Flood Assessment Report

JOB NUMBER: 23-1883

Waikato Greenfield Racing Hub

PROJECT

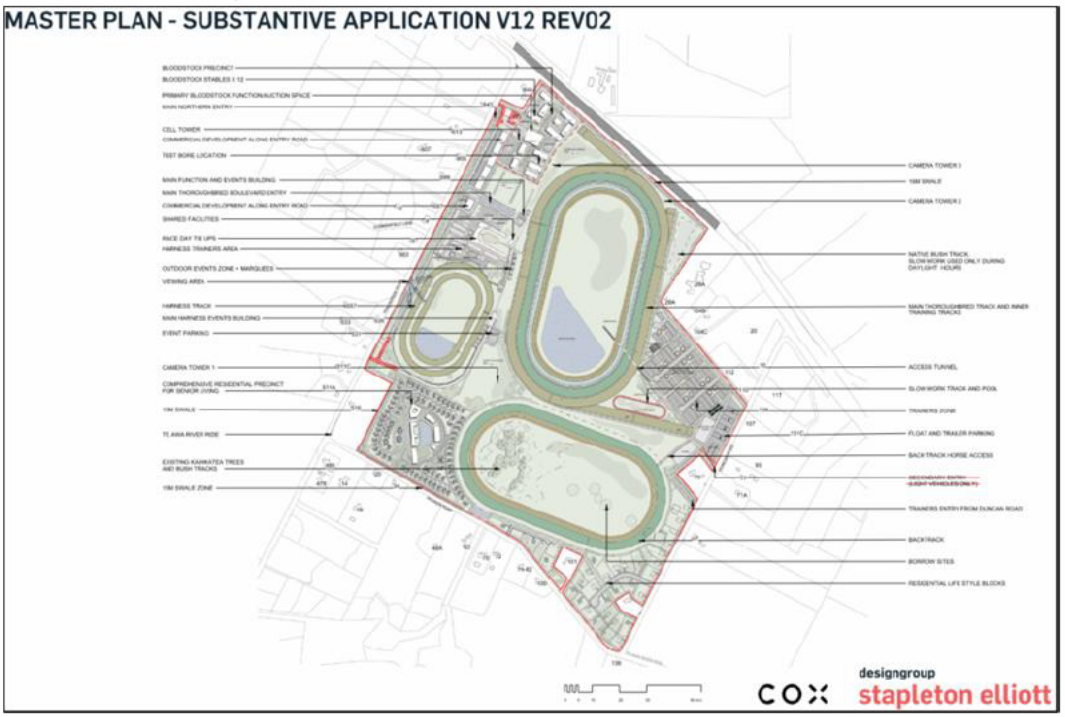
Waikato Thoroughbred Racing Inc

CLIENT

SUBSTANTIVE APPLICATION – REV 7
1 October 2026

Flood Model Assessment Report

23-1883 **Waikato Greenfield Racing Hub** **Pencarrow Road, Waikato**



Prepared for: Waikato Thoroughbred Racing Inc **Project no:** 23-1883

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Revision	Date	Status	Authorised by:
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1 INTRODUCTION AND SCOPE

1.1 Introduction

Waikato Thoroughbred Racing Inc has engaged BCD Group to complete a Substantive Application work for the proposed Waikato Thoroughbred Racing Greenfield Site – Pencarrow Road. Part of this involves flood modelling of the site to determine both existing flooding and potential impacts of the proposed development, with the intent of demonstrating if and/or how the site could be developed for the purpose of this project while managing and mitigating any effects to the upstream and downstream environments.

Report objectives are to:

- Identify flooding risks on the developed site, including:
    - Providing an estimation of top of flood level across the site to aid with potential finished flood levels during large storm events.
- Identify if the proposed stormwater swales have sufficient capacity to handle any modelled flooding across the site.
    - Providing an estimation of flows in the site's existing agricultural drains during large storm events.
    - Provide approximate capacities for the culverts within the site.
- Identify any upstream or downstream impacts resulting from the proposed development and how these impacts might be addressed/mitigated.
- Provide potential solutions to resolve flooding on site, and upstream and downstream effects.

1.2 Statement of Experience

I am a Chartered Professional Engineer (NZ) in both Civil and Water Engineering. I have 12 years working experience in engineering consulting in New Zealand and have led the design on land development projects, public infrastructure and private developments. Carrying out civil design, reviewing civil design work, managing construction works have all been the primary components of my role. I also lecture and teach Civil Engineering at the University of Waikato School of Engineering and am a Producer Statement Author with Auckland Council. I have visited the subject site and am familiar with the site and the surrounding environment; and understand the ethical obligation as a Chartered Professional Engineer completing this work.

1.3 Fast Track Referral Feeback

BCD had initially carried out flood modelling of the existing site as part of the Fast Track Referral application.

Multiple parties (Hamilton City Council, Waikato Regional Council, Department of Conservation, Hamilton Aiprot, Heritage New Zealand, Waipa District Council, Waikato District Council, and Waikato Tainui) provided feedback on the Fast Track Referral Application,

The feedback of Hamilton City Council and Waikato Regional Council has specific relevance to the stormwater modelling. The feedback and responses are set out below

1.3.1 Hamilton City Council

- Existing assessment identifies catchments, quantifies inflows and outflows, and outlines an approach at achieving hydraulic neutrality.
- Detailed Design will need to confirm
    - How proposed stormwater infrastructure will respond to the final site layout
        - How the site's proposed terrain response is addressed in Section 4.2,
        - How the site's proposed stormwater infrastructure will respond and mitigate any potential downstream is address in Section 5.

- Primary and secondary overland flow paths
    - The performance of the primary overland flow paths (swales) is addressed in Section 4.2.1 and Section 4.2.2.
- Storage capacity
    - The amount of additional stormwater that requires storage to achieve hydraulic neutrality for each catchment is address in section 4.3.

- There will need to be ongoing consideration of the downstream effects and avoidance of any impacts on the site.
    - Section 4.3 shows that the design is considering the upstream and downstream impacts.
    - This requirement for the consideration to be ongoing is address in in Section 8.

1.3.2 Waikato Regional Council

- Water courses (drains) are being infilled. Although of low ecological value, they do provide habitat value and connectivity to downstream environments
    - The proposed design methodology will use swales to replace the habitat lost and the outlets for the proposed swales will match those of the existing drains (addressed in Section 2.4). Flows at these outlets are maintained even in very small storm events, such as a 12 EY event (addressed in Section 4.3)
- The site includes a wetland, within the Kahikatea Stand. Although a remnant of high ecological value, the following policies and standards apply:
    - Policy LF-P2 in the Waikato Region Policy: Ensure that the outstanding values of a fresh water body that result in that water body being identified as an outstanding fresh water body, and the significant values or wetlands, are protected and where appropriate enhanced.
    - Policy 6 of the Nation Policy Statement for Fesh Water Management: There is no further loss of extent of natural wetlands, their values are protected, and their restoration is promoted.
    - Part 3, Subpart 1 (Natural Inland Wetlands) of the National Environmental Standards for Freshwater (NES-F)
        - Clause 52: Where the taking, use, damming, of diversion of water outside, but within a 100 m setback, from a natural inland wetland is a non-complying activity if it – results, or is likely to result, in the complete or partial drainage of all or part of a natural wetland
    - The wetland is discussed in Section 3.3.1.2 and its pre and post development depths are discussed in Section 4.2.1.3

2 PROJECT BACKGROUND AND DESCRIPTION

2.1 Project Description

The purpose of the project is to create a unique, world class racing hub designed for horse training, racing and other equine related activities, while bringing the expertise and strength of the local racing fraternity together in a centralised location.

This enables the local racing industry to be more streamlined, competitive, sustainable and future focused while bringing potential international investment and creating a 'destination' for horse racing in New Zealand, also increasing tourism opportunities for the wider region.

A key driver behind the proposed greenfield equine hub and racecourses is enabling the consolidation of four separate racecourse facilities (Te Rapa, Waipa and Cambridge thoroughbred courses, and the Cambridge harness track).

These facilities duplicate assets and resources and, given their current condition require significant levels of upgrades and investment to provide fit-for-purpose facilities that meet the higher standards of the modern-day racecourse experience. The retirement of these areas also frees up significant tracts of land within existing urban areas for future development, increasing housing supply.

To support the development's financial viability and enhance the site's long-term vibrancy as a racing, entertainment, commercial and community precinct, the proposal includes a range of complementary activities on the remaining land. These include equine support services, rural residential housing, a retirement living community, a village centre and a bloodstock sales precinct.

2.2 Site Description

The site is located 6 km south-east of Hamilton and 4.5 km west of Cambridge, in the Waikato District. The site is rural pasture used for cattle raising, with the occasional neighbouring lifestyle block. The site is bounded to its northeast by SH1, to its southeast by Duncan Road, to its southwest by Hooker Road, and its northeast by Pencarrow Road.



Figure 1: Site Location

Based on BCD Survey data (17th of November 2025), the existing site is crossed by a series of drainage channels, most of which collect into either a main channel running westward through the site centre or collect into a drain running alongside State Highway 1 (SH1). The main existing central drain begins on the site, near to Duncan Road. After crossing the site, it runs under Pencarrow Road via a 200mm culvert and continues to flow westward (through a series of low-lying hills) before turning north, connecting with the Mangaonua Stream.



Figure 2: Site Location - zoomed in

The site appears quite flat, with only 7 m of elevation change from its southern high points (~58 m RL) and northern low points (~51 m RL), resulting in an average ~0.4% gradient across the 2 km distance. However, the site does appear to have some quite notable ridgelines, splitting the site into four catchments that the assessment must consider.

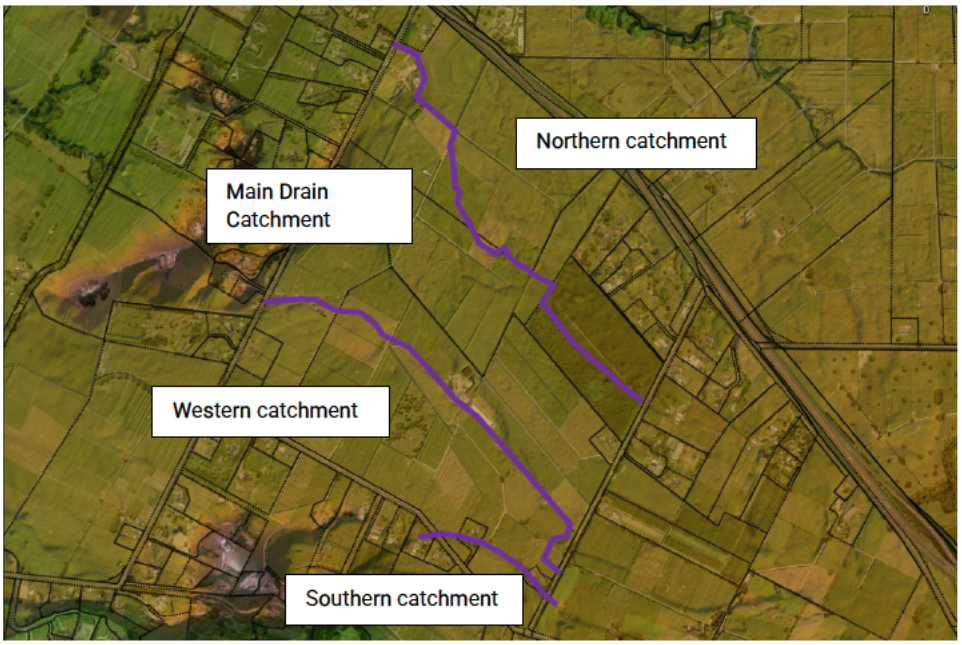


Figure 3: Site Location - ridgelines across the site (purple) split the site

These catchments are:

- Northern Catchment – flows into the drain running along the southern side of SH1.
    - Catchment area upstream of site: 43 Ha
    - Catchment area within the site: 40 Ha
- Main Drain Catchment – Discussed above, this is the main drain running through the site
    - Catchment area upstream of site: 54 Ha
    - Catchment area within the site: 79 Ha

- Western Catchment – falls to a sump/culvert at the sites western corner, after the culvert flows west to Waikato River.
    - Catchment area upstream of site: 7 Ha (very little inflow reaches the site due to Duncan Road. The main existing inflow to this sub catchment is an offshoot from the main Catchment that occurs within the site.
    - Catchment area within the site: 41 Ha
- Southern Catchment – Small catchment at the southern corner of the site, flows south to the Waikato River
    - Catchment area upstream of site: 7 Ha
    - Catchment area within the site 4 Ha

Each of these catchments extends upstream from the site, with flow originating from across Duncan Road (flows to the site's north, west, and south drain away from the site and towards either the Mangaonua Stream or the Waikato River). However, each of the catchments appears short, with Cambridge Road acting as a barrier for flows from the eastern direction.

2.3 Consultation

BCD conducted flood modelling in accordance with Waikato Stormwater Runoff Modelling Guidelines and Regional Infrastructure Technical Specifications Version 2 using GEO-HECRAS software.

The site is not located in any flood hazard zone under the Waikato District Council Land Resource maps. Nor is the site subjected to any River Flooding or Flood management.

BCD also consulted Earth Science NZ 'Flood hazard across Aotearoa New Zealand' (1% AEP rainfall events) and used it to help validate modelling results. The Earth Science NZ data shows that depth of the flooding on site remains below 1m.

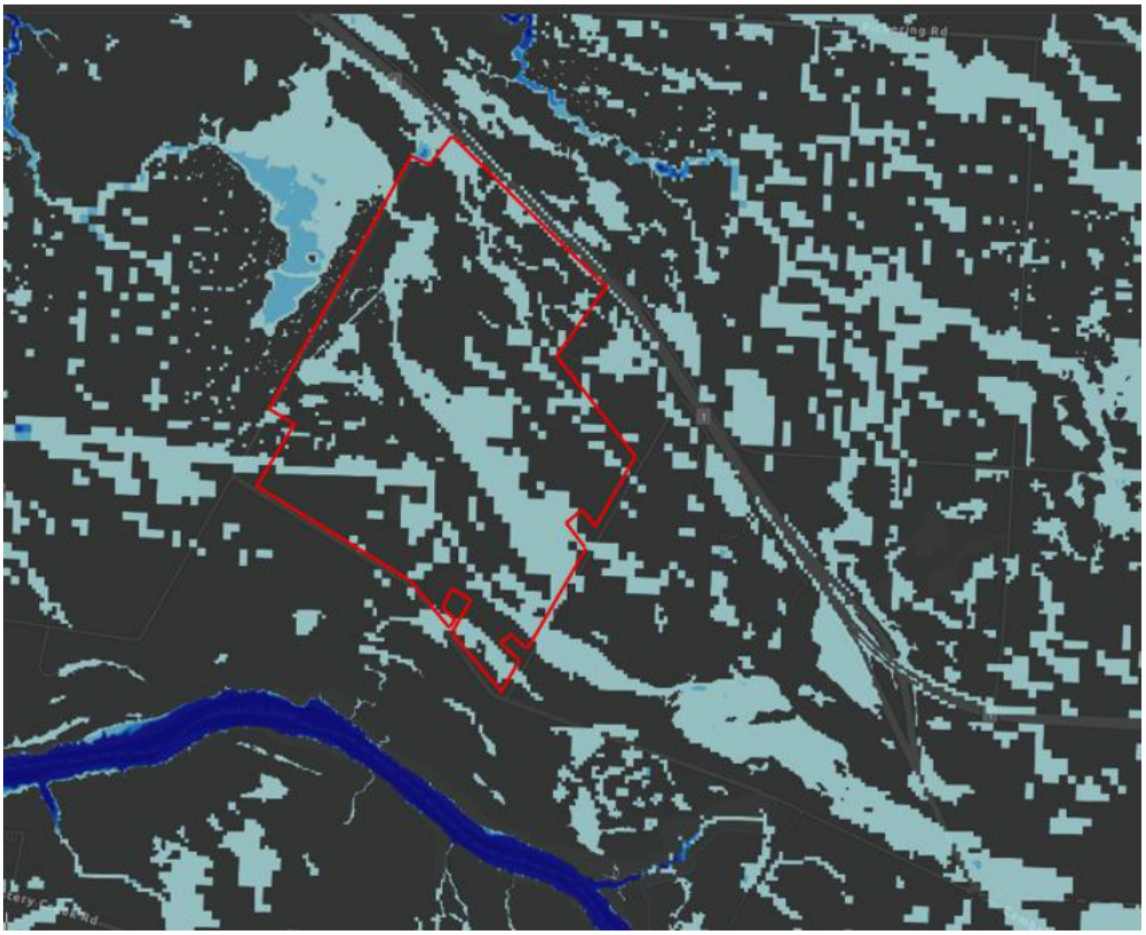


Figure 4: NIWA Flood Hazard Results

2.4 Design Methodology

The design methodology for on-site development and stormwater management will be coordinated with the Masterplan, the architect, and with the three waters design. The site's proposed development design approach will require:

- Infill of part or all existing drains
- Setting FFL platforms that are above the 1% AEP flood depths for the location,
- Ensuring building platforms do not clash with or obstruct overland flow.
- Inclusions of new swales, ponds, and pipework to convey stormwater under both the 10% and 1% AEP events (and lesser events). The new stormwater network will account for the proposed site layout, roads, buildings etc. The new swales will also act to help replace the habitat lost be the removal of the existing drains.
- Attenuation ponds to be sufficiently sized to ensure that the proposed development runoff matches the existing runoff. This hydraulic neutrality should be ensured for both individual catchments and the site in its totality. As part of this hydraulic neutrality:
    - Peak upstream and downstream flows are to remain unchanged.
    - There is to be no impact on the roads, cycle way, and SH1.

The proposed master plan shows that the proposed landscaping will include multiple trees along the roadways, in parallel with the new swales. The trees will not be located within the swales and will be planted either with sufficient distance to ensure their roots do not interfere with the swales or with some form of root barrier. As the proposed

swales will be designed to have sufficient capacity to covey the 1% AEP within its boundaries, the trees will not obstruct the overland flows.

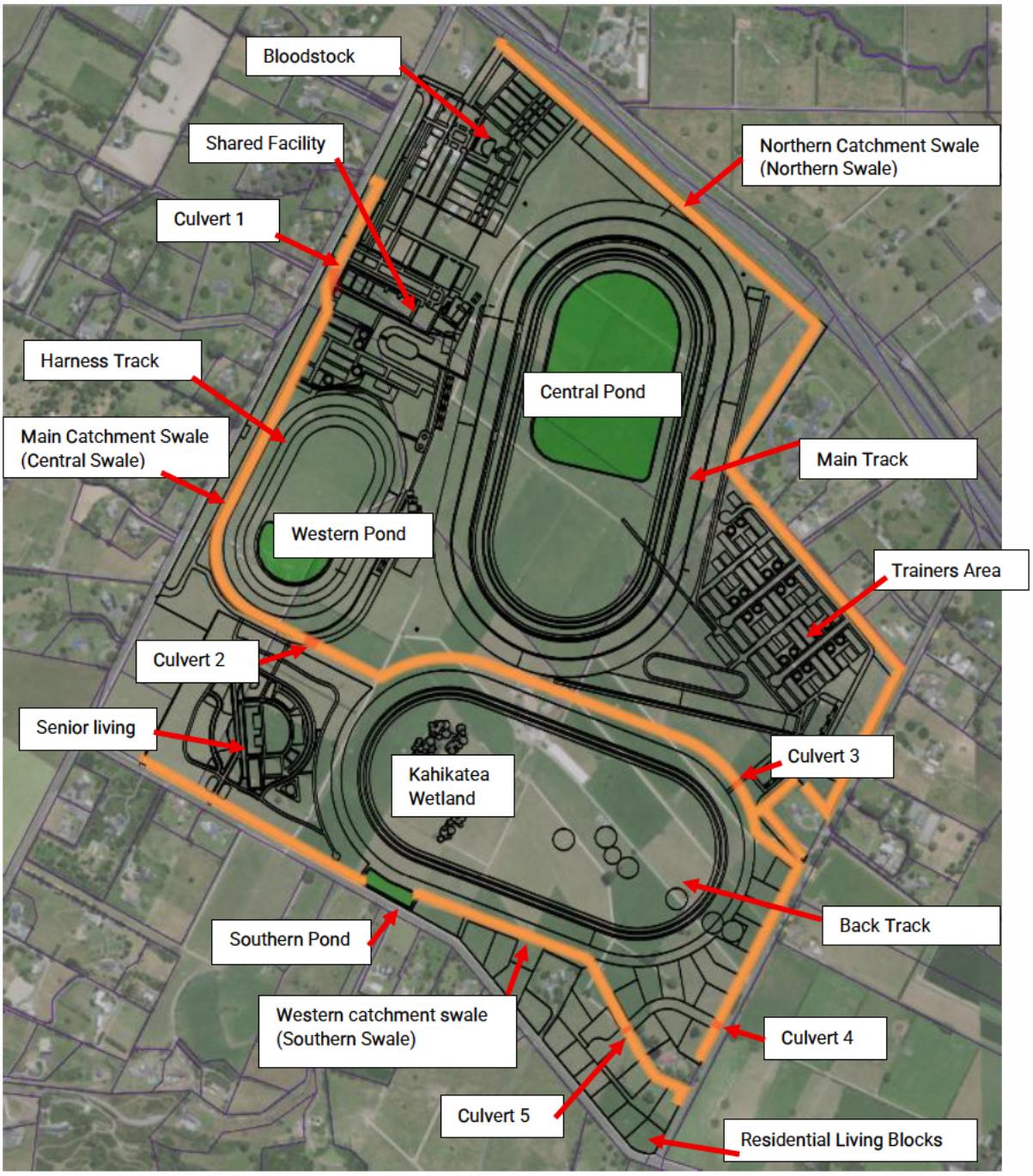


Figure 5: Proposed developed site with swales (orange) and attenuation ponds (green)

3 GEO-HECRAS STORMWATER MODEL

3.1 Summary

BCD built the GEO-HECRAS stormwater models based on Waikato Stormwater Runoff Modelling Guideline and Regional Infrastructure Technical Specifications and using the following inputs:

- A design surface of the proposed development.

