REPORT

Tonkin+Taylor

# Rotorua Stormwater Masterplan

## Concept options assessment

**Prepared for**
Rotorua Lakes Council
**Prepared by**
Tonkin & Taylor Ltd
**Date**
April 2021
**Job Number**
1010988.1000.v2

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# Document Control

| **Title: Rotorua Stormwater Masterplan – Concept Options Assessment** | | | | | |
|---|---|---|---|---|---|
| **Date** | **Version** | **Description** | **Prepared by:** | **Reviewed by:** | **Authorised by:** |
| 27/01/2021 | 1 | Draft for client review | J Hodson | M Pennington | D Bouma |
| 08/04/2021 | 2 | Final draft | J Hodson | M Pennington | D Bouma |
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**Distribution:**

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| Rotorua Lakes Council | 1 electronic copy |
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## Appendix D : Cost estimate breakdowns

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# Executive summary

Rotorua Lakes Council (RLC) has engaged Tonkin & Taylor Ltd (T+T) to undertake a stormwater masterplanning process for selected catchments of Rotorua City. The purpose of the Rotorua stormwater masterplan process is to identify, design and construct integrated flood management solutions that will facilitate planned future urban growth across the city. This report summarises the first stage of the stormwater masterplanning process which is a concept options assessment for the major growth areas across the city. The purpose of this first stage is to identify the preferred flood management approaches that will be progressed to subsequent feasibility assessment and design stages.

Because the focus of the project is to facilitate future urban growth, the spatial scope of the study is limited to the following subcatchments where major growth areas have been identified (red subcatchments in Figure E1 below):

- The eastern subcatchments (Subcatchments 1, 2, 3, 4, and 5).
- The Utuhina Catchment (Subcatchments 13, 14 and 15).

The focus of this study is flood management only. Onsite stormwater management issues such as reticulation and water quality treatment will be assessed by developers as part of subdivision design and is outside the scope of this study. Other catchment-wide stormwater management issues such as elevated sediment loads from upstream land use and erosion, channel erosion and nutrient loads into the lake are also outside the scope of this study.

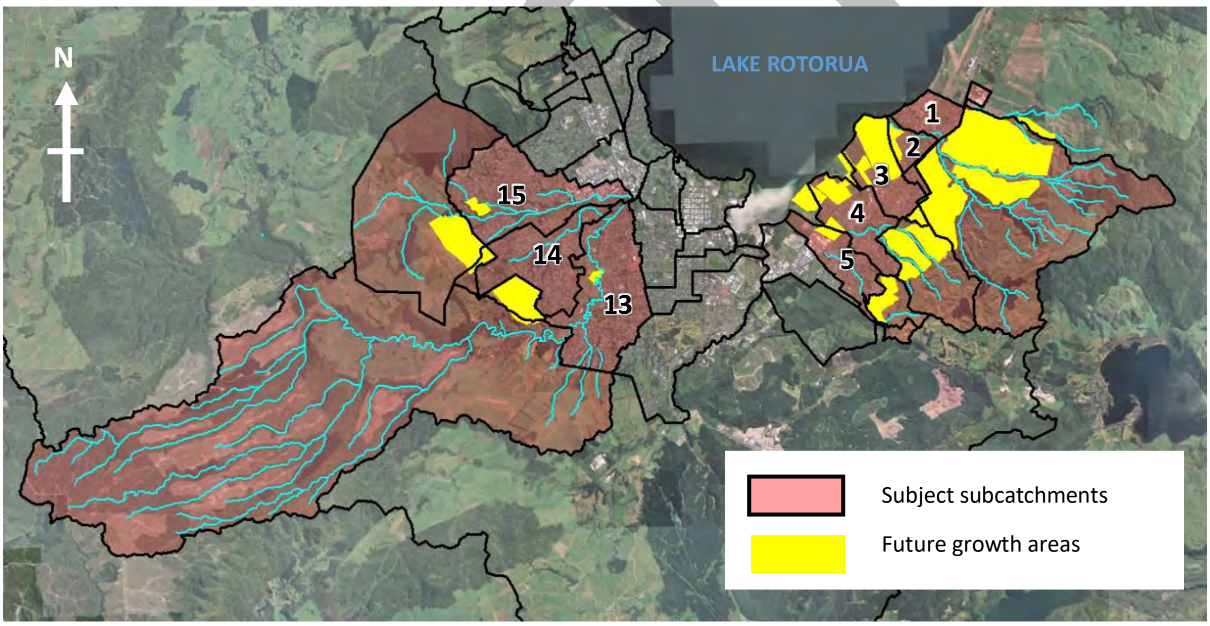


*Figure E1: Spatial scope of the study*

The sizing of the flood mitigation options in this study are based only on an assumed requirement to mitigate for changes in runoff from new urban (i.e. mitigation for growth), as this is the minimum requirement under the Resource Management Act 1991 (RMA) for land use changes. However, mitigation of existing flooding issues and increases in runoff due to climate change and intensification are all relevant factors for RLC's wider stormwater infrastructure and flood management planning and have therefore been given consideration in selection of the preferred option.

The options that were selected to be assessed as part of this study are based on the following general principles:

- Focus on offsite options (i.e. outside the boundaries of the growth cell) and options that address multiple development areas where possible. Isolated onsite options (e.g. detention storage within the growth cells) have already been the subject of prior studies.
- Give preference to increasing existing conveyance capacity, flow diversion or land use controls to reduce/offset runoff before considering attenuation/detention where possible, as these solutions will likely have lower capital costs.
- Lower parts of catchments should adopt a “pass flows forward” approach (i.e. no detention of runoff) where possible. Ideally detention should only be considered further upstream in catchments (in accordance with BOPRC stormwater guidelines).
- Where detention within, or upstream, of a development area is not possible but can be incorporated further downstream, an increase in conveyance capacity may be required downstream to pass unattenuated flows without causing flooding. However, mitigation options should avoid increases in peak flows to existing pipe networks as reticulated network upgrades are generally very expensive and are of limited value in conveying large flood events.
- Where possible, use open channels rather than pipes for conveyance of stormwater runoff.
- Give preference to utilising land already owned by Council rather than private property. Avoid property purchase where possible, but consider where it is cheaper/simpler than other solutions.
- Give preference to solutions that can also address existing flood issues, future climate change and intensification (i.e. in addition to meeting the needs for new development areas).

The preferred flood mitigation approaches and recommendations for next steps are summarised below:

**Subcatchments 1 and 2:**

- Given the current uncertainty in the hydrology and hydraulics associated with these catchments, it is prudent at this stage to assume that detention storage within the future urban growth areas in these catchments will be required. This assumption should be reviewed following collection of further data as recommended below.
- We recommend that further work be undertaken to determine the conveyance capacity of the Waingaehe Stream and bridges from Te Ngae Road to the lake. This work will need to include a survey of the Te Ngae Road bridge, and the Waingaehe Stream downstream of the bridge and detailed hydraulic modelling.
- We recommend that further work be undertaken to confirm the hydrological parameters for the eastern catchments. Collection of site specific rainfall and stream flow data in the eastern catchments will allow a significant reduction in the uncertainty that is inherent in the flood analyses to date.
- Once the existing capacity of the Waingaehe Stream is known and there is more certainty around catchment hydrology, a decision can be made on the preferred approach for flood management. This will include confirming an appropriate balance between provision of detention storage in the growth cells, and utilisation of conveyance capacity, and whether it is possible to divert flow from Subcatchment 1. The optimal result may well be a combination of all three options.
- If upgrade of conveyance capacity downstream of Te Ngae Road is part of the preferred approach, collaboration with BOPRC occuring at an early stage of the project will be required to get acceptance in principle for the proposed approach due to the Waingaehe Sstream being part of a rated flood scheme.

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**Subcatchment 3:**

- For Subcatchment 3 we recommend adopting a pass flows forward approach (i.e. no detention of post-developed flows) with conveyance of primary flows in a new trunk main under Galvin Road and utilising the road network (including the existing open channel if retained) as an overland flowpath.
- We recommend that further work be undertaken to confirm the hydrological parameters that should be used for the eastern catchments as mentioned for Subcatchments 1 and 2.
- Further investigations should be undertaken by RLC around land purchase or easement creation to secure an outlet to the lake at the end of Galvin Road. If the option of using the existing paper road is preferred the effect of this on the Waingaehe Stream should be assessed as part of the future work recommended for Subcatchments 1 and 2.
- The mitigation approach for the top of Subcatchment 3 should be further investigated as part of the plan change structure plan for the Ngati Whakaue Lands.

**Subcatchment 4:**

- For Subcatchment 4 we recommend adopting a pass flows forward approach (i.e. no detention of post-developed flows) for development in the lower catchment (Owhatiura South and Vaughan Road Developments) and providing centralised detention for the Wharenui Development upstream of Morey St (Option 4A).
- This option would involve construction of two flood detention dams upstream of Morey St and upgrades to the existing Owhatiura Eastern Drain and Vaughan Road culvert and a new drain for the developments along Vaughan Road.
- We recommend that further work be undertaken to confirm the hydrological parameters for the eastern catchments as mentioned for Subcatchments 1 and 2.
- As part of subsequent engineering feasibility/preliminary design stage the following issues should be addressed for the preferred option:
  - Consideration of the proposed development layout to ensure that the road/pipe networks will convey 100 year flows to the channels/dams.
  - Geotechnical investigations and engineering feasibility assessments should be undertaken for the proposed dam sites.
  - Optimisation of the dam configuration and whether the storage can be increased to address existing flooding or mitigate the effects of climate change for the Melrose Ave area.
  - Refinement of the concept to ensure that the dams can act independently of one another and to confirm at what stage of development the second dam is required.
  - Slope instability risk to neighbouring properties.
  - A dambreak assessment should be undertaken to determine the required PIC rating of the proposed dams.
  - Confirmation of the flood management requirements that BOPRC will expect to see met in a resource consent application including whether the attenuation to 80% of pre-development flows is required.
  - More detailed hydraulic modelling of the downstream channel should be undertaken to confirm the scope of the required conveyance upgrades.
- We recommend early engagement with Waka Kotahi to discuss conveyance upgrades across Te Ngae Road.

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**Subcatchment 5:**

- For Subcatchment 5 we recommend progressing a decentralised detention approach (Option 5A) to engineering feasibility/preliminary design based on RLC's direction that a coordinated mitigation approach between the landowners will not be feasible.
- This option involves construction of two separate flood detention dams - one dam would be located on private property within the existing stream gully near Link Road while the other dam would be located within the council-owned stream reserve near Shane Place.
- We recommend that further work be undertaken to confirm the hydrological parameters that should be used for the eastern catchments as mentioned for Subcatchments 1 and 2.
- As part of subsequent engineering feasibility/preliminary design stage the following issues should be addressed for the preferred option:
  - Consideration of the proposed development layout and the assessment of the ability of the existing road/pipe network to convey flow to the dam sites.
  - Assess consentability of all proposed dams given the recent National Policy Statement for Freshwater Management 2020.
  - Geotechnical investigations and engineering feasibility assessments should be undertaken for the proposed dam sites.
  - Whether the size of Dam 5-1 can be increased to address existing flooding or mitigate the effects of climate change in the reach downstream of the dam.
  - Slope instability risk to neighbouring properties.
  - A dambreak assessment should be undertaken to determine the required PIC rating of the proposed dams.
  - Confirmation of the flood management requirements that BOPRC will expect to see met in a resource consent application including whether attenuation to 80% of pre-development flows is required.

**Utuhina Catchment:**

- For the Utuhina Catchment we recommend adopting a detention based approach that is a combination of onsite detention (as proposed by WSP/Opus for Pukehangi Heights) and offsite detention (the three dam sites discussed in Section 5.5.1).
- Depending on what flood management objectives are required, the size of these dams may be able to be decreased in subsequent design stages or all of the dams may not be required. Significant opportunity exists for optimisation of the design of these dams due to the interdependencies in their design.
- As part of subsequent engineering feasibility/preliminary design stage the following issues should be addressed for the preferred option:
  - Investigate whether mitigation for existing flooding issues, climate change and/or intensification can be achieved with a combination of onsite and offsite detention. This will require clarification from RLC on what their wider flood management objectives for the Utuhina Catchment are beyond just catering for growth.
  - Assess consentability of of all proposed dams given the recent National Policy Statement for Freshwater Management 2020.
  - Geotechnical investigations and engineering feasibility assessments should be undertaken for the proposed dam sites.
  - Slope instability risk to neighbouring properties.

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- A dambreak assessment should be undertaken to determine the required PIC rating of the proposed dams.
- Confirmation of the flood management requirements that BOPRC will expect to see met in a resource consent application including whether the requirement for attenuation to 80% of pre-development flows is required.

- We recommend that collaboration with BOPRC occurs at an early stage to get acceptance in principle for the proposed approach and to confirm which model should be used to assess the preliminary design (the Greater Utuhina Catchment Model developed by BOPRC or the TUFLOW model developed by T+T).

**This report has been prepared for the exclusive use of our client Rotorua Lakes Council, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement. The mitigation options presented within this report are concept level only and their feasibility requires further investigation and design. Some mitigation options are located within private property, but we note no consultation with landowners have been undertaken as part of this study. The hydrological parameters/methods and the sizing or location of the proposed mitigations should not be relied on for any other context or purpose and this report should not be used as an engineering reference for stormwater/flood management for any proposed development without further consultation with Rotorua Lakes Council.**

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# 1 Introduction

Several growth areas, generally located within the eastern and south-western parts of Rotorua City, are currently proposed within the Lake Rotorua Catchment. Flood management for these catchments has been the subject of numerous prior studies and in some cases localised flood mitigation options have been identified and tested in hydraulic models. Rotorua Lakes Council (RLC) has engaged Tonkin & Taylor Ltd (T+T) to undertake a stormwater masterplanning process on a city-wide scale that will progress flood management enabling works to relieve flood-related impediments to growth within Rotorua.

## 1.1 Purpose

Figure 1.1 below provides a roadmap of how the stormwater masterplan process will generally be undertaken. The purpose of the Rotorua stormwater masterplan project is to identify, design and construct integrated flood management solutions that will facilitate planned future urban growth across the city. This report summarises the first stage of the stormwater masterplanning process which is a concept options assessment for the major growth areas across the city. The purpose of this first stage is to identify the preferred flood management approaches that will be progressed to subsequent feasibility assessment and design stages.

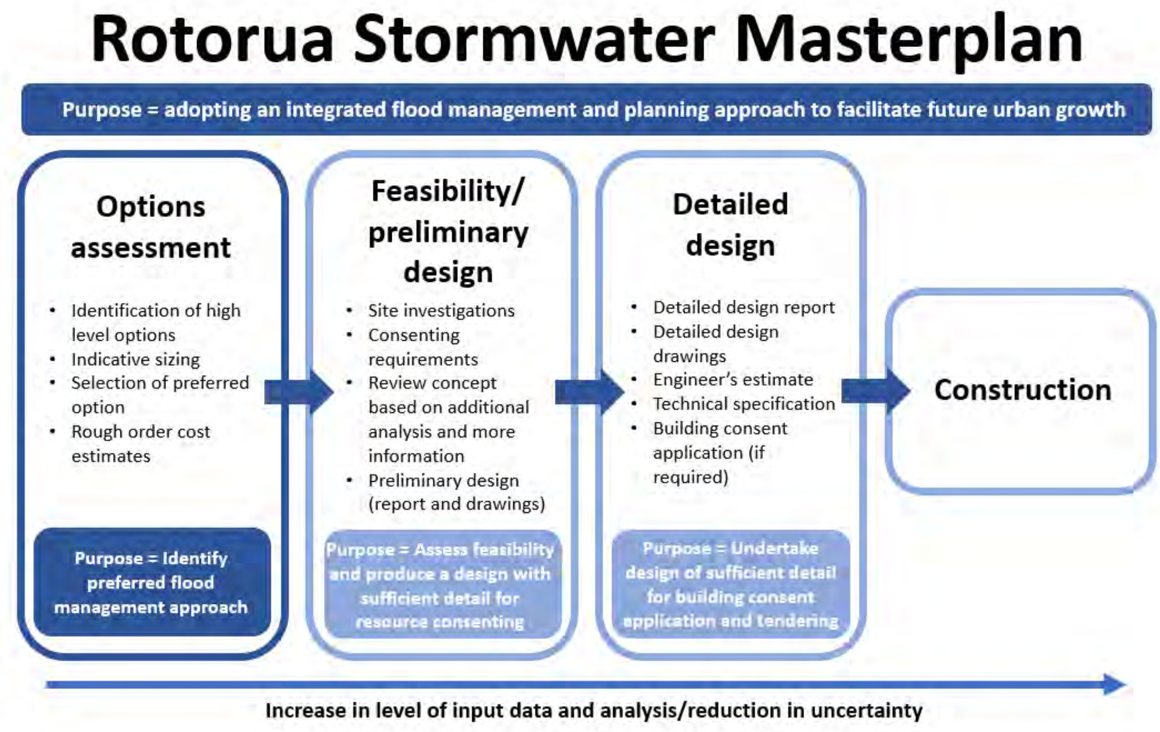


*Figure 1.1: Project roadmap for the stormwater masterplan process*

## 1.2 Scope

Because the focus of the project is to facilitate future urban growth, the spatial scope of the study is limited to the following subcatchments where major growth areas have been identified (red subcatchments in Figure 1.2 below):

- The eastern subcatchments (Subcatchments 1, 2, 3, 4, and 5).
- The Utuhina Catchment (Subcatchments 13, 14 and 15).

The focus of this study is flood management only. Onsite stormwater management issues such as reticulation and water quality treatment will be assessed by developers as part of subdivision design and is outside the scope of this study. Other catchment-wide stormwater management issues such as elevated sediment loads from upstream land use and erosion, channel erosion and nutrient loads into the lake are also outside the scope of this study.

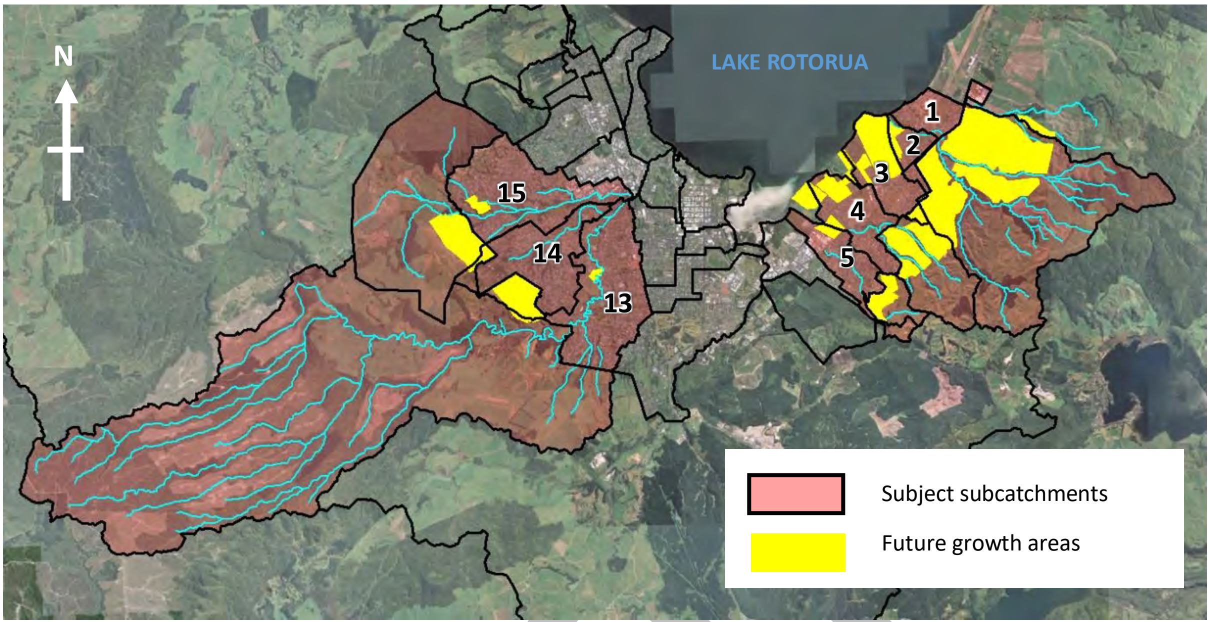


*Figure 1.2: Spatial scope of the masterplan project*

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# 2 Methodology

The general methodology adopted for the assessment of concept options is as follows:

- Identify the development areas for which flood mitigation is to be provided (summarised in Section 3).
- Review previous studies to understand what options have already been assessed by others and the main existing flooding issues in each subcatchment (summarised in Section 4).
- Identify the suite of flood mitigation options to be assessed as part of this study (refer Section 2.2).
- Develop hydraulic and hydrological models to demonstrate the performance and inform the indicative sizing of the mitigation options (refer Section 2.3).
- Prepare rough order cost estimates for each option (refer Section 2.4).
- Qualitatively assess options and make recommendations on preferred options (summarised in Section 5).

## 2.1 Flood management objectives

There are broadly four different objectives that can be achieved for flood alleviation of urban areas. These are:

- Mitigation of existing flooding issues.
- Mitigation for increases in runoff as a result of new urban areas ("greenfield" development).
- Mitigation for increases in runoff as a result of expected climate change.
- Mitigation for increases in runoff as a result of intensification of existing urban areas ("brownfield" development).

The sizing of the flood mitigation options in this study are based only on an assumed requirement to mitigate for changes in runoff from new urban areas (i.e. mitigation for growth), as this is the minimum requirement under the Resource Management Act 1991 (RMA) for land use changes. However, all four objectives are relevant for RLC's wider stormwater infrastructure and flood management planning and have therefore been given consideration in selection of the preferred option. This approach was agreed with RLC at the project inception.

Mitigation for growth for the purposes of this study is taken to mean no increase in catchment wide flood levels on land that is not owned by RLC. However, in some cases peak flow rate mitigation (i.e. the reduction of post-development peak flowrate to match pre-development flowrate) is suitable as a proxy to achieve this objective and has been used to assess the mitigation options. Further discussion on how the pre-developed and post-developed scenarios are defined for this study is provided in Section 2.3.2.

For the purposes of identifying the preferred flood management approach, the flood mitigation options have been assessed for a 100 year average recurrence interval (ARI) storm event (rainfall and lake level combination). In subsequent design stages, consideration will also need to be given to more frequent events for flood management and potentially also less frequent events for meeting safety requirements (e.g. dam safety for detention based solutions).

The flood management objectives that will be required for the preferred flood management approach should be confirmed with Bay of Plenty Regional Council (BOPRC) as the consenting authority in subsequent design stages. This may require the use of different rainfall temporal patterns or durations, return periods (as discussed above), hydrological methods or mitigation targets (such as targeting 80% of pre-developed flowrates).

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## 2.2 Option identification

Identification of the flood mitigation options that are assessed as part of this study has been done through a workshop process attended by RLC and T+T specialists. Figure 2.1 below shows the option identification framework that has been developed as part of this process and applied to each subcatchment to identify possible mitigation options. This framework proposes that there are four main ways that flood effects of urban growth can be addressed to meet the requirements of the RMA: increase conveyance capacity; catchment diversion; attenuation/detention; and land use solutions/property purchase. There are a number of different ways each of these approaches can be implemented, examples of which are bulleted below in Figure 2.1. Not all of these solutions are appropriate for every subcatchment/site, but the framework was used to ensure all possible mitigation options had been considered.

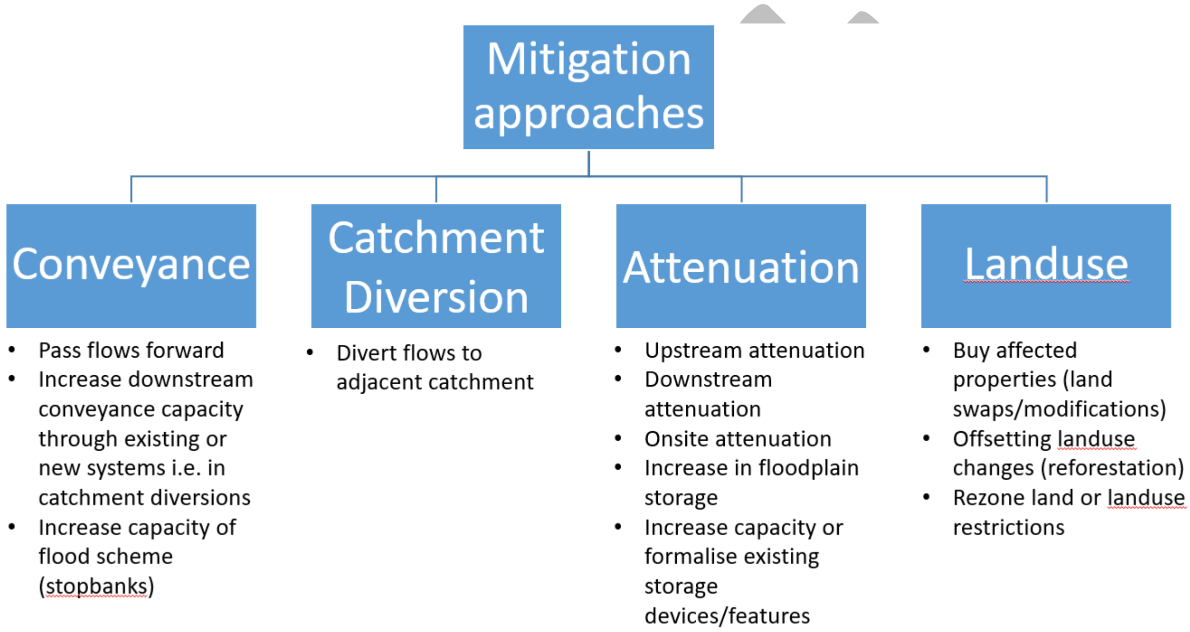


*Figure 2.1: Framework to identify possible mitigation options*

In addition to the framework above, a number of guiding principles were established for selecting the mitigation options that would be assessed:

- Focus on offsite options (i.e. options not located within the subdivision itself) and options that address multiple development areas where possible. Isolated onsite options have already been the subject of prior studies.
- Give preference to increasing existing conveyance capacity, diversion or landuse before considering attenuation/detention where possible, as these solutions will likely have lower capital costs.
- Only consider detention in the upper part of catchments (in accordance with BOPRC stormwater guideline suggestions). Lower parts of catchments should adopt a "pass flows forward" approach (i.e. no detention of runoff) where possible.
- Where detention at source is not adopted (i.e. detention is achieved elsewhere in the catchment), an increase in conveyance capacity may be required downstream to pass unattenuated flows without causing flooding. However, mitigation options should avoid increases in peak flows to existing pipe networks as reticulated network upgrades are generally very expensive and are of limited value in conveying large flood events.

- Where possible, use open channels rather than pipes for conveyance of stormwater runoff.
- Give preference to utilising land already owned by Council rather than private property. Avoid property purchase where possible, but consider where it is cheaper/simpler than other solutions.
- Give preference to solutions that can also address existing flood issues, future climate change and intensification (i.e. in addition to meeting the needs for new development areas).

The mitigation options have been primarily developed to mitigate the effects of development and this is the basis for their indicative sizing. Where the options can be scaled to address the other three objectives (e.g. reduce existing flooding, allow for future climate change, and allow for intensification of existing areas), these opportunities are identified. However, flood mitigation options which do not mitigate the effects of new development areas have not been considered. There may be other options RLC wish to consider to reduce the impacts of existing flood issues but which do not support upstream development, however these are outside the scope of this study.

It is important to note that the flood management approaches considered by this study vary in scale and by the number of potentially affected parties. Some options provide catchment wide solutions, whilst others provide site-specific benefit. This conceptual level stormwater works programme has not yet sought feedback from community, stakeholders or partners and therefore some options may need to adapt as feedback is received in subsequent stages.

## 2.3 Indicative sizing of options

### 2.3.1 Design rainfall

The design rainfall used for this study is based on NIWA's HIRDS v4. The HIRDs design temporal patterns have been used and rainfall depths are based on the RCP8.5 scenario for a 2090 climate change horizon. A nested rainfall profile was not used because this can lead to peak flow estimates that are higher than "typical" rainfall events which have varying return periods across different durations. This can result in outlet sizes that do not provide sufficient attenuation of actual storm events[1].

Four storm durations have been assessed to ensure that interventions are adequately sized for a range of storm profiles. This included the 1 hour, 6 hour, 12 hour and 24 hour HIRDs design storms.

### 2.3.2 Hydrological/hydraulic modelling

A number of different analysis approaches and tools have been used to provide indicative sizing of the proposed mitigation options.

A 2D direct rainfall model of the subcatchments covered by this study has been developed using TUFLOW software. The details of the model build and model results are summarised in the model build report (refer Appendix A). The TUFLOW model has been calibrated to the Utuhina Stream Gauge at Depot Street. Because of the complexities of infiltration in pumiceous soils, the upper rural part of the Utuhina Catchment has been modelled using a lumped catchment approach in HEC-HMS.

We note there is higher uncertainty in the TUFLOW model's use in the eastern subcatchments where no calibration has been undertaken. The reason for the lack of calibration in eastern catchments is the unavailability of reliable pairs of rainfall and flow monitoring data. There is a gauge in the Waingaehe Catchment but the rainfall-runoff response in the data appears inconsistent and so is not

[1] Refer conference paper 'Does Your Detention Meet Your Intention?' by Groves et. al. 2020 Water NZ Stormwater Conference



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considered reliable (also noted in WSP/Opus' 2001 hydrological assessment for the Rotokawa Catchment), in part because there is no rain gauge located in the catchment.

There are also a number of other limitations of the TUFLOW model and recommendations for further refinement which are noted in the model build report. However, the results of the TUFLOW model are considered suitable for the purposes they have been used for within this study.

The TUFLOW model has been used to provide indicative sizing of detention based options in the Utuhina Catchment. Given the presence of large tributary catchments and the large amount of floodplain storage it is important to use the TUFLOW model to assess potential effects such as coincidence of peak flows and volume effects (i.e. where downstream flood levels can increase as a result of increased runoff volume even when peak flowrates are unchanged).

