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APPENDIX F

Stormwater Runoff and Flood Effects Assessment

REPORT

Stormwater Runoff and Flood Effects Assessment

Mission Special Character Zone

Prepared for
Marist Holdings (Greenmeadows) Limited

Prepared by
Tonkin & Taylor Ltd

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Exceptional thinking together

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Executive summary

The Mission Special Character Zone is a proposed residential development for approximately 570 residential properties located near Napier. The 246 ha site is currently owned by Marist Holdings (Greenmeadows) Ltd and they have engaged T+T to carry out a stormwater effects assessment

The primary goals of the assessment are to:

1 Assess the downstream flood effects of the proposed development
2 Assess whether the proposed development can meet the goals outlined in Hawke's Bay Regional Council Stormwater Management Guidelines (HBRC SMG) and the standards set out in Rule 42 of the Hawkes Bay Regional Resource Management Plan

Assessing these items, will help determine whether the development can proceed as a permitted activity, which is needed for a certificate of compliance.

The development proposes to increase impervious surface across the site which, if unmitigated, has the potential to increase runoff volume, produce higher peak flows and faster hydrological response times than the current situation runoff volume and peak flow rates, thereby increasing the potential for downstream flooding and increased erosion. Increased erosion can cause downstream sedimentation and turbidity problems in the watercourses.

The stormwater effects assessment has:

1 Assessed unmitigated hydrological effects
2 Assessed the downstream effects of unmitigated runoff on flood levels and flood duration (through the development of a hydraulic model)
3 Developed a stormwater concept design to mitigate the effects
4 Quantified the downstream changes in flooding caused as a result of the development with mitigation included.
5 Provided a water quality concept design.

The proposed development will increase impervious coverage by approximately 43 ha for land draining to the west to the Turirau Stream and by 3.4 ha for land draining to the east to the Taipo Stream. However runoff to the east will be mitigated through landscape planting, which will result in a net reduction in runoff.

The increase in impervious surface for land draining to the west represents approximately 2.5% of the Turirau catchment area (~1720 ha) and 3% of the catchment upstream of a farmland area which is known to flood (~1345 ha). Irrespective of the mitigation works at the Mission Special Character Zone site, there is farmland located downstream from the site which will continue to flood to a similar degree to what is currently experienced. The farmland is a former natural wetland that was drained by creating man made channels to improve drainage.

Without mitigation, the downstream flood levels will increase by between 10mm and 20mm however the increase is insufficient to make a discernible difference to flood extents. The duration of flooding from the water quality storm (30mm rainfall over 24 hours) will increase by approximately 3 hours in comparison with a predicted flood duration of approximately 50 hours. This is considered representative of a flood event which may occur more frequently than once per year. For less frequent events (e.g. 2 year ARI, 10 year ARI and 100 year ARI) the increase in flood duration is less pronounced as the period of flooding increases. Increases in the order of 1 hour are likely.

To be consistent with HBRC SMG requirements for areas upstream of floodplains, flood mitigation options were considered to reduce the downstream effects and to attenuate post-development peak flows to 80% of predevelopment peak flows.

The concept developed to meet these requirements was to develop a series of stormwater management ponds which will capture runoff from each of the gullies, attenuate peak flows and reduce the volume of runoff at critical times, thereby delaying the hydraulic response times downstream.

The analysis showed that peak flows have been reduced to 78% and 73% of the predevelopment peak flows for a climate change and no climate change scenario respectively. Furthermore the concept design shows that the post-development flows are approximately 67% and 53% of the pre-development flows for 2 year ARI and 10 year ARI events, thereby meeting another guideline from the HBRC SMG.

The effect of the mitigation is to reduce downstream flood levels by between 10mm and 20mm for the water quality storm, 2 year ARI and 10 year ARI. We expect flood levels to be very similar to existing flood levels in the 100 year ARI.

Overall we conclude that there is no downstream negative effect caused by the development. The concept design included in this report is likely to cause a small reduction in downstream flood levels.

The stormwater ponds have been designed with appropriate outlet configurations and storage volume to provide water quality treatment that meets the HBRC SMG. That is:

- Water quality control volume based on 30mm of runoff from catchment impervious surfaces.
- 50% of the water quality volume allocated to permanent pond storage.
- 1.2 times the water quality volume released as extended detention over 24 hours.
- Removal of 75% of total suspended solids

Through appropriate vegetation selection and topographic contouring it may be possible in subsequent design stages to change the stormwater management ponds into wetlands, or add wetland pond elements to them. Both wetlands and ponds promote sedimentation, however wetlands also promote biological uptake of contaminants for water quality treatment. The choice of vegetation which can withstand both long, dry periods and also relatively deep inundation depths and multiple day flood durations will require advice from an appropriate expert.

During the detailed design stages of this project, there will be opportunities to refine and optimise the stormwater design, however based on the assessment and recommendations from this report, Marist Holdings (Greenmeadows) Ltd should be confident that the proposed development can be carried out in a manner that does not cause negative downstream effects from flooding and does not create adverse water quality effects. The stormwater discharge from the proposed development will therefore meet the permitted activity standards of Rule 42 of the Hawke's Bay Regional Resource Management Plan as it will not reduce of the ability of the receiving channel to convey flood flows, cause any bed scouring or bank erosion of the receiving channel, or cause the production of conspicuous oil or grease films, scums or foams, or floatable or suspended materials in any receiving water body after reasonable mixing.

1 Introduction

The Mission Special Character Zone (MSCZ) is a proposed residential development for approximately 570 residential properties located near Napier. The 246 ha site is currently owned by Marist Holdings (Greenmeadows) Ltd and they have engaged T+T to carry out a stormwater effects assessment for the development to support a certificate of compliance application and plan change.

The western side of the proposed development lies within the Tutaekuri River Catchment and the eastern side drains to the Taipo Stream. The predominant land use is currently dry land cattle farming and pine plantation.

2 Scope

The primary goals of the assessment is to:

1 Assess the downstream flood effects of the proposed development

2 Assess whether the proposed development can meet the goals outlined in Hawke's Bay Regional Council Stormwater Management Guidelines (HBRC SMG) and the standards set out in Rule 42 of the Hawkes Bay Regional Resource Management Plan

Assessing these items, will help determine whether the development can proceed as a permitted activity, which is needed for a certificate of compliance.

3 HBRC Stormwater Management Guidelines

The HBRC SMG propose the following goals for development stormwater runoff effects:

Storm peak discharge control

The following recommendations identify the most relevant peak discharge control:

1. Where there are existing flooding problems downstream and in the absence of a catchment study that evaluates a potential project in a given location depending on the location within a catchment (as per Section 7.1.2 in the HBRC SMG), it is recommended that the post-development peak discharge for the 100-year storm for a new project be limited to 80% of the pre-development peak discharge.
2. In terms of intermediate storm control, it is recommended that the 2- and 10-year post-development peak discharges do not exceed the 2- and 10-year pre- development peak discharges.

Runoff volume control

For the purposes of this Guideline, erosion control criteria is regarded as 1.2 times the water quality volume that should be live storage provided within the stormwater management practice to be infiltrated or released over a 24-hour period.

Water quality control

Attenuate and treat over 24 hours, and/or infiltrate the water quality volume determined by using the 90% storm as shown in Figure 6-5 of HBRC 2009, using the City of Christchurch method of determining the first flush volume in their Waterways, Wetlands and Drainage Guide (2003). For the location of the proposed development, referencing Figure 6-5 of the HBRC SMG, this equates to approximately the first 30mm of rainfall runoff from the impervious area.

4 Existing Site Description

The proposed development area is approximately 246 ha, and approximately 80% (194 ha) drains westwards into the Turirau Stream (a tributary of the Tutaekuri River), and 20% (52 ha) drains east towards the Mission Estate Vineyard and Taipo Stream. The total upstream area of the Turirau Stream at the confluence with the Tutaekuri is approximately 1720 ha.

Figure 4-1 outlines the sub-catchments for the proposed development, with sub-catchments C1 to C3 draining west, and sub-catchments c4 to C7 draining east. The entire catchment area of the Turirau Stream is presented in Figure 6-1 in the “Hydrology” section.

The land use on the westward draining catchments include dry land cattle farming and pine plantation, as shown by the aerial image in Figure 4-1. A significant portion of sub-catchment C1 (Figure 4-1) is currently pine forest plantation. As the plantation reaches maturity the plantation will be harvested. For the purposes of this assessment, we have assumed the land use is pasture (i.e. a harvested condition).

The westward draining sub-catchments fall from 140m RL to 30m RL over an approximate distance of 1.5km (refer Figure 4-3).

The western catchments flow into farmland, historically formed by draining swampland. The low-lying area drains through a series of manmade channels and culverts before flowing into the Tutaekuri River, as shown in Figure 4-2. Flooding has been a reported issue for the farmland (Goodier 2000) and therefore the HBRC SMG requirement to limit the 100 year ARI post-development peak discharge to 80% of the pre-development discharge is appropriate. The total upstream area of the farmland is approximately 1345ha, noting that some downstream catchments may also influence flooding in this area due to the flat topography.

The land use on the eastern side of the site (sub-catchments C4, C5, C6 and C7) is currently dry land cattle farming and drains east towards the Mission Estate Vineyard before entering the Taipo Stream, which discharges into the Main Outfall Channel (Figure 4-2). The Taipo Stream and Main Outfall Channel are urban streams modified from their natural state.

The eastern draining sub-catchments fall from 140m RL to 15m RL over a distance of 500m, and have a shorter hydrological response compared to the westward draining slopes.

LINZ soil maps of the area indicate that soil types are typically poorly drained across the site. The slopes comprise of silty soils with grassland cover (refer Figure 4-4).

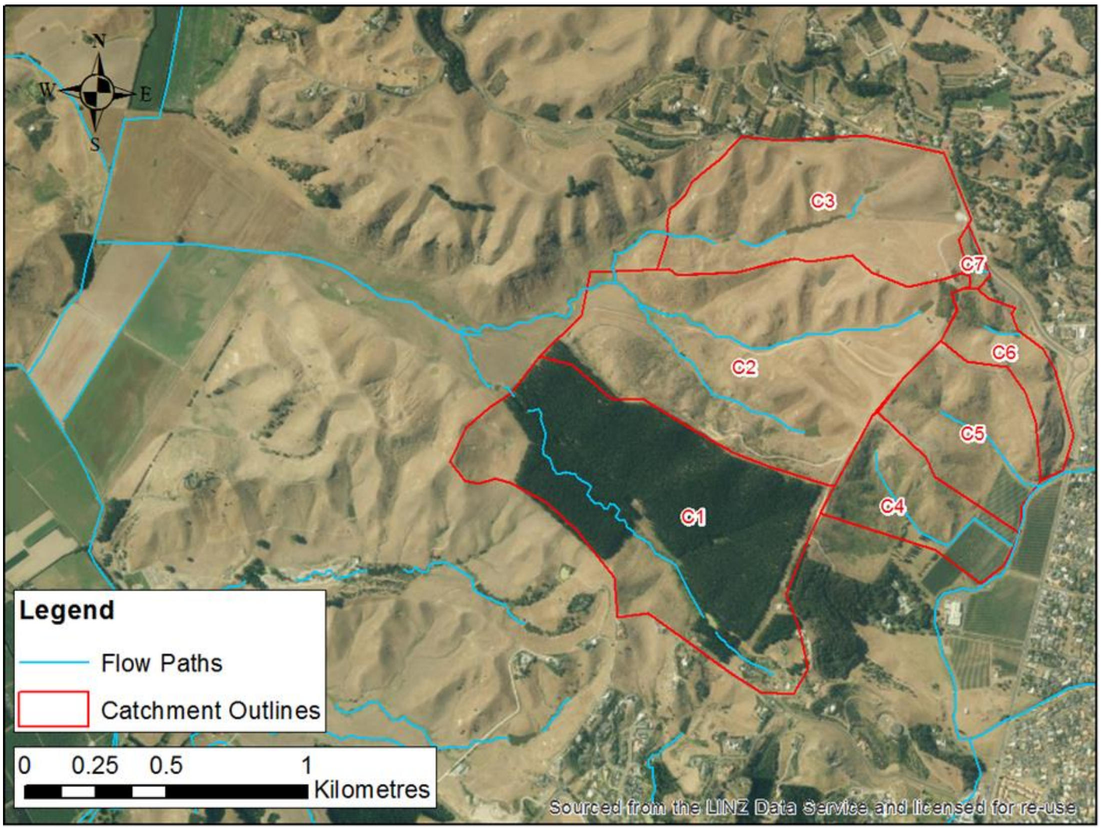

Figure 4-1: Aerial photograph showing the proposed development area and sub-catchments

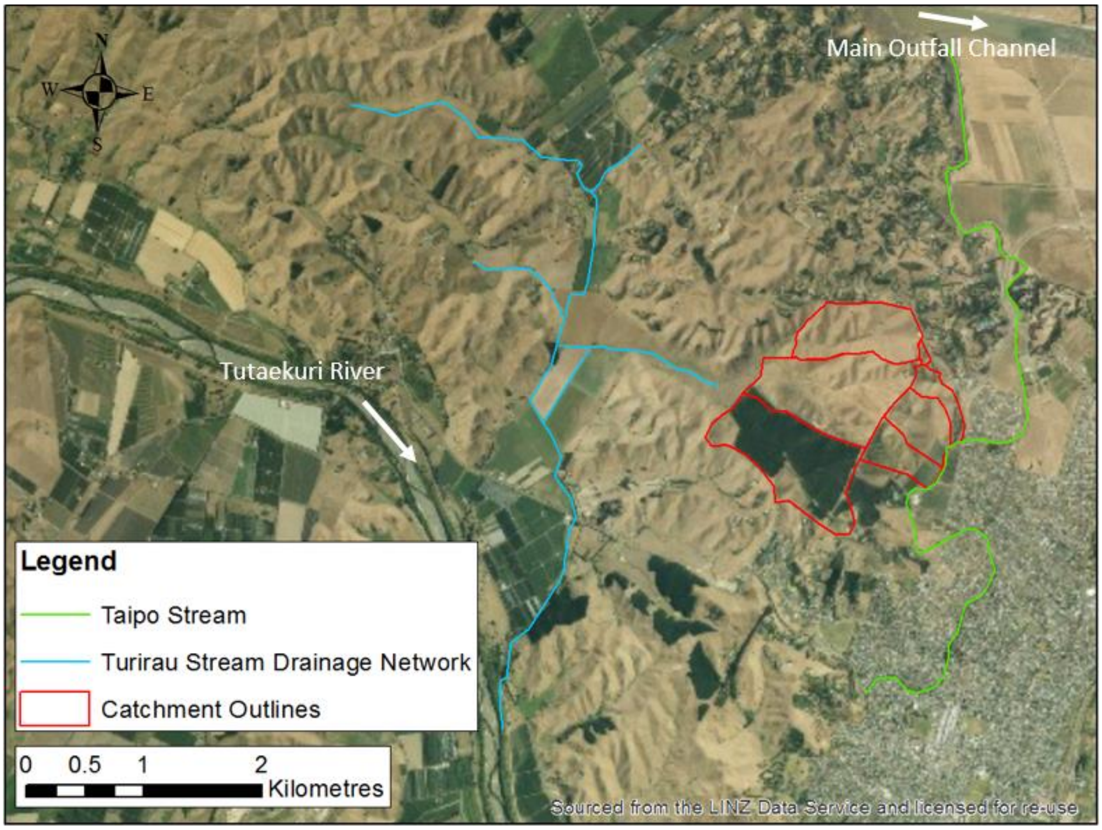

Figure 4-2: Location of development sub-catchments with respect to receiving environments

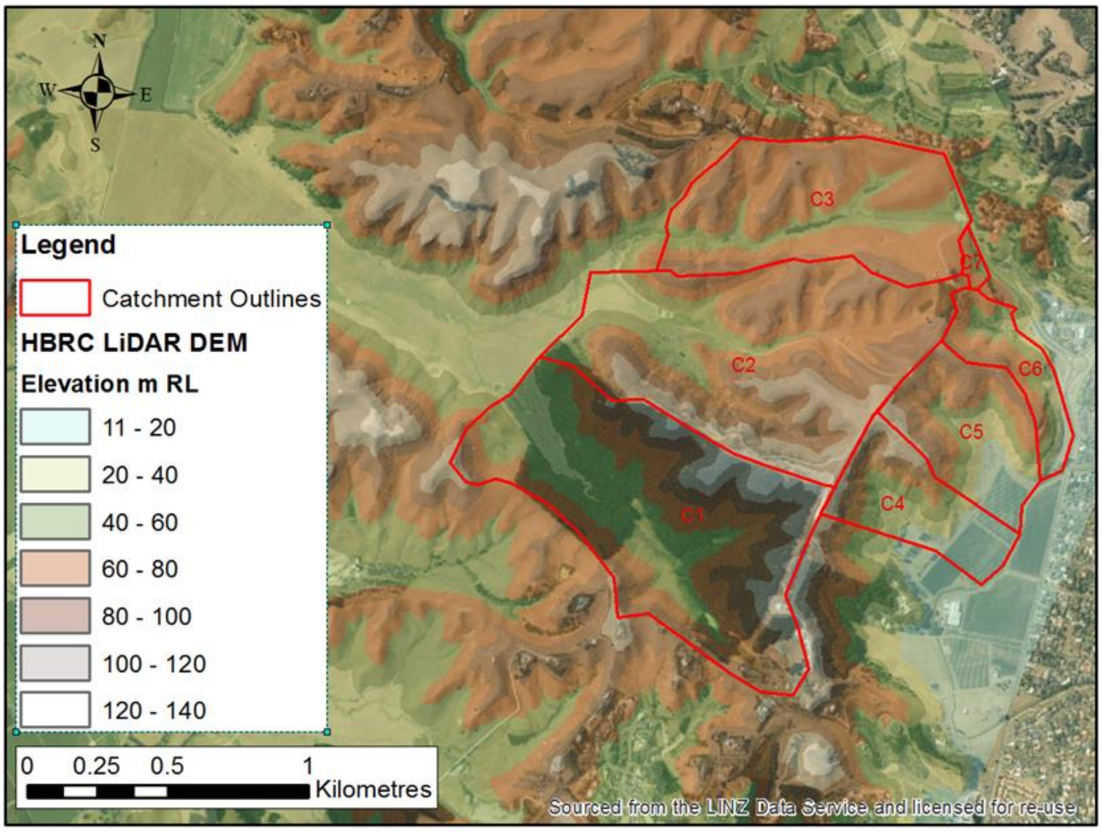

Figure 4-3: Contour banded digital elevation model of the site with sub catchment boundaries

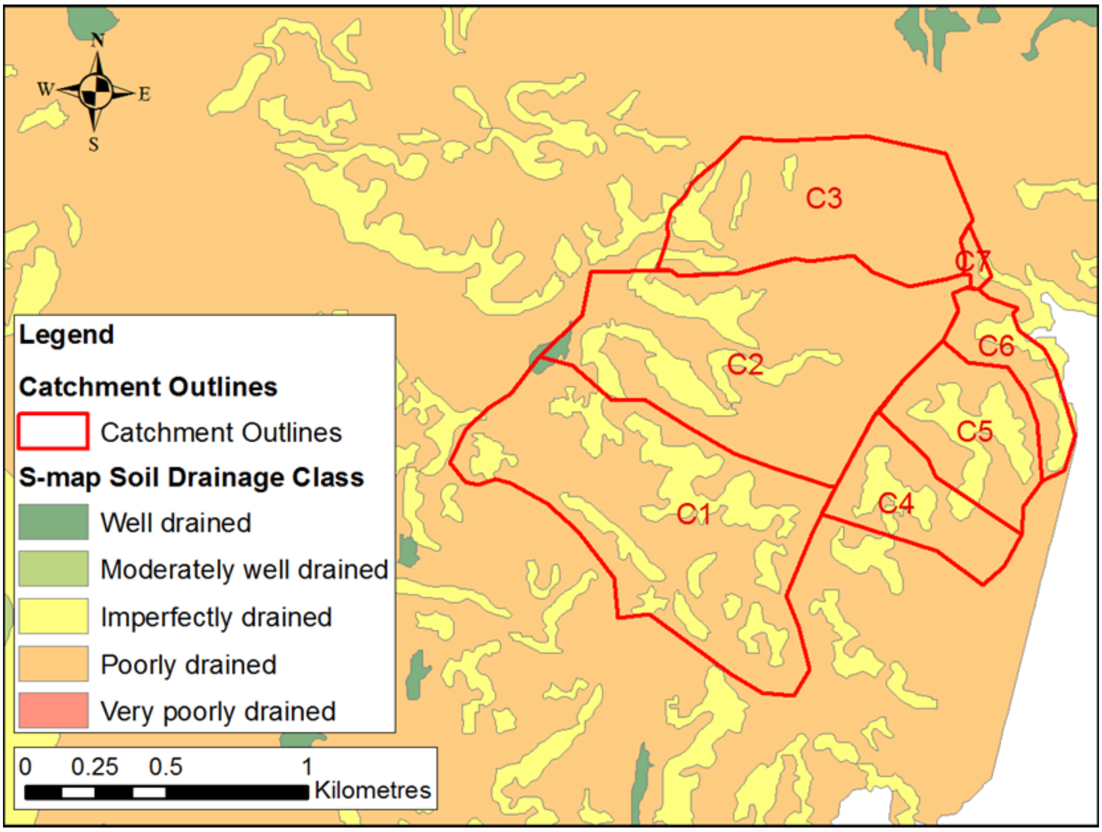

Figure 4-4: Landcare Research S-Map (2016 release) soil drainage category map

5 Proposed Development

The proposed development has a catchment area of 246 ha and approximately 570 properties are planned. The final contouring of the proposed site has not yet been finalised however it is unlikely to change the existing sub-catchment area distribution significantly.

The proposed layout of the development and the direction of drainage amongst the sub-catchments is shown in Figure 5-1. The residential areas are shown in grey, and proposed vegetative cover is shown in green. The direction of drainage is indicated in blue. The site to the left of the orange dashed line drain westwards and to the right drains eastwards. The yellow dashed lines indicate sub catchment divides of the westwards draining catchments.

Our assessment has been made on the basis that the impervious runoff from sub-catchment C7 will be captured and drained to the west, whilst the pervious component will drain to the east. This is reasonable given its location along the divide between catchments.

The percentage of impervious area for residential lot areas has been estimated at 75%. This is similar to nearby developments in Napier.

The total impervious area for the entire development including roads and residential lots will be approximately 20%. This is significantly lower than nearby developments in Napier. The 20% increase represents a 43 ha increase in imperviousness draining west and 3.4 ha increase draining east. The distribution of impervious area in the proposed development is shown in Table 5-1.

On the eastward draining slopes a significant amount of landscape planting is proposed with a small area covered by residential lots.

Table 5-1: Impervious area in sub-catchments in proposed development

Sub Catchments	Area (ha)	% Area Impervious Post-Development
C1	76	6%
C2	73	23%
C3	45	48%
C4	18	4%
C5	22	10%
C6	11	4%
C7	1.3	0%

A 43 ha increase in imperviousness draining west represents 2.5% of the Turirau catchment area (~1720 ha) and 3% of the catchment upstream of the farmland area which floods (~1345 ha).

5.1 Description of potential effects

This section describes the potential effects if there is no mitigation in place. The subsequent stages of this report will quantify the effects and the requirements for mitigation if necessary.

Change in land use from predominantly cattle farming to residential has the potential to alter the stormwater runoff due to the effects on water quality and water quantity. These are caused by the increase in impervious area and changes in land use. Without mitigation, the increased impervious surface area will lead to increased runoff volume, higher peak flows and shorter hydrological response times than the current situation. Increased peak flows can also lead to increased erosion which can cause loss of land, downstream sedimentation and higher turbidity in streams and watercourses.

In the eastward draining sub-catchments there is extensive planting proposed (more details provided later) which may result in a stormwater quantity and quality improvements.

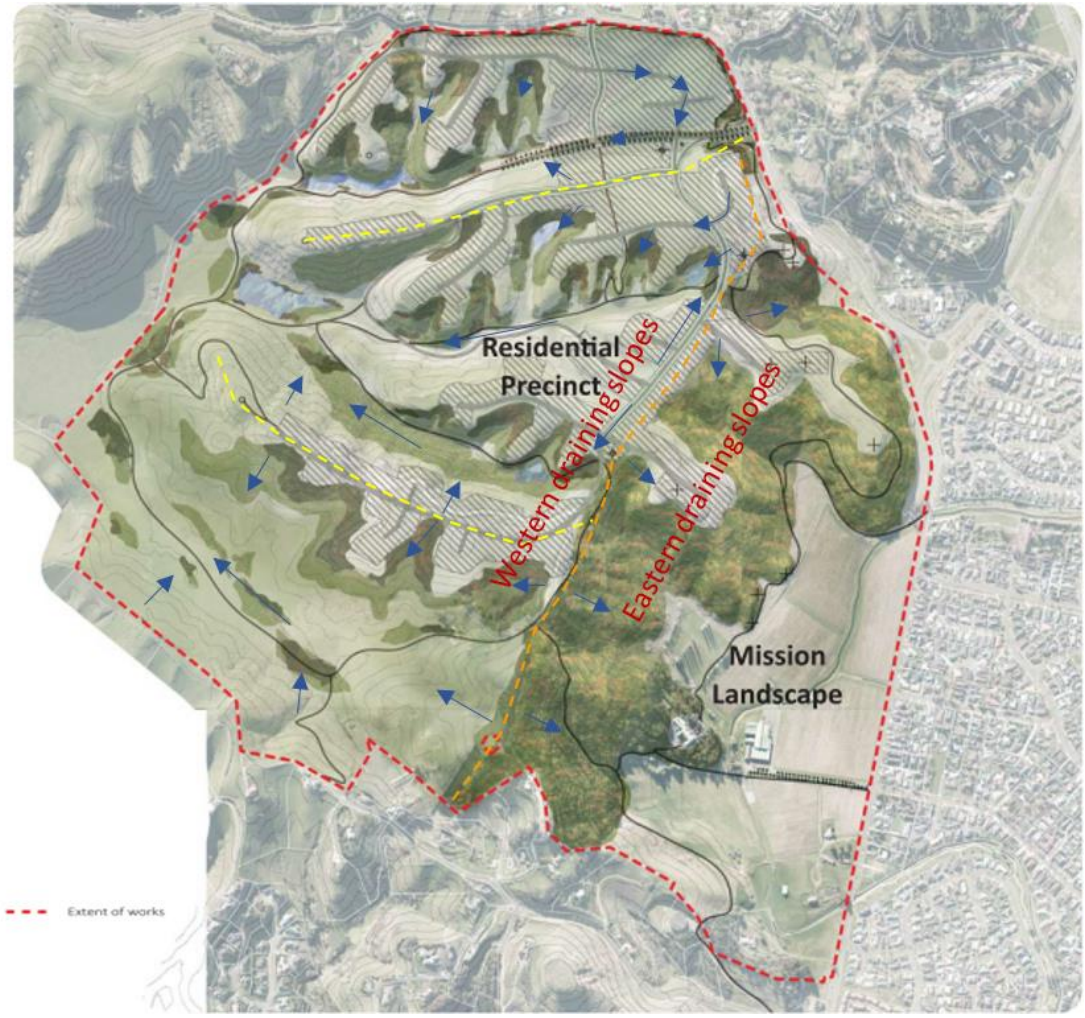

Figure 5-1: Proposed development plan with runoff flow directions indicated

6 Hydrology

This section describes the methodology used to carry out the stormwater runoff assessment. The hydrological assessment provides the inflows for the hydraulic model which is used to assess the hydraulic flood effects.

6.1 Hydrological methodology

An assessment has been carried using the SCS hydrological method to calculate excess rainfall and convert it to runoff. Due to potential downstream effects, the catchment assessment includes all of the downstream catchment. The catchment extents (including west draining sub-catchments) are shown in Figure 6-1.

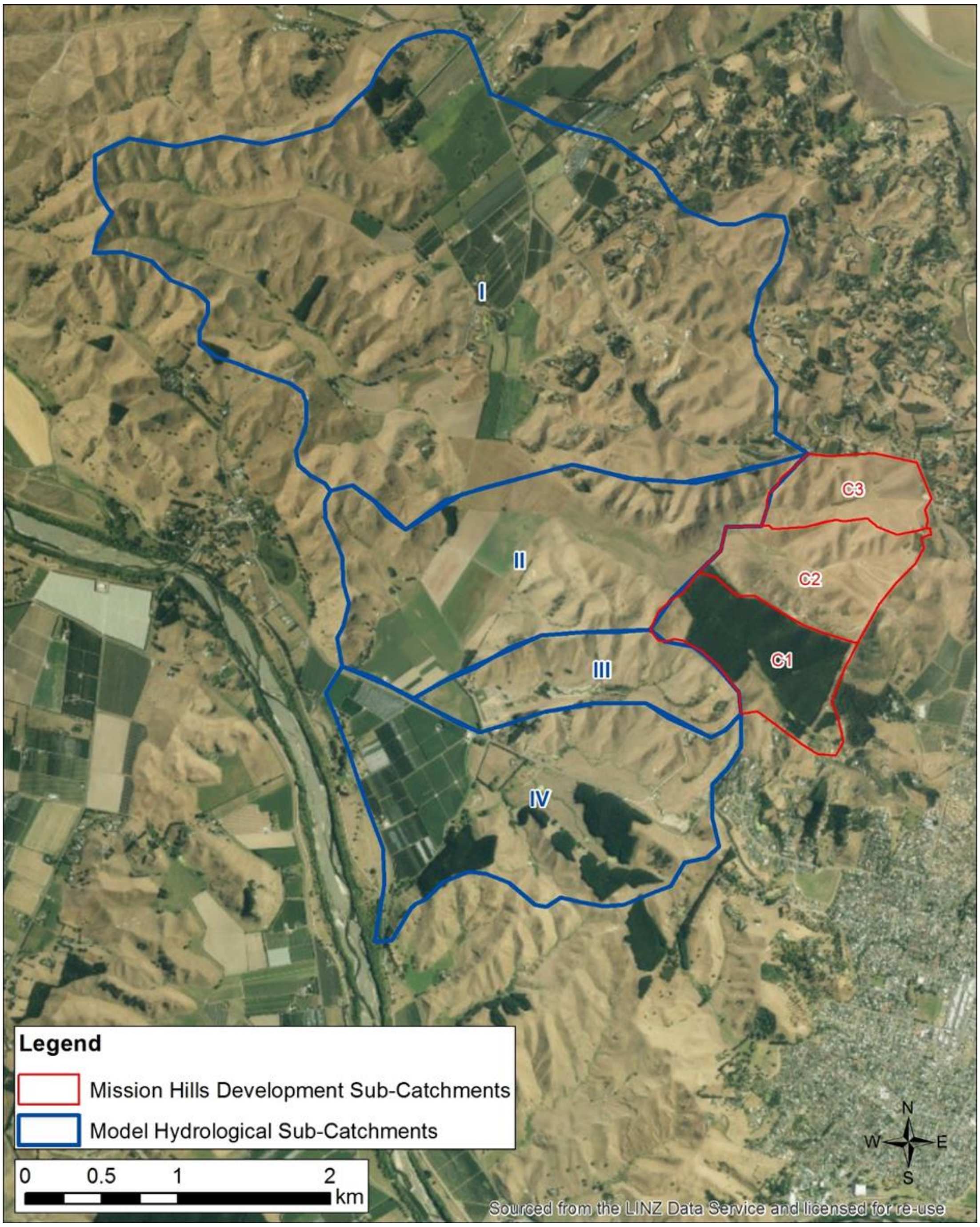

Figure 6-1: Model sub-catchment outlines

A temporal rainfall distribution was developed based on the 'embedded' rainfall approach referred to as the Chicago Method. The method uses a rainfall distribution where short duration, high intensity storms are embedded in longer duration, higher volume storms. This has the benefit of assessing peak flows for critical storm durations, whilst also assessing the effects of volume changes associated with longer storm durations. Therefore flood effects relating to both volume and conveyance can be assessed by a single rainfall hyetograph for each recurrence interval storm.

Additional information regarding rainfall and runoff is provided in the following subsections.

6.2 Design Rainfall

Design rainfall depths were sourced from NIWA's HIRDs v3 (Table 6-1). HIRDs v3 uses a more recent data set than those shown in Section 6.1.3 of the HBRC SMG. However we note that generally the depths are within 5% of each other in both sources.

Design rainfall depths adjusted for climate change effects are shown in Table 6-1. Adjustments for climate change effects are discussed in Section 6.2.1.

Table 6-1: HIRDs V3 Rainfall Depths (mm) at development site

Duration	≤1 hr				1 < Duration ≤24 hr				Duration > 24 hr	
mins	10	20	30	60	120	360	720	1440	2880	4320
hours	0.167	0.333	0.5	1	2	6	12	24	48	72
ARI										
2	7	10	12	18	25	41	56	78	95	108
2 + CC	8	11	14	21	29	48	66	91	*	*
10	12	17	21	31	42	67	89	120	147	166
10 + CC	13	20	25	36	49	78	104	140	*	*
50	19	28	35	51	67	102	134	175	216	243
100	23	34	43	63	81	123	159	206	253	286
100 + CC	27	40	50	73	95	143	186	240	296	334

**not run for durations longer than 24 hour*

The water quality storm rainfall hyetograph was estimated 30mm depth rainfall event as recommended in Section 6.3.3 of the HBRC SWMG. A 24 hour duration was chosen for consistency of the duration of the other design storm hyetographs used.

The design rainfall approach makes the assumption that a design rainstorm produces a flood event of similar return period and that the same rainfall occurs over the entire catchment. These are appropriate assumptions for catchment analyses and effects assessments.

Rainfall hyetographs were generated using this method for the combination of events shown in Table 6-2.

Table 6-2: Design rain storm events

ARI	**Duration**		
	24 hours	**48 hours**	**72 hours**
Water Quality Storm	✓		
2	✓	✓	✓
10	✓	✓	✓
50	✓	✓	✓
100	✓	✓	✓

6.2.1 Climate Change Effects

The effects of climate change on rainfall to 2090 were assessed for 2 year ARI, 10 year ARI and 100 year ARI events, for 24 hour duration storms.

Rainfall was increased by 16.8% to account for climate change to 2090 in accordance with HBRC SMG. This is based of 2.1 degree C of warming and an 8% increase in rainfall per degree of warming.

It is noteworthy that the mid-range temperature rise scenario (RCP 4.5) from the June 2016 Ministry recommends 1.7 degree C of warming for Hawkes Bay to 2120. Gaining agreement with regulatory authorities on appropriate climate change projections for subsequent design stages should be sought. The 16.8% increase is considered appropriate for this report.

6.3 Runoff Model

To assess the effects of stormwater runoff a hydrological model was created using HEC_HMS. The SCS Curve Number (CN) approach was used to determine hydrological losses and the transformation of the excess rainfall to runoff was calculated using the SCS Unit Hydrograph.

