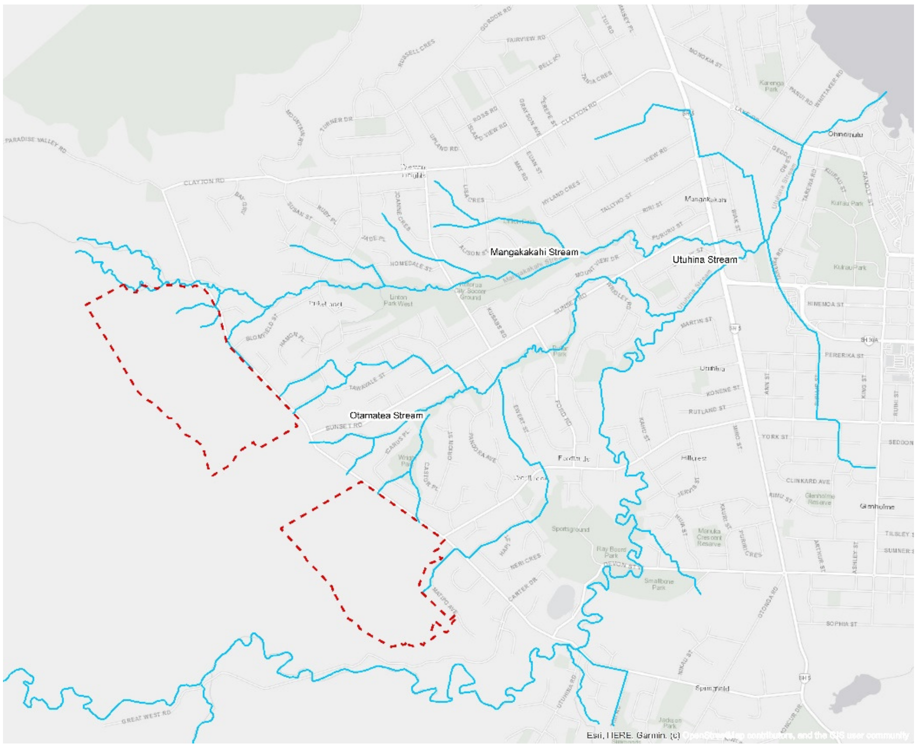
Rotorua Lakes Council

PC2 - PUKEHANGI HEIGHTS

STORMWATER REPORT

19 AUGUST 2020



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PC2 - PUKEHANGI HEIGHTS
STORMWATER REPORT

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REV	DATE	DETAILS
01	17/08/2020	Collation of previous reporting and modelling information early August 2020.
02	19/08/2020	Minor wording amendments and adding linkages to other documents.

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GLOSSARY

Term	Definition
Annual exceedance probability (AEP)	AEP is the chance or probability of a natural hazard event (usually a rainfall or flooding event) occurring annually and is usually expressed as a percentage. Bigger rainfall events occur (are exceeded) less often and will therefore have a lesser annual probability.
Areal Reduction Factor	An adjustment factor applied to point estimates of rainfall to account for the effect of catchment area. The ARF effectively reduces the rainfall estimated from gauge data.
Average recurrence interval (ARI)	The average number of years that it is predicted between events of a given magnitude occurs. Also known as the return period.
Catchment	The area contributing flow to a given point on a drainage system
Conveyance	The means by which water is transferred from one place to another. Natural systems include rivers and streams, whereas built systems include stormwater pipes and drains.
Depression storage	These are small low points in the topography of the land which can store precipitation that would otherwise become runoff.
Detention	Water that enters a stormwater device and is temporarily detained, before being released slowly.
Direct runoff	Water which arrives rapidly after the onset of precipitation; also called storm runoff.
DTM	Digital Terrain Model, often used interchangeably with DEM which is a Digital Elevation Model. A DTM is a digital representation of the landscape allowing 3D analyses.
Ephemeral Stream	Generally, a small stream or upper reaches of a stream that flows only in direct response to precipitation.
Erosion	The process where rock and soil are removed, transported, and repositioned by the action of running water, ice, wind, waves, currents, and mass wasting.
Hydrograph	The changes in flow (either water level or volume) over time.
Hyetographs	The changes in rainfall intensity over the duration of an event.
Impermeable / impervious surface	A surface through which water cannot pass (e.g. roof, concrete)
Infiltration	The process of water on the ground surface entering the soil.

Term	Definition
Infiltration Rate	Velocity or speed at which water enters the soil. It is usually measured by the depth (in mm) of the water layer that can enter the soil over time (usually one hour). The infiltration rate depends on soil texture (the size of the soil particles) and soil structure (the arrangement of the soil particles), crusts or films and head (water depth).
Orifice	An outlet of a specific diameter which restricts flows.
Overland flow	Water that flows over the ground surface. May be caused either by the soil being saturated or when rainfall intensity exceeds the infiltration capacity
Overland flow paths	The path taken by overland flow.
Pervious area	Any area covered in vegetation or garden.
Post-development	Site condition after proposed development has been completed (including existing and new buildings and roadways)
Pre-development	Existing site condition prior to proposed (re)development (including existing buildings and roadways).
PMP (Probable Maximum Precipitation)	Theoretically the greatest depth of precipitation for a given duration that is meteorologically possible over an area at a time.
Retention	Reducing the volume of runoff through disposal/reuse on site. Water that enters a stormwater device and does not leave via an outflow pipe. This can include water lost to exfiltration, reuse and evapotranspiration
Runoff	The flow of water across the ground or an artificial surface generated by rain falling on it
SCS curve number	The SCS curve number method is a simple, widely used and efficient method for determining the approximate amount of runoff from a rainfall even in an area. The curve number is based on the area's hydrologic soil group, land use, land condition and hydrologic condition
Slope	A slope is the rise or fall of the land surface. Refer to the equal area method found in TP108 to calculate the slope required for hydrology calculations
Slope stability	Slope stability is the potential of soil-covered slopes to withstand and undergo movement. The stability is determined by shear stress and shear strength of the soil.
Soakage	The process of water entering the ground (see infiltration).
Time of Concentration (ToC)	The time taken for water to travel from the catchment boundary to the catchment outlet i.e. the minimum duration of a rainstorm necessary so that all parts of the catchment are contributing to runoff.
Watercourse	Natural or artificial channel which conveys runoff

1 PROJECT BACKGROUND

1.1 PURPOSE

To support the district's housing needs, Rotorua Lakes Council (Council) have lodged a Plan Change Application to rezone approximately 150 hectares of rural land southwest of Pukehāngī in Rotorua.

This report has been prepared to determine that there is at least one approach for onsite stormwater management such that the adverse flooding effects resulting from future development can be adequately managed. This report provides the technical documentation of the stormwater assessment of the modelling carried out for the future proposed development for the plan change. For the purposes of this reporting, the definition of adverse effects downstream are increased flood levels in response to the key design events modelled (2 & 1% Annual Exceedance Probability (AEP)).

This report summarises previous work and more recent analysis undertaken since May 2020 to confirm that stormwater from the Plan Change Area (also known as the Pukehāngī Heights Plan Change area) can be managed on site without significant adverse flooding effects.

1.2 SITE DESCRIPTION

The Pukehāngī Heights development is located to the west of Rotorua, adjacent to the suburbs of Sunnybrook and Pomare. It is situated on fairly steeply sloped land with elevations from 310mRL to 400mRL (Moturiki Vertical Datum) and covers an area of approximately 150 ha. Figure 1-1 shows the site in relation to Rotorua and key features and locations within the downstream catchment.

Figure 1-1: Development Site and Key Locations

Several existing overland flow paths have been identified, with varying discharge points into the existing stormwater networks on Pukehāngī Road. These networks discharge into the Mangakakahi and Otamatea Streams, both of which are tributaries of the Utuhina Stream.

The area was identified as a Future Growth Area in the District Plan (Rotorua Lakes Council (RLC), 2016), being currently un-developed pasture. The site comprises three separate developments: Hunts Farm, Sunny Downs and Te Arawa Group Holdings (TAGH). The development sites are shown in Figure 1-2, alongside other key images Figures 1-3 to 1-5 showing the current status

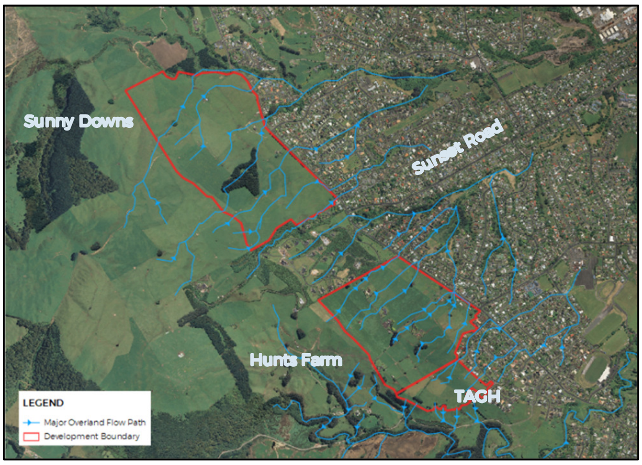


Figure 1-2: Development Sites & Overland Flow Paths

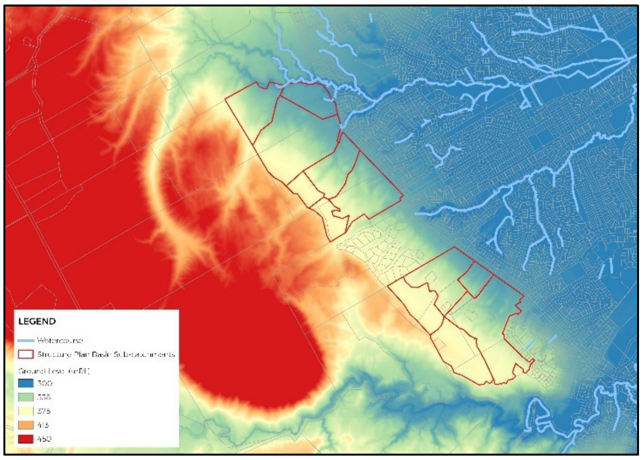


Figure 1-3: Topography

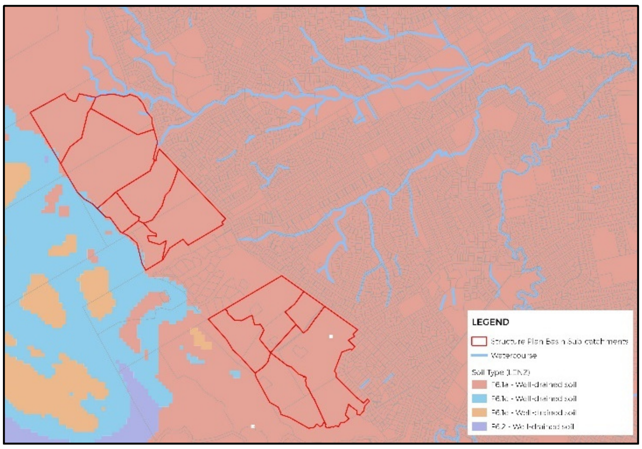


Figure 1-4: Soil Types

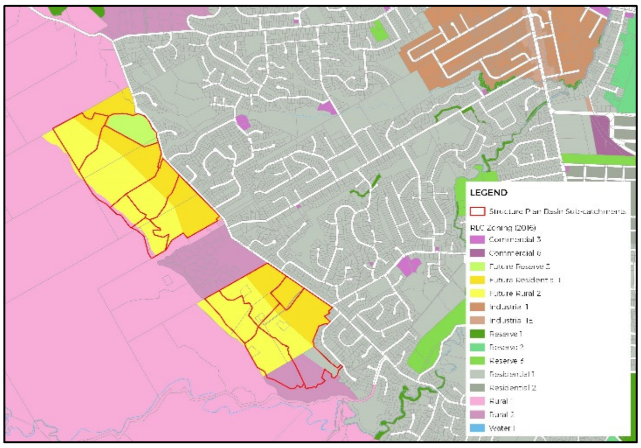


Figure 1-5: District Plan Zoning

1.3 PROPOSED PLAN CHANGE

The proposed Plan Change has a site area of approximately 150 ha and will be primarily of low-density residential land use, with smaller pockets or medium density and commercial land uses. The full total of proposed properties is not yet known.

The final contouring of the proposed site has not yet been finalised either. However, any proposed stormwater management approach should seek to account for the larger catchment area that extends further west up to the ridgeline (approximately 100 ha additional), either through:

- Conveying these upslope areas of rural runoff safely through the development to the appropriate discharge locations along Pukehāngī Road, or
- Capturing additional flow within the proposed stormwater management structures on site.

The proposed layout of the Structure plan as notified is shown in Figure 1-6, and the location of existing overland flow paths are shown above in Figure 1-2

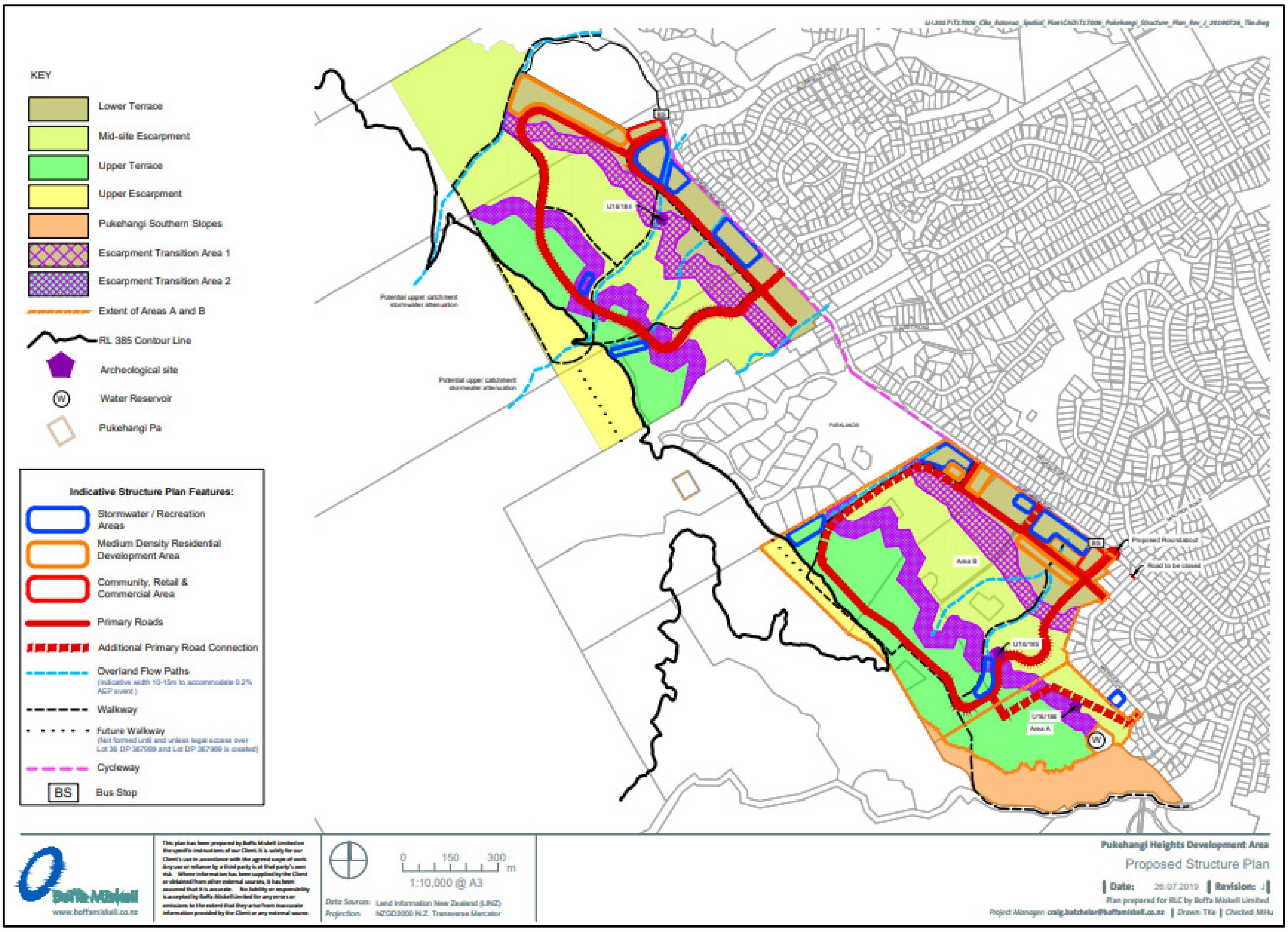


Figure 1-6: Notified Structure Plan (Boffa Miskell, 2020)

Our assessment has been made on the basis that both impervious and pervious runoff from the site, and the upper rural catchment, will be captured and drained to one of twelve stormwater basins, as indicated in blue in the figure above. Flow from the basins into the existing stormwater networks will then be controlled by a series of orifices.

This is a relatively conservative assumption, as the area has high permeability soils; the future development could incorporate an element of stormwater disposal to ground, where geotechnically sound to do so.

1.4 DESCRIPTION OF POTENTIAL EFFECTS

Land development increases stormwater runoff volumes due to increases in impervious area, particularly during the initial phase of a rain event, but also shifts the nature of the contaminant discharges due to the residential and roading activities which take place. The additional runoff volumes, flows, and contaminant discharges can have negative impacts on downstream receiving environments.

In order to avoid potential adverse effects, Rotorua Lakes Council (Council) will require implementation of stormwater management solutions. The application of stormwater volume and quality mitigation practices is typically referred to as water sensitive design (WSD).

The mitigation devices are designed to mimic a more natural flow regime, increasing the time of concentration to reduce peak runoff and velocity through provision of storage and providing treatment to remove some contaminants at source or prior to discharge. Council will require the future stormwater systems design to incorporate a treatment train (series of treatment stages from source to the outfall) to remove gross pollutants as well as sediments, metals and hydrocarbons.

1.5 TIMELINE

This section contains the details of key events in relation to the development of this report. Subsequent sections detail the current best understanding following the process / timeline described below:

- October 2017 – Preparation of an initial advice for stormwater management approaches for developments above Pukehāngī Road.
- 2017 - 2018 – Development of Conceptual Stormwater Masterplans and initial sizing of potential attenuation basins (using attenuation basin spreadsheet modelling) to control peak flow to lower than the estimate pre-development flow rate for a range of different AEP's and durations (up to 24 hours) with different rainfall distributions. In line with Section 7.1.1 of the Regional Council guidance (BOPRC-01, 2012) , where in the absence of a catchment study that – '*in catchments where flooding problems do exist, it is recommended that the post-development peak discharge for the 100-year storm for a new development be limited to 80% of the pre-development peak discharge*'.
- January 2019 – Development of downstream flood risk assessment up to the 24-hour duration storm event, identifying that the 80% rule would yield no adverse effects along the Mangakakahi and Otamatea streams.
- March 2019 – Peer review of works from Tonkin and Taylor. Reports amended to address elements of Peer Review. Key concern in relation to the impacts on the Lower Utuhina stream referenced in reporting, could not be closed out until the Regional Council model became available.
- November 2019 – Meeting with Regional Council to agree on how to share information between models and that the plan change should use the available information to help portray the effects. Agreement to:
    - Use of the Rotorua Lake Council models for urban infrastructure impacts and overland flow in upper urban tributaries (Mangakakahi and Otamatea streams), and
    - Use of Regional Council model for overall catchment flood effects, with Regional Council overlays taking precedence.
- April 2020 – Receipt of 3-day nested design storms from Regional Council to allow use of Regional Council models for effects determination (the BOPRC approach). Initial testing of these identified that the previous approach to sizing the necessary infrastructure showed an adverse effect when compared to the additional volumetric impacts of the 3-day design storm.
- April – now – Ongoing iteration of basin sizing to achieve no adverse effects on peak water levels downstream with Regional Council 3-day design storm.
- July 2020 – Revision to Structure Plan (Revision M) and alignment on imperviousness percentage values with Regional Council.

At the time of submission of this evidence (August 17th), there are some ongoing works, that are not yet identified within the report, namely:

- Revisions to the modelling results presented in latter sections, due to recent agreements with Regional Council on imperviousness and structure plan amendments, namely Scenario 14 modelling outputs;
- Effects assessment using urban imperviousness set to maximum District Plan allowances ('City Future" scenario);

These assessments will follow during the subsequent time up to the hearing.

Further, an option could take place to investigate the potential to undertake effects assessment of 24-hour nested storm on resultant development infrastructure needs and performance. As the requirement to mitigate the volumetric impacts of the 72-hour nested storm is unduly conservative and not in line with current Regional guidance.

2 PROPOSED STORMWATER MANAGEMENT APPROACH

2.1 BACKGROUND

Previous reports contain the background as to the selection of the proposed stormwater management approach identified for the Plan Change area that has been developed in stages as areas have been incorporated into the work undertaken by WSP.

The original advice in relation to potential stormwater management approaches is contained with an (Opus, 2017) memo that 'investigates on a high level basis the constraints to development from a stormwater drainage perspective, as well as identifying the potential for a series of low impact development concepts and strategies that could be utilized for the management of the post development stormwater'.

This has been further tailored through specific further commissions through 2018 and 2019 to develop Conceptual Stormwater Master Plans (C.SMP) across most of the areas contained within the Structure Plan area identified in Figure 1-6. These C. SMPs have been shared as part of the Plan Change notification process, specifically:

— Opus (2017) Pukehāngī Road (Sunny Downs) development – Concept Stormwater masterplan (001-3C1672.00-Chch-01-SWMasterplanning-Rev1 – Updated March 2019).

— Opus (2017) Pukehāngī Road (Hunts Farm) development – Concept Stormwater masterplan (002-3C1672.00-Chch-01-SWMasterplanning– Updated March 2019).

— WSP Opus (2018) Te Arawa Group Holdings Development, Rotorua – Concept Stormwater masterplan (Rev 2 - Infiltration) (content updated March 2019).

The area identified as Area 12 on Figure 1-6 and subsequent work is the only area that has not followed the Conceptual Stormwater Management Plan development approach. Please note that this area is referenced as Area 13 (in place of Area 12)

Additionally, a further report relating to the downstream flood risk assessment for this development has been provided. This work covers the areas of the Mangakakahi and Otamatea Streams that serve the areas downstream of the development area before their confluence with the Utuhina:

— WSP Opus (2019) Pukehāngī Heights Development Area – Flood Risk Assessment (002-VO6-3C1672.00-CHCR-00-FloodHazardAssessment – updated March 2019).

Reference to this report should also be made for the state of anticipated flooding risks within the Plan Change site itself. This report contains key guidance on the development of the site to avoid the potential risks of stormwater flooding affecting the development site itself, through recommendations to preserve and enhance the overland flow paths through the site and develop appropriate low impact design methods (on-lot through to community scale) to control the water at source.

The notified Structure Plan (Figure 1-6) was finalised with the information from the Flood Risk Assessment report to help identify the scale, location of the potential stormwater management devices to support the development.

This Stormwater Report refers to each of these previous works appropriately. The focus of this report is to share the works from May 2020, in conjunction with Bay of Plenty Regional Council (BOPRC), supported by their consultants (River Edge Consulting and Blue Duck Consulting). This report largely supersedes the downstream flood report, identified above, due to the changes in approach taken as a result of receiving the latest model for the Utuhina catchment. The key findings from these studies above, relating to the chosen approach for stormwater mitigation for this area are:

- Development is taking place upstream of an existing urban area through which there are known concerns with flooding.
- The development should seek to mitigate the impacts such that the impacts downstream are no worse than current.
- The selection of 'dry' attenuation basins was due to several factors including the existing ground conditions, the need to balance the peak flows resulting from the development and that the site is shown to lay within areas defined as potentially be susceptible to landslides.
- The siting of these proposed locations was predominantly to suit existing infrastructure crossing Pukehāngī Road. No works have been undertaken to assess the quality and condition of these assets. This is recommended for future design stages.
- The topography of the area presents many challenges to siting basins, but in most locations allows for these structures to be located below the existing ground levels and allow for appropriate servicing of development. A key consideration has been the level and connection through to the existing infrastructure that crosses Pukehāngī Road and maintaining the discharge points from the development to be like those pre-development, preventing further effects to adjacent properties.
- Low impact approaches to stormwater management have the potential to further enhance the developmental impacts and volumetric requirements for mitigation. Through this plan change assessment work, we have taken a precautionary approach, in allowing for a low rate of infiltration on site. This is largely to reflect a worst-case scenario where soakage may not be feasible.
- The design and delivery of the resultant 'dry' attenuation basins should be undertaken by competent professionals due the geotechnical and geological conditions on site.

The above works have had the impact of potentially impacting the notified Structure Plan to one that accommodates these larger basins and resultant changes to land uses surrounding them. The approach of this Stormwater Report is to capture these latest revisions within the works that identify the potential flood risks downstream.

Finally, it is worth noting at this point that the work presented contains one such approach to manage the downstream flooding risks. This approach can be further refined at later stages when further development information is available, this could see the location, size and subcatchment approach presented herein being delivered differently to suit the housing needs for the district. Changes would be captured through appropriate assessments during subsequent stages of the development process.

2.1.1 WATER QUALITY

The 'dry' stormwater basins provide for some water quality treatment through the settlement of sediment, and associated contaminants. Through appropriate vegetation selection and topographic contouring, it may be possible in subsequent design stages to add wetland pond elements to them.

Should this choice be promoted, further, detailed investigations would be required to assess the appropriate methods for amending the current soil profiles to 'retard' and retain water within them.

Both wetlands and 'dry' basins promote sedimentation, however wetlands also promote biological uptake of contaminants for water quality treatment. The choice of vegetation that can withstand both long, dry periods and relatively deep inundation depths and multiple day flood durations will require input from appropriate specialists. Both stormwater management basins and stormwater wetlands are consistent with the Low Impact Design aspirations of the Conceptual Stormwater Management Plan.

We envision some form of 'first flush' treatment would be provided prior to these basins to manage the highest risk run-off in terms of quality. This could either be done at a catchment level, or in a distributed manner following source control principles. For example, through use of rain gardens and swales along the road corridors. These could also be used to dispose of the 'first flush' to ground to reduce the overall run-off volume post development, if geotechnically feasible to do so.

2.1.2 WATER QUANTITY

The 'dry' attenuation basin controls discussed throughout the document relate to managing the downstream flood effects from developing the current land use. As discussed, unmitigated development can lead to an increase in the speed, the volume and the peak flows., which can result in adverse flood related effects downstream.

Section 2.2 presents the further details of the assets that have been tested to support the structure plan approach to the development. The performance objective of these assets is discussed below.

As discussed above, the approach is generally conservative and represents the upper bound values, as the potential for upstream stormwater treatment systems and disposal to ground have not been considered.

2.2 PROPOSED MITIGATION

2.2.1 ATTENUATION CONCEPT AND SIZING

The approach, therefore, has been to focus on providing appropriate mitigation activities to balance the peak flow rates in line with the guidance and key assumption received during 2018 from BOPRC and contained within (BOPRC-01, 2012). Namely, that in the absence of a catchment model, peak flows post-development be limited to no more than 80% of the pre-development magnitude.

The approach adopted by WSP is to ensure the basin discharge in a controlled manner, and at or below pre-development of a range of different events. This is different to the practice of using a nested storm, as this results in a very high pre-development flow rate which does not represent 'typical' rainfall events. The nested profile forces the pre-development discharge rate to equal the upstream catchments Time of Concentration (Tc) and not that of the wider downstream catchment.

Section 1.5 shows the timeline of activity and that this work presented in Section 2.2 refers to the initial work and is placed here for context. Please note, this approach has been further refined and tested, as detailed in subsequent sections as more information has been made available to the project team, including the delivery of a catchment model from the Regional Council in April 2020 to enable the assessment to take place through the Utuhina to the lake.

2.2.1.1 RAINFALL USED

Prior to April 2020, the rainfall input data was sourced from HIRDS V4 from NIWA. HIRDS V4 presents depth-duration-frequency data for a range of future climate scenarios, based on Representative Concentration Pathway (RCP) as defined by the IPCC. This data was utilised for the purposes of the initial assessment in 2019. For the avoidance of doubt, we initially used the rainfall data representing RCP6.0 for the period 2081 – 2100 to size the initial basin infrastructure.

We have utilised several hyetographs to check on the performance of the proposed mitigations to achieve the key philosophies identified above, including the

- Triangle hyetograph
- TP108 style nested storm profile
- North Island - Probable Maximum Precipitation (PMP) profile
- CCC profile

At this stage, we had not included for the HIRDS v4 temporal rainfall profiles for different areas, noting the Peer Review commentary (Tonkin and Taylor, 2019) that this approach was appropriate for this stage of the process. We agree with the peer reviewers' sentiments that rainfall distribution and timing would require confirmation during future design stages to ensure the storage efficiency of the assets is appropriate for its intended purpose across a range of durations and event magnitudes.

As identified above, we have at this stage assumed that the stormwater management delivery will be through end of pipe stormwater attenuation areas (dry areas that accept flow). We expect that this approach can be improved on during subsequent stages with further distributed approaches to deliver Low Impact Design.

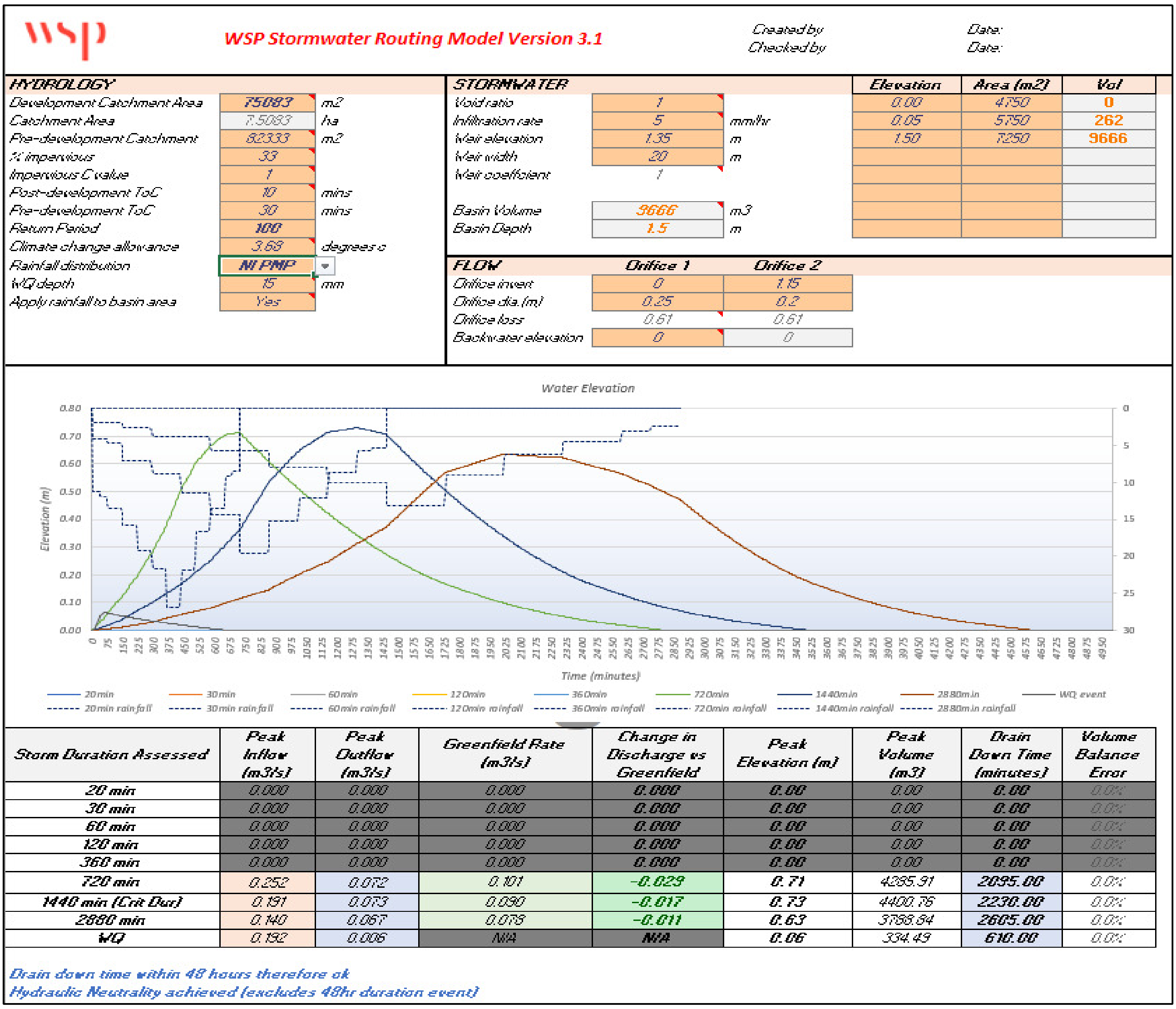
2.2.1.2 RUNOFF

The runoff assessment has been based on a fixed run-off model routed through a linear reservoir. This approach is typically conservative relative to other methods, as it under-estimates the pre-development longer duration flow rates (not accounting for increasing soil saturation over time) and over-estimates total event run-off post development (not accounting for initial losses / initial abstraction).

Testing by WSP has confirmed that this approach typically results in a larger attenuation basin when compared to using more detailed hydrological models such as SCS CN or Horton. Given the later hydrological approaches better approximate run-off processes, fixed run-off is therefore the more conservative approach.

In addition, for this high-level study, we have considered how the proposed development (and in this case end of development storage) could perform in differing design ensembles of rainfall (discussed above in Section 2.2.1.1), return periods and durations to assist with understanding the performance and risks.

The section below identifies the key features of the Attenuation Design Model (an example output is shown in Figure 2-1), utilised for this assessment to assist with the initial sizing of the infrastructure based upon the key requirements for the Regional Council (BOPRC-01, 2012) and (BOPRC-02, 2012). This assessment was further checked against the four rainfall distributions identified (temporal patterns) in Section 2.2.1.1 above.



WSP Stormwater Routing Model Version 3.1

Created by
Checked by
Date:
Date:

HYDROLOGY		
Development Catchment Area	75083	m2
Catchment Area	7.5083	ha
Pre-development Catchment	82333	m2
% impervious	33	
Impervious C value	1	
Post-development ToC	10	mins
Pre-development ToC	30	mins
Return Period	100	
Climate change allowance	3.68	degrees c
Rainfall distribution	NI PMP	
WQ depth	15	mm
Apply rainfall to basin area	Yes	

STORMWATER		
Void ratio	1	
Infiltration rate	5	mm/hr
Weir elevation	1.35	m
Weir width	20	m
Weir coefficient	1	
Basin Volume	9666	m3
Basin Depth	1.5	m

Elevation	Area (m2)	Vol
0.00	4750	0
0.05	5750	262
1.50	7250	9666

FLOW	Orifice 1	Orifice 2
Orifice invert	0	1.15
Orifice dia. (m)	0.25	0.2
Orifice loss	0.61	0.61
Backwater elevation	0	0

Storm Duration Assessed	Peak Inflow (m3/s)	Peak Outflow (m3/s)	Greenfield Rate (m3/s)	Change in Discharge vs Greenfield	Peak Elevation (m)	Peak Volume (m3)	Drain Down Time (minutes)	Volume Balance Error
20 min	0.000	0.000	0.000	0.000	0.00	0.00	0.00	0.0%
30 min	0.000	0.000	0.000	0.000	0.00	0.00	0.00	0.0%
60 min	0.000	0.000	0.000	0.000	0.00	0.00	0.00	0.0%
120 min	0.000	0.000	0.000	0.000	0.00	0.00	0.00	0.0%
360 min	0.000	0.000	0.000	0.000	0.00	0.00	0.00	0.0%
720 min	0.252	0.072	0.101	-0.029	0.71	4285.91	2095.00	0.0%
1440 min (Crit Dur)	0.191	0.073	0.090	-0.017	0.73	4400.76	2230.00	0.0%
2880 min	0.140	0.067	0.078	-0.011	0.63	3758.64	2605.00	0.0%
WQ	0.192	0.006	N/A	N/A	0.06	334.49	610.00	0.0%

Drain down time within 48 hours therefore ok
Hydraulic Neutrality achieved (excludes 48hr duration event)

Figure 2-1: Example output of the attenuation spreadsheet used to assess the initial basin sizing requirements (for the avoidance of doubt, the image presents trials using dummy data).

