.595
Subbasin-10_Perv	0.014	0.225	1 January 2000, 12:20	1.411

Global Summary Results for Run "10yr_FAB v2"

Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 10yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_10yr_164mm_CoPv4
Compute Time: 31Jan2025, 16:29:51 Control Specifications: 48hr

Show Elements: Elements Volume Units: MM 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-10_Imp	0.023	0.638	1 January 2000, 12:14	3.595
Subbasin-10_Perv	0.014	0.225	1 January 2000, 12:20	1.411
Subbasin-11_Imp	0.020	0.575	1 January 2000, 12:13	3.184
Subbasin-11_Perv	0.012	0.205	1 January 2000, 12:19	1.250
Subbasin-12_Imp	0.042	1.128	1 January 2000, 12:15	6.602
Subbasin-12_Perv	0.025	0.389	1 January 2000, 12:22	2.567
Subbasin-13_Imp	0.018	0.561	1 January 2000, 12:12	2.915
Subbasin-13_Perv	0.011	0.213	1 January 2000, 12:16	1.166
Subbasin-14_Imp	0.042	1.184	1 January 2000, 12:14	6.637
Subbasin-14_Perv	0.026	0.420	1 January 2000, 12:19	2.606
Subbasin-15_Imp	0.010	0.331	1 January 2000, 12:11	1.654
Subbasin-15_Perv	0.006	0.128	1 January 2000, 12:14	0.649
Subbasin-16_Imp	0.011	0.349	1 January 2000, 12:11	1.747
Subbasin-16_Perv	0.007	0.134	1 January 2000, 12:14	0.686
Subbasin-17_Imp	0.001	0.034	1 January 2000, 12:15	0.197
Subbasin-17_Perv	0.011	0.174	1 January 2000, 12:21	1.138
Subbasin-18_Imp	0.046	1.332	1 January 2000, 12:13	7.351
Subbasin-18_Perv	0.005	0.086	1 January 2000, 12:19	0.523
Subbasin-19_Imp	0.039	1.178	1 January 2000, 12:12	6.267
Subbasin-19_Perv	0.004	0.078	1 January 2000, 12:17	0.446
Subbasin-2_Imp	0.026	0.748	1 January 2000, 12:13	4.060
Subbasin-2_Perv	0.016	0.268	1 January 2000, 12:18	1.594
Subbasin-20_Imp	0.026	0.788	1 January 2000, 12:12	4.067
Subbasin-20_Perv	0.000	0.000	1 January 2000, 12:15	0.001
Subbasin-21_Imp	0.037	1.141	1 January 2000, 12:12	5.950
Subbasin-21_Perv	0.000	0.000	1 January 2000, 12:16	0.001
Subbasin-22_Imp	0.025	0.766	1 January 2000, 12:12	3.950
Subbasin-22_Perv	0.003	0.052	1 January 2000, 12:15	0.281
Subbasin-23_Imp	0.034	1.044	1 January 2000, 12:12	5.427
Subbasin-23_Perv	0.004	0.070	1 January 2000, 12:16	0.386
Subbasin-24_Imp	0.059	1.667	1 January 2000, 12:13	9.308
Subbasin-24_Perv	0.007	0.107	1 January 2000, 12:19	0.663
Subbasin-25_Imp	0.042	1.191	1 January 2000, 12:13	6.649
Subbasin-25_Perv	0.005	0.076	1 January 2000, 12:19	0.473
Subbasin-26_Imp	0.085	2.480	1 January 2000, 12:13	13.590
Subbasin-26_Perv	0.000	0.000	1 January 2000, 12:19	0.001
Subbasin-3_Imp	0.017	0.507	1 January 2000, 12:12	2.708
Subbasin-3_Perv	0.010	0.184	1 January 2000, 12:17	1.063
Subbasin-4_Imp	0.012	0.371	1 January 2000, 12:12	1.924
Subbasin-4_Perv	0.007	0.138	1 January 2000, 12:16	0.756
Subbasin-43_Imp	0.000	0.008	1 January 2000, 12:20	0.057

Global Summary Results for Run "10yr_FAB v2"

Project: FAB_Swale_Sizing   Simulation Run: 10yr_FAB v2

Start of Run: 01Jan2000, 00:00   Basin Model: 10yr_Pr v2
End of Run: 03Jan2000, 00:00   Meteorologic Model: TP108_10yr_164mm_CoPv4
Compute Time: 31Jan2025, 16:29:51   Control Specifications: 48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-43_Imp	0.000	0.008	1 January 2000, 12:20	0.057
Subbasin-43_Perv	0.001	0.010	1 January 2000, 12:32	0.085
Subbasin-44_Imp	0.013	0.416	1 January 2000, 12:11	2.123
Subbasin-44_Perv	0.008	0.158	1 January 2000, 12:15	0.834
Subbasin-45_Imp	0.017	0.550	1 January 2000, 12:11	2.751
Subbasin-45_Perv	0.000	0.000	1 January 2000, 12:14	0.001
Subbasin-46_Imp	0.019	0.569	1 January 2000, 12:12	3.019
Subbasin-46_Perv	0.012	0.208	1 January 2000, 12:17	1.185
Subbasin-47_Imp	0.022	0.657	1 January 2000, 12:12	3.483
Subbasin-47_Perv	0.013	0.240	1 January 2000, 12:17	1.368
Subbasin-48_Imp	0.100	2.917	1 January 2000, 12:13	15.834
Subbasin-48_Perv	0.050	0.858	1 January 2000, 12:18	5.072
Subbasin-49_Imp	0.131	3.704	1 January 2000, 12:14	20.873
Subbasin-49_Perv	0.088	1.435	1 January 2000, 12:20	9.003
Subbasin-5_Imp	0.021	0.614	1 January 2000, 12:12	3.278
Subbasin-5_Perv	0.013	0.223	1 January 2000, 12:17	1.287
Subbasin-6_Imp	0.008	0.262	1 January 2000, 12:11	1.335
Subbasin-6_Perv	0.005	0.100	1 January 2000, 12:15	0.524
Subbasin-7_Imp	0.015	0.451	1 January 2000, 12:12	2.352
Subbasin-7_Perv	0.009	0.168	1 January 2000, 12:16	0.923
Subbasin-8_Imp	0.001	0.036	1 January 2000, 12:13	0.196
Subbasin-8_Perv	0.003	0.049	1 January 2000, 12:19	0.294
Subbasin-9_Imp	0.000	0.002	1 January 2000, 12:19	0.013
Subbasin-9_Perv	0.000	0.004	1 January 2000, 12:31	0.032

Western Catchment Proposed Scenario – HMS Inflow hydrograph summary 100yr Storm

Global Summary Results for Run "100yr_FAB v2"

Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 100yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_100yr_292mm
Compute Time: 31Jan2025, 16:31:06 Control Specifications: 48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Central_Pond	0.535	2.405	1 January 2000, 14:14	133.026
Junction-Swale 13	0.108	4.956	1 January 2000, 12:15	29.464
Junction-Swale14	0.177	8.505	1 January 2000, 12:15	49.598
Junction-Swale8	0.136	5.445	1 January 2000, 12:18	35.389
Junction-10	0.036	1.500	1 January 2000, 12:17	9.522
Junction-11	0.032	1.363	1 January 2000, 12:16	8.435
Junction-12	0.067	2.608	1 January 2000, 12:19	17.432
Junction-13	0.030	1.386	1 January 2000, 12:14	7.768
Junction-14	0.067	2.796	1 January 2000, 12:17	17.581
Junction-15	0.017	0.838	1 January 2000, 12:13	4.380
Junction-16	0.018	0.880	1 January 2000, 12:13	4.628
Junction-17	0.012	0.403	1 January 2000, 12:24	2.803
Junction-18	0.051	2.443	1 January 2000, 12:16	14.391
Junction-19	0.044	2.189	1 January 2000, 12:14	12.270
Junction-2	0.041	1.783	1 January 2000, 12:16	10.755
Junction-20	0.026	1.389	1 January 2000, 12:13	7.342
Junction-21	0.037	1.987	1 January 2000, 12:14	10.740
Junction-22	0.028	1.446	1 January 2000, 12:13	7.733
Junction-23	0.103	4.943	1 January 2000, 12:15	28.848
Junction-24	0.000	0.000	31 December 1999, 24:00	0.000
Junction-25	0.046	2.176	1 January 2000, 12:16	13.016
Junction-26	0.085	4.248	1 January 2000, 12:15	24.528
Junction-3	0.027	1.221	1 January 2000, 12:15	7.173
Junction-4	0.020	0.913	1 January 2000, 12:14	5.097
Junction-43	0.001	0.033	1 January 2000, 12:31	0.286
Junction-44	0.022	1.034	1 January 2000, 12:13	5.624
Junction-45	0.017	0.982	1 January 2000, 12:12	4.967
Junction-46	0.031	1.378	1 January 2000, 12:15	7.996
Junction-47	0.035	1.587	1 January 2000, 12:15	9.226
Junction-48	0.149	6.603	1 January 2000, 12:16	39.478
Junction-49	0.220	8.917	1 January 2000, 12:17	57.024
Junction-5	0.033	1.481	1 January 2000, 12:15	8.684
Junction-6	0.014	0.656	1 January 2000, 12:13	3.535
Junction-7	0.024	1.105	1 January 2000, 12:14	6.229
Junction-8	0.004	0.155	1 January 2000, 12:18	0.986
Junction-9	0.000	0.011	1 January 2000, 12:32	0.093
Reservoir-Wairoa_Pond	0.220	6.253	1 January 2000, 12:30	56.873
Sink-To_Awakeri_Wetland	0.220	6.253	1 January 2000, 12:30	56.873
Sink-1	0.535	2.405	1 January 2000, 14:14	133.026
Subbasin-10_Imp	0.023	1.092	1 January 2000, 12:16	6.488
Subbasin-10_Perv	0.014	0.461	1 January 2000, 12:23	3.034

Global Summary Results for Run "100yr_FAB v2"

Project: FAB_Swale_Sizing   Simulation Run: 100yr_FAB v2

Start of Run: 01Jan2000, 00:00   Basin Model: 100yr_Pr v2
End of Run: 03Jan2000, 00:00   Meteorologic Model: TP108_100yr_292mm
Compute Time: 31Jan2025, 16:31:06   Control Specifications: 48hr

Show Elements: All Elements   Volume Units: MM 1000 M3   Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-10_Imp	0.023	1.092	1 January 2000, 12:16	6.488
Subbasin-10_Perv	0.014	0.461	1 January 2000, 12:23	3.034
Subbasin-11_Imp	0.020	0.988	1 January 2000, 12:15	5.747
Subbasin-11_Perv	0.012	0.420	1 January 2000, 12:22	2.688
Subbasin-12_Imp	0.042	1.911	1 January 2000, 12:17	11.914
Subbasin-12_Perv	0.025	0.793	1 January 2000, 12:26	5.518
Subbasin-13_Imp	0.018	0.985	1 January 2000, 12:13	5.261
Subbasin-13_Perv	0.011	0.439	1 January 2000, 12:18	2.507
Subbasin-14_Imp	0.042	2.028	1 January 2000, 12:16	11.979
Subbasin-14_Perv	0.026	0.861	1 January 2000, 12:23	5.602
Subbasin-15_Imp	0.010	0.590	1 January 2000, 12:12	2.984
Subbasin-15_Perv	0.006	0.266	1 January 2000, 12:16	1.396
Subbasin-16_Imp	0.011	0.621	1 January 2000, 12:12	3.153
Subbasin-16_Perv	0.007	0.279	1 January 2000, 12:16	1.475
Subbasin-17_Imp	0.001	0.058	1 January 2000, 12:17	0.356
Subbasin-17_Perv	0.011	0.356	1 January 2000, 12:25	2.447
Subbasin-18_Imp	0.046	2.289	1 January 2000, 12:15	13.266
Subbasin-18_Perv	0.005	0.176	1 January 2000, 12:22	1.125
Subbasin-19_Imp	0.039	2.048	1 January 2000, 12:14	11.311
Subbasin-19_Perv	0.004	0.160	1 January 2000, 12:20	0.959
Subbasin-2_Imp	0.026	1.287	1 January 2000, 12:15	7.328
Subbasin-2_Perv	0.016	0.551	1 January 2000, 12:21	3.427
Subbasin-20_Imp	0.026	1.389	1 January 2000, 12:13	7.340
Subbasin-20_Perv	0.000	0.000	1 January 2000, 12:18	0.002
Subbasin-21_Imp	0.037	1.987	1 January 2000, 12:14	10.737
Subbasin-21_Perv	0.000	0.000	1 January 2000, 12:18	0.002
Subbasin-22_Imp	0.025	1.349	1 January 2000, 12:13	7.129
Subbasin-22_Perv	0.003	0.108	1 January 2000, 12:17	0.604
Subbasin-23_Imp	0.034	1.827	1 January 2000, 12:13	9.795
Subbasin-23_Perv	0.004	0.145	1 January 2000, 12:18	0.830
Subbasin-24_Imp	0.059	2.853	1 January 2000, 12:16	16.799
Subbasin-24_Perv	0.007	0.219	1 January 2000, 12:23	1.424
Subbasin-25_Imp	0.042	2.038	1 January 2000, 12:16	11.999
Subbasin-25_Perv	0.005	0.157	1 January 2000, 12:23	1.017
Subbasin-26_Imp	0.085	4.247	1 January 2000, 12:15	24.526
Subbasin-26_Perv	0.000	0.000	1 January 2000, 12:22	0.002
Subbasin-3_Imp	0.017	0.878	1 January 2000, 12:14	4.887
Subbasin-3_Perv	0.010	0.379	1 January 2000, 12:20	2.286
Subbasin-4_Imp	0.012	0.653	1 January 2000, 12:13	3.473
Subbasin-4_Perv	0.007	0.285	1 January 2000, 12:18	1.624

Global Summary Results for Run "100yr_FAB v2"

Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 100yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_100yr_292mm
Compute Time:31Jan2025, 16:31:06 Control Specifications:48hr

Show Elements: All Elements Volume Units: MM 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-4_Imp	0.012	0.653	1 January 2000, 12:13	3.473
Subbasin-4_Perv	0.007	0.285	1 January 2000, 12:18	1.624
Subbasin-43_Imp	0.000	0.014	1 January 2000, 12:24	0.103
Subbasin-43_Perv	0.001	0.021	1 January 2000, 12:38	0.183
Subbasin-44_Imp	0.013	0.734	1 January 2000, 12:13	3.832
Subbasin-44_Perv	0.008	0.326	1 January 2000, 12:17	1.792
Subbasin-45_Imp	0.017	0.981	1 January 2000, 12:12	4.964
Subbasin-45_Perv	0.000	0.000	1 January 2000, 12:16	0.002
Subbasin-46_Imp	0.019	0.990	1 January 2000, 12:14	5.448
Subbasin-46_Perv	0.012	0.428	1 January 2000, 12:19	2.548
Subbasin-47_Imp	0.022	1.142	1 January 2000, 12:14	6.286
Subbasin-47_Perv	0.013	0.492	1 January 2000, 12:19	2.940
Subbasin-48_Imp	0.100	5.035	1 January 2000, 12:15	28.576
Subbasin-48_Perv	0.050	1.758	1 January 2000, 12:21	10.902
Subbasin-49_Imp	0.131	6.316	1 January 2000, 12:16	37.671
Subbasin-49_Perv	0.088	2.931	1 January 2000, 12:23	19.353
Subbasin-5_Imp	0.021	1.067	1 January 2000, 12:14	5.917
Subbasin-5_Perv	0.013	0.458	1 January 2000, 12:20	2.767
Subbasin-6_Imp	0.008	0.464	1 January 2000, 12:13	2.408
Subbasin-6_Perv	0.005	0.207	1 January 2000, 12:17	1.126
Subbasin-7_Imp	0.015	0.788	1 January 2000, 12:13	4.244
Subbasin-7_Perv	0.009	0.345	1 January 2000, 12:18	1.985
Subbasin-8_Imp	0.001	0.062	1 January 2000, 12:15	0.355
Subbasin-8_Perv	0.003	0.100	1 January 2000, 12:21	0.631
Subbasin-9_Imp	0.000	0.003	1 January 2000, 12:23	0.023
Subbasin-9_Perv	0.000	0.008	1 January 2000, 12:37	0.070

APPENDIX 5 – HMS Subbasin Parameters

HMS Subbasin Parameters

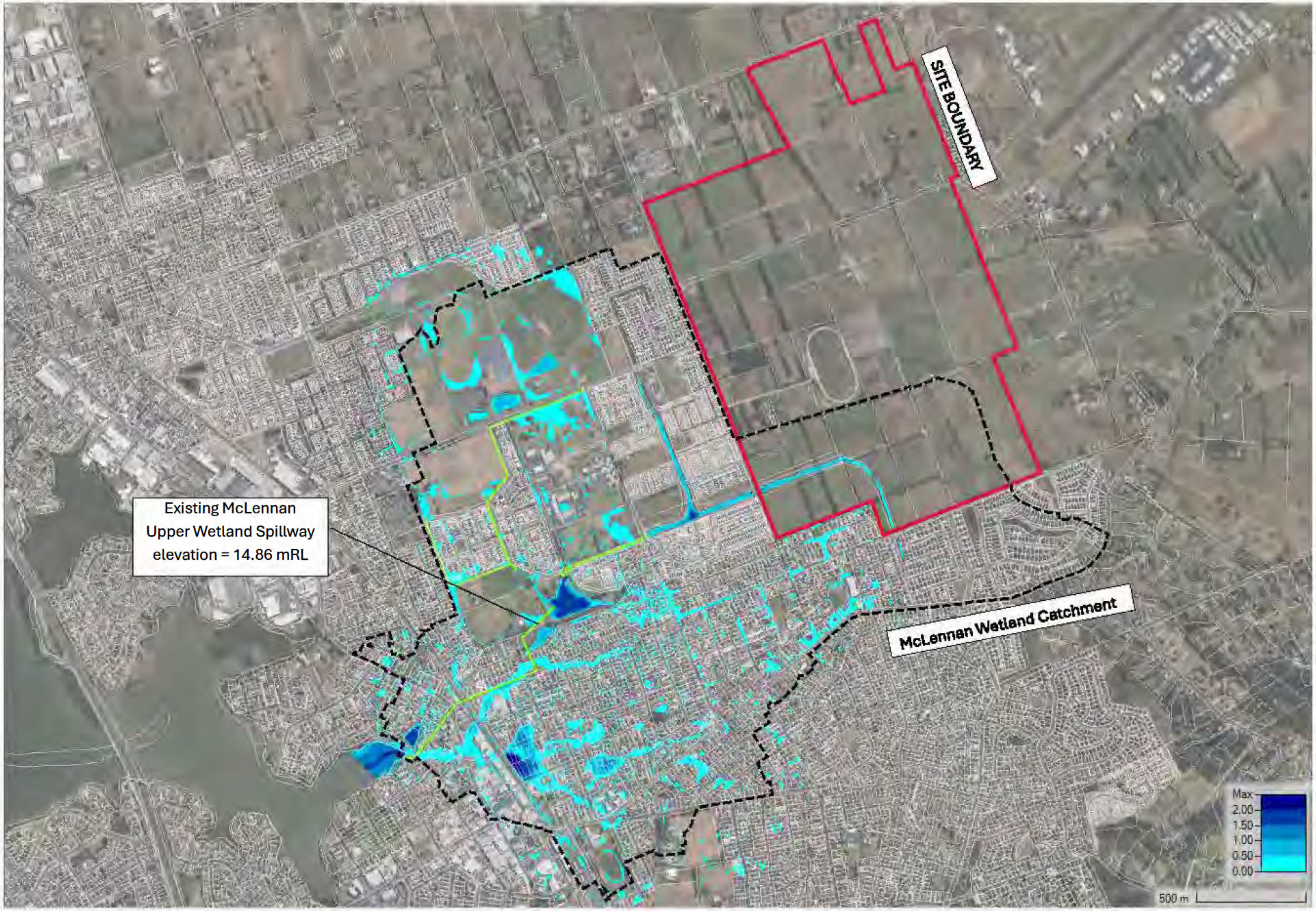
										2& 10yr			100yr
Subbasin Name	**Area KM2**	**Ia**	**CN**	**10 yr Channelisation**	**100 yr Channelisation**	**Slope**	**Length**	**Tc (hr)**	**Tp (hr)**	**Tp (min)**	**Tc (hr)**	**Tp (hr)**	**Tp (min)**
Subbasin-3_Imp	0.017033298	0	98	0.6	0.8	0.005	249	0.17	0.11	6.7	0.22	0.15	9.0
Subbasin-3_Perv	0.010439763	5	74	0.8	1	0.005	249	0.29	0.20	11.8	0.37	0.24	14.7
Subbasin-4_Imp	0.012104485	0	98	0.6	0.8	0.005	199	0.15	0.10	5.8	0.19	0.13	7.7
Subbasin-4_Perv	0.007418878	5	74	0.8	1	0.005	199	0.25	0.17	10.1	0.32	0.21	12.7
Subbasin-5_Imp	0.020622708	0	98	0.6	0.8	0.005	248	0.17	0.11	6.7	0.22	0.15	8.9
Subbasin-5_Perv	0.012639724	5	74	0.8	1	0.005	248	0.29	0.20	11.7	0.37	0.24	14.7
Subbasin-6_Imp	0.008394651	0	98	0.6	0.8	0.005	168	0.13	0.09	5.2	0.17	0.12	6.9
Subbasin-6_Perv	0.005145109	5	74	0.8	1	0.005	168	0.23	0.15	9.1	0.28	0.19	11.3
Subbasin-7_Imp	0.014793379	0	98	0.6	0.8	0.005	208	0.15	0.10	6.0	0.20	0.13	8.0
Subbasin-7_Perv	0.009066909	5	74	0.8	1	0.005	208	0.26	0.17	10.4	0.33	0.22	13.0
Subbasin-8_Imp	0.001235953	0	98	0.6	0.8	0.005	292	0.19	0.12	7.5	0.25	0.17	10.0
Subbasin-8_Perv	0.002883891	5	74	0.8	1	0.005	292	0.33	0.22	13.1	0.41	0.27	16.3
Subbasin-9_Imp	7.95984E-05	0	98	0.6	0.8	0.005	772	0.35	0.24	14.2	0.47	0.32	18.9
Subbasin-9_Perv	0.000318394	5	74	0.8	1	0.005	772	0.62	0.41	24.8	0.78	0.52	31.0
Subbasin-10_Imp	0.022613315	0	98	0.6	0.8	0.005	341	0.21	0.14	8.3	0.28	0.18	11.0
Subbasin-10_Perv	0.013859773	5	74	0.8	1	0.005	341	0.36	0.24	14.5	0.45	0.30	18.1
Subbasin-11_Imp	0.020031255	0	98	0.6	0.8	0.005	309	0.19	0.13	7.8	0.26	0.17	10.3
Subbasin-11_Perv	0.012277221	5	74	0.8	1	0.005	309	0.34	0.23	13.6	0.42	0.28	16.9
Subbasin-12_Imp	0.041525483	0	98	0.6	0.8	0.005	413	0.23	0.16	9.4	0.31	0.21	12.5
Subbasin-12_Perv	0.025205998	5	74	0.8	1	0.005	413	0.41	0.27	16.4	0.51	0.34	20.5
Subbasin-13_Imp	0.018336621	0	98	0.6	0.8	0.005	202	0.15	0.10	5.9	0.20	0.13	7.8
Subbasin-13_Perv	0.011450669	5	74	0.8	1	0.005	202	0.26	0.17	10.2	0.32	0.21	12.8
Subbasin-14_Imp	0.041751729	0	98	0.6	0.8	0.005	328	0.20	0.13	8.1	0.27	0.18	10.8
Subbasin-14_Perv	0.025589769	5	74	0.8	1	0.005	328	0.35	0.23	14.1	0.44	0.29	17.6
Subbasin-15_Imp	0.010402251	0	98	0.6	0.8	0.005	141	0.12	0.08	4.6	0.15	0.10	6.2
Subbasin-15_Perv	0.006375573	5	74	0.8	1	0.005	141	0.20	0.13	8.1	0.25	0.17	10.1
Subbasin-16_Imp	0.010989676	0	98	0.6	0.8	0.005	147	0.12	0.08	4.7	0.16	0.11	6.3
Subbasin-16_Perv	0.006735608	5	74	0.8	1	0.005	147	0.21	0.14	8.3	0.26	0.17	10.4
Subbasin-17_Imp	0.00124184	0	98	0.6	0.8	0.005	398	0.23	0.15	9.2	0.31	0.20	12.2
Subbasin-17_Perv	0.01117656	5	74	0.8	1	0.005	398	0.40	0.27	16.0	0.50	0.33	20.0
Subbasin-18_Imp	0.046238727	0	98	0.6	0.8	0.005	304	0.19	0.13	7.7	0.26	0.17	10.2
Subbasin-18_Perv	0.005137636	5	74	0.8	1	0.005	304	0.34	0.22	13.4	0.42	0.28	16.8
Subbasin-19_Imp	0.03942415	0	98	0.6	0.8	0.005	244	0.17	0.11	6.6	0.22	0.15	8.8
Subbasin-19_Perv	0.004380461	5	74	0.8	1	0.005	244	0.29	0.19	11.6	0.36	0.24	14.5
Subbasin-20_Imp	0.025583108	0	98	0.6	0.8	0.005	190	0.14	0.09	5.6	0.19	0.13	7.5
Subbasin-20_Perv	0.00001	5	74	0.8	1	0.005	190	0.25	0.16	9.8	0.31	0.20	12.3
Subbasin-21_Imp	0.037425015	0	98	0.6	0.8	0.005	212	0.15	0.10	6.0	0.20	0.13	8.1
Subbasin-21_Perv	0.00001	5	74	0.8	1	0.005	212	0.26	0.18	10.6	0.33	0.22	13.2
Subbasin-22_Imp	0.024847836	0	98	0.6	0.8	0.005	189	0.14	0.09	5.6	0.19	0.12	7.5
Subbasin-22_Perv	0.002760871	5	74	0.8	1	0.005	189	0.24	0.16	9.8	0.31	0.20	12.2
Subbasin-23_Imp	0.034139092	0	98	0.6	0.8	0.005	205	0.15	0.10	5.9	0.20	0.13	7.9
Subbasin-23_Perv	0.003793232	5	74	0.8	1	0.005	205	0.26	0.17	10.3	0.32	0.22	12.9
Subbasin-24_Imp	0.058550445	0	98	0.6	0.8	0.005	326	0.20	0.13	8.0	0.27	0.18	10.7
Subbasin-24_Perv	0.006505605	5	74	0.8	1	0.005	326	0.35	0.23	14.0	0.44	0.29	17.6
Subbasin-25_Imp	0.041822229	0	98	0.6	0.8	0.005	324	0.20	0.13	8.0	0.27	0.18	10.7
Subbasin-25_Perv	0.004646914	5	74	0.8	1	0.005	324	0.35	0.23	14.0	0.44	0.29	17.5
Subbasin-26_Imp	0.085485204	0	98	0.6	0.8	0.005	296	0.19	0.13	7.5	0.25	0.17	10.1
Subbasin-26_Perv	0.00001	5	74	0.8	1	0.005	296	0.33	0.22	13.2	0.41	0.27	16.5
Subbasin-27_Imp	0.039518373	0	98	0.6	0.8	0.005	474	0.26	0.17	10.3	0.34	0.23	13.7
Subbasin-27_Perv	0.024220939	5	74	0.8	1	0.005	474	0.45	0.30	18.0	0.56	0.37	22.5
Subbasin-28_Imp	0.037213715	0	98	0.6	0.8	0.005	318	0.20	0.13	7.9	0.26	0.18	10.5
Subbasin-28_Perv	0.022808406	5	74	0.8	1	0.005	318	0.35	0.23	13.8	0.43	0.29	17.3
Subbasin-29_Imp	0.020485589	0	98	0.6	0.8	0.005	228	0.16	0.11	6.3	0.21	0.14	8.5
Subbasin-29_Perv	0.012555683	5	74	0.8	1	0.005	228	0.28	0.18	11.1	0.35	0.23	13.9
Subbasin-30_Imp	0.01713693	0	98	0.6	0.8	0.005	208	0.15	0.10	6.0	0.20	0.13	8.0
Subbasin-30_Perv	0.010503279	5	74	0.8	1	0.005	208	0.26	0.17	10.4	0.33	0.22	13.0
Subbasin-31_Imp	0.0266613	0	98	0.6	0.8	0.005	284	0.18	0.12	7.3	0.24	0.16	9.8
Subbasin-31_Perv	0.016340796	5	74	0.8	1	0.005	284	0.32	0.21	12.8	0.40	0.27	16.0
Subbasin-32_Imp	0.032406998	0	98	0.6	0.8	0.005	320	0.20	0.13	7.9	0.26	0.18	10.6
Subbasin-32_Perv	0.019862354	5	74	0.8	1	0.005	320	0.35	0.23	13.9	0.43	0.29	17.3
Subbasin-33_Imp	0.020363444	0	98	0.6	0.8	0.005	275	0.18	0.12	7.2	0.24	0.16	9.6
Subbasin-33_Perv	0.01248082	5	74	0.8	1	0.005	275	0.31	0.21	12.6	0.39	0.26	15.7
Subbasin-34_Imp	0.023545281	0	98	0.6	0.8	0.005	298	0.19	0.13	7.6	0.25	0.17	10.1
Subbasin-34_Perv	0.014430979	5	74	0.8	1	0.005	298	0.33	0.22	13.2	0.41	0.28	16.5
Subbasin-35_Imp	0.021359882	0	98	0.6	0.8	0.005	263	0.17	0.12	7.0	0.23	0.15	9.3
Subbasin-35_Perv	0.013091541	5	74	0.8	1	0.005	263	0.30	0.20	12.2	0.38	0.25	15.2

HMS Subbasin Parameters

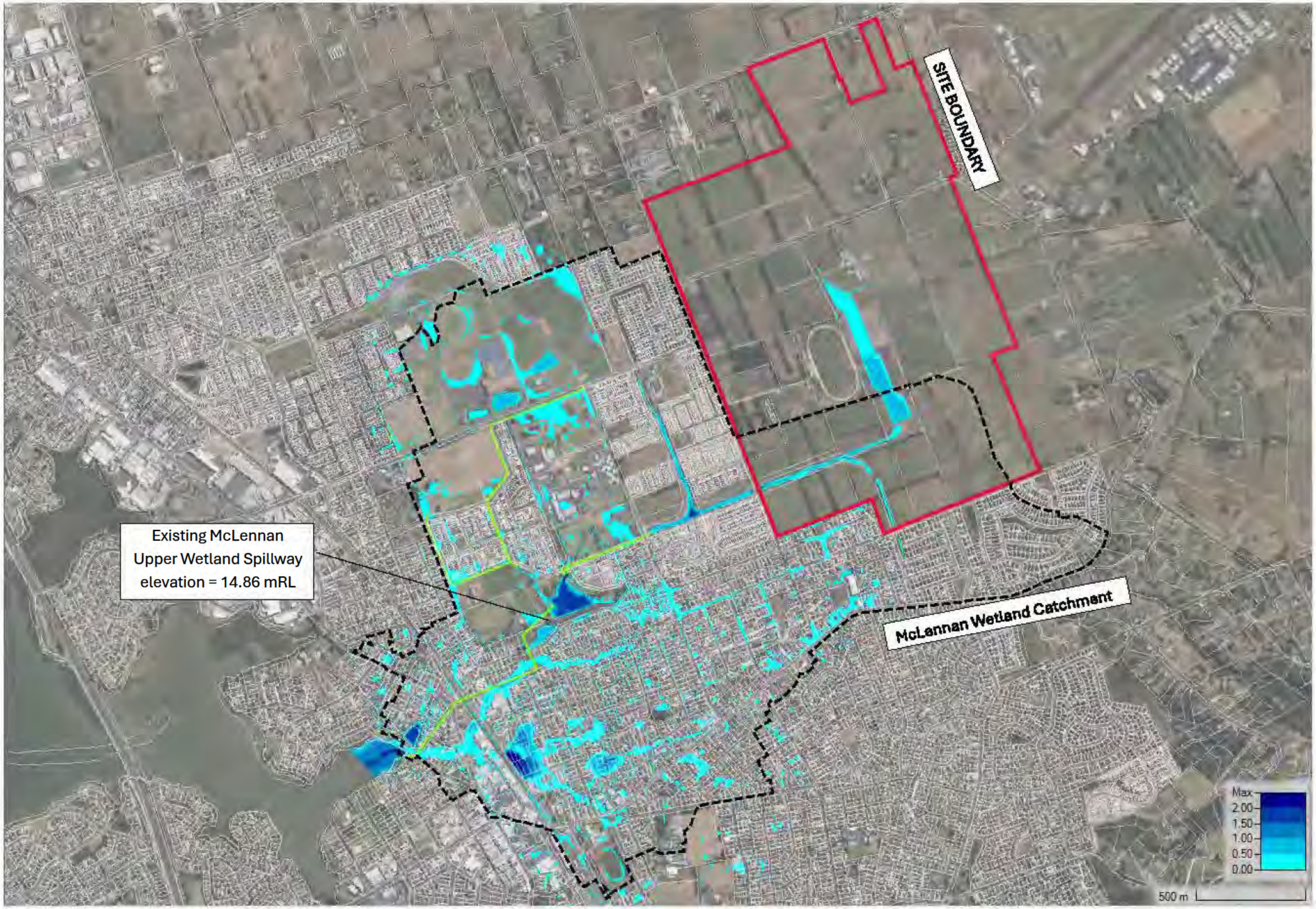
										2& 10yr			100yr
Subbasin Name	**Area KM2**	**Ia**	**CN**	**10 yr Chan**	**100 yr Cha**	**Slope**	**Length**	**Tc (hr)**	**Tp (hr)**	**Tp (min)**	**Tc (hr)**	**Tp (hr)**	**Tp (min)**
Subbasin-36_Imp	0.039314092	0	98	0.6	0.8	0.005	397	0.23	0.15	9.1	0.30	0.20	12.2
Subbasin-36_Perv	0.024095734	5	74	0.8	1	0.005	397	0.40	0.27	16.0	0.50	0.33	20.0
Subbasin-37_Imp	0.090018313	0	98	0.6	0.8	0.005	556	0.29	0.19	11.4	0.38	0.25	15.2
Subbasin-37_Perv	0.010002035	5	74	0.8	1	0.005	556	0.50	0.33	20.0	0.62	0.42	25.0
Subbasin-38_Imp	0.06471901	0	98	0.6	0.8	0.005	389	0.23	0.15	9.0	0.30	0.20	12.0
Subbasin-38_Perv	0.007191001	5	74	0.8	1	0.005	389	0.39	0.26	15.8	0.49	0.33	19.7
Subbasin-39_Imp	0.000807606	0	98	0.6	0.8	0.005	3013	0.87	0.58	34.9	1.16	0.77	46.5
Subbasin-39_Perv	0.001884414	5	74	0.8	1	0.005	3013	1.52	1.02	60.9	1.90	1.27	76.2
Subbasin-40_Imp	3.53456E-06	0	98	0.6	0.8	0.005	70	0.07	0.05	2.9	0.10	0.06	3.9
Subbasin-40_Perv	0.003531027	5	74	0.8	1	0.005	70	0.13	0.08	5.1	0.16	0.11	6.4
Subbasin-41_Imp	1.84914E-05	0	98	0.6	0.8	0.005	206	0.15	0.10	5.9	0.20	0.13	7.9
Subbasin-41_Perv	0.018472944	5	74	0.8	1	0.005	206	0.26	0.17	10.4	0.32	0.22	13.0
Subbasin-42_Imp	4.34312E-05	0	98	0.6	0.8	0.005	393	0.23	0.15	9.1	0.30	0.20	12.1
Subbasin-42_Perv	0.000390881	5	74	0.8	1	0.005	393	0.40	0.26	15.9	0.50	0.33	19.9
Subbasin-43_Imp	0.000358811	0	98	0.6	0.8	0.005	831	0.37	0.25	14.9	0.50	0.33	19.9
Subbasin-43_Perv	0.000837226	5	74	0.8	1	0.005	831	0.65	0.43	26.0	0.81	0.54	32.6
Subbasin-44_Imp	0.013356467	0	98	0.6	0.8	0.005	173	0.13	0.09	5.3	0.18	0.12	7.1
Subbasin-44_Perv	0.008186221	5	74	0.8	1	0.005	173	0.23	0.15	9.2	0.29	0.19	11.6
Subbasin-45_Imp	0.017303138	0	98	0.6	0.8	0.005	143	0.12	0.08	4.7	0.16	0.10	6.2
Subbasin-45_Perv	0.00001	5	74	0.8	1	0.005	143	0.20	0.14	8.2	0.25	0.17	10.2
Subbasin-46_Imp	0.018989449	0	98	0.6	0.8	0.005	237	0.16	0.11	6.5	0.22	0.14	8.7
Subbasin-46_Perv	0.011638694	5	74	0.8	1	0.005	237	0.28	0.19	11.4	0.36	0.24	14.2
Subbasin-47_Imp	0.021910077	0	98	0.6	0.8	0.005	239	0.16	0.11	6.5	0.22	0.15	8.7
Subbasin-47_Perv	0.013428757	5	74	0.8	1	0.005	239	0.29	0.19	11.4	0.36	0.24	14.3
Subbasin-48_Imp	0.02289379	0	98	0.6	0.8	0.007	320	0.18	0.12	7.2	0.24	0.16	9.6
Subbasin-48_Perv	0.014031677	5	74	0.8	1	0.007	320	0.31	0.21	12.5	0.39	0.26	15.7
Subbasin-49_Imp	0.02289379	0	98	0.6	0.8	0.023	687	0.21	0.14	8.3	0.28	0.18	11.1
Subbasin-49_Perv	0.014031677	5	74	0.8	1	0.023	687	0.36	0.24	14.5	0.45	0.30	18.2
Subbasin-50_Imp	0.015564955	0	98	0.6	0.8	0.005	295	0.19	0.13	7.5	0.25	0.17	10.0
Subbasin-50_Perv	0.001729439	5	74	0.8	1	0.005	295	0.33	0.22	13.1	0.41	0.27	16.4
Subbasin-51_Imp	0.01739647	0	98	0.6	0.8	0.005	186	0.14	0.09	5.5	0.18	0.12	7.4
Subbasin-51_Perv	0.001932941	5	74	0.8	1	0.005	186	0.24	0.16	9.7	0.30	0.20	12.1
Subbasin-52_Imp	0.006789462	0	98	0.6	0.8	0.005	260	0.17	0.12	6.9	0.23	0.15	9.2
Subbasin-52_Perv	0.00119814	5	74	0.8	1	0.005	260	0.30	0.20	12.1	0.38	0.25	15.1
Subbasin-53_Imp	0.01968575	0	98	0.6	0.8	0.005	196	0.14	0.10	5.7	0.19	0.13	7.7
Subbasin-53_Perv	0.002187306	5	74	0.8	1	0.005	196	0.25	0.17	10.0	0.31	0.21	12.5
Subbasin-54_Imp	0.02003173	0	98	0.6	0.8	0.005	210	0.15	0.10	6.0	0.20	0.13	8.0
Subbasin-54_Perv	0.002225748	5	74	0.8	1	0.005	210	0.26	0.18	10.5	0.33	0.22	13.1
Subbasin-55_Imp	0.021908226	0	98	0.6	0.8	0.005	206	0.15	0.10	5.9	0.20	0.13	7.9
Subbasin-55_Perv	0.002434247	5	74	0.8	1	0.005	206	0.26	0.17	10.4	0.32	0.22	13.0
Subbasin-56_Imp	0.036180878	0	98	0.6	0.8	0.005	423	0.24	0.16	9.5	0.32	0.21	12.7
Subbasin-56_Perv	0.004020098	5	74	0.8	1	0.005	423	0.42	0.28	16.7	0.52	0.35	20.8
Subbasin-57_Imp	0.015826636	0	98	0.6	0.8	0.005	193	0.14	0.09	5.7	0.19	0.13	7.6
Subbasin-57_Perv	0.001758515	5	74	0.8	1	0.005	196	0.25	0.17	10.0	0.31	0.21	12.5
Subbasin-58_Imp	0.005716976	0	98	0.6	0.8	0.005	263	0.17	0.12	7.0	0.23	0.15	9.3
Subbasin-58_Perv	0.001008878	5	74	0.8	1	0.005	263	0.30	0.20	12.2	0.38	0.25	15.2
Mill_Road_IMPERV	0.149238427	0	98	0.6	0.8	0.005	700	0.33	0.22	13.3	0.44	0.30	17.7
Mill_Road_PERVIOUS	0.063977979	5	74	0.8	1	0.005	700	0.58	0.39	23.3	0.73	0.48	29.1
2A_1_IMPERV	0.176659119	0	98	0.6	0.8	0.005	400	0.23	0.15	9.2	0.31	0.20	12.3
2A_1_PERV	0.075710806	5	74	0.8	1	0.005	400	0.40	0.27	16.1	0.50	0.33	20.1
2A_2_IMPERV	0.074556397	0	98	0.6	0.8	0.008	250	0.15	0.10	5.9	0.20	0.13	7.9
2A_2_PERV	0.040145831	5	74	0.8	1	0.008	250	0.26	0.17	10.3	0.32	0.21	12.9
2A_4_IMPERV	0.00001	0	98	0.6	0.8	0.005	400	0.23	0.15	9.2	0.31	0.20	12.3
2A_4_PERVIOUS	0.101798013	5	74	0.8	1	0.005	400	0.40	0.27	16.1	0.50	0.33	20.1
2B_1_IMPERV	0.0996	0	98	0.6	0.8	0.007	320	0.18	0.12	7.2	0.24	0.16	9.6
2B_1_PERV	0.0498	5	74	0.8	1	0.007	320	0.31	0.21	12.5	0.39	0.26	15.7
2B_2_IMPERV	0.1313	0	98	0.6	0.8	0.023	687	0.21	0.14	8.3	0.28	0.18	11.1
2B_2_PERV	0.0884	5	74	0.8	1	0.023	687	0.36	0.24	14.5	0.45	0.30	18.2
2B4_1_IMPERV	0.044295802	0	98	0.6	0.8	0.005	250	0.17	0.11	6.7	0.22	0.15	9.0
2B4_1_PERV	0.029530534	5	74	0.8	1	0.005	250	0.29	0.20	11.8	0.37	0.25	14.7
2B4_2_IMPERV	0.183337146	0	98	0.6	0.8	0.014	700	0.24	0.16	9.8	0.33	0.22	13.0
2B4_2_PERV	0.107674197	5	74	0.8	1	0.014	700	0.43	0.28	17.1	0.53	0.36	21.3
2B4_3_IMPERV	0.107990324	0	98	0.6	0.8	0.0075	400	0.20	0.14	8.1	0.27	0.18	10.9
2B4_3_PERV	0.063422889	5	74	0.8	1	0.0075	400	0.36	0.24	14.2	0.44	0.30	17.8
SW Pond 2 Ex													
Subbasin-SW Pond 2 Ex_Perv	0.152738169	5	74	0.8	1	0.009	650	0.46	0.31	18.6	0.58	0.39	23.2
SW Pond 3 Ex													
Subbasin-SW Pond 2 Ex_Perv	0.003531027	5	74	0.8	1	0.008	330	0.31	0.20	12.3	0.38	0.26	15.4

APPENDIX 6 – RAS Western Model & Results

Western Catchment 50%AEP – Baseline Flood Extents

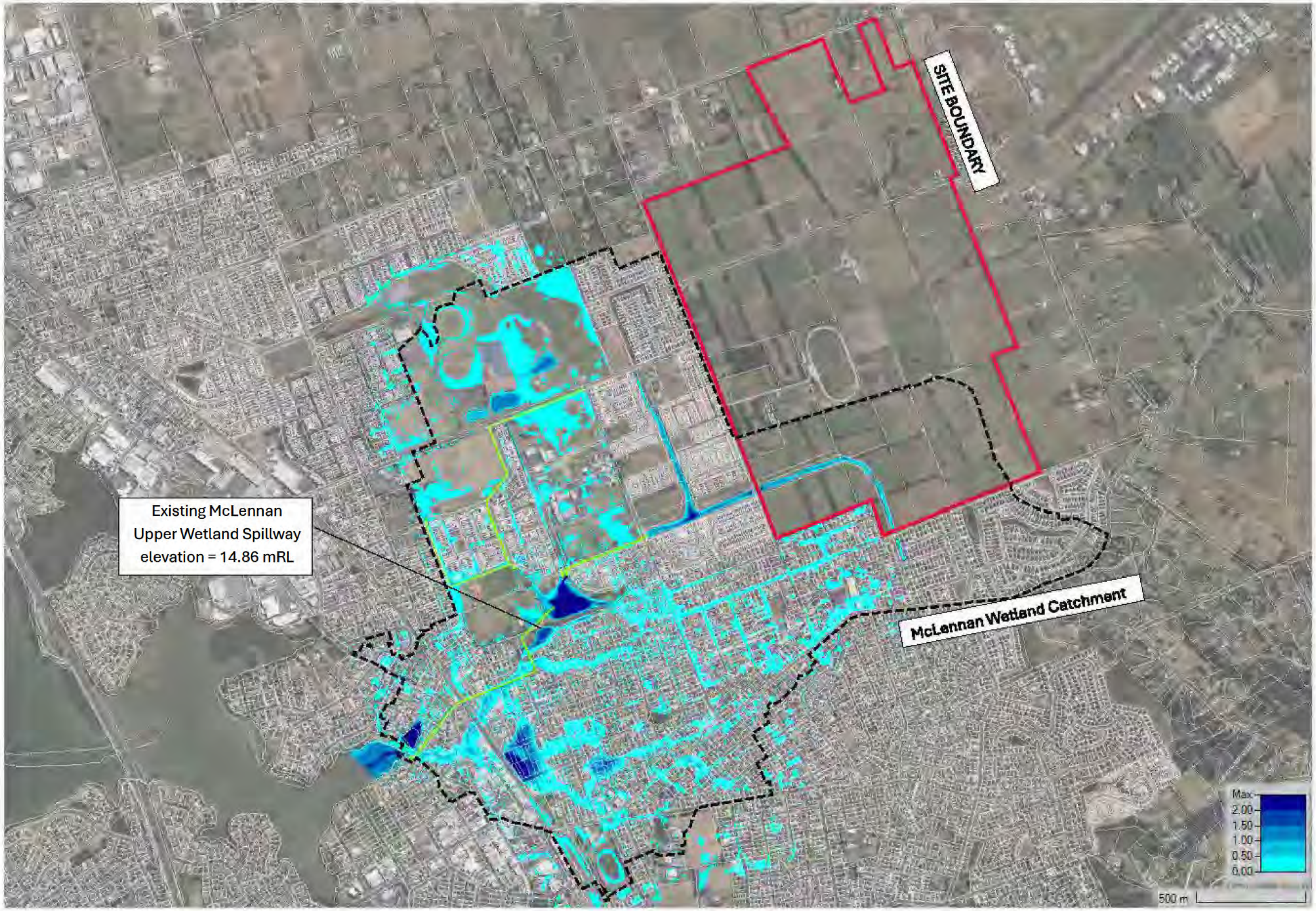


Western Catchment 50%AEP – Post Development Flood Extents

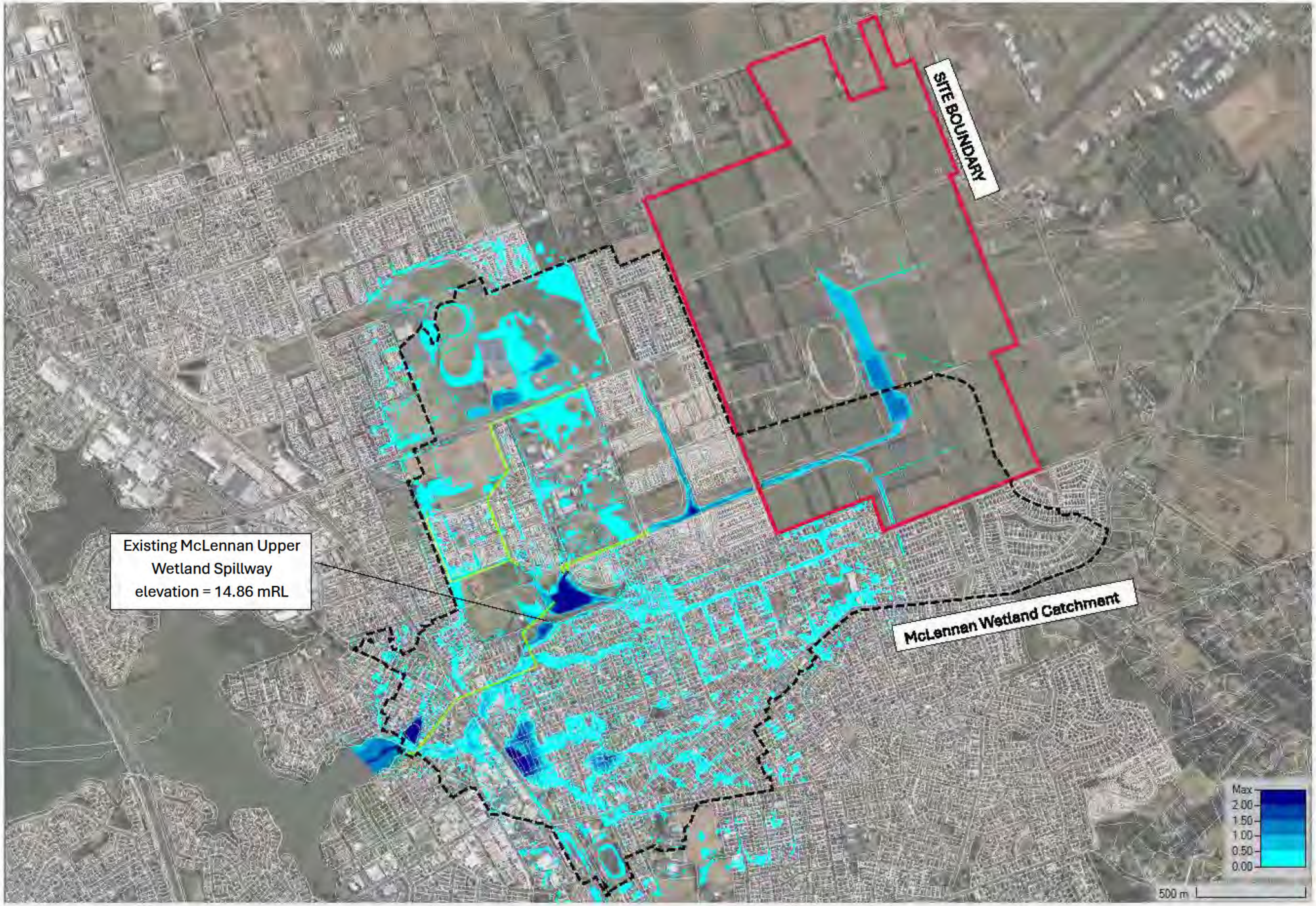


Western Catchment 50%AEP – Flooding level Comparison Extents (Post Development minus Existing: ie blue = decrease and red = increase)

Western Catchment 10%AEP – Baseline Flood Extents

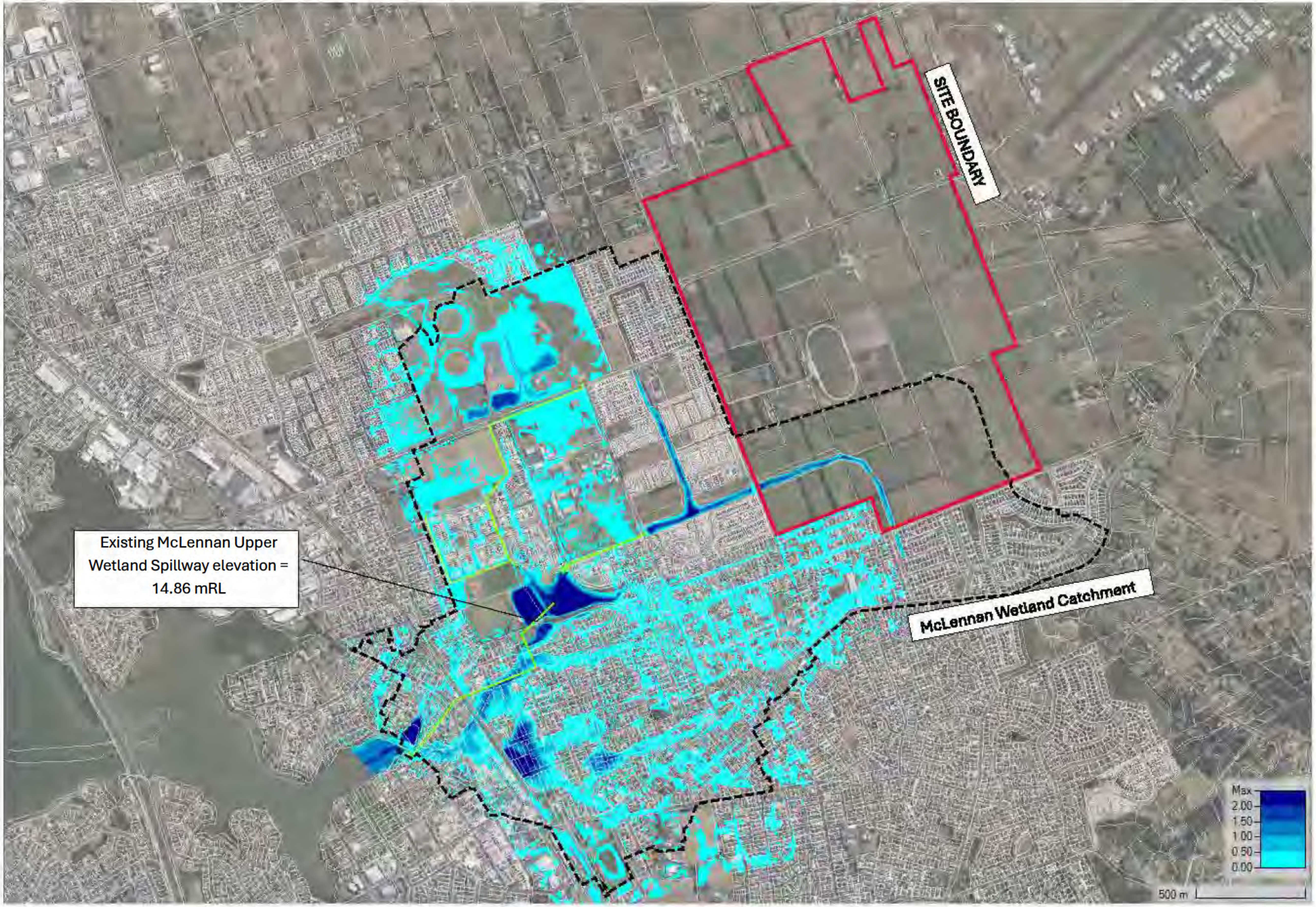


Western Catchment 10%AEP – Post Development Flood Extents

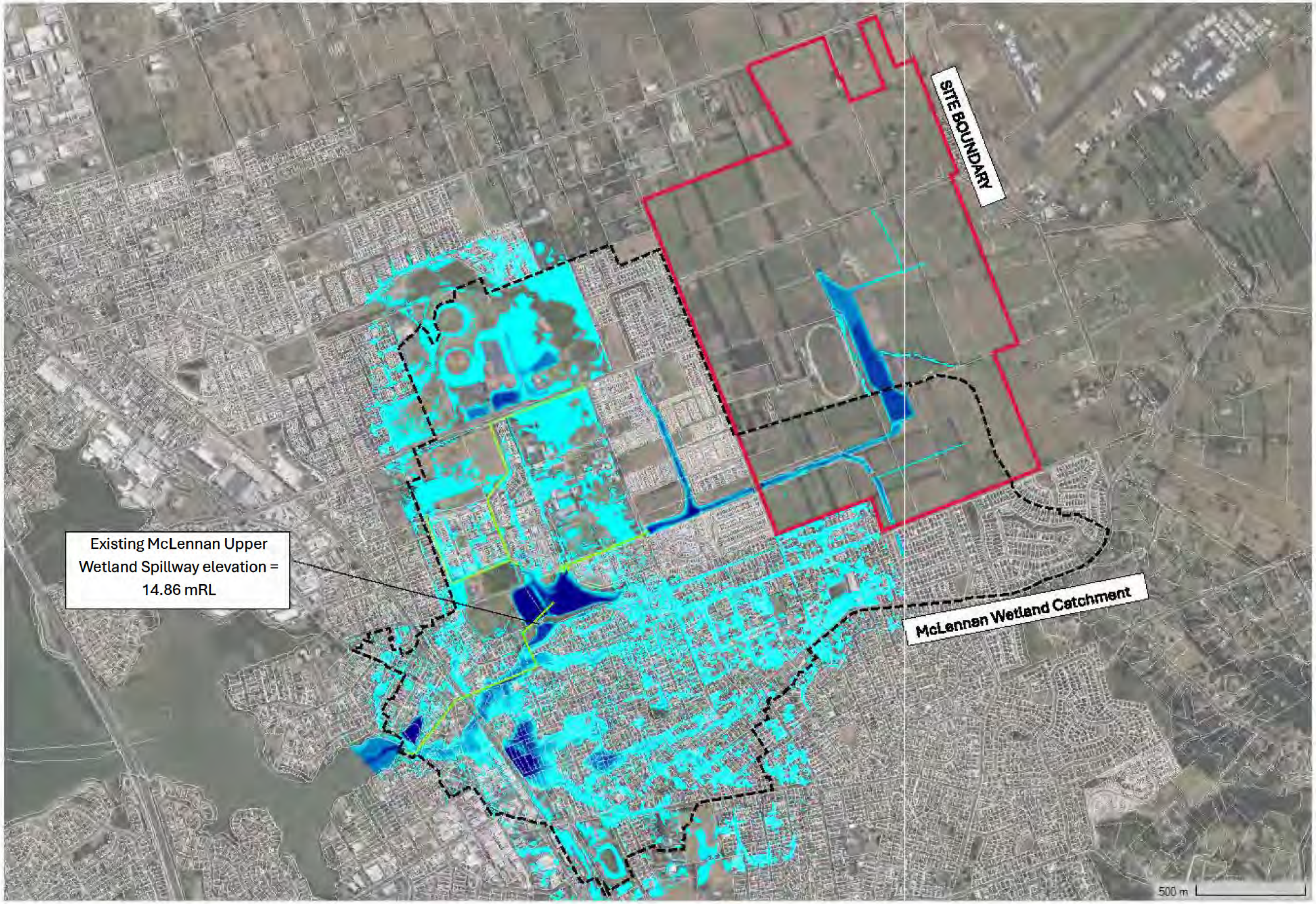


Western Catchment 10%AEP – Flooding level Comparison Extents (Post Development minus Existing: ie blue = decrease and red = increase)

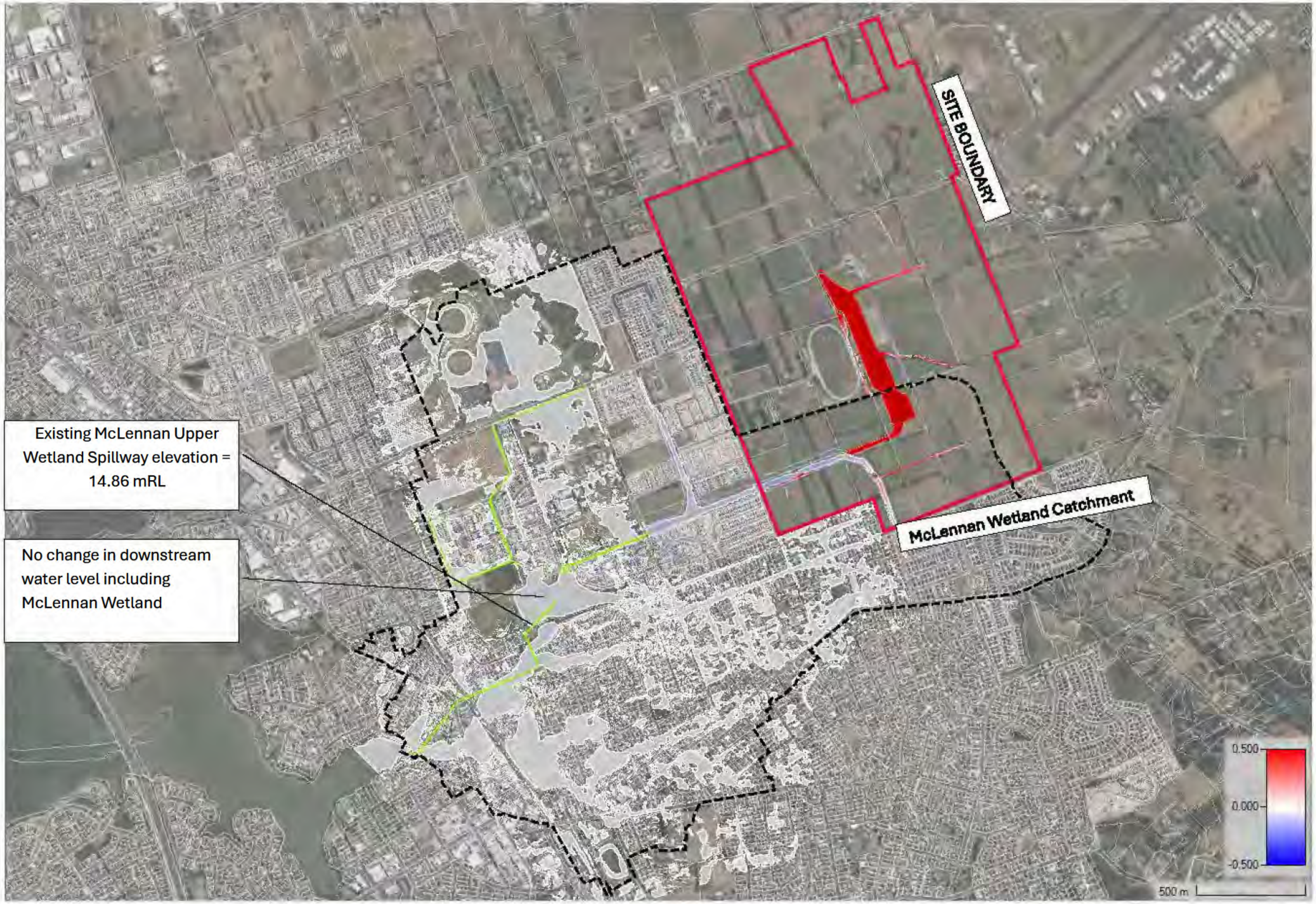
Western Catchment 1%AEP – Baseline Flood Extents



