Preliminary Flood Model Assessment Report

JOB NUMBER: 23-1883

WAIKATO THOROUGHBRED RACING GREENFIELD HUB

PROJECT

WAIKATO THOROUGHBRED RACING

CLIENT

DRAFT REFERRAL – REV 5

16th APRIL 2026

Preliminary Flood Assessment Report

FAST TRACK REFERRAL APPLICATION **PENCARROW ROAD, WAIKATO**

Prepared for: Waikato Thoroughbred Racing
Project status: Referral

Date: 16-4-2026
Project no: 25-1883
Revision: 5

Prepared by:

Tim Durkin-Wright
CIVIL ENGINEER
BCD Group Ltd

Reviewed and approved for release by:

Callum Davison
HAMILTON OFFICE MANAGER
BCD Group Ltd

Contents

1 INTRODUCTION AND SCOPE

Waikato Thoroughbred Racing has engaged BCD Group to complete a FAST TRACK Referral Application work for the proposed Waikato Thoroughbred Racing Greenfield Site – Pencarrow Road. Part of this due diligence 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.

This report focusses on the flood assessment, the objectives of which are to:

- Identify existing sources of onsite flooding, including:
  - Providing an estimation of flows in the site's existing agricultural drains during large storm events
  - Providing an estimation of top of flood level across the site to aid with potential Finished flood levels during large storm events
- Provide potential solutions to resolve flooding on site.

1.1 Project Description

The purpose of the Project is to create a unique, world class greenfield 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.

1.2 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 into a drain running along 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.

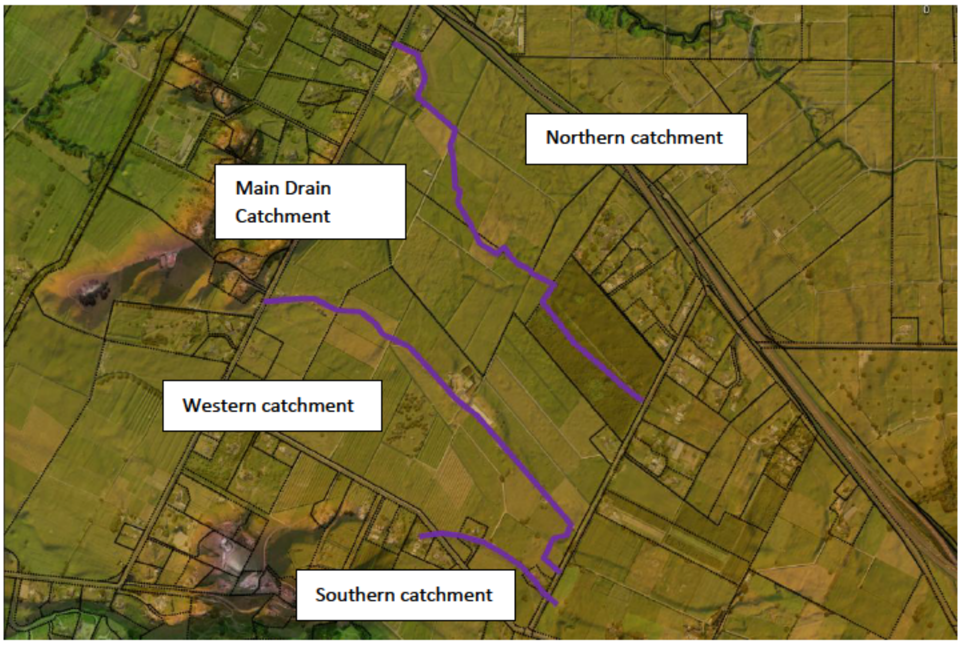

Figure 3: Site Location – ridgelines across the site (purple) split the site.

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 with 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 with 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
    - Catchment area with the site: 41Ha
- 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 with 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.

1.3 Consultation

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

As discussed in BCD pre-application due diligence, 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 area. The resource maps do indicate that the main drain running through the site is classified as a stream.

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

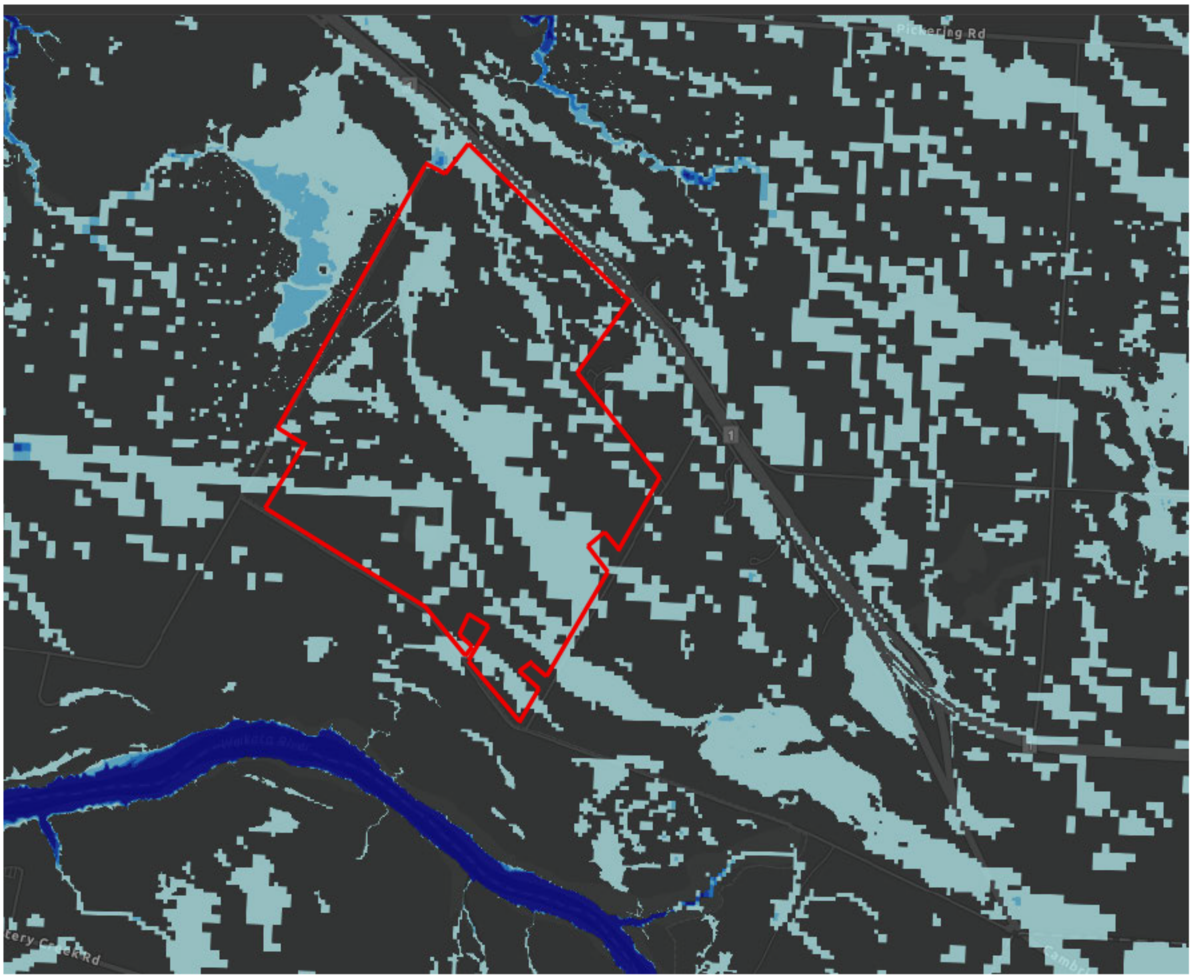

Figure 4: NIWA Flood Hazard results.