The hydrological losses in the SCS CN approach are determined from initial abstraction and curve number. The hydrological soil type, impervious coverage and land use was used to derive a "weighted-CN". An initial abstraction value of 5mm is applied to pervious areas and 0mm in impervious areas. The excess rainfall is transformed into a discharge runoff hydrograph using the SCS unit hydrograph. The standard peak rate factor for the SCS unit hydrograph was used (peak rate factor 484) and this is common practice in New Zealand (e.g. TP108 uses peak rate factor 484). The time of concentration was calculated using the SCS approach which is determined using the slope (by equal area method) and longest flow path distance.

The CN values were determined from hydrological soil group (HSG) analyses and land cover estimates. The HSG class was determined from a comparison of LINZ soil maps (refer Figure 4-4) with definitions provided by NCRS HNEH (2004, Chapter 7). The LINZ soil maps indicate that the soils are generally poorly drained, which is considered to be consistent with the "high runoff potential" definition from NCRS HNEH (2004, Chapter 7). The land use was assumed "pasture" with "fair" hydrologic condition from NCRS HNEN (2004, Chapter 9) and was based on observed ground cover.

A comparison of the hydrological parameters for pre-development and post development for the catchment shown is summarised in Table 6-3. Detailed hydrological parameters are provided in Appendix B. For post-development scenarios, a weighted curve number which includes the impervious surface component was adopted.

Table 6-3: Comparison of pre-development and post development CN

Sub Catchments	Area (ha)	% Impervious post development	CN Pre-Development	CN Post-Development
C1	76	6%	78	80
C2	73	23%	79	84
C3	44.3 – Pre 44.7 - Post	48%	79	92
C4	18	4%	79	76
C5	22	10%	79	78
C6	11	4%	79	76
C7	1.7 – Pre 1.3 - post	0%	79	79

As shown in Table 6-3, the CN generally increases for sub-catchments on the western side of the site due to the increase in impervious area proposed by the development.

It should be noted that for sub-catchment C7, the impervious area will drain westwards post development (as part of catchment C3).

There is landscape planting proposed on the eastern side of the development which formed part of the Structure Plan. A vegetation plan contained within the Structure Plan is reproduced in Appendix A of this report and identifies the area as East Hill Face Woodland. Furthermore Plan Change Design Outcome 16 requires that the area "is to be planted with trees established prior to subdivision of the Residential Precinct and is to be retained as a woodland to achieve the following specific outcomes:

- *A high amenity landscape comprising a mix of deciduous and evergreen species.*
- *A green skyline and backdrop to the Mission landscape when viewed from Church Road.*
- *The screening of houses in the Residential Precinct when viewed from Church Road.*
- *Provision for paths and 'art cabins' within the woodland (see Design Outcome 19).*
- *Long term retention of the woodland backdrop while providing for individual trees to be selectively harvested on an on-going basis. Such harvesting is not to apply to trees on the upper slopes so that the skyline retains a permanent screening function and is to be undertaken so that the green woodland backdrop is maintained."*

A detailed vegetation plan will be available at subsequent design stages, however on the basis of the Structure Plan (Appendix A) and the existing land use (as shown below in Figure 6-2Table 6-1) we believe that there will be an overall decrease in CN for the east facing slopes.

Figure 6-2: Land cover for eastern draining slopes

6.4 Unmitigated hydrology results

This section provides a summary of the hydrological results without any stormwater mitigation for catchments draining to the west. The catchments draining to the east are not included because there will be a reduction in runoff volume and peak discharge based on the proposed planting in comparison with the existing land use.

For catchments draining west, the hydrological results are provided at the five locations shown in Figure 6-1.

A comparison of pre-development and unmitigated post-development peak flows for the proposed development catchment is shown in Table 6-4. For all other sub-catchments (i.e. the downstream ones), the peak flows for the combined Mission Special Character Zone Development Sub-catchment (i.e. C1, C2, C3) are shown in Table 6-4.

Table 6-4: Results comparison for un-mitigated post-development peak discharges

Storm Duration	Event	West flowing catchments			East flowing catchments		
		Pre-Dev'pment Peak Discharge (m^3/s)	Post-Dev'pment Peak Discharge (m^3/s)	% of Pre Dev'pment Peak Discharge	Pre-Dev'pment Peak Discharge (m^3/s)	Post-Dev'pment Peak Discharge (m^3/s)	% of Pre Dev'pment Peak Discharge
24 hr	WQV	0.46	0.64	137%	0.13	0.12	92%
	2	5.6	7.3	129%	2.7	2.5	93%
	2 + CC	7.0	8.9	127%	3.4	3.2	94%
	10	12.1	14.6	121%	5.8	5.5	95%
	10 + CC	15.3	18.0	118%	7.4	7.0	96%
	50	23.1	26.6	115%	11.2	10.8	96%
	100	29.7	33.6	113%	14.3	13.9	97%
	100 + CC	36.0	40.2	112%	17.3	16.9	97%
48 hr	2	6.2	7.8	126%	3.0	2.8	94%
	10	13.3	15.6	117%	6.4	6.1	96%
	50	24.6	27.7	112%	11.8	11.5	97%
	100	31.1	34.6	111%	14.9	14.6	97%
	100 + CC	37.5	41.3	110%	18.0	17.6	98%
72 hr	2	6.7	8.2	123%	3.2	3.0	95%
	10	13.7	15.9	116%	6.6	6.3	96%
	50	25.3	28.2	111%	12.1	11.8	97%
	100	31.8	35.1	110%	15.3	14.9	98%
	100 + CC	38.2	41.7	109%	18.3	17.9	98%

A comparison of the hydrographs for a 24 hour duration storm are provided in Figures 6-3 to 6-7 without climate change. The results for all scenarios are provided in Appendix C[1].

[1] The results provided in Appendix C include mitigated hydrology which is not presented in the report until Section 8. The results are all included together in one Appendix to avoid unnecessary duplication of information.

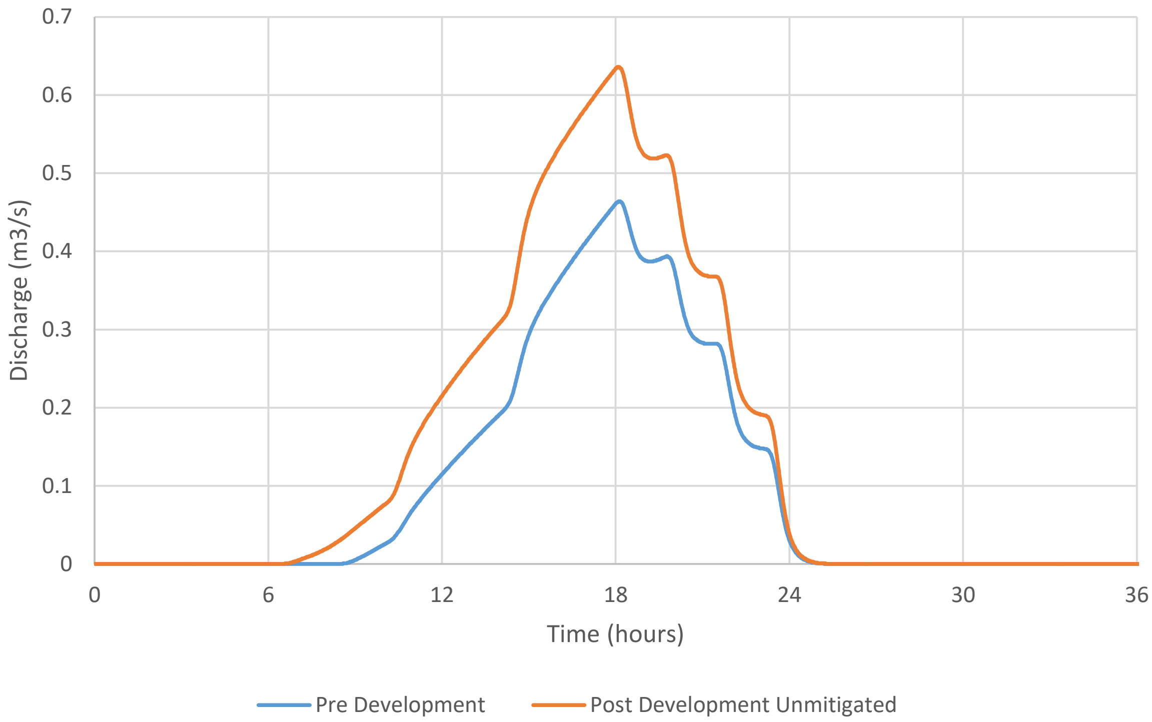

Figure 6-3: Water Quality Storm; comparison of pre-development flows and unmitigated post-development flows for MSCZ Development sub-catchments

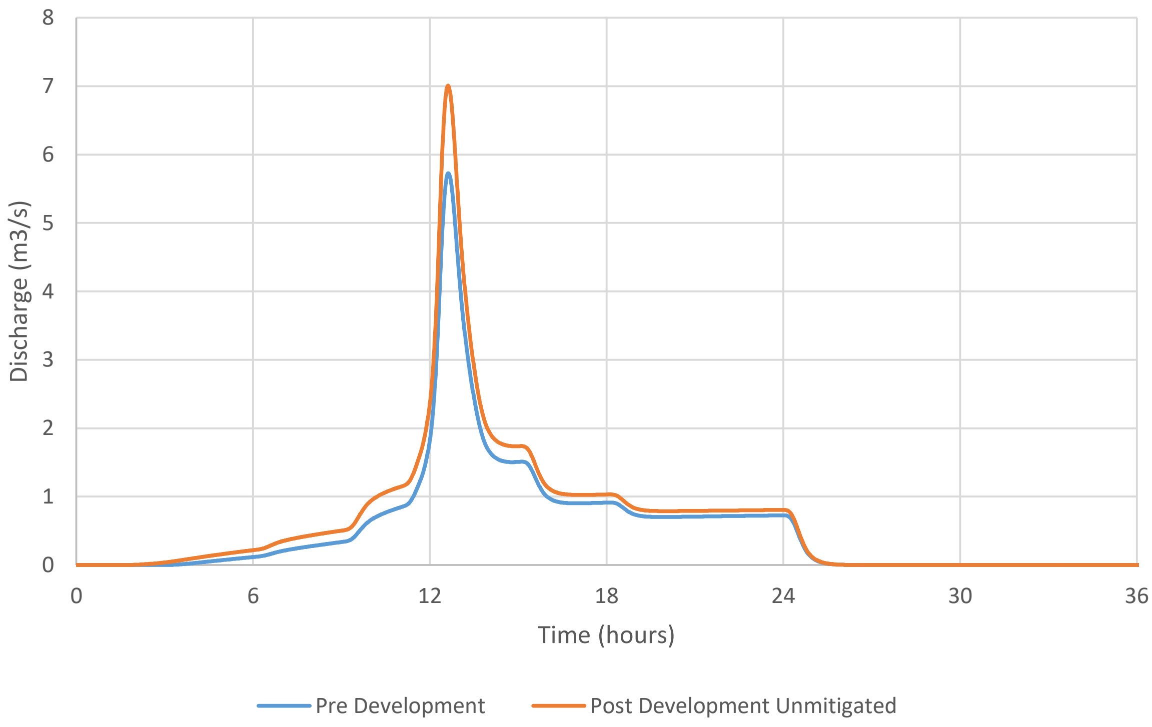

Figure 6-4: 2 year ARI, 24 hour storm duration; comparison of pre-development flows and unmitigated post-development flows for MSCZ Development sub-catchments

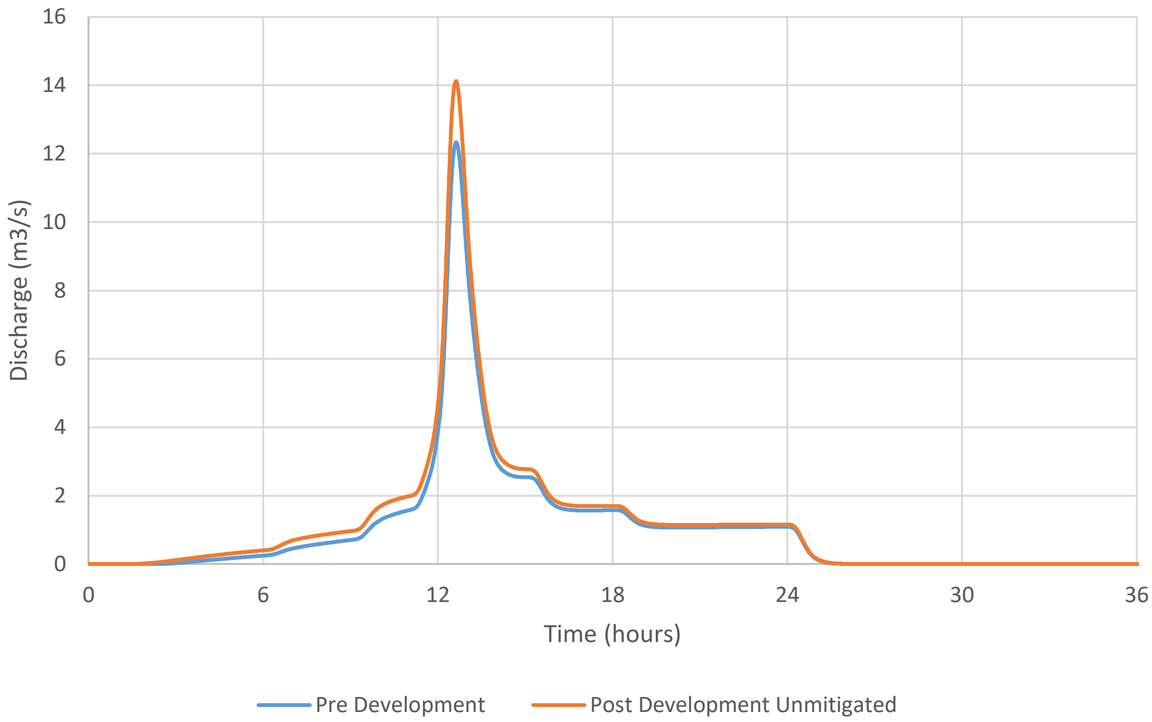

Figure 6-5: 10 year ARI, 24 hour storm duration; comparison of pre-development flows and unmitigated post-development flows for MSCZ Development sub-catchments

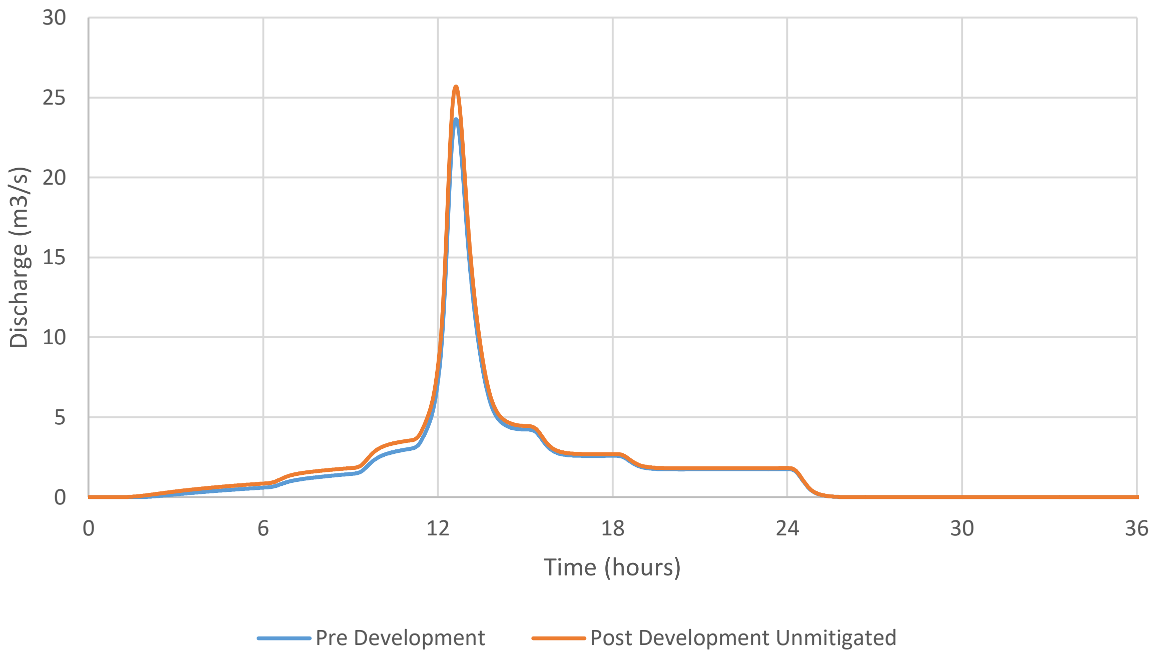

Figure 6-6: 50 year ARI, 24 hour storm duration; comparison of pre-development flows and unmitigated post-development flows for MSCZ Development sub-catchments

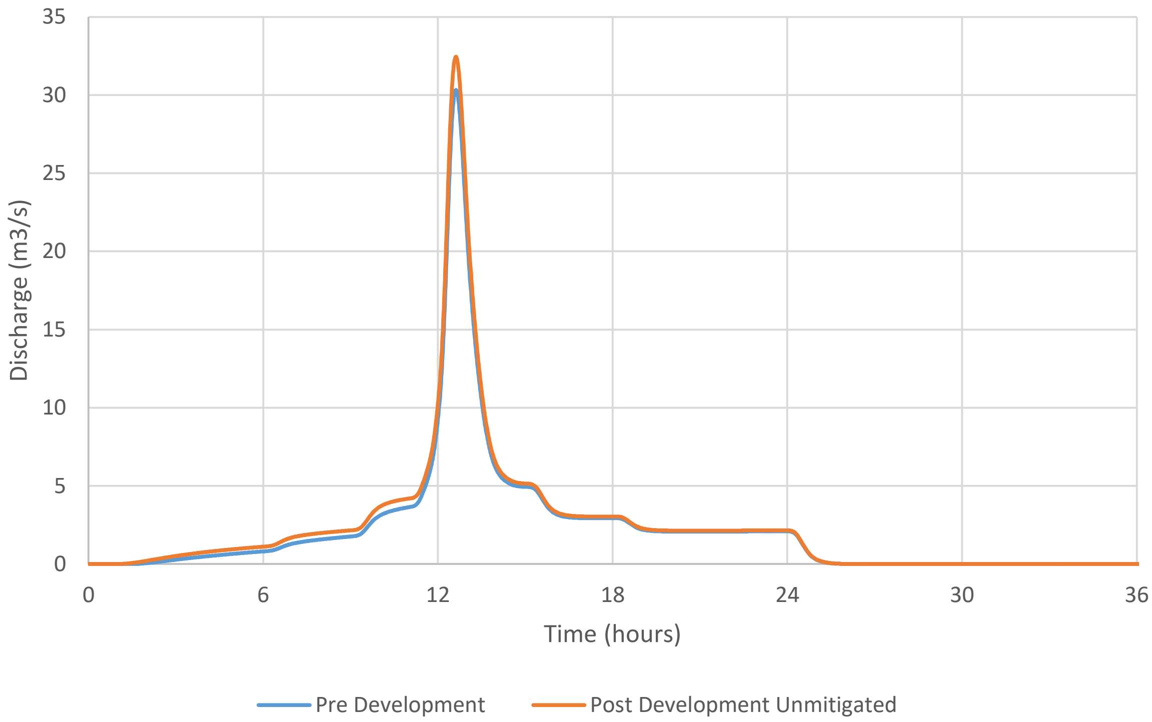

Figure 6-7: 100 year ARI, 24 hour storm duration; comparison of pre-development flows and unmitigated post-development flows for MSCZ Development sub-catchments

The results demonstrate that for the western sub-catchments there are peak flow increases of between 10% and 28% in comparison with the pre-development peak flows.

A comparison of hydrographs from the wider catchment is provided for the water quality storm and the 50 year ARI storm in Figure 6 8 and Figure 6 9 respectively. The names and locations of the wider catchments are shown in Figure 6-1, noting that the Mission Special Character Zone catchments have been merged into one output “MSCZ” for pre-development and post-development scenarios.

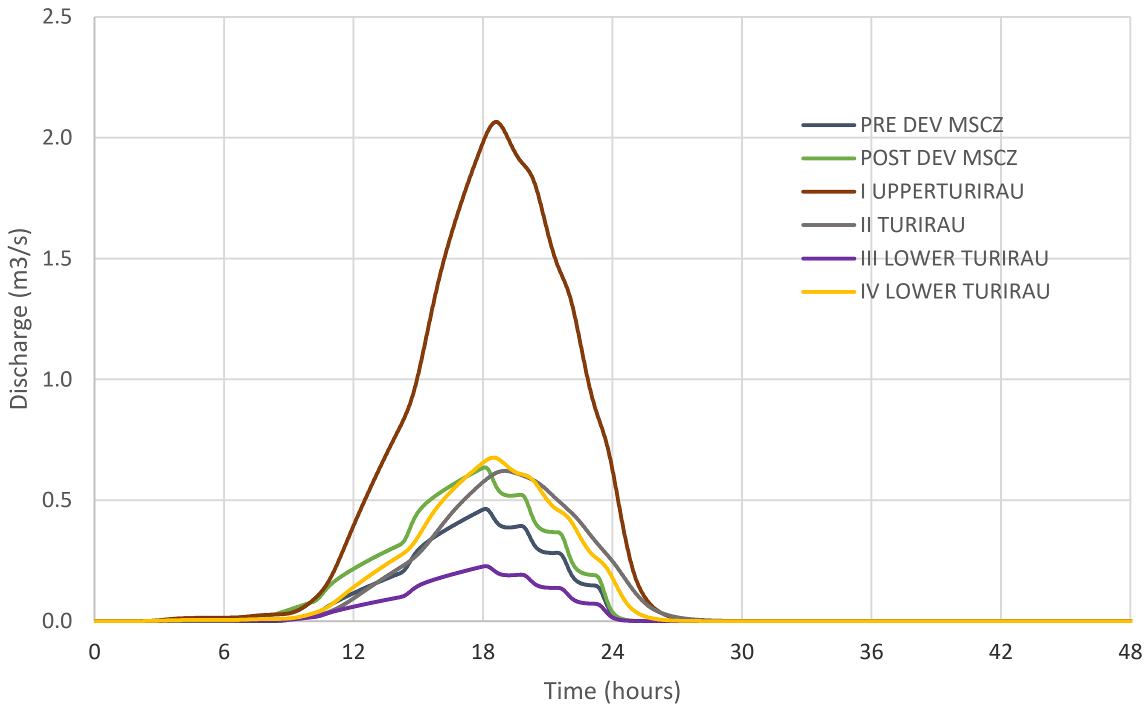

Figure 6-8 Comparison of catchment hydrographs for water quality storm

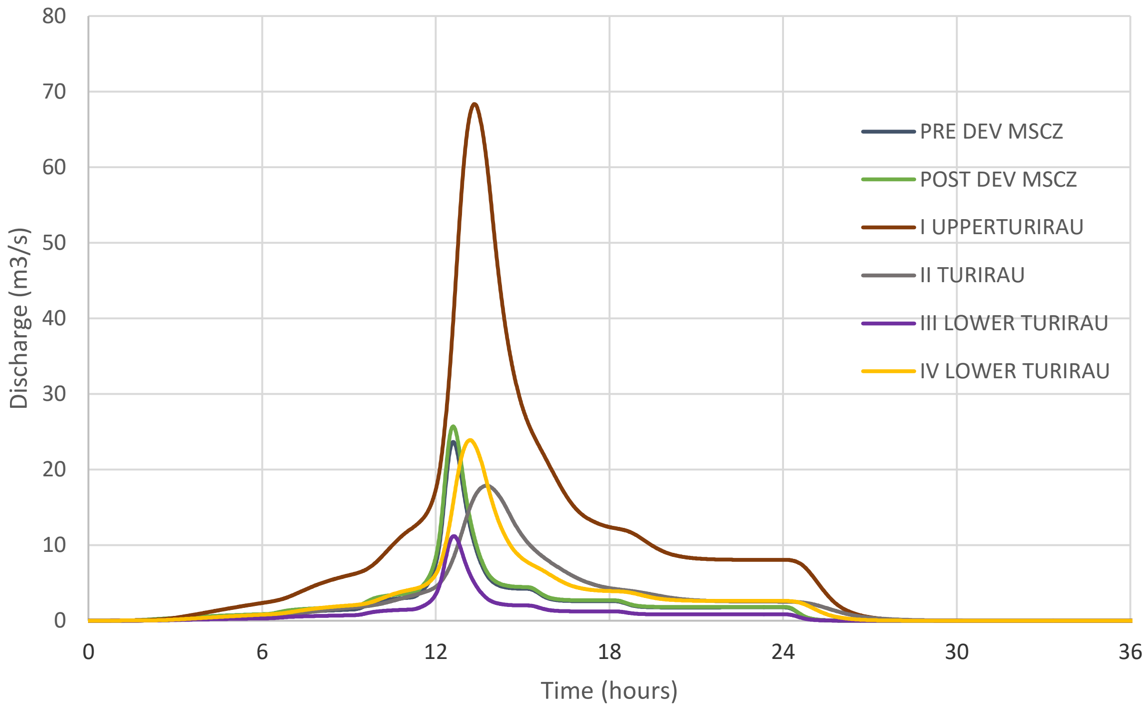

Figure 6-9 Comparison of catchment hydrographs for the 50 year ARI storm.

The hydrographs help demonstrate that the MSCZ Development catchment contribute approximately 11% of the runoff volume in the downstream catchment. The small change in discharge and volume that can be observed in the figures relative to the wider catchment can also be observed. The effect of this change will be discussed in Section 7.

6.4.1 Peak flow comparison with other methods

A comparison between Rational Method calculated 100 year ARI peak discharges and SCS peak discharges is presented in Table 6-5 where the "C" value in the rational method has been adjusted to produce similar SCS peak discharges. This has been carried out as a sense check for the SCS results.

The rainfall intensity (mm/hr) for the Rational Method has been adjusted to reflect an intensity that is similar to the time of concentration.

Table 6-5: Comparison of peak discharge from Rational method and SCS method

Catchment	Area (ha)	SCS Peak Discharge (m^3/s)	Tc (SCS) (mins)	100 yr ARI Rainfall Intensity (mm / hr)	Runoff Coefficient "C"	Rational Peak discharge Q (m3/s)
MHSCZ Combined	194	36.0	45	72	0.8	31
I	878	105.5	47	72	0.65	114
II	273	27.5	40	72	0.5	27
III	91	17.3	46	72	0.8	15
IV	284	36.9	46	72	0.6	34

The comparison between the SCS peak discharges and the Rational method peak discharges indicates that "C" values (runoff coefficient) are at the higher end of what would be expected for pervious, undeveloped land uses. However given that the land has been defined by the LINZ soil maps as "poorly drained", a higher "C" should be expected to show agreement with the SCS approach. Furthermore a higher "C" value should also be expected for 100 year ARI storms since the ground is often saturated by the time the peak rainfall occurs. The 'storage' is represented in the SCS method, but can only be represented by a higher "C' value using the rational method.

Overall we believe that the comparison between methods suggests that the peak discharge estimated is reasonable.

7 Hydraulic Effects

This section of the report describes the methodology used to carry out a hydraulic effects assessment of the unmitigated stormwater runoff. The purpose of assessing the unmitigated effects is to determine whether mitigation is required.

7.1 Hydraulic effects methodology

The effects of the unmitigated stormwater runoff was carried out using a hydraulic model, and the process is described in the following sub-sections.

7.1.1 Hydraulic model build

A computational hydraulic model was constructed using DHI MIKE Flood Software (version 2014). The MIKE Flood model comprises a 1 dimensional (1D) representation of the main Turirau Stream drainage channel and a 2D representation of the floodplain and overland flowpaths.

The vertical datum used in the model is Napier Local Authority Datum 1962. This is consistent with the LiDAR data and previous work undertaken by HBRC. The horizontal coordinate system is NZTM 2000.

The cross sections for the 1D model were sourced from stream cross section surveys carried out during September 2017 supplemented with information from Goodier (2000), and LiDAR data from 2003 sourced from Hawkes Bay Regional Council. A summary of the September 2017 cross section survey is provided Appendix D.

The modelled 1D and 2D extents are shown in Figure 7-1 including the "loading" locations of the various catchments (i.e. where the catchment hydrology is added to the hydraulic model). The key hydraulic features are presented in the shaded elevation map in Figure 7-2, which shows the inland basin (i.e. a topographic depression) that the MSCZ catchment and upstream catchments drain to (i.e. the blue area in the figure).

There are two culverts of relevance to our modelled areas of interest in the Turirau River:

1 The culvert under Puketapu Road which is immediately downstream of the inland basin,

2 The culvert under Springfield Road at the downstream extent of the model.

The downstream boundary (at Springfield Road) is represented by a stage-discharge relationship reflecting the culvert performance, assuming inlet control.

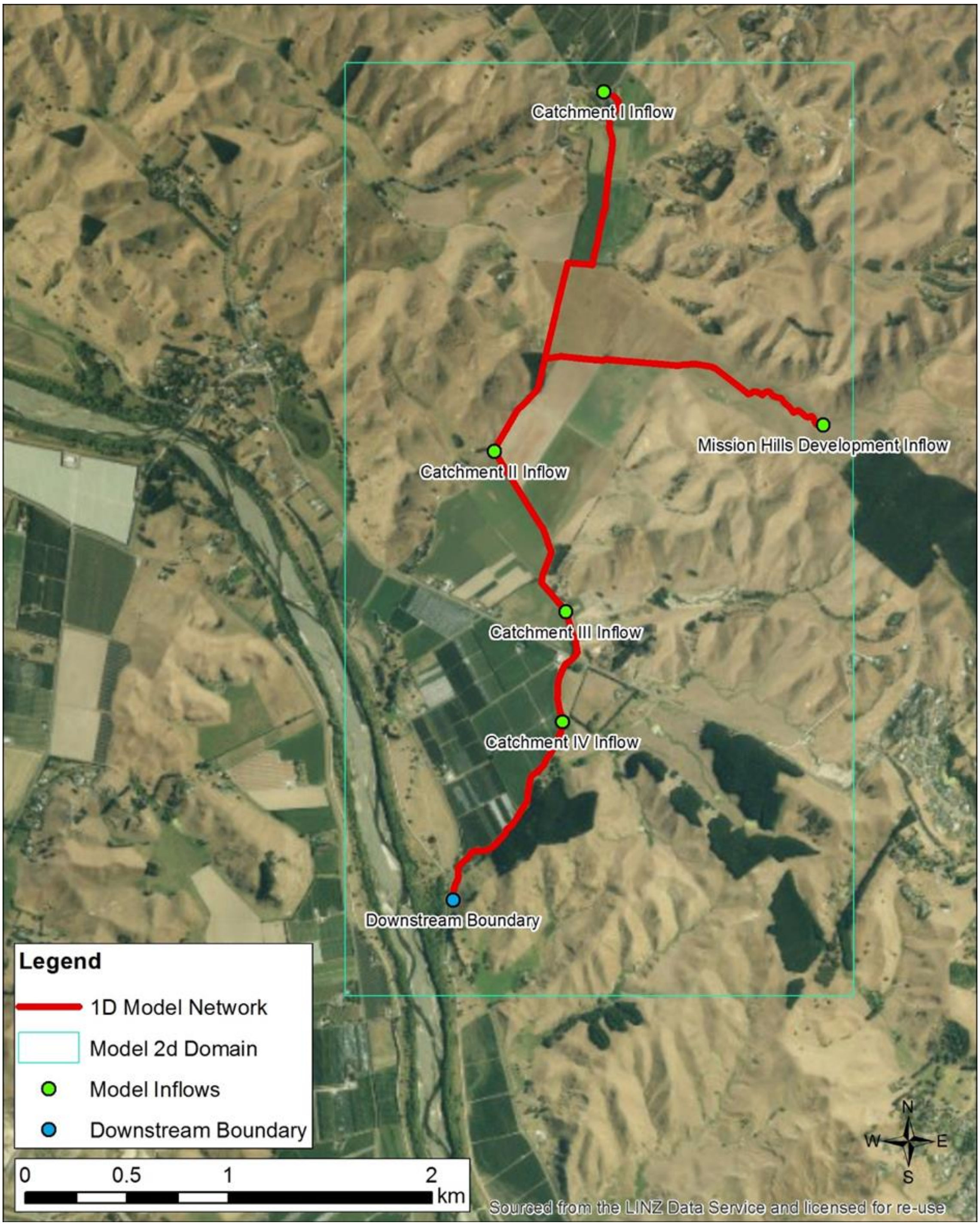

Figure 7-1: Model Schematisation

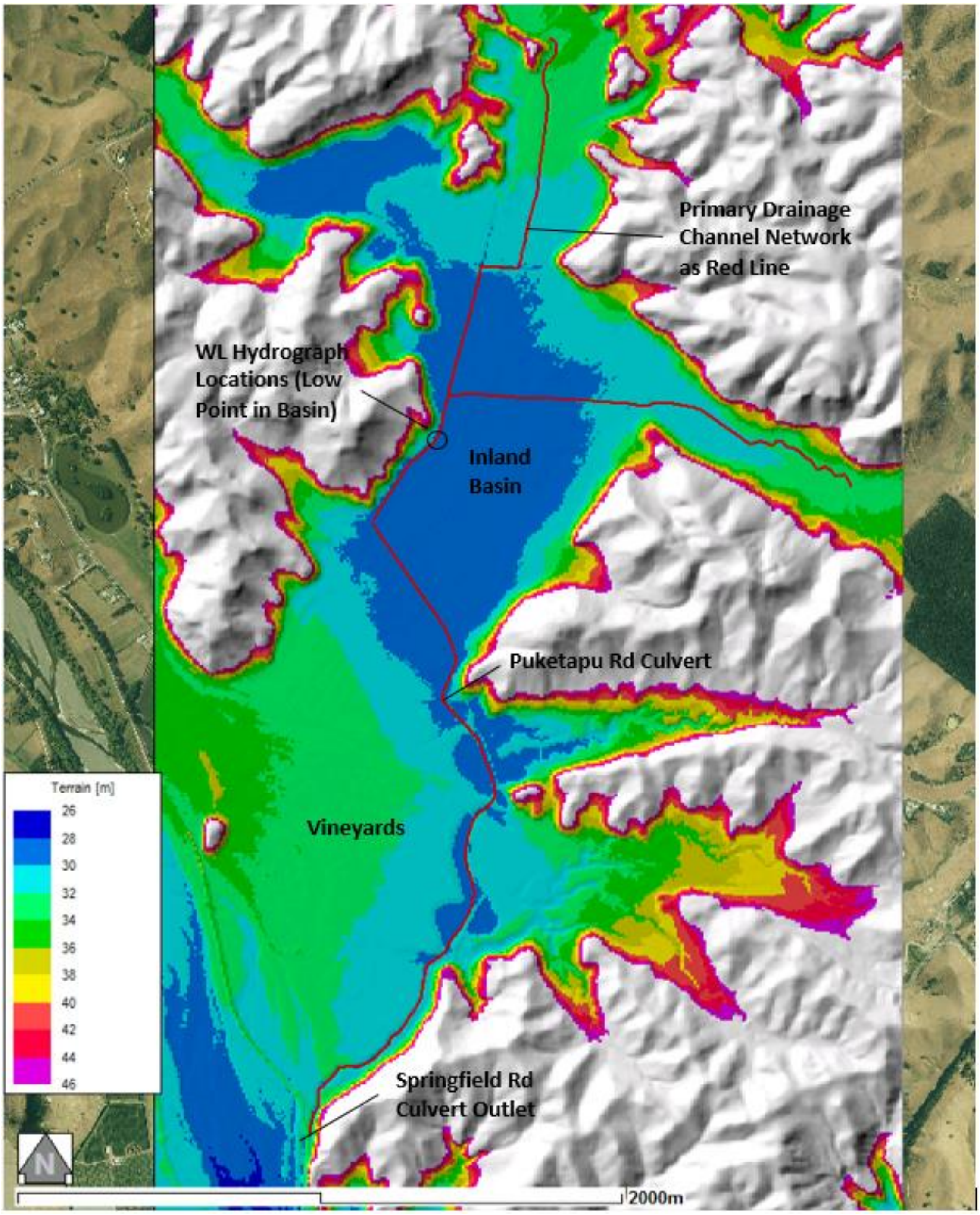

Figure 7-2: Shaded elevation map and critical model feature

7.1.2 Scenarios

For the unmitigated stormwater effects assessment, the scenarios in Table 7-1were assessed using the hydraulic model for 24 hour duration storm events.