Western Catchment 1%AEP – Post Development Flood Extents

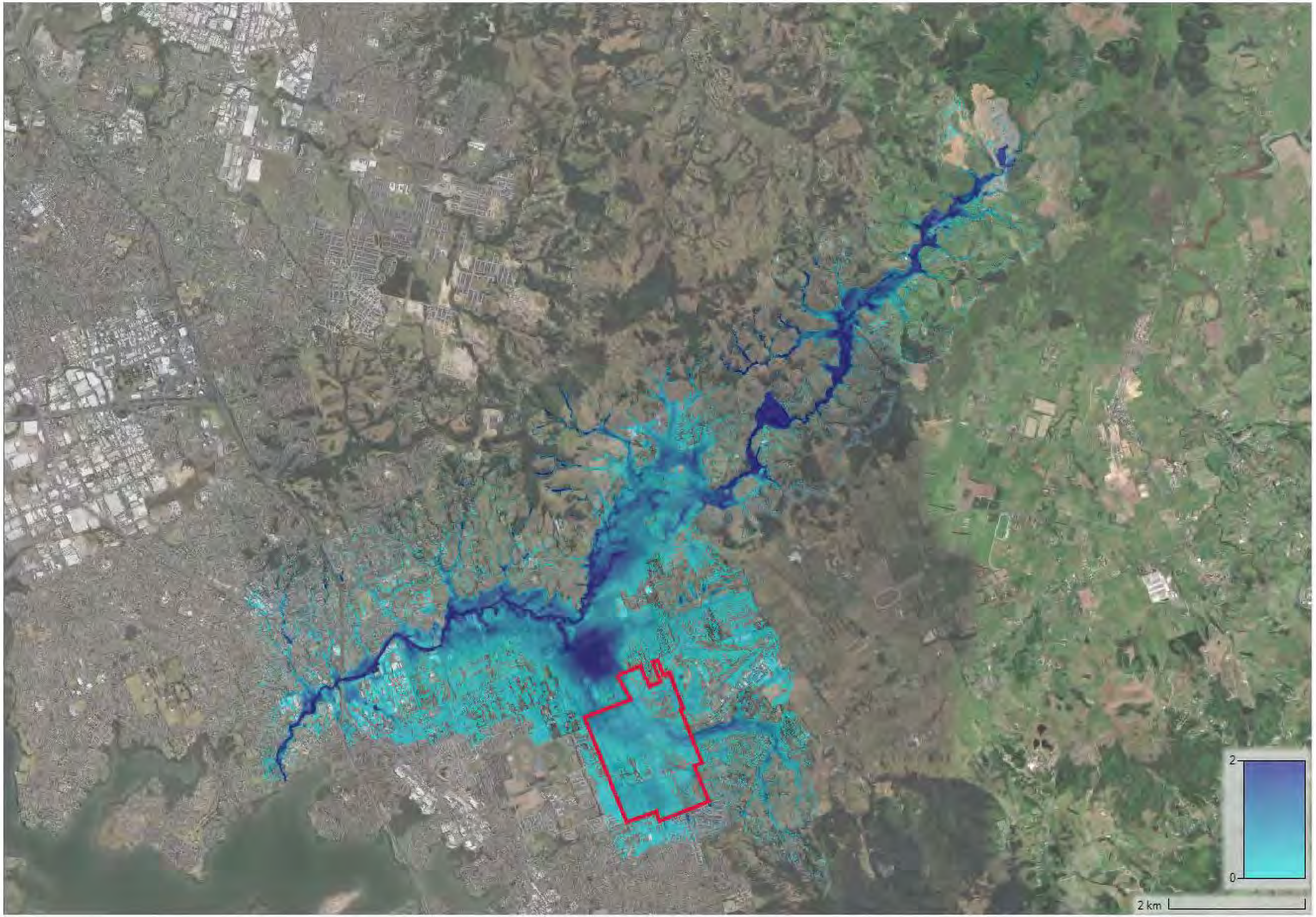


Western Catchment 1%AEP – Flooding level Comparison Extents (Post Development minus Existing: ie blue = decrease and red = increase)

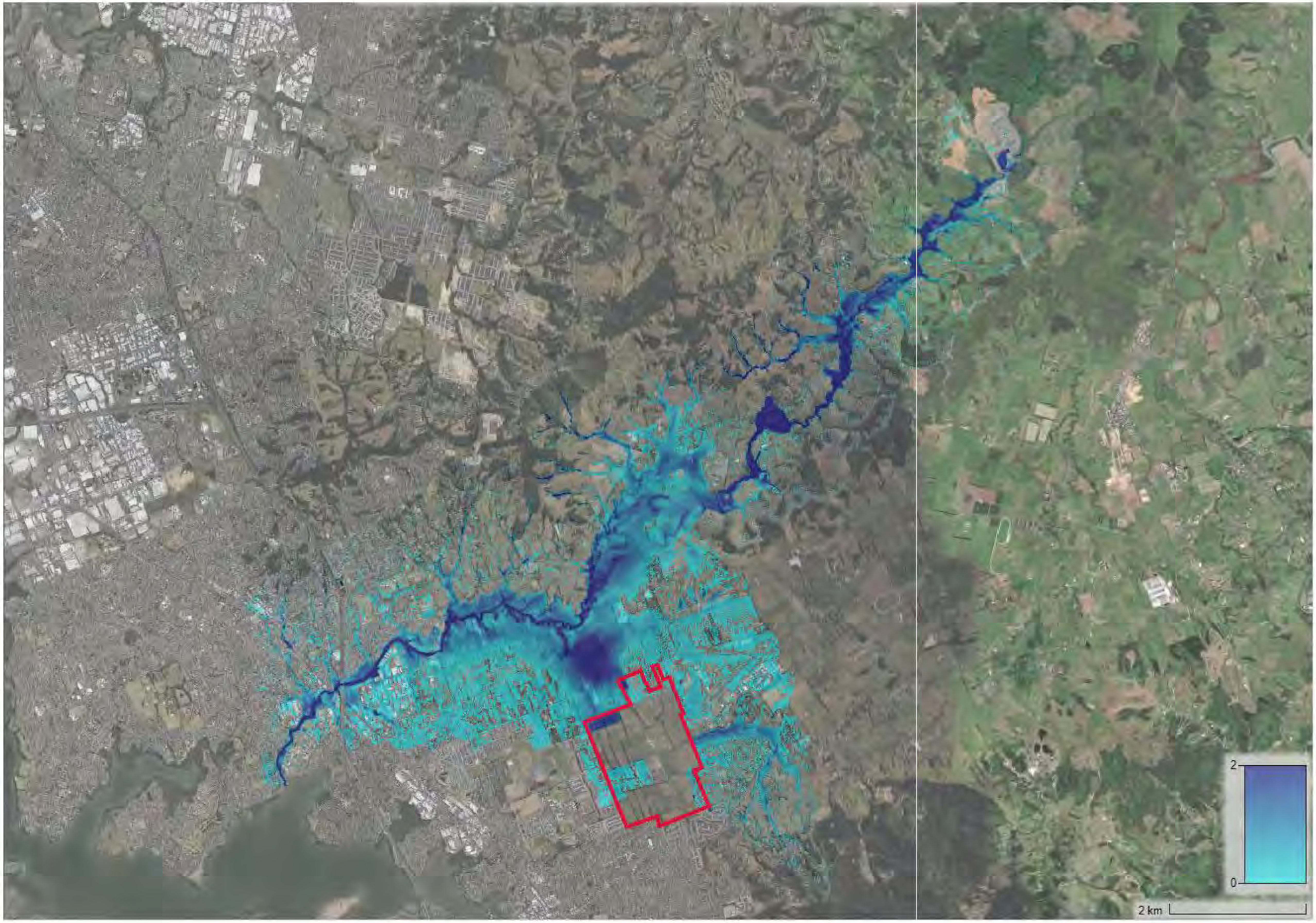


APPENDIX 7 – RAS Eastern Model & Results

Eastern Catchment 1%AEP – Existing Flood Extents

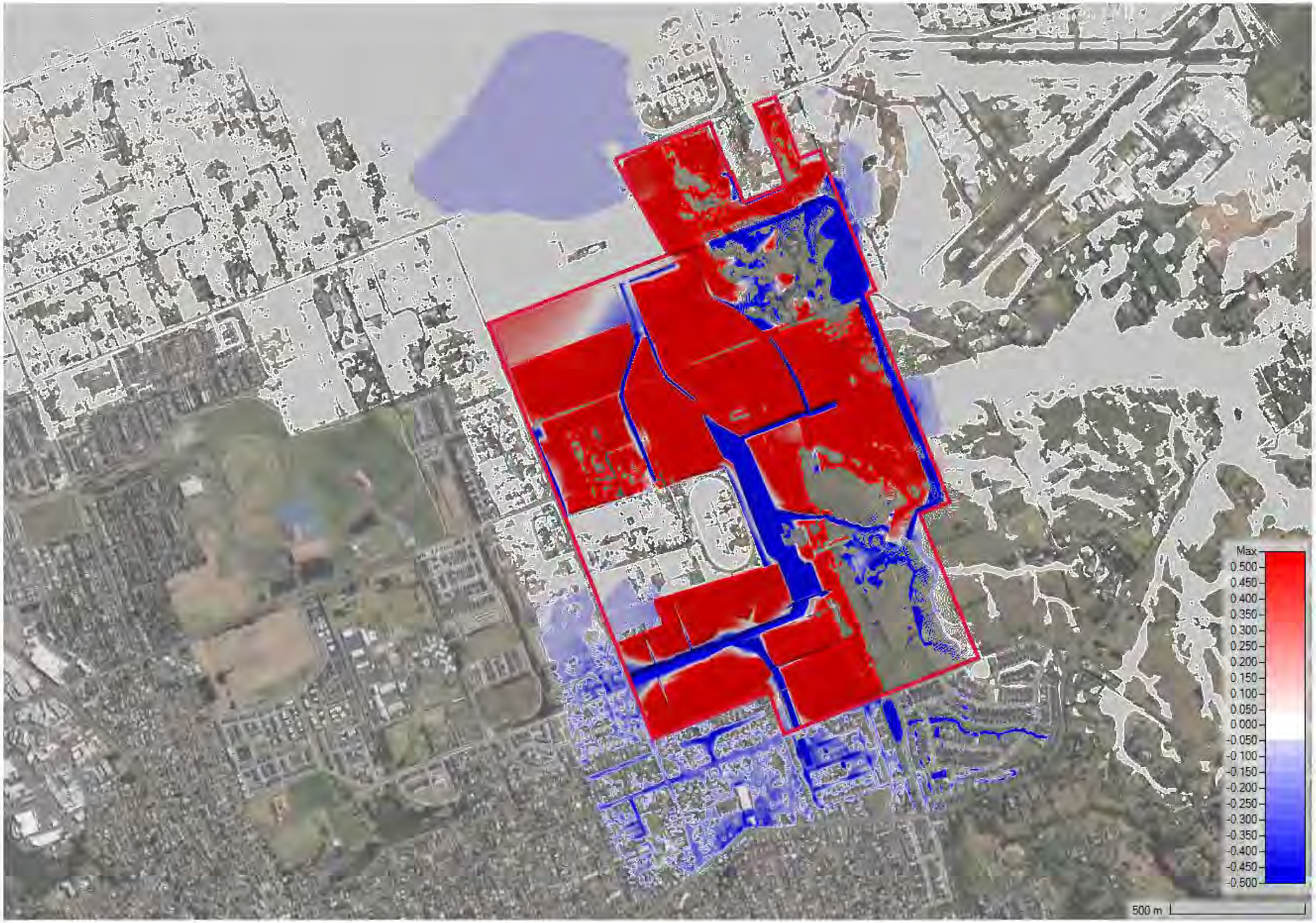


Eastern Catchment 1%AEP – Post Development Flood Extents

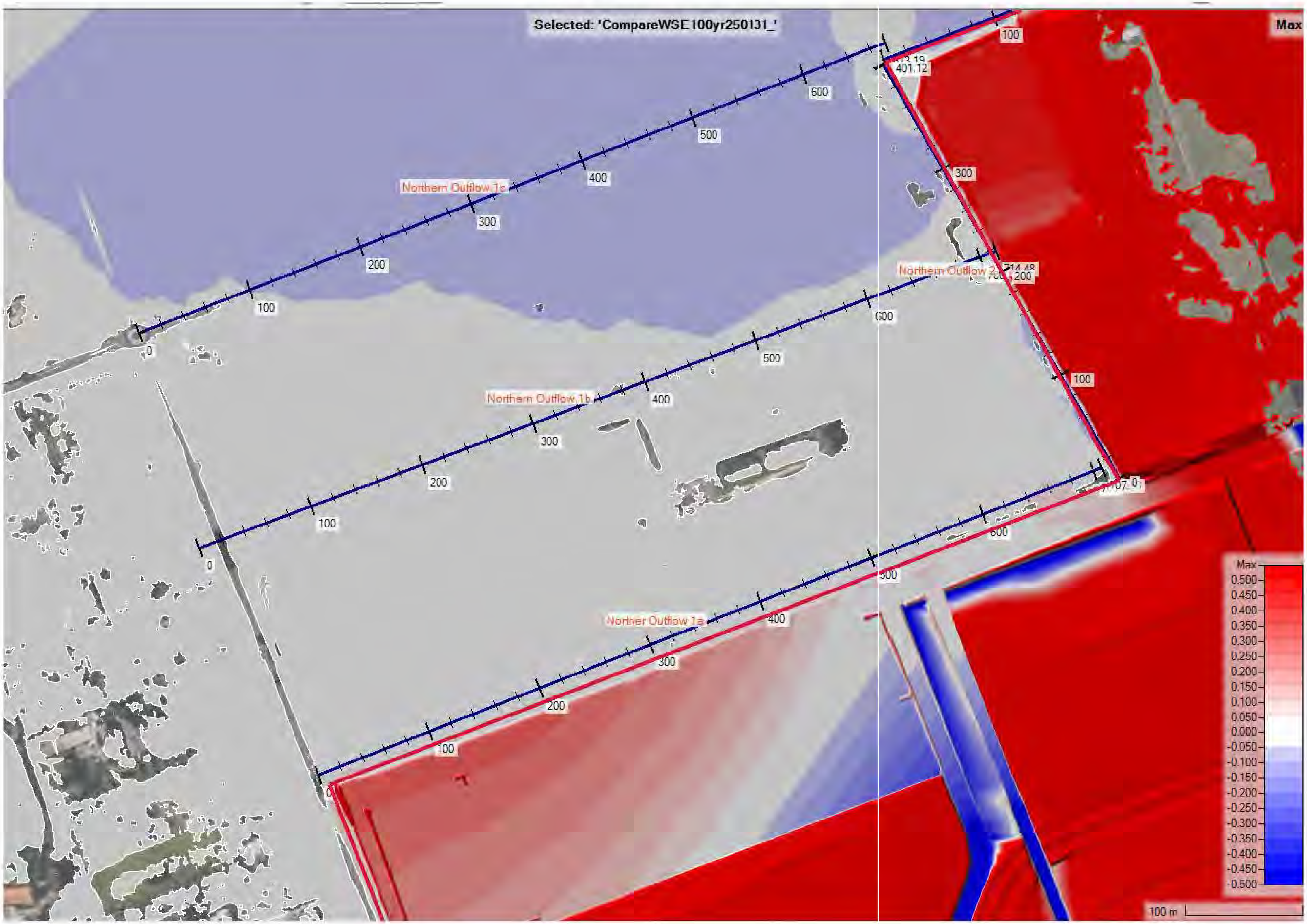


Eastern Catchment 1%AEP – Flooding level Comparison Extents (Post Development minus Existing) : ie blue = decrease and red = increase)

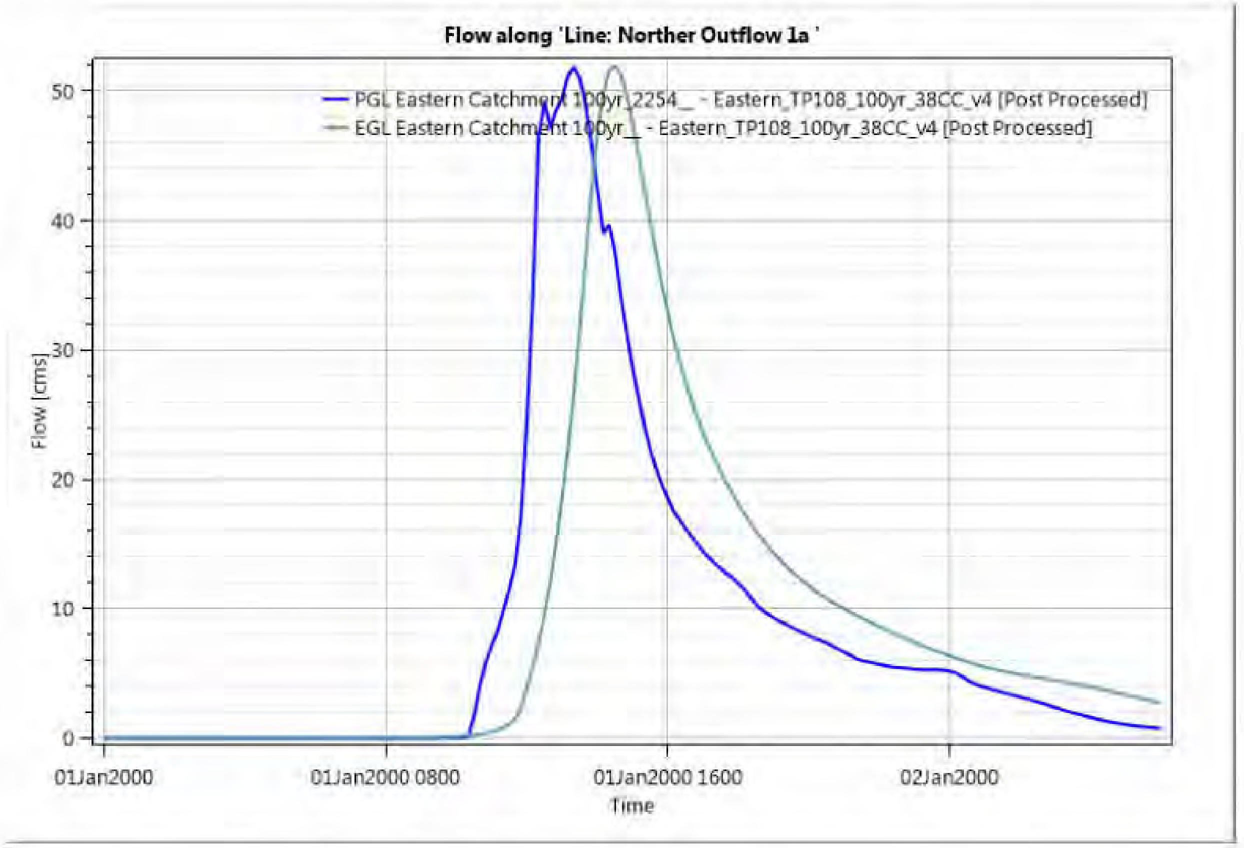
Eastern Catchment 1%AEP – Flooding level Comparison Extents (Post Development minus Existing) : ie blue = decrease and red = increase)

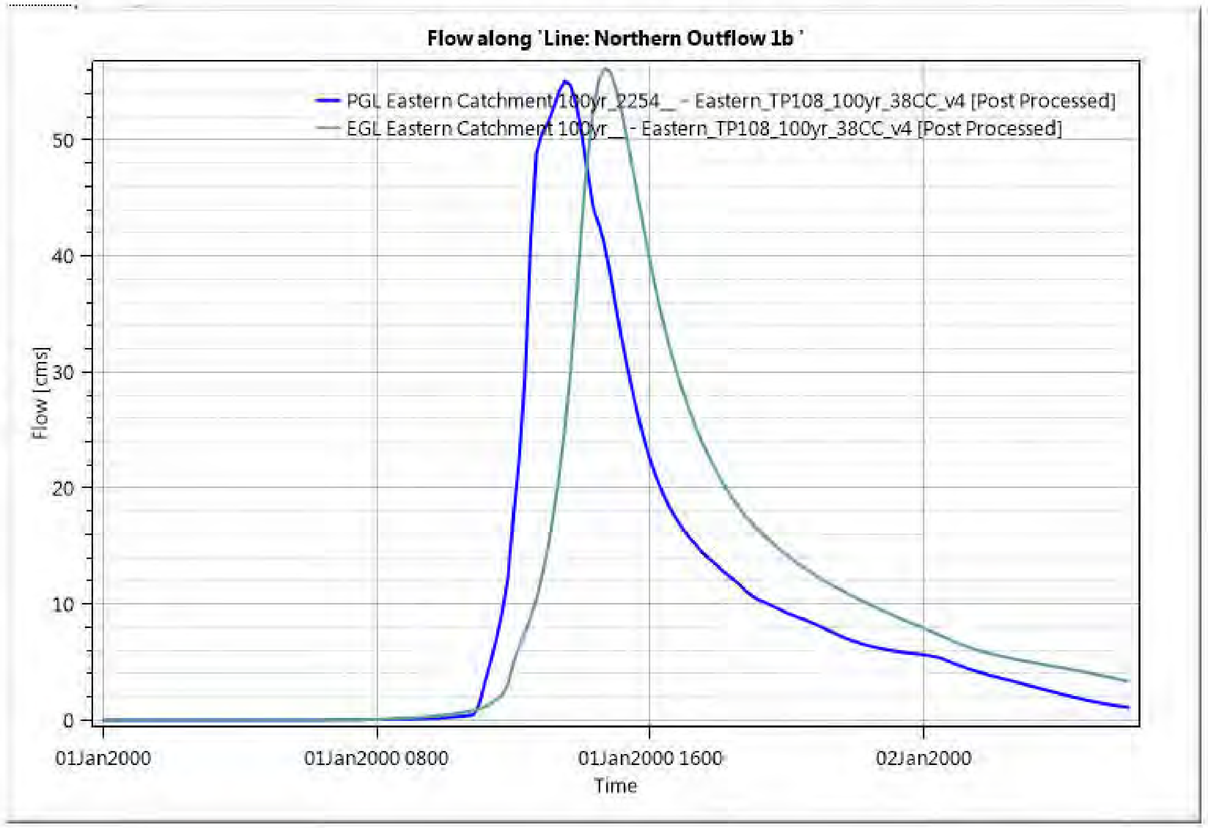


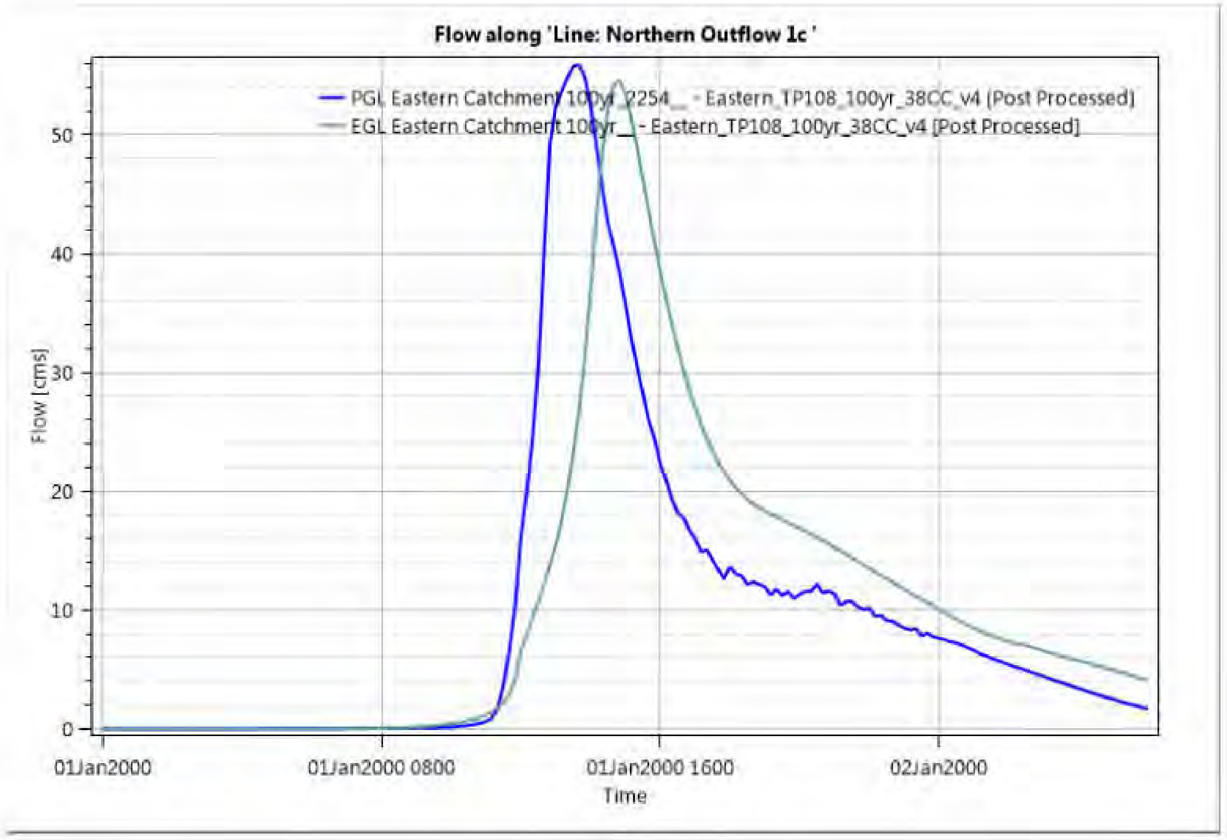
Eastern Catchment 1%AEP – Flooding level Comparison Extents (Post Development minus Existing) : ie blue = decrease and red = increase)



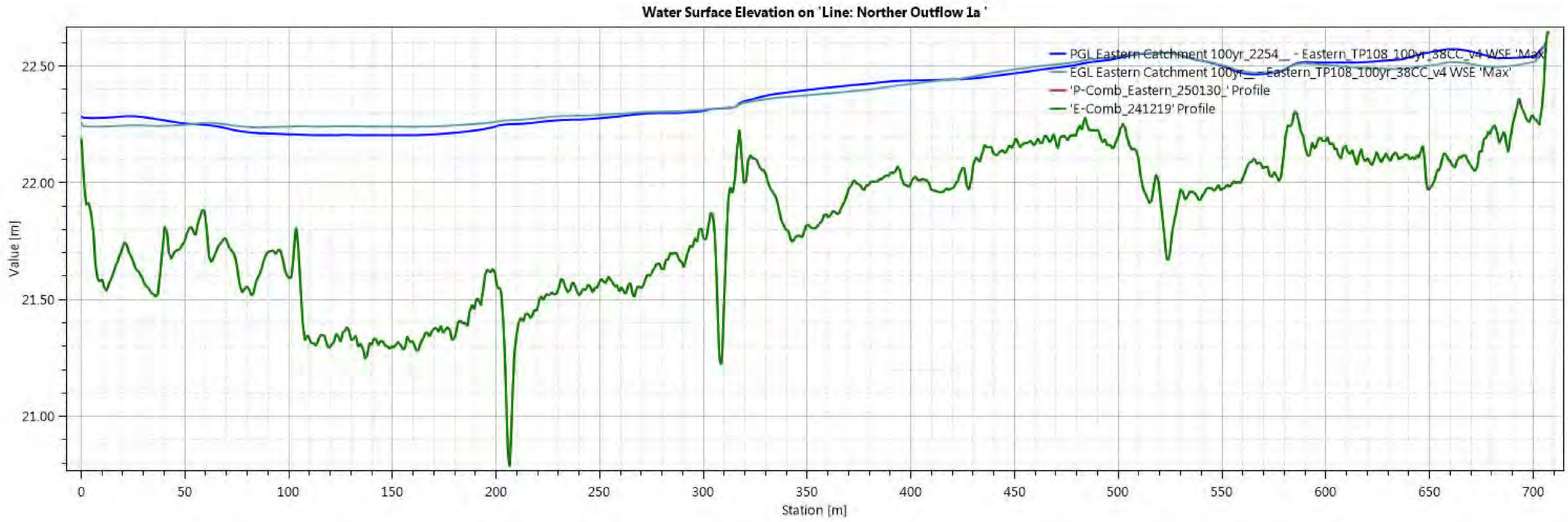
Eastern Catchment 1%AEP – Flow Comparison Extents (Post Development versus Existing)



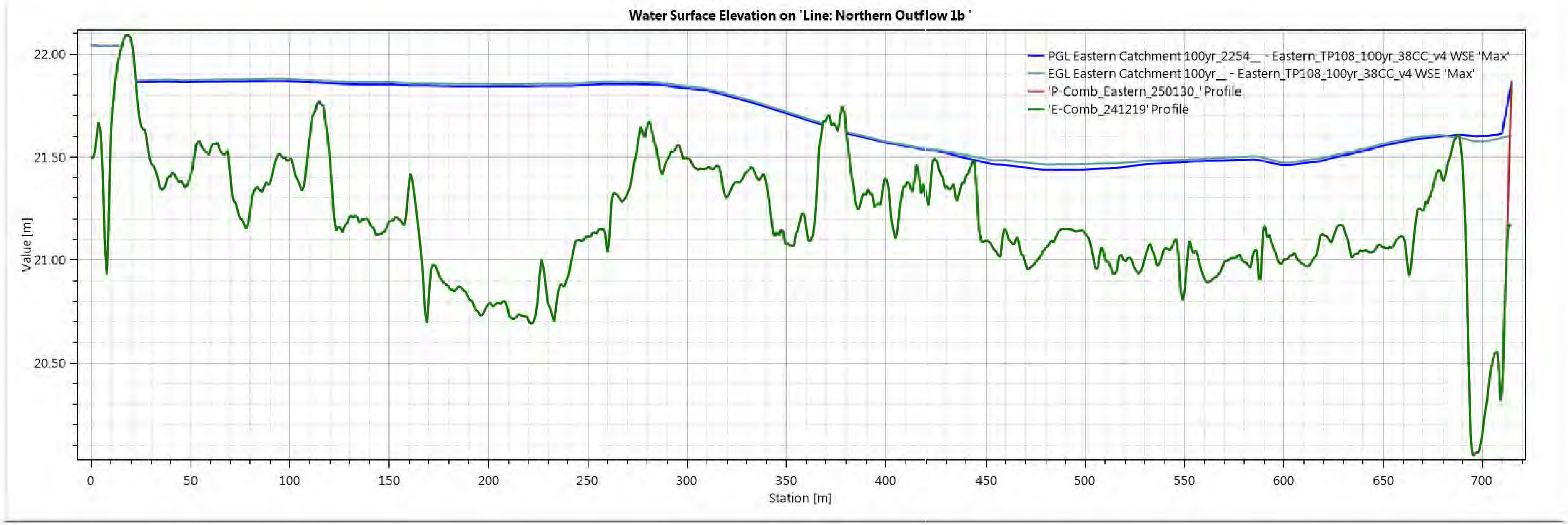




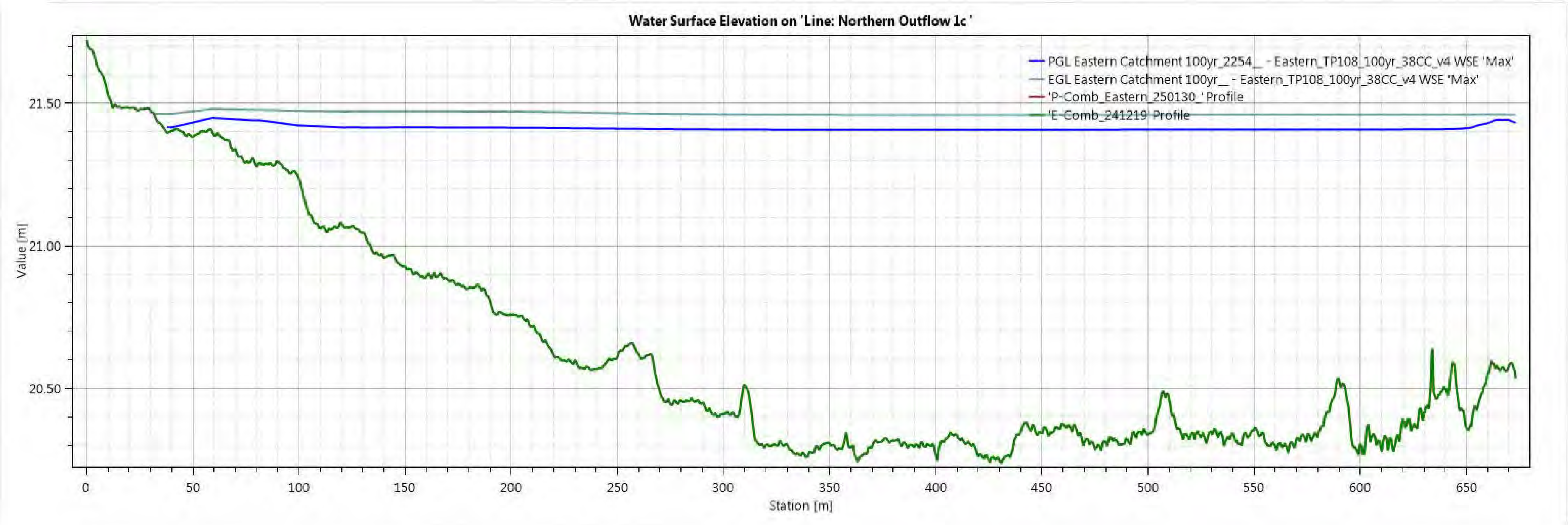
Eastern Catchment 1%AEP – Flood Level Comparison - Section 1A



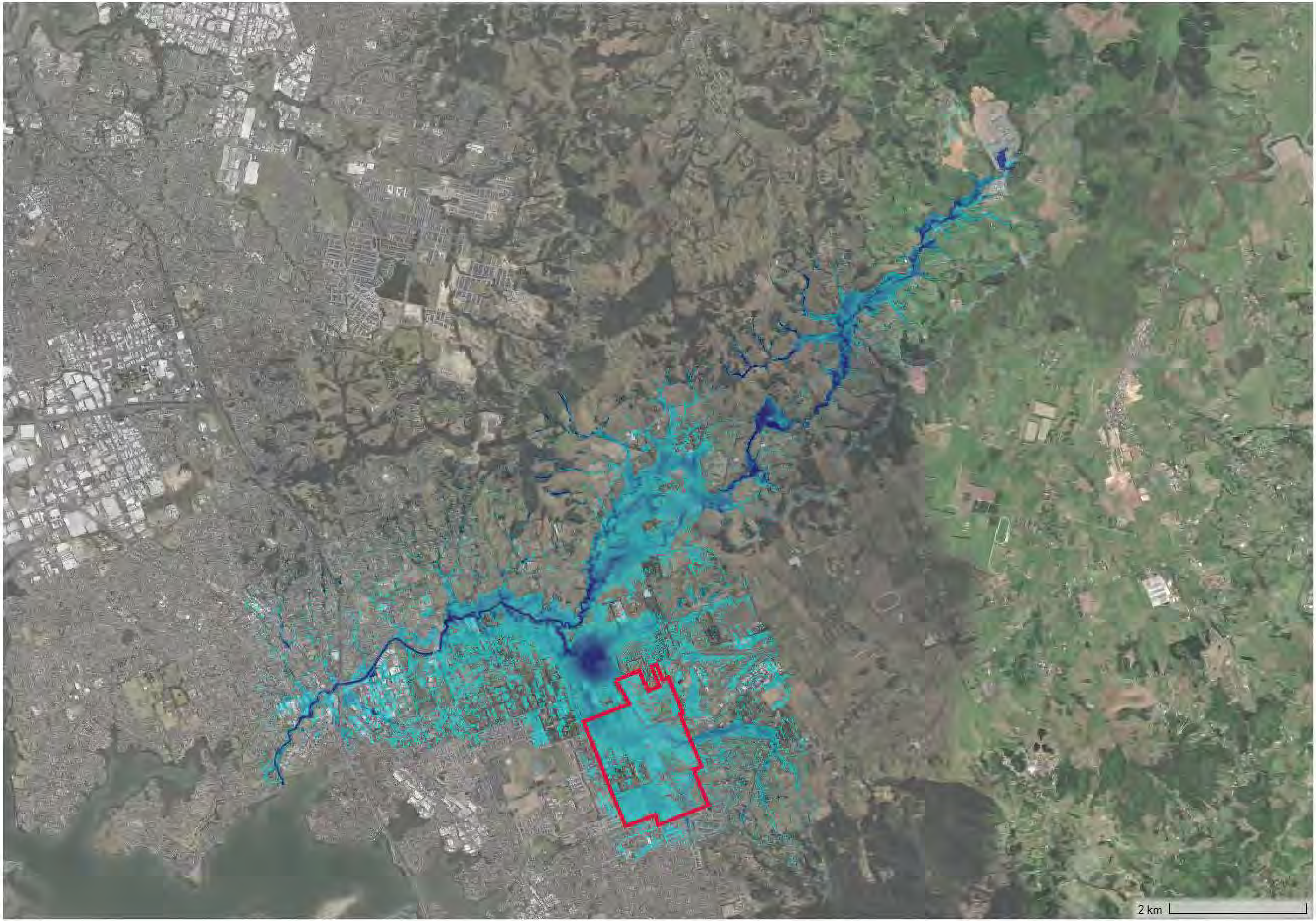
Eastern Catchment 1%AEP – Flood Level Comparison Extents - Section 1B



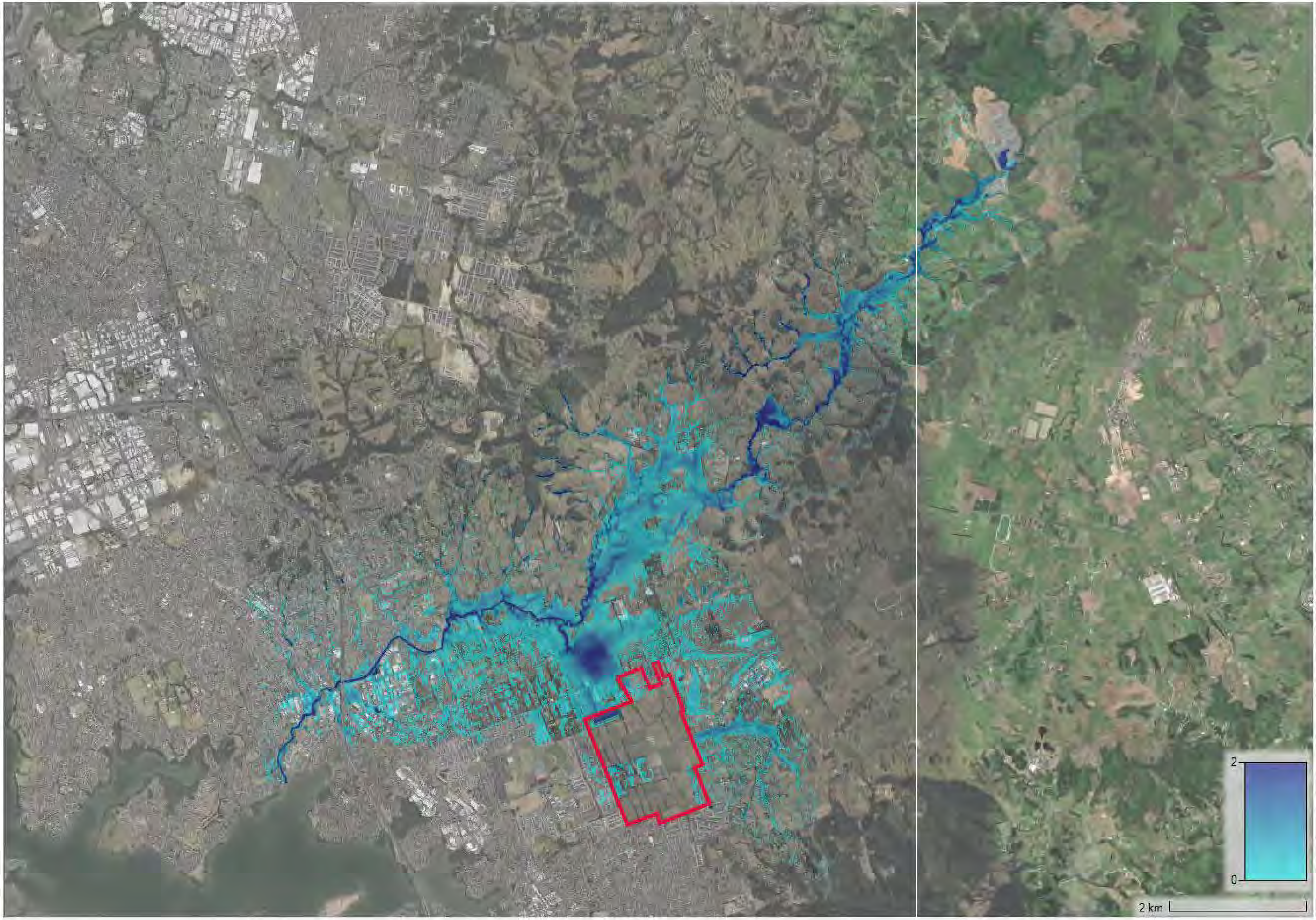
Eastern Catchment 1%AEP – Flood Level Comparison Extents - Section 1C



Eastern Catchment 10%AEP – Existing Flood Extents

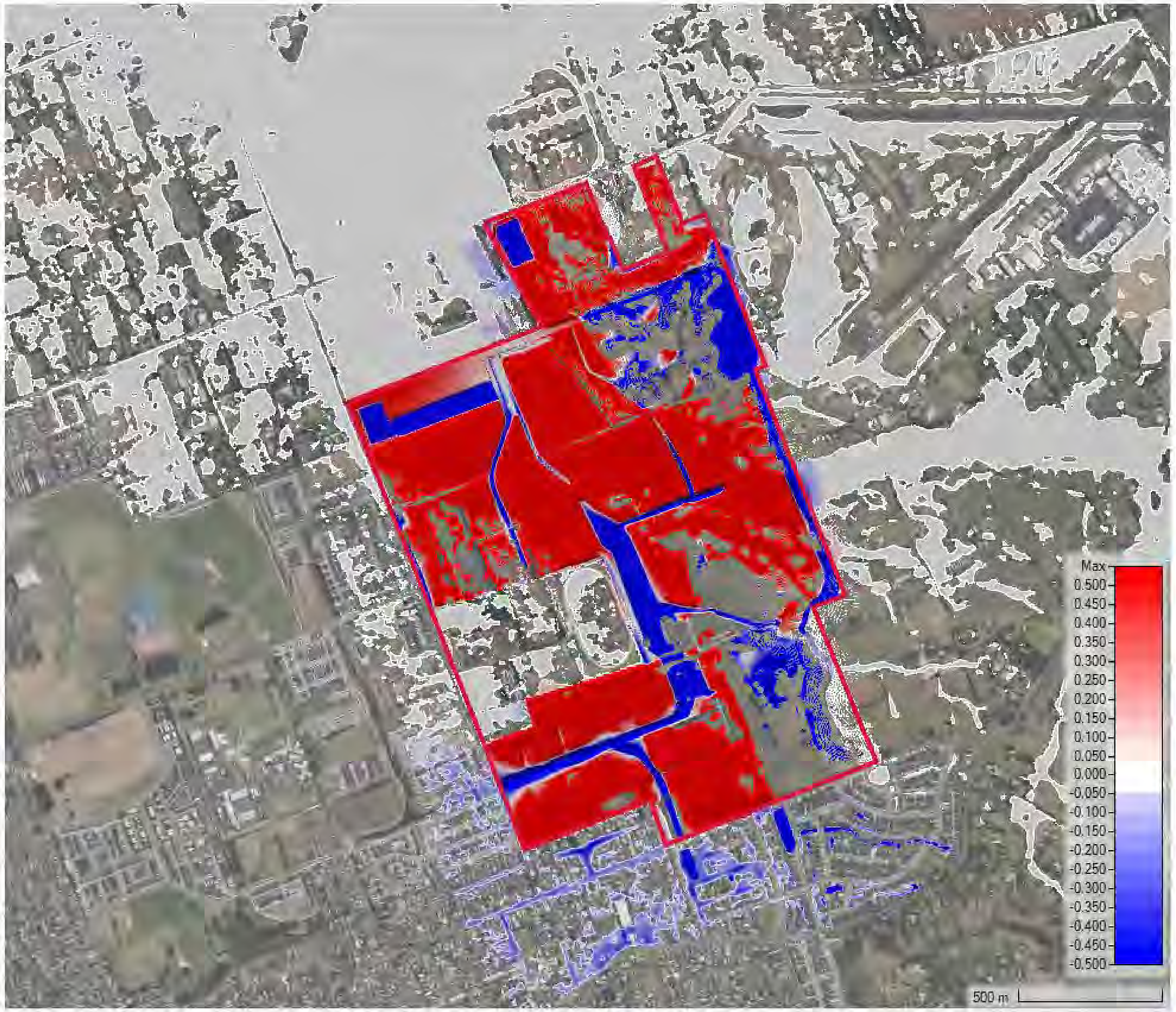


Eastern Catchment 10%AEP – Post Development Flood Extents

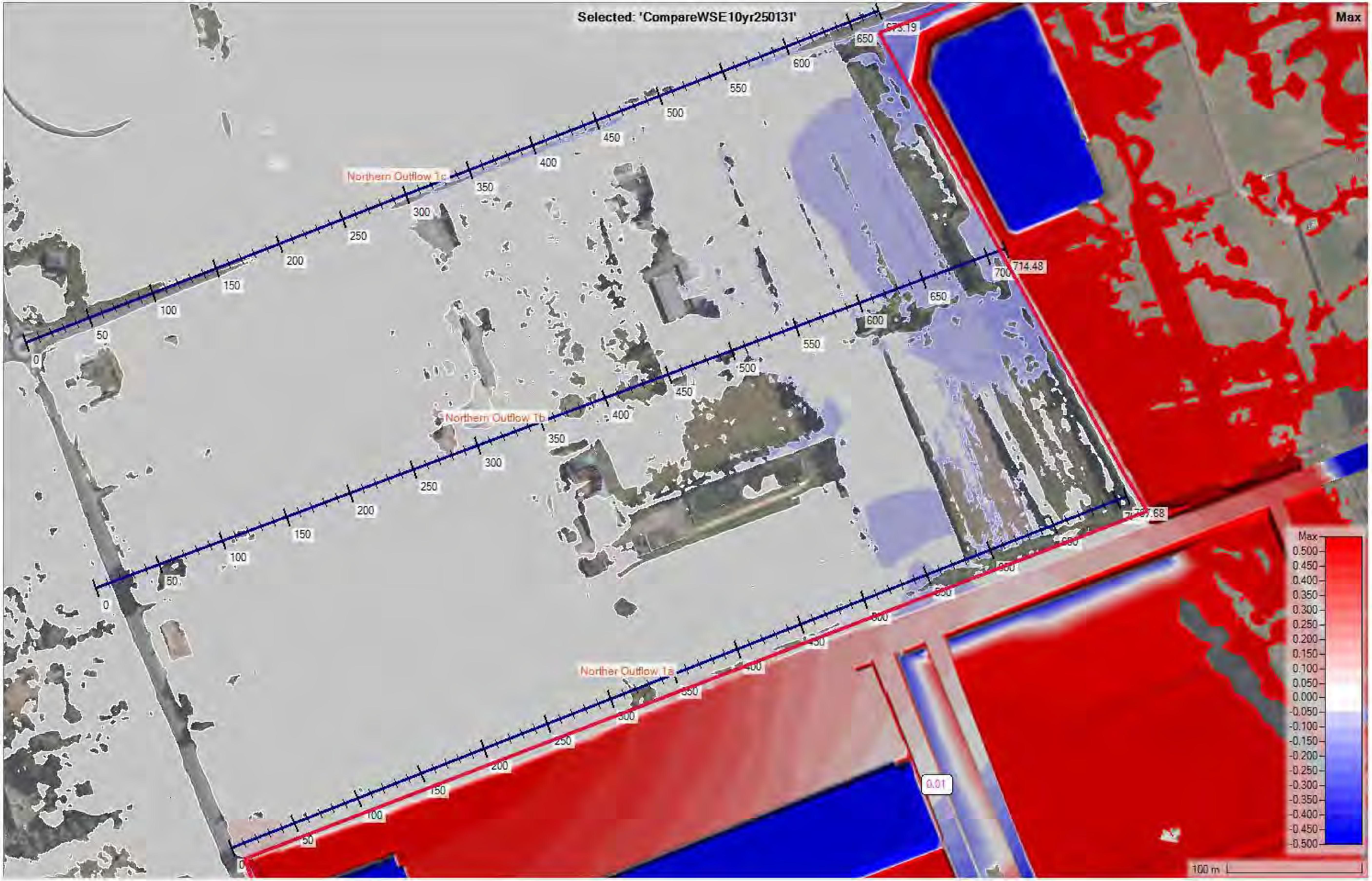


Eastern Catchment 10%AEP – Flooding level Comparison Extents (Post Development minus Existing) : ie blue = decrease and red = increase)

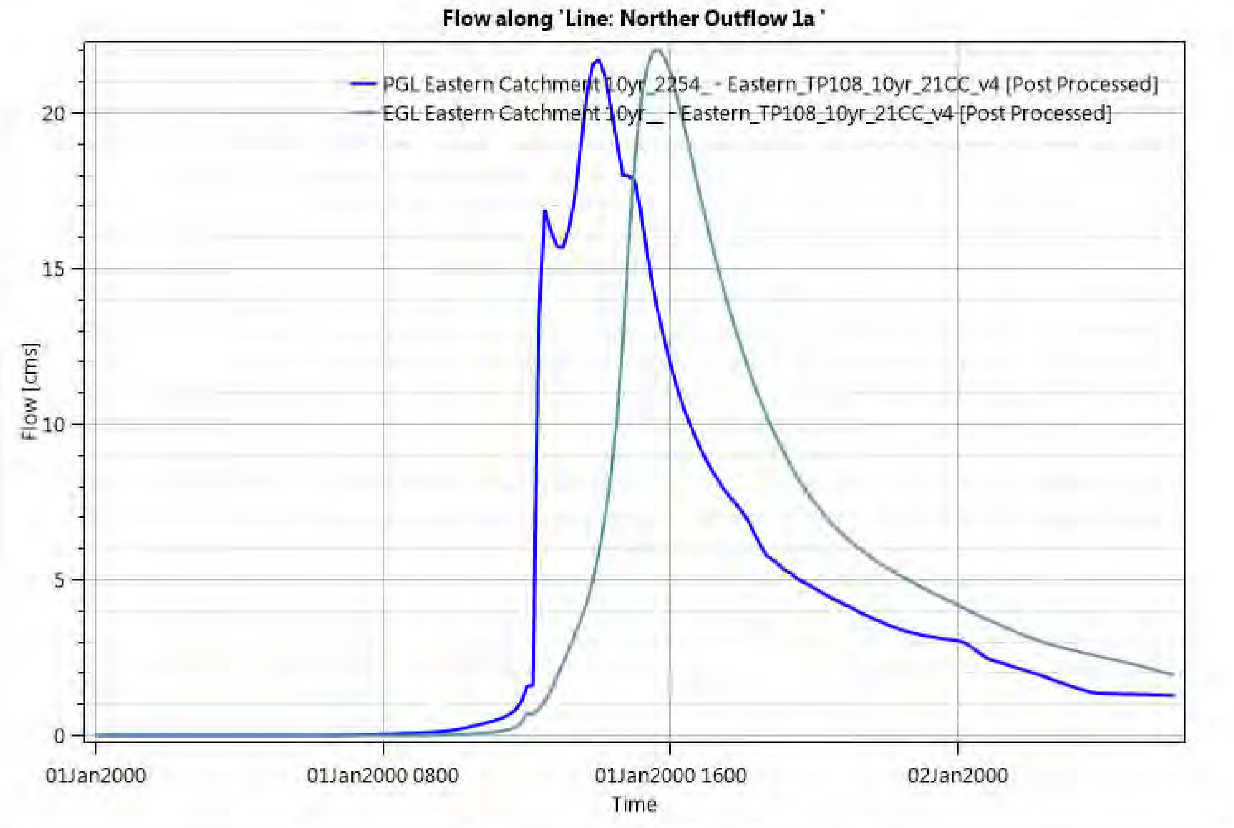
Eastern Catchment 10%AEP – Flooding level Comparison Extents (Post Development minus Existing) : ie blue = decrease and red = increase)

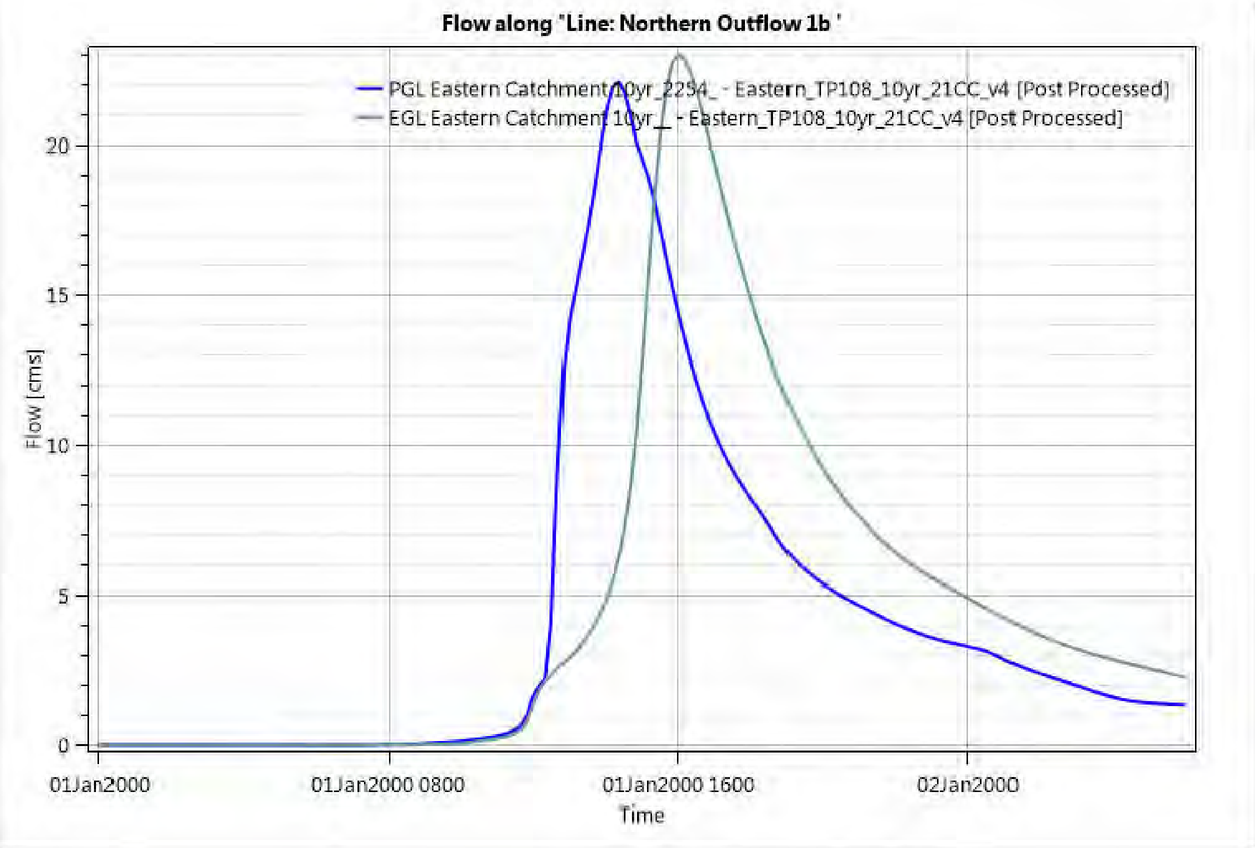


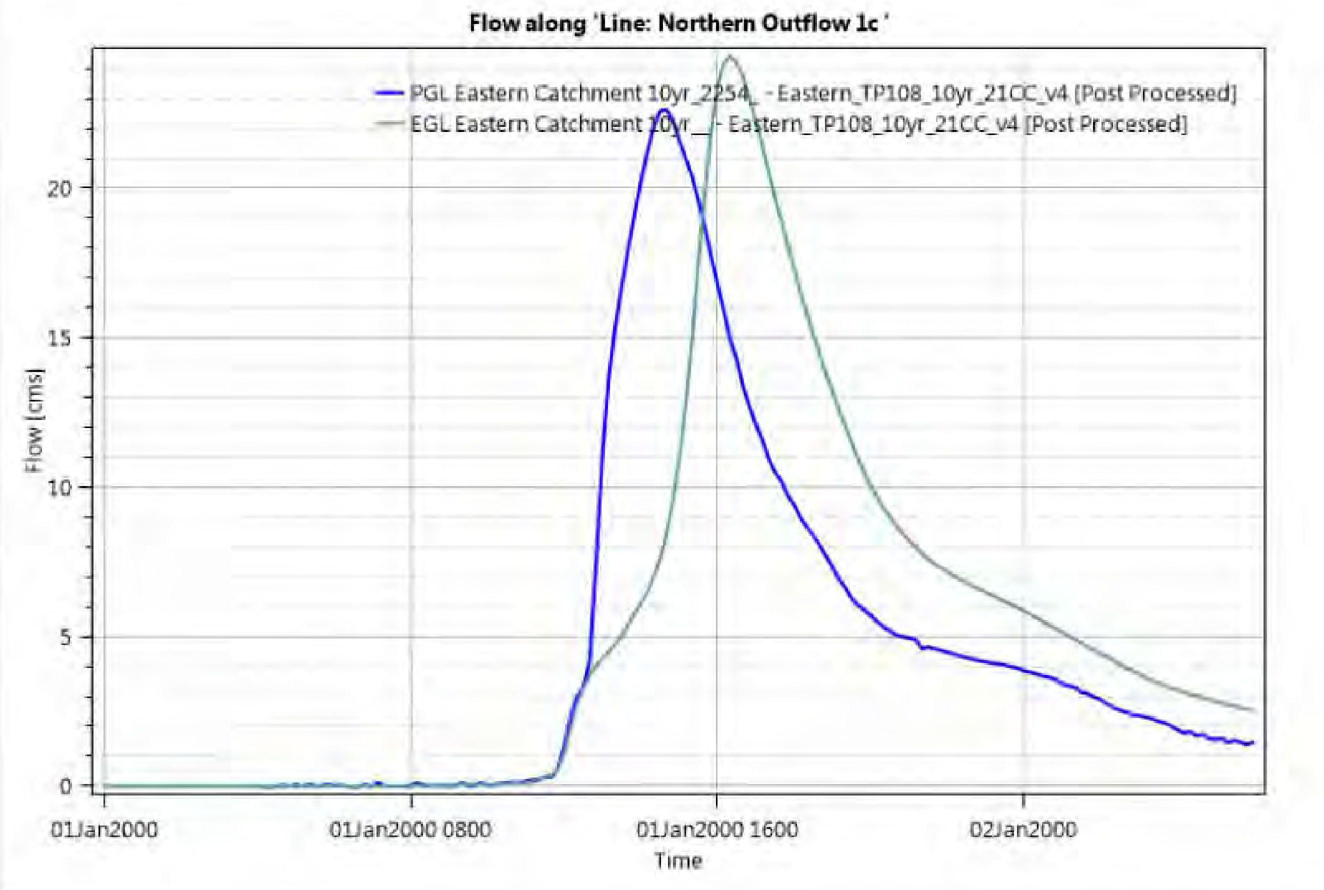
Eastern Catchment 10%AEP – Flooding level Comparison Extents (Post Development minus Existing) : ie blue = decrease and red = increase)



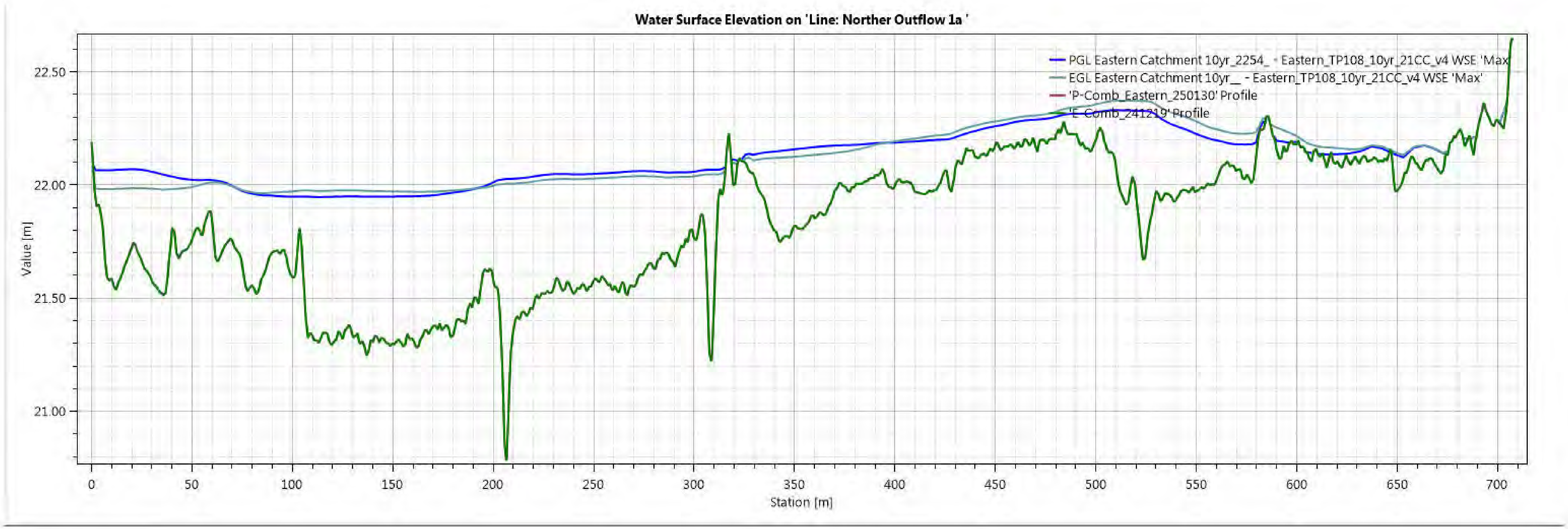
Eastern Catchment 10%AEP – Flow Comparison (Post Development versus Existing)