2.2.2 ATTENUATION DESIGN MODEL

2.2.2.1 MODEL SET-UP

From the information presented in the previous section, the following assumptions were made:

— In relation to the vegetation type, the land is largely well managed pasture and as such the hydrological characteristics for this have 'limited vegetation' applied.

— High moisture level & moderated well drained soil based on Landcare Research database.

— The rainfall intensity and depth data are based on NIWA High Intensity Rainfall System (HIRDS V4) dataset for a site within the site to the east of Pukehāngī Road located at the coordinates of latitude: -38.1373, longitude: 176.1789 (WGS84). The RCP 6.0 depth data for the period 2081 – 2100 has been the input basis for the model, with an allowance for Areal reduction based on the generalised New Zealand parameters taken from the HIRDS V4 Technical report.

— Min time of concentration is 10 minutes as defined in (RLC, 2004) for residential areas.

- Attenuation areas are calculated as homogenous waterbodies.
- Exfiltration rate from the basins wetted area was set to have a constant infiltration rate of 5mm/hr for the base of the basins. Based on type of soil, moisture level and soil drainage understood to be largely prevalent across the development area, this is believed to be a conservative value. It is recommended that this is reviewed in line with specific ground investigations later to ensure the soil can accept the centralisation of potential infiltration areas.
- Varying return periods between 50 % and 1 % AEP and durations from 20 - 2880 mins.
- The run-off coefficient C for the Greenfield Equivalent Flow Estimation was set to be 0.2 (as per Table 5.2 (RLC, 2004), defined to be High Soakage gravel, sandy and volcanic soil types with pasture and grass cover, with no slope correction as per Table 5.3).
- The run-off coefficient C for the post development state was developed through GIS to identify the blended value for the sub-catchment, with an imperviousness run off percentage of 100%. Please note, that the calculations will change with further iterations of the Structure Plan. Further, more refined work will be required in subsequent stages when development layouts are more complete.
- No adjustment correction was applied to the run-off coefficient.
- No initial losses have been allowed for at this stage to support this high-level assessment.

2.2.3 INCLUSION OF DRY DETENTION BASINS WITHIN THE MODEL

Prior to the receipt of the Regional Council model, we developed initial basin sizing as above for each of the sub-catchments and these were added into the ICM hydraulic model as 1d storage node with a controlling outlet in the form of one, or two, orifices (as identified from the spreadsheet model).

Following receipt of the 72-hour nested storms from BoPRC, it was clear that the sizing of these assets was not able to mitigate the impacts satisfactorily due to the 'suggested' input parameters from Regional Council to deliver a precautionary approach to assessing the effects of land use zone changes.

We note that all the largest flood events observed in the lower catchment have been rainfall events with a duration of around 12 hours, which aligns with the catchments estimated Tc, and not 72-hour duration events as BoPRC have adopted.

The nested storm is intended to determine the peak discharge by nesting a range of intensities equivalent to the same AEP. This ensures that the catchments Tc is matched, so long as the base duration is greater than the catchments Tc. Whilst this is a good approach to determine peak flow, it can be overly conservative in terms of run-off volume, as the peak discharge is combined which the large run-off volume of longer duration events. The same applies, when using a particularly long base event duration to account for antecedent conditions. The result is a run-off volume likely to be greater than any single rainfall event would generate.

Table 2-1 below identifies the scale and key basin attributes that have been incorporated into the proposed precautionary approach for mitigating stormwater through the provision of twelve basins, upon which the subsequent results section reports.

The location, configuration and details of the devices as used in the modelling process is one solution that meets the objective to avoid adverse flooding effects downstream and is consistent with the Conceptual Stormwater Master Plan. Subsequent design phases will be required to identify that changes to the development outline contained in the Structure Plan can achieve similar outcomes.

2.2.4 THE POST DEVELOPMENT LAYOUT AND MODELLING APPROACH

2.2.4.1 SUB-CATCHMENTS

The sub-catchment boundaries align with either parcel boundaries or ground contours and were attributed to a node based on the ground contours and the road and reticulation layout. These were then amended to support the modelling assessments and transfer of information between the Regional Council and Councils models.

2.2.4.2 PRE - DEVELOPMENT

The catchments around the development area were adapted to enable an easy transfer of information between modelling platforms used to identify the potential impacts of the development (i.e. the sub-catchments were edited to match the MIKE sub catchments (provided by the Regional Council) as used within the Greater Utuhina Catchment Model (*Subcatchments20190923.shp*). The time of concentration values were adjusted accordingly to match the change in geometry.

Existing modelled sub-catchments downstream of the development i.e. to the east of Pukehāngī Road, were not changed. Figure 2-2shows the model extents and sub-catchment delineation for the pre-development scenario.

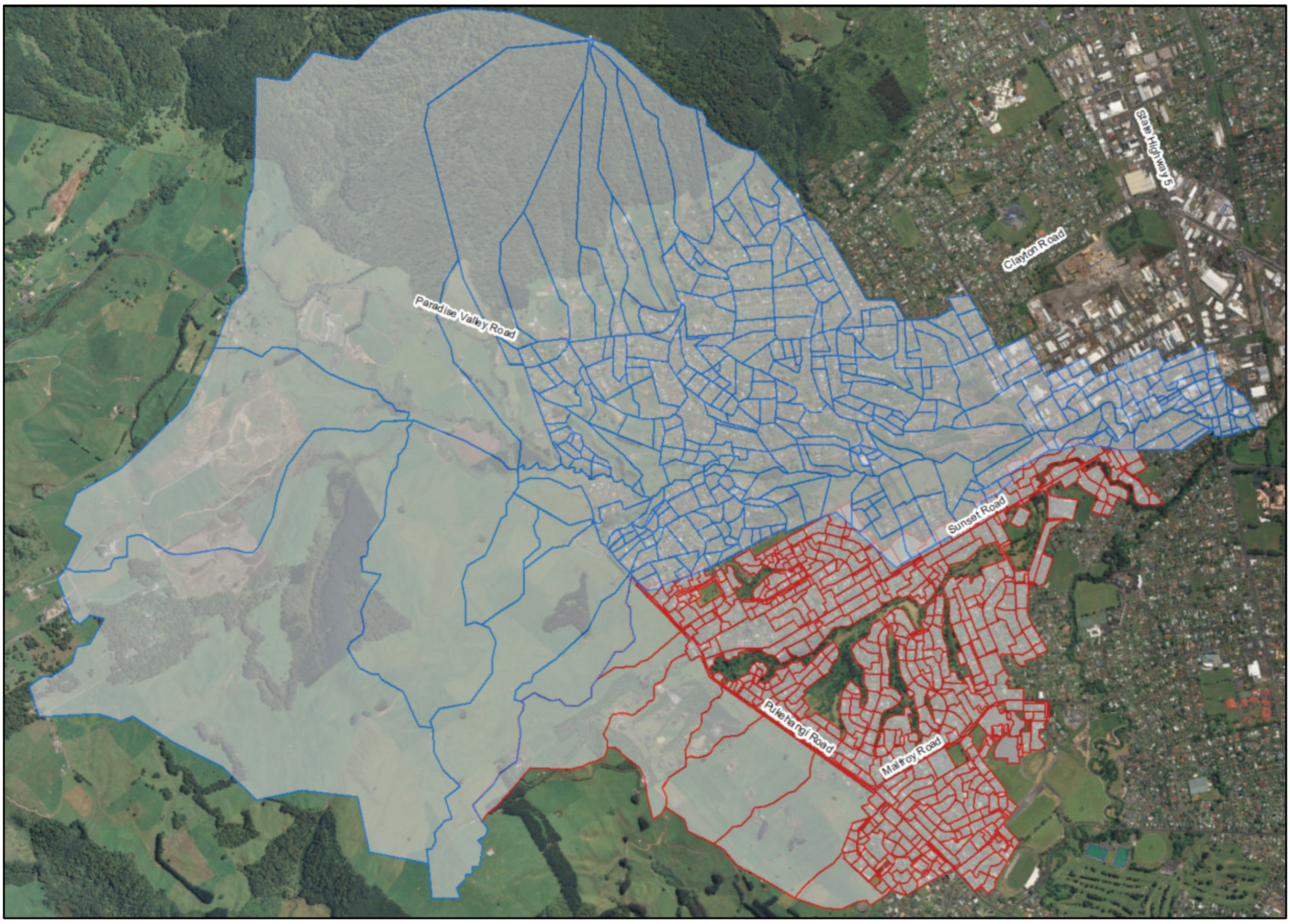


Figure 2-2: Modelling Approach in ICM – Sub-catchments for the Pre-Development state.

The post development scenario including a potential approach to mitigation of the effects is included in Section 2.2.4.

The sub-catchments within the development areas were edited based on the shapefile, provided by BOPRC, to match the Structure Plan. Please note that for these subsequent figures that:

— Red arrows – represent the flow contribution direction from the sub-catchment into either a node (in the pre-development state) or through one of the Basins. Discharge from the basins would be controlled either via orifices or emergency spillways and linked into the same modelled node.

— Purple dotted line – represents one potential approach, in the form of a modelled engineered channel to route flow from the upper basins/catchments through the development to the discharge location. These have been reviewed for high level feasibility and have enough gradient to achieve the performance warranted. Specific design requirements are required to enable the 'safe' conveyance of flows from the upper catchment through the development site.

In addition, for the larger upstream rural subcatchment (above the proposed plan change), we propose that this is routed through the development separately to maintain the critical ephemeral overland flow path (as discussed in previous reports). Subcatchment 40 would be split (at the plan change boundary) as shown in Figure 2-4 for the larger catchment to the west of the Sunny Downs Development area.

We propose to capture and convey this overland flow path through the development to avoid this impacting on the ability of the proposed development basins to manage the stormwater impacts of the development.

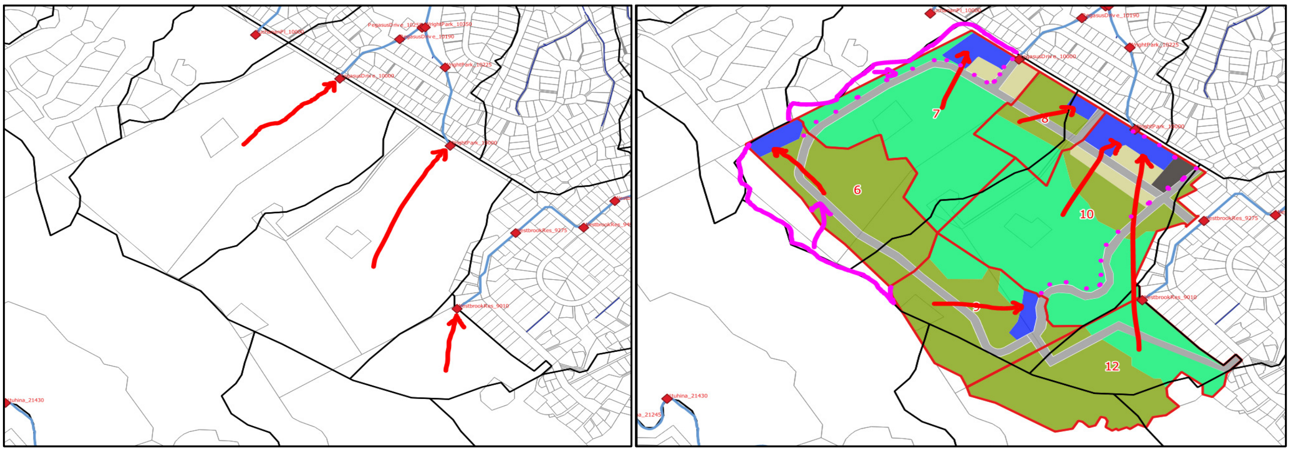


Figure 2-3: Otamatea Stream (Catchment 14 – incorporating TAGH & Hunts Farm) – Pre- (left) and Post-Development (right) Scenarios – Hydrological Catchment Routing

The PC2 level stormwater masterplan identified the potential to create a water sensitive development and distribute the storage of stormwater across the development as shown in the Structure Plan. Figure 2-3 shows an approach to distributing the inflows into the networks downstream, as tested in the modelling to date. For this area, we are proposing to remove contributory flow from WestbookRes_9010 model node and put through Basin 10 to discharge into WrightPark_10000 node, such that:

— Basins 6, 7 and 8 enter the Otamatea urban network at PegasusDrive_10000

— Basins 9 and 10 enter the Otamatea urban network at WrightPark_10000

— WestbrookRes_9010 – will now no longer drain the TAGH land.

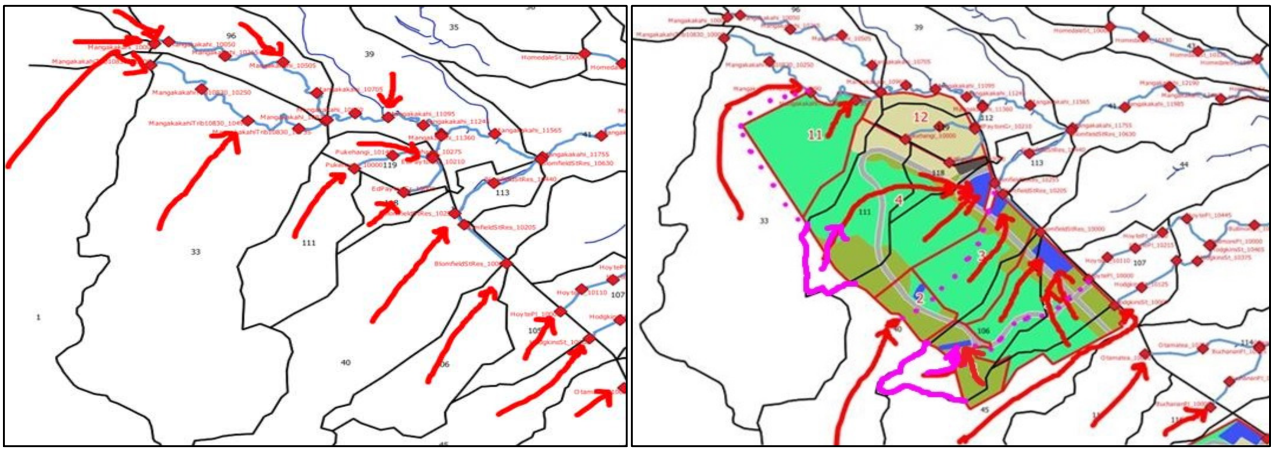


Figure 2-4: Mangakakahi (Catchment 15– incorporating Sunny Downs) – Pre (left) and Post Development (right) Scenarios – Hydrological Catchment Routing

Remaining contributing rural areas upstream of the development areas. were included within the Plan Change area following a precautionary approach. In truth, these areas upstream are small enough to be safely routed (except for the two major overland flow paths identified running to the north of the current Parklands Estate site that sits between the proposed Plan Change site.

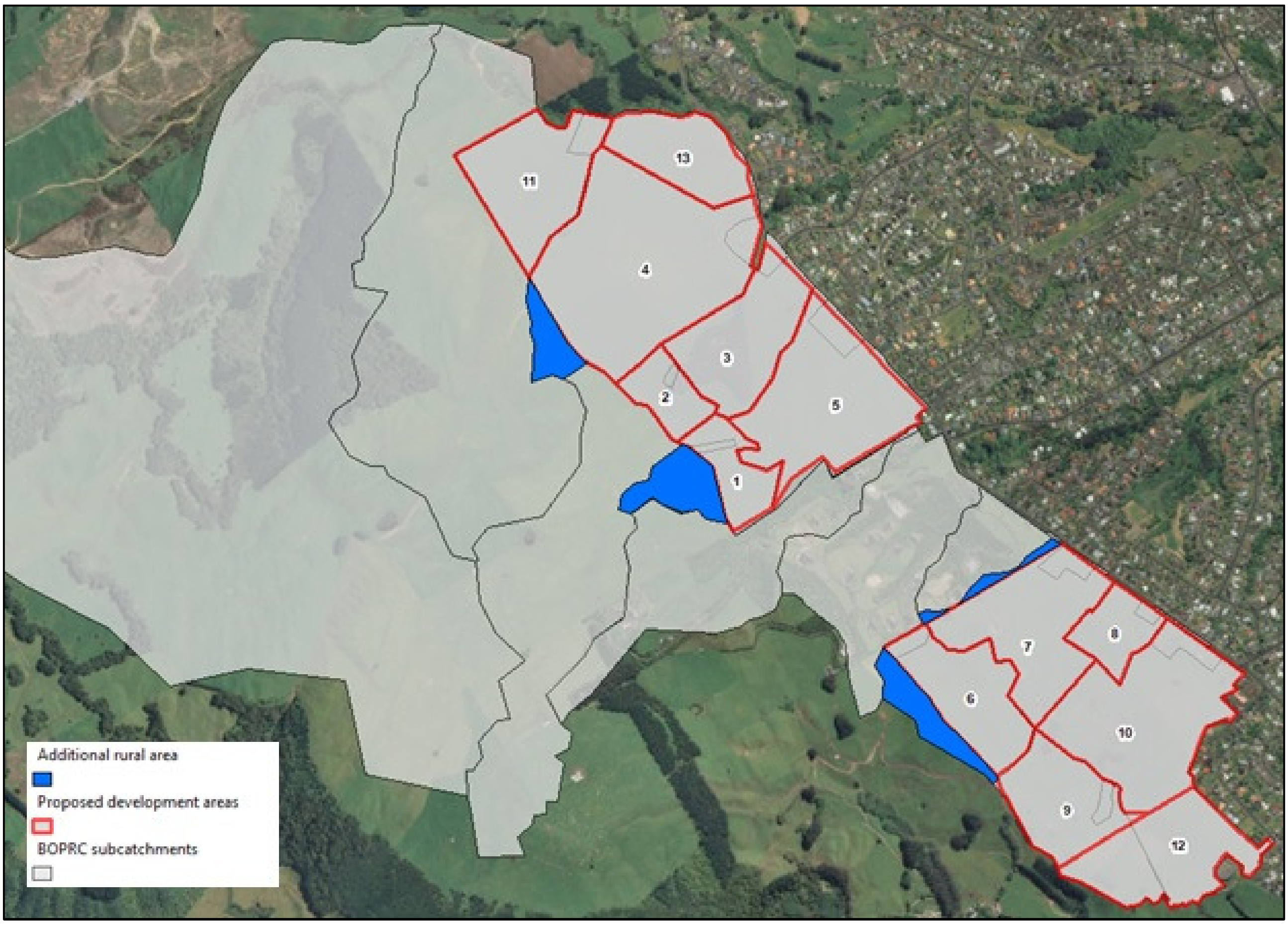


Figure 2-5: Subcatchment amended to represent the Proposed development (blue areas identify areas outside the proposed development that are currently incorporated into the stormwater management approach).

Amendments to the pre-development landform shift where the water will discharge to the downstream network. Please see Figure 2-4 for an example of this shift in where the flows will then be distributed. The development process will change the land form and introduce a positive site drainage system that collects, conveys and stores the excess stormwater.

For this plan change assessment phase and to simplify matters (whilst the final development layout is not yet properly understood) it is assumed that the stormwater for each developed sub-catchment (Sub-catchments 1 – 13 shown on the figures above) are end of pipe development dry detention basins, whose storage are controlled by orifices and discharged either into the existing pipe network or an open channel at specific node points.

The locations selected are like those identified receiving nodes within the Mike Flood model, thanks to early liaison in 2019 between the Regional Council team and WSP. Table 2-1 below shows the latest (July 2020) proposed mitigation approach for the post–development state (Scenario 14), based upon recent changes to Structure Plan layout. Note that imperviousness percentages are identified in subsequent sections.

Table 2-1 – Attenuation Basin and key modelling setup information (Scenario 14)

Basin Subcatchment	Catchment Area (ha)	Upstream Rural Area included (ha)	Model Basin Reference	Maximum Top Water Area (ha)*[1]	Available Storage Depth (m)	Proposed Orifice Details	Potential PIC Assessment Required	PIC assessment under 2019 Proposals
1	8.2	3.6	BasinStorage1	0.73	2	100mm @ 0m 150mm @ 1.5m	N	N
2	3.9		BasinStorage2	0.5	2	125mm @ 0m 100mm @ 1.65m	N	N
3	10.5		BasinStorage3	0.83	2.3	170mm @ 0m 150mm @ 1.15m	N	N
4	29.5	2.3	BasinStorage4	2.30	2.25	300mm @ 0m 200mm & 1.7m	N	Y
5	16.2		BasinStorage5	1.43	2.1	225mm @ 0m 200mm @ 1.4m	N	N
6	12.3	3.2	BasinStorage6	1.00	2.25	170mm @ 0m 125mm@ 1.75m	N	N
7	14.3	0.8	BasinStorage7	0.95	2.25	200mm @ 0m 150mm @ 1.55m	N	N
8	4		BasinStorage8	0.5	2	110mm @ 0m 125mm @ 1.25m	N	N
9	12.7		BasinStorage9	1.40	2.25	125mm @ 0m 150mm @ 1.65m	N	POSS
10	25.5		BasinStorage10	2.20	2.4	225mm @ 0m 300mm @ 1.1m	N	Y
11	11.6		BasinStorage11	0.75	2	225mm @ 0m 225mm @ 1.2m	N	N
13	7.6		PR Pond (13)	1.4	1.7	150mm @ 0m 220mm @ 0.6m	N	N
TOTAL	**156.3**	**9.9**	TOTAL	**13.975**				

*[1] *Note – This represents the Top Water plan area. Additional area will be required to reflect specific design requirements for slope buffers, access allowance etc.*

3 MODELLING APPROACH

This section describes the hydraulic modelling approach taken to carry out the stormwater assessments. The hydrological assessment provides the inflows for hydraulic modelling, which has been used to assess the hydraulic flood effects.

Both the pre-development and the post-development cases were modelled to allow monitoring and comparison of the predicted flows within the area at various points of interest.

3.1 HYDROLOGICAL MODEL

The SCS runoff model (NRCS, 1986) is a well-established approach suited to rural catchments which has been modified to estimate combined runoff for pervious and impervious surfaces referred to as a 'Curve Number (CN) in line with the approach used in New Zealand and overseas. The CN is based on soil characteristics, plant cover, level of impervious area and surface storage. Values presented in this report have been adapted from (ARC, 1999), based upon New Zealand experience and application across the past couple of decades.

As there are no measured flow data for the two tributary catchments, sufficiently high within each catchment, the Catchment 15 model was 'calibrated' to the downstream gauge on the Utuhina Stream and the same values applied to Catchment 14, given they have similar geology.

This original calibration was undertaken by another consultant using a different run-off model (New UK). WSP converted this hydrology (similar in approach to CN) to the equivalent SCS CN for consistency with other RLC and BoPRC models. To do this, the CN was estimated and iteratively fine-tuned changed until the new model flows matched the original model. The CN value obtained is like that used in other areas around Rotorua, with a similar approach to Ia (Initial Abstraction).

3.1.1 HYDROLOGIC SOIL GROUP

The SCS approach uses four soil group categories; A, B, C and D, which range from low to high runoff potential. Catchment 14 has dominant soil type of F6.1a (see Figure 1-4), which is characterised to have good drainage potential. All curve numbers were therefore based on the hydrological soil group A.

Post development pervious areas have been raised to hydrological soil group B in line with (NRCS, 1986) guidance. it is conservative to use a lower CN for existing vegetated areas (Group A) with a Group B, CN value for future vegetated areas post development. This provides for a potential reduction in permeability of the soil due to development earthworks. This value was reached by solving equations for CN using the runoff coefficient value of 0.2 as guided by the Rotorua Civil Engineering Industry Standard 2000 (Version 2004).

3.1.2 HYDROLOGIC CONDITION

Hydrologic condition is accounted for during determination of cover type. Pervious urban areas are assumed to have good hydrologic condition (surface infiltration capacity), while impervious areas are assumed to have an imperviousness of 98% and be directly connected to a drainage system.

The proposed approach to identify the change in cover type from development and its use in the model is to create a blended CN for each post-development sub-catchment, factoring the proportional area of each land use type (cover) and multiplying that by the CN.

3.1.2.1 COVER TYPE

Cover type was determined by undertaking a desktop assessment of aerial photography. Several cover types were identified in line with (ARC, 1999) classifications and as above adapted based on New Zealand experience: The assigned sub-catchment cover types and their corresponding curve numbers are detailed in Table 3-1.

Table 3-1: Curve numbers for sub-catchments – Affected by the potential plan change area ONLY – Scenario 14*1

Cover Description	Impervious Area (%)	Impervious CN	Pervious Area (%)	Pervious CN	Blended Curve Number
	1	2	3	4	= ((1*2) + (3*4))/100
Rural -Pre-Development	0	98	100	21	21
Rural – Post Development*2	0	98	100	29	29
Rural 2 Residential*3: Min. Average lot size 4000 m^2	20	98	80	29	43
Residential 1 – Average lot size 600 m^2	70	98	30	29	77
Residential Medium Density: Average lot size 450 m^2	80	98	20	29	84
Commercial and business	85	98	15	29	88
Streets/roads: sealed	62.5	98	37.5	29	72

*1 *Existing urban areas use SCS CN numbers identified as part of previous hydraulic model build works.*

*2 *To represent improved hydrological efficiency from amendments to the current rural land topography.*

*3 *Planned landscaping of this area to include increased revegetation to support visual and nutrient work.*

Central to the approach taken is the satisfactory selection of the parameters identified in Table 3-1. To support this, Table 3-2 identifies the source and includes discussion on the suitability of each chosen value.

Table 3-2: Discussion on parameter selection presented in Table 3-1

Parameter	Source	Discussion
Rural – CN 21	CN 17 – AC TP108, Wharenui Plan Change CN24 – Hawkins et al (2001)	Previous studies in New Zealand relating to volcanic soils use of CN curve numbers 17 or 24. The original Catchment 15 model's calibration was back-calculated to an equivalent of CN21 within this range
Rural Post Dev – CN29	Equivalency to (NRCS, 1986) tables approach.	Note not statistically showing a similar impact as Group A (CN30) to Group B (CN39) in (NRCS, 1986) tables – More conservative.
Rural Residential - 20% imperviousness	Tauranga CC SMG Guidance. No set limit in RLC Plans.	Quick GIS assessment of Parklands Development shows broad agreement. Steeper topography would self-limit. Note – intent for increased revegetation as buffer (would encourage a CN value for pervious areas comparable (or lower) to CN 21.
Commercial – 85% Imperviousness	Plan Change intent	Small Commercial centres allowed, with assumed low-level off-street parking.

Parameter	Source	Discussion
Medium Density – 80% imperviousness	Plan Change rule (limiting to 80% imperviousness)	Precautionary approach.
Residential – 70% imperviousness	Plan Change rule (limiting lots to 55% imperviousness. Wharenui Plan Change Allowance for sub-roads/private ways of 15%	Precautionary approach.
Road Reserves – 62.5% imperviousness	Development Code & Nutrient Calculations on proposed form of roads to be 'narrow' for safety.	Roading Major (20m corridor, 8m road, 1.5m footpath x 2) – i.e. 11m. Thus imperviousness = 55% Roading Additional (16m corridor, 7m road, 1.5m footpath x 2) - i.e. 10m. Thus imperviousness = 62.5% - Deemed Precautionary.
Imperviousness CN Curve 98	(NRCS, 1986) tables	No change – Precautionary

The distribution of impervious area in the proposed plan change area as reported in Structure Plan (Revision M) is shown in Table 3-4. The values in Table 3-1 have been shared and determined collaboratively with Regional Council at several meetings throughout April – August 2020.

3.1.2.2 ANTECEDENT RAINFALL CONDITION

All CNs are calculated for average antecedent rainfall conditions. The BOPRC approach has a nested storm profile (also known as the Chicago profile), which is shaped to ensure the catchment is saturated prior to the peak of the storm and typically has little sensitivity to initial condition at peak flow.

The 'suggested' approach for this catchment from BOPRC, delivers a 72-hour nested event hyetograph where, for any specified duration, from 10-minutes through to 72-hours, the maximum intensity of rainfall for each duration has the same Annual Exceedance Probability (AEP). This 'type-hyetograph', however, does not represent any measured historical rainstorm.

When combined with the correct time of concentration this allows the catchment runoff analysis to operate on the relevant duration embedded within the nested storm. However, for attenuation design this results in the peak flow rate of a shorter duration event being combined which the much higher run-off volume of a long duration event which essentially has no meaningful statistical probability when used for design purposes.

As a result, the use of 'nested' storms tends to produce a much higher peak discharge when compared to either normalised storm hyetographs (based on 'typical' observed storm events), flood frequency analysis using observed flow data, or other industry standard temporal patterns, like the Probable Maximum Precipitation (PMP).

At this stage to allow for compatibility with the BOPRC model, this approach has been used, However, given the current subdivision regulations around the Region, where typically 24-hour duration nested storms have been deemed to be enough to allow for the assessment of stormwater management, we consider this approach is conservative for attenuation design purposes. Further discussion of this is contained in Appendix C, relating to a high-level assessment of the hydrometric information available for the catchment.

Gaining agreement with regulatory authorities on appropriate parameters for subsequent design stages should be sought, such as the continued use of the 'suggested' 72-hour nested storm approach which is beyond that typically used for design of attenuation systems.

3.1.3 DESIGN RAINFALL

Design rainfall depths were sourced from NIWA's HIRDs v4 for the original C.SMP work and the sizing of potential 'dry' attenuation basins for the plan change area, as described in Section 2.2.

During early discussions, with the Regional Council, it was agreed that we should use their Greater Utuhina Catchment Model (GUCM) as the basis of our investigations to determine the impacts of development on the riverine catchment downstream. This facilitated the production of a whole of catchment overview of the risk through the tributaries and Lower Utuhina catchment itself.

Following this approach, Regional Council provided rainfall hyetographs that represent a 'fully centred' nested synthetic time series, with a depth-duration-probability relationship calculated at each sub-catchment centroid from NIWA's HIRDS v4 spatially varying coefficients (Blue Duck Design, 2019). This report contains further information on the approach taken to model the catchments.

This design rainfall approach assumes that the rainstorm produces a flood event of similar return period and includes for spatial and areal variability across the catchment.

A review of the statistical record and model performance (presented in Appendix C), shows that the rainfall approach 'suggested' is conservative in nature given that the time of concentration for the Utuhina catchment is less than 12 hours. It is worth noting (discussed later in this report) that the impact of running events that are larger than the time of concentration for the catchment will lead to larger volumetric storage requirements to achieve the levels of performance required to prevent impacts downstream.

Therefore, these assumptions are deemed precautionary and as such broadly suitable for the catchment analyses and effects assessments, to identify the potential scale and magnitude of assets required to service the plan change land use changes.

Rainfall hyetographs were generated using this method for the events shown in Table 3-3 and these were used with the assessment of stormwater mitigation needs through the Council's models. The premise for the assessment has been to review the comparative response because of land use changes, focussing in on the differential of effects for changing this variable alone.

Table 3-3: Design rain storm events (provided by Regional Council)

% AEP	Current Day	2130 (+3.68ºC)
10	✓	✓
2	✓	✓
1	✓	✓
0.2	✓	✓

Gaining agreement with regulatory authorities on alternative approaches for rainfall assumptions for subsequent design stages should be sought.

3.1.4 CLIMATE CHANGE EFFECTS

The above rainfall received from Regional Council, includes for effects of climate change on rainfall out to 2130. (BOPRC, 2019) identified within this draft guidance document that for consideration of the impacts of new development, a 3.68 degrees temperature increase out to 2130 should be incorporated into the assessment. This represents a Relative Concentration Pathway of 8.5.

The RCP 8.5 pathway assumes that current global emissions will continue to exponentially rise with large scale reversion to coal use i.e. this presents the worst-case low probability scenario (at the edge of the 90%ile confidence range). The adoption of this RCP is justified by an awareness that current emissions are tracking this pathway, and a desire to design infrastructure to be resilient, even for a low probability outcome.

However, this emission pathway assumes no reduction in increases in emissions over the coming century, and abundant fossil fuel use in future production with a large return to the use of coal. It has been suggested that fossil fuel use at this rate will result in the depletion of all known coal and oil resources by 2070 (Wang et al., 2017).

Based on current policies and pledges made by countries around the world (including New Zealand), this emission's pathway (driven by anthropogenic emissions) has a low probability of eventuating. Therefore, the requirement to use 3.68°C warming for future climate change may result in overly conservative design. This does however, allow a high level of conservatism in design to deliver a resilient design that could offer a potentially higher level of service. A more pragmatic approach might be to provide additional space for expansion of the attenuation areas in the future, once the likely outcome is better understood.

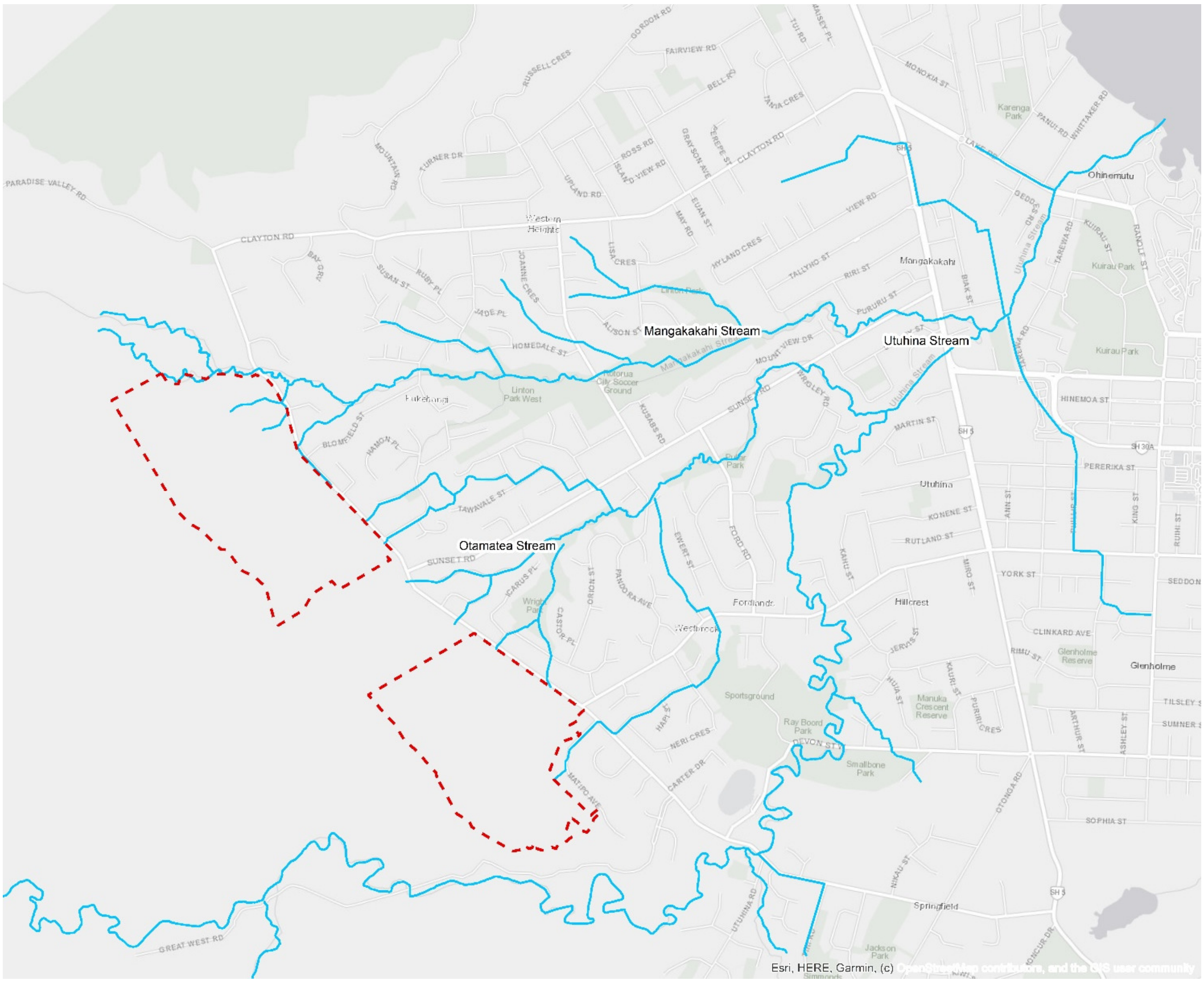
Updated guidance provided, (MFE, 2018) refers to four RCP (Representative Concentration Pathways) scenarios. The RCP 6.0 scenario can be considered a "middle of the road" prediction of climate change and has been adopted by several territorial authorities for similar catchment wide and effects-based studies.