Longer storm durations could be investigated, however the hydrological runoff response in the post mitigated scenarios for the 48 and 72 hour durations is similar to the 24 hour storm runoff response and is very likely to yield similar results. The 48 and 72 hour storm durations will require longer duration model runs which is computationally intensive and may not be necessary. This is discussed further later on.

Table 7-1: Scenarios investigated with hydraulic model

ARI	Scenario	
	Pre Development	Post Development
WQ Storm	✓	✓
2	✓	✓
10	✓	✓
100 CC	✓	✓

7.1.3 Results

Flood maps for the scenarios shown in Table 7-1 are provided in Appendix E. The flood maps show the flood depth and extents for pre-development, unmitigated post-development and differences between the two respectively.

Furthermore flood level time series is provided at the Inland Basin location shown in Figure 7-1 to show changes in duration and timing as a result of the unmitigated development. The water level timeseries are presented in Figure 7-3 to Figure 7-6. Note that the initial drop in water level (most noticeable in Figure 7-3) is an artefact of the model initial conditions where the numerical model is required to be 'wet' at start up to aid in stability and should be ignored.

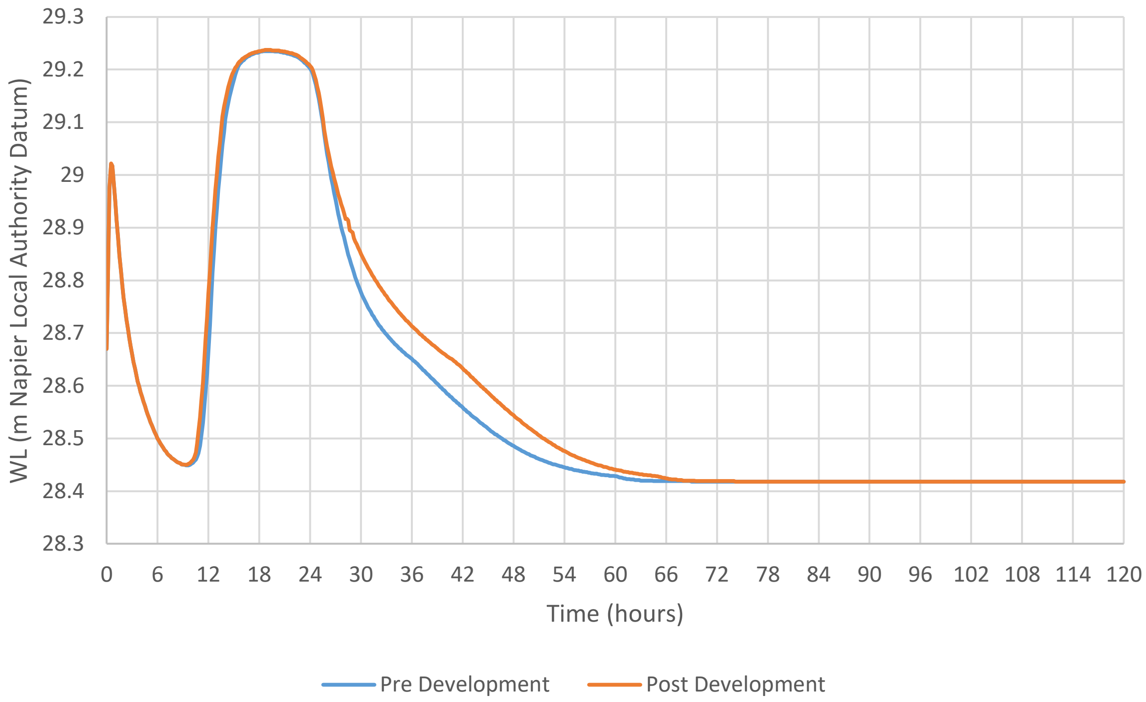

Figure 7-3: Water quality storm; comparison of water levels for pre-development and unmitigated post-development

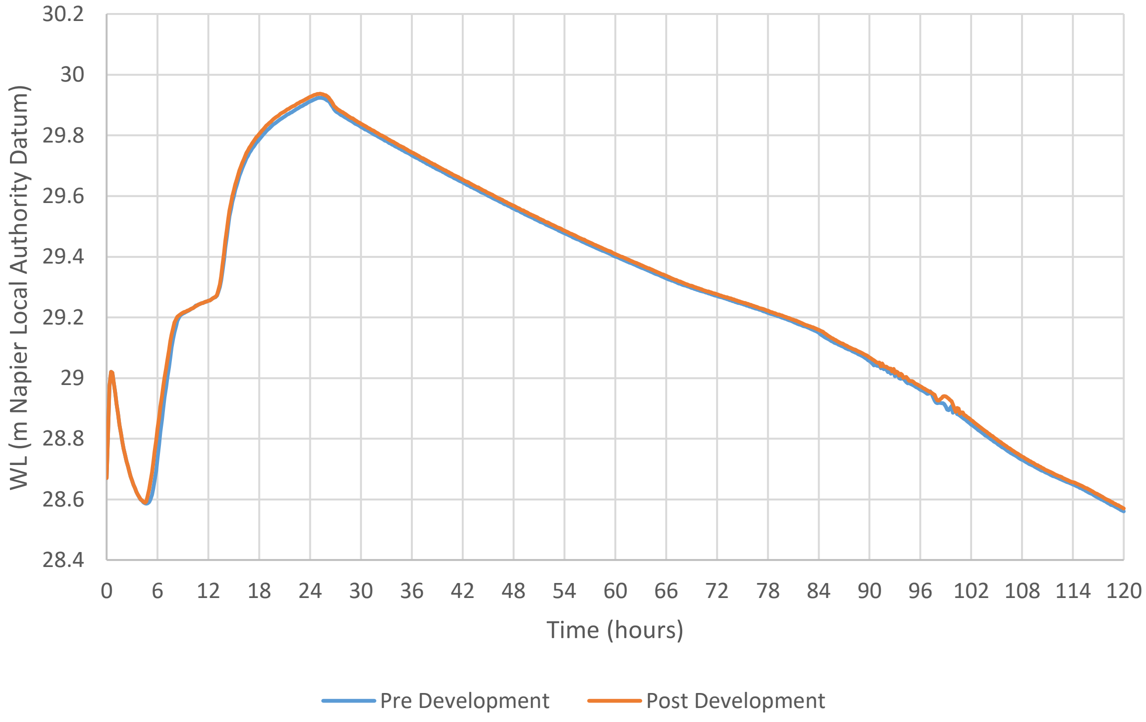

Figure 7-4: 2 year ARI storm; comparison of water levels for pre-development and unmitigated post-development

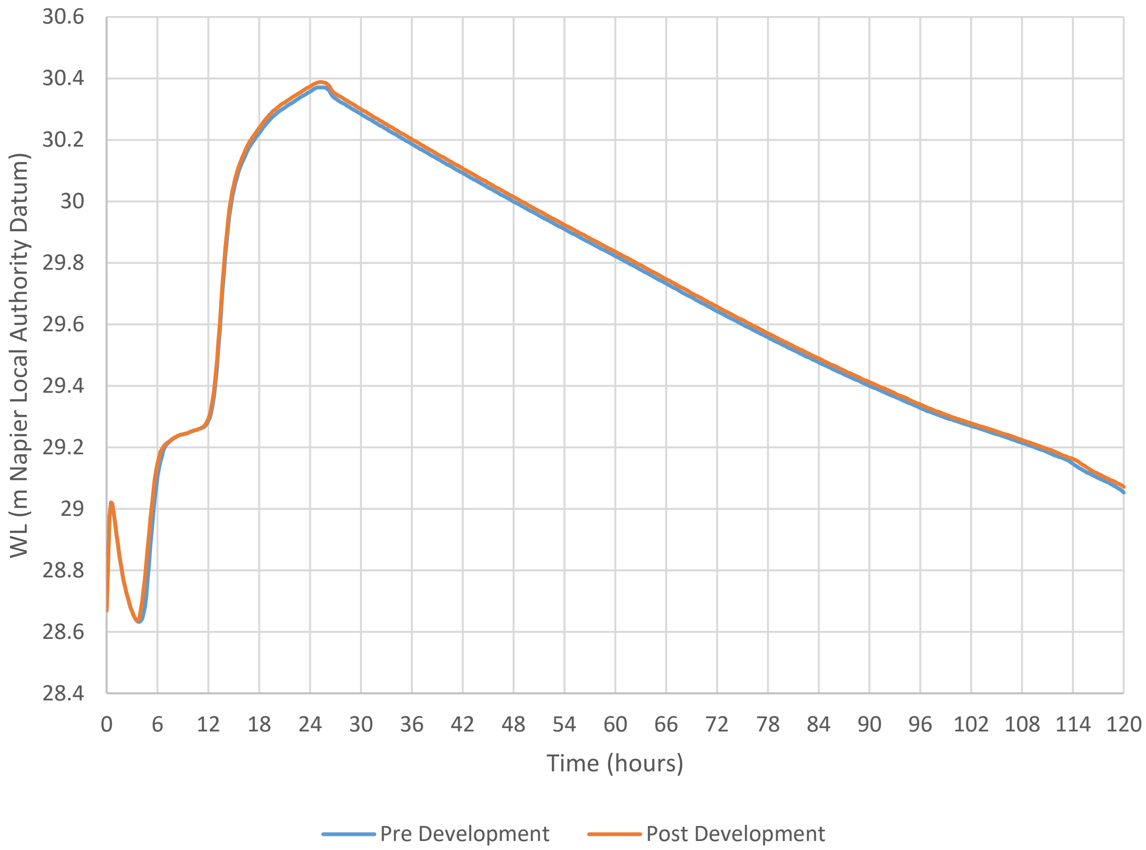

Figure 7-5: 10 year ARI storm comparison of water levels for pre-development and unmitigated post-development

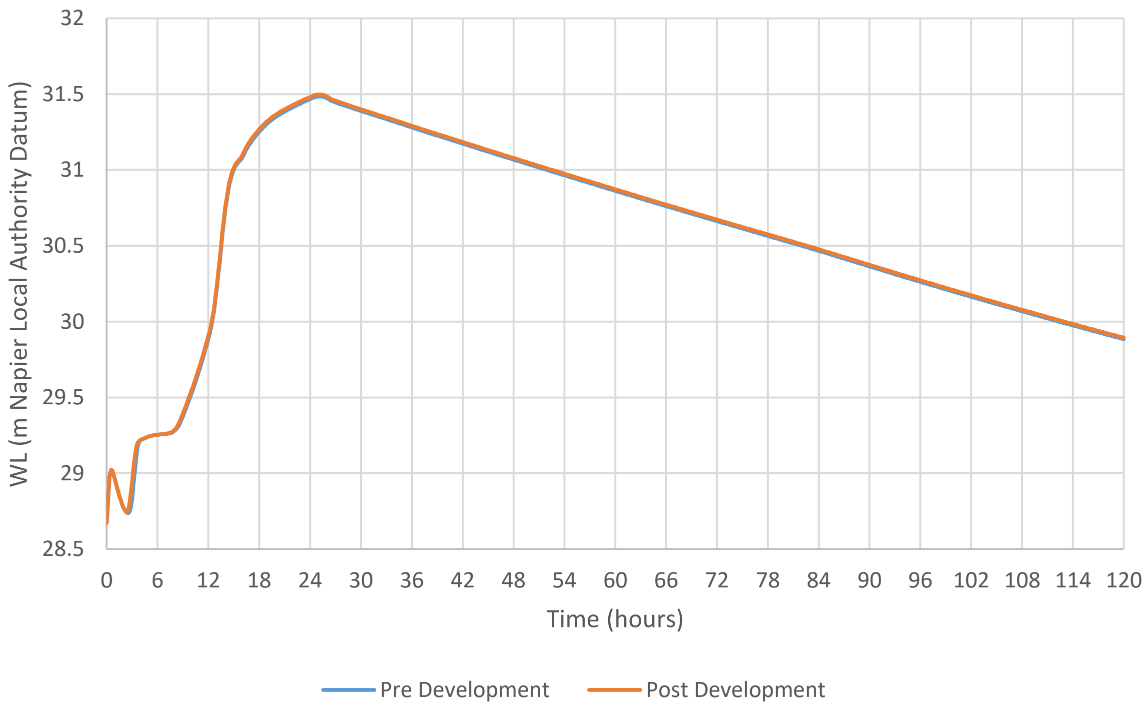

Figure 7-6: 100 year ARI storm; comparison of water levels for pre-development and unmitigated post-development

7.2 Hydraulic effects results

The flood maps presented in Appendix E show that there are small changes in downstream water level (in the order of 10-20mm) as a result of unmitigated release of stormwater from the site. The increase is insufficient to make a discernible difference to flood extents.

The change in water level is also presented by the timeseries presented in Figure 7-3 to Figure 7-6 for the Inland basin area.

Figure 7-3 to Figure 7-6 show that the duration of flooding increases by approximately 3 hours for the Water Quality storm out of a total flood duration of approximately 50 hours. The increase in the duration of flooding is less pronounced for more extreme events (typically less than 1 hour increases) and the total duration of flooding increases; thereby further reducing the relative effect.

The relatively small changes in peak water levels and duration of high water levels is a consequence of the large topographic depression at the Inland Basin which attenuates large volumes of runoff. The results suggest that the inland basin is relatively insensitive to small changes in peak discharges and volumes from the MSCZ catchment.

8 Mitigation of Flood Effects

The previous section identified that without any mitigation there are small increases in downstream water level (10mm to 100mm), and small increases in the duration of flooding (up to 3 hours).

To be consistent with HBRC SMG and to investigate whether the effects can be minimised, this section provides a stormwater management approach for mitigating the water quantity effects.

The following steps identify the processes involved in developing the flood mitigation strategy:

1. Develop a flood mitigation design concept based around reducing peak discharge to 80% of pre-development flows. This is consistent with the HBRC Waterway Guidelines and the Structure Plan rules.
2. Assess the downstream hydraulic effects of the concept design using a hydraulic model
3. Subject to the results of the previous section, revise the design to minimise downstream effects and repeat the exercise.

The flood mitigation concept is based around stormwater attenuation (storage). Soakage tests were carried out to determine whether discharge to ground was feasible. Unfortunately the soakage tests indicated that soakage to ground was not an appropriate approach. Stormwater re-use has the potential to reduce runoff volume in frequent storm events, however due to the relatively low density of the proposed development it is unlikely to be a suitable approach for reducing effects in larger storm events. Stormwater re-use has not been considered further at this stage of the design process.

The following sub-sections describe the mitigation design and effects assessment.

8.1 Stormwater quantity approach

The stormwater quantity controls discussed in this section relate only to the catchments draining to the west. This is because there a reduction in peak flows and runoff volume draining to the east caused by the changes in land use. If the quantum of planting on the eastwards catchments reduces from that shown on the concept plans, then further analysis of these catchments may be required which could require mitigation works.

8.1.1 Flood mitigation design concept

The concept for the stormwater quantity approach is to control the effects caused by increased runoff through a series of stormwater management ponds (or wetlands). They will be arranged to capture runoff from each of the gullies. The ponds will provide attenuation to reduce peak flows, reduce the volume of runoff at critical times and delay the hydraulic response times downstream.

The design for the stormwater management ponds follows the guidance set out in the HBRC WGSM. These include:

- The ability to reduce the 100 year ARI peak discharge (including climate change) to 80% of pre-development peak discharge
- A recommendation that the 2- and 10-year post-development peak discharges do not exceed the 2- and 10-year pre- development peak discharges.
- Water quality control volume based of 30mm of runoff from catchment impervious surfaces.
- 50% of the water quality volume allocated to permanent pond storage.
- 1.2 times the water quality volume released as extended detention over 24 hours.
- Removal of 75% of total suspended solids

The location of the proposed ponds are shown in Figure 8-1 which shows a concept sketch in plan of how the ponds will be positioned in the natural contours in each of the major gullies. The ponds will be formed by constructing an earth embankment at the downstream end of the gulley, through which an outlet and spillway will be designed and constructed.

The method for passing stormwater flows to the ponds will be confirmed at subsequent stages of design however it is likely to be via conventional curb and channel collection. Runoff will then be piped to the base of the gulley, where it can be conveyed overland to the pond. Outlet energy dissipation is a likely requirement for the discharge locations into the gullies.

For concept design, the ponds were sized to minimise their impounded height (keeping below 3m depth of water during the most extreme event modelled), and conform to the sites existing contours in the gullies. The HBRC 2003 LiDAR was used to determine storage versus elevation relationships for the pond locations. The location of the downstream bund and the outlet configuration was modified through an iterative approach until the performance requirements were met.

Table 8-1 shows the indicative surface area, maximum depth and outlet configurations for each of the ponds. The final earthworks, geotechnical details, outlet arrangement, outlet dimensioning, and scour protection details will be confirmed in detailed design. The final earthworks, geotechnical details, outlet arrangement, outlet dimensioning, and scour protection details will be confirmed in detailed design.

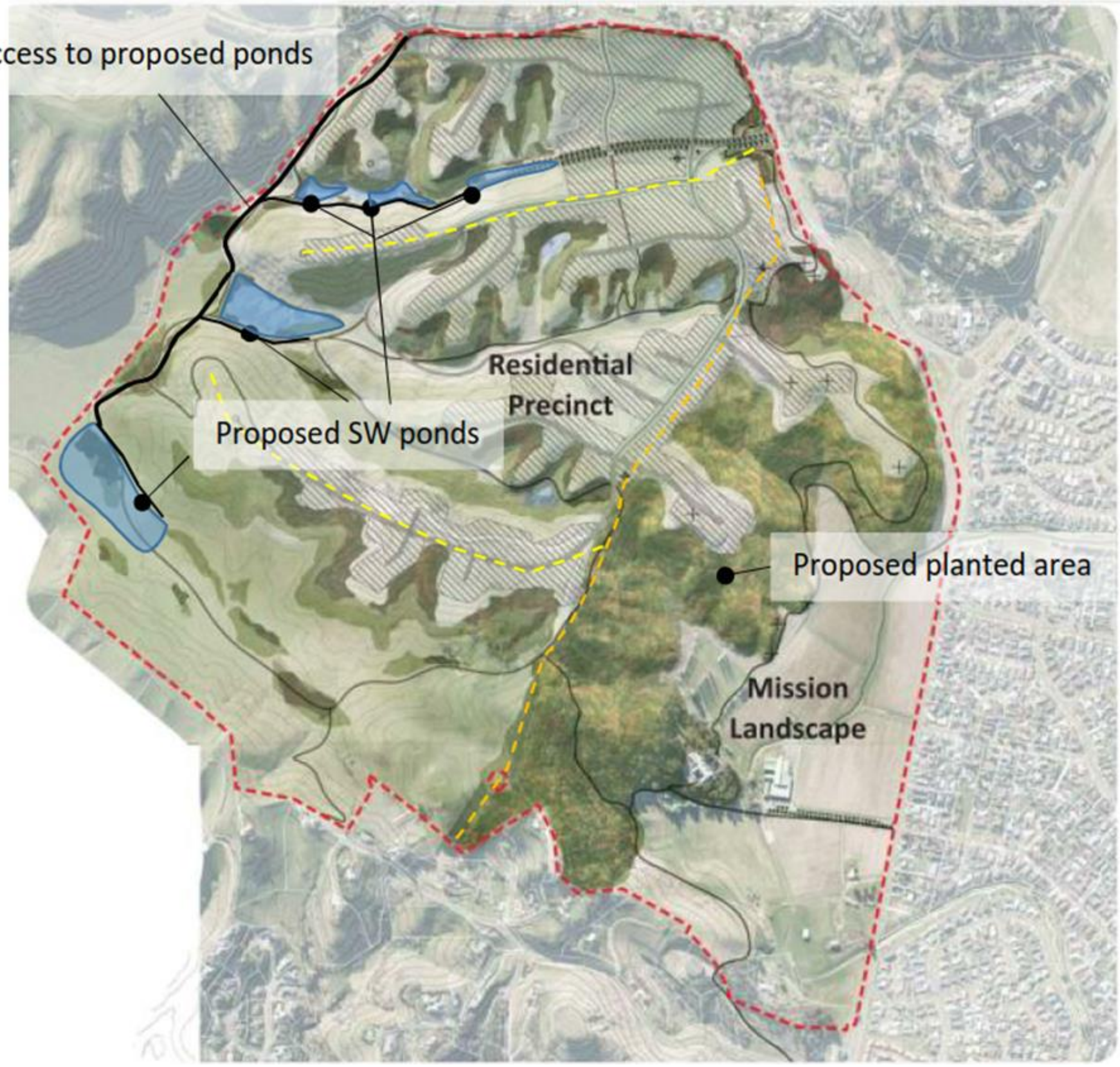

Figure 8-1: Proposed pond locations

Table 8-1: Pond key parameters for stormwater quantity control

	100 year ARI + CC		**Outlet configuration for 100 year ARI**
	Indicative surface area (m^2)	**Maximum depth of impounded water (m)**	
Catchment 1 Pond	8,400	3	100mm dia pipe for EDV release at 0.5m depth + 1.2m dia pipe at 1m depth + spillway at 3m depth
Catchment 2 Pond	10,540	3	300mm dia pipe for EDV release at 0.5m depth + 1.2m dia pipe at 1m depth + spillway at 3m depth
Catchment 3 Pond A	5,323	3	100mm dia pipe for EDV release at 0.5m depth + 1.2m dia pipe at 1m depth + spillway at 3m depth
Catchment 3 Pond B	6,400	3	100mm dia pipe for EDV release at 0.5m depth + 1.2m dia pipe at 1m depth + spillway at 3m

	100 year ARI + CC		Outlet configuration for 100 year ARI
	Indicative surface area (m^2)	**Maximum depth of impounded water (m)**	
Catchment 3 Pond C	7,640	3	100mm dia pipe for EDV release at 0.5m depth + 1.2m dia pipe at 1m depth + spillway at 3m

8.1.1.1 Comparison of volume estimates using other methods

We consider the method provided in the previous sub-section as the most appropriate method for assessing the likely pond requirements of the Mission Special Character Zone. Nonetheless, additional "sense checks" are provided below.

The difference in runoff between pre-development and post-development hydrology when the post development flow exceed the predevelopment can be used as a lower estimate of pond volume. It does not consider the need to attenuate to 80% of pre-development discharge and does not consider outlet configuration. Therefore the volumes estimated using this methodology should be less than the concept design.

Figure 8-2 to Figure 8-4 show a comparison of pre-development and post-development hydrographs for the 100 year ARI design storms for west flowing catchments. Table 8-2 shows a comparison of theoretical pond storage volume for each of the ponds created by calculating the difference in volume between pre-and post-development discharge where post development discharge exceeds pre development discharge (i.e. area under the curve) in comparison to the proposed pond volume calculated using the hydrological routing model in this analysis.

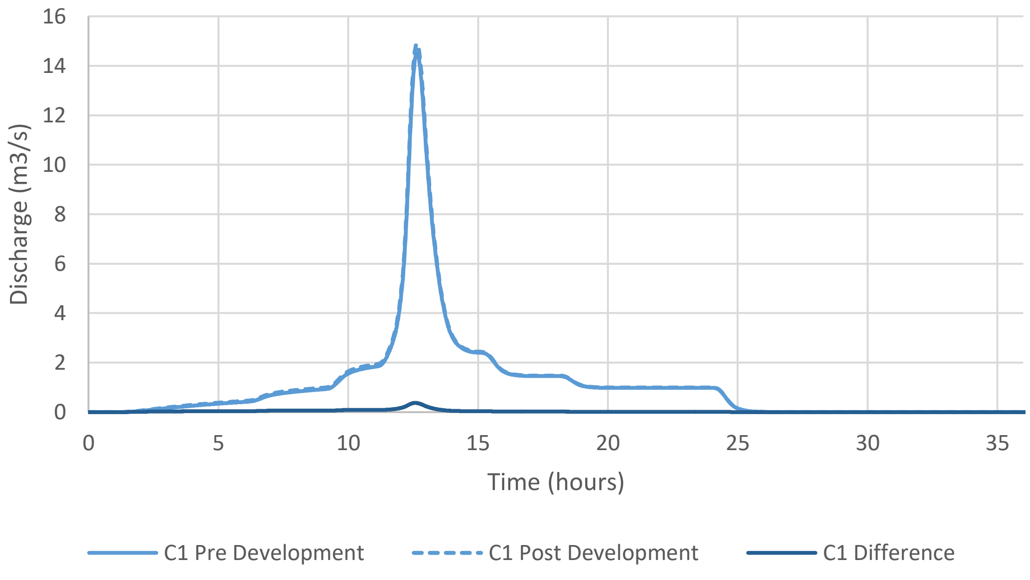

Figure 8-2: Comparison of pre-development and post-development runoff for 100 year ARI design storm with allowance for climate change for MSCZ catchment 1

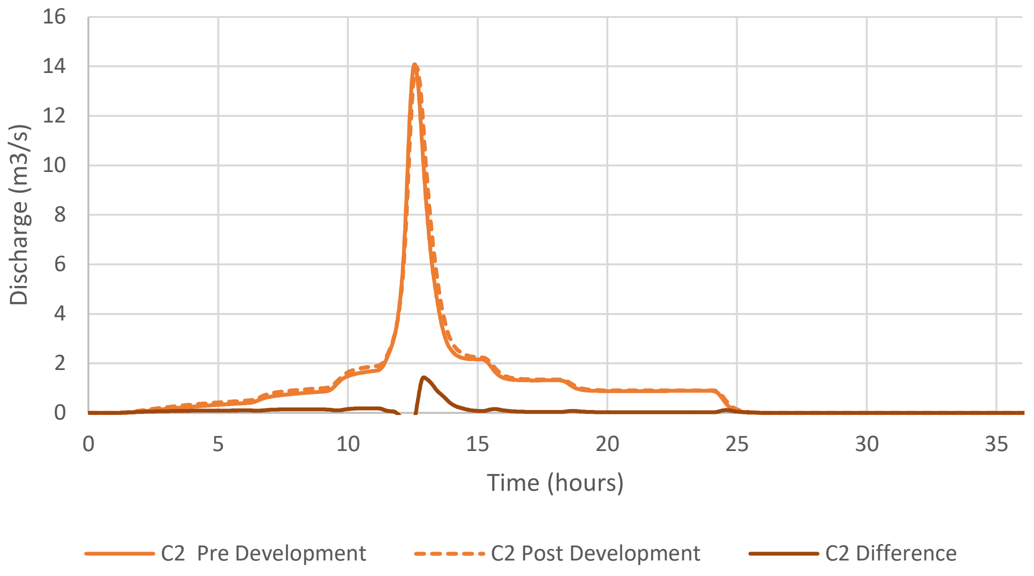

Figure 8-3: Comparison of pre-development and post-development runoff for 100 year ARI design storm with allowance for climate change for MSCZ catchment 2

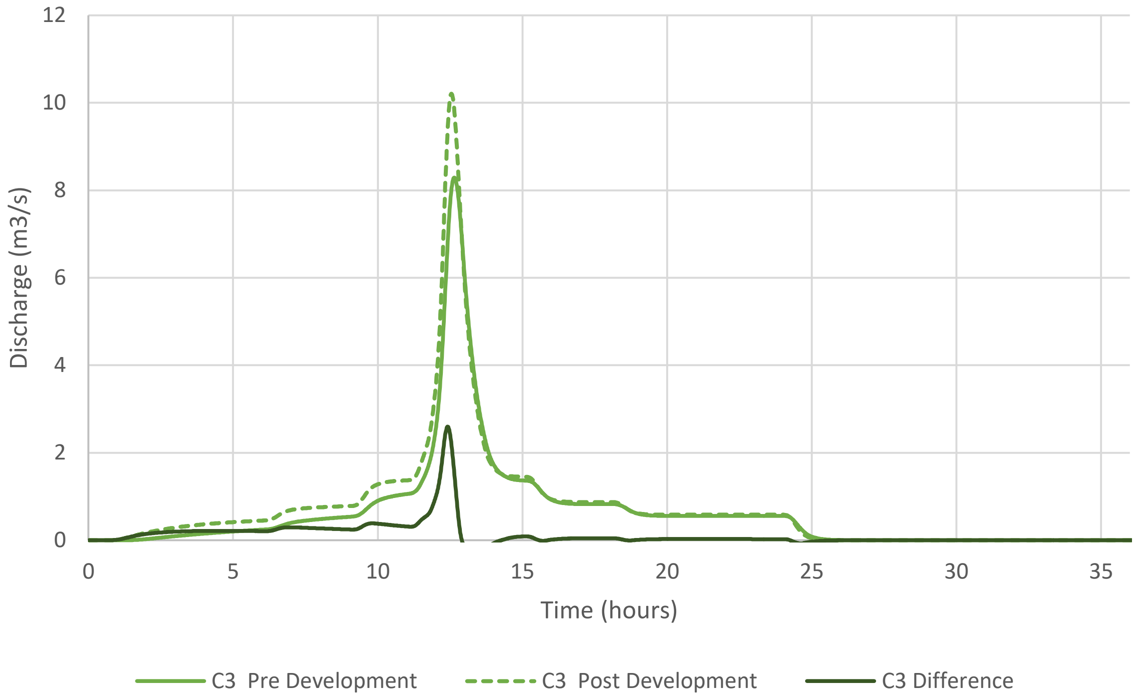

Figure 8-4: Comparison of pre-development and post-development runoff for 100 year ARI design storm with allowance for climate change for MSCZ catchment 3

Table 8-2: Comparison of theoretical minimum storage requirements for 100 year ARI adjusted for climate change pond volume against proposed pond volume

MSCZ Catchment	Volume of Minimum Storage Required Between Pre and Post Dev Hydrographs (m3)	Proposed pond volume for Volume applied for proposed 80% of pre dev peak discharge (for 2y ARI to 100y CC ARI) determined using routing model (m3)
Catchment 1	4,082	24,500
Catchment 2	10,782	27,400
Catchment 3	16,398	32,000

The comparison provides confidence in the concept design because the proposed pond volumes are significantly higher than those derived using from the hydrological comparison method.

Another method is proposed for determining the volume needs for storage in the HBRC SMG (Section 6.1.4.2). The approach is limited in the guidance to 2 and 10 year ARI storms and makes some simplistic assumptions. At the request of HBRC, we have compared the 10 year ARI storage volume using the HBRC SMG method using the parameters shown in Table 8-3.

To implement the method, we assumed a post-development peak discharge based on a rational method estimate of 10 year ARI peak flows, as shown in Table: 8-4. The peak 10 year ARI flows were then applied to the volume calculation shown in Table: 8-4.

Table 8-3: Rational method parameters for MSCZ west draining catchments for HBRC SWMG pond sizing calculations

Pond	Catchment Area	tc (min)	Duration (min)	i 60 min (mm/h)	Runoff Coefficient	Rational Method Storm Peak Discharge (m3/s)
C1	78	45	60	31	0.6	4.03
C2	70	45	60	31	0.7	4.22
C3	44	45	60	31	0.8	3.03

Table: 8-4 Comparison of calculated pond volumes between HBRC SWMG method and proposed pond volumes derived from hydrological analysis

Pond	WQV (m3)	Dead Storage (m3)	EDV (m3)	Rational Method Estimated Volume Using HBRC WGSM (m3)	Total Pond Volume Using HBRC WGSM for 10y ARI (m3)	Proposed pond volume for Volume applied for proposed 80% of pre dev peak discharge (for 2y ARI to 100y CC ARI) determined using routing model (m3)
C1	1477	739	1772	18,135	21,762	23,978
C2	4848	2424	5818	16,275	22,785	30,057
C3	6413	3207	7695	10,230	16,368	25,988

The results of the comparisons show that the proposed pond volumes exceed the indicative volume comparison methodologies provided above. This should help to provide further confidence that the pond volumes are sufficient.

8.1.2 Mitigated hydrology results

Table 8-5 provides a comparison of the peak pre-development discharge and the peak post-development discharge after mitigation. It is provided to determine whether the 80% of pre-development peak flow guideline has been met. A comparison of pre-development hydrographs, unmitigated post development hydrographs and mitigated post development hydrographs is provided in Appendix C and the inflows and outflows for each of the proposed ponds are provided in Appendix F.

Table 8-5: Comparison of mitigated post-development flows for catchments draining to the west

Storm Duration	Event (ARI)	Pre-Development Discharge (m^3/s)	Post-Development Discharge (m^3/s)	% of Pre-Development Discharge
24 hr	2	5.6	3.8	68%
	2 + CC	7.0	4.4	62%
	10	12.1	6.4	53%
	10 + CC	15.3	7.3	48%
	50	23.1	11.3	49%
	100	29.7	16.5	56%
	100 + CC	36.0	26.6	74%
48 hr	2	6.2	4.0	65%
	10	13.3	6.7	51%
	50	24.6	12.3	50%
	100	31.1	17.5	56%
	100 + CC	37.5	27.5	73%
72 hr	2	6.7	4.2	62%
	10	13.7	6.9	50%
	50	25.3	12.8	50%
	100	31.8	18.1	57%
	100 + CC	38.2	27.9	73%

A comparison of the results with the HBRC SMG is provided in Table 8-5

Table 8-6: Comparison of mitigated hydrology with HBRC SMG

HBRC SMG Item	Result
The ability to reduce the 100 year including climate change effects design storm to 80% of pre-development peak discharge	Requirement exceeded. Peak flows reduced to 78% in 100 year ARI with allowance for climate change, and 73% of pre-development flows in 100 year ARI without allowance for climate change.
It is recommended that the 2- and 10-year post-development peak discharges not exceed the 2- and 10-year pre- development peak discharges for 24 hour storm duration.	This requirement is considerably exceeded since the post-development flows are approximately 67% and 53% of the pre-development flows for 2 year ARI and 10 year ARI events respectively.

8.2 Downstream effects of concept mitigation design

Flood maps showing peak flood depths, flood extent and a comparison between pre-development and post-development are provided in Appendix G.

Figure 8-5 provides a comparison of flood level at the Inland Basin for pre-development and the post-development scenarios for a 2 year ARI, with and without mitigation. Figure 8-6 provides a finer resolution of the same information so that the differences can be seen more clearly.