1.4 Design Methodology

The design methodology for on-site development and stormwater management will coordinate with the Masterplan, the architect, and with the three waters design. It is expected that regardless of the site layout and the design approach, the following will be required:

- 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. The new stormwater network will account for the proposed site layout, roads, buildings and etc.
- Attenuation ponds to be sufficiently sized to ensure that the post development runoff matches the pre-development runoff. This hydraulic neutrality should be ensured for both individual catchments and the site in its totality. As part of this hydraulic neutrality:
  - Upstream and downstream flows are to remain unchanged.
  - There is to be no impact on the roads, cycle way, and SH1.

2 GEO-HECRAS STORMWATER MODEL

2.1 Summary

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

- 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 and post-developed flows based on the NIWA HIRDS V4 RCP6.0 2081-2100 24-hour rainfall events intensity.
- Infiltration and land use, based on if the area was Agricultural, Roadway, or Residential (see section 2.3.3)
- A mesh size of 10 m^2 (30 m^2 was used initially to check the upstream catchment)

2.2 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 the stormwater did pond on the easter 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.

2.3 Modelling Methodology

2.3.1 Model Extents

Initial modelling showed that Cambridge Road was the upstream extent of the catchment impacting the site, along with confirming the behaviour of the downstream catchments. This allowed 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. BCD set the model boundary along the ridge of SH1 and included a boundary condition (allowing outflow) at Pickering Road.
- 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 5: Site Model Extents

2.3.1 Modelled Ground Surface and Surface changes

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 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. Furthermore, the survey investigated the on-site and surrounding culverts. Most of the culverts were less than 600mm in diameter, allowing 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.

2.3.2 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 all open channels, levies, and roads. The drain break lines were derived from the Top-of-bank/Bottom-of-bank/channel centre lines determined by the survey.

2.3.3 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).

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
Agricultural	**0.03**	**0%**	**84**	**0.1**
Roadway	**0.013**	**100%**	**99**	**0.1**
Residential	**0.016**	**50%**	**91**	**0.1**

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 - 0.05	0.045	0
Grassland	8	Grassland/Herbaceous	0.025 - 0.05	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 - 0.085	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 - 0.085	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

2.3.4 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. These rainfall values are used for both pre- and post-development BCD used the following RCP6.0 2081-2100 24-hour rainfalls for Pencarrow Road.

- 10% AEP: 4.88 $^{mm}/_{hr}$
- 5%: AEP: 5.64 $^{mm}/_{hr}$
- 2%: AEP: 6.73 $^{mm}/_{hr}$
- 1%: AEP: 7.60 $^{mm}/_{hr}$

Rainfall intensities (mm/hr) :: RCP8.5 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	68.2	47.8	38.4	25.9	16.8	8.00	4.82	2.86	1.65	1.19	0.935	0.776
2	0.500	75.6	52.9	42.4	28.6	18.7	8.84	5.34	3.15	1.82	1.31	1.03	0.853
5	0.200	101	70.7	56.7	38.1	24.8	11.7	7.05	4.13	2.38	1.71	1.34	1.11
10	0.100	121	84.5	67.6	45.3	29.4	13.9	8.33	4.88	2.81	2.02	1.58	1.31
20	0.050	143	99.0	79.2	53.0	34.3	16.2	9.67	5.64	3.25	2.32	1.83	1.51
30	0.033	156	108	86.3	57.7	37.3	17.5	10.5	6.11	3.52	2.51	1.97	1.63
40	0.025	165	115	91.4	61.1	39.5	18.6	11.1	6.46	3.70	2.65	2.08	1.72
50	0.020	173	120	95.7	63.9	41.3	19.4	11.6	6.73	3.86	2.76	2.17	1.79
60	0.017	179	124	99.1	66.1	42.7	20.0	12.0	6.96	3.99	2.85	2.23	1.84
80	0.013	190	131	105	69.9	45.1	21.1	12.6	7.31	4.19	2.99	2.34	1.94
100	0.010	198	137	109	72.7	46.9	22.0	13.1	7.60	4.35	3.10	2.43	2.01
250	0.004	232	160	128	84.8	54.6	25.4	15.1	8.77	5.00	3.57	2.79	2.30