Given the above, it is suggested that the design rainfalls 'suggested' by Regional Council for use in this assessment allow for a precautionary approach to the assessment. Gaining agreement with regulatory authorities on appropriate climate change projections for subsequent design stages should be sought.

3.1.4.1 RAINFALL DISTRIBUTION

A 'fully-centred' temporal rainfall distribution was developed based on the 'embedded' rainfall approach, often referred to as the Chicago or Nested Storm Method. The method uses a rainfall distribution where short duration, high intensity storms are embedded in longer duration, higher volume storms. Blue Duck Consulting (2019) contains more information on the derivation of the utilised design storms. Section 3.1.2.2 contains some discussion on the use of this dataset and its precautionary approach. In short, the approach contains 1% AEP peak intensities for a range of rainfall durations (all with equal probability) nested together within a 72-hour storm depth.

This has the benefit of being a computationally efficient manner for assessing the peak flows for critical storm durations but use of a nested storm beyond the receiving environments Tc can result in overly conservative design in terms of attenuation. Hence the nested storm is typically used for estimat



ion of peak discharge rather than volumetric assessment, for the reasons outlined above.

Gaining agreement with regulatory authorities on appropriate distribution for subsequent design stages should be sought.

3.1.5 LAND USE AMENDMENTS

The received structure plan approach (revision M, July 2020) and area land use calculations undertaken, within the proposed C.SMP basin sub-catchment approach (as one approach for managing the site development stormwater). Table 3-4 contains the assessment results, showing the distribution of different land use types across each of the basin sub-catchments. This directly influences runoff changes for each of these basins as each land use has different run off characteristics.

Table 3-4: Approximate Land Use Sizes per Area (based on Structure Plan Rev M, July 2020)

Basin Subcatchment	Approx. Area (Ha)	Existing Land Use	Structure Plan Land Use	Area (Ha)
1	8.2	Rural	Residential	3.4
			Basins	0.7
			Roads	0.5
			Additional Post Dev Rural	3.6

Basin Subcatchment	Approx. Area (Ha)	Existing Land Use	Structure Plan Land Use	Area (Ha)
2	3.9	Rural	Residential	2.4
			Basins	0.5
			Roads	0.8
			Rural Residential 2	0.2
3	10.5	Rural	Residential	4.4
			Basins	0.8
			Roads	0.6
			Rural Residential 2	4.7
4	29.5	Rural	Commercial	0.4
			Medium Density	1.8
			Basins	2.3
			Residential	9.8
			Roads	2.7
			Additional Post Dev Rural	2.3
			Rural Residential 2	10.2
5	16.2	Rural	Residential	7.7
			Basins	1.4
			Roads	1.7
			Rural Residential 2	5.3
6	12.3	Rural	Residential	6.8
			Basins	1
			Roads	1.3
			Additional Post Dev Rural	3.2
7	14.3	Rural	Medium Density	0.5
			Basins	1
			Residential	4.2
			Roads	1.3
			Additional Post Dev Rural	0.8
			Rural Residential 2	6.5
8	4.0	Rural	Residential	2.7
			Basins	0.5
			Roads	0.9
9	12.7	Rural	Residential	9.4
			Basins	1.4
			Roads	1.8
			Rural Residential 2	0.1
10	25.5	Rural	Commercial	0.3

Basin Subcatchment	Approx. Area (Ha)	Existing Land Use	Structure Plan Land Use	Area (Ha)
			Basins	2.2
			Medium Density	0.6
			Residential	9.4
			Roads	3.2
			Rural Residential 2	9.8
11	11.6	Rural	Basins	0.8
			Rural Residential 2	10.8
13	7.6	Rural	Medium Density	6.2
			Basins	1.4
TOTALS	**156.3**	**Rural**	**Mixed**	**156.3**

3.1.6 DISCUSSION

The key findings from the hydrological assessment in Appendix C, suggests that for the Utuhina catchment, and studies relating to the effects of development, that:

- The rainfall and flow frequency analyses show no consistent relationship between large rainfall and large floods. For example, the largest flow event has a 7.5% AEP, yet the associated rainfall was a 1.4% AEP event. Antecedent conditions likely mask any simple relationship;
- The response of Utuhina Stream to rainfall is very quick, with the peak flow reached typically between 4-7 hours after the rainfall;
- The rainfall events corresponding to the six largest flows in the Utuhina catchment lasted between 6-12 hours, except for the largest event, 29 April 2018. However, even during that event the 'bulk' of the rainfall and the response of the stream lasted over only 8-hours;
- Comparison with other temporal distributions demonstrated that the rainfall events tend to be short, and begin to 'fit' the typical nested storm events only over longer durations i.e. 12-hours or longer;
- The use of a 72-hour storm nested hyetograph for modelling is likely to produce overconservative results. The local rainfall and flow data show that storms are typically less than 12 hours, with a quick response time and sharp 'peak' in the resulting hydrograph; and
- Using a longer duration rainfall event (nested event) to derive runoff in the Utuhina catchment would therefore produce over extreme flows, greater than the rainfall AEP event applied to the model, that are unlikely to occur. The same applies when used to sized attenuation basins.

3.1.7 ASSUMPTIONS

- The Structure Plan (July 2020) has been used as the basis of the modelling assessment and is representative of the potential development.
- Key input parameters (% imperviousness and resultant Curve Numbers) are as described above and appropriate for the level of relative change assessment.

- Subsequent stages after rezoning will work to test the approach utilised to support a Precautionary assessment for Plan Change and work with regulatory authorities through to the Consent Application phase.
- Soakage/Infiltration has not been tested across the whole plan change area. Indications from work carried out on a portion of Sunny Downs site (WSP Opus, 2018) is that the lower terraces have the potential for relatively high soakage rates, but the site does vary. The report contains eight test locations, whose potential soakage rates are all significantly greater than the rates utilised for this plan change assessment. At the very least testing and site-specific design could represent an opportunity to manage and treat stormwater more efficiently and effectively for the development that proposed here. A precautionary approach has therefore been used.

3.2 HYDRAULIC APPROACH

3.2.1 SOFTWARE

The hydraulic assessment was carried out using a variety of methods to enable the determination of impacts across the whole Utuhina catchment.

- Infoworks ICM v9.5, a computational hydraulic modelling software package developed by Innovyze.
- Mike Flood v 2017 Service Pack 2 - a computational hydraulic modelling software package developed by the Danish Hydraulics Institute (DHI).

Both packages are designed to perform one and two-dimensional computational modelling of flow. Both are used throughout New Zealand and extensively used worldwide for modelling urban drainage and open-channel flow and hydraulic structures through rivers.

Both are appropriate for resolving the assessments being undertaken to assess the impact of land use change on the flows through the model and deriving the impacts of development and mitigation on flood hazard. It should be noted significant further improvement has been added to these models since a benchmarking assessment of their suitability to resolve 'typical' flood modelling problems (EA, 2009).

3.2.2 MODEL COVERAGE AND KEY INFORMATION

3.2.2.1 ROTORUA LAKES COUNCIL URBAN STORMWATER MODELS

Rotorua Lakes Council (Council) commissioned the development of two integrated urban catchment models during 2017 to support the understanding of current catchment flooding issues across these two catchments. Figure 2-2 identifies the spatial extent of the Council models representing these two tributary catchments of the Utuhina Stream. The model outlets are their respective confluences with the stream (as described above).

Opus International Consultants Ltd (now WSP New Zealand Limited) undertook the model build and system performance reporting for Catchment 14 (in Infoworks ICM v7.5) and Jeff Booth Consulting Limited (JBCL) delivered similar outputs for Catchment 15 (in Infoworks ICM v6.5).

In the intervening period since these projects were completed, WSP has updated both models to the most recent model version of ICM (v9.5) and amended the modelling of Catchment 15 to be of a similar approach to that in Catchment 14, as shown below:

Table 3-5: Comparison of Catchment models.

Field	Catchment 14	Catchment 15
Watercourse	Otamatea	Mangakakahi
ICM Model version	V 9.5	
2D Area	390 ha	143 ha
Catchment Area		1286 ha
Rural Catchment		866 ha
Urban Catchment		400 ha
Grid	100/25 m^2	
Model approach	Sub-catchments	
Vertical Datum	Moturiki 1953	
Downstream condition (1% AEP 2130) at Utuhina River. Preliminary time-series data with peak (received 20th May)	285.472m (OTAMATEA 4286)	283.958m (MANGAKAKAHI 10232)
Outfall Level	280.965m	280.400m
Runoff surface	Impervious	
Runoff volume type	SCS-CN	
Fixed runoff coefficient	1	
Routing model	SCS	
Routing type	Abs	
Routing value	0.1	
Initial loss type	SCS	
Initial loss value	0	

The modelling has the key hydrological approaches taken from (Opus, 2018), presented in Section 3.1. Further details are available within the report.

2D SURFACE

A 2D mesh surface has been included in the model. It is based on the supplied Digital Terrain Model (DTM). The mesh has the following attributes:

— Min triangle - 25 m^2

— Max triangle - 100 m^2

— Boundary condition – Normal hydraulic condition (where no condition was applied).

This is appropriate for a broad catchment scale assessment of the impacts of changes in the catchment and is in line with studies delivered across New Zealand. Some models in certain areas apply a finer resolution but these tend to be to support the understanding of either smaller geographic areas or of hydraulic issues at specific structures.

SURFACE ROUGHNESS

Table 3-6 shows the range of Manning's 'n' surface values for Land Use based on industry guidance, from (Auckland Council, 2011), further developed in (Capacity (now Wellington Water), 2013).

Table 3-6: Typical 2D Manning's 'n' Roughness Values.

Land Use	Manning's 'n' values
Urban Residential	0.08 – 0.12
Industrial / Commercial	0.1 – 0.5
Roads	0.013 – 0.02
Grass	0.03 – 0.06
Gardens / Dense Vegetation	0.06 – 0.15

A default Manning's 'n' surface roughness of 0.1 has been applied to represent both the residential and upstream rural area with specific roughness zones added as below:

- Road parcels have been imported into the network as roughness zones and assigned a roughness of 0.013.
- Further roughness zones have been digitised manually; open spaces, reserves and the upstream rural area have been assigned a roughness of 0.07, the upper part of the Otamatea stream has been assigned a roughness of 0.15 due the dense vegetation observed on the site visit.

Roughness is a key calibration parameter in 2D modelling and for the purposes of this comparative assessment, the values selected are in line with good practices undertaken across New Zealand.

BOUNDARY CONDITIONS

The Regional Council modelling team supplied boundary condition files from the model for the 1% AEP 2130 events, in agreement with the team these were extended to provide levels for the full 72 hours of the storm event modelled by assuming a recession at a similar rate to the end of the storm (see Figure 3-1).

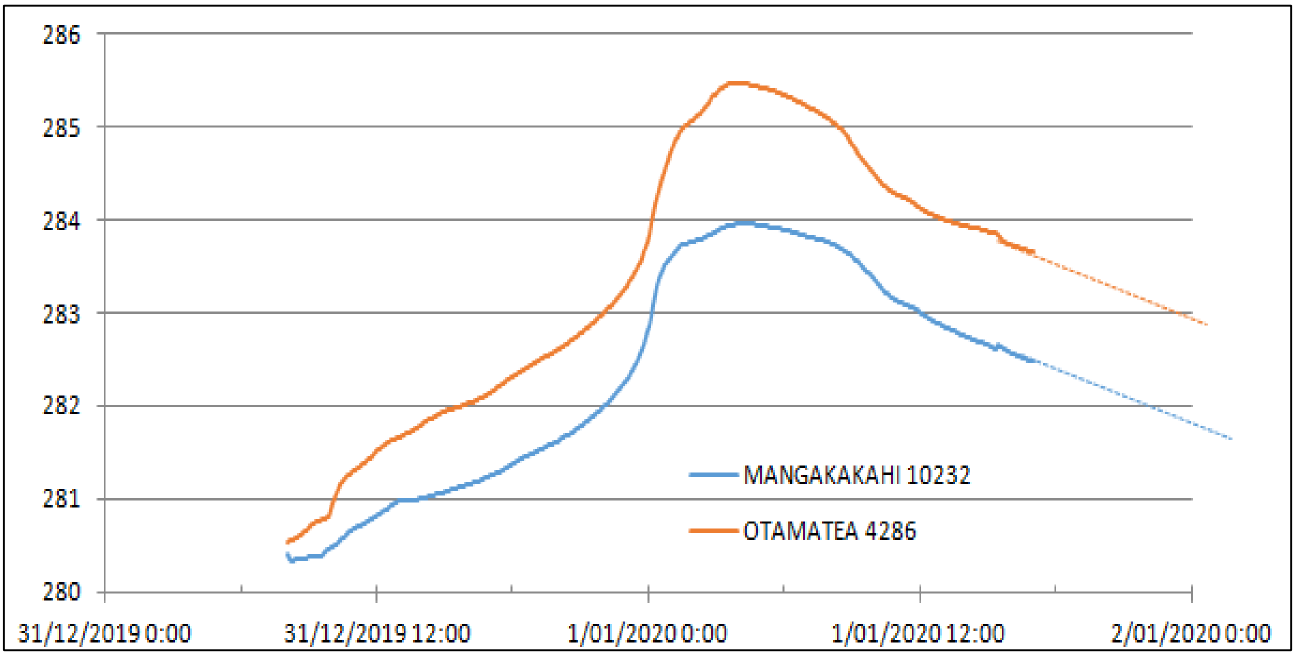


Figure 3-1: Boundary conditions for the 1% AEP 2130 event

DISCUSSION

Although the hydraulic models are largely uncalibrated, it is considered to provide a realistic indication of both the existing flood hazard and the potential effects of the plan change rezoning.

While there may be some uncertainty regarding the precise numbers (i.e. the exact depths and velocities), the relative changes between the different scenarios are likely to be representative

To recognise the uncertainty within the hydraulic model, and the fact that shallow flooding of short duration does not pose a hazard, all areas where the depth of flooding is less than 0.1m have been removed from subsequent analyses/comparisons.

3.2.2.2 BAY OF PLENTY – GREATER UTUHINA CATCHMENT MODEL (GUCM)

Regional Council commissioned the development of an update to previous Utuhina Stream and catchment modelling to support the understanding current flooding issues, shown in Figure 3-2. The total catchment area is in the order of 60 square kilometres. The GUCM became available for use in May 2020 to support the effects assessment.

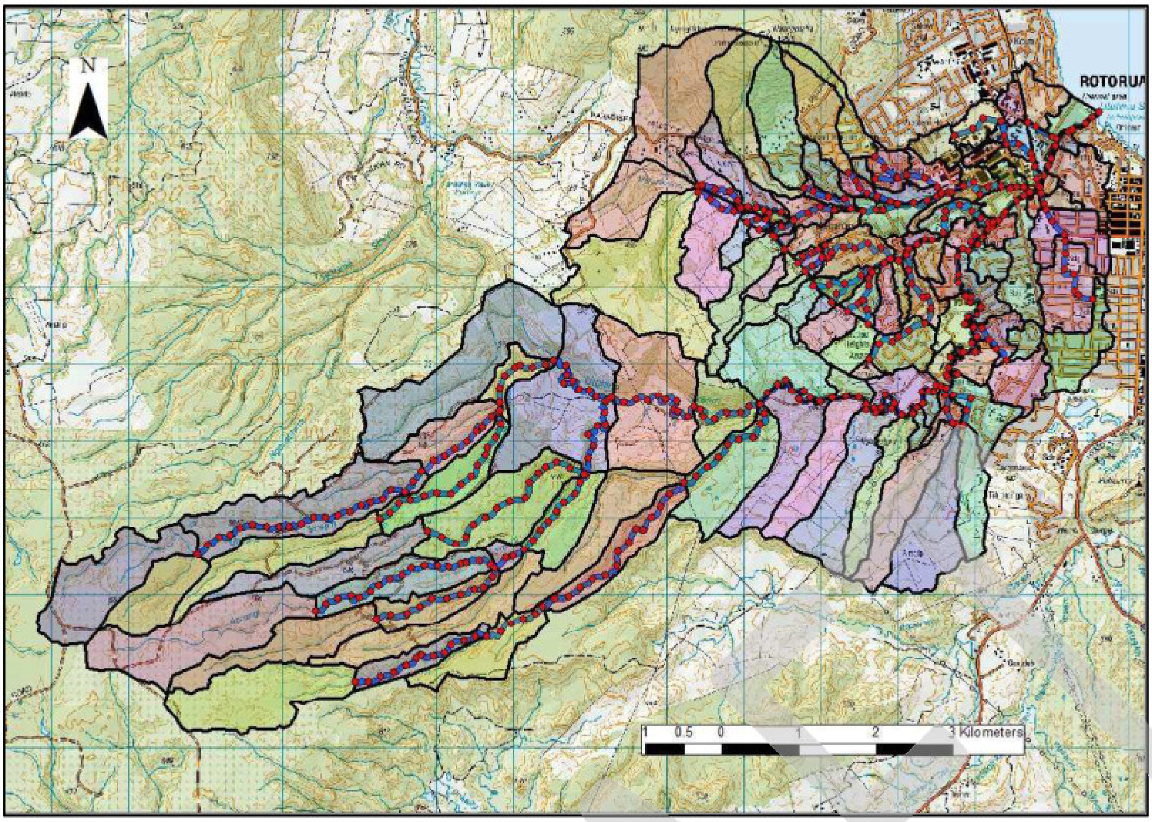


Figure 3-2: GUCM Catchment - Subcatchment, routing branches and nodes

The model consists of two parts – the hydrological modelling to support the creation of runoff for use within the hydraulic model then resolves the detailed relationships between flows, water levels, waterway capacities and storage volumes. The combination of this are outputs that help to determine the impact/effect of the development.

At time of writing, we have received a draft summary of the hydrological modelling establishment approach (West, 2019), identifying that 122 sub-catchments have been created represented the catchment from source through to the discharge into Lake Rotorua. Currently and without access to the database itself, the tool and the 'preliminary calibration' reported seem to be operating in a fashion that is tending towards its suitability for a conservative assessment of hazard within the Utuhina catchment.

At this stage, no documentation on the hydraulic model build or calibration with the existing river level gauge at Depot St, has been supplied to assess the approach and suitability. Given that the Regional Council (during 2019) identified that this model would need to be used to assess the impacts of development on the existing flooding issues in the Lower Utuhina catchment, it is appropriate to assume we are using the best available information at this time.

To recognise the uncertainty within the hydraulic model, and the fact that shallow flooding of short duration does not pose a hazard, all areas where the depth of flooding is less than 0.1m have been removed from subsequent analyses/comparisons.

3.2.2.3 DISCUSSION AND USE

THE COUNCIL MODELS

The Council models were developed to understand the performance and levels of service of the Councils Urban Stormwater network. The models can deliver a catchment level assessment of the flood risk resulting from application of rainfall and an understanding of the below ground network performance. These models have been used to determine the effects of this plan change in the following ways:

- Determination of risk (depth and velocity) from the plan change area through to locations where the GUCM modelling will take preference, and;
- An assessment of the relative effects at key locations throughout the network down to confluence with the Utuhina, including the networks influenced by the performance of the Linton Park structure.

THE REGIONAL COUNCIL MODEL (GUCM)

The GUCM was developed to understand the flooding risk throughout the catchment and to help understand the levels of protection and levels of service for various flood infrastructure through the catchment. The model delivers a catchment level assessment of the flood risk resulting from application of rainfall. This model has been used to determine the effects of plan change through:

- The determination of risk from the plan change area through the whole model 2d Domain, including depth, velocity and duration of inundation.

Where there is discrepancy or overlap in results, between the two models, the Regional Council model will take precedence for consistency and application.

3.2.3 MODEL INTEGRATION

As agreed with the Regional Council, we have identified a series of 'loading' nodes that are used to share modelling outputs from the Council models and feed them into the Regional Council model. We have prepared modelling results that allow for a purely comparative study between the existing conditions and the proposed development as modelled.

To facilitate this, ICM model hydrographs have been shared at these locations for the identified flood events (inclusive of climate change allowances), for the nodes identified in Figure 3-3 below for both the pre-development (current land use) state and the post development with mitigated state. From this point, the Regional Council prepare the hydrological and hydraulic modelling to deliver the results for analysis. Note that some of these would in effect be zero inflow sub-catchments as a result of the topographic changes assumed as part of the Plan Change stormwater management approach and shown in Figure 2-3.

Figure 3-4shares the approximate area that the Regional Council model will not be able to determine the effects of the development, due to the schematisation of the model. In these areas, the Council models will be used.

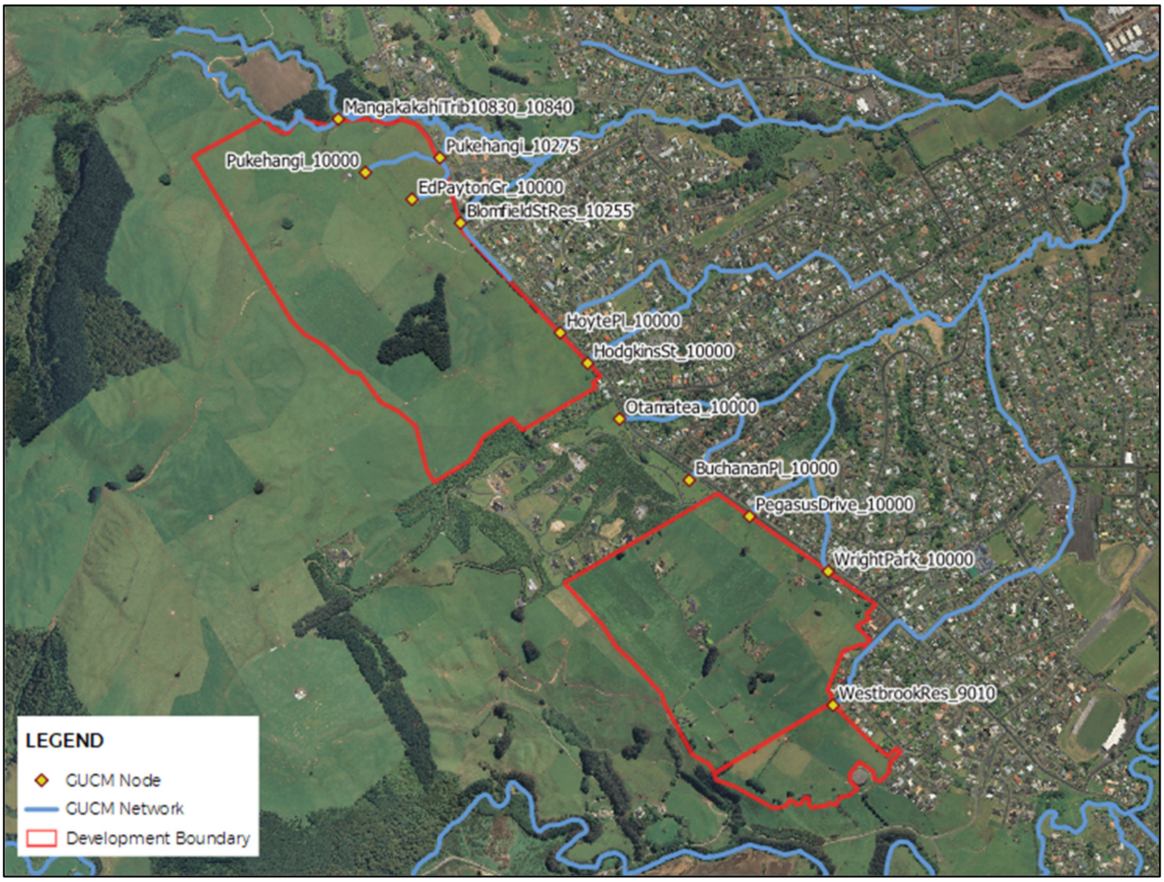


Figure 3-3: Location of the linking (loading) nodes between the two modelling packages.

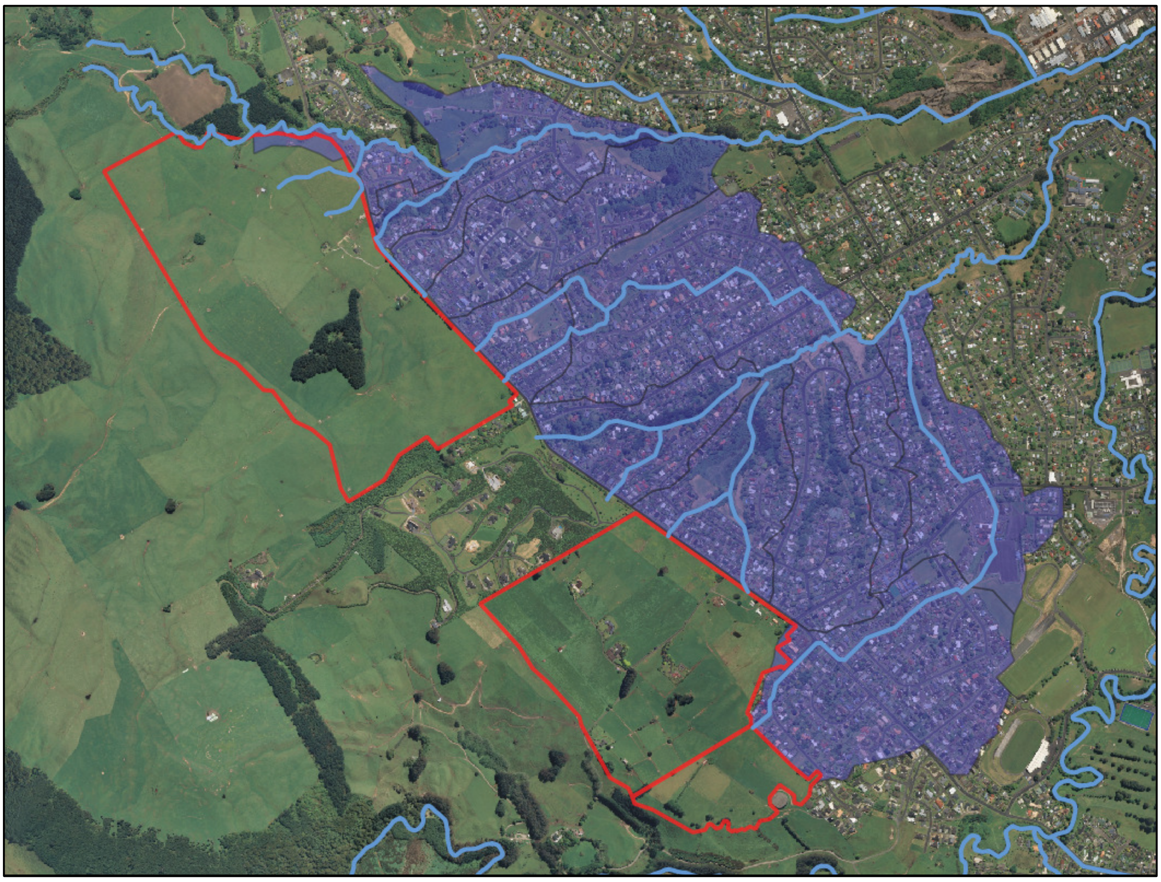


Figure 3-4 – Purple area representing the area where the Council models will share flood hazard impact information.

3.2.4 DISCUSSION ON THE MODELS

A review of the modelling performance in relation to the empirical data presented in Table 3-7, suggest that the models are on the conservative side – i.e. both models have flows that are greater than the flows recorded from the available gauged record (15 years) reviewed. Table 3-8 below shows the peak flows recorded with the GUCM model received from Regional Council consultants in early August 2020. The key findings are that:

- In comparison with the three statistical approaches, based on 15 years of empirical record all the model approaches are likely to be over-estimating the peak flow arriving at the Utuhina gauge in Depot Street.
- It is noted that previous statistical analyses reference a longer record by combining a previous gauge at Lake Road, which may alter this analysis.
- The flow in a river reflects the integration of the spatial and temporal distribution of rainfall, and all those processes that affect the rainfall-runoff relationship within the catchment. These processes may not be constant through time, or with changing magnitude of the rainfall event. Consequently, the frequency distribution of rainfall may be significantly different to the frequency distribution of flow. A rainfall event with a given AEP may not produce a flow with the same AEP. The tables presented below reflect this in that the current modelling approaches identify that the rainfall approach used does not create an equivalent flow return period. With the hydraulic models having flows that are greater than the statistically driven flows based on the record.

- The tables below, where the nested storm approach is producing flows greater than that empirically measured for the period of record reviewed, reflects the aspiration that the effects of development have an account for appropriate and precautionary antecedent conditions (saturated).

Table 3-7: Design flows (m^3/s) for Utuhina at Depot St (2005-2020), rounded to 1dp (see Appendix C for more details)

AEP	Gumbel	GEV	PEV
10%	27.5	27.4	27.2
2%	36.7	33.4	33.5
1%	40.5	35.4	35.9

Table 3-8: Peak Flows (m^3/s) from the GUCM model (located approximately at the Depot St gauge), with the application of rainfall design storms of the following AEP magnitude.

ORIGINAL GUCM BASE				Climate Change - Original GUCM		Climate Change - BASE (with WSP inputs)				Climate Change - Scenario 3 (with WSP inputs)			
10%	2%	1%	0.2%	10%	2%	10%	2%	1%	0.2%	10%	2%	1%	0.2%
28.3	46.2	53.9	71.0	45.2	68.4	51.9	74.4	81.9	93.0	50.0	73.0	81.7	93.2

3.2.5 ASSUMPTIONS

This hydraulic analysis includes the following assumptions;

- LiDAR resolution provides adequate representation of the terrain; The extent of the model provides enough detail of flooding effects in the floodplain, suitable for a relative assessment of the impacts of the development;
- The existing urban stormwater catchment models are at an appropriate level to understand the downstream primary system network performance;
- The GUCM conservatively assesses the flood risk through the catchment (with the identified areas of gaps in coverage resulting from the model schematisation) and presents the best available tool to assess the comparative effects resulting from the land use zone changes proposed for this plan change.

4 MODELLING RESULTS

4.1 MODEL STATUS

The current models and their results are an evolving picture as the consultation between the Council and the Regional Council continues. During the past three months, since receipt of the GUCM., WSP has been working to provide results to support the risk assessment modelling within GUCM. At the time of writing up this section, not all the results are available to support as during August 2020, further modelling amendments have been made to facilitate the hearing.

Therefore, the work below reports on the performance of the latest full set of modelling results, known as 'Bigger Basins' (Scenario 03). This represents a reasonable assessment at this stage and subsequent sections will detail the findings of these modelling outputs and compare them with the inputs used for the more recent simulation (Scenario 14 – post structure plan and revised imperviousness values) to identify the likely outcome of these simulations.

It is recognised that this qualitative assessment is not ideal and that there is some subjectivity to this approach in lieu of quantitative outputs, however to support the process and to allow for the effort to take place, this is deemed an appropriate surrogate for the time being. It is intended that the updated modelling outputs and analyses will be presented to support the subsequent stages.

4.1.1 SCENARIO 03

The information presented above in the Sections 2 and 3 relate to the proposed stormwater management response to Structure Plan Revision M (received in July 2020), the stormwater management concept for which is referred to as Scenario 14. For clarity, Scenario 03 incorporates the work from previous iterations of the Structure Plan. The key differences are:

- The amount of residential area has increased in Scenario 14;
- The basin sizes have been increased on the Structure Plan Rev M (Scenario 14);
- The total amount of area contributing through the basins has decreased (Scenario 14);
- The addition of further road corridors through the Structure Plan Rev M (in Scenario 14);
- Further changes to agreed parameters and imperviousness coverage, following on from further workshops with Regional Council in August).
  - Scenario 14 – uses Table 3-1 values
  - Scenario 03 – uses values as presented in Table 4-1 (representing a more conservative approach, compared to the proposed Plan Change rules)

The calculations to support these changes, have identified that the proposed basin areas identified are appropriate, but that some of the basins would benefit from being marginally deeper (as presented in Table 2-1). The two key elements are that the residential area has been increased with an increased residential imperviousness approach, but the previous cautious modelling approach to include a buffer around each basin helps to balance out this increase.

Table 4-2 identifies the net impacts of total modelled area and weighted CN. Both approaches keep the additional 'smaller' rural catchment (shown in Figure 2-2) above the development site managed through the proposed attenuation structures.

Table 4-1: Curve numbers for sub-catchments – Affected by the potential plan change area ONLY – Scenario 03

Cover Description	Impervious Area (%)	Impervious CN	Pervious Area (%)	Pervious CN	Blended Curve Number
	1	2	3	4	= ((1*2) + (3*4))/100
Rural -Pre-Development	0	98	100	21	21
Rural – Post Development	0	98	100	29	29
Rural 2 Residential - Min. Average lot size 4000 m^2	20	98	80	29	51
Residential 1 – Average lot size 600 m^2	55	98	45	29	66
Residential Medium Density - Average lot size 450 m^2	85	98	15	30	89
Commercial and business	85	98	15	30	89
Streets/roads: sealed	100	98	0	29	98

Table 4-2: Weighted CN for each area

Area Reference	Scenario 03 (**Table 4-1**)		Scenario 14 (**Table 3-1**)	
	Catchment Area (ha)	Weighted CN	Catchment Area (ha)	Weighted CN
1	8.6	47.00	8.2	49.81
2	4.2	67.52	3.9	74.09
3	10.8	56.16	10.5	60.40
4	30.0	59.97	29.4	59.59
5	17.2	60.46	16.2	64.22
6	12.6	56.13	12.3	60.63
7	14.1	55.48	14.3	56.66
8	4.5	64.53	4.0	76.04
9	13.5	67.67	12.7	76.16
10	26.4	62.12	25.5	62.47
11	12.1	51	11.6	42.80
13	7.9	89	7.6	84.2
Totals	**161.9**		**156.3**	

The result of this is that the modelling results that are contained below are now no longer the latest or most appropriate. As discussed, above this is looking to be resolved soon. In the meantime, however subsequent sections share the results from the Scenario 03 reviewed in Section 4.2 onwards would be similar (or worse) to those that are expected to be shown from the Scenario 14 simulation results and analysis.

4.1.2 COMPARISON OF PEAK FLOWS FROM SCENARIO 03 AND SCENARIO 14.

The figures in Appendices A and B, show the outcomes of the model simulations for Scenario 03 (A1 – A13 for the GUCM). The results shared between the two models at the locations identified in Figure 3-3 is presented in Table 4-3 overleaf.

A review of these tables shows that the delta change of peak flows between pre and post is reduced for each of the linking node locations, compared to the results presented in subsequent sections. It is noted that peak flow is not the sole determinant of the effect in a flooding context, but this is used as a proxy for assessing the flooding at this initial phase.

There are more locations in Scenario 14 that have a relative drop in peak flows post development, compared to those for Scenario 3. The percentage change relative to the current scenario is further proof that Scenario 14 has lowered the peak flows from the development both relative against Scenario 03 and the current case, representing reduced peak flows off site for the key design events of the 2% and 1% with a 3.68-degree climate change allowance.

There are marginal increases in the 0.2% peak discharges when comparing Scenario 14 to the baseline, however in comparison to the modelled Scenario 03 results, these peaks are lower.

The assertion that a lower peak flow will result in a benefit downstream is a fair one given that within the current modelling setups, the peak flows and top water elevations are driven almost entirely by the intense shorter duration events within the larger 'nested' rainfall profile. The catchment is saturated, with most basins 'filling or filled before the peak of the storm event occurs.