HEC-HMS models have been used to provide indicative sizing for detention based options in the eastern subcatchments. That is because there is higher uncertainty in the TUFLOW model results for these catchments due to the lack of calibration. These catchments are also generally more linear (i.e. no large tributary catchments) and generally do not have significant floodplain storage so matching pre-developed peak flows is a suitable proxy for not increasing downstream flood levels. HEC-HMS is also a more suitable tool for assessing offline detention devices which are proposed for some of the eastern subcatchments. This is because it is more difficult in direct rainfall models to redirect overland flow to proposed detention areas in the model when they are not natural drainage points for a development area. That is, to simulate the effects of development in a direct rainfall model a "post-development" digital elevation model is needed, whereas when using a lumped catchment approach this does not need to be generated. A summary of the HEC-HMS model results for the various ponds and dams discussed in Section 5 is presented in Appendix B.

For conveyance based options, peak flowrate estimates have been made using a combination of the TUFLOW model and the Waikato Regional Council's runoff modelling guideline (TR2020/06). Required channel sizes have been estimated based on uniform flow calculations using Manning's equation.

While the hydrological methods used are considered appropriate for the purposes of this study (i.e. identification of the preferred flood management approaches), the hydrological method and analysis tools used to design the preferred options should be reviewed in subsequent design stages to ensure they are suitable for the specific context.

### 2.3.3 Development scenarios

In order to isolate the effects of future urban growth on flooding, the "pre-development" scenario is defined as present day land use with future climate and maximum permitted intensification of existing urban areas (as per the District Plan). That is, there is an inbuilt assumption that all urban land has been developed to the maximum allowable state, which is not necessarily the current case. The "post-developed" scenario is defined as future land use scenario where all currently zoned areas (as per the District Plan) have been developed as well as any areas where plan changes are underway or pending. The post-development scenario also has future climate and maximum permitted intensification. The scenarios are summarised in Table 2.1.

The same base digital elevation model (DEM) has been used in both the pre-development and post-development scenarios and is based on RLC's 2011 and 2018 LiDAR survey. In most cases the proposed earthworks associated with the developments that have been assessed are unknown and these areas have just been assumed to have no changes in ground levels. However, in some cases where it is known that earthworks are proposed that would affect floodplain storage or change overland flowpaths, these are represented in the post-development scenario (these exceptions are noted in Section 5).



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**Table 2.1: Summary of pre-development and post-development scenarios**

| Scenario | Land use | Climate scenario |
|---|---|---|
| Pre-development scenario | Present day (2020) urban extents (with full intensification of all existing urban areas) | Future 2090 climate based on RCP8.5 |
| Post-development scenario | Future land use based on all currently consented urban areas and impending/current plan change applications (with full intensification of all existing urban areas) | Future 2090 climate based on RCP8.5 |

For flood mitigation solutions that involve detention the assessment of mitigation is relative. That is, the post-development flood levels/flows are compared to pre-development flood levels/flows. As discussed in Section 2.1, detention-based options in this study are at this stage only sized to prevent flood levels/flows increasing as a result of future urban growth. The predicted increases in runoff from climate change and intensification in the existing urban areas has not been allowed for in the detention based options developed in this study (based on how the pre-development scenario is defined), but should be considered when proceeding with future feasibility and design stages as there may be opportunity to increase the size of some detention options to address the wider flood mitigation criteria (i.e. considerations other than simply allowing for the effects of development of urban growth cells).

However, for flood mitigation solutions that involve conveyance they are sized to convey the full post-development peak flowrates regardless of pre-development peak flows as is typically done for design of conveyance networks. Therefore, conveyance based solutions account for predicted increases in flows due to climate change and intensification.

## 2.4 Rough order cost estimates

Rough order cost estimates for each option have been prepared to provide an understanding of indicative cost. The estimates are generally broken down into a number of key construction elements with an item for minor and unscheduled options taken as a percentage of the total physical work costs. Estimates of quantities are based on the concepts described in Section 5 and estimates of rates are based on recently tendered rates for similar items (where available) and engineering judgement.

A contingency of 40% has been applied to the physical works costs due to the high-level nature of the estimate (i.e. estimates are based on concepts - no design has been done at this stage). Allowance for professional fees (design, consenting and construction monitoring) has also been included as a percentage of the physical works cost. The cost estimate totals are presented with a range of -10% to +30% to reflect uncertainty in estimating project costs.

The full cost estimate breakdowns for each option are included in Appendix D.

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# 3 Proposed development

This section summarises the future urban growth areas which are the focus of the stormwater masterplan and for which mitigation options have been assessed. The future urban growth areas are based on areas that are currently zoned in the Rotorua Lakes District Plan but are undeveloped as well as areas that currently are, or will be, subject to plan change applications. For plan change areas, assumed zoning for this study is based on latest available layout plans. The various development areas are at different stages of development and the status of each at the time of writing is noted below.

## 3.1 Eastern subcatchments

Proposed greenfield development within the eastern subcatchments is generally located alongside Lake Rotorua and to the south-east of Te Ngae Road (SH30) and totals approximately 700 ha. The key development areas are shown in Figure 3.1 and are as follows:

- Wharenui Block – Proposed residential subdivision to the south-east of Morey Street in Subcatchment 4 already zoned under the District Plan. Harrison Grierson (HG) is currently undertaking subdivision and dry detention basin design for Stages 1-3 on behalf of Whakaue Property Trust. The Wharenui Block contains 10 potential development stages in total. Subdivision consent has been granted for Stage 1 and a stormwater basin to provide attenuation for Stage 1 has already been constructed.
- Ngati Whakaue Lands – Refers to the remaining area to the south-east of SH30 within Subcatchments 1 and 2. This area is also owned by Ngati Whakaue and will be a mix of residential, rural residential and commercial/industrial. The proposed zoning for this area was based on the alternative business zone arrangement shown on Boffa Miskell's Rotorua Upper Eastside Structure Plan dated May 2020 (refer Appendix C). This area is currently zoned as rural and will require a plan change.
- Owhatiura South – Proposed 15 ha residential and industrial subdivision north of SH30 on the boundary of Subcatchments 4 and 5. Some initial layouts have been provided to RLC that include a stormwater attenuation pond but subdivision consent has not yet been applied for (refer Appendix C).
- Link Road/Spence Block – Proposed 30 ha residential developments in the upper part of Catchment 5. These are currently proposed as two separate developments by different landowners. Discussions between RLC and the landowners have been underway about the intention to develop this land, but it is our understanding that no development layouts have been produced to date.

The remaining future urban areas are currently zoned areas that have not yet been developed. These include the rural and residential zoned areas north of Vaughan Road and the residential zoned area in the upper part of Subcatchment 5 (Link Rd and Spence Block).

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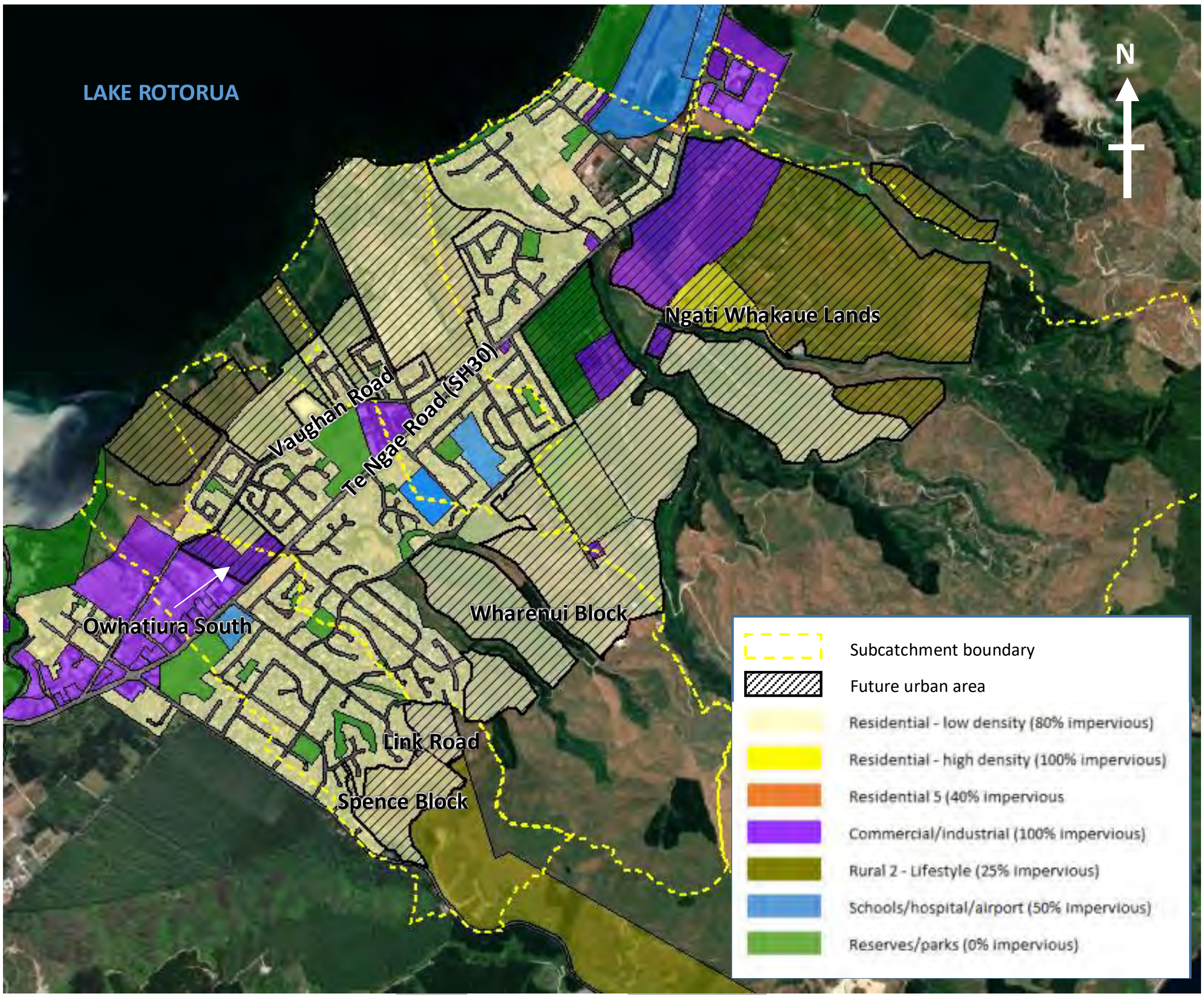


*Figure 3.1: Proposed development in the eastern subcatchments*

## 3.2 Utuhina Catchment

Proposed greenfield development within the Utuhina Catchment is generally located to the south east of the city and totals approximately 160 ha. The key development areas are shown in Figure 3.2 and are as follows:

- Pukehangi Heights – Proposed 138 ha residential development on the western side of Pukehangi Road. This includes three different developments – Te Arawa Group Holdings (TAGH) and Hunts Farm blocks in the Otamatea Subcatchment (Subcatchment 14) and the Sunny Downs block in the Mangakakahi Subcatchment (Subcatchment 15). RLC has lodged a plan change application for the rezoning of this land from rural. The plan change hearing for this development had just concluded at the time of writing. The plan change application is based on a flood management approach developed by WSP/Opus using dry detention basins that would be located within the site[2]. The structure plan for Pukehangi Heights is shown in Appendix C.
- Freedom Village development – Proposed 7 ha retirement village being developed by Rotorua Lakes Land Holdings Ltd. WSP/Opus completed an assessment of the required pond size to

[2] PC2 – Pukehangi Heights Stormwater Report. Prepared by WSP/Opus for Rotorua Lakes Council. Dated 12 August 2020. Revision 1.

provide attenuation[3]. However, it is our understanding that this development is not part of the Pukehangi plan change application, despite the fact this land is not currently zoned for residential development.

- Diamond Street – Proposed 10 ha residential development in the Mangakakahi Subcatchment (Subcatchment 15). This area is currently zoned 'Residential 1' in the District Plan. Cheal is undertaking the subdivision design and a subdivision consent application has been made to RLC. Cheal is also designing an onsite attenuation solution involving a stormwater basin.
- Titoki Place – Proposed 4 ha Kainga Ora development in the Utuhina Catchment (Subcatchment 13). A preliminary development plan has been provided to RLC but a subdivision consent has not yet been lodged. T+T has prepared a stormwater management plan for this development on behalf of RLC for Kainga Ora.

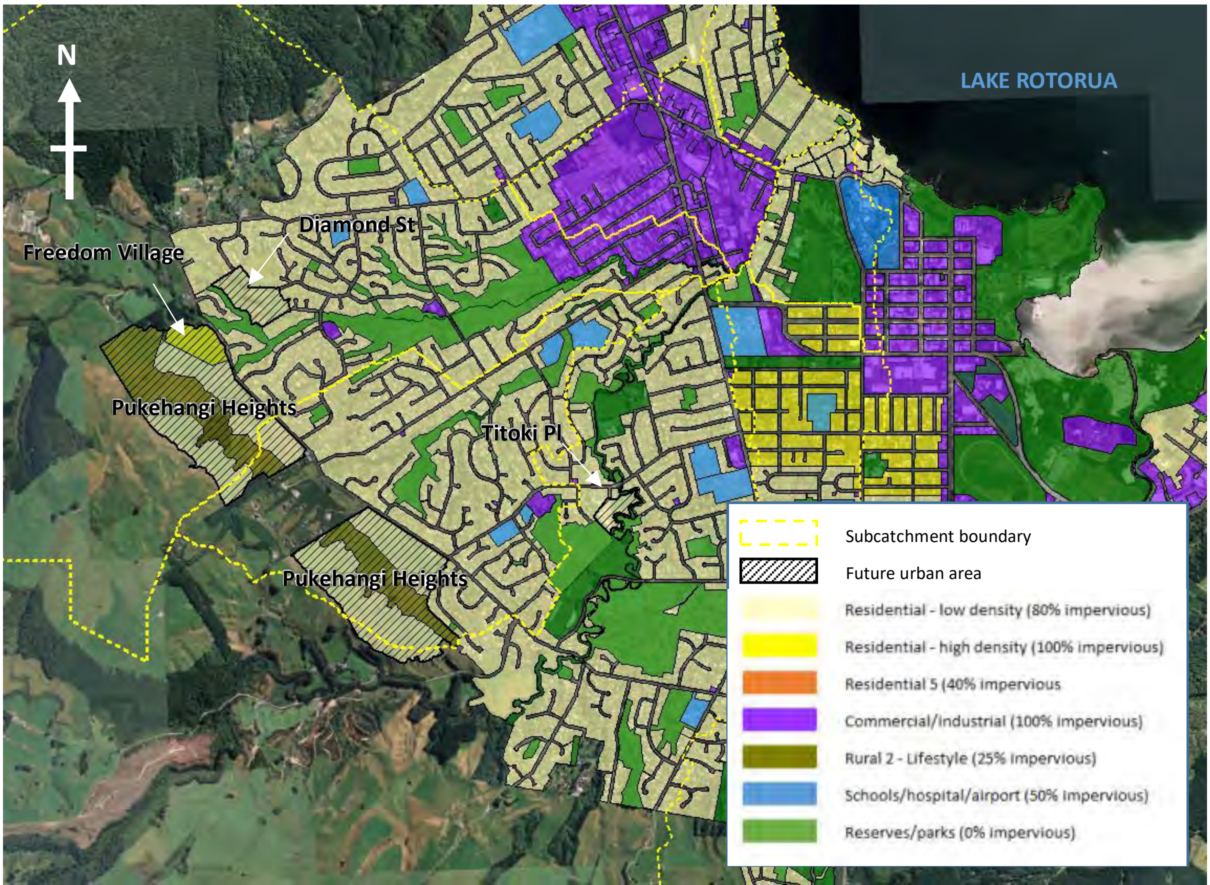


*Figure 3.2: Proposed development in the Utuhina Catchment*

[3] Memorandum titled 'Pukehangi Road Site Development Stormwater Ponds Flood Modelling'. From Avinash Gangurde (WSP/Opus) to Peter Moodie (Lysaght). Dated 1 May 2020.

# 4 Existing flood issues and previous studies

This section presents a brief description of each subcatchment, its key existing flooding issues and a summary of optioneering that has been undertaken to date.

The information regarding existing flood issues has been compiled from RLC's 2010 overview report for their urban subcatchments[4] and the city-wide catchment management plan[5].

## 4.1 Subcatchments 1 and 2

Subcatchments 1 and 2 are located to the south-east of Lake Rotorua, with drainage generally flowing north-west into the lake. These subcatchments have a total area of some 1420 hectares, with about 180 ha of this being in predominantly urban land use. The remaining 1240 ha is currently in rural land use, with parts of this being the subject of future urban growth. This urban/rural split is shown in Figure 4.1. Subcatchment 1 includes the rural area that drains to Lake Rotorua via the open channel known as Bridal Stream (immediately to the south of the airport) and constitutes approximately 220 ha of the combined catchment. Subcatchment 2 is the rural area that drains to Lake Rotorua via the Waingaehe Stream and constitutes approximately 1200 ha of the combined catchment. The flood performance of the Waingaehe Stream is controlled by a flood scheme located in the downstream reach that is administered by BOPRC.

The key existing flood issues and key drainage features for these subcatchments are summarised below:

1. Surface flooding associated with high lake levels occurs at lake edge properties.
2. There has been some isolated, small-scale flooding from streams at Holdens Bay.
3. The pipes exiting into Lake Rotorua are subjected to siltation restricting the flow capacity and causing local flooding from time to time at the lower reaches. High lake levels during winter further restrict the hydraulic capacities of these stormwater pipes.
4. High groundwater tables exist throughout the urban part of Subcatchment 1 (specifically along the lake front and around Cooper and Willow Avenues) and in the centre of the urban part of Subcatchment 2. High groundwater in Subcatchment 2 has required extensive sub-soil drains and a pump station to be installed.
5. There is an existing detention structure known as "Gee Block dam" on the southern side of SH30 that provides some informal attenuation.

The locations of these flooding hotspots are shown in Figure 4.1 below.

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[4] Overview of 29 Urban Stormwater Catchments. Prepared by Rotorua Lakes Council. Dated 19 December 2017. Report reference RDC-505444.

[5] Rotorua Urban Area Stormwater Catchment Management Plan (Working Draft for Consultation). Prepared by Rotorua Lakes Council. Date Unknown. Report reference RDC-834908.

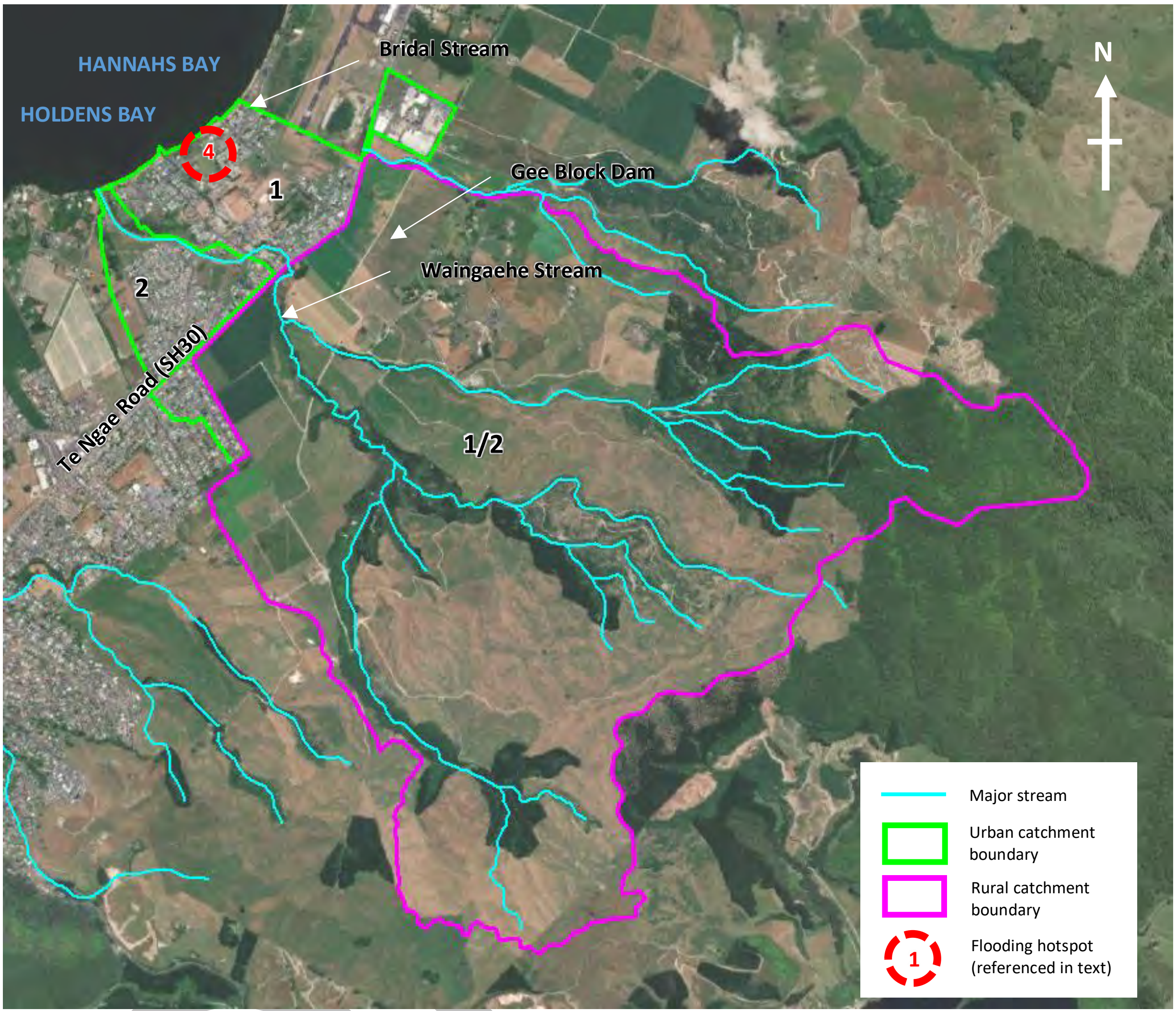


*Figure 4.1: Subcatchments 1 and 2 extents and known flooding hotspots*

It is our understanding that RLC has not undertaken any detailed flood modelling within these catchments. However, BOPRC did undertake a review of the capacity of the Waingaehe Stream in 2008[6]. This study estimated the present day (i.e. no climate change and current land use) 100 year ARI peak flow to be approximately 17 $m^3/s$. Following on from this work Beca also prepared a Catchment Management Plan for Ngati Whakaue Tribal Lands for this catchment in 2009[7] and used an InfoWorks model to assess the catchment flows. The flood management approach proposed by Beca for Subcatchment 1 was to construct one offline pond (refer Figure 4.2) and for Subcatchment 2 the proposed approach was to construct five online dams and five offline ponds to attenuate peak flows to pre-developed levels (refer Figure 4.3). For both subcatchments it was proposed that all lifestyle blocks would have to provide 10 $m^3$ of roof runoff storage.

[6] Waingaehe Stream Hydraulic Capacity Review, Jonathan Freeman, Engineering Hydrologist & Peter West, Environmental Engineer, BOPRC, February 2008
[7] Ngati Whakaue Catchment Management Plan. Prepared for Ngati Whakaue Tribal Lands. Dated 16 October 2009. Revision D.

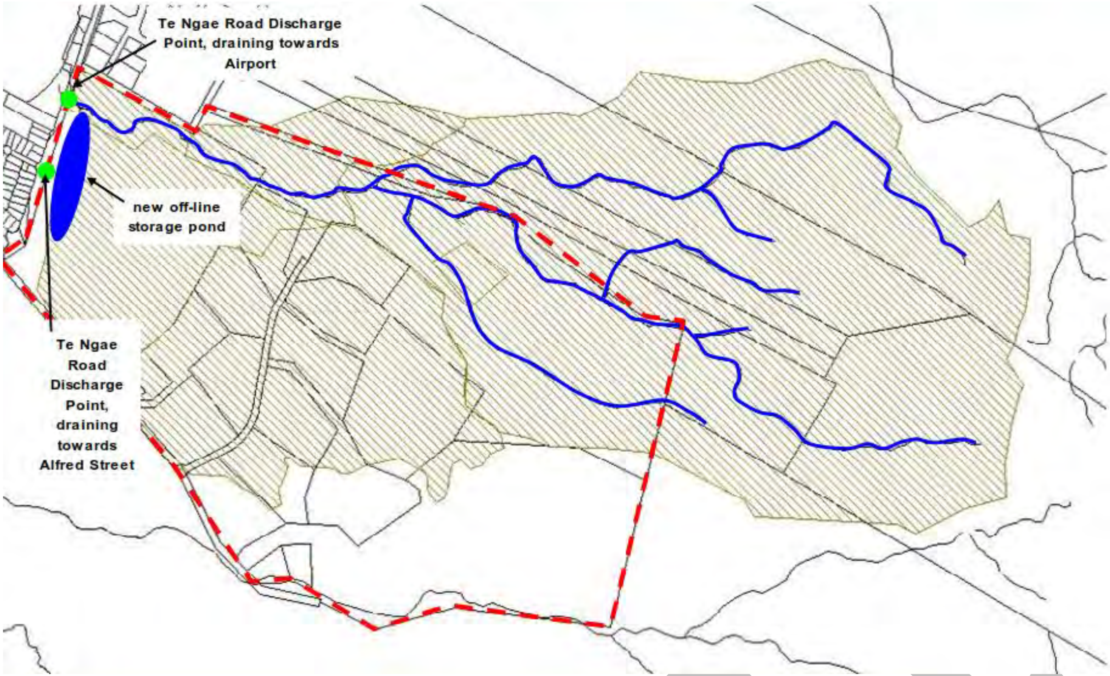


*Figure 4.2: Proposed pond location for Subcatchment 1 (Beca, 2009)*

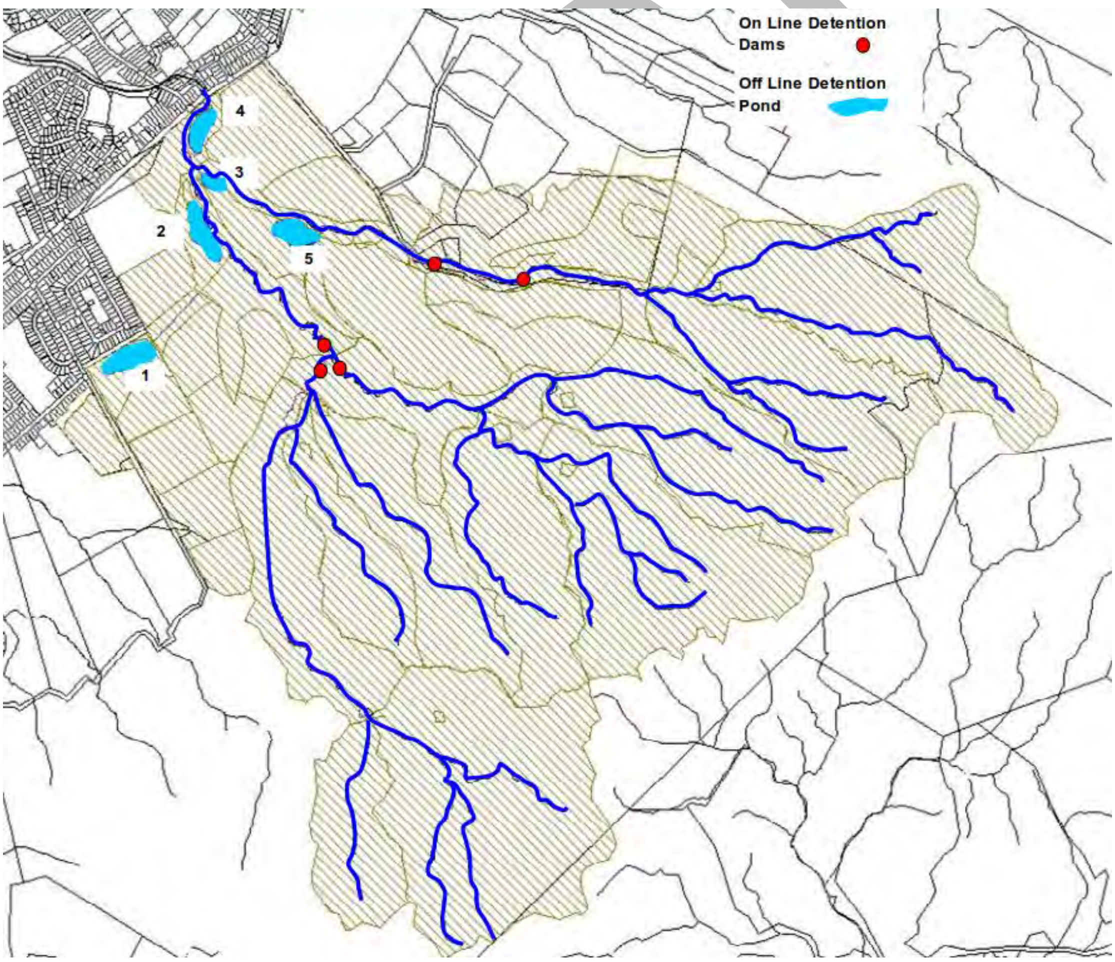


*Figure 4.3: Proposed pond/dam locations for Subcatchment 2 (Beca, 2009)*

## 4.2 Subcatchment 3

Subcatchment 3 is located to the south-east of Lake Rotorua, with drainage generally flowing north-west into the lake. This subcatchment has a total area of some 200 ha, with about 190 ha of this being predominantly classed as urban land use, although a large portion of this area still remains undeveloped. This urban/rural split is shown in Figure 4.5.

Subcatchment 3 has various stormwater reticulation networks that generally discharge to one of the three drains that flow northward to Lake Rotorua. The main drain and pipe system along Owhata Rd conveys most of the stormwater from the catchment to the lake. Five culverts cross SH30 conveying runoff from the top of the catchment down to the lake.

The key existing flood issues for Subcatchment 3 include:

1 The stormwater pipelines and culverts at Reeve Rd and Te Ngae Rd have a history of surcharging during high rainfall events with minor surface flooding of the road and surrounding property.
2 The houses and road on the corner of Galvin Rd and Te Roro O Te Rangi Roadway experiences regular flooding from runoff due to inadequate management of stormwater from the higher lying farmland on the southern side.