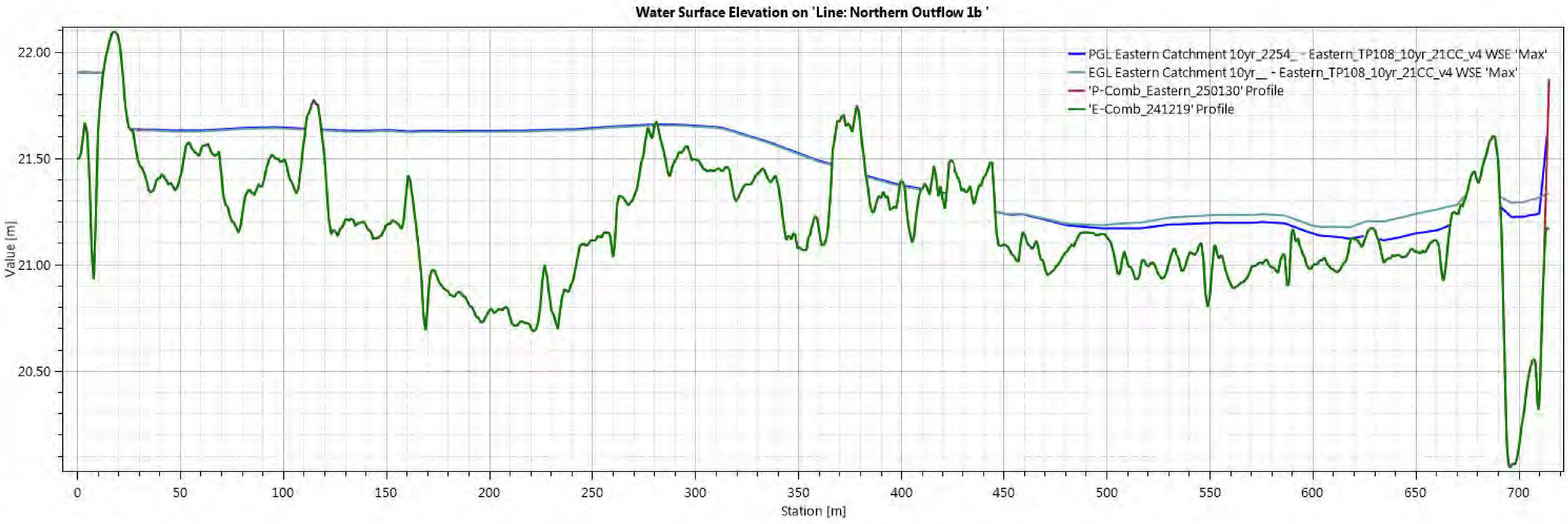




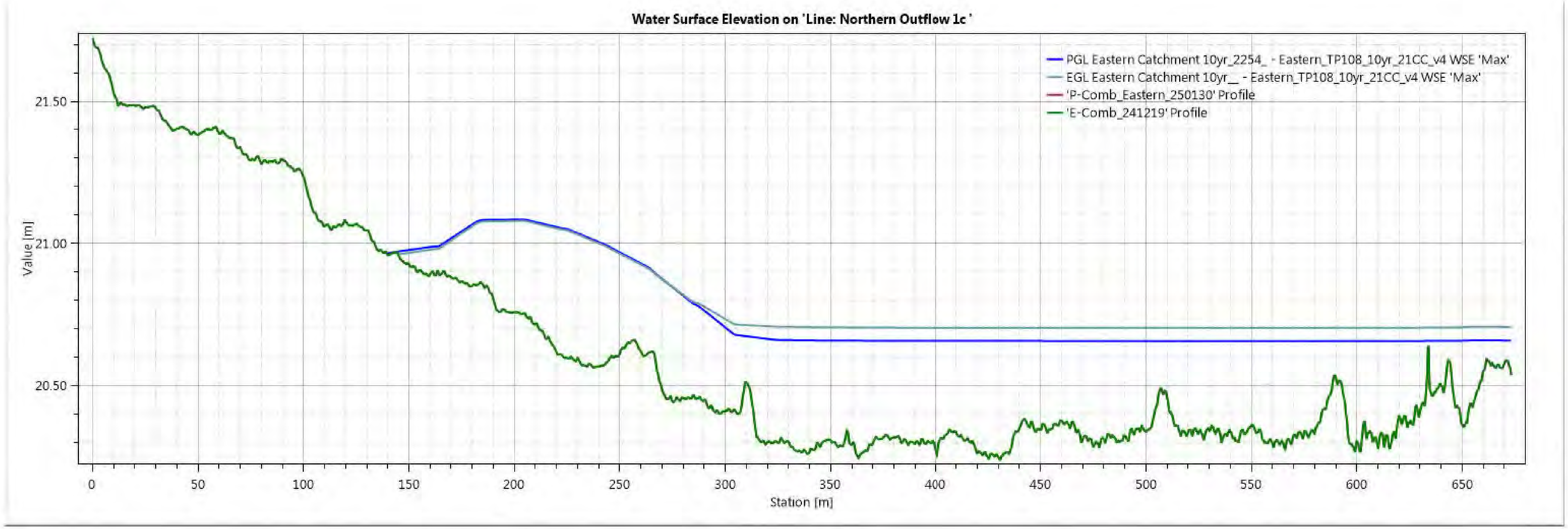
Eastern Catchment 10%AEP – Flood Level Comparison Extents - Section 1A



Eastern Catchment 10%AEP – Flood Level Comparison Extents - Section 1B

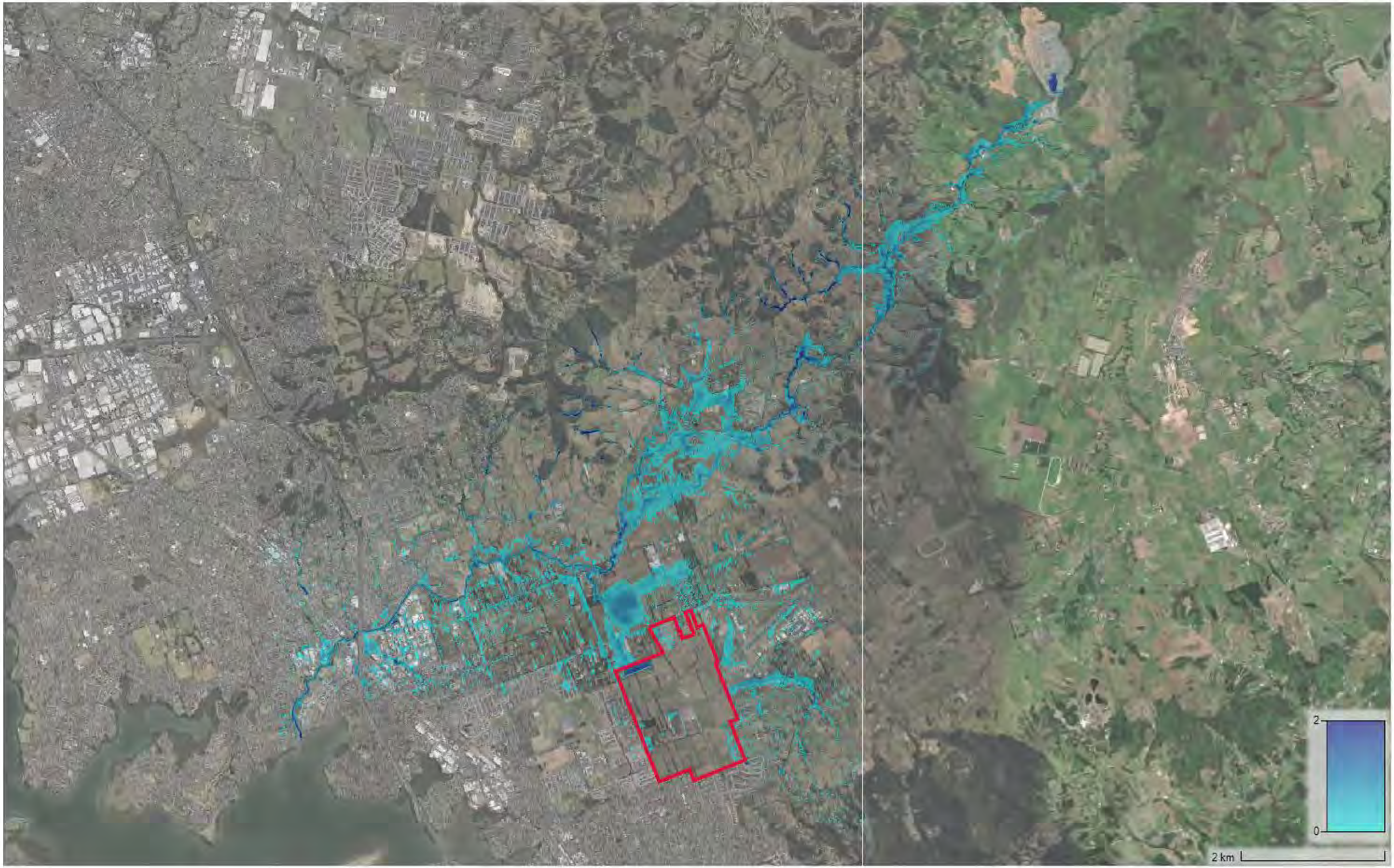


Eastern Catchment 10%AEP – Flood Level Comparison Extents - Section 1C



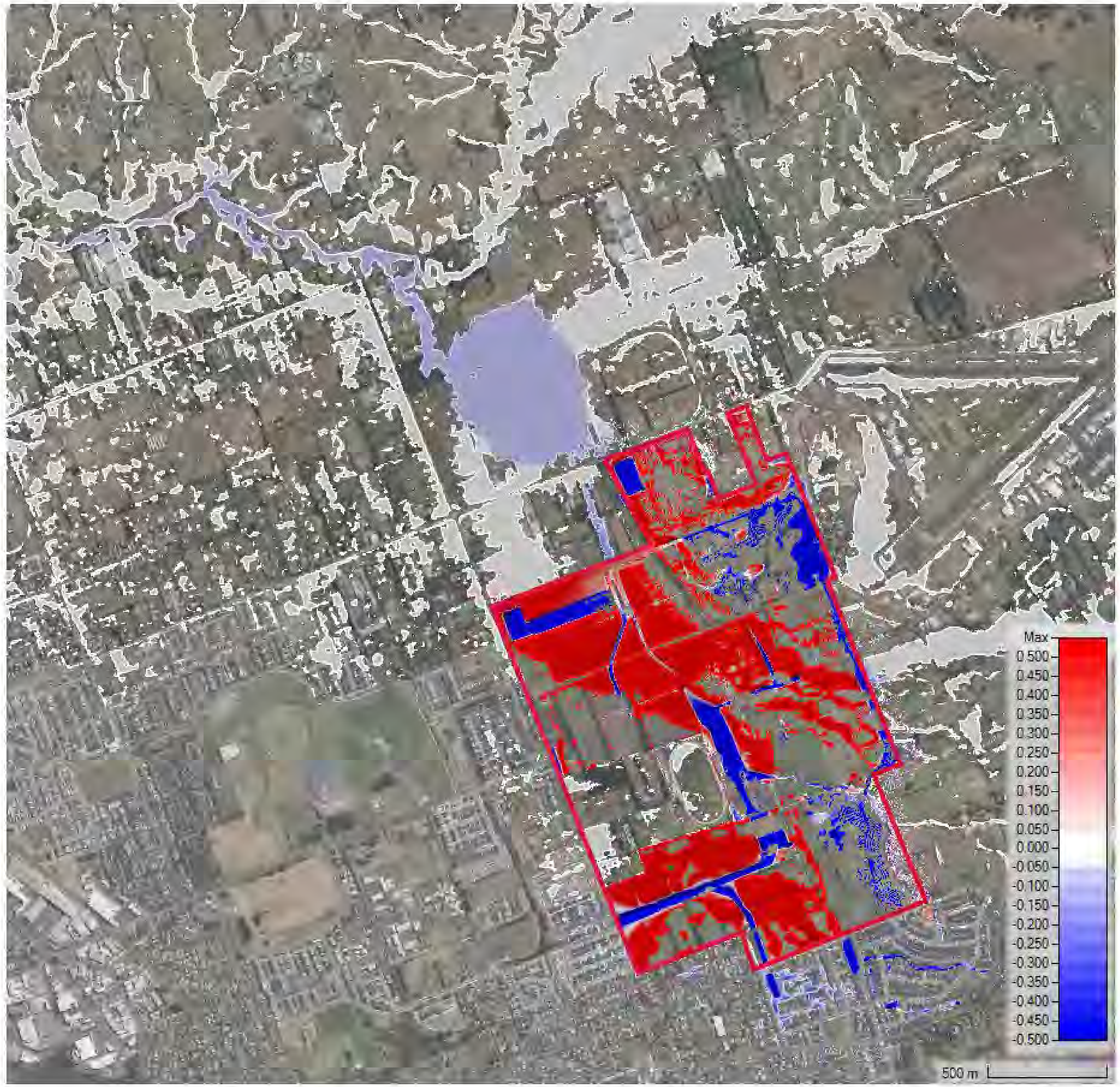
Eastern Catchment 50%AEP – Existing Flood Extents

Eastern Catchment 50%AEP – Post Development Flood Extents

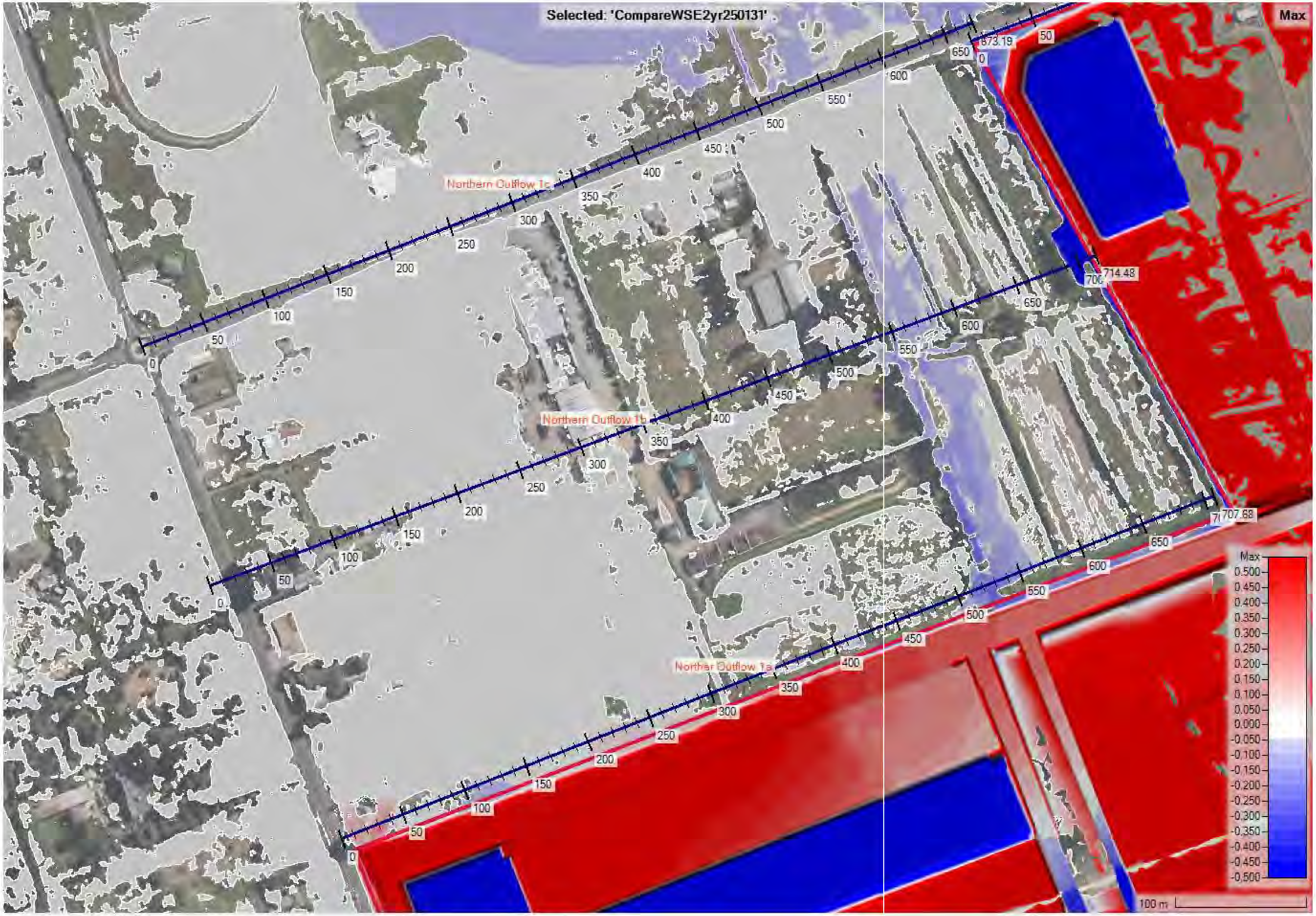


Eastern Catchment 50%AEP – Flooding level Comparison Extents (Post Development minus Existing) : ie blue = decrease and red = increase)

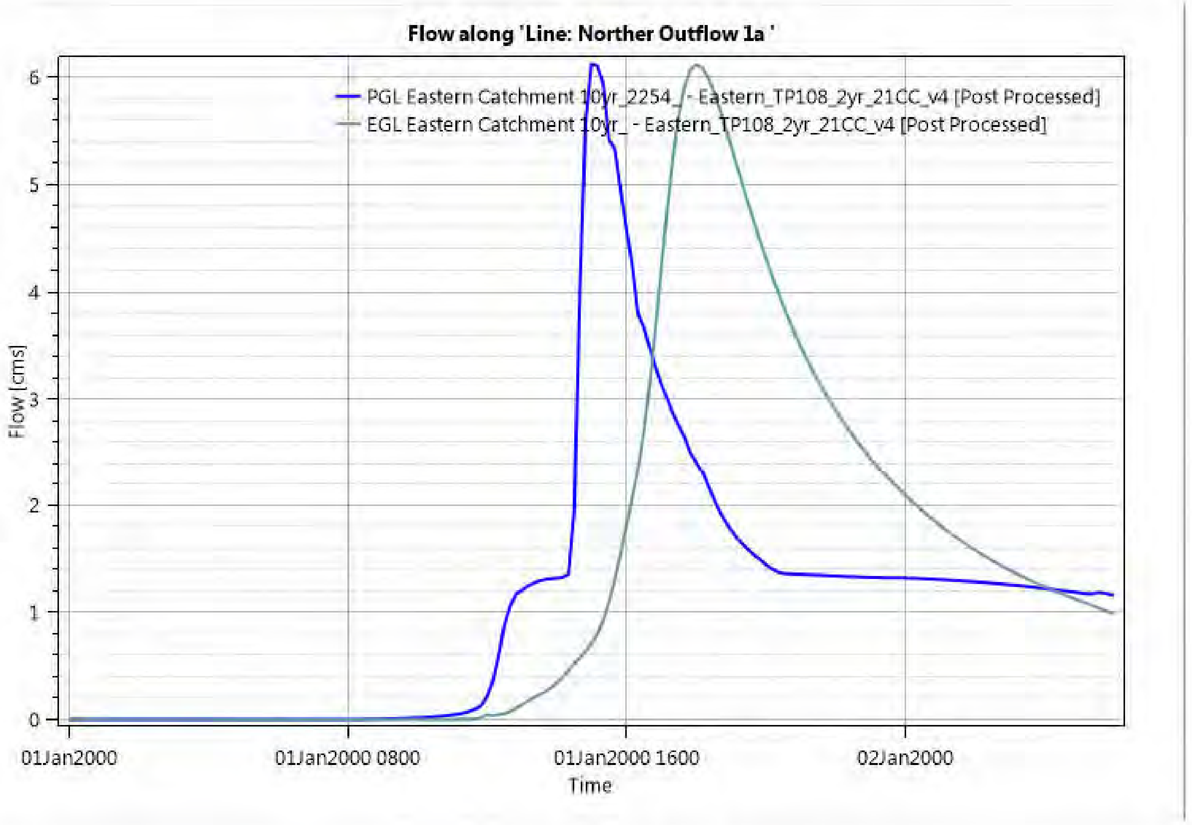
Eastern Catchment 10%AEP – Flooding level Comparison Extents (Post Development minus Existing) : ie blue = decrease and red = increase)

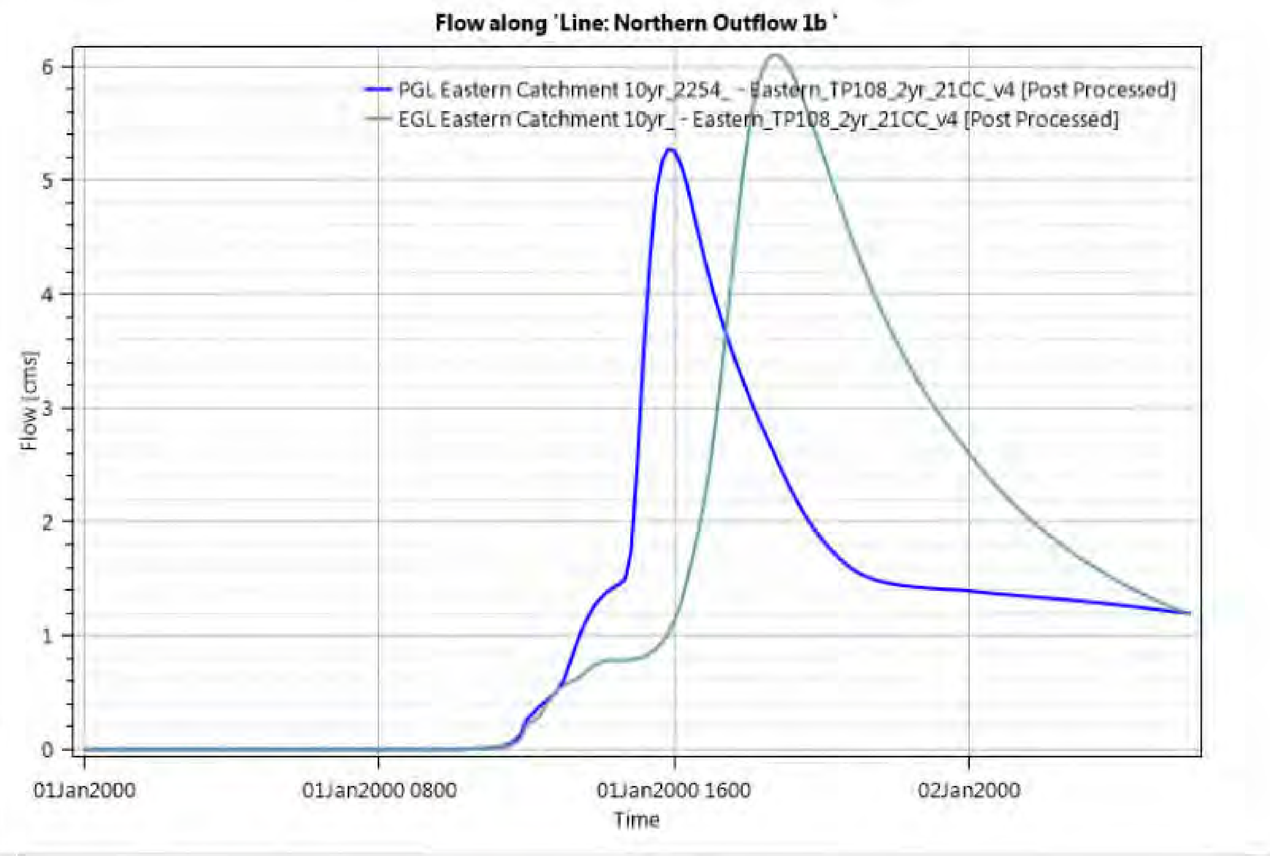


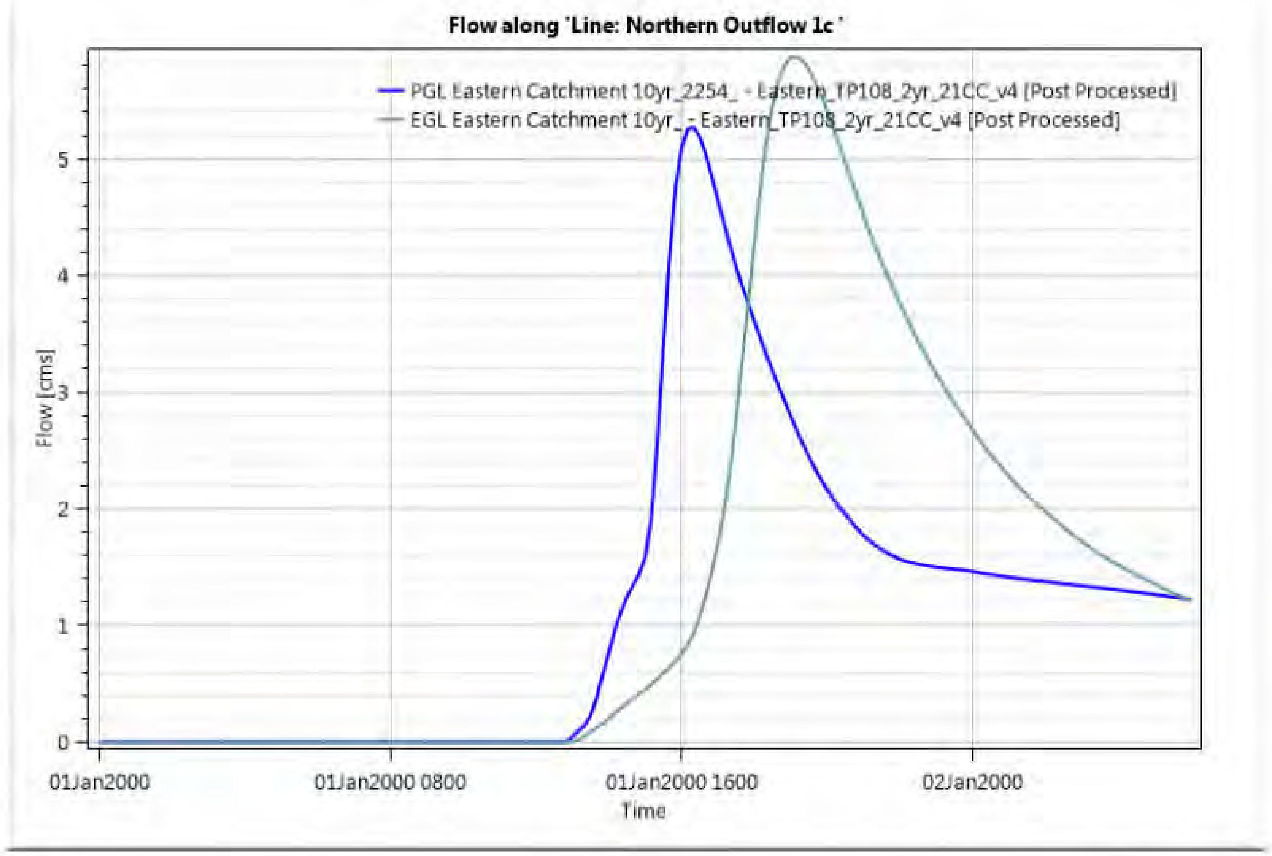
Eastern Catchment 10%AEP – Flooding level Comparison Extents (Post Development minus Existing) : ie blue = decrease and red = increase)



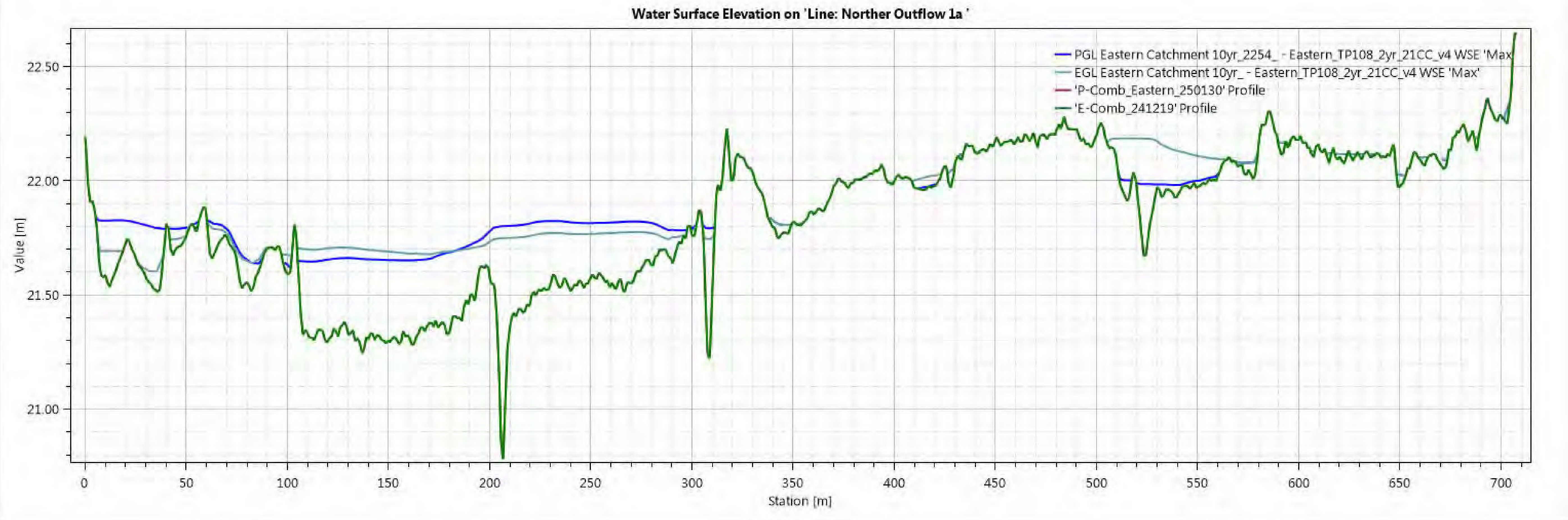
Eastern Catchment 50%AEP – Flow Comparison (Post Development versus Existing)



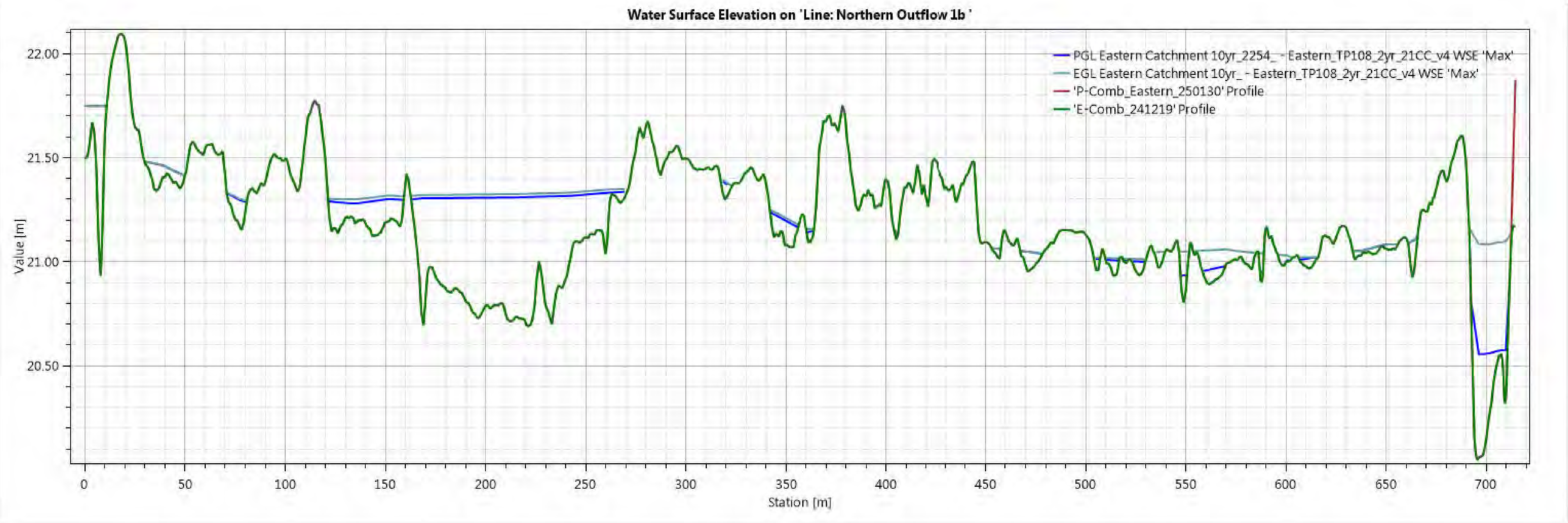




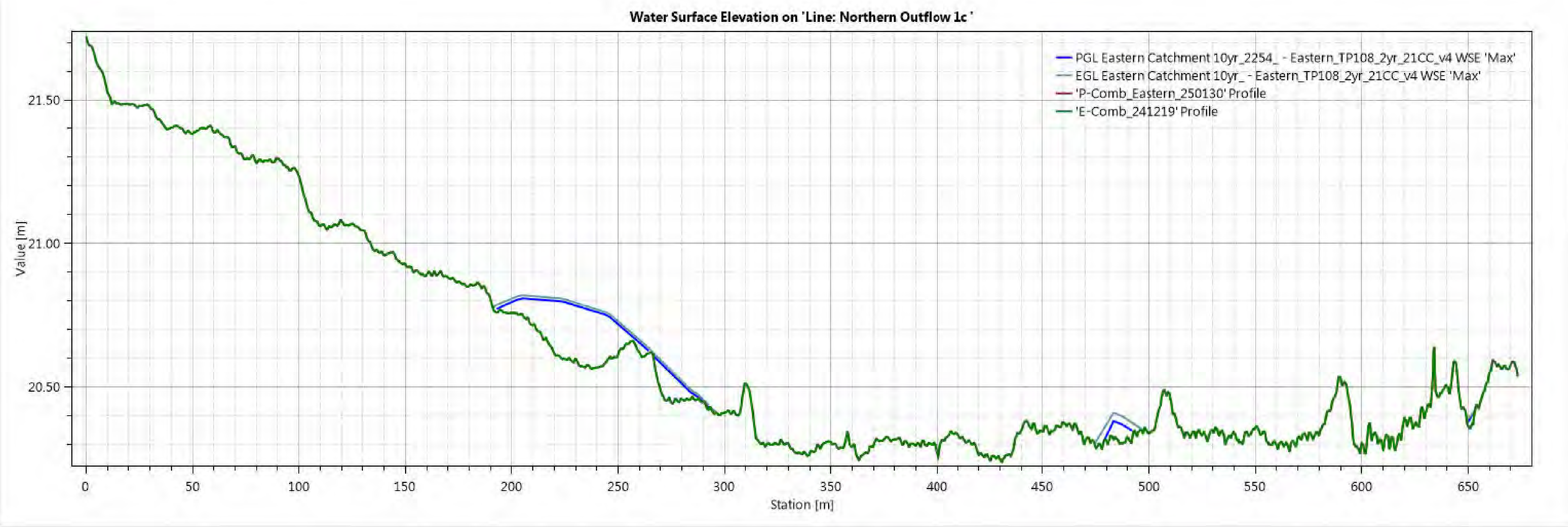
Eastern Catchment 50%AEP – Flood Level Comparison Extents - Section 1A



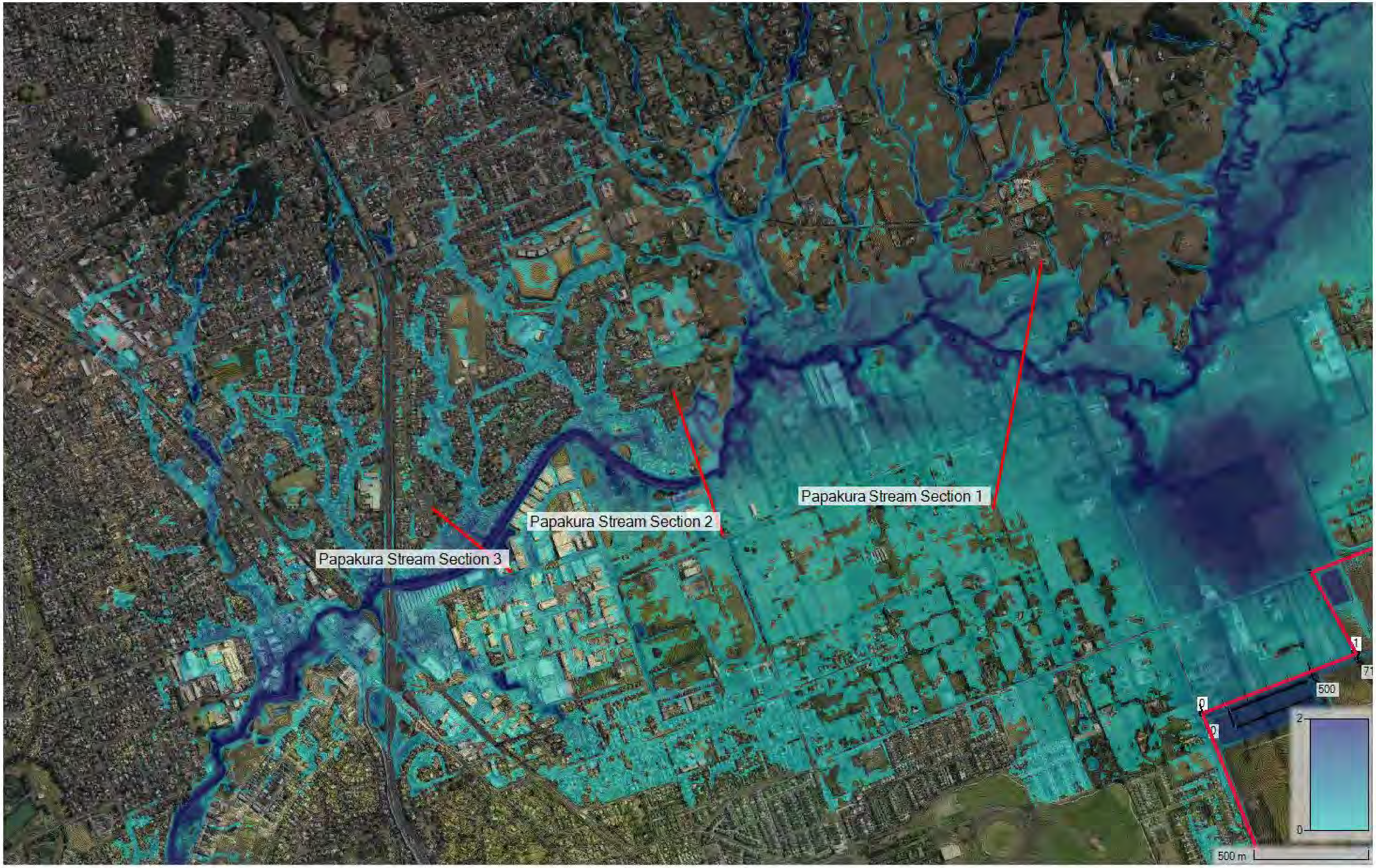
Eastern Catchment 50%AEP – Flood Level Comparison Extents - Section 1B



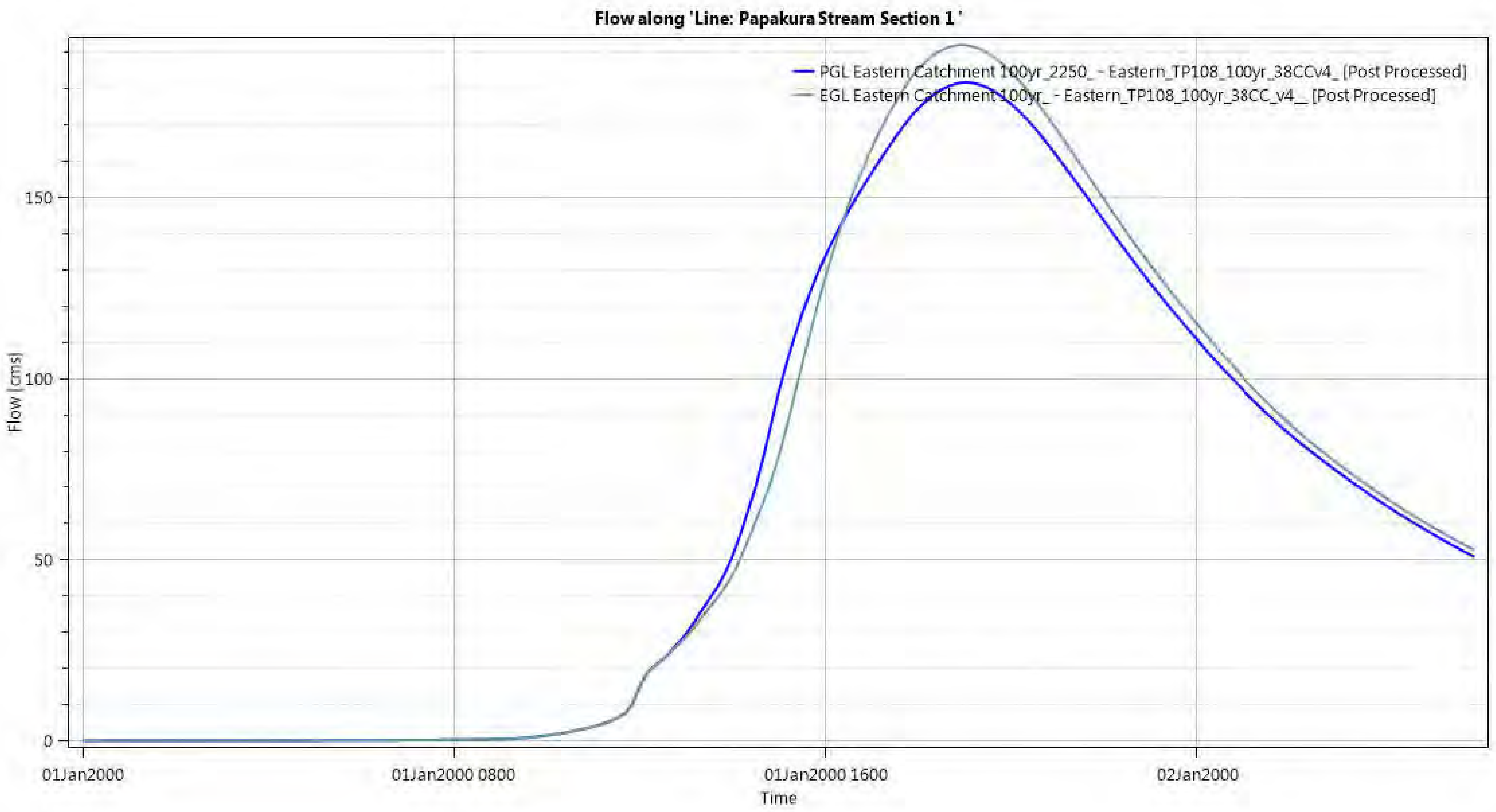
Eastern Catchment 50%AEP – Flood Level Comparison Extents - Section 1C



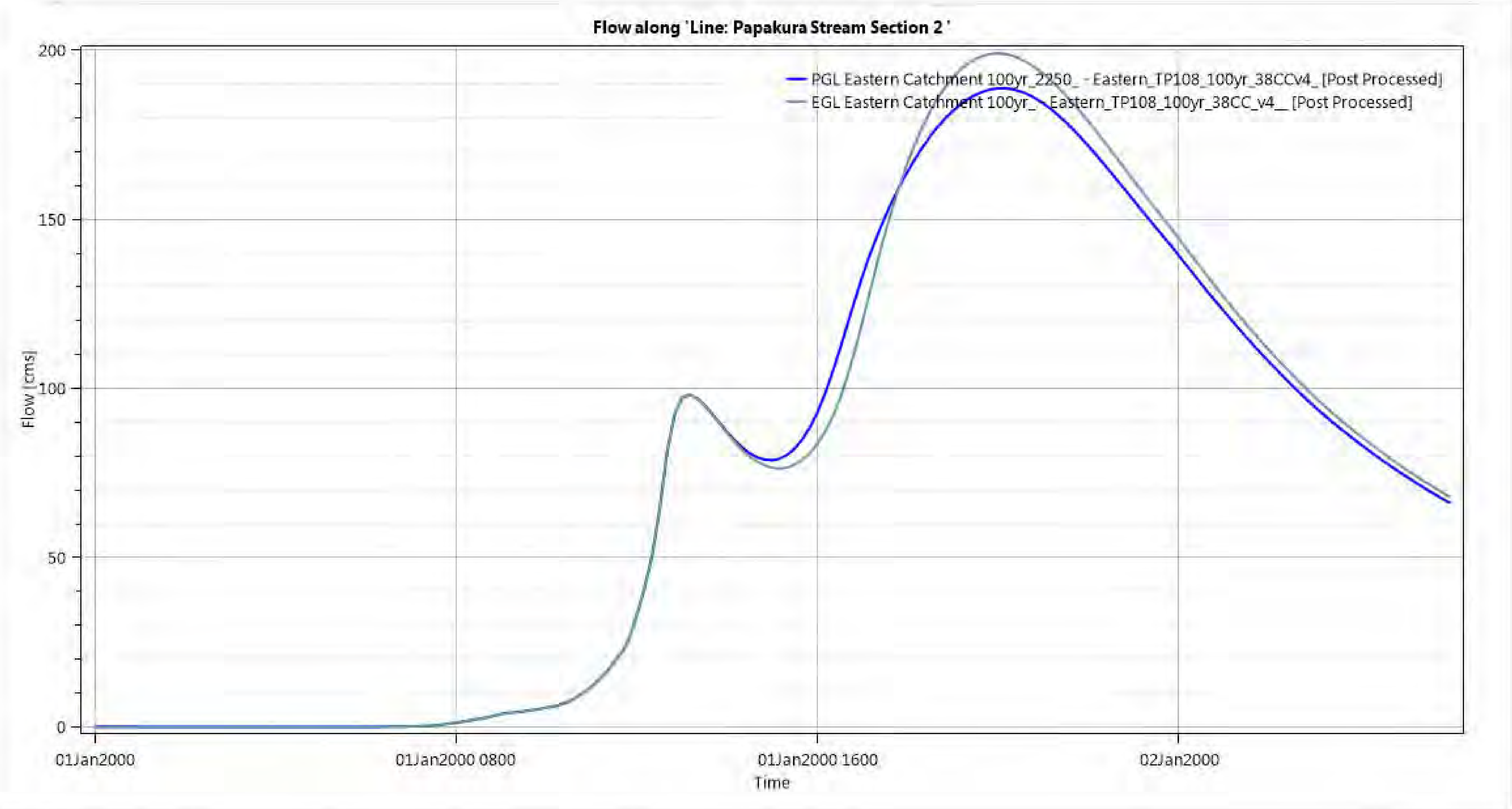
Eastern Catchment – Papakura Stream Peak Flow comparison (Cross section Locations)



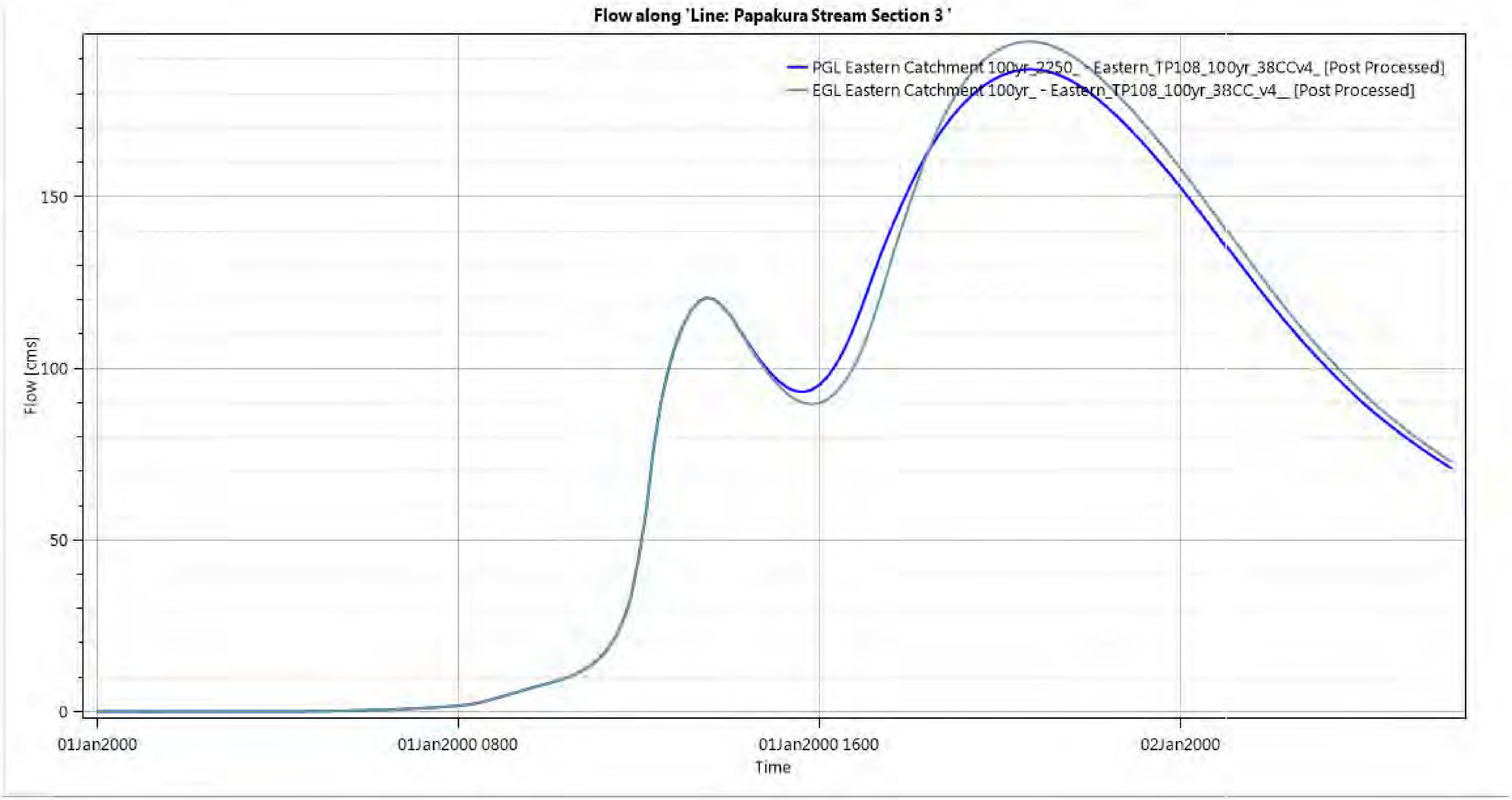
Eastern Catchment – 10% AEP Papakura Stream Peak Flow comparison (Papakura Cross section 1)



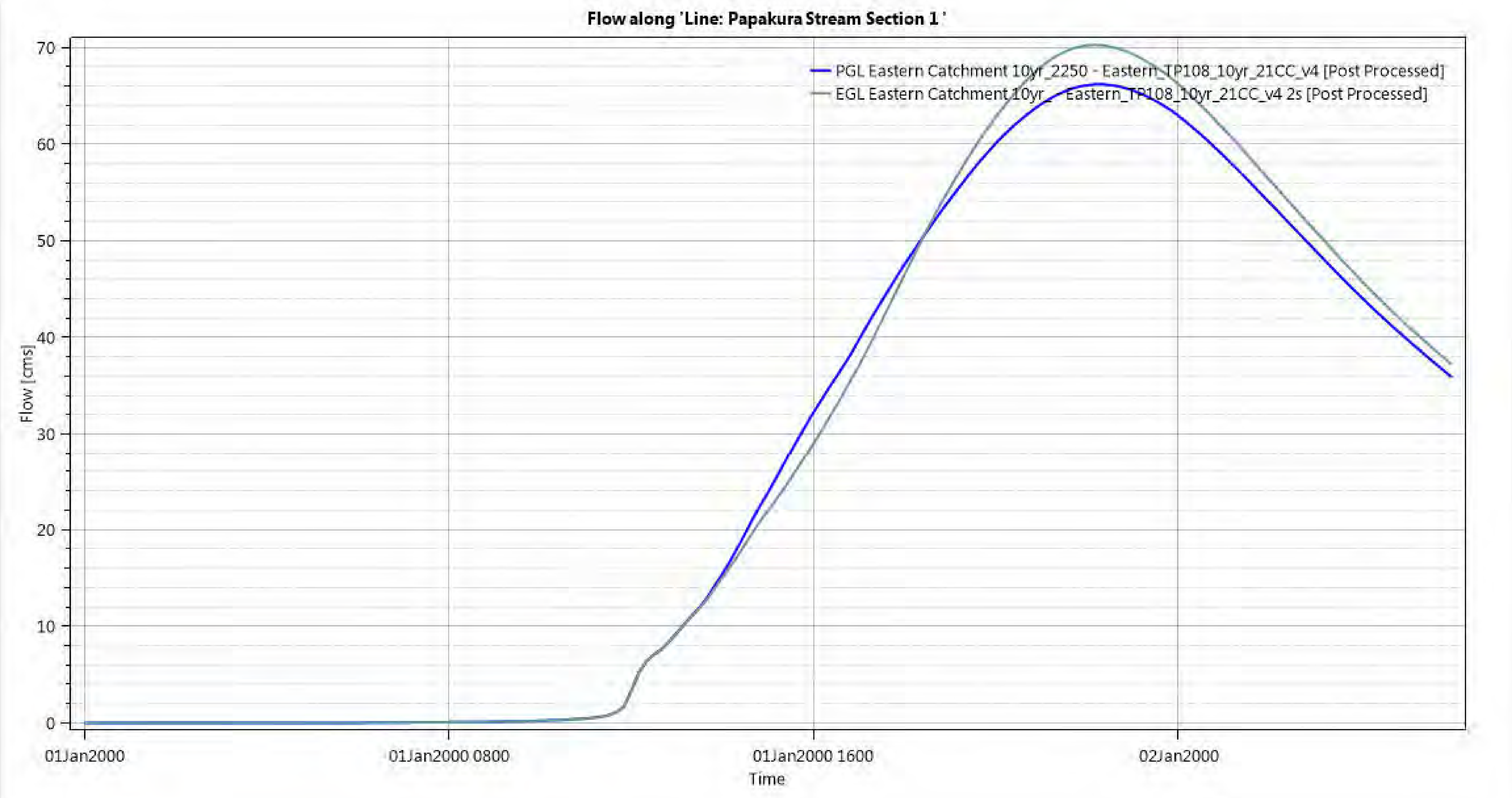
Eastern Catchment – 10% AEP Papakura Stream Peak Flow comparison (Papakura Cross section 2)



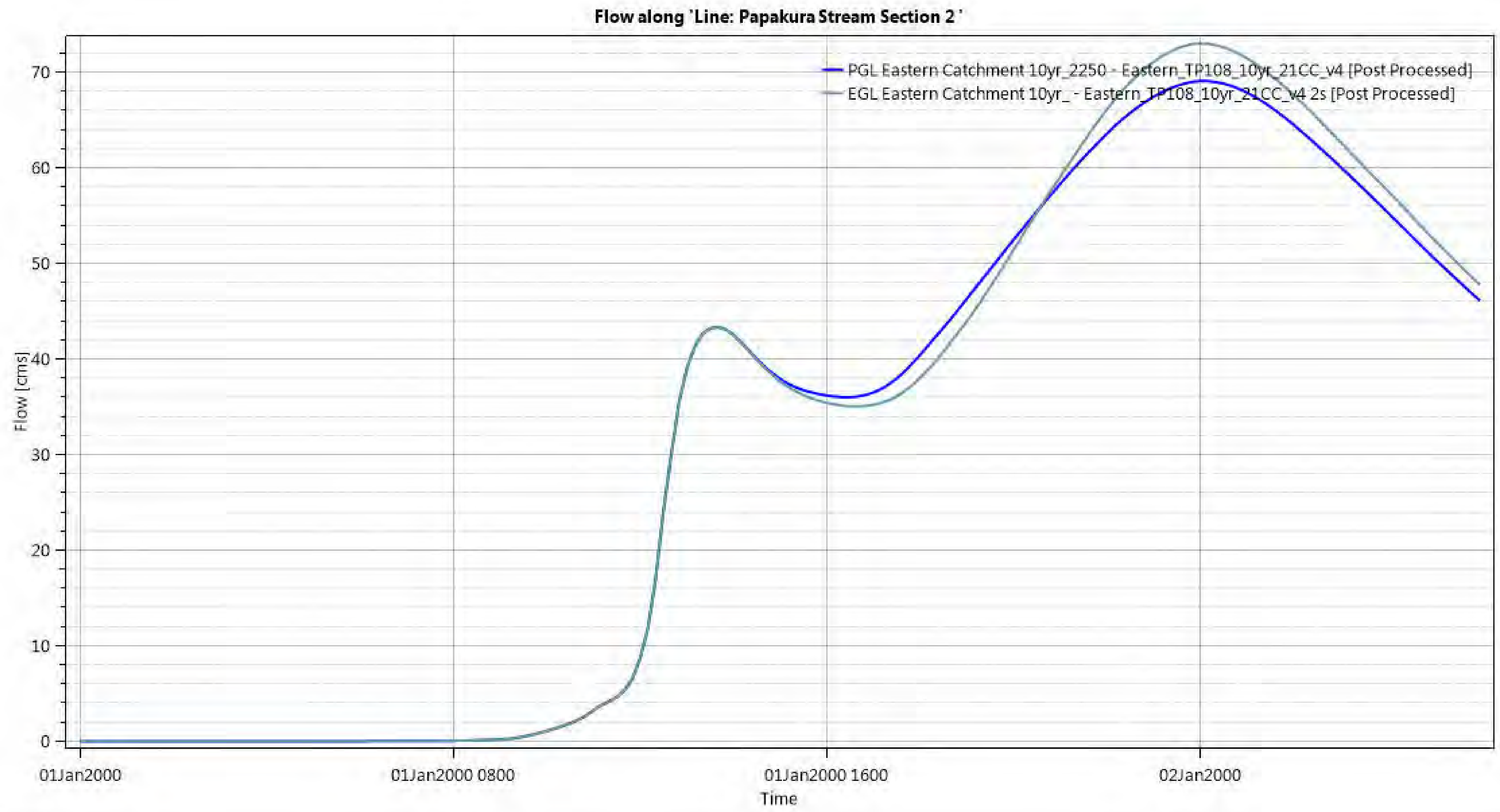
Eastern Catchment – 10% AEP Papakura Stream Peak Flow comparison (Papakura Cross section 3)



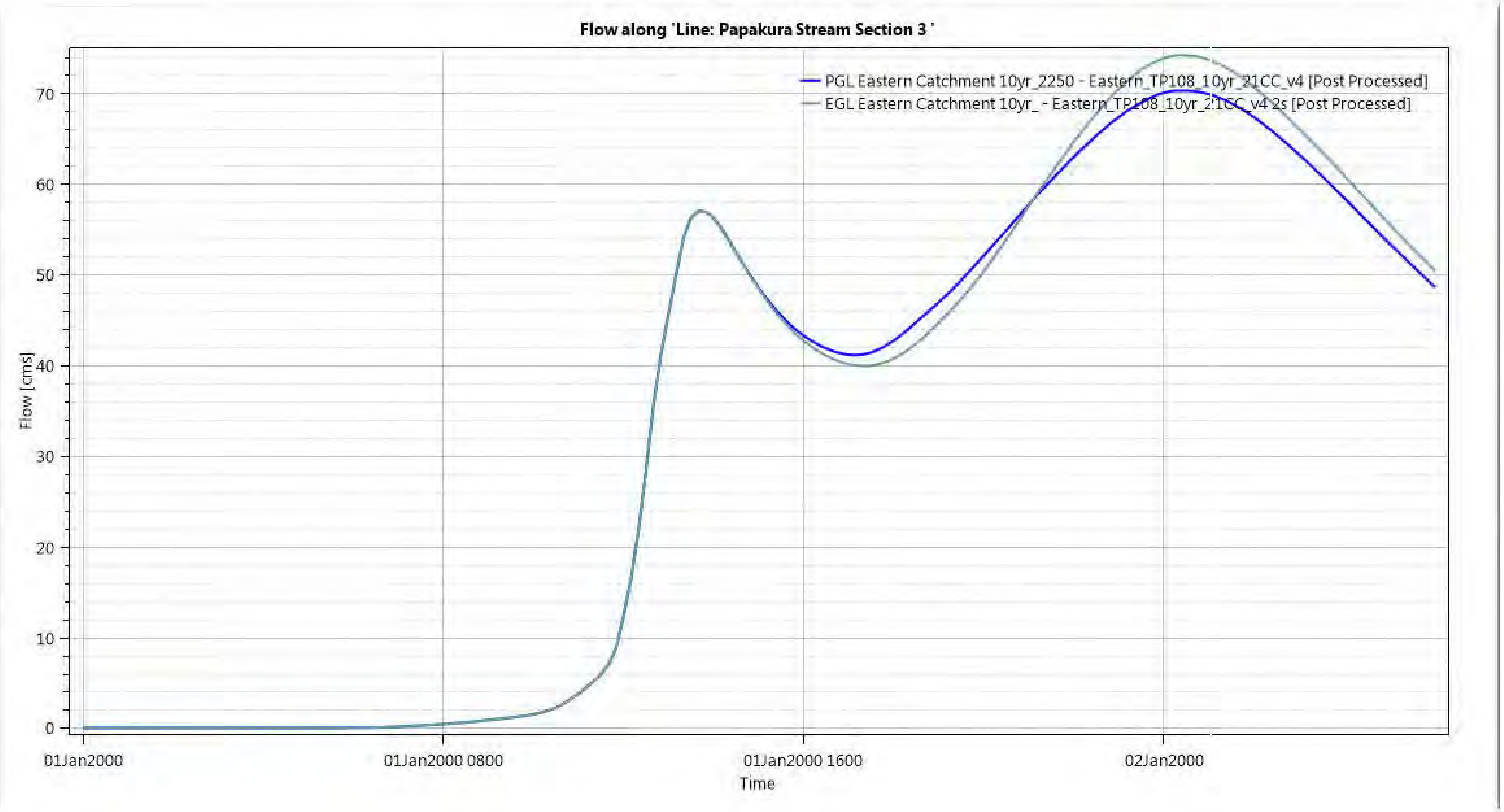
Eastern Catchment – 1% AEP Papakura Stream Peak Flow comparison (Papakura Cross section 1)



Eastern Catchment – 1% AEP Papakura Stream Peak Flow comparison (Papakura Cross section 2)



Eastern Catchment – 1% AEP Papakura Stream Peak Flow comparison (Papakura Cross section 3)



APPENDIX 8 – HMS EASTERN INFLOW HYDROGRAPHS

Eastern Catchment Proposed Scenario– HMS model setup for Inflow hydrograph

Eastern Catchment Proposed Scenario– HMS Inflow hydrograph summary 2yr Storm

Project: FAB_Swale_Sizing Simulation Run: 2yr_FAB v2

Start of Run: 01Jan2000, 00:00
End of Run: 03Jan2000, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