Therefore, any reduced peak flow would inevitably have a reduced peak water level as this event flows through the catchment. This is particularly the case for the two tributaries of the Mangakakahi and the Otamatea Streams, however it is not certain that this would hold true for the lower Utuhina area where the volumetric impacts in the areas were flooding is already experienced.

However, the GUCM modelling identifies (in Figures A2, A5 and A8) that the benefits from the proposed mitigation measures on the urban primary and secondary stormwater networks, reduce the further downstream you are. The green shading becomes less dark tending to no net change in peak water levels in the lower part of the catchments as other tributaries take up the available capacity to reduce surcharging levels.

The above therefore suggests that the subsequent analysis is still valid and will be similar in terms of the effects identified, in that reduced peak flows will support an upper catchment benefit in the 1% events. For the 10% events, the benefits are more widespread than the 1%.

For the more extreme event modelled, the 0.2% AEP event, has parts of the catchment benefit whilst others showing an effect, by the proposed approach to concentrate flows into certain parts of the catchment.

Table 4-3: Comparative results for the 'loading' nodes between the two models (discharge in m³/s). Percentages identified relates to the percentage change from the Base state.

Event	Scenario	MangakakahiTrib 10830_10490	New Area 11	Amended Rural 33	Pukehangi_10275	Pukehangi_10000	EdPaytonGr_10000	BlomfieldStRes_10255
2%	Base*1	17.6	0	0	2.1	1.4	0.4	4.6
	3	12.9	1.1	4.8	2.3	0	0	3.4
	% Change	73%	Inc	Inc	110%	0%	0%	74%
	14	12.9	0.2	4.7	0.5	0	0	2.6
	% Change	73%	Inc	Inc	24%	0%	0%	57%
1%	Base*1	21.8	0	0	2.4	1.8	0.5	5.7
	3	15.9	2.2	5.8	0	0	2.6	7.7
	% Change	73%	Inc	Inc	0%	0%	520%	135%
	14	15.9	0.6	5.8	1	0	0	4.1
	% Change	73%	Inc	Inc	42%	0%	0%	72%
0.20%	Base*1	33.1	0	0	4	2.6	0.6	8.6
	3	24.2	3.9	9	3.3	0	0	18.6
	% Change	73%	Inc	Inc	83%	0%	0%	216%
	14	24.2	2.8	8.9	3.1	0	0	14.8
	% Change	73%	Inc	Inc	78%	0%	0%	172%
Event	Scenario	HoytePl_10000	HodgkinsSt_10000	Otamatea _10000	BuchananPl_10000	PegasusDrive_10000	WrightPark_10000	WestbrookRes_9010
2%	Base*1	1	1.9	1.5	0.5	2.1	2.1	0.5
	3	0.3	1.7	1.5	0.5	0.8	0.7	0
	% Change	30%	89%	100%	100%	38%	33%	0%
	14	0.3	1.7	1.5	0.5	0.5	0.6	0
	% Change	30%	89%	100%	100%	24%	29%	0%
1%	Base*1	1.2	2.3	1.9	0.7	2.6	2.6	0.6
	3	0.3	2.1	1.9	0.7	1.6	1.6	
	% Change	25%	91%	100%	100%	62%	62%	0%
	14	0.3	2.1	1.9	0.7	1.2	0.7	0
	% Change	25%	91%	100%	100%	46%	27%	0%
0.20%	Base*1	1.7	3.6	2.9	1	3.9	4.1	0.9
	3	2	3.2	2.9	1	4.9	5.8	0
	% Change	118%	89%	100%	100%	126%	141%	0%
	14	1.8	3.2	2.9	1	4.4	3.7	0
	% Change	106%	89%	100%	100%	113%	90%	0%

*1 Base V2 Results shared – additional overland flow paths identified at HoytePl_10000, PegasusDrive_10000 and WestbrookRes_9010 not previously discovered or shared at time of Scenario 03 – Incorporated into Scenario 14 simulations pre and post development (currently underway with Regional Council).

4.2 DOWNSTREAM EFFECTS – URBAN STORMWATER NETWORK.

4.2.1 PERFORMANCE OF PRIMARY NETWORK

The existing Council reticulated (primary) stormwater network serves the existing suburbs through a combination of below ground piped networks, connecting through the urban environment through to open channels and natural, ephemeral streams that traverse private properties. As such, it is important to test and understand the impact of the proposals on the primary networks as well as the overall flooding.

Figure 4-1 and Figure 4-4 below demonstrate that the proposed development provides for a decrease in the overall catchment peak stormwater depths at nodes through the catchment. There are areas that show a minor increase in the peak water levels at these points along the upper branch of the Mangakakahi Stream, above the Linton Park Detention Basin, however there is no discernible pattern to where these increases/decreases occur.

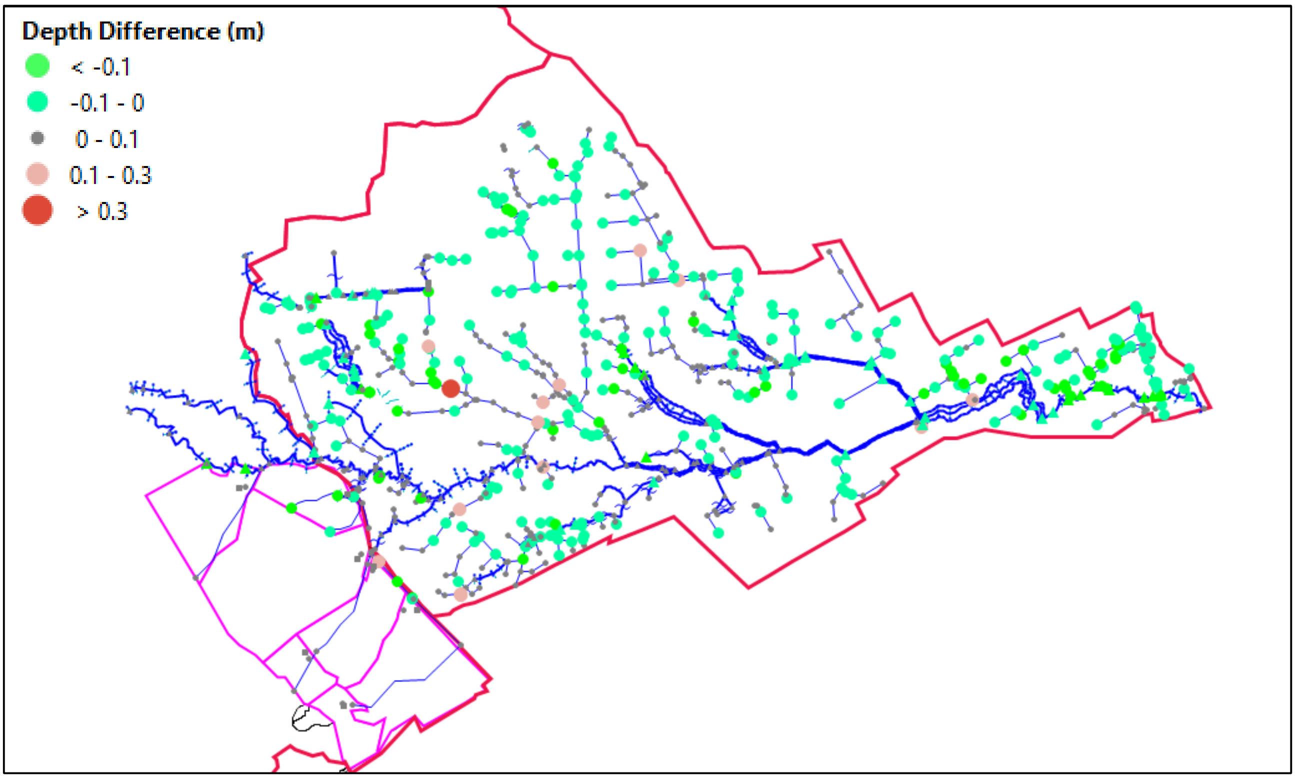


Figure 4-1: Difference map – Post development (Scenario 3) minus pre-development 1% AEP plus climate change event – Mangakakahi Stream (Catchment 15)

Reference to Table 4-3 identifies that Scenario 3, the 1% AEP +CC event, has an increased peak flow of 2m^3/s through the branch (identified by the node BlomfieldStRes_10255 on Figure 3-3). This is predominantly the result of capturing all the predevelopment flow that used to be distributed along Pukehāngī Road within the Sunny Downs development area and concentrating it into the three key proposed attenuation basins (3, 4 and 5) to discharge safely into the gully.

This represents a negative effect along this gully and a positive effect elsewhere for the Scenario 03 approach, with peak water levels and velocities increased compared to the base case.

At this stage, please note that there are no overland flow paths identified on the image for the Plan Change area due to the modelling approach and as development plans would shift the current landform.

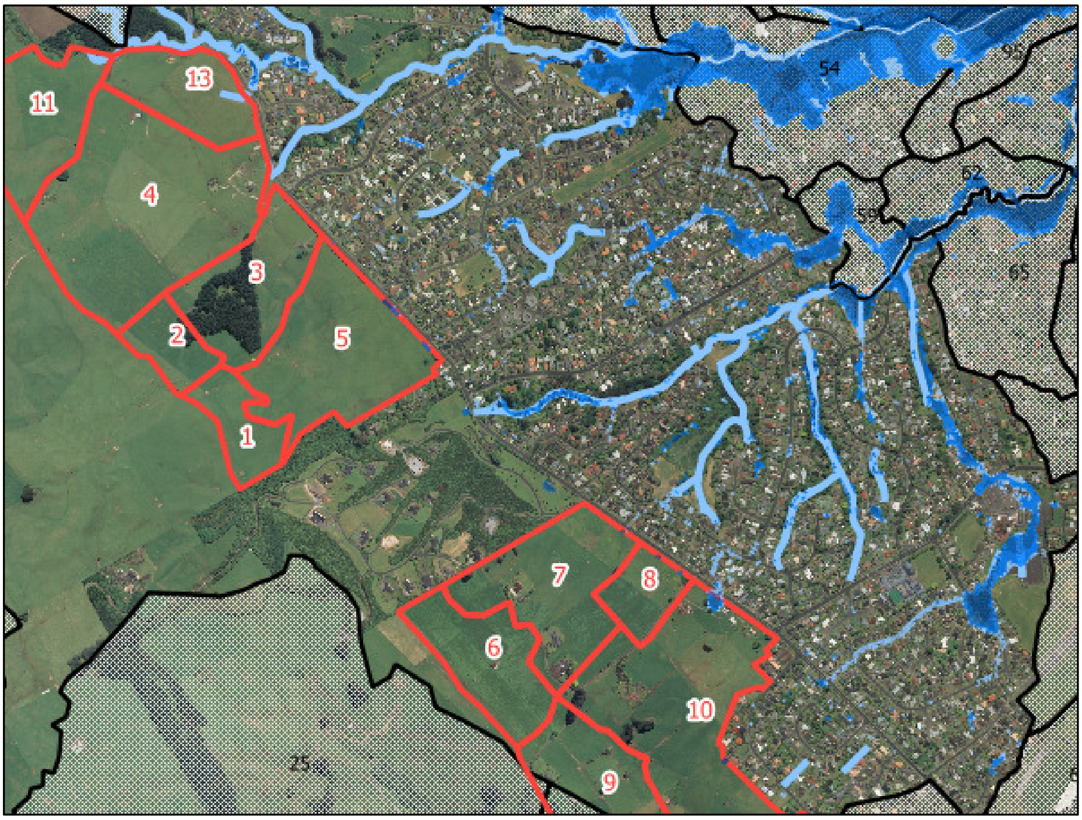


Figure 4-2: Flood depth map (Pre-development) 1% AEP plus climate change.

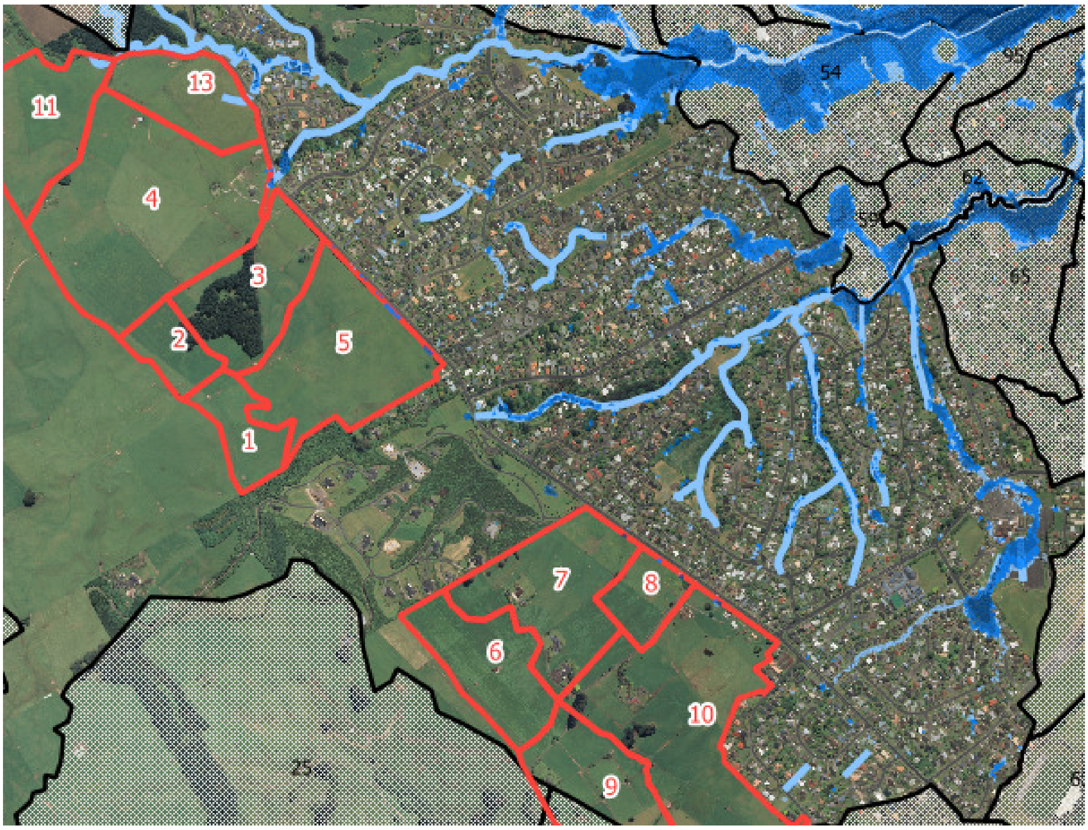


Figure 4-3: Flood depth map (Scenario 3) 1% AEP plus climate change

Scenario 14 improves the peak flow performance throughout the model further (see Table 4-3) and would help to deliver further reduced water levels across the catchment nodes and hence improve the post event flooding shown in Figure 4-3. BlomfieldStRes_10255, has a reduced peak flow in comparison to the current condition. The flow differences are reversed in comparison to the Scenario 3 reported above.

From a review of previous simulations of the GUCM, this reduced peak flow into the gully would assist in reducing the water levels and velocities further in these areas to help deliver an overall benefit to the catchment downstream in relation to the peak water levels resulting from this specific design event simulation. The Otamatea Stream shows a similar, if not lower benefit in peak water levels at nodes across the catchment.

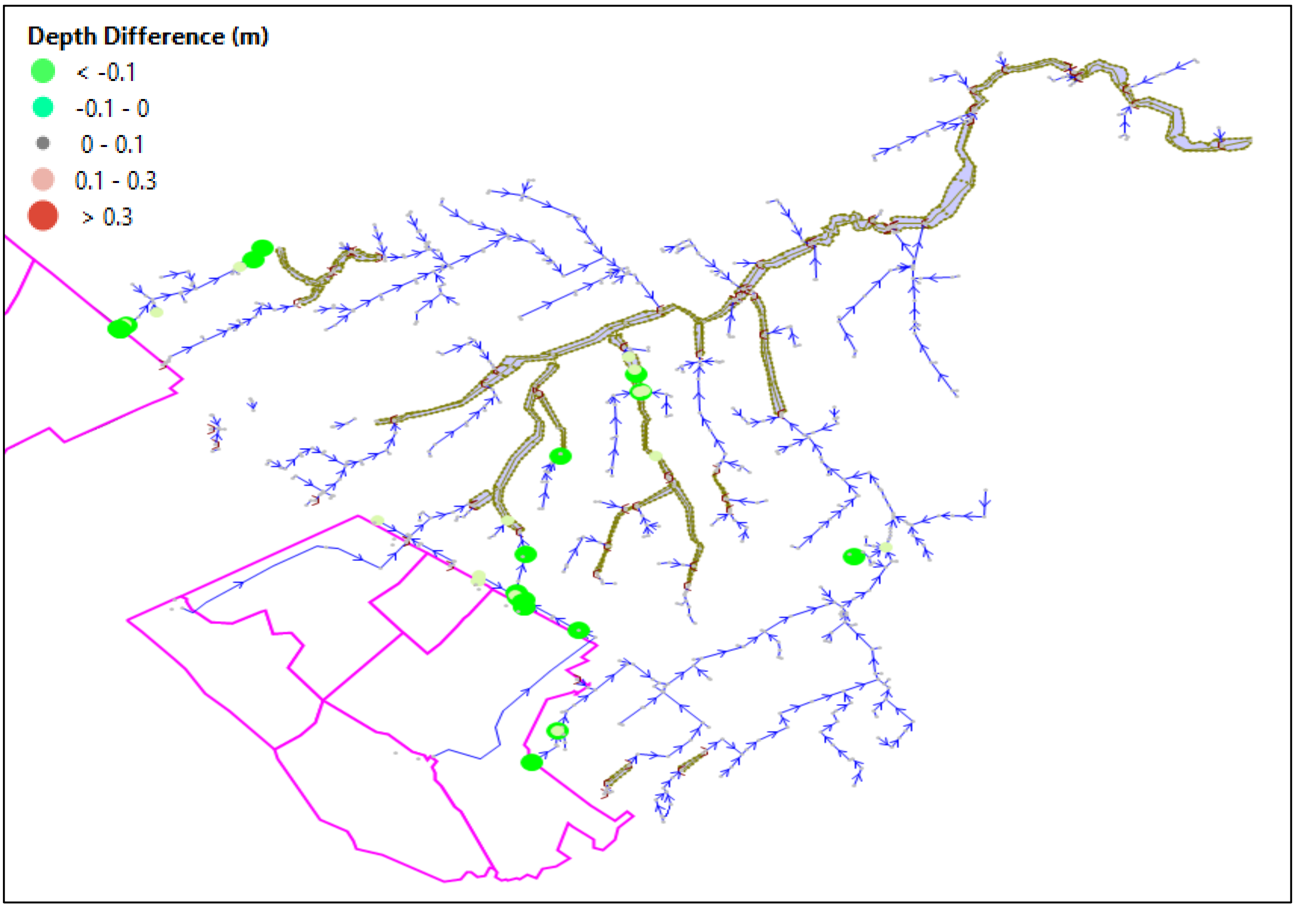


Figure 4-4: Difference map – Post development (Scenario 3) minus pre-development 1% AEP plus climate change event – Otamatea Stream (Catchment 14).

Delivery of upstream attenuation storage will affect the resultant hydrograph through the downstream network. The above figures show that the peak water levels are reduced through the approach taken to have large 'dry' basins to manage the stormwater impacts of the plan change development area.

In relation to the design events simulated for this plan change, the broad impacts are shown through comparing the flow hydrographs at the locations shown below in Figure 4-5.

The flow rate will remain elevated compared to the existing condition for a longer period. This effect can't easily be resolved without managing the volumetric impact of the development through soakage or other means.

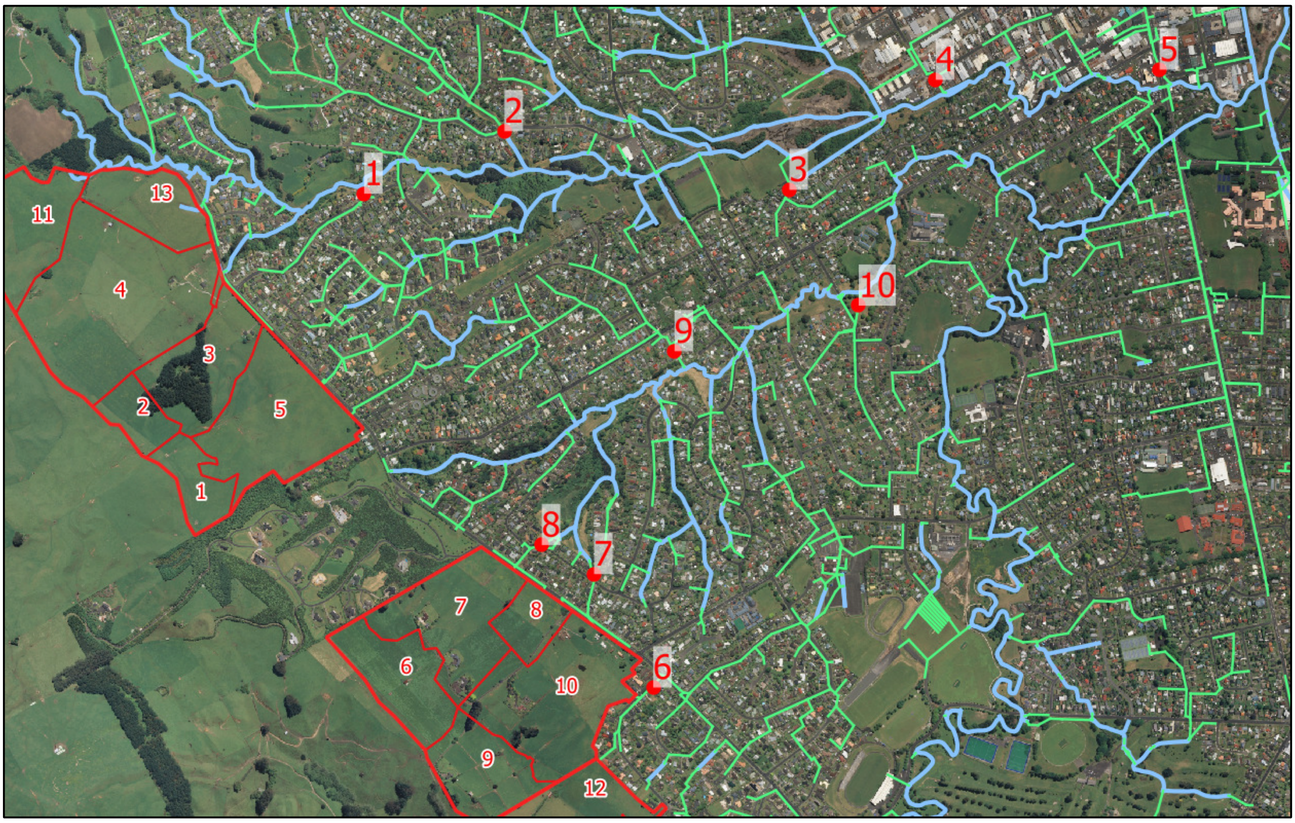


Figure 4-5: Selection nodes for comparison of flow performance in the primary stormwater reticulation network.

The following graphs show the flow hydrographs along a pipe at the approximate locations identified above. They relate to the Scenario 3 and existing 1% plus climate change (+CC) simulations. The flow graphs are presented in order and a short discussion of the results is presented after the set.

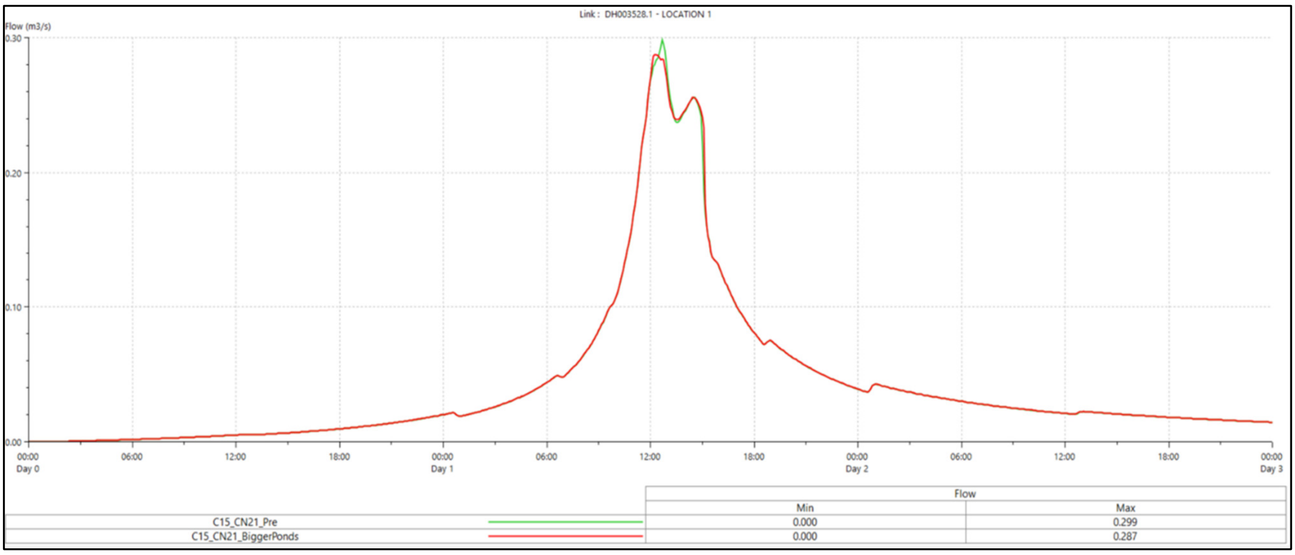


Figure 4-6: Mangakakahi - Location 1 – Comparison of flows within the primary stormwater network.

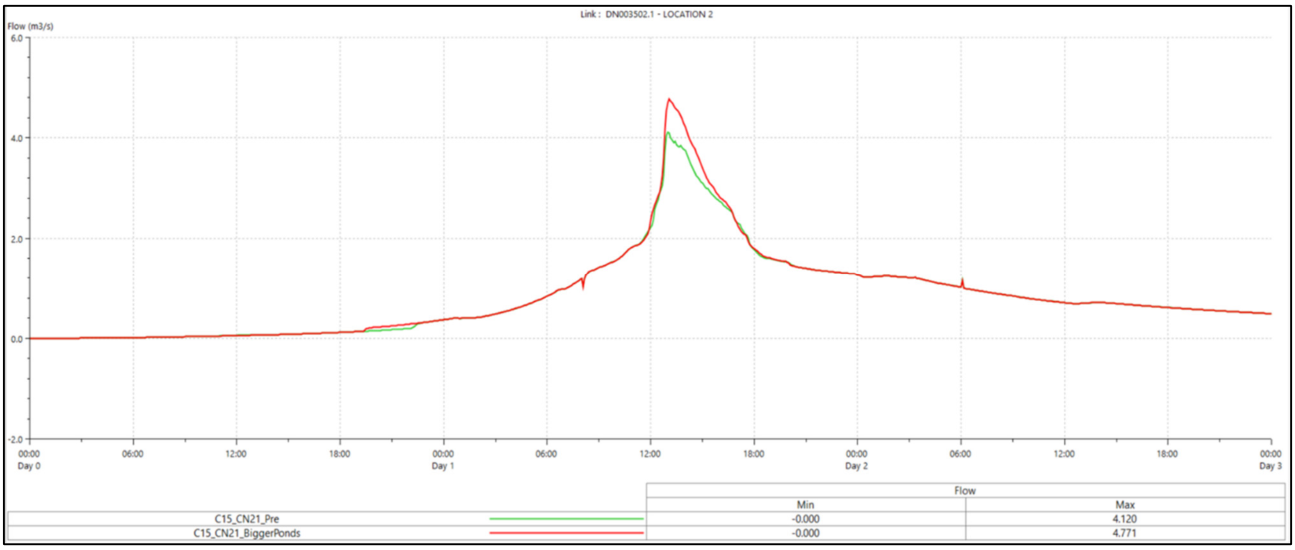


Figure 4-7: Mangakakahi - Location 2 - Comparison of flows within the primary stormwater network.

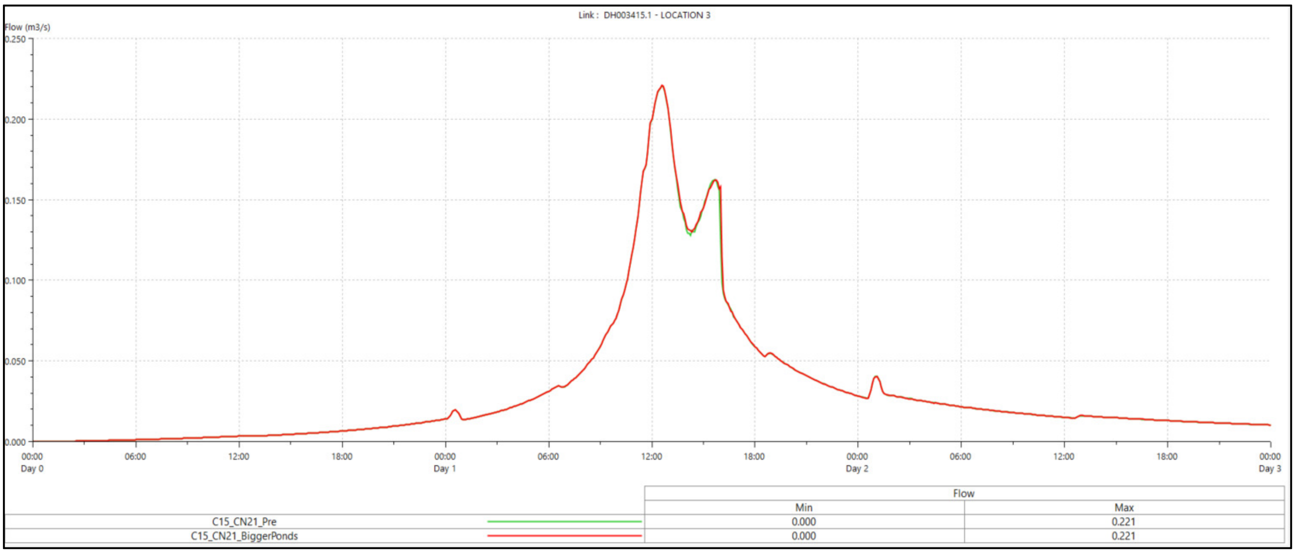


Figure 4-8: Mangakakahi - Location 3 - Comparison of flows within the primary stormwater network.

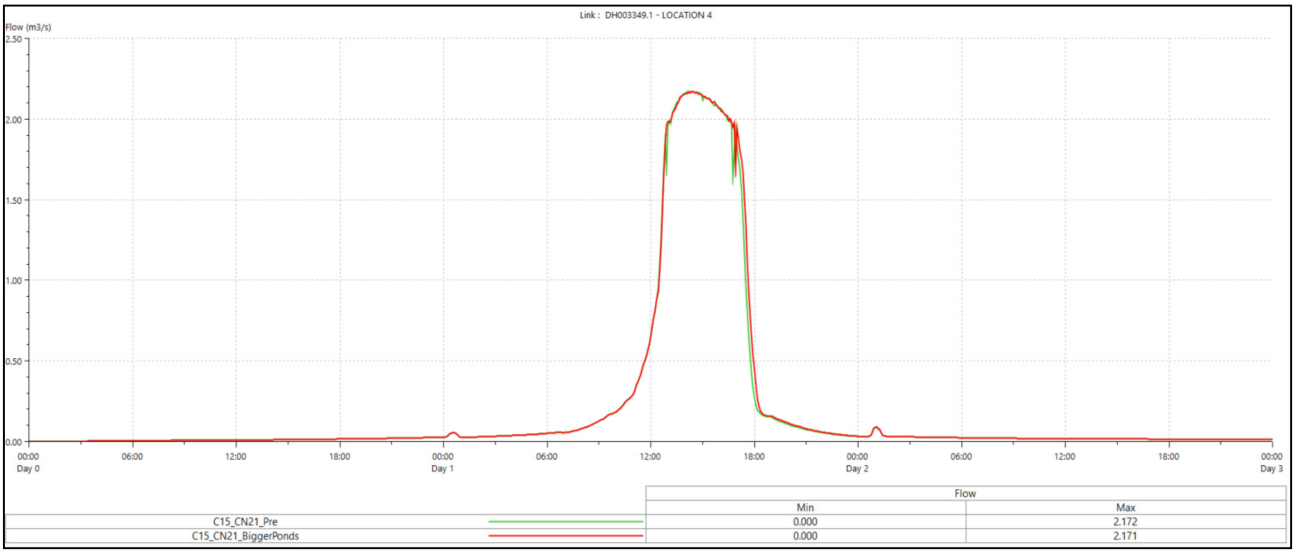


Figure 4-9: Mangakakahi - Location 4 - Comparison of flows within the primary stormwater network.

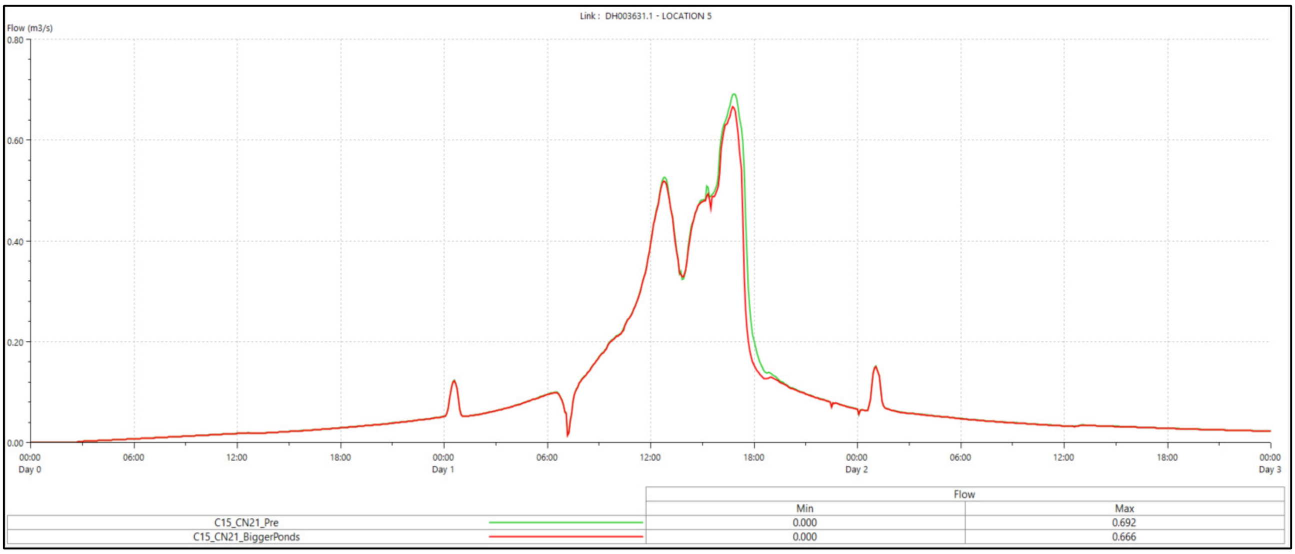


Figure 4-10: Mangakakahi - Location 5 - Comparison of flows within the primary stormwater network.