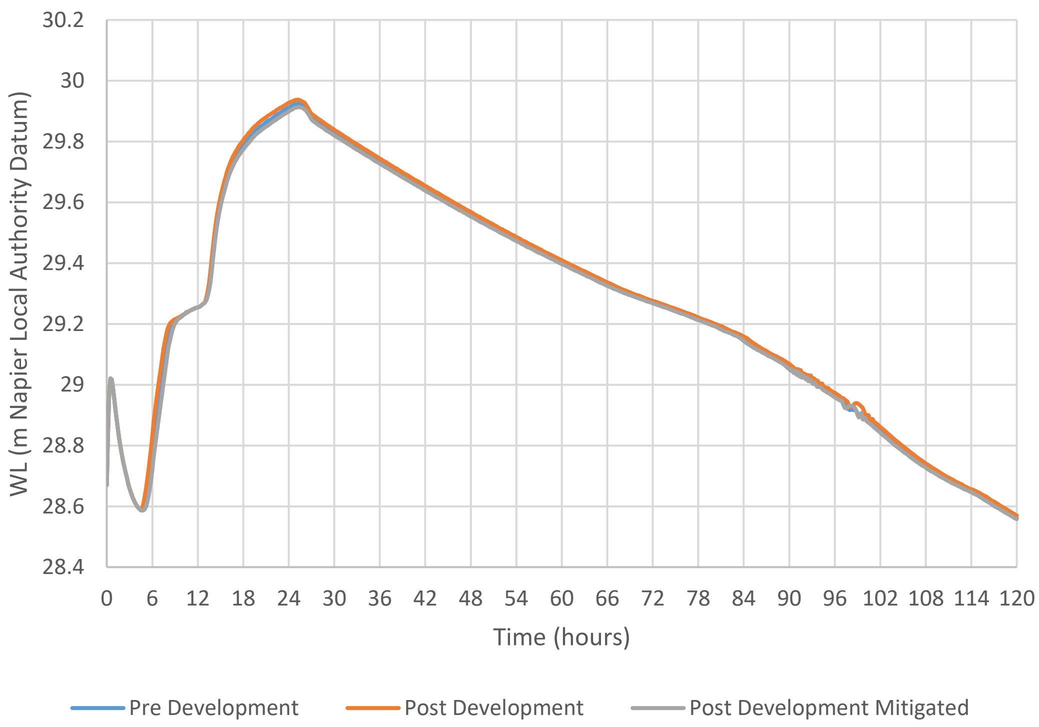

Figure 8-5 2y ARI storm event WL hydrographs in inland basin

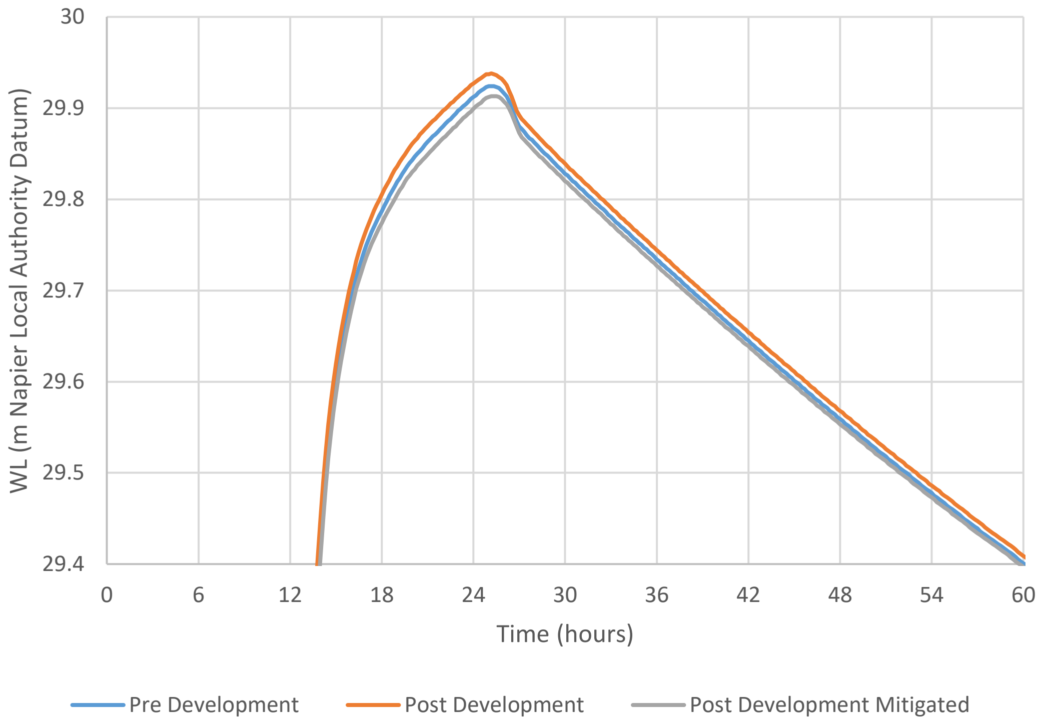

Figure 8-6 2 y ARI storm event WL hydrograph in inland basin zoomed in on peak

The flood maps show that there is a small reduction in downstream flood levels for the water quality storm, 2 year ARI design storm and 10 year ARI design storm. The results indicate that the flood levels will be very similar (i.e. less than 10mm difference) in the 100 year ARI design storm scenario.

The flood level timeseries has only been provided for the 2 year ARI on the basis that the unmitigated scenarios showed that the adverse effects were more noticeable in the more frequent events. Additional timeseries figures can be provided to demonstrate the point further if required.

Additional hydrological scenarios can be considered for mitigated hydraulic analysis for longer duration storm events (e.g. 48 hour and 72 hour). However on the basis that the hydrological runoff response in the mitigation scenarios for the 48 and 72 hour durations is similar to the 24 hour storm runoff response (refer Table 8-5 and Appendix C), we consider it likely to yield similar results. The 48 and 72 hour storm durations will require longer duration model runs which are computationally intensive and considered unnecessary.

9 Stormwater Quality

The concept design demonstrated in the previous section incorporates stormwater ponds as a method of providing flood attenuation to reduce downstream effects.

Stormwater ponds also provide water quality treatment through the capture of sediment, and associated contaminants. Through appropriate vegetation selection and topographic contouring it may be possible in subsequent design stages to change the stormwater management ponds into wetlands, or add wetland pond elements to them. Both wetlands and ponds promote sedimentation, however wetlands also promote biological uptake of contaminants for water quality treatment. The choice of vegetation which can withstand both long, dry periods and also relatively deep inundation depths and multiple day flood durations will require advice from an appropriate expert.

Both stormwater management ponds and stormwater wetlands are consistent with the Low Impact Design guidelines in the Hawke's Bay Waterway Guidelines. However we refer to stormwater management ponds for the remainder of this report.

The stormwater quality approach builds on the stormwater quantity approach by utilising the ponds in the western draining catchments for quality treatment as well as quantity control. The drivers for the ponds were determined from the HBRC SMG, notably:

- Water quality control volume based on 30mm of runoff from catchment impervious surfaces.
- 50% of the water quality volume allocated to permanent pond storage.
- 1.2 times the water quality volume released as extended detention over 24 hours.
- Removal of 75% of total suspended solids

The ponds have been designed so that the extended detention volume (EDV) is released gradually over 24 hours by locating small diameter outlets below the primary outlets at 0.5m of water depth above the dead storage volume level. The primary outlets can be detailed as either culverts or weirs with their inverts located just above the extended detention volume, at 1m water depth. High level emergency spillways will be integrated into the design to prevent water exceeding crest levels of 3m water depth.

The primary outlet levels and configuration will be confirmed during detailed design; however due to the oversized nature of these ponds (to meet the 80% pre-development 100 year ARI peak flow requirement) there should be a high degree of confidence that the water quality controls can be accommodated within the proposed pond areas.

Table 9-1 provides a summary of the key pond requirements for water quality treatment.

Table 9-1: Pond key parameters for water quality treatment

Pond	Dead volume = 50% WQV (m^3)	Water Quality Volume WQV (m^3)	Extended Detention Volume =1.2WQV (m^3)
Catchment 1	740	1,480	1,776
Catchment 2	2400	4,800	5,800
Catchment 3 Pond A	1965	3,930	4,715
Catchment 3 Pond B	820	1,640	1,968
Catchment 3 Pond C	422	844	1,013

Further design consideration regarding forebay design, planting and access requirements for the ponds should be considered further at the detailed design. There are no fundamental problems with any of these based on the concept proposed in this report.

Stormwater runoff from the eastward sloping catchment should be collected via a curb and channel collection. Runoff should be passed through a gross pollutant trap and coarse filtration before being discharged via a level spreader onto the proposed planting areas. The level spreader is intended to reduce the potential for erosion and increase the area of planting that the stormwater discharge is exposed to. Downstream of the level spreader, the proposed planting will continue to treat stormwater contaminants through a combination of biological processes and filtration. No further treatment is considered necessary.

10 Conclusions

This report has been prepared to assess the downstream effects of the proposed development of the Mission Special Character Zone by Marist Holdings (Greenmeadows) Ltd. It has considered the measures required to meet the stormwater goals outlined in the Hawke's Bay Regional Council Stormwater Management Guidelines (HBRC SMG) and the standards set out in Rule 42 of the Hawkes Bay Regional Resource Management Plan in regards to the downstream receiving environment (Rule 42b) and quantity (Rule 42a). Meeting these goals would demonstrate that the development can proceed as a permitted activity.

The development proposes to increase impervious surface across the site which, if unmitigated, has the potential to increase runoff volume, produce higher peak flows and faster hydrological response times than the current situation runoff volume and peak flow rates, thereby increasing the potential for downstream flooding and increased erosion. Increased erosion can cause downstream sedimentation and turbidity problems in the watercourses.

The proposed development will increase impervious coverage by approximately 43 ha for land draining to the west to the Turirau Stream and by 3.4 ha for land draining to the east to the Taipo Stream. However runoff to the east will be mitigated through landscape planting, which will result in a net reduction in runoff.

The increase in impervious surface for land draining to the west represents approximately 2.5% of the Turirau catchment area (~1720 ha) and 3% of the catchment upstream of a farmland area which is known to flood (~1345 ha). Irrespective of the mitigation works at the Mission Special Character Zone site, the farmland will continue to flood to a similar degree to what is currently experienced.

Without mitigation, the downstream flood levels will increase by between 10mm and 20mm however the increase is insufficient to make a discernible difference to flood extents. The duration of flooding from the water quality storm (30mm rainfall over 24 hours) will increase by approximately 3 hours in comparison with a predicted flood duration of approximately 50 hours. This is considered representative of a flood event which may occur more frequently than once per year. For less frequent events (e.g. 2 year ARI, 10 year ARI and 100 year ARI) the increase in flood duration is less pronounced as the period of flooding increases. Increases in the order of 1 hour are likely. The relatively small changes in peak water levels and duration of high water levels is a consequence of the large topographic depression at the Inland Basin which attenuates large volumes of runoff. The results suggest that the inland basin is relatively insensitive to small changes in peak discharge and volumes from the MSCZ catchment.

To be consistent with HBRC SMG requirements for areas upstream of floodplains, flood mitigation options have been considered to reduce the downstream effects and to attenuate post-development peak flows to 80% of predevelopment peak flows.

The concept developed to meet these requirements was to develop a series of stormwater management ponds which will capture runoff from each of the gullies, attenuate peak flows and reduce the volume of runoff at critical times, thereby delaying the hydraulic response times downstream.

The analysis has shown that peak flows have been reduced to 78% and 73% of the predevelopment peak flows for a climate change and no climate change scenario respectively. Furthermore the concept design shows that the post-development flows are approximately 67% and 53% of the pre-development flows for 2 year ARI and 10 year ARI events, thereby meeting another guideline from the HBRC SMG.

The effect of the mitigation is to reduce downstream flood levels by between 10mm and 20mm for the water quality storm, 2 year ARI and 10 year ARI. We expect flood levels to be very similar to existing flood levels in the 100 year ARI.

Overall we conclude that there is no downstream negative effect caused by the development. The concept design included in this report is likely to cause a small reduction in downstream flood levels.

The stormwater ponds have been designed with appropriate outlet configurations and storage volume to provide water quality treatment that meets the HBRC SMG. That is:

- Water quality control volume based on 30mm of runoff from catchment impervious surfaces.
- 50% of the water quality volume allocated to permanent pond storage.
- 1.2 times the water quality volume released as extended detention over 24 hours.
- Removal of 75% of total suspended solids

Through appropriate vegetation selection and topographic contouring it may be possible in subsequent design stages to change the stormwater management ponds into wetlands, or add wetland pond elements to them. Both wetlands and ponds promote sedimentation, however wetlands also promote biological uptake of contaminants for water quality treatment. The choice of vegetation which can withstand both long, dry periods and also relatively deep inundation depths and multiple day flood durations will require advice from an appropriate expert.

The primary outlet levels and configuration will be confirmed during detailed design; however due to the oversized nature of these ponds (to meet the 80% pre-development 100 year ARI peak flow requirement) there should be a high degree of confidence that the water quality controls can be accommodated within the proposed pond areas.

During the detailed design stages of this project, there will be opportunities to refine and optimise the stormwater design, however based on the assessment and recommendations from this report, Marist Holdings (Greenmeadows) Ltd should be confident that the proposed development can be carried out in a manner that does not cause negative downstream effects from flooding and does not create adverse water quality effects. The stormwater discharge from the proposed development will therefore meet the permitted activity standards of Rule 42 of the Hawke’s Bay Regional Resource Management Plan as it will not reduce of the ability of the receiving channel to convey flood flows, cause any bed scouring or bank erosion of the receiving channel, or cause the production of conspicuous oil or grease films, scums or foams, or floatable or suspended materials in any receiving water body after reasonable mixing.

11 References

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12 Applicability

This report has been prepared for the exclusive use of our client Marist Holdings (Greenmeadows) Limited, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
	pp
....	....
Hamish Smith	Andy Pomfret
Water Resources Engineer	Project Director

....

Jon Rix

Senior Water Resources Consultant

HSMI
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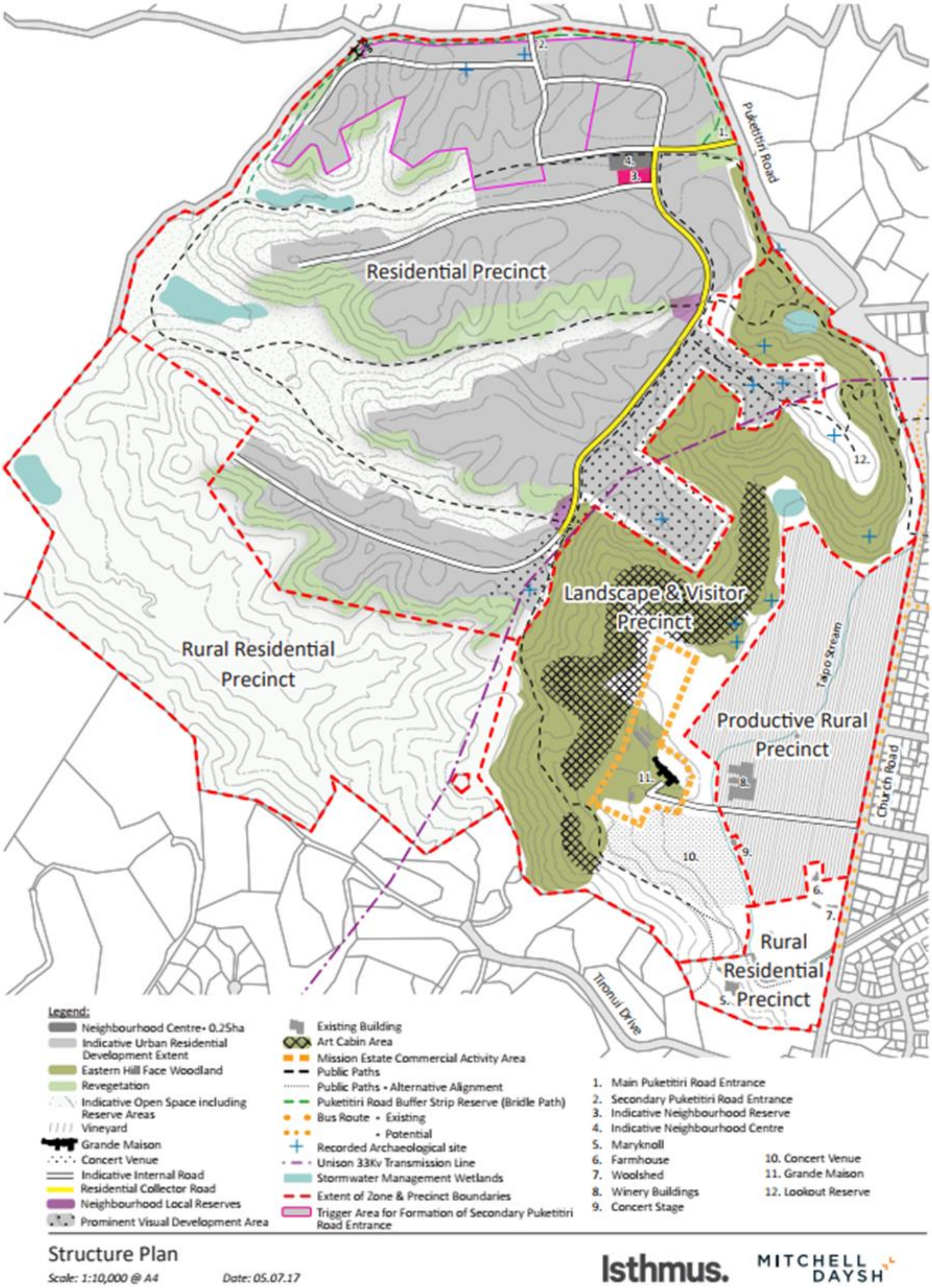
Appendix A: Structure Plan Vegetation East Hill Face Woodland



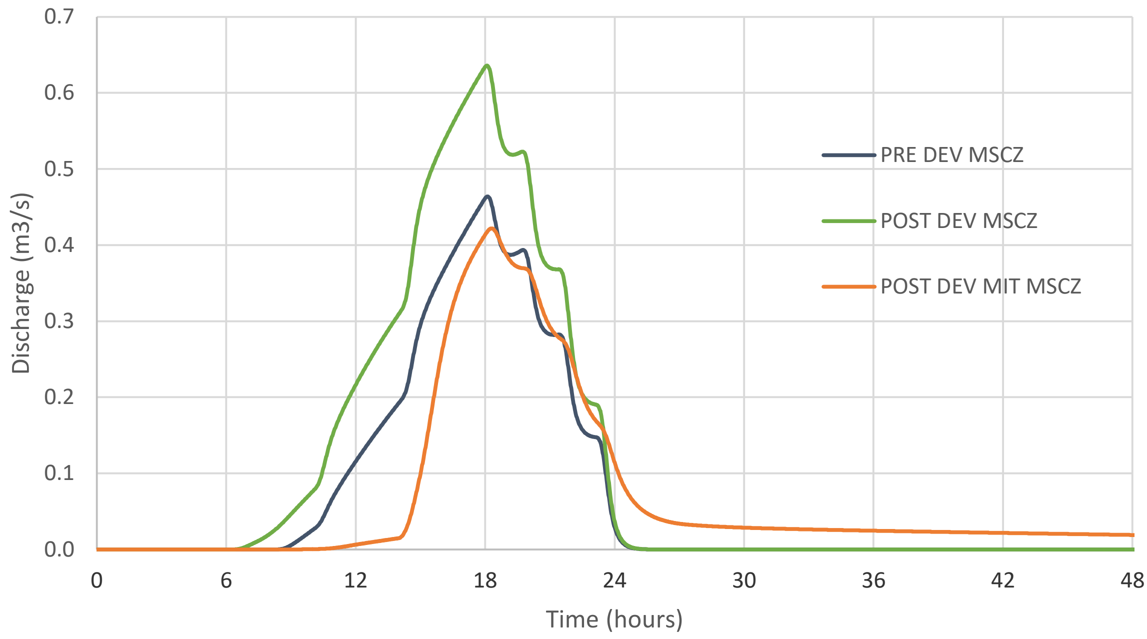
Appendix B: Hydrological Parameters

Catchment	Area (ha)	Stream Length (km)	Slope (m/m)
MHSCZ C1	76	1.56	0.019
MHSCZ C2	73	1.41	0.025
MHSCZ C3	45 – Pre 45.4 - Post	1.49	0.018
MHSCZ C4	18	860	0.12
MHSCZ C5	22	640	0.12
MHSCZ C6	11	200	0.17
MHSCZ C7	1.7 – Pre 1.3 - Post	100	0.2
Upper Turirau I	878	4.55	0.006
Turirau II	273	3.12	0.001
Lower Turirau III	91	2.01	0.017
Lower Turirau IV	284	3.43	0.005

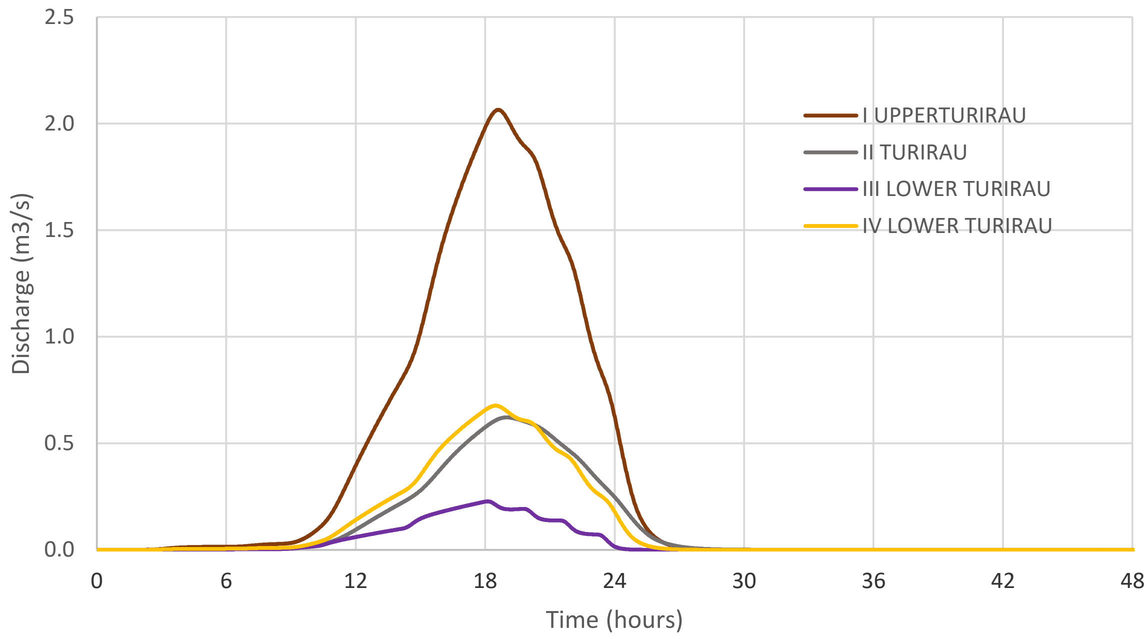
	Pre Development				Post Development			
Catchment	**Tc (min)**	**% Pervious**	**Ia (mm)**	**CN**	**Tc (min)**	**% Pervious**	**Ia (mm)**	**CN**
MHSCZ C1	47	99	5	78	46	94	4.7	81
MHSCZ C2	40	99	5	79	38	78	3.9	84
MHSCZ C3	46	99	5	79	39	52	2.6	92
MHSCZ C4	10	99	5	76	10	96	5	76
MHSCZ C5	10	99	5	78	10	90	5	78
MHSCZ C6	10	99	5	76	10	96	5	76
MHSCZ C7	10	99	5	79	10	99	5	79
Upper Turirau I	107	99	5	79	-	-	-	-
Turirau II	142	99	5	79	-	-	-	-
Lower Turirau III	46	99	5	79	-	-	-	-
Lower Turirau IV	94	99	5	79	-	-	-	-

Appendix C: Hydrology comparison of pre-development and post-development flows

A)

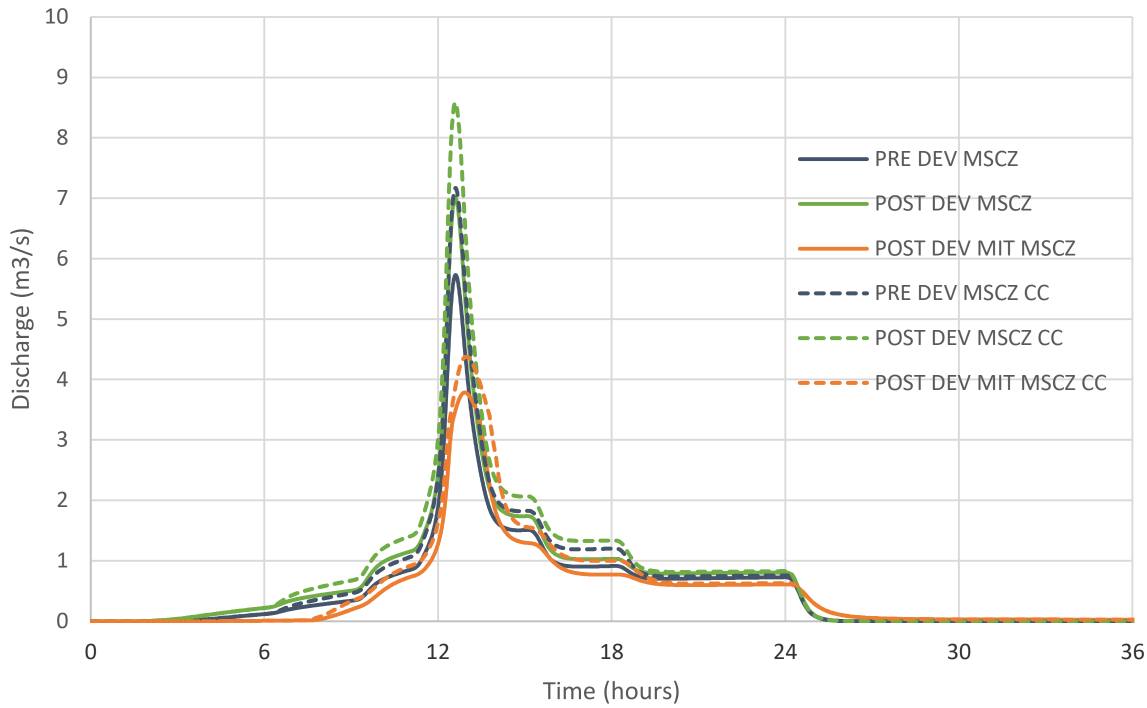


B)

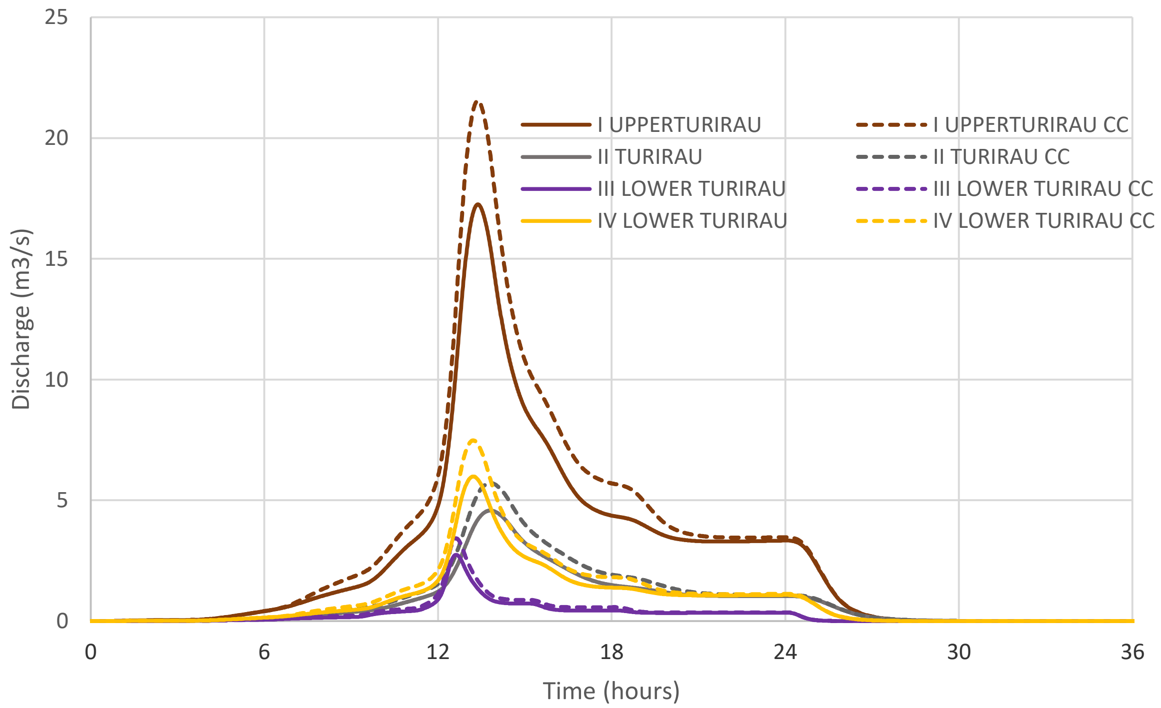


- **Figure 12-1 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; Water Quality Storm**

A)

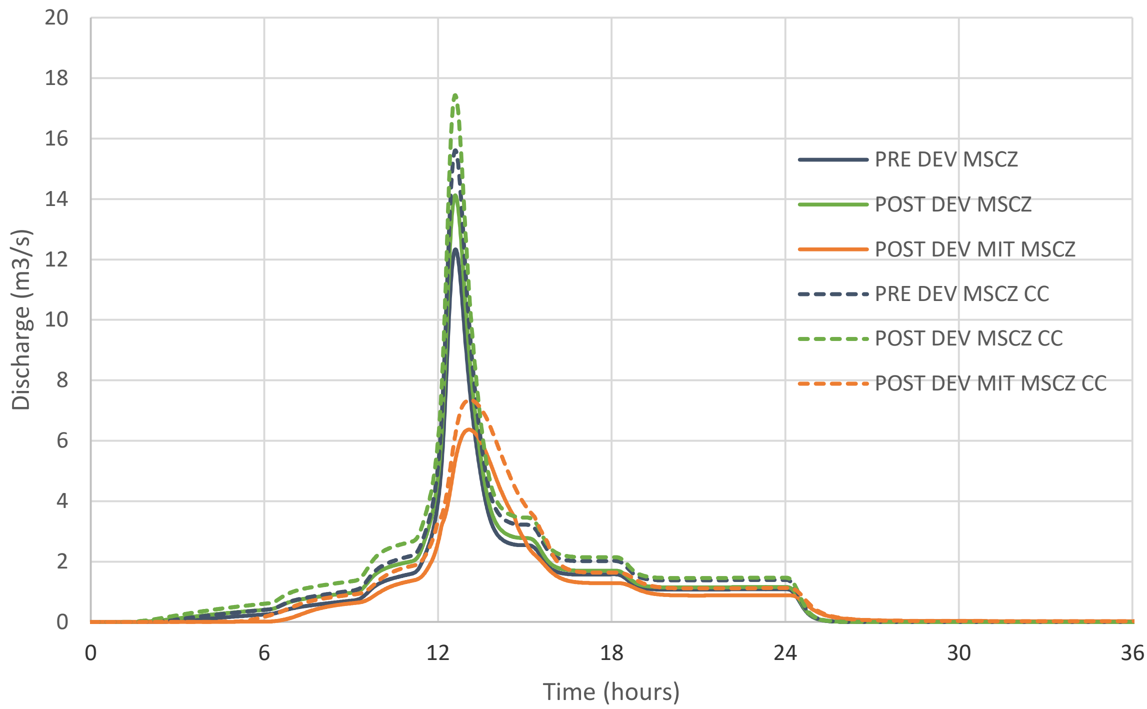


B)

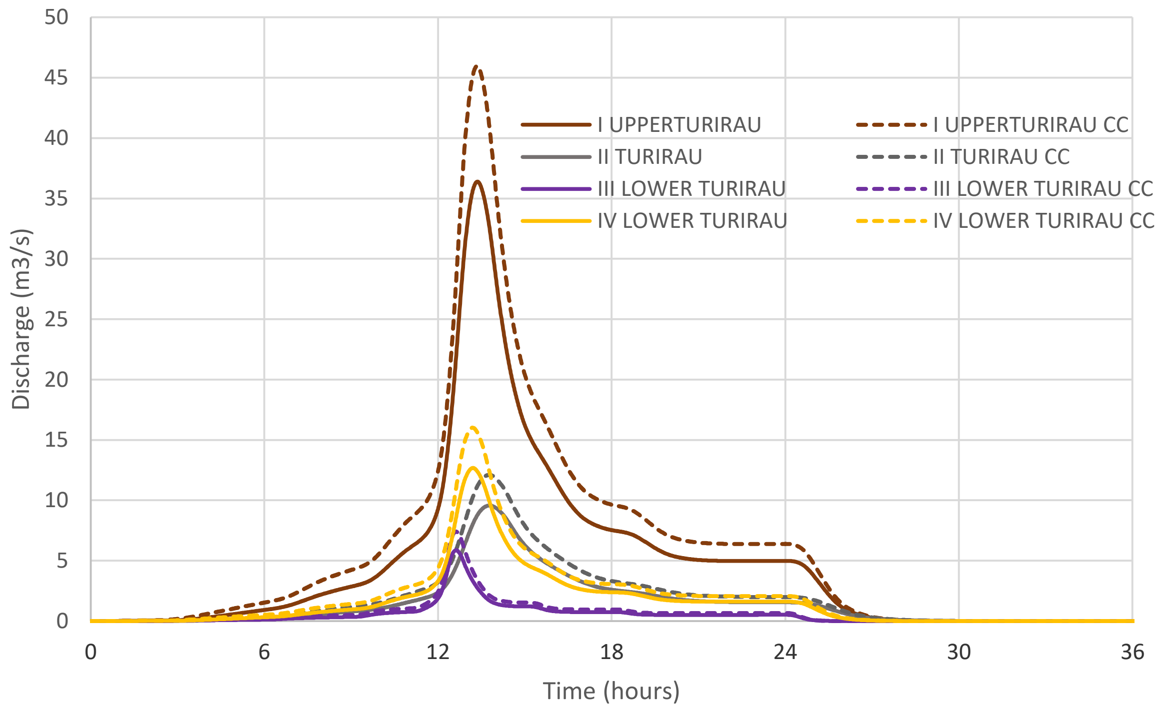


- **Figure 12-2 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 2 year ARI, 24 hour storm duration (with and without climate change)**

A)