Figure 6: NIWA Rainfall intensities

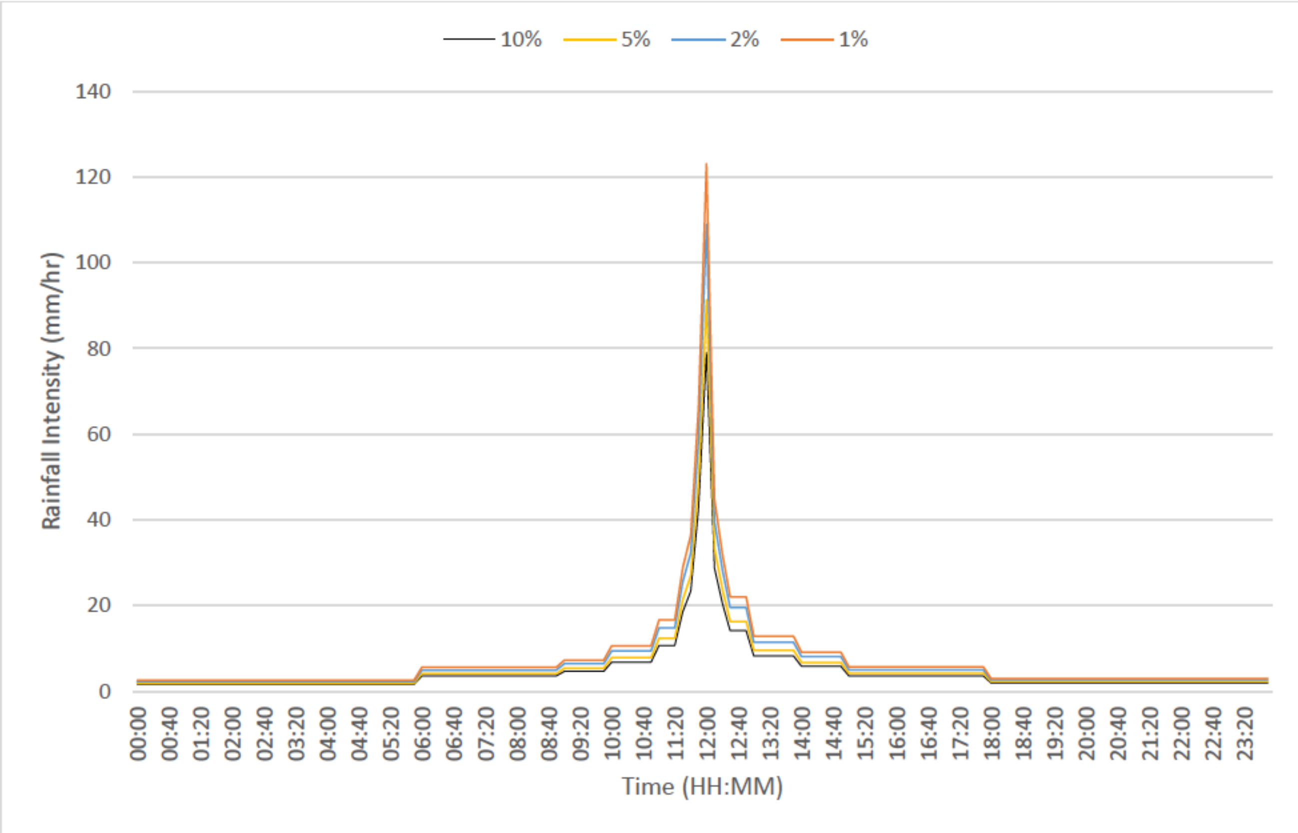

Figure 7: Nested Storm events.

3 GEO-HECRAS STORMWATER MODEL RESULTS

3.1 Flooding Overview

The flood modelling results confirmed that the 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 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.

The onsite flooding is a result of ponding, rather than lack of downstream capacity. This was confirmed by running the model with an unconstrained boundary condition along Pencarrow road (i.e. preventing the road and small culverts from acting as an obstruction. The results were largely identical. The percentage error was 0.46% of the model, which is acceptable.

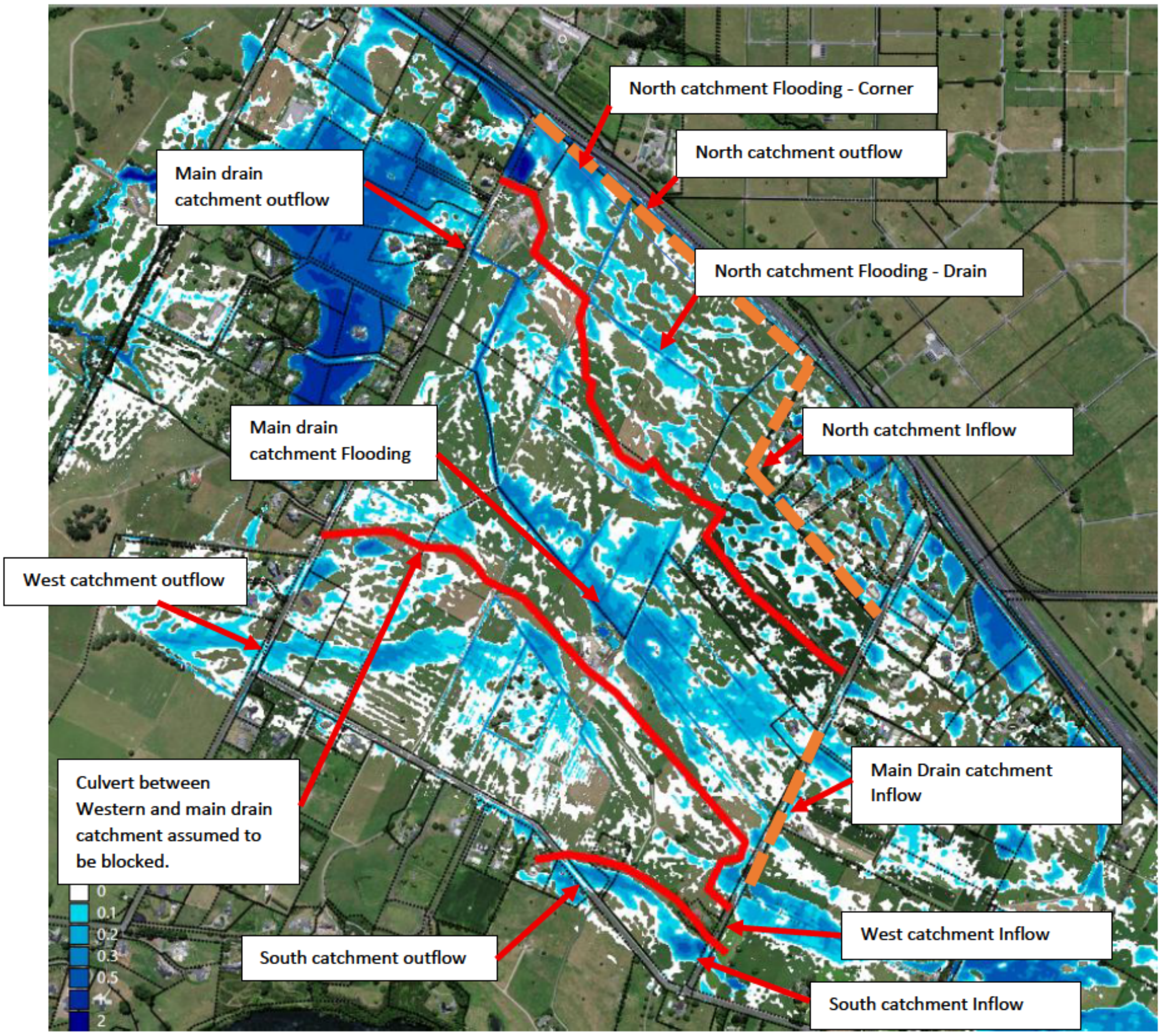

Figure 7: Flooding overview – catchment inflow and outflow locations

3.2 Catchment Inflows and Outflows.

The modelling gave the following inflow and outflow results for each of the catchments (Table 2). Any proposed designs on the site must ensure that the outflows are not exceeded in their totality and the inflows are not decreased in their totality.

While it may be possible to shift flow from catchment to another i.e. increasing the outflow flow from one catchment while decreasing the outflow from another catchment, this would require agreement with the Waikato Regional Council. Due to a floodplain being present downstream of the subject site, it is crucial that the outflow from the main drain catchment is not being exceeded as a result of the development.

Table 2: Catchment Inflow and Outflows

North Catchment	Inflow	Outflow
10%	0.40 m^3/s	2.4 m^3/s
5%	0.76 m^3/s	3.16 m^3/s
2%	1.36 m^3/s	4.51 m^3/s
1%	1.79 m^3/s	5.59 m^3/s

Main Drain Catchment	Inflow	Outflow
10%	2.80 m^3/s	4.80 m^3/s
5%	3.39 m^3/s	6.57 m^3/s
2%	4.48 m^3/s	9.19 m^3/s
1%	5.52 m^3/s	11.32 m^3/s

West Catchment	Inflow	Outflow
10%	0.28 m^3/s	2.55 m^3/s
5%	0.34 m^3/s	3.71 m^3/s
2%	0.44 m^3/s	5.49 m^3/s
1%	0.51 m^3/s	6.75 m^3/s

South Catchment	Inflow	Outflow
10%	0.18 m^3/s	0.08 m^3/s
5%	0.36 m^3/s	0.09 m^3/s
2%	0.69 m^3/s	0.23 m^3/s
1%	0.79 m^3/s	0.40 m^3/s

The inflow and outflow results indicate the following:

- The main drain catchment has the largest outflow and a notable sized inflow (all inflow rates being above 1 $^{m3}/_{s}$),
- The north and west catchments have notable outflows; however, only the north has a notable inflow. All the west catchment's inflows are below 1 $^{m3}/_{s}$. Due to this; the western catchment shows the largest percentage increase between its inflow and outflow. While the northern and main drain catchments approximately double in flow, the western catchment increase ten-fold.
- The southern catchment has very small inflows and even small outflows and appears to be more of a flooded depression that an overland flow path.