The locations of these flooding hotspots are shown in Figure 4.5 below.

It is our understanding that RLC has not undertaken any detailed flood modelling within this catchment and that no optioneering for stormwater/flood management has been undertaken to date by other consultants.

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*Figure 4.4: Subcatchment 3 extent and known flooding hotspots*

## 4.3 Subcatchment 4

Catchment 4 is located to the south-east of Lake Rotorua, with drainage generally flowing north-west into the lake. The catchment has a total area of some 450 hectares, with about 190 ha of this being in predominantly urban land use. The remaining 260 ha is currently in rural land use, with parts of this being the subject of future urban growth. This urban/rural split is shown in Figure 4.5. Much of the rural area has been considered for urban growth, although parts of the area indicated as urban are currently undeveloped and have also been considered for future growth.

Drainage to the south of Morey Street currently consists of ephemeral flow paths which converge to two main gullies which are culverted under Morey Street. Downstream of Morey Street, flow from the upper catchment converges and is conveyed through the Melrose Ave area via a timber lined drain. The Melrose Ave drain discharges under SH30 via two box culverts and then is conveyed in a grassed open channel to the lake passing through two more large culverts at Vaughan Road and Carroll Place.

The key existing flood issues for Subcatchment 4 include:

1 The Melrose Ave Drain is under capacity and flooding occurs along the reach of the drain on average every 10 years.

2 The main open drain from Te Ngae to Vaughan Rd cannot contain the flow and overland flow occurs.

3 The Warwick Drive – Lynwood Place overland flow path is under capacity and flooding occurs.

4 Significant ponding occurs on the corner of Tennyson Drive and Vaughan Road.

5 The Eastlake Drain between Vaughan Road and Carroll Place is at capacity.

The locations of these flooding hotspots are shown in Figure 4.5 below.

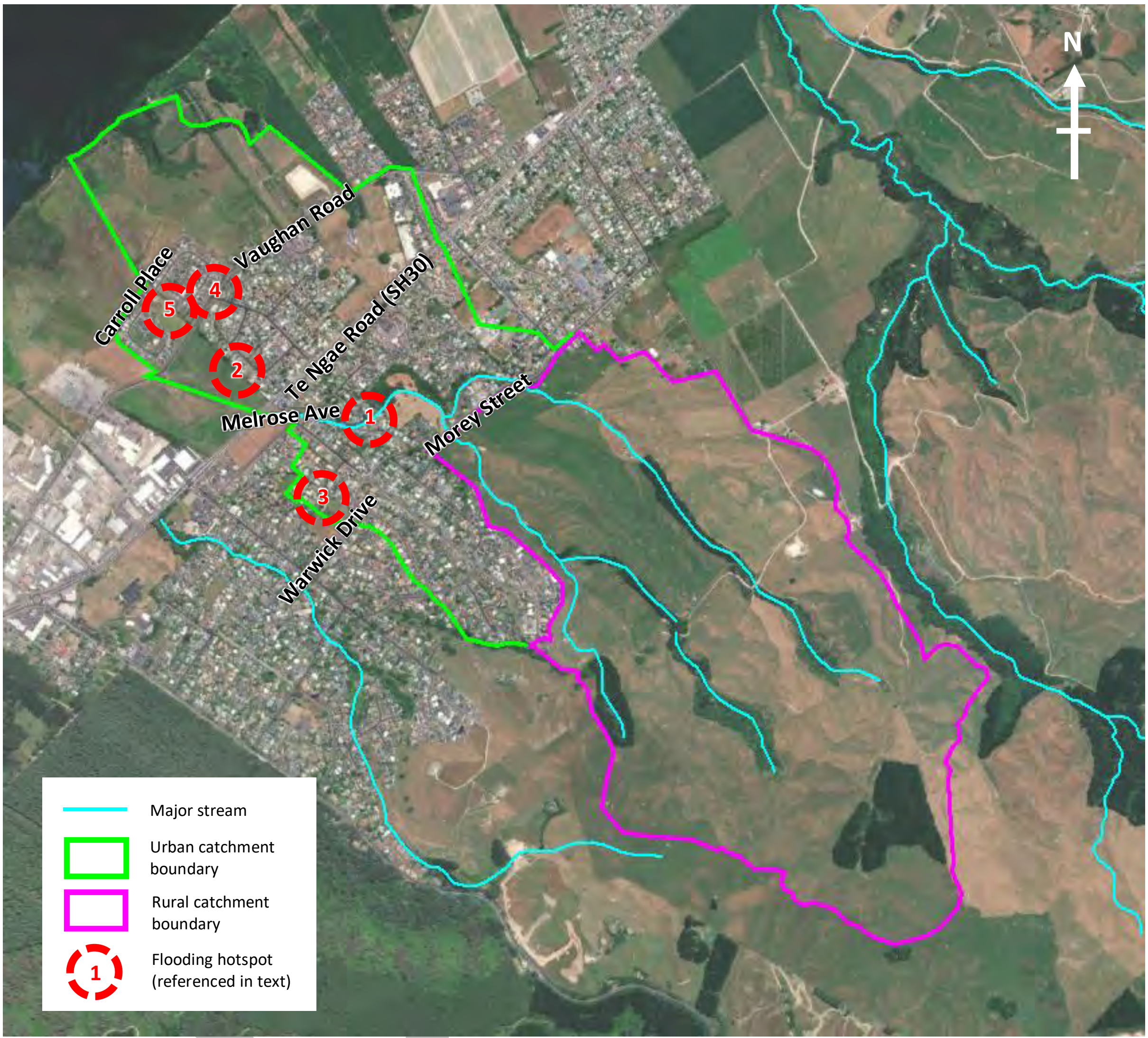


*Figure 4.5: Subcatchment 4 extent and known flooding hotspots*

Subcatchment 4 was assessed in Beca's 2009 Catchment Management Plan for Ngati Whakaue Tribal Lands. The flood management approach proposed by Beca was to construct three online detention dams and nine offline detention ponds to attenuate peak flows to pre-development levels (refer Figure 4.6). It was proposed that all lifestyle blocks would have to provide 10 $m^3$ of roof runoff storage.

In 2017 WSP/Opus was engaged by RLC to develop a InfoWorks ICM hydraulic model for Subcatchment 4 and identify high level options for addressing existing flooding. The options identified by WSP/Opus included detention storage in the gullies upstream of Morey Street and detention downstream of Morey Street (within 28 Morey Street)[8].

[8] Catchment 4 Stormwater Model Sensitivity Testing and Option Modelling. Prepared by WSP/Opus for Rotorua Lakes Council. Dated October 2017. Report reference 0300-OP01-3C142304.

HG is currently undertaking subdivision and stormwater design for the first three stages of the development upstream of Morey Street. The proposed solution is similar to the concept from the Beca 2009 Catchment Management Plan with three dry detention basins providing flood management and water quality treatment for Stages 1-3 (refer Figure 4.7)[9]. Basin 1 has already been constructed. It is noted that these dry detention basins are being designed to attenuate to 80% of pre-developed peak flows.

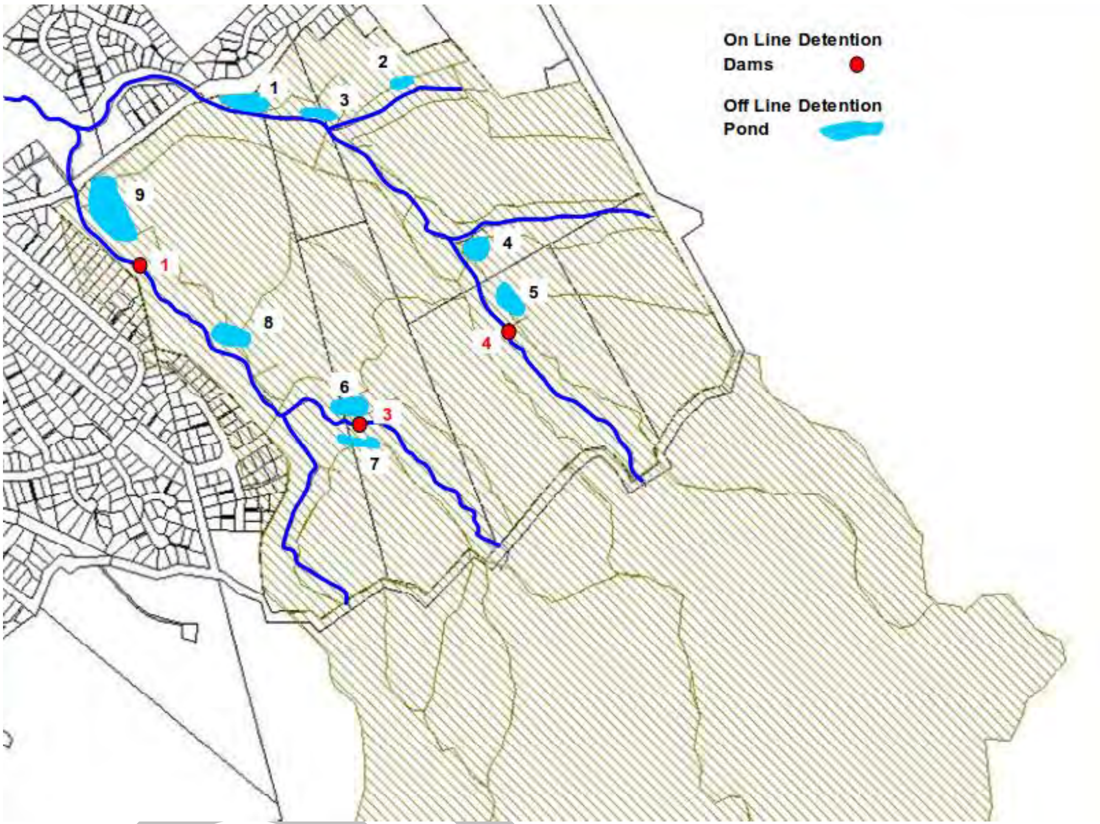


*Figure 4.6: Proposed pond/dam locations (Beca, 2009)*

[9] Wharenui Road, Rotorua – Detailed Design Stormwater Management Plan. Prepared by HG for Whakaue Property Trust. Dated August 2020. Report reference R001v8-AK144247-SMP-JMSC.

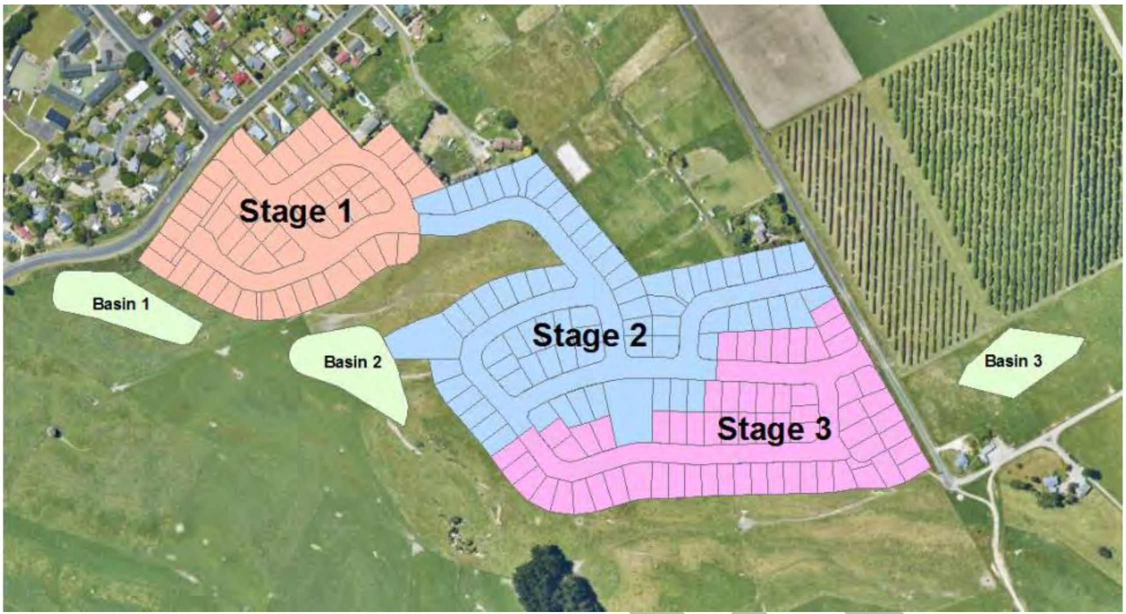


*Figure 4.7: Proposed dry detention basin locations (HG, 2020)*

## 4.4 Subcatchment 5

Catchment 5 is located to the south-east of Lake Rotorua, with drainage generally flowing north-west into the lake. The catchment has a total area of some 250 ha, with about 150 ha of this being in predominantly urban land use. The remaining 100 ha is currently in rural land use, with parts of this being the subject of future urban growth. This urban/rural split is shown in Figure 4.8.

The drainage network in Subcatchment 5 generally consists of localised pipe networks draining to an open channel known as the Selwyn-Larcy Drain. This drain varies in size and construction material and is generally located on private property with culverts at road crossings.

The key existing flood issues for Subcatchment 5 include:

6 The inlet screen of the pipe-culverts under Te Ngae Rd regularly blocks during major rain events due to large amount of debris from native bush just upstream of the inlet. Resultant flooding affects Neil Hunt Park, Lynmore School, and Te Ngae Rd. This also affects Larcy Road.

7 The Selwyn-Larcy Drain is timber lined for the last 70 m. The balance of this drain is poorly formed and its level of service is limited. Access is restricted, and residents regularly dump garden refuse near the stream. Flooding in this area has been observed about every five years.

8 Significant flooding of the industrial area downstream of Vaughan Rd including 70 Vaughan Rd (Infracore site) and the two residential properties at 80 and 82 Vaughan Rd.

The locations of these flooding hotspots are shown in Figure 4.8 below.

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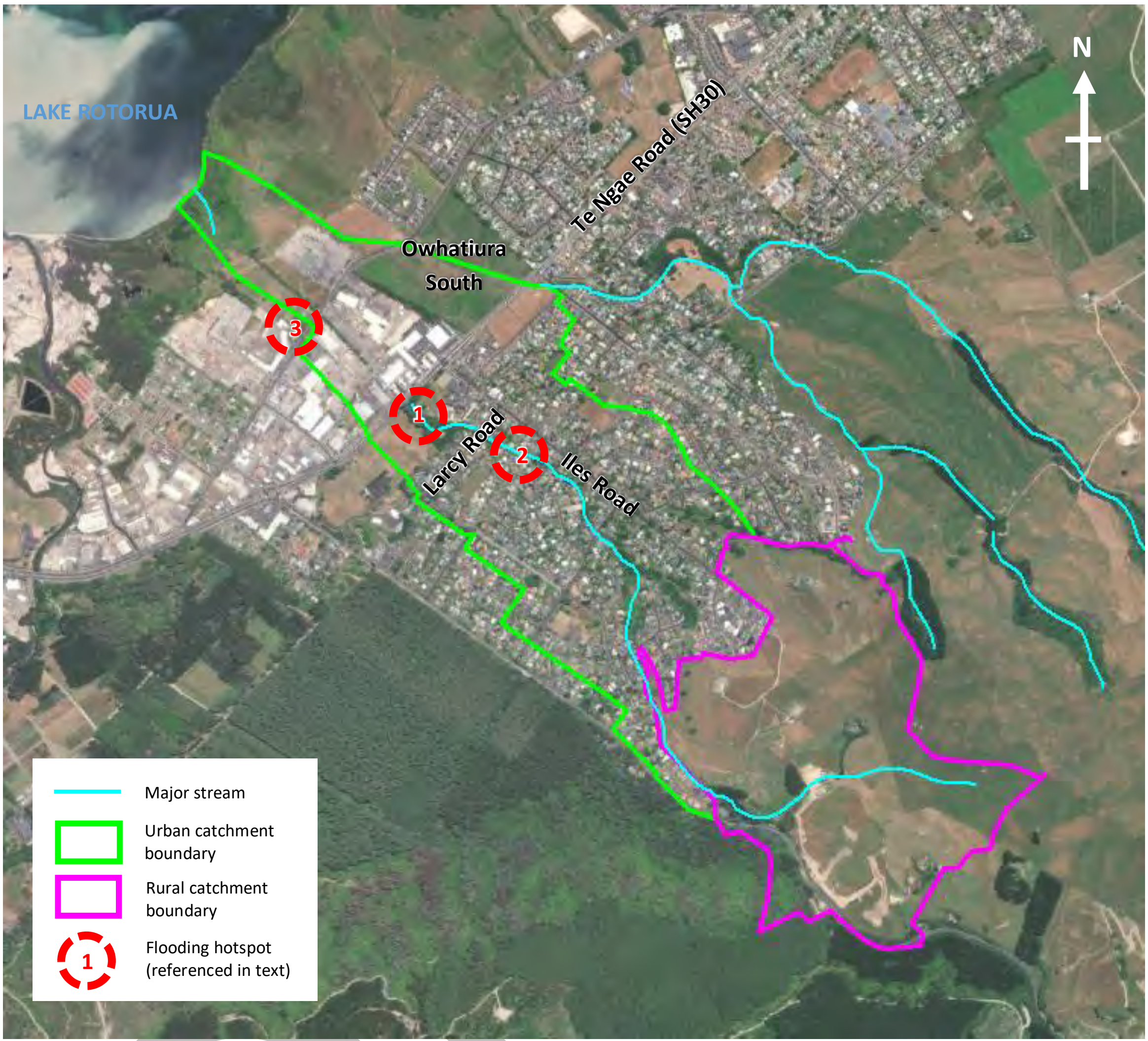


*Figure 4.8: Subcatchment 5 extent and known flooding hotspots*

In 2016 WSP/Opus was engaged by RLC to develop an InfoWorks ICM hydraulic model for Subcatchment 5 and identify high level options for addressing existing flooding. The options identified by WSP/Opus included the following options[10,11]:

- Construction of 6 earth dams within the upper catchment to attenuate peak flows (refer Figure 4.9).
- Box culvert to increase conveyance capacity downstream of Larcy Road.
- Increase capacity of pipe down Iles Road and discharge into the drain on the eastern side of the Owhatiura South block (as opposed to the Selwyn/Larcy drain) (refer Figure 4.10).
- Combination of increasing capacity of pipe down Iles Road and storage using earth dams.

[10] Catchment 5 Stormwater Model Flood Alleviation Option Modelling. Prepared by WSP/Opus for Rotorua Lakes Council. Dated 2016. Report reference 002-3C1423-Chch-00.
[11] Memorandum titled "Catchment 5 Optioneering – Option 4 Amendment (Revision 1)". Prepared by WSP/Opus for Rotorua Lakes Council. Dated 29 November 2016.

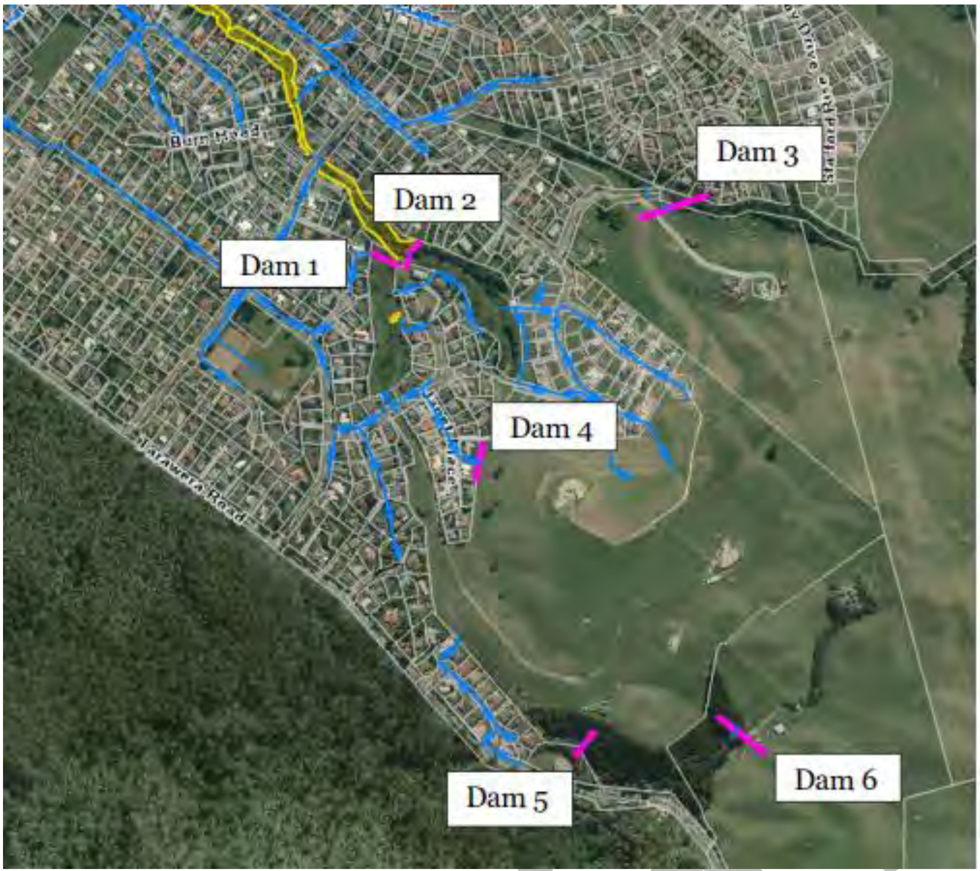


*Figure 4.9: Proposed dam locations (WSP/Opus, 2016)*


*Figure 4.10: Option to increase conveyance down Iles Road (WSP/Opus, 2016)*

In 2019 Pattle Delamore Partners (PDP) was engaged by RLC to investigate the number of residential dwellings that could be mitigated for by constructing flood detention dams in the gully near Walford Drive. Two dams were considered by PDP in the gully to the west of Walford Drive in addition to the Redwood Park detention pond which has already been constructed (refer Figure 4.11).

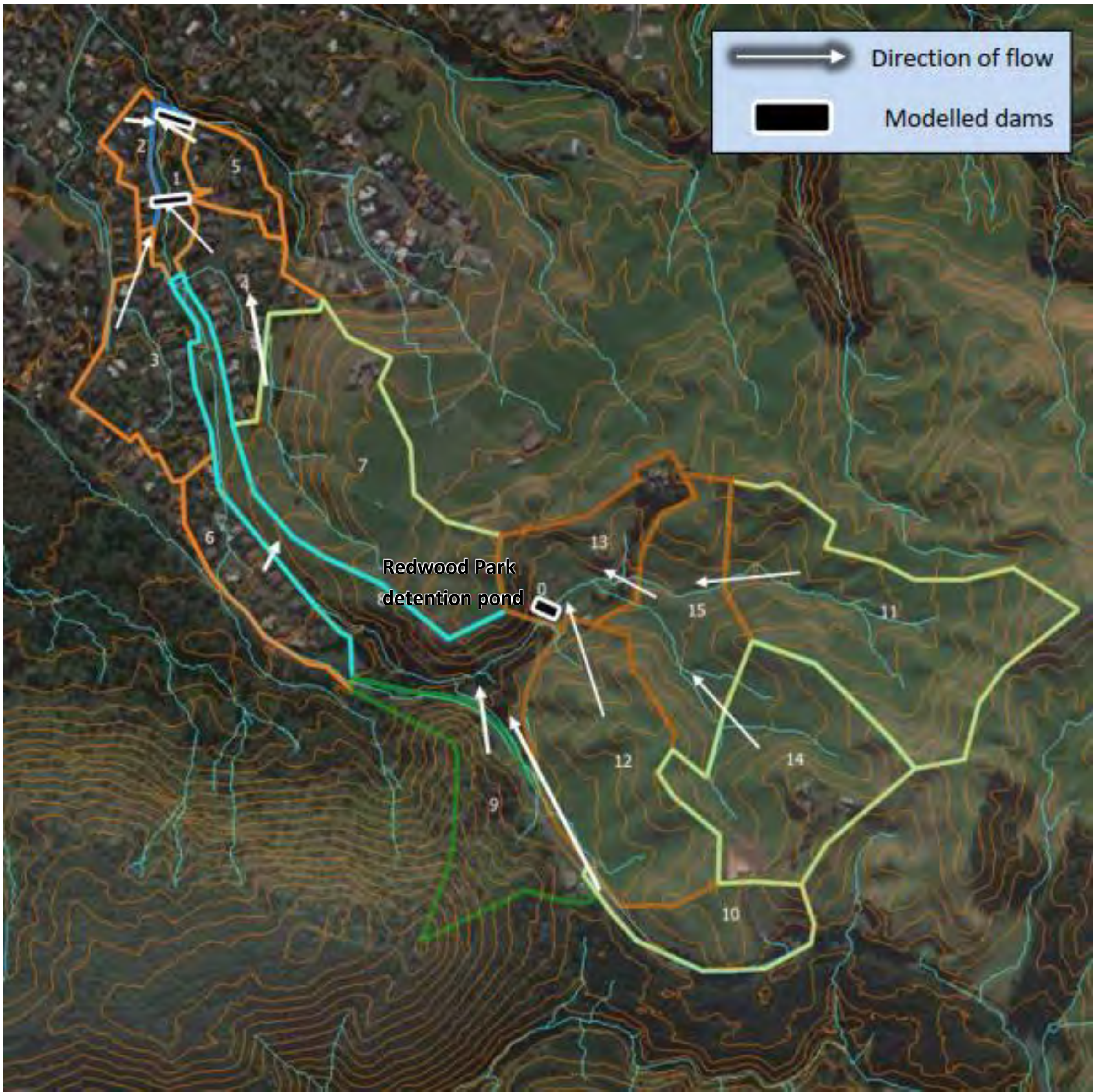


*Figure 4.11: Proposed dam locations (PDP, 2019)*

## 4.5 Utuhina Catchment (Subcatchments 13, 14 and 15)

Subcatchments 13, 14 and 15 are located in the catchment of the Utuhina Stream (refer Figure 4.12). The flood performance of the Utuhina stream in the lower part of the catchment is controlled by a flood scheme located in the downstream reach that is administered by BOPRC.

Subcatchment 13 contains the main branch of the Utuhina Stream. Subcatchment 13 is located south-west of Lake Rotorua and has its downstream location near to the confluence of the Utuhina and Mangakakahi Streams.

The major surface waterway draining Subcatchment 14 is the Otamatea Stream, a tributary of the Utuhina Stream. Subcatchment 14 is located south-west of Lake Rotorua and has its downstream location near to the confluence of the Utuhina and Otamatea Streams. The urban part of the subcatchment is located down-slope of Pukehangi Road, except for the Parklands development which is up-slope from Pukehangi Road. The rural part of the catchment is connected to the urban part via a small number of culverts under Pukehangi Road. These generally drain north-east toward the Otamatea Stream, which passes through the Wright Park area. Wright Park was assessed as a possible location for detention storage by PDP in a 2019 study[12].

Subcatchment 15 is located to the south-west of Lake Rotorua, and immediately north-west of Subcatchment 14. The Mangakakahi Stream is the major surface water feature that drains this catchment. Subcatchment 15 terminates at the confluence of this waterway and the Utuhina Stream. Pukehangi Road separates the current urban area from the rural parts of the subcatchment. There is an existing flood detention dam in Linton Park that provides some peak discharge control within the lower part of the Mangakakahi Stream. There has been some work previously undertaken by RLC to assess whether the capacity of this dam can be increased to address existing flood issues and the predicted effects of climate change[13].

Because Subcatchments 13, 14 and 15 are not directly connected to Lake Rotorua, any change in flow in these subcatchments can affect the downstream urban subcatchments known as Subcatchment 11, 12, 16 and 17 (not described in this report). BOPRC has indicated that the downstream stretch of the Utuhina Stream does not have any excess capacity and cannot receive any more flow than it currently does. There are also existing out of bank flood issues associated with the Mangakakahi and Utuhina Stream in the Old Taupo Road area.

The key existing flood issues for Subcatchments 13, 14 and 15 include:

**Catchment 13**

1. Significant flooding associated with the Kahu St, Whitworth Rd and Konene St drains towards the front of John Paul's College occurs during storm events about every 2 years.
2. The roundabout at Old Taupo/Devon/Otonga roads is a low point and is subjected to surface flooding about once every five years. The flooding is mainly from secondary flows and blocked cesspits.

**Catchment 14:**

3. The culvert under Sunset Rd has insufficient capacity for large rainfall events.
4. The culvert under Ford Rd was upgraded around 2008/9 to eliminate surface flooding that made the road impassable. However, the downstream capacity issues continue to cause flooding at this location about every 5 years.

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[12] Memo titled "Wright Park Stormwater Detention basin – Feasibility Assessment". Prepared by PDP for Rotorua Lakes Council. Dated 7 August 2019.

[13] Stormwater Urban Catchment 15 Linton Park Detention Basin Upgrading Report. Prepared by Rotorua District Council Works Division. Dated June 2010. Report reference RDC-122650.

**Catchment 15:**

5 Overland flow from the upper catchment off Mount Ngongotaha flows along Clayton Rd and down Mountain Rd into the piped network at the intersection. This piped system is under capacity and excess flows cross over Clayton Rd and then run through private property into Garrick Place.

6 Flooding at the Industrial area near Riri St occurs during large rainfall events.

7 Flooding of Edmund Rd.

The locations of these flooding hotspots are shown in Figure 4.12 below.