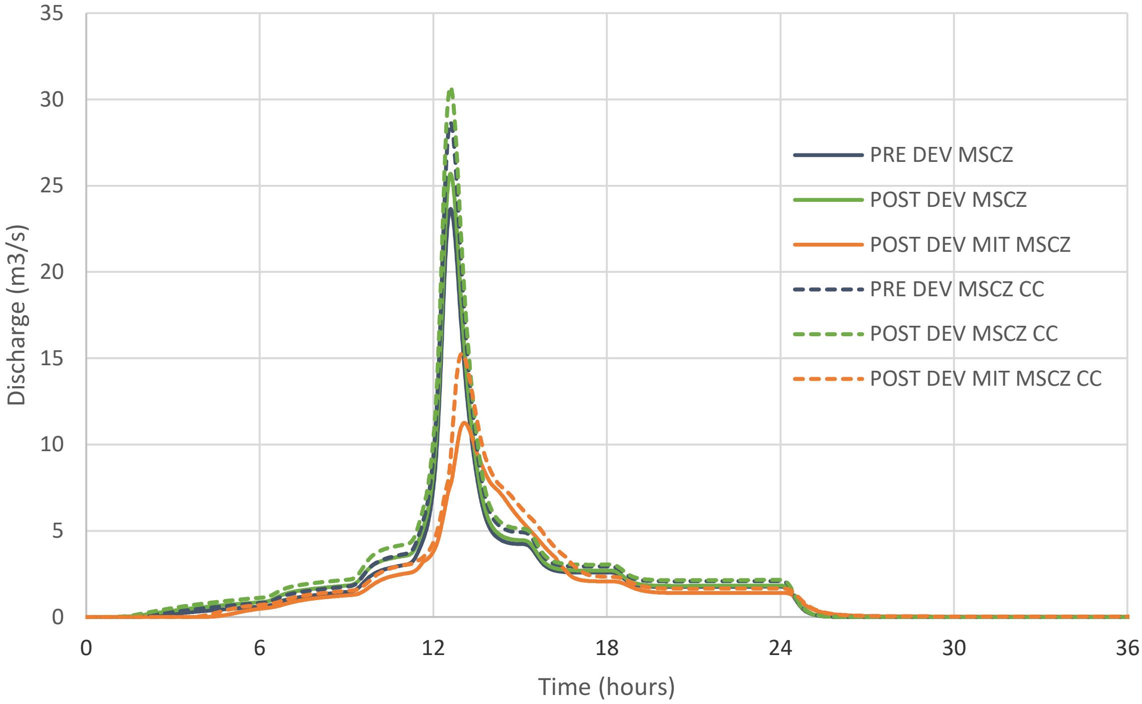


B)

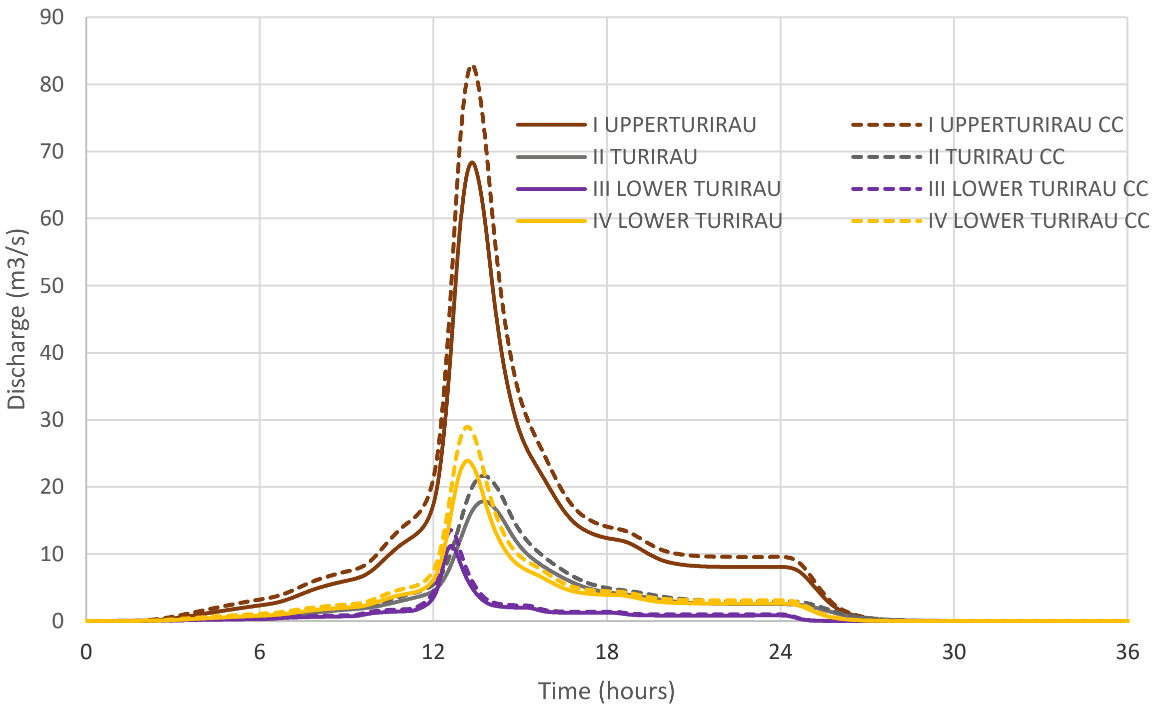


- **Figure 12-3 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 10 year ARI, 24 hour storm duration (with and without climate change)**

A)

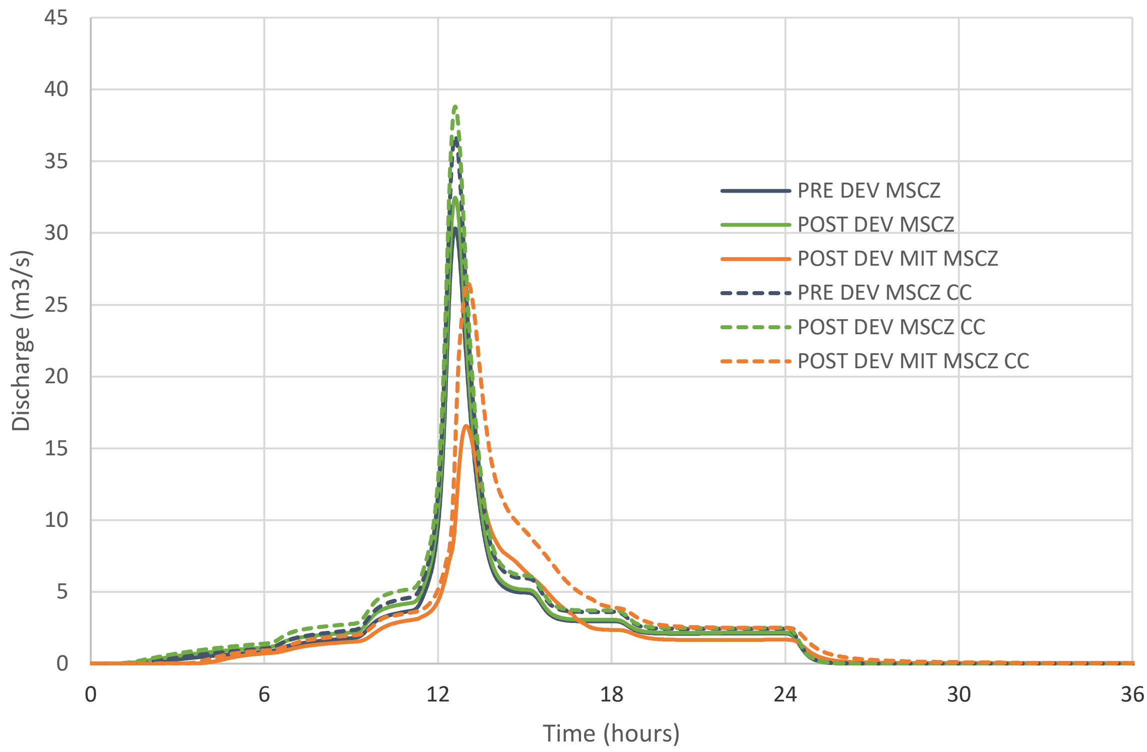


B)

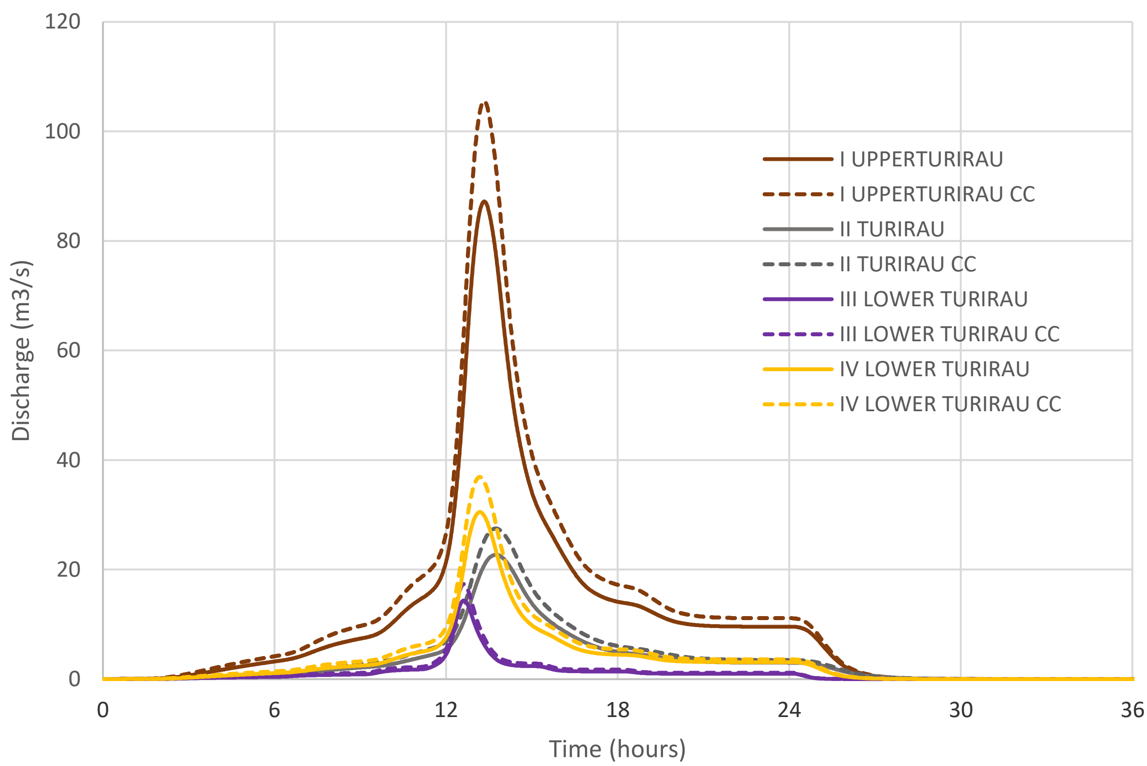


- **Figure 12-4 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 50 year ARI, 24 hour storm duration**

A)

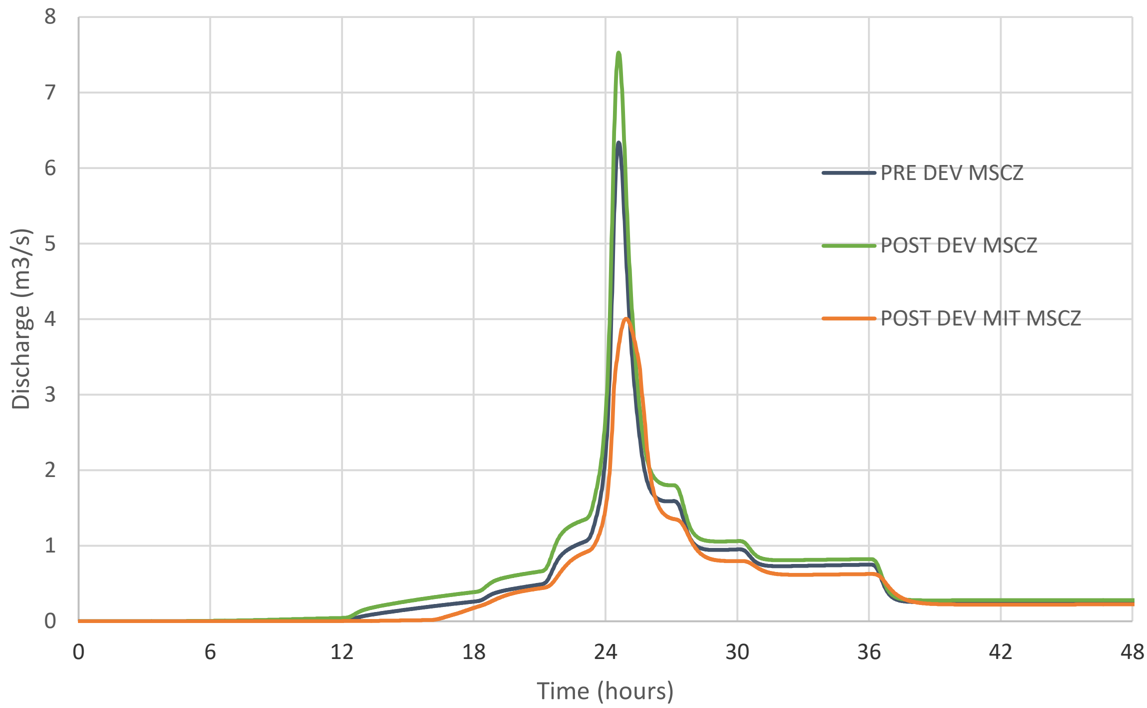


B)

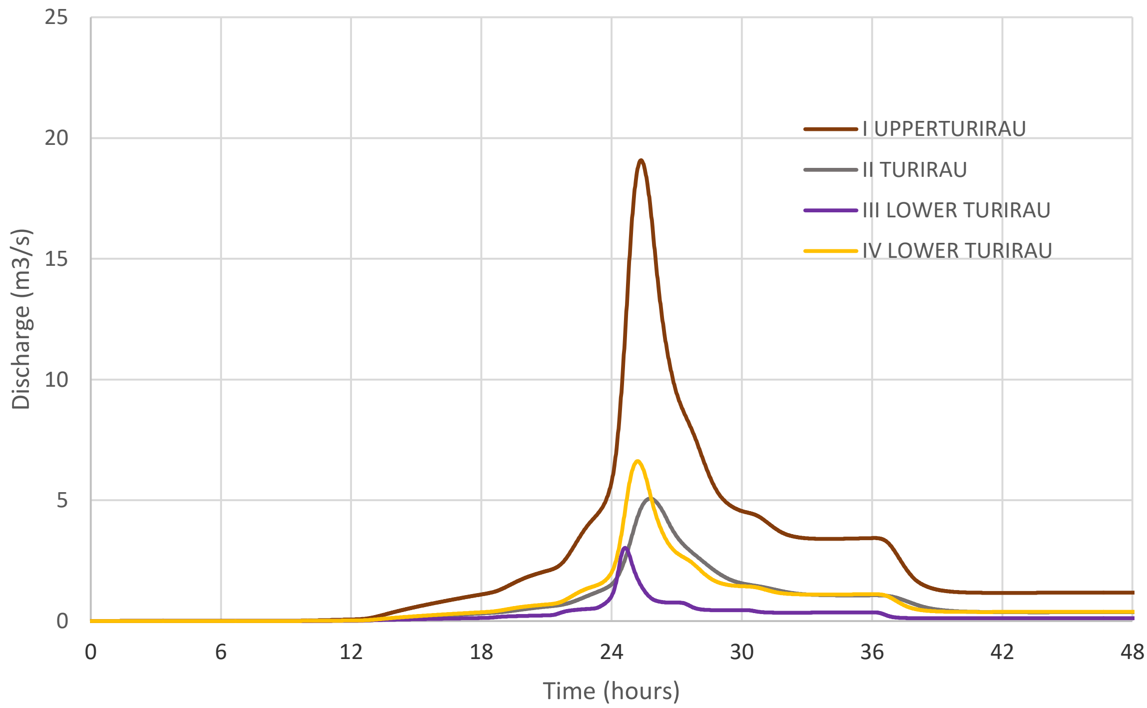


- **Figure 12-5 A& B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 100 year ARI, 24 hour storm duration (with and without climate change)**

A)

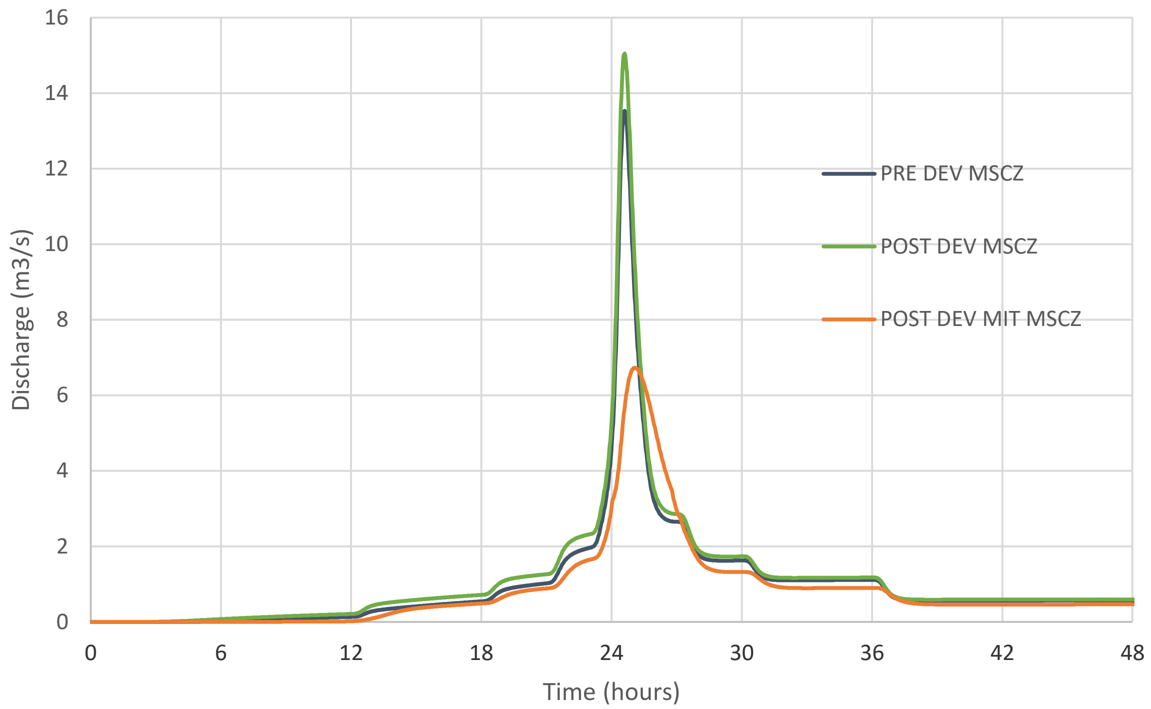


B)

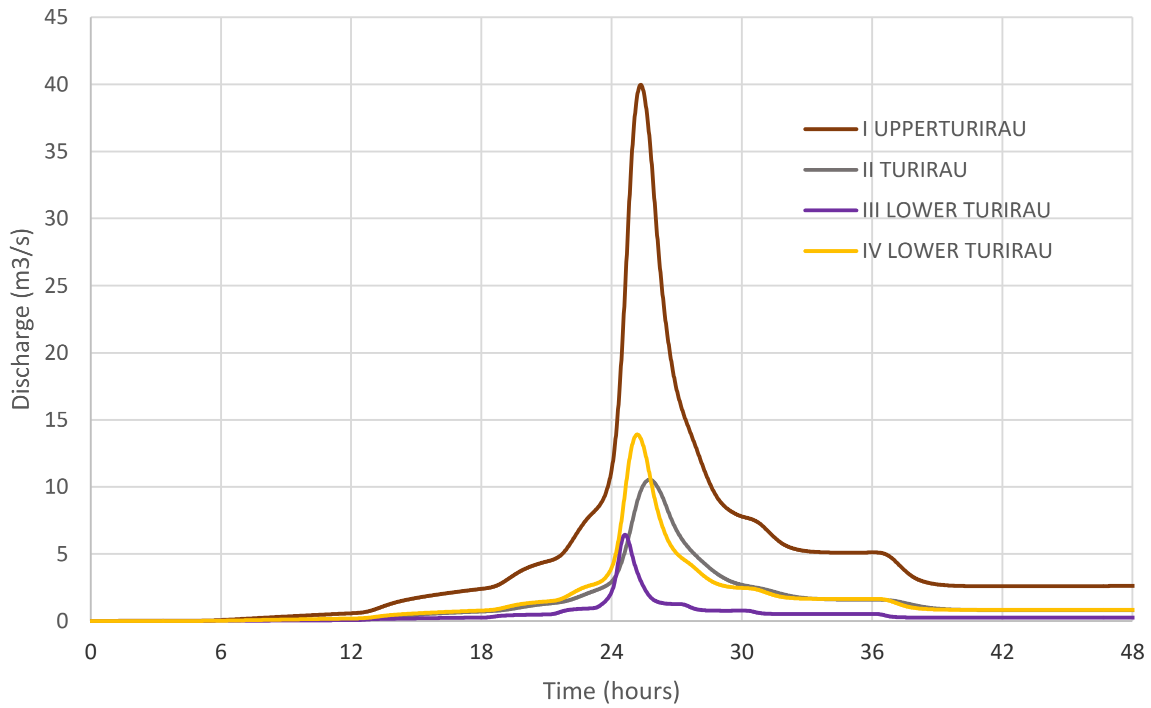


- **Figure 12-6 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 2 year ARI, 48 hour storm duration (without climate change)**

A)

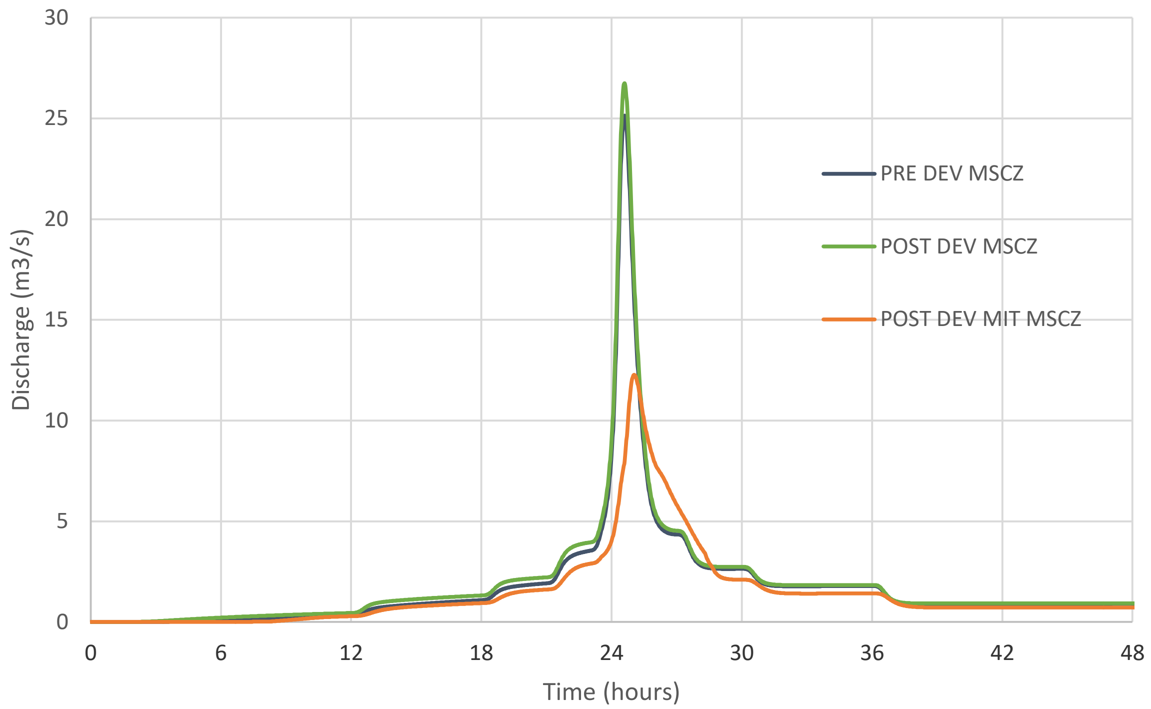


B)

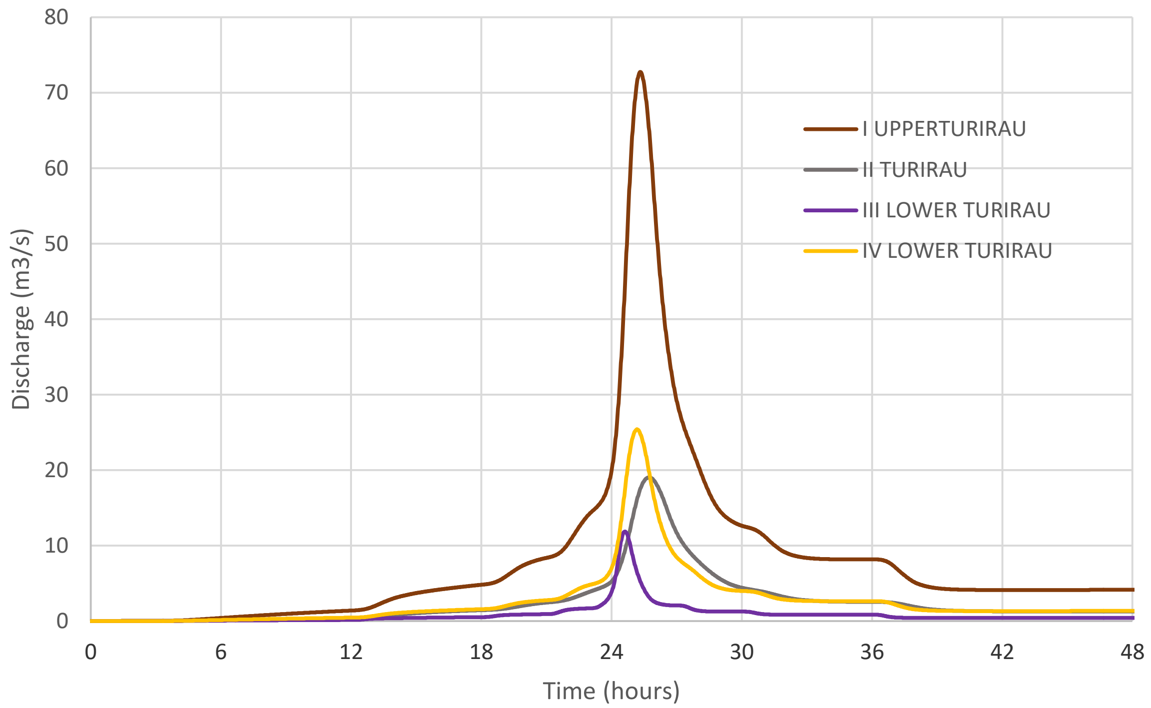


- **Figure 12-7 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 10 year ARI, 48 hour storm duration (without climate change)**

A)

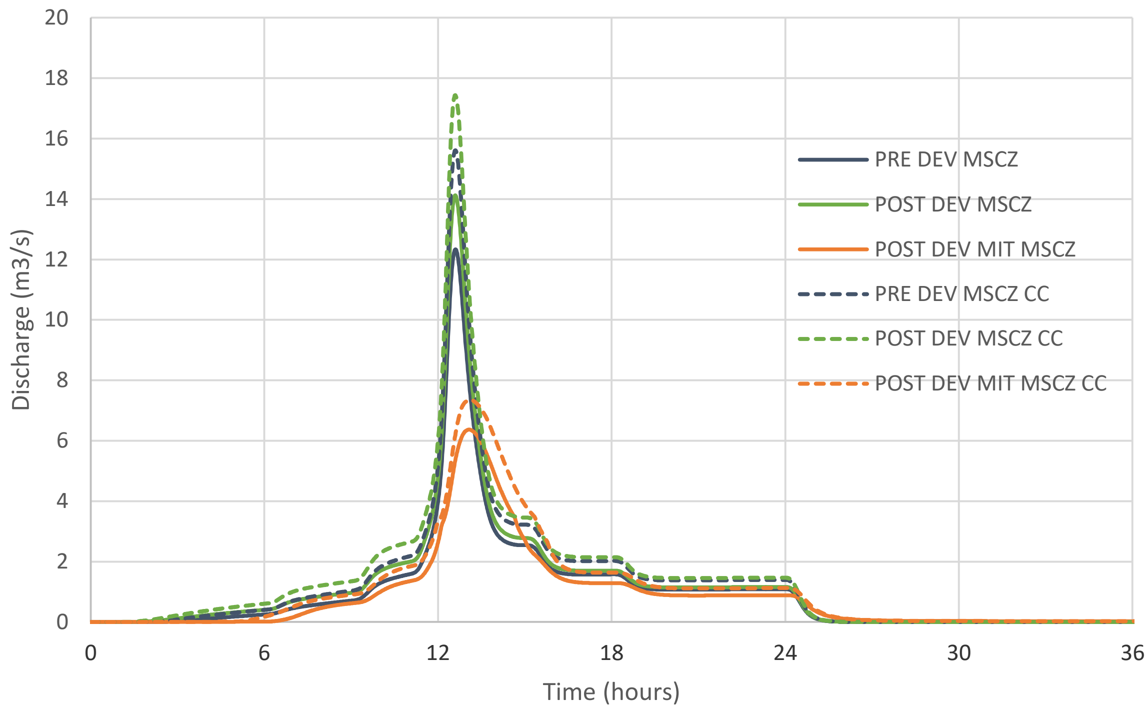


B)

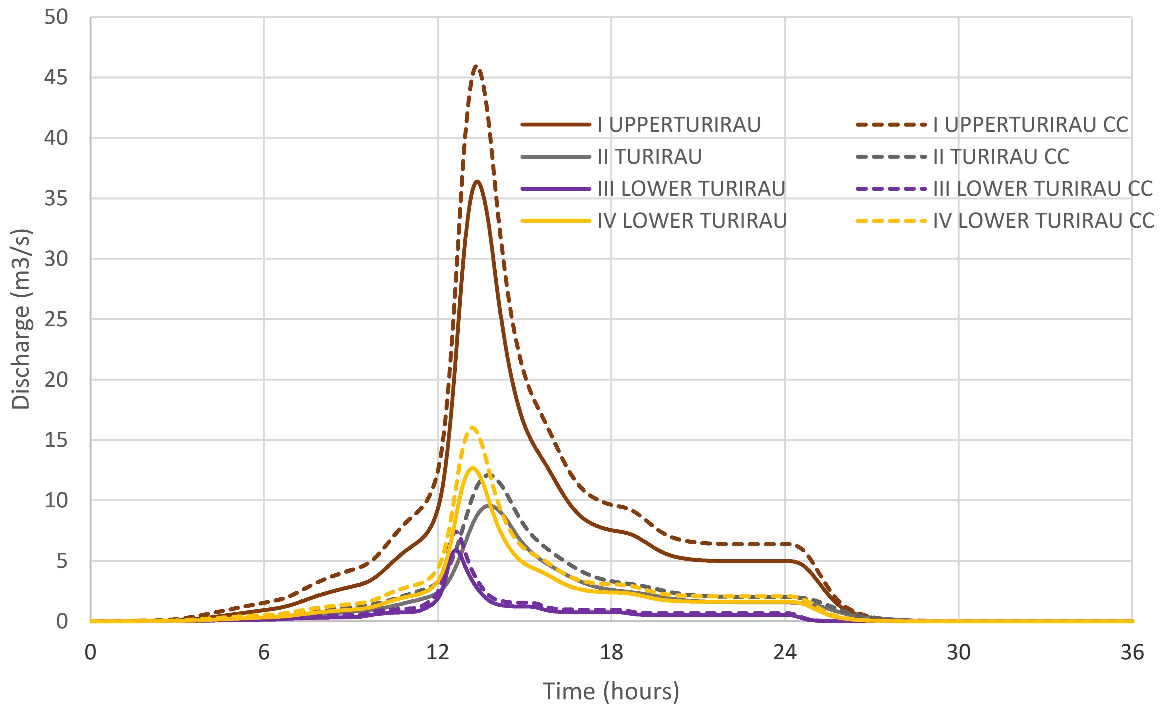


- **Figure 12-8 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 50 year ARI, 48 hour storm duration (without climate change)**

A)

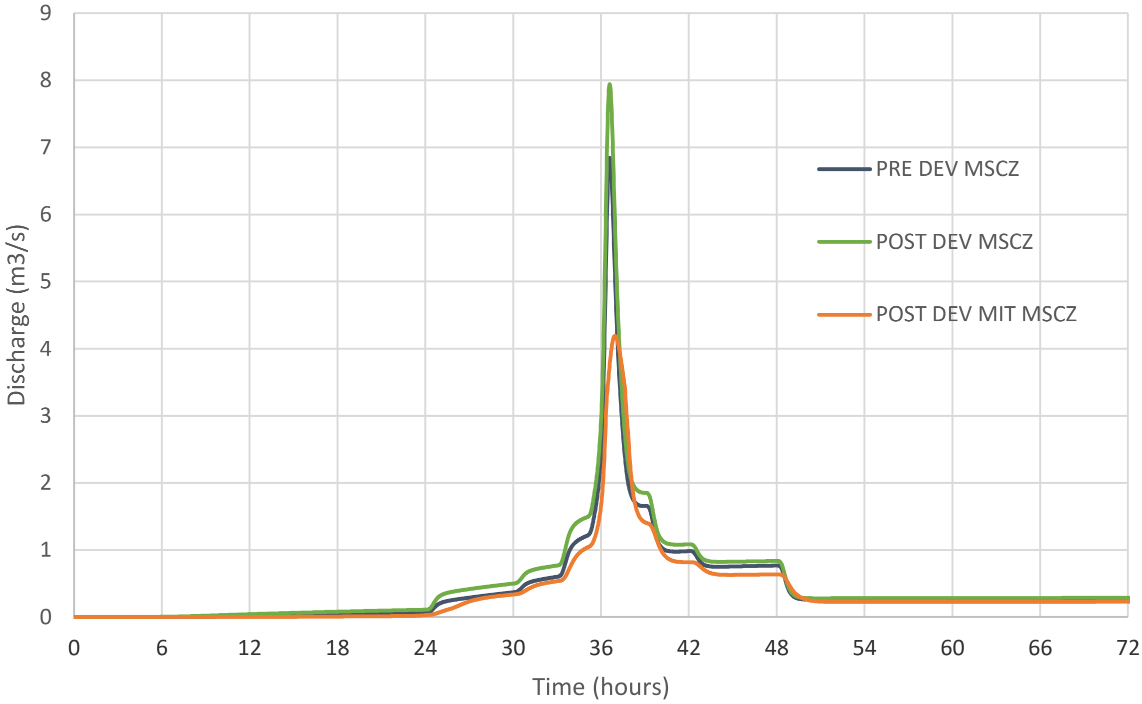


B)