3.3 Flooding Depths

The analysis assessed flood depths (and flood elevations) at locations of notable flooding across the site, generally in the flooding areas, overland flow paths, and depressions discussed above. To ensure useful results, the analysis did not take results inside any of the drains as the drains will be infilled. 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.

Table 3: Flooding locations - Depth and Elevation

Simulation	Northern Corner	Northen Drain Flooding	Main Drain Flooding - west	Main Drain Flooding - east	Western overland flow	Southern Depression
10%	0.32 m	0.17 m	0.39 m	0.35 m	0.26 m	0.54 m
5%	0.35 m	0.19 m	0.42 m	0.37 m	0.28 m	0.57 m
2%	0.38 m	0.22 m	0.45 m	0.40 m	0.31 m	0.60 m
1%	0.40 m	0.24 m	0.47 m	0.42 m	0.32 m	0.62 m

Simulation	Northern Corner	Northen Drain Flooding	Main Drain Flooding - west	Main Drain Flooding - east	Western overland flow	Southern Depression
Ground Level	52.95 m RL	54.05 m RL	53.92 m RL	54.24 m RL	54.52 m RL	55.85 m RL
10%	53.27 m RL	53.23 m RL	54.31 m RL	54.59 m RL	54.78 m RL	56.39 m RL
5%	53.30 m RL	53.25 m RL	54.34 m RL	54.61 m RL	54.80 m RL	56.42 m RL
2%	53.33 m RL	53.28 m RL	54.37 m RL	54.64 m RL	54.83 m RL	56.45 m RL
1%	53.35 m RL	53.29 m RL	54.39 m RL	54.66 m RL	54.84 m RL	56.47 m RL

The flooding and flow depth results indicated the following:

- The typical flood depths on the existing site are around 400-450 mm, with deeper flooding locations being isolated to drains or small depressions – such as the southern catchment.
- The Top Water Level varies across the site by over 3 m; however, this variance is due to the variation in ground level. Finished floor levels for any buildings will have to account for this variance; however, it is likely the individual buildings will have different finished floor levels.
- Due to the upstream catchment being reasonably small and the floodplains being spread out over a large area, the resultant Top Water Level and flood depths are not overly sensitive to the rainfall intensities – with the difference between 10% and 1% AEP events being only around 80 mm.

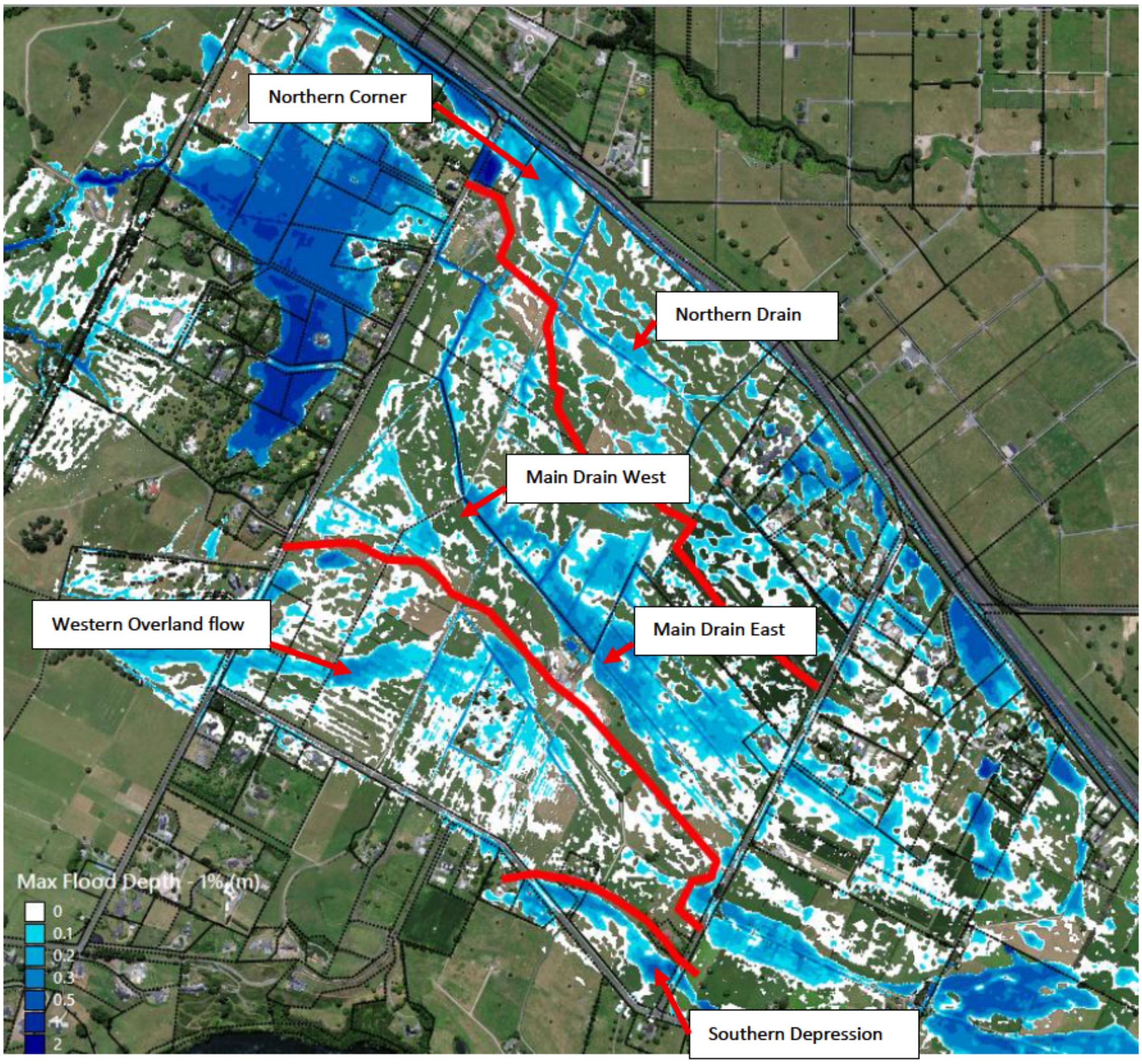

Figure 8: Flooding overview – flooding locations

3.4 Flooding Velocities

The flood mapping build based on the model results indicated that the 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.