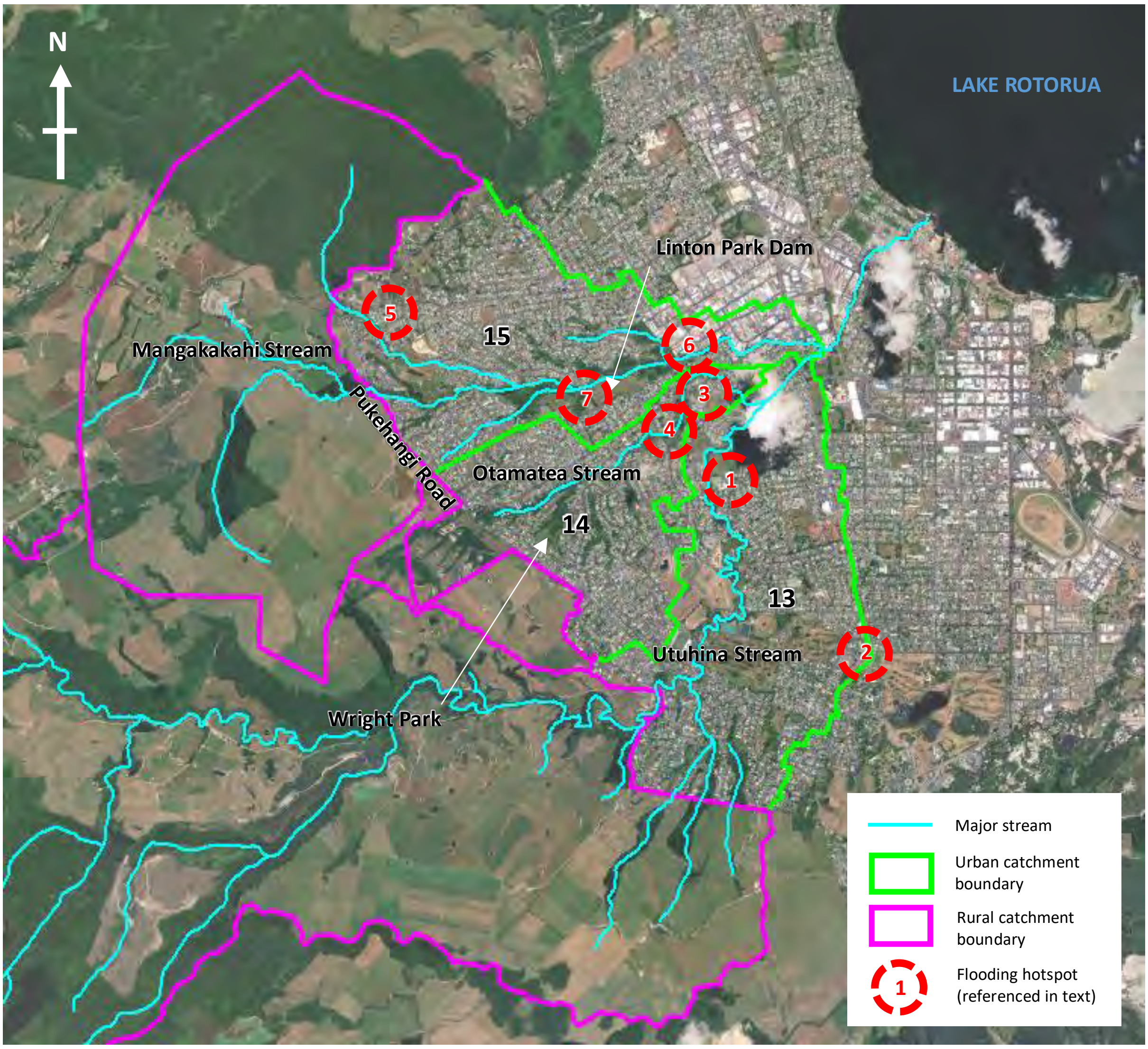


*Figure 4.12: Extents of Subcatchments 13, 14 and 15 and known flooding hotspots*

RLC has previously developed various flood models for these subcatchments (using ICM Innovyze and AULOS software), but do not currently have a model for the entire Utuhina Catchment. BOPRC has recently completed a new hydrological and hydraulic model for the Utuhina Stream Catchment for the purposes of assessing the capacity of the BOPRC rated flood scheme. The hydrological modelling was completed by Blue Duck Design and uses a non-linear reservoir hydrological method[14]. The hydraulic model that has been built by DHI is a 1D hydraulic model. This model was considered for use as part of this study but was not available within the required timeframes. However, it may be a requirement of BOPRC to test any flood management solutions using this model as part of subsequent design stages.

WSP/Opus was engaged by RLC in 2020 to investigate onsite flood mitigation solutions for the Pukehangi Heights development (the main area of development in the Utuhina Catchment) to support the plan change application. The proposed solution to mitigate the flood effects of the Pukehangi Heights development is to construct 12 dry detention basins located throughout the development (refer Figure 4.13). These basins would have a total footprint of approximately 14 ha.

The Freedom Village, Titoki Place and Diamond Street Developments are all proposed to have stormwater attenuation devices to manage flood flows as discussed in Section 3. The designs of these are being completed by the developers separately to this study.

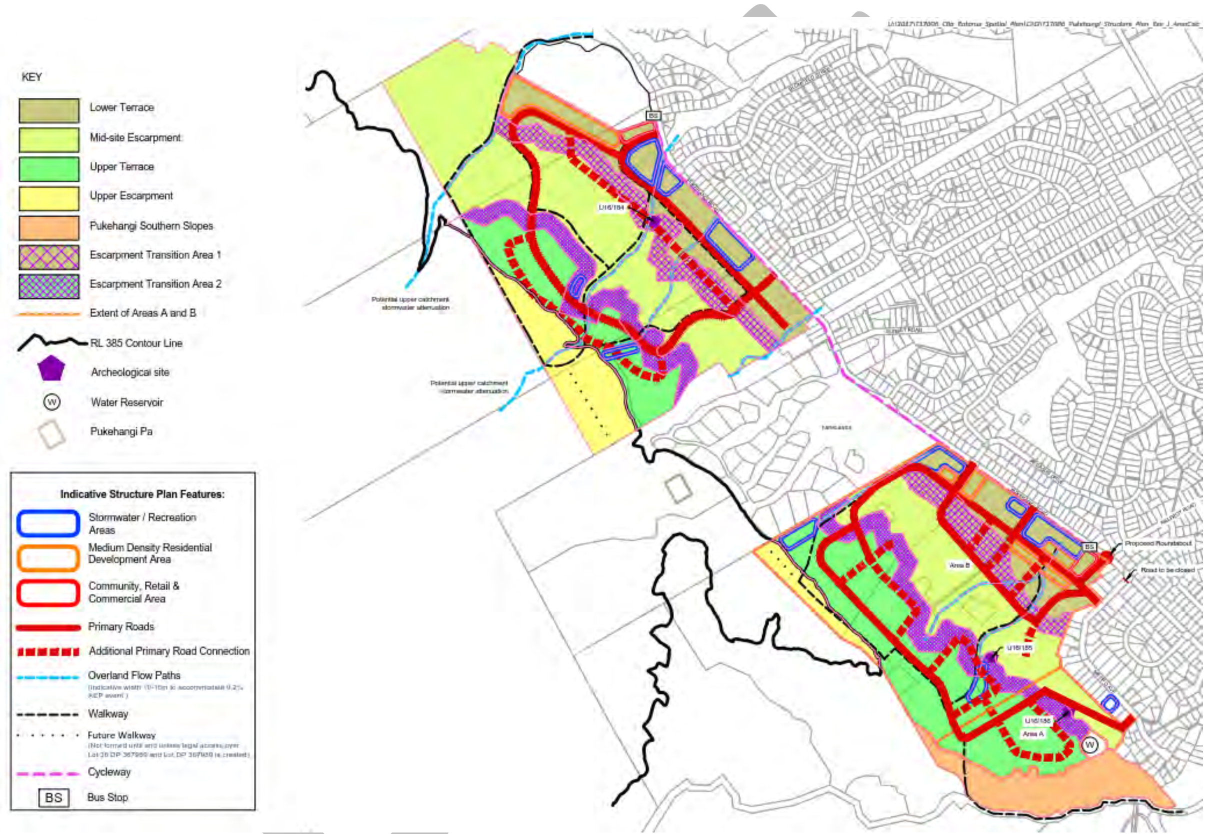


*Figure 4.13: Proposed structure plan for Pukehangi Heights (Source: Boffa Miskell figure Revision K dated 21 April 2020)*

[14] Utuhina Hydrological Model Establishment. Prepared by Blue Duck Design Ltd. for Bay of Plenty Regional Council. Dated 26 September 2019.

# 5 Conceptual mitigation options

This section describes the conceptual mitigation options, presents indicative option sizing and presents a recommendation on the preferred option for each subcatchment. It should be noted that for the purposes of this study (i.e. identification of the preferred flood management approach), these options have been developed only to a concept level. This means that more detailed analysis is required as part of subsequent design stages to assess the feasibility of these concepts and further develop the design. This may mean that the layout or sizing of these solutions will change in subsequent design stages.

While consideration has been given to the practicality of directing catchment runoff mitigation options such as flood detention dams, no concept for the conveyance elements required to achieve this has been developed. This is because it will require a combination of road layout, subdivision layout and urban design concepts to determine the appropriate alignment and sizes. The road layout in particular provides a good opportunity to convey overland flowpaths to the detention areas and this needs to be considered by developers as development plans are progressed. Geological, land ownership or other constraints will need further consideration in future stages.

## 5.1 Subcatchments 1 and 2

The growth area within Subcatchments 1 and 2 for which flood mitigation options have been assessed is the portion of the Ngati Whakaue Lands development which is currently unzoned and requires a plan change (refer Figure 5.1). These two subcatchments have been assessed together because they encompass one development area and there are opportunities for an integrated approach across both subcatchments. Three options have been assessed to mitigate the effects of this development area. These include:

- Centralised detention upstream of Te Ngae Road.
- Improved conveyance across Te Ngae Road to the lake (with no detention).
- Diversion of Subcatchment 1 flows to the Waingaehe Stream (Subcatchment 2).

These options are discussed further in the following subsections.

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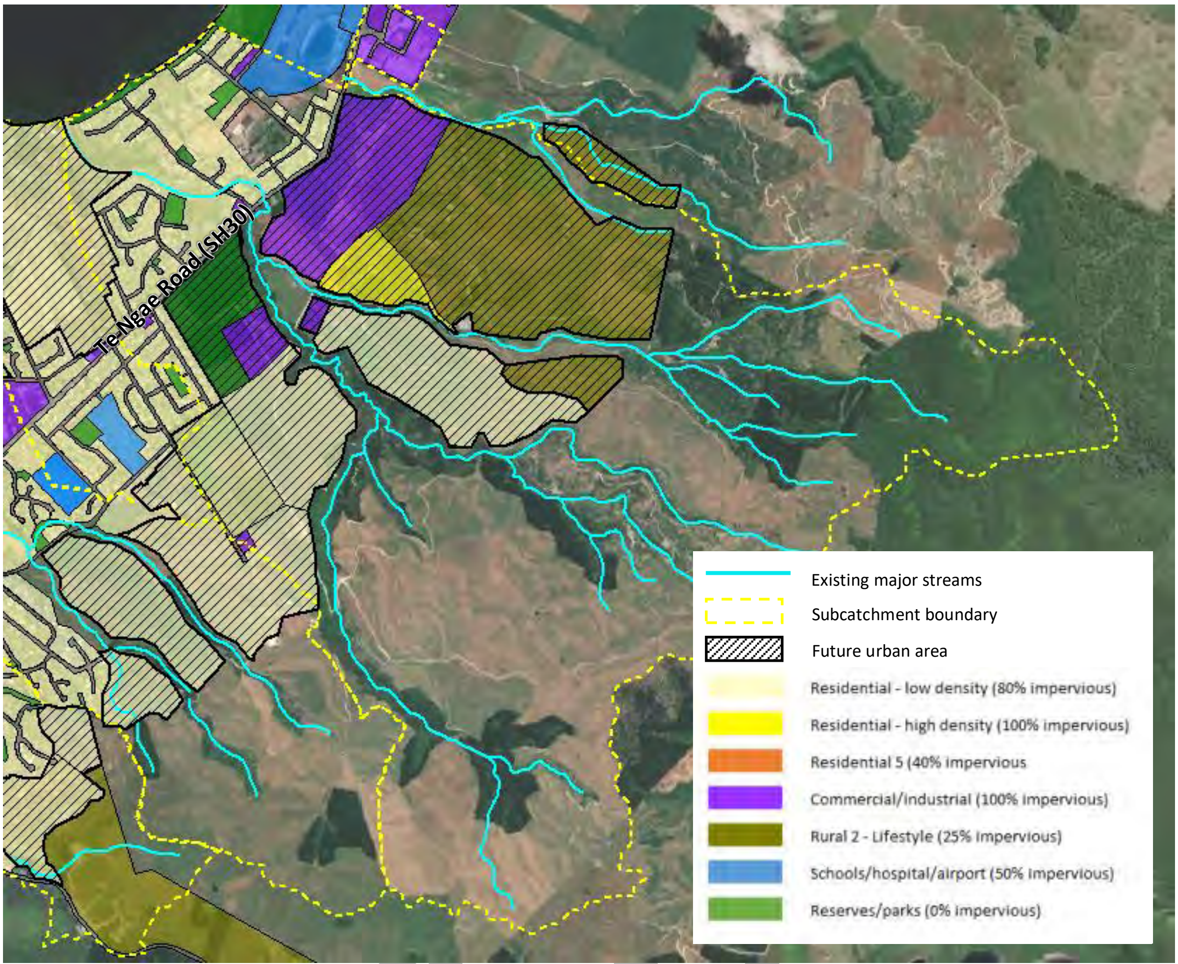


*Figure 5.1: Development areas to be mitigated within Subcatchments 1 and 2*

### 5.1.1 Option 1/2A - decentralised detention

Option 1/2A involves construction of a number of flood detention ponds and dams throughout the development area. Dams have been proposed where there are existing natural topographical features that can be utilised to detain runoff. Where dams are not possible because of topography, flood detention ponds have been proposed assuming that storage volume will have to be created by excavation of a basin. Subcatchments 1 and 2 are further split into a number of different catchment areas for the different detention structures and these are shown in Figure 5.2 along with the potential device locations.

Table 5.1 presents a summary of the indicative pond/dam sizing. The dam crest elevations are based on providing some nominal freeboard above the 100 year water level, however conveyance of less frequent events to meet dam safety requirements has not been assessed and may determine the required crest level. The pond footprint areas are based on 3 m deep (including nominal freeboard) square ponds with sloped sides.

We have assessed the detention designs for multiple durations of design rainfall events to ensure attenuation objectives are met for a range of storm profiles. The ponds and dams are sized to provide attenuation of post-development peak flows to pre-development levels for the 1 hr and 6 hr events. Longer duration/lower intensity storms (e.g. the 12 hr and 24 hr events) are not predicted to result in downstream flooding so attenuation for these events has not been targeted. Matching of peak flows has only been done for the respective development area served by each detention device. In subsequent design stages coincidence of peaks flows from these devices will need to be assessed to ensure post-development peak flows at Te Ngae Road do not exceed pre-development peak flows (unless there is sufficient capacity in the Waingaehe to convey these flows).

A centralised "online" detention structure has been considered for this option as opposed to a decentralised approach. However, this is not preferred due to the uncertainty in hydrology and hydrogeology for this catchment. Online devices carry more risk of being significantly over- or under-engineered where significant uncertainty exists relative to offline devices and may also have a larger ecological impact. Offline structures also allow development to be staged with devices sequentially constructed as new stages are developed and typically do not have the same ecological impact. Offline devices also have the ability to provide stormwater management function such as water quality treatment and extended detention (for mitigation of stream erosion risk) if a permanent pond is able to be maintained.

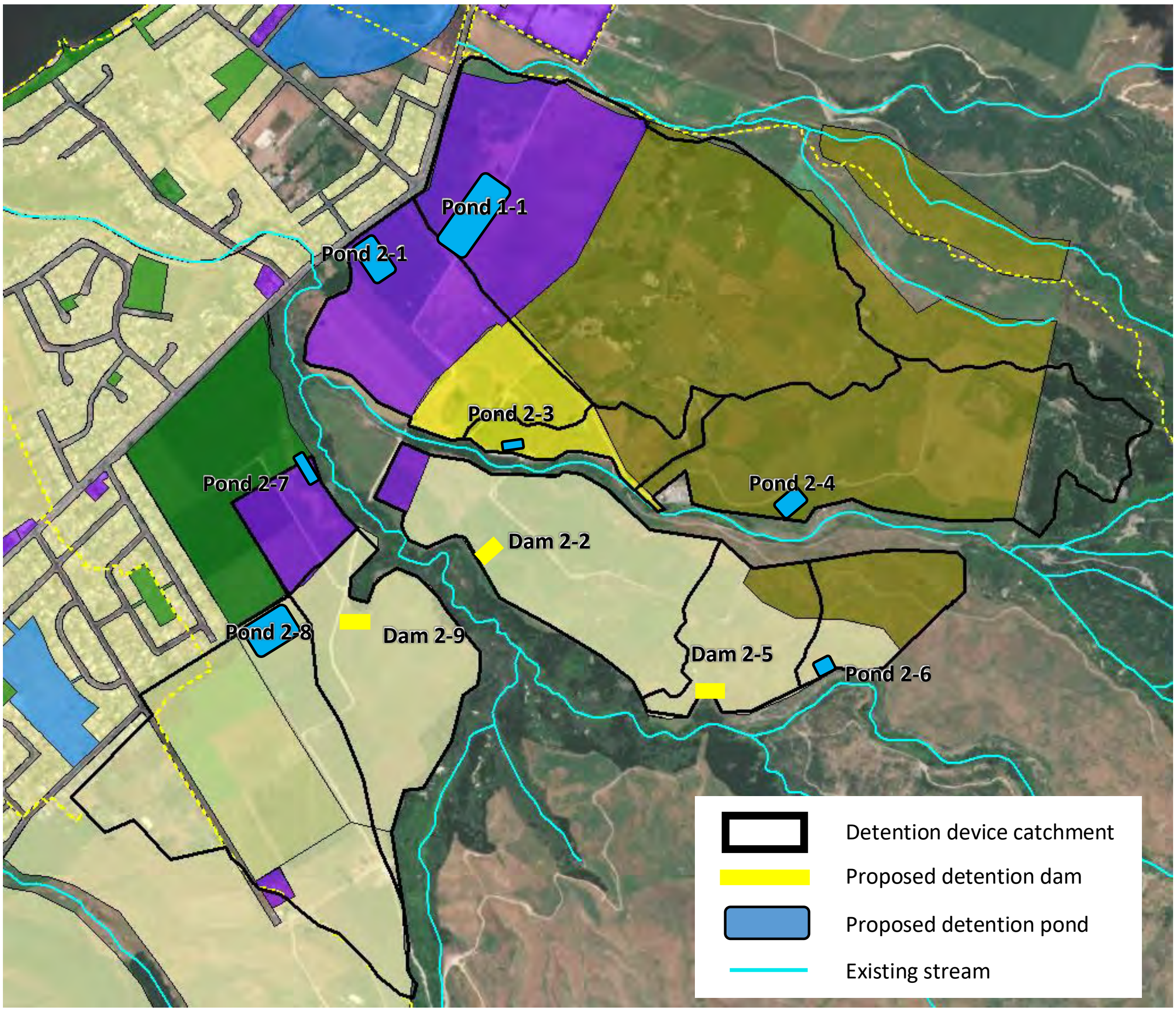


*Figure 5.2: Location of all proposed ponds and dams (Option 1/2A) (ponds shown in blue, dams in yellow)*

**Table 5.1: Summary of detention sizing for Option 1/2A**

| Structure reference | Structure type | Dam crest elevation /height | Pond footprint | 100 yr live storage volume |
|---|---|---|---|---|
| 1-1 | Pond | | 2.8 ha | 63,000 $m^3$ |
| 2-1 | Pond | | 1.6 ha | 35,000 $m^3$ |
| 2-2 | Dam | 319 m RL / 8.5 m | | 40,000 $m^3$ |
| 2-3 | Pond | | 0.3 ha | 6,000 $m^3$ |
| 2-4 | Pond | | 1.0 ha | 22,000 $m^3$ |
| 2-5 | Dam | 327.5 m RL / 5.5 m | | 11,000 $m^3$ |
| 2-6 | Pond | | 0.5 ha | 9,000 $m^3$ |
| 2-7 | Pond | | 0.5 ha | 9,000 $m^3$ |
| 2-8 | Pond | | 2.2 ha | 54,000 $m^3$ |
| 2-9 | Dam | 325 m RL / 15 m | | 36,000 $m^3$ |

**Northern ponds**

The northern part of Catchment 2 generally grades northward towards Te Ngae Road and is generally flatter than the upper parts of Catchment 2. This part of the development consists of the main commercial/industrial precinct for the development as well as the bulk of the proposed rural lifestyle zoning and some residential area (refer Figure 5.3).

Pond 1-1 will require a storage volume of approximately 63,000 $m^3$ and approximately 2.8 ha of land. The currently proposed location of the detention pond is alongside Gee Road because this is an area of flat, low lying land where stormwater has been informally detained by the embankment. This location provides some flexibility in whether the pond discharges to Bridal Stream or is diverted into the Waingaehe (refer to Option 1C). There is significant opportunity to optimise the design because the location and shape of the pond is adaptable and can be relatively easily modified as the layout of the commercial area and upstream rural lifestyle area are developed. In particular, based on the density of current development in the area, we consider that the 25% impervious assumption for the rural lifestyle land use upstream may over-estimate the likely density of development.

Utilising the existing Gee Block dam has been considered for Pond 1-1, however, it is our understanding that it was not designed, constructed or consented as a formal detention structure and therefore it is highly unlikely to be suitable to be relied upon to further mitigate the effects of development. Also given the low height of the embankment this existing storage is also very inefficient and its replacement with an engineered pond will optimise developable area.

Pond 2-1 will require a storage volume of approximately 35,000 $m^3$ and approximately 1.6 ha of land. The currently proposed location of the detention pond is alongside Te Ngae Road. However, given the flat topography, the location and shape of the pond could easily be adjusted to suit the proposed development layout if required in subsequent design stages.

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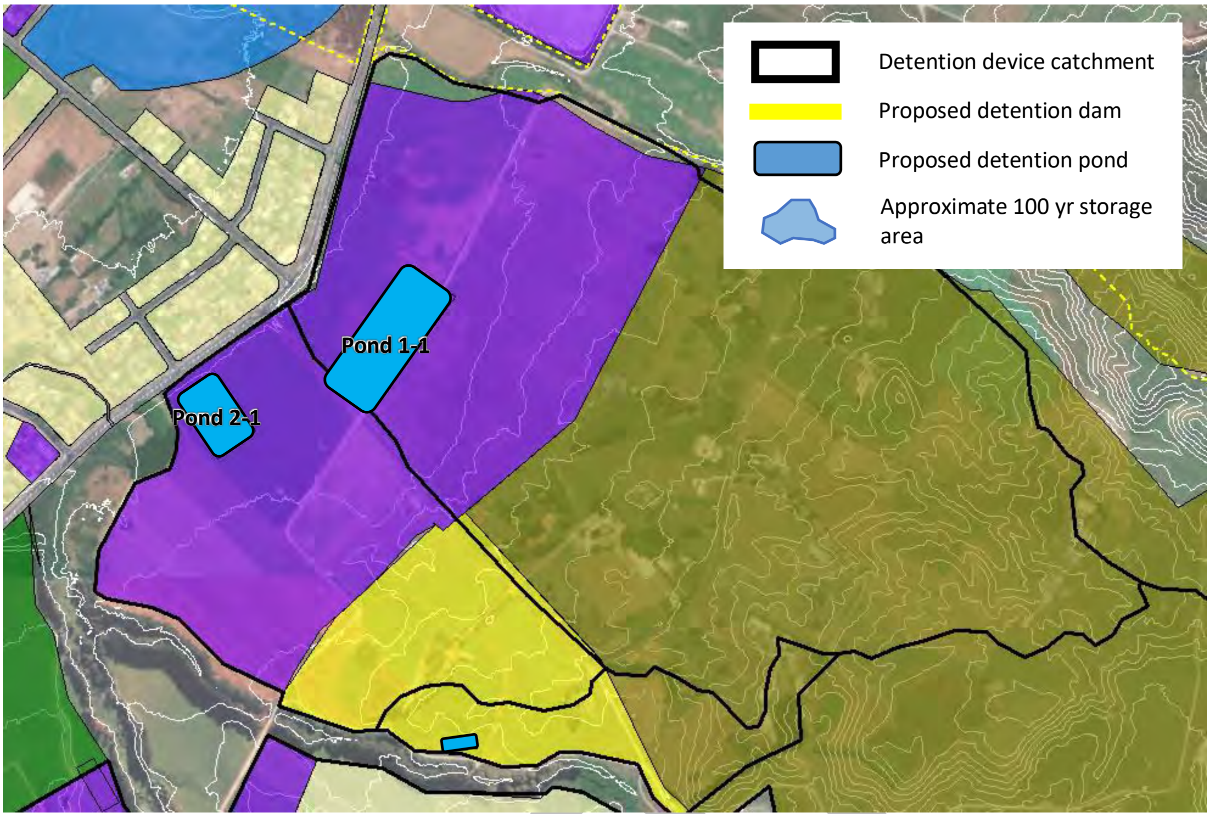


*Figure 5.3: Proposed location of northern ponds (2 m contours sourced from RLC's GeyserView)*

**Central ponds/dams**

This central area of Catchment 2 comprises complex topography with a mixture of residential, rural lifestyle and some commercial/industrial land (refer Figure 5.4).

Dam 2-2 is proposed to be located in the existing gully that runs through the middle of the proposed residential area. Dam 2-2 will require approximately 40,000 m$^3$ of storage and have a height of approximately 8.5 m. However, there may be some scope to increase the size of this dam to provide offset mitigation for other parts of Subcatchment 2 where detention is difficult or uneconomic.

The proposed size of Dam 2-2 includes for mitigation of areas which do not naturally drain to the dam such as the commercial/industrial land to the west and the portions of the residential area to the south-east or north-west. The approach to mitigating these areas will need further investigation in subsequent design stages to see whether they are able to be drained to the dam or if an offset mitigation approach for these areas is feasible.

Pond 2-3 will require a storage volume of approximately 6,000 m$^3$ and approximately 0.3 ha of land. It is currently located on relatively flat land adjacent to the Waingaehe Stream. This is the smallest pond catchment considered. Optimisation of the design in subsequent stages should investigate whether offset mitigation could be applied to this area to remove the need for a pond here. However, we also note that this is an area with a high development density proposed and other stormwater management objectives such as water quality treatment may require a stormwater pond in this location.

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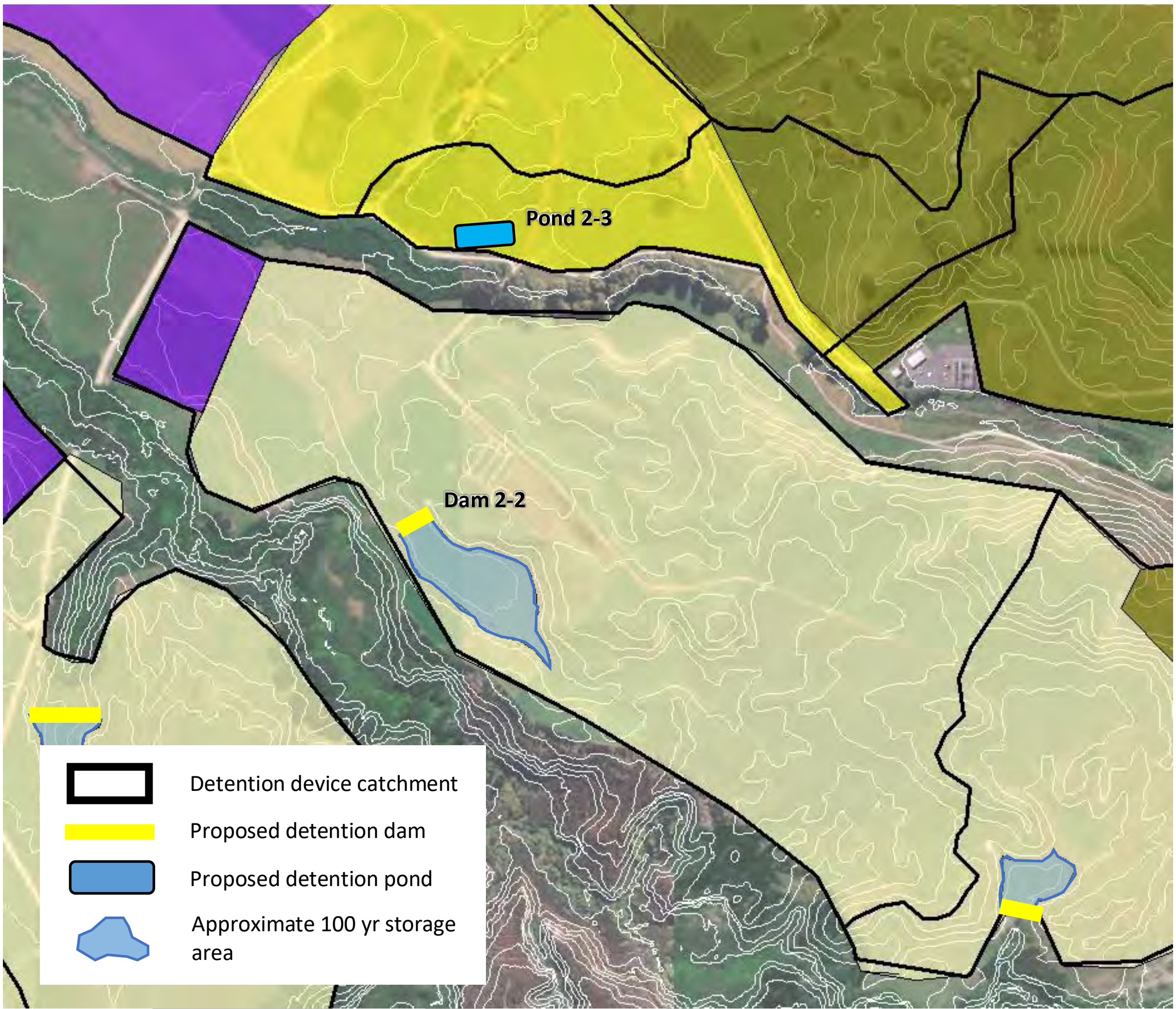


*Figure 5.4: Proposed location of central ponds/dams (2 m contours sourced from RLC's GeyserView)*

**Eastern ponds/dams**

This area of catchment 2 is likely to be a mixture of low density residential and rural lifestyle (refer Figure 5.5). There is a high degree of uncertainty regarding the likely road layouts, access, earthworks and impervious surfaces that are likely in this catchment. The topography is also highly variable and therefore draining impervious areas to communal detention will likely prove challenging but appears possible. Feasibility will have to be reviewed in subsequent stages when further information about the proposed subdivision design is available.