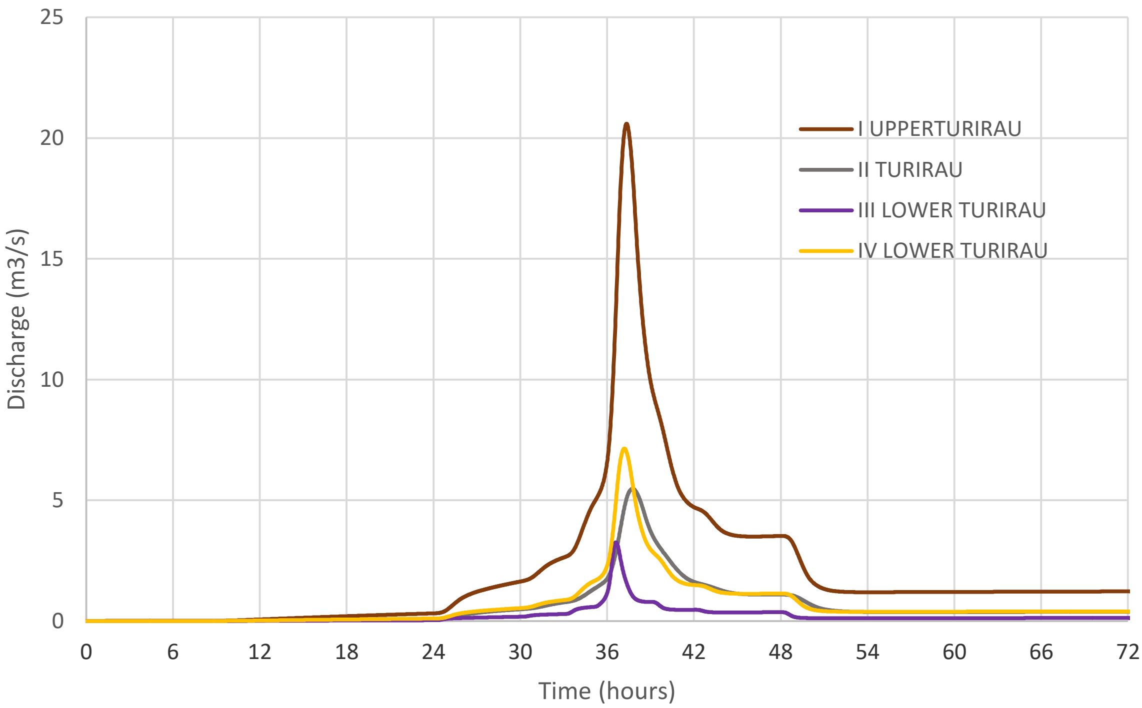


- **Figure 12-9 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 100 year ARI, 48 hour storm duration (with and without climate change)**

A)

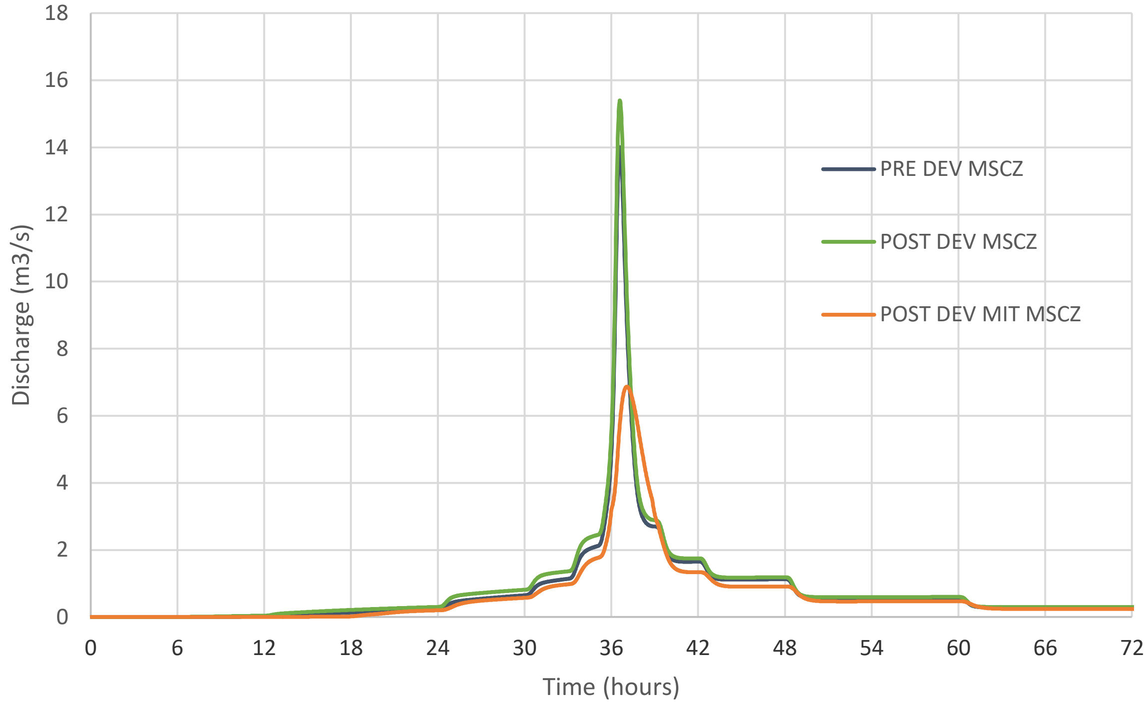


B)

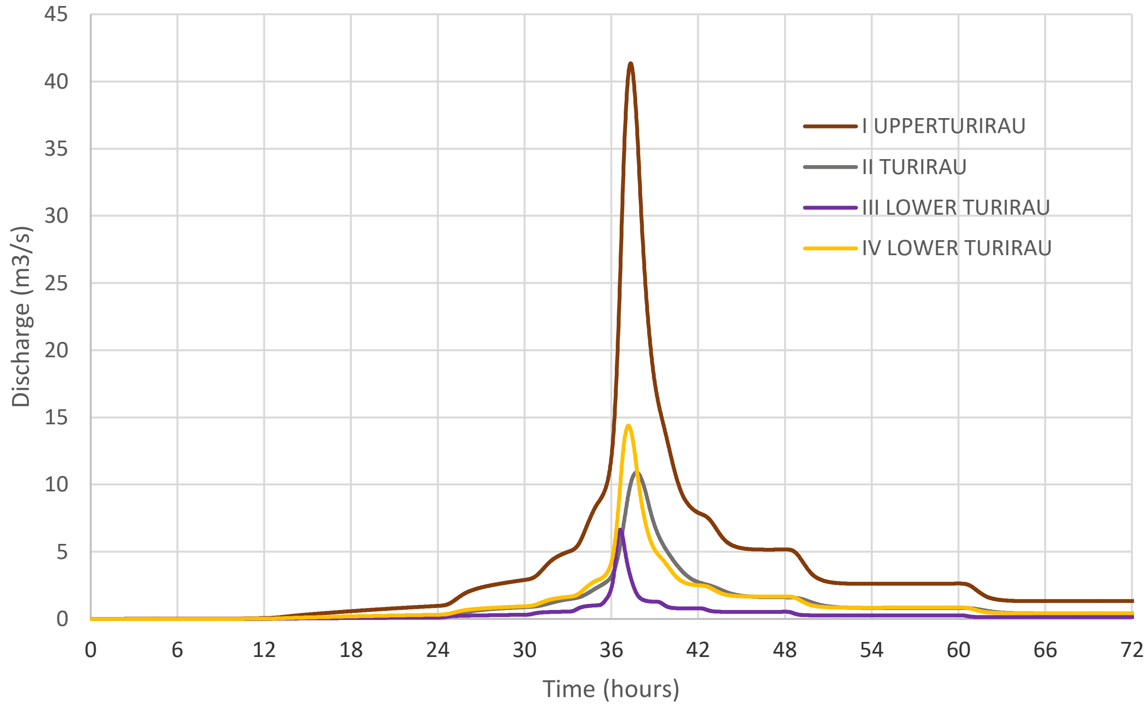


- **Figure 12-10 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 2 year ARI, 72 hour storm duration (without climate change)**

A)

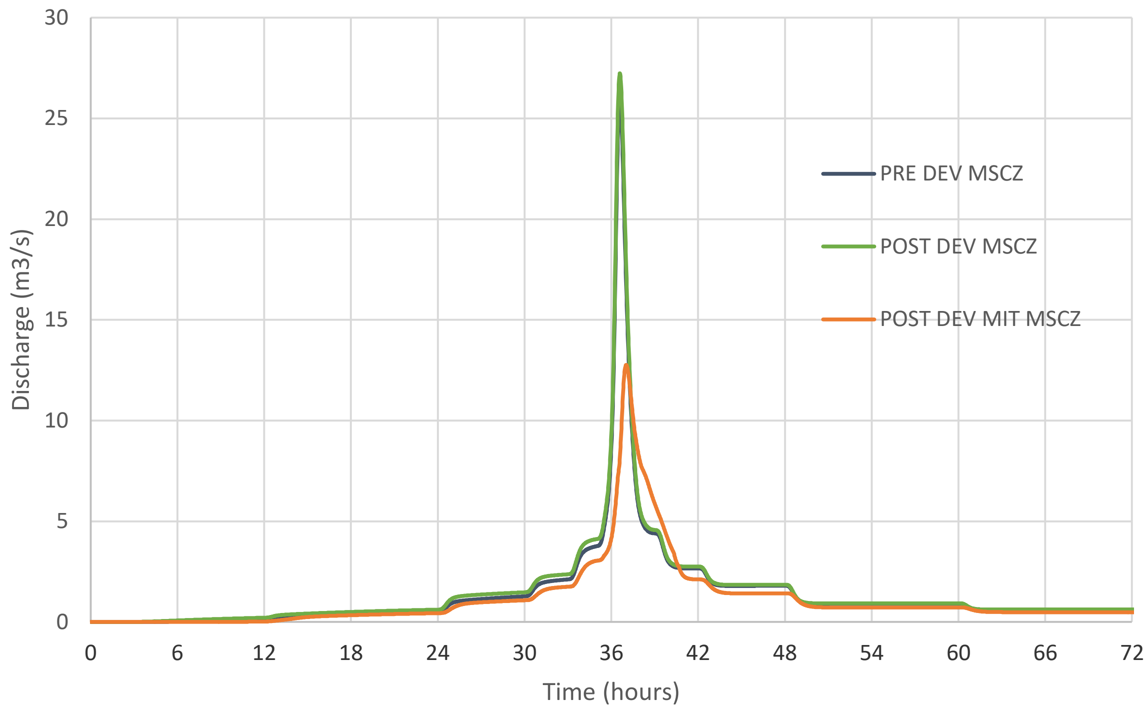


B)

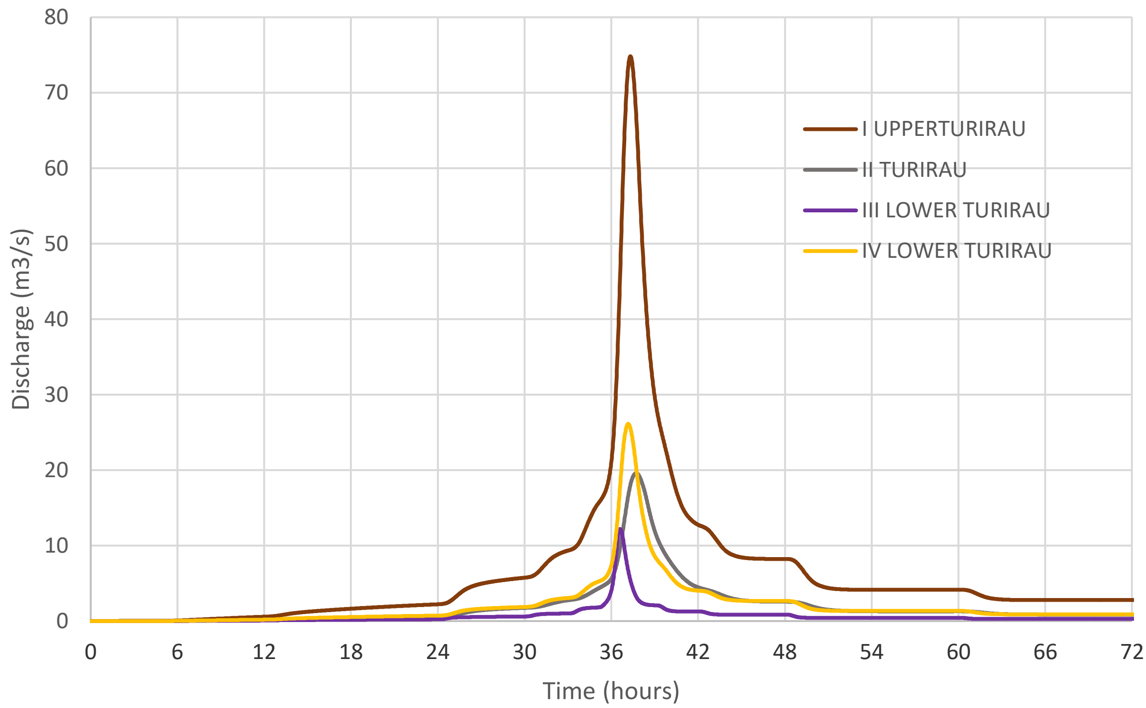


- **Figure 12-11: Comparison of pre-development flows and post-development flows for MSCZ catchment; 10 year ARI, 72 hour storm duration (without climate change)**

A)

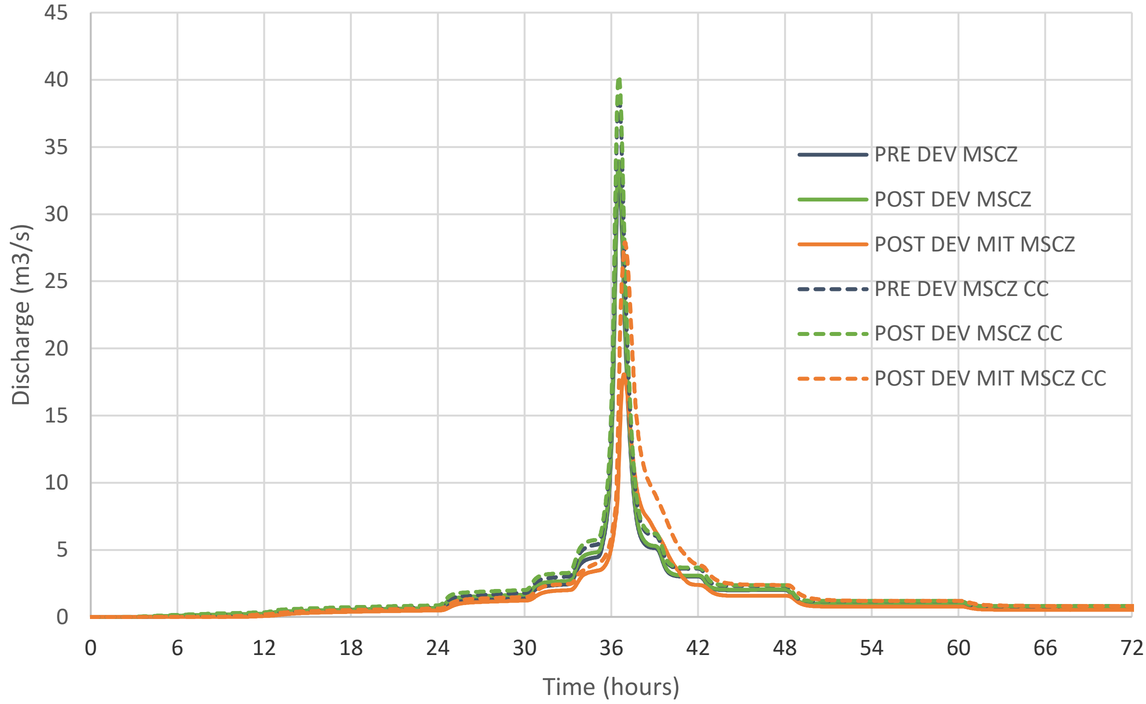


B)

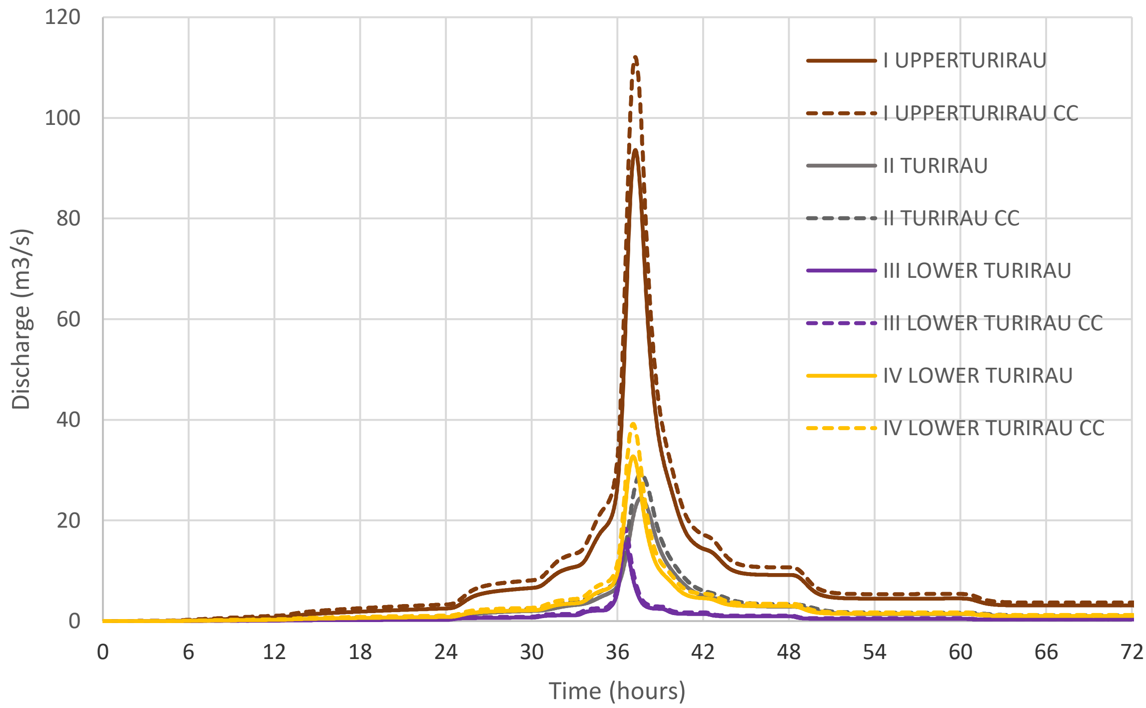


- **Figure 12-12: Comparison of pre-development flows and post-development flows for MSCZ catchment; 50 year ARI, 72 hour storm duration(without climate change)**

A)



B)



- **Figure 12-13 A & B: Comparison of pre-development flows and post-development flows for MSCZ catchment; 100 year ARI, 72 hour storm duration (with and without climate change)**

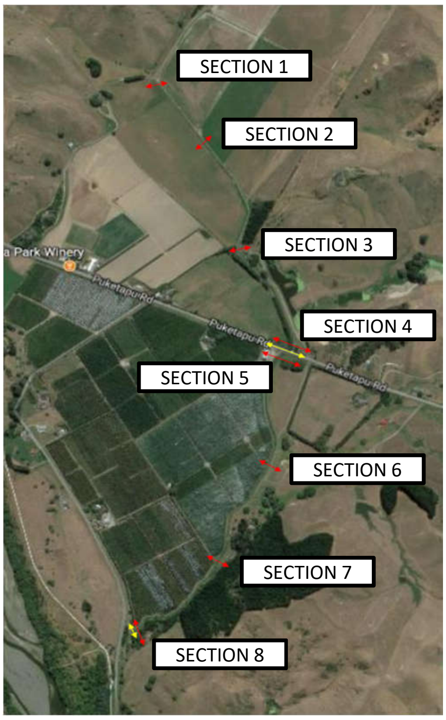
Appendix D: Cross Section Survey

Tuarau Stream Cross Sections

Upstream → Downstream

Left → Right

SURVEYING THE BAY
HAWKE'S BAY | NEW ZEALAND



Notes

Coordinates are in Hawkes Bay 2000
Heights are related to NZ Vertical Datum 2016
Survey undertaken by P.Durham
Survey Date: 4/09/2017

Section 1	Top of Bank	411390.37	815416.10	19.691
	Bottom of Bank	411389.36	815415.50	18.439
	Centre of Ditch	411388.22	815414.09	17.960
	Bottom of Bank	411386.99	815413.30	18.377
	Top of Bank	411386.39	815412.72	19.331

Section 2	Top of Bank	411541.99	815195.49	19.628
	Bottom of Bank	411541.38	815194.98	18.347
	Centre of Ditch	411539.99	815193.96	17.828
	Bottom of Bank	411538.94	815192.42	18.266
	Top of Bank	411538.59	815191.65	19.095

Section 3	Top of Bank	411656.42	814817.80	19.455
	Bottom of Bank	411654.60	814818.39	18.419
	Centre of Ditch	411652.11	814818.37	17.313
	Bottom of Bank	411649.85	814819.71	18.359
	Top of Bank	411649.30	814820.27	19.054

Section 4	Top of Bank	411836.74	814433.77	20.144
	Bottom of Bank	411834.79	814434.80	18.162
	Centre of Ditch	411833.61	814435.48	17.807
	Bottom of Bank	411829.93	814434.22	18.334
	Top of Bank	411826.82	814433.79	20.062

	Invert Level 1.80ø	411827.21	814428.65	17.369
	Crown Level	411827.18	814428.54	19.567
	Ground Level	411826.83	814428.20	19.824
	Ground Level	411826.01	814426.83	20.063
	Ground Level	411825.13	814425.81	20.323
	Ground Level	411823.95	814424.09	20.933
	Ground Level	411822.89	814423.03	21.191
	Edge of seal	411822.02	814421.64	21.387
	Centre of Road	411820.02	814418.35	21.54
	Edge of seal	411818.29	814414.98	21.352
	Ground Level	411816.79	814413.19	21.096
	Ground Level	411814.77	814411.00	20.599
	Ground Level	411813.10	814408.94	19.424
	Crown Level	411812.82	814408.67	19.248
	Invert Level 1.80ø	411812.72	814408.67	17.402

Levels taken from above the upstream opening across the road to the downstream opening.


Upstream - Puketapu Rd


Downstream - Puketapu Rd

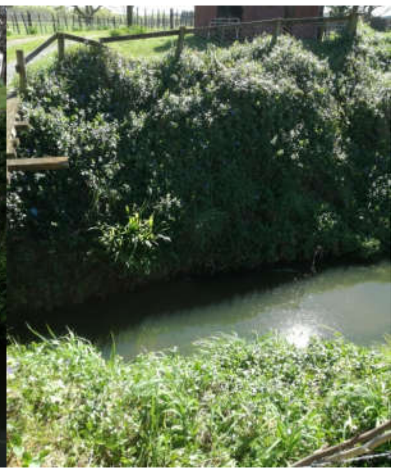
Section 5	Top of Bank	411811.84	814402.87	19.066
	Bottom of Bank	411811.06	814403.22	17.925
	Centre of Ditch	411809.98	814404.48	17.565
	Bottom of Bank	411807.62	814406.12	18.144
	Top of Bank	411806.98	814406.95	19.317

Section 6	Top of Bank	411767.08	814024.69	19.599
	Bottom of Bank	411765.95	814024.99	18.031
	Centre of Ditch	411764.26	814026.50	17.528
	Bottom of Bank	411762.29	814026.54	17.98
	Top of Bank	411761.17	814026.99	19.434


Section 7	Top of Bank	411588.21	813635.71	20.926
	Bottom of Bank	411585.80	813637.06	18.137
	Centre of Ditch	411584.29	813637.88	17.714
	Bottom of Bank	411582.84	813638.14	17.977
	Top of Bank	411580.95	813639.32	20.703

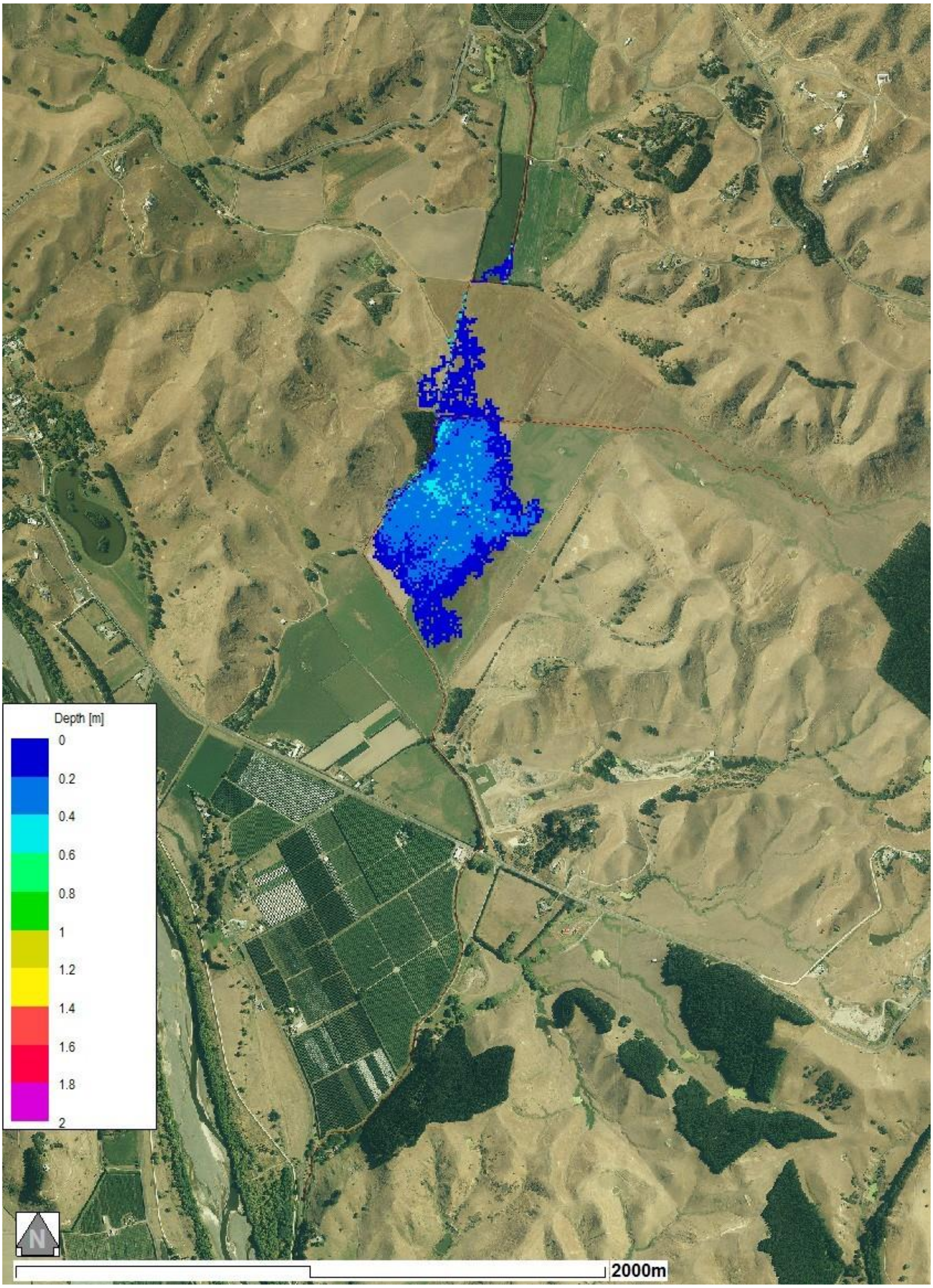

Section 8	Top of Bank	411325.65	813391.26	20.205
	Bottom of Bank	411324.04	813393.12	17.707
	Centre of Ditch	411323.26	813393.94	17.475
	Bottom of Bank	411321.80	813394.77	18.048
	Top of Bank	411320.03	813397.77	21.471



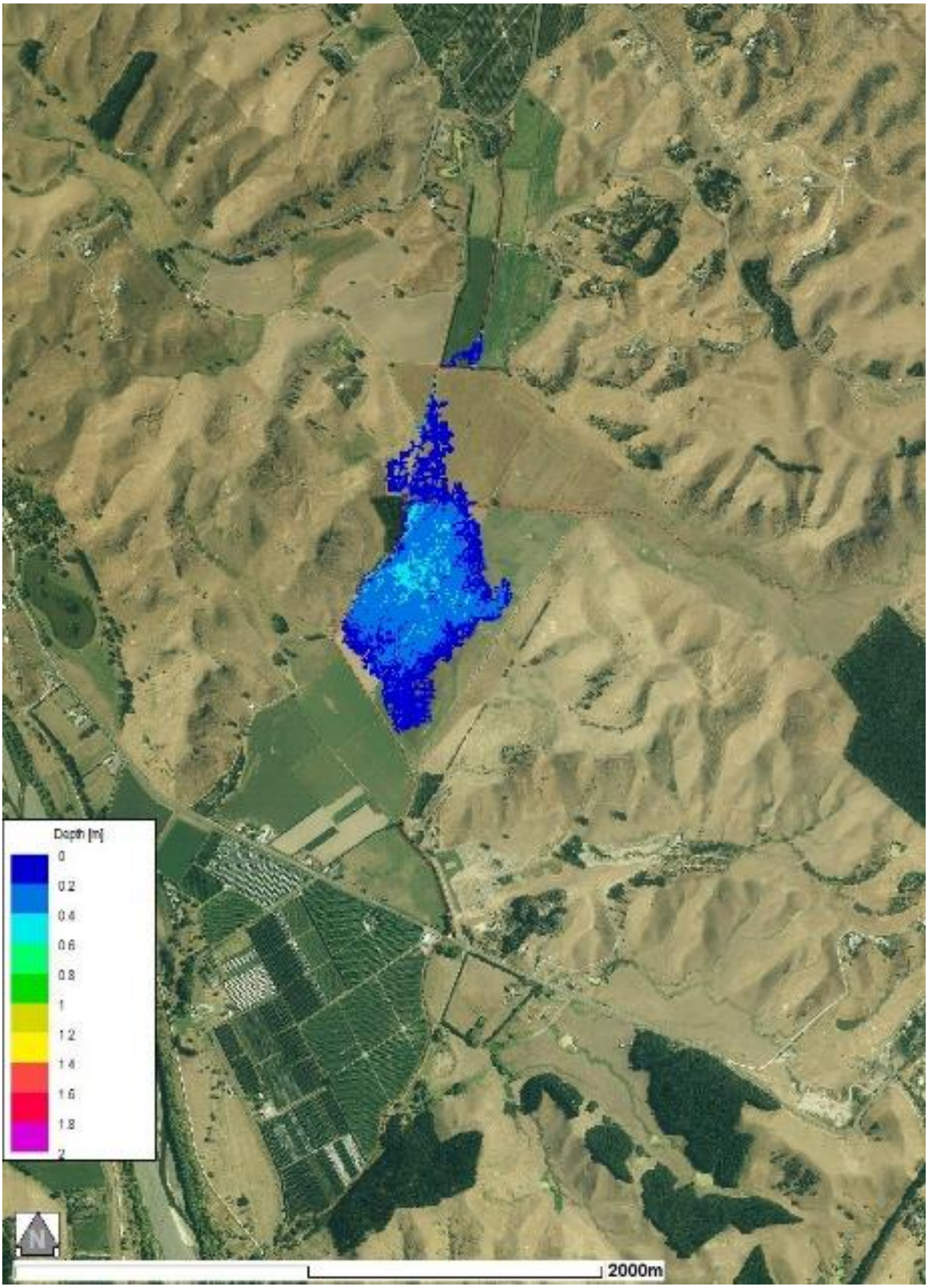


	Bridge Deck	411323.36	813388.17	21.051
	Bridge Deck	411320.05	813391.08	21.196
	Bridge Deck	411317.37	813393.62	21.375

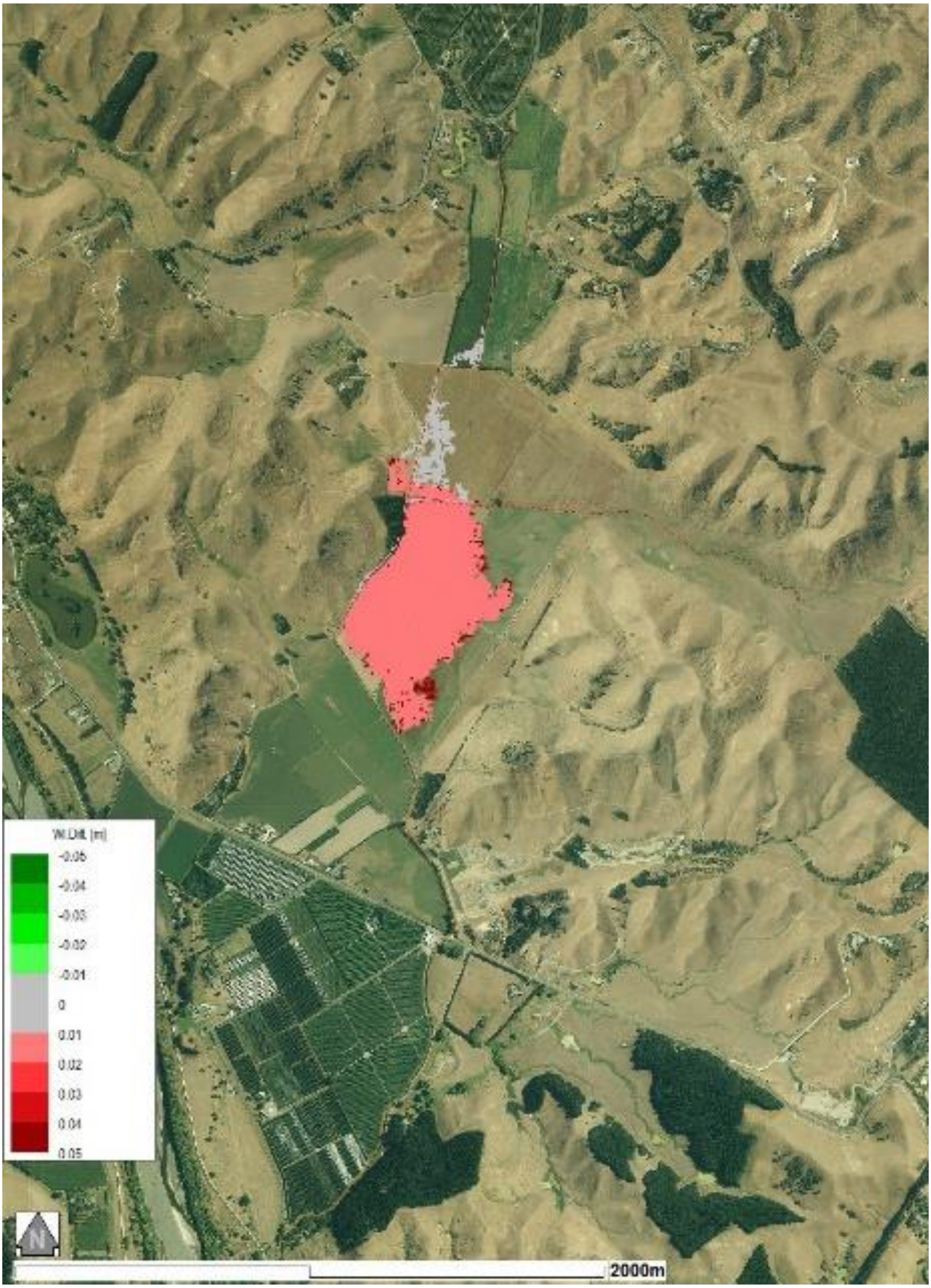
Appendix E: Hydraulic Modelling Flood Maps for Pre and Post Development (no Mitigation)