Basin Model: 2yr_Pr v2
Meteorologic Model: TP108_2yr_86mm
Control Specifications: 48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Junction-27	0.06420	0.65636	1 January 2000, 12:16	4.16871
Junction-28	0.06297	0.70676	1 January 2000, 12:14	4.08864
Junction-29	0.03433	0.41057	1 January 2000, 12:13	2.22892
Junction-30	0.03111	0.37784	1 January 2000, 12:12	2.02031
Junction-31	0.04699	0.53929	1 January 2000, 12:13	3.05104
Junction-32	0.05554	0.62268	1 January 2000, 12:14	3.60631
Junction-33	0.03631	0.41885	1 January 2000, 12:13	2.35798
Junction-34	0.04128	0.46899	1 January 2000, 12:14	2.68062
Junction-35	0.03693	0.43006	1 January 2000, 12:13	2.39777
Junction-36	0.06601	0.70589	1 January 2000, 12:15	4.28632
Junction-37	0.10365	1.27654	1 January 2000, 12:17	7.96574
Junction-38	0.07211	0.96316	1 January 2000, 12:14	5.54229
Junction-39	0.26627	1.06932	1 January 2000, 12:49	13.66252
Junction-40	0.00353	0.02959	1 January 2000, 12:12	0.13637
Junction-41	0.01849	0.12701	1 January 2000, 12:16	0.71343
Junction-50	0.01729	0.24395	1 January 2000, 12:13	1.32914
Junction-51	0.01933	0.29190	1 January 2000, 12:12	1.48554
Junction-52	0.00799	0.11110	1 January 2000, 12:13	0.59688
Junction-53	0.02187	0.32847	1 January 2000, 12:12	1.68102
Junction-54	0.02226	0.33106	1 January 2000, 12:12	1.71057
Junction-55	0.02434	0.36321	1 January 2000, 12:12	1.87081
Junction-56	0.04020	0.52847	1 January 2000, 12:15	3.08960
Junction-57	0.01759	0.26408	1 January 2000, 12:12	1.35148
Junction-58	0.00673	0.09327	1 January 2000, 12:13	0.50259
Reservoir-Swale_Storage	0.65143	1.24773	1 January 2000, 13:14	44.14202
Reservoir-SW_Pond_2	0.15274	0.05980	1 January 2000, 18:15	7.03891
Reservoir-SW_Pond_3	0.02755	0.07489	1 January 2000, 13:03	1.68934
Sink-Outflow_1	0.65143	1.24773	1 January 2000, 13:14	44.14202
Sink-Outflow_2	0.15274	0.05980	1 January 2000, 18:15	7.03891
Sink-Out_3	0.02755	0.07489	1 January 2000, 13:03	1.68934
Subbasin-Outflow 2 Ex	0.15274	0.82457	1 January 2000, 12:25	5.88644
Subbasin-Outflow 3 Ex	0.02755	0.17724	1 January 2000, 12:18	1.06193
Subbasin-27_Imp	0.03980	0.54490	1 January 2000, 12:15	3.22852
Subbasin-27_Perv	0.02440	0.13368	1 January 2000, 12:24	0.94019
Subbasin-28_Imp	0.03904	0.58008	1 January 2000, 12:13	3.16651
Subbasin-28_Perv	0.02393	0.14677	1 January 2000, 12:20	0.92213
Subbasin-29_Imp	0.02128	0.33416	1 January 2000, 12:12	1.72622
Subbasin-29_Perv	0.01304	0.08734	1 January 2000, 12:17	0.50270
Subbasin-30_Imp	0.01929	0.30585	1 January 2000, 12:12	1.56466
Subbasin-30_Perv	0.01182	0.08114	1 January 2000, 12:16	0.45565
Subbasin-31_Imp	0.02913	0.44223	1 January 2000, 12:13	2.36293
Subbasin-31_Perv	0.01786	0.11292	1 January 2000, 12:19	0.68812

Project: FAB_Swale_Sizing   Simulation Run: 2yr_FAB v2

Start of Run: 01Jan2000, 00:00   Basin Model: 2yr_Pr v2
End of Run: 03Jan2000, 00:00   Meteorologic Model: TP108_2yr_86mm
Compute Time:DATA CHANGED, RECOMPUTE   Control Specifications:48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-32_Imp	0.03443	0.51165	1 January 2000, 12:13	2.79296
Subbasin-32_Perv	0.02110	0.12911	1 January 2000, 12:20	0.81335
Subbasin-33_Imp	0.02251	0.34283	1 January 2000, 12:13	1.82617
Subbasin-33_Perv	0.01380	0.08786	1 January 2000, 12:18	0.53181
Subbasin-34_Imp	0.02560	0.38465	1 January 2000, 12:13	2.07604
Subbasin-34_Perv	0.01569	0.09806	1 January 2000, 12:19	0.60457
Subbasin-35_Imp	0.02289	0.35065	1 January 2000, 12:13	1.85699
Subbasin-35_Perv	0.01403	0.09055	1 January 2000, 12:18	0.54078
Subbasin-36_Imp	0.04093	0.58353	1 January 2000, 12:14	3.31961
Subbasin-36_Perv	0.02508	0.14468	1 January 2000, 12:22	0.96672
Subbasin-37_Imp	0.09328	1.23271	1 January 2000, 12:16	7.56629
Subbasin-37_Perv	0.01036	0.05418	1 January 2000, 12:26	0.39945
Subbasin-38_Imp	0.06490	0.92911	1 January 2000, 12:14	5.26436
Subbasin-38_Perv	0.00721	0.04181	1 January 2000, 12:22	0.27792
Subbasin-39_Imp	0.07988	0.64850	1 January 2000, 12:40	6.47924
Subbasin-39_Perv	0.18639	0.54936	1 January 2000, 13:13	7.18328
Subbasin-40_Imp	0.00000	0.00006	1 January 2000, 12:10	0.00029
Subbasin-40_Perv	0.00353	0.02953	1 January 2000, 12:12	0.13608
Subbasin-41_Imp	0.00002	0.00029	1 January 2000, 12:12	0.00150
Subbasin-41_Perv	0.01847	0.12677	1 January 2000, 12:16	0.71193
Subbasin-50_Imp	0.01557	0.23473	1 January 2000, 12:13	1.26249
Subbasin-50_Perv	0.00173	0.01084	1 January 2000, 12:19	0.06665
Subbasin-51_Imp	0.01740	0.27977	1 January 2000, 12:11	1.41105
Subbasin-51_Perv	0.00193	0.01358	1 January 2000, 12:15	0.07449
Subbasin-52_Imp	0.00679	0.10427	1 January 2000, 12:13	0.55070
Subbasin-52_Perv	0.00120	0.00776	1 January 2000, 12:18	0.04617
Subbasin-53_Imp	0.01969	0.31476	1 January 2000, 12:12	1.59673
Subbasin-53_Perv	0.00219	0.01523	1 January 2000, 12:16	0.08430
Subbasin-54_Imp	0.02003	0.31760	1 January 2000, 12:12	1.62479
Subbasin-54_Perv	0.00223	0.01522	1 January 2000, 12:16	0.08578
Subbasin-55_Imp	0.02191	0.34839	1 January 2000, 12:12	1.77700
Subbasin-55_Perv	0.00243	0.01670	1 January 2000, 12:16	0.09381
Subbasin-56_Imp	0.03618	0.50956	1 January 2000, 12:15	2.93467
Subbasin-56_Perv	0.00402	0.02275	1 January 2000, 12:23	0.15493
Subbasin-57_Imp	0.01583	0.25306	1 January 2000, 12:12	1.28371
Subbasin-57_Perv	0.00176	0.01224	1 January 2000, 12:16	0.06777
Subbasin-58_Imp	0.00572	0.08756	1 January 2000, 12:13	0.46371
Subbasin-58_Perv	0.00101	0.00651	1 January 2000, 12:18	0.03888

Eastern Catchment Proposed Scenario – HMS Inflow hydrograph summary 10yr Storm

Project: FAB_Swale_Sizing   Simulation Run: 10yr_FAB v2

Start of Run: 01Jan2000, 00:00   Basin Model: 10yr_Pr v2
End of Run: 03Jan2000, 00:00   Meteorologic Model: TP108_10yr_170mm_CoPv4
Compute Time: 24Jan2025, 20:51:53   Control Specifications: 48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Junction-27	0.06374	1.40175	1 January 2000, 12:16	9.11297
Junction-28	0.06002	1.45357	1 January 2000, 12:14	8.58151
Junction-29	0.03304	0.85538	1 January 2000, 12:13	4.72400
Junction-30	0.02764	0.72596	1 January 2000, 12:12	3.95179
Junction-31	0.04300	1.06547	1 January 2000, 12:14	6.14812
Junction-32	0.05227	1.26401	1 January 2000, 12:14	7.47309
Junction-33	0.03284	0.81733	1 January 2000, 12:13	4.69582
Junction-34	0.03798	0.93207	1 January 2000, 12:14	5.42957
Junction-35	0.03445	0.86639	1 January 2000, 12:13	4.92561
Junction-36	0.06341	1.46027	1 January 2000, 12:15	9.06586
Junction-37	0.10002	2.49703	1 January 2000, 12:17	15.92133
Junction-38	0.07191	1.94717	1 January 2000, 12:14	11.44670
Junction-39	0.26627	2.57331	1 January 2000, 12:52	33.13708
Junction-41	0.01849	0.35950	1 January 2000, 12:16	1.98117
Junction-50	0.01729	0.49487	1 January 2000, 12:13	2.75294
Junction-52	0.00799	0.22763	1 January 2000, 12:13	1.24836
Junction-53	0.02187	0.66694	1 January 2000, 12:12	3.48177
Junction-54	0.02226	0.67202	1 January 2000, 12:12	3.54296
Junction-55	0.02434	0.73730	1 January 2000, 12:12	3.87485
Junction-55_56_Culv...	0.06454	1.76968	1 January 2000, 12:13	10.27408
Junction-56	0.04020	1.07148	1 January 2000, 12:15	6.39923
Junction-57	0.01759	0.53619	1 January 2000, 12:12	2.79921
Reservoir-Swale_Stor...	0.88660	16.04138	1 January 2000, 12:16	124.17540
Reservoir-SW_Pond_2	0.15274	0.63895	1 January 2000, 13:10	17.37989
Sink-Outflow_1	0.88660	16.04138	1 January 2000, 12:16	124.17540
Sink-Outflow_2	0.15274	0.63895	1 January 2000, 13:10	17.37989
Subbasin-27_Imp	0.03952	1.07932	1 January 2000, 12:15	6.51934
Subbasin-27_Perv	0.02422	0.37727	1 January 2000, 12:23	2.59363
Subbasin-28_Imp	0.03721	1.10292	1 January 2000, 12:13	6.13913
Subbasin-28_Perv	0.02281	0.39732	1 January 2000, 12:19	2.44238
Subbasin-29_Imp	0.02049	0.64139	1 January 2000, 12:12	3.37950
Subbasin-29_Perv	0.01256	0.23799	1 January 2000, 12:17	1.34450
Subbasin-30_Imp	0.01714	0.54174	1 January 2000, 12:12	2.82707
Subbasin-30_Perv	0.01050	0.20414	1 January 2000, 12:16	1.12472
Subbasin-31_Imp	0.02666	0.80710	1 January 2000, 12:13	4.39831
Subbasin-31_Perv	0.01634	0.29336	1 January 2000, 12:18	1.74981
Subbasin-32_Imp	0.03241	0.96046	1 January 2000, 12:13	5.34617
Subbasin-32_Perv	0.01986	0.34493	1 January 2000, 12:19	2.12692
Subbasin-33_Imp	0.02036	0.61833	1 January 2000, 12:13	3.35934
Subbasin-33_Perv	0.01248	0.22551	1 January 2000, 12:18	1.33648
Subbasin-34_Imp	0.02355	0.70570	1 January 2000, 12:13	3.88426
Subbasin-34_Perv	0.01443	0.25578	1 January 2000, 12:19	1.54531

Project: FAB_Swale_Sizing   Simulation Run: 10yr_FAB v2

Start of Run: 01Jan2000, 00:00   Basin Model: 10yr_Pr v2
End of Run: 03Jan2000, 00:00   Meteorologic Model: TP108_10yr_170mm_CoPv4
Compute Time: 24Jan2025, 20:51:53   Control Specifications: 48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Subbasin-35_Imp	0.02136	0.65233	1 January 2000, 12:13	3.52374
Subbasin-35_Perv	0.01309	0.23940	1 January 2000, 12:18	1.40187
Subbasin-36_Imp	0.03931	1.11818	1 January 2000, 12:14	6.48564
Subbasin-36_Perv	0.02410	0.39476	1 January 2000, 12:21	2.58023
Subbasin-37_Imp	0.09002	2.37344	1 January 2000, 12:16	14.85029
Subbasin-37_Perv	0.01000	0.14868	1 January 2000, 12:25	1.07104
Subbasin-38_Imp	0.06472	1.84809	1 January 2000, 12:14	10.67667
Subbasin-38_Perv	0.00719	0.11847	1 January 2000, 12:21	0.77003
Subbasin-39_Imp	0.07988	1.29502	1 January 2000, 12:40	13.17798
Subbasin-39_Perv	0.18639	1.56246	1 January 2000, 13:11	19.95911
Subbasin-40_Imp	0.00000	0.00012	1 January 2000, 12:10	0.00058
Subbasin-40_Perv	0.00353	0.08278	1 January 2000, 12:11	0.37811
Subbasin-41_Imp	0.00002	0.00059	1 January 2000, 12:12	0.00305
Subbasin-41_Perv	0.01847	0.35904	1 January 2000, 12:16	1.97812
Subbasin-50_Imp	0.01557	0.46814	1 January 2000, 12:13	2.56775
Subbasin-50_Perv	0.00173	0.03074	1 January 2000, 12:18	0.18519
Subbasin-52_Imp	0.00679	0.20791	1 January 2000, 12:13	1.12006
Subbasin-52_Perv	0.00120	0.02197	1 January 2000, 12:18	0.12830
Subbasin-53_Imp	0.01969	0.62752	1 January 2000, 12:12	3.24754
Subbasin-53_Perv	0.00219	0.04305	1 January 2000, 12:16	0.23422
Subbasin-54_Imp	0.02003	0.63325	1 January 2000, 12:12	3.30462
Subbasin-54_Perv	0.00223	0.04311	1 January 2000, 12:16	0.23833
Subbasin-55_Imp	0.02191	0.69461	1 January 2000, 12:12	3.61419
Subbasin-55_Perv	0.00243	0.04731	1 January 2000, 12:16	0.26066
Subbasin-56_Imp	0.03618	1.01637	1 January 2000, 12:15	5.96875
Subbasin-56_Perv	0.00402	0.06470	1 January 2000, 12:22	0.43048
Subbasin-57_Imp	0.01583	0.50451	1 January 2000, 12:12	2.61091
Subbasin-57_Perv	0.00176	0.03461	1 January 2000, 12:16	0.18830
Subbasin-58_Imp	0.00572	0.17460	1 January 2000, 12:13	0.94313
Subbasin-58_Perv	0.00101	0.01845	1 January 2000, 12:18	0.10804

Eastern Catchment Proposed Scenario – HMS Inflow hydrograph summary 100yr Storm

Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 100yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_100yr_298mm
Compute Time: 24Jan2025, 20:52:13 Control Specifications: 48hr

Show Elements: All Elements Volume Units: MM 1000 M3 Sorting: Alphabetic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Junction-27	0.06374	2.44536	1 January 2000, 12:20	17.01497
Junction-28	0.06002	2.56700	1 January 2000, 12:17	16.02267
Junction-29	0.03304	1.52967	1 January 2000, 12:15	8.82025
Junction-30	0.02764	1.30844	1 January 2000, 12:14	7.37845
Junction-31	0.04300	1.89389	1 January 2000, 12:16	11.47925
Junction-32	0.05227	2.23305	1 January 2000, 12:17	13.95312
Junction-33	0.03284	1.45642	1 January 2000, 12:16	8.76764
Junction-34	0.03798	1.65293	1 January 2000, 12:16	10.13763
Junction-35	0.03445	1.54495	1 January 2000, 12:15	9.19667
Junction-36	0.06341	2.55474	1 January 2000, 12:18	16.92701
Junction-37	0.10002	4.16941	1 January 2000, 12:20	28.61318
Junction-38	0.07191	3.29250	1 January 2000, 12:17	20.57156
Junction-39	0.26627	4.66047	1 January 2000, 13:08	65.25941
Junction-40	0.00353	0.17270	1 January 2000, 12:12	0.79407
Junction-41	0.01849	0.72248	1 January 2000, 12:18	4.15429
Junction-50	0.01729	0.84576	1 January 2000, 12:15	4.94748
Junction-51	0.01933	1.03749	1 January 2000, 12:13	5.52964
Junction-52	0.00799	0.39422	1 January 2000, 12:15	2.25776
Junction-53	0.02187	1.16086	1 January 2000, 12:13	6.25729
Junction-54	0.02226	1.16768	1 January 2000, 12:14	6.36726
Junction-55	0.02434	1.28036	1 January 2000, 12:14	6.96372
Junction-55_56_Culvert	0.06454	2.99123	1 January 2000, 12:15	18.46417
Junction-56	0.04020	1.80127	1 January 2000, 12:18	11.50045
Junction-57	0.01759	0.93641	1 January 2000, 12:13	5.03064
Junction-58	0.00673	0.33098	1 January 2000, 12:15	1.90113
Reservoir-SW_Pond_2	0.15274	4.14206	1 January 2000, 12:28	33.62547
Reservoir-SW_Pond_3	0.02755	0.98821	1 January 2000, 12:23	7.24933
Sink-Outflow_2	0.15274	4.14206	1 January 2000, 12:28	33.62547
Sink-Out_3	0.02755	0.98821	1 January 2000, 12:23	7.24933
Subbasin-Outflow 2 Ex	0.15274	4.65679	1 January 2000, 12:28	34.30427
Subbasin-Outflow 3 Ex	0.02755	1.00483	1 January 2000, 12:20	6.18860
Subbasin-27_Imp	0.03952	1.79219	1 January 2000, 12:18	11.57514
Subbasin-27_Perv	0.02422	0.74896	1 January 2000, 12:28	5.43983
Subbasin-28_Imp	0.03721	1.86195	1 January 2000, 12:16	10.90008
Subbasin-28_Perv	0.02281	0.79248	1 January 2000, 12:22	5.12260
Subbasin-29_Imp	0.02049	1.09726	1 January 2000, 12:14	6.00033
Subbasin-29_Perv	0.01256	0.47768	1 January 2000, 12:19	2.81992
Subbasin-30_Imp	0.01714	0.93200	1 January 2000, 12:13	5.01948
Subbasin-30_Perv	0.01050	0.41031	1 January 2000, 12:18	2.35896
Subbasin-31_Imp	0.02666	1.36656	1 January 2000, 12:15	7.80923
Subbasin-31_Perv	0.01634	0.58695	1 January 2000, 12:21	3.67002

Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2

Start of Run: 01Jan2000, 00:00 Basin Model: 100yr_Pr v2
End of Run: 03Jan2000, 00:00 Meteorologic Model: TP108_100yr_298mm
Compute Time:24Jan2025, 20:52:13 Control Specifications:48hr

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

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Subbasin-31_Perv	0.01634	0.58695	1 January 2000, 12:21	3.67002
Subbasin-32_Imp	0.03241	1.61675	1 January 2000, 12:16	9.49217
Subbasin-32_Perv	0.01986	0.69012	1 January 2000, 12:22	4.46095
Subbasin-33_Imp	0.02036	1.05068	1 January 2000, 12:15	5.96454
Subbasin-33_Perv	0.01248	0.45178	1 January 2000, 12:21	2.80310
Subbasin-34_Imp	0.02355	1.19395	1 January 2000, 12:15	6.89654
Subbasin-34_Perv	0.01443	0.51143	1 January 2000, 12:22	3.24110
Subbasin-35_Imp	0.02136	1.11193	1 January 2000, 12:15	6.25642
Subbasin-35_Perv	0.01309	0.48020	1 January 2000, 12:20	2.94025
Subbasin-36_Imp	0.03931	1.86492	1 January 2000, 12:17	11.51529
Subbasin-36_Perv	0.02410	0.78603	1 January 2000, 12:25	5.41172
Subbasin-37_Imp	0.09002	3.92267	1 January 2000, 12:20	26.36681
Subbasin-37_Perv	0.01000	0.29451	1 January 2000, 12:30	2.24638
Subbasin-38_Imp	0.06472	3.08930	1 January 2000, 12:17	18.95652
Subbasin-38_Perv	0.00719	0.23613	1 January 2000, 12:25	1.61505
Subbasin-39_Imp	0.07988	2.04910	1 January 2000, 12:52	23.39760
Subbasin-39_Perv	0.18639	3.00143	1 January 2000, 13:26	41.86181
Subbasin-40_Imp	0.00000	0.00022	1 January 2000, 12:10	0.00104
Subbasin-40_Perv	0.00353	0.17249	1 January 2000, 12:12	0.79304
Subbasin-41_Imp	0.00002	0.00101	1 January 2000, 12:13	0.00542
Subbasin-41_Perv	0.01847	0.72164	1 January 2000, 12:18	4.14888
Subbasin-50_Imp	0.01557	0.79219	1 January 2000, 12:15	4.55907
Subbasin-50_Perv	0.00173	0.06145	1 January 2000, 12:21	0.38841
Subbasin-51_Imp	0.01740	0.96717	1 January 2000, 12:13	5.09552
Subbasin-51_Perv	0.00193	0.07761	1 January 2000, 12:17	0.43412
Subbasin-52_Imp	0.00679	0.35438	1 January 2000, 12:15	1.98868
Subbasin-52_Perv	0.00120	0.04407	1 January 2000, 12:20	0.26908
Subbasin-53_Imp	0.01969	1.08318	1 January 2000, 12:13	5.76604
Subbasin-53_Perv	0.00219	0.08675	1 January 2000, 12:18	0.49125
Subbasin-54_Imp	0.02003	1.08944	1 January 2000, 12:13	5.86738
Subbasin-54_Perv	0.00223	0.08667	1 January 2000, 12:18	0.49988
Subbasin-55_Imp	0.02191	1.19631	1 January 2000, 12:13	6.41702
Subbasin-55_Perv	0.00243	0.09509	1 January 2000, 12:18	0.54670
Subbasin-56_Imp	0.03618	1.68947	1 January 2000, 12:18	10.59756
Subbasin-56_Perv	0.00402	0.12885	1 January 2000, 12:26	0.90288
Subbasin-57_Imp	0.01583	0.87396	1 January 2000, 12:13	4.63569
Subbasin-57_Perv	0.00176	0.06974	1 January 2000, 12:18	0.39495
Subbasin-58_Imp	0.00572	0.29761	1 January 2000, 12:15	1.67454
Subbasin-58_Perv	0.00101	0.03701	1 January 2000, 12:20	0.22659

Stormwater Pond 2 Sizing

PWL = 19.40 mRL

Outlets

SMAF Outlet = ø180mm

2yr/10yr Outlet = 2m cutout in DN1200 Scruffy dome @ 20.40 mRL

100yr Spillway -

100yr Emergency Spillway @ 21.50 mRL 70m Long

2yr

Ex

Summary Results for Subbasin: Subbasin-Outflow 2 Ex

Project: FAB_Swale_Sizing   Simulation Run: 2yr_FAB v2
Subbasin: Subbasin-Outflow 2 Ex

Start of Run:	01Jan2000, 00:00	Basin Model:	2yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_2yr_86mm
Compute Time:	14Jan2025, 13:57:54	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Dischar...	0.82457 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:25
Precipitation Volu...	13.13564 (1000 M3)	Direct Runoff Volume:	5.88644 (1000 M3
Loss Volu...	7.24920 (1000 M3)	Baseflow Volume:	0.00000 (1000 M3
Excess Volu...	5.88644 (1000 M3)	Discharge Volume:	5.88644 (1000 M3

Pr

Summary Results for Reservoir: Reservoir-SW_Pond_2

Project: FAB_Swale_Sizing   Simulation Run: 2yr_FAB v2
Reservoir: Reservoir-SW_Pond_2

Start of Run:	01Jan2000, 00:00	Basin Model:	2yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_2yr_86mm
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Inflow:	2.01036 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:13
Peak Discharge:	0.05980 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 18:15
Inflow Volume:	11.01380 (1000 M3)	Peak Storage:	8.38783 (1000 M3)
Discharge Volume:	7.03886 (1000 M3)	Peak Elevation:	20.20325 (M)

10yr

Ex

Summary Results for Subbasin "Subbasin-Outflow 2 Ex"

Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2
Subbasin: Subbasin-Outflow 2 Ex

Start of R...	01Jan2000, 00:00	Basin Model:	10yr_Pr v2
End of R...	03Jan2000, 00:00	Meteorologic Model:	TP108_10yr_170mm_CoPv4
Compute Ti...	14Jan2025, 14:00:31	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Disch...	2.34580 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:2
Precipitation Vol...	25.96580 (1000 M3)	Direct Runoff Volume:	16.35578 (1000
Loss Vol...	9.61002 (1000 M3)	Baseflow Volume:	0.00000 (1000 M
Excess Vol...	16.35578 (1000 M3)	Discharge Volume:	16.35578 (1000

Pr

Summary Results for Reservoir "Reservoir-SW_Pond_2"

Project: FAB_Swale_Sizing Simulation Run: 10yr_FAB v2
Reservoir: Reservoir-SW_Pond_2

Start of Run:	01Jan2000, 00:00	Basin Model:	10yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_10yr_170mm_CoPv4
Compute Time:	22Jan2025, 14:25:09	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Inflow:	4.18201 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:13
Peak Discharge:	0.63895 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 13:10
Inflow Volume:	23.3276 (1000 M3)	Peak Storage:	13.5827 (1000 M3)
Discharge Volume:	17.3799 (1000 M3)	Peak Elevation:	20.66176 (M)

100yr

Ex

Summary Results for Subbasin "Subbasin-Outflow 2 Ex"

Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2
Subbasin: Subbasin-Outflow 2 Ex

Start of Run:	01Jan2000, 00:00	Basin Model:	100yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_100yr_298mm
Compute Time:	14Jan2025, 14:01:39	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Disch...	4.17135 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:2ٍ8
Precipitation Vol...	45.51652 (1000 M3)	Direct Runoff Volume:	34.30427 (1000 M
Loss Vol...	11.21225 (1000 M3)	Baseflow Volume:	0.00000 (1000 M
Excess Vol...	34.30427 (1000 M3)	Discharge Volume:	34.30427 (1000 M

Pr

Summary Results for Reservoir "Reservoir-SW_Pond_2"

Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2
Reservoir: Reservoir-SW_Pond_2

Start of Run:	01Jan2000, 00:00	Basin Model:	100yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_100yr_298mm
Compute Time:	22Jan2025, 14:36:45	Control Specifications:	48hr

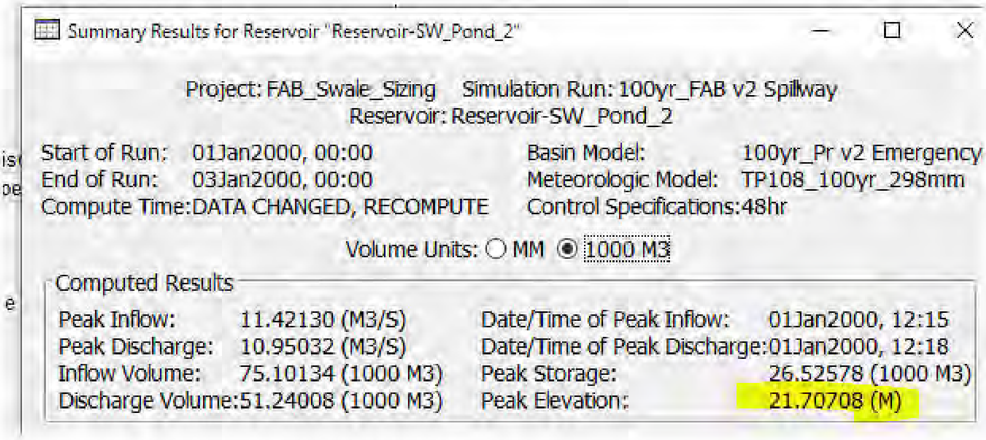
Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Inflow:	7.27751 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:14
Peak Discharge:	4.14206 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:28
Inflow Volume:	42.5314 (1000 M3)	Peak Storage:	22.2910 (1000 M3)
Discharge Volume:	33.6255 (1000 M3)	Peak Elevation:	21.37843 (M)

100yr

Pr Emergency



Stormwater Pond 3 Sizing

PWL = 25.40 mRL

Outlets

SMAF Outlet = ø68mm

2yr/10yr/100yr Outlet 0.7m long Manhole Cutout @ 26.16 mRL (2yr tailwater)

100yr Emergency Spillway @ 27.00 mRL 20m Long

Freeboard Top of bund = 27.30 mRL

2yr

Ex

Summary Results for Subbasin "Subbasin-Outflow3 Ex"

Project: FAB_Swale_Sizing   Simulation Run: 2yr_FAB v2
Subbasin: Subbasin-Outflow 3 Ex

Start of Run:	01Jan2000, 00:00	Basin Model:	2yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_2yr_86mm
Compute Time:	14Jan2025, 15:24:24	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Discharge:	0.17724 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:18
Precipitation Volume:	2.36971 (1000 M3)	Direct Runoff Volume:	1.06193 (1000 M3
Loss Volume:	1.30778 (1000 M3)	Baseflow Volume:	0.00000 (1000 M3
Excess Volume:	1.06193 (1000 M3)	Discharge Volume:	1.06193 (1000 M3

Pr

Summary Results for Reservoir "Reservoir-SW_Pond_3"

Project: FAB_Swale_Sizing   Simulation Run: 2yr_FAB v2
Reservoir: Reservoir-SW_Pond_3

Start of Run:	01Jan2000, 00:00	Basin Model:	2yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_2yr_86mm
Compute Time:	14Jan2025, 15:24:24	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Inflow:	0.36600 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:13
Peak Discharge:	0.07489 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 13:03
Inflow Volume:	1.96810 (1000 M3)	Peak Storage:	1.02456 (1000 M3)
Discharge Volume:	1.68934 (1000 M3)	Peak Elevation:	26.28592 (M)

10yr

Ex

Project: FAB_Swale_Sizing   Simulation Run: 10yr_FAB v2
Subbasin: Subbasin-Outflow 3 Ex

Start of Run:	01Jan2000, 00:00	Basin Model:	10yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_10yr_170mm_CoPv4
Compute Time:	14Jan2025, 15:23:16	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Discharge:	0.50244 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:18
Precipitation Volume:	4.68432 (1000 M3)	Direct Runoff Volume:	2.95064 (1000 M3)
Loss Volume:	1.73368 (1000 M3)	Baseflow Volume:	0.00000 (1000 M3)
Excess Volume:	2.95064 (1000 M3)	Discharge Volume:	2.95064 (1000 M3)

Pr

Summary Results for Reservoir "Reservoir-SW_Pond_3"

Project: FAB_Swale_Sizing   Simulation Run: 10yr_FAB v2
Reservoir: Reservoir-SW_Pond_3

Start of Run:	01Jan2000, 00:00	Basin Model:	10yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_10yr_170mm_CoPv4
Compute Time:	14Jan2025, 15:23:16	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Inflow:	0.76620 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:13
Peak Discharge:	0.48687 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:21
Inflow Volume:	4.18280 (1000 M3)	Peak Storage:	1.50543 (1000 M3)
Discharge Volume:	3.79048 (1000 M3)	Peak Elevation:	26.63437 (M)

100yr

Ex

Summary Results for Subbasin "Subbasin-Outflow 3 Ex"

Project: FAB_Swale_Sizing   Simulation Run: 100yr_FAB v2
Subbasin: Subbasin-Outflow 3 Ex

Start of Run:	01Jan2000, 00:00	Basin Model:	100yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_100yr_298mm
Compute Time:	14Jan2025, 15:23:46	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Discharge:	0.90075 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:20
Precipitation Volume:	8.21133 (1000 M3)	Direct Runoff Volume:	6.18861 (1000 M3)
Loss Volume:	2.02273 (1000 M3)	Baseflow Volume:	0.00000 (1000 M3)
Excess Volume:	6.18861 (1000 M3)	Discharge Volume:	6.18861 (1000 M3)

Pr

Summary Results for Reservoir "Reservoir-SW_Pond_3"

Project: FAB_Swale_Sizing   Simulation Run: 100yr_FAB v2
Reservoir: Reservoir-SW_Pond_3

Start of Run:	01Jan2000, 00:00	Basin Model:	100yr_Pr v2
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_100yr_298mm
Compute Time:	14Jan2025, 15:23:46	Control Specifications:	48hr

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Inflow:	1.19668 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:15
Peak Discharge:	0.87376 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:23
Inflow Volume:	7.64268 (1000 M3)	Peak Storage:	1.82041 (1000 M3)
Discharge Volume:	7.26074 (1000 M3)	Peak Elevation:	26.86261 (M)

100yr

Pr Emergency

Summary Results for Reservoir "Reservoir-SW_Pond_3"

Project: FAB_Swale_Sizing Simulation Run: 100yr_FAB v2 Spillway
Reservoir: Reservoir-SW_Pond_3

Start of Run:	01Jan2000, 00:00	Basin Model:	100yr_Pr v2 Emergency
End of Run:	03Jan2000, 00:00	Meteorologic Model:	TP108_100yr_298mm
Compute Time:	14Jan2025, 15:24:06	Control Specifications:	48hr

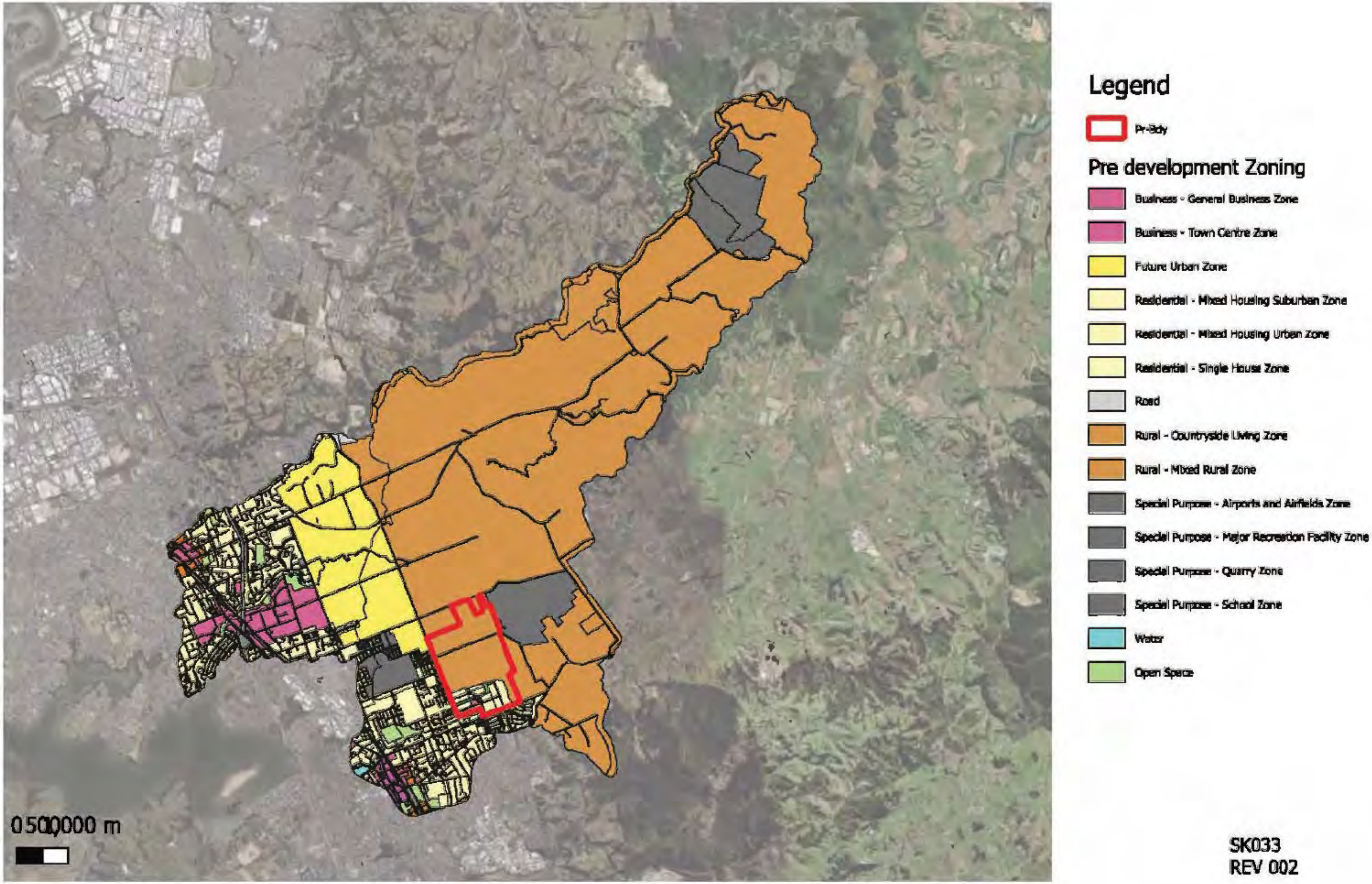
Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Inflow:	2.83192 (M3/S)	Date/Time of Peak Inflow:	01Jan2000, 12:13
Peak Discharge:	2.78946 (M3/S)	Date/Time of Peak Discharge:	01Jan2000, 12:14
Inflow Volume:	16.34647 (1000 M3)	Peak Storage:	2.34982 (1000 M3)
Discharge Volume:	14.33669 (1000 M3)	Peak Elevation:	27.19183 (M)

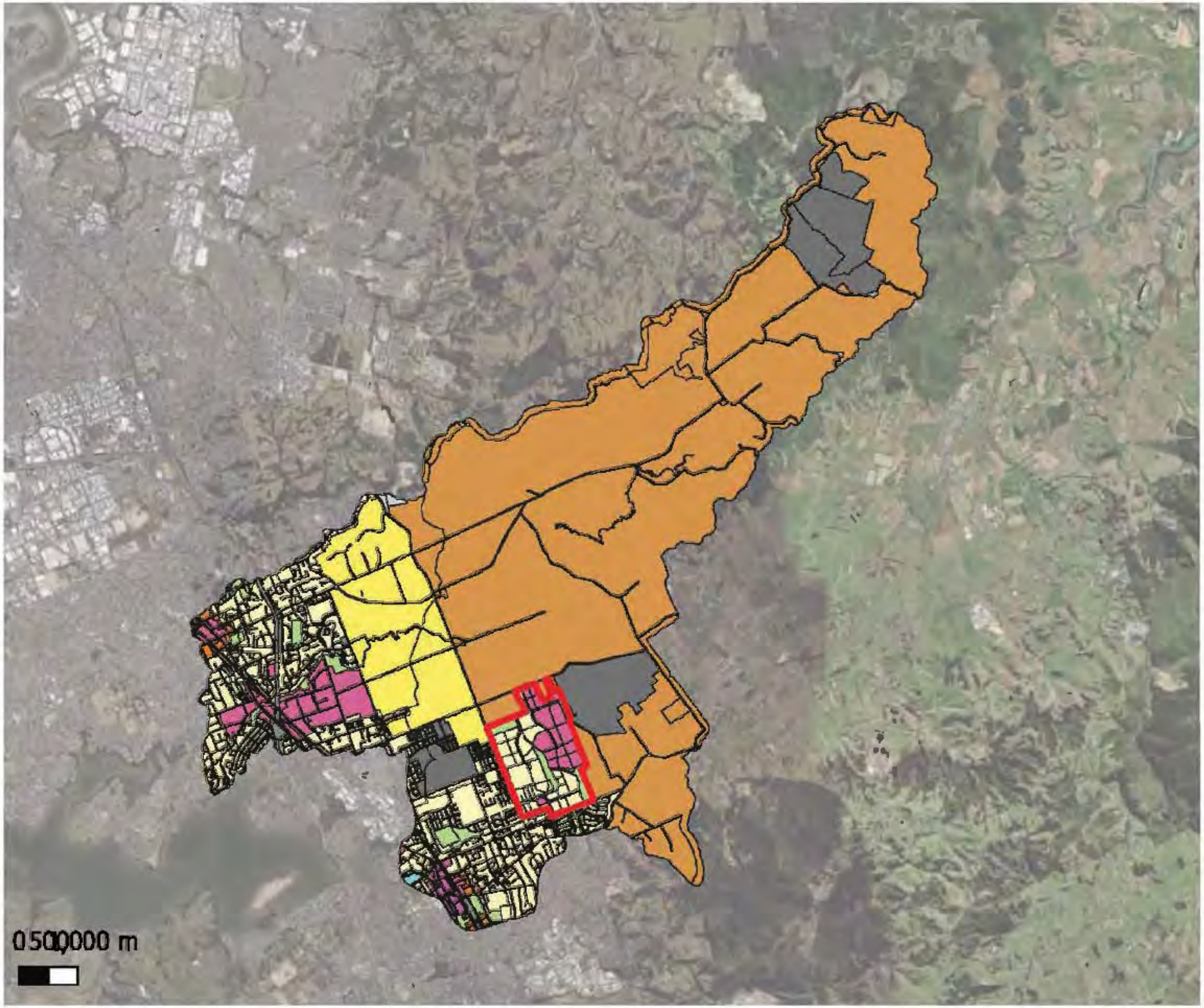
APPENDIX 9 – Zoning

Pre development Zoning



SK033
REV 002

Post development Zoning



Legend

- Pr-Bdy

Post development Zoning

- Business - General Business Zone
- Business - Town Centre Zone
- Future Urban Zone
- Residential - Mixed Housing Suburban Zone
- Residential - Mixed Housing Urban Zone
- Residential - Single House Zone
- Road
- Rural - Countryside Living Zone
- Rural - Mixed Rural Zone
- Special Purpose - Airports and Airfields Zone
- Special Purpose - Major Recreation Facility Zone
- Special Purpose - Quarry Zone
- Special Purpose - School Zone
- Water
- Open Space

SK032
REV 002

Rain on Grid Infiltration layer Parameters

Zone Name	Impervious %	CN	Ia	S	Ab_Ratio
Business - General Business Zone	0.9	95.6	0.5	11.7	0.043
Business - Heavy Industry Zone	0.9	95.6	0.5	11.7	0.043
Business - Light Industry Zone	0.9	95.6	0.5	11.7	0.043
Business - Local Centre Zone	0.9	95.6	0.5	11.7	0.043
Business - Mixed Use Zone	0.9	95.6	0.5	11.7	0.043
Business - Neighbourhood Centre Zone	0.9	95.6	0.5	11.7	0.043
Business - Town Centre Zone	1	98	0	5.2	0.001
Coastal - Coastal Transition Zone	0.6	88.4	2	33.3	0.06
Coastal - General Coastal Marine Zone	1	98	0	5.2	0.001
Future Urban Zone	0.6	88.4	2	33.3	0.06
Open Space	0.1	76.4	4.5	78.5	0.057
Open Space - Community Zone	0.1	76.4	4.5	78.5	0.057
Open Space - Informal Recreation Zone	0.1	76.4	4.5	78.5	0.057
Open Space - Sport and Active Recreation Zone	0.4	83.6	3	49.8	0.06
Residential - Mixed Housing Suburban Zone	0.6	88.4	2	33.3	0.06
Residential - Mixed Housing Urban Zone	0.6	88.4	2	33.3	0.06
Residential - Single House Zone	0.6	88.4	2	33.3	0.06
Residential -Terrace Housing and Apartment Buildings Zone	0.7	90.8	1.5	25.7	0.058
Road	0.85	94.4	0.75	15.1	0.05
Rural - Countryside Living Zone	0.1	76.4	4.5	78.5	0.057
Rural - Mixed Rural Zone	0.1	76.4	4.5	78.5	0.057
Special Purpose - Airports and Airfields Zone	0.9	95.6	0.5	11.7	0.043
Special Purpose - Major Recreation Facility Zone	0.5	86	2.5	41.3	0.06
Special Purpose - Quarry Zone	0.9	95.6	0.5	11.7	0.043
Special Purpose - School Zone	0.7	90.8	1.5	25.7	0.058
Special Purpose Zone	0.7	90.8	1.5	25.7	0.058
Strategic Transport Corridor Zone	0.9	95.6	0.5	11.7	0.043
Water	1	98	0	5.2	0.001

APPENDIX 10 – STAGE 2 & 3 AWAKERI WETLANDS REQUIREMENTS

AWAKERI WETLANDS STAGE 2

DESIGN REQUIREMENTS

Healthy Waters

Design Office

Awakeri Wetlands – Stage 2
Design Requirements

Project	AWAKERI WETLANDS STAGE 2
Document	DESIGN REQUIREMENTS
Issue and Revision Record	
Status	Issued
Version	1
Date of Issue	27/11/2023
Author and Designer	Jesse Peeters Senior Healthy Waters Specialist
Reviewed	Amelia Cunningham Design Office Team Manager
Approved for Issue	Amelia Cunningham Design Office Team Manager

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Auckland Council
Te Kaunihera o Tāmaki Makaurau

Table of Contents

1.0 Project extent and staging

The project area for the Awakeri Wetlands is shown in Figure 1below and is broken into three stages by location:

- Stage 1: Awakeri Wetlands between Grove Road, Cosgrave Road and Walters Road
- Stage 2: Culvert crossing Cosgrave Road and connection to Stage 1
- Stage 3: Awakeri Wetlands between Cosgrave Road, Old Wairoa Road and the pond upstream of Old Wairoa Road

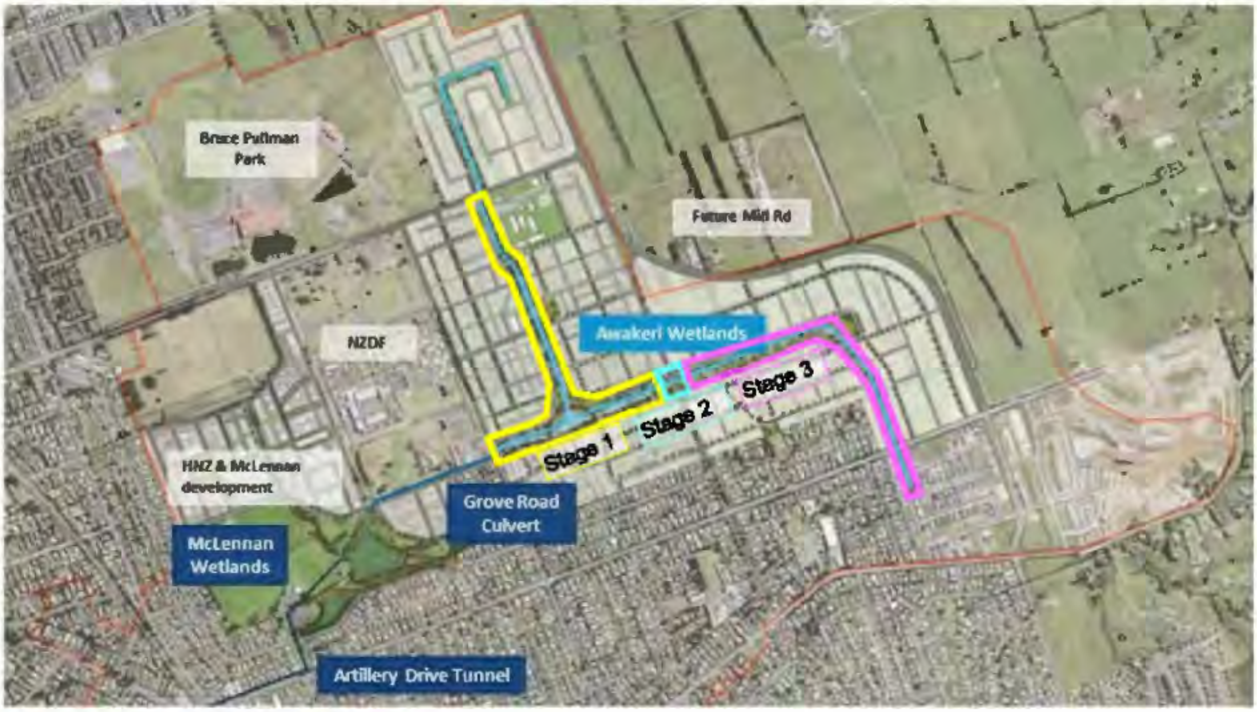

Figure 1 Awakeri Wetlands Staging

2.0 Design criteria

2.1 Standards, manuals and publications

The design and physical works shall be done in accordance with the version current at the time of the work of the following standards, manuals and publications except where amended by these Principal's Requirements:

a) Auckland Council Standard Specifications

b) The Building Act

c) Health and Safety in Employment Act

d) Resource Management Act

e) Maritime Safety Regulation

f) New Zealand Standards and codes

g) Auckland Council Code of Practice for Land Development

h) Auckland Transport Code of Practice

i) NZ Transport Agency Standards and Guidelines Manual

j) NZ Transport Agency Standard Specifications and Publications.

k) NZ Transport Agency Bridge Manual

Where the above does not explicitly cover all parts or issues relating to the design or construction, other codes or standards, such British, Australian or American that are applicable in respect of a part or issue shall apply where agreed by the Engineer.

Standards, manuals and publications shall be read in the following order of priority:

a) Acts of Parliament

b) The Principal's Requirements

c) Auckland Council Standard Specifications

d) Auckland Council Code of Practice for Land Development

e) Auckland Transport Code of Practice

f) Regulatory authority standards, specifications and guidelines

g) Australian/New Zealand Standards and guidelines

h) NZ Transport Agency specifications, standards and guidelines

i) British Standards

j) United States Standards.

Where a guideline document allows for different options or where engineering judgement is required, a design report or technical memorandum shall be provided.

2.2 Safety in Design

Safety in design must be considered throughout all stags of design and shall include a register, reporting and workshop to discuss and document the options considered for each element of the design.

Safety in design shall include input from Auckland Council Healthy Waters Operations, Community Facilities, Auckland Transport, Watercare and any other parties who will be involved with the asset throughout its design life.

Safety in design considerations shall be made for all key features including, but not limited to:

- Culvert
- Wingwall/headwalls
- Road, footpath and berm
- Pedestrian crossing

- Debris clearance
- Fall protection
- Stormwater connections
- Groundwater cutoff barrier installation
- Watermain protection methodology

2.3 Mana whenua partnership

A partnership was formed between mana whenua and the Awakeri Wetlands project design team during the design of Stage 1 of the Awakeri Wetlands. This partnership must be transitioned to the new designer to ensure that mana whenua continued to be partners of the project.

Regular huis/meetings are expected as part of this, an initial hui shall be arranged by the new designer and mana whenua to agree on hui frequency and level of involvement. The iwi who were part of the partnership are listed below. Key staff / contacts may have changed since and it is the designers responsibility to identify the current representatives of the iwi.

Table 1 Mana whenua representatives

Iwi group	Representatives	Contact
Ngati Tamaoho	Lucie Rutherford Hero Potini Zachary Sirett Edith Tuhimata (previously Ngati Te Ata Waiohua)	[redacted] zac@tamaoho.maori.nz
Te Ākitai Waiohua	Nigel Denny Karen Wilson	kaitiaki@teakitai.com
Ngai Tai ki Tamaki	Jonathan Billington James Brown	kaitiaki@ngaitaitamaki.iwi.nz
Ngāti Te Ata Waiohua	Karl Flavell	karl.flavell@ngatiteata.iwi.nz

2.4 Watercare consultation and approvals

Watercare's Waikato No.1 watermain is in close proximity to the project and Watercare approval will therefore be needed prior to any work taking place. Previous correspondence with Watercare has indicated the following:

- Any shutdowns of the Waikato No.1 Watermain would need to be planned in advance with up to 2 years notice given to Watercare. This requirement may change depending on Watercares scheduled shutdowns so communications should be made with Watercare during the design phase to confirm.
- Finite Element Analysis (FEA) will be required and provided to Watercare to demonstrate that effects on the Waikato No.1 Watermain can be managed.
- Any design of temporary support or ground improvements shall be reviewed and approved by Watercare.

2.5 Local board consultation

The designer shall contact the local board to provide regular updates and accommodate feedback from the local board as required. The frequency of updates shall be determined based on an initial meeting with the local board to agree on frequencies.

2.6 Consents

All required consents (resource, discharge, landowner, etc.) and approvals (Watercare EPA and Works Over Approval, Auckland Transport EPA, etc) shall be obtained by the designer.

2.7 Existing services

The designer shall be responsible for liaising with utility providers and designing protection for of all known services which conflict with the proposed work, including but not limited to:

- Waikato No.1 Transmission Watermain (1200mm diameter CLS pipe)
- Fibre optic cable for Watercare treatment plant controls (critical watercare infrastructure)
- Wastewater pipes and wastewater rising mains (225mm dia. rising main)
- Local watermains
- Overhead power lines
- Underground power cables
- Underground communications cables
- Fibre optic cables
- Roads

Approval from Watercare shall be obtained by the designer for the proposed works around the Waikato No.1 Watermain. This shall include approval of any short term (during construction) and long-term (post construction) protection methods and/or support required for the Waikato No.1 Watermain.

All underground and overhead services shall be protected and/or diverted during the works, with approvals gained from the relevant service providers where required.

2.8 Minimum design requirements

General design features of the Cosgrave Road Culvert (Stage 2 of the Awakeri Wetlands) are shown in the Specimen Design Drawings. The Specimen Design Drawings shall be referred to and significant deviations from the key features shown in these drawings shall be documented and approved by Auckland Council.

It is the designer's responsibility to determine the final design criteria, however the following section provides minimum requirement and sets out Auckland Councils expectations.

2.8.1 Crossing type

Stage 2 of the Awakeri Wetlands is proposed to be a multi-barrel culvert to convey water under Cosgrave Road.

The preferred culvert type is to use box culverts due to the following benefits:

- Larger air gap between low flow water level and soffit of culvert, as the flat top provides an increased width of air gap compared to a circular culvert. This will allow more debris to float through the culvert without getting impinged at the entrance.
- Larger capacity per width of cross section compared to circular culverts, therefore higher capacity, lower velocities and less erosion protection needed.

Sizing has been calculated in the *Awakeri Wetlands Stage 2 Specimen Design Report (Auckland Council, 2019)* as twin 2m (H) x 3m (W) box culverts.

Another size that would be acceptable is three 1.5m (H) x 2.5m (W) box culverts. The reduction in height and width per culvert unit means a shallower permanent water depth and larger air gap could potentially be achieved due to a reduced thickness of roof slab on the smaller culverts.

Final sizing of the box culverts is to be provided by the designer.

2.8.2 Culvert alignment (vertical/horizontal)

The vertical and horizontal alignment of the culverts shall be such that they do not adversely affect the integrity of any existing structures (i.e. Waikato No.1 Watermain, other services) and considers the safety, operation and maintenance considerations and risks described in the *Awakeri Wetlands Stage 2 Specimen Design Report (Auckland Council, 2019).*

The low flow depth of water in the culvert is controlled by a downstream weir within Stage 1 of the Awakeri Wetlands. The low flow water level is 22.25 m RL. The design shall consider the safety, operational and maintenance aspects associated with the depth of water in the culvert. An air gap shall be provided in the culvert between the low flow water level and the soffit of the culvert to allow small debris to flow through.

The length of the culvert shall be confirmed based on discussions with Auckland Transport in regards to any future road modifications planned and other constraints determined by the designer.

2.8.3 Design Life

A design life of not less than 100 years shall be allowed for, taking into consideration the low pH / aggressive ground conditions such as potential acid sulphate soils. Further information is available in the *Hydrogeology Assessment of Effects (GHD, 2016)* and the *Acid Sulfate Soils Management Plan (GHD, 2017).*

2.8.4 Design flow

The design of the culvert shall be able to convey up to the 1% AEP storm event without the immediate upstream water level surcharging above RL23.80m RL. Catchment flows for the culvert are outlined in Table 2, provided the catchment, development and impervious area assumptions from the Awakeri Wetlands Stage 1 Detailed Design Report are met.

Table 2 Target Hydraulic Capacity Requirements

Storm Event	Peak flow (m^3/s)
50% AEP (2yr ARI)	5.7
10% AEP (10yr ARI)	14.6
1% AEP (100yr ARI)	23.0

2.8.5 Blockage assessment

A blockage assessment shall be provided which outlines the likelihood and consequence of various blockage scenarios. The final allowance for blockage must be agreed with Auckland Council prior to finalization of the design.

2.8.6 Hydraulic design parameters

Hydraulic design parameters are described in the *Awakeri Wetlands Stage 2 Specimen Design Report (Auckland Council, 2019)*. This report provides a hydraulic design for various options, however the designer is not limited to these options.

The designer will need to provide an updated hydraulic design for any solutions outside the options considered in the *Awakeri Wetlands Stage 2 Specimen Design Report (Auckland Council, 2019)*. Table 3 outlines the hydraulic design parameters that shall be applicable to any option. These performance requirements must be met for Stages 1 and 3 of the Awakeri Wetlands to perform as intended.

Table 3 Hydraulic design parameters

Assumption	Value	Source
Low flow water level	22.25 m RL	Awakeri Wetlands Scheme Design (GHD, 2016)
1% AEP tailwater level	23.25 m RL	Awakeri Wetlands Scheme Design Hydraulic model (GHD, 2017).
Maximum 1% AEP upstream water level	23.80 m RL	Selected based on not increasing flood levels upstream in comparison to the Awakeri Wetlands Scheme Design model. Value of the 1% AEP water level at the upstream weir used.
Invert level of Waikato No.1 Watermain	23.25 m RL	Watercare As-built (at centre of proposed infrastructure alignment). Level to be confirmed by the designer.
Awakeri Wetlands channel invert U/S and D/S end	21.45 m RL	Awakeri Wetlands Scheme Design (GHD, 2016)

2.8.7 Minimum external design loads

The designer shall determine the design load parameters for the crossing with Auckland Transport.

Minimum design load parameters are available in Table 4.

Table 4 Live loads

Item	Load allowance	Reference

HN vehicle loads	3.5 kPa x 3 m wide uniform load *plus* 2 x 120 kN axle loads at 5 m ctrs, or 12 kPa surcharge pressure (as appropriate)	Bridge Manual (SP/M/022) – Section 3.2.2 and Figure 3.1 and Section 3.4.12
HO vehicle loads	3.5 kPa x 3 m wide uniform load *plus* 2 x 240 kN axle loads at 5 m ctrs, or 24 kPa surcharge pressure (as appropriate)	

2.8.8 Inlet and outlet structures

An inlet / outlet structure shall be provided at each end of the culverts. The headwall/wingwalls shall be similar to the Grove Road culvert outlet wingwalls and shall include:

- Wingwalls/headwall shall be parallel with the road (ie. straight concrete retaining walls)
- Material shall be concrete with an exposed aggregate finish (sandblasted or similar)
- Height of the headwall shall be minimised to minimise the fall height from above.
- Mitigation to discourage access by the public, such as planting.
- Access to be provided for clearing blockages
- Safety barrier / fence to mitigate fall height. Safety fence to match the fence on top of the Grove Road Culvert outlet structure at the McLennan Wetland.
- Erosion / scour protection. Type and extent of erosion protection to minimise visual impact on surrounding environment and align with materials used in Stage 1 of the Awakeri Wetlands.

Figure 2 Culvert headwall/wingwalls

2.8.9 Buoyancy

The culvert shall be designed for the effects of buoyancy for suitable scenarios determined by the designer based on nearby groundwater monitoring data and Awakeri Wetlands design information.

2.8.10 Dewatering

Any dewatering systems used shall be designed and operated so that related settlement does not exceed limits prescribed by the Resource Consent Conditions in both long term and short term (during construction) scenarios.

2.8.11 Groundwater cut-off barrier

Previous assessments, as described in the *Hydrogeology Assessment of Effects (GHD, 2016)* have indicated that a vertical groundwater cut-off barrier is required around the perimeter of the culvert excavation, and the base of the excavation should be lined with a low permeability material to reduce groundwater inflows during construction.

The vertical groundwater cut-off barrier is also required around permanent excavations at either end of the culvert to manage long-term groundwater drawdown and associated settlement effects.

Groundwater barrier requirements

- The cut-off barrier is required to have a maximum permeability of 1×10^{-8} in order to mitigate groundwater drawdown to an acceptable level.
- The base of the excavation during culvert construction shall also have a maximum permeability of 1×10^{-8}.
- The vertical groundwater cut-off barrier was proposed as a bentonite-cement slurry wall. The slurry wall was proposed as minimum 600mm wide and 7.0m deep.
- The mix design for the slurry wall shall be provided by the designer, with lab testing and in-situ field testing to demonstrate that the required permeability can be met and sufficient curing will be achieved.
- Final design of the slurry wall shall be provided by the designer.

Quality assurance

- In-situ QA coring and sampling shall be undertaken during construction to confirm that the required permeability requirements have been met for the installed wall.
- For every hundred meters of the slurry wall, the proper curing of the slurry wall shall be checked as follows:
    - Coring should be done no earlier than 14 days in at least 4 locations in the central axis of the cut off wall and all the way to the full depth of the cut off wall.
    - Coring will enable checking of cut off wall consistency, depth, verticality and width. Holes to be grouted back
    - Number of locations to be drilled will be reviewed and may change in view of the encountered results, as further coring could be necessary.
- Where QA coring or sampling shows non-compliance, the slurry wall shall be repaired to achieve compliance. This may involve re-excavating and reinstalling the slurry wall in some sections.

Contingency plan for obstructions

A contingency plan is required where the slurry wall installation encounters underground obstructions. The Engineer shall be consulted when an obstruction is encountered to determine an appropriate action. Actions could include:

- Investigating the extent of the obstruction through coring, or additional excavating
- Coring through timber obstructions using an excavator mounted coring tool.
- Cutting and removing the obstruction.
- Realigning the cut-off barrier to go around the obstruction.

Figure 3 Slurry wall photo

2.8.12 Culvert groundwater ingress and settlement considerations

The following groundwater and settlement considerations shall be made when designing the culvert:

- The infrastructure conveying water under the road shall be fully sealed to prevent leakage of groundwater and sediments into the culvert over the design life of the structure. This requirement is important for mitigating the risk of groundwater drawdown and settlement of the surrounding peat soils.
- Post tensioning shall be considered if the structure has joints that are at risk of leaking due to ground movement.
- Settlement of the structure (due to the weight of the structure and/or backfill) shall be assessed and the impact of any predicted settlement on adjacent structures and services shall be considered and mitigated. Use of lightweight backfill shall be considered where/if appropriate.
- Settlement of the ground around the structure (due to any anticipated effects on adjacent groundwater and soil) shall be assessed and the impact of any predicted settlement on adjacent structures and services shall be considered and mitigated.

- Lightweight backfill such as polyrock may be required to reduce the load on subsoils and manage ground settlement.
- Stiffening of the culvert bedding may also be required by including a geogrid raft or flowable fill to mitigate differential settlement across the culvert.