Pond 2-4 will require a storage volume of approximately 22,000 $m^3$ and approximately 1 ha of land. Pond 2-6 will require a storage volume of approximately 9,000 $m^3$ and approximately 0.5 ha of land. Both these ponds are located in relatively flat areas adjacent to the tributaries of the Waingaehe Stream. The catchment for Pond 2-4 is particularly long and has two distinct subcatchments. It may not be possible to drain all flow in a 100 year event to the proposed pond location, in which case this pond may need to be split into two separate detention devices. This should be investigated further in subsequent stages when further information about the proposed subdivision design is available.

Dam 2-5 is located in the confined gully to the south of the mostly residential development area. Dam 2-5 will require approximately 11,000 $m^3$ of storage and have a height of approximately 5.5 m.

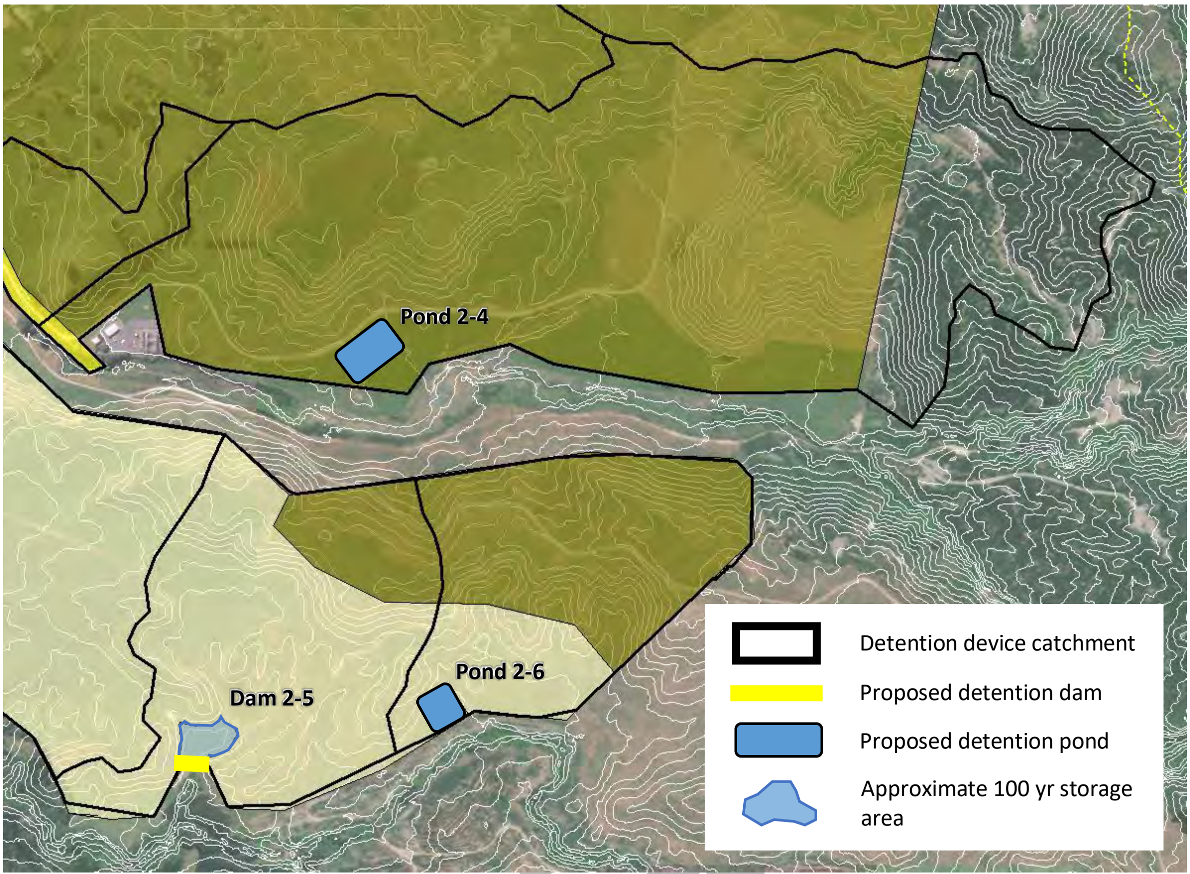


*Figure 5.5: Proposed location of eastern ponds/dams (2 m contours sourced from RLC's GeyserView)*

**Western ponds/dams**

This area of Catchment 2 is proposed to be predominantly residential with one commercial/industrial precinct and a proposed park/reserve area.

Pond 2-7 will require a storage volume of approximately 9,000 $m^3$ and approximately 0.5 ha of land. Pond 2-7 is currently shown on the lower terrace adjacent to the Waingaehe (but above the floodplain) where development potential is lower but may also be located on the upper terrace. Feasibility on situating the pond on the lower terrace will depend on road layout, earthworks and subdivision design.

Pond 2-8 will require a storage volume of approximately 54,000 $m^3$ and approximately 2.2 ha of land. Currently the design assumes that the part of Subcatchment 3 west of Wharenui Road is able to be diverted to Pond 2-8. Given the topography, the location of the pond can be adjusted along the northern boundary to adapt to the road layout, earthworks and subdivision design. Subject to Council approval, it may be possible to move the pond onto the parks/reserve area thereby allowing additional development to take place in the proposed residentially zoned land. Unlike other detention areas in the development, Pond 2-8 is located some distance away from the nearest stream branch. Consideration will have to be given of how to convey discharge from the pond to the stream.

Dam 2-9 is currently proposed to be located in the ephemeral stream gully to the east of this area. Dam 2-9 will require approximately 36,000 $m^3$ of storage and have a height of approximately 15 m. This is a relatively high dam for the volume of storage achieved. We recommend investigating whether Pond 2-8 and Dam 2-9 could be combined as one pond in future design stages.

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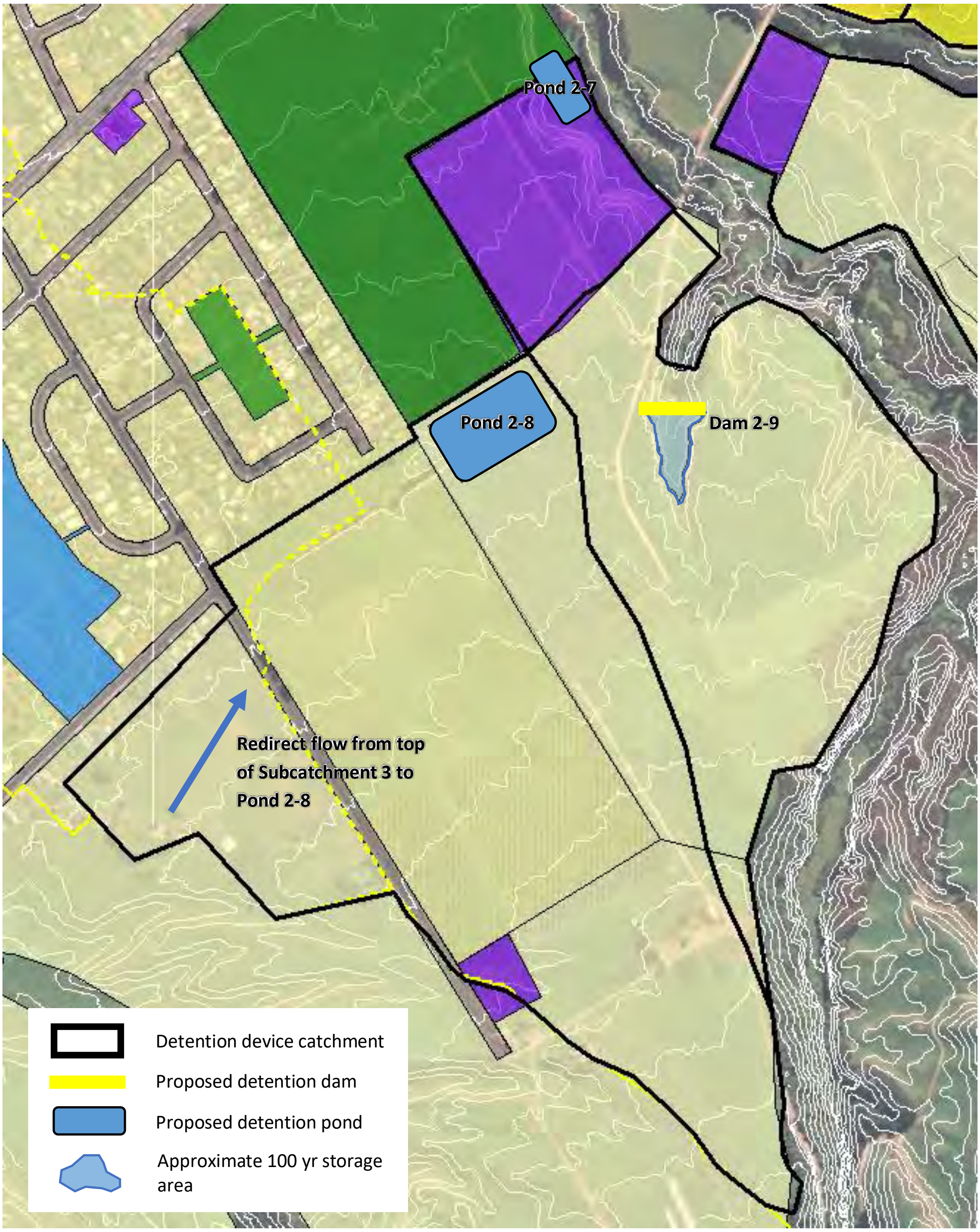


*Figure 5.6: Proposed locations of western pond/dams (2 m contours sourced from RLC's GeyserView)*

### 5.1.2 Option 1/2B – conveyance without detention

Option 1/2B involves conveyance upgrades from Te Ngae Road to the lake, with no attenuation of post-development flows from the upper catchment.

**Waingaehe Stream (Subcatchment 2)**

For subcatchment 2 (the Waingaehe Stream) the estimated post-development unattenuated peak flow is approximately 60 $m^3/s$ (including climate change to 2090). This would be the required conveyance capacity of the Waingaehe to allow upstream development to occur without any upstream detention devices. While it has been noted by RLC that there appears to become some spare capacity in the Waingaehe Stream, we understand no assessment of its current conveyance capacity has been undertaken to date. BOPRC did conclude that the present day 100 year flow was 17 $m^3/s$ (pre-development, no climate change) in their 2008 study for the Waingaehe Stream and found that freeboard requirements for the stream were met for this flow.

The Waingaehe Stream appears to have longitudinal grade of approximately 1% immediately downstream of Te Ngae Road based on LiDAR information, flattening to approximately 0.2-0.3% closer to the lake. The depth of the channel also appears to decrease from 2-2.5 m deep near Te Ngae Road to approximately 1.5 m deep nearer to the lake. The corridor that is currently owned by RLC for the stream is generally 40-50 m wide, with the lowest width being approximately 18 m wide downstream of Te Ngae Road (refer Figure 5.7).

Assuming uniform flow and a grade of 0.3%, a channel with a top width of 27 m would be required to convey the estimated 100 year unattenuated peak flow (assuming a trapezoidal profile with 6 m base width, 2 m depth and side slopes of 5:1 (H:V)). While currently available information indicates it may be possible to achieve using the existing stream corridor, more detailed analysis based on additional stream survey data is required to determine the existing capacity of the Waingaehe Stream and to assess the hydraulics of any proposed upgrade. The hydraulics of the Te Ngae Road and Robinson Ave bridges also needs to be assessed as these may act as hydraulic controls.


*Figure 5.7: Waingaehe Stream corridor (downstream of Te Ngae Road)*

**Bridal Stream (Subcatchment 1)**

The estimated 100 year post-development peak flow for Subcatchment 1 is approximately 20 $m^3/s$. There is very little grade available downstream of Te Ngae Road and Bridal Stream would need to be approximately 1.5 m deep with a top width of 15 m at 0.25% grade to convey these flows. However, this assumes uniform flow, but we note that high tailwater levels from the lake may impact on capacity. This is significantly larger than the existing channel. It is noted that Hannah's Reserve provides a potential alternative discharge location to the lake.

Currently flow from Subcatchment 1 is piped from Te Ngae Road and along Alfred Road in a pipe network ranging from 900 mm to 1050 mm in diameter. For this pipe network to convey primary flows (i.e. flow from a 10 year ARI event – approximately 10 $m^3/s$) the pipe would need to be upgraded to approximately 1800 mm dia. There would also need to be sufficient capacity to convey 10 $m^3/s$ of secondary flow (i.e. flow from a 100 year ARI event minus primary flow) along Alfred Road. The feasibility of this will depend on road cross section geometry and property levels and should be investigated further in subsequent design stages if this is identified as the preferred option.

Overall, the required conveyance capacity along Alfred Road and the lack of grade in the lower part of the catchment, present potential fatal flaws for this option for Bridal Stream.

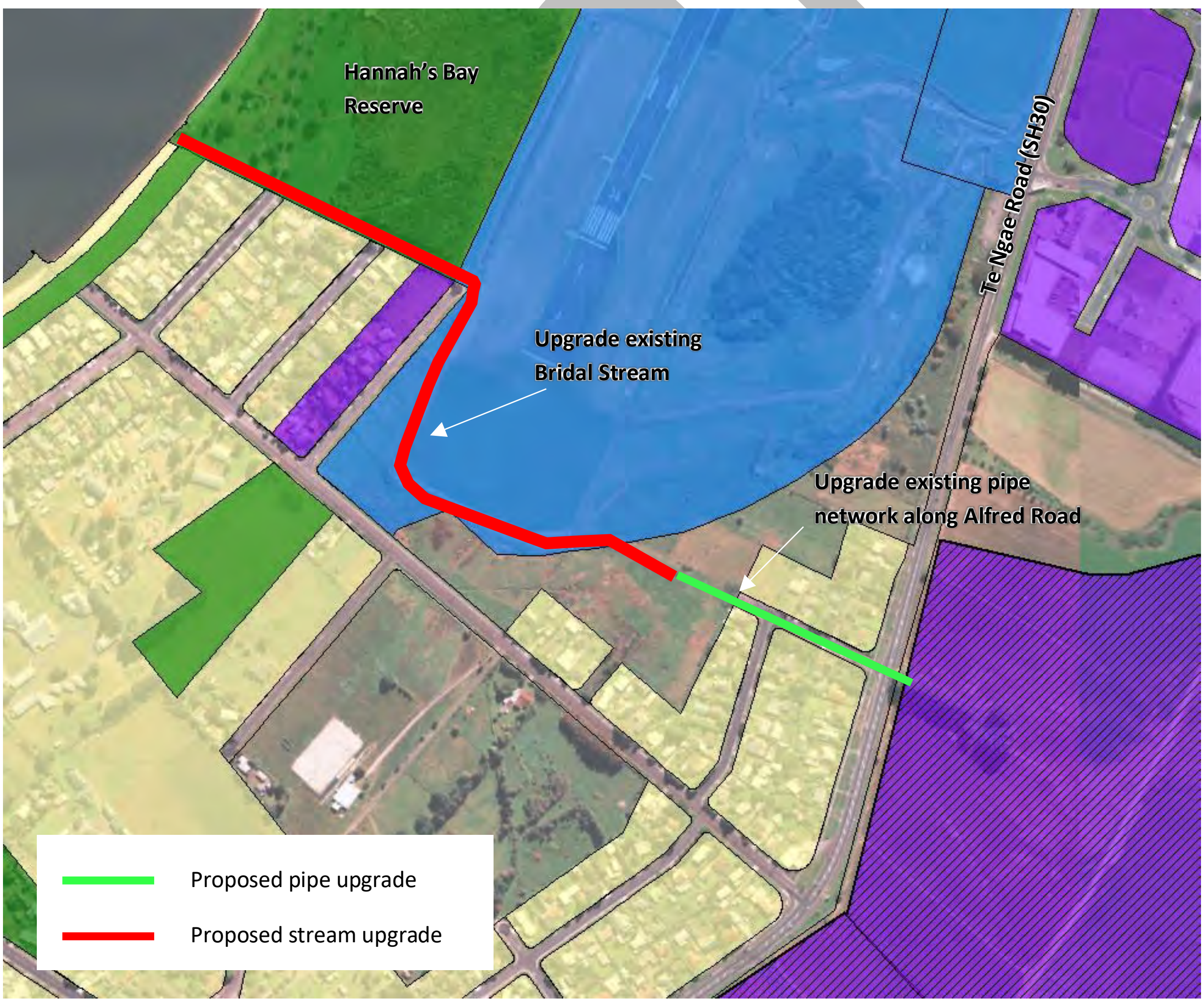


*Figure 5.8: Proposed location of conveyance upgrades for Option 1B*

### 5.1.3 Option 1/2C – catchment diversion

Option 1C involves diversion of the flow from Subcatchment 1 into the Waingaehe Stream (Subcatchment 2). Assuming this option involves detention of Subcatchment 1 as per Option 1A, this will add approximately 6 $m^3/s$ of flow to the Waingaehe Stream. Assuming detention is provided in Subcatchment 2, this will increase 100 year ARI peak flows in the Waingaehe Stream by approximately 10-15% in 2090.

We note that significant uncertainty still exists around the hydrology of the Waingaehe Catchment and the hydraulic capacity of the Waingaehe Stream. Therefore, further work would have to be undertaken to confirm the capacity of the Waingaehe Stream before the feasibility of this option could be confirmed. There may also be potentially cultural issues if this is viewed as the diversion of water from one catchment into another catchment by Mana Whenua.

### 5.1.4 Recommendation – Subcatchments 1 and 2

Table 5.2 below summarises the qualitative options assessment that has been undertaken for Subcatchments 1 and 2. Option 1/2C can be applied to either Option 1/2A or 1/2B (i.e. it is not mutually exclusive to these options). For the purposes of the options assessment, it is assumed that Option 1/2B is feasible (yet to be confirmed) and that it has been adopted in conjunction with Option 1/2C (i.e. no detention for either Subcatchment with diversion of Subcatchment 1 into the Waingaehe Stream).

We recommend that further work be undertaken to determine the conveyance capacity of the Waingaehe Stream and bridges from Te Ngae Road to the lake prior to making a decision on the preferred option for Subcatchment 1/2. This work will need to include a survey of the Waingaehe Stream downstream of Te Ngae Road and detailed hydraulic modelling.

We also recommend that further work be undertaken to confirm the hydrological parameters that should be used for the eastern catchments to improve the confidence in peak flow estimates. We note that these catchments have been assessed by multiple consultants using a variety of methods and there is a wide range in peak flow estimates. This is due to the complexity of infiltration/interflow dynamics in pumiceous soils which are not well captured in traditional rainfall/runoff methods as well as potentially erroneous data capture from the Waingaehe Stream gauge and lack of a rain gauge in this catchment.

Once the existing capacity of the Waingaehe Stream is known and there is more certainty around catchment hydrology, a decision can be made on whether the preferred approach for flood management includes provision of detention or conveying unattenuated flows to the lake, and whether it is possible to divert flow from Subcatchment 1. The optimal result may well be a combination of all three options.

If the decentralised detention approach is the preferred approach, an estimate of the capacity of the Waingaehe Stream will determine what level of attenuation is required (e.g. attenuation to more or less than 100% of pre-development flows) and whether detention devices may also need to provide mitigation for climate change effects. A decentralised detention approach also allows for reserving any excess capacity in the Waingaehe Stream for potential landuse changes in this catchment beyond current planning horizons. We note that there is a high level of uncertainty around the proposed detention concept because the size of the development area and the number of subcatchments makes the concept highly dependent on subdivision layout and staging.

If the conveyance/no detention approach is the preferred approach we recommend that collaboration with BOPRC occurs at an early stage to get acceptance in principle for the proposed approach due to this stream being part of a rated flood scheme. However, we do not anticipate that

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this approach will be viable for Bridal Stream at this stage and so attenuation or catchment diversion will still likely be required for Subcatchment 1.

We note that the proposed flood detention dams and conveyance upgrades only provide a flood mitigation function. Onsite stormwater management devices would likely still be required to provide functions such as extended detention (for mitigation of stream stability risk, if required) and water quality treatment.

**Table 5.2: Summary of qualitative options assessment for Subcatchment 1/2**

| | **Decentralised detention (Option 1/2A) for both subcatchments** | **Conveyance without detention (Option 1/2B) for Subcatchment 2 and catchment diversion for Subcatchment 1 (Option 1/2C)** |
|---|---|---|
| Rough order cost estimate (including professional fees) | $28.6 to $41.3 million | Not assessed but likely considerably lower than Option 1/2A |
| Ability to mitigate flooding | • Provides mitigation for growth<br>• Climate change effects potentially not an issue if sufficient capacity exists in Waingaehe | • Provides mitigation for growth<br>• Addresses climate change effects |
| Ability to address other stormwater objectives (e.g. water quality, erosion etc.) | • Extended detention and water quality treatment could be achieved in the proposed ponds and dams (if a permanent pool of water is maintained) | • Extended detention and water quality treatment will likely have to be achieved in separate stormwater management devices |
| Speed of implementation | • Design and consenting process required for multiple dams/ponds | • Relatively quick implementation. No constraint on development staging |
| Constructability and maintenance | • Constructability of the multiple ponds/dams differ dramatically and need to be assessed on a case by case basis<br>• Dams will require ongoing dam safety management | • Potentially no additional maintenance requirements |
| Potential risks (engineering, consenting etc.) | • Suitability of the underlying soils for construction of an earth embankment dam<br>• Dam break risk to downstream properties | • Potentially higher consequence if hydrological estimates are incorrect<br>• BOPRC may not agree with a conveyance based/no attenuation approach out of principle |
| Social/cultural/environmental impacts | • Function of flood detention dam near residential dwellings would have to be carefully communicated to community | • Consultation with iwi may need to be undertaken to get agreement on catchment diversion due to cultural values |
| Land purchase required? | • Potentially requires designations to be established for each device, and land to be vested in RLC | • Not likely |

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| | Decentralised detention (Option 1/2A) for both subcatchments | Conveyance without detention (Option 1/2B) for Subcatchment 2 and catchment diversion for Subcatchment 1 (Option 1/2C) |
|---|---|---|
| Other considerations | • The subdivision, pipe network and road network design for the developments need to consider how 100 year flows will be conveyed to detention devices/open channels | • The feasibility of diverting Subcatchment 1 into the Waingaehe requires further investigation |

## 5.2 Subcatchment 3

The growth areas within Subcatchment 3 for which mitigation has been assessed are shown in Figure 5.9. Because the development areas near Galvin Road are generally located in the bottom half of the subcatchment the recommended mitigation approach for these areas is to not attenuate flows but rather “pass flows forward” so peak flows from these areas do not coincide with peak flows from the upper catchment. This approach requires sufficient conveyance capacity for unattenuated peak flows.

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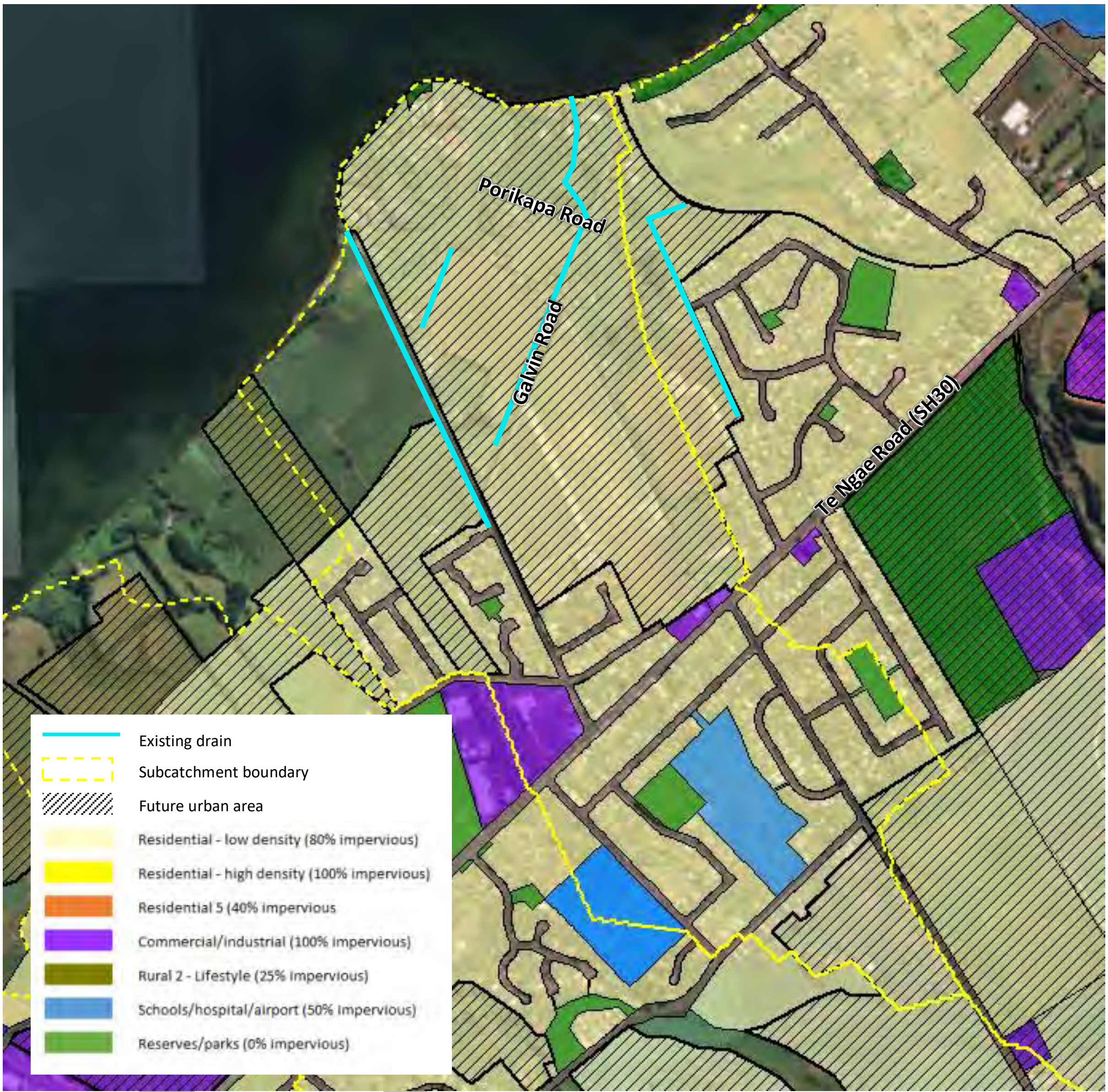


*Figure 5.9: Development areas to be mitigated in Subcatchment 3*

### 5.2.1 Option 3

It is proposed that flood management in this area is done by ensuring that adequate capacity is provided in the road corridor to convey post-developed 100 year flows. This will likely require installation of trunk main stormwater pipes for primary flows with secondary flows being conveyed on the road surface and existing roadside open channel (if retained). Utilising the existing roadside open channel for primary flows has been considered but it is a relatively shallow drain so a reticulated pipe network south of Galvin Road would likely not be able to connect to it. Some indicative locations of where trunk mains may be located are shown in Figure 5.10.

It is proposed that all development south of Galvin Road drain northward to a trunk main located in Galvin Road. It is estimated that primary flows could be up to 10 $m^3$/s and secondary flows could be up to 7 $m^3$/s. Some preliminary calculations show that the trunk main west of Te Roro o Te Rangi Roadway would likely need to be in the order of 1200-1500 mm dia. while the trunk main east of Te Roro o Te Rangi Roadway would likely need to be in the order of 1800 mm dia. Currently the trunk main alignment is shown as continuing along the paper road and discharging into the

Waingaehe Stream. Further investigation would have to be undertaken to determine if this is feasible given the presence of stop banks in the lower reach of the Waingaehe Stream and whether there is capacity in the stream to receive this flow. Alternatively, the open channel along the rear of 75, 83 and 95 Galvin Road could be formalised, and a drainage corridor established to protect this outlet (either by easement or land purchase). We note there appears to be an existing wetland area at 14 and 22 Umukaria Road that may be able to be incorporated into this outlet to provide water quality treatment, however this may require additional land purchase by RLC.

Depending on land ownership, the large vacant block of land at 18 Hinemoa Point Road may not be able to discharge to Umukaria Road and may instead have to drain back to Owhata Road because of topographical constraints. However, the existing Owhata Road Drain is quite deep and if this road is not urbanised the trunk main shown draining south-west on Hinemoa Point Road may be able to discharge to the Owhata Road Drain.

There is also a currently undeveloped rural block south of Porikapa Road at the top of Catchment 3 (see Figure 5.9). It is proposed that mitigation for this area is achieved by diverting runoff into Subcatchment 2, however the feasibility of this will depend on the proposed development layout. It is noted that only a portion of primary flows needs to be diverted into Subcatchment 2 to offset the increase in runoff as a result of development. Otherwise, this area would likely require onsite detention as it drains to an existing pipe network. Currently this area is included in the sizing of Pond 2-8 (refer Section 5.1.1).