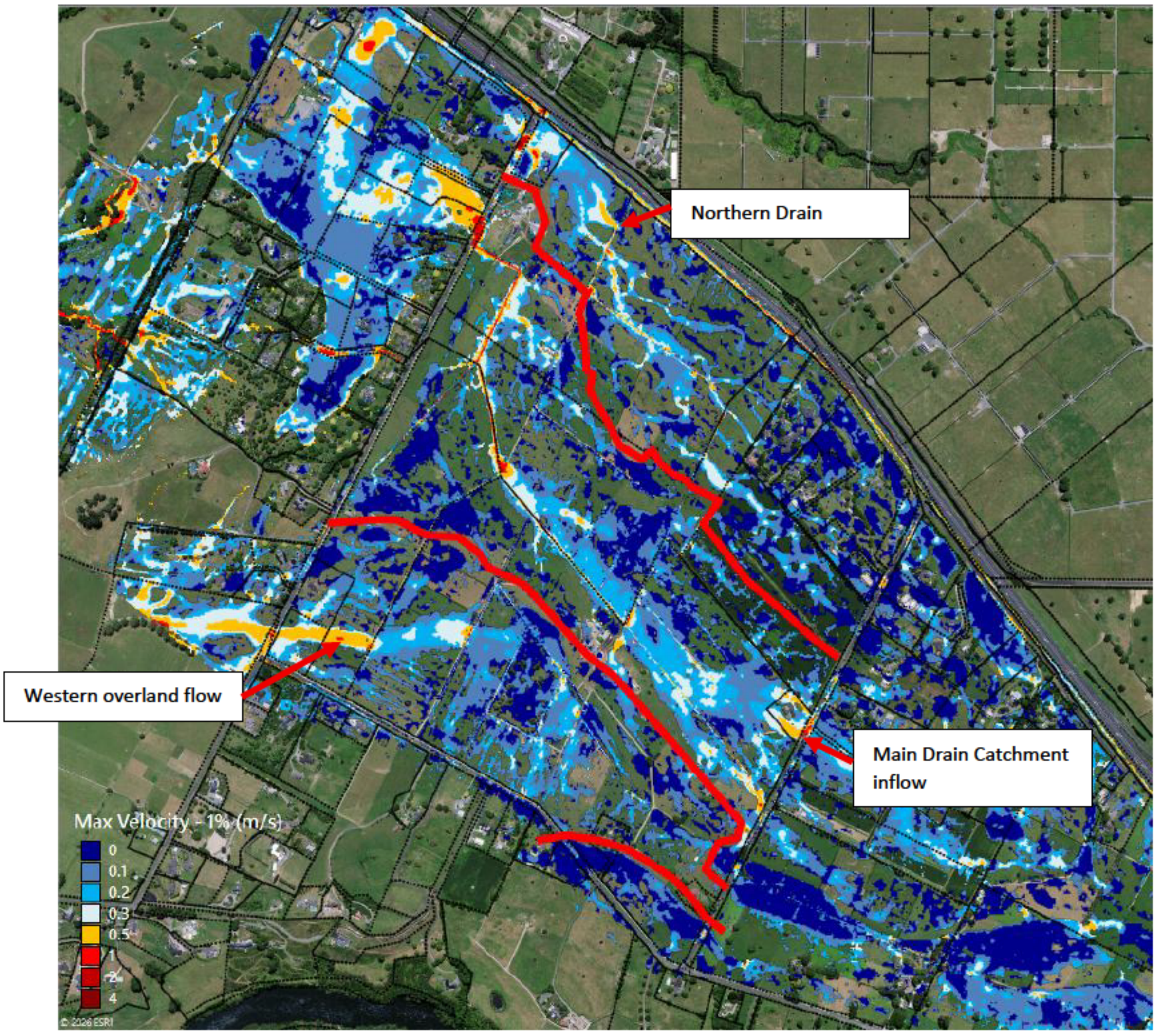

Figure 9: Flooding overview – flooding velocities

4 DESIGN RECOMMENDATIONS

BCD has received a masterplan of the proposed site; however, this masterplan has not been finalised and a preliminary design surface for use in a post-development model has not been created at this stage. However, based on the modelling results, we can provide some high-level design recommendations for handling stormwater on-site.

4.1 Southern Catchment

Due to this catchment mostly involving a depression with relatively small inflows and outflows, BCD recommends that the Southern catchment be incorporated into the Western Catchment by filling in the depression and directing the flow northward (the inflow should be kept the same).

Although this would increase the outflow from the Western Catchment, the increase in flow will not be substantial. It should be possible for the western catchment to still achieve hydraulic neutrality, with the use of wetlands and the attenuation pond located in the Harness Racing Track.

Table 4: Change to Western Catchment

Simulation	Western Catchment outflow Pre-development	Western Catchment outflow Plus, South Catchment	Amount Change	Percentage Change
10%	2.55 m^3/s	2.66 m^3/s	0.11 m^3/s	4.3%
5%	3.71 m^3/s	3.85 m^3/s	0.14 m^3/s	5.1%
2%	5.49 m^3/s	5.77 m^3/s	0.28 m^3/s	3.8%
1%	6.75 m^3/s	7.07 m^3/s	0.32 m^3/s	4.7%

4.2 Western Catchment

There is an existing drain running along Hooker Road, between the site and the cycle path. This drain runs to an existing sump and culvert at the intersection of Pencarrow Road and Hooker Road (and is the location where the Western Catchment currently drains.)

To provide additional capacity, BCD recommends installing a swale drain running along the Hooker Road site boundary, with the western catchment sloped in this direction. As the catchment already slopes in a westward direction, this should not require a significant amount of work. The new drain will end at this site's boundary with 516 Pencarrow Road, with flows using the existing Hooker Road drain for the final 100 m between the site and existing culvert and sump. The site will still be expected to handle the 10% AEP event via a primary network (i.e. no overland flows) and to achieve hydraulic neutrality in the 1% AEP events (i.e. at downstream end the discharge at Hooker Road remaining unchanged).

BCD recommends a series of swales running across the catchment, for the most part following either the proposed roads or proposed racing tracks. These swales will capture both the upstream inflow (from both the western and southern catchments), along with runoff from the site itself. The swales then will direct the flow into two areas of attenuation -an irrigation pond located within the Harness Racing Track and a wetland located at the site's proposed southern entrance.

The actual size of these swales will be coordinated once the masterplan for the site and land usage is finalised and a preliminary design surface created as the size of this drains will be heavily dependent on: the proposed ultimate impervious area of the site; the operation of the wetlands/attenuation ponds; how the site is sloped, and the ultimate design of pipes and swales feeding into it. An approximate minimum size can be found by taking the inflows from the upstream western and southern catchment.