- Waikato's 2021 1 m LIDAR, NZVD2016 was used to create a surface for the upstream catchment
- Topographical survey carried out by BCD on the 17th of November 2025 was adopted to create an existing surface within the subject site.
- Pre-development rainfalls are based on NIWA Historical 24-hour rainfall events (see Section 3.3.4)
- Infiltration and land use, based on if the area was Agricultural, Roadway, or Residential (see section 2.3.3)
- A model mesh size of 10 m^2 (30 m^2 was used initially to check the upstream catchment)

3.2 Catchment Check and Initial Modelling

Although Cambridge Road appeared to provide a barrier against upstream flow, this was not initially known for certain. It was possible that flow could pond on the eastern side of Cambridge Road before eventually overtopping and contributing to the runoff on site. This is what appears to happen under the NIWA flood hazard model; however, it is possible that, given the NIWA model's size, Cambridge Road elevation obstruction was not properly included within the model.

BCD used Waikato's 2021 1 m LIDAR, NZVD2016 (sourced from LINZ) and some high-level modelling to check the upstream catchment affecting the proposed site during storm events up to a 1% AEP event. The extents of this model were based on contours (derived from LIDAR) and included all areas that could potentially (if allowed to pond and overtop) contribute to the proposed site. The intent was, based on the initial modelling, to cut the modelled area down to only the area that was impacting the site during the 1% AEP.

The initial model was constructed using a 2D flow area with a hexagonal cell size of 30 m^2. This model included 2D break lines to ensure model properly recognised the main open channels, levies, and roads. As the initial model intent was to determine the catchment extent, rather than the results, 30 m^2 cells are acceptable.

The catchment check determined that, while stormwater did pond on the eastern side of Cambridge Road, the stormwater did not overtop with either volume or flow rates significant enough to impact the downstream catchment. Subsequent modelling set the upstream catchment boundary at Cambridge Road as a result.

3.3 Modelling Methodology

3.3.1 Model Extents

Initial modelling showed that was the upstream extents of the catchments impacting the site was Cambridge Road. The initial modelling also indicated the behaviour of the downstream catchments, allowing BCD to select appropriate extents for the model.

- Northward flows would run into the drain running along SH1 and be unable to immediately cross the state highway. They would follow the drain west until they reached Pickering Road, where they would flow through the underpass and overland towards the Mangaomapu Stream. BCD set the model boundary along the Mangaomapu Stream to ensure the impact between SH1 and Mangaomapu Stream could be analysed.
- Westward flows run across Pencarrow Road and through a series of rolling hills. These hills have a notable impact on the downstream catchment, as they act as a flow restriction on the drain, resulting in the area to the west of Pencarrow Road (including Summerfield Lane) becoming a floodplain. BCD set the Model boundary along these hills with a boundary condition on the drain immediately downstream of the hill. This ensured the impact of this floodplain was accounted for in the model.
- Southern flows run across Pencarrow Road and Hooker Road and drain smoothly towards the Waikato River. The model boundary and boundary conditions were set a little downstream of Pencarrow Road and Hooker Road to ensure the model accounted for any impacts to downstream flow.



Figure 6: Site Model Extent

3.3.2 Modelling Ground Surface and Surface Changes

3.3.2.1 Existing Model

For the existing model, BCD used Waikato's 2021 1 m LIDAR, NZVD2016 (sourced from LINZ) to create a surface for the upstream catchment, as with the catchment check model. The existing site model combined this LIDAR with a survey surface of the site, surveyed by BCD on the 17th of November 2025, which covered the proposed site area – the survey surface taking precedence over the LIDAR surface.

The survey also determined the Top-of-bank/Bottom-of-bank/channel centre lines for the on-site drains, providing more detail to these areas and allowing the model to use them as breaklines. Furthermore, the survey investigated the on-site and surrounding culverts. Most of the culverts were less than 600mm in diameter, allowing the analysis to assume that they were blocked for the purposes of the modelling; however, a culvert was found near the site's northern corner (on the SH1 drain) the diameter of which was confirmed to be 700mmØ by the survey. To account for flows through this culvert, the model assumed a 700 mm gap at this location.

3.3.2.2 Proposed Model

The proposed development's design surface included the three proposed racing tracks (the main track, back track, and harness track), attenuation ponds, proposed swales, and the site proposed roadways. Generally, the design surface was sloped towards the nearest swale. As the design is still being developed, the surface did not included localised surfacing or buildings.

The proposed swales will include five culverts, with approximate capacities to be determined as part of the analysis. As a results, the analysis included two proposed development models.

- Proposed Development - Open channels: the ground surface assumed the swales were completely open, allowing BCD to observe the unobstructed flow within each channel, allowing the analysis to determine the approximate capacities required for each culvert.
- Proposed Development – Blocked Channels; the ground surface assumed that the swales were completely blocked by the proposed roads (i.e. a blocked culvert). This allowed BCD to observed how the site might flood in the event of the culverts blocking.

The proposed development models were constructed similarly to the existing model, except with a design surface taking precedence over both the survey surface and the LIDAR. For the Proposed Development – Open Channels model, stamped geometry was used to continue the swales designed geometry through the proposed roadways.

The proposed design included swales and irrigation ponds as a means of stormwater management. Top-of-bank and bottom-of-bank centre lines were used as break lines to ensure the swales and ponds were modelled correctly. The tops of the tracks were also detailed. For the Proposed Development – Block Channels, the mode also used the road crests above the culverts as a break line to ensure the mesh did not skip over the road.

In regard to the wetlands, BCD notes that the Kahikatea Stand is currently dry and the existing surrounding drains actively drain away from the strand. Their removal should not damage the natural wet land or cause further loss.

The proposed development flood modelling has assumed the exclusion of stormwater mitigation and control structures, such as culverts, weirs, and piped connections between the swales and irrigation ponds.

To resolve this issue, the site's design includes a swale near 70 Duncan Road's southern boundary and a culvert connecting it to the Main Catchment Swale. Designs for this culvert will ensure it is too big to block.

3.3.3 Model 2D Flood Area

The model used hexagonal cell size of 10 m^2 and included 2D break lines to ensure the model properly recognised the important open channels, ponds, levies, tacks, and roads, as noted above)

3.3.4 Model Land Use and Infiltration

The model used both land use and infiltration layer to model the differing land uses within the catchment (such as roads, residential areas, and rural areas) would have on overland flow and infiltration. Curve number values, with the model uses to determine infiltration, were derived from the Waikato Stormwater Runoff Modelling Guideline (where available).

Some of the surfaces (Roadways and Parking) and (Agricultural and Track surfacing) are currently equivalent but have been made distinct to ensure their properties can be modified independently (if required).

The masterplan shows the landscape trees being planted with sufficient distance to not obstruct flow. The planting is also along a single line, meaning the trees are unlikely to reduce runoff significantly over grass land.

Due to the high-water table encountered on site, the development was considered to be poorly drained, with a soil type C and equivalent Curve Numbers assumed. The upstream catchment is noted to be well drain. As a result, the curve number for the upstream agricultural area was lower to 79.

Table 1: Land Use types, roughness's, and infiltrations – typical manning's n (from HEC_RASE 2D User's Manual) and Curve Number (from Waikato Stormwater Runoff Modelling Guideline (where available), Modelling Urban River catchment: A Case Study of Berop River – Tanjong Malim (where not)

Land Use Type	Manning's N	Imperviousness	Curve Number (Soil type C)	Initial Abstraction Ratio
Upstream Agricultural	0.045	0%	79	0.05
Agricultural	0.045	0%	86	0.05
Site Landscaping	0.030	0%	86	0.05
Roadway	0.016	100%	98	0.05
Open Water	0.035	100%	98	0.05
Commercial	0.020	90%	96.8	0.05
Paved parking lots, roofs, driveways	0.016	100%	98	0.05
1-8 Arce (town houses)	0.1	65%	93.8	0.05
Trainers Area	0.02	85%	96.8	0.05

1-2 Acre	0.05	25%	89	0.05
Track Surface 1	0.03	0%	86	0.05
Track Surface 2	0.03	0%	86	0.05
Track Surface 3	0.03	0%	86	0.05
Buildings	1.00	100%	98	N/A

Land Use	Given Land Use ID	Hydrological Soil Class			
		A	B	C	D
Existing Land Use					
Fruit Farm	1	61	70	77	80
Rubber	2	64	73	88	90
Mangrove Swamp	3	77	80	83	86
Road & Railways	4	90	90	90	90
Cleared Land	5	70	80	85	87
Urban Land	6	83	90	93	96
Oil Palm	7	50	66	80	87
Various Plants	8	62	70	78	81
Mix Farming	9	67	72	81	86
Rangeland	10	66	77	88	94
Shrubland	11	55	66	80	87
Pasture	12	61	77	85	91
Future Land Use					
Commercial	1	89	92	94	95
Educational	2	57	72	81	86
Flood Plain	3	39	61	74	80
Green Area	4	39	61	74	80
Industrial	5	81	88	91	93
Residential	6	77	85	90	92

Suggested Manning's n and Percent Impervious for C-CAP Land Cover

C-CAP Class	C-CAP Value	C-CAP Land Cover Type	Range of n (HEC-RAS 2D Manual)	Suggested Initial n	Percent Impervious
Nodata	0	N/A	N/A	N/A	N/A
Unclassified	1	N/A	N/A	N/A	N/A
Developed Land	2	Developed, High Intensity	0.12 - 0.20	0.15	90
	3	Developed, Medium Intensity	0.08 - 0.16	0.12	65
	4	Developed, Low Intensity	0.06 - 0.12	0.08	35
	5	Developed, Open Space	0.03 - 0.05	0.035	10
Agricultural Land	6	Cultivated Crops	0.020 - 0.05	0.05	0
	7	Pasture/Hay	0.025 - 0.06	0.045	0
Grassland	8	Grassland/Herbaceous	0.025 - 0.06	0.04	0
Forest Land	9	Deciduous Forest	0.10 - 0.20	0.10	0
	10	Evergreen Forest	0.08 - 0.16	0.15	0
	11	Mixed Forest	0.08 - 0.20	0.12	0
Scrub Land	12	Shrub/Scrub	0.07 - 0.16	0.08	0
Palustrine Forested Wetlands	13	Palustrine Forested Wetland	0.045 - 0.15	0.08	50
	14	Palustrine Scrub/Shrub Wetland	0.045 - 0.15	0.08	50
	15	Palustrine Emergent Wetland (Persistent)	0.05 - 0.065	0.06	75
Estuarine Wetlands	16	Estuarine Forested Wetland	0.045 - 0.15	0.08	50
	17	Estuarine Scrub/Shrub Wetland	0.045 - 0.15	0.08	50
	18	Estuarine Emergent Wetland	0.05 - 0.065	0.06	75
Barren Land	19	Unconsolidated Shore	0.023 - 0.03	0.03	0
	20	Barren Land (Rock/Sand/Clay)	0.023 - 0.03	0.03	0
	24	Tundra	0.023 - 0.03	0.03	0
	25	Perennial Ice/Snow	0.023 - 0.03	0.03	50
Water and Submerged Lands	21	Open Water	0.025 - 0.05	0.035	100
	22	Palustrine Aquatic Bed	0.025 - 0.05	0.035	100
	23	Estuarine Aquatic Bed	0.025 - 0.05	0.035	100

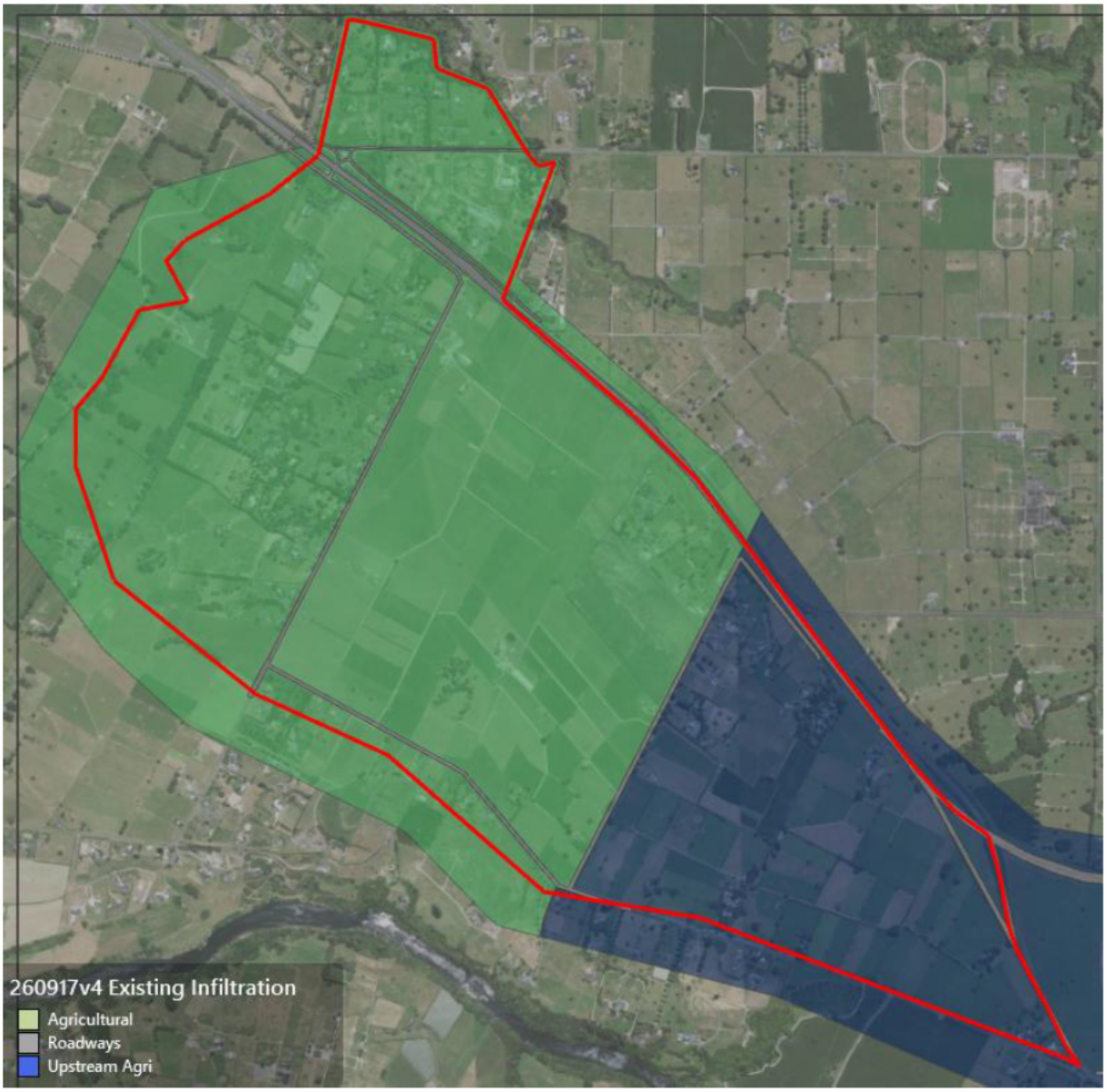


Figure 7: Existing Land Use and Infiltration (model area in red)



Figure 8: Proposed Land use and infiltration (model area in red)

3.3.5 Model Rainfall

BCD simulated rain based on the nested storm event detailed in the Waikato Storm Runoff Modelling Guidelines. These are based on a 24-hour rainfall intensity.

To account for the impacts of climate change, the post development flood modelling (for both the open channels and blocked channels), used the RITS climate change adjusted NIWA historical rainfall intensities.

The pre-development flood modelling used both the unmodified NIWA historical rainfalls (from checking the hydraulic neutrality of outflows from the site) and the climate change adjusted rainfalls (for comparing the upstream and downstream depths and checking the neutrality of inflows into the site). The latter also was required to isolate the impact of the site from the impact of non-site associated runoff. To reduce the number of simulations required, the comparison analysis covered less events than the on-site analysis, although the range was maintained and the standard events (1%, 2%, 10%, 50%) were used under both analyses.

To account for the impact of small (more frequent than one year) event, BCD extrapolated off the NIWA rainfall.

Note: AEP = Annual Exceedance probability and is used for large events while EY = Each year is used for events that happen multiple times a year. Due to the natural of probability, a 63% AEP event is equivalent to a 1 EY event (i.e. an event with an average frequency of one year has a 63% probability of occurring within a given year).

Table 2: HIWA Rainfall intensities

Rainfall Event	Historical	Climate change adjusted
12 EY	**1.48 $^{mm}/_{hr}$**	**1.93 $^{mm}/_{hr}$ (RITSv2 adjustment)**
4 EY	-	**2.43 $^{mm}/_{hr}$ (RITSv2 adjustment)**
2 EY	**2.15 $^{mm}/_{hr}$**	**2.81 $^{mm}/_{hr}$ (RITSv2 adjustment)**
63% AEP	-	**3.25 $^{mm}/_{hr}$ (RITSv2 adjustment)**
50% AEP	**2.65 $^{mm}/_{hr}$**	**3.41 $^{mm}/_{hr}$ (RITSv2 adjustment)**
20% AEP	-	**4.51 $^{mm}/_{hr}$ (RITSv2 adjustment)**
10% AEP	**4.04 $^{mm}/_{hr}$**	**5.35 $^{mm}/_{hr}$ (RITSv2 adjustment)**
5% AEP	-	**6.19 $^{mm}/_{hr}$ (RITSv2 adjustment)**
2% AEP	**5.54 $^{mm}/_{hr}$**	**7.40 $^{mm}/_{hr}$ (RITSv2 adjustment)**
1% AEP	**6.22 $^{mm}/_{hr}$**	**8.36 $^{mm}/_{hr}$ (RITSv2 adjustment)**