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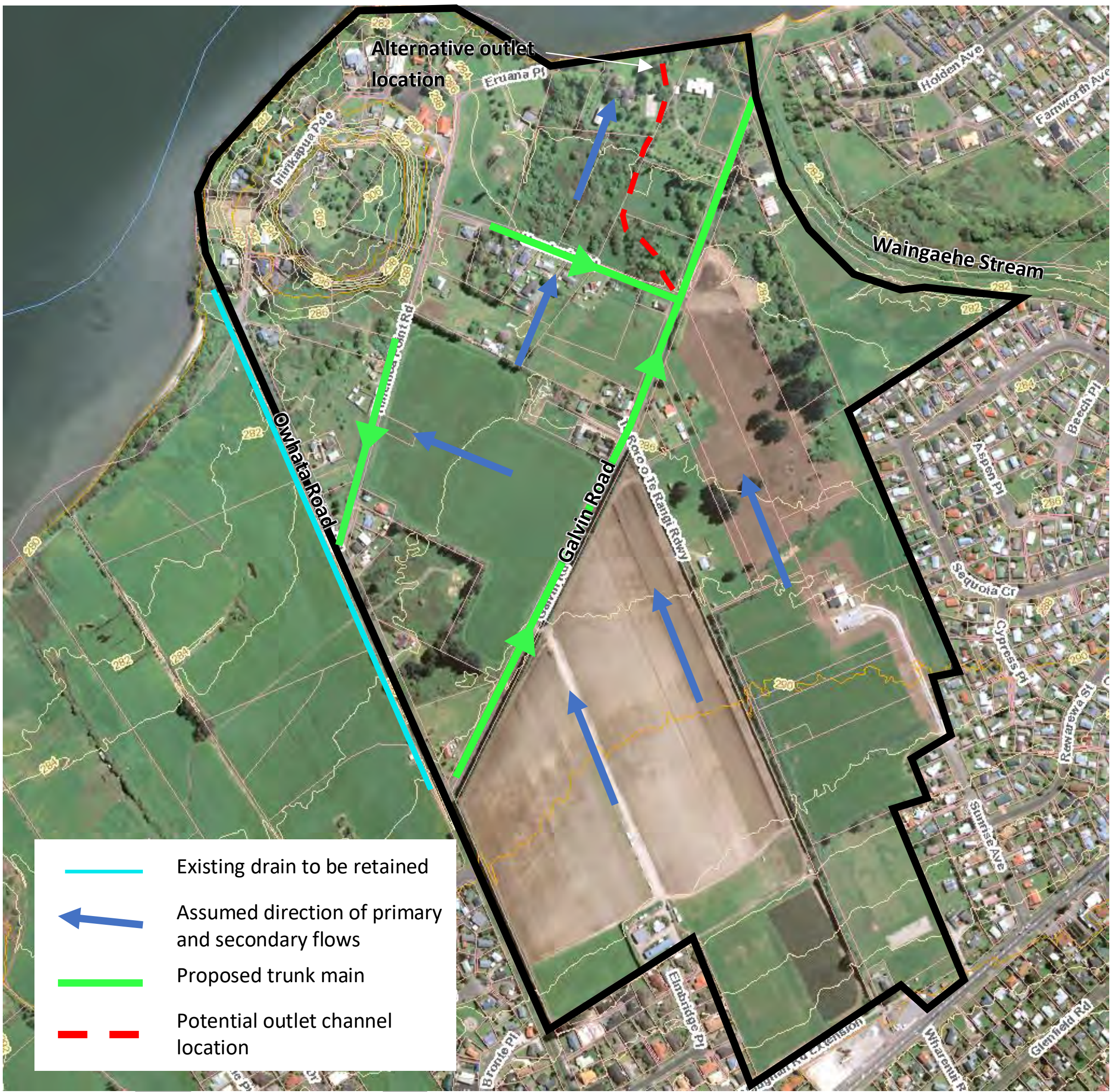


*Figure 5.10: Proposed location of stormwater trunk mains for Subcatchment 3*

### 5.2.2 Recommendation

For Subcatchment 3 the preferred approach is the pass flows forward approach (no detention of post-developed flows) with conveyance of primary flows in a new trunk main under Galvin Road and utilising the road network (including existing roadside open channel if retained) as an overland flowpath.

Further investigations should be undertaken by RLC around land purchase or easement creation to secure an outlet at the end of Galvin Road. If the option of using the existing paper road is preferred, the effect of this on the Waingaehe Stream should be assessed as part of the future work recommended for Subcatchments 1 and 2 (refer Section 5.1.4). The feasibility of the mitigation approach for the top of Subcatchment 3 should be further investigated as part of the flood management approach for Subcatchment 1 and 2.

We also recommend that further work be undertaken to confirm the hydrological parameters that should be used for the eastern catchments to improve the confidence in peak flow estimates. We note that these catchments have been assessed by multiple consultants using a variety of methods and there is a wide range in peak flow estimates. This is due to the complexity of infiltration/interflow dynamics in pumiceous soils which are not well captured in traditional rainfall/runoff methods as well and potentially erroneous data capture from the Waingaehe Stream gauge.

## 5.3 Subcatchment 4

The growth areas within Subcatchment 4 for which mitigation has been assessed include Wharenui Block, Owhatirua South and the development areas near Vaughan Road (refer Figure 5.11).

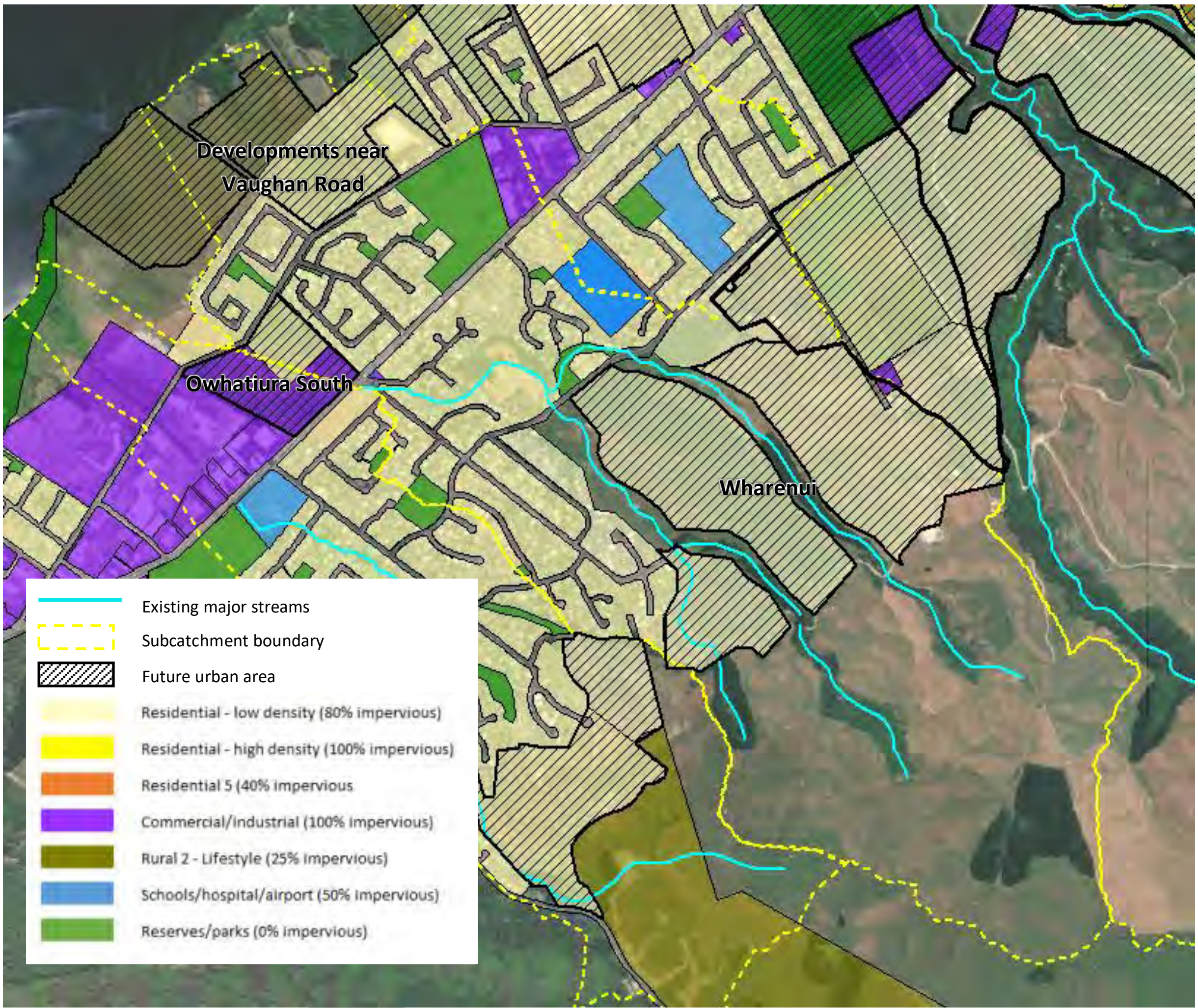


*Figure 5.11: Development areas to be mitigated within Subcatchment 4*

Mitigation for Owhatiura South will be to redirect piped and overland flows so that this development drains to Catchment 4, specifically the open channel on the eastern boundary of this development area (referred to herein as the Owhatiura Eastern Drain) (refer Figure 5.14). While this may not be possible for the entire development area, specifically the north-west corner, enough of the flow from the development should be able to be redirected so that post-developed flows are no greater than pre-developed flows.

Because Owhatiura South and the development areas near Vaughan Road are located in the bottom half of the subcatchment, the recommended mitigation approach for these areas is to not attenuate flows but rather pass flows forward so peak flows from these areas do not coincide with peak flows from the upper catchment. For this approach it is proposed that a new open channel is created from Vaughan Road (near the intersection with Carroll Place) to the lake (refer Figure 5.14). This will secure an outlet for future development along Vaughan Road and will reduce the overland flow heading westward along Vaughan Road to the intersection of Tennyson Drive which is an area known for flooding. For passing flows forward from Owhatiura South the Owhatiura Eastern Drain will require upgrade. The sizing of this channel (and associated culverts/bridges) depends on the approach taken in the upper catchment and is discussed further in the following subsections.

Two approaches have been considered for mitigating the effects of development in the Wharenui Block:

- Detention upstream of Morey Street.
- Improved conveyance through the Melrose Ave area and larger downstream conveyance to convey unattenuated flows from upper catchment (i.e. no detention).

These options are discussed further in the following subsections.

### 5.3.1 Option 4A – centralised detention

**Upper catchment**

Option 4A involves construction of two flood detention dams to mitigate development in the upper catchment (refer Figure 5.12) in addition to conveyance upgrades to allow a pass flows forward approach in the lower catchment (refer Figure 5.14). One dam would be located in the eastern gully immediately upstream of Morey Street (Dam 4-1), while the other dam would be located further upstream of Morey Street in the western gully (Dam 4-2).

We have assessed the detention designs for multiple durations of design rainfall events to ensure attenuation objectives are met for a range of storm profiles. These dams are sized to provide attenuation of post-development peak flows to pre-development levels for the 1 hr and 6 hr events. Longer duration/lower intensity storms (e.g. the 12 hr and 24 hr events) are not predicted to result in downstream flooding so attenuation for these events has not been targeted. Table 5.3 presents a summary of the indicative option sizing. The dam crest elevations at this stage are based on providing some nominal freeboard above the 100 year ARI water level, however conveyance of less frequent events to meet dam safety requirements (as outlined in the NZSOLD Dam Safety Guidelines) has not been assessed and will determine the required crest level.

**Table 5.3: Summary of detention sizing for Option 4A**

| Structure reference | Structure type | Crest elevation | Dam height | 100 yr live storage volume |
|---|---|---|---|---|
| 4-1 | Dam | 307.5 m RL | 3.5 m | 20,000 $m^3$ |
| 4-2 | Dam | 315 m RL | 7 m | 43,000 $m^3$ |

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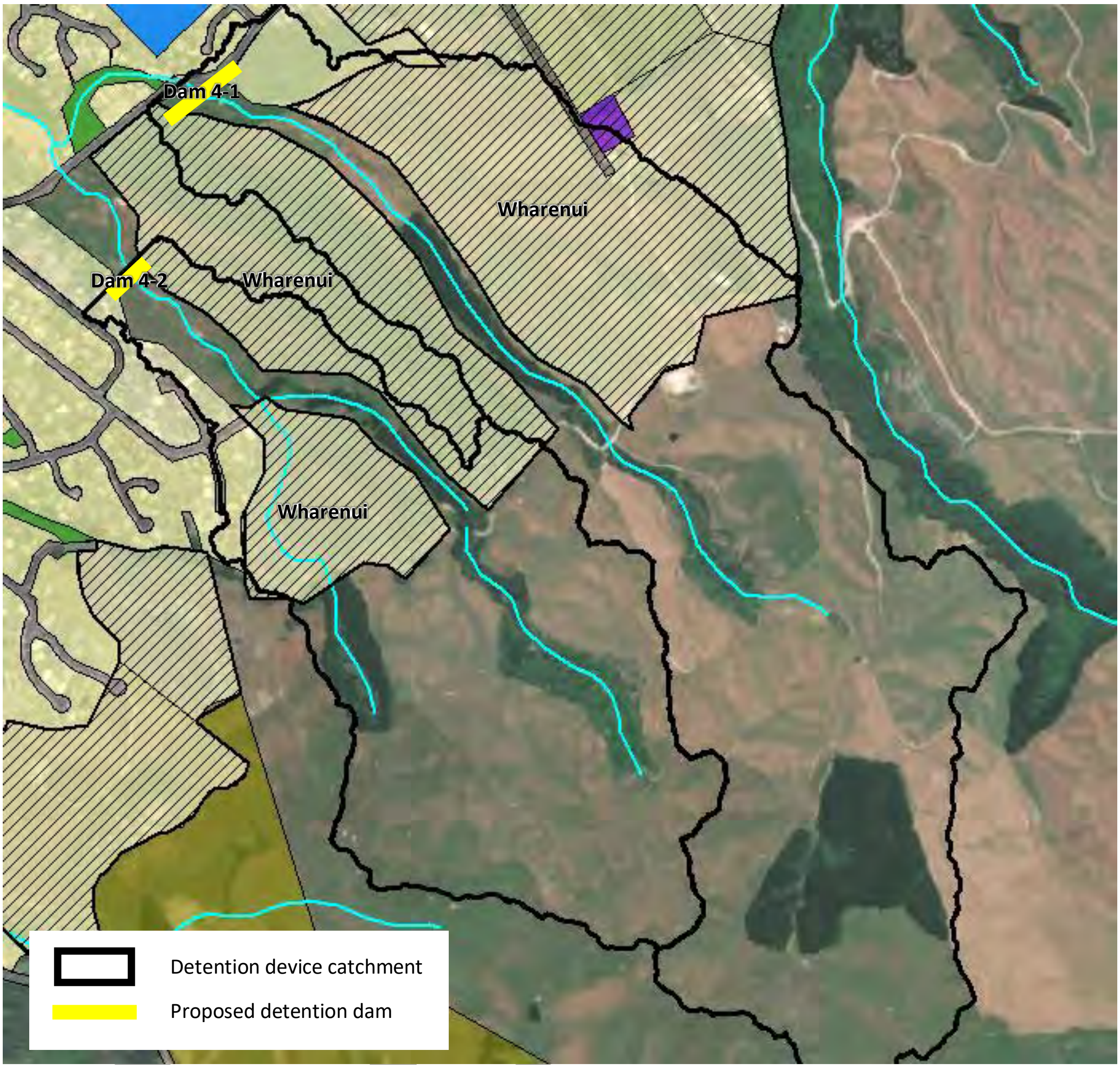


*Figure 5.12: Location of proposed flood detention dams for Subcatchment 4*

The upstream detention concept comprises the following features:

- Eastern Gully diversion and attenuation.
- Increase in the culvert passing underneath Morey Street from the eastern gully from 1600 mm dia. to 1800 mm dia.
- Spillway channel to the existing natural storage area in the western gully upstream of Morey Street.
- New western gully flood alleviation dam.

The location of these features is shown in Figure 5.13. The detention dam in the eastern gully will have spillway that is designed to divert flows in excess of the culvert capacity under Morey street into the western gully. The flows passing through the eastern gulley culvert will increase as a result of increasing the culvert size, but due to the significant reduction in road overflow the overall peak flows will reduce. There is potential to optimise the eastern culvert and outlet configuration further to control flood flows for more frequent events. The opportunity to increase the culvert size further to direct more flows to the Morey Street Reserve and reduce flood flows from the western gully should be investigated further in subsequent design stages.

The diversion structure requires an approximately 120 m long embankment running along the eastern side of Morey Street at a level of 307.5 m RL and a spillway channel approximately 18 m wide to the west. It is estimated the spillway channel needs to be able to convey flows up to approximately 12 $m^3/s$ in a 100 year event. However, we note that given this dam will be a large dam there may be the need to convey less frequent events to meet dam safety guidelines. There is potential to use the fill cut from the spillway channel to build the dam embankment. At this stage the proposed crest level is consistent with the level used by Basin 1 for the 1st stage of the Wharenui development. It is noted that in subsequent design more flood storage may be provided by raising the embankment higher. However, the effect of this on Basin 1 would need to be considered and discussed with the developer.

We understand that RLC is proposing an upgrade to Morey Street to add a cycleway. If this option is adopted as the preferred option, the design of the dam along Morey Street will need to consider the proposed road upgrades.

This option does provide some scope to scale up the solution to mitigate existing downstream flooding. Opportunities to further mitigate flood flows include:

- Adding a third detention dam in the central gully that is located in between the east and western gullies (currently unattenuated).
- Raising the road embankment on the upstream side of Morey Street (west gully). However, the effect of this on nearby properties would need to be assessed as the bottom of these properties are situated at approximately 305 m RL.
- Excavating the west gully area immediately upstream of Morey St to increase available storage, and use this fill for one of the detention dams (or within the subdivision earthworks).

We also note that the current detention concept for the Wharenui development assumes that all development has occurred. However, in reality this development will be staged. Therefore, further refinement of concept is required to ensure that the concept works with the two dams acting independently of one another and also to determine at which stage in development a second dam is required.

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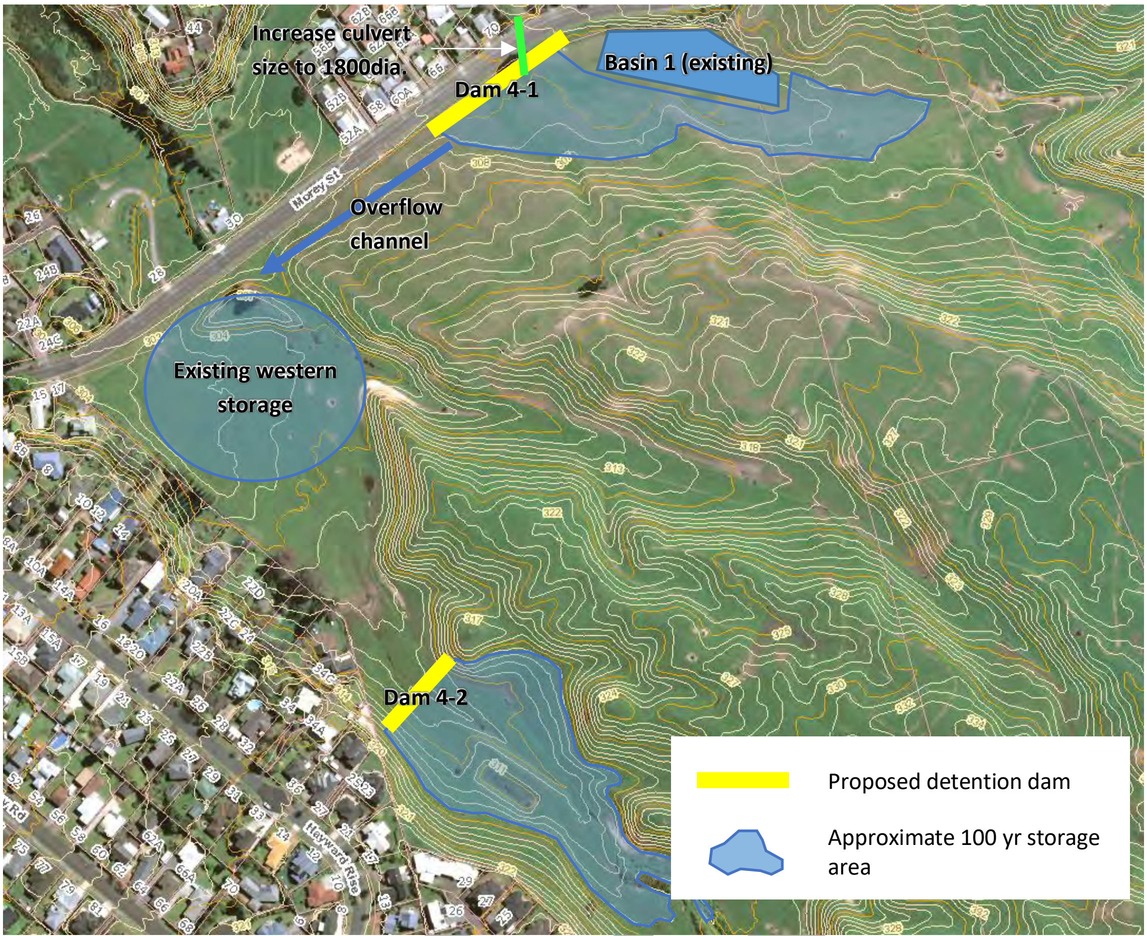


*Figure 5.13: Proposed location of Dam 4-1 and Dam 4-2 for Option 4A (base image sourced from RLC's Geyserview)*

**Lower Catchment**

Option 4A would also include conveyance upgrades of the Owhatiura Eastern Drain and creation of a new open channel to service the development near Vaughan Road. The size of the drains required for this option are summarised in Table 5.4 below. The location of these upgrades is shown in Figure 5.14.

The estimated 100 year design flow for the Te Ngae Road Culverts is 15 $m^3/s$. While it appears from high level calculations that the Te Ngae Road culverts have sufficient capacity to pass the 100 year event, this will need to be assessed further, as there are known flooding issues upstream in Melrose Place. We understand that there are proposed state highway upgrades to Te Ngae Road (not part of the stormwater masterplan project). If this option is progressed to engineering feasibility/preliminary design, there will need to be engagement with Waka Kotahi (the NZ Transport Agency) to collaborate on a solution for conveyance in the vicinity of Te Ngae Road.

The estimated 100 year design flow for the Owhatiura Eastern Drain (and associated culverts) is 23 $m^3/s$ for Option 4A. Based on a high level assessment of hydraulics it appears that sufficient capacity exists in the section of the Owhatiura Eastern Drain from Vaughan Road to Carroll Place. Therefore it is anticipated that only the section of drain from Te Ngae Road to Vaughan Road will need to be upgraded to facilitate development of Owhatiura South.

Based on high level calculations the Carroll Place bridge appears to have sufficient capacity but the culvert that conveys flow under Vaughan Road will require upgrading. High level calculations show that a bridge/culvert approximately the size of the Carroll Place bridge is required (2 m high x 6.4 m wide). However, more detailed hydraulic modelling of the downstream channel should be undertaken to confirm the scope of the required conveyance upgrades.

The estimated design flows for Owhatiura Eastern Drain are based on detention of the Wharenui Development to pre-development peak flows and diversion of all flow from the Owhatiura South development into the channel. We note that the calculated peak flows from this study are higher than work done by other consultants. We recommend that further work be undertaken to confirm the hydrological parameters that should be used for the eastern catchments to improve the confidence in the sizing of the required conveyance and detention sizing.

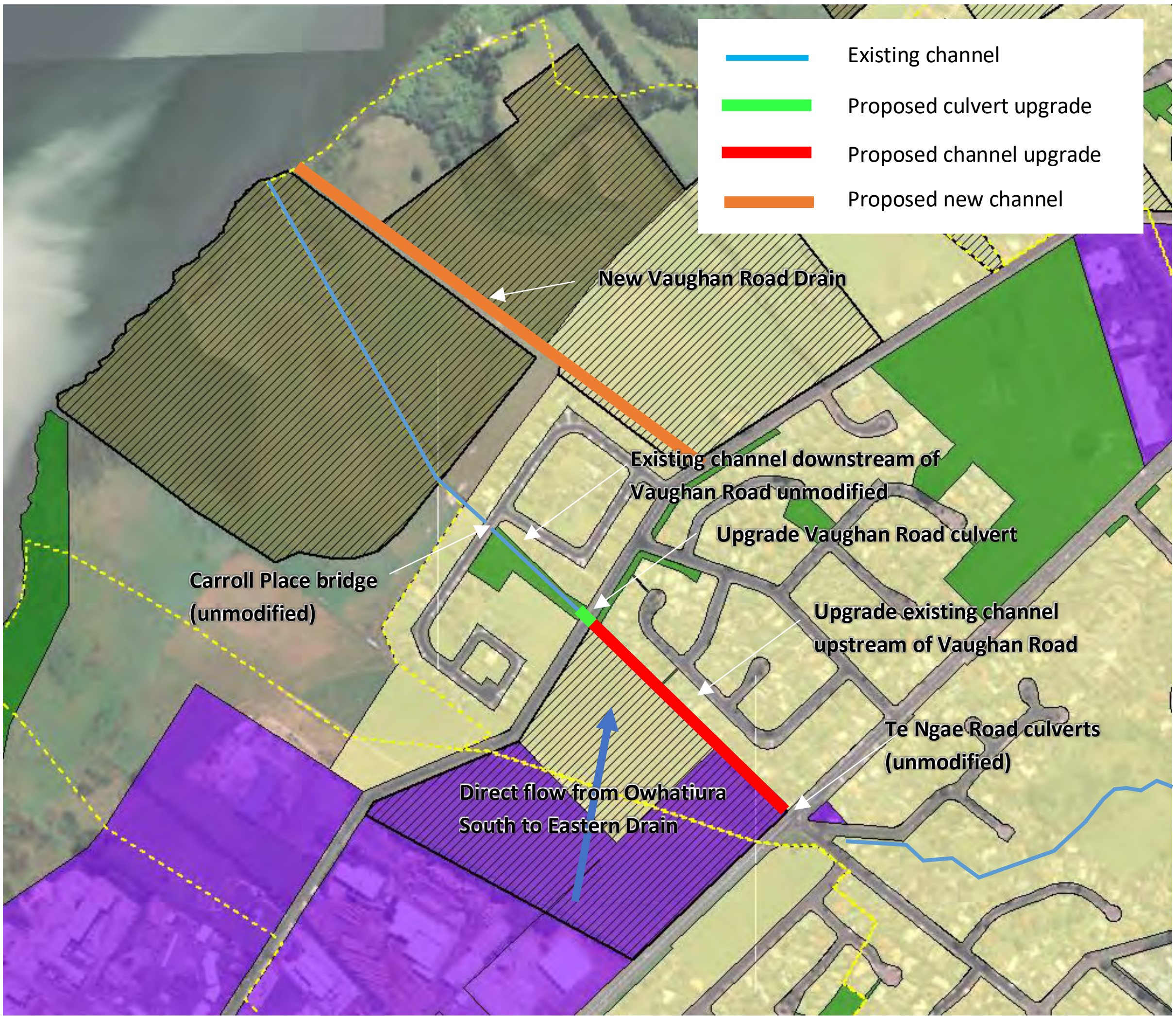


*Figure 5.14: Location of proposed conveyance upgrades in lower catchment for Option 4A*

**Table 5.4: Summary of proposed open channel upgrades for Option 4A**

| Channel location | Estimated 100 year design flow | Required channel size |
|---|---|---|
| Owhatiura Eastern Drain (Te Ngae Road to Vaughan Road) | 23 $m^3/s$ | 2 m deep, 12.5 m top width at 1% grade |
| Owhatiura Eastern Drain (Vaughan Road to Carroll Place) | 23 $m^3/s$ | No change |
| Vaughan Rd Drain | 11 $m^3/s$ | 1.5 m deep, 10 m top width at 1% grade |

### 5.3.2 Option 4B - conveyance without detention

Option 4B involves conveyance upgrades from Morey Street to the lake, with no attenuation of post-development flows from the upper catchment. This option includes the following features:

- Upgrade of the Morey Street culverts.
- Upgrade of the Melrose Ave channel.
- Upgrade of the Te Ngae Road culverts.
- Upgrade of the Owhatiura Eastern Drain.
- Upgrade of the Vaughan Road culvert.
- Creation of a new open channel to service the developments near Vaughan Road (same concept as Option 4A).

The indicative sizing of the drain upgrades is summarised in Table 5.5. The location of the proposed conveyance upgrades is shown in Figure 5.15.

For Option 4B the estimated 100 year design flow for the Morey Street Culverts and Melrose Ave Drain is 19 $m^3/s$. This would require upgrading both the existing Morey Street Culverts. It is estimated that a 2.5 m wide x 1.5 m high box culvert would be required under Morey Street for each of the main gullies. The existing Melrose Ave Drain is already known to be under capacity and would require upgrade for this option. The sizing presented in Table 5.5 is based on a naturalised trapezoidal channel from 28 Morey Street to Melrose Ave. Along Melrose Ave a timber walled u-channel is proposed given the lack of width available in the berm. This may require some power poles to be relocated.

It is proposed that a drainage corridor be established for the Melrose Ave Drain upgrade by compulsory land acquisition and several dwellings and appurtenant structures removed to allow for the widening of the existing channel. At a minimum, seven properties would be affected (refer Figure 5.16). Some of the cost of the land acquisition could likely be recouped by sale of some of the properties back to market with revised boundaries. Land swaps in newly developed land in the area could be investigated to minimise disruption for displaced residents. However, it is recommended that RLC seek legal advice on this if it is the preferred option.

The estimated 100 year design flow for the Te Ngae Road culverts would be approximately 22 $m^3/s$ for Option 4B. This will require upgrading the culvert with high level calculations showing at least another 1.5 m x 1.5 m box culvert would be required in addition to the two existing culverts.

The estimated 100 year design flow for the Owhatiura Eastern Drain (and associated culverts) is 32 $m^3/s$ for Option 4B. Based on a high level assessment of hydraulics it appears that the Owhatiura Eastern Drain from Te Ngae Road to Vaughan Road and from Vaughan Road to Carroll Place will need to be upgraded to convey these flows. The culvert that conveys flow under Vaughan Road will require upgrading. High level calculations show that a bridge/culvert approximately the size of the Carroll Place bridge (2 m high x 6.4 m wide) may just provide enough capacity. Therefore the Carroll Place bridge is assumed to not require upgrade for this option. However, more detailed hydraulic modelling of the downstream channel should be undertaken to confirm the scope of the required conveyance upgrades.