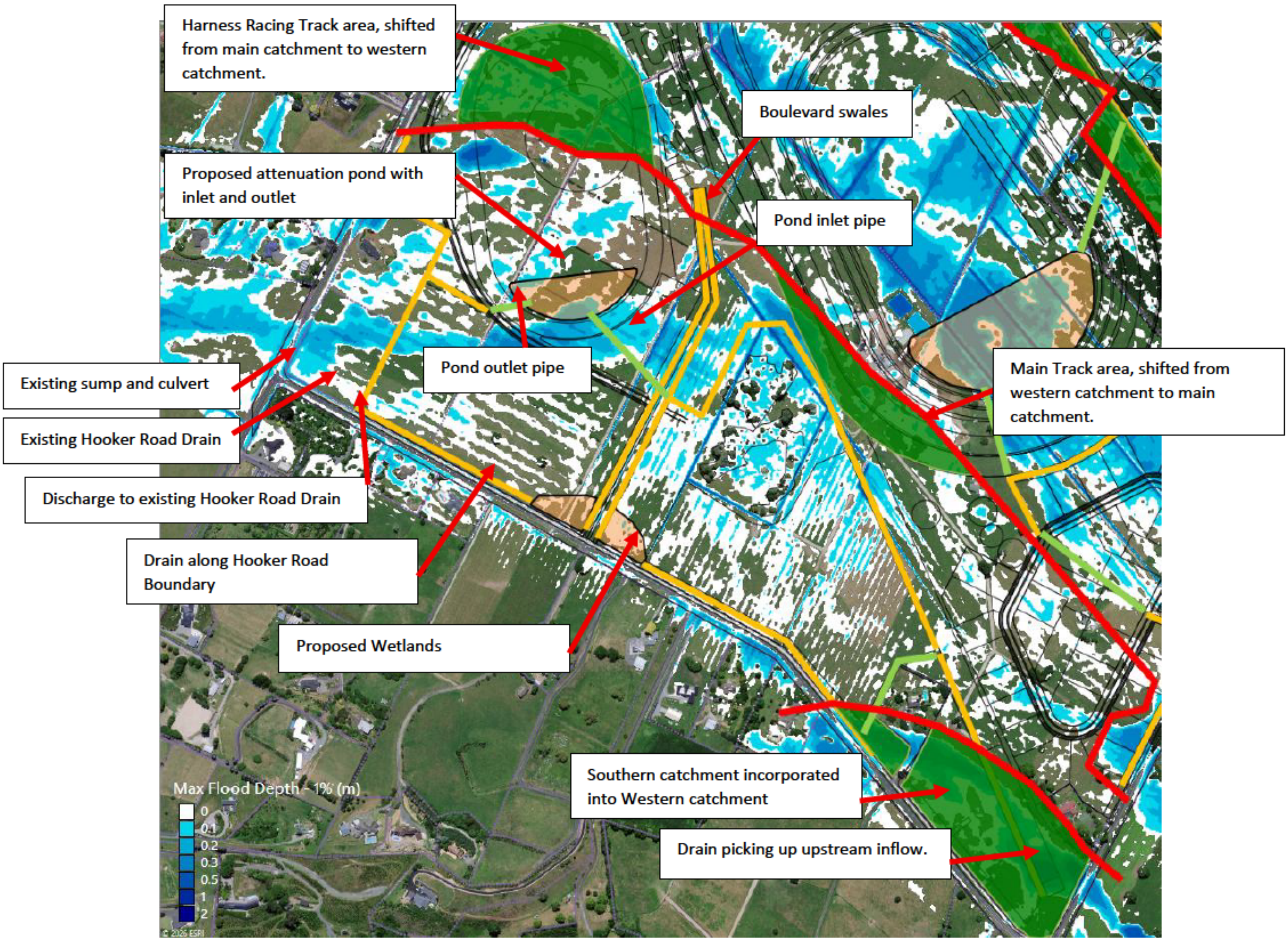

Figure 10: Design Recommendations – western catchment (Red: existing catchment, yellow: proposed swales, green: proposed pipes, green hatch: area that will change catchment, orange hatch: proposed ponds)

An approximate maximum size for the swales can be found by taking the outflows from the western catchment and adding the difference between the western catchment's inflow and outflow (accounting for the runoff coefficient onsite increasing from 0.3 to 0.6) and adding the southern catchment's inflow (to account for this catchment being combined into the western catchment).

Table 5: Range of Drain Flows - Western catchment

Simulation	Estimated max outflow
10%	5.0 $m^3/_s$
5%	7.44 $m^3/_s$
2%	11.12 $m^3/_s$
1%	13.78 $m^3/_s$

The average slope of the drain along the Hooker Road boundary is 0.23%. Assuming a drain with 1:2 sides, the drain would require the following dimensions to convey the maximum flow under each storm event. The results show that the drain will need a width of 5.8 m to convey the 10% AEP event. A width of 8.3 m would be needed to convey that 1% AEP event.

Table 6: Western Drain - Sizing

Simulation	Base Width	Depth	Top Width
10%	1.8 m	1.00 m	5.8 m
5%	2.0 m	1.14 m	5.56 m
2%	2.25 m	1.36 m	7.69 m
1%	2.7 m	1.40 m	8.3 m

A similar method was used to approximate the required stormwater attenuation. BCD used the models time varying flow results to determine the existing outflow volume and the difference between the inflow and outflow volume. The volume difference was then added to the existing outflow volume to get a predicted post-development outflow volume. The time-varying outflow results were then multiplied by the factor required to achieve the same post-development outflow volume.

Based on the peak pre-development flow and the simulation time-step, the calculation then determined the detention volume required, should the developed outflow be restricted to the peak existing outflow.

Table 7: Western Attenuation (split between wetland and Harness Racing Track Pond)

Simulation	Volume Required
10%	6,200 m^3
5%	7,600 m^3
2%	8,500 m^3
1%	8,500 m^3

4.3 Northern Catchment

This catchment already naturally drains northward to the drain running along SH1. BCD proposes that the northern catchment maintains this existing inclination and continue to use the SH1 drain as an outlet, with the catchment topography altered to remove ponding locations. Although it would be preferable to avoid a drain along the northern boundary and have multiple points of discharge to the SH1 drain, this will likely not work with the need to achieve hydraulic neutrality.

The design recommendation proposes smoothing the existing slopes out to remove depressions and ponding within the Northern Catchment. As with the western catchment, BCD proposes that the 10% storm event be handled via a series of pipes/swales, capturing and directing runoff to the drain in a controlled manner and to achieve hydraulic neutrality in the 1% AEP events.

BCD recommends a swale along the northern boundary of the catchment, both capturing the inflow from the upstream catchment and runoff from the developed northern catchment. The swale running along the northern boundary will run parallel to the SH1 swale and will use an existing 400mm culvert as a low flow outlet from the site.

The northern catchment includes proposed carparking and roadways, which will increase the impervious area of the site and therefore its runoff. It is expected that the post-developed flows from the catchment will require attenuation. The only location for a detention pond (based on the current Masterplan and site design) is in the northern corner. The current site design is using the Back Track for wastewater discharge. Due to the location of the Back Track and Main Track, feeding this pond will require running the swale along the northern boundary (the main catchment's swale will need to run between the Back Track and Main Track to avoid the grandstand).

The size of this swale will be heavily dependent on the proposed ultimate impervious area of the site, the operation of the attenuation ponds, how the site is sloped, and the ultimate design of pipes and swales feeding into it.