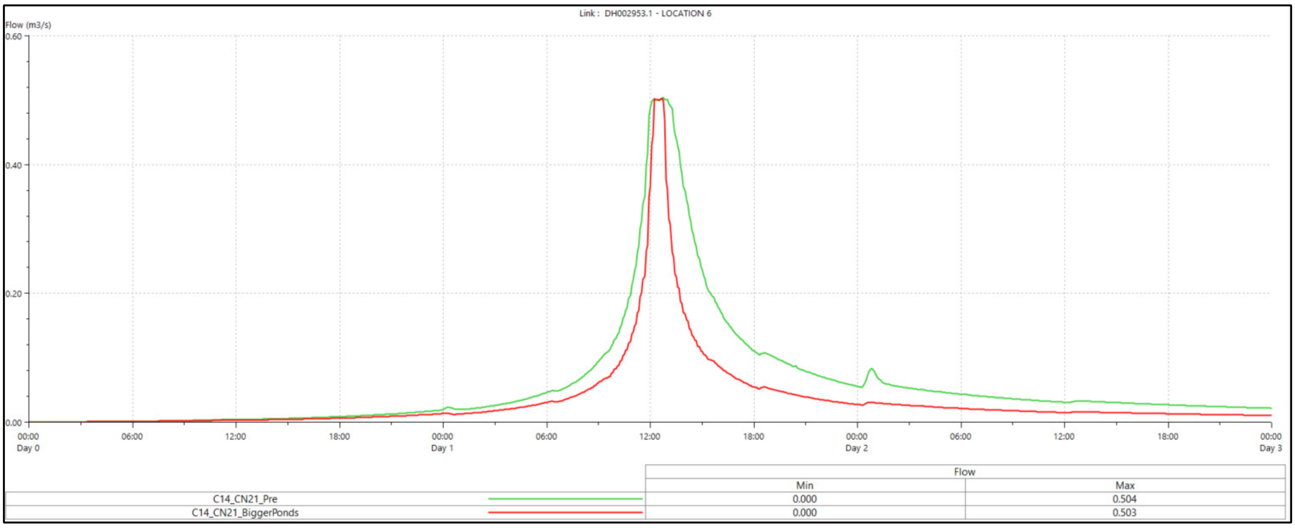


Figure 4-11: Otamatea – Location 6 - Comparison of flows within the primary stormwater network.

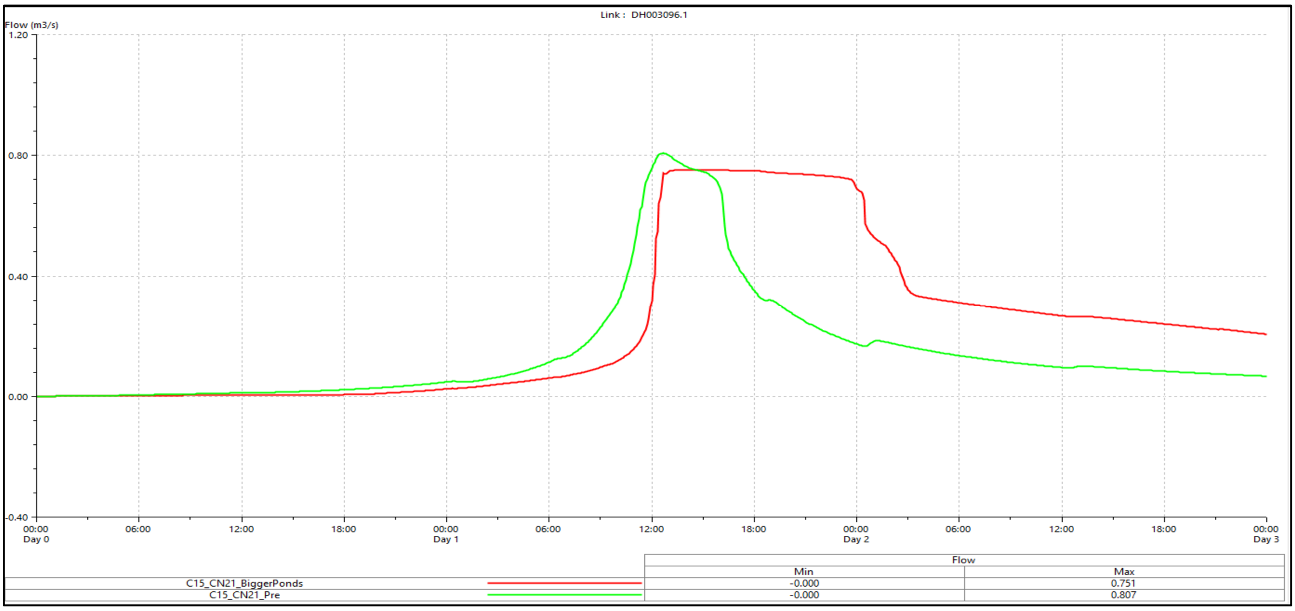


Figure 4-12: Otamatea – Location 7 - Comparison of flows within the primary stormwater network.

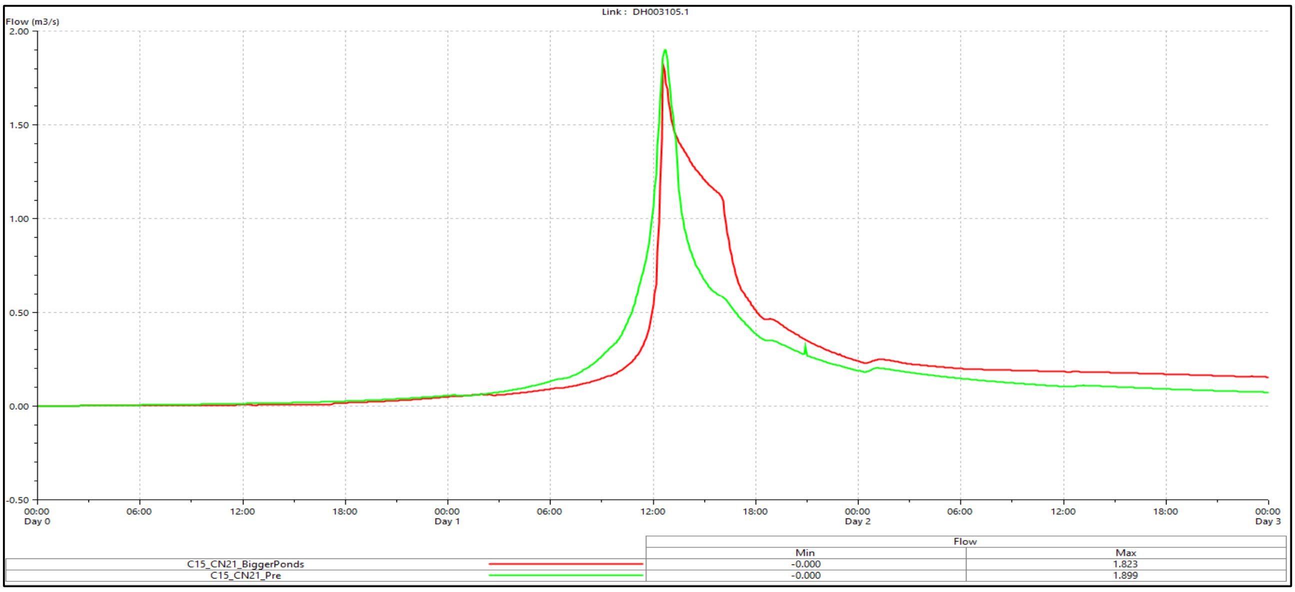


Figure 4-13: Otamatea – Location 8 - Comparison of flows within the primary stormwater network.

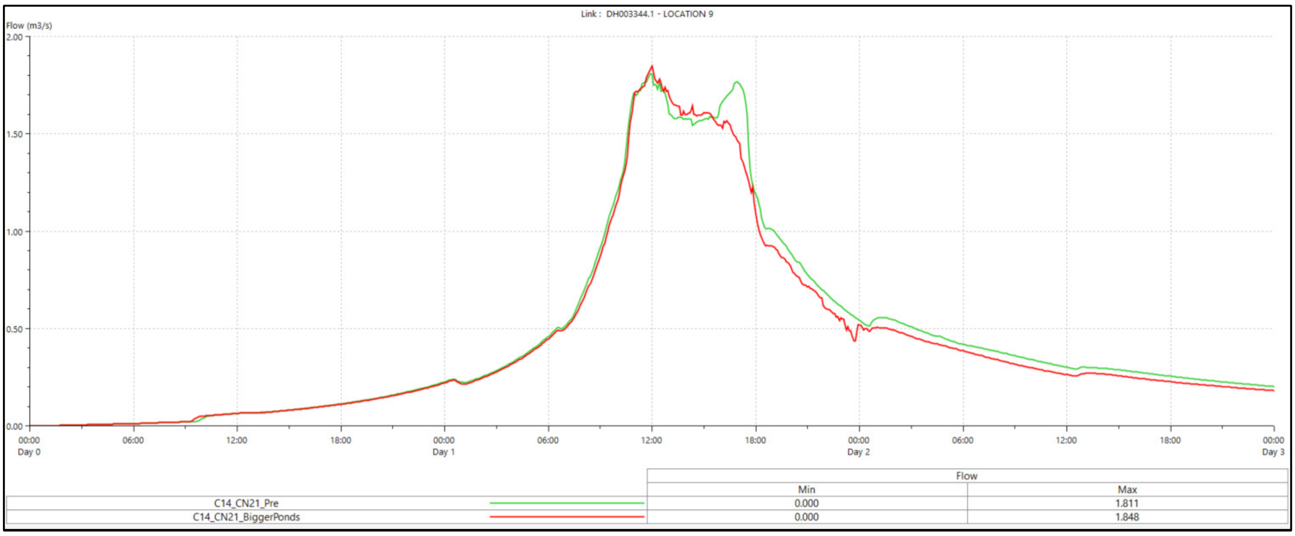


Figure 4-14: Otamatea – Location 9 - Comparison of flows within the primary stormwater network.

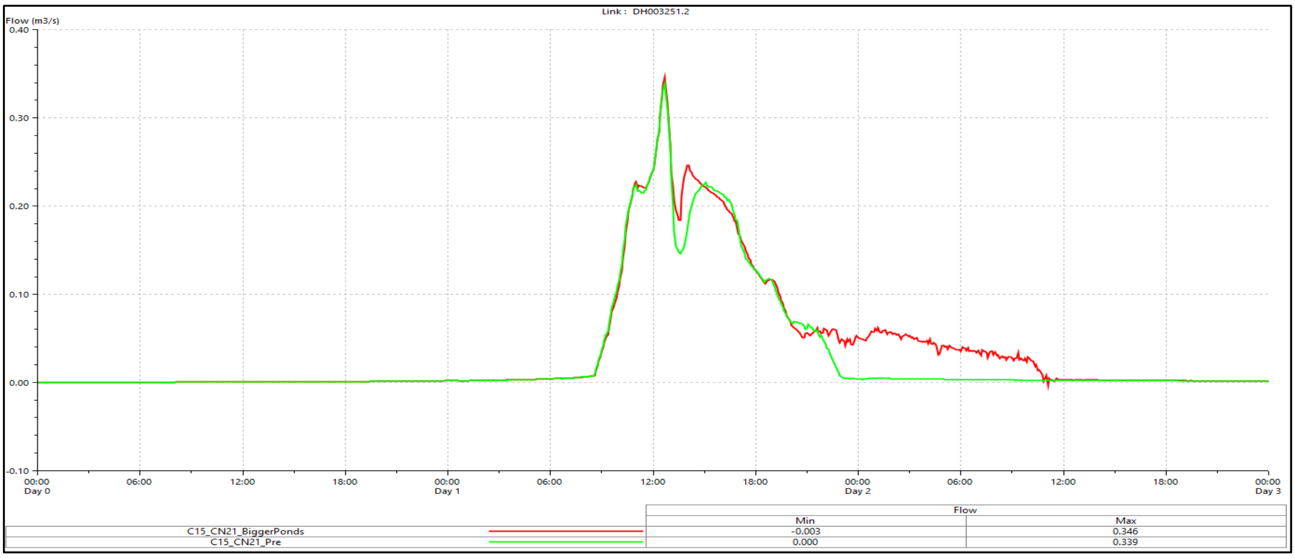


Figure 4-15: Otamatea – Location 10 - Comparison of flows within the primary stormwater network.

A review of the figures above, show that the primary network can both temporarily benefit and effect the flow regime underground. A review of these suggest, in line with Figure 4-1 and Figure 4-4, that the peak water levels can be positively impacted. This reduction in the peak water level, enables the side branches of the urban network to increase their peak (see locations 2, 9 and 15 in Figure 4-7, Figure 4-14 and Figure 4-15 for examples of this).

Nearer the development the effects on the flow regime within the primary stormwater network are more marked as would be expected, locations 1, 6, 7 & 8 (Figure 4-6, Figure 4-11, Figure 4-12 and Figure 4-13) show these effects in different ways.

- Location 1 (Figure 4-6) shows a side branch into the Mangakakahi sufficiently close to the plan change site to show a slightly minor reduction in peak flow from the stormwater network as flow is constrained from being released into the channel through the altered flow regime in the channel. The effect is short lived and doesn't affect the flow in any other way as shortly after the peak the post development flow is slightly raised to release the same volume;
- Location 6 (Figure 4-11) shows the positive impacts of rerouting the urban network from the TAGH land through the Hunts Farm, thereby reducing peak flows and volumes through the network in Matipo Ave (as one potential route for stormwater disposal).
- Locations 7 and 8 (Figure 4-12 and Figure 4-13) show the effects of the basins retaining the additional development volumes. The peak flows are reduced through the basins, but the flow is elevated after the peak has passed for some time. The key impacts being that flow is elevated at 50% of the peak flow for 15 hours (Location 7) and 6 hours (Location 8). This could have an impact on the primary stormwater network should further rain events happen in the subsequent 48 hours as the basins seek to drain down.

Locations 3 and 4 (Figure 4-8 and Figure 4-9) show no change in performance relative to the base event. These two locations are sufficiently affected by the presence of the Linton Park Detention basin. This attenuation or flood storage area is the main influence on network performance in and around the Park area. Further discussion of the basin itself is included below in Section 4.2.2 but a review of the primary stormwater network that discharge into the park area and downstream through the industrial estate suggests that there is no net impact on the primary system performance.

It is recommended that future stages of the development process seek to minimise the impact of this elongated flow through appropriate design and control, such that the basins are sized to drain down within industry good practice guidelines to be as available for subsequent rainstorms. The designers are urged to consider the storage efficiency of each of the 'dry' basins to deliver effective risk mitigation downstream.

As such, it is recommended that a specific Stormwater Management Plan for the proposed development is delivered, that identifies any changes in runoff characteristics generated from the resulting development layouts or change in land use and propose measures to mitigate the effects, as identified in the Plan Change[1] documentation supplied.

This could identify the specific approaches that could help to minimise the period of elongated flow, through soakage or further amendments to the approaches for the hydrological parameters

[1] Performance Standard - A5.2.3.4.7 (Residential Zone) and A5.2.4.4.4 (Rural 2 Zone)

for design, the overall form and density of the proposed plan change, specific targeted revegetation approaches throughout the development or delivery of leading water sensitive design approaches.

4.2.2 PERFORMANCE OF THE LINTON PARK DETENTION BASIN

The Linton Park Detention basin is located on the Mangakakahi downstream of the proposed plan change area (see Figure 1-1). The basin provides flood attenuation for areas downstream within the Mangakakahi and Utuhina stream catchment, by holding flow back from the catchments upstream.

In a 1% AEP +CC event, the park is inundated with water for a period of between 6 and 90 hours. The modelling information presented from the GUCM, shows that there are depressions within the park that once inundated would be wet for longer than 72 hours for such an event.

The modelling in relation to Scenario 3 suggests that the water is present for up to 2 hours longer in areas that predevelopment was largely inundated for a period of between 12 – 24 hours. This can be seen with reference to Figure 4-16 and Figure 4-17 shows the results from the GUCM modelling.

Figure 4-16: Time (in hours) of inundation for the 1% AEP +CC event

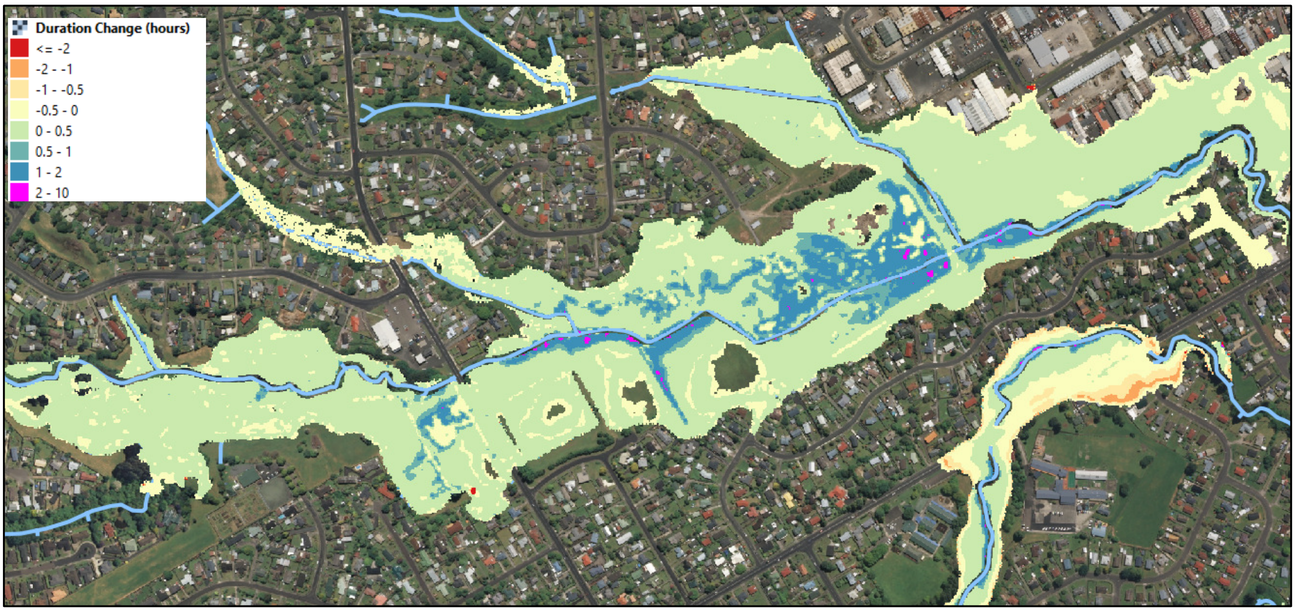


Figure 4-17: Change in time (in hours) of inundation for the 1% AEP +CC event

Relatively speaking this increase in inundation time would not cause additional issues in relation to the ability for the vegetation to survive. Good practice seeks to keep vegetation saturated for no longer than 3 days. An increase of between 5 and 10 % of time on top of the 24 – 36 hours of inundation is not enough in itself to represent a detrimental impact to the park's performance or aesthetics and therefore would not present too many concerns at this stage.

Similarly, it is my opinion at this stage, although I am not a geotechnical engineer, that there are no additional concerns in this relatively small increase in duration of time compared to the 24 hours plus of inundation with water adjacent to the detention bund.

The location of these longer differences (between 0.5 and 2 hours) are adjacent to the Mangakakahi Stream itself, through the park and towards the lower lying areas at the centre of the park, lying within the Councils land ownership. The primary stormwater network that discharges through these areas (to the west, north and south of the basin) are serving areas that are largely above 288 m RL.

For the 1% AEP event, the modelling identifies that the water level in the park is c 287.7m RL and is above 287.5mRL for approximately 3 hours. As Figure 4-8 shows for one of these branches discharging into the Park the impacts of the development and the extended inundation period across parts of the basin are not affecting the networks ability to serve the surrounding land uses as is currently the case.

At this stage, it is perceived that Linton Park detention area may be assisting the development in mitigating it's impacts. This is a difficult assessment to deliver in all reality due to the many complexities involved, however a review of the difference mapping produced from the GUCM model for the Scenario 3 work, suggests that this is perhaps not a critical issue.

Appendix A figures A2, A5, A8 and A11 show the differences between the post and pre development modelling, showing that the peak water level benefits generally erode as you travel further down the catchment. Should the Linton Park detention basin be further providing a benefit to the downstream environments, then one might expect to see the benefits further translated downstream.

Except for the 10% result (A11), this is not showing to be the case, suggesting that the detention basin here is overtopping (the DEM shows the bund has a low spot approximately at 287.2m RL). As a result, the upstream networks are the areas that have the greatest impacts and the wider catchment effects/contributions then override as more tributaries and urban networks affect the flow characteristics.

During August 2020, a query has been raised as to the impact of the existing urban area being developed out to the limits allowed within the District Plan and the impact that this future baseline condition would have on the potential effects from the plan change in relation to the catchment risk profile and performance. Work has recently commenced on this scenario and it is expected that this would be tabled either in advance of the expert caucusing in late August 2020 or prior to the hearing.

4.3 DOWNSTREAM EFFECTS – FLOODING

4.3.1 CATCHMENT FLOOD MAPPING

4.3.1.1 OVERVIEW

The hydraulic computational models described above were run for the three climate adjusted events listed in Table 3-3 for both the existing land use and the plan change land use proposals.

4.3.1.2 WATER LEVELS & EXTENT

GREATER UTUHINA CATCHMENT MODELLING

Figures A-1, A-4, A-7 and A-10 show the peak water level and extent during the flood flows for the three design events and the 10% AEP event for the existing state – the baseline case.

Figure A-2, A-5, A-8 and A-11 show the differences in peak flood depths resulting from the proposed development and the mitigations identified, noting that the simulations received to date incorporate the settings identified in ***Scenario 03***.

COUNCIL MODELS

Figures B-1, B-4, B-7 and B-10 show the peak water level and extent during the flood flows of the three design events and the 10% AEP event for the existing state – the baseline case.

The outputs from the Council models are at this stage more difficult to deliver a difference map due to the way the software is configured. To create the post development state within the model, requires you to amend constituent parts of the model. Once this is done, you need to re-mesh your ground surface.

This results in ground surfaces that are subtly different for each 'scenario' created, making comparisons a little hard to achieve and these have not been completed for this reporting. Figures B2, B3, B5, B6, B8, B9, B11 and B12 will be available in advance of the hearing.

4.3.1.3 FLOW VELOCITY

GREATER UTUHINA CATCHMENT MODELLING

Figure A-3, A-6, A-9 and A-12 show the differences in peak flood velocities resulting from the proposed development and the mitigations identified, noting that the simulations received to date incorporate the settings identified in ***Scenario 03***.

4.3.2 DISCUSSION

For the purposes of the plan change assessment, a precautionary approach has been taken in consultation with the Regional Council. The results presented here represent the development Scenario 3, this is in the process of being superseded with Scenario 14.

Table 4-3 presents the changes in peak flows for the key model exchange nodes and should help to identify that there is the expectation that the performance of the Scenario 14 models would be at least similar if not better than that presented in the figures in Appendices A and B, at mitigating the effects of the development (and in the lower return periods) and reducing peak flood levels.

The Scenario 3 results as presented in the Appendix A show the general trend that the development with the mitigation approaches taken can identify that the plan change can be carried out in a way that avoids increasing peak water levels downstream and can provide some wider catchment benefits. An initial analysis in line with (BOPRC, 2014) has not be fully completed as a complete picture of the hazard is not fully available through utilising the GUCM model alone. The Council models will have completed the additional modelling and mapping to fill the area identified in Figure 3-4, where the GUCM model has no current coverage, in advance of the hearing.

4.3.2.1 2% AEP SIMULATIONS +CC

Reviewing the results for the GUCM, for the more likely event of the 2% AEP event (Figure A2), shows that the water levels are decreased throughout the whole catchment as a result of the proposed plan change stormwater approach.

4.3.2.2 1% AEP SIMULATIONS +CC

For the 1% event (Figure A5), the results show for the components of the GUCM, the post-development state improves the downstream levels less than the 2% event. The benefit is also only really felt in the Mangakakahi and Otamatea streams.

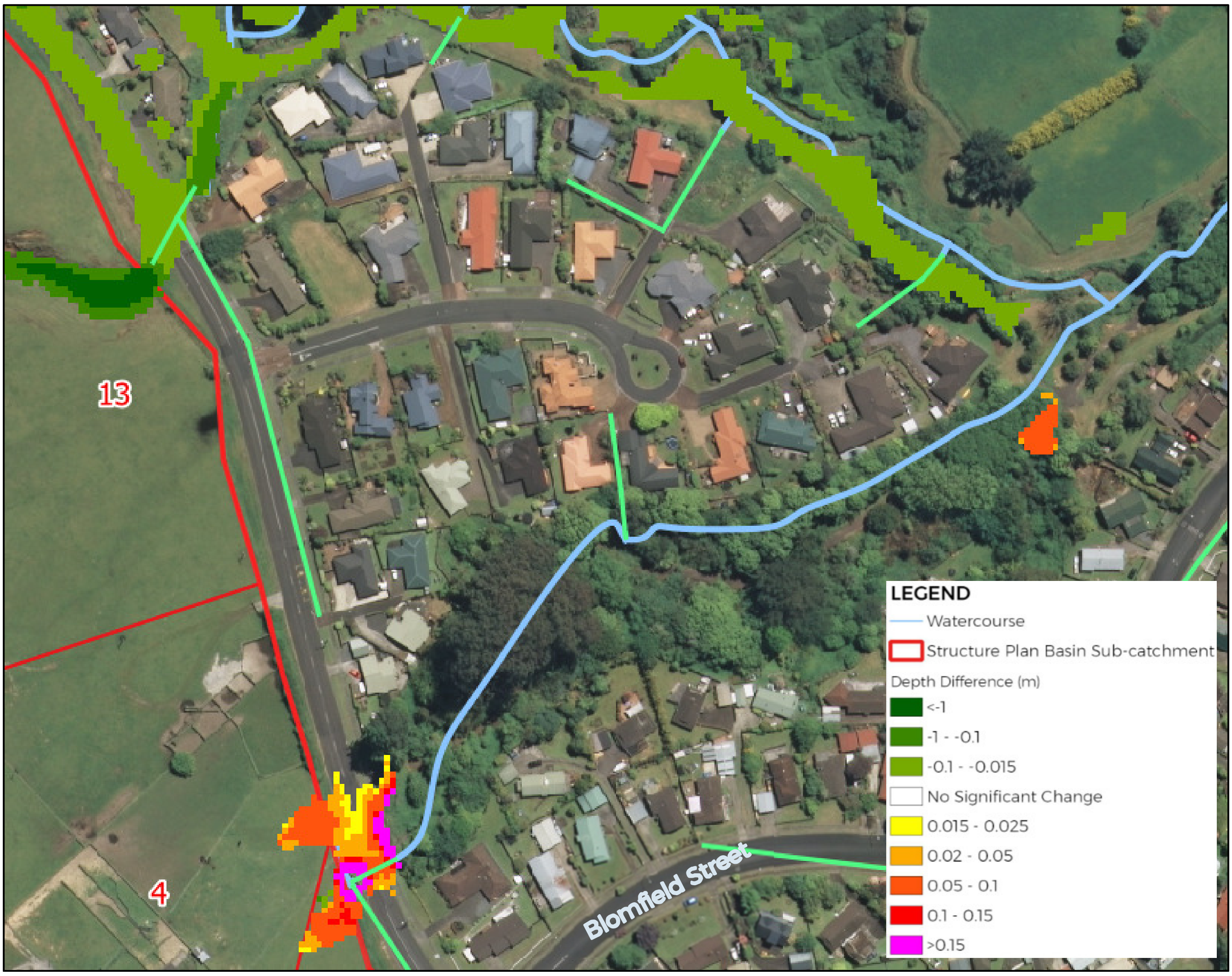


Figure 4-18: Section of the GUCM modelling results for the 1% +CC – Depth difference maps

Looking at Figure 4-18, shows that for the gully north of Blomfield Street, depths and velocities are increased in the Scenario 3 outputs, in line with an increase in the peak flows discharging to this location (see BlomfieldStRes_10255 node).

The consequence of this is limited to this southern gully through to the confluence with the Mangakakahi Stream. A review of the peak water levels in the Council model suggests that the water is contained within the gully and will not affect adjacent properties.

Furthermore, Scenario 14 does show that the amendments made between Scenario 3 and 14 will reduce the peak water levels as the peak flows are lower than the modelled pre-development state. Qualitatively, this would suggest that the Scenario 14 would decrease the water levels and velocities in this area.

Reviewing this area from with the Council models, shows that for the Scenario 3 outputs, the depths, flows and velocities are increased within gully. The velocities stay higher for longer as a result of the attenuation and slower release of the water. This is as expected with the attenuation of peak flows and presents a challenge for urban development that will require further effort to satisfactorily address.

With the velocity remaining above 1m/s for 5.5 hours in the post development state compared to just over three hours in the current conditions, there is an elongated period at which velocities are elevated. To date, there have been no assessments of the critical sheer stress for the soils along these streams, however work. Further works will be required through the design process to assess this hazard and determine approaches to satisfactorily address this.

Remedial measures, including more regular maintenance of the gully (to reduce the risk of blockage further downstream), erosion control measures to reduce the velocities may be required to mitigate this for key points through the channels downstream to avoid increasing erosion risks for adjacent landowners.

Elsewhere, the peak velocities were equal to or less when comparing the post development case to the base case. Thus, the potential for erosion should be no worse than in the existing gullies.

4.3.2.3 0.2% AEP SIMULATIONS +CC

For the 0.2% event (Figure A8), the results show for the components of the GUCM, the post development state reduces the downstream levels less than the 2% event. The benefit is also only really felt in the tributaries of the Utuhina once more, although there are areas across the catchment that are showing to have elevated water levels compared to the existing scenario. The area identified above as showing some effects on the 1% have a similar effect on this model scenario.

After the confluence of the southern branch with the Mangakakahi, peak water levels are increased by up to 50mm. This localised impact then reduces through back to no significant change to the current conditions, through the Linton Park detention area, as shown below in Figure 4-19. Furthermore, a similar effect is seen on the Otamatea Stream, largely contained with Wright Park recreation reserve.

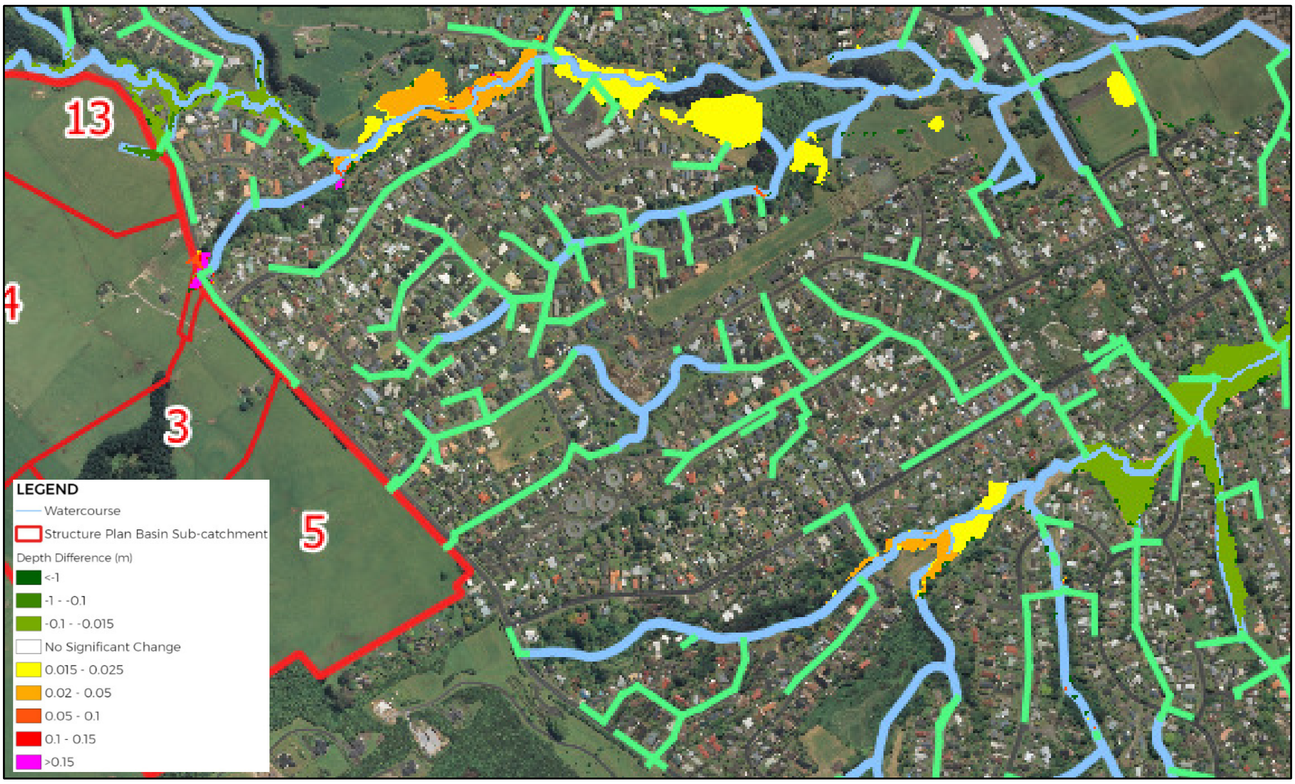


Figure 4-19: Section of the GUCM modelling results for the 0.2% +CC – Depth difference maps (green lines represent the Councils stormwater network)

A review of the areas that are showing to have an increased water level for the 0.2% AEP event shows that some buildings are affected during this extreme event. Across the catchment, two dwellings and an outbuilding are shown to experience an increased water level greater than 50mm.

5 FUTURE WORKS

In preparation for the hearing and the caucusing further modelling analysis is required that has yet not been possible due to the late agreement on critical parameters. These include:

- Determination of the urban stormwater network performance using the Council models with the agreed final Structure Plan Revision M (Scenario 14) and imperviousness percentages with the current urban stormwater networks downstream. These are currently being updated to reflect the changes. Please note that Section 4 contains an assessment against Scenario 03 (with the available reporting) and this is subject to change,
- Similar assessment with the downstream urban environment imperviousness also increased as per current District Plan rules. This assessment requires the existing models to be amended following on from recent meetings with Regional Council in August.
- Caucusing between Regional Council and Council consultants to determine the extent of matters that are agreed on prior to the hearing. This represents a further opportunity to present more supporting evidence to offset the issues raised above.

Subsequent to the plan change approval process, stormwater management represents a great opportunity to deliver improved outcomes across a catchment that experiences flooding. The presence of existing development flooding issues downstream necessitates that the future stages of delivering development onto the Pukehāngī Heights plan change area arrive at an appropriate approach to mitigating (or improving) its effects relative to the current conditions.

Outside of the Plan Change process, Council is working on the development of a Stormwater Masterplan for the district that will identify approaches for efficiently and effectively managing catchment stormwater to both:

- Respond to existing and future flooding issues in catchments, and
- Respond to the existing need for good quality and safe housing as well as enable for the future growth of Rotorua.

This approach may recommend that the on-site mitigation measures proposed herein can be reduced or removed in favour of other stormwater management approaches across the catchment to achieve the same outcomes as above.

6 SUMMARY

The development proposes to increase impervious surface across the site, which if unmitigated, has the potential to increase runoff volume and peak discharge as well as decrease hydrological response times compared to the current situation. Unmitigated, this increases the potential for downstream flooding and increased erosion. Increased erosion can cause downstream sedimentation, turbidity problems and affect the bank stability in the watercourses.

To be consistent with Regional Council requirements for areas upstream of floodplains, options have been considered to reduce the downstream effects and to attenuate post-development peak flows. The approach to meet these requirements was to develop a series of stormwater management basins which will capture runoff from each area of development, attenuate peak flows and reduce the volume of runoff at critical times, thereby delaying the hydraulic response times downstream.

The assessments carried out to date represent a precautionary approach given the nature of the modelling approach, the multiple assumptions that have gone into their development. Examples of this include, as mentioned above:

- The choice of imperviousness cover utilised for the calculations for the post-development state, including the decision to model the residential at 70% imperviousness. This is a current potential outcome in the proposed rules for the site. This would represent a density of development that is in excess of the 'current' surrounding areas with sampling suggested that the imperviousness was in the order of 45% on lot with an allowance for 15% for the roads.
- The choice of rainfall event 'suggested' to prove that there is a viable solution to managing stormwater resulting from the proposed changes to the land use. The 72-hour approach was suggested to minimise the modelling effort as it is deemed to represent an appropriate method for deriving 'antecedent conditions' – A 24-hour nested storm would achieve a similar outcome but would have a significantly reduced volume required to meet a similar level of service.
- The climate change approach, as discussed above, is precautionary representing a trend to RCP 8.5 and a 3.68-degree temperature change by 2130.
- The assumed imperviousness is believed to be conservative and increasing the SCS Curve Number from 21 to 29 for the developed sub-catchments is conservative compared to conventional calculations.
- The runoff generated from the modelling approaches are higher than those empirically derived for the catchment from the reviewed record. This allows

Based on the evidence available to hand for the current city state, we can conclude that with the proposed mitigation measures there are only minor adverse effects on flood hazard (in relation to the peak water levels or the peak velocities) experienced through the main riverine environments, for the 1% event now and into the future.