2.8.13 Road reinstatement

The design shall include details for road reinstatement after the culvert is installed which shall be agreed with Auckland Transport, but at a minimum shall include:

- A pedestrian crossing for Cosgrave Road shall be designed and approval shall be obtained by Auckland Transport and any other relevant parties. The designer shall liaise with Auckland Transport and any other relevant parties to determine a suitable crossing detail (ie. signalised, island, zebra crossing).
- The Cosgrave Road corridor shall be upgraded to align with the future road cross section. The designer shall liaise with Auckland Transport to determine the future road cross section details.
- The extent of road corridor upgrade shall include along the full frontage property boundary of the Awakeri Wetlands designation.
- Preference is to avoid the need for road safety barriers (crash barriers). Providing a setback for inlet/outlet structures is a preferred method for managing the risk of collisions with the culvert or associated structures rather than installing crash barriers.
- Consultation with Auckland Transport is required to confirm whether a Traffic Impact Assessment is required and to plan any road closures, diversions or traffic management required during the work.

2.8.14 Scour and erosion protection

The design shall include the 2 year, 10 year and 100 year ARI peak flow scenarios for assessment of scour and erosion potential by carrying out a permissible shear stress analysis. Scour protection shall be designed and installed at the upstream and downstream ends of the structure to adequately mitigate scour and erosion.

Large riprap and concrete shall be avoided where possible for scour protection. Naturalised methods of scour protection such as planting is preferred. Oversizing culverts to minimise velocity, energy and shear stress is preferred over providing scour protection to dissipate high energy. Geosynthetic materials to reinforce plant roots such as geoweb and enkamat are preferred over hard engineering solutions.

Consideration to lining the underwater base of the wetland with geotextile filter fabric, enkamat and gravel shall be made to avoid scour, discharge of sediment and soft exposed peat which can be a safety hazard. This also provides a distinct base that can be identified during maintenance or desilting.

Figure 4 Geoweb scour protection, enkamat and gravel lining for underwater wetland base

2.8.15 Planting

Planting is required between the footpath and the culvert headwall to discourage access. Planting shall meet the following requirements:

- Species to match those used in Stage 1 of the Awakeri Wetlands.
- Zones, mixes and planting layouts/clumps to match those in Stage 1 of the Awakeri Wetlands.
- Planting areas to include 100mm of aged arbor mulch. Processed wood chips are not acceptable.
- Planting areas shall include a minimum of 300mm topsoil or local peat soil.
- All planted areas below the 10% AEP flood level shall be covered with 100% biodegradable coconut matting or similar with jute mesh (no plastic mesh).
- All planted areas above the 10% AEP flood event shall be covered with a minimum of 100mm aged arbor mulch.

2.8.16 Obstruction Management Plan

The designer shall prepare an obstruction management plan which outlines the approach that needs to be taken if an obstruction is encountered during construction. It is highly likely that large buried kauri logs will be discovered when excavating. The management plan shall include the following response actions for the discovery of an obstruction:

- Determine the nature of the obstruction.
- Determine whether the obstruction clashes with the proposed work.
- Recommend a response action and seek approval from Auckland Council before proceeding.

- Options for managing buried obstruction/kauri log that clashes with the proposed infrastructure include, in order of preference:
    - Leave the kauri log in place if it doesn't clash with any key infrastructure such as weirs, boardwalks, footpaths or culverts.
    - Leave the kauri log in place and realign the proposed infrastructure to avoid the log.
    - Cut and remove part of the kauri log to avoid the proposed infrastructure, leaving the remainder in the ground.
    - Complete removal of the obstruction/kauri log, stockpile on site or place the kauri log in an approved location within the wetland.
    - Other options may be identified and proposed by the designer.

2.8.17 Reinstatement of permanent surface water drainage features

The reinstatement of surface drainage systems shall be designed and constructed such that the existing conveyance and inlet capacities are maintained or improved where they have been disrupted by the Contract Works.

2.8.18 Geotechnical design criteria

Recent geotechnical investigations are provided in the *Geotechnical Investigations Report (GHD 2016)* and the *Geotechnical and Ground Settlement Effects Report (GHD, 2016).* This is for information only and the designer is responsible for preparing updated geotechnical investigations and assessments to support their design.

2.8.18.1 Ground conditions

The ground conditions for the project area are described in the *Geotechnical Investigations Report (GHD 2016)* and the *Geotechnical and Ground Settlement Effects Report (GHD, 2016).* This is for information only and the designer is responsible for preparing updated geotechnical investigations and assessments to support their design.

2.8.18.2 Groundwater

Groundwater information is provided in the *Hydrogeology Assessment of Effects (GHD, 2016).* This is for information only and the designer is responsible for gathering updated groundwater information and preparing an updated hydrogeology assessment.

2.8.18.3 Seismic design

The designer shall consider seismic hazards and liquefaction risks in the design of the project.

Some information is provided in the *Geotechnical and Ground Settlement Effects Report (GHD, 2016)*. This is for information only and the designer is responsible for preparing updated seismic hazard and liquefaction assessments..

2.8.18.4 Geotechnical Baseline Report (GBR)

The designer shall prepare a GBR for the project.

The purpose of the GBR is to provide a single source contract document containing measurable contractual descriptions of the geotechnical conditions to be anticipated or to be assumed to be anticipated during construction. In the event of the project running into difficulties due to ground conditions, the GBR can be used to decide if the conditions are unforeseen and therefore create potential for a claim or fall within the conditions expected at the site. This does not present any ambiguous interpretation of conditions or any uncertainty. Only measurable, quantitative terms used.

The GBR shall present a very concise contractual summary of the ground model that the Contractor should allow for when tendering for the construction.

The GBR shall be prepared in general accordance with the Essex, R.J., 1996, Geotechnical Baseline Reports for Underground Construction: Guidelines and Practices published by the American Society of Civil Engineers, as amended by this contract.

The GBR shall contain:

- A brief summary description of the material types expected to be encountered.
- The estimated amounts and distribution of different materials along the alignment, typically presented as an estimate of the percentage of the project (for example, linear metres of tunnel) that each material type will make up. This shall be given as a predicted range to allow for geological uncertainty.
- Geotechnical and groundwater parameters, and expected behaviours, for each of these materials, given as a predicted range to allow for geological uncertainty. Include strength, permeability, grain size, mineralogy, predicted pumping rates, predicted settlement and any other aspects which could impact on construction. Wherever possible, these shall be expressed in quantitative terms, and should present the expected distribution envelope of each parameter within the range.
- Descriptions of geotechnical and man-made sources of potential difficulty or hazards that could impact the construction process (such as boulders, bedrock variability, contaminated groundwater, subsurface obstructions, unstable slopes, adjacent activities).
- A description of the anticipated construction methodology with which the baselines are associated. The baseline statements should be clear that the ground can be expected to behave differently with alternative tools, methods, sequences and equipment.

The GBR shall not contain:

- Ambiguous or vague interpretations
- Descriptions or parameters that cannot be easily measured or assessed and recorded during construction
- Qualitative terms such as 'large' or 'major' unless these are clearly defined.

2.8.19 Ground improvements

The designer shall identify in its Detailed Design Report:

- What ground improvements, if any, are proposed
- The methods used to quantify their extent and effectiveness, and

- The precedent that has been followed in their development.

The methods and precedent shall be referenced in the Detailed Design Report. The effectiveness of ground improvement shall be demonstrated by field testing.

2.8.20 Operation and Maintenance

A Draft Operations and Maintenance (O&M) Manual shall be supplied with the proposed design solution to enable the proposed solution to be fully assessed and understood by the asset owner. The Draft O&M Manual will include maintenance of fittings, plant and machinery requirements, traffic management, isolation, dewatering (and treatment/disposal of this water), pipe inspection, sediment/debris removal and decommissioning. Auckland Council may engage a third party operations expert to review the acceptability of the solution and the O&M Manual.

The design shall provide for safe personnel access to enable a walk-through of the full length of the new infrastructure for operational and maintenance purposes.

The designer shall finalise O&M manuals, containing such information and details as are necessary for Auckland Council to carry out the operation and cost-effective maintenance of the system. The contents and format of these O&M manuals shall be subject to the approval of Auckland Council and a third party expert.

2.9 Handover

A plan for handover of the asset to Auckland Council shall be prepared by the designed and reviewed / approved by Auckland Council during the design phase. The handover plan shall include:

- Confirm timeframes for handover, allowing for a minimum 12 month defects liability period.
- Confirm which departments will own and maintain each asset.
- Duration of planting maintenance by the contractor prior to handover to Auckland Council.
- Responsibility for completing resource consent conditions including groundwater and settlement monitoring.

3.0 Design deliverables

3.1 Preliminary Design

3.1.1 Definition

In the Preliminary Design the drawings and technical documentation are developed to the point where a resource consent application may be lodged if required.

It is expected that no significant changes to line, level, sizing or construction techniques will be required at detailed design. The purpose of detailed design will be to finalize structural calculations and produce construction drawings.

3.1.2 Safety in design

The Contractor shall conduct a safety in design process for the works.

At the Preliminary Design Stage, the Contractor shall take account within the design of:

- Provision and maintenance of a safe work environment for whole of asset life
- Provision and maintenance of safe plant and structures
- Provision and maintenance of safe systems of work
- Safe use, handling, and storage of plant, substances, and structures
- Provision of adequate facilities for the welfare at work of workers.

At Preliminary Design Phase, a Safety in Design Risk Assessment Register is required to identify current risk exposure in construction, operation, maintenance and decommissioning, plus the proposed risk treatment or improvement opportunity for the preferred option and any residual risks with commentary on how they should be managed.

3.1.3 Preliminary design requirements

As a minimum, the designer shall undertake the following tasks:

- Develop options for variations of the concept design including, but not limited to, construction methodology, vertical and horizontal alignments, exact locations of features, materials. The purpose of this exercise is for the designer to ensure that an optimum solution is developed for detailed design that has the lowest whole-of-life cost whilst still meeting all other project objectives and providing a safe working environment throughout the life of the asset.
- Identifying and scoping any additional investigations the designer considers necessary to properly complete the design
- Preliminary design report. The designer shall prepare a brief Preliminary design report as described below.
- Preliminary design drawings. The drawings shall be sufficiently detailed to support the required consent applications. The designer shall allow for all drawings and supporting documents required for a consent but they shall include as a minimum:
    - General arrangement drawings
    - Long sections and elevations
    - Working areas
    - Preliminary erosion and sediment control plans
    - Preliminary traffic management plans.

3.1.4 Preliminary design report and drawings

The Contractor shall submit a Preliminary Design Report covering the following:

- Hazards identified and mitigated through the safety in design process followed (either appended with a summary within the report or a detailed section within the report).
- Outline specifications for all key components including all structures and materials.

- Key assumptions made.
- Utility diversions (if required).
- Key risks and risk management.
- Recommendations including confirmation that the recommended option meets the project objectives or otherwise.
- Commentary on hydraulic performance.
- A clear statement of the internal quality control and quality assurance procedures adopted in developing the design options.
- Stakeholder and end-user requirements and confirmation that the preferred option meets agreed asset owner requirements.
- Benefits of the option, and how they can be measured during or after project implementation.

3.2 Detailed design

3.2.1 Definition

In the Detailed Design Stage, the design is finalised and complete construction drawings, specifications and any monitoring, QC plans etc are produced.

3.2.2 Safety in design

A safety in design assessment is required for the Detailed Design. This safety in design process should be continued from the Preliminary design phase. The designer should define the methodology used for the risk assessment during this phase for agreement with Auckland Council. Auckland Council will identify the stakeholders that will be involved in the Safety in Design process.

At the end of the detailed design, a Safety in Design Risk Register and Report is required. All residual health and safety risks in construction, operation, design for exceedance, maintenance and demolition or disposal shall be clearly described and conveyed. Where relevant, these should be noted on the construction drawings if they are to be managed during construction.

A stand-alone safety in design report is required. All other reports will require a safety in design section as part of the design reporting.

3.2.3 Detailed design requirements

As a minimum, the Contractor shall undertake the following tasks:

- Scope confirmation workshop. The designers project manager and lead technical staff shall all attend this workshop. The purpose of the workshop will be to confirm the preferred solution and determine any preferences for final details. The designer shall prepare minutes of the workshop.
- Review the preliminary design including; confirmation of the findings of the preliminary design report costings, benefits, etc.
- Carry out all calculations required to finalize materials, structural strengths etc.

- Prepare fully detailed Construction Drawings including:
    - Geotechnical information (bore logs) and existing underground and overhead services shall be shown on the long sections.
    - Detailed construction site access, storage and site office areas.
    - Locations of existing utility services including details of diversions where required.
    - Waikato No.1 Watermain protection details.
    - Groundwater and settlement monitoring plan.
    - Structural drawings.
    - Any H&S issues and design mitigation assumptions that may affect or be affected by construction methodology.
- Review Standard Specifications and prepare Particular Specifications.
- Update safety in design and prepare report suitable for hand over to Constructors.
- Prepare a Geotechnical Baseline Report and agree with Auckland Council.
- Submit draft drawings, specifications and plans to Auckland Council for review.
- Finalize pre-construction risk register.
- Detailed design report as below.
- Address comments on the detailed design report and drawings. The designer shall ensure that the drawings and documents are properly reviewed and a QA undertaken by the designer prior to submission. Where any errors or omissions are found that Auckland Council considers to be a failure of QA or review, the drawings and documents shall be returned to the designer without further comment. The formal review will only be undertaken once the QA issues have been resolved. Auckland Council may appoint a peer review in addition to internal reviews. The designer shall allow for collating and assessing the reviews from Auckland Council and shall produce a Construction Documents and Drawing set that adequately addresses the comments.

3.2.4 Detailed design report and drawings

The designer shall prepare and submit a Detailed Design Report. The Final Design Report need not repeat all the detail of the Preliminary design report but should cover the following:

- Details of the final design and proposed construction methodology
- Safety in design considerations in the design process
- The Contractor's assessment of hazards and risks (including H&S) to be managed, if appropriate, a summary of risks eliminated or minimised through design can also be included
- Significant changes to the design or departures from the approved Preliminary Design Report
- Summary of residual risks at contract stage and for whole of life highlighting any remaining high risk items

- Updates to any technical or specialist reports as a result of amendments since the Preliminary design report
- Advice on any variations that may be required to Resource Consents as a result of such changes.
- Any departures from the Auckland Council Code of Practice and Standard Specifications
- Detailed advice on producer statement requirements and the level of construction monitoring required by the designer in order to be able to deliver the producer statements.

Construction drawings, labelled "For Construction", shall be prepared on the standard Auckland Council title block and include the same information as the Preliminary Design drawings plus the following:

- Intended purpose of the asset (if appropriate)
- Geotechnical information (bore logs) and existing underground and overhead services shall be shown on the long sections
- Updated and more detailed construction site access, storage and site office areas
- Utility services diversions
- Groundwater and settlement monitoring plan.
- Structural drawings.
- Safety in design considerations to be addressed during construction, operation, maintenance and demolition or disposal.

The designer shall facilitate a meeting with Auckland Council to present and discuss the detailed design and the draft construction documentation. The objective of the design meeting is to agree what changes (if any) are required to the Draft Construction Documentation before a final version of the documentation is signed off.

Any review does not remove any responsibility from the designer for the correctness or appropriateness of the design or construction documents.

3.2.5 Quality assurance requirements

The designer shall prepare a project review and audit schedule for the Detailed Design and Construction Documentation. This will identify:

- One or more named reviewers accountable for reviewing technical outcomes, and technical reviews planned. Unless otherwise agreed with Auckland Council, the reviewers shall be those named in the Proposal. Where Auckland Council considers that less qualified staff than those shown in the Proposal are offered, revised rates shall be agreed.
- The designers named Project Director or Sponsor will be accountable for reviewing overall project delivery, and project outcome reviews. Auckland Council has an expectation that there will be a scope review (10%), a progress review (50%) and an outcome review (90%) as a minimum

- All documents will have a QA review before being delivered to Auckland Council, including draft reports. The report should include a QA and document control page to identify author, reviewer and version control for drafts.

4.0 Hold points

In addition to reviews of the design packages and elements described above, specific review of the following design items is also required by Auckland Council as they become available:

- Construction methodology.
- Staging of Stage 2 and 3.
- Groundwater management design.
- Peer review for groundwater drawdown assessment and settlement assessment (long term and short term), buoyancy assessment, culvert foundation/bedding design and road backfill design.
- Waikato No.1 Watermain effects assessment and protection design.
- Groundwater and Settlement Monitoring and Contingency Plan.
- Material specifications including shop drawings for pre-cast concrete structures.
- Groundwater cut-off barrier / slurry wall mix design and testing documentation.
- Erosion and scour protection design.
- Obstructions management plan (approach for managing clashes with buried kauri logs etc).

Additional hold points will be required for the construction phase and these will be outlined by Auckland Council prior to construction.

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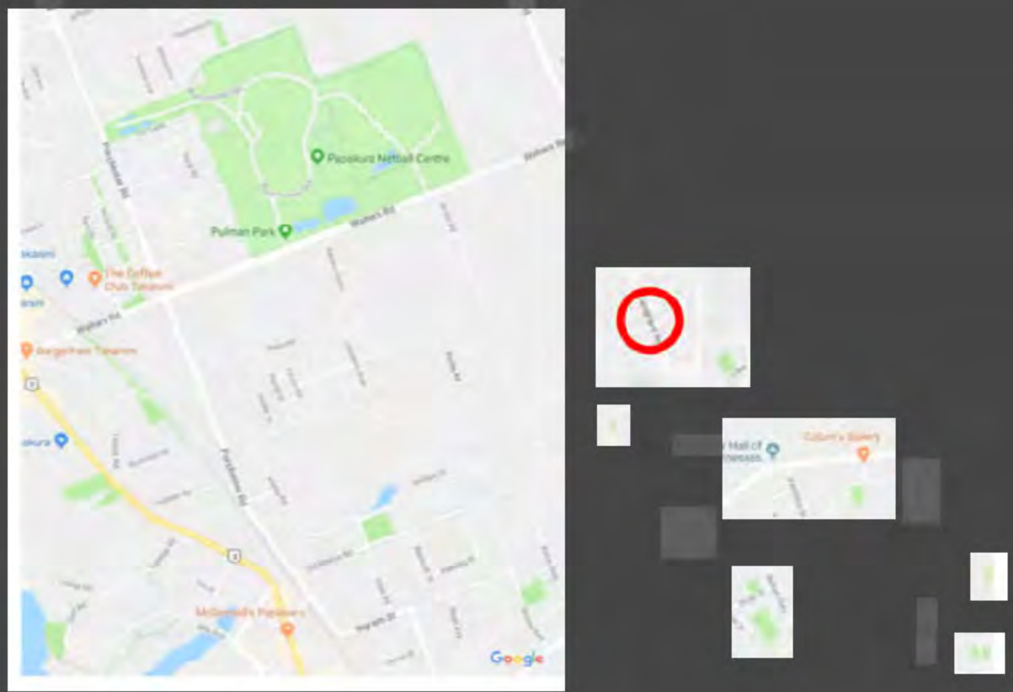

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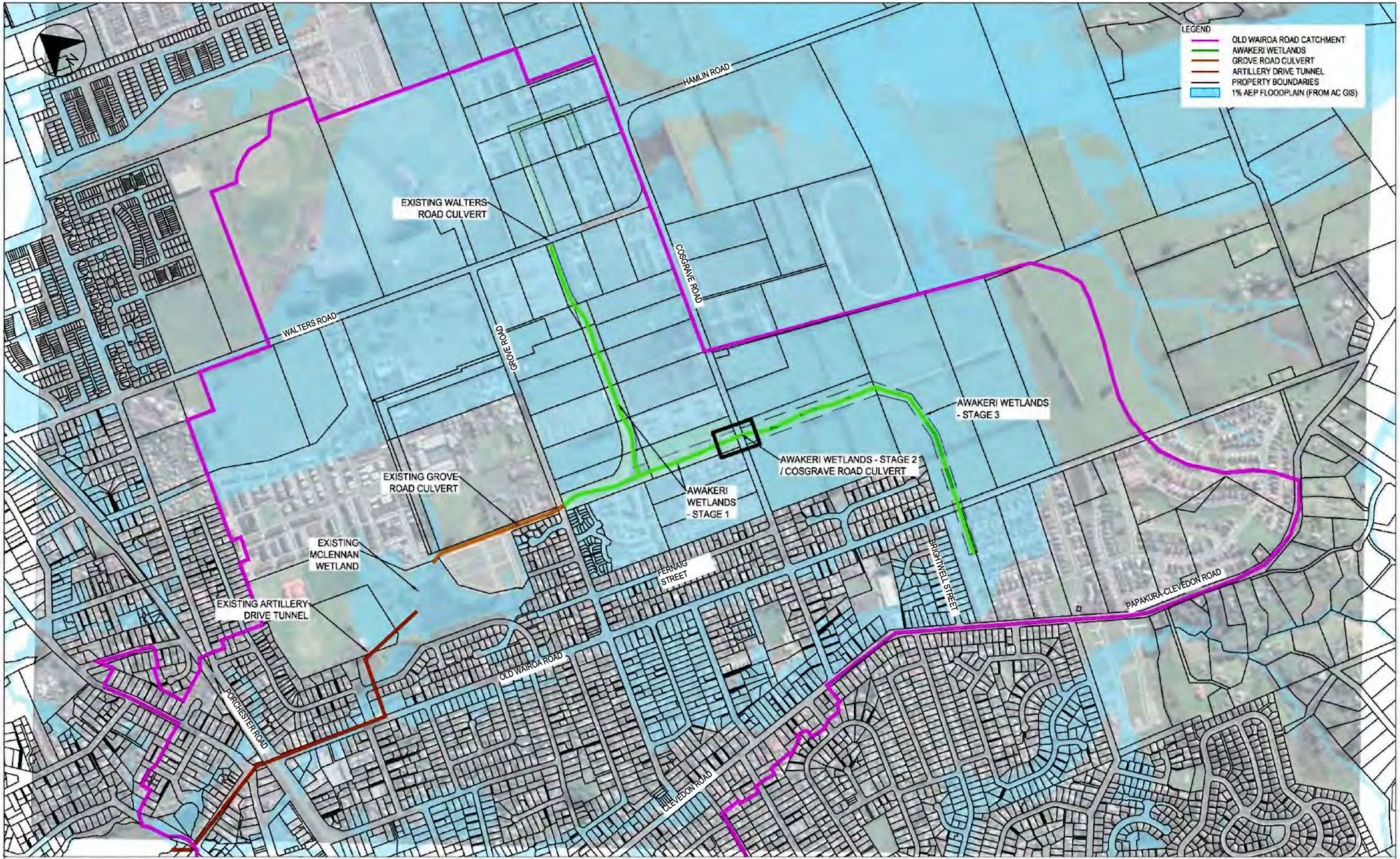

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					DESIGNED BY: J PEETERS		10.10.23
B	10.10.23	SPECIMEN DESIGN	JP	AC	CHECKED BY: A. CUNNINGHAM		10.10.23
A	16.04.19	DRAFT	JP	AC	APPROVED FOR RELEASE BY: A CUNINGHAM		10.10.23

Healthy Waters
Design Office
AUCKLAND COUNCIL, LEVEL 3, BLEDISLOE HOUSE

Auckland Council
Te Kaunihera o Tāmaki Makaurau

TITLE:
AWAKERI WETLANDS - STAGE 2
COSGRAVE ROAD CULVERT - SPECIMEN DESIGN
GREATER SCHEME LOCATION PLAN

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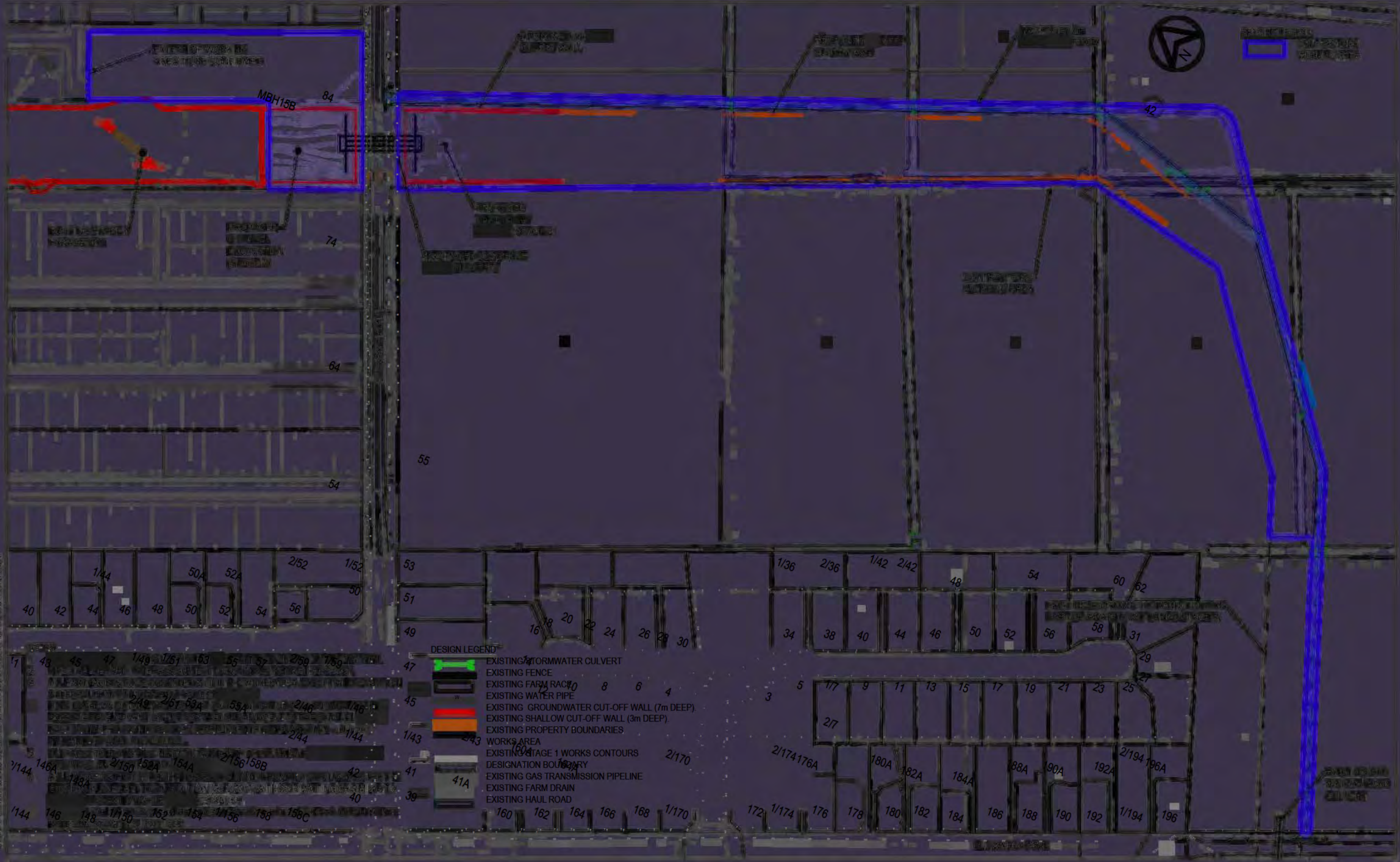

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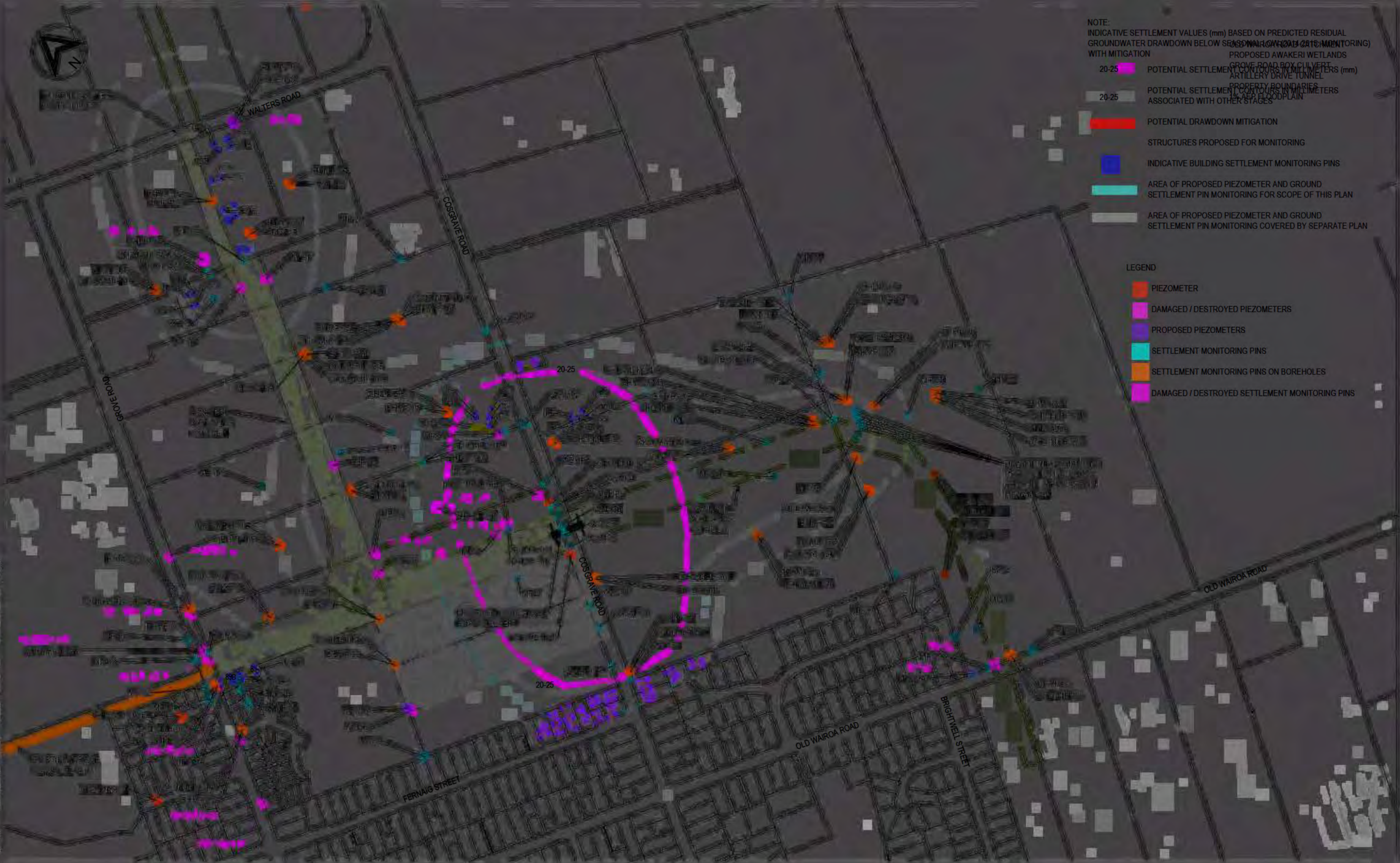
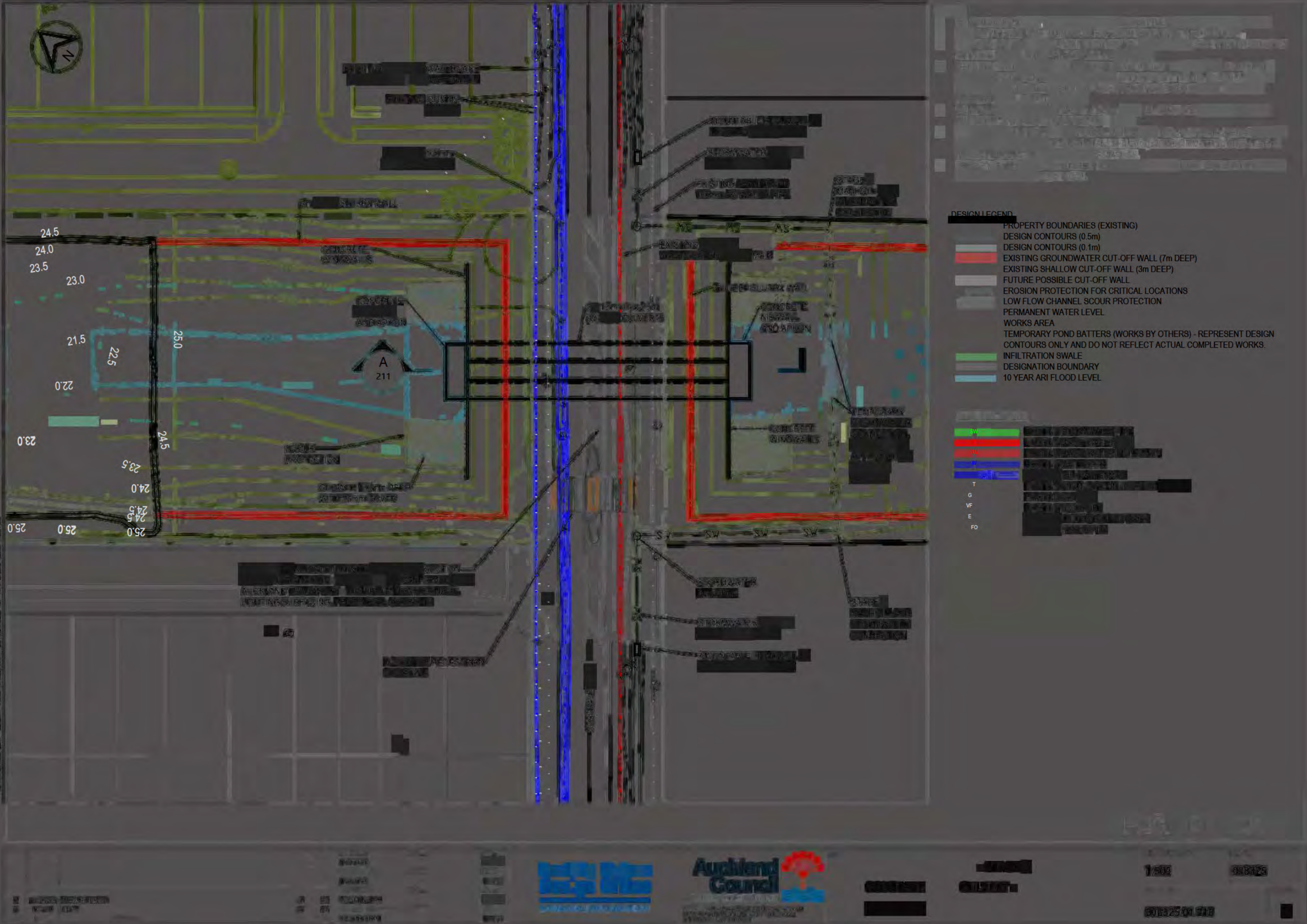


Auckland Council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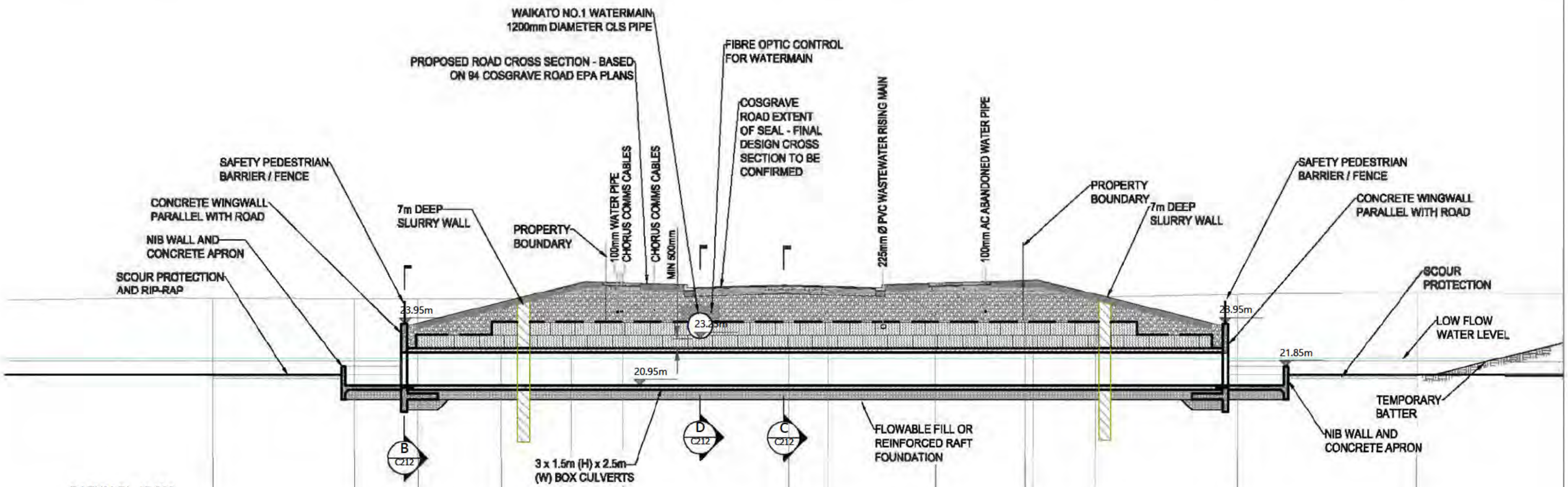
DESIGN LEGEND
PROPERTY BOUNDARIES (EXISTING)
DESIGN CONTOURS (0.5m)
DESIGN CONTOURS (0.1m)
EXISTING GROUNDWATER CUT-OFF WALL (7m DEEP)
EXISTING SHALLOW CUT-OFF WALL (3m DEEP)
FUTURE POSSIBLE CUT-OFF WALL
EROSION PROTECTION FOR CRITICAL LOCATIONS
LOW FLOW CHANNEL SCOUR PROTECTION
PERMANENT WATER LEVEL
WORKS AREA
TEMPORARY POND BATTERS (WORKS BY OTHERS) - REPRESENT DESIGN CONTOURS ONLY AND DO NOT REFLECT ACTUAL COMPLETED WORKS.
INFILTRATION SWALE
DESIGNATION BOUNDARY
10 YEAR ARI FLOOD LEVEL
Auckland Council

NOTES:
1. SCHEME DESIGN ONLY. FINAL DESIGN TO BE DETERMINED DURING DETAILED DESIGN.
2. PIPE DIAMETER AND MATERIALS SUBJECT TO CONSTRUCTION METHODOLOGY.
3. ALL EARTHWORKS TO BE CONTAINED WITHIN THE WORKS AREA, EXCEPT WHERE WRITTEN APPROVAL FROM LANDOWNERS HAS BEEN OBTAINED
4. DETAILS SHOWN OUTSIDE THE DESIGNATION ARE INDICATIVE ONLY AND REPRESENT A POSSIBLE DEVELOPMENT LAYOUT FOR THE LAND ADJACENT TO THE CHANNEL. THIS IS BASED ON THE 'TAKANINI CASCADES DEVELOPMENT FRAMEWORK PLAN - JULY 2017' PREPARED BY AUCKLAND COUNCIL.
5. SURFACES TO BE PLANTED IN ACCORDANCE WITH THE AUCKLAND COUNCIL TAKANINI CASCADES LANDSCAPING PLAN SHEETS 1 TO 16.
6. ALL EXPOSED SURFACES TO BE PROGRESSIVELY AND PROMPTLY STABILISED AS PER THE EROSION AND SEDIMENT CONTROL PLAN (GHD, 2017) WITH COIR MATTING OR SIMILAR TO PROVIDE PROTECTION WHILE PLANTS ESTABLISH.
7. SET-OUT LEVELS AND CO-ORDINATES TO BE BASED OFF THE BENCHMARK ORIGIN POINTS NOTED ON DRAWING 51-33411-V001.



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(A) SECTION - COSGRAVE ROAD CULVERT LONGSECTION
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SPECIMEN DESIGN

Plot Date: 10-Oct-23 1:57 pm

REVISION	DATE	AMENDMENT	BY	APPD.
B	10.10.2023	SPECIMEN DESIGN	JP	AC
A	18.04.19	DRAFT	JP	AC

	SIGNED:	DATE:
DRAWN BY: J PEETERS		10.10.23
DESIGNED BY: J PEETERS		10.10.23
CHECKED BY: A CUNNINGHAM		10.10.23
APPROVED FOR RELEASE BY: A CUNNINGHAM		10.10.23

Healthy Waters
Design Office
AUCKLAND COUNCIL, LEVEL 5, BLEDISLOE HOUSE

TITLE:
AWAKERI WETLANDS - STAGE 2
COSGRAVE ROAD CULVERT - SPECIMEN DESIGN
SECTIONS

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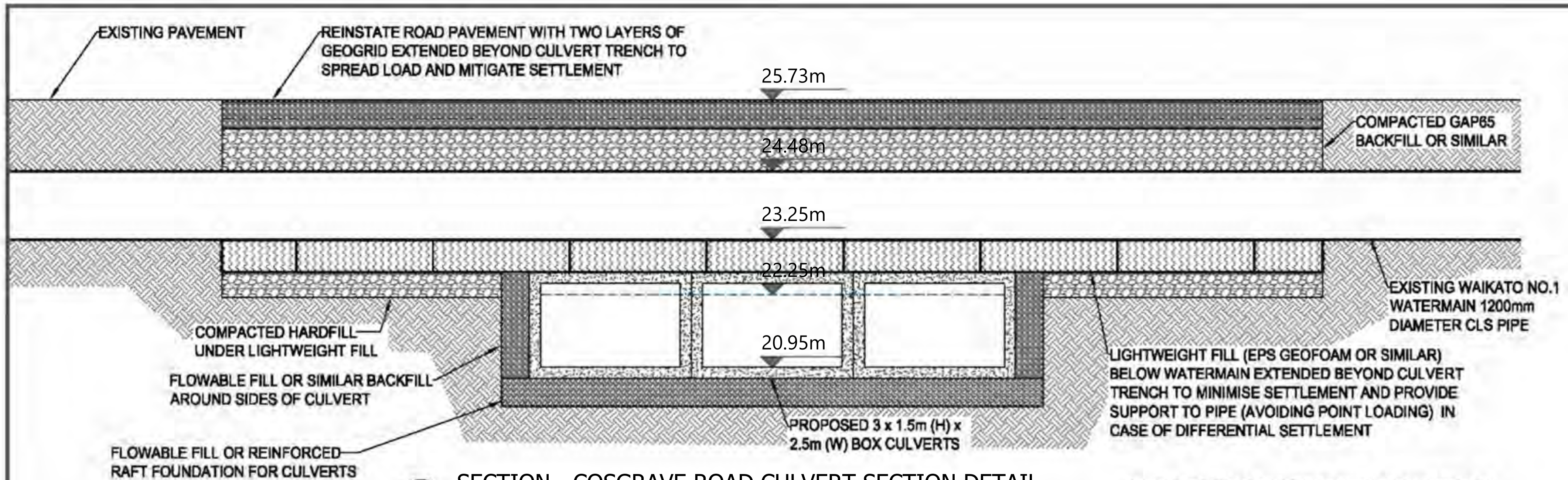
WBS No: 008325
DRAWING No: 008325.01.211
REVISION: B



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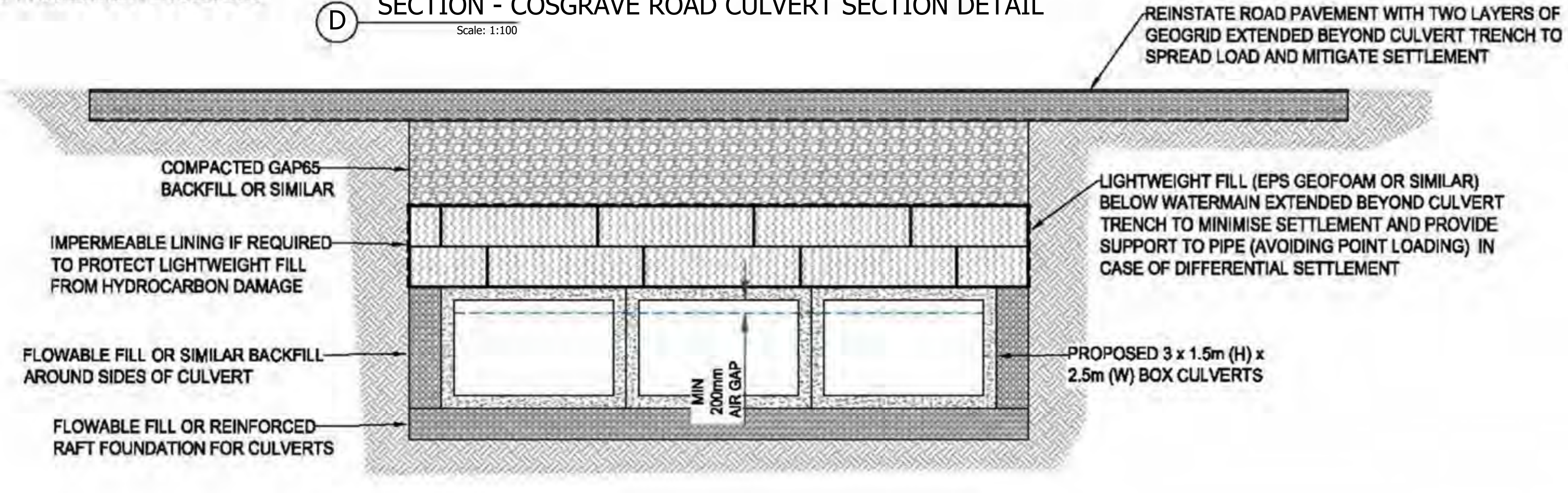

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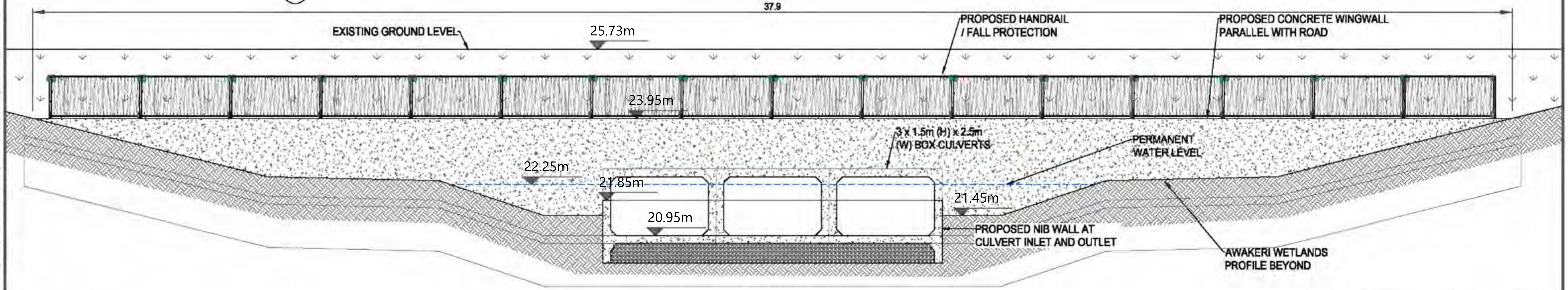

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NOTES:
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Healthy Waters Design Office
AUCKLAND COUNCIL, LEVEL 3, BLEDISLOE HOUSE

TITLE:
AWAKERI WETLANDS - STAGE 2
COSGRAVE ROAD CULVERT - SPECIMEN DESIGN
SECTION DETAILS

ORIGINAL SCALE A3: 1:100
WBS No.: 008325
DRAWING No.: 008325.01.212
REVISION: B

AWAKERI WETLANDS STAGE 3

DESIGN REQUIREMENTS

Healthy Waters

Design Office

Awakeri Wetlands – Stage 3
Design Requirements

Auckland Council
Te Kaunihera o Tāmaki Makaurau

Project	AWAKERI WETLANDS STAGE 3
Document	DESIGN REQUIREMENTS
Issue and Revision Record	
Status	Issued
Version	1
Date of Issue	27/11/2023
Author and Designer	Jesse Peeters Senior Healthy Waters Specialist
Reviewed	Amelia Cunningham Design Office Team Manager
Approved for Issue	Amelia Cunningham Design Office Team Manager

Limitation: *Auckland Council accepts no liability or responsibility whatsoever for or in respect of any use of or reliance upon this report by any third party.*

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Table of Contents

1.0 Project extent and staging

The project area for the Awakeri Wetlands is shown in Figure 1 below and is broken into three stages by location:

- Stage 1: Awakeri Wetlands between Grove Road, Cosgrave Road and Walters Road
- Stage 2: Culvert crossing Cosgrave Road and connection to Stage 1
- Stage 3: Awakeri Wetlands between Cosgrave Road, Old Wairoa Road and the pond upstream of Old Wairoa Road

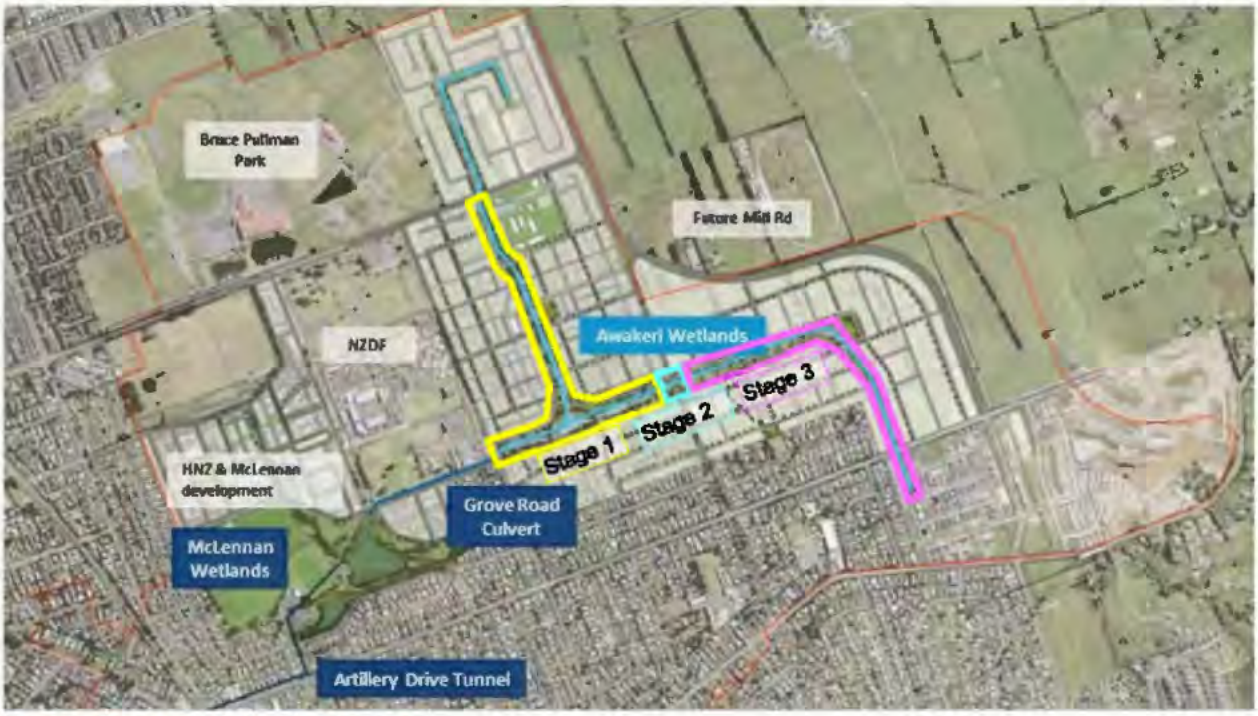

Figure 1 Awakeri Wetlands Staging

2.0 Design criteria

2.1 Standards, manuals and publications

The design and physical works shall be done in accordance with the version current at the time of the work of the following standards, manuals and publications except where amended by these Design Requirements:

a) Auckland Council Standard Specifications

b) The Building Act

c) Health and Safety in Employment Act

d) Resource Management Act

e) Maritime Safety Regulation

f) New Zealand Standards and codes

g) Auckland Council Code of Practice for Land Development

h) Auckland Transport Code of Practice

i) NZ Transport Agency Standards and Guidelines Manual

j) NZ Transport Agency Standard Specifications and Publications.

k) NZ Transport Agency Bridge Manual

Where the above does not explicitly cover all parts or issues relating to the design or construction, other codes or standards, such British, Australian or American that are applicable in respect of a part or issue shall apply where agreed by the Engineer.

Standards, manuals and publications shall be read in the following order of priority:

a) Acts of Parliament

b) The Design Requirements

c) Auckland Council Standard Specifications

d) Auckland Council Code of Practice for Land Development

e) Auckland Transport Code of Practice

f) Regulatory authority standards, specifications and guidelines

g) Australian/New Zealand Standards and guidelines

h) NZ Transport Agency specifications, standards and guidelines

i) British Standards

j) United States Standards.

Where a guideline document allows for different options or where engineering judgement is required, a design report or technical memorandum shall be provided.

2.2 Safety in Design

Safety in design must be considered throughout all stags of design and shall include a register, reporting and workshop to discuss all of the options which are to be considered for each element of the design. Safety in design considerations shall be made for all key features including but not limited to those described in Section 1.3 of this document.

Safety in design shall include input from Auckland Council Healthy Waters Operations, Community Facilities, Auckland Transport, Watercare and any other parties who will be involved with the asset throughout its design life.

2.3 Mana whenua partnership

A partnership was formed between mana whenua and the Awakeri Wetlands project design team during the design of Stage 1 of the Awakeri Wetlands. This partnership must be transitioned to the new designer to ensure that mana whenua continued to be partners of the project.

Regular huis/meetings are expected as part of this, an initial hui shall be arranged by the new designer and mana whenua to agree on hui frequency and level of involvement. The iwi who were part of the partnership are listed below. Key staff / contacts may have changed since and it is the designers responsibility to identify the current representatives of the iwi.

Table 1 Mana whenua representatives

Iwi group	Representatives	Contact
Ngati Tamaoho	Lucie Rutherford Hero Potini Zachary Sirett Edith Tuhimata (previously Ngati Te Ata Waiohua)	[redacted] zac@tamaoho.maori.nz
Te Ākitai Waiohua	Nigel Denny Karen Wilson	kaitiaki@teakitai.com
Ngai Tai ki Tamaki	Jonathan Billington James Brown	kaitiaki@ngaitaitamaki.iwi.nz
Ngāti Te Ata Waiohua	Karl Flavell	karl.flavell@ngatiteata.iwi.nz

2.4 Watercare consultation and approvals

Watercare's Waikato No.1 watermain is in close proximity to the project and Watercare approval will therefore be needed prior to any work taking place. Previous correspondence with Watercare has indicated the following:

- Any shutdowns of the Waikato No.1 Watermain would need to be planned in advance with up to 2 years notice given to Watercare. This requirement may change depending on Watercares scheduled shutdowns so communications should be made with Watercare during the design phase to confirm.
- Finite Element Analysis (FEA) will be required and provided to Watercare to demonstrate that effects on the Waikato No.1 Watermain can be managed.
- Any design of temporary support or ground improvements shall be reviewed and approved by Watercare.

2.5 Local board consultation

The designer shall contact the local board to provide regular updates and accommodate feedback from the local board as required. The frequency of updates shall be determined based on an initial meeting with the local board to agree on frequencies.

2.6 Consents

All required consents (resource, discharge, landowner, etc.) and approvals (Watercare EPA and Works Over Approval, Auckland Transport EPA, etc) shall be obtained by the designer.

2.7 Services

The designer shall be responsible for incorporating the design of any protection or relocation of any known existing services or proposed future services which conflict or cross the proposed work, including but not limited to:

- Fibre optic cables
- Wastewater pipes and wastewater rising mains
- Local watermains
- Overhead power lines
- Underground power cables
- Underground communications cables
- Roads

Approvals shall be obtained from all relevant utility providers where required and clearances shall be provided as per the relevant standards and utility provider requirements.

2.8 Minimum design requirements

General design features of the Awakeri Wetlands Stage 3 scope are shown in the Awakeri Wetlands Stage 3 Specimen Design Drawings. The Specimen Design Drawings shall be referred to and significant deviations from the key features shown in these drawings shall be documented and approved by Auckland Council.

It is the designer's responsibility to determine the final design criteria, however the following section provides minimum requirement and sets out Auckland Councils expectations.

2.8.1 Key features of design

Stage 3 of the Awakeri Wetlands must include the following features:

- Low flow channel and wetland bench
- Erosion protection
- 2.5m wide shared path
- Boardwalks
- Staircases
- Informal stepping logs
- Removable bollards
- Overland flowpaths
- Weirs
- Stormwater connections
- Road culvert crossing
- Debris screen

- Groundwater cut-off barrier
- Planting, mulching and erosion control matting
- In-situ swamp kauri

2.8.2 Layout, framework plan and wider context

A framework plan (shown in Figure 2 was prepared by Auckland Council for development adjacent to the Awakeri Wetlands to ensure co-ordination between the Awakeri Wetlands layout and the adjacent development layout.

A park and neighbourhood centre is proposed at the eastern end of the Awakeri Wetlands and interaction between the wetland, park and neighbourhood centre is required. The designer shall communicate with Auckland Council Parks to ensure the park and wetland are well planned and co-ordinated.

Figure 2 Awakeri Wetlands Stage 3 Framework Plan

2.8.3 Design Life

A design life of not less than 100 years shall be allowed for infrastructure assets unless agreed otherwise with Auckland Council, taking into consideration the low pH / aggressive ground conditions such as potential acid sulphate soils. Further information is available in the *Hydrogeology Assessment of Effects (GHD, 2016)* and the *Acid Sulfate Soils Management Plan (GHD, 2017)*.

2.8.4 Design flow

The design of the culvert shall be able to convey up to the 50% AEP, 10% AEP and 1% AEP storm event without resulting in:

- Flooding of the proposed shared paths or boardwalks in the 50% AEP event
- Surcharging of the pipe network in the developments beyond the Awakeri Wetland boundary during the 10% AEP event.
- Surcharging of overland flowpaths of developments within the catchment during the 1% AEP event.

Catchment flows for Stage 3 of the Awakeri Wetlands are outlined in Table 2, provided the catchment, development and impervious area assumptions from the Awakeri Wetlands Stage 1 Detailed Design Report are met. If these assumptions are altered in the developers proposal, then the flows shall be recalculated based on the updated assumptions.

Table 2 Peak flows in the Awakeri Wetlands (Stage 3)

	MIKE11 modelled peak flow (m^3/s)		
Chainage (m)	50% AEP	10% AEP	1% AEP
500	5.7	14.6	23
600	5.6	14.3	22.6
700	5.2	13.5	21.3
800	4.9	12.6	19.9
900	4.5	11.6	18.5
950	4.3	11.2	18.0
1000	1.9	5.8	9.6
1100	1.8	5.6	9.2
1200	1.6	5.3	8.7
1300	1.5	5.1	8.2
1400	1.4	5.0	7.9
1500	1.4	3.2	4.7

2.8.5 Awakeri Wetlands low flow channel and wetland bench

The Awakeri Wetlands low flow channel is typically 800mm deep and varies in width. The invert of the channel is flat, with a step in elevation at each weir location. Some localised deeper areas are proposed.

On the edges of the low flow channel is a wetland bench where the water level varies from 200mm to 0mm (refer to Figure 3). The wetland bench provides a safety warning prior to reaching the deeper water, and includes wetland planting for shade, habitat and water quality benefits.

The low flow channel and wetland bench shall be as per the Specimen Design Drawings unless modified and approved via the resource consent process. In particular levels shall not be modified as these form the basis of the groundwater and settlement effects assessment.

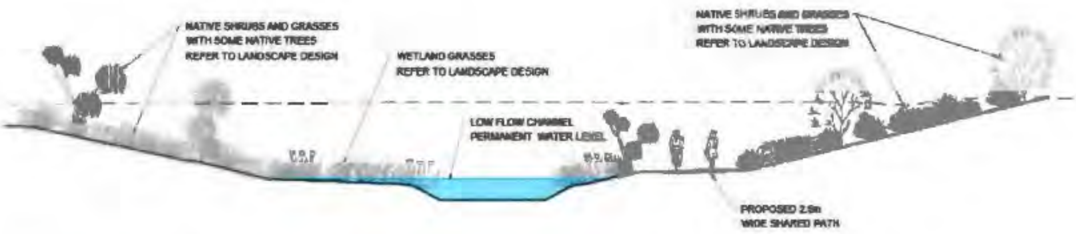

Figure 3 Typical cross section

2.8.6 Scour and erosion protection

The design shall include the 2 year, 10 year and 100 year ARI peak flow scenarios for assessment of scour and erosion potential by carrying out a permissible shear stress analysis. Scour protection shall be designed and installed at key areas along the channel where the applied shear stress exceeds the permissible shear stress of the surface and a structure or boundary is at risk of undercutting or damage.

Large riprap and concrete shall be avoided where possible for scour protection. Naturalised methods of scour protection such as planting is preferred. Oversizing culverts to minimise velocity, energy and shear stress is preferred over providing scour protection to dissipate high energy. Geosynthetic materials to reinforce plant roots such as geoweb and enkamat are preferred over hard engineering solutions.

Consideration to lining the underwater base of the wetland with geotextile filter fabric, enkamat and gravel shall be made to avoid scour, discharge of sediment and soft exposed peat which can be a safety hazard. This also provides a distinct base that can be identified during maintenance or desilting.

The specimen design drawings provide a high level indication of where erosion protection materials are likely to be required based on protecting hard assets. Monitoring of unreinforced areas is an acceptable approach where scour does not risk undermining structures or properties.