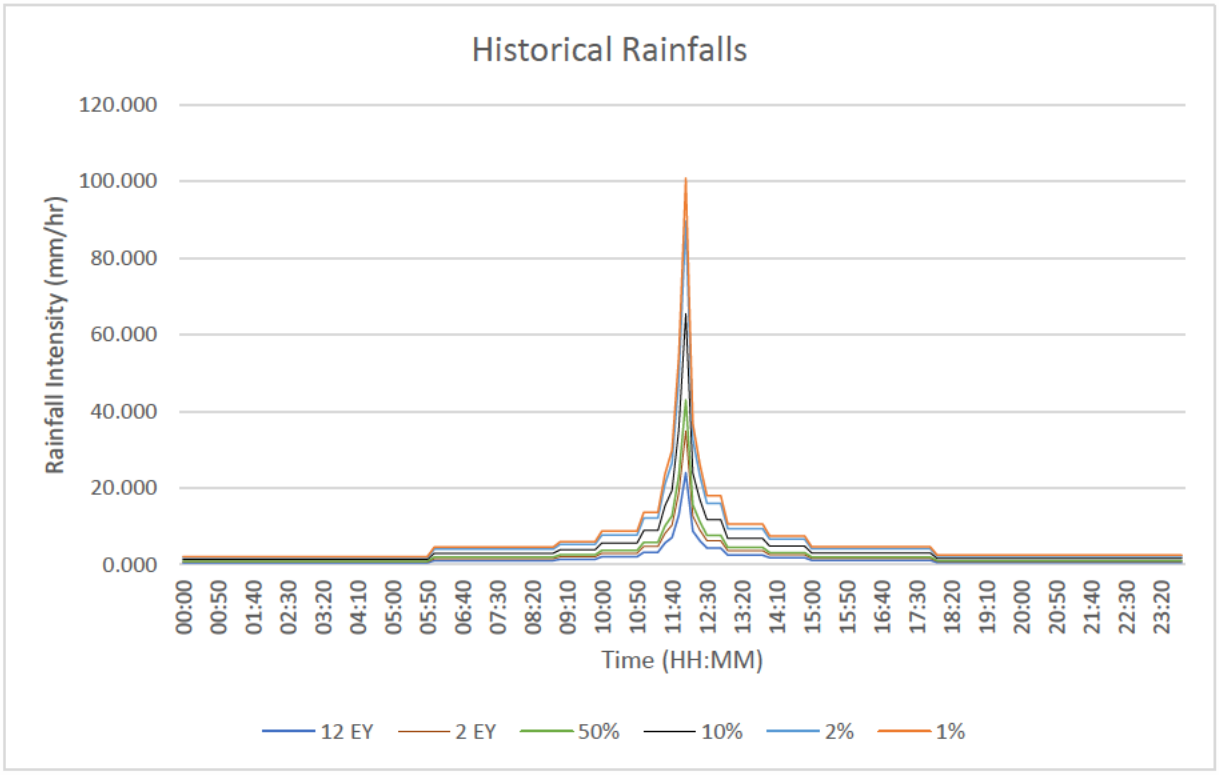


Figure 9: Historical Nested Rainfalls

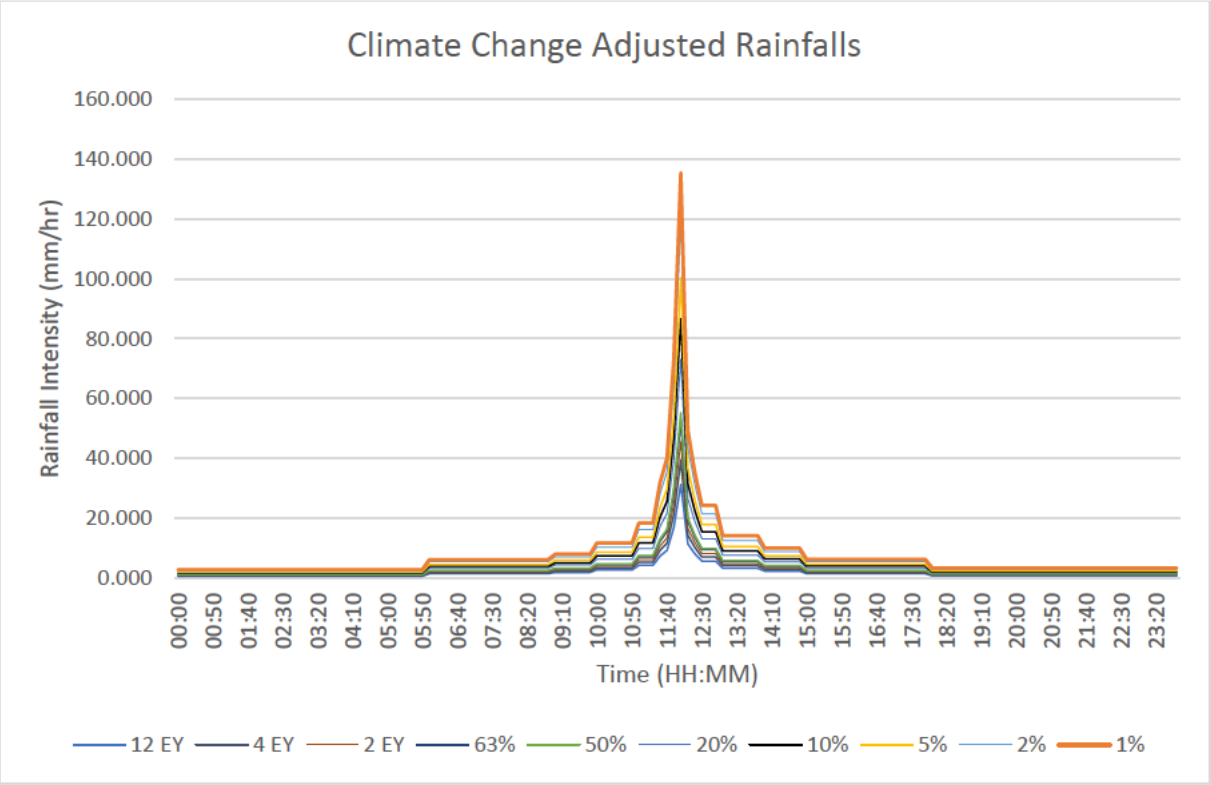


Figure 10: Climate Change Adjusted Nested Rainfalls

4 GEO-HECRAS STORMWATER MODEL RESULTS

4.1 Existing Site Results

The existing flood modelling results confirmed that the existing site can be split into four catchments, based on its existing landform. The main drain, west, and south catchments have discrete points of outflow, while the northern catchment outflow is spread out along the northern boundary with some concentration at northern corner of the site. Each catchment has an inflow from upstream, with the southern catchment's inflow being discrete while the main drain and north catchment inflows are more spread. The main western catchment inflow is an overflow from the main catchment.

The northern catchment shows two areas of floodplain - one towards the northern corner of the site and one centred around one of the catchments drains. The main drain catchment shows a large floodplain area in the middle of the site, focused on the main drain running through the site. Inflow to the western catchment appears minor and mostly an overflow from the main catchment. Although the catchments still have flooding, the western and southern catchments show more concentrated overland flow paths rather than floodplains.

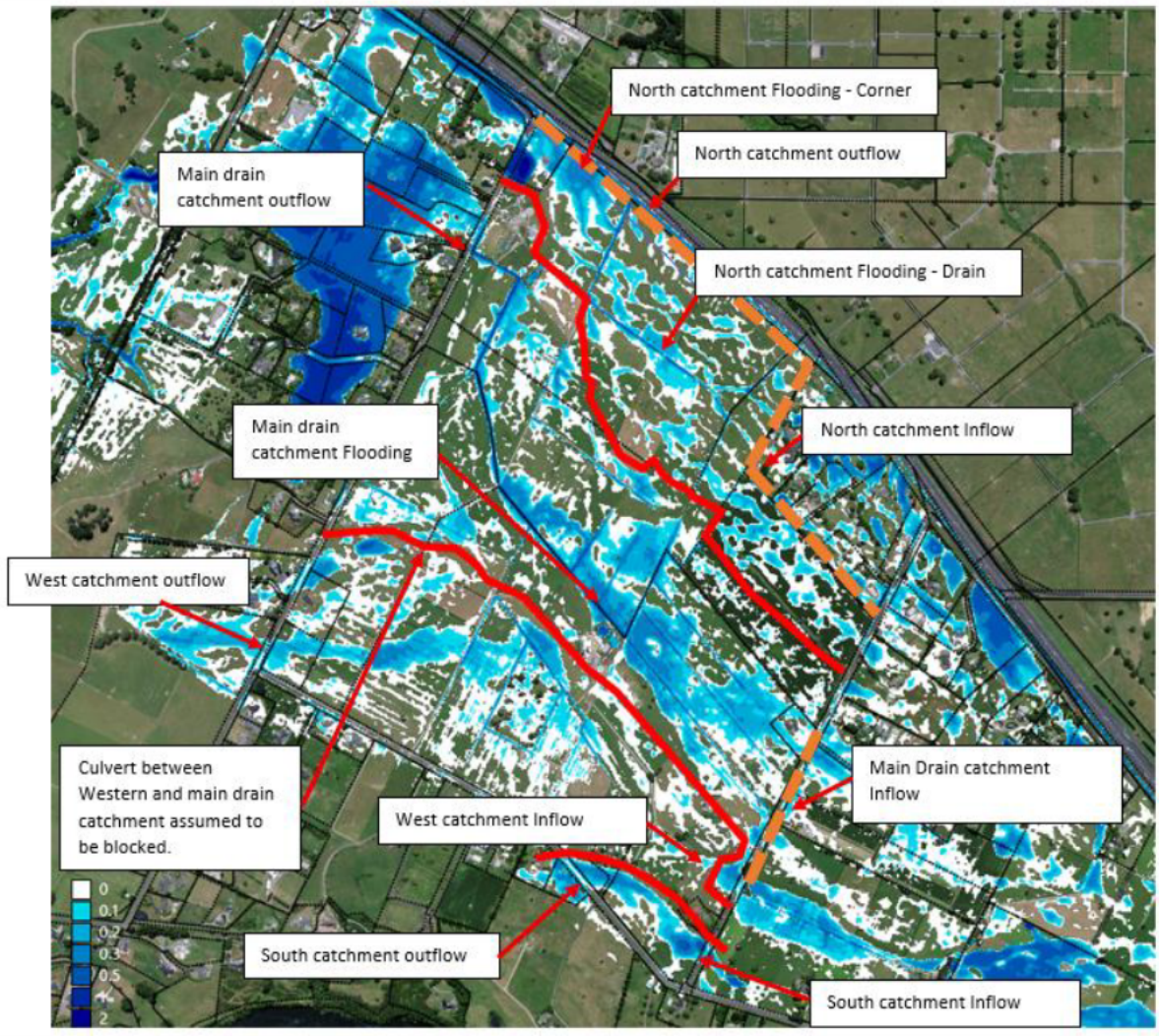


Figure 11: Existing Flooding Overview - catchment inflow and outflow locations - <100mm of flooding is shown in white

- The main drain catchment has the largest outflow and a notable sized inflow.
- The north and west catchments have notable outflows; however, only the north has a notable inflow.
- The southern catchment has very small inflows and even smaller outflows and appears to be more of a flooded depression that an overland flow path.

The flood mapping results indicated that the existing site has only minor issue in terms of flood velocities. Apart from some very isolated locations within the drains, the site has no flood velocities over 1 m/s. The site does have a few areas where the flood velocity sites between 0.5 – 1 m/s while outside of a drain. These are:

- Overflow from one of the northern catchments drains.
- Inflow (across Duncan Road) into the main drain catchment
- Western overland flow.

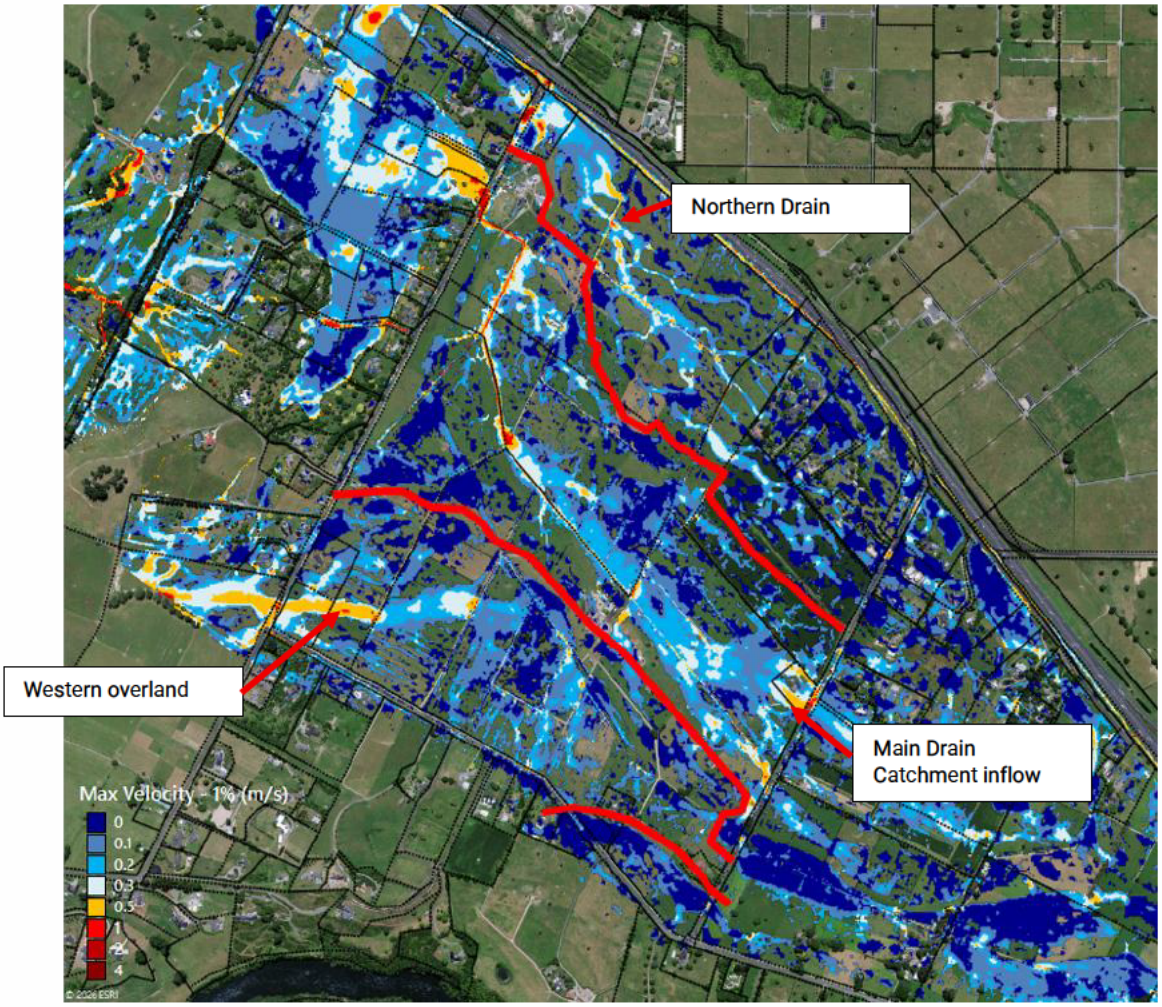


Figure 12: Existing Flooding Overview - flooding velocities

4.2 Proposed Development Results

4.2.1 Flooding Depths

4.2.1.1 Open Channels

The "proposed development – open channels" flood modelling results showed that, even during a 1% AEP event, most of the proposed site would either not experience flooding or would experience flooding of less than 100 mm. Most of the flooding would occurs inside the proposed racing tracks (with ponding inside the racing tracks largely contained within the proposed irrigation ponds) or within the vicinity of the proposed swales. The current swales generally have sufficient capacity to convey stormwater through the site.

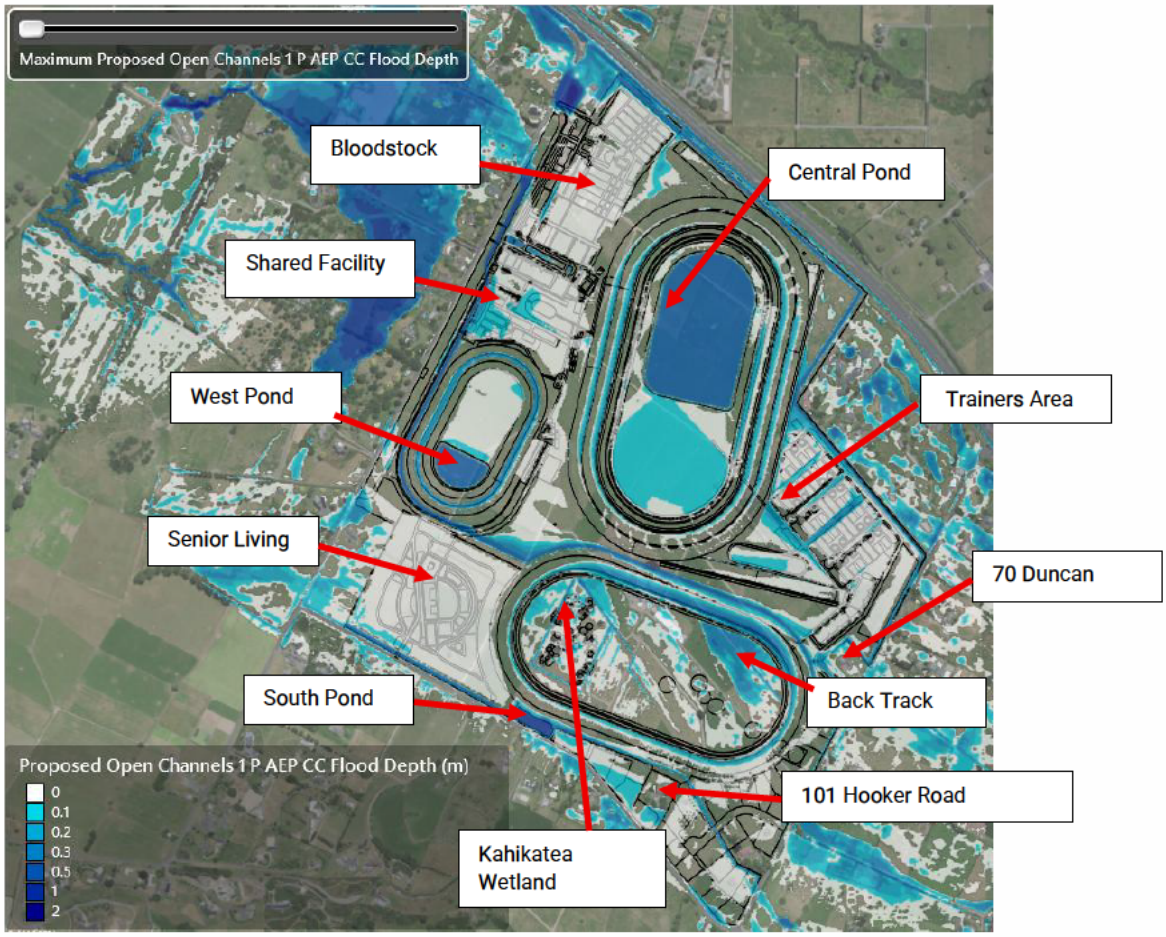


Figure 13: Existing Flooding Overview - Catchment Inflow and Outflow Locations

The landuse that experience flooding (within the Shared Facilities and Trainer areas) are generally close to the swales, this flooding is minor and intended to be mitigated through EPA/Building Consent. There are two locations of notable non-pond/non-swale flooding within the proposed development's current design. The trainer's area to the east and the bloodstock to the west.

Flooding in the Trainer's area appears to the result of very localised low points either blocking an area from to the northern swale and/or experience tipping from the swale. The design surface has ensured these flooded areas are largely contained outside the buildings and within the roadways or landscaped areas i.e. areas where flooding is more acceptable.

Further surface detailing in detailed design will reduce and limit flooding to these areas.

Some flooding does coincide with the proposed tree landscaping area. However, the landscaped trees generally form a single line with sufficient distance between trunks. Overland flow in parallel with the treeline will quickly shift to either side of the line (the swale of the road) where it is no longer obstructed. Overland flow cross the

treeline only has to cross one line of trees that offer a very minor obstruction before entering the swales. It is unlikely that the trees will pose an impact.

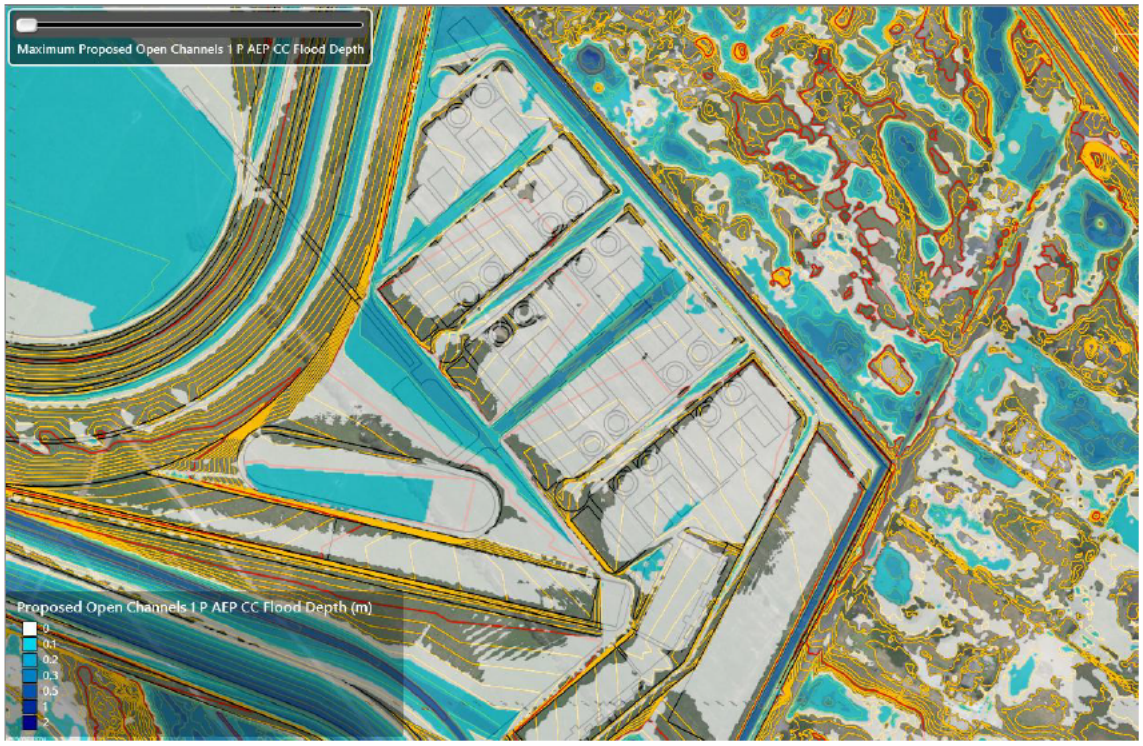


Figure 14: Flooding in the Trainers Area - Red Contours (1 m), yellow contours (0.1 m)

Although most of the bloodstock, shared facility, and senior living areas experience some flooding during the 1% AEP, the flood depths are less than 100mm. Buildings within these areas can achieve sufficient flood protection via lifting the surface to remove flooding, a 300 mm freeboard (as specified under the Regional Infrastructure Technical Specifications) or local landscaping. Further surface modelling will attempt to remove this minor ponding.

4.2.1.2 Blocked Culverts

The "proposed development – blocked culvert" flood modelling results showed that, in comparison to the "proposed development – open culvert" flood modelling results, the impacts of blockages to Culvert-4 (see locations on Figure 15) was largely insignificant, only a slight increase in flooding extent into the Residential living blocks (see Figure 16). A blockage to Culvert-3 would have only a minor impact.

A blockage to Culvert-2 would result in a large increase in flooding within the area between the harness track, the main track, and the main catchment swale (shown in Figure 15). This area is currently proposed to be an overflow carpark.