- A slight minor effect from the development for the less likely events, in relation to elevated flows and increased water depths has been identified (0.2% +CC). The consequence of this has not yet been calculated across the whole model.

- The approach taken, however, can have a positive impact for the effects from the more likely events analysed (the 10% and the 2% (both with climate change), allowing a wider catchment area downstream to benefit the primary and secondary systems.
- The stormwater basins have been designed with appropriate outlet configurations and storage volume to provide water quantity management and as part of future stages will help to achieve the key water quality treatment needs identified within Regional Council guidelines.
- The Scenario 14 modelling to date, shows that the proposed stormwater basins can reduce peak discharges, from the developed catchment by 22% for the 2% AEP +CC event and 28% for the 1% AEP +CC event. This compares to the Scenario 3 work presented above that had a decrease of 12% for the 2% AEP +CC event and an increase of 8% for the 1% AEP +CC event.

The proposed approaches are shown to reduce the peak flow effects for all events up to the 1% AEP (Scenario 3 – 10 % and 2%, with Scenario 14 likely to show that the 1%AEP can follow a similar benefit). This reduced flow results in reduced velocities; however, the hydrological regime could be impacted through elongating the time at which velocities are over the threshold value for the waterways downstream.

At this stage, it is recognised that the more frequent events are the events that 'form' the channels with the larger events acting as a flush of the system. The design of the attenuation basins and the orifices are such that the channels through the existing areas benefit from the proposed approach. Future stages should seek to assess the impacts of the stormwater approaches on the erosion hazard downstream.

The attenuation requirements for managing the effects of a 72-hour storm, also should not be underestimated. The volume stored directly impacts on the length of time at which the downstream networks are elevated compared to the current conditions. This influences the primary network availability and response and the impact on the urban waterways, in relation to the erosion potential.

Reducing the requirements to mitigate the volumes for a 72-hour storm, would have a further impact of reducing the length of time at which peak velocities are above velocities that could result in erosion of the waterways downstream for the events greater than 10%.

During the detailed design stages of this project, there may be opportunities to refine and optimise the stormwater design, however based on the assessment and recommendations from this report, Council should be confident that the proposed plan change can be carried out in a manner that could avoid negative downstream flooding and water quality effects. There is enough evidence to identify that the stormwater management system should be delivered prior to significant development on site to avoid potential effects during the development phasing.

Water sensitive design elements must be incorporated in the development of each lot and the road corridor to deliver both stormwater quantity and quality requirements. A stormwater treatment train comprising multiple treatment steps from source to outlet must be provided to treat all stormwater runoff and allows for opportunities to reduce the volumetric impacts of development, particularly for the lower return period events. This would assist in reducing the effects.

Through appropriate vegetation selection, topographic contouring and geotechnical advice, it may be possible to change the stormwater management basins into wetlands or add wetland pond elements to them. Both wetlands and basins promote sedimentation, however wetlands also promote biological uptake of contaminants for water quality treatment.

The choice of vegetation (or through specific design) which can withstand both long, dry periods and relatively deep inundation depths and multiple day flood durations will require advice from an appropriate expert. Alternatively, separate treatment of the 'first flush' may be provided prior to the flood attenuation

This report considers a conceptual stormwater management system for the Pukehāngī Heights area for the purpose of deciding whether there are downstream flood risk effect impediments to a proposed Plan Change. The investigation has therefore been targeted at determining whether a solution can be found that meets the objective of avoiding increased peak water levels, rather than providing the detailed design information needed for consenting and construction purposes.

Finally, Council is currently developing a District wide Stormwater Masterplan, that may recommend for this overall catchment, that the on-site mitigation measures proposed herein can be reduced or removed in favour of other stormwater management approaches across the catchment to achieve similar outcomes as above.

CONCLUSIONS

To support the proposed Plan Change documentation, stormwater management devices have been conceptually designed to address downstream flooding and seek to improve water quality outcomes as a result of including detention to help reduce the peak flows.

The location, configuration and details of the devices as used in the modelling process is **one solution** that meets the objectives of having no adverse effects on peak flood water levels downstream and is consistent with the Conceptual Stormwater Master Plan. With the information available at present, this demonstrates there are no fatal flaws in terms of stormwater management that should prevent the Plan Change proceeding.

Therefore, the solution reached should cover any reasonable urban development. It is noted however, that other solutions / configurations of devices may also be feasible both on site and off site to mitigate the impacts identified within, and these could be explored in a detailed design phase.

To facilitate this and outside of the Plan Change process, Council is working on the development of a Stormwater Masterplan for the district that will identify approaches for efficiently and effectively managing catchment stormwater to both:

- Respond to existing and future flooding issues in catchments, and
- Respond to the existing need for good quality and safe housing as well as enable for the future growth of Rotorua.

This approach may recommend that the on-site mitigation measures proposed herein can be reduced or removed in favour of other stormwater management approaches across the catchment to achieve the same outcomes as above.

7 DISCLAIMERS AND LIMITATIONS

This report ('**Report**') has been prepared by WSP exclusively for the Rotorua Lakes Council('**Client**') in relation to identifying the stormwater management considerations relating to proposed Plan Change 2 ('**Purpose**') and in accordance with the Contract with the Client. The findings in this Report are based on and are subject to the assumptions specified in the Report WSP accepts no liability whatsoever for any reliance on or use of this Report, in whole or in part, for any use or purpose other than the Purpose or any use or reliance on the Report by any third party.

In preparing the Report, WSP has relied upon data, surveys, analyses, designs, plans and other information ('**Client Data'**) provided by or on behalf of the Client. Except as otherwise stated in the Report, WSP has not verified the accuracy or completeness of the Client Data. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations in this Report are based in whole or part on the Client Data, those conclusions are contingent upon the accuracy and completeness of the Client Data. WSP will not be liable in relation to incorrect conclusions or findings in the Report should any Client Data be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to WSP

APPENDIX A

FLOOD MODELLING MAPS

Map Reference	Model Source	AEP Event	Scenario
A-1	GUCM	2% with CC	Flood Map – Depth - Existing State
A-2	GUCM	2% with CC	Depth Difference map – Existing state minus development plus proposed mitigation – Scenario 03
A-3	GUCM	2% with CC	Velocity Difference map – Existing state minus development plus proposed mitigation – Scenario 03
A-4	GUCM	1% with CC	Flood Map – Depth - Existing State
A-5	GUCM	1% with CC	Depth Difference map – Existing state minus development plus proposed mitigation – Scenario 03
A-6	GUCM	1% with CC	Velocity Difference map – Existing state minus development plus proposed mitigation – Scenario 03
A-7	GUCM	0.2% with CC	Flood Map – Depth - Existing State
A-8	GUCM	0.2% with CC	Depth Difference map – Existing state minus development plus proposed mitigation – Scenario 03
A-9	GUCM	0.2% with CC	Velocity Difference map – Existing state minus development plus proposed mitigation – Scenario 03
A-10	GUCM	10% with CC	Flood Map – Depth - Existing State
A-11	GUCM	10% with CC	Depth Difference map – Existing state minus development plus proposed mitigation – Scenario 03
A-12	GUCM	1% with CC	Velocity Difference map – Existing state minus development plus proposed mitigation – Scenario 03
A-13	GUCM	10% with CC	Duration of inundation – Difference map