Figure 4 Geoweb scour protection, enkamat and gravel lining for underwater wetland base

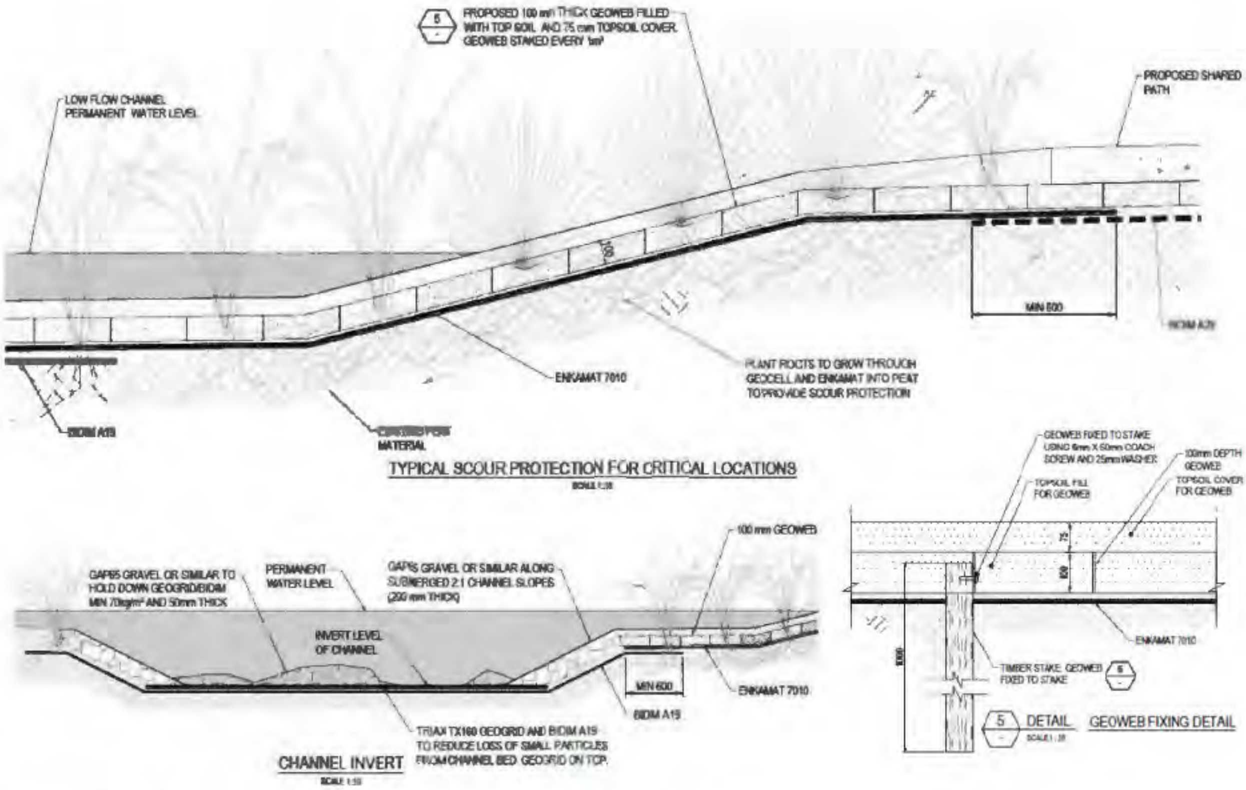

Figure 5 Scour protection details for Stage 1 of the Awakeri Wetlands

2.8.7 Shared path

A shared path must be designed to follow the approximate alignment shown in the Specimen Design Drawings. Final alignments may be adjusted to align with the final development lot layout. The shared path shall be a similar detail to the shared path in the Awakeri Wetlands Stage 1. The structural design of the path shall be provided by the designer, with the following minimum requirements:

- Minimum 2.5m width
- Maximum gradients in accordance with Auckland Transport and Auckland Council Shared Path standards.
- Design loading to be determined by structural engineer and shall include allowance for light maintenance vehicle loading, defined as:
    - A vehicle or combination of vehicles having a gross vehicle weight not exceeding 7.2 kN consisting of 3 axle loads of 2.4 kN each, spaced 1500 mm apart. Each axle load shall consist of two wheel loads of 1.2 kN each spaced at 500 mm centre to centre. Each wheel load shall be applied over a square not greater than 150 mm x 150 mm. Typical examples include a power carrier or a 4-wheel motorcycle towing a trailer.
- Exposed aggregate finish using the same river pebble aggregate (Longburn Pebble)
- Red oxide (Peter Fell 468 or similar) shall be added to the concrete mix at key areas to act as a warning for pedestrians / cyclists at path intersections and at entry/exits to boardwalks. Red coloured strips shall be created in a similar way to the Awakeri Wetlands Stage 1 design. Final location of red strips to be agreed with Auckland Council.

- Concrete strength to be specified by designer.
- River stone drainage channel on the uphill side of the path to prevent groundwater flowing across the path surface
- Foundation consisting of geoweb filled with drainage metal to allow flow of water underneath path without loss of material (similar to permeable paving basecourse)
- Geotextile under geoweb to prevent drainage metal mixing with subsoils
- Control joints at 3m spacing
- Dowel bars at control joints to minimise movement

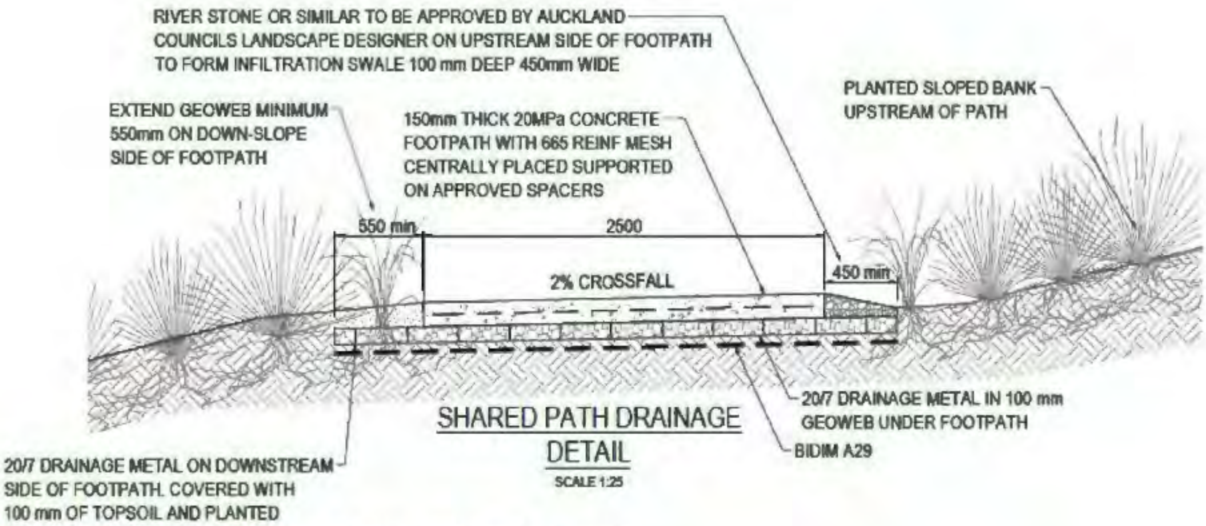

Figure 6 Footpath detail from Awakeri Wetlands Stage 1

Figure 7 Footpath photo from Awakeri Wetlands Stage 1

2.8.8 Staircases

Staircases may be required where the maximum gradient of a path exceeds allowance longitudinal slope for shared paths. If a staircase is proposed, there must be an alternative accessible route available. Staircases shall be a similar detail to the staircases in the Awakeri Wetlands Stage 1. Minimum requirements for the staircases are:

- Red coloured concrete to match the warning strips on shared path.
- Exposed aggregate tread.
- Hand rail matching the Awakeri Wetland Stage 1 staircases.
- Cycle ramp to match Awakeri Wetlands Stage 1 staircases.
- Minimum 2.5m wide to match shared path.
- Tread depth to be 360mm and tread height to be 120mm.

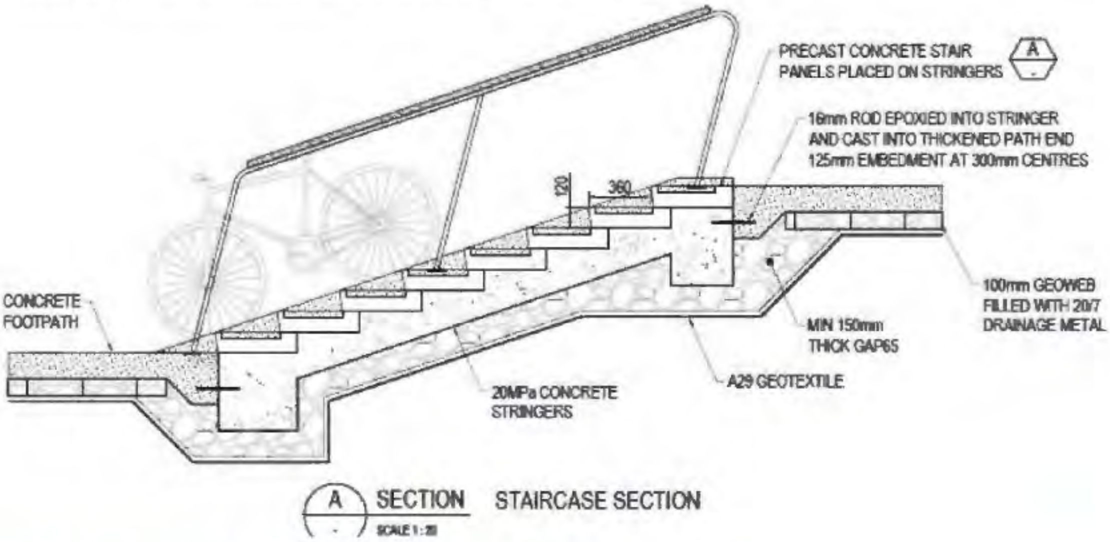

Figure 8 Staircase detail from Awakeri Wetlands Stage 1

Figure 9 Photo of staircase from Awakeri Wetlands Stage 1

2.8.9 Informal stepping logs

Informal stepping logs shall be designed to provide informal access from the shared path to useful connections within the development or points of interest within the Awakeri Wetlands, where a formal shared path or staircase is not required. The informal stepping logs shall use a similar detail to the informal stepping logs in Stage 1 of the Awakeri Wetlands, including:

- Two sizes of stepping logs to be used to create variability:
    - Size one: 1500mm (L) x 250mm (W) x 150mm (D)
    - Size two: 2000mm (L) x 300mm (W) x 200mm (D)
- Leading edge and face needs to be refined with a 15-20mm chamfer around top face.
- Timber for stepping log to be Eucalyptus or similar approved.
- Stepping logs to have concrete footings which shall be specified by the designer.

Figure 10 Informal stepping logs photo from Awakeri Wetlands Stage 1

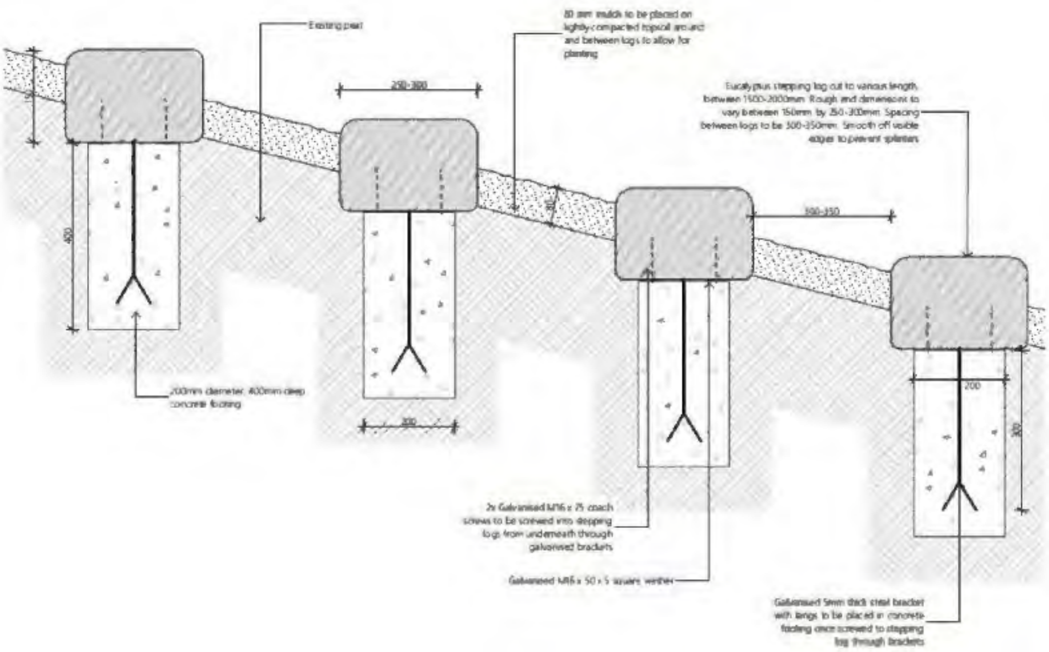

Figure 11 Stepping log detail

2.8.10 Boardwalks

Boardwalks shall be designed at the approximate locations shown in the Specimen Design Drawings. Final localities/alignments may be adjusted to align with the final development lot layout. The boardwalks shall use a similar detail to the Awakeri Wetlands Stage Boardwalks including:

- Straight decking shall be 140mm x 45mm decking.
- Weaving pattern decking shall be 90mm x 45mm decking.
- Timber decking pattern with the direction weaving pattern, same Tonka hardwood timber species, stainless steel plate running longitudinally between the decking pattern.
- Timber kerb.
- Shallow concrete pad foundations designed by a structural engineer.
- Width to be 2.5m between inside of kerbs.

Figure 12 Boardwalk decking (left) and Boardwalk foundations (right)

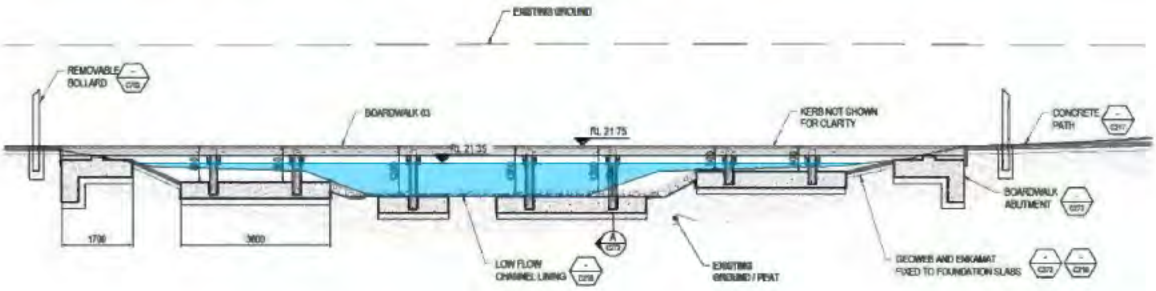

Figure 13 Boardwalk detail from Awakeri Stage 1

2.8.11 Weirs

Weirs shall be designed at the locations and to the levels shown in the Specimen Design Drawings. No changes must be made to this unless the effects are assessed and approved as part of a resource consent.

Weirs shall use a similar detail to the weirs installed in Awakeri Wetlands Stage 1 including:

- Minimum 6m deep PVC sheetpiles, final depth to be specified by the designer based on geotechnical and/or hydrogeological advice.
- Scour pool with 4m deep PVC sheetpiles around perimeter and concrete base.

- Hardwood (Tonka) timber capping on all sheetpiles.
- Fish passage to be designed with input from a qualified freshwater ecologist and to have a similar design to those in Awakeri Wetlands Stage 1.
- Weirs shall be water tight to maintain an upstream water level at the crest of the timber capping.
- Allowance for stormwater connections through the scour pool sheet piles.

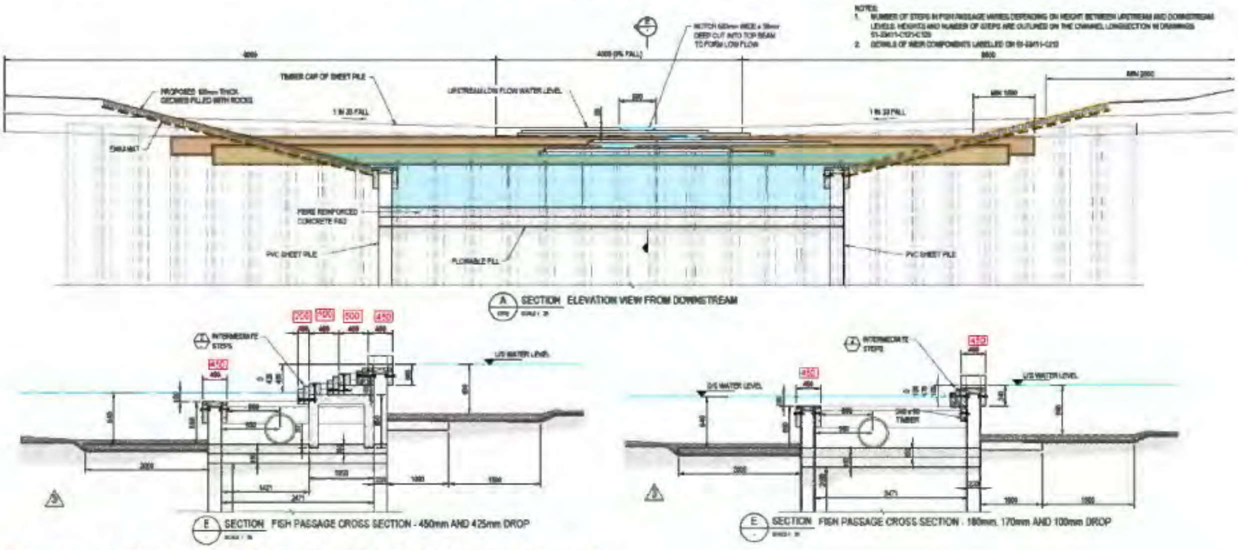

Figure 14 Weir detail from Awakeri Wetlands Stage 1

Figure 15 Photo of weirs from Awakeri Wetlands Stage 1

2.8.12 Stormwater Connections

Stormwater pipes shall only be connected into the Awakeri Wetlands at weir locations. One stormwater pipe connection shall discharge into each side of each weir scour pool. The stormwater connection shall use a similar detail to the stormwater connections in Awakeri Wetlands Stage 1 including:

- Low flow pipe connection to the scour pool to have a maximum size of 500mm OD PE100 SDR17. Slope of pipe to be determined by designer.

- Overflow manhole with scruffy dome at the upstream end of the low flow pipe. The lip level of the manhole shall be set below the 10 year ARI event level where possible. Final levels to be determined by the designer.
- Stormwater outfall pipe upstream of the overflow manhole to be made of PE100 SDR17. Slope and pipe diameter to be determined by the designer. This pipe will penetrate the slurry wall and a specific detail is required to reseal the slurry wall around the pipe. Refer to Awakeri Wetlands Stage 1 detail.
- Pipe bedding and support to be determined by the designer, considering soft peat soils and other geotechnical ground conditions.
- Manhole specifications to be determined by the designer, considering soft peat soils, low pH, potential acid sulphate soils. This could include using micro-silica concrete additives, increased concrete strength, Hydura products etc.

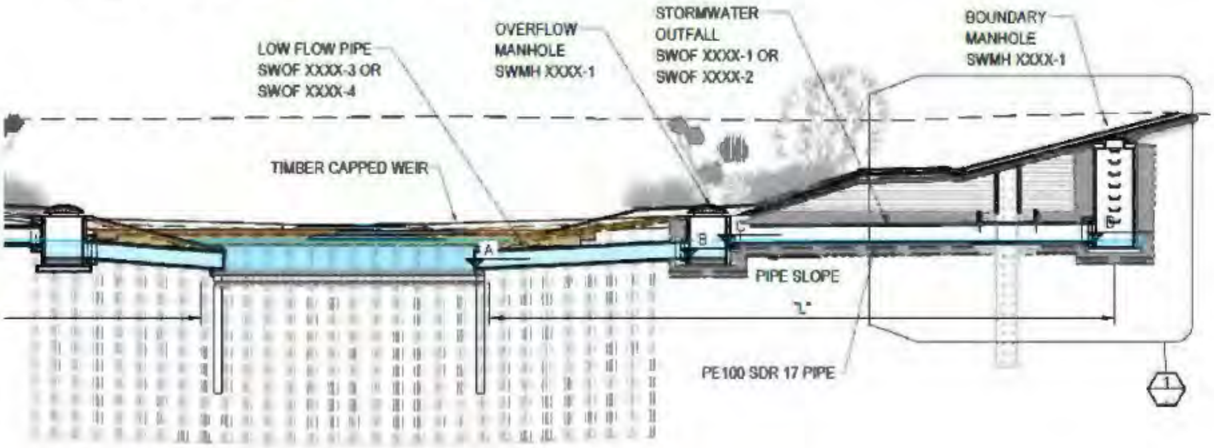

Figure 16 Stormwater connection detail from Awakeri Wetlands Stage 1

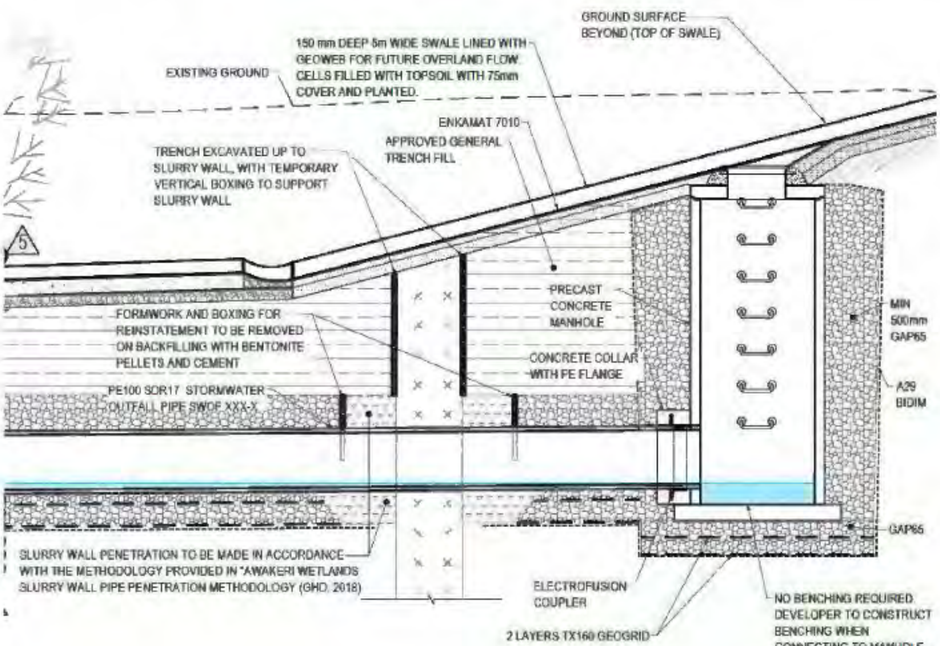

Figure 17 Slurry wall penetration detail

2.8.13 Overland flowpaths

Overland flowpaths within the adjacent development shall be designed as per the Auckland Council Stormwater Code of Practice. Overland flowpath locations shall be co-ordinated with the development lot and road layouts and connections shall generally be located to align with the weirs such that water flows towards the bubble up manholes of the stormwater connections. This allows the scour protection around the bubble up manholes to be utilised for overland flow.

Overland flowpath connections shall be designed to match the Awakeri Wetlands Stage 1 design including:

- A dip in the shared path with a maximum gradient of 1:10
- Minimum base width of 2m, to be confirmed by designed based on flow rate.
- Erosion protection along the overland flowpath to be specified by the designer but to generally included planting of native grasses as the main erosion control method. Geoweb and enkamat can be used in conjunction with native grass roots to provide additional protection.

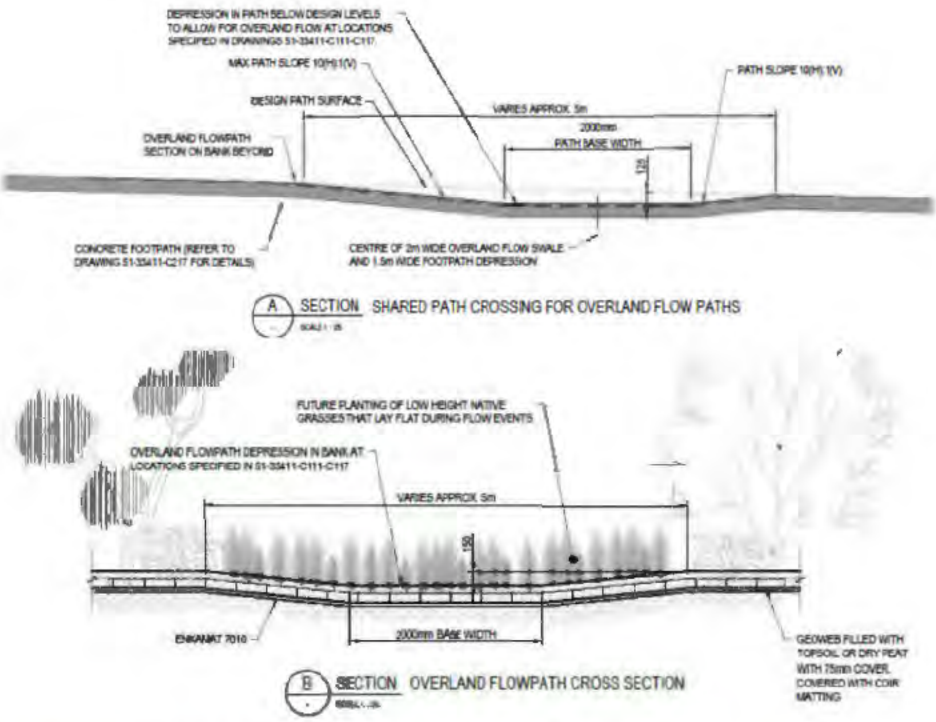

Figure 18 Overland flowpath detail from Awakeri Wetlands Stage 1

2.8.14 Wastewater crossings

Wastewater crossings may be required to convey wastewater within the adjacent developments. The designer shall determine any wastewater pipe crossing locations and design these to avoid future excavations within the Awakeri Wetlands area.

2.8.15 Wastewater connections

A toilet block is likely to be required at the adjacent park. A wastewater connection shall be designed for the toilet block and any other facilities that require a wastewater connection within the project or adjacent park.

2.8.16 Culverts

A road culvert crossing shall be designed at the approximate location shown in the Specimen Design Drawings. The following section provides minimum requirements for the design of the road crossing culvert.

2.8.16.1 Blockage assessment

A blockage assessment shall be provided which outlines the likelihood and consequence of various blockage scenarios for the culverts. The final allowance for blockage must be agreed with Auckland Council prior to finalization of the design.

Auckland Councils preliminary view is that a safety grill on the inlet and outlet of the culvert would not be required due to its expected short length, large height/width and location in the upper part of the catchment, however the requirement of a safety grill would need to be considered as part of the design.

2.8.16.2 Minimum external design loads

The designer shall determine the design load parameters for the crossing with Auckland Transport.

Minimum design load parameters are available in Table 3.

Table 3 Live loads

Item	Load allowance	Reference
HN vehicle loads	3.5 kPa x 3 m wide uniform load *plus* 2 x 120 kN axle loads at 5 m ctrs, or 12 kPa surcharge pressure (as appropriate)	Bridge Manual (SP/M/022) – Section 3.2.2 and Figure 3.1 and Section 3.4.12
HO vehicle loads	3.5 kPa x 3 m wide uniform load *plus* 2 x 240 kN axle loads at 5 m ctrs, or 24 kPa surcharge pressure (as appropriate)	

2.8.16.3 Culvert alignment (vertical/horizontal)

The vertical and horizontal alignment of the culverts shall be such that they do not adversely affect the integrity of any existing structures and considers safety, operation and maintenance considerations.

The design shall consider the safety, operational and maintenance aspects associated with the depth of water in the culvert. The culvert shall generally be placed with its invert level matching the bed level of the wetland, and therefore will include 800mm of permanent water within it. The remainder will be an air gap to allow debris to flow through.

2.8.16.4 Culvert groundwater ingress and settlement considerations

The following groundwater and settlement considerations shall be made when designing culverts within Stage 3 of the Awakeri Wetlands:

- The infrastructure conveying water under the road shall be fully sealed to prevent leakage of groundwater and sediments into the culvert over the design life of the structure. This requirement is important for mitigating the risk of groundwater drawdown and settlement of the surrounding peat soils.
- Post tensioning shall be considered if the structure has joints that are at risk of leaking due to ground movement.
- Settlement of the structure (due to the weight of the structure and/or backfill) shall be assessed and the impact of any predicted settlement on adjacent structures and services shall be considered and mitigated. Use of lightweight backfill shall be considered where/if appropriate.
- Settlement of the ground around the structure (due to any anticipated effects on adjacent groundwater and soil) shall be assessed and the impact of any predicted settlement on adjacent structures and services shall be considered and mitigated.
- Lightweight backfill such as polyrock may be required to reduce the load on subsoils and manage ground settlement.
- Stiffening of the culvert bedding may also be required by including a geogrid raft or flowable fill to mitigate differential settlement across the culvert.

2.8.16.5 Inlet and outlet structures

An inlet / outlet structure shall be provided at each end of the culverts. The headwall/wingwalls shall be similar to the Grove Road culvert outlet wingwalls and shall include:

- Wingwalls/headwall shall be parallel with the road (ie. straight concrete retaining walls)
- Material shall be concrete with an exposed aggregate finish (sandblasted or similar)
- Height of the headwall shall be minimised to minimise the fall height from above.
- Mitigation to discourage access by the public, such as planting.
- Access to be provided for clearing blockages
- Safety barrier / fence to mitigate fall height. Safety fence to match the fence on top of the Grove Road Culvert outlet structure at the McLennan Wetland.
- Erosion / scour protection. Type and extent of erosion protection to minimise visual impact on surrounding environment and align with materials used in Stage 1 of the Awakeri Wetlands.

Figure 19 Culvert headwall/wingwalls

2.8.16.6 Buoyancy

The culvert shall be designed for the effects of buoyancy for suitable scenarios determined by the designer based on nearby groundwater monitoring data and Awakeri Wetlands design information.

2.8.16.7 Road reinstatement

The design shall include details for road reinstatement after the culvert is installed which shall be agreed with Auckland Transport, but at a minimum shall include:

- A pedestrian crossing shall be designed and approval shall be obtained by Auckland Transport any other relevant parties. The designer shall liaise with Auckland Transport and any other relevant parties to determine a suitable crossing detail (ie. signalised, island, zebra crossing).
- The road corridor shall be upgraded to align with the future road cross section. The designer shall liaise with Auckland Transport to determine the future road cross section details.

2.8.17 Dewatering

Any dewatering systems used shall be designed and operated so that related settlement does not exceed limits prescribed by the Resource Consent Conditions in both long term (post-construction) and short term (during construction) scenarios.

The designer shall also consider how groundwater will be managed during construction.

2.8.18 Groundwater cut-off barrier

Previous assessments, as described in the *Hydrogeology Assessment of Effects (GHD, 2016)* have indicated that a vertical groundwater cut-off barrier (7m deep) is required around the perimeter of certain areas of the wetland excavation.

A shallower slurry wall (3m deep) was also proposed along the perimeter of remaining (lower risk) areas to manage groundwater during construction. The location of the proposed slurry wall is shown in the Specimen Design Drawings.

Groundwater barrier requirements

- The cut-off barrier is required to have a maximum permeability of 1×10^{-8} in order to mitigate groundwater drawdown to an acceptable level.
- The base of the excavation during culvert construction shall also have a maximum permeability of 1×10^{-8}.
- The vertical groundwater cut-off barrier was proposed as a bentonite-cement slurry wall. The slurry wall was proposed as minimum 600mm wide and 7.0m deep.
- The mix design for the slurry wall shall be provided by the designer, with lab testing and in-situ field testing to demonstrate that the required permeability can be met and sufficient curing will be achieved.
- Final design of the slurry wall shall be provided by the designer.

Quality assurance

- In-situ QA coring and sampling shall be undertaken during construction to confirm that the required permeability requirements have been met for the installed wall.
- For every hundred meters of the slurry wall, the proper curing of the slurry wall shall be checked as follows:
    - Coring should be done no earlier than 14 days in at least 4 locations in the central axis of the cut off wall and all the way to the full depth of the cut off wall.
    - Coring will enable checking of cut off wall consistency, depth, verticality and width. Holes to be grouted back
    - Number of locations to be drilled will be reviewed and may change in view of the encountered results, as further coring could be necessary.
- Where QA coring or sampling shows non-compliance, the slurry wall shall be repaired to achieve compliance. This may involve re-excavating and reinstalling the slurry wall in some sections.

Contingency plan for obstructions

A contingency plan is required where the slurry wall installation encounters underground obstructions. The Engineer shall be consulted when an obstruction is encountered to determine an appropriate action. Actions could include:

- Investigating the extent of the obstruction through coring, or additional excavating
- Coring through timber obstructions using an excavator mounted coring tool.
- Cutting and removing the obstruction.
- Realigning the cut-off barrier to go around the obstruction.


Figure 20 Slurry wall photo

2.8.19 Debris Screen

A debris screen shall be designed immediately upstream of the Cosgave Road culvert. The purpose of the debris screen is to catch large objects that float down the network during large storms which could block the culvert inlet such as mattresses, cars, woody debris, vegetation or other large items.

The debris screen shall use a similar detail to the debris screen in Awakeri Wetlands Stage 1 including:

- Dead hardwood gum trees or similar embedded into the ground in an array designed to capture large debris.
- A maximum opening size of 800mm shall be achieved along the screen in regards to distance between each log post.
- Suitable embedment and a concrete ring for support shall be designed to ensure the logs are stable during operation.
- Dead trees shall also be designed to be easy to replace after damage or at the end of their design life.
- Additional dead trees shall be placed around the screen to achieve a natural aesthetic, rather than installing the minimum number of dead trees to achieve performance.
- Ends of dead trees shall be charred to represent a burnt forest appearance to align with the Awakeri Wetlands Stage 1 design.

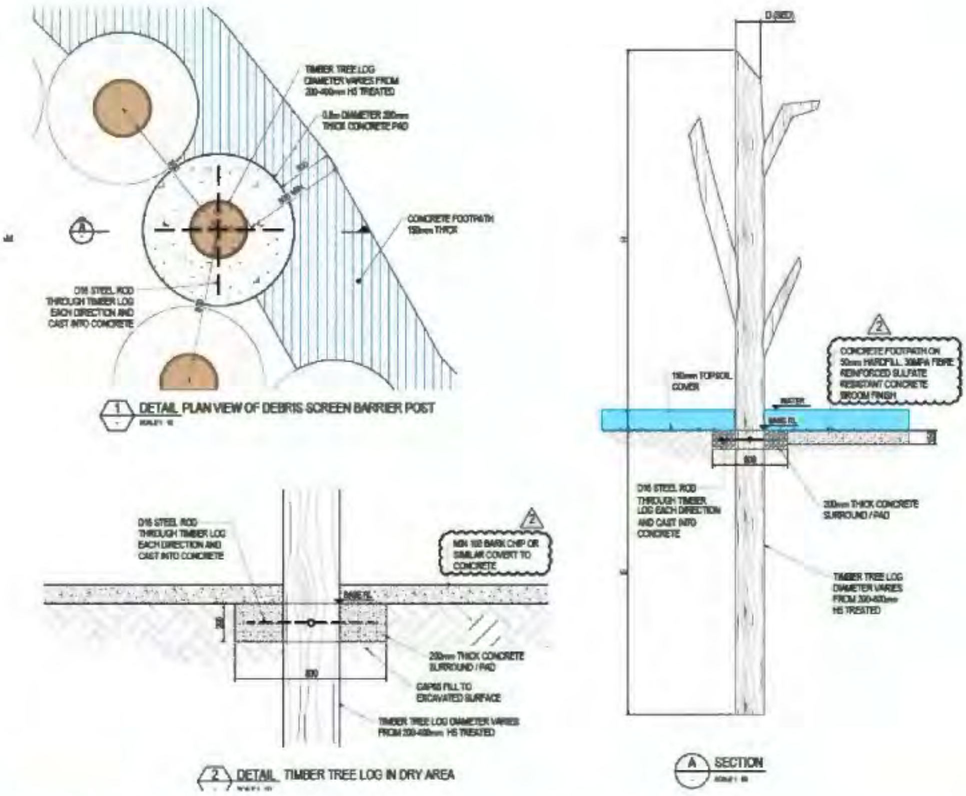

Figure 21 Debris screen detail from Awakeri Wetlands Stage 1

Figure 22 Debris screen from Awakeri Wetlands Stage 1

2.8.20 Water supply

A water supply pipeline shall be designed within the project to service drinking water fountains, toilets and any other assets within the project or adjacent park area that require water. This shall include at least one water meter connection at the boundary, or possibly multiple if multiple connection points achieves a better design outcome.

2.8.21 Lighting

Lighting design for the project shall be provided, including electrical design. This shall include for the adjacent park area.

2.8.22 Signage and wayfinding

Signage and wayfinding shall be designed to match the signage and wayfinding design for Awakeri Wetlands Stage 1. Locations and quantity of signs shall be determined by the landscape designer and reviewed/approved by Auckland Council. Signage and Wayfinding shall include:

- Entry plinth signs
- Entry blade signs
- Directional bollards
- Flood warning steel plaques
- Information boards

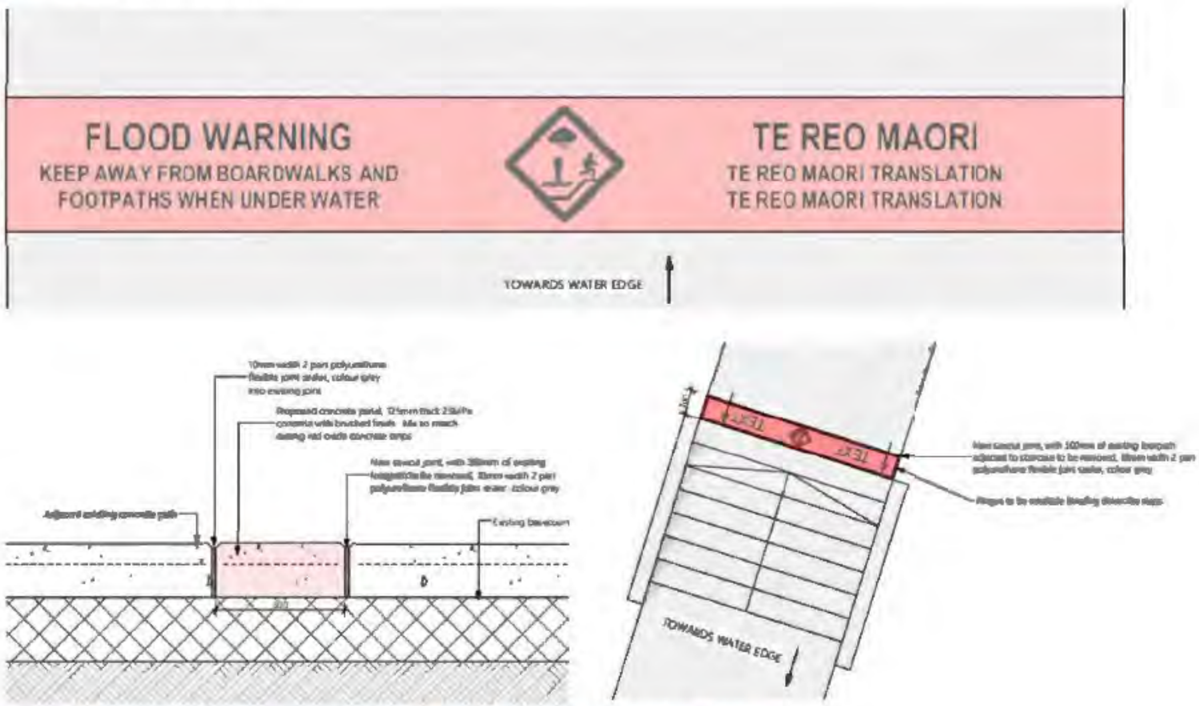

Figure 23 Flood warning plaque detail from Awakeri Wetlands Stage 1

2.8.23 Furniture

Furniture shall be designed to match the furniture design for Awakeri Wetlands Stage 1. Locations and quantity of furniture shall be determined by the landscape designer and reviewed/approved by Auckland Council. Furniture includes:

- Streetscape Statesman seats and benches
- Streetscape Mondo Accessible Picnic Set
- Streetscape Pan Bin
- Scope Cycle Rack

- Blok Drinking Fountain

Figure 24 Furniture details from Awakeri Wetlands Stage 1

2.8.24 Removable bollards

Removable bollards shall be designed to match the Awakeri Wetlands Stage 1 design. Removable bollards shall be located at each shared path entrance and at each end of each boardwalk. Refer to the Awakeri Wetlands Landscape Design Drawings for more details.

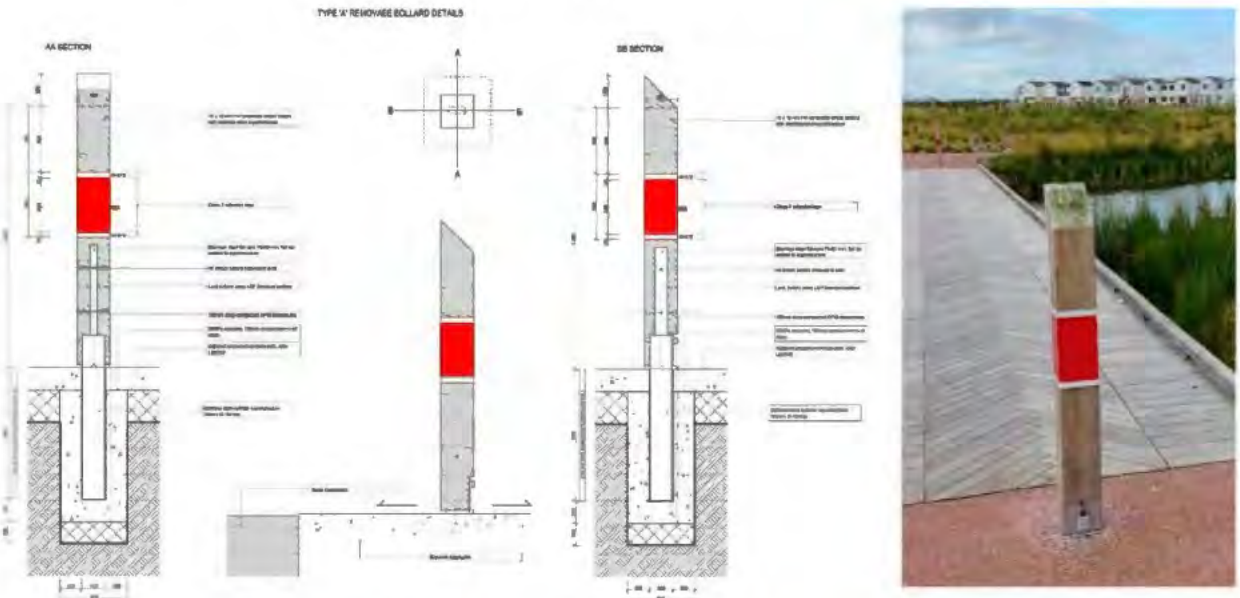

Figure 25 Removable bollard detail from Awakeri Wetlands Stage 1

2.8.25 Planting

Planting shall meet the following requirements:

- Species to match those used in Stage 1 of the Awakeri Wetlands.
- Zones, mixes and planting layouts/clumps to match those in Stage 1 of the Awakeri Wetlands.
- A CPTED analysis shall be used when designing planting zones and shall include the following:
    - Clear sightlines to be maintained along boardwalk crossings, entry points and street interfaces
    - Dense, high growth planting zones to be limited to areas where viewshafts are not required.
    - Refer to planting zones of the Awakeri Stage 1 planting plan for examples.
- All plants shall be eco-sourced from the appropriate ecological district.
- Planting areas to include 100mm of aged arbor mulch. Processed wood chips are not acceptable.
- Planting areas shall include a minimum of 300mm topsoil or local peat soil.
- All planted areas below the 10% AEP flood level shall be covered with 100% biodegradable coconut matting or similar with jute mesh (no plastic mesh).
- All planted areas above the 10% AEP flood event shall be covered with a minimum of 100mm aged arbor mulch.

2.9 Handover

A plan for handover of the asset to Auckland Council shall be prepared by the designed and reviewed / approved by Auckland Council during the design phase. The handover plan shall include:

- Confirm timeframes for handover, allowing for a minimum 12 month defects liability period.
- Confirm which departments will own and maintain each asset.
- Duration of planting maintenance by the contractor prior to handover to Auckland Council.
- Responsibility for completing resource consent conditions including groundwater and settlement monitoring.

3.0 Design deliverables

3.1 Preliminary Design

3.1.1 Definition

In the Preliminary Design the drawings and technical documentation are developed to the point where a resource consent application may be lodged if required.

It is expected that no significant changes to line, level, sizing or construction techniques will be required at detailed design. The purpose of detailed design will be to finalize structural calculations and produce construction drawings.

3.1.2 Safety in design

The Contractor shall conduct a safety in design process for the works.

At the Preliminary Design Stage, the Contractor shall take account within the design of:

- Provision and maintenance of a safe work environment for whole of asset life
- Provision and maintenance of safe plant and structures
- Provision and maintenance of safe systems of work
- Safe use, handling, and storage of plant, substances, and structures
- Provision of adequate facilities for the welfare at work of workers.

At Preliminary Design Phase, a Safety in Design Risk Assessment Register is required to identify current risk exposure in construction, operation, maintenance and decommissioning, plus the proposed risk treatment or improvement opportunity for the preferred option and any residual risks with commentary on how they should be managed.

3.1.3 Preliminary design requirements

As a minimum, the designer shall undertake the following tasks:

- Develop options for variations of the concept design including, but not limited to, construction methodology, vertical and horizontal alignments, exact locations of features, materials. The purpose of this exercise is for the designer to ensure that an optimum solution is developed for detailed design that has the lowest whole-of-life cost whilst still meeting all other project objectives and providing a safe working environment throughout the life of the asset.
- Identifying and scoping any additional investigations the designer considers necessary to properly complete the design
- Preliminary design report. The designer shall prepare a brief Preliminary design report as described below.
- Preliminary design drawings. The drawings shall be sufficiently detailed to support the required consent applications. The designer shall allow for all drawings and supporting documents required for a consent but they shall include as a minimum:
    - General arrangement drawings
    - Long sections and elevations
    - Working areas
    - Preliminary erosion and sediment control plans
    - Preliminary traffic management plans.

3.1.4 Preliminary design report and drawings

The Contractor shall submit a Preliminary Design Report covering the following:

- Hazards identified and mitigated through the safety in design process followed (either appended with a summary within the report or a detailed section within the report).
- Outline specifications for all key components including all structures and materials.

- Key assumptions made.
- Utility diversions (if required).
- Key risks and risk management.
- Recommendations including confirmation that the recommended option meets the project objectives or otherwise.
- Commentary on hydraulic performance.
- A clear statement of the internal quality control and quality assurance procedures adopted in developing the design options.
- Stakeholder and end-user requirements and confirmation that the preferred option meets agreed asset owner requirements.
- Benefits of the option, and how they can be measured during or after project implementation.

3.2 Detailed design

3.2.1 Definition

In the Detailed Design Stage, the design is finalised and complete construction drawings, specifications and any monitoring, QC plans etc are produced.

3.2.2 Safety in design

A safety in design assessment is required for the Detailed Design. This safety in design process should be continued from the Preliminary design phase. The designer should define the methodology used for the risk assessment during this phase for agreement with Auckland Council. Auckland Council will identify the stakeholders that will be involved in the Safety in Design process.

At the end of the detailed design, a Safety in Design Risk Register and Report is required. All residual health and safety risks in construction, operation, design for exceedance, maintenance and demolition or disposal shall be clearly described and conveyed. Where relevant, these should be noted on the construction drawings if they are to be managed during construction.

A stand-alone safety in design report is required. All other reports will require a safety in design section as part of the design reporting.

3.2.3 Detailed design requirements

As a minimum, the Contractor shall undertake the following tasks:

- Scope confirmation workshop. The designers project manager and lead technical staff shall all attend this workshop. The purpose of the workshop will be to confirm the preferred solution and determine any preferences for final details. The designer shall prepare minutes of the workshop.
- Review the preliminary design including; confirmation of the findings of the preliminary design report costings, benefits, etc.
- Carry out all calculations required to finalize materials, structural strengths etc.

- Prepare fully detailed Construction Drawings including:
  - Geotechnical information (bore logs) and existing underground and overhead services shall be shown on the long sections.
  - Detailed construction site access, storage and site office areas.
  - Locations of existing utility services including details of diversions where required.
  - Groundwater and settlement monitoring plan.
  - Structural drawings.
  - Any H&S issues and design mitigation assumptions that may affect or be affected by construction methodology.
- Review Standard Specifications and prepare Particular Specifications.
- Update safety in design and prepare report suitable for hand over to Constructors.
- Prepare a Geotechnical Baseline Report and agree with Auckland Council.
- Submit draft drawings, specifications and plans to Auckland Council for review.
- Finalize pre-construction risk register.
- Detailed design report as below.
- Address comments on the detailed design report and drawings. The designer shall ensure that the drawings and documents are properly reviewed and a QA undertaken by the designer prior to submission. Where any errors or omissions are found that Auckland Council considers to be a failure of QA or review, the drawings and documents shall be returned to the designer without further comment. The formal review will only be undertaken once the QA issues have been resolved. Auckland Council may appoint a peer review in addition to internal reviews. The designer shall allow for collating and assessing the reviews from Auckland Council and shall produce a Construction Documents and Drawing set that adequately addresses the comments.

3.2.4 Detailed design report and drawings

The designer shall prepare and submit a Detailed Design Report. The Final Design Report need not repeat all the detail of the Preliminary design report but should cover the following:

- Details of the final design and proposed construction methodology
- Safety in design considerations in the design process
- The Contractor's assessment of hazards and risks (including H&S) to be managed, if appropriate, a summary of risks eliminated or minimised through design can also be included
- Significant changes to the design or departures from the approved Preliminary Design Report
- Summary of residual risks at contract stage and for whole of life highlighting any remaining high risk items
- Updates to any technical or specialist reports as a result of amendments since the Preliminary design report

- Advice on any variations that may be required to Resource Consents as a result of such changes.
- Any departures from the Auckland Council Code of Practice and Standard Specifications
- Detailed advice on producer statement requirements and the level of construction monitoring required by the designer in order to be able to deliver the producer statements.

Construction drawings, labelled “For Construction”, shall be prepared on the standard Auckland Council title block and include the same information as the Preliminary Design drawings plus the following:

- Intended purpose of the asset (if appropriate)
- Geotechnical information (bore logs) and existing underground and overhead services shall be shown on the long sections
- Updated and more detailed construction site access, storage and site office areas
- Utility services diversions
- Groundwater and settlement monitoring plan.
- Structural drawings.
- Safety in design considerations to be addressed during construction, operation, maintenance and demolition or disposal.

The designer shall facilitate a meeting with Auckland Council to present and discuss the detailed design and the draft construction documentation. The objective of the design meeting is to agree what changes (if any) are required to the Draft Construction Documentation before a final version of the documentation is signed off.

Any review does not remove any responsibility from the designer for the correctness or appropriateness of the design or construction documents.

3.2.5 Ground improvements

The designer shall identify in its Detailed Design Report:

- What ground improvements, if any, are proposed
- The methods used to quantify their extent and effectiveness, and
- The precedent that has been followed in their development.

The methods and precedent shall be referenced in the Detailed Design Report. The effectiveness of ground improvement shall be demonstrated by field testing.

3.2.6 Quality assurance requirements

The designer shall prepare a project review and audit schedule for the Detailed Design and Construction Documentation. This will identify:

- One or more named reviewers accountable for reviewing technical outcomes, and technical reviews planned. Unless otherwise agreed with Auckland Council, the reviewers shall be those named in the Proposal. Where Auckland Council considers that less qualified staff than those shown in the Proposal are offered, revised rates shall be agreed.
- A named Project Director or Sponsor accountable for reviewing overall project delivery, and project outcome reviews planned. Auckland Council has an expectation that there will be a scope review (10%), a progress review (50%) and an outcome review (90%) as a minimum
- All documents will have a QA review before being delivered to the Auckland Council, including draft reports. The report should include a QA and document control page to identify author, reviewer and version control for drafts.

3.3 Geotechnical design criteria

Recent geotechnical investigations are provided in the *Geotechnical Investigations Report (GHD 2016)* and the *Geotechnical and Ground Settlement Effects Report (GHD, 2016)*. This is for information only and the designer is responsible for preparing updated geotechnical investigations and assessments to support their design.

3.3.1 Ground conditions

The ground conditions for the project area are described in the *Geotechnical Investigations Report (GHD 2016)* and the *Geotechnical and Ground Settlement Effects Report (GHD, 2016)* This is for information only and the designer is responsible for preparing updated geotechnical investigations and assessments to support their design.

3.3.2 Groundwater

Groundwater information is provided in the *Hydrogeology Assessment of Effects (GHD, 2016)*. This is for information only and the designer is responsible for gathering updated groundwater information and preparing an updated hydrogeology assessment.

3.3.3 Seismic design

The designer shall consider seismic hazards and liquefaction risks in the design of the project.

Some information is provided in the *Geotechnical and Ground Settlement Effects Report (GHD, 2016)*. This is for information only and the designer is responsible for preparing updated seismic hazard and liquefaction assessments..

3.3.4 Geotechnical Baseline Report (GBR)

The designer shall prepare a GBR for the project.

The purpose of the GBR is to provide a single source contract document containing measurable contractual descriptions of the geotechnical conditions to be anticipated or to be assumed to be anticipated during construction. In the event of the project running into difficulties due to ground conditions, the GBR can be used to decide if the conditions are unforeseen and therefore create potential for a claim or fall within the conditions expected at the site. This does not present any ambiguous interpretation of conditions or any uncertainty. Only measurable, quantitative terms used.

The GBR shall present a very concise contractual summary of the ground model that the Contractor should allow for when tendering for the construction.

The GBR shall be prepared in general accordance with the Essex, R.J., 1996, Geotechnical Baseline Reports for Underground Construction: Guidelines and Practices published by the American Society of Civil Engineers, as amended by this contract.

The GBR shall contain:

- A brief summary description of the material types expected to be encountered.
- The estimated amounts and distribution of different materials along the alignment, typically presented as an estimate of the percentage of the project (for example, linear metres of tunnel) that each material type will make up. This shall be given as a predicted range to allow for geological uncertainty.
- Geotechnical and groundwater parameters, and expected behaviours, for each of these materials, given as a predicted range to allow for geological uncertainty. Include strength, permeability, grain size, mineralogy, predicted pumping rates, predicted settlement and any other aspects which could impact on construction. Wherever possible, these shall be expressed in quantitative terms, and should present the expected distribution envelope of each parameter within the range.
- Descriptions of geotechnical and man-made sources of potential difficulty or hazards that could impact the construction process (such as boulders, bedrock variability, contaminated groundwater, subsurface obstructions, unstable slopes, adjacent activities).
- A description of the anticipated construction methodology with which the baselines are associated. The baseline statements should be clear that the ground can be expected to behave differently with alternative tools, methods, sequences and equipment.

The GBR shall not contain:

- Ambiguous or vague interpretations
- Descriptions or parameters that cannot be easily measured or assessed and recorded during construction
- Qualitative terms such as 'large' or 'major' unless these are clearly defined.

3.4 Operation and Maintenance

A Draft Operations and Maintenance (O&M) Manual shall be supplied with the proposed design solution to enable the proposed solution to be fully assessed and understood by the asset owner. The Draft O&M Manual will include maintenance of fittings, plant and machinery requirements, traffic management, isolation, dewatering (and treatment/disposal of this water), pipe inspection, sediment/debris removal and decommissioning. Auckland Council may engage a third party operations expert to review the acceptability of the solution and the O&M Manual.

The design shall provide for safe personnel access to enable a walk-through of the full length of the new infrastructure for operational and maintenance purposes.

The designer shall finalise O&M manuals, containing such information and details as are necessary for Auckland Council to carry out the operation and cost-effective maintenance of the system. The contents and format of these O&M manuals shall be subject to the approval of Auckland Council and a third party expert.

4.0 Hold points

In addition to reviews of the design packages and elements described above, specific review of the following design items is also required by Auckland Council as they become available:

- Construction methodology.
- Staging of Stage 2 and 3.
- Groundwater management design.
- Peer review for groundwater drawdown assessment and settlement assessment (long term and short term), buoyancy assessment, culvert foundation/bedding design and road backfill design.
- Groundwater and Settlement Monitoring and Contingency Plan.
- Material specifications including shop drawings for pre-cast concrete structures.
- Groundwater cut-off barrier / slurry wall mix design and testing documentation.
- Erosion and scour protection design.
- Obstructions management plan (approach for managing clashes with buried kauri logs etc).
- Landscape design drawings and report.
- Planting plans.
- Preliminary design (Report and Drawings)
- Detailed design (Report and Drawings)

Additional hold points will be required for the construction phase and these will be outlined by Auckland Council prior to construction.

APPENDIX 11 – STAGE 1 AWAKERI WETLANDS DESIGN REPORT

Auckland Council

Awakeri Wetlands - Stage 1A

Detailed Design Report

September 2017

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1. Introduction

The Awakeri Wetlands, also known as Takanini Stormwater Conveyance Channel (TSCC) or Takanini Cascades, forms the fourth stage of a greater scheme to provide stormwater servicing for the Takanini south-east area. The Awakeri Wetlands will pass forward flows from Old Wairoa Road, Cosgrave Road, Walters Road and Grove Road, for which there is currently no formal drainage system, to a box culvert at Grove Road. The Grove Road Box Culvert conveys flows from the Awakeri Wetlands to the McLennan Wetland. During large storm events, flow is attenuated in the McLennan Wetland before being discharged to the Pahurehure inlet via the proposed Artillery Drive tunnel. Refer to Drawing 51-33411-C001 (in Appendix C) for an overview of the greater scheme. The Awakeri Wetlands construction will take approximately 2-3 years to complete.

The Awakeri Wetlands consists of approximately 2.3 km of open waterway that will contain the existing 1% AEP floodplain, allowing the surrounding land to be comprehensively developed.

Resource Consent was granted in September 2016. A Notice of Requirement was approved in October 2016 for the designation of land to allow the development of the TSCC.

The designation corridor will allow for the construction of the Wetlands which will convey low flow and the full 1% AEP flow from the catchment. It will deliver an open public space with the provision for cycleways and footpaths that will increase the connectivity between new urban areas and allow for the development of the Special Housing Takanini Strategic Areas (including Special Housing Areas 2A, 2B and Wallace) and area 2B4 which is currently zoned Future Urban.

The Awakeri Wetlands are proposed to be constructed in three discrete stages, these are shown in Figure 1 below. Stage 1 is split into Stages 1A and 1B; where 1A includes bulk earthworks and hydraulic structures for the wetlands to operate for stormwater management. Stage 1B incudes additional structures and landscaping features. This detailed design report is for Stage 1A of the Awakeri Wetlands.

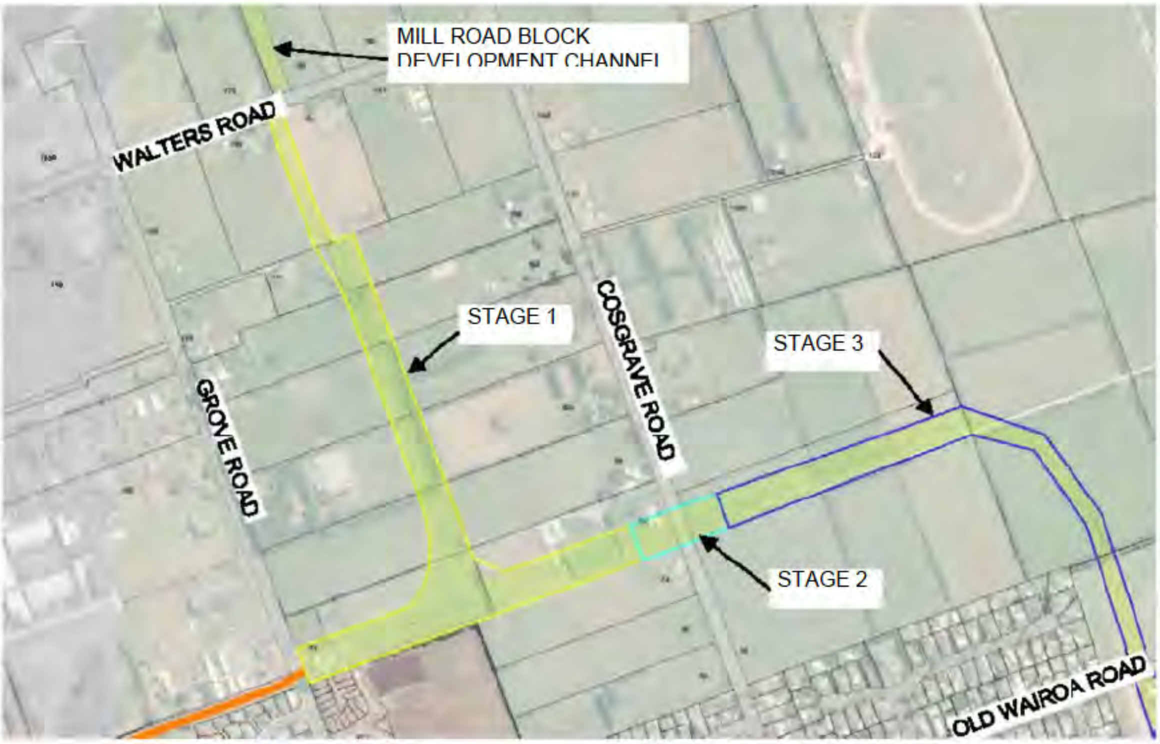


Figure 1 Awakeri Wetlands Staging

1.1 Purpose

The purpose of this report is to document the detailed design of Stage 1A of the Awakeri Wetlands.

1.2 Scope

The scope of this report is to:

- Document the detailed design of Stage 1A of the Awakeri Wetlands.
- Document the design philosophy and design practices relating to the detailed design.
- Provide a record of the key decisions and Safety in Design provisions.
- Document the anticipated maintenance requirements and project risks.

The scope of Stage 1A of the Awakeri Wetlands project includes design of the features required for the channel to operate for stormwater management, primarily as a stormwater conveyance system. This includes:

- Bulk earthworks.
- Erosion and scour protection.
- Weirs.
- Footpaths.
- Boardwalks.
- Culverts.
- Groundwater cut-off barriers.
- Other works required for construction to make the stormwater system operational.

The scope of Stage 1A of the Awakeri Wetlands excludes additional structures such as high level pedestrian bridges or urban design / landscaping features which will be included in Stage 1B. Planting will happen concurrently during Stage 1A, however the specifics relating to planting are documented separately by Auckland Council.

The planting forms an essential part of the erosion protection regime. Discussions with Auckland Council have been undertaken to coordinate the type of plants, as discussed further in Section 4.7.1 and in Section 6.5. As part of this liaison Auckland Council have confirmed that the plants being selected will have extensive root systems. The root systems of the planting are relied on as one strategy for mitigating scour within the channel as discussed within this report and is an integral part of the design.

1.3 Assumptions and Limitations

This report has been prepared by GHD for Auckland Council and may only be used and relied on by Auckland Council for the purpose agreed between GHD and the Auckland Council as set out in this report.

GHD otherwise disclaims responsibility to any person other than Auckland Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Auckland Council and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

2. Project Overview

2.1 Awakeri Wetlands

The proposed Awakeri Wetlands will extend from 989-999 Papakura-Clevedon Road in the south-east to 91 Grove Road in the west. A northern branch will extend northwards towards Walters Road.

In general the Awakeri Wetlands will provide stormwater servicing for future development of Areas 2A, 2B and part of Area 4 (2B4) of the Takanini Structure Plan. At present the area is significantly impacted by the 1% AEP (Annual Exceedance Probability) floodplain, restricting development of the area.