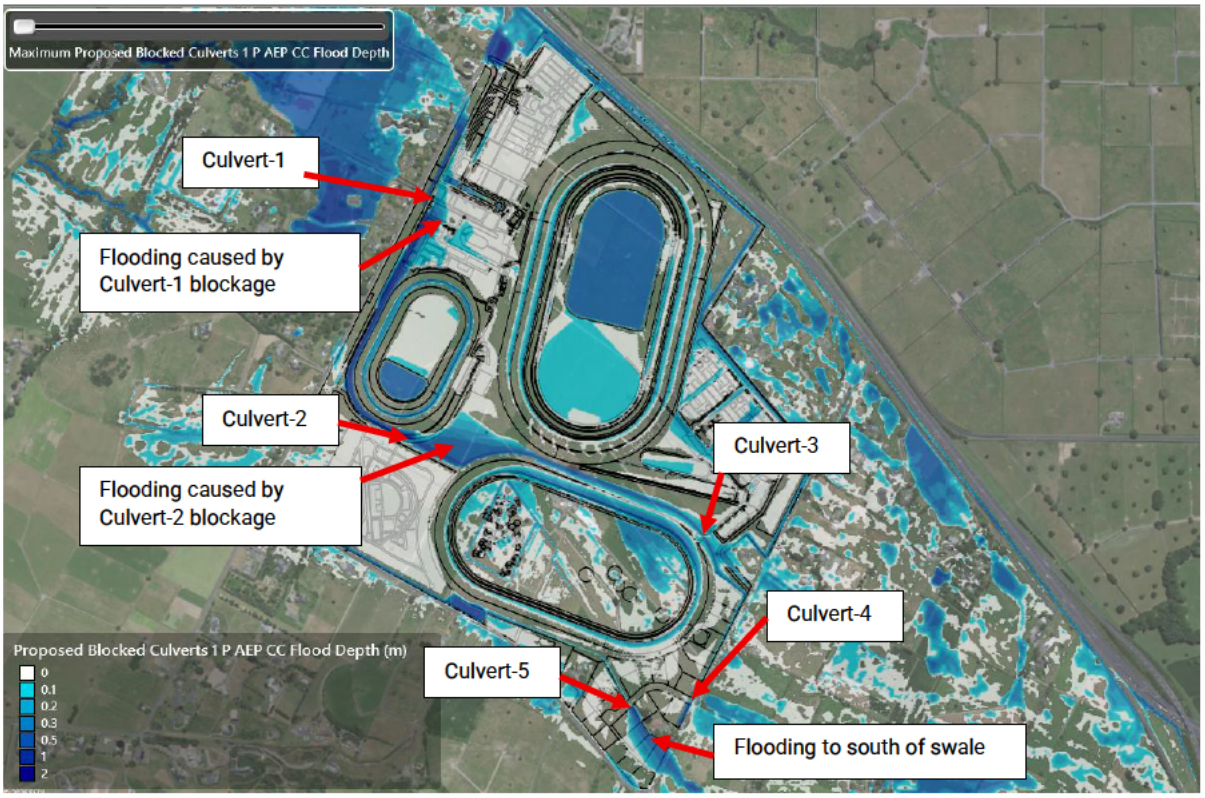


Figure 15: Locations of Culverts and Flooding with Blocked Culverts

The results showed that a blockage to Culvert-5 will results in flooding to the south of the western catchment swale. However, this flooding is very localised (outflow from the south-catchment does not increase due to its blockage, See Section 4.3.1.4).



Figure 16: Flooding from the western catchment swale due to a blockage at Culvert-5

The impact of a blockage to Culvert-1 is far more significant, with a notable increase in flooding at the shared facilities' location. However, the proposed design is for the Culvert-1 to be a box culvert of significant size. Combined with the gradual slope of the swales and the fact that large debris would be caught by Culvert-2 and Culvert-3 prior, the likelihood of Culvert-1 blocking is very low.

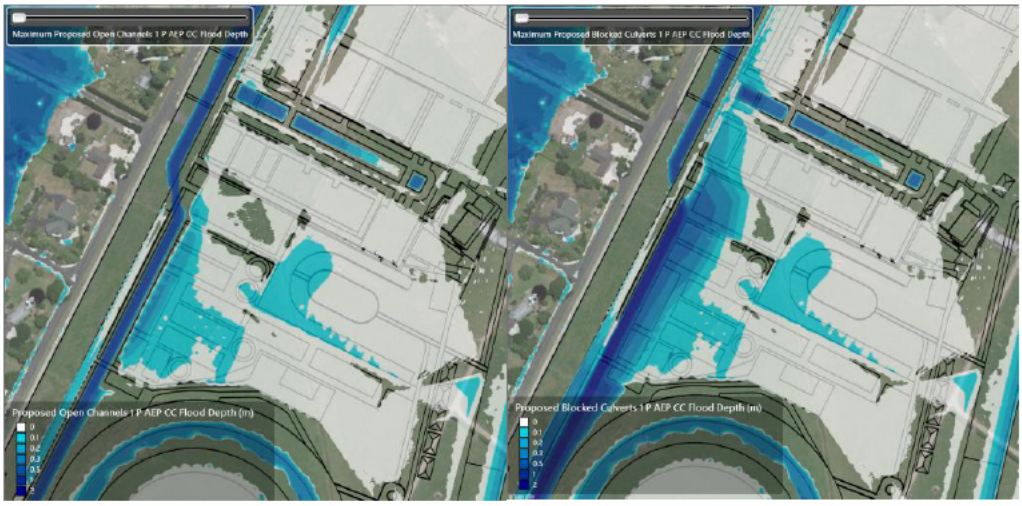


Figure 17: Flooding from the main catchment swale due to a blockage at Culvert-1.

4.2.1.3 Important Locations

The analysis assessed flood depths (and flood elevations) at locations of notable flooding and building locations across the site. It should be noted that while Geo-HECRAS accounts for infiltration loss between precipitation and initial runoff (i.e. the water that infiltrates as it hits the ground), it does not account for infiltration from flooding (i.e. soakage from a flooded depression). Due to this, these values can be considered conservative.

The analysis checked the depths of the ponds as a means to confirm the current runoff volumes from the harness and main tracks. As a result, the depths within the northern pond were check despite current plans to combined it with the central pond.

The results noted below are with the culverts unblocked and the channel's fully open. It is noted that the Kahikatea Wetland obtains a flood depth of 0.14 m in the 12 EY event.

Table 3: Flooding Depths with open Channels

Simulation	Central Pond	South Pond	Western Pond	Kahikatea Wetland	Back Track East
12 EY	0.06 m	0.88 m	0.12 m	0.14 m	0.06 m
4 EY	0.08 m	1.00 m	0.18 m	0.15 m	0.10 m
2 EY	0.11 m	1.07 m	0.23 m	0.16 m	0.13 m
63% AEP	0.13 m	1.10 m	0.26 m	0.16 m	0.15 m
50% AEP	0.16 m	1.16 m	0.30 m	0.17 m	0.18 m
20% AEP	0.27 m	1.35 m	0.44 m	0.16 m	0.25 m
10% AEP	0.36 m	1.48 m	0.56 m	0.19 m	0.30 m
5% AEP	0.45 m	1.59 m	0.67 m	0.20 m	0.35 m
2% AEP	0.58 m	1.72 m	0.84 m	0.20 m	0.39 m
1% AEP	0.68 m	1.78 m	0.97 m	0.21 m	0.40 m

Table 4: Flooding Depths with Open Channels

Simulation	Bloodstock	Shared Facility	East Trainers Area
12 EY	0.02 m	0.04 m	0.34 m
4 EY	0.03 m	0.06 m	0.37 m
2 EY	0.04 m	0.07 m	0.38 m
63% AEP	0.04 m	0.07 m	0.40 m
50% AEP	0.05 m	0.08 m	0.41 m
20% AEP	0.06 m	0.10 m	0.45 m
10% AEP	0.07 m	0.12 m	0.47 m
5% AEP	0.08 m	0.13 m	0.49 m
2% AEP	0.09 m	0.14 m	0.51 m
1% AEP	0.09 m	0.16 m	0.53 m

Table 5: Flooding Elevations with Open Channels

Simulation	Central Pond	South Pond	Western Pond	Kahikatea Wetland	Back Track West
Ground Level	53.05 m RL	53.60 m RL	52.90 m RL	55.10 m RL	54.53 m RL
12 EY	53.10 m RL	54.48 m RL	53.02 m RL	55.14 m RL	54.54 m RL
4 EY	53.13 m RL	54.61 m RL	53.08 m RL	55.15 m RL	54.58 m RL
2 EY	53.16 m RL	54.96 m RL	53.13 m RL	55.16 m RL	54.62 m RL
63% AEP	52.18 m RL	54.71 m RL	53.16 m RL	55.17 m RL	54.64 m RL
50% AEP	52.21 m RL	54.76 m RL	53.20 m RL	55.17 m RL	54.66 m RL
20% AEP	53.32 m RL	54.96 m RL	53.34 m RL	55.19 m RL	54.74 m RL
10% AEP	53.41 m RL	55.08 m RL	53.46 m RL	55.20 m RL	54.79 m RL
5% AEP	53.50 m RL	55.19 m RL	53.57 m RL	55.21 m RL	54.84 m RL
2% AEP	53.63 m RL	55.32 m RL	53.73 m RL	55.22 m RL	54.87 m RL
1% AEP	53.73 m RL	55.39 m RL	53.86 m RL	55.22 m RL	54.88 m RL

Table 6: Flooding Elevations with open channels

Simulation	Bloodstock	Shared Facility	East Trainers Area
Ground Level	53.70 m RL	54.88 m RL	54.43 m RL
12 EY	53.72 m RL	54.93 m RL	54.77 m RL
4 EY	53.73 m RL	54.94 m RL	54.80 m RL
2 EY	53.74 m RL	54.95 m RL	54.81 m RL
63% AEP	53.74 m RL	54.95 m RL	54.82 m RL
50% AEP	53.75 m RL	54.96 m RL	54.84 m RL
20% AEP	53.76 m RL	54.99 m RL	54.88 m RL
10% AEP	53.77 m RL	55.00 m RL	54.90 m RL
5% AEP	53.78 m RL	55.01 m RL	54.92 m RL
2% AEP	53.79 m RL	55.03 m RL	54.94 m RL
1% AEP	53.79 m RL	55.04 m RL	54.96 m RL

The results were also analysed using flood depths across the site under the assumption that the culverts were blocked. As noted above, the flooding in most locations was largely unchanged. The table's below show the locations that did experience a difference in flooding due to blocked culverts, along with the respective change.

The tables also show the difference in depth and elevation immediately upstream of Culvert-1, Culvert-2, and Culvert-5, as these culverts are having the most notable impact on flooding extent.

The results indicated that a blocked Culvert-1 would only increase the flooding in around the Shared Facility during a 20% AEP event or greater.

Table 7: Flooding Depths with blocked culverts (the +depth is increase over the open-channel depth)

Simulation	Shared Facility (front)	Overflow Carparking (Harness)	Rural Residential
12 EY	0.01 m	0.01 m	0.03 m
4 EY	0.01 m	0.01 m	0.04 m
2 EY	0.02 m	0.04 m (+0.03 m)	0.04 m
63% AEP	0.02 m	0.14 m (+0.12 m)	0.07 m (+0.02 m)
50% AEP	0.02 m	0.21 m (+0.19 m)	0.11 m (+0.06 m)
20% AEP	0.28 m (+0.24 m)	0.31 m (+0.28 m)	0.23 m (+0.17 m)
10% AEP	0.32 m (+0.38 m)	0.38 m (+0.35 m)	0.31 m (+0.24 m)
5% AEP	0.43 m (+0.37 m)	0.43 m (+0.39 m)	0.38 m (+0.30 m)
2% AEP	0.50 m (+0.43 m)	0.51 m (+0.46 m)	0.48 m (+0.39 m)
1% AEP	0.55 m (+0.47 m)	0.57 m (+0.52 m)	0.55 m (+0.46 m)

Table 8: Flooding Elevation with blocked culvert (the +elevation is the increase over the open-channel depth)

Simulation	Shared Facility (front)	Overflow Carparking (Harness)	Rural Residential
Ground Level	53.78 m RL	53.79 m RL	56.21 m RL
12 EY	53.79 m RL	53.80 m RL	56.24 m RL
4 EY	53.79 m RL	53.80 m RL	56.25 m RL
2 EY	53.80 m RL	53.83 m RL (+0.03 m)	56.25 m RL
63% AEP	53.80 m RL	53.93 m RL (+0.12 m)	56.28 m RL (+0.02 m)
50% AEP	53.80 m RL	54.00 m RL (+0.19 m)	56.32 m RL (+0.06 m)
20% AEP	54.06 m RL (+0.24 m)	54.10 m RL (+0.28 m)	56.44 m RL (+0.17 m)
10% AEP	54.15 m RL (+0.38 m)	54.17 m RL (+0.35 m)	56.52 m RL (+0.24 m)
5% AEP	54.21 m RL (+0.37 m)	54.22 m RL (+0.39 m)	56.59 m RL (+0.30 m)
2% AEP	54.28 m RL (+0.43 m)	54.30 m RL (+0.46 m)	56.69 m RL (+0.39 m)
1% AEP	54.33 m RL (+0.47 m)	54.36 m RL (+0.52 m)	56.76 m RL (+0.46 m)

The depths analysis also compared the depths within the Kahikatea Wetland to ensure that the proposed design was not worsening the wetland, as per Waikato Regional Council requirements.

The results showed that flooding within the wetland would be largely unchanged during <10% AEP events, even with climate change taken into account. Even in the >10% AEP events, the difference is only ~50 mm. The area would receive less flooding during larger events; however, it would not be dry. Detailed design around the back track will pipe more stormwater runoff into the wetland.

Table 9: Flooding Depths at the Kahitatea Wetland

Rainfall	Existing - Historical	Existing - Climate Change	Proposed	Difference
12 EY	0.12 m	0.14 m	0.14 m	0.00 m
2 EY	0.15 m	0.16 m	0.16 m	0.00 m
50% AEP	0.15 m	0.17 m	0.17 m	0.00 m
10% AEP	0.17 m	0.20 m	0.19 m	-0.01 m
2% AEP	0.21 m	0.25 m	0.20 m	-0.05 m
1% AEP	0.22 m	0.26 m	0.21 m	-0.05 m

4.2.2 Flood Velocities

The flood modelling results showed most of the site would experience velocities of <0.5 $^{m}/_{s}$, with higher velocity flow largely confined to the swales. The velocity results showed a couple of local overland flow paths outside of the swales, one running through the shared facility and one running between the Main track and Harness Track, through the Overflow carpark. Detailed design will ensure these flows the contained within the stormwater management system.

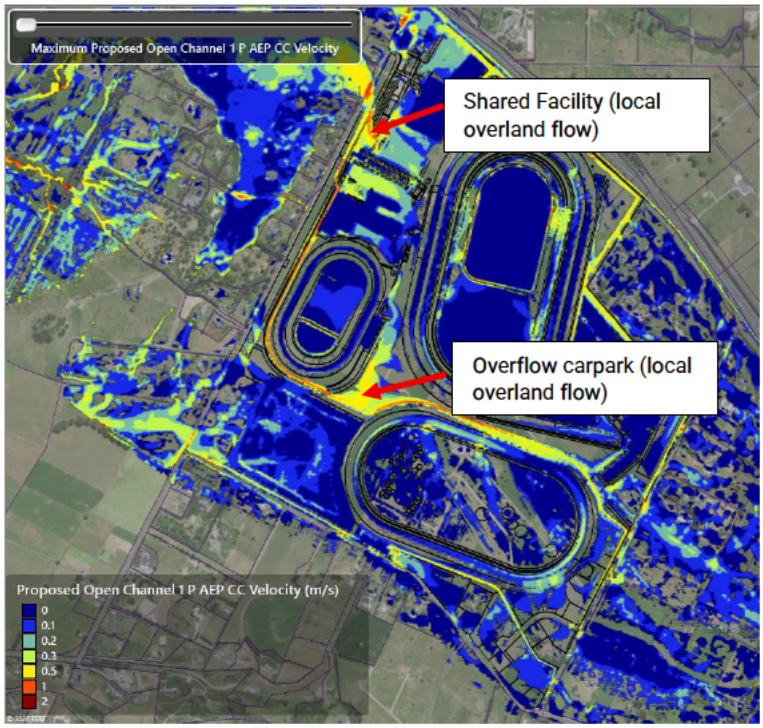


Figure 18: Proposed Development Flood Velocities with Open Channels

A blockage at Culvert-1 would increase the velocities through the Shared Facility, with the overflow creating an overland flow path parallel to the swale with velocities >1 $^{m}/_{s}$. However, as noted above, Culvert-1 will be design with sufficient size and capacity to not block.

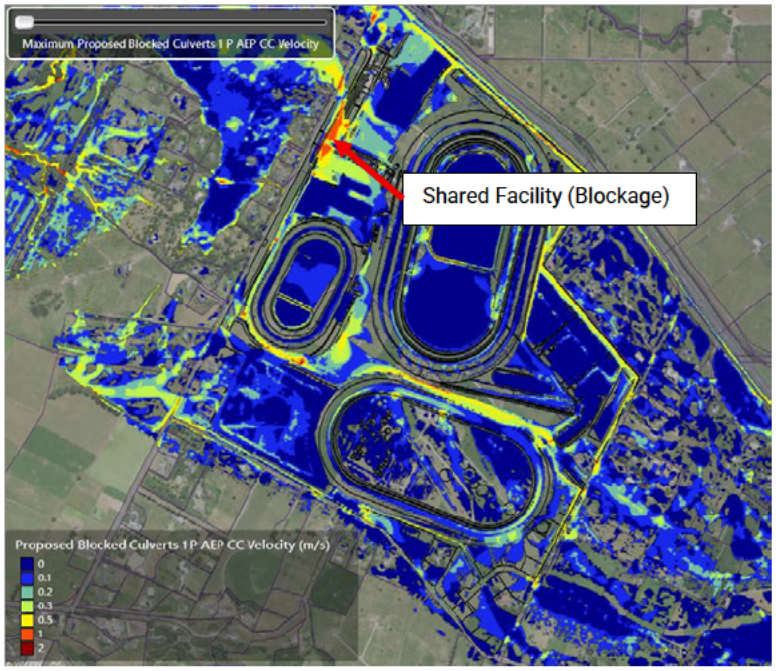


Figure 19: Proposed Development Flood Velocities - Blocked Channels

Table 10: Flooding Velocities in notable overland flow paths

Rainfall	Shared Facility Overland Flow	Shared Facility – Blocked Culvert	Shared Carpark
12 EY	-	0.30 m/s	0.32 m/s
4 EY	-	0.34 m/s	0.38 m/s
2 EY	-	0.38 m/s	0.41 m/s
63% AEP	-	0.40 m/s	0.43 m/s
50% AEP	-	0.42 m/s	0.45 m/s
20% AEP	0.82 m/s	0.46 m/s	0.52 m/s
10% AEP	0.88 m/s	0.83 m/s	0.57 m/s
5% AEP	0.93 m/s	1.32 m/s	0.61 m/s
2% AEP	0.99 m/s	1.91 m/s	0.67 m/s
1% AEP	1.02 m/s	2.21 m/s	0.70 m/s

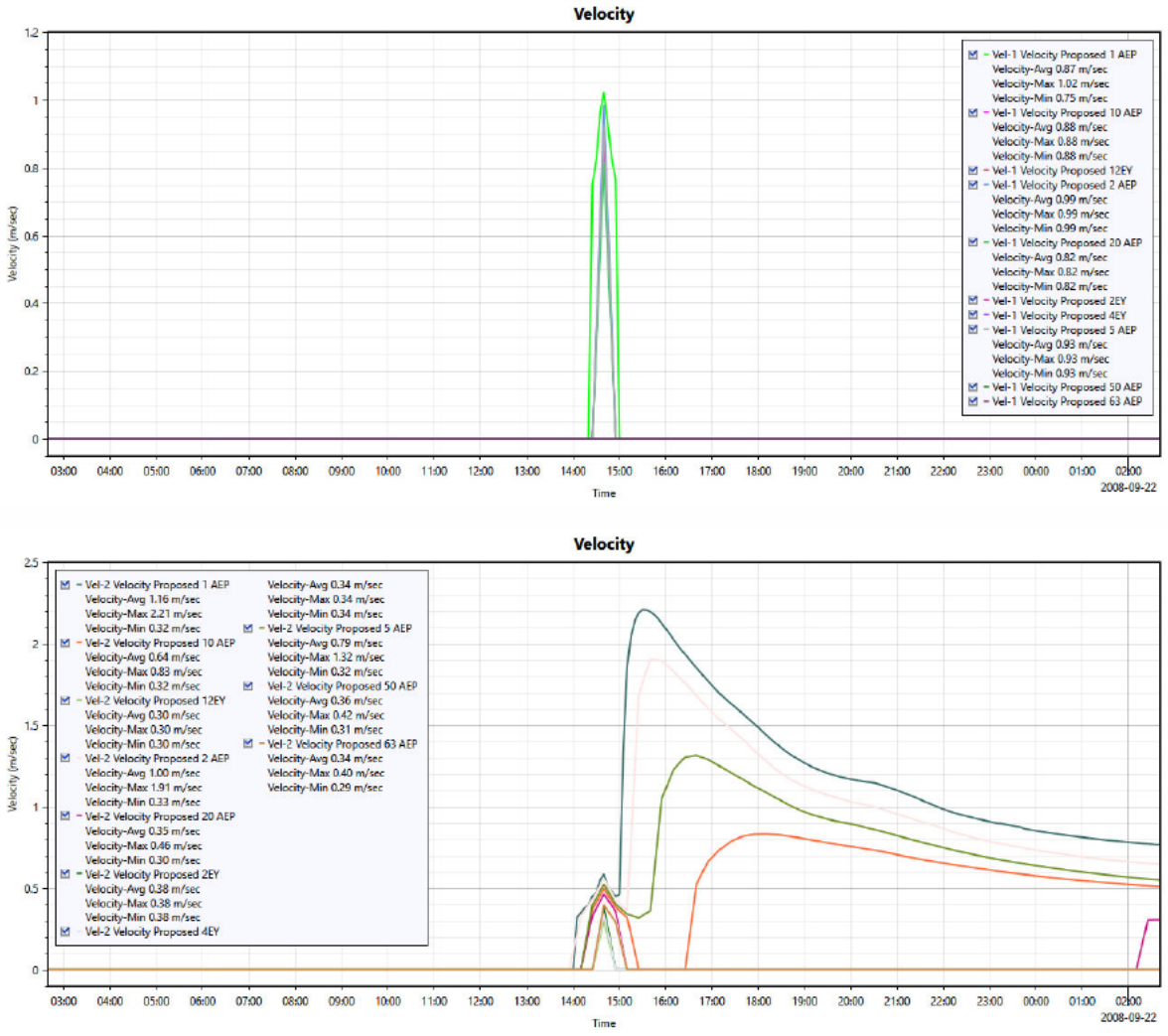


Figure 20: Flood Velocity Graphs - Shared Facility Overflow (top), Shared Facility Blocked Culvert (bottom)

4.2.3 Upstream and Downstream Impacts

4.2.3.1 Overview

Proposed designs for the site are being undertaken to ensure that the peak outflows from the proposed site (which includes climate change) do not exceed the peak existing historical outflows from any <1% AEP storm event. The designs are also being undertaken to ensure the peak inflow for the proposed site exceed (i.e. don't obstruct) the climate change adjusted existing inflows. Climate change adjusted existing is used in this instance to ensure that any exceedance in the proposed site is not simply an increase due to climate change.