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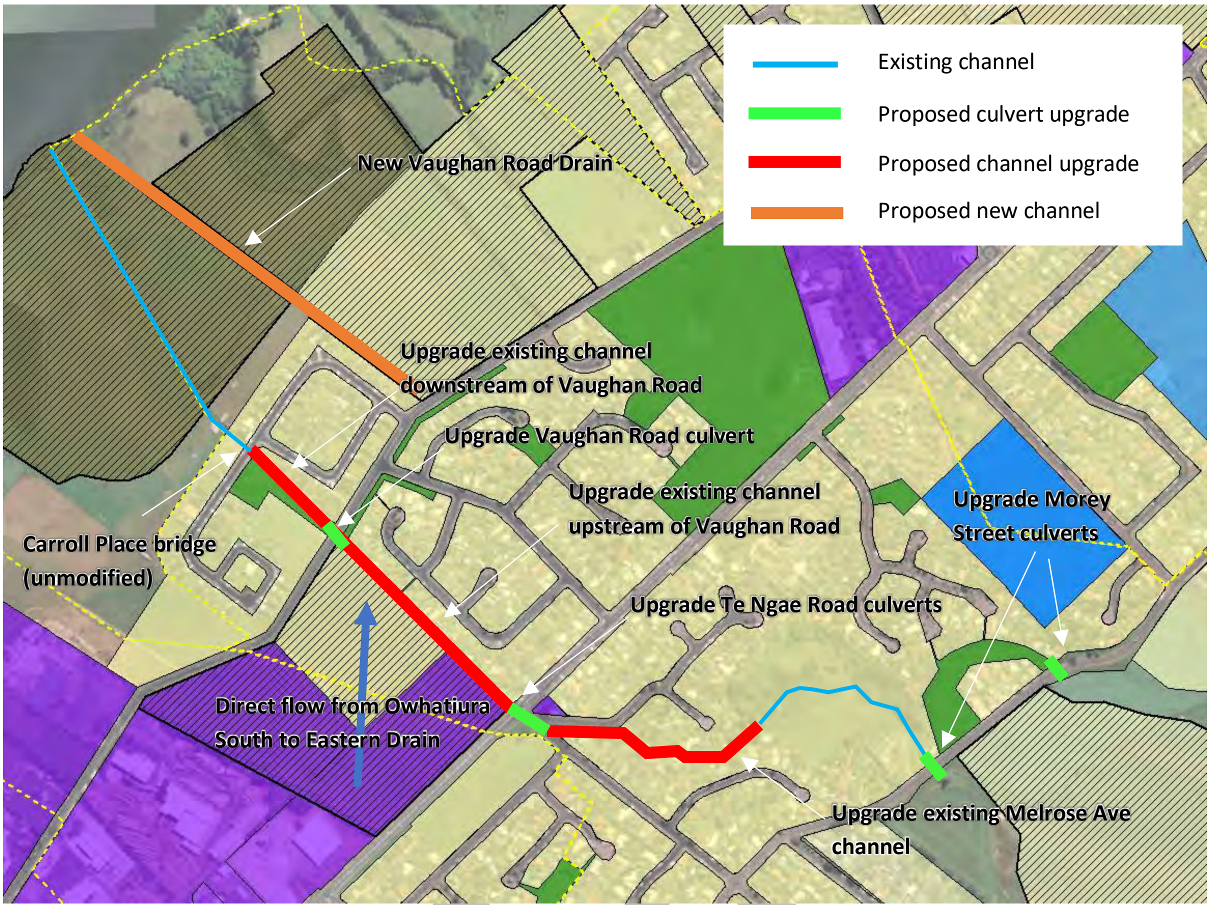


*Figure 5.15: Location of proposed conveyance upgrades for Option 4B*


*Figure 5.16: Possibly affected properties from upgrade of Melrose Ave Drain (current drain alignment shown in blue)*

**Table 5.5: Summary of open channel upgrades for Option 4B**

| Channel location | Estimated 100 year design flow | Required channel size |
|---|---|---|
| Melrose Ave Drain (naturalised section) | 19 $m^3/s$ | 1 m deep, 15 m top width at 0.5% grade |
| Melrose Ave Drain (timber U channel) | 19 $m^3/s$ | 1 m deep, 5 m wide at 0.5% grade |
| Owhatiura Eastern Drain (Te Ngae Road to Vaughan Road) | 32 $m^3/s$ | 2 m deep, 13 m top width at 1% grade |
| Owhatiura Eastern Drain (Vaughan Road to Carroll Place) | 32 $m^3/s$ | 2.5 m deep, 18 m top width at 0.2% grade |
| Vaughan Rd Drain | 11 $m^3/s$ (same as Option 4A) | 1.5 m deep, 10 m top width at 1% grade |

### 5.3.3 Recommendation

Table 5.6 below summarises the qualitative options assessment that has been undertaken for Subcatchment 4. Option 4A is the preferred option that should be progressed to engineering feasibility/preliminary design. This option allows growth of the upstream Wharenui Block to occur with less disruption to existing landowners and requires fewer landowner approvals than the conveyance only option. It also is a more conventional approach and keeps more future options open for flood management.

The current detention concept for the Wharenui development assumes that all development has occurred. However, in reality this development will be staged. Therefore, further refinement of the concept is required to ensure that the concept works with the two dams acting independently of one another and also to determine at which stage in development a second dam is required.

This option has the potential to address existing flooding in Melrose Ave by increasing the size of the upstream storage areas. This should be investigated further in engineering feasibility/preliminary design. Alternatively, if flooding in Melrose Ave needs to be addressed with an increase in conveyance capacity, the scale of the required upgrade will be much lower with attenuation of upstream development.

We note that the proposed flood detention dams and conveyance upgrades only provide a flood mitigation function. Onsite stormwater management devices would likely still be required to provide functions such as extended detention (for mitigation of stream stability risk, if required) and water quality treatment.

We also recommend that further work be undertaken to confirm the hydrological parameters that should be used for the eastern catchments to improve the confidence in peak flow estimates. We note that these catchments have been assessed by multiple consultants using a variety of methods and there is a wide range in peak flow estimates. This is due to the complexity of infiltration/interflow dynamics in pumiceous soils which are not well captured in traditional rainfall/runoff methods, a lack of site specific rainfall data, as well as potentially erroneous data capture from the Waingaehe Stream gauge.

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**Table 5.6: Summary of qualitative options assessment for Subcatchment 4**

| | **Centralised detention (Option 4A)** | **Conveyance without detention (Option 4B)** |
|---|---|---|
| Rough order cost estimate (including professional fees) | $8.2 to $12.1 million | $10.1 to $14.8 million (includes $3.5 million of property purchase, some of which could be recouped) |
| Ability to mitigate flooding | • Provides mitigation for growth<br>• Addresses existing downstream flooding/climate change effects in lower catchment<br>• Potentially can address existing downstream flooding/climate change effects in Melrose Ave area if size of dams increased | • Provides mitigation for growth<br>• Addresses existing downstream flooding/climate change effects in Melrose Ave area and in lower catchment |
| Ability to address other stormwater objectives (e.g. water quality, erosion etc.) | • Provides opportunity to address existing channel erosion issues<br>• Extended detention and water quality treatment will likely have to be achieved in separate stormwater management devices | • Provides opportunity to address existing channel erosion issues<br>• Extended detention and water quality treatment will likely have to be achieved in separate stormwater management devices |
| Speed of implementation | • Land for dams already owned by developer but design and consenting process required for two dams prior to construction<br>• Land for Vaughan Drain not currently owned by RLC | • Dependant on the mechanism required for obtaining a drainage corridor through Melrose Ave. Compulsory acquisition of land can be a lengthy process if required<br>• Land for Vaughan Drain not currently owned by RLC |
| Constructability and maintenance | • Ongoing dam safety management and maintenance for two dams | • Construction access to Melrose Ave drain could be difficult |
| Potential risks and opportunities (engineering, consenting etc.) | • Slope instability risk to nearby properties<br>• Suitability of the underlying soils for construction of an earth embankment<br>• Dam break risk to downstream properties<br>• Land acquisition (if required) for Vaughan Road drain may be difficult<br>• Conveyance upgrades in lower catchment provide opportunity to address existing lateral spread issues with the proximity of drains to dwellings | • Multiple landowner approvals may be required to be feasible<br>• BOPRC may not agree with a conveyance based/no attenuation approach out of principle<br>• Conveyance upgrades in lower catchment provide opportunity to address existing lateral spread issues with the proximity of drains to dwellings<br>• Land acquisition (if required) for Vaughan Road drain and Melrose Ave drain may be difficult |
| Social/cultural/environmental impacts | • Function of flood detention dam near residential dwellings would have to be carefully communicated to community | • Compulsory acquisition of properties (if required) may not be viewed favourably by the community |

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| | Centralised detention (Option 4A) | Conveyance without detention (Option 4B) |
|---|---|---|
| Land purchase required? | • Land purchase may be required for Vaughan Road drain | • Land purchase may be required for Vaughan Road drain<br>• Land purchase may be required through Melrose Ave area |
| Other considerations | • The subdivision, pipe network and road network design for the developments need to consider how 100 year flows will be conveyed to detention devices/open channels | • The subdivision, pipe network and road network design for the developments need to consider how 100 year flows will be conveyed to detention devices/open channels |

## 5.4 Subcatchment 5

The growth areas within Subcatchment 5 for which flood mitigation options have been assessed include the Link Road, Spence block development areas and Owhatiura South (refer Figure 5.17). Mitigation for Owhatiura South will be to redirect piped and overland flows so that this development drains to Catchment 4. While this may not be possible for the entire development area (specifically the north-west corner), enough of flow from the development should be able to be redirected so that post-developed flows are no greater than pre-developed flows. This concept is discussed further in Section 5.3.

For mitigation for the Link Road/Spence Block development mitigation options are limited due to topographical constraints and existing flooding issues in the catchment. The following options have been considered for these developments:

- Decentralised detention.
- Centralised detention and diversion down Iles Road.

These options are discussed further in the following subsections.

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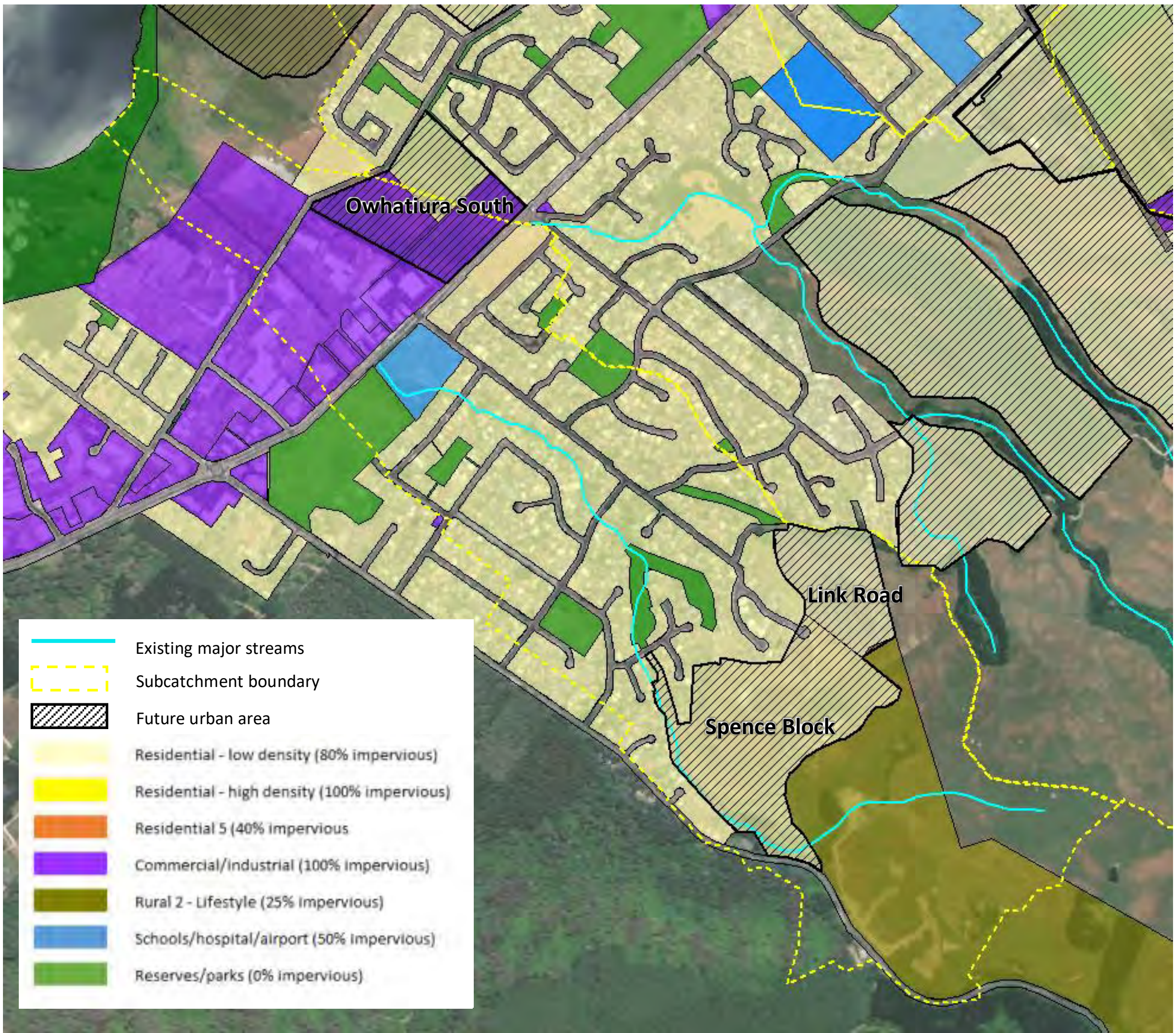


*Figure 5.17: Development areas to be mitigated within Subcatchment 5*

### 5.4.1 Option 5A - decentralised detention

Option 5A involves construction of two separate flood detention dams to mitigate development in the upper catchment (refer Figure 5.18). One dam would be located on private property within the existing stream gully near Link Road (Dam 5-1) while the other dam would be located within the council-owned stream reserve near Shane Place (Dam 5-2). Dam 5-1 would provide attenuation for the Link Road development area while Dam 5-2 would provide attenuation for the Spence Block development area.

We have assessed the detention designs for multiple durations of design rainfall events to ensure attenuation objectives are met for a range of storm profiles. These dams are sized to provide attenuation of post-development peak flows to pre-development levels for the 1 hr and 6 hr events. Longer duration/lower intensity storms (e.g. the 12 hr and 24 hr events) are not predicted to result in downstream flooding so attenuation for these events has not been targeted. Table 5.7 presents a summary of the indicative option sizing. The dam crest elevations are based on providing some nominal freeboard above the 100 year ARI water level, however conveyance of less frequent events to meet dam safety requirements has not been assessed and may determine the required crest level.

**Table 5.7: Summary of detention sizing for Option 5A**

| Structure reference | Structure type | Crest elevation | Dam height | 100 yr live storage volume |
|---|---|---|---|---|
| 5-1 | Dam | 328 m RL | 5 m | 10,000 $m^3$ |
| 5-2 | Dam | 313 m RL | 7 m | 22,000 $m^3$ |

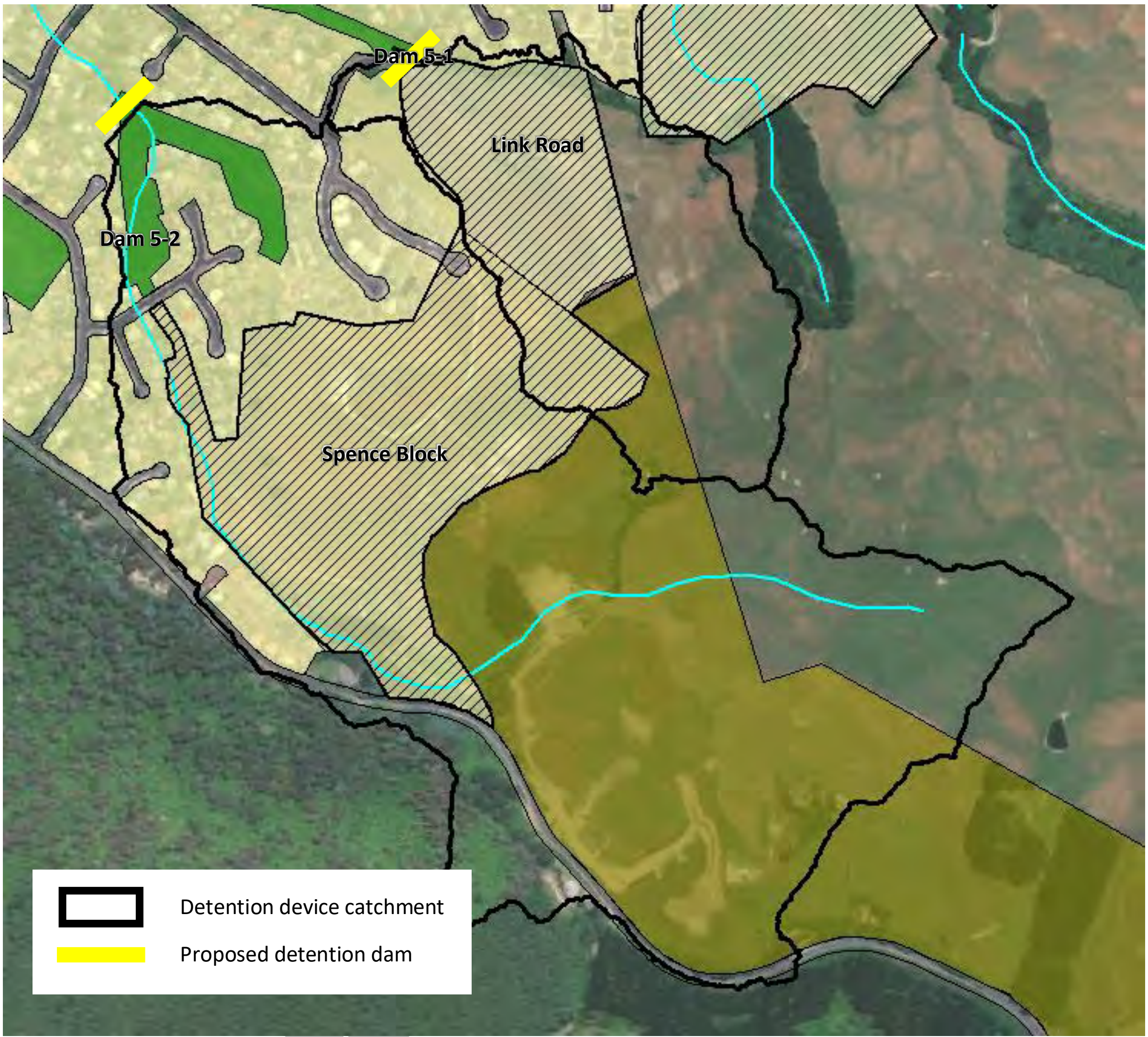


*Figure 5.18: Location of proposed flood detention dams for Subcatchment 5*

Figure 5.19 and Figure 5.20 show the proposed location of Dam 5-1 and Dam 5-2.

For Dam 5-1 we note there is potential to either mitigate development from outside the immediate catchment (i.e. increase the catchment size), and/or over-attenuate flood flows to reduce downstream flood effects. For each additional metre of embankment elevation above the proposed crest height, there is an additional 10,000 $m^3$ (approximately) of storage volume available to provide flood mitigation and/or reduce the downstream flooding issues. This could be investigated further in engineering feasibility/preliminary design.

Dam 5-1 and the upstream flood storage will either require raising of the existing accessway to 15 and 15A Link Road by up to 3 m, subject to the acceptable level of service for flooding of the accessway, or realigning/relocating the accessway (e.g. access off Butler Place).

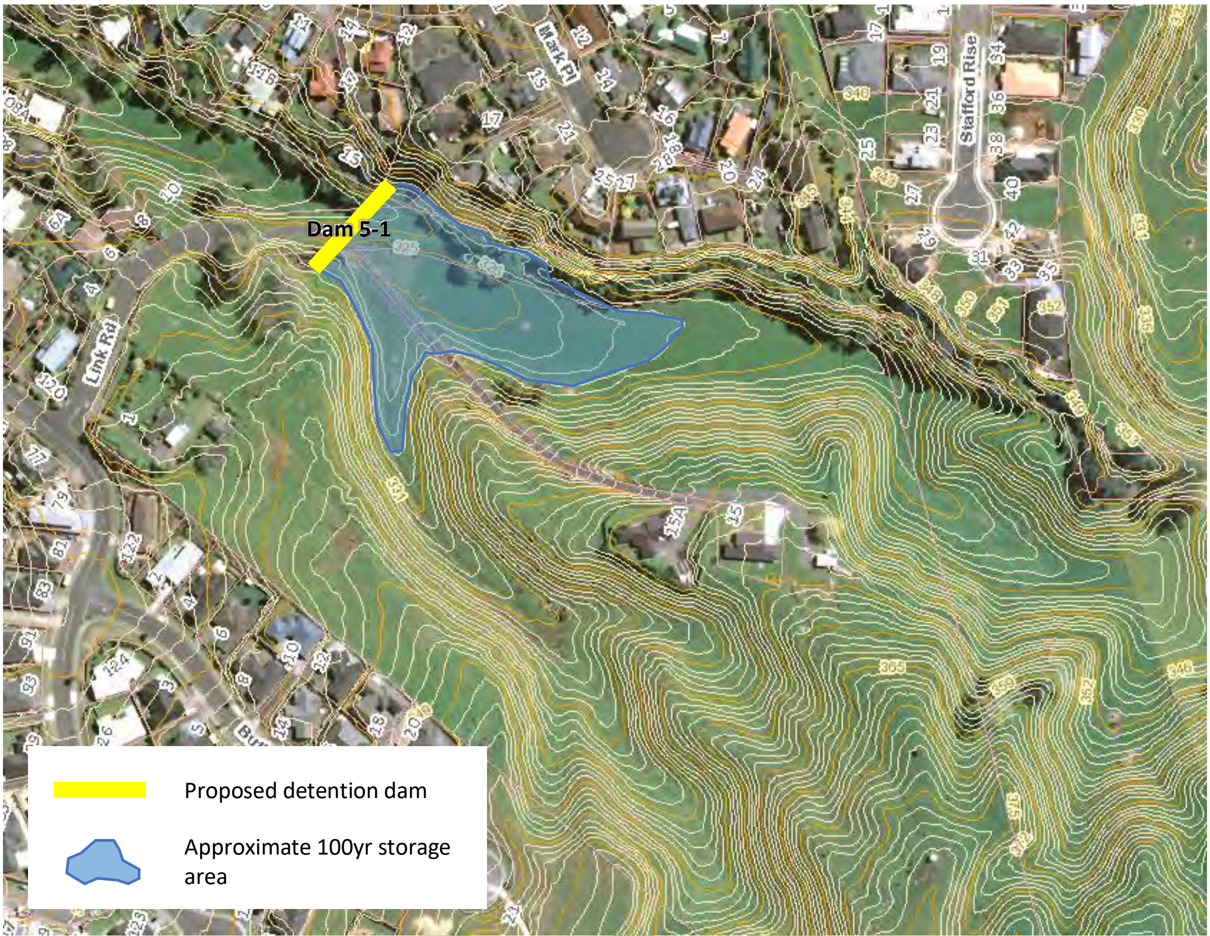


*Figure 5.19: Proposed location of Dam 5-1 for Option 5A (base image sourced from RLC's Geyserview)*

Dam 5-2 is located within council-owned stream reserve, however the 100 year peak water level is predicted to inundate some of the slope to the rear of 39 Walford Drive. The proximity of the houses/properties at the top of the stream bank are a key constraint on how much storage can be achieved (refer Figure 5.20). It appears that the lowest floor levels around the gully are approximately 315 m RL.

The site has difficult access due to the built-up nature of the area, therefore landowner discussions and approvals will be required for this option to progress. We also note that there are properties located at the top of the banks (above the upstream inundation area) and the effects of flooding and dewatering on the slope stability will require further investigation, as feasibility of the option is assessed in future stages.

Another key consideration for this option is the conveyance requirements to get flow to Dam 5-2. Because the dam is located slightly downstream from the development the ability of existing reticulated pipe networks and secondary flowpaths through Janet Place, Walford Drive, Butler Place and Iles Road to pass increased flows to the dam will need further consideration once site layout plans for the development are confirmed.

If sufficient capacity is not available in the existing pipe networks, the most economical solution would likely be to direct flow to the gully to the west and under Walford Drive. This would possibly require upgrading the existing culverts under Walford Drive.

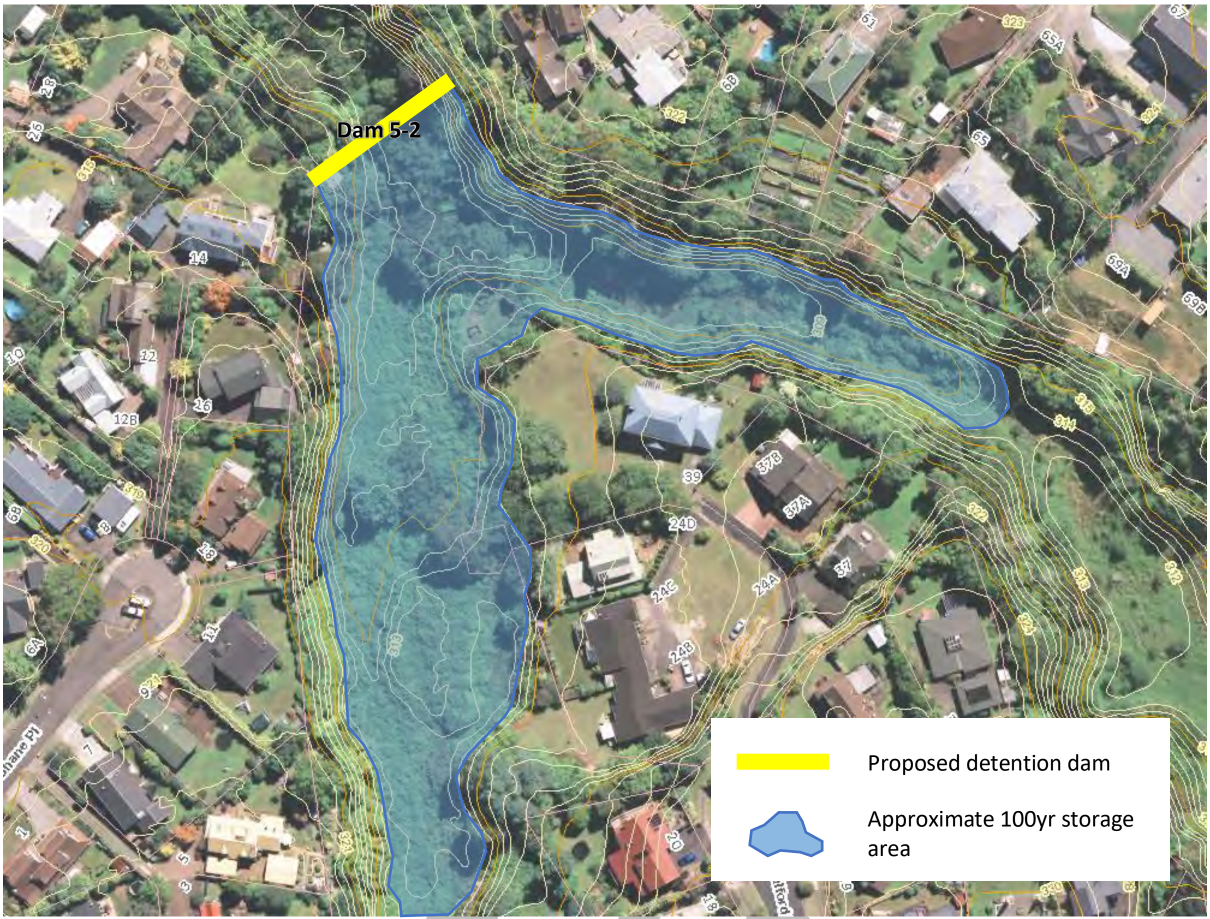


*Figure 5.20: Proposed location of Dam 5-2 for Option 5A (base image sourced from RLC's Geyserview)*

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### 5.4.2 Option 5B - catchment diversion and detention

Option 5B involves construction of one flood detention dam in the same location as Dam 5-1 described for Option 5A (refer Section 5.4.1). It is proposed that this flood detention dam provides attenuation for the entire development area in the upstream catchment by redirecting flow from the Spence Block north-east into the gully near Link Road. This could be done by diverting only primary (i.e. piped) flows using the reticulated stormwater network or by using the road network to also divert secondary/overland flows. If only primary flows are diverted, secondary flows will still drain to the north-west unattenuated, however they would likely be lower than pre-developed flows due to the diversion of primary flows. Sufficient grade is likely to be available to do this as the lowest development level will be approximately 370 m RL and the top of the gully at the top of Butler Place is at about 355 m RL.

If feasible this option would have the ability to mitigate for development and could also potentially address existing flooding depending on how much catchment can be diverted and the level of attenuation that can be achieved in the dam. The required storage volume to achieve this was not assessed as part of this study. However, it also requires a coordinated approach for both development areas (Link Rd and Spence Block). RLC has advised that such a solution is not likely to be possible because of a lack of collaboration between land owners and so have instructed that this option not be progressed any further.