The proposed channel will:

- Provide for the full 1% AEP flows, effectively removing the floodplain from surrounding land.
- Offer an ecological corridor (both terrestrial and aquatic) that would otherwise not be provided.
- Deliver stormwater servicing for development within the catchment area that is not currently presented.
- Afford an open space with significant amenity value and the provision for pedestrian linkages and cycleways.

The Awakeri Wetlands consists of two main branch channels; the main channel and the northern branch channel.

Main channel

The main channel has a length of 1.55 km of open waterway, ranging in depth between 2 m and 4 m below ground level. The channel has an approximate gradient of 0.28% and a total width (at the 1% AEP water level) ranging from 20 m to 37 m. The low flow water width is typically 14 m wide but varies substantially in width and depth.

Northern channel

The northern channel has a length of 0.7 km of open waterway, ranging in depth between 2.4 m and 3.8 m below ground level. The channel has an approximate gradient of 0.20% and a total width (at the 1% AEP water level) of approximately 25 m. The low flow water width is typically 14 m wide but varies substantially in width and depth.

The Awakeri Wetlands is designed with a meandering low flow series of discrete water bodies or wetlands, with a permanent water depth of about 0.8 m controlled by weirs at approximately 100 m centres longitudinally along the base of the channel. These provide an ecological benefit and limit groundwater drawdown. Generally the low flow channel base width varies between 3-6 m and has side slopes of 2H:1V, with an intermediate flat wetland bench. Above the wetland bench are riparian planted channel banks with slope batters 4H:1V integrated into landscape features such as shared paths and play areas.

Figure 2 provides a typical cross section of the Awakeri Wetlands.

Figure 2 Typical section of the Awakeri Wetlands

2.2 Catchment area

The Awakeri Wetlands stormwater catchment (shown in Figure 3) represents the area to be serviced by the Awakeri Wetlands for stormwater conveyance.

The area is approximately 162 hectares (ha) and consists of areas 2A (50.3 ha), 'Mill Road Block' (16.4 ha), 2B4 (57.3 ha) and 2B (38.0 ha) as shown as a dotted purple line in Figure 3.

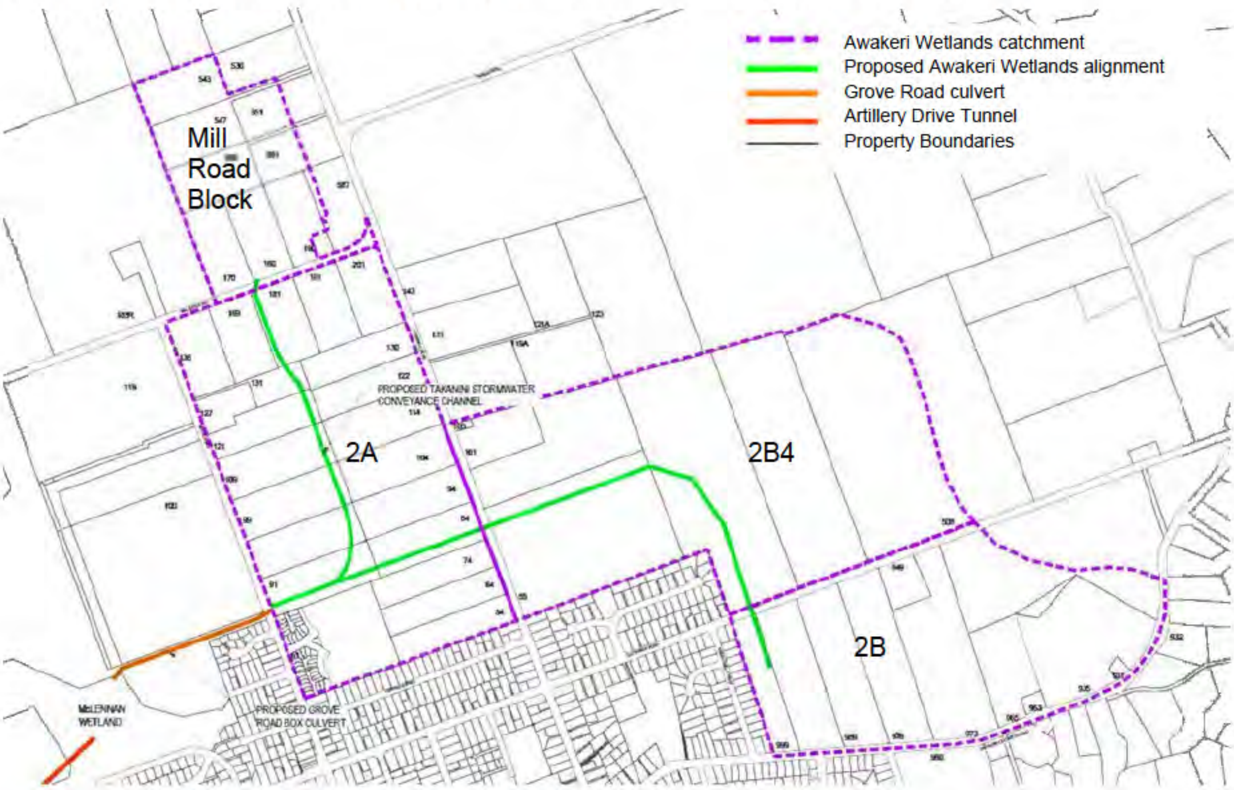


Figure 3 Awakeri Wetlands catchment

2.3 Takanini Stormwater Scheme

The Awakeri Wetlands is part of the Takanini Stormwater Scheme (refer Drawing 51-33411-C001) to reduce flooding for events up to the 1% AEP and provide servicing for the greater Old Wairoa Road catchment. The Takanini Stormwater Scheme is comprised of four sections including:

Part 1 - Artillery Drive Tunnel

A new 2.5 m diameter tunnel that will extend over approximately 1.1 km from the McLennan Wetland to the Pahurehure Inlet. This effectively forms the downstream outlet for the stormwater scheme. The Artillery Drive Tunnel project is currently under construction (2017).

Part 2 - McLennan Wetland

Constructed in 2002, this wetland already receives stormwater from the Housing New Zealand development and Papakura Military Camp through to Bruce Pulman Park in the north; and Willis Road and Clevedon Road to the south. The wetland provides attenuation and treatment for the greater catchment before discharge. Currently the wetland passes forward flows to the Gills Road pond and will continue to do so in the future with only high flows being conveyed through the new Artillery Drive tunnel.

The McLennan Wetland is designed to accept flows from the Old Wairoa Road catchment, which includes the catchment area of the Awakeri Wetlands. The McLennan Wetland has been included in a hydrological model, built and held by Auckland Council. The model indicates that there is enough storage to attenuate flows to an acceptable level which the Artillery Drive Tunnel has been designed in accordance with.

Part 3 - Grove Road Culvert

A new culvert that will convey flows from the Awakeri Wetlands catchment to the McLennan Wetland.

The location of the Grove Road Culvert was altered from the location shown in the Grove Road Structure Plan. The structure plan showed the channel running through the middle of 61 Grove Road and connecting to the proposed Grove Road Culvert at Matheson Street.

The property at 61 Grove Road has subdivision consent and physical works on site are near completion for Stage 1 of their development. As a consequence; the route defined in the Structure Plan is no longer viable. The optimal location for the box culvert connection was therefore to the north of the northern boundary of 61 Grove Road. This allows minimal dissection of private properties and optimises the drainage potential of the surrounding land.

The Grove Road Culvert has been designed by Jacobs (NZ) Ltd and is a separate project to the Awakeri Wetlands.

Construction is currently underway with completion expected April / May 2018.

Part 4 - Awakeri Wetlands

As outlined in this report, a new 2.3 km open channel that will convey flows from part of the Old Wairoa Road catchment (Old Wairoa Road in the south-west to Walters Road in the north) to the Grove Road Culvert. The construction of the channel will take 2-3 years for completion of all stages.

2.4 Zoning and Special Housing Areas

The zoning of the catchment is based on the Unitary Plan zoning. Area 2B4 is currently zoned Future Urban, and therefore a similar level of development has been assumed to the surrounding areas and assumptions made based on existing information from Auckland Council, as described in this report.

2.5 Network Discharge Consent

The Old Wairoa Road Catchment Management Plan (CMP) (PDC, 2004) defines the catchment boundary for the McLennan Wetland. In 2010 the boundary shown in the CMP increased to include part of the Takanini South Catchment through CMP Variation 33738 (2010). This additional area is shown as the 'Wallace' area.

A "trunk stormwater conveyance system to serve areas 2A, 2B and 2B4" is consented under the Network Discharge Consent 34887 (NDC). The Awakeri Wetlands is the proposed infrastructure for servicing these areas and the Wallace area to the north.

2.6 Draft Central Papakura ICMP

The Draft Central Papakura Integrated Catchment Management Plan ICMP (PDC, 2007) documents the overarching stormwater conveyance approach for the catchment. The ICMP outlines a potential alignment for the Awakeri Wetlands.

The ICMP alignment is similar to the main channel alignment proposed in this report; with the main difference at the eastern end where the ICMP alignment splits into two channels. The ICMP channel excludes the proposed Northern Branch channel and services part of the 2A catchment using a piped stormwater system.

2.7 Concept design

The concept design was developed by GHD in July 2014 as part of the Notice of Requirement process and is described in the Takanini Stormwater Conveyance Channel Infrastructure Report (GHD, 2014). The Concept Design concluded that a conveyance channel was the most beneficial and recommended stormwater solution for the catchment, compared to a piped solution, or piped / pond hybrid system.

Refer to the Plan amendment 48 – Takanini stormwater conveyance corridor (Auckland Council, 2014) for more detail.

2.8 Scheme design

The scheme design was developed by GHD in July 2016 as part of the Resource Consent process and is described in the Takanini Stormwater Conveyance Channel Stormwater Report (GHD, 2016a). The Scheme Design outlined the key features, effects and mitigation of effects for the TSCC.

3. Existing Environment

The following section provides a brief description of known future works and a general description of how the existing environment will be affected by the proposed works.

3.1 Site setting

3.1.1 Land use

The majority of land within the conveyance catchment has historically been pastoral with large lifestyle blocks and a relatively low intensive nature. Recently, large areas of land have begun developing into residential areas to a high density.

Consents have already been obtained for development of sites within the catchment, subject to temporary stormwater solutions, on the proviso that once the Awakeri Wetlands are built, these sites will be connected to it. These include:

- The Grove at 61 Grove Road (Equinox Group).
- 54, 64, 74 and 94 Cosgrave Road.
- Kauri Flats School at 181 and 191 Walters Road.
- 201 Walters Road.
- Twin Parks Estate at 989 to 999 Papakura-Clevedon Road (Cappella Papakura Developments Ltd).
- Papakura Residential at 965 Old Wairoa Road and 965 to 973 Papakura-Clevedon Road (Cabra Investments Ltd).
- Part of the Montgomery development at 881 to 899 Papakura-Clevedon Road.

These sites are at different stages of development, from concept stage to bulk earthworks. Houses have been established at 61 Grove Road (The Grove) and at the Cappella development (Twin Parks Estate). Additional houses are currently still under construction within both of these developments.

The developments above are shown on drawing 51-33411-C006 (see Appendix C).

Other properties that have expressed their intention to develop within the next 12 months include:

- 169 Walters Road.
- 122 Cosgrave Road.
- 130 Cosgrave Road.
- 99 Grove Road.

3.1.2 Topography

The catchment is essentially flat in nature; except for the eastern portion where it falls from approximately 67 m over a distance of 0.8 km to 26 m; with an average slope of about 3 %.

From here; the catchment falls from an RL of 26 m over 1.7 km to an RL of 22 m at Grove Road. This provides an average slope for the flat portion of about 0.24 %.

3.1.3 Existing stormwater and features

There is no formalised drainage across the catchment with small dissected channels and farm drains connecting to roadside table drains. The existing natural streams in the region are very short and have little to nil baseflow during the summer months (PDC, 2007).

The roadside table drains along Cosgrave Road and Walters Road collect overland flow and have limited conveyance capability. These roadside drains are deeply incised, up to about 2 to 2.5 m in depth. Generally, the roadside drains store water and discharge to ground soakage when water tables are low over summer. Figure 4 shows the table drain on Cosgrave Road.

Figure 4 Cosgrave Road table drain

To the west of Grove Road and south of Fernaig Street and Pukeroa Place stormwater is reticulated. Most of these flows are directed to the wetland located in McLennan Park. This wetland (the McLennan wetland) is designed to attenuate and treat flows from the Old Wairoa Road catchment before discharge via Gills Pond to the Pahurehure Inlet and is discussed further in Section 3.2.

Figure 5 McLennan wetland

3.1.4 Existing flooding

The vast majority of the Awakeri Wetlands catchment area and a portion of the Takanini South catchment to the north-west are predicted to be inundated in a 1% AEP storm event to a depth of 300 to 500 mm. Extensive ponding has been observed during rainfall events, particularly in winter when the groundwater table is high. This is primarily a result of ineffective stormwater drainage, but is also due to flat topography, high groundwater tables and limited soakage capacity of the peat fields.

3.1.5 Geological setting and extent of peat

The geotechnical investigation conducted by GHD in 2016 confirms that the ground beneath the area is predominantly made up of peats, organic silts and sands. Further details of the peat are discussed in the Takanini Stormwater Conveyance Channel Geotechnical Investigations Report (GHD, 2016c).

The organic peat typically extends to a depth of 20 m below ground level and is extensive throughout the entire Stage 1A of the Awakeri Wetlands. An approximate extent of the peat is shown in Figure 6 below.

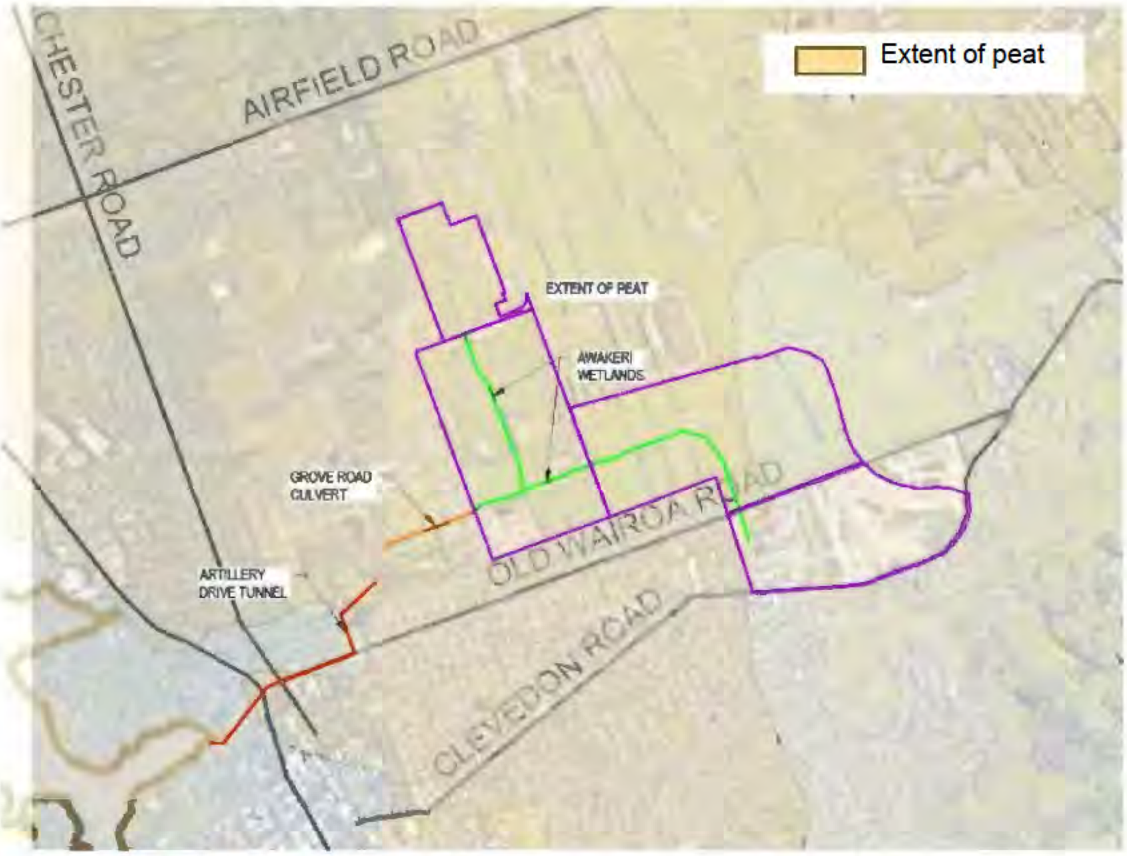


Figure 6 Extent of peat (PDP, 2006)

The Takanini area is known to be underlain by a significant peat aquifer.

Geological units described generally as peat in this area consist of a material that ranges from humic, fibrous peat to amorphous organic clay and are generally horizontally stratified. This is further discussed in the Takanini Stormwater Conveyance Channel Geotechnical Investigations Report (GHD, 2016c).

3.1.6 Surface water and discharge to ground

The majority of stormwater in the undeveloped areas of the Awakeri Wetlands catchment and surrounding rural areas enters the ground via direct infiltration. Impervious surfaces in areas designated as rural discharge to ground soakage or open channels. Soakage test results indicate some of the highest soakage rates were found within peat areas. However, sample testing indicated the peat also had low permeability.

The stormwater from developed areas are generally conveyed via pipe networks or swales and will generally be piped into the Awakeri Wetlands at the weir locations.

3.1.7 Groundwater

Groundwater level monitoring data has been collected over the past 33 months (depending on location) to establish seasonal variation in groundwater levels. This data is included in Takanini Stormwater Conveyance Channel Geotechnical Investigation Report (GHD, 2016c) and in the Takanini Stormwater Conveyance Channel Stage 1 Groundwater and Settlement Monitoring and Contingency Plan (GSMCP) (GHD, 2017). The latest data can be provided on request.

Depths to groundwater in the shallow unconfined aquifer system range from 0.0 m in the eastern part of the subject site to 1.0 m to 1.5 m near Cosgrove Road and are >1.5 m depth in the south western part of the site near Grove Road.

3.1.8 Existing utilities

Existing services are outlined in Drawing 51-33411-C008 which include:

Stormwater

As already noted, a large portion of the Awakeri Wetlands catchment area is not serviced by a formal stormwater network. The developed and developing areas of the catchment typically include stormwater attenuation which discharge at predevelopment levels to the roadside table drains or existing stormwater networks at the catchment extents.

Water

Watercare Services Limited (WSL) through Veolia Water provides reticulated potable water to residential properties within the Awakeri Wetlands catchment area along Cosgrave Road, Walters Road and Grove Road. The following water assets are known to be within the area:

- 1200 mm diameter CLS water pipe along the west side of Cosgrave Road (Waikato No.1 Trunk Watermain discussed further below)
- 100 mm diameter PVC pipe along the west side of Cosgrave Road.
- 100 mm diameter AC abandoned water pipe along the east side of Cosgrave Road.
- 250 mm diameter PE pipe along the east side of Grove Road.
- 100 mm diameter PVC pipe along the west side of Grove Road.
- 175 mm diameter CLS and 100 mm diameter AC abandoned water pipes along the east side of Grove Road.
- 100 mm diameter AC pipe along the south side of Walters Road.
- PE pipes within the development at 61 Grove Road (along Saddleback Crescent, Bellbird Street and Stitchbird Crescent).

New water pipes are proposed along Walters Road by developers and these are discussed further in the Walters Road culvert Detailed Design Report (GHD,2017).

Waikato No. 1 trunk watermain

A 1,200 mm diameter watermain owned by Watercare Services Ltd runs along the western side of Cosgrave Road and has an estimated depth to invert varying between approximately 2.5 m to 3.0 m. This is a strategic main, supplying the bulk of potable water to Auckland.

There is a fibre optic cable above the watermain for communication purposes with a direct link to the Waikato Treatment Plant.

Wastewater

The Takanini Sewer which runs through Bruce Pulman Park is the proposed wastewater discharge location for developments within the Awakeri Wetlands catchment. The closest current (2017) connection point to the Awakeri Wetlands is at 169 Walters Road which is at a 525 RCRRJ pipe and manhole.

Currently, there are two known rising mains in the area which discharge to the Takanini Sewer at 169 Walters Road. One is along Grove Road from the 61 Grove Road development, and the other along Cosgrave Road from the Cappella and Cabra developments. These rising mains discharge to the north at the watercare trunk line near Walters Road, which is the proposed connection for future wastewater.

There is no existing wastewater servicing for the undeveloped areas within the catchment. As development of the catchment commences, wastewater servicing is being constructed by developers. The wastewater is owned and operated by Veolia.

The residential areas adjacent to the catchment such as Fernaig Street and Corkill Place are reticulated with wastewater and water services. Refer to Drawing 51-33411-C008.

Gas

A 356 OD PE Vector high pressure gas transmission pipeline traverses through areas 2B and 2B4 with an average depth of cover of 900 mm and has a 12 m wide designation. The gas main travels in a north-south direction between Settlement Road and Hamlin Road, as shown in Drawing 51-33411-C008.

Power

Historically, power has been transmitted in overhead lines. Some new developments such as at Old Wairoa Road are installing underground power systems. Hence there is a mixture of overhead and underground power throughout the area.

There are no significant known high voltage feeds in this area.

Telecom and Vodafone

There are existing Telecom and Vodafone services along Cosgrave Road, Grove Road and the local roads adjacent to the Awakeri Wetlands catchment.

3.2 McLennan wetland

Existing and consented wetland

The McLennan wetland was constructed in 2002, this wetland already receives stormwater from the Housing New Zealand development and Papakura Military Camp through to Bruce Pulman Park in the north; and Willis Road and Clevedon Road to the south. The wetland provides attenuation and treatment for of the Old Wairoa Road catchment as per Figure 7.

Figure 7 McLennan wetland sub-catchment map (Old Wairoa Road CMP Variations, 2009)

The wetland currently has an embankment top level of RL 16.00 m and an emergency spillway level of RL 15.1 m.

Network Discharge Consent 37205, 33738 and 33538 specify that prior to any further development commencing in areas 2A, 2B or 2B4 (ie. The construction of the Awakeri Wetlands) the following works will be undertaken:

- Increase of embankment level from RL 16.0 m to RL 16.2 m.
- Increase of spillway level from RL 15.1 m to RL 15.4 m.

3.3 Capacity downstream

The capacity of the downstream network has been considered and discussed in the *Takanini Stormwater Conveyance Channel Stormwater Report (GHD, 2016a).*

3.4 Water quality

For the pre-developed scenario, during the Water Quality rainfall event (1/3 50% AEP event), rainfall onto the Awakeri Wetlands catchment is expected to soak through the soil, with little runoff being produced.

For the developed areas adjacent to the proposed Awakeri Wetlands catchment; water quality treatment is provided by the McLennan Wetland discussed in Section 3.2. The efficiency of the upper McLennan Wetland has been estimated at 72% (PDC, 2004).

There is another stormwater treatment pond at the downstream end of the Old Wairoa Road catchment; the Gills Road Pond. The Gills Road Pond provides stormwater treatment for the Old Wairoa Road catchment prior to discharging to the Pahurehure Inlet.

There is a requirement for developments in the area to discharge stormwater into soakage devices, which will mitigate some contaminants from entering the downstream receiving environment during small rainfall events (<15mm).

4. Methodology and Design Parameters

4.1 Design requirements

The Awakeri Wetlands has been designed to accommodate the following elements:

1. Convey the 1% AEP flows that are conveyed to the designation boundary wholly within the channel extent and subsequently within the designation. The design does not include earthworks outside the designation within private developments that would be required to get overland flow from the adjacent land into the channel. The design assumes that these works will be undertaken by the developers in accordance with their own designs.
2. Provide a permanent water level to support the development of a natural aquatic ecosystem.
3. Provide low flow operation levels of the channel at a suitable depth to allow piped flow from adjacent catchment areas to flow with a free discharge at low flows (not drowned) where practical.
4. Provide suitable 1% AEP flow levels in the channel to allow properties at the catchment extents to design overland flow paths with sufficient capacity and grade to discharge to the channel.
5. Provide a safe environment for the community and for those staff undertaking the operation and maintenance of the channel.
6. Provide for additional amenity value within the designated area where possible.
7. Make provision of the development of footpaths and cycleways.

4.1.1 Design standards

The design requirements for relevant components of the Awakeri Wetlands are discussed in this section. The relevant design guides and reference material that have been referred in this report include, but are not limited to:

Wetland / channel and hydraulic structures (culverts, pipes, weirs) design

- TP108 (Auckland Regional Council, 1999)
- TP10 (Auckland Regional Council, 2003)
- Queensland Urban Drainage Manual (Department of Energy and Water Supply, 2013)
- Auckland Council Stormwater Code of Practice (Auckland Council, 2015)
- Hydraulics of Precast Concrete Conduits (Concrete Pipe Association of Australasia, 1997)

Shared paths and boardwalks

- Auckland Transport Code of Practice (Auckland Transport, 2017)
- Tracks and Outdoor Visitor Structures SNZ HB 8630:2004 (SNZ, 2004)

Design criteria for the Awakeri Wetlands has been summarised in Table 1 below, based generally on the above publications.

Table 1 Design requirements and considerations

Constraint		Design principle	Discussion
Hydrology	Conveyance	Allow for conveying up to the 1% AEP flow from the catchment.	Section 4.5
Hydrology	Soakage / first 15mm rainfall in catchment goes to soakage	Low flow water level to accommodate reduced recharge from small rainfall events.	Section 4.4
Hydrology	Climate change	Hydrological parameters to allow for climate change to 2090 values.	Section 4.4
Hydrology	Scour	Control scour of channel bed and banks to acceptable levels.	Section 6
Hydrogeology	Peat soils (permeability)	Consider high permeability of peat soils.	Sections 4.3.1 and 7
Hydrogeology	Groundwater levels	Consider high groundwater levels during construction, seepage and floatation risk for structures.	Section 10
Hydrogeology	Groundwater levels (drawdown)	Risk of lowering groundwater levels and inducing settlement of adjacent land. Consider effect of works on groundwater levels and settlement risk.	Section 4.3.1
Hydrogeology	Groundwater/soil chemistry	Low pH of groundwater/soil and potential sulphates in groundwater/soil. Consider low pH and sulphate impact on structures and materials.	Section 4.3
Hydrogeology	Groundwater/soil chemistry (drawdown)	Consider impact of groundwater drawdown on chemistry of groundwater and soils. Risk of lowering pH further and Acid Sulphate Soils.	Section 4.3
Geotechnical	Peat soils (strength)	Peat to ~20 m depth. Consider impact of low strength peat in terms of erodibility, settlement, bearing and difficulties with piling.	Sections 5,6,7 and 10
Geotechnical	Slope stability	Consider low strength of soils and wetland batters. Maximum slope of batters typically 4:1 (or 2:1 where low height, underwater and stabilised with aggregate).	Section 5
Development	Primary drainage	Allow for stormwater connections into the channel at levels which allows the extent of the catchment to be drained by gravity.	Section 8
Development	Secondary drainage / overland flow	Allow for overland flow connections into the channel at levels which allows the extent of the catchment to be drained by gravity.	Section 8
Ecological	Temperature	Consider temperature of low flow water during consideration of depth, width, shading, materials and operation.	Section 5
Ecological	Fish passage	Allow for fish passage at structures that may cause barrier such as culverts and weirs.	Sections 7 and 10

	Evaporation	Consider evaporation effects and lowering of water level / drying out of wetland benches.	Section 5.1.5
	Habitat	Consider habitat of wetlands including planting, water quality, flow velocities, materials.	Section 5
Planning	Designation	All permanent works to fit within the designation boundary or land owned by Auckland Council. No earthworks outside the designation, unless agreed with landowners.	Section 5
	Resource consent	Works to be in accordance with resource consent conditions	Section 5
Public space	Public access	Shared paths provided along the wetlands corridor. Boardwalks to provide access between both sides of the wetlands. Paths and boardwalks to be designed in accordance with ATCOP (Auckland Transport, 2017).	Sections 11 and 12
Operational	Debris	Blockage risk of culverts to consider impact on flooding and conveyance of flow.	Section 10
	Sediment deposition	Consider build-up of sediment in the wetland and how this will be inspected and cleaned out.	Section 6.7
	Maintenance	Access to be provided for inspections and maintenance of weirs, culverts, boardwalks, paths and wetland.	Sections 11 and 12
Cultural	Design philosophy	Involve Mana Whenua in the design process and incorporate iwi philosophy in the stormwater design where possible.	At hui's and incorporated throughout design
Safety	Public safety	Safety in design for areas that could be accessed by public: low flow water body, paths, culverts and boardwalks.	Section 13
	Operational safety	Safety in design for areas accessed by operations staff: low flow water body, weirs, paths, culverts and boardwalks.	Section 13
	Construction safety	Safety in design for the construction of the wetlands and associated structures.	Section 13

4.2 Geotechnical background

Geotechnical parameters have been derived from the investigations carried out as part of the Awakeri Wetlands Scheme Design.

Table 2 from the geotechnical investigation describes the geotechnical parameters for the site. This includes key soil parameters. Additional information in relation to the depth of peat, groundwater levels and geological logs is available in the Takanini Stormwater Conveyance Channel Geotechnical Investigations Report – Technical Report C (GHD, 2016c).

Table 2 Range of geotechnical parameters

Geological Unit	Bulk Unit Weight (kN/m³)	Undrained Shear Strength S_u (kPa)	Effective Strength Parameters		Young Modulus (MPa)		Poisson's Ratio		Estimated Permeability k (m/sec)(Note 1)
			Effective Cohesion c' (kPa)	Effective Friction Angle Φ' (degrees)	Undrained (E_u)	Drained (E')	Undrained v_u	Drained v'	
Cohesive Fill	14	6 - 12	3 - 6	26 - 30	0.6	0.4	0.5	-	10^{-4} to 10^{-5}
Puketoka Formation Organic soils / Peat	11	0 - 20	0 - 5	25 - 36	0.5 - 0.8	0.4 - 0.6	0.5	0.1-0.15	$k_v = 10^{-4}$ to 10^{-6} $k_h = 10^{-5}$ to 10^{-7}
Puketoka Formation Alluvial clays/silts	16 - 20	30 - 60	5 - 10	27 - 34	4 - 10	2 - 8	0.5	0.2 - 0.3	$k_v = 10^{-4}$ to 10^{-6} $k_h = 10^{-5}$ to 10^{-7}
Kaawa Formation Sands	18 - 22	-	-	30 - 34	-	40 - 90	0.5	0.2 - 0.4	$k_v = 10^{-3}$ to 10^{-6} $k_h = 10^{-4}$ to 10^{-7}
ECBF Residual Soils	17 - 20	100 - 230	10 - 13	26 - 34	-	45 - 75	0.5	0.2 – 0.3	$k_v = 10^{-4}$ to 10^{-6} $k_h = 10^{-5}$ to 10^{-7}
ECBF Rock (Sandstone)	20 - 22		20	34 - 36	-	90 - 150	0.5	0.25	$k_v = 10^{-3}$ to 10^{-5}

4.3 Hydrogeology and settlement

The hydrogeology of the area and effect of the Awakeri Wetlands are discussed in the Takanini Stormwater Conveyance Channel Hydrogeology Assessment of Effects (GHD, 2016d).

4.3.1 Groundwater dewatering

The Awakeri Wetlands are constructed below the seasonal low groundwater level in some areas. This has the potential to cause dewatering of the groundwater within adjacent land. Dewatering of the adjacent peat can cause ground settlement and potentially damage adjacent structures.

Slurry wall

To mitigate potential dewatering, a slurry wall was installed at critical locations during the enabling works for the Awakeri Wetlands. The design of the slurry wall is further discussed in the Takanini Stormwater Conveyance Channel Groundwater and Ground Settlement Effects Report (GHD, 2016e) and the extents of the slurry walls are shown in Figure 8 below.

The slurry wall is 7 m deep and a minimum of 600 mm wide, consisting of a cement / bentonite mix. The slurry wall was installed by excavating a trench under slurry and replacing the peat material with the cement-bentonite mix.

The top 1 m of the slurry wall will be excavated through when forming the Awakeri Wetlands final contours. This is considered acceptable and part of the design. The top 150 mm of the slurry wall will be covered with topsoil to protect the top of the wall from damage.

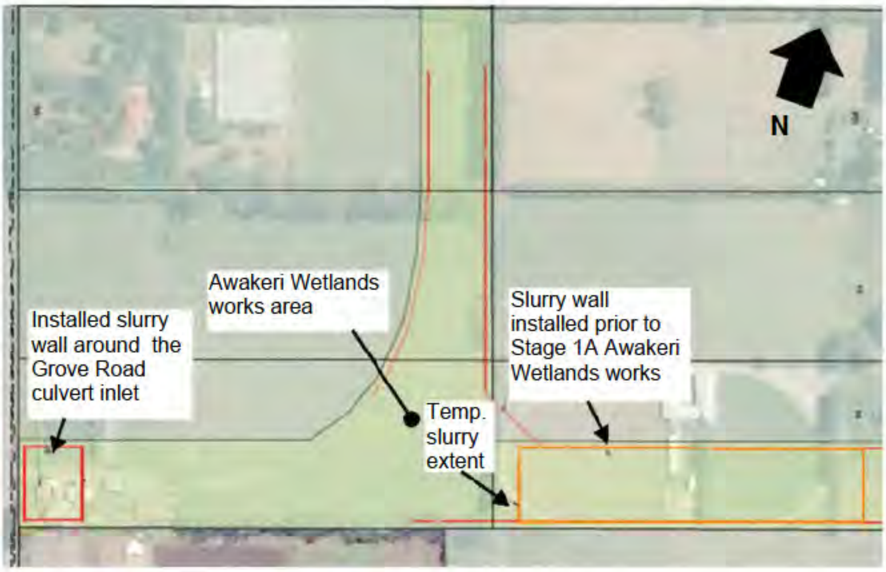


Figure 8 Slurry wall extent

4.3.2 Settlement effects

As discussed above, the Awakeri Wetlands has the potential to cause settlement of adjacent land. This can have some positive effects prior to development of land, but can also have adverse effects to existing housing and infrastructure.

This risk has been considered in the Takanini Stormwater Conveyance Channel Groundwater and Ground Settlement Effects Report (GHD, 2016e) as part of the resource consent assessment.

Groundwater and settlement will be monitored in accordance with the GSMCP (GHD, 2017) during construction.

4.3.3 Downstream water chemistry

The downstream receiving environment is the McLennan wetland. The McLennan wetland environment is an acidic environment due to the nature of the area. The peat soils generate low pH groundwater and hence the pH of the water in the wetland has been measured as low as 5.

The flow from the channel has the potential to have a low pH due to the inflow of groundwater and the nature of the soils in the area. The risk of causing adverse effects on the downstream environment is considered to be low given than the downstream environment is currently subject to low pH.

An Acid Sulphate Soils Management Plan (GHD, 2017n) has been prepared to address this risk.

4.4 Hydrological parameters

The following section outlines the hydrological parameters assumed for the catchment. TP108 (Auckland Regional Council, 1999) is the general approach used for the hydrological assessment. The TP108 methodology has been used in the modelling software MIKE11 for calculation of flows and channel flow.

4.4.1 Prescribed catchment

The proposed catchment area outlines the area that the stormwater conveyance channel can service for the 1% AEP event as described and outlined in Section 2.2. This is controlled by the channel depth, capacity and the topography of the catchment. Refer to Figure 9 for an outline of the catchment.

4.4.2 Design rainfall and climate change

24 hour rainfall

For this project the design rainfall has been derived from Auckland Council's TP108 (Auckland Regional Council, 1999) with a 24-hour storm profile. The 24-hour total rainfall for each of the design storms without climate change allowances are presented in Table 3 below:

Table 3 Design rainfall

Rainfall event	24 hr rainfall (mm)
1% AEP	220
2% AEP	200
5% AEP	165
10% AEP	140
20% AEP	110
50% AEP	70

Climate change

The adopted climate change scenario for this project is to year 2090, as per the AC Stormwater COP (Auckland Council, 2015). The MfE Guidance for local government (New Zealand Climate Change Office, 2008) recommends a warming value of 2.1°C for the 2090 A1B mid-range scenario.

Based upon a 24-hour storm, the effect on rainfall per degree rise is set out in Table 4 (New Zealand Climate Change Office, 2008).

Table 4 Adopted climate change scenarios

Rainfall event	Percentage increase in rainfall
1% AEP	8.0 % increase per 1°C rise
2% AEP	8.0 % increase per 1°C rise
5% AEP	7.2 % increase per 1°C rise
10% AEP	6.3 % increase per 1°C rise
20% AEP	5.4 % increase per 1°C rise
50% AEP	4.3 % increase per 1°C rise

Design rainfall values

The adopted 24-hour design rainfall with climate change to 2090 used in the design is as shown in Table 5.

Table 5 Adopted design rainfall

Rainfall event	24 hr rainfall (not including climate change) (mm)	24 hr design rainfall including climate change (mm)
1% AEP	220	256
2% AEP	200	234
5% AEP	165	190
10% AEP	140	158
20% AEP	110	122
50% AEP	70	76

4.4.3 Modelling and hydrological parameters

Impervious areas

The Maximum Probable Development (MPD) impervious areas for the catchment have been assumed using the Draft Papakura Central ICMP as a base. The impervious areas in the ICMP are generally equal to or greater than the maximum allowable in the Proposed Auckland Unitary Plan zoning. The Proposed Auckland Unitary Plan allows for 60% maximum impervious area in catchment 2A and 2B. Area 2B4 is currently zoned Future Urban in the Proposed Auckland Unitary Plan, however it is expected that this land will be rezoned in the near future.

The impervious areas from the ICMP have been adjusted to account for additional impervious area from the Mill Road Block, as discussed below.

The Mill Road Corridor is proposed to run through areas 2B and 2B4, as shown in Figure 9. The alignment and size of the Mill Road Corridor has not been confirmed; however, for the purpose of this report, a possible route has been assumed which allows for a corridor approximately 1 km long, 20 m wide and 100% impervious. This additional impervious area will slightly increase the maximum impervious area (MPD) scenario as per the values in Figure 9. The three sub-catchments that Mill Road runs through will have impervious areas increased from 60% to 63%. This has been allowed for in our design flow for the Awakeri Wetlands.

Figure 9 outlines the impervious area assumptions used for calculation of design flows for the Awakeri Wetlands.

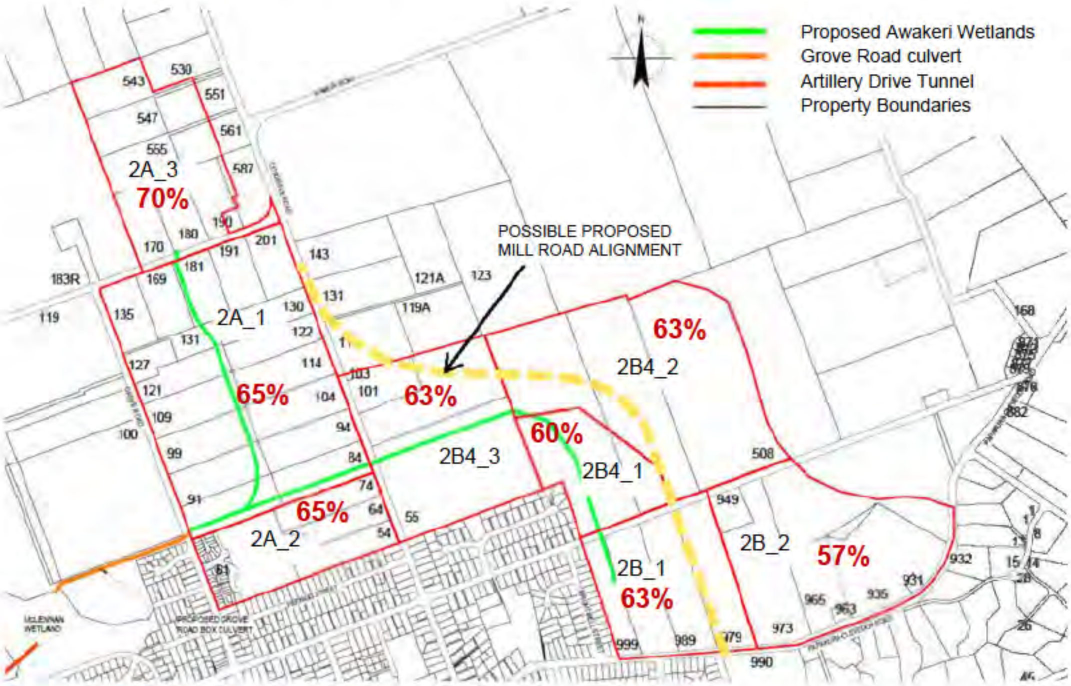


Figure 9 Impervious areas

Design curve numbers

An SCS Curve Number (CN) of 74 has been used for peat soils for the predevelopment scenario as per the Papakura ICMP, as per TP108. The post-developed scenario also uses a CN of 74 for pervious areas based on likely imported fill characteristics or existing peat soils as per above.

This aligns with the curve numbers being used by developers in the catchment.

Geotechnical observations indicate that the top crust of the soil can harden when exposed to oxygen and sheds water. This gives further support to using a curve number of 74.

An SCS Curve Number (CN) of 98 has been used for impervious areas as per the Papakura ICMP, this aligns with TP108 and other industry standards.

Channelisation factor

Channelisation factors as per Table 6 below were used.

Table 6 Channelisation factors

Surface	Factor	
Impervious areas	0.8	This is considered appropriate due to the fact that developers are required to implement recharge pits which will increase the time of concentration as water needs to pass through the granular material before discharging through a pipe. In addition, the catchment is very flat and overland flow to the channel for events greater than the 10% AEP event does not follow direct routes to the channel. Overland flow is expected to pass through “green corridors” in some areas.
Pervious areas	1.0	This is considered appropriate as the pervious areas in the catchment are expected to sheet flow onto the impervious areas once saturated with no formalised drainage pathways. In small events, water will likely soak into the ground before reaching the impervious areas. In larger events, the water will be slowed by grass / vegetation before sheet flowing onto the impervious areas.

The channelisation factors in Table 6 were used for the 50%, 10% and 1% AEP events. A sensitivity check was carried out by changing the channelisation factor for impervious areas to 0.6 for the 10% AEP model. This resulted in an increase in flow of less than 5% in the 10% AEP. This is expected to have a negligible effect on the water level in the channel, and therefore using a channelisation factor of 0.8 for impervious areas for all storm events has been considered reasonable; given the flat catchment, possible use of open stormwater systems for some areas of the catchment and recharge pits / soakage devices.

Time of concentration

The values for flow length and time of peak flow have been derived from calculations based on the TP108 methodology. The slopes and catchment lengths consider the developed slopes of the catchment draining to the proposed channel and therefore in some cases are slightly steeper than the existing gradient. These consider:

- Channel flow in the main channel.
- Pervious and impervious flow over the reduced length.

Depression storage

The significant area of flat land within the catchment area currently has the ability to store significant volumes of runoff.

Post development with the Awakeri Wetlands in place, the flow path lengths and depression storage will be significantly reduced due to filling and grading of the land towards the channel. GHD has used reduced channel lengths to reflect the geometric layout of the proposed conveyance channel layout within the catchment.

For impervious and pervious areas; depression storage of 0 and 5 mm respectively, has been used. These are the recommended values in Auckland Council’s TP108 (Auckland Regional Council, 1999).

Recharge pits

The development controls have a requirement for storage and soakage to ground for the first 15 mm of rainfall through the installation of recharge pits. We consider that the soakage will have negligible effect on the peak flows from larger events such as the 50%, 10% and 1% AEP events (which have been modelled). Therefore the 15mm soakage criteria has not been explicitly considered in the model, however, some representation is present in the consideration of channelisation factors. The presence of soakage devices has only been considered in the model for selection of channelisation factors to account for drainage pathways.

Attenuation

Generally there is limited attenuation in the catchment, as the proposed Awakeri Wetlands will convey post-development flows.

The exception is for the sub-catchment which is currently under development by Cabra Investments Ltd (refer to Figure 10). A permanent stormwater pond has been consented to attenuate flows from the Cabra development up to the 1% AEP event to pre-development levels.

The effect of the pond has been flow routed by GHD and incorporated into the hydraulic model. The peak discharge from the pond in the 1% AEP event has been modelled as 3.6 m^3/s.

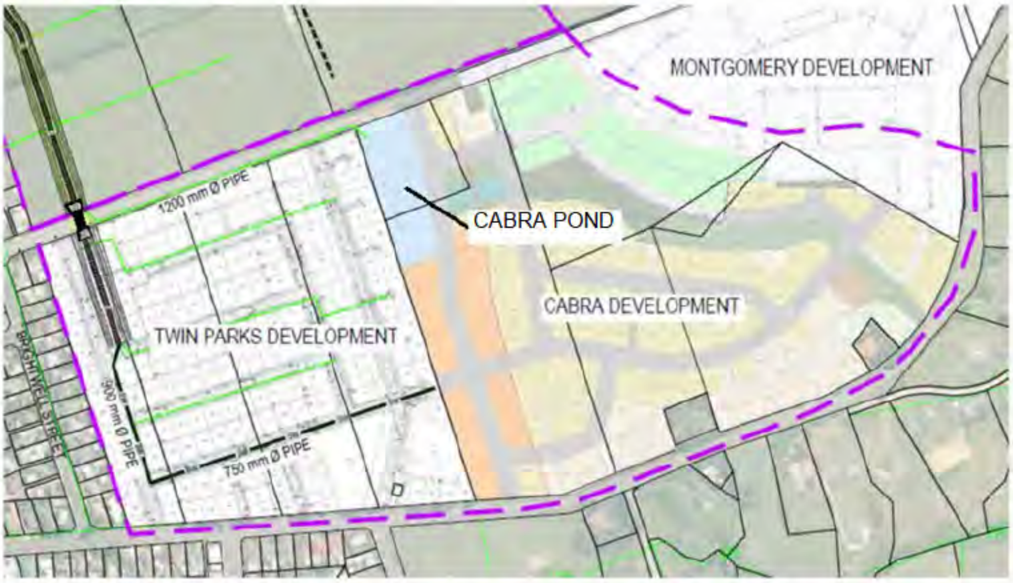


Figure 10 Cabra development and attenuation pond

4.5 Design flows

4.5.1 GHD 1D / 2D coupled model

The catchment and scheme design channel have been modelled in a 1D / 2D coupled model to determine peak flow in the catchment and flood levels within the catchment and channel. The channels were modelled using MIKE11 (1D model) and the surface has been modelled in MIKE 21 (2D model).

The sub-catchment runoff was computed by the model; using the parameters outlined in Section 4.4.

The model predicts a peak flow at the downstream end of the conveyance channel of 37.9 m^3/s for the 1% AEP storm event.

Sub-catchment loading

The sub-catchments were loaded into the Awakeri Wetlands in the GHD model as per Table 7 below (refer to Figure 9 for sub-catchment boundaries).

Table 7 Loading of sub-catchments

Sub - catchment	Loading	Explanation	Impervious area
2A_1	Distributed load along the northern branch channel.	Represents multiple incoming pipes and overland flow paths as per the expected development.	65%
2A_2	Distributed load along the main channel.	Represents multiple incoming pipes and overland flow paths as per the expected development.	65%
2A_3	Point load at top of northern branch channel.	The Mill Road Block is expected to discharge to the top of the branch channel.	70%
2B4_1	Distributed load along the channel.	Represents multiple incoming pipes and overland flow paths as per the expected development.	60%
2B4_2	Point load at CH 950.	Represents an incoming pipe or open channel connection. This sub-catchment is large and it is expected that the developer will need to construct an open channel to service their development which will connect into the Awakeri Wetlands at CH 950.	63%
2B4_3	Distributed load along the main channel.	Represents multiple incoming pipes and overland flow paths as per the expected development.	63%
2B_2	Point load downstream of the Old Wairoa Road Culvert.	Represents the proposed connection location of the Cabra Pond discharge pipe.	57%
2B_1	Point load at top (upstream end) of the main channel.	Assumes the development discharge to the top of the channel via a pipe or overland flowpath.	63%

The *modelled flow and hydraulic grade line are plotted on the channel longsections on drawings 51-33411-C121-C129.*

MIKE11 model outputs

Drawings 51-33411-C111-C117 shows the modelled Awakeri Wetlands and the chainage along the channel. Refer to Table 8 for MIKE11 model outputs. Further model outputs are included in Appendix A.

Table 8 MIKE11 model outputs - design peak flows

	MIKE11 modelled peak flow (m^3/s)		
Chainage (m)	50% AEP	10% AEP	1% AEP
Main Channel			
0	10.2	25.2	40.2
100	10.1	25.0	39.9
150	10.0	24.9	39.7
200	6.0	15.4	24.5
300	5.9	15.2	24.0
400	5.8	14.8	23.5

	MIKE11 modelled peak flow (m^3/s)		
Chainage (m)	50% AEP	10% AEP	1% AEP
500	5.7	14.6	23
600	5.6	14.3	22.6
700	5.2	13.5	21.3
800	4.9	12.6	19.9
900	4.5	11.6	18.5
950	4.3	11.2	18.0
1000	1.9	5.8	9.6
1100	1.8	5.6	9.2
1200	1.6	5.3	8.7
1300	1.5	5.1	8.2
1400	1.4	5.0	7.9
1500	1.4	3.2	4.7
Northern Channel			
60	4.1	9.4	15.0
200	3.7	8.5	14.4
300	3.0	7.0	11.5
400	2.4	5.5	9.2
500	2.0	4.4	7.4
600	1.6	3.6	6.0
700	1.2	2.8	4.6

4.5.1 Validation of flows

A HEC-HMS model was prepared by GHD to compare and confirm the predicted flows from the MIKE11 modelling. The peak flow predicted by the HEC-HMS model in the Awakeri Wetlands at Grove Road is 39.1 m^3/s.

The channel was represented in HEC-HMS as a series of reaches linked together with junctions. Lag time for each reach was based on expected flow velocities along the length of each reach. Velocities and corresponding lag times for each reach have been assumed as per Table 9.

Table 9 Lag times and flow velocity

Reach	Velocity (m/s)	Lag time (min)
Main channel		
CH 0 - 160	1.50	1.8
CH 160 - 550	1.00	6.5
CH 550 - 950	1.00	6.7
CH 950 - 1400	0.80	9.4
CH 1400 - 1540	0.50	4.7
Northern branch		
CH 0 - 300	0.75	6.7
CH 300 - 550	0.90	4.6

The catchment was represented by a series of sub-catchments which were split into separate impervious and pervious catchments, with the catchment parameters as per Section 4.4. Each sub-catchment was loaded into the channel at junction points. This is expected to give a good representation of the flow at each junction. However between junctions the flow rate could be deduced from interpolation.

The Cabra pond has been represented in HEC-HMS by a reservoir linked to an Elevation-Area Function and an Elevation-Discharge Function which was derived from the pond routing carried out by GHD.

The flow predicted by the HEC-HMS model matches the MIKE11 modelling and confirms that the peak flow predictions are within acceptable levels of accuracy suitable for the purpose of this design.

Refer to Appendix B for HEC-HMS model outputs.

4.6 Sensitivity analysis

A sensitivity analysis was carried out on the impervious area assumptions for the catchment. The 1% AEP model was run using a base of 70% impervious area for each sub-catchment, adjusted further as above for the Mill Road Corridor (+3% for the three sub-catchments that Mill Road runs through). This resulted in a less than 3% increase in flow for the 1% AEP event which is expected to have a negligible effect on the water level in the channel.

4.7 Hydraulic modelling

The Awakeri Wetlands Scheme design was modelled in MIKE11 to determine the hydraulic grade line in the channel for the 50%, 10% and 1% AEP events. The model was checked using spreadsheet calculations based on Bernoulli's energy principle and Manning's flow equation (using Flowmaster).

Channel cross sections were input into the model at 20 m spacing. Channel cross sections, roughness, culverts and catchment parameters were used to match the values described in Section 4.4 and of this report.

The model confirms that the channel design is adequate for conveying the 1% AEP event with adequate freeboard. In addition, the hydraulic grade line is maintained at a low enough level to provide drainage of the surrounding land developments; this is further discussed in Section 8. Refer to Drawing 51-33411-C121-C127.

Refer to Appendix A for the MIKE11 model outputs.

4.7.1 Channel hydraulic parameters

Manning's numbers

The adopted Manning's numbers for the Awakeri Wetlands align with the recommended values in Christchurch City Council's Waterways, Wetlands and Drainage Guide (Christchurch City Council, 2003). The above publication was used as it contains Manning's numbers for stream surfaces that are similar to the proposed vegetation and channel profiles of the proposed Awakeri Wetlands.

Adopted Manning's n numbers

The following Manning's numbers have been used for the hydraulic analysis. These have been selected assuming:

1. The low flow channel is maintained to keep clear of obstructions and prevent excessive weed growth. n = 0.030
2. The wetland plants are lay flat species and will flatten during flood events. n = 0.045

3. The flax and native grasses on the channel bank are maintained to keep clear of excessive weeds. The plant species are assumed as a mixture of those that can flatten during flood events with some heavier shrubs less than 1 m tall. n = 0.060

4. The grass is maintained at a short length and specimen trees are scattered throughout the floodplain. n = 0.045

Table 10 Manning's numbers for conveyance channel design

Section	Surface Cover	Manning's number (n)
Low flow channel	Naturalised channel with pools and slight channel meander	0.030
Wetland bench	Wetland grasses	0.045
Channel bank	Flax and native grasses (<1 m tall)	0.060
Floodplain	Mowed grass with footpath and specimen trees	0.045

4.7.2 Culverts

There are two culverts within Stage 1 of the Awakeri Wetlands.

Losses through the culverts were checked using Bernoulli's Energy equation and the losses were determined to be sufficiently low to not significantly impact the hydraulic grade line of the channel. This is further discussed and detailed under Section 10.

5. Channel Design

5.1 Design basis

The design of the Awakeri Wetlands has been driven by a number of factors. These are recorded below along with a brief commentary of the effects of each on other aspects of the design.

- The design philosophy in having weirs along the channel length is to maintain low flow water as high as is practical in order to limit the groundwater drawdown and provide for the development of aquatic habitats.
- A second parameter is that the weirs should not cause more than a modest rise in the 1% AEP flow profile.
- The design has considered the ability to drain all of the catchment with minimal site filling to maintain minimum freeboard to habitable floor levels.
- The setting of the 1% AEP flood level has been determined at sufficient depth to allow the channel to operate as an open waterway whilst minimising the overall depth and allowing overland flow from the catchment extents to flow by gravity to the channel and be unaffected by backwater effects from the flood level in the channel.
- During low flow there will be a series of discrete water bodies or wetlands. Each water body will be nominally 100 m long and be separated by a weir structure to maintain a permanent water surface.
- The wetland bench channel is important for flow, ecological, aesthetic and safety reasons. The wetland bench will contain plants, whereas the low flow channel will be deep enough to prevent or limit plant growth.

5.1.1 Channel geometry

Defined zones

The channel has been designed to allow for the following zones:

1. **Low flow channel**

   A meandering low flow channel with a permanent water depth varying between 0.5 m – 1.0 m (typically 0.8 m) controlled by the weirs at 100 m centres longitudinally along the base of the channel. The base of the low flow channel typically varies between 3 -6 m wide with slope batters 2H:1V. The 2H:1V batters are generally only 0.6 m high, below water level and are lined with a granular material, hence it is considered acceptable from a safety and slope stability perspective to have these greater than 4H:1V.

2. **Wetland bench**

   A slightly meandering wetland bench above the low flow channel that varies in width as the low flow channel meanders within it. The wetland bench is part of the permanent flow channel and the intention is for this zone to be within the permanent water level provided for by the weirs. The wetland bench will be planted with wetland species, is nominally flat and has a permanent water depth of 0-0.2 m. The wetland bench provides ecological, water quality and safety benefits.

3. **10% AEP water level**

   The channel bank is battered at 4H: 1V or flatter to a height between 0.70 m and 1.5 m to allow for conveyance of the 10% AEP. The batters will incorporate riparian planting, as per the planting plan in the Urban and Landscape Design Analysis Report (GHD, 2014) and as specified by Auckland Council. Paths and boardwalks extend through this area.

4. **1% AEP water level**

   The 1% AEP flood area is above the 10% AEP flood level and includes a mix of planted areas, paths, play space, grass and trees.

Flooded areas

The extent and depth of flooding from the 50%, 10% and 1% AEP events has been extensively discussed with Auckland Council. Auckland Council have adopted a design that has paths and boardwalks that will be submerged in events greater than the 50% AEP event and as such will not be available for public use.

In general the velocities in the channel are relatively low (<1 m/s) except at weir locations and 40 m upstream of the Grove Road culvert inlet.

Side slopes / channel batters

Generally, slope batters have been designed at 4H:1V or flatter to fit in with the landscape design and as per the recommendations from the Geotechnical Investigations Report (Technical Report C).

Steeper batters (2H:1V) in the low flow channel have been considered suitable for the following reasons:

- Being fully submerged improves slope stability from a geotechnical perspective and discourages access by pedestrians – hence improving safety.
- Having a granular lining improves stability from an erosion perspective and provides a stable / traversible surface if accessed by pedestrians.
- Low height (approx. 0.6 m) allows the slopes to be traversed by pedestrians who may enter the channel.

The channel sections have been modelled in the Geotechnical Investigations Report (Technical Report E).

Overall depth and width

The overall depth of the channel has been designed as shallow as possible for the following reasons:

- Minimise groundwater drawdown and associated potential ground settlement of adjacent land and structures.
- Maintain stable side slopes / channel batters.
- Minimise excavation volumes.

The main channel ranges in depth from between 1.9 m to 4.0 m bgl to the base of the channel. The overall total width of the main channel at the 1% AEP water level ranges from 20 m to 37 m.

The northern branch channel ranges in depth between 2.4 m to 3.8 m bgl to the base of the channel. The total overall width of the northern branch channel at the 1% AEP level ranges from 12 m to 27 m.

Planting

Planting has been selected by Auckland Council and generally consists of native grass species and sedges that would lay flat during large flow events. Tree species will have most of their mass above the 1% AEP event and therefore would not have a significant impact on the channel roughness. These include cabbage tree and kahikatea.

Paths

There are shared paths and boardwalks within the Awakeri Wetlands which allows public access along the corridor. Parts of the paths are expected to flood occasionally, with paths and boardwalks closer to the water surface flooding more frequently. Paths within the 10 year ARI event flow typically have alternative routes which would allow public access around flooded areas.

5.1.2 Channel alignment

The overall alignment of the corridor is linear, however the low flow channel varies in width, depth and direction to create variation in habitat. Refer to Drawing 51-33411-C211 for typical sections of the channel.

5.1.3 Channel bed slope

The overall gradient of the main channel from Old Wairoa Road at IL 23.97 m at the top of the channel falls to IL 19.80 m at Grove Road over a distance of approximately 1.55 km. This is an approximate overall gradient of 0.28%.

The overall gradient of the northern branch channel from Walters Road at IL 21.48 m at the top of the channel falls to IL 20.10 m at the junction with the main branch over a distance of approximately 0.70 km. This is an approximate overall gradient of 0.20%.

There are 9 major weirs designed along both channels. At very low flow, the hydraulic gradient is flat. The bed of the channels between each weir is also flat except where there is a variation in depth.

5.1.4 Low flow channel

The low flow channel depth has been selected based on a combination of water quality, flow characteristics, safety and industry guidelines.

The width of the low flow channel varies significantly along the alignment of the Awakeri Wetlands. The low flow channel for the Awakeri wetlands is unique in that it operates as both a conveyance channel and a wetland, therefore typical design guidelines for channels **or** wetlands cannot be applied directly – however the design has been based on the principles of a number of different design guidelines as discussed below.

The permanent water level in the channel varies throughout its length with a depth ranging from 800 mm to 1200 mm for the deeper sections and 200 mm for the wetland bench areas. These depths align with the principles for design of wetlands in Auckland Council's TP10.

No design recommendations for low flow channel depths or widths have been found in any Auckland Council or New Zealand design standards for similar channel designs.

The Queensland Urban Drainage Manual (Department of Energy and Water Supply, 2013) recommends a depth of 0.45 m for a low flow channel with a base width of 2.0 m.

The width of the Awakeri Wetlands is within this order of magnitude and has similar proportions, however it is typically deeper and wider due the specific project requirements including:

- Sufficient flow capacity required for conveyance of large events.

- Low velocities during low flow to minimise erosion.
- Fixed water levels to manage groundwater drawdown impacts.
- Safety (shallow / 200 mm around the perimeter of the open water and maximum depth that allows an adult to walk through).
- Water quality (sufficient volume/depth to manage temperature fluctuations).
- Variety (varying depths for ecological purposes).
- Aligns with the principles for design of wetlands in TP10.

5.1.5 Evaporation

There is a risk of lowering of the water level in the low flow channel due to evaporation. Lowering of the water level in the low flow channel could result in the following key issues:

- Drying out of the wetland bench areas.
- Die off of wetland plants.
- Odour issues.

It is expected that the wetland grasses in the wetland bench adjacent to the low flow channel, and some of the larger plant and tree species in the riparian margin (cabbage tree, kahikatea) will provide shading to the channel. This will help control temperature and evaporation while also providing additional ecological benefit.

As discussed above, planting will provide some mitigation for this risk, however additional management is recommended for the operation and maintenance of the channel, including:

- Monitoring of water levels.
- If evaporation issues are found to be an issue, additional mitigation can be installed to recharge water into the wetlands. This could include pumping from a nearby water source into the wetlands.

An assessment of historical rainfall and evaporation rates in the area has been undertaken to assess the likelihood and scale of this risk (provided in Appendix D), however there are a number of factors that cannot be modelled accurately and hence monitoring should be the key tool for assessing this risk.

5.1.6 Water balance

Main channel - above Cosgrave Road

Based on ground water balance models above Cosgrave Road we expect the dry summer low flow to have a surplus of water and a base flow in excess of 3 l/s. Historically, groundwater levels rise above the channel level, and hence there is expected to be flow from groundwater into the channel.

Main channel - Cosgrave to Grove Road

Groundwater has been observed in this area to drop lower than the proposed low flow water level during dry periods. Due to this, and from evaporation, we expect that there could be a net loss of water in the lower part of this area (near Grove Road) during extended dry periods.

Northern channel

The northern channel low flow water level is set close to the seasonal low groundwater level. For most of the year, the groundwater is typically expected to be above this level and therefore a baseflow is likely to be achieved in the channel for most of the year. This flow could decrease during extended dry periods. Monitoring of low flow water levels throughout the Awakeri Wetlands is recommended.

5.1.7 Operational water levels

The operational water levels for the 10% AEP and 1% AEP flows vary along the channel/wetland but typically are in the order of those shown in Table 11.