To maintain the current ecological values of the existing drains and streams, the proposed design maintained the existing outfalls with the design intent of maintaining the existing flows at these outfalls for even minor storm events.

Further, designs for the site are being undertaken to ensure that the flooding (both upstream and downstream of the site) does not increase because of the proposed development during a <1% AEP storm event. To isolate the proposed site's impact (i.e. not include the impact of climate change adjusted runoff from other upstream and downstream areas), this part of the analysis compared the proposed site with a climate change adjusted existing site. All other parts of the analysis used historical rainfalls for the existing site.

A high-level overview of the site, and its upstream and downstream catchment, determined that the proposed development will have a largely negligible impact of the both the upstream and downstream depths. The present surface design shows very minor increases in upstream and downstream ponding, largely within the 20-30 mm range, which can be considered the margin of error for the modelling.

These increases are all immediately adjacent to the site. In regard to the downstream impacts, they are the result of the flood model not including the flood mitigation infrastructure (weirs, connections between ponds and swales etc) which will be included in the detailed design. In regard to the upstream impacts, they are the result of minor changes to Duncan Road. The site's design surface has no obstructions between Duncan Road and the proposed flooding swales, and the flooding swales have been shown not to overtop back towards Duncan Road.

Duncan Road will be further detailed in the detailed design stage to remove any upstream impacts, with the inclusion of sumps and culverts running across the road in required.

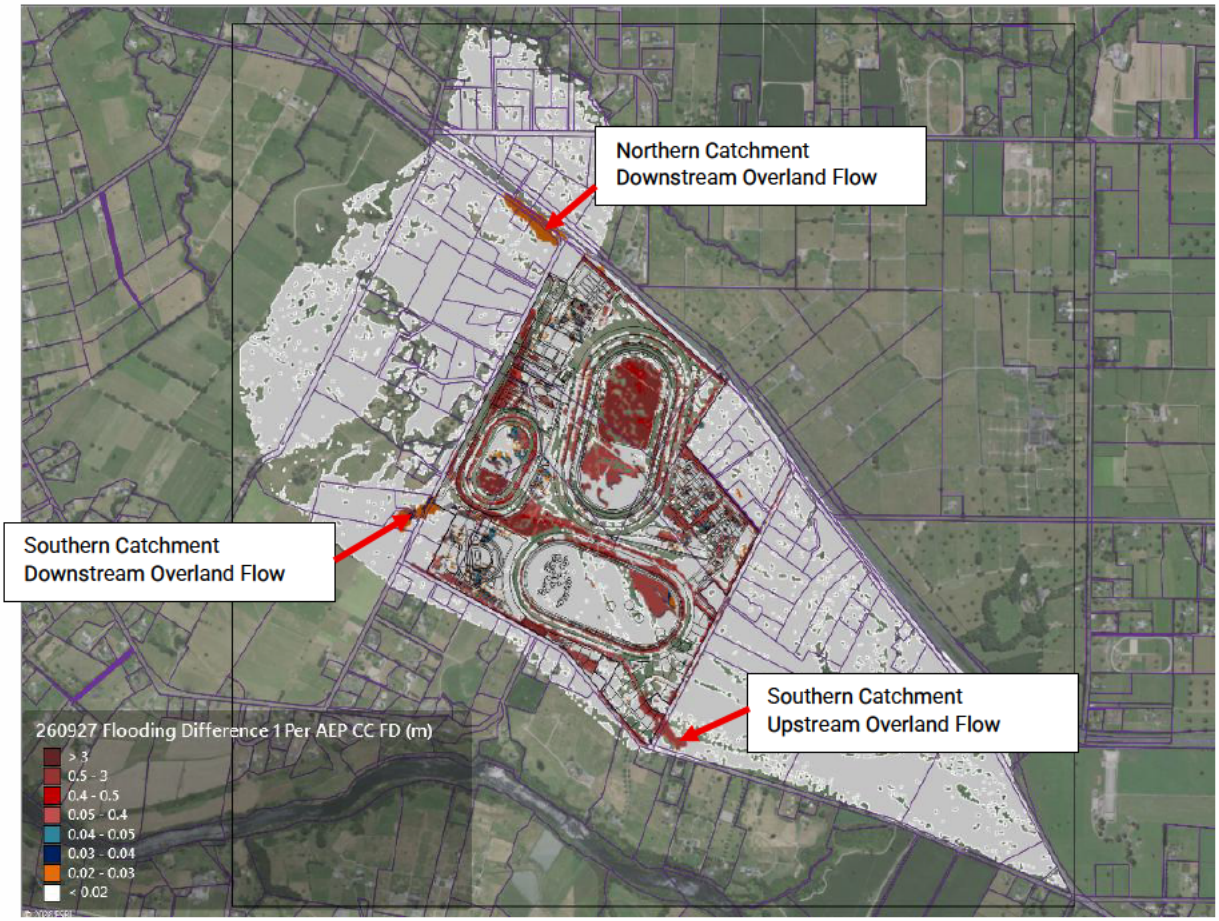


Figure 21: Comparison Between the proposed and existing (climate change) site. White shows area with <20 mm of flooding difference

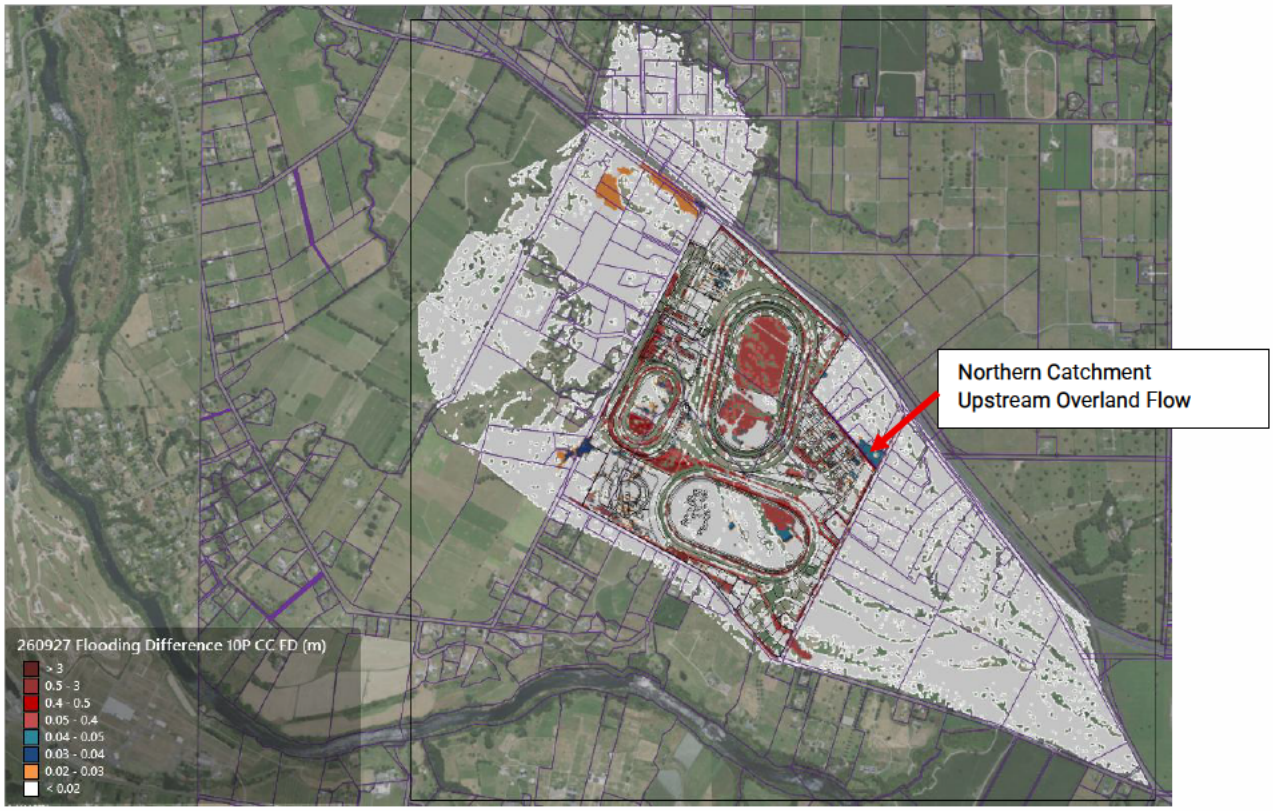


Figure 22: Comparison between the proposed and existing (climate change) site. Whie shows areas with <20 mm of flooding difference



Figure 23: Comparison between the proposed and existing (climate change) site. White shows areas with <20 mm of flooding difference

4.2.3.2 *Northern Catchment*

The results showed that the proposed development would not have a large-scale negative impact on the upstream section of the northern catchment, with the inflows into the proposed site increasing (i.e. the proposed development is not causing any large-scale flow blockage or obstructing) even with climate change applied to the pre-development.

The results do show that 104C Duncan Road would experience a very localised increase in flooding, on the scale of 40 mm. As the peak inflows from the northern catchment are higher in the proposed design, the model shows no obstruction (i.e. free flow from 104C Duncan Road into the site), and the increase is very localised, the difference is the result of a slight differences in the modelled terrain of this area. The difference will be removed as the surface and model become more detailed as part of detailed design, **which will ensure there is no additional flooding on 104C Duncan Road.**

The results indicate that the proposed development will impact the downstream sections of the northern catchment if mitigation measures are not introduced as part of the design. Outflows from the site increase during all events. As the North Catchment flooding swale contains no culverts, the blocked swales have no impact of the results.

Table 11: North Catchment Inflow and Outflows

North Catchment	Inflow – Existing (Climate Change)	Inflow – Proposed Open Channel	% change	Inflow – Proposed Blocked Channel	% change	Outflow – Existing (Historic)	Outflow – Proposed Open Channel	% change	Outflow – Proposed Blocked Channel	% change
12 EY	0.06 m^3/s	0.07 m^3/s	17%	0.07 m^3/s	17%	0.09 m^3/s	0.22 m^3/s	144%	0.22 m^3/s	144%
2 EY	0.14 m^3/s	0.15 m^3/s	7%	0.15 m^3/s	7%	0.28 m^3/s	0.64 m^3/s	129%	0.64 m^3/s	129%
50% AEP	0.23 m^3/s	0.27 m^3/s	17%	0.27 m^3/s	17%	0.42 m^3/s	1.09 m^3/s	160%	1.09 m^3/s	160%
10% AEP	0.58 m^3/s	0.76 m^3/s	31%	0.76 m^3/s	31%	1.36 m^3/s	2.87 m^3/s	111%	2.87 m^3/s	111%
2% AEP	1.37 m^3/s	1.42 m^3/s	4%	1.42 m^3/s	4%	2.54 m^3/s	4.58 m^3/s	80%	4.58 m^3/s	80%
1% AEP	1.77 m^3/s	1.86 m^3/s	5%	1.86 m^3/s	5%	3.12 m^3/s	5.43 m^3/s	74%	5.44 m^3/s	74%

A review of the time varying results noted that in addition to having a larger peak flow, the proposed development off-peak flows will be higher than in the existing for most of the post peak storm period. This increase volume will have an impact on the downstream catchment and will be mitigated as much as possible.

BCD has calculated, at a high level, the volume the site would need to attenuate in order to keep reduce the storm event to its pre-development levels for the entire duration. Detailed design will provide flow structures to ensure outflow off the site remains consistent with the existing case.

This additional storage outlined in Table 13, will mitigate the additional flows calculated in Table 12, to ensure hydraulic neutrality. This will be detailed in the engineering design phase.

Table 12: Additional Detention Required for proposed northern catchment

	Additional Detention Required
12 EY	2,634 m^3
2 EY	6,417 m^3
50% AEP	8,587 m^3
10% AEP	19,090 m^3
2% AEP	31,648 m^3
1% AEP	40,606 m^3

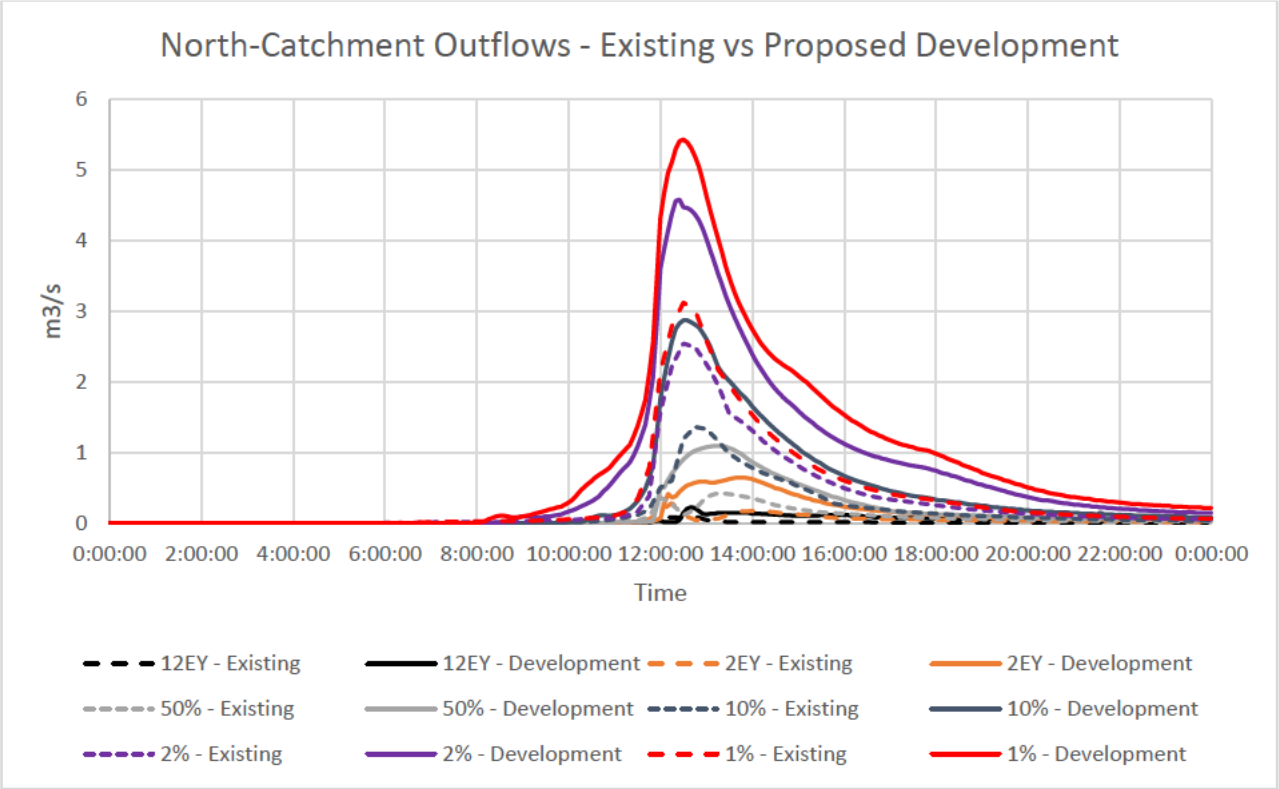


Figure 24: North Catchment - Time varying results

4.2.3.3 Main Catchment

70 Ducan Road

As discussed, initial results indicated that the site could have an impact on 70 Duncan Road and the current design has already accounted for this flooding issue. Subsequent coordination with the architect and client regarding the Masterplan indicated that negotiations with the owners of 70 Duncan Road have resulted in a proposal to build a timber noise wall along the boundary between the site and 70 Duncan Road.

Although the current modelling assessment does not consider the impact of every individual wall or building, due to the model's current level of detail and the likelihood of the building layouts changing, the sound wall around 70 Duncan Road required an exception.

Its location on a major overland flow path and its set location (on a boundary) meant it would have a large impact (if unmitigated) that was unlikely change with relocation.

The wall is currently proposed as a 2 m high timber wall with a series of 100mm culverts running beneath to provide an outlet for flow. This structure was represented in GeoHECRAS as a 4 m wide wall (to ensure the 1 m grid topography didn't create an unintended gap) with 1 m slits stamped at 15 m intervals to represent the 100mm culverts. These gaps were included on only the northwest and southwest sides as modelling indicated that culverts on the northeast side would result in flow from the site running into 70 Duncan Place.

The wall's width in GeoHECRAS meant that the proposed swale along with site's side of the wall was partly obstructed. This is considered conservative, as an obstructed swale carries less flow.

A visual comparison of existing (climate change) and proposed development indicated that the flooding within 70 Duncan Road would not increase notably as a result of the timber noise wall. Due to limitations of representing the wall, its adjacent swale, and the culverts within GeoHECRAS, the visual comparison is considered acceptable as proof of concept.

Detailed design will ensure that the culverts are sized and located as required to ensure that 70 Duncan Road does not flood.

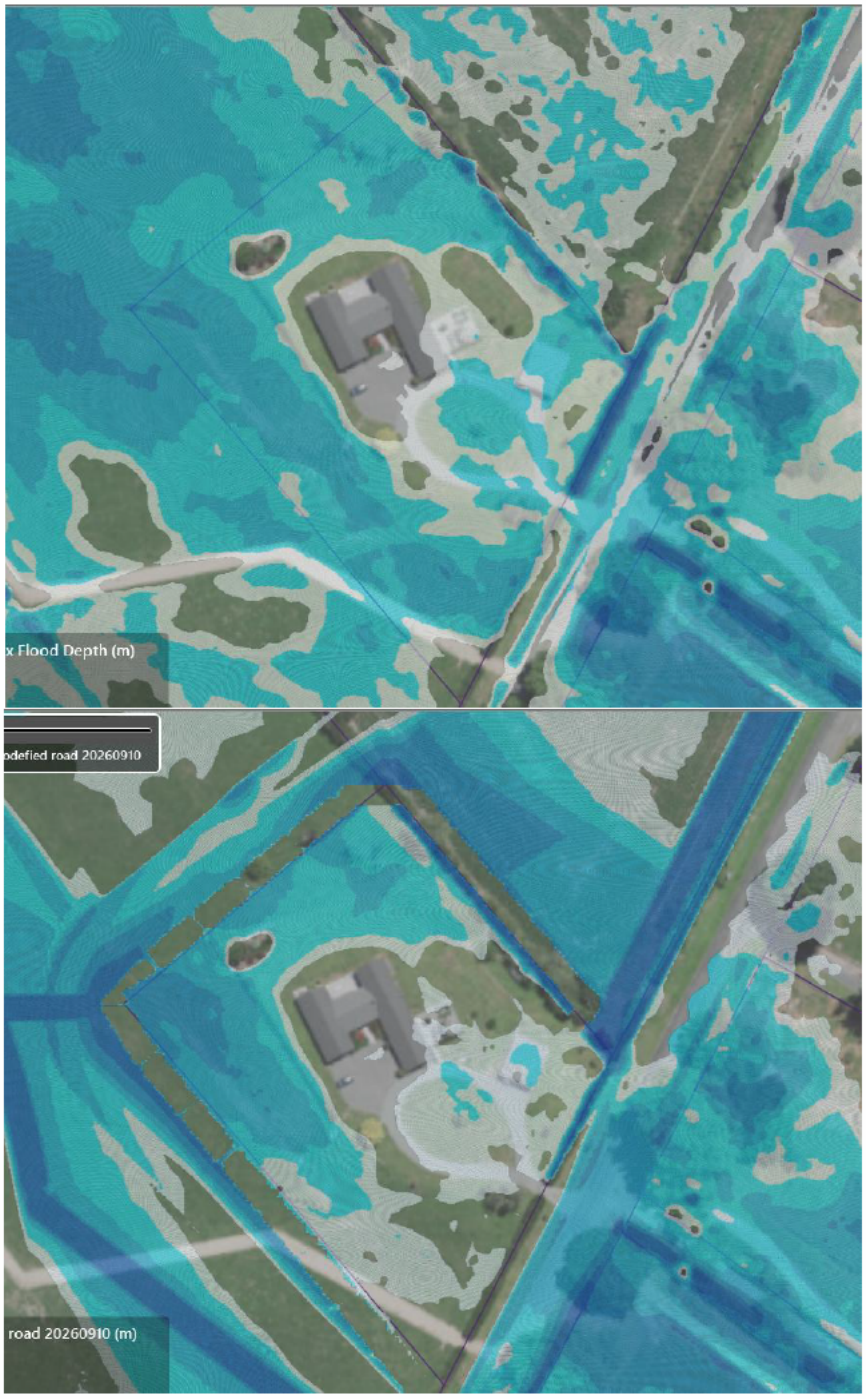


Figure 25: Flooding within 70 Duncan Road with wall

Rest of Catchment

The results showed that the proposed design will have a very minor impact on the upstream section of the main catchment, when compared with the climate change adjusted existing flooding. This difference is minor (2-3% decrease in peak flow) and the result of slight changes to Duncan Road.