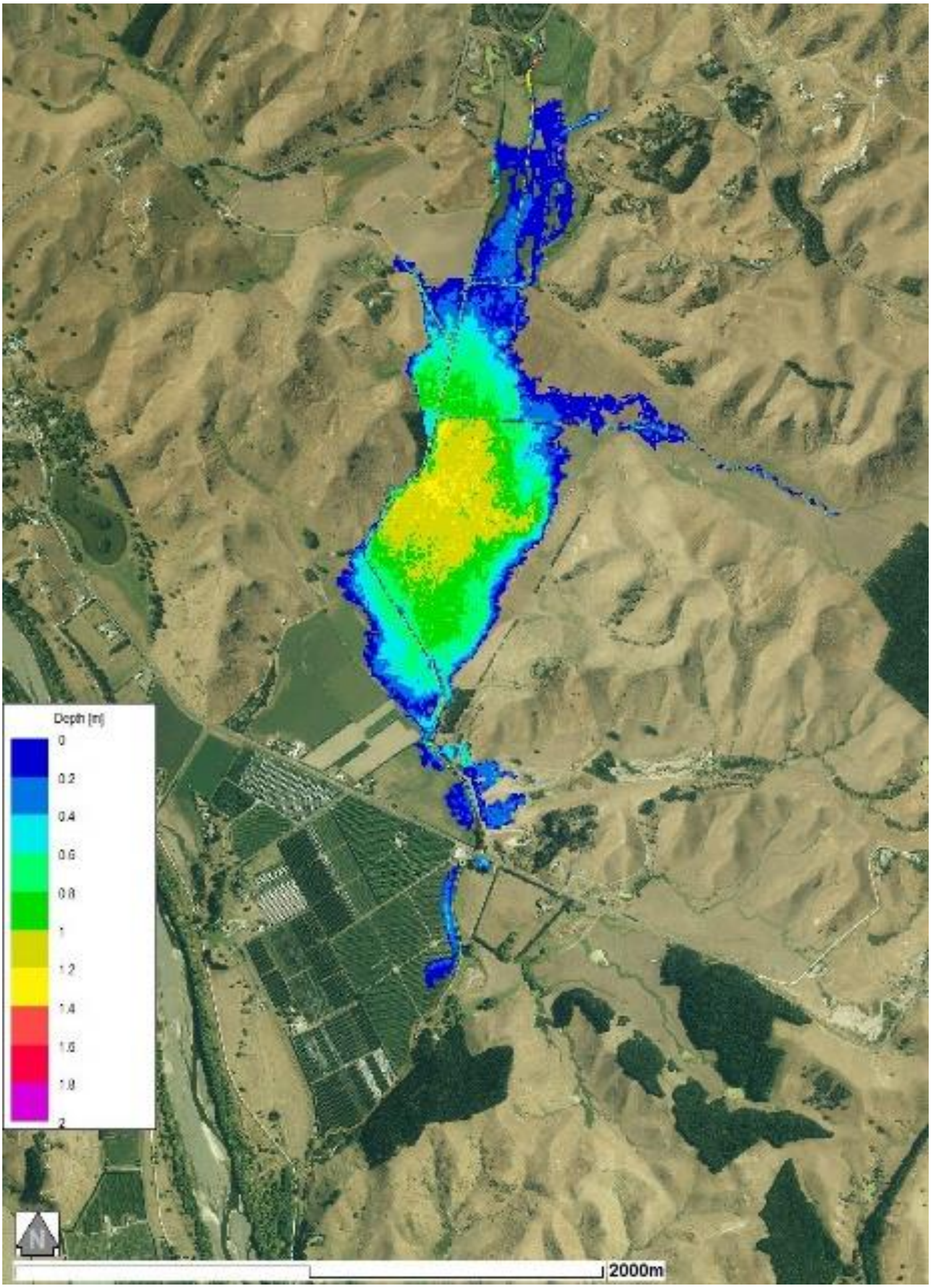
- **Figure 12-14: Flood inundation map showing peak water depth for the WQV Storm for pre development conditions**



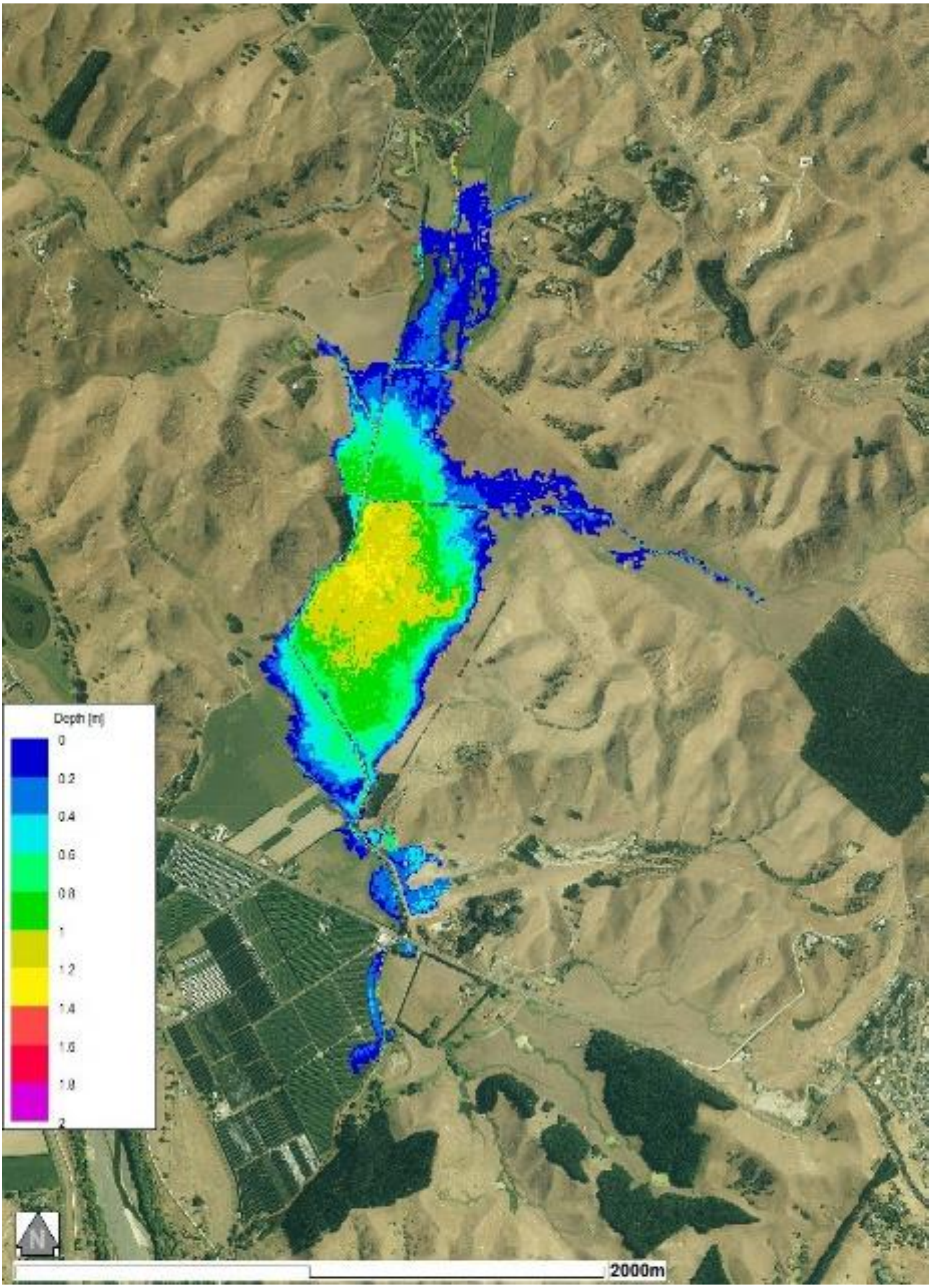
- **Figure 12-15: Flood inundation map showing peak water depth for the WQV Storm for post development unmitigated conditions**



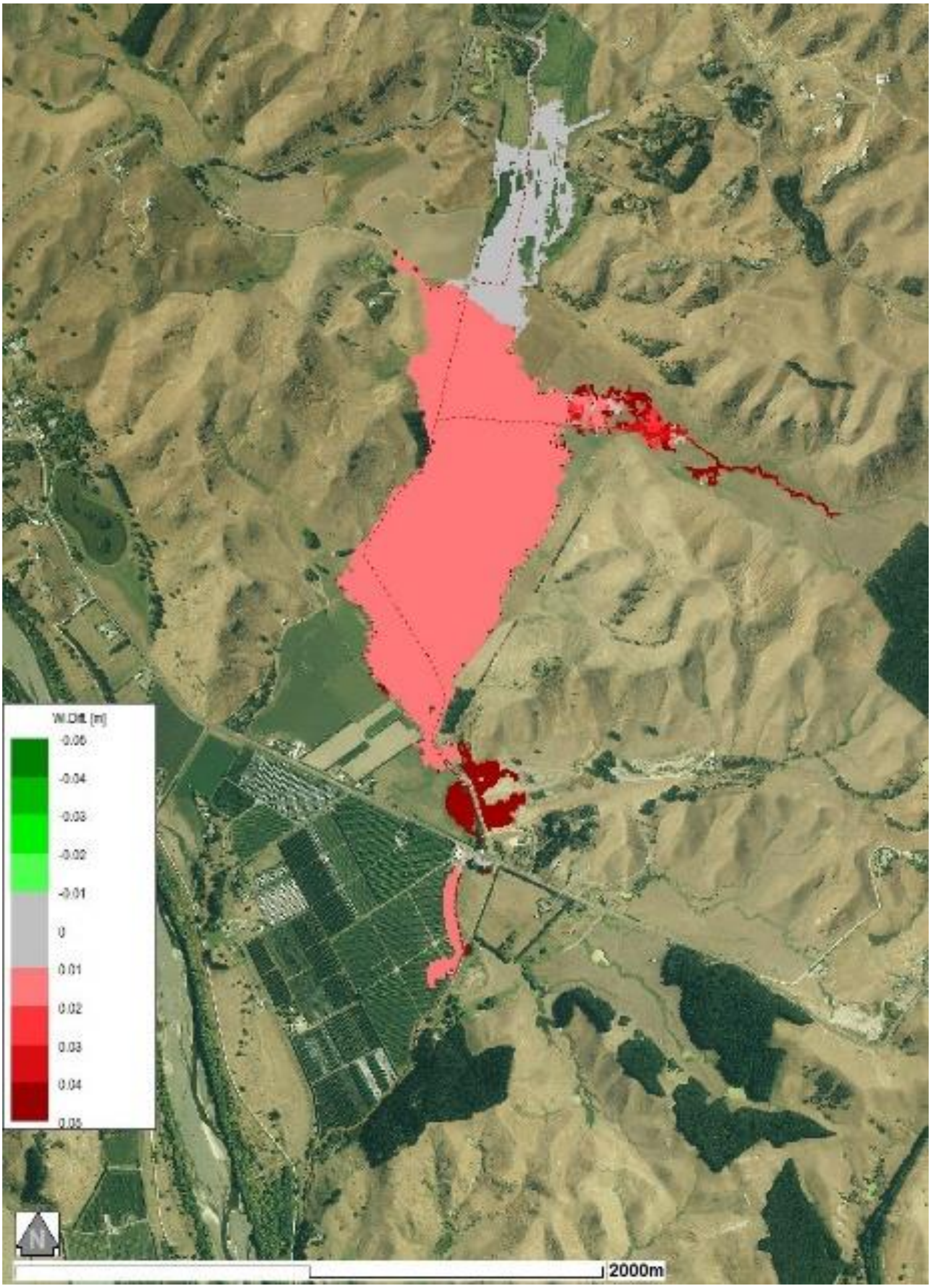
- **Figure 12-16: Flood peak flood level difference map showing differences between the post development unmitigated case and the pre development for the WQV Storm. Green indicates a decrease in water levels, grey, within 1cm and red, an increase.**



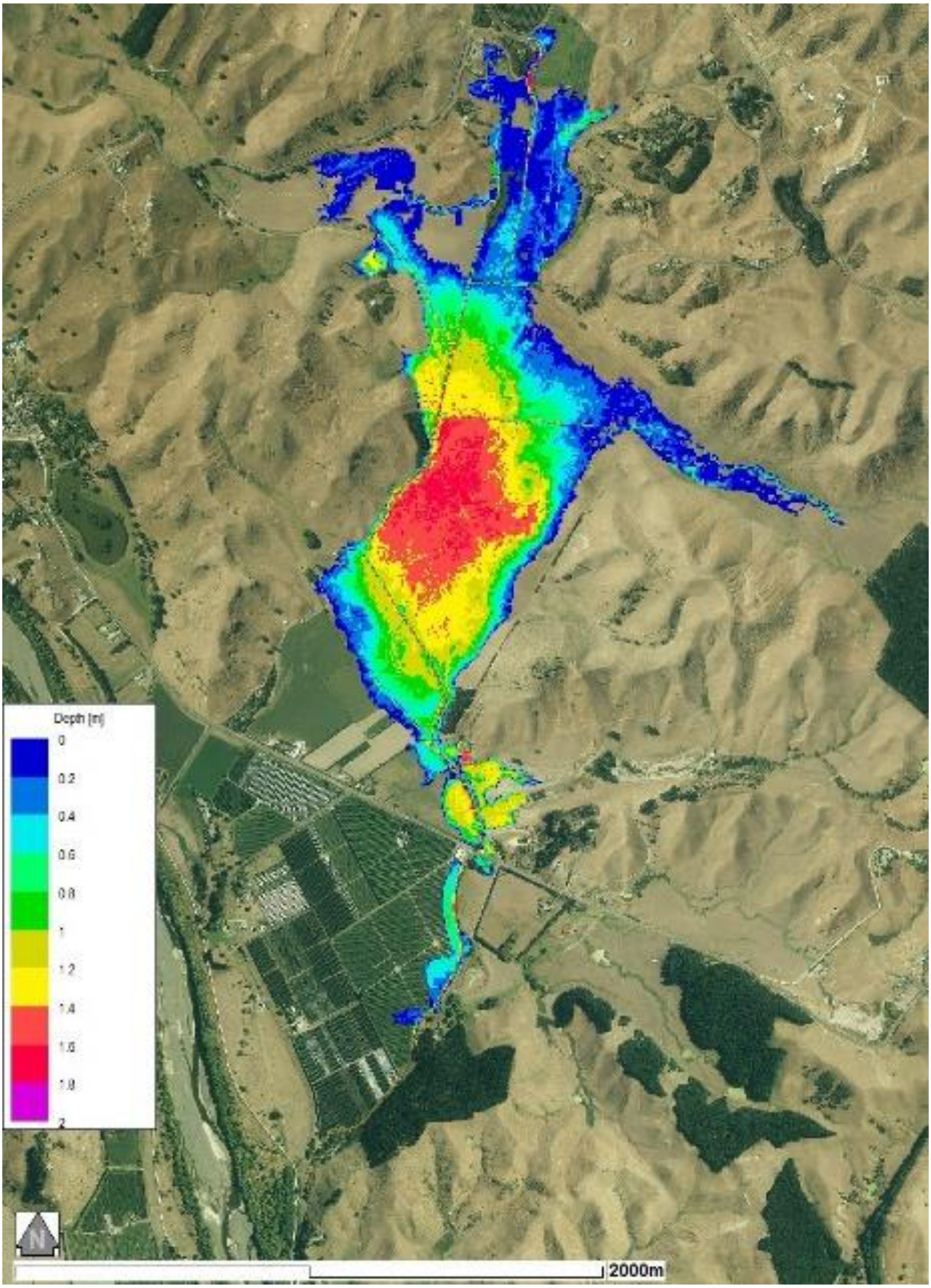
- **Figure 12-17: Flood inundation map showing peak water depth for the 2y ARI Storm for pre development conditions**



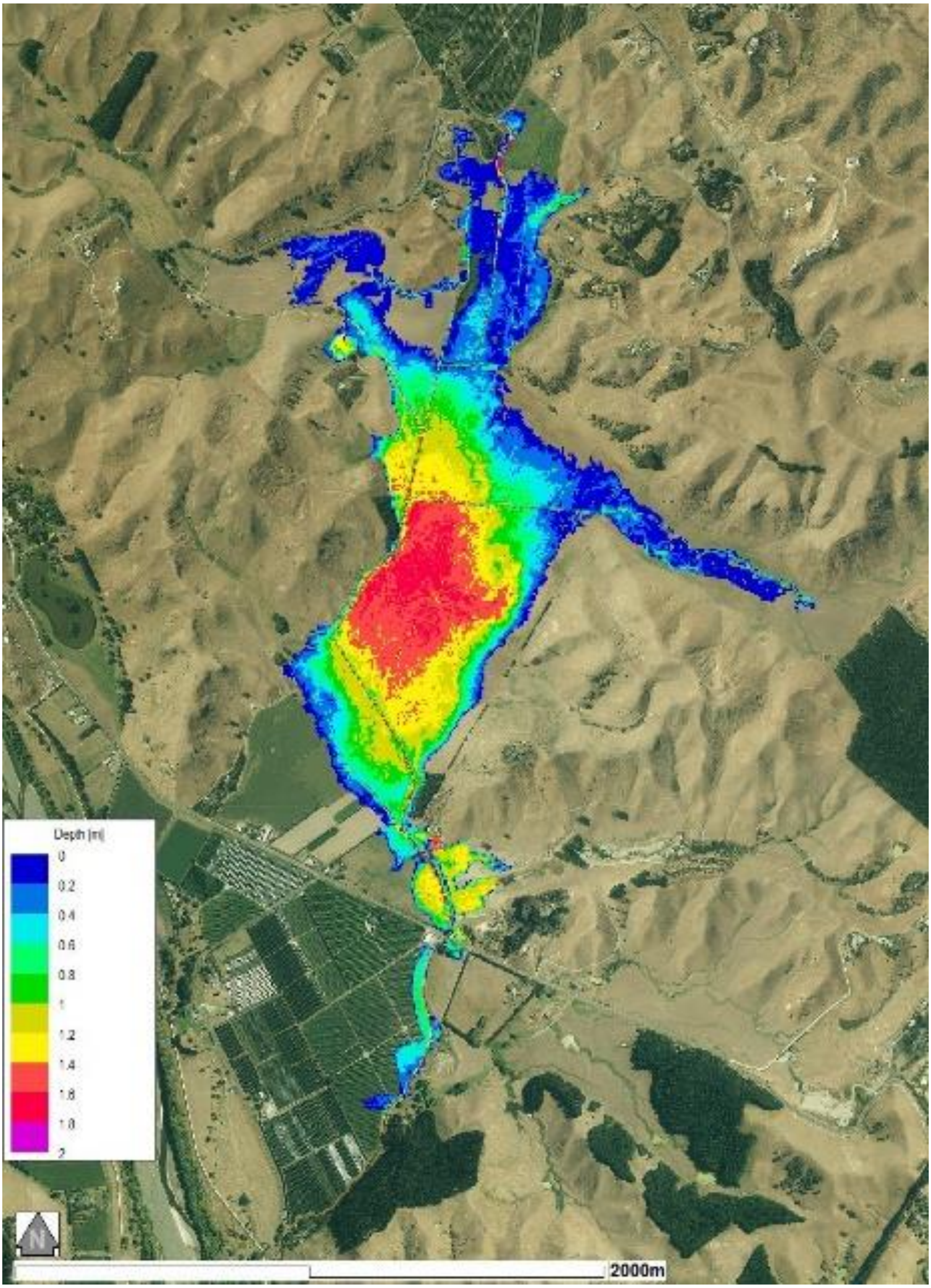
- **Figure 12-18: Flood inundation map showing peak water depth for the 2y ARI Storm for post development unmitigated conditions**



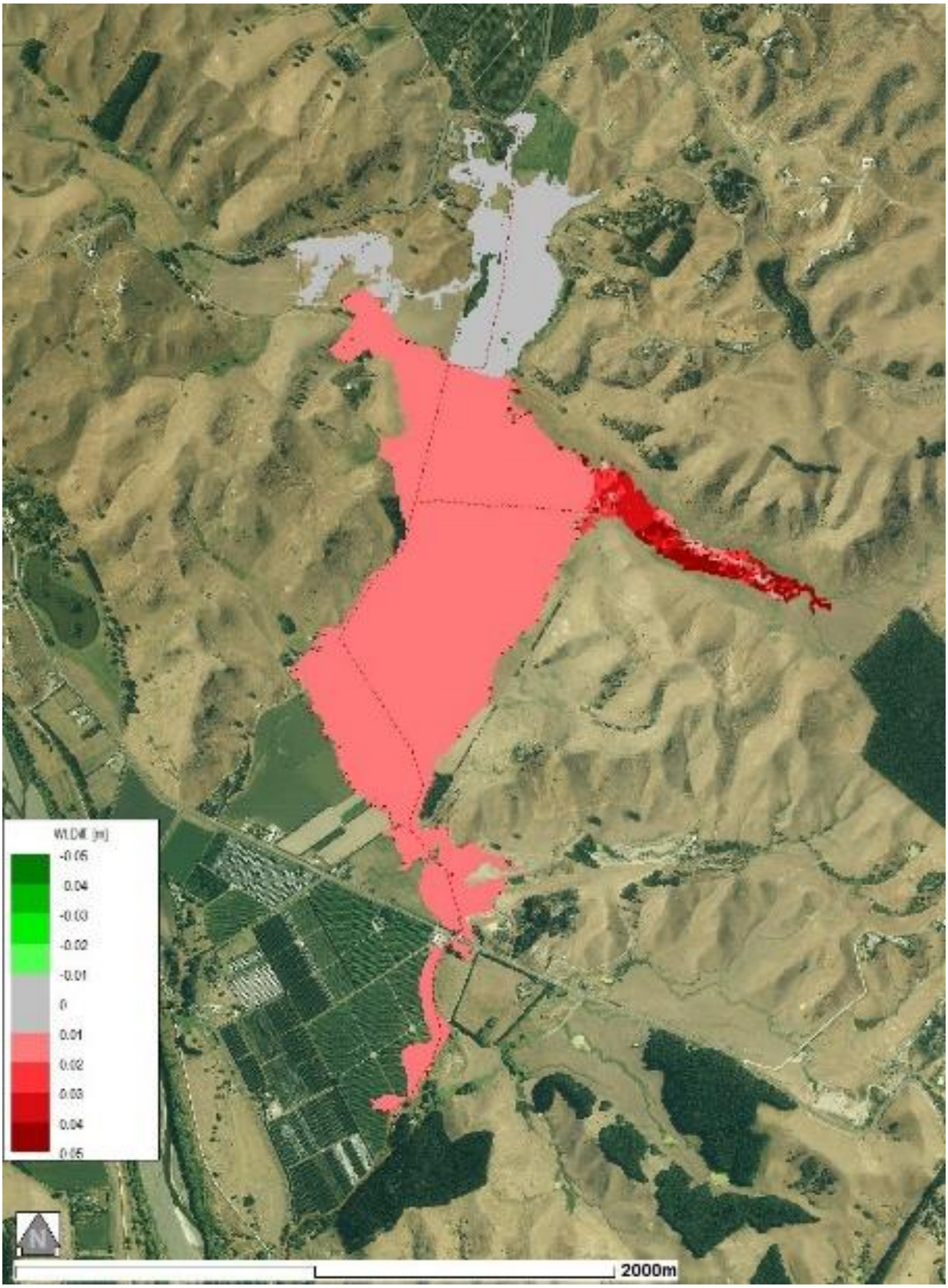
- **Figure 12-19: Flood peak flood level difference map showing differences between the post development case and the pre development for the 2y ARI storm. Green indicates a decrease in water levels, grey, within 1cm and red, an increase.**



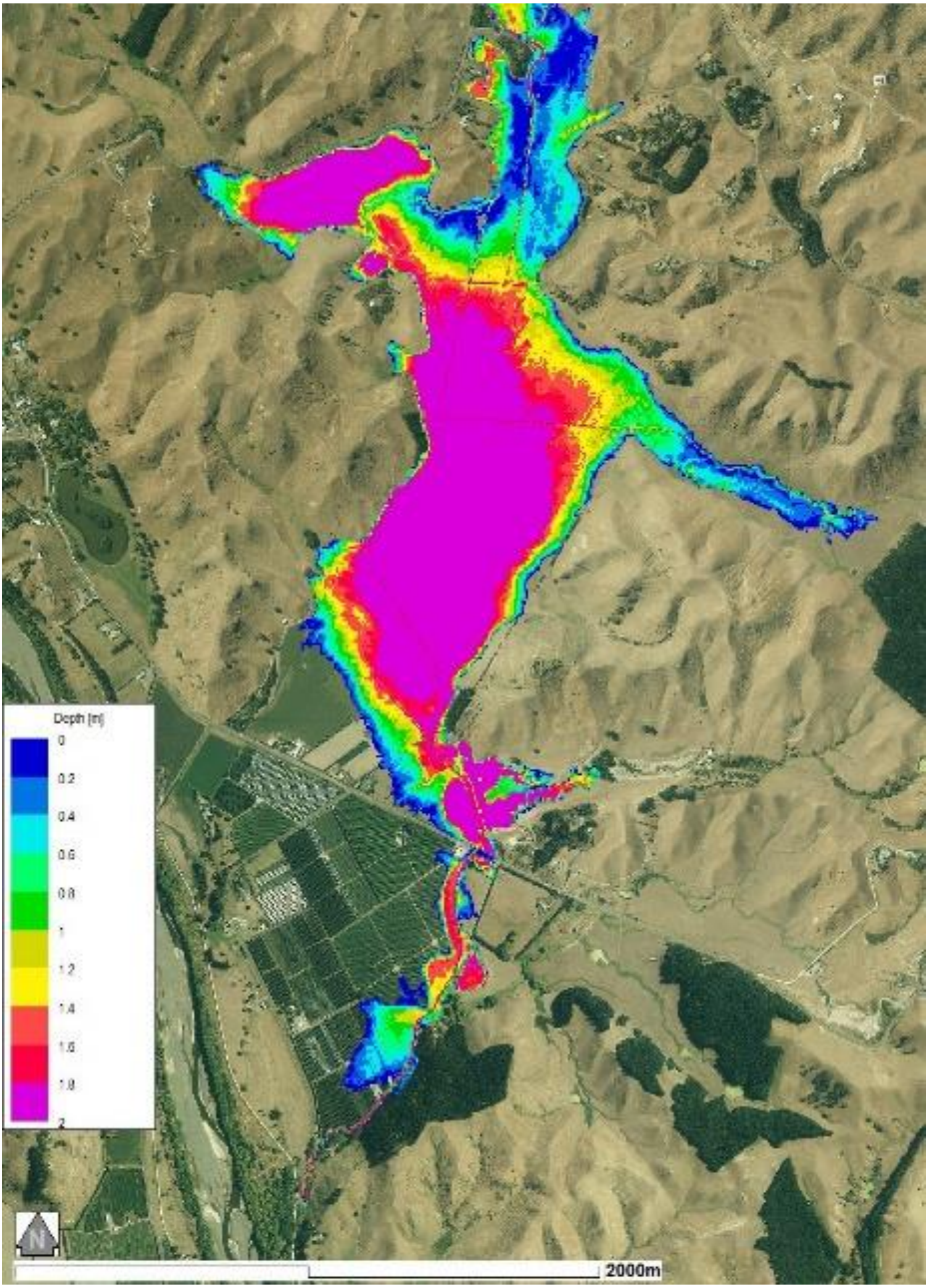
- **Figure 12-20: Flood inundation map showing peak water depth for the 10y ARI Storm for pre development conditions**



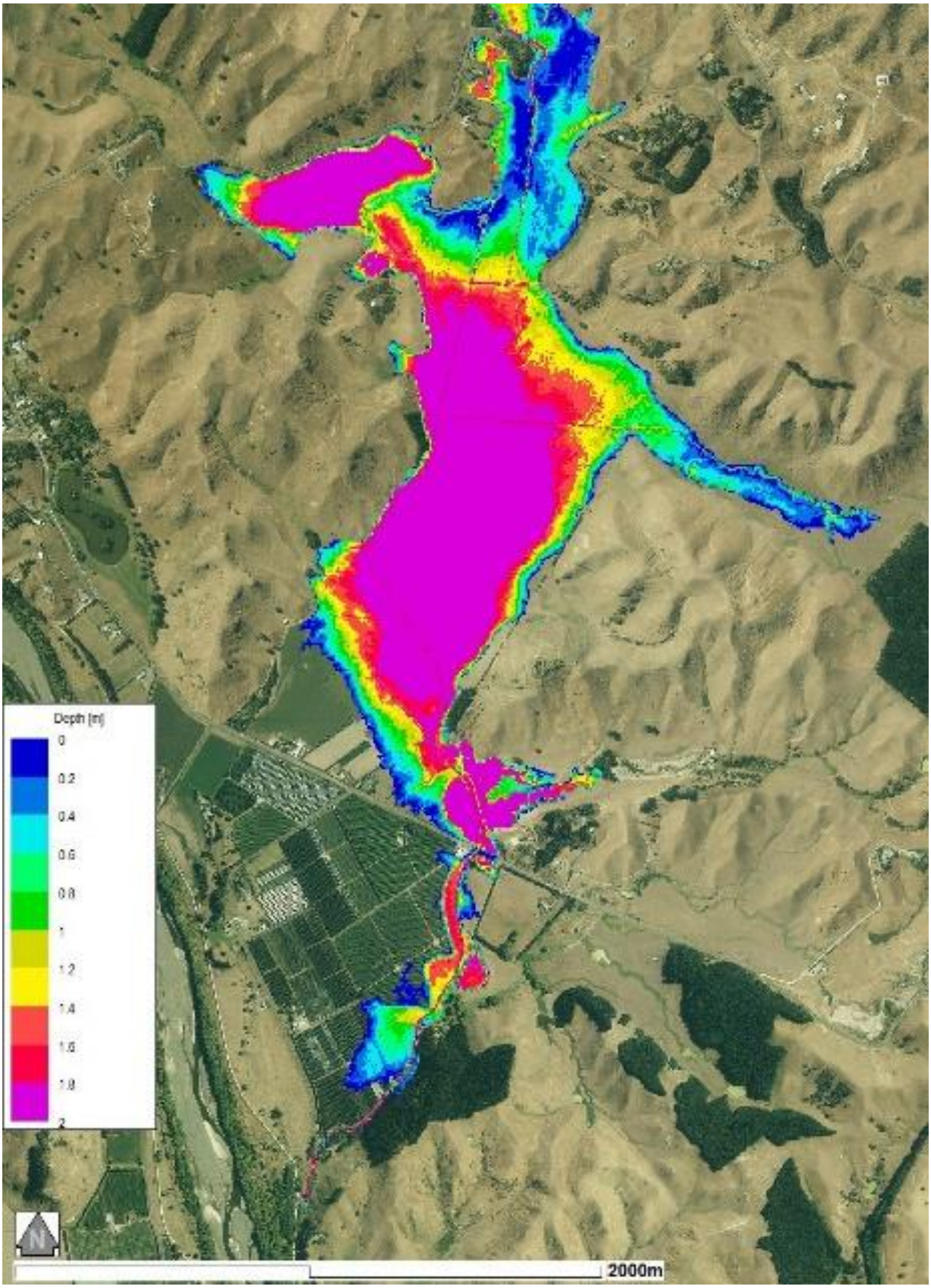
- **Figure 12-21: Flood inundation map showing peak water depth for the 10y ARI storm for post development unmitigated conditions**



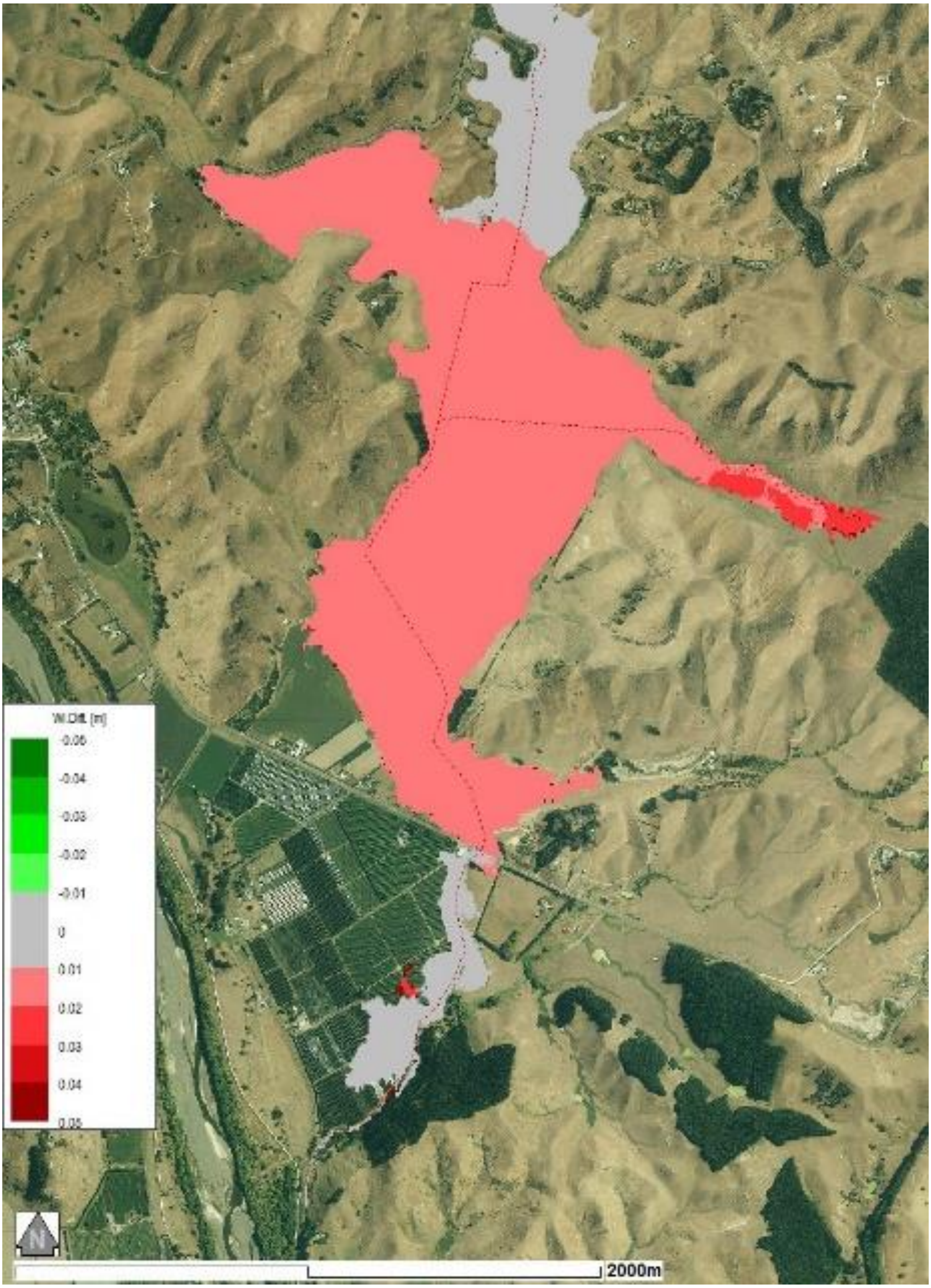
- **Figure 12-22: Flood peak flood level difference map showing differences between the post development unmitigated case and the pre development for the 10y ARI storm. Green indicates a decrease in water levels, grey, within 1cm and red, an increase.**