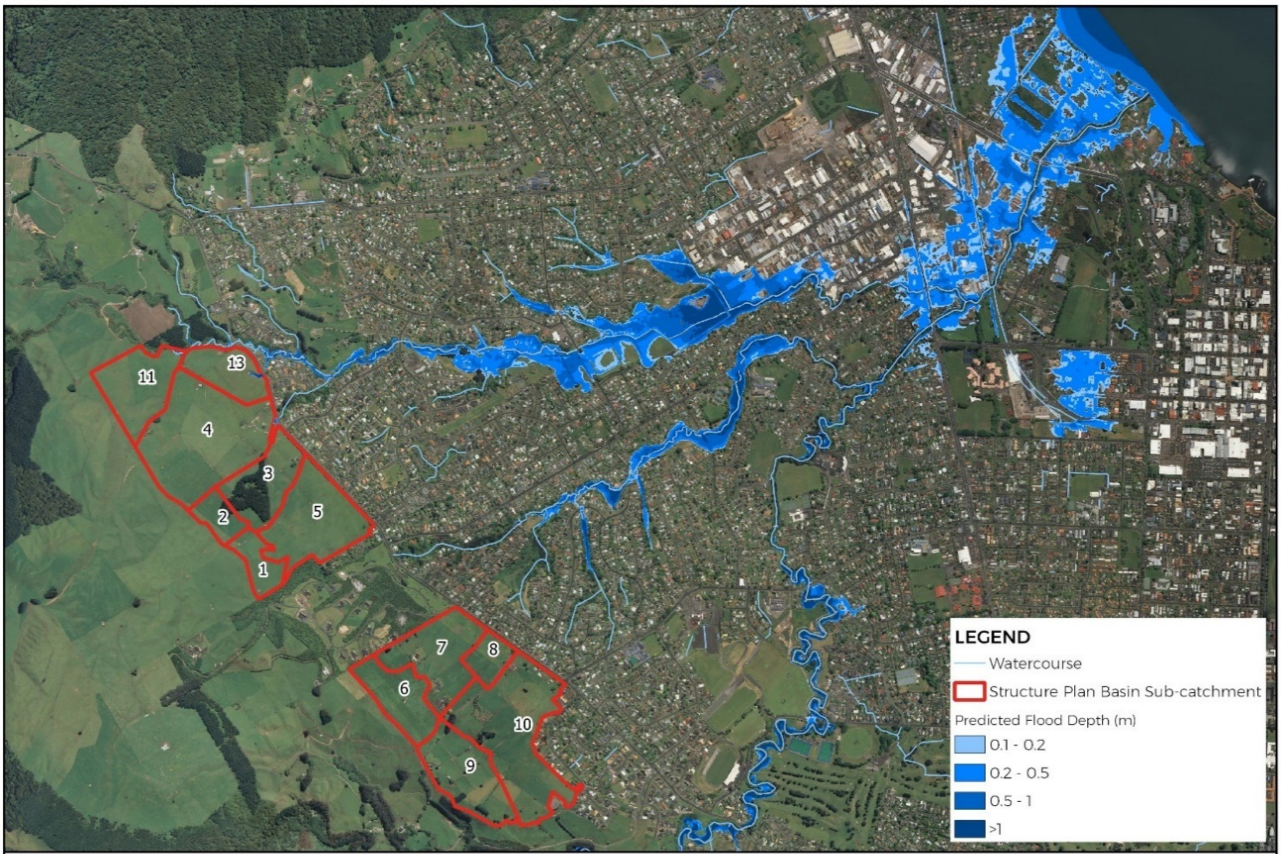


Figure A1 - 2% AEP +CC GUCM Pre-Development Predicted Flood Depths

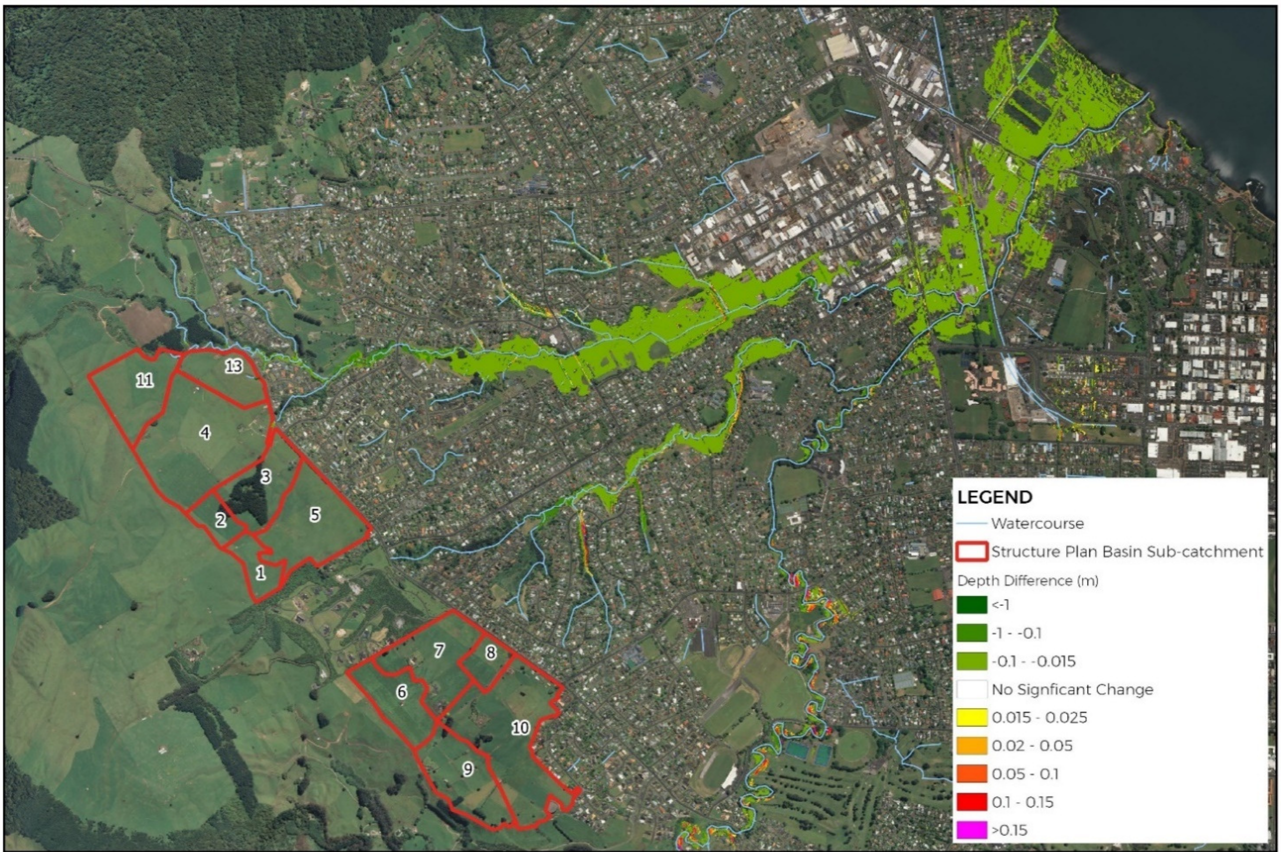


Figure A2 - 2% AEP +CC GUCM Predicted Difference in Flood Depths

Figure A3 - 2% AEP +CC GUCM Predicted Difference in Velocity

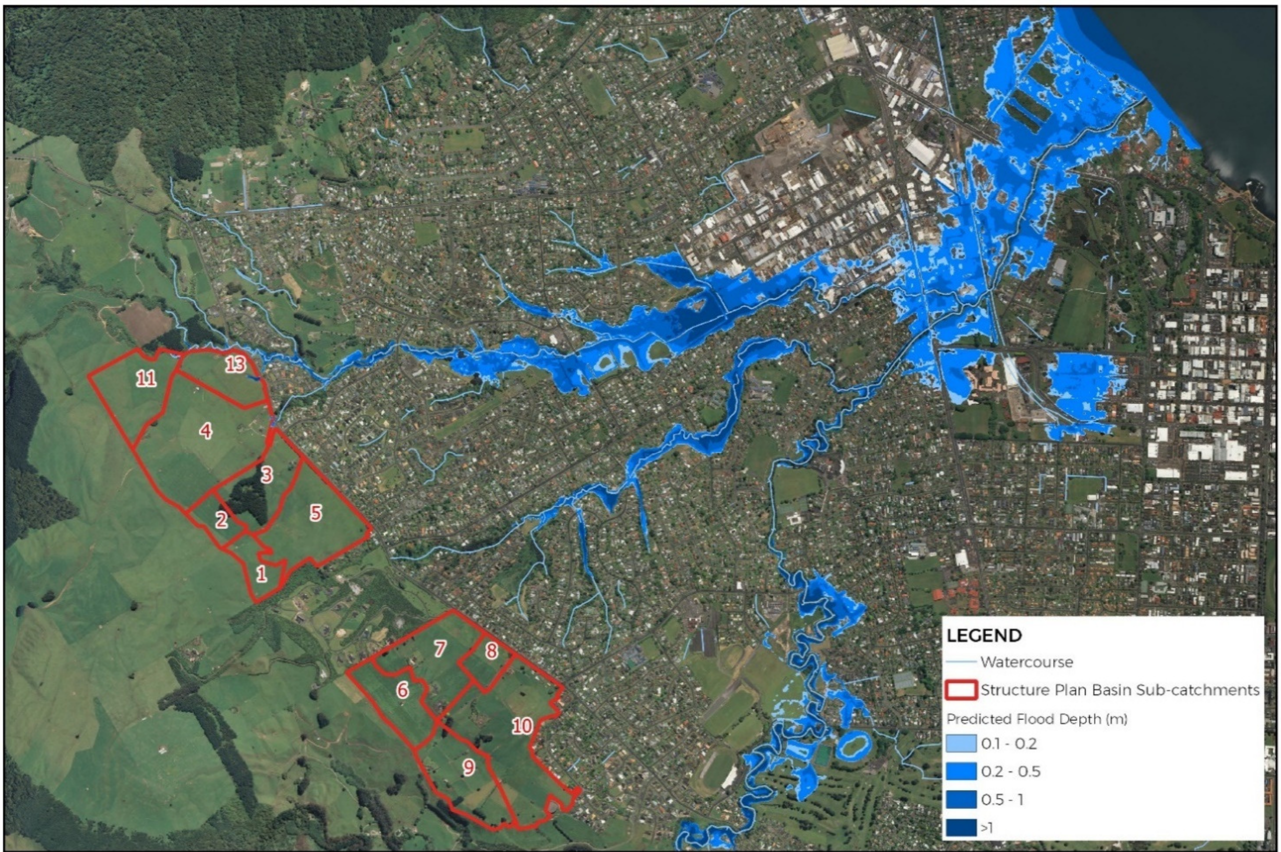


Figure A4 - 1% AEP +CC GUCM Pre-Development Predicted Flood Depths

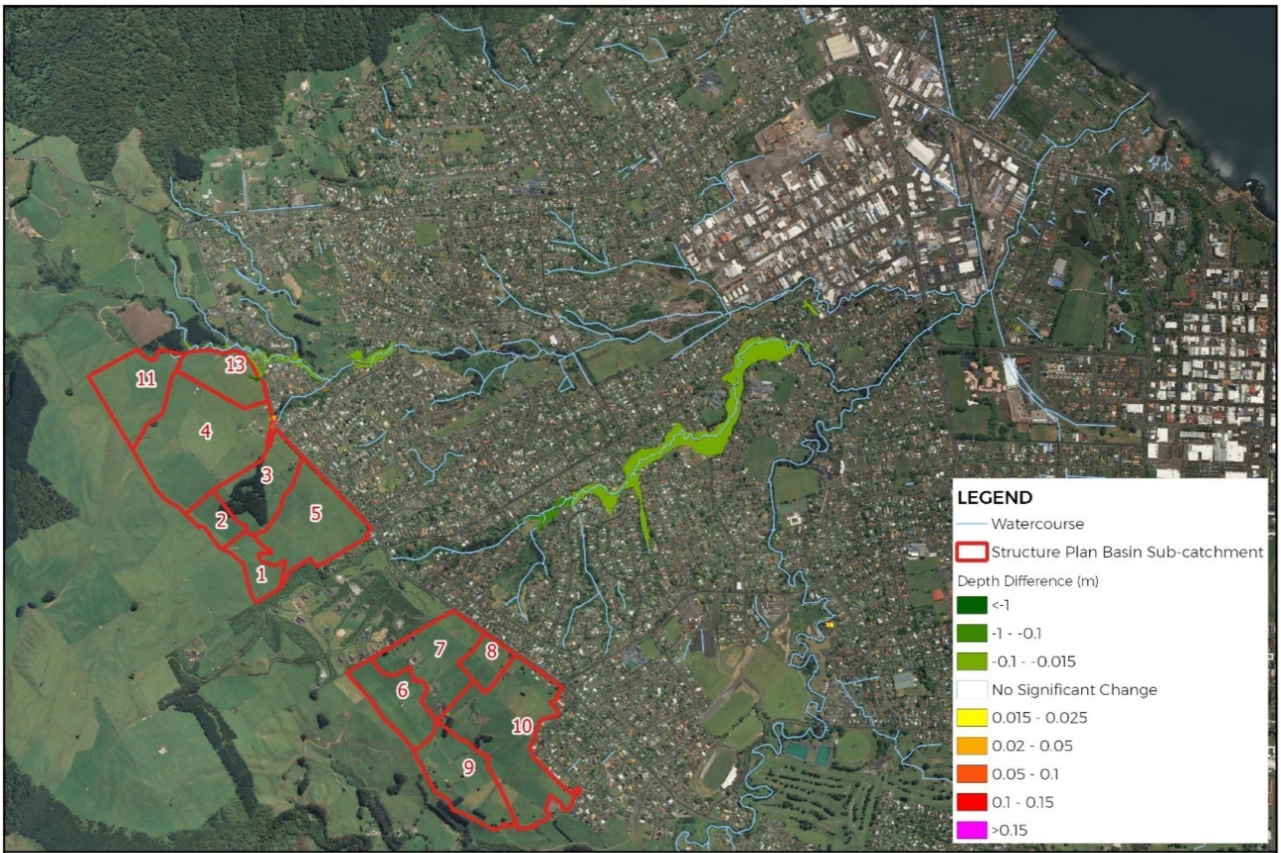


Figure A5 - 1% AEP +CC GUCM Predicted Difference in Flood Depths

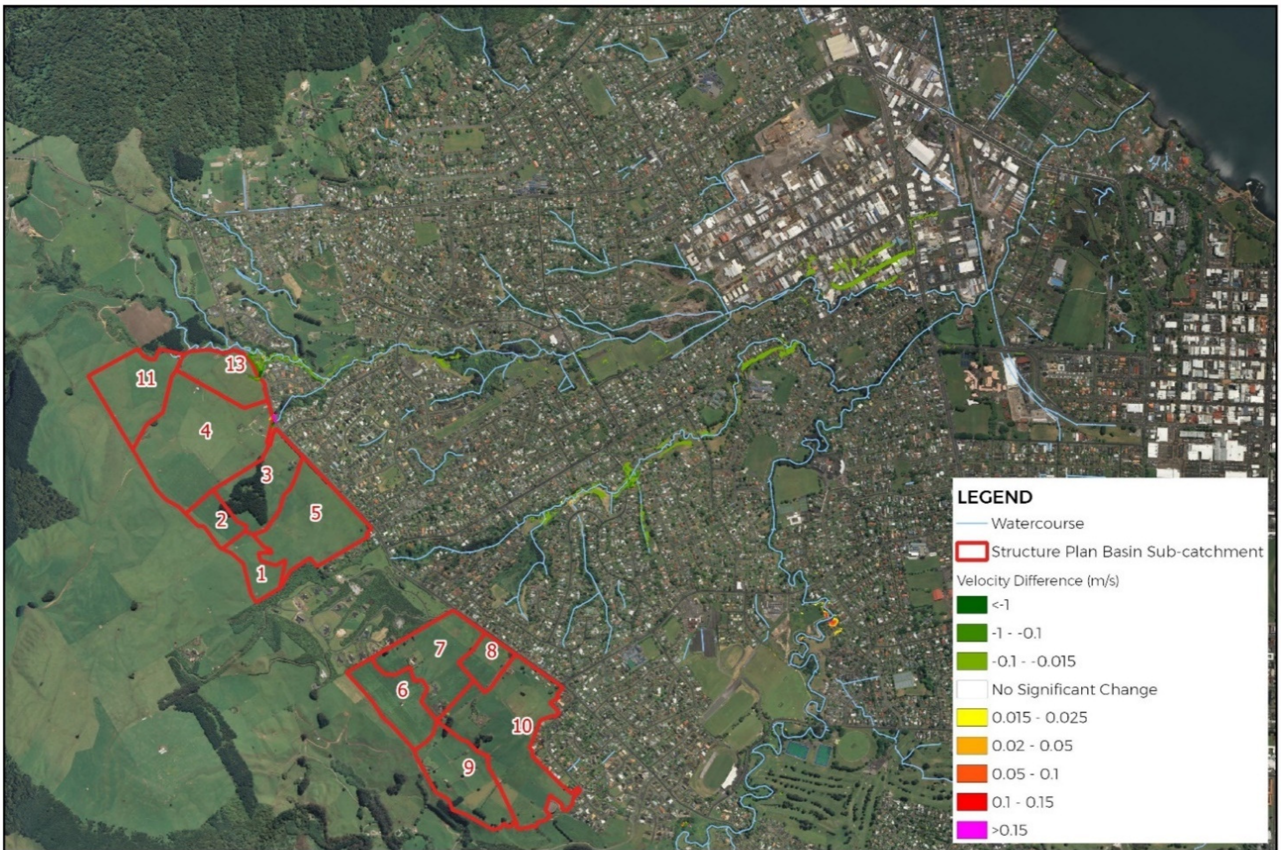


Figure A6 - 1% AEP +CC GUCM Predicted Difference in Velocity

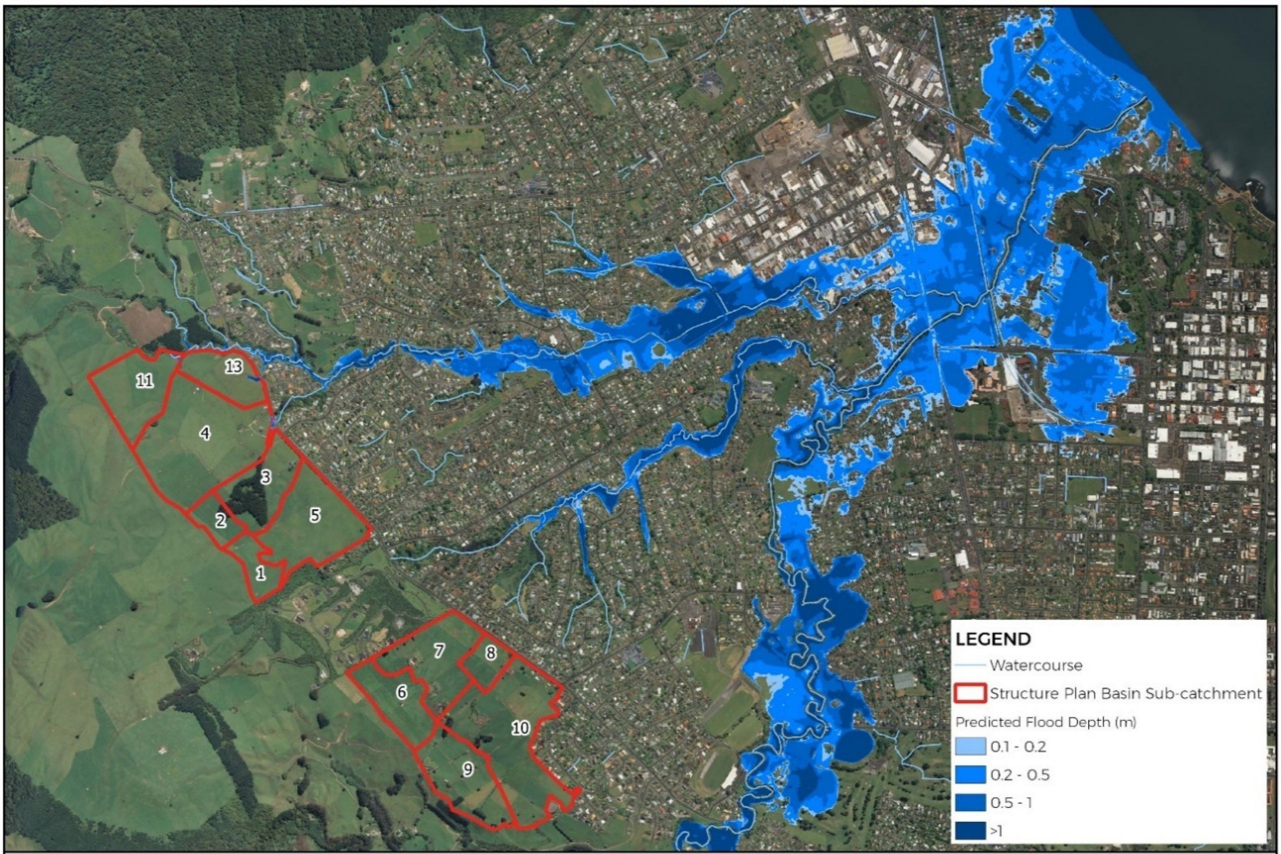


Figure A7 - 0.2% AEP +CC GUCM Pre-Development Predicted Flood Depths

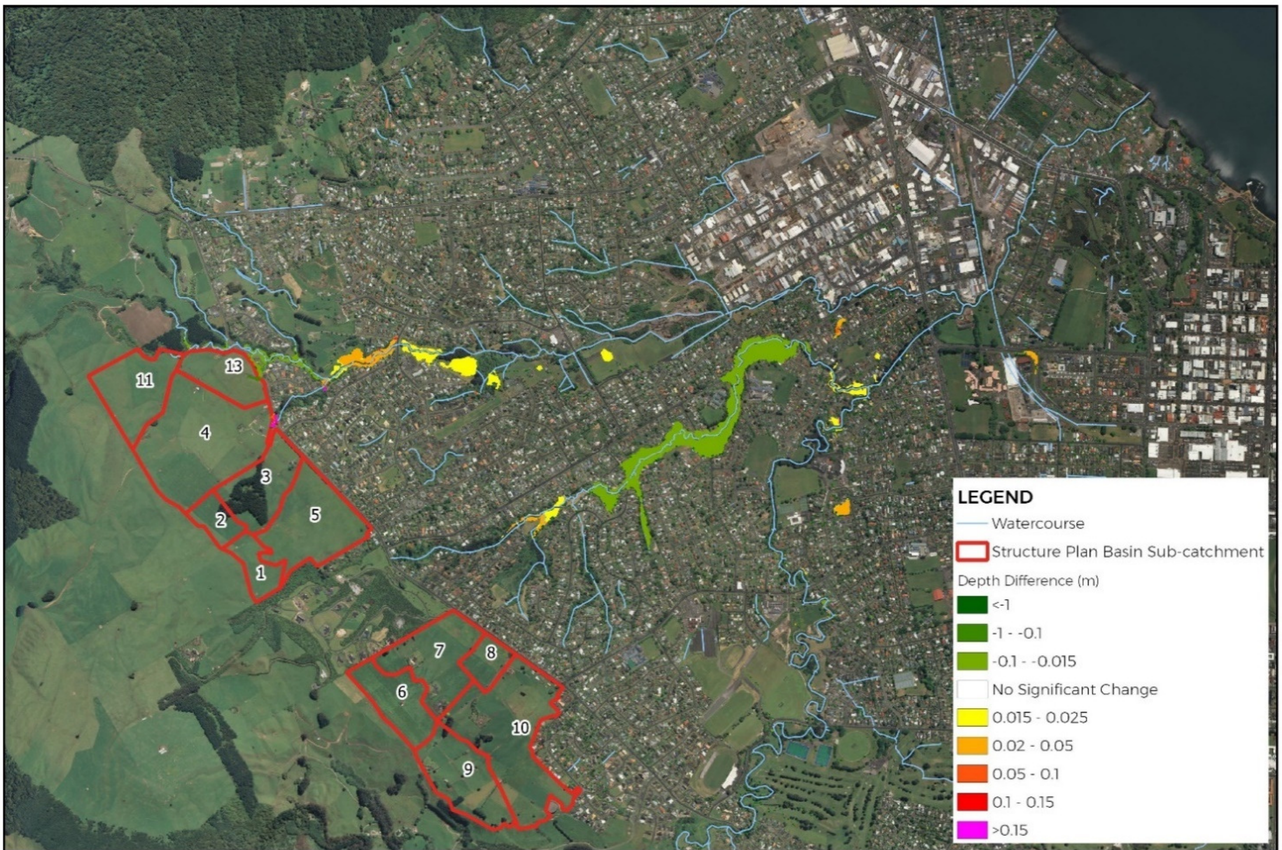


Figure A8 - 0.2% AEP +CC GUCM Predicted Difference in Flood Depths

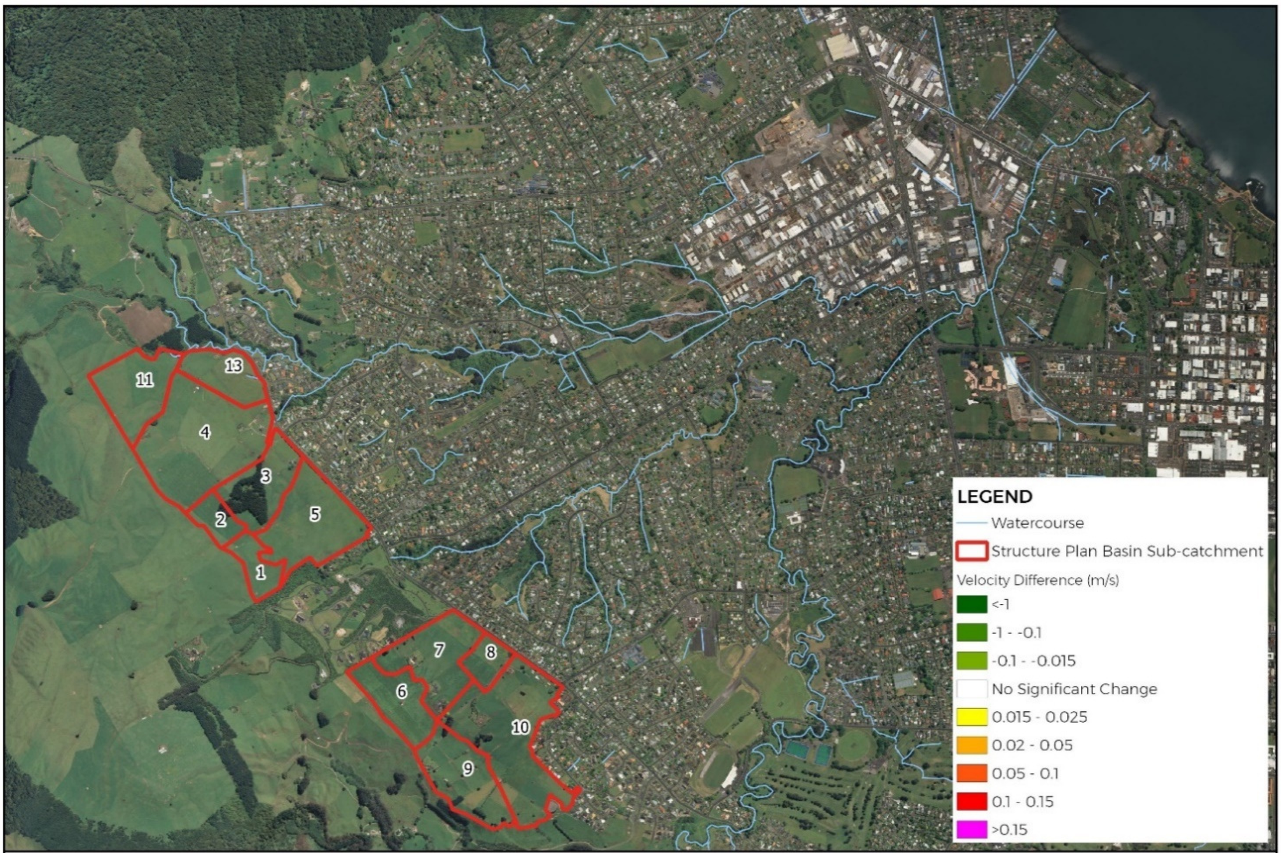


Figure A9 - 0.2% AEP +CC GUCM Predicted Difference in Velocity

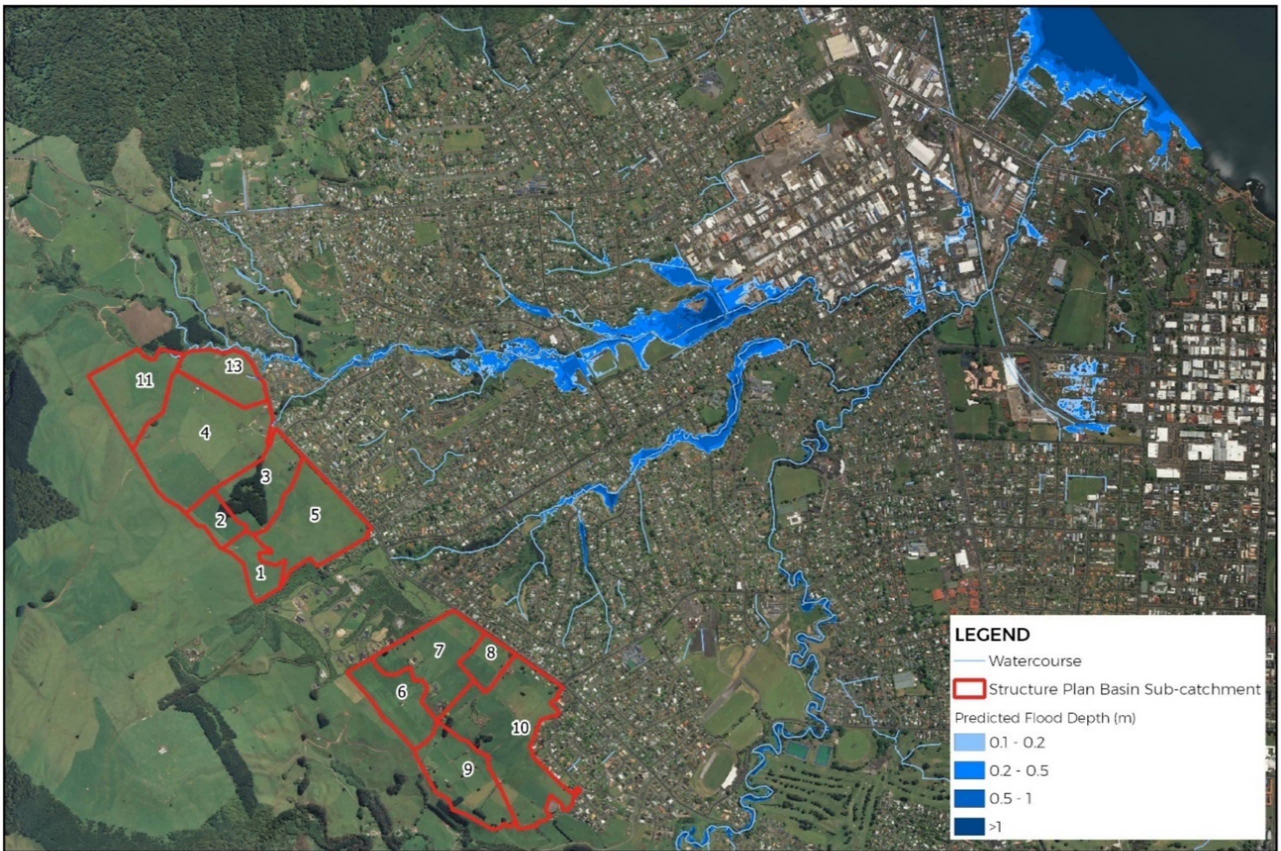


Figure A10 - 10% AEP +CC GUCM Pre-Development Predicted Flood Depths

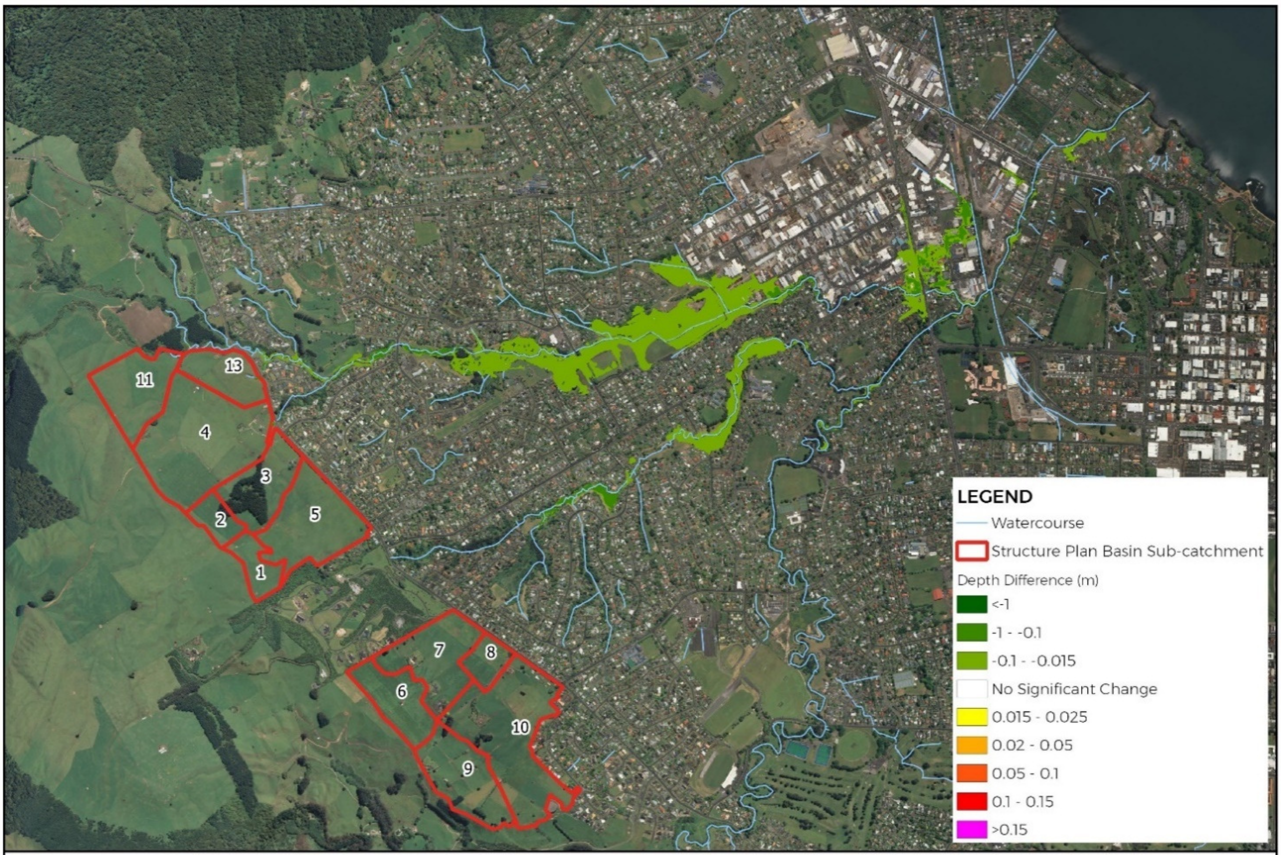


Figure A11 - 10% AEP +CC GUCM Predicted Difference in Flood Depths

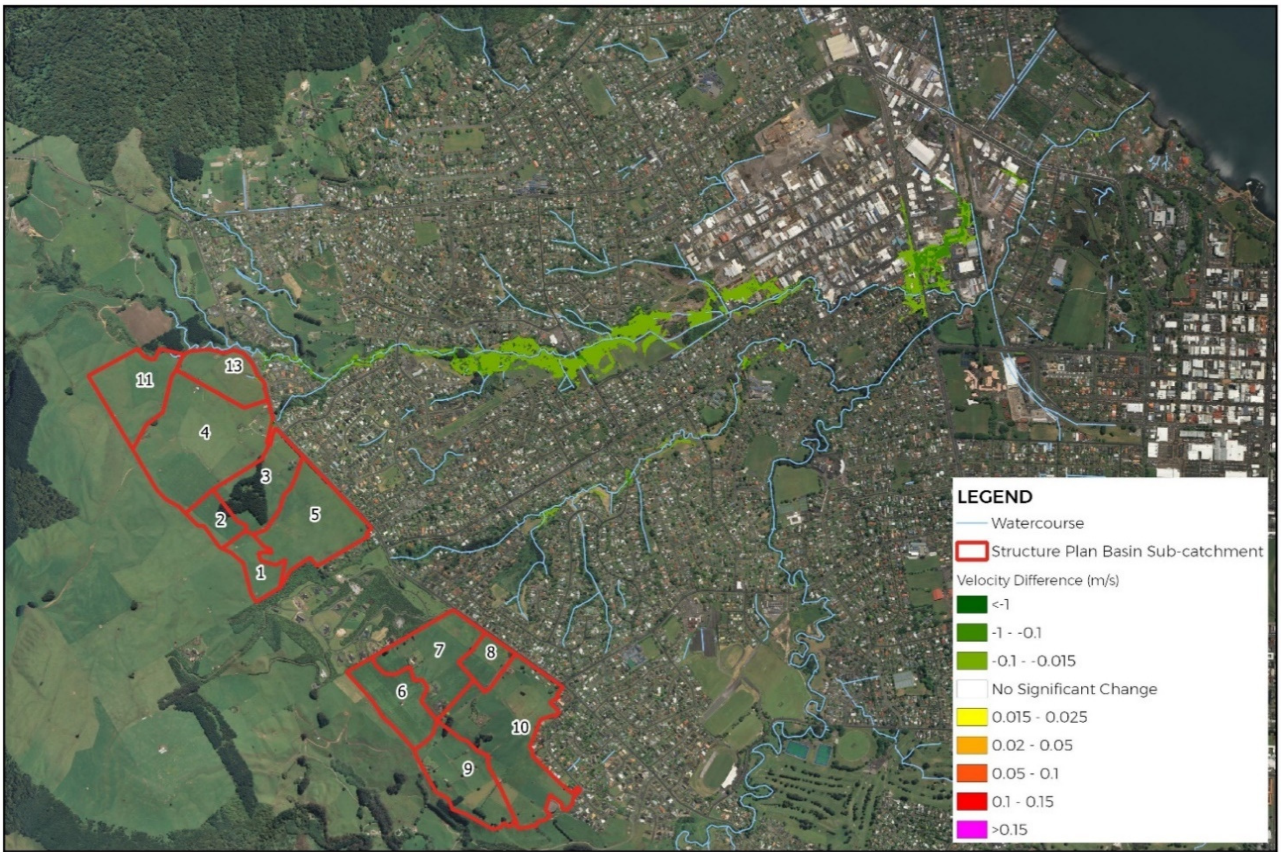


Figure A12 - 10% AEP +CC GUCM Predicted Difference in Velocity

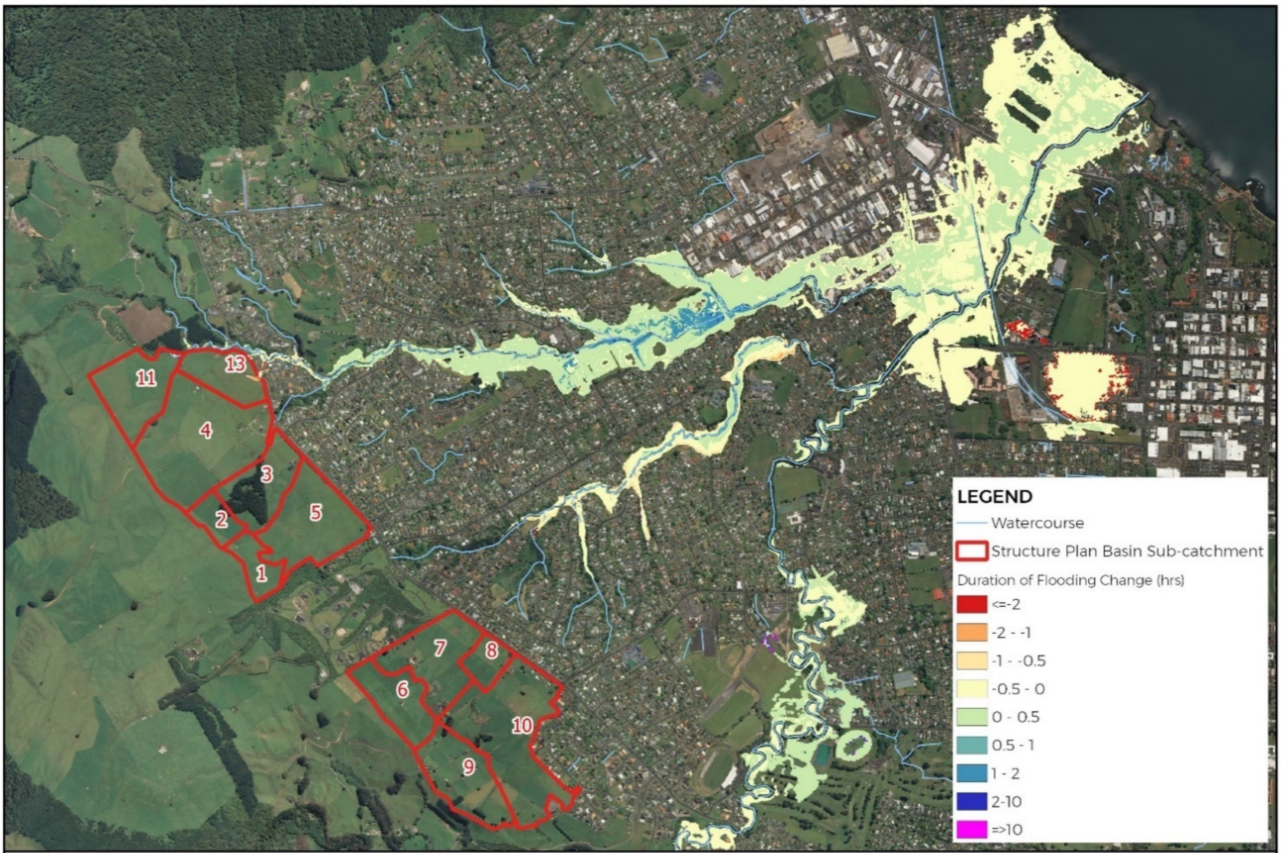


Figure A13 - 1% AEP +CC GUCM Predicted Difference in Flood Duration

APPENDIX B

FLOOD MODELLING MAPS

Map Reference	Model Source	AEP Event	Scenario	Comments
B-1	COUNCIL	2% with CC	Flood Map – Depth - Existing State	Produced
B-2	COUNCIL	2% with CC	Depth Difference map – Existing state minus development plus proposed mitigation – Scenario 03	Not yet available
B-3	COUNCIL	2% with CC	Velocity Difference map – Existing state minus development plus proposed mitigation – Scenario 03	Not yet available
B-4	COUNCIL	1% with CC	Flood Map – Depth - Existing State	Produced
B-5	COUNCIL	1% with CC	Depth Difference map – Existing state minus development plus proposed mitigation – Scenario 03	Not yet available
B-6	COUNCIL	1% with CC	Velocity Difference map – Existing state minus development plus proposed mitigation – Scenario 03	Not yet available
B-7	COUNCIL	0.2% with CC	Flood Map – Depth - Existing State	Produced
B-8	COUNCIL	0.2% with CC	Depth Difference map – Existing state minus development plus proposed mitigation – Scenario 03	Not yet available
B-9	COUNCIL	0.2% with CC	Velocity Difference map – Existing state minus development plus proposed mitigation – Scenario 03	Not yet available
B-10	COUNCIL	10% with CC	Flood Map – Depth - Existing State	Produced
B-11	COUNCIL	10% with CC	Depth Difference map – Existing state minus development plus proposed mitigation – Scenario 03	Not yet available
B-12	COUNCIL	1% with CC	Velocity Difference map – Existing state minus development plus proposed mitigation – Scenario 03	Not yet available

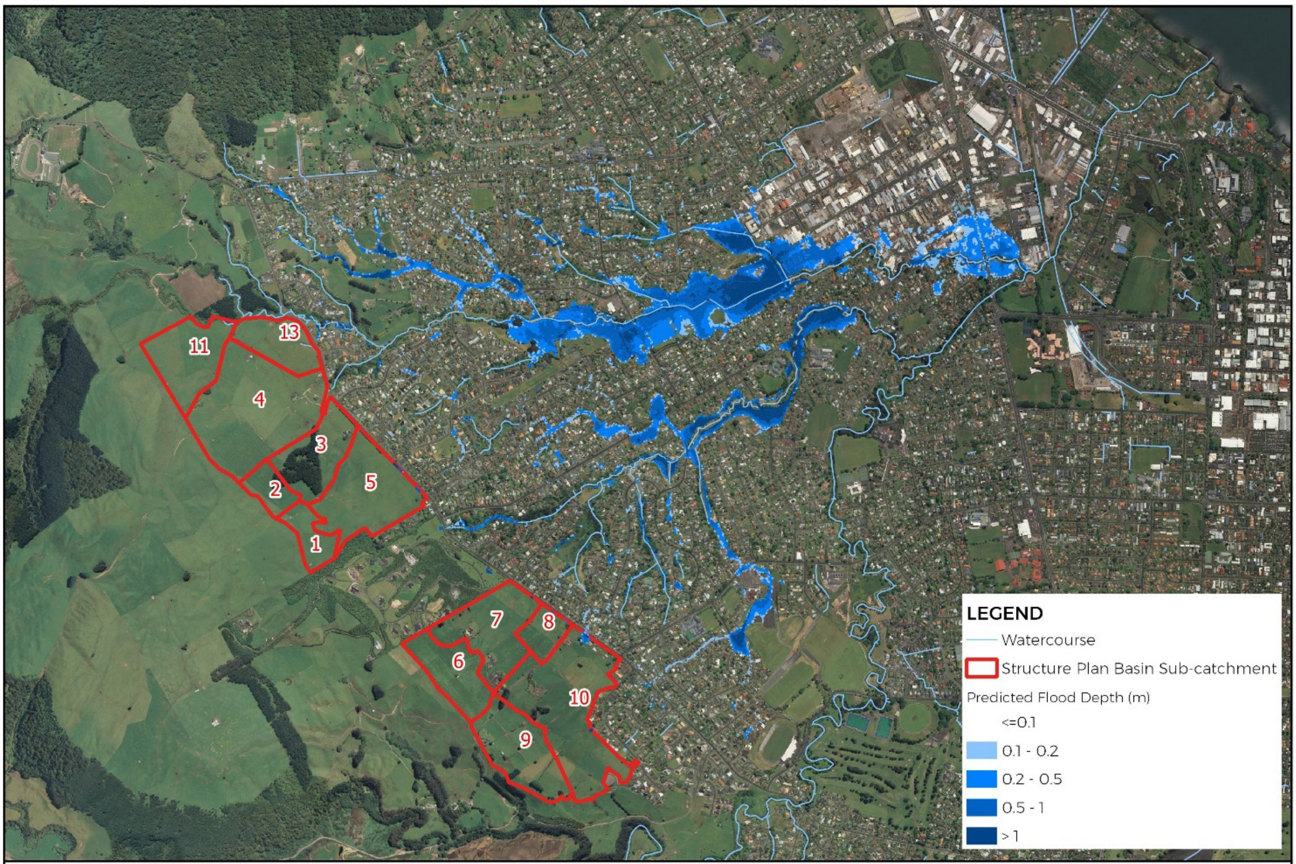


Figure B1 - 2% AEP +CC Council Pre-Development Predicted Flood Depths

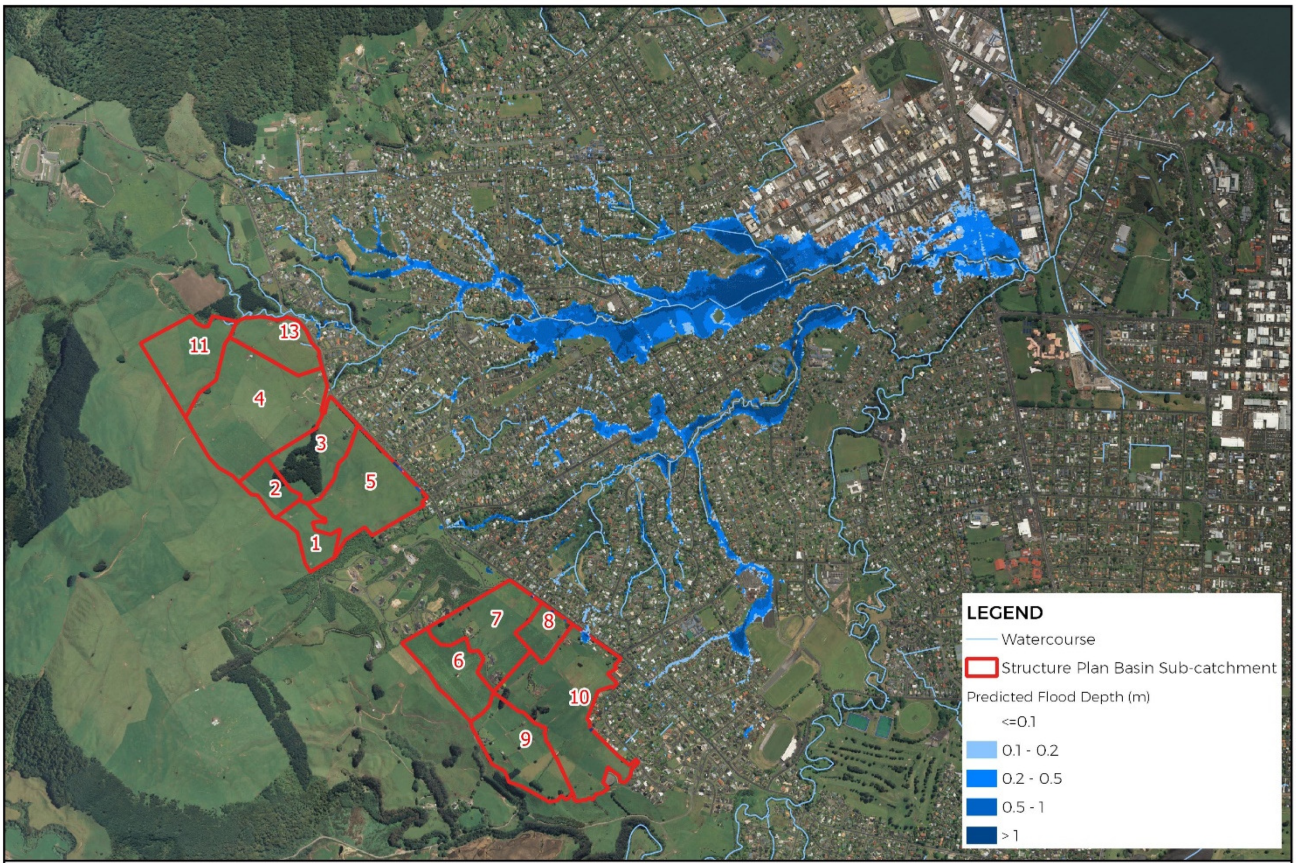


Figure B4 - 1% AEP +CC Council Pre-Development Predicted Flood Depths

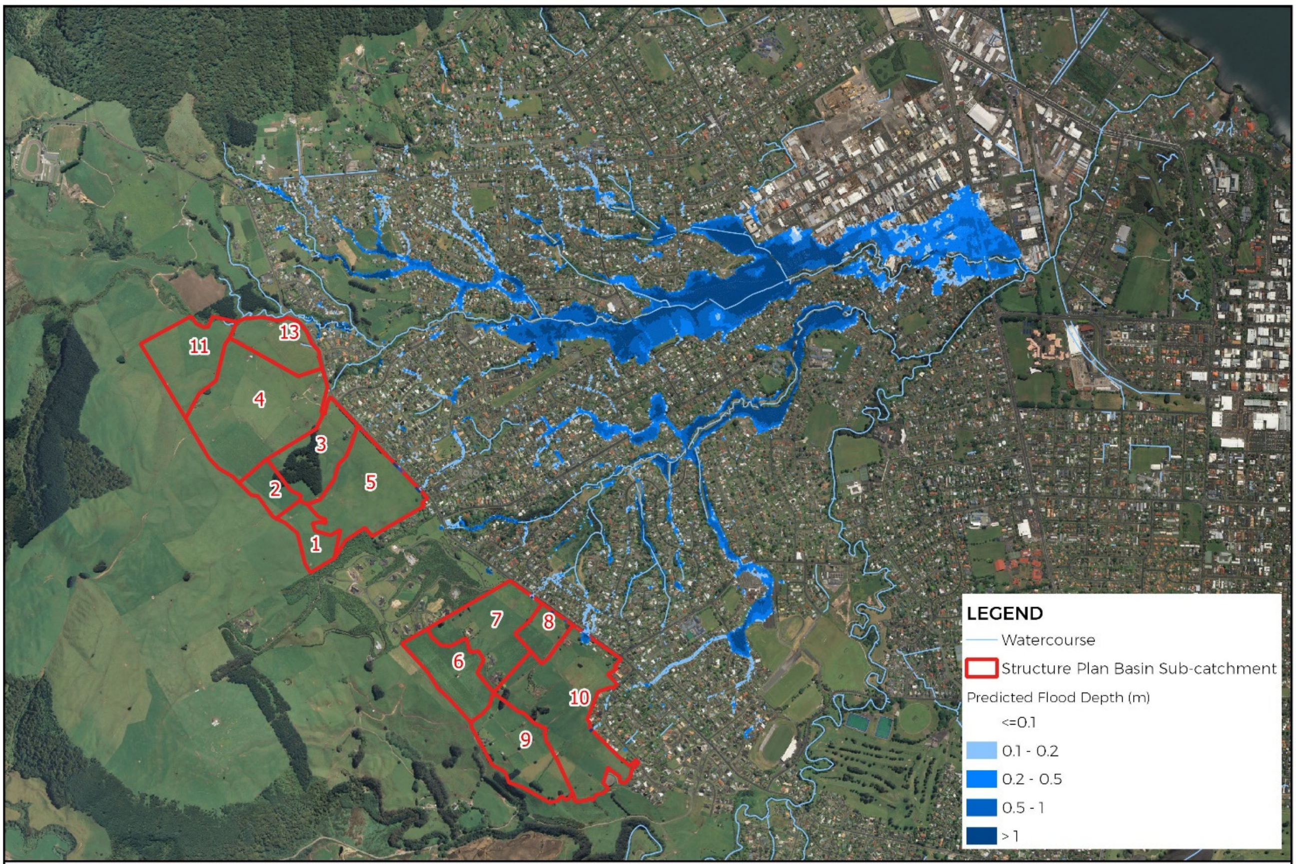


Figure B7 - 0.2% AEP +CC Council Pre-Development Predicted Flood Depths

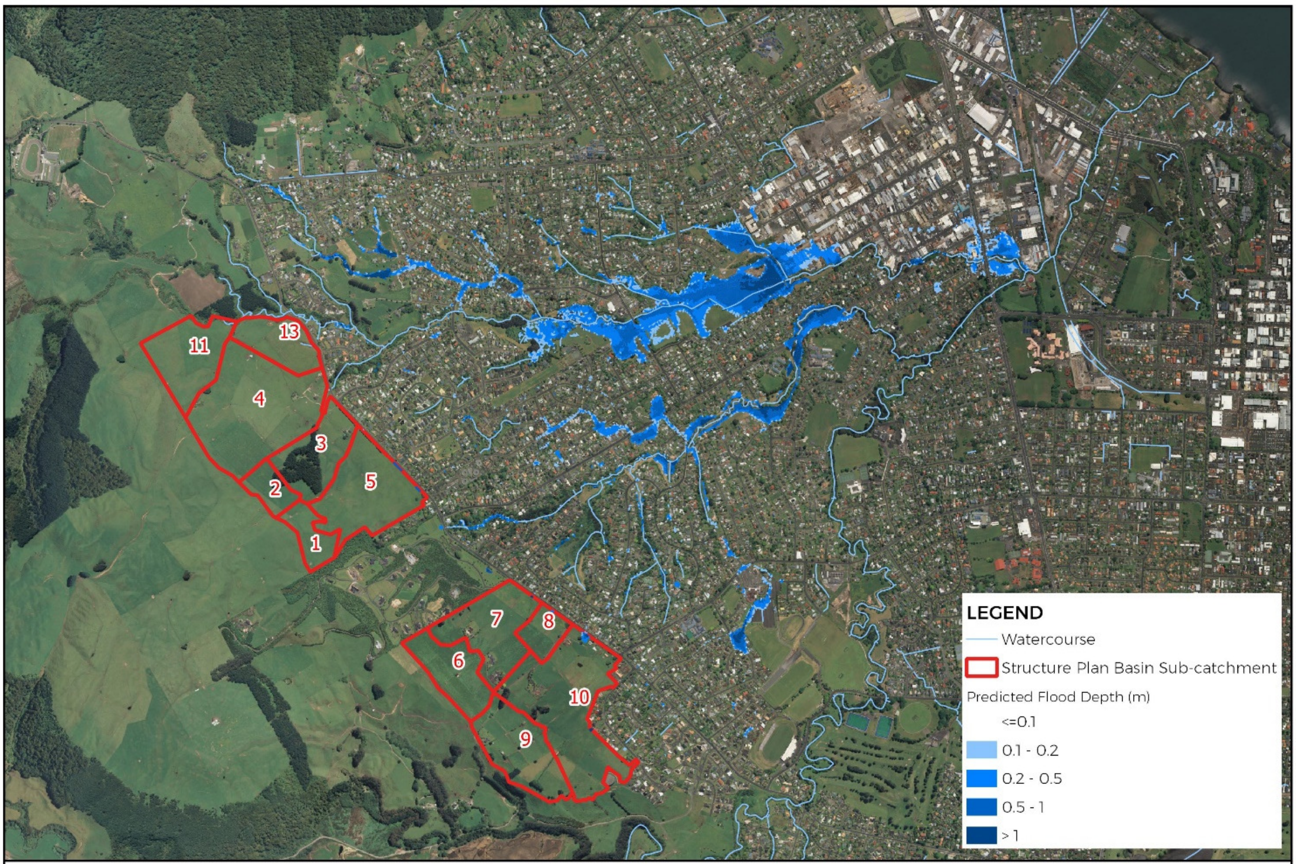


Figure B10 - 10% AEP +CC Council Pre-Development Predicted Flood Depths

APPENDIX C

UTUHINA HYDROLOGICAL WORK MEMO – JULY 2020

Memorandum

To	Liam Foster
Copy	Jack McConchie
From	Lizzie Fox
Office	Wellington
Date	17 August 2020
File/Ref	
Subject	Utuhina at Depot St Rainfall and Flow Analysis

Introduction

A computational hydraulic model was developed to assess the potential impact of proposed development and infrastructure on the flood hazard in the Utuhina catchment. Bay of Plenty Regional Council suggest that a 72hr nested storm should be used in the model to account for antecedent conditions and that the flows from such an event would need to be mitigated.

This memo reviews the flow and rainfall data to characterise the rainfall-runoff processes and define the critical storm in the Utuhina catchment.

Available empirical data

There is one flow gauge on the Utuhina Stream; Utuhina at Depot St. There are no rain gauges within the Utuhina catchment, with the three nearest gauges located within a 1km of the catchment boundary (Figure 1). The available hydrometric data are summarised in Table 1.

The available hydrometric data are used to define the rainfall-runoff relationship for large storm events, and to determine the most appropriate temporal rainfall distribution for use in the computational hydraulic model.

Table 1: Summary of hydrometric data within and around the Utuhina catchment.

Site name	Recording Authority	Data type	Start Date	End Date	Record Length
Utuhina at Depot St	BOPRC	Flow	Sep 2005	May 2020	15 years
Ngongotaha at Relph Rd	BOPRC	Rainfall	Jul 2018	Jun 2020	2 years
Rotorua at Whakarewarewa	BOPRC	Rainfall	Jan 1901	Jun 2020	119 years
Rotorua at Ews	NIWA	Rainfall	Sep 2015	Apr 2020	5 years

WSP
Wellington
L9 Majestic Centre
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+64 4 471 7000
wsp.com/nz

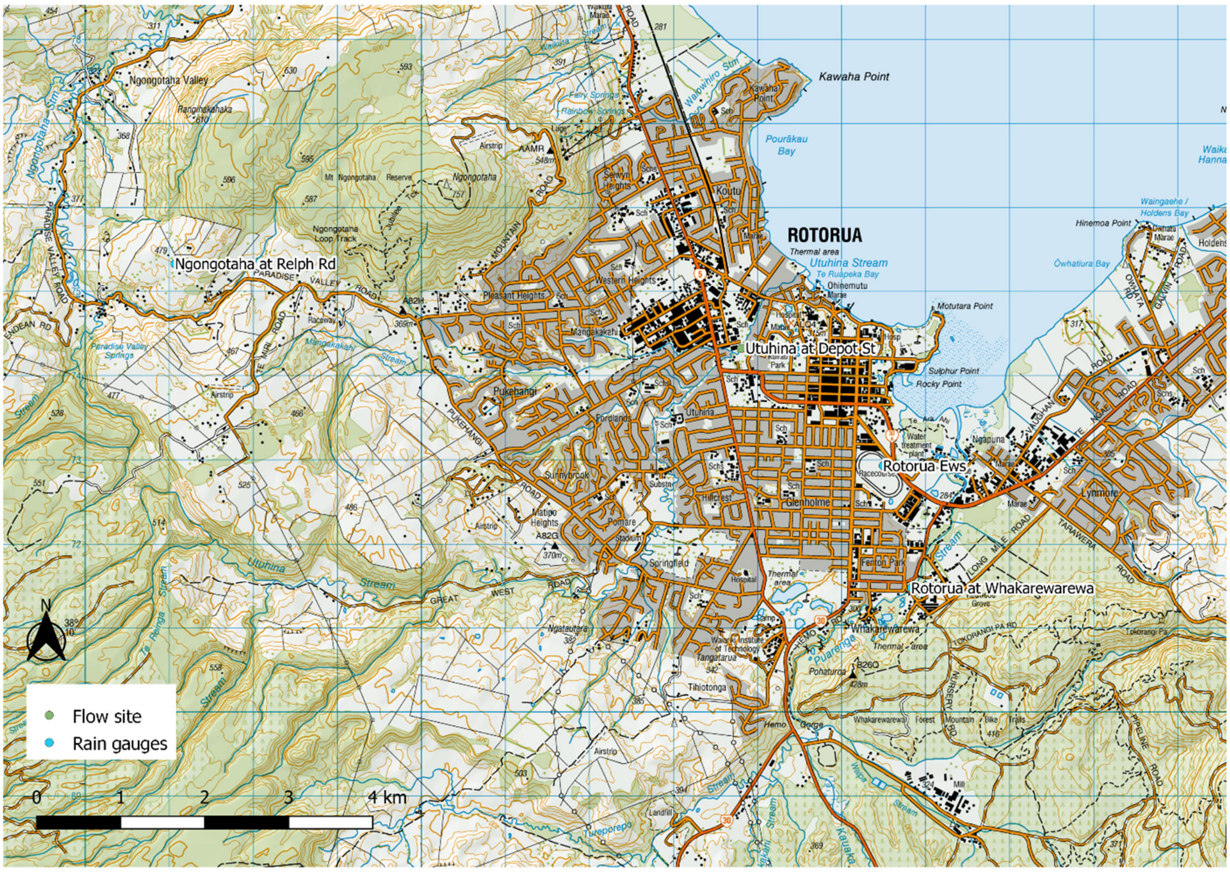

Figure 1: Available hydrometric data near the Utuhina flow gauge.

Flow and rainfall flood frequency analysis

Flow and rainfall data was obtained from the Bay of Plenty Regional Council (BOPRC) Environmental Data Portal on the 24 July, 2020. These data included flow from Utuhina at Depot St hydrometric station, and rainfall from the Rotorua at Whakarewarewa gauge. The Rotorua rain gauge is considered to best reflect rainfall in the Utuhina catchment as it provides a long record for analysis (> 100 years), is located near the eastern boundary of the Utuhina catchment (and therefore likely receives similar rainfall), and the record includes many of the most recent large flood events in the Utuhina catchment i.e. April 2018.