The model assumes no culverts across Duncan Road and excludes many of its swales (due to cell size and GeoHECRAS encountering problems if break lines are too closely located). As noted previously, GeoHECRAS also assumes that ponded runoff does not infiltrate. Further, the comparison is with the climate change adjusted existing flooding (undertaken to help mitigate the impacts of climate change on the upstream catchment).

Regardless, the designs for Duncan Road will be further detailed in the detailed design to ensure that the inflow either remains unchanged or increases during each event.

The Proposed Development – Open Channel results indicated that the sites outflows will have a significant impact on the downstream sections of the main catchment if mitigation measures are not introduced as part of the design . Outflows are increasing under each event; however, the off-peak flows are generally lower under the proposed development.

By contrast, the Proposed Development – Blocked culverts results show a decrease in outflows in comparison to the existing development for all <2% AEP events (with 2% AEP having the same peak in both existing and proposed). This indicates the restricting the culvert capacities may work as a mitigation measures. Of course, as noted in Section 4.2.1.2, restricting the capacities too much will cause additional flooding.

Table 13: Main Catchment inflow and Outflows

Main Catchment	Inflow – Existing (Climate Change)	Inflow – Proposed Open Channel	% change	Inflow – Proposed Blocked Channel	% change	Outflow – Existing (Historic)	Outflow – Proposed Open Channel	% change	Outflow – Proposed Blocked Channel	% change
12 EY	0.02 m^3/s	0.08 m^3/s	300%	0.07 m^3/s	250%	0.04 m^3/s	0.05 m^3/s	25%	0.00 m^3/s	-100%
2 EY	0.36 m^3/s	0.43 m^3/s	19%	0.43 m^3/s	19%	0.47 m^3/s	1.00 m^3/s	113%	0.01 m^3/s	-98%
50%	0.72 m^3/s	0.75 m^3/s	4%	0.75 m^3/s	4%	0.88 m^3/s	1.85 m^3/s	110%	0.06 m^3/s	-93%
10%	1.92 m^3/s	1.93 m^3/s	1%	1.87 m^3/s	-3%	2.02 m^3/s	4.45 m^3/s	120%	0.80 m^3/s	-60%
2%	3.52 m^3/s	3.46 m^3/s	-2%	3.45 m^3/s	-2%	4.35 m^3/s	8.68 m^3/s	100%	4.35 m^3/s	0%
1%	4.51 m^3/s	4.40 m^3/s	-2%	4.40 m^3/s	-2%	5.60 m^3/s	10.58 m^3/s	89%	6.91 m^3/s	23%

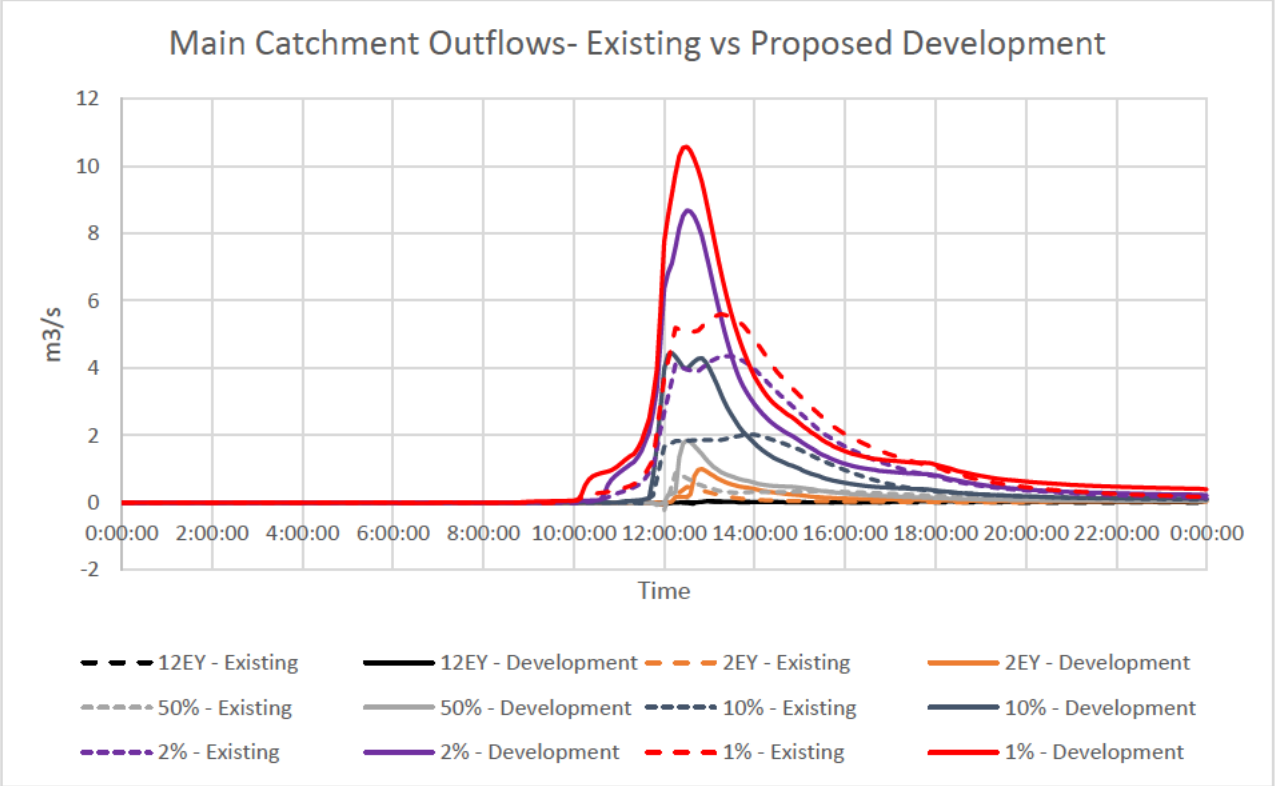


Figure 26: Main Catchment - Time varying results for the main catchment

BCD has calculated, at a high level, the volume the site would need to attenuate in order to keep reduce the storm event to its pre-development levels for the entire duration.

Table 14: Additional Detention Required for proposed main catchment

	Additional Detention Required
12 EY	746 m³
2 EY	4,361 m³
50%	4,797 m³
10%	12,758 m³
2%	21,874 m³
1%	26,263 m³

4.2.3.4 Western Catchment

As the main Western Catchment inflow is an off shoot of the main catchment, its upstream impact is accounted for in Section 4.2.3.3.

The results indicate that proposed development will have a small impact on the downstream sections of the western catchment if mitigation measures are not introduced as part of the design.

Table 15: West Catchment Outflows

West Catchment	Outflow – Existing (Historic)	Outflow – Proposed Open Channel	% change	Outflow – Proposed Blocked Channel	% change
12 EY	0.11 m^3/s	0.21 m^3/s	91%	0.21 m^3/s	-91%
2 EY	0.26 m^3/s	0.46 m^3/s	77%	0.46 m^3/s	-77%
50%	0.39 m^3/s	0.65 m^3/s	67%	0.65 m^3/s	-67%
10%	1.25 m^3/s	1.55 m^3/s	24%	1.55 m^3/s	24%
2%	2.76 m^3/s	2.90 m^3/s	5%	2.48 m^3/s	-10%
1%	3.56 m^3/s	3.58 m^3/s	1%	3.01 m^3/s	-15%

A review of the time varying results noted that the post-development off-peak flows are only marginally higher than in the pre-development for most of the post peak storm period.

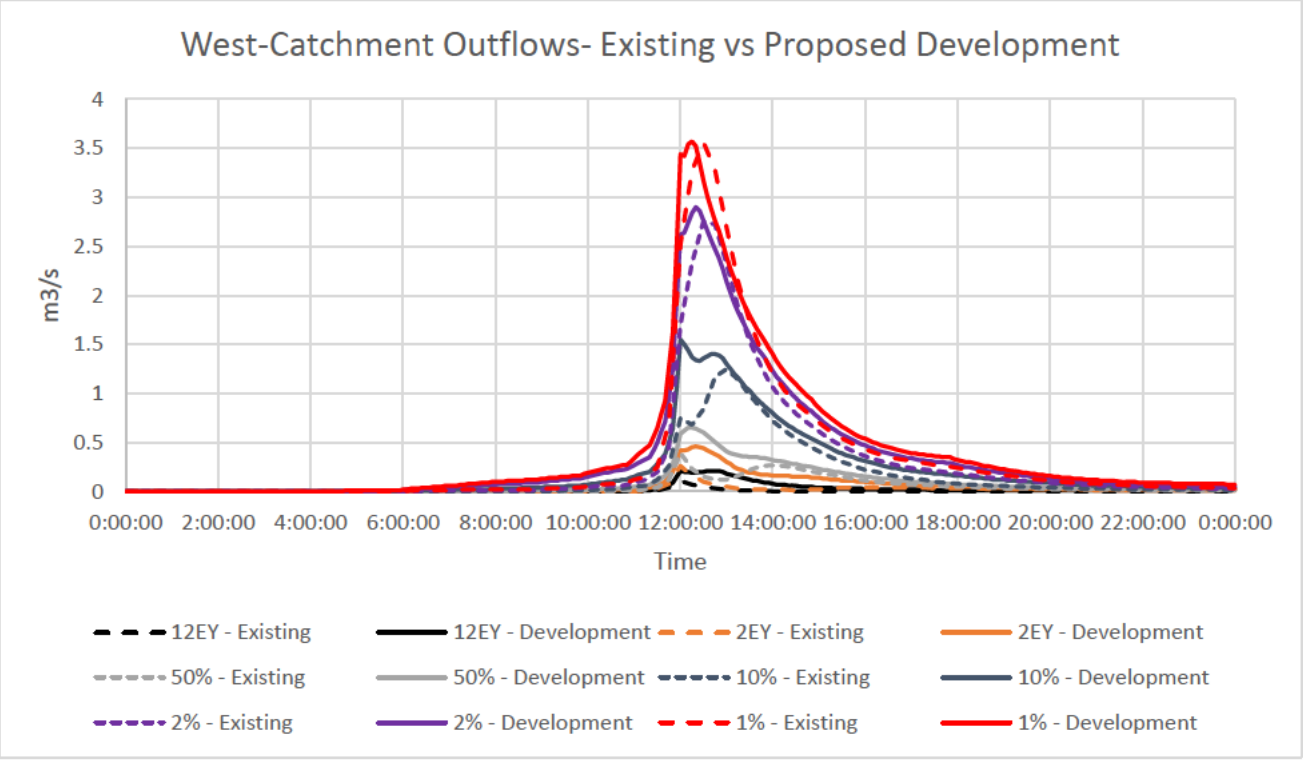


Figure 27: West Catchment – Time varying results for the >10% AEP results

BCD has calculated, at a high level, the volume the site would need to attenuate in order to keep reduce the storm event to its pre-development levels for the entire duration.

Table 16: Additional Detention Required for proposed west catchment

	Additional Detention Required
12 EY	1,687 m^3
2 EY	3,178 m^3
50%	3,127 m^3
10%	5,368 m^3
2%	5,820 m^3

1%	5,667 m³

4.2.3.5 *Southern Catchment*

The results showed that the proposed development would have a small, localised impact on the main overland flow path immediately upstream of the site. As with Section 4.2.3.3, this increase is the result of slight changes to Duncan Road, as the peak inflows are higher in the proposed development. The issue will be resolved as Duncan Road is further detailed.

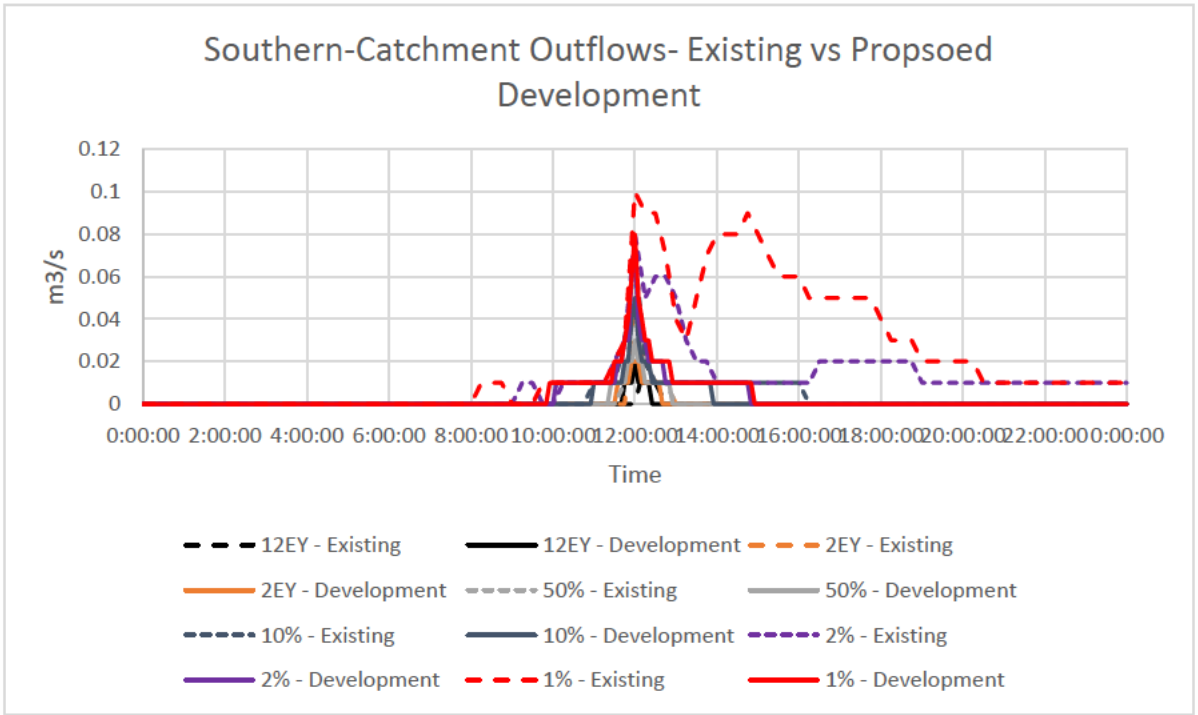
Overall, the inflows into the proposed site are increasing (i.e. the proposed development is not causing large scale blocking, restricting, or obstructing any flow.)

The 'Proposed – Blocked Channels" model indicated that a blockage to Culvert-5 would result in the South Catchment inflow decreasing. As a result, designs for Culvert-5 should ensure that it will not block.

The results indicate that proposed development will not impact the downstream sections of the southern catchment. This is due to most of its flow being re-directed to the western catchment

Table 17: South Catchment Inflow and Outflows

Main Catchment	Inflow – Existing (Climate Change)	Inflow – Proposed Open Channel	% change	Inflow – Proposed Blocked Channel	% change	Outflow – Existing (Historic)	Outflow – Proposed Open Channel	% change	Outflow – Proposed Blocked Channel	% change
12 EY	0.00 m^3/s	0.00 m^3/s	0%	0.00 m^3/s	0%	0.02 m^3/s	0.02 m^3/s	0%	0.02 m^3/s	0%
2 EY	0.00 m^3/s	0.02 m^3/s	-	0.02 m^3/s	-	0.05 m^3/s	0.02 m^3/s	-60%	0.02 m^3/s	-60%
50%	0.00 m^3/s	0.06 m^3/s	-	0.06 m^3/s	-	0.05 m^3/s	0.03 m^3/s	-40%	0.03 m^3/s	-40%
10%	0.19 m^3/s	0.25 m^3/s	32%	0.12 m^3/s	-37%	0.07 m^3/s	0.05 m^3/s	-29%	0.05 m^3/s	29%
2%	0.36 m^3/s	0.56 m^3/s	56%	0.18 m^3/s	-50%	0.08 m^3/s	0.07 m^3/s	-13%	0.07 m^3/s	-13%
1%	0.48 m^3/s	0.69 m^3/s	44%	0.22 m^3/s	-54%	0.10 m^3/s	0.08 m^3/s	20%	0.08 m^3/s	-20%



5 MITIGATION RECOMMENDATIONS

5.1 Overview

BCD has received a masterplan of the proposed development and a high-level design surface for the proposed development has been created. However, the design surface is still being detailed. The proposed development flood modelling has assumed the exclusion of stormwater mitigation and control structures, such as culverts, weirs, and piped connections between the swales and irrigation ponds to outlets. This exclusion is due to the GeoHECRAS software being limited in its ability to both include and accurately model these structures in a 2D stormwater model.

Based on the modelling results, we can provide some catchment and locations specific design recommendations for handling stormwater on-site.

5.2 Northern Catchment

Flood mitigation for the northern catchment will involve some weir structures within the swales to slow down the smaller (<10% AEP) flows and retain them more within the swales. The mitigation will also involve the use of the central pond for attenuation, with controlled piped inlets and outlets between the northern catchment swale and the central pond.

5.3 Main Catchment

Flood mitigation for the main catchment will involve some weir structures within the swales to slow down the smaller (<10% AEP) flows.

It is noted that outflows results from the “Proposed - Culverts Blocked” scenarios are lower than the existing for all <2% AEP events. The Culvert-2 will, therefore, be sized to hold back some of the flow without causing additional flood issues within the site. Culvert-2 is the best candidate for this as it appears to attenuate a reasonable amount of flooding within an area designated for overflow carparking.

Under sizing Culvert-1 risks flooding the shared facility while under sizing Culvert-4 risks flooding the residential building area. Under sizing Culvert-3 does not appear to have any benefits in attenuation.

The mitigation will also involve the use of the central and western ponds for attenuation, with controlled piped inlets and outlets between the main catchment swale and the northern attenuation pond. As the Main Catchment will share the central pond with the northern catchment, designs for the pond’s outlet structures will need to consider that the northern pond’s inflow does not increase the outflow to the main catchment.

5.4 Western Catchment

Flood mitigation for the western catchment will involve some weir structures within the swales to slow down the flows. The mitigation will also involve the use of the southern pond for attenuation along with the ponds within the Senior Living Precinct. The results indicated that the western catchment is mostly achieving it required attenuation.

5.5 Southern Catchment

The Southern Catchment has no downstream impacts; however, the model results do show some upstream impacts resulting from changes to Duncan Road. Mitigation here will involve further detailing to Duncan Road to ensure the upstream flooding does not increase. Culvert-5 should have sufficient size and capacity to avoid blockage.

6 CULVERT CAPACITIES

The Proposed Development – Open channel model results indicate the approximate capacities required by each culvert to meet the open channel flow.

As noted above, the proposed design will use the culverts as a form mitigation, to help the proposed design reduce outflows from the main and western catchments. As a results, the culvert design capacities may be smaller than the capacity required for the open channel.

Table 18: Culvert Capacities

Event	Culvert-1	Culvert-2	Culvert-3	Culvert-4	Culvert-5
12 EY	0.34 m3/s	0.52 m3/s	0.23 m3/s	0.11 m3/s	0.12 m3/s
4 EY	0.51 m3/s	0.78 m3/s	0.36 m3/s	0.15 m3/s	0.18 m3/s
2 EY	0.86 m3/s	1.00 m3/s	0.43 m3/s	0.19 m3/s	0.22 m3/s
63%	1.20 m3/s	1.15 m3/s	0.53 m3/s	0.21 m3/s	0.25 m3/s
50%	1.59 m3/s	1.33 m3/s	0.59 m3/s	0.23 m3/s	0.31 m3/s
20%	2.74 m3/s	1.95 m3/s	0.89 m3/s	0.34 m3/s	0.46 m3/s
10%	3.91 m3/s	3.36 m3/s	1.37 m3/s	0.40 m3/s	0.63 m3/s
5%	5.53 m3/s	4.81 m3/s	1.77 m3/s	0.49 m3/s	0.86 m3/s
2%	8.06 m3/s	7.10 m3/s	2.84 m3/s	0.58 m3/s	1.13 m3/s
1%	9.85 m3/s	8.69 m3/s	3.65 m3/s	0.71 m3/s	1.40 m3/s

7 POND VOLUMES

While the current model does not include stormwater mitigation and control structures, such as culverts, weirs, and piped connections between the swales and irrigation ponds, the volume required for the attenuation ponds can be calculated by summing the volume already being used and the additional volume required to achieve hydraulic neutrality.

As a check, a HEC-HMS model was run with the same inputs GeoHECRAS model. The runoff volumes produced by the Geo-HECRAS software were ~25% lower than the volumes produced by the HEC-HMS.

It should be noted, as previously mentioned in the report, that GeoHECRAS also does not account for infiltration from ponded depressions. Meaning, on a flat catchment, it will also likely overcalculate the amount of runoff that reaches the site.

While the difference between GeoHECRAS and HEC-HMS these had no effect of the depths and little impact on the peak flows, the concern is that the attenuation volumes calculated using GeoHECRAS's results may be too small.