Table 11 Operational water levels

Channel zone	Typical water level above channel invert (m)
Low flow	0.80-1.20
10% AEP	1.40
1% AEP	1.70

6. Scour protection

Scour and erosion potential is an important consideration for the design of the Awakeri Wetlands. Scour and erosion of the channel could potentially result in poor amenity, discharge of sediment into the downstream receiving environment and bank stability issues for adjacent structures.

The peat soils which the Awakeri Wetlands will be constructed in are particularly soft and susceptible to scour and undercutting. Evidence of this can be observed at the McLennan Wetlands, the Bruce Pulman Park ponds and the table drains on Cosgrave Road and Walters Road, where the channel banks are being undercut by the open water surface.

Potentially high velocities and shear stresses in the channel pose the biggest risk of scour and erosion to the channel banks. Velocities are expected to be low during small rainfall events and scour and erosion is not considered to be an issue. In larger events, such as the 1% and 10% AEP, velocity and shear stress is higher and scour and erosion protection has been incorporated in the design to address this.

6.1 Scour and erosion protection design philosophy

Two approaches have been considered for the scour and erosion protection design.

1. **Hard engineered, fully mitigated approach**

   This approach would include providing scour and erosion protection along the entire channel banks within the 10% AEP flow area. This would consist of a mixture of granular material / rip rap and other proprietary devices such as Geoweb, blown bags and Reno mattresses applied extensively throughout the channel.

   The hard engineering solution would provide a reduced chance that retrospective scour and erosion protection would need to be installed post-construction, however it would also require a significantly higher up-front cost and overall would result in a less attractive asset from a landscaping, ecological and public amenity perspective.

2. **Risk based approach**

   This approach would consist of providing scour and erosion protection at critical areas only where the consequence of scour and erosion could cause damage to key structures and would impact directly on the performance of the channel as a hydraulic asset and amenity feature. This would consist of installing granular material and other proprietary devices such as Geoweb and Reno-mattresses at locations around key structures and at high velocity locations in the channel such as:

   - Where the paths are in proximity to the low flow channel (key structures).
   - At weir locations (high velocities).
   - Around boardwalk or bridge locations (key structures).
   - At culvert inlet and outlets (high velocities).
   - Along steep slopes (high risk).
   - Within the low flow channel (high risk).

   The remainder of the areas within the 10% AEP flow area are at risk of scour and erosion, however the immediate consequence is considered low because any issues can be detected during inspections and remediated if required. This risk based approach would be adopted, coupled with on-going monitoring of scour and erosion and that this should form part of the operation and maintenance manual for the Awakeri Wetlands.

If scour does become an issue in these areas and it is left to continue over a period of time (ie. monitoring and maintenance is not undertaken), it is possible that the consequence could become critical such that:

- Deep vertical channel slopes are formed.
- Channel batters approach adjacent properties and/or cause slope instabilities.

This philosophy has been discussed with Auckland Council who have indicated that this is the preferred approach and that there is a strong driver to keep the Awakeri Wetlands as natural as possible.

6.1.1 Discussion with Auckland Council

As mentioned above, Auckland Council have requested that the Awakeri Wetlands are naturalised as much as possible and that exposed granular material is kept at a minimum. The scour risks have been communicated and it has been agreed that a higher level of scour risk will be accepted by Auckland Council in conjunction with monitoring to keep a natural finish to the channel batters.

Monitoring will be undertaken in accordance with the Operation and Maintenance Manual for the channel which has been prepared by Auckland Council.

Auckland Council have advised that the selected plants will have extensive root systems that are expected to provide reinforcement to the soil and on this basis have adopted that the risk based approach is their preferred option.

6.1.2 Discussion of recommended approach

The risk based approach is the recommended option. The risk based approach has the following key benefits over the hard engineered solution:

- **Cost effective:** Reduced construction costs due to reduced quantities of imported construction materials. This approach requires special attention to monitoring and retrospective remediation for repairing any identified areas, however it allows economical use of resources as additional mitigation can be applied to known areas of scour that are not possible to predict in advance such as:
  - Soft, loose ground conditions encountered at low flow channel level.
  - Obstructions / logs encountered and left in place during construction.
- **Aesthetics:** The risk based approach allows increased areas of planting in natural soils to give more vegetation cover and less exposed granular material.
- **Environmental:** The risk based approach allows greater density and extent of planting in natural soils. This provides more wildlife habitat, shading for the low flow channel and materials are less prone to heating up to help manage temperature of the low flow channel.

6.1.3 Adoption of risk based approach in design

The primary method of scour protection throughout the channel is reliance on the root systems of the plants to reinforce the channel banks and soils. At areas within the Awakeri Wetlands that are considered at high risk of scour or where scour will have a high consequence, planting is proposed to be coupled with Geoweb and Enkamat to reinforce the plant roots.

In areas where velocities and shear stresses are especially high, the Geoweb will be filled with granular material or a mixture of granular material and topsoil for planting.

Granular material by itself has only been accepted by Auckland Council where it is permanently submerged and not visible. Where there is a high risk of scour above water, such as around the weir locations; a mixture of granular material and topsoil for planting is proposed within the Geoweb cells.

Further detailed of how this approach has been implemented are provided in the following sections.

6.2 Channel velocity

The peak 1% AEP flow velocity has been calculated and is approximately 1.3 m/s just upstream of Grove Road before dropping into the fish passage leading down to the Grove Road culvert inlet structure (Refer to Table 12). At Cosgrave Road the peak 1% AEP velocity is 1 m/s. The northern channel typically has velocities less than 1.0 m/s. This excludes the peak localised velocity of water flowing over the weir sections, where there is an expected increase.

Average velocities have been calculated along the channel and are noted in Table 12.

Table 12 Average channel velocities

	10% AEP		1% AEP	
Chainage (m)	Q (m3/s)	V (m/s)	Q (m3/s)	V (m/s)
Main Channel				
0	25.2	1.34	40.2	1.71
100	25.0	1.01	39.9	1.2
200	15.4	0.84	24.5	1.06
300	15.2	0.84	24.0	1.06
400	14.8	0.81	23.5	1.02
500	14.6	0.44	23.0	0.6
600	14.3	0.79	22.6	0.93
700	13.5	0.75	21.3	0.89
800	12.6	0.71	19.9	0.88
900	11.6	0.78	18.5	1.00
1000	5.8	0.68	9.6	0.83
1100	5.6	0.64	9.2	0.81
1200	5.3	0.62	8.7	0.79
1300	5.1	0.45	8.2	0.55
1400	5.0	0.31	7.9	0.93
1500	3.2	0.42	4.7	0.43
Northern Channel				
60	9.4	0.78	15.0	0.87
200	8.5	0.63	14.4	0.76
300	7.0	0.59	11.5	0.73
400	5.5	0.54	9.2	0.74
500	4.4	0.49	7.4	0.67
600	3.6	0.78	6.0	0.80
700	2.8	0.55	4.6	0.31

The velocities in Table 12 represent the average velocities over the full cross sectional flow area. These velocities are low and are generally less than 1 m/s. In storm events smaller than the 1% and 10% AEP, velocities are expected to be lower.

It is estimated that the velocity at the downstream end of the main channel is approximately 0.6 m/s in the 50% AEP and 0.3 m/s in the 100% AEP. These velocities are low and are not expected to cause significant scour or erosion in the channel.

6.3 Shear stress

Bed shear stress in the channel has been extracted from the MIKE11 model as per Table 13 below.

Table 13 Average channel shear stress

	10% AEP		1% AEP		
Chainage (m)	Q (m^3/s)	Bed Shear Stress (N/m^2)	Q (m^3/s)	Bed Shear Stress (N/m^2)	Comments
Main Channel					
0	25.2	17.51	40.2	27.40	
100	21.9	9.15	37.6	12.65	
200	14.0	9.86	24.2	10.61	
300	13.7	6.99	23.6	10.73	
400	13.4	6.42	23.2	9.81	
500	13.1	1.43	22.7	8.26	
600	12.9	6.21	22.3	8.37	
700	12.2	5.56	21	7.77	
800	11.5	5.05	19.8	7.51	
900	10.7	6.18	18.5	9.80	
1000	5.9	4.70	9.6	6.86	
1100	5.7	4.12	9.2	6.52	
1200	5.4	3.89	8.7	6.18	
1300	5.1	2.04	8.2	3.02	
1500	2.8	1.04	4.7	1.02	
Northern Branch					
60	9.4	6.42	15.0	7.82	
200	8.5	4.78	14.4	5.82	
300	7.0	4.30	11.5	5.90	
400	5.5	3.33	9.2	4.95	
500	4.4	8.55	7.4	12.13	500 mm deep channel drops into 800 mm deep channel
600	3.6	3.67	6.0	4.90	
700	2.8	0.65	4.6	0.87	

Erosion occurs when the shear stress of the flow exceeds the strength of the soil particles along the surface of the bed. Peat soil is highly erodible and the bed shear stresses from the channel flow is expected to surpass the critical value for exposed peat. The critical shear stress values for peat is typically highly variable depending on the level of decomposition and disturbance, and can range from <1 N/m^2 up to 5 N/m^2 (Tuukkanen, T., H. Marttila, and B. Klove, 2014)

The bed shear stress experienced in the Awakeri Wetlands peaks at approximately 27-30 N/m^2. The higher values are predicted mostly at the downstream end of the main channel, at the confluence of the two channels and at the weir locations.

The peat soil in this catchment is therefore susceptible to sediment transport and as a result, the channel bed is proposed to be protected with a Bidim+Geogrid composite material held down with 70kg/m^2 of GAP 65 hardfill. This will minimise the degradation and the aggradation of the channel invert.

6.4 Scour and erosion risk

The surface cover of different zones within the channel provides varying levels of resistance against scour and erosion. Table 14 outlines the surface cover types and the expected performance in regards to scour and erosion due to flow in the channel.

Table 14 Scour and erosion risk for channel zones

Zone	Surface Cover	Risk of scour / erosion	Protection measures
Typical channel areas			
Low Flow Channel	Naturalised channel with pools and slight channel meander	High susceptibility to scour and erosion.	Geogrid+Bidim composite with gravel lining along base of the channel and 1(V):2(H) submerged slopes.
Wetland Bench	Wetland grasses	Low risk. Wetland grasses will slow velocities and roots will strengthen soils.	No additional protection required.
Channel bank	Sedges and native grasses. Small unrestrictive trees with mass of branches above 1% AEP.	Low risk. Roots of grasses and trees will strengthen channel banks.	No additional protection required.
Floodplain	Sedges, with footpath and specimen trees	Low risk. Roots of grasses and sedges will naturally protect from scour and erosion.	No additional protection required.
Critical areas			
Between paths and water body	Sedges and native grasses. Small unrestrictive trees with mass of branches above 1% AEP.	Low risk. Roots of grasses and trees will strengthen channel banks. High consequence of undercutting paths.	Geoweb along slope with soil / gravel infill and planted.
Weirs	Sedges and native grasses. Small unrestrictive trees with mass of branches above 1% AEP.	High risk. Velocities high as water flows over weir.	Reno Mattress and Gabion basket immediately downstream of weir. Geoweb with GAP80 and Geoweb with planting and topsoil around weir.

Boardwalks	Boardwalk abutments and adjacent ground	High risk, high consequence. Undercutting of different materials at boardwalk and bridge locations.	GAP65 backfill immediately around structure and Geoweb with a mixture of granular material and planted topsoil

The low flow channel side slopes are proposed to be lined with a granular material to prevent scouring of the sides of the low flow channel and to provide solid side slopes which would facilitate safety aspects if someone were to enter the channel. The base of the channel will be protected with a Geogrid / Bidim composite, weighed down with GAP65 at 70kg/m^2.

6.5 Planting

Wetland plants, sedges, native grasses and small trees have an ability to withstand the generally expected velocities (<1.4 m/s) without adverse effect. The planting of the channel will provide stability to the soils to resist against scour and erosion once plants are established.

In early years before plants and roots are fully established, the channel will be more susceptible to erosion. However it is unlikely that the catchment will be fully developed within this time, and therefore peak runoff and velocities are expected to be much less, hence mitigating this risk.

Plants are being relied on to provide the bulk of the erosion and scour protection for the channel and the ability of their roots to provide this function has been confirmed by the planting designer at Auckland Council. The channel slopes will need to be protected temporarily while the plants are establishing. The planting designer indicates that within 1-2 years the roots will have grown extensively through the soils and will aid greatly in stability of the slopes. Coir matting is proposed to provide temporary protection to the channel slopes during the 1-2 year establishment phase. Coir matting will slowly biodegrade over time but typically lasts for a period of 2-3 years.

Full development of the catchment is not expected to be completed for some years after the construction of the channel. As such peak flow rates will be less than the MPD scenario during this time. This will allow additional time for the wetland plants to become established and grow.

As discussed in Table 14, soil filled Geoweb with Enkamat underneath will be installed at critical areas where the consequence of scour is high (such as around footpaths and structures).

6.5.1 Geoweb

Geoweb is frequently used in slope protection and stormwater channel applications. It provides structural confinement of topsoil/vegetation and granular materials such as sand, gravel and larger rock or stone. An example is shown in Figure 11 below.

Figure 11 Geoweb (https://www.geofabrics.co)

The Geoweb proposed in the Awakeri Wetlands project will typically extend from the lower side of footpaths down and across the wetland bench of the low flow. Hence it will be partially submerged. Native grasses and shrubs will be planting into the Geoweb cells in accordance with the planting plan. The Geoweb will provide confinement for the topsoil and some support for the planting across the surface of the channel. The cells will also aid in capturing sediments and material that may be lost due to erosion, before these materials are discharged downstream.

6.5.2 Enkamat

Enkamat is proposed beneath the Geoweb to provide additional support for the plant roots. Native grasses and shrubs that are planted within the Geoweb cells and above the Enkamat layer will be small and will take time to establish. During the establishment phase, the roots will grow through the Enkamat, providing additional reinforcement to the plants. This will reduce the risk of plants being lost during storm events and is expected to reduce the long-term maintenance costs of the channel.

Enkamat is usually installed close (25 mm) to the ground surface, because typically the surface above the Enkamat is grassed with a turf grass with shallow roots. Having the Enkamat at a shallow depth provides support to the shallow roots. Given that the planting in the Awakeri Wetlands consists of larger native grass species with deeper roots, the Enkamat has been proposed deeper, and to be laid beneath the Geoweb (100 mm deep).

Enkamat is shown in Figure 12 below:

Figure 12 Enkamat example

As a proof of concept, a high level trial was undertaken which consisted of planting two plants within a container on top of a layer of Enkamat (MacMat R) with approximately 100 mm of topsoil above it.

6.5.3 Proof of concept trial

Two plants were selected which were readily available from a local store which represented an approximation of the type of plants that would be used within the relevant areas of the TSCC. The two plants used were:

- *Carex secta* (Purei)
- *Phorium tenax* (New Zealand Flax)

MacMat R was used which is Enkamat with a layer of steel wire reinforcement. A schematic is shown in Figure 13.

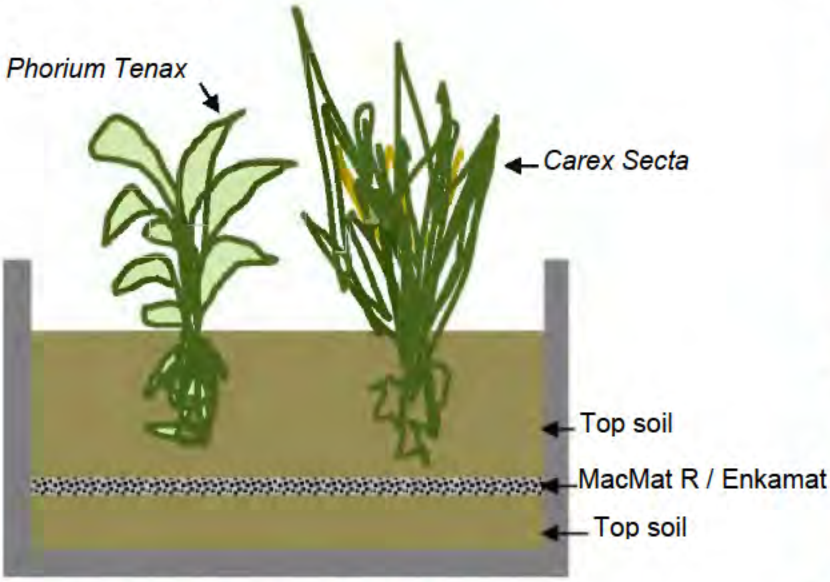


Figure 13 Trial set up 05/06/2017

After 2 months, the plants were removed to observe whether the roots had grown through the Enkamat.

Figure 14 shows some photos after 2 months indicating that the roots had grown through and interwoven with the Enkamat, hence indicating that the Enkamat will provide some support to the plants.

Figure 14 End of trial 05/08/2017

6.6 Confluence main channel and northern branch (Ch 200)

At the confluence of the main channel and northern branch there is a large body of water. This body of water is expected to provide energy dissipation as the two channels come together. Given the low velocities in each channel and the large volume of water at this confluence, the erosion and scour potential of the flow is expected to be low. The expected bed shear stresses are expected to be resisted by the bidim+geogrid composite with the aggregate covering.

The purpose of the aggregate is to hold down the bidim+geogrid on the bed of the channel. The geogrid by itself is neutrally buoyant, and therefore requires aggregate to reduce the chance of it coming loose.

No additional or special scour protection is proposed at this location, other than what is already proposed in the previous discussion.

6.7 Sediment deposition

6.7.1 Estimated sediment deposition

The typical runoff from a developed Auckland catchment will be in the order of 0.5 t/ha/annum. This is based on soil types generally consisting of Waitemata clays and would occur when all bulk earthwork development has been completed and individual housing sites are developed. In the case of this development there is expected to be areas of recent peat alluvium as per the existing soils, in addition, there is expected to be imported fill from developers. Slopes in this catchment are very flat and therefore it is expected that the runoff will be towards the lower range of any variance around 0.5 t/ha/annum.

The steep portion of the 2B catchment will drain to a stormwater pond at the Cabra Development site, and therefore sediment removal is expected for this area.

We can also expect that a portion of sediment will be entrained and passed through the system down to the McLennan wetland and Pahurehure Inlet during high flow events. We therefore expect the residual sediment deposition in the channel to be in the order of 0.25 t/ha/annum. If this deposition is evenly distributed along the channel, then the catchment area/channel length (155 ha / 2,100 m = 0.74 ha / lineal meter) relates to an annual deposition of 18 kg per lineal meter of channel per annum. We would expect some of this to be deposited below the permanent water level.

6.7.2 Maintenance

The annual estimated deposition rate is between 1.0 - 1.5 mm/annum. At this rate, it would take between 60-100 years for 100 mm of sediment to build up along the channel. This may not be distributed evenly, and would likely be distributed along the wetland planting area, the main low flow channel and behind the weirs. It is expected that maintenance to remove sediment would be required approximately every 20-50 years. This has been allowed for by provided access to key areas.

This has been discussed and agreed with Auckland Council Healthy Waters Operations Team.

6.8 Other channel features

6.8.1 Swamp Kauri and obstructions

There is a high likelihood of digging up timber including Swamp Kauri tree logs, stumps and trunks during the excavation. In accordance with the Curatorial Framework, any Kauri that is dug up and removed from the channel shall be retained on site and stockpiled for potential re-use. If the Kauri cannot be excavated without damaging the channel batters then it shall be left in place unaltered and flagged within the design team for potential utilisation in-situ within the channel design.

Logs or obstructions cannot be excavated where they penetrate the channel profile as backfilling to form the channel profiles is not practical.

An assessment will need to be made on a case-by-case basis on how to utilise the Swamp Kauri and other timber. Hydraulic, geotechnical and structural risks will also need to be considered.

Options for managing logs or obstructions include:

- Leaving the obstruction in place within the channel profile.
- Cutting the obstruction and leaving the remainder in place.

These options will need to consider how the obstructions will integrate into the final form of the channel, including assessment of any hydraulic issues, scour and erosion issues and landscaping elements.

7. Weirs

In order to maintain a permanent waterbody within the wetland channel, a series of weirs at notional 100 m centres will be used to maintain these bodies of water. The depth of water behind each weir is 800 mm with a depth of 200 mm along the wetland bench. As well as providing for aquatic habitat, the permanent water level will assist in reducing groundwater drawdown and associated potential settlement by maintaining the groundwater at a level higher than the channel invert.

The top surface of the weirs ranges between 9 m to 14 m across.

The step between each weir varies from 0.18 m to 0.45 m to give an overall average gradient along the full channel length and to facilitate fish passage and to provide hydraulic controls. At medium and high flows these weirs will be totally drowned. The depth of the 1% AEP event flow above the top of the weir level has been calculated as up to 1 m deep.

As the flow increases (during a flood event), the flow over the weir increases and the flow in the channel downstream of the weir raises at a faster rate until the weir is almost drowned. Prior to the weir being drowned the flow becomes critical over the weir and the velocities will be at a maximum. The extent of increase will depend on the difference in water level above and below the weir.

7.1 Main structure

The main structure of the weir consists of PVC sheetpiles down to 6 m below ground. This is required to create an impermeable hydraulic cut-off and to maintain stability of the weir structure. An impermeable cut-off is required to maintain the permanent body of water behind the weir to reduce permanent groundwater lowering within adjacent land which can cause ground settlement.

PVC sheetpiles were selected because they provide the following advantages:

- Durable and resistant to acid / low pH (compared to steel sheetpiles).
- Easy to install along the channel following excavation of the ground profile.
- Reduced risk of ground settlement (compared to a concrete structure due to weight).
- Top surface of the weir can be readjusted to account for any movement by alterations to the timber facing on top of the weir.

The proposed method of installation is to drive the sheetpiles and leave approximately 200 mm high to allow any negative skin friction forces to dissipate. Following this rest period, the PVC sheets will be cut to length and capped with a hardwood timber beam. The beam has a low point in the middle and slopes up gradually towards the channel batters. This provides hydraulic benefits in larger storm events by concentrating flow towards the centre of the channel.

Stability calculations for the weir are provided in Appendix D.

7.1.1 PVC sheet pile stability

The stability of the sheet pile walls have been checked based on kick-out calculations, as attached in Appendix D. The calculation shows that the sheet pile is stable under the proposed conditions.

The Wallap software was also used to determine displacement of the sheet pile. Displacement of the sheet pile under the proposed conditions is less than 50 mm at the top of the sheet pile. This is considered acceptable.

7.2 Fish passage

Fish passage is provided for at each weir. A series of timber beams and notches downstream of the main weir provides a series of 50 mm steps which water will flow down to create a passage for fish to climb. The overall drop varies per weir between 180-450 mm, but with the largest individual step of 50 mm.

Refer to drawings 51-33411-C213 for fish passage details.

During drier periods when flows reduce below 5 l/s, the fish passage will be restricted to eels and other good climbing species. A very low flow fish passage during dry periods will not be available unless make up water is introduced at the top of the channel.

7.3 Erosion and scour protection

A Reno Mattress and gabion basket at the downstream end of the weirs is proposed for dissipating energy from water flowing over the weirs. This also provides support and energy dissipation for the proposed incoming stormwater pipes from adjacent developments.

Rock and soil filled Geoweb around the weirs provides additional erosion protection at critical locations.

Refer to drawing 51-33411-C215 for details of the erosion and scour protection at the weir locations.

8. Stormwater Connections

Development within the catchment of the Awakeri Wetlands are expected to discharge primary and secondary flows into the channel. Primary flows (10% AEP) are expected to enter the channel via piped networks. Secondary flows (1% AEP) are expected to enter the channel via overland flow.

8.1 Development connections to channel

The channel has been designed with a shallow depth to reduce potential for groundwater drawdown and ground settlement. The channel therefore requires a wide, shallow flow depth to allow connections for servicing the 10% AEP. Swales or multiple small diameter shallow pipes would be favourable for draining the catchment once developed due to the shallow channel.

Lateral connections to allow properties to drain have been assumed as piped flow, where practical, for events up to the 10% AEP. Overland flow paths will be required to convey flows up to the 1% AEP event.

Drawings 51-33411-C218-C219 shows the proposed outlet detail for connections to the channel.

Piped connections to the channel will typically enter at the permanent water level. Piped connections are required to discharge at the base of the 4H:1V channel banks, typically downstream of the proposed weirs.

Key benefits of discharging downstream of the weir locations are:

- Limit outlet structures and associated energy dissipation to areas where energy dissipation is already required to control flow over the weirs.
- Allows maximum steepness of the hydraulic gradient of the piped flow and as such limiting pipe sizes to their respective minimum size.
- Increased cover over the discharging pipe.
- Visually less prominent within the riparian and wetland planting between the weir structures.

8.2 Pipe connections

PE stormwater pipe outfalls will be installed as part of the Stage 1A works with one upstream manhole for developers to connect into.

Manholes are located on the outside of the slurry wall compared to the channel where relevant. This allows developers to connect into the manholes with their stormwater discharges without excavating through the slurry wall. This reduces the risk of the slurry wall being compromised in the future which could result in lowering of groundwater and the associated long-term settlement issues.

Stormwater outfalls have been sized based on an indicative development layout plan for the catchment. The indicative layout plan was provided by Auckland Council and is titled “Takanini Cascades Development Framework Plan July 2017”. Based on this plan, possible road levels were determined from which possible overland flow and stormwater catchments have been proposed as per the GHD Drawings 51-33411-C601-C613 attached in Appendix C.

The possible catchments were used for calculating pipe sizes for the stormwater discharge pipes. These sizes would need to be checked prior to developers connecting into these locations, and alterations to the outfalls may be required depending on the final catchment draining to each area.

The proposed pipe sizes and details are outlined in Table 15 below:

Table 15 Pipe outfalls

Pipe	Pipe diameter (mm)	Pipe slope (m/m)	10% AEP flow (m^3/s)	10% HGL slope (m/m)	Velocity (m/s)	Location
SWOF-80A-1	630	0.01	0.565	0.011	2.34	D/S Weir
SWOF-80A-2	710	0.01	0.678	0.008	2.21	D/S Weir
SWOF-180A-1	500	0.01	0.187	0.004	1.23	D/S Weir
SWOF-180A-2	500	0.01	0.372	0.016	2.45	D/S Weir
SWOF-330A-1	500	0.01	0.213	0.005	1.40	D/S Weir
SWOF-330A-2	710	0.01	0.812	0.012	2.65	D/S Weir
SWOF-440A-1	630	0.01	0.518	0.009	2.15	D/S Weir
SWOF-440A-1	560	0.01	0.495	0.015	2.60	D/S Weir
SWOF-100B-1	500	0.01	0.309	0.011	2.03	D/S Weir
SWOF-260B-1	400	0.01	0.138	0.007	1.42	D/S Weir
SWOF-260B-2	500	0.01	0.231	0.006	1.52	D/S Weir
SWOF-300B-1	400	0.01	0.146	0.008	1.50	Culvert wall
SWOF-300B-2	800	0.01	0.909	0.008	2.34	Culvert wall
SWOF-340B-1	560	0.01	0.483	0.015	2.53	D/S Weir
SWOF-340B-2	630	0.01	0.684	0.016	2.83	D/S Weir
SWOF-480B-1	630	0.01	0.672	0.015	2.78	D/S Weir
SWOF-480B-2	560	0.01	0.353	0.014	2.32	D/S Weir
SWOF-570B-1	355	0.01	0.107	0.008	1.41	Low flow channel
SWOF-680B-1	710	0.01	0.680	0.008	2.16	Culvert wall

8.2.1 Pipe outfall support

Pipe outfalls typically enter the channel 200 mm below the low flow water level and will therefore be partially drowned (less than half) during low flow. This assists with energy dissipation and reduces the visual impact of the pipes while still allowing suitable maintenance access.

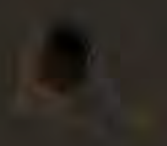
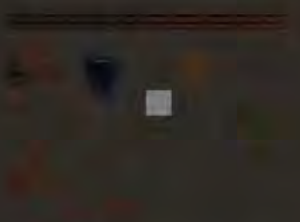
The philosophy for the pipe outfall design includes:

- Minimising the visual impact of the outlets.

- Managing scour and erosion of material around the pipes.
- Managing loss of material via seepage through backfill.
- Connecting into Geoweb material.
- Allowing practical maintenance and access.

A number of options have been considered for the detail around the pipe outlets. Table 16 outlines the options considered.

Table 16 Pipe outfall options

Option	Diagram	Advantages	Disadvantages
Standard concrete headwalls structure		• Effective management of seepage, scour and erosion	• High visual impact • Heavy
No facing around pipe		• Low visual impact	• Risk of scour and erosion around outfall • Risk of seepage and loss of material around pipe
Rip rap surround		• Medium visual impact • Effective management of seepage, scour and erosion	• Un-natural materials for the area
Mitred concrete headwall		• Effective management of seepage, scour and erosion	• Medium visual impact
Hessian bag headwall		• Effective management of seepage, scour and erosion	• Medium visual impact
Timber frame		• Effective management of scour and erosion	• Medium visual impact • Moderate risk of scour and erosion
PE flange fixed to Geo web		• Low visual impact • Reasonable management of scour and erosion when combined with Geoweb	• Moderate risk of scour and erosion

8.2.2 Selected option

Auckland Council have advised that the visual impact of the pipe outfalls is a key requirement and should be reduced as much as possible. The timber frame and rip rap surround options were not considered suitable by the landscape architect.

The PE flange fixed to Geoweb option is the selected option.

The PE flange option provides a reduced visual impact of the pipe outfalls as they can be integrated and hidden by the surrounding landscape features. Geoweb will surround the pipe outfalls and will be fixed to the PE pipe.

The flange of the pipe will overlap the Geoweb to prevent it from coming loose around the pipe. The flange is proposed to be 75 mm wide from the outer wall of the PE pipe to the outer diameter of the flange.

The Geoweb will be filled with a mixture of rock and soil and planted. Rock will be placed around the pipe outfall to a distance of at least 300 mm to minimise scour around the pipe outlet.

8.2.3 Timing

The benefits of installing the outfalls as part of the Awakeri Wetlands project are:

- Pipes are installed through the slurry wall in a controlled manner, and reinstatement of the slurry wall hydraulic barrier can be monitored and achieved to a high quality.
- Auckland Council can control the appearance of the outfalls, including size, material and headwall structures.
- Auckland Council can control the locations of the outfalls to align with the design of the Awakeri Wetlands (i.e. typically downstream of weirs where practical.

8.3 Overland flow

Overland flow will need to be conveyed to the channel via secondary overland flow paths from development within the adjacent land. The design of these flow paths will be undertaken by the developers of the land. Overland flow paths for developments are usually designed along walkways or roads. This will be done by individual developers as and when infrastructure for particular development is implemented.

The channel has been designed with a depth to allow sufficient hydraulic grade from the furthermost extent of the catchment to the channel. Some areas will require fill by the developer due to the existing topography sloping away from the catchment. The possible drainage solution considered uses pipes to convey the primary flow (10% AEP). Developers may use swales and water sensitive design rather than piped networks, however assuming pipes provides a conservative assessment.

The Awakeri Wetlands design includes bank protection at locations where overland flow is expected down the banks. Scruffy dome manholes are provided upstream of the pipe outfalls to allow the capture of some overland flow where overland flow paths are expected to align with pipe outfalls.

Given that development of the catchments and alignment of overland flow within the catchment is highly dependent on developers, the locations allowed for in the Awakeri Wetlands project are indicative, and alternative locations may be installed in the future by developers.

9. Grove Road Culvert Inlet

The Grove Road Box Culvert and the inlet structure for it has been designed by Jacobs, who have provided an invert level of the culvert of 17.5 m. The culvert entry has a tapered mouth to provide more efficient inlet conditions. The culvert mouth has an invert level of 17.6 m. The mouth transitions into an apron which slopes up to RL19.1 m.

The downstream weir of the Awakeri Wetlands has an RL of 20.55 m (lowest point). Therefore a 1.5 m vertical transition is required between the inlet structure/apron and the last weir of the Awakeri Wetlands. This section outlines the design of this transition. Drawings 51-33411-C221-C223 outlines the concept.

Design principle

The key considerations for the design of the transition between the Awakeri Wetlands and the Grove Road Box Culvert inlet structure include:

- Lower velocities to control erosion / scour.
- Flood level to achieve suitable freeboard for Grove Road.
- Fish passage.
- Controlling groundwater drawdown.

The key design features include:

- A series of discrete pools formed using PVC sheet piles and timber facing to assist fish passage between the Grove Road culvert and the TSCC.
- There is a 150 mm step between each pool allowing each pool to cascade into each other via low points created using the weirs. Each 150 mm step is broken up into three 50 mm steps which is formed using timber. This provides a 50 mm maximum jump for fish travelling up the passage.
- The average longitudinal slope of the fish passage is approximately 13.3H:1V.
- The pools are approximately 2.4 m wide allowing some shading from adjacent planting of native grasses and shrubs.
- This defined channel has capacity up to 1 m^3/s before water spills over other sections of the weir and flows across the adjacent ground slope.
- The ground adjacent to the fish passage has an approximate slope of 5% with a 0.5 m drop over a timber faced concrete wall at the base of the slope. The purpose of the wall is to allow a reduction of the overall slope leading down to the culvert headwall to reduce velocities and shear stresses along the slope. This will allow the slope to be planted with native grasses and shrubs, which will hide the concrete and improve the aesthetics at this location.
- Geoweb and Enkamat is proposed across the slope to reduce the risk of scour during storm events and to help stabilise the plants and soil along the slope.
- The last weir of the Awakeri Wetlands is located at the top of the slope and is approximately 35 m long with an RL of 20.55 m at the centre (lowest point). The level of the weir varies across its length with areas of RL 20.80 m and RL 20.90 m to control the flow regime and manage scour risk of the downstream slope.
- This weir sets the permanent water level in the upstream channel, which is maintained to control the groundwater level.

- This last weir outside of the controlled fish passage incorporates a drop of RL 20.80 to RL 20.50. This drop concentrates the energy dissipation where there is a high level of erosion protection prior to flowing down the overall slope to the culvert mouth.

9.1.1 Scour and erosion

High flow events up to the 1% AEP event are not expected to produce the highest velocities, as the flow will be drowned out at the culvert entry; rather, the smaller events will produce the critical velocities for erosion and scour. Velocities are expected to reach up to 3-4 m/s for the critical storm events along the surface leading down to the Grove Road culvert inlet structure. These velocities are expected to be acceptable for planting and will be dissipated using a strategically placed concrete wall and apron at the downstream end of the slope and hardfill immediately downstream of the weir.

A combination of Geoweb and Enkamat is proposed along the slope down to the inlet. The Geoweb will be filled with:

- A mixture of 75% soil and 25% GAP65. This will allow the surface to be planted and will provide some reinforcement to the plant roots as discussed in Section 6.5.3.
- GAP65 strip 1 m wide downstream of the last weir at the top of the slope leading down to the culvert mouth to manage sufficient energy dissipation over this weir.

9.1.2 Groundwater drawdown

The weir at the top of the slope will maintain the permanent water level in the channel. Downstream of this weir, the proposed ground level will drop into the Grove Road Culvert Inlet. To prevent groundwater drawdown due to the deeper cut; a physical groundwater cut-off barrier has been constructed to surround the entire inlet structure, as per Drawing 51-33411-C221.

The barrier is a 7 m deep slurry wall and has been installed. A similar barrier has also been installed upstream near Cosgrave Road to mitigate groundwater drawdown due to the deep cut of the channel.

10. Crossings

The Awakeri Wetlands includes a number of proposed vehicular crossings. The Cosgrave Road culvert and Old Wairoa culvert are part of Stages 2 and 3 and therefore have not been discussed in this report.

Crossings within the Stage 1 Awakeri Wetlands include

- A proposed culvert on the northern channel at Chainage 300.
- A proposed culvert on the northern channel at Chainage 700.

The proposed culverts are standard culvert units and the suppliers shall provide the structural design details based on the requirements discussed within this section.

This report, along with the specification outlines the design requirements for each of the culverts.

Table 17 Culvert details

Culvert location	Internal dimensions	Length	Description
Northern Channel CH300	1.5 m (H) x 2 m (W)	20 m	Provides a channel vehicle crossing for developments on adjacent sides of the channel. Allows for a standard 20 m wide road corridor including 2 lanes, footpaths and berms.
Northern Channel CH700	1.5 m (H) x 2 m (W)	8 m	Provides a one-way entry or exit for the Kauri Flats School. Single lane and footpath.

Key features for the culverts include:

- The culvert has a notionally flat gradient and operates under a hydraulic grade line above culvert invert.
- The design hydraulic grade line is such that the culvert does not result in any noticeable increase in flow depth in the channel upstream of each of the culverts.
- The units will be post-tensioned together to resist differential settlement.
- The culvert is designed to allow for continuation of the low flow channel beneath the road to extend the ecological corridor past this constraint.
- Sufficient height and width to allow for safe maintenance and inspections (1.5 m high).
- Flows partially full with low velocity during all storm events for safety.
- Shallow low flow water depth within the culvert for safety (0.8 m depth).
- Earth / gravel fill on top of the wing wall structures to continue channel profile up to the culvert and allow planting to hide the concrete wing wall structures.
- Allowance for up to 40% blockage without significant increase in headwater depth.
  - Based on the principles of the Stormwater CoP and a best practicable approach, the Stormwater Code of Practice recommends 50% blockage, but given the environment and level of risk, 40% has been considered appropriate.

10.1.1 Northern Channel Chainage 300 Crossing

This crossing is shown on Drawing 51-33411-C230. This crossing is proposed as a twin 1.5 m (H) x 2 m (W) x 20 m (L) concrete box culvert with 45 degree wingwalls.

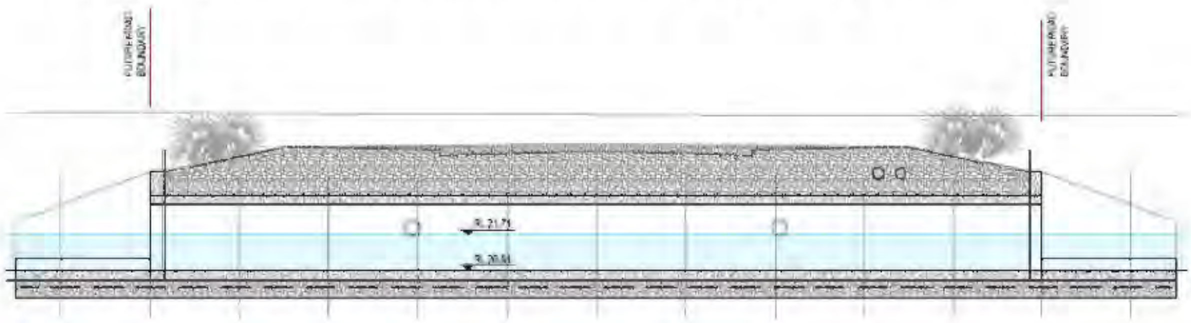


Figure 15 Northern Channel CH 300 culvert crossing

10.1.2 Northern Channel Chainage 700 Crossing

This crossing is shown on Drawing 51-33411-C240. This crossing is proposed as a twin 1.5 m (H) x 2 m (W) x 8 m (L) concrete box culvert with 45 degree wingwalls.

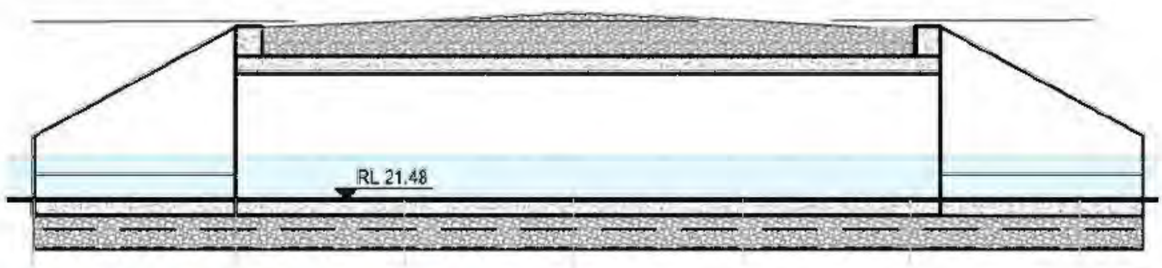


Figure 16 Northern Channel CH 700 culvert

10.2 Hydraulic design

10.2.1 Hydraulic losses

Losses through the culverts were checked using Bernoulli's Energy equation and the losses were determined to be sufficiently low to not significantly impact the hydraulic grade line of the channel. Calculations are provided in Appendix D and summarised Table 18 below.

Table 18 Culvert losses

Culvert	Exit loss (m) 1% AEP	Friction loss (m) 1% AEP	Entry loss (m) 1% AEP	Total loss through culvert (m) 1% AEP	Upstream water level 1% AEP (m RL)
CH300B	0.067	0.045	0.067	0.179	22.64
CH700B	0.019	0.003	0.019	0.041	23.00

10.2.2 Blockage

A blockage assessment for the culverts was undertaken, where it was assumed that the inlet of the culverts were blocked by 0%, 25% and 40% to determine the impact on hydraulic operation of the Awakeri Wetlands system. The results are outlined in Table 19 below

Table 19 Culvert blockage

Culvert	Upstream water level / (in brackets) = increase in headwater depth (m RL)			Upstream water level from culvert calculation (m RL)
	0% blockage	25% blockage	40% blockage	
CH300B	22.64 (0.000)	22.71 (0.070)	23.14 (0.500)	22.64
CH700B	23.00 (0.000)	23.00 (0.000)	23.00 (0.000)	23.00

The blockage assessment indicates that the culvert at chainage 300B can tolerate up to 25% blockage without any noticeable adverse effects on upstream water levels. At blockages greater than 25%, upstream water levels would increase. At 40% blockage the water level upstream of the culvert could increase up to 0.5 m. The water level would still be maintained 440 mm below the road level. This increase could create a backwater effect up the Awakeri Wetlands with water levels approximately 0.1 m below the road level at chainage 700B (the MoE crossing). This is considered acceptable as no significant flooding of floor levels is anticipated at this level of blockage.

The blockage assessment indicates that the culvert at Chainage 700B can tolerate up to 40% blockage of the inlet area without any adverse effect on upstream water levels.

Blockage of up to 40% is considered unlikely for culverts of this size based on the likely type of debris that may be floating down the corridor during large storm events such as wooden pallets, logs, mattresses, containers, shrink-wrap or car bodies.

10.3 Structural design requirements

As discussed above, the supplier shall provide a structural design for the culverts.

The structural design should be in accordance, but not limited to the requirements in drawings 51-33411-S001-S002 and Table 20 below.

Table 20 Culvert design requirements

Feature	Note
Culvert internal dimensions	1.5 m (H) x 2 m (W) twin box culverts
Design working life	100 years
Exposure classification	XA2
Minimum concrete cover to reinforcement	Minimum 65 mm
Minimum concrete strength	50 MPa
Concrete specification	Refer to B610 Concrete Construction
Importance level	2
Site subsoil class for seismic design	D
Surcharge loads on proposed ground level	HN-HO-72 traffic loads
Longitudinal post-tensioning	Longitudinal post-tensioning cables are to be provided within the ducts in the corners of the culverts as per the Contact Drawings.
Cross-bolting of twin culvert units	Each twin culvert unit shall be bolted to the adjacent unit as per the drawings.

10.4 Geotechnical design parameters for culverts

A typical design philosophy for the culverts has been chosen to provide consistency throughout the corridor and ease of construction, maintenance and aesthetics.

Ground support and fill

GAP65 granular hardfill is proposed for underneath the culverts. Two layers of TX160 Geogrid are typically proposed within the GAP 65 layer to create a stiffened raft. This will reduce the potential differential settlement of the culvert units.

GAP65 granular hardfill is also proposed for around the sides of the culverts and above the culverts as backfill.

Geotechnical unit weight parameters

Given the variable nature of the geotechnical testing results, a range of unit weights have been considered for the existing peat soil for each fill scenario; 11 kN/m^3 and 13 kN/m^3, respectively.

The assumed unit weight parameters for other materials are outlined in Table 21 below.

Table 21 Material unit weight parameters

Material	Unit weight (kN/m^3)
GAP65 fill	20
Concrete	25
Water	10

10.5 Settlement assessment

A settlement analysis for the culvert has been undertaken based on a range of soil parameters and site conditions to determine the likely risk of settlement issues during construction and long-term post construction.

Groundwater level

Two possible groundwater level assumptions have been considered to provide a long-term scenario estimate and a construction scenario estimate:

Long-term scenario: assumes the weirs in the Awakeri Wetlands are operating effectively and the groundwater level is maintained at the weir level.

Construction scenario: The water level in the culvert is empty and the groundwater is at the invert level of the culvert.

Settlement predictions

The calculations are provided in Appendix D and summarised in Table 22.

Table 22 Predicted settlement

Scenario	Possible range of settlement (mm)	
	Culvert CH300B	Culvert CH700B
During construction	80-110	100-125
Long-term settlement Design Scenario	40-75	70-90
Long-term settlement Worst case	55-90	75-100

Based on the settlement assessment discussed above, it is recognised that some ground settlement will occur as a result of installing and operating the culvert. This is likely to occur mostly in the short term (during construction), where the dewatering and excavation of the existing peat soils will cause the most settlement. During the construction stage, the settlement should be closely monitored and any difference in the final levels can be made good by increasing the level of backfill to reinstate the affected area back up to the design level.

In the long-term, further minor settlement is also likely to occur. This is not expected to be a significant issue as the effect on hydraulics of the channel (if the invert levels drops by 100-200 mm) is minimal, and there are no services currently proposed above or below the culverts. Future services crossing above or below the culvert could include pipes for wastewater, water, power ducts, fibre and a road. These services will need to be designed to tolerate the predicted settlement of each of the culverts.

10.6 Buoyancy assessment

A buoyancy assessment of the proposed culvert designs has been undertaken to ensure that the design will be sufficient to prevent floatation.

10.6.1 Assessment area

Buoyancy has been estimated by calculating the weight of water displaced by the culvert (uplift force) and the weight of the culvert structure itself (resisting downward force). The factor of safety against buoyancy is determined by dividing the resisting force (mass) by the uplift force. The culvert assessment area includes:

- The culvert structure itself.
- GAP65 between the top of the culvert and the assumed design surface level (cover).
- Any permanent water in the culvert (see groundwater level scenarios below).

The geotechnical unit weight parameters for water, concrete and backfill are as presented in Table 21.

10.6.2 Water level

Two possible water level scenarios have been considered to assess buoyancy in different worst case scenarios:

Flooding scenario: groundwater level is at the ground surface level and the water level in the culvert is at the permanent water level (800 mm above the culvert invert, as set by the downstream weir). Any water depth above ground surface level (during floods) does not create any greater buoyancy risk as the weight of the water cancels out the buoyancy force.

Poor operation scenario: groundwater level is at the top of the culvert and the culvert is empty i.e. the permanent water level in the culvert is at the invert level. This case assumes the downstream weir is ineffective at maintaining a permanent water level within the culvert. This is unlikely to occur during wet weather / flood events, therefore the backfill above the culvert has been considered dry / above the groundwater level for this scenario.

10.6.3 Applied factors of safety

Construction soils can be variable and it is good practice to apply a factor of safety (FoS) to decrease the downward force of backfill. Generally, if the weight of the structure is the primary force resisting flotation, then a FoS of 1.0 is adequate. If friction or cohesion of the backfill are the primary forces resisting floatation, then it would be appropriate apply a safety factor to account for the variability of the soil properties. Therefore, the following factors of safety have been applied for the following scenarios:

Flooding scenario: FoS of 1.0. The backfill above the culvert is saturated so the resisting force is largely provided by the culvert structure rather than the backfill. Friction and cohesion of the soil has been ignored and only the self-weight of the backfill has been considered. Therefore, the backfill does not need an additional factor of safety.

Poor operation scenario: FoS of 0.9. The dry backfill above the culvert exerts a significant downward force compared with the weight of the concrete culvert. Given the variability of the soil, it is appropriate to multiply the backfill downward force by a factor of safety, which essentially gives a more conservative estimate of the total downward force.

10.6.4 Buoyancy predictions

Buoyancy along the length of the culvert has been calculated at 2 m intervals. The units have been considered as individual unconnected units, which is the conservative scenario given that they are expected to be tensioned together. In all cases, the culvert structure (including backfill) is sufficiently weighted to prevent floatation. The buoyancy factor of safety predicted for the different water level scenarios is presented in Table 23 below:

Table 23 Predicted buoyancy factor of safety

Design scenarios	Buoyancy Factor of Safety	
	Culvert CH300B	Culvert CH700B
Flooding – groundwater is at ground level and the water level in the culvert is at the permanent water level.	1.8-2.2	1.4-1.5
Poor operation – groundwater level is at the top of the culvert and the culvert is empty.	1.4-1.9	1.2-1.4

An additional 'worst case' scenario has been considered as a sensitivity check which assumes groundwater is at the ground level and the culvert is empty. This provides a minimum FoS greater than 1.0 which is considered acceptable, hence the floatation risk for this culvert is low. This scenario could be possible during construction when the water in the channel is being pumped.

This assessment is conservative as friction and cohesion of the surrounding soils is ignored.

Refer to Appendix D for the buoyancy assessment.

11. Paths

Stage 1 of the Awakeri Wetlands includes approximately 1,300 m of shared path within the works area. Paths within the Stage 1 works area will be constructed as part of this stage and are discussed in the sections below.

11.1 Path alignment and levels

The alignment and levels of the paths were designed and provided by Auckland Council. The alignment and levels were incorporated into the contour design of the Awakeri Wetlands by GHD. Slight adjustments were made where required and these have been confirmed by Auckland Council.

The resulting path alignments are available on drawings 51-33411-C111-C117 and long sections on drawings 51-33411-C151-C161.

Details of the paths are shown on drawing 51-33411-C217.

11.2 Path details

11.2.1 Typical section

The width of the paths is typically 2.5 m of formalised / paved area, within a 4 m wide corridor that slopes at 4% towards the low flow channel to provide natural drainage to the channel.

The paths generally consist of 100 mm thick concrete, with a 100 mm thick layer of Geoweb filled with 20/7 drainage metal underlying the concrete slab. This detail allows groundwater which is expected to seep out of the upstream slope to pass beneath the concrete footpath. This will minimise staining of the paths and minimise build up of slime and debris on the paths.

11.2.2 Drainage

The paths are generally cut into the slope batters of the channel, and therefore will potentially have surface water and/or groundwater flowing towards them from the channel batters. The flow rate is generally expected to be low. A 750 mm wide shallow drainage channel is proposed along the upstream side of the footpath to capture surface water from small rainfall events and any groundwater that seeps out of the upstream slope. This will minimise staining and slippage as discussed above.

The drainage channel will comprise of river stones / pebbles restrained in Geoweb cells, which connect with the drainage metal beneath the footpath. This allows water to be collected in the drainage channel on the upstream side of the footpath where it can soak through the drainage metal and the perforated Geoweb cells as well as soaking into the ground.

11.2.3 Foundation support for the paths

As discussed above, a 100 mm thick layer of Geoweb filled with 20/7 drainage metal will underlie the concrete footpath. While 20/7 drainage metal is not typically relied on for its strength, the confinement provided by the Geoweb cells will provide suitable strength for supporting the footpath. A similar detail is used in the design of permeable pavements where storage of water within pavement base-course is required.

11.2.4 Concrete reinforcement

As discussed above, the footpaths will be 100 mm thick, 20 MPa concrete using sulphate resisting cement (SR type). There is high risk of cracking to the footpaths due to the soft ground and potential settlement and movement of the ground underneath. To mitigate this risk, the concrete is proposed to have macro-synthetic fibre reinforcement.

Synthetic fibre reinforcement, unlike steel, is resistant to low pH / acidic conditions, which is present in the groundwater and soils of the site. Using fibre reinforcing is also expected to provide reduced construction timeframes compared to standard steel mesh reinforcing. Therefore, providing an overall saving in cost, time and risk.

The proposed application of macro-synthetic reinforcement is proposed as 3.0 kg/m^3 of concrete, but this will depend on the product of fibre reinforcement that the Contractor selects, and therefore is subject to the manufacturers requirements. The 3.0 kg/m^3 is based on Figure 17 below.

	Mesh type (all 150mm centres) standard wire sizes			
	668	**665**	**500E**	**663**
	Typical application Residential driveway or parking area, footpaths, etc.	Typical application Heavy duty residential driveway, residential slab on grade (single storey) or light commercial.	Typical application Residential slab on grade (compliance with code)	Typical application Commercial or industrial warehouse or loading area (external)
Slab Thickness	Forta Ferro dosage (kg per cubic metre of concrete)			
100mm	2.5kg/m^3	3.0kg/m^3	3.5kg/m^3	4.5kg/m^3
125mm	2.0kg/m^3	2.5kg/m^3	3.0kg/m^3	3.5kg/m^3
150mm	2.0kg/m^3	2.0kg/m^3	2.5kg/m^3	3.0kg/m^3

Figure 17 Fibre reinforcement dosage (http://fbsltd.co.nz)

11.2.5 Control joints

Control joints will be required as per Auckland Transports Code of Practice. It is expected that these will be formed through a hit and miss pouring methodology of the footpath, but could be achieved in other ways such as saw cutting. The contractor will confirm the proposed methodology.

11.2.6 Surfacing

Two different paving types are proposed as per the Landscape Plan:

1. Stevensons 'Riviera' exposed aggregate concrete with Peter Fell 468 oxide added to the mix.

2. Stevensons 'Harvest' exposed aggregate concrete with 5% black oxide added to the mix.

These paving types have been provided by Auckland Council and the locations for use are specified in the Takanini Cascades General Arrangement Plans – Hardworks, drawings L8102 – L8117 (Auckland Council, 2017).

An F5E exposed aggregate surfacing is proposed for the finishing of these pavement types.

This is in line with the curatorial framework considerations that were put together by the Auckland Council landscape designer, and Iwi representatives. The curatorial framework requests that the footpaths acknowledge the 'red earth' definition of the name Papakura.

Stevensons 'Riviera' aggregate gives a 'red earth' appearance as shown in Figure 18 below.

Figure 18 Stevensons Riviera exposed aggregate concrete

This style of surfacing has the following benefits:

- Aligns with curatorial framework.
- The footpath will be subject to channel flow and therefore is at risk of staining, the expected staining colour would be orange / brown, and hence would be less noticeable with this finishing.
- Given that the area is wet and is subject to flood flows, there is a risk of the pavement becoming slippery. An exposed aggregate finishing will help to mitigate this by providing grip, however maintenance / cleaning of the footpath will be the primary mitigation for this risk.
- Suitable for walking and cycling.

11.3 Taupo ash layer

Ash layers are present throughout the soil in Takanini. The level of the Taupo ash layer is proposed to be marked where possible, however this has not yet been incorporated into the design. This requirement should be considered by the Contractor to determine what the most effective way of marking this within the works. This should be agreed with Auckland Council and the Engineer.

12. Boardwalks

Stage 1 of the Awakeri Wetlands includes approximately 290 m of boardwalks within the works area. Boardwalks within the Stage 1 works area will be constructed as part of this stage and are discussed in the sections below.

12.1 Boardwalk alignment

The alignment of boardwalks were designed and provided by Auckland Council. The alignments were incorporated into the contour design of the Awakeri Wetlands by GHD. Slight adjustments were made where required which were checked by Auckland Council.

The resulting boardwalk alignments are available on drawings 51-33411-C261-C264.

Details of the boardwalks are shown drawings 51-33411-C265-C266.

12.2 Boardwalk details

12.2.1 Curatorial framework

A curatorial framework has been provided by Auckland Council which collates the aspiration of Mana Whenua, the AC Landscaping Team and other stakeholders.

Key points for the boardwalks are:

- Boardwalk construction to have environmental sensitivity design and prioritise a 'light touch' on the landscape.
- The timber boardwalk decking will acknowledge the 'red earth' meaning of the name Papakura.
- Timber used is to be environmentally sensitive. Local or native timber is to be prioritised.

12.2.2 Typical section

The boardwalks are typically 2.23 m wide between the kerbs, but with a total width of the decking of 2.7 m. The structure consists of timber kerbs, timber decking, timber joists, timber bearers and timber posts which attach to a concrete footing to spread the load of the boardwalk onto the soft peat ground below.

12.2.3 Decking

The decking typically consists of 45 mm x 140 mm timber decking panels with each panel 2.7 m long. Hardwood is proposed for the decking that is resistant to low pH and frequent wetting.

The Hardwood Jarrah is proposed for the decking timber. Native timbers to NZ were considered as per the curatorial framework, however none of the native timbers were suitable to achieve sufficient durability or the 'red earth' look. While not native to New Zealand Jarrah presents the following advantages:

- Very durable.
- Dark red colour.
- Resistant to acid / low pH.

Jarrah is available in 150 x 50 nominal size, which is suitable for the decking timber. This same species is proposed for the kerb and packers underneath the kerbs.

Figure 19 shows Jarrah being used in a marine setting for a waterfront platform.

Figure 19 Jarrah waterfront platform (http://www.fqtimber.com)

12.2.4 Support structure

The structure beneath the decking consists of:

- 190 mm x 90 mm joists to support the decking.
- 190 mm x 90 mm bearers to support the joists.
- The bearers are bolted onto 200 mm SED posts.
- The posts are connected to a concrete footing below ground.

The joists, bearers and posts are proposed as treated pine, in accordance with the specification and drawings. The treatment requirements for each component is outlined in Table 24 below.

Table 24 Timber boardwalk components

Component	Location	Material	Grade	Treatment	Species
Decking	Frequently in contact with water	Hardwood	F11	H4	Jarrah
Joists, bearers	In contact with water (low pH)	Sawn timber	SG8(wet) / G8	H6	Pinus Radiata
Posts	In contact with aggressive soils and water	SED posts	NZS 3605	H6	Pinus Radiata

12.2.5 Footing

A 400 mm thick concrete footing is proposed to support the boardwalk and spread loads onto the peat soils below.

The 400 mm thick concrete slab will typically be 3.6 m x 3.0 m for four posts, or 1.8 m x 3.0 m slab for two posts at the ends of the boardwalk.

This footing has been designed to spread the load across a sufficient surface area to accommodate the strength of the peat below.

The concrete footing shall meet the requirements outlined in Table 25 below.

Table 25 Boardwalk footing requirements

Component	Specification
Concrete type	Sulphate Resistant (SR type)
Strength	50 MPa
Reinforcement	668 steel mesh
Exposure classification	XA-2
Cover	Min 65 mm

12.2.6 Foundation

The concrete footing is proposed to be laid a 150 mm thick layer of flowable fill poured directly onto the excavated peat surface.

The Bidim+Geogrid composite material will tie into the foundation to mitigate scour around the footings.

13. Safety in design

Safety has been considered throughout the design process. Each component of the Awakeri Wetlands has been designed with safety as a key consideration.

The following section provides a summary of the safety considerations for the channel design.

13.1 Low flow channel

The low flow channel has been designed to discourage entry by the public through dense wetland planting on the edges of the water body. If someone were to enter the low flow channel, the key features below have been incorporated into the design to reduce safety risks:

- Flow velocity very low.
- Shallow depth maintained by weirs (0.5-1.2 m).
- 2:1 side slopes lined with granular material. As such, the ability for someone to walk up this submerged slope without slipping is enhanced.
- Wetland bench of varying width provides warning of imposing deep water. The wetland bench also acts as a safety bench to assist anyone climbing out of the channel and reduces the chance of people falling into the deeper section.
- Riparian margin and wetland planting creates barrier to entry.

13.2 Weirs

The water level drop between weirs varies from 0.18 m - 0.45 m. This drop is into 800 mm deep water. This is a relatively small drop and a safe falling height, however given that there is water either side of the weirs, there is an associated safety risk. Key safety features and considerations for the weirs include:

- Small drop height between weirs.
- Wetland bench and planting on both sides of the weirs discourages access to weirs by public.
- Timber capping provides a lip at the weir surface that could be held on to if required, likewise with the fish passage structure.
- Low flow channel safety features on both sides of the weir as described in Section 13.1 above.

13.3 Paths

The paths within the channel provide a key amenity feature for the public. As with any public asset, there are some associated risks as outlined below:

- **Falls and trips:** The shared paths will be standard surfacing, that would be familiar to most users, hence minimising fall and trip hazards. This consideration should form part of the operation and maintenance plan to allow frequent maintenance and clearing of the paths, as if plant debris, dirt or slime is allowed to build up on the paths, then the risk of falls and trips would be increased. A gravel drainage strip on the upstream side of the paths is proposed to minimise groundwater seepage or surface flows from frequently flowing across the paths. This will reduce the chance of slime build up and slippery paths.
- **Sight:** Generally, sight distances should not be an issue with the alignment of the paths, as provided by Auckland Council, given that the proposed planting is generally less than

1 m tall grasses and shrubs with some largely amenity trees. Furthermore the alignment of the paths and the nature of the environment is expected to make cyclists ride cautiously and be aware of their surroundings, given the natural environment, proximity to open water, vertical and horizontal curvature of paths, planting and reduced width of the shared path.

- **Proximity to water:** The path alignments occasionally run alongside and over open water. The wetland bench and planting provides a shallow depth of water and a natural barrier to the deeper water which would restrict anyone who veers off the paths from falling into the deeper water.
- **Flooding:** The paths levels are designed above the 50% AEP water level as per ATCOP. Signage is proposed to warn the public of flooding. In these circumstances alternative routes are available which bypass flooded areas. For larger events where the paths are flooded, alternative routes will be available to give access throughout the Awakeri Wetlands alignment. Furthermore, flow velocities in the channel are low, and therefore the safety risks associated with flooding of the paths is considered low.

13.4 Crime Prevention through Environmental Design (CPTED)

The urban and landscape designer has carried out a CPTED analysis as part of a separate report for the TSCC.

13.5 Culverts

13.5.1 Northern Channel Chainage 300 Crossing

This culvert is approximately 18 m long and will have a permanent water depth of 0.8 m within the 2 m high box culvert as a continuation of the low flow channel. The permanent water body and the planting at each end of the culvert will discourage interaction with the culvert by the public.

13.5.2 Northern Channel Chainage 700 Crossing

This culvert is approximately 10 m long and will have a permanent water depth of 0.5 m within the 2 m high box culvert as a continuation of the low flow channel. The permanent water body and the planting at each end of the culvert will discourage interaction with the culvert by the public.

13.5.3 Safety in design features

Key safety in design features and considerations for the culverts are:

- Entry into the culvert is discouraged by planting in the channel at each end and a permanent water level that is continuous between the channel and the culvert.
- Low velocity and low turbulence during low flow conditions.
- Shallow depth of water within the culvert.
- Fencing mitigates falls from the top of the headwalls. Planting behind the headwalls and fencing reduces the risk of anyone accessing the top of the headwall and being in a position where falling is possible.
- No inlet or outlet grills to eliminate risk of people getting stuck against them.