- **Figure 12-23: Flood inundation map showing peak water depth for the 100y + CC ARI Storm for pre development conditions**



- **Figure 12-24: Flood inundation map showing peak water depth for the 100y + CC ARI storm for post development unmitigated conditions**



- **Figure 12-25: Flood peak flood level difference map showing differences between the post development case and the pre development for the 100y + CC ARI storm. Green indicates a decrease in water levels, grey, within 1cm and red, an increase.**

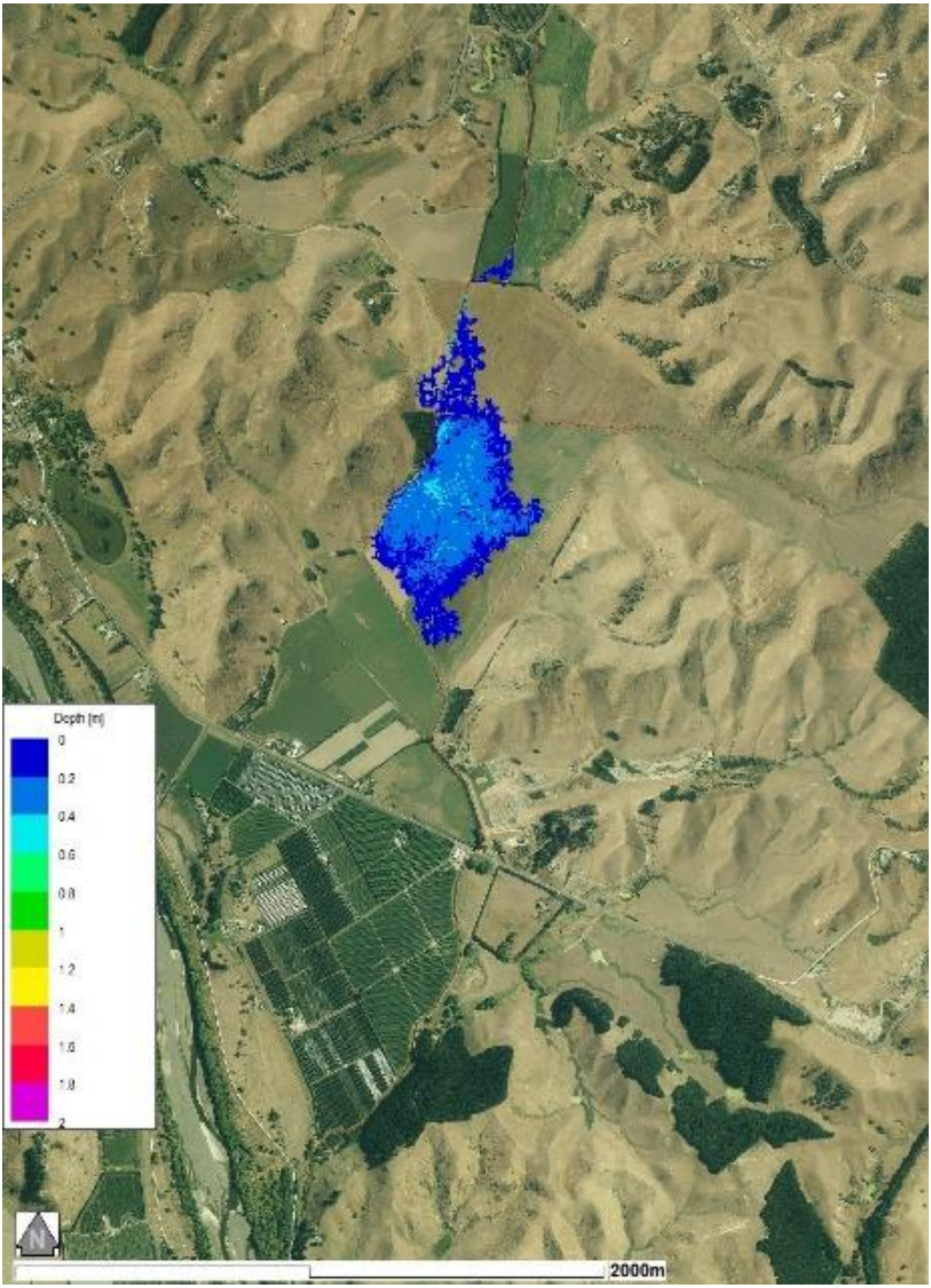
Appendix F: Stormwater Management Pond Peak Inflows and Outflows

Storm Duration	Event (Year ARI)	Sub-Catchment 1 Pond		Sub-Catchment 2 Pond	
		Peak Inflow	Peak Outflow	Peak Inflow	Peak Outflow
24 hr	2	2.4	1.8	2.7	2.0
	2 + CC	3.0	2.1	3.4	2.2
	10	5.1	3.2	5.6	3.2
	10 + CC	6.4	3.6	6.9	3.7
	50	9.7	6.3	10.2	5.0
	100	12.3	9.0	12.9	7.6
	100 + CC	14.9	11.2	15.5	10.0
48 hr	2	2.6	1.9	3.0	2.1
	10	5.6	3.4	6.0	3.4
	50	10.2	6.9	10.7	5.4
	100	12.9	9.6	13.3	8.0
	100 + CC	15.5	11.7	15.9	10.4
72 hr	2	2.8	2.1	3.1	2.1
	10	5.7	3.5	6.1	3.4
	50	10.5	7.2	10.9	5.6
	100	13.1	9.9	13.5	8.2
	100 + CC	15.7	11.9	16.1	10.6

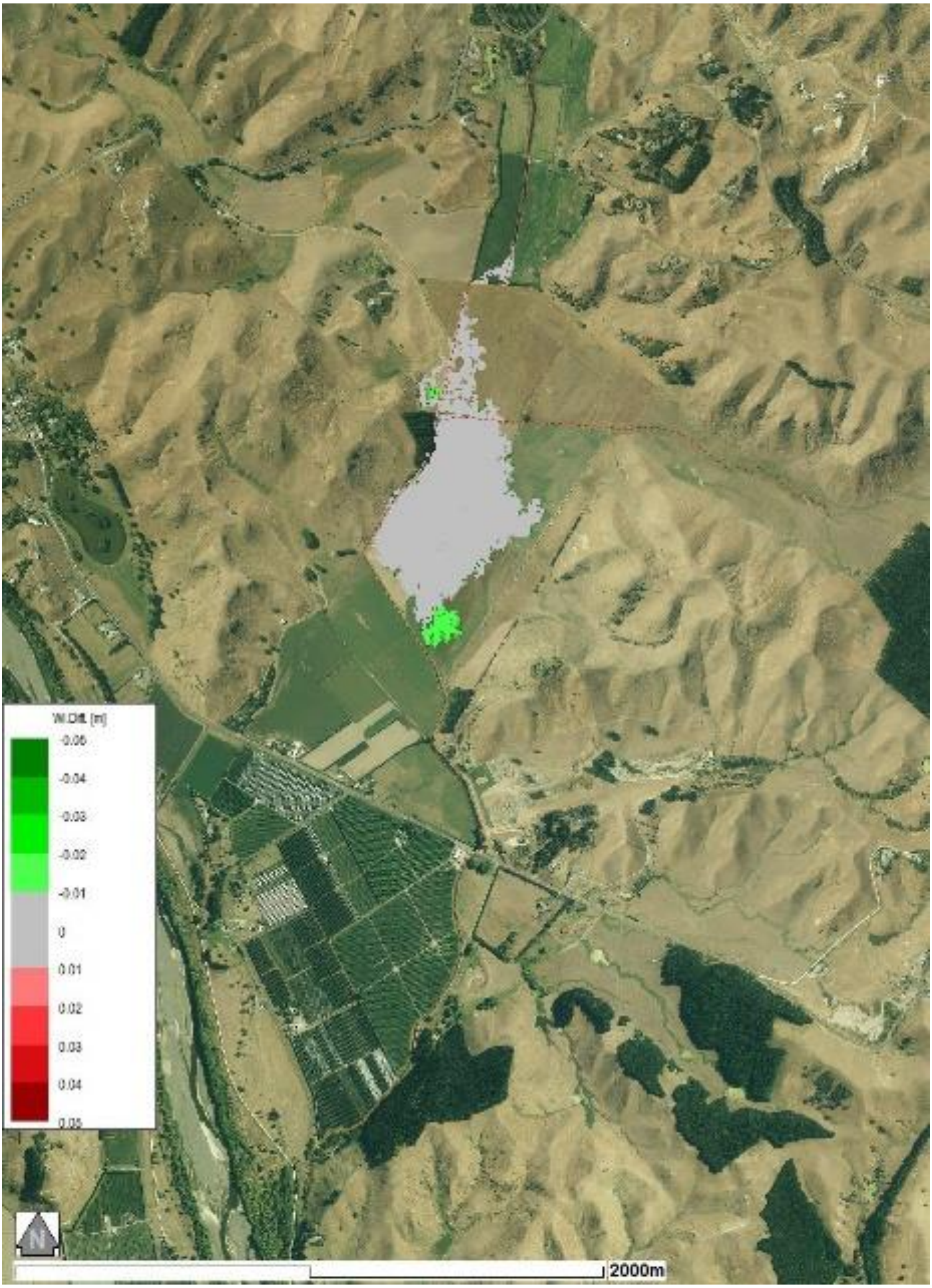
Storm Duration	Event (Year ARI)	Sub-Catchment 3 Pond 1		Sub-Catchment 3 Pond 2*		Sub-Catchment 3 Pond 3*	
		Peak Inflow	Peak Outflow	Peak Inflow	Peak Outflow	Peak Inflow	Peak Outflow
24 hr	2	1.3	0.9	1.3	1.1	1.20	0.01
	2 + CC	1.6	1.1	1.6	1.4	1.54	0.01
	10	2.5	1.8	2.7	2.4	2.69	0.01
	10 + CC	3.0	2.4	3.4	3.1	3.52	0.01
	50	4.3	3.8	5.5	5.0	5.70	0.01
	100	5.3	4.9	7.0	6.5	7.45	0.01
	100 + CC	6.3	5.9	8.4	7.9	9.02	6.17
48 hr	2	1.4	0.9	1.4	1.2	1.33	0.01
	10	2.6	1.9	2.8	2.5	2.81	0.01
	50	4.3	3.9	5.6	5.1	5.82	0.01
	100	5.4	5.0	7.1	6.6	7.55	0.01
	100 + CC	6.3	6.0	8.5	8.0	9.11	6.24
72 hr	2	1.4	1.0	1.5	1.2	1.39	0.01
	10	2.6	1.9	2.8	2.5	2.85	0.01
	50	4.4	3.9	5.6	5.1	5.87	0.01
	100	5.4	5.0	7.1	6.6	7.59	0.01
	100 + CC	6.3	6.0	8.5	8.0	9.15	6.27

***Sub-catchment 3 pond 2 and 3 inflows are from the upstream pond, and some upstream contributing catchment area, so will not be equal to preceding pond in series outflow**

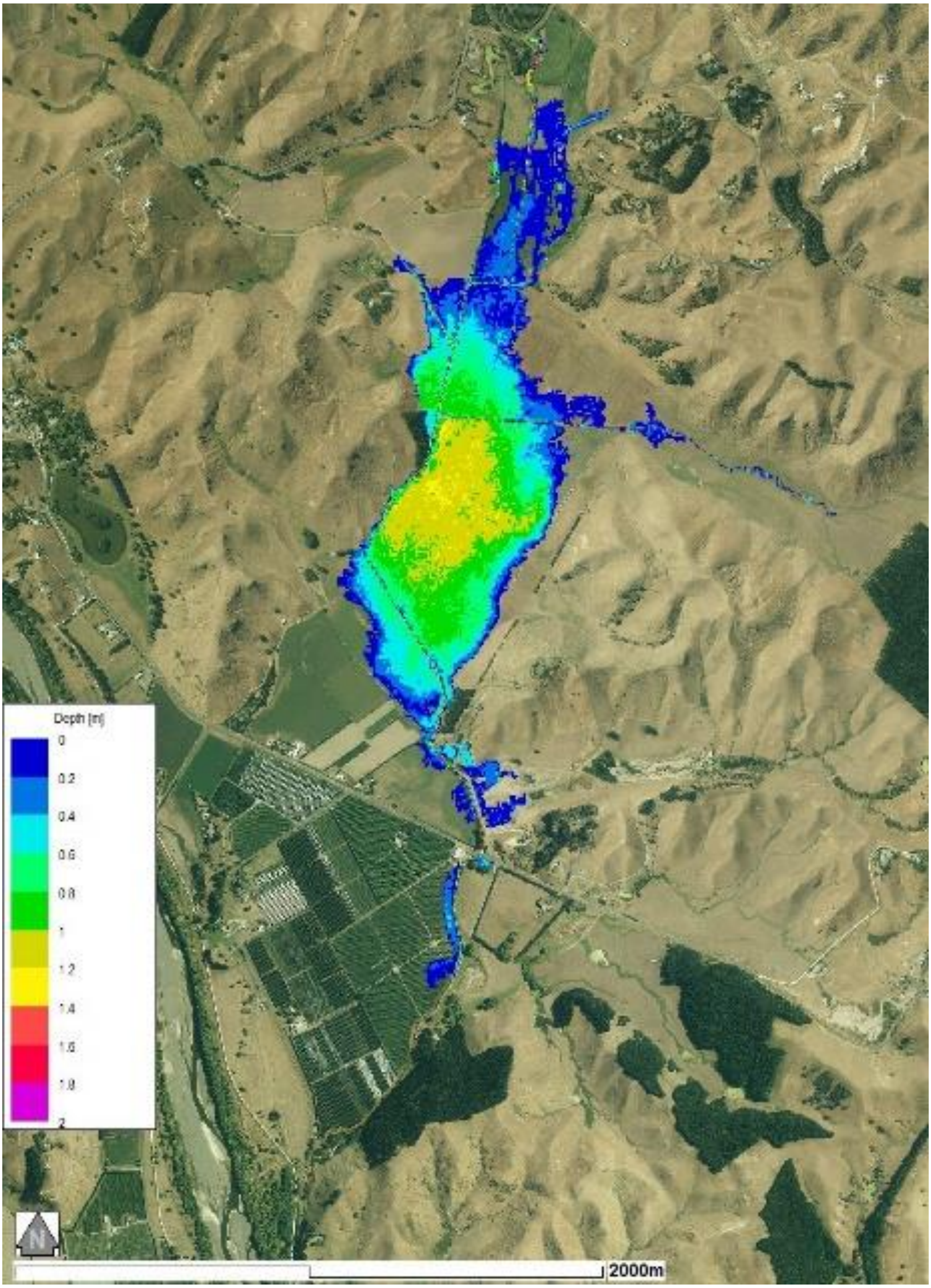
# Appendix G:	Hydraulic Modelling Flood Maps for Post Development



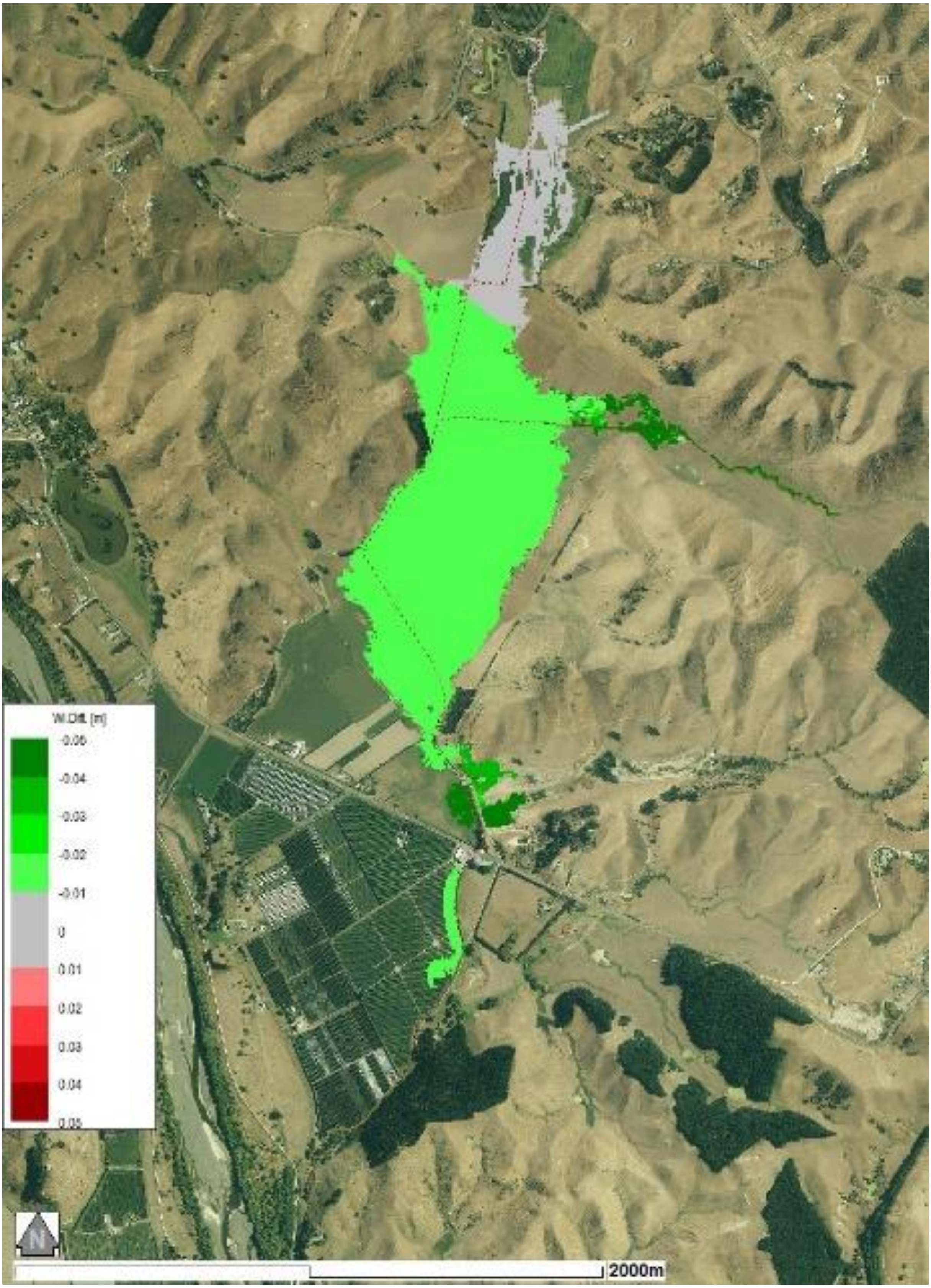
- **Figure 12-26: Flood inundation map showing peak water depth for the WQV Storm for post development mitigated conditions**



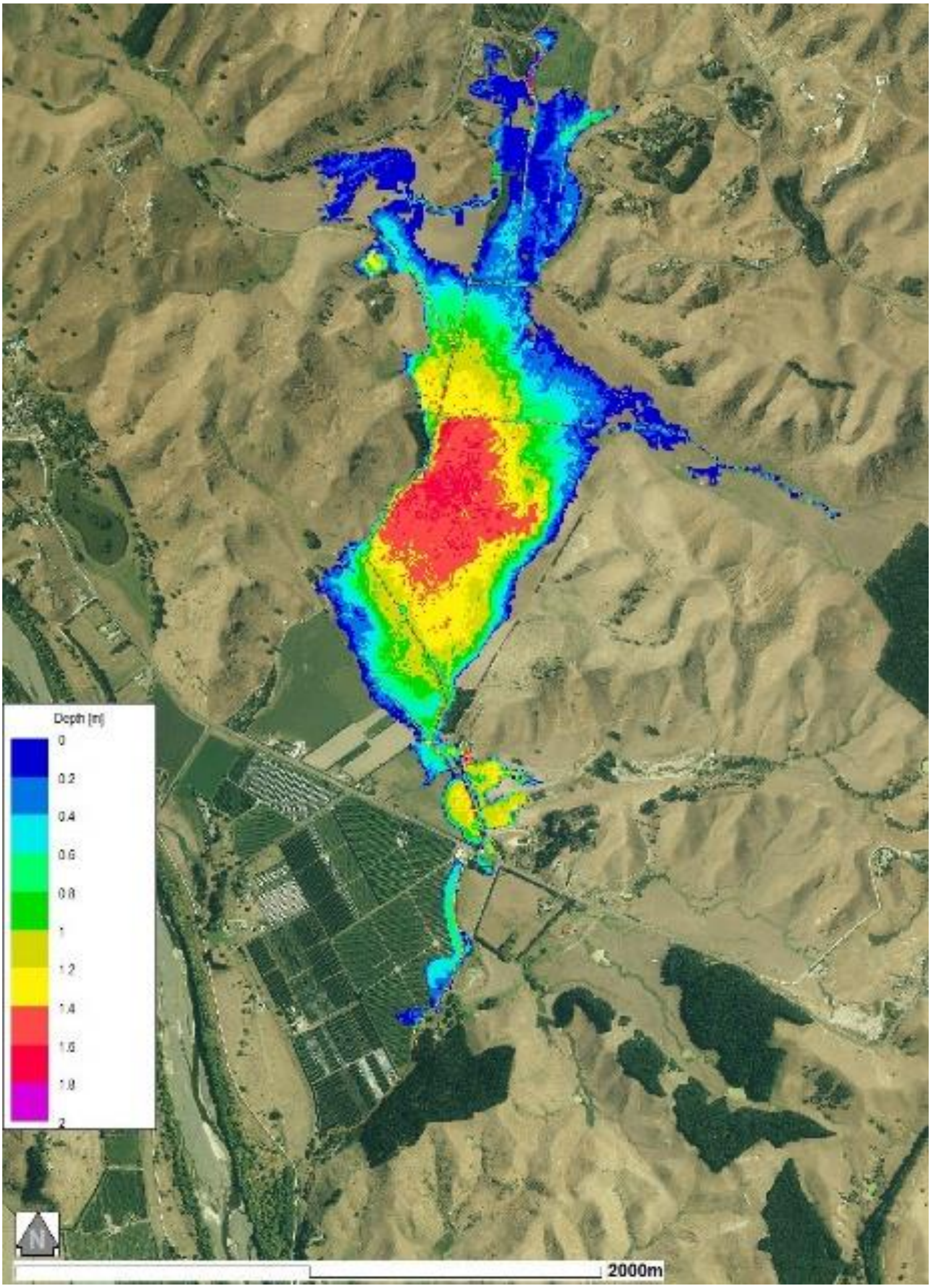
- **Figure 12-27: Flood peak flood level difference map showing differences between the post development mitigated case and the pre development for the WQV Storm. Green indicates a decrease in water levels, grey, within 1cm and red, an increase.**



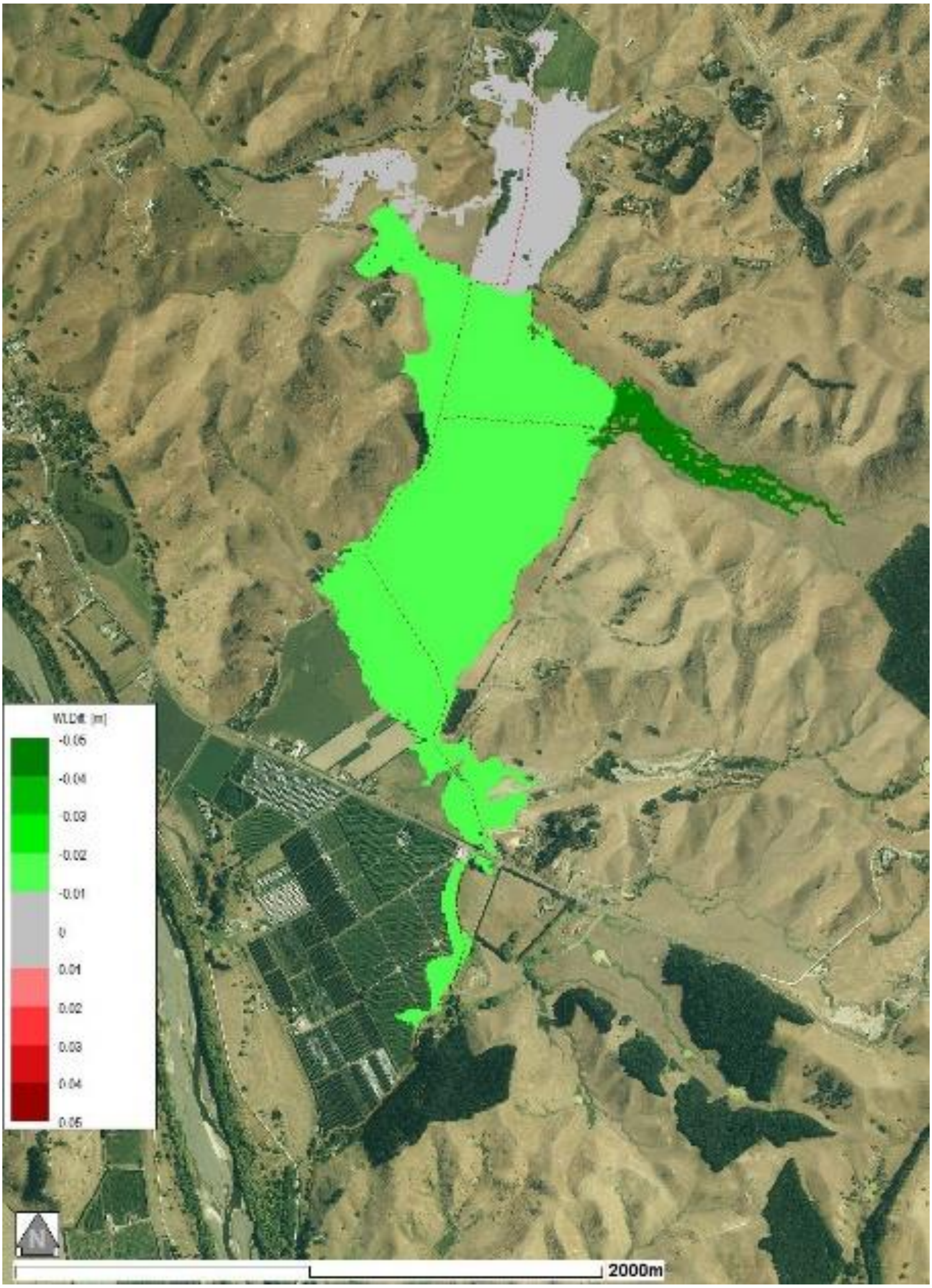
- **Figure 12-28: Flood inundation map showing peak water depth for the 2y ARI storm for post development mitigated conditions**



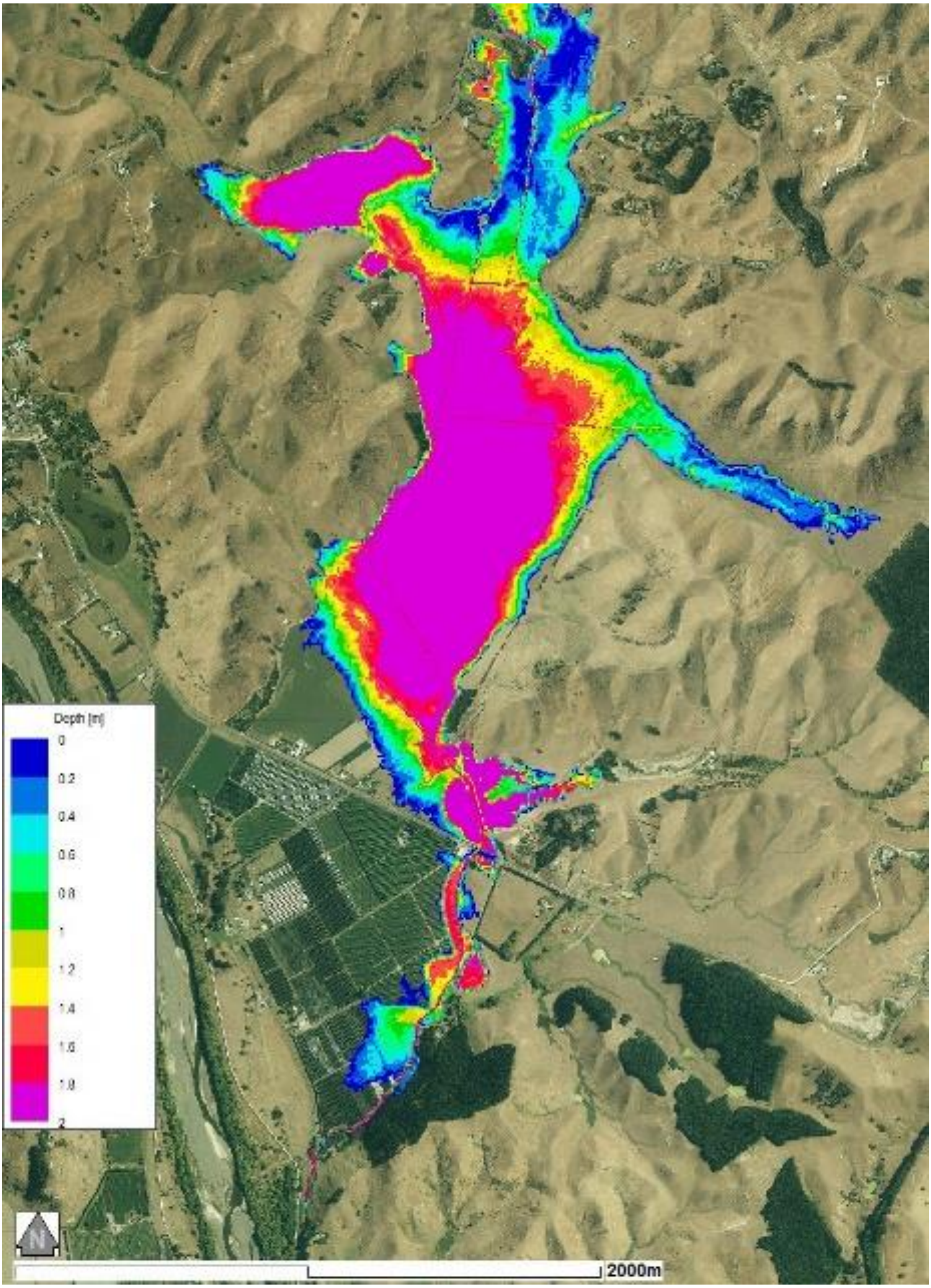
- **Figure 12-29: Flood peak flood level difference map showing differences between the post development mitigated case and the pre development for the 2y ARI storm. Green indicates a decrease in water levels, grey, within 1cm and red, an increase.**



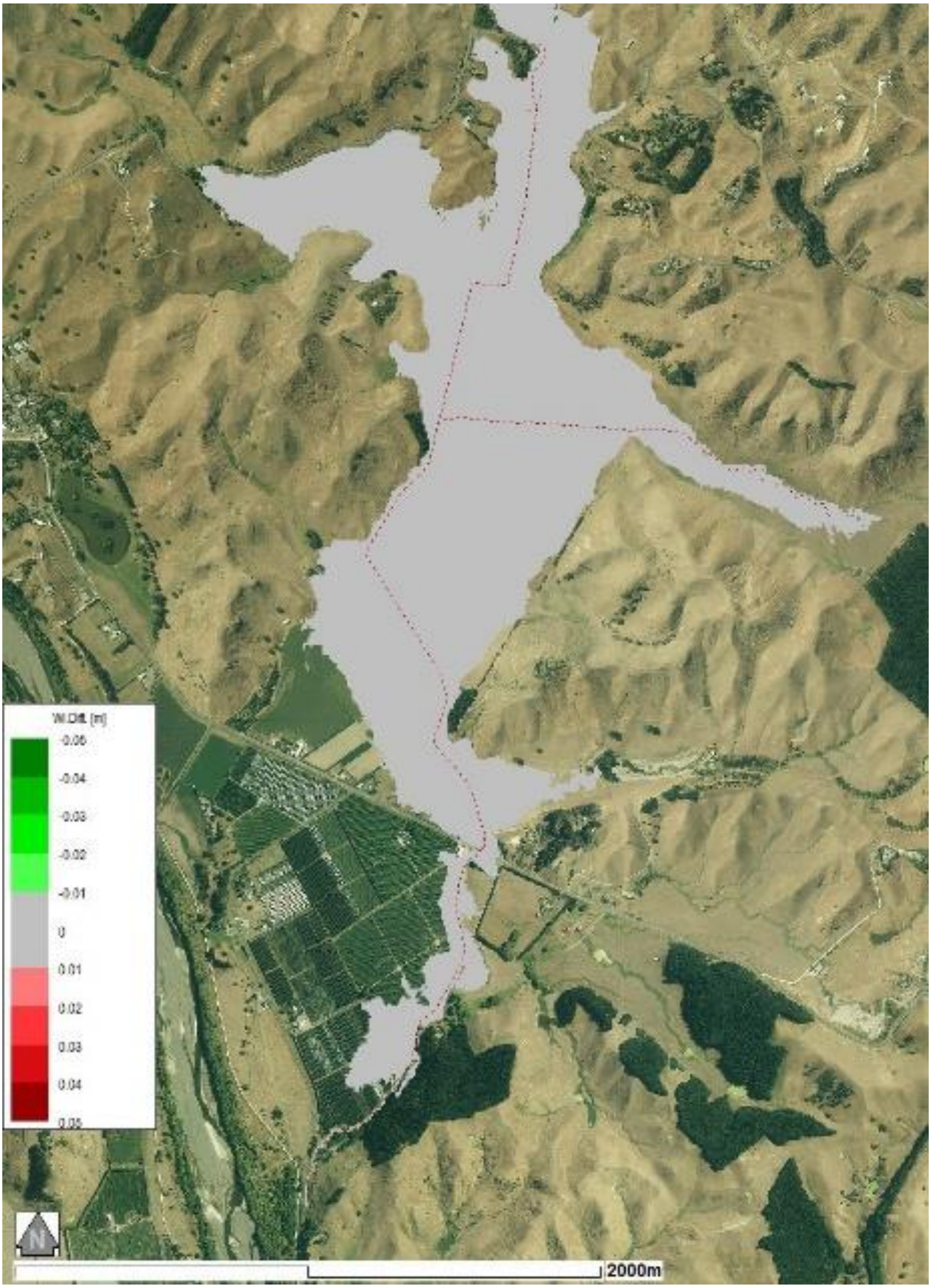
- **Figure 12-30: Flood inundation map showing peak water depth for the 10y ARI storm for post development mitigated conditions**



- **Figure 12-31: Flood peak flood level difference map showing differences between the post development mitigated case and the pre development for the 10y ARI storm. Green indicates a decrease in water levels, grey, within 1cm and red, an increase.**



- **Figure 12-32: Flood inundation map showing peak water depth for the 100y + CC ARI storm for post development mitigated conditions**



- **Figure 12-33: Flood peak flood level difference map showing differences between the post development mitigated case and the pre development for the 100y + CC ARI storm. Green indicates a decrease in water levels, grey, within 1cm and red, an increase.**