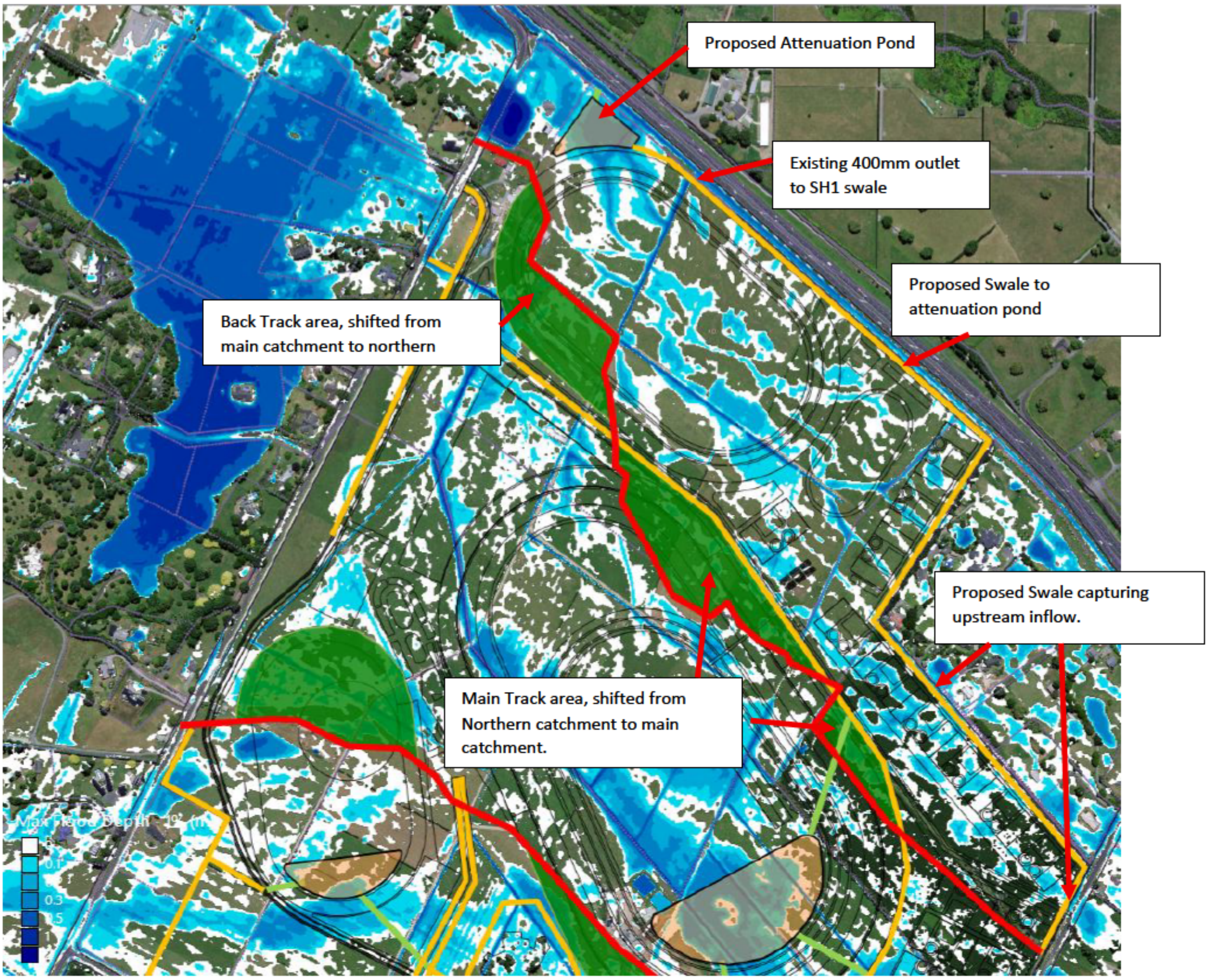

Figure 11: Design Recommendations – northern catchment (Red: existing catchment, yellow: proposed swales, green: proposed pipes, green hatch: area that will change catchment, orange hatch: proposed ponds)

An approximate maximum size of the swales can be found by taking the outflows from the northern catchment and adding the difference between the northern catchment's inflow and outflow (accounting for the runoff coefficient onsite increasing from 0.3 to 0.6).

Table 8: Potential Flows - Northern Catchment

Simulation	Estimated max outflow
10%	4.40 m^3/s
5%	5.56 m^3/s
2%	7.66 m^3/s
1%	9.39 m^3/s

The average slope of the drain along the boundary is SH1 0.23%. Assuming a drain with 1:2 sides, the drain would require the following dimensions to convey the maximum flow under each storm event. The results show that the drain will need a width of 3.8 m to convey the 10% AEP event. A width of 7.1 m would be needed to convey that 1% AEP event.

Table 9: Northern Drain - Sizing

Simulation	Base Width	Depth	Top Width
10%	1.0 m	0.7 m	3.8 m
5%	1.2 m	0.9 m	4.8 m
2%	1.9 m	1.1 m	6.3 m
1%	2.3 m	1.2 m	7.1 m

The approximate required stormwater attenuation volume was defined in the similar manner as attenuation volumes for the western catchment. BCD used the models time varying flow results to determine the existing outflow volume and the difference between the inflow and outflow volume. The volume difference was then added to the existing outflow volume to get a predicted post-development outflow volume. The time-varying outflow results were then multiplied by the factor required to achieve the same post-development outflow volume.

Based on the peak pre-development flow and the simulation time-step, the calculation then determined the detention volume required, should the developed outflow be restricted to the peak existing outflow.

Table 10: Northern Attenuation

Simulation	Volume Required
10%	9,200 m^3
5%	13,700 m^3
2%	20,900 m^3
1%	24,700 m^3

4.4 Main Drain Catchment

The main drain catchment has several risks not present in the other catchments:

- A very high outflow (4.80 m3/s in the 10% AEP) and a high inflow (2.80 m3/s in the 10% AEP), preventing a significant portion of the run-off from being moved or redirected to another catchment by shifting an equivalent area into the main catchment.
- The rise in elevation at Pencarrow Road prevents running a main stormwater swale along the downstream boundary of the catchment.

The existing catchment slopes westward towards Pencarrow Road. Presently this slope stops about 150 m from the road, with the catchment then rising upwards to Pencarrow Road. Runoff from the catchment is directed northward via an existing drain to a point where the site continues to slope towards Pencarrow Road. There is an existing culvert at this location that runs under Pencarrow Road. The Master plan has proposed locating its north-western road near this existing drain.

BCD recommends recontouring the catchment to remove depressions and locations of ponding, and to ensure the catchment slopes towards this north-western road. The road will then slope towards the existing culvert and include a roadside swale. Although not quite on the boundary, this would locate the swale both near the boundary and near the existing drain and culvert, taking advantage of the site's natural landform while keeping the drain towards the edge of the site.

The Masterplan has proposed locating the main Racetrack near the centre of the site, where it blocks off the entire main catchment. The Masterplan also proposed located a series of lifestyle lots near the main inlets of upstream inflow.

While re-contouring the catchment towards the culvert will help the catchment with flooding, the site is still expected to handle the 10% AEP event via a primary network (i.e. no overland flows) and to achieve hydraulic neutrality in the 1% AEP events (i.e. discharge through the culvert and across Pencarrow Road remaining unchanged).

BCD recommends running several swales from the main points of upstream inflow, switching to a piped system were necessary to run under the lifestyle blocks. This will ensure the inflow is captured and contained. These swales would run westward until they reach the Main Track, whereupon they would merge, with the combined swale running counterclockwise around the Main Track until it reaches the northern road proposed by the masterplan. The swale will then follow this northern road westward until it meets the swale near the boundary.