As a result, the required attenuation volumes have been increased by 25% over the volume's calculated by GeoHECRAS's results. As this accounts for GeoHECRAS's undercalculation of runoff from rainfall but does not account for GeoHECRAS's overcalculation of the volume reaching the site, these values are likely conservative and will be refined in detailed design.

Table 19: Northern and Western Pond Volumes

Event	Northern Pond		Western Pond		Volume required to achieve hydraulic neutrality		Difference
Maximum Volume	100,326 m³		15,249 m³				
Location	Track Runoff	Spare Capacity	Track Runoff	Spare Capacity	Northern Catchment	Main Catchment	
12 EY	4,653 m³	95,674 m³	1,133 m³	14,117 m³	3,293 m³	933 m³	105,565 m³
2 EY	10,225 m³	90,101 m³	2,199 m³	13,050 m³	8,021 m³	5,451 m³	89,679 m³
50%	14,886 m³	85,440 m³	2,891 m³	12,358 m³	10,598 m³	5,996 m³	81,204 m³
10%	33,693 m³	66,634 m³	5,560 m³	9,689 m³	23,863 m³	15,948 m³	36,513 m³
2%	54,674 m³	45,652 m³	8,491 m³	6,758 m³	39,560 m³	27,343 m³	-14,493 m³*
1%	64,314 m³	36,012 m³	9,961 m³	5,288 m³	50,758 m³	32,829 m³	-42,286 m³*

**these additional volumes are detained in dedicated overflow areas on the site, namely the temporary carpark. Refer to the mitigation outlined below*

Event	Southern Pond		Volume required to achieve hydraulic neutrality	Difference
Maximum Volume	4,005 m³			
Location	Volume used	Spare Capacity	Southern Catchment	
12 EY	1,814 m³	2,191 m³	2,109 m³	83 m³
2 EY	2,424 m³	1,581 m³	3,973 m³	-2,391 m³*
50%	2,663 m³	1,343 m³	3,909 m³	-2,566 m³*
10%	3,808 m³	198 m³	6,710 m³	-6,513 m³*
2%	4,848 m³	-843 m³	7,275 m³	-8,118 m³*
1%	5,006 m³	-1,001 m³	7,084 m³	-8,085 m³*

these additional volumes are detained in dedicated overflow areas on the site, namely the temporary carpark. Refer to the mitigation outlined below

Although the volumes calculated above are likely conservative, they do indicate the ponds do not presently meet the attenuation requirements. The additional attenuation volume for the northern/main catchment will be accounted for in the following ways and in the following order, as required, once the additional volumes are refined:

- Installing weirs to use more of the Main Swales' capacity.
- Encouraging additional ponding within the Back Tack. Analysis of the model shows that the Back Track is presently overflowing into the Main Catchment Flooding swale. As the Back Track's area is largely within the existing western catchment, this overflow is having a notable impact. Some form of masonry wall along the Back Tack's northern ridge will reduce this overflow and, therefore, the attenuation requirements.
- Increase the depth of the Western Pond by 1m.
- Expanded the area of the Western Pond.
- Under sizing Culvert-2 for >10% AEP events, allowing the proposed design to use the overflow carpark as additional storage.

  As noted in the discussion of the wetland, one of the detailed design intents already involves directing this flow towards the wetland.
- Installing weirs to use more of the Main Catchment Swales' capacity.
- Expanding the Central Pond, if required.

The additional attenuation volume for the western catchment will be accounted for by

- Installing weirs to use more of the Western Swales' capacity.
- The ponds located within the Senior Living Precinct.

8 MITIGATION OF CLIMATE CHANGE IMPACTS

The stormwater management plan for the development will reduce the impact of climate change via two methods.

1. The post-development flood modelling of flood depths and velocities used, as indicated above, climate change adjusted rainfall intensities to ensure that that onsite flooding accounts for climate change.
2. The final sizing of the stormwater attenuation ponds will use historical rainfall for the existing runoff and climate change adjusted rainfall for the post-development runoff. This will ensure that the ponds will restrict downstream outflow to the historical existing, not just the existing situation under a climate change adjusted scenario. This ensures the management plan not only mitigates the impact of the site on the downstream, but the impact of climate change.

9 CURRENT CONCLUSIONS

The flood modelling determined that:

- Under the proposed design the site will not experience any notable (>100 mm) locations of flooding that coincide with proposed buildings, provided that Culvert-1 (located near the shared facility) and Culvert-5 (located in the Rural Living Precinct) are designed to avoid any blockage.
- The Northern and Main catchment will require notable attenuation to match the historical existing outflows. The Northern catchment will use the central pond while the main catchment will use the Central and Western ponds. The main catchment can also utilise Culvert-2 and the overflow carparking area for attenuation during some of the larger storm events; however, the preference will be to increase the size of the western pond.
- The northern ridge of the Back Track will also have some form of masonry wall to help retain runoff within the track, reducing overflow to the Main Catchment and reducing the needs for additional infiltration. This will tie into the detailed design intention of directing more water towards the wetland.
- Excluding the swales, the site has two local overland flows: one through the through the Shared facility, the other through the overflow carparking area.
- Even without the stormwater mitigation infrastructure (weirs and connections to culverts), the depth impacts upstream and downstream of the site are minor, localised, and close to the site's boundary.
- Due to the area being flat and the proposed design surface still being in flux, it is important that this is an ongoing consideration.
- The proposed swales are adequate to cater for the design flows.

For minor events, the Kahikatea stand floods the same under the proposed design as it does under the existing. It can be concluded that the proposed development will not worsen the wetland.

10 LIMITATIONS

This report has been prepared for our client for their purposes. It is not to be relied upon or used out of context by any other person without reference to BCD. The reliance by other parties on the information or opinions contained in this report shall, without prior review and agreement in writing, be at such parties' sole risk.

Modelling completed for this report was undertaken in GeoHECRAS Version 6.1.0.3985, based on a survey of the site. If the surrounding surface changes, the model inputs would need to be updated to ensure the results are still accurate and valid.

Appendix A Model Review Memo

Memorandum

Peer Review of Flood Model Assessment

Waikato Greenfield Racing Hub

Introduction

BCD Group have worked on a flood model as part of assessing flood risk and mitigation for the proposed racing hub

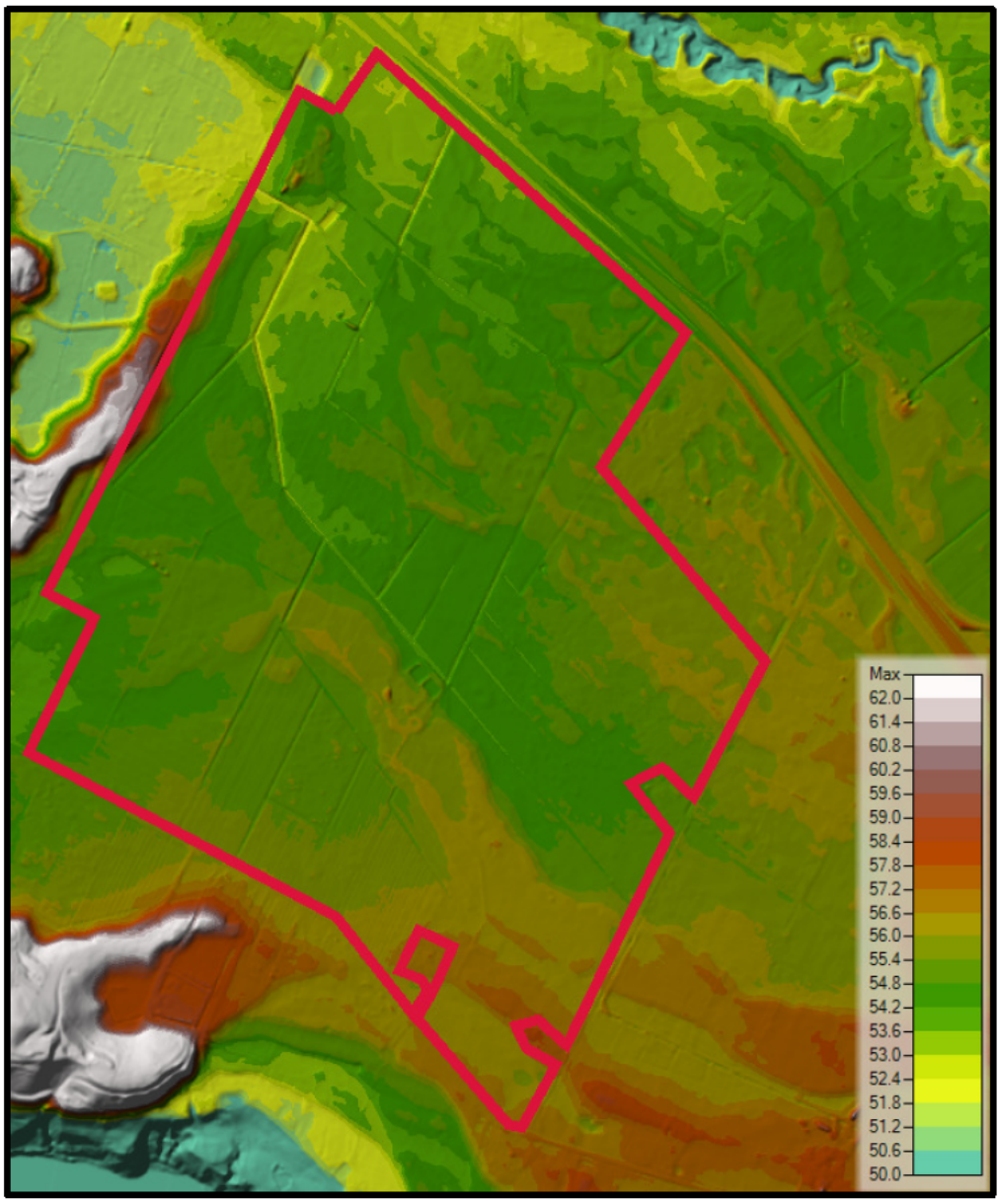
This peer review is based on the receipt of the GeoHEC-RAS flood model files and the flood assessment report dated 28th August 2026

GeoHEC-RAS and HEC-RAS use the same algorithm engine but operate in a different environment. The former requires an ARCGIS license and is used in the analysis and preparation of engineering design surfaces and structures. The latter is just a flood model software that is free to download. To this end the working environments are slightly different but the base file requirements are still the same.

Proposed site

Figure 1 shows the topography within the site. The ground levels range from about RL58m to RL53m with numerous farm drains convey run-off generally to the northern apex of the site.

Figure 1 – Site topography



The proposed site layout

Figure 2 shows the proposed features of the site for the mitigation structures.

Figure 2 – Proposed site features and mitigation features



Figure 5: Proposed developed site with swales (orange) and attenuation ponds (green). Note, the current proposed is to have just a large central pond, rather than a northern and central pond

Scenarios and plan data

In the first instance 4 scenarios will be checked. These are

1. Existing environment with historical rain,
2. Existing environment with climate change rain,
3. Proposed land-use with open channels and climate change rain,
4. Proposed land-use with blocked culverts and climate change rain.

Scenario 3 is to assume that flows can pass unimpeded through stopbanks or otherwise where culverts exist or are proposed to exist.

Scenario 4 assumes water is blocked where culverts exist or are proposed to exist.

It is presumed that scenarios 3 and 4 reflect the worst-case scenarios for both extremes.

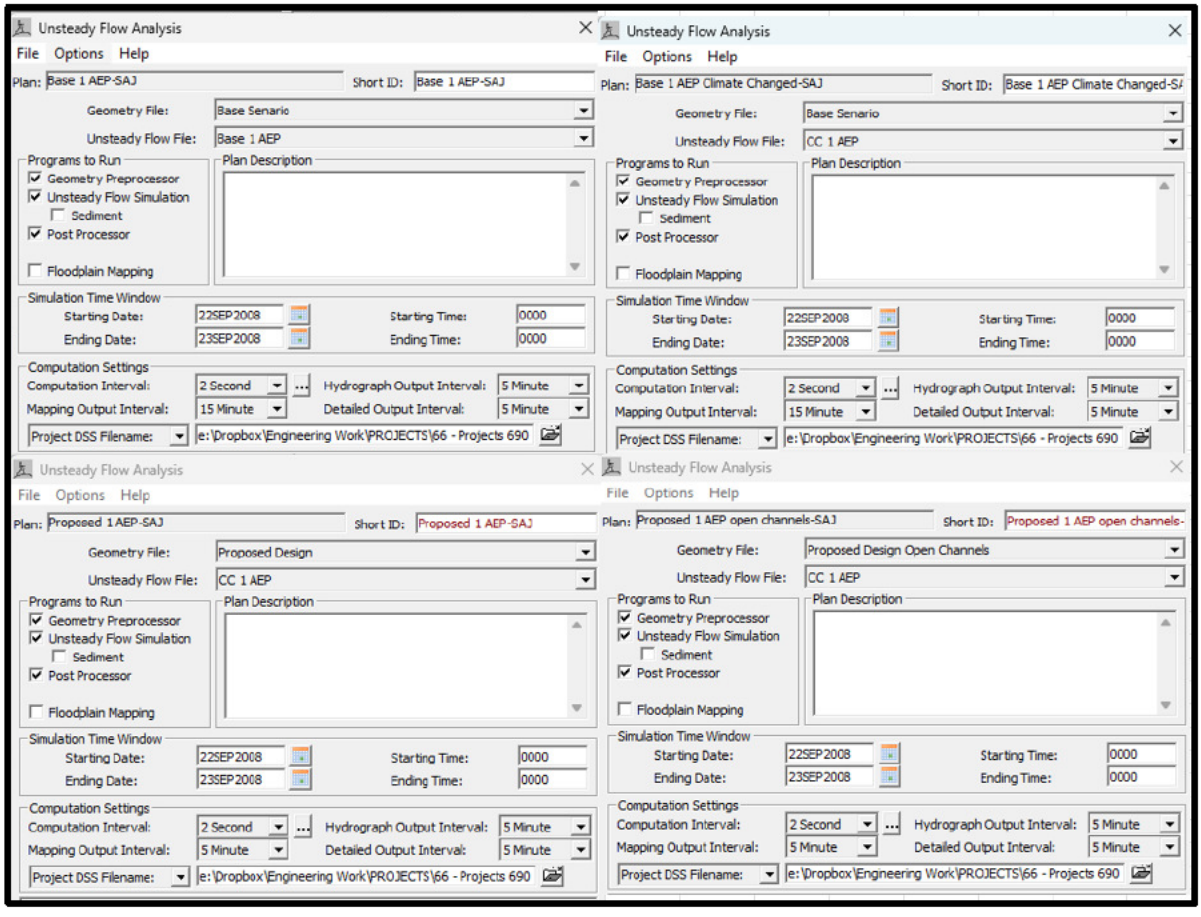
I created and ran my own plans of the 4 scenarios to ensure they run and to compare the results with those provided.

Figure 1 shows a screenshot of the geometric file associations. Figure 2 shows the Plan files.

Figure 1 – Geometry file associations

Type	RAS Geometry Layers	Terrain	Manning's n	Infiltration
Geometry	Base Senario	20260817 Pre-Development LIDAR + Survey + Stamp 231883_Base 1 AEP Climate Changed	260803 Land Cover Existing 231883	260901 Existing Infiltration
Geometry	Proposed Design Open Channels	260826 design surface Open Channels with LIDAR and Survey	260804 Land Cover Proposed Harness Track V3 231883	260826 Proposed Infiltration
Geometry	Proposed Design	260826 design surface blocked culverts with LIDAR and Survey	260804 Land Cover Proposed Harness Track V3 231883	260826 Proposed Infiltration

Figure 2 – All scenarios plan files



Terrain

The latest LiDAR DEM file was uploaded from the LINZ website. It was compared to the existing surface used in scenarios 1 and 2. I can confirm it is correct and that NZVD has been used.

I reviewed the proposed ground surfaces

260826 design surface Open Channels with LIDAR and Survey_Proposed 2 AEP open channels.260tif

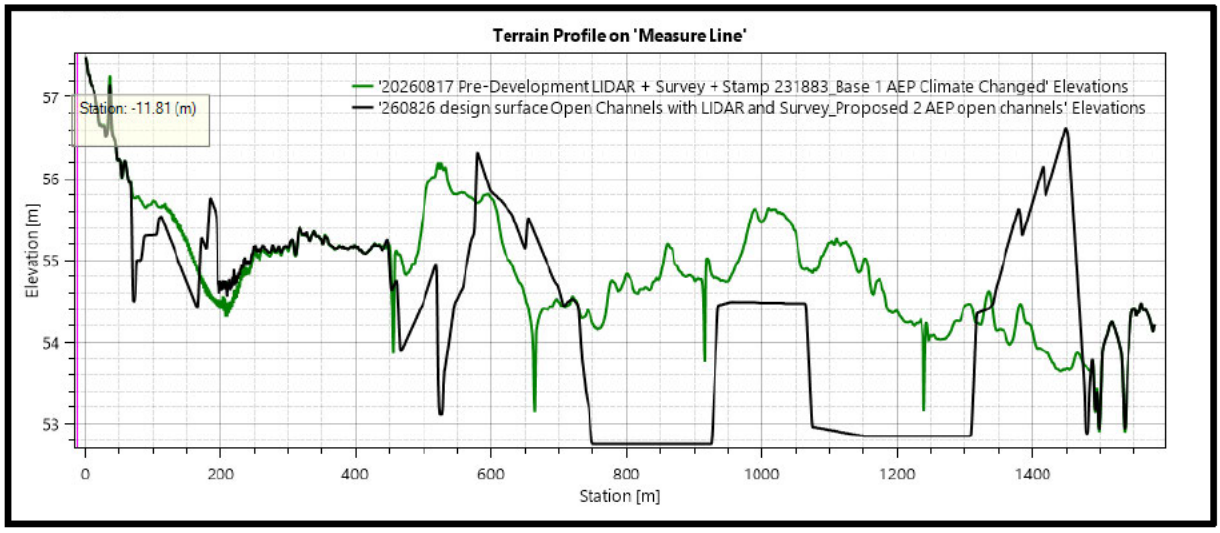
and

260826 design surface blocked culverts with LIDAR and Survey_Proposed 2 AEP.260tif

They were imported from a design surface, not by terrain modification method. The report did not state any design surface elevation for the ponding areas. I assume they match the design drawings. The bases of the ponds for northern, central and western are RL52.85m, RL52.75m and RL52.90m respectively.

A south-to-north transect from boundary to boundary through the main course is shown in Figure 3 for existing and proposed open channel. It shows the cut for the north and central ponds are about 1.5m to 2.0m deep.

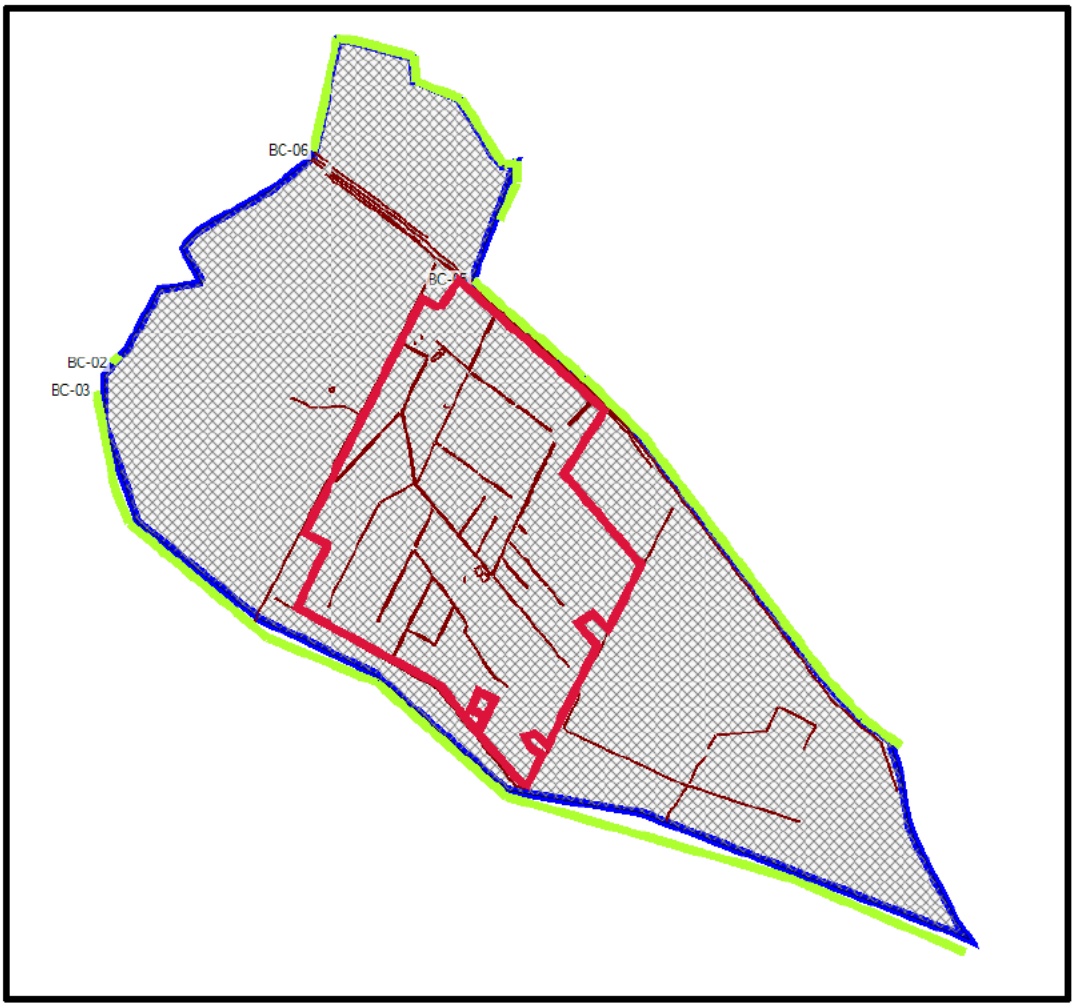
Figure 3 – South-to-north terrain transects



Geometry

Figure 4 shows the exiting geometry. The grid cells are hexagonal (available in GeoHEC-RAS only and are 15m x 15m with an average cell size of 192m^2, not 15m^2 as quoted in the report. 141 breaklines are used with 15m spacings. Many seemed to be redundant along drains and not enforced. Some along the road centre-line/high point are mis-aligned. Breaklines are useful to refine the grid along high points and drains, and the spacings should comfortably work is it is 3rd the size of the main grid. I am not sure if it makes a material difference to the results but it is untidy. No refinement areas have been used and that seems reasonable. However, it may be informative to run the 1% AEP existing model with a 5m grid just to see if it makes a difference to the results (it will but is it worth?). Ther number of cells is only 27,000 and takes only 10 minutes to run of the BCD machine so it worth a try taking about 90 minutes to run.