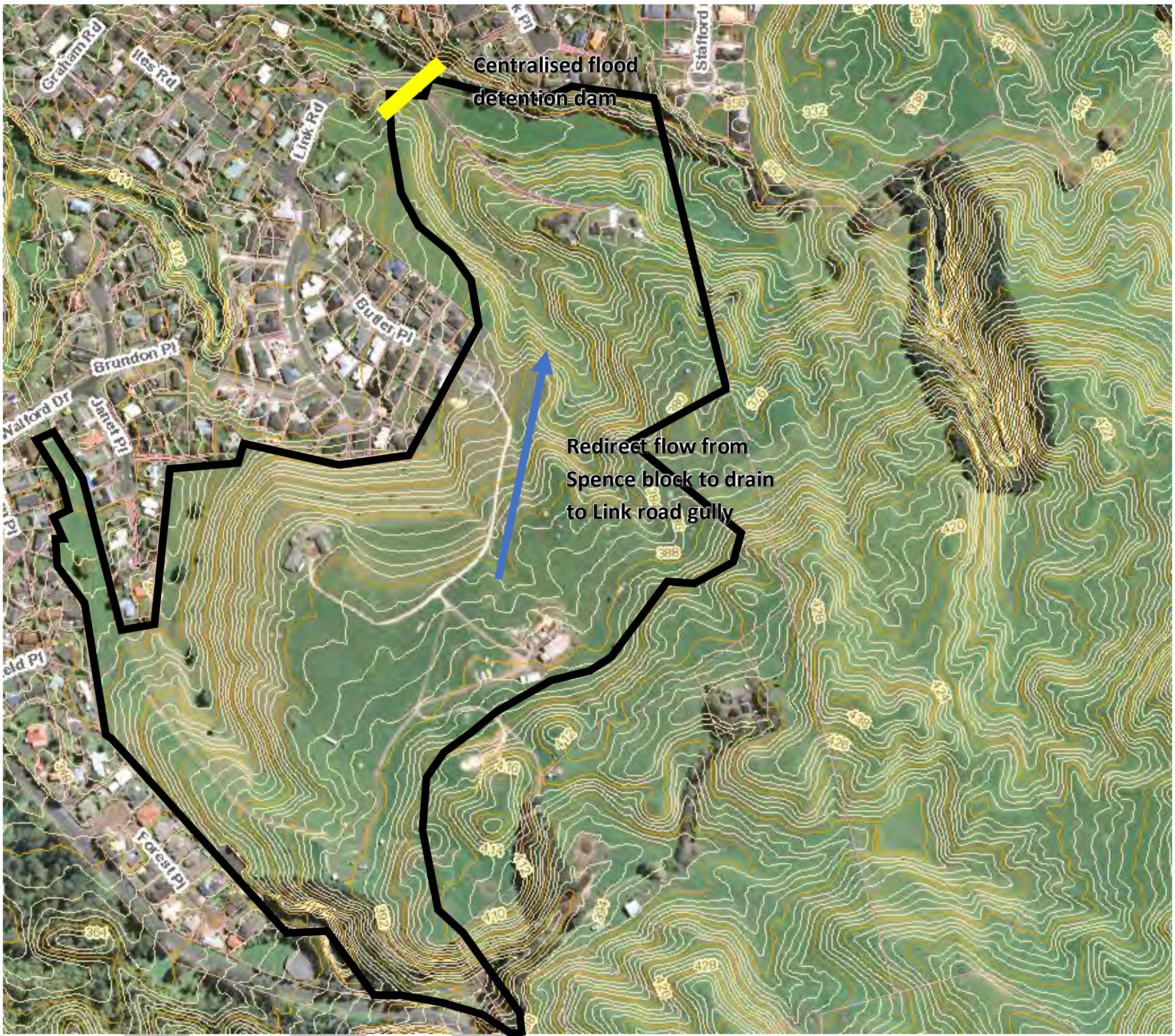


*Figure 5.21: Proposed location of dam for Option 5B (base image sourced from RLC Geyserview)*

### 5.4.3 Recommendation

Table 5.8 below summarises the qualitative options assessment that has been undertaken for Subcatchment 5. Based on RLC's direction that a coordinated mitigation approach between the landowners will not be feasible (Option 5B), Option 5A is the preferred option that should be progressed to engineering feasibility/preliminary design. However, we note that if it was determined that Option 5B was feasible with regards to land ownership issues, it would likely be the preferred option given the potential to address existing flooding issues downstream.

Conveyance based options are not considered suitable for Subcatchment 5 given the existing downstream flooding issues and the difficultly in upgrading a very long length of downstream network often without an easement for open channels.

There is some potential that if the size of Dam 5-1 can be increased, that it could help mitigate some of the existing flood issues or the effects of climate change. This should be investigated further in engineering feasibility/preliminary design. Another key feasibility issue to be investigated with this concept is slope instability risk to neighbouring properties.

We note that these flood detention dams only provide a flood mitigation function. Onsite stormwater management devices would likely still be required to provide functions such as extended detention (for mitigation of stream stability risk) and water quality treatment.

Opportunities for flood mitigation in this catchment are limited due to the existing downstream flooding and topographical constraints. Should the preferred concept not prove feasible in subsequent stages then there is a risk that development cannot take place to the density indicated by the land use zoning and/or greater use of at-source mitigation may be required, which will put the onus on developers to come up with solutions.

We also recommend that further work be undertaken to confirm the hydrological parameters that should be used for the eastern catchments to improve the confidence in peak flow estimates. We note that these catchments have been assessed by multiple consultants using a variety of methods and there is a wide range in peak flow estimates. This is due to the complexity of infiltration/interflow dynamics in pumiceous soils which are not well captured in traditional rainfall/runoff methods, lack on site specific rainfall data, as well as potentially erroneous data capture from the Waingaehe Stream gauge.

**Table 5.8: Summary of qualitative options assessment for Subcatchment 5**

| | **Decentralised detention (Option 5A)** | **Catchment diversion and detention (Option 5B)** |
|---|---|---|
| Rough order cost estimate (including professional fees) | $3.6 to 5.3 million | Not assessed but likely lower than Option 5A |
| Ability to mitigate flooding | • Provides mitigation for growth<br>• Potentially can address some existing downstream flooding or effects of climate change if size of Dam 5-1 can be increased | • Provides mitigation for growth<br>• Potentially can address existing downstream flooding or effects of climate change (likely to a greater degree than Option 5A) |
| Ability to address other stormwater objectives (e.g. water quality, erosion etc.) | • Extended detention and water quality treatment will likely have to be achieved in separate stormwater management devices | • Extended detention and water quality treatment will likely have to be achieved in separate stormwater management devices |
| Speed of implementation | • Land already owned by RLC for Dam 5-2 and land owned by developer for Dam 5-1 but design and consenting process required for two dams prior to construction | • Requires more coordination with developers but only requires one dam |
| Constructability and maintenance | • Constructability of Dam 5-2 will be difficult because of limited access/laydown areas, proximity to dwellings and because of ecological value in the gully. Dam 5-1 also in close proximity to dwellings.<br>• Ongoing dam safety management and maintenance for two dams | • Ongoing dam safety management and maintenance for one dam |
| Potential risks and opportunities (engineering, consenting etc.) | • Slope instability risk to nearby properties<br>• Suitability of the underlying soils for construction of an earth embankment | • Slope instability risk for nearby properties<br>• Suitability of the underlying soils for construction of an earth embankment |

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| | Decentralised detention (Option 5A) | Catchment diversion and detention (Option 5B) |
|---|---|---|
| | • Safety in design should be considered to mitigate health and safety risk of deep water near dwellings<br>• Dam break risk to downstream properties | • Safety in design should be considered to mitigate health and safety risk of deep water near dwellings<br>• Dam break risk to downstream properties |
| Social/cultural/environmental impacts | • Some loss of ecological value in gully for Dam 5-2 (however gully not permanently inundated)<br>• Function of flood detention dam near residential dwellings would have to be carefully communicated to community | • Function of flood detention dam near residential dwellings would have to be carefully communicated to community |
| Land purchase required? | • Potentially would require a designation to be established on private land for Dam 5-1.<br>• Landowner consultation and approvals required for Dam 5-2. Potentially land purchase required | • Potentially would require a designation to be established on private land for Dam 5-1. |
| Other considerations | • Capacity of pipe network and overland flowpaths between the development area and dams needs to be assessed. Upgrades may be required. | • RLC have advised that a coordinated detention solution with the two developments in the upper catchment is likely not feasible so this option has not been considered further |

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## 5.5 Utuhina Catchment (Subcatchments 13, 14 and 15)

The growth areas within the Utuhina Catchment for which mitigation has been assessed include Pukehangi Heights, Freedom Village development, Titoki Place and Diamond Street (refer Figure 5.22).

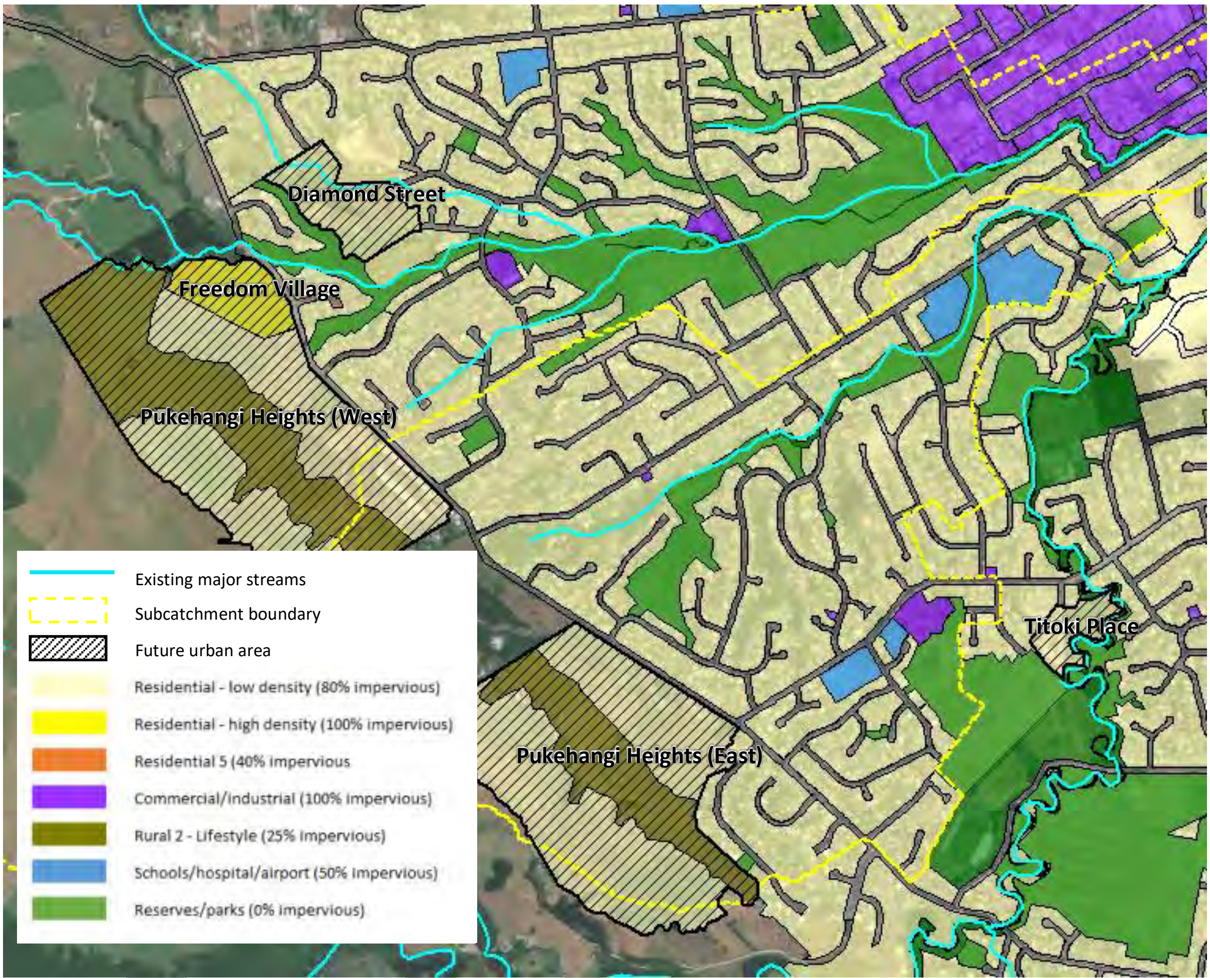


*Figure 5.22: Development areas to be mitigated within the Utuhina Catchment*

Mitigation options for development within the Utuhina Catchment is constrained by existing downstream capacity and the fact development is located in the upper part of the catchment, limiting conveyance based options. Mitigation for Diamond Street and Titoki Place will be onsite detention using stormwater detention basins/ponds.

For mitigation of Pukehangi Heights and Freedom Village, several sites have been identified to provide detention storage. These include:

- Linton Park (upgrade of the existing dam and new dam at Edmund Road).
- Wright Park.
- 25 Paradise Valley Road (referred to herein as the Mangakakahi Dam).

The location of these dam sites, and their respective catchment areas, are shown in Figure 5.23.

A purely conveyance based option is not considered suitable for the Utuhina Catchment given the existing downstream issues and the difficultly in upgrading a very long length of downstream network.

The plan change process for Pukehangi has been ongoing in parallel with the masterplan project supported by the onsite detention concept being designed by WSP/Opus. The draft Hearing Commissioners Report for Plan Change 2 for Pukehangi Heights was released towards the end of this study and includes a provision which states that stormwater effects for Pukehangi Heights development must be “managed solely within the…development area without the need to reply on upstream or downstream mitigation options”[15]. Therefore, the detention dam options presented in this section are intended to provide an alternative to the onsite detention concept, and/or provide additional capacity to allow for other considerations like downstream flooding, intensification, and climate change.

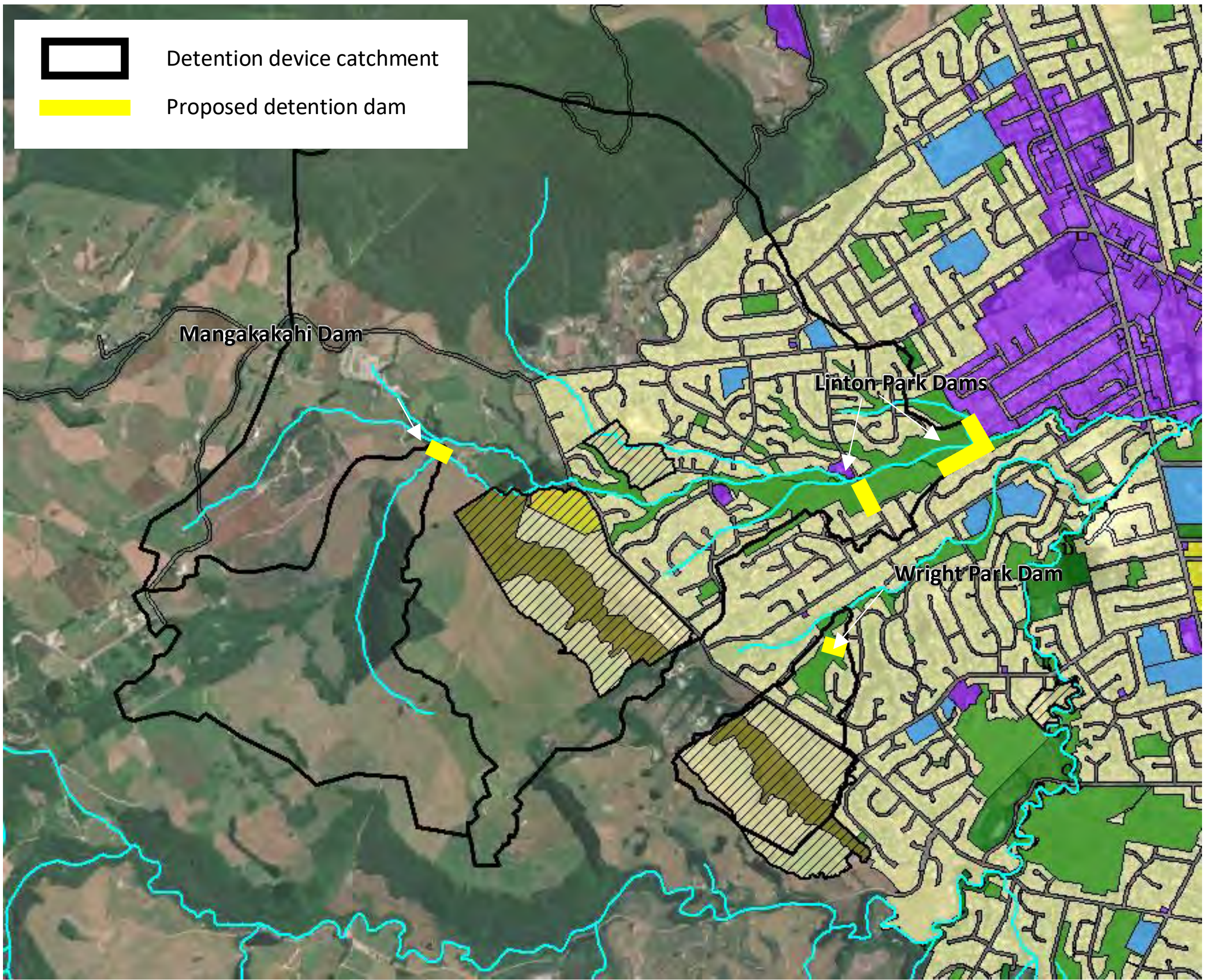


*Figure 5.23: Proposed location of Utuhina detention dams*

### 5.5.1 Option 6

Three options consisting of different combinations of the four proposed dams have been assessed as part of the options assessment:

- Wright Park Dam and Linton Park Dams.
- Wright Park Dam and Mangakakahi Dam.
- Wright Park Dam, Mangakakahi Dam and Linton Park Dams.

[15] Objective 2a from the draft plan change provisions (Appendix 2). Report accessed from https://letstalk.rotorualakescouncil.nz/District-Plan-Plan-Change-2-Pukehangi-Heights on 19 January 2020

The modelling showed that the option with all three of these detention structures is potentially required to mitigate for growth when no onsite detention is provided within Pukehangi Heights. Therefore, all three dam sites are discussed below. No assessment of the required dam sizes to only mitigate for intensification and existing flooding issues (assuming mitigation for growth is achieved onsite) was undertaken as part of this study.

Given the presence of large tributary catchments and a large amount of floodplain storage in the Utuhina Catchment a peak flow mitigation approach is not suitable and the TUFLOW model has been used to assess potential increases in downstream flood levels. This is because peak flow mitigation on its own does not address potential effects such as coincidence of peak flows and volume effects (i.e. where downstream flood levels can increase as a result of increase runoff volume even when peak flowrates are unchanged), which are of particular concern around the Depot Street area.

We have assessed the detention designs for multiple durations of design rainfall events to ensure attenuation objectives are met for a range of storm profiles. These dams are sized to result in no increase in downstream flood levels for the 1 hr and 6 hr events. Longer duration/lower intensity storms (e.g. the 12 hr and 24 hr events) are not predicted to result in downstream flooding so attenuation for these events has not been targeted.

It is noted that this concept assumes the flow diversions proposed in WSP/Opus' design for Pukehangi Heights are in place. This includes diversion of all flow from Pukehangi Heights West to the Mangakakahi Stream (rather than discharging to existing stormwater pipe networks on the northern side of Pukehangi Road) and diversion of all flow from Pukehangi Heights East to Pukehangi Road (rather than some discharging to the existing stormwater pipe network near Matipo Ave).

The potential upper-bound storage of these dams is summarised in Table 5.9. This assumes that the storages are maximised through additional excavation and maximising the dam heights within practicable limits. However, we note that, depending on what flood management objectives are required (mitigation of existing flooding, climate change, intensification etc.), the size of these dams may be able to be decreased in subsequent design stages or all the dams may not be required. Significant opportunity exists for optimisation of the design of these dams due to the interdependencies in their design.

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**Table 5.9: Summary of detention sizing for Utuhina Catchment**

| Structure reference | Structure type | Crest elevation | Pond/dam height | Maximum likely 100 yr live storage volume |
|---|---|---|---|---|
| 14-1 (Wright Park) | Dam | 297 m RL | 7 m | 80,000 $m^3$ |
| 15-1 (Linton Park) | Dam | 288 m RL | 2.5 m | 122,000 $m^3$ |
| 15-2 (Edmund Rd) | Dam | 291 m RL | 3 m | 58,000 $m^3$ |
| 15-3 (Mangakakahi) | Dam | 345 m RL | 17 m | 130,000 $m^3$ |

**Wright Park Dam**

The proposed Wright Park Dam is located within the stream gully between 51 Icarus Place and 12 Helena Place (refer Figure 5.24). This gully consists of three gullies that converge at the site of the proposed dam. From a high level assessment of topographical constraints it appears the dam could be constructed to 297 m RL which is approximately the maximum height water can be impounded before flooding of nearby dwellings will occur (as assessed with available LIDAR data). At approximately 293 m RL, impounded water would start to encroach into some properties along the eastern most gully in the storage area, however it is noted that this would only occur during an extreme rainfall event. In order to maximise the storage behind the dam and provide connectivity between all three gullies some of the ridgeline between the western and central gully could be excavated if required. The proposed excavation would be approximately 17,000 m$^3$. With this crest level and excavation the total storage volume behind the Wright Park Dam would be approximately 80,000 m$^3$.

Another key consideration for this option is the conveyance requirements to get unattenuated flow to Wright Park. Because the dam is located slightly downstream from the development, the ability of existing reticulated pipe networks and secondary flowpaths across Pukehangi Road and Pegasus Drive to pass increased flows to the dam will need further consideration. Currently the preference would be to pass all secondary flow down Pegasus Drive as the other overland flowpath near 362 Pukehangi Road and 49A Pegasus Drive is currently on private property.

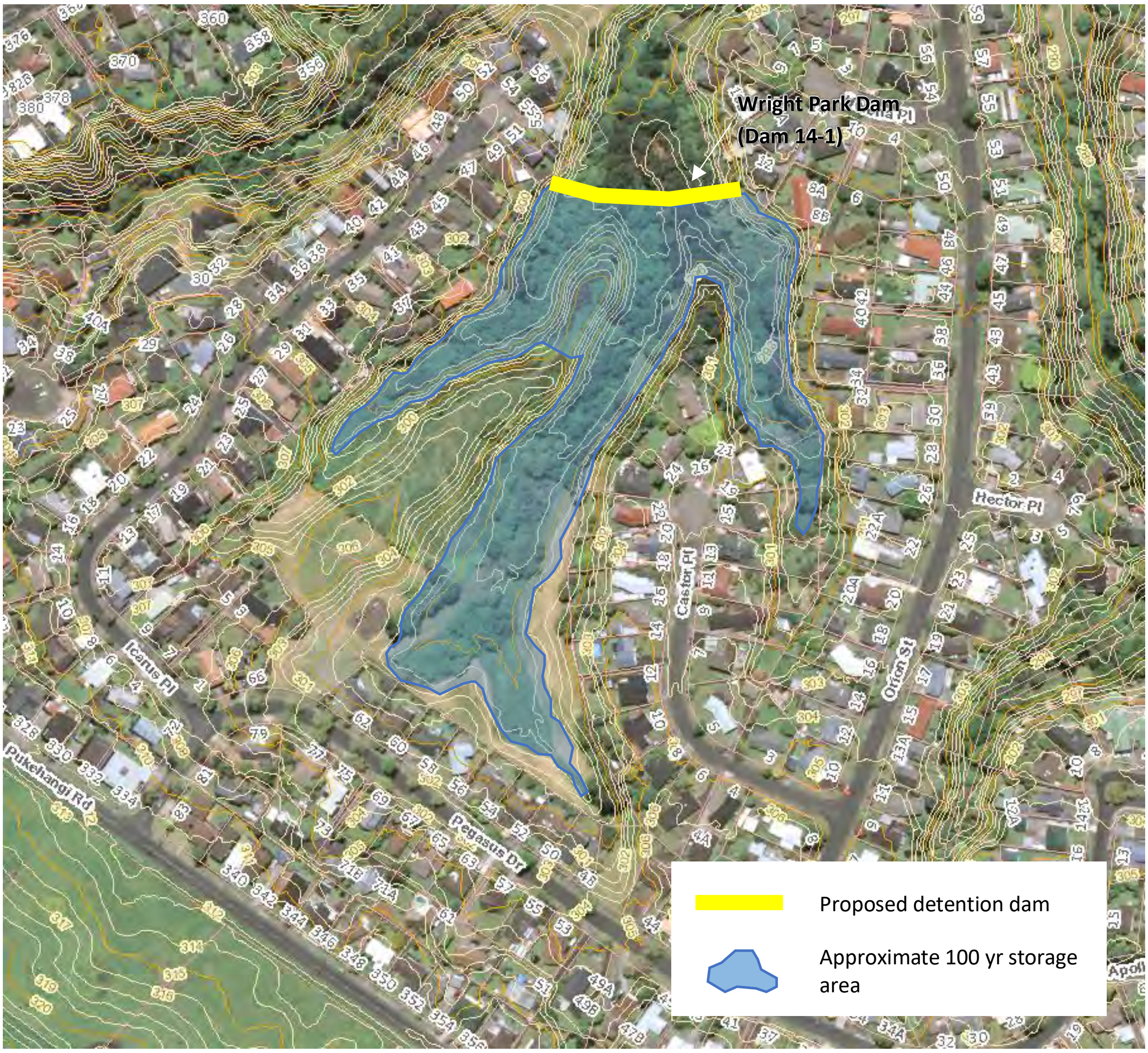


*Figure 5.24: Proposed location of Wright Park Dam (base image sourced from RLC's Geyserview)*

**Linton Park Dams**

For Linton Park it is proposed to upgrade the existing Linton Park Dam and provide a new dam at Edmund Road, immediately upstream of the existing Linton Park Dam (refer Figure 5.25). The existing embankment could be raised to 288 m RL and approximately 50,000 m$^3$ could be excavated behind the dam (if required) in the Linton Park reserve area which is currently scrubland. The outlet culvert under the embankment would also be replaced, and a new more robust spillway incorporated.

This would create a total volume of 122,000 $m^3$ – an increase of 90,000 $m^3$ compared to the existing dam. The proposed excavation would involve excavation behind the dam to 285.8 m RL. This would require an average excavation depth of approximately 1 m over 4.6 ha.

Currently the bund height of the Linton Park Dam appears to vary between approximately 287.4 m RL to 287.8 m RL, based on RLC LiDAR data. 288 m RL has been selected as the proposed bund height as this is the maximum level to which inundation could occur before entering the surrounding private properties.

For the new dam at Edmund Road a new bund could be built to 291 m RL on the upstream side of the road along with altering the inlet configuration of the box culvert under Edmund Road. This would create a total storage of 58,000 $m^3$ – an increase of 54,000 $m^3$ compared to the existing road embankment (currently at 289.5 m RL). This would utilise the existing Linton Park West reserve for flood storage. The embankment would need to extend around the southern boundary of the reserve area for some distance to protect properties such as 18 and 20 Edmund Road from becoming inundated.

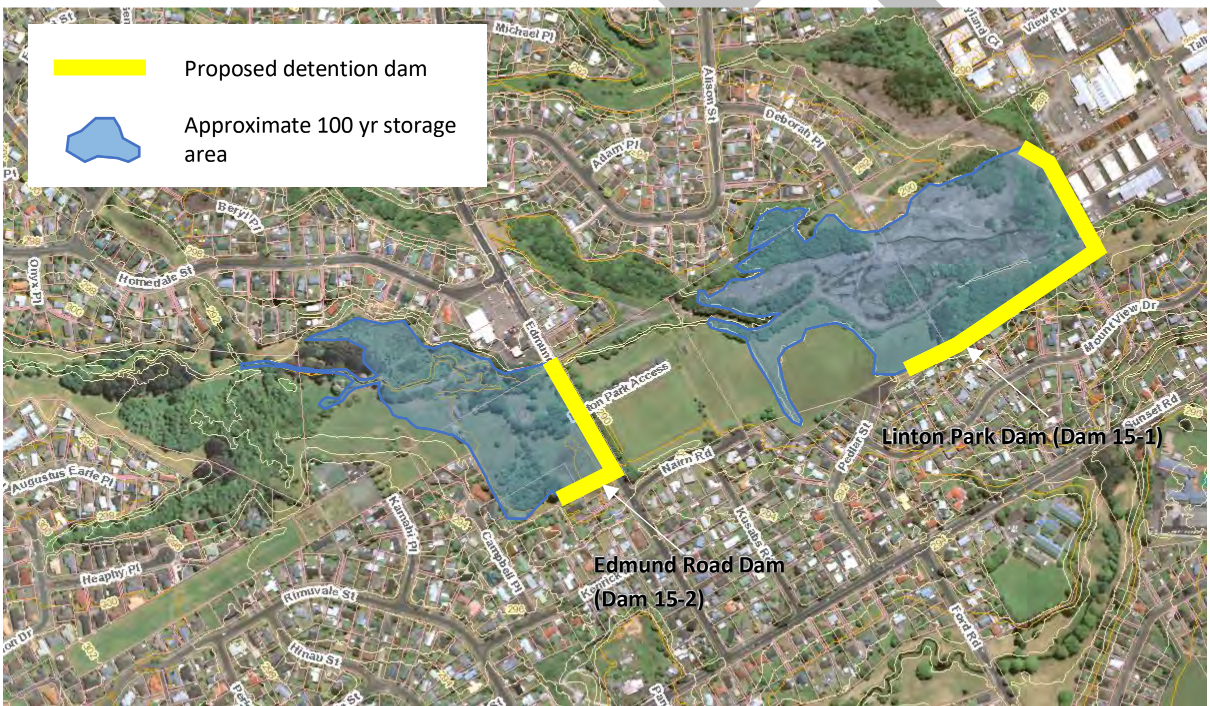


*Figure 5.25: Proposed location of Linton Park Dams (base image sourced from RLC's Geyserview)*

### Mangakakahi Dam

The proposed Mangakakahi Dam is located on the true right branch of the Mangakakahi Stream, approximately 900 m upstream of Pukehangi Road (refer Figure 5.26). It is located on farmland that we understand is owned by the developer of the Pukehangi Heights development. The dam could be constructed to 345 m RL which is approximately the height of the small knoll that separates the two parts of the dam. This would create approximately 130,000 $m^3$ of storage.

Several potential storage sites within the upper Mangakakahi Catchment have been investigated however, there are limited sites where storage can be achieved. This site has been selected due to its storage efficiency (large storage volume relative to the required volume of the embankment) and upstream catchment size. The dam would result in the loss of approximately 100 m of existing naturalised stream and the storage area would inundate approximately 450 m of stream (although only during extreme rainfall events).