The flow record analysed starts on the 18 September 2005 and ends on the 4 May 2020; providing approximately 15-years data for analysis. The data was recorded as instantaneous values, either at 15-minute or 5-minute intervals. The actual rainfall record began much earlier, 9 January 1901, providing 119 years of rainfall data. However, rainfall was recorded at daily or hourly timesteps until 1992, when data became sub-hourly.

During the 15-year flow record, a peak flow of just under 28.5m^3/s was recorded on the 29 April, 2018. Other large flow events include 20 August 2014 (28.2m^3/s) and 29 January 2001 (27.9m^3/s) (Figure 2). The minimum flow recorded was 0.96m^3/s on 22 March, 2020.

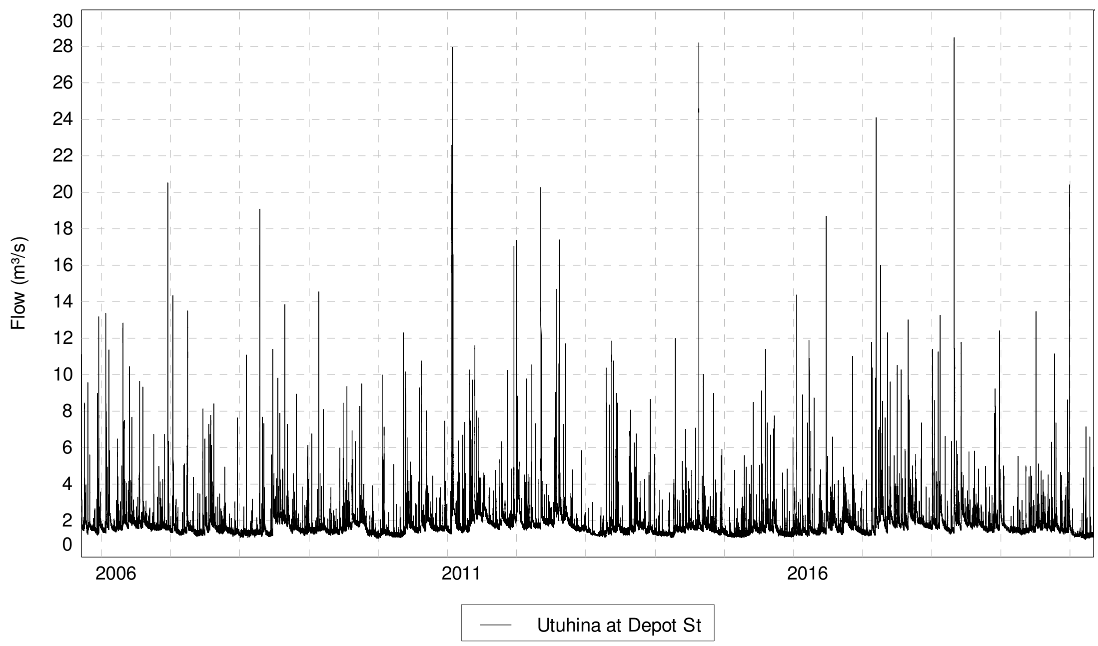

Figure 2: Flow data obtained from BOPRC on the 24 July 2020 for the Utuhina at Depot St hydrometric site. Data from September 2005 to May 2020

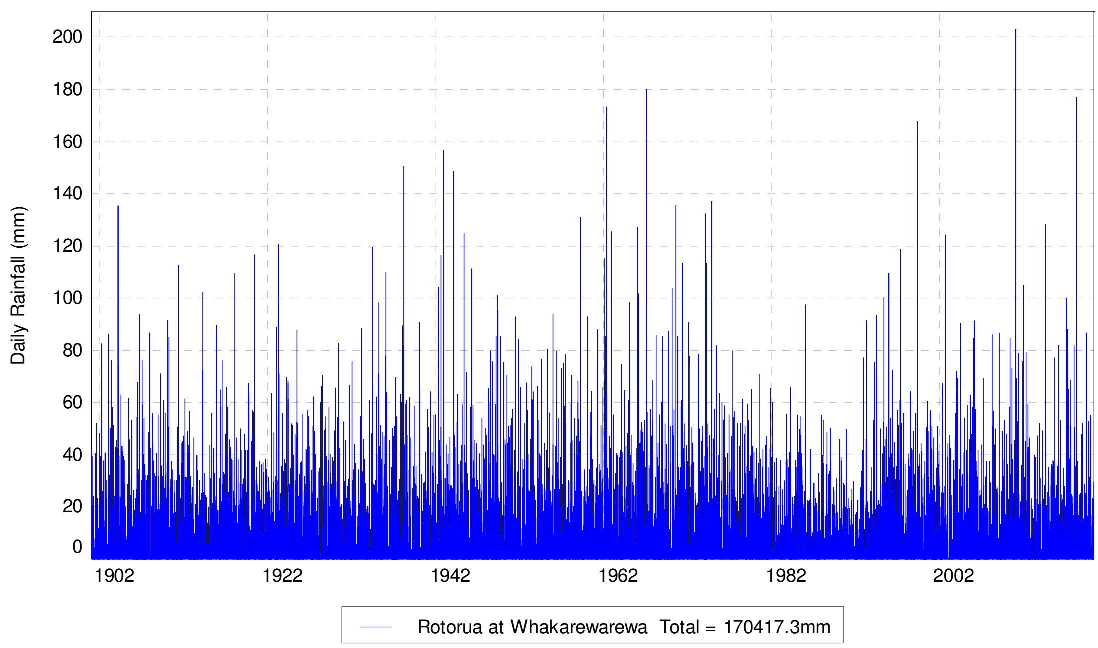

Figure 3: Rainfall record from the Rotorua at Whakarewarewa gauge. Data from January 1901 to June 2020.

No independent data quality check was conducted on the data; however, as it is collected and processed by Bay of Plenty Regional Council it is assumed accurate and quality assured.

To derive design rainfalls and flows for the Utuhina catchment, flood frequency analysis is required. Three types of statistical distribution were assessed for how well they modelled the annual flood maxima series (i.e. Gumbel, Pearson 3 (PE3) and GEV). The distribution which provided the best fit to the annual maxima series was then used to estimate the annual exceedance probabilities (i.e. AEPs), or average recurrence intervals (i.e. ARIs), of each design event. The criteria adopted in this study were:

- The distribution that provided the best-fit through all the data points (i.e. annual maximum flood and rainfall);

- The distribution with the most realistic shape; and
- The distribution that provides the closest approximation to the extreme values.

While this process may appear subjective, in most cases the choice of a specific statistical distribution for the annual maxima series results in relatively minor differences in the estimated duration-intensity-frequency table; at least for the relatively more frequent events.

It should be noted that there is significant uncertainty when there is a high degree of extrapolation. As a rule of thumb, AEPs should not be extrapolated beyond twice the length of the record (Davie, 2008). NIWA, however, have argued that values can be extrapolated to five times the length of record. Using either method, the extrapolation of these extreme events should be treated with caution as the uncertainty of estimates increases rapidly with increasing magnitude e.g. the uncertainty of the magnitude of the 1% AEP event is much larger than that of the 2% AEP design event.

Design flows

Of the three distributions, the PE3 is considered the most suitable; although there is little difference between this and the GEV, until events exceed a 0.5% AEP (i.e. a 1-in-200-year ARI) (Figure 4). Therefore, either of these distributions could be used, but it is considered that the PE3 distribution provides a better 'shape' and therefore more appropriate design flows.

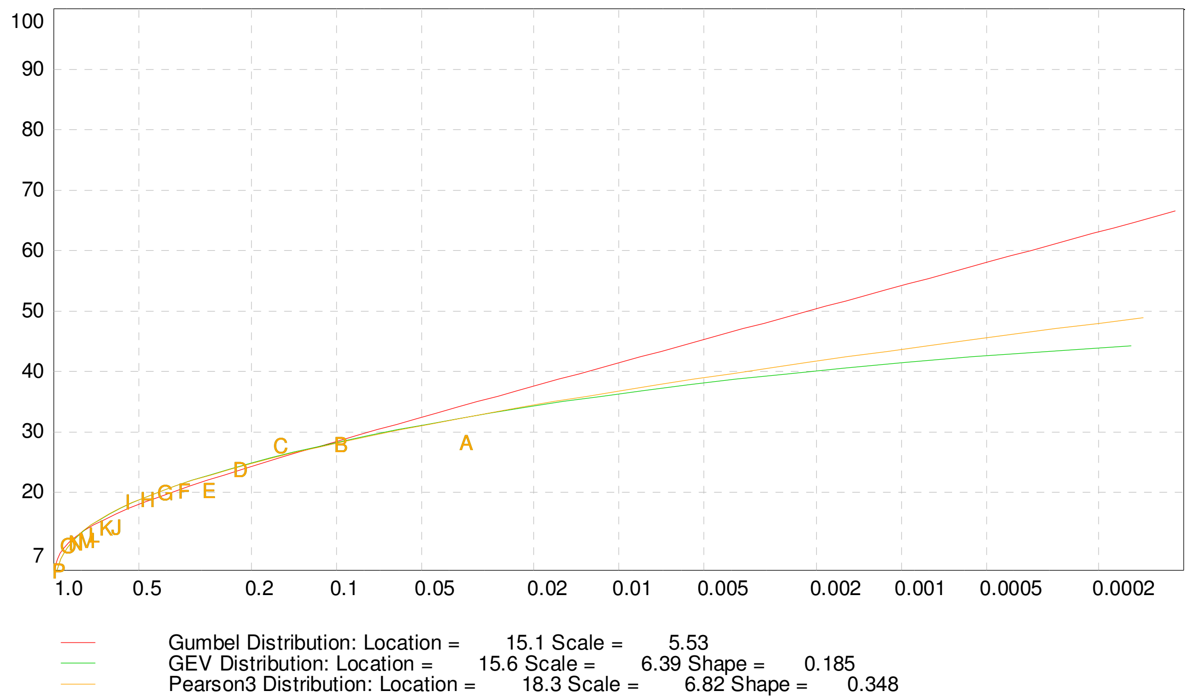

Figure 4: Frequency distribution of annual maxima series of Utuhina at Depot St (2005-2020).

The Gumbel distribution does not fit the large April 2018 event as well as the PE3 or GEV, and plots significantly higher than the other distributions. Therefore, if a Gumbel distribution was used this event would have a higher probability of occurring in any given year (i.e. 8.5% AEP using Gumbel, vs. 7.7% AEP using GEV and 7.5% AEP using PE3). It is considered that the use of a Gumbel distribution would provide magnitudes of design events which are too conservative.

The peak discharges of various design events are displayed in Table 2.

Table 2: Design flows (m³/s) for Utuhina at Depot St (2005-2020), rounded to 1dp. Values in bold are suggested to be used to represent flood flows at various frequencies and magnitudes for the site.

ARI (years)	AEP (%)	GUMBEL Design Flows (m³/s)	GEV Design Flows (m³/s)	PE3 Design Flows (m³/s)
2.33	43	18.3	19.1	19.1
5	20	23.4	24.0	23.9
10	10	27.5	27.4	27.2
20	5	31.5	30.2	30.1
50	2	36.7	33.4	33.5
100	1	40.5	35.4	35.9
200	0.5	44.4	37.2	38.1

Design rainfalls

Of the three statistical distributions, PE3 is considered the most suitable for the rainfall record. This is because the PE3 distribution provides a better 'shape' and a better fit to the more extreme rainfalls.

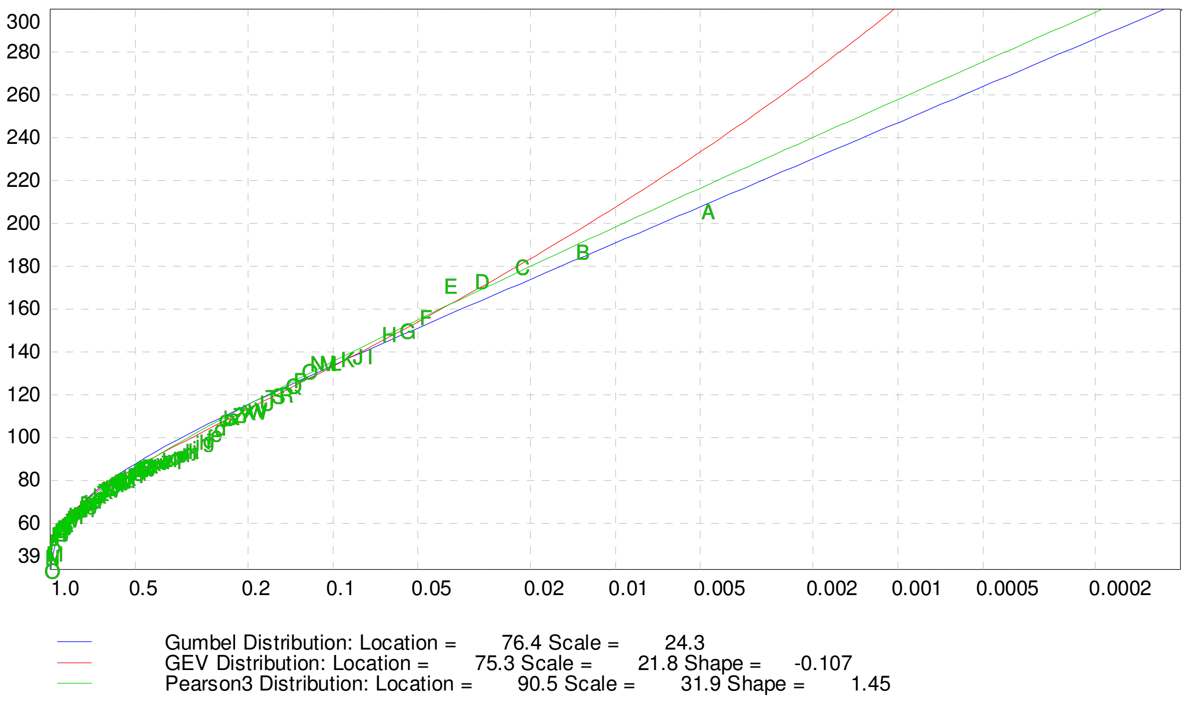

Figure 5: Frequency distribution of annual maxima 24-hr rainfall depths of Rotorua at Whakarewarewa (1901-2020).

Table 3: Design rainfall depths (mm) for 24-hr event for Rotorua at Whakarewarewa (1901-2020), rounded to 1dp. Values in bold are suggested to be used to represent rainfall depths at various frequencies and magnitudes for the Utuhina catchment.

ARI (years)	AEP (%)	GUMBEL 24-hr design rainfalls	GEV 24-hr design rainfalls	PE3 24-hr design rainfalls
2.33	43	90.5	88.3	88.3
5	20	112.9	110.8	112.7
10	10	131.2	130.8	133.1

20	5	148.7	151.6	152.5
50	2	171.4	181.0	177.4
100	1	188.4	205.1	195.8
200	0.5	205.3	230.9	214.0

Large flood events

Using the PE3 distribution for both the flows and rainfall, the six largest floods measured at the Utuhina flow gauge were analysed and the rainfall-runoff relationship quantified.

The six floods are shown in Table 4 and compared against the rainfall records in Figure 6. Note that the Ngongotaha rain gauge has only been operational since July 2018, therefore only has rainfall for the 6th largest event, i.e. that of December 2019.

Table 4: Six largest flow events measured at the Utuhina at Depot St gauge from 2005 to 2020. Compared against the Rotorua at Whakarewarewa rain gauge 24hr, 12hr and 6hr depths.

Flood event	Peak flow (m^3/s)	AEP (%)	ARI (years)	24-hr Rainfall (mm)	AEP (%)	ARI (years)	12-hr Rainfall (mm)	AEP (%)	ARI (years)	6-hr Rainfall (mm)	AEP (%)	ARI (years)
29 April 2018	28.48	7.5	13.3	187.31	1.4	72.4	166.67	0.4	261.8	152.22	0.5	199.4
20 August 2014	28.20	8.0	12.5	138.58	8.2	12.1	128.61	1.8	56.2	119.64	1.4	69.3
29 January 2011	27.94	8.5	11.7	139	8.1	12	138.0	1.2	85	117	1.6	63.5
12 March 2017	24.10	19.2	5.2	109.95	21.9	4.6	95.35	6.8	14.8	81.39	5.2	19.2
18 December 2006	20.54	35.1	2.8	28.0	100	<1	25.5	100	<1	22.5	52	1.9
25 December 2019	20.42	35.8	2.8	60.21	86.4	1.2	49.59	41	2.4	48.07	17.3	5.8

The analysis shows that for the largest rainfall event, a 1.4% AEP event for 24 hr and even more extreme for 12hr and 6hr durations, did not result in a large flood event. The next two largest rainfall and flow events were of a similar 'magnitude/frequency' i.e. August 2014 and January 2011 using 24hr rainfall. However, the shorter duration rainfalls were much more extreme.

Therefore, there is no consistent relationship between a large magnitude, infrequent rainfall event and corresponding flood. The shorter duration rainfalls are generally much more extreme than the corresponding flow event. Other factors, such as antecedent conditions, therefore impact the rainfall-runoff relationship for a specific storm event and these must be considered when modelling the most 'extreme' flood events.

The lag time i.e. the time between the peak in rainfall and flow, for the Utuhina catchment is very short; as is the time of rise. This is despite the rain gauge being just outside the catchment boundary. Within one hour of rainfall starting, flow begins to increase. The flood is reached within 4-7 hours across the six events. This is to be expected, as the catchment is relatively small; 61.3km^2.

Furthermore, the rainfall events are of a short duration. Except for the April 2018 event, rainfall generating these large flows lasts between 6-12 hours. Using a nested storm greater than 24 hours, and more likely 12 hours, is therefore unrealistic. The resulting runoff from such an event would be excessive and not represent large storms in the catchment.

Figure 7 shows for the largest flow event, 29 April 2018. Although rainfall began at midday 28 April, the 'bulk' of the rain did not fall until the 29 April; and then over an 8-hour period. Furthermore, the rainfall event shows that that there was a 'dip' midway through the recorded rainfall; similar to during the March 2017 event. The other rainfall and flow events analysed also demonstrate the quick response time and relatively short duration of the rainfall event.

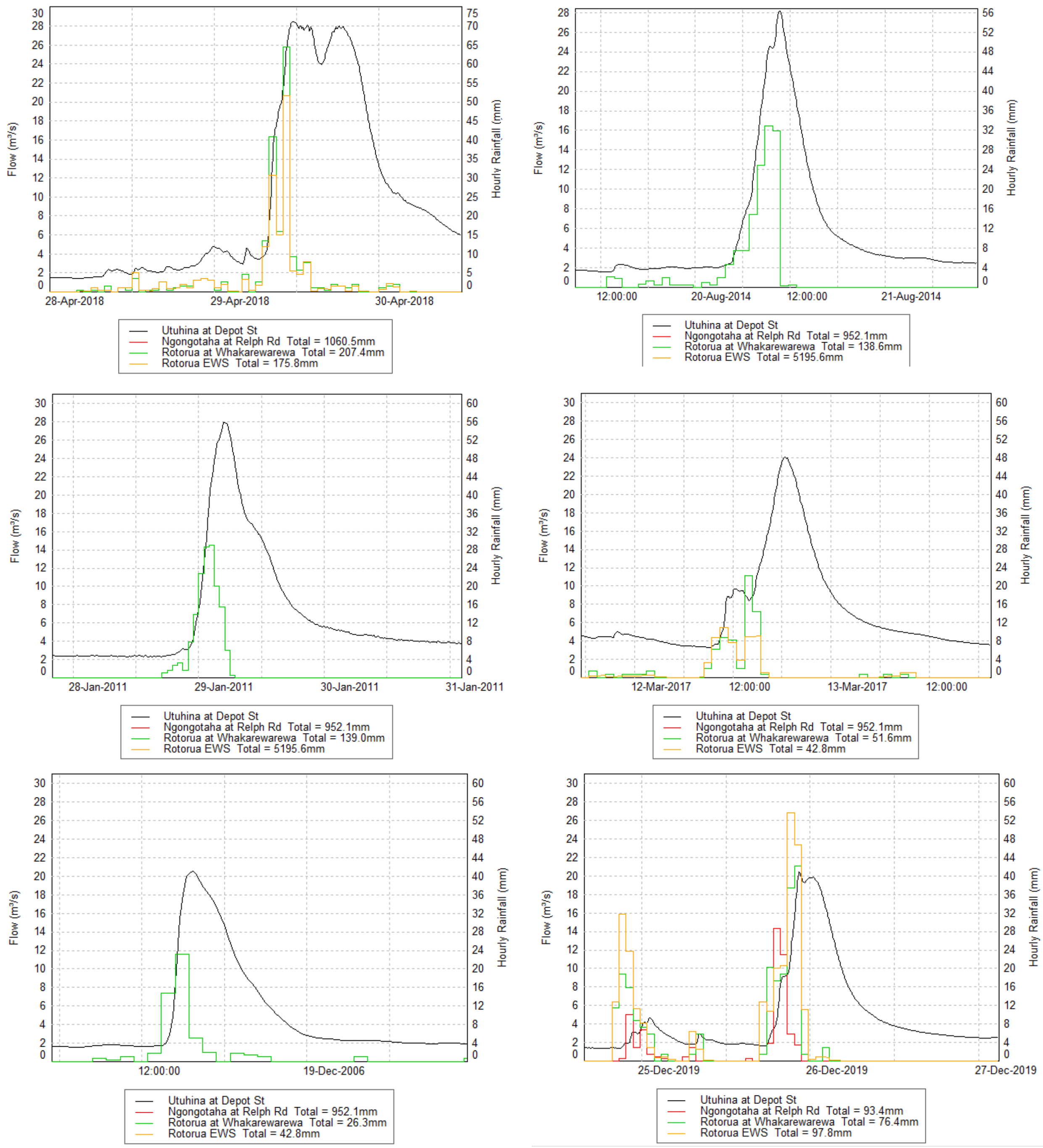

Figure 6: *Comparison of empirical hourly rainfall data to the Utuhina at Depot St flow record for the six largest flow events in the 15-year record.*

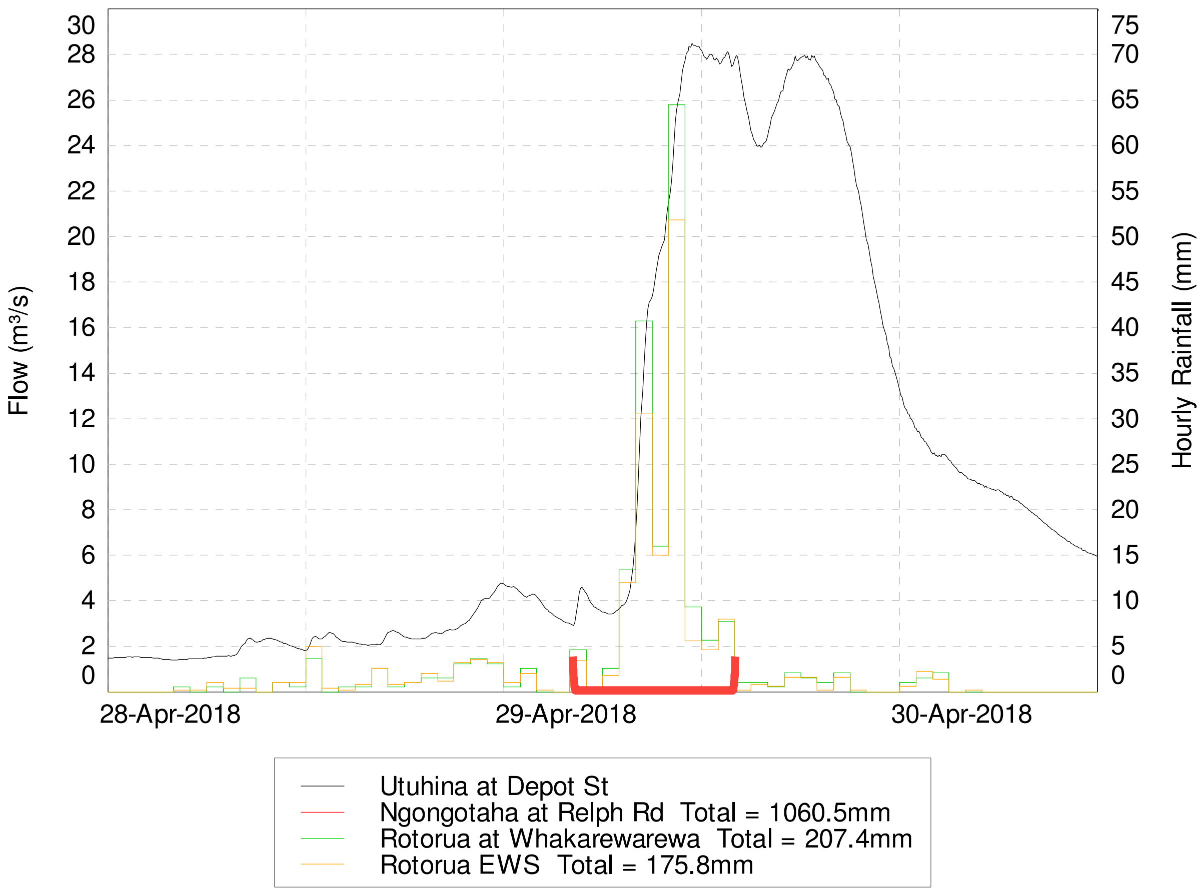

Figure 7: *Largest flood event (29 April 2018) compared with nearest rain gauges to the catchment. Red brackets showing where 'bulk' of rainfall fell in an 8-hour window.*

Rainfall temporal distribution

Five of the six rainfall events were then compared against temporal rainfall distributions for the most common generalised temporal rainfall distributions applied in New Zealand; TP108 (a nested storm), Probable Maximum Precipitation (PMP), and the HIRDS nested storm. Note the 18 December 2006 storm was excluded from the analysis as the data was not recorded at the same level of accuracy as the other 5 storms (was recorded at sporadic 15-min and hourly intervals compared to minute-intervals).

TP108 uses a nested hyetograph where, for any specified duration, from 10-minutes through to 24-hours, the maximum intensity of rainfall for each duration has the same Annual Exceedance Probability (AEP). This 'type-hyetograph', however, does not represent any measured historical rainstorm. When combined with the correct time of concentration, this allows the catchment runoff analysis to operate on the relevant duration embedded within the nested storm. It has been validated for catchments up to 12km², but only for catchments in Auckland. This temporal distribution is similar to the 72-hr nested storm that BOPRC suggest should be used for flood modelling of the Utuhina catchment.

The use of the TP108 distribution tends to produce a much higher peak discharge when compared to either actual storm hyetographs, or the Probable Maximum Precipitation (PMP). Consequently, the use of the TP108 rainfall distribution can lead to conservative design and greater expenditure than required to provide the desired level of service.

The Probable Maximum Precipitation (PMP) temporal distribution, in contrast to TP108, was derived from autographic rainfall charts from North Island storms, using a temporal pattern of average variability, as proposed by Pilgrim et al. (1969 & 1975). This method is aimed at producing, from the recorded intense bursts of a given duration, a temporal pattern with an average variation in intensities, together with a most likely sequence of these varying intensities. The temporal sequences were then 'smoothed' to reduce any inconsistencies within the temporal pattern. The PMP provides temporal distribution for various storm durations; from 1-hour up to 96-hours (Tomlinson & Thompson, 1992). As this method was derived using empirical data, it may be more representative. However, the method did not consider any recent storm data from the project area, where most of the empirical data records only begin in the mid-1980s.

A temporal design storm methodology was developed as part of the recent review of HIRDS. A reconnaissance study was undertaken of storm hyetographs using a conventional analysis of suitably long records from clusters of rain gauges throughout New Zealand. This involved about 70 rain gauges measuring at 15-minute intervals or less and having a long common record length of at least 30 years. These gauges were subsequently split into six regions across the country. It was found that an asymmetric hyperbolic tangent function provided a simple and robust model for cumulative hyetographs when using the empirical data. Although there was little regional difference between the cumulative hyetographs for short durations, variability increased with storm duration. There is no apparent influence of return period on the results. For most cases when a duration of 24-hours or less is used, the generic New Zealand-wide hyetograph varies little from those of the six regions. This is not the case for longer storm durations (NIWA, 2018).

The HIRDS approach uses actual temporal rainfall records like the PMP temporal distribution but includes more recent data and a greater range across the country. However, it requires further investigation for storm durations less than 1 hour, and more gauges with sufficient length of record to make substantive progress in empirical calculations of design hyetographs (NIWA, 2018).

Using the available high-resolution rainfall data from Rotorua at Whakarewarewa, and the only gauge that covers all events, the empirical rainfall data was compared to the different temporal distributions for the 6-hr and 12-hr patterns, based on the storm lengths and response time from the gauge as shown in Figure 6 and Figure 7.

For the 6-hour rainfall events, the empirical data suggests that the nested storm of HIRDS for the North of North Island, New Zealand, is a closer fit, with a relatively uniform distribution that increases in the latter half of the event.

For the 12-hour, the TP108 nested storm is a better fit; however, the largest rainfall events analysed in the Utuhina catchment last less than 8 hours. As the TP108 nested storm has the 'bulk' of the rainfall falling over 10% of the event, using the longer 12-hour distribution is 'skewing' the actual empirical data, as rainfall is not falling over a 12-hour period but during a shorter window of time.

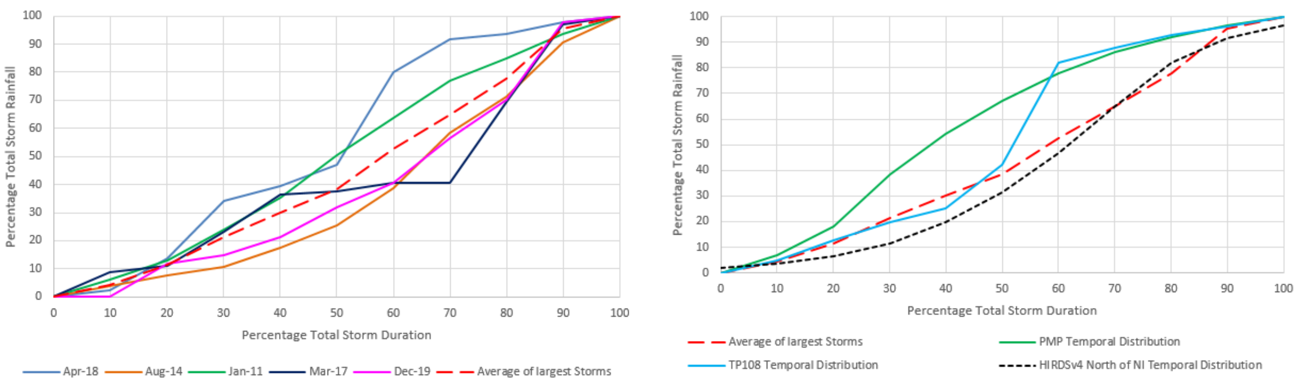

Figure 8: *6-hour storm duration comparison of empirical rainfall data at Rotorua at Whakarewarewa to different temporal distributions.*

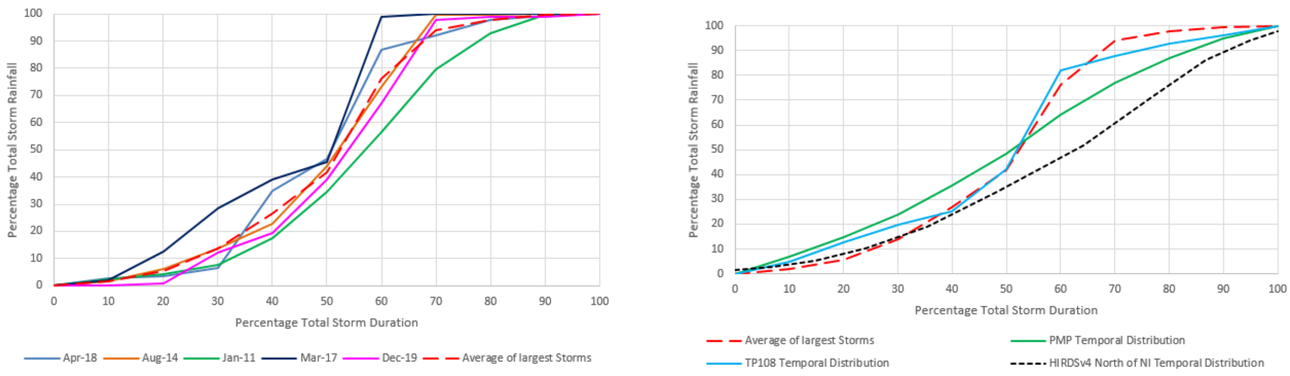

Figure 9: 12-hour Storm duration comparison of empirical rainfall data at Rotorua at Whakarewarewa to different temporal distributions.

Comparison with modelled rainfall

The empirical data from the Rotorua at Whakarewarewa gauge was compared with the rainfall data that was used in the 72-hr nested storm modelled. These rainfall depths were derived by dividing up the larger Utuhina catchment into 42 sub-catchments, and interpolating the design rainfall using HIRDS v4 design depths. These ranged from 190mm to 225mm for the 10% AEP, 259-305mm for the 2% AEP and 290 to 341mm for the 1% AEP. The averages across the entire modelled area were 280mm, 282mm and 317mm respectively. These were compared with the empirical rainfall from the Rotorua gauge, and the HIRDSv4 design rainfall depths at the same location, and are displayed in Table 5.

The design rainfalls currently used in the computational hydraulic model are higher than the empirical data and the HIRDS v4 data for the same site. The Utuhina catchment has higher headwaters than where the rain gauge is located, so it is not unsurprising that these values are higher; from orographic enhancement. These values are likely to be conservative, and may therefore generate greater runoff than would be expected. However, this cannot be quantified without empirical rainfall data in the Utuhina catchment, which is not available at present

Table 5: Comparison of design rainfall depths of those used in modelling, from the empirical record and from HIRDS v4

ARI (years)	AEP (%)	72-hr nested storm design rainfall depths (average) (mm)	Empirical Rotorua at Whakarewarewa design rainfall depths (mm)	HIRDS v4 design rainfall depths at Rotorua at Whakarewarewa (mm)
10	10	208	196	**200**
50	2	282	247	**272**
100	1	317	267	**305**

Summary

The above analysis demonstrates the following:

- A flow gauge within the Utuhina catchment can be used to calibrate a flood model, however, there is no rain gauge within in the catchment;

- The nearest rain gauge, less than 6km to the east, is the Rotorua at Whakarewarewa gauge. Data from this gauge can be used to infer rainfall in the Utuhina catchment, and therefore the rainfall-runoff relationship;
- Flood frequency analysis demonstrated that, although a relatively short record (15 years), there are several events (particularly the large 29 April 2018 flood) that can be used for model calibration. The record can also be used to derive design flows when assessing the potential impact of development on the flood hazard in the catchment;
- The rainfall and flow frequency analyses show no consistent relationship between large rainfall and large floods. For example, the largest flow event has a 7.5% AEP, yet the associated rainfall was a 1.4% AEP event. Antecedent conditions likely mask any simple relationship;
- The response of Utuhina Stream to rainfall is very quick, with the peak flow reached typically between 4-7 hours after the rainfall;
- The rainfall events corresponding to the six largest flows in the Utuhina catchment lasted between 6-12 hours, except for the largest event, 29 April 2018. However, even during that event the 'bulk' of the rainfall and the response of the stream lasted over only 8-hours;
- Comparison with other temporal distributions demonstrated that the rainfall events tend to be short, and begin to 'fit' the typical nested storm events only over longer durations i.e. 12-hours or longer;
- The design rainfalls used in the 72-hr model are higher than those from the empirical record and HIRDS v4 at the same site. It is likely the design rainfalls used are conservative, however, the Utuhina catchment does have higher elevations which likely receive greater rainfall. Without empirical data from the Utuhina catchment it cannot be confirmed if the values used are too high;
- The use of a 72-hour storm nested hyetograph for modelling is likely to produce overconservative results. The local rainfall and flow data shows that storms are typically less than 12 hours, with a quick response time and sharp 'peak' in the resulting hydrograph; and
- Using a longer duration rainfall event to derive runoff in the Utuhina catchment would therefore produce over extreme flows unlikely to occur.

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