Figure 4 – Existing geometry



There are 4 boundary files with a Normal Depth of varying gradients. This seems reasonable. Pencarrow and Hooker Roads overflow at few points and personally I would have used the road's high point as the grid boundary and shorter outflow boundaries where they do occur. The most downstream boundary to the west seems a reasonable distance with a low risk of backwater impacts due to the 5m drop in terrain on a large potential flooded area.

Landcover

The following files have been used for landcover

260803 Land Cover Existing 231883

260804 Land Cover Proposed Harness Track V3 231883

For the existing landcover the Manning's n is 0.03 for "agriculture". I would have used 0.045 for thick grass and shallow flows.

Figure 5 shows the n-values used in the proposed scenario.

Figure 5 – Landcover for proposed

ID	Name	ManningsN
0	NoData	0.045
1	Agricultural	0.03
2	Open Water	0.035
3	Commercial	0.02
4	Residential	0.035
5	Paved parking lots, roofs, driv...	0.016
6	1-8 acre (town houses)	0.1
7	Trainers Area	0.02
8	1-2 acre	0.05
9	Roadways	0.016
10	Track Surface 1	0.03
11	Track Sufrace 2	0.03
12	Track Surface 3	0.03

I assume IDs 3, 4 and 6 are solid buildings (does it include the grandstand?) The n values should be at least 1. Apart from my earlier comment on "agriculture", the other values are okay.

infiltration

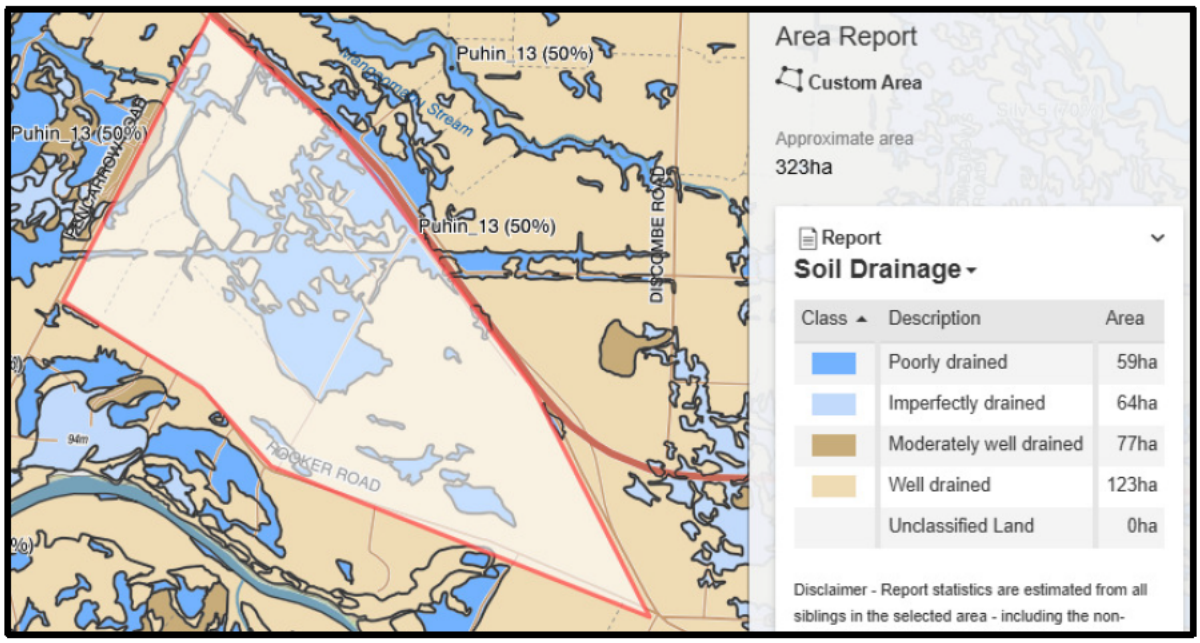
The files used are

260901 Existing Infiltration

260826 Proposed Infiltration

For the existing infiltration the impervious curve number is 98 and 95 for roadways and residential areas which is fine. Agriculture is 86 suggesting very poorly drained grasslands. A quick check on the Landcare Research S-maps gives the output in Figure 6. About 2/3rds is well or moderately well drained. This high curve number needs to be justified.

Figure 6 – Soil map



Rainfall

The rainfall has been expressed in mm/hr average (Table 2) instead of the usual mm in 24-hours. The values are correct, but total mm depths help in check in the run-off volumes from the model. The distribution used is correct and the 1% AEP event is correct in the model.

Results

The 1% AEP event for the Existing and open channel models were compared for the received models and the one I ran on my machine. The only change I made was manning's n was input as 0.045 whereas the model received was blank.

Interestingly my model was 20-40mm lower around the whole catchment than the BCD model. I don't think it is important, but I wanted to ensure the model could run.

I check the results for maximum water levels at a number of locations to ensure the return period vs. WL was correct and they were.

Volume check

I don't particularly trust the hydrology component of HEC-RAS. Figure 7 shows the run-off volumes in mm for the 1% AEP scenarios using HEC-HMS with an area of 5.35km2 and curve number of 86 and Ia of 2.1mm. Table 1 shows the summary of run-off in mm based on the log files of the received results files compared to the simple HEC-HMS analysis.

Figure 7 – HEC-HMS outputs

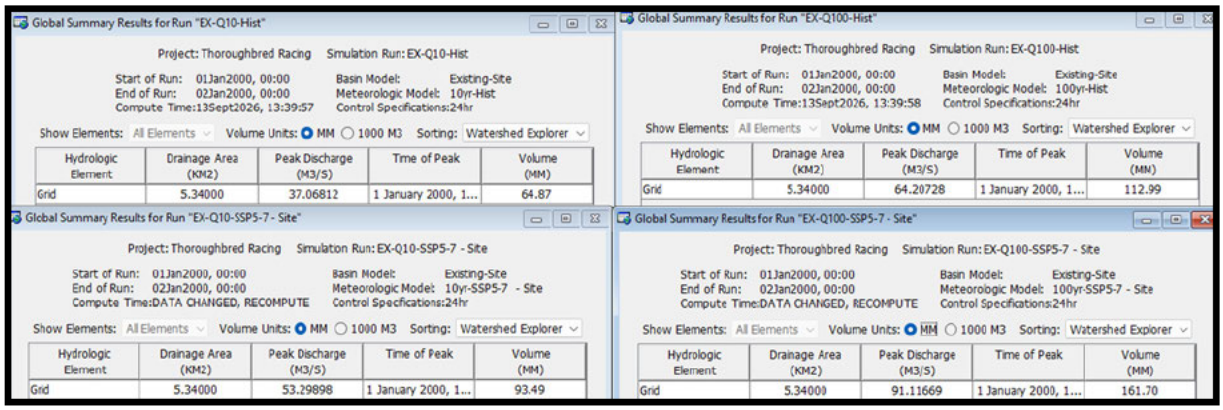


Table 1 – Comparison of GeoHEC-RAS and HEC-HMS volumes of run-off for existing terrain (mm)

	10% AEP			1% AEP		
	Total rain	**Geo-HEC-RAS**	**HEC-HMS**	**Total rain**	**Geo-HEC-RAS**	**HEC-HMS**
Existing	97	44	64	149	87	113
Proposed	128	75	94	200	137	161

The GeoHEC-RAS values are about 15 to 30% lower than expected. With a curve number of 86 the volumetric run-off should be about 70-80% depending on the absolute rain depth (the more rain the closer to 80%). This needs to be checked.

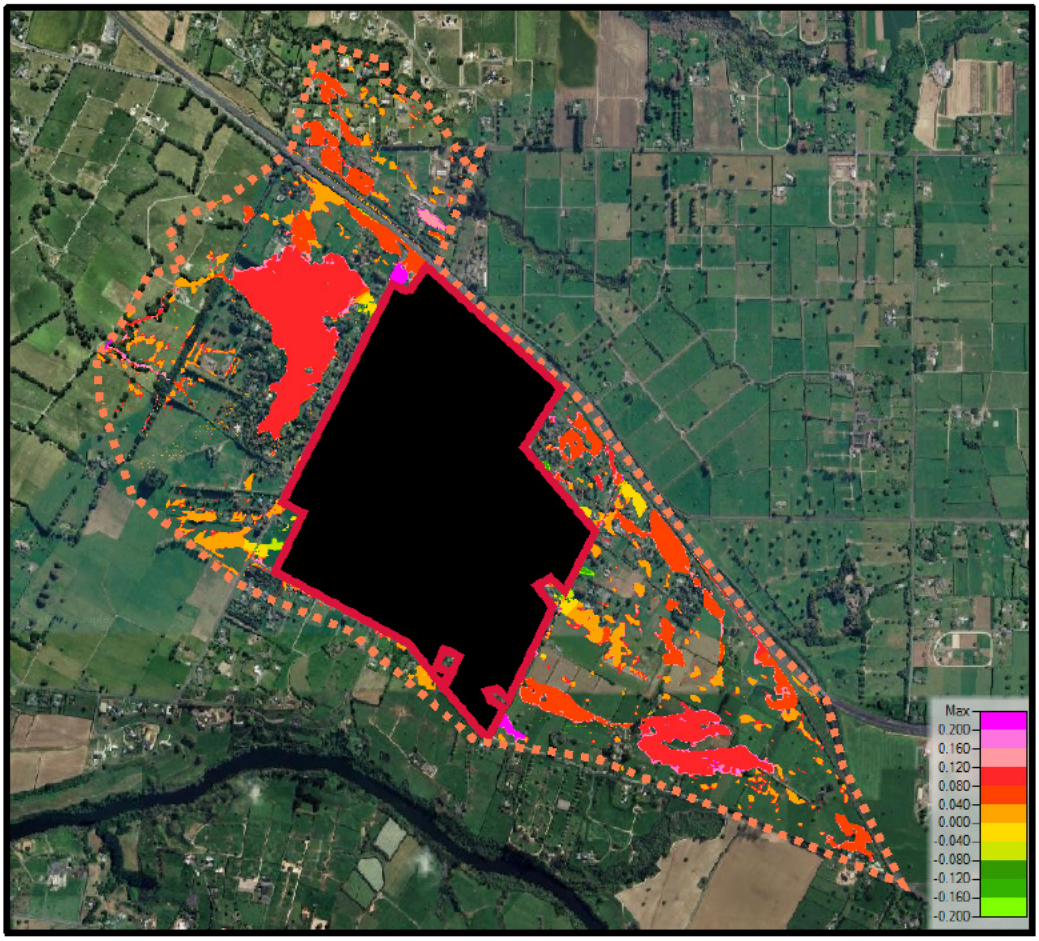
It would be good to have a calculation of the total new impervious area.

Flood displacement

My understanding of RITS is that pre-development flows are based on current rainfall and post-development is based on climate adjusted rainfall. Section 4.2.3 of the report shows no impact for climate change rain only.

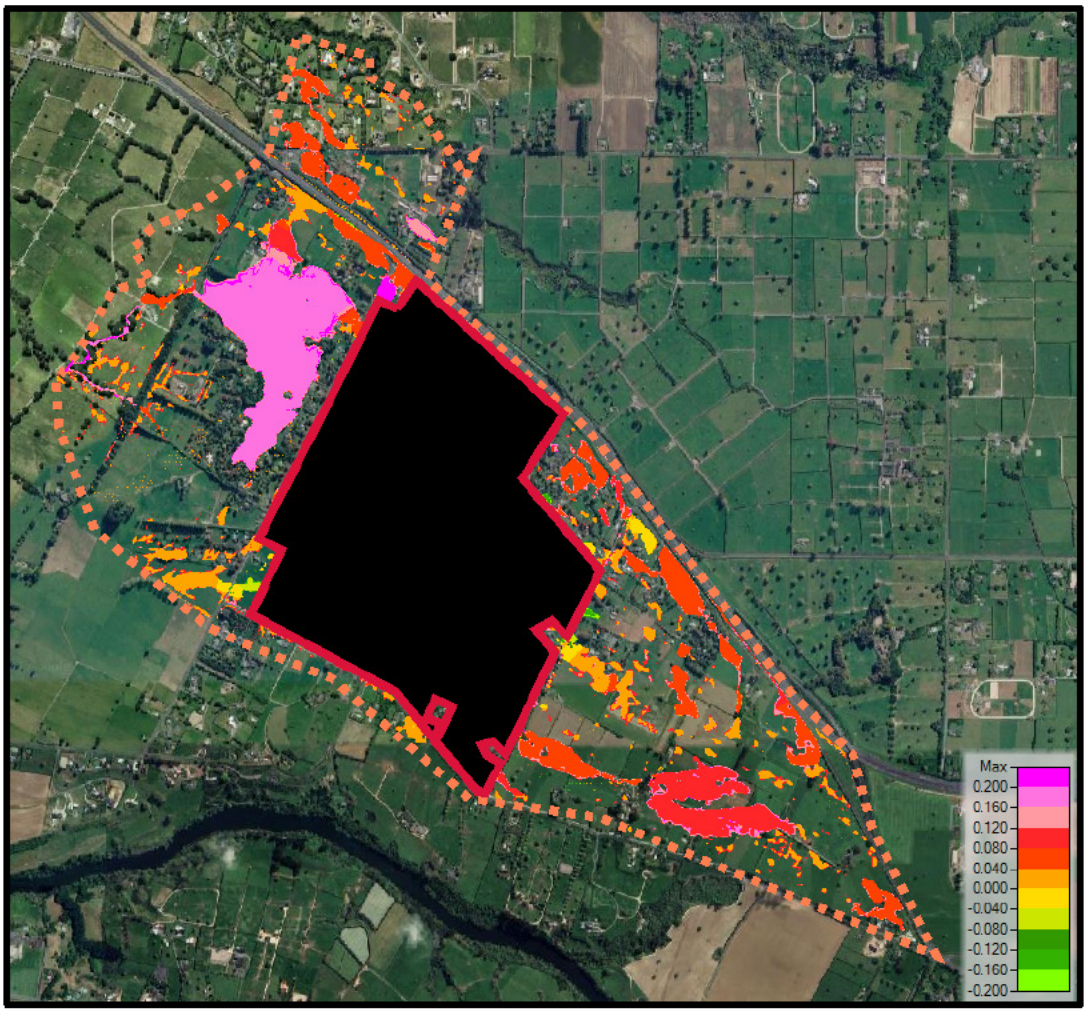
I have created a flood displacement map, Figure 8, for the proposed scenario with the blocked culverts against the existing situation. I have redacted the site because it is not important for this comparison. Parts of the neighbouring properties upstream and downstream have flood levels rise of at least 90mm. This may not be acceptable to Council.

Figure 8 – Flood displacement for proposed culvert blocked



I have created a flood displacement map, Figure 9, for the proposed scenario with the open channels against the existing situation. I have redacted the site because it is not important for this comparison. Parts of the neighbouring properties upstream and downstream have flood levels rise of at least 180mm. This may not be acceptable to Council.

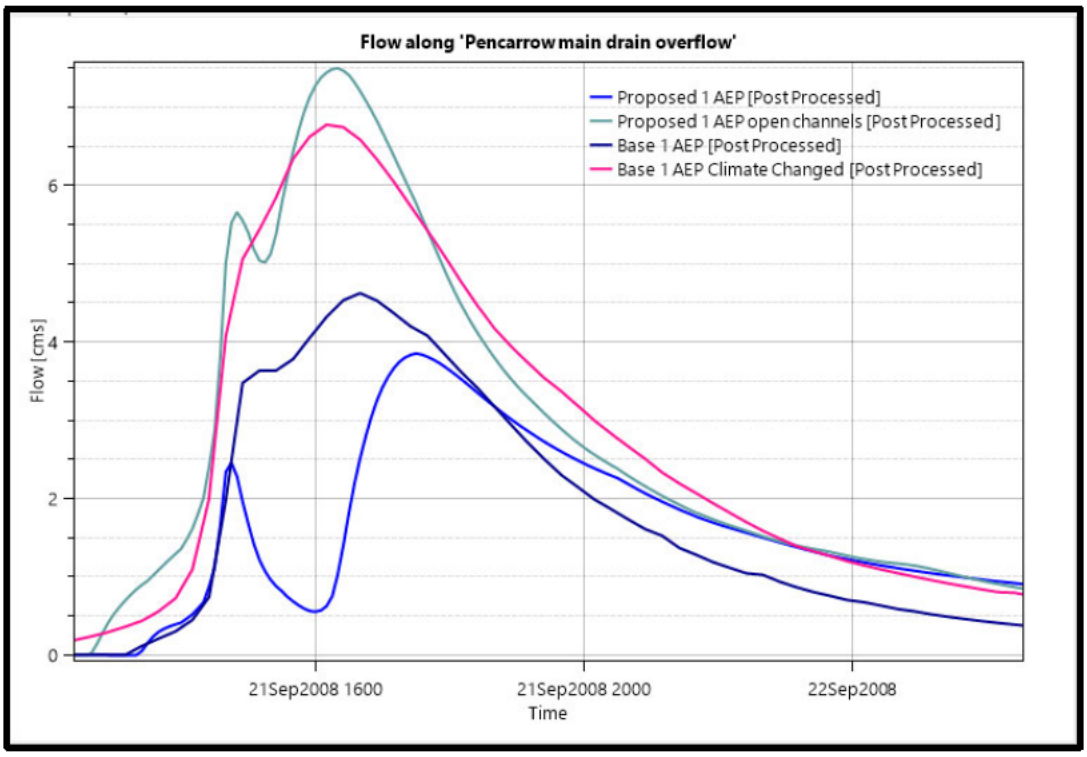
Figure 9 – Flood displacement for proposed culvert blocked



Flow mitigation

I have produced flow hydrographs over Pencarrow Road at the Main Drain Catchment outflow, see Figure 10. This is for the 1% AEP only. The existing overflow peak is 4.5m^3/s, the proposed blocked channels is 3.8m^3/s, the proposed open channel is 7.5m^3/s and the existing terrain with climate changed rain is 6.7m^3/s. According to RITS requirement blocked drains work but open channels will impact downstream.

Figure 10 – Main catchment outflow



Conclusion

The general set up of the model is acceptable but with the possible changes:

1. Better breaklines cell spacings, maybe try 5m,
2. Try a smaller general cell size, maybe 5m,
3. N-value for agricultural might be too smooth,
4. Check other n-values in proposed land-use

My biggest concern is the run-off.

1. Is the soil type correct? Is it type C, the soil maps suggest a lower curve number?
2. The volume check for the excess rainfall does not compute and seems to produce a volume that's too low. Can you check this?

Are you using the correct rainfall for the pre-development comparison to comply with RITS?

The problem with using an "open channel" and "blocked culvert" approach is that you don't really know the real answer until you apply something realistic. I get you are

trying to get a feel for where the flooding is problematic but you can't know you have provided the right mitigation until culverts are applied.

Addendum

The memo was supplied to BCD on the 13th of September. I report the following:

Better break-line alignment has been implemented.

A smaller cell size of 5m was tested with no material change in results.

N value for agriculture now 0.045.

Run-off soil type C accepted due to known waterlogging/poor drainage than soil type suggests.

Volume check still does not work out for rainfall excess expectation in GeoHEC-RAS. I understand BCD are requesting support from the software supplier. My concern is that more storage for mitigation may be required than is currently shown. To expedite the preliminary design process BCD have offered the following.

> 'As a check, a HEC-HMS model was run with the same inputs GeoHECRAS model. The runoff volumes produced by the Geo-HECRAS software were ~25% lower than the volumes produced by the HEC-HMS. This raised concerns regarding GeoHECRAS's ability to calculate infiltration, resulting is less runoff from an equivalent rainfall. It should be noted, as previously mentioned in the report, GeoHECRAS also does not account for infiltration from ponded depressions. Meaning, on a flat catchment, it will also likely overcalculate the amount of runoff that reaches the site. While the difference between GeoHECRAS and HEC-HMS these had no effect of the depths and little impact on the peak flows, the concern is that the attenuation volumes calculated using GeoHECRAS's results may be too small. As a result, the required attenuation volumes have been increased by 25% over the volume's calculated by GeoHECRAS's results.'

I consider this a reasonable, pragmatic interim approach while the volume error is resolved.

In conclusion, the modelling is fit-for-purpose in terms of setting design parameters in readiness for more detailed modelling of culverts, storage and final flood design levels.

Dr Steven Joynes

28th September 2026

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