Swale running clockwise around the track was considered; however, it would have required coordination with the grandstand and likely required a series of large culverts running below the grandstand.

The main catchment includes proposed carparking, roadways, buildings, and tracks, that will increase the impervious area of the site and therefore its runoff. It is expected that the run-off from the post-developed catchment will require attenuation. The masterplan proposed locating this attenuation at the south-east end of the main track. The catchment can be fed (via pipework) from the swale as it circles the main track. The proximity of the swale will also allow for the outlet to be established.

The size of the swales will be heavily dependent on the proposed site's ultimate impervious area, the operation of the wetlands/attenuation ponds, how the site is sloped, and the ultimate design of pipes and swales feeding into it.

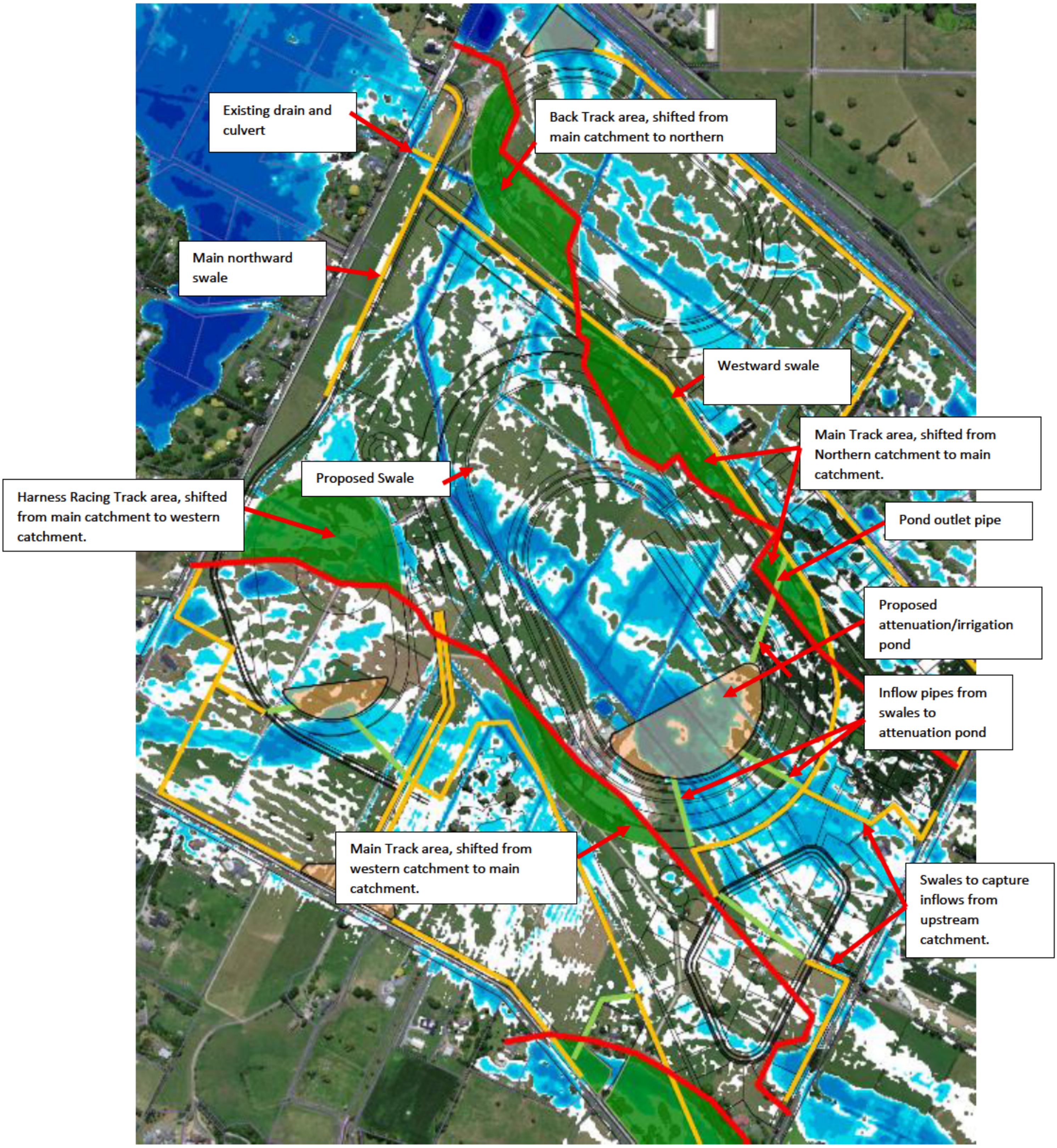

Figure 12: Design Recommendations – main drain catchment (Red: existing catchment, yellow: proposed swales, green: proposed pipes, green hatch: area that will change catchment, orange hatch: proposed ponds)

An approximate maximum swale size can be found by taking the outflows from the main catchments and adding the difference between the main catchment's inflow and outflow (accounting for the runoff coefficient onsite increasing from 0.3 to 0.6).

Table 11: Potential Flows - Main Catchment

Simulation	Estimated max outflow
10%	6.80 m^3/s
5%	9.75 m^3/s
2%	13.90 m^3/s
1%	17.12 m^3/s

The average slope of the drain near the boundary is 0.26%. Assuming a drain with 1:2 sides, the drain would require the following dimensions to convey the maximum flow under each storm event. The results show that the drain will need a width of 5.92m to convey the 10% AEP event. A width of 8.7m would be needed to convey that 1% AEP event.

Table 12: Drain Sizing - Main Drain Catchment

Simulation	Base Width	Depth	Top Width
10%	1.8 m	1.03 m	5.92 m
5%	2.1 m	1.2 m	6.90 m
2%	2.25 m	1.4 m	7.85 m
1%	2.7 m	1.50 m	8.7 m

The approximate required stormwater attenuation volume was defined in the similar manner to attenuation volumes for the western catchment. BCD used the models time varying flow results to determine the existing outflow volume and the difference between the inflow and outflow volume. The volume difference was then added to the existing outflow volume to get a predicted post-development outflow volume. The time-varying outflow results were then multiplied by the factor required to achieve the same post-development outflow volume.

Based on the peak pre-development flow and the simulation time-step, the calculation then determined the detention volume required, should the developed outflow be restricted to the peak existing outflow.

Table 13: Main Catchment Attenuation

Simulation	Volume Required
10%	7,400 m^3
5%	9,500 m^3
2%	11,900 m^3
1%	14,900 m^3

5 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 will, as indicated above, use 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.

6 CURRENT CONCLUSIONS

Based on the above analysis, the proposed development should be able to handle stormwater from both on-site runoff and from the upstream catchment.

The existing site does not suffer significantly from flooding, and the proposed development should be able to handle stormwater by ensuring the site's natural slopes are maintained and complement with boundary drains and pipes/swales.

There are no significant flooding issues resulting from the project and all potential adverse stormwater issues can be mitigated by standard drainage and stormwater management measures/devices. The proposed measures will also mitigated the impact of climate change on the downstream catchments with is a considerable benefit.

7 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 5.1.0.2113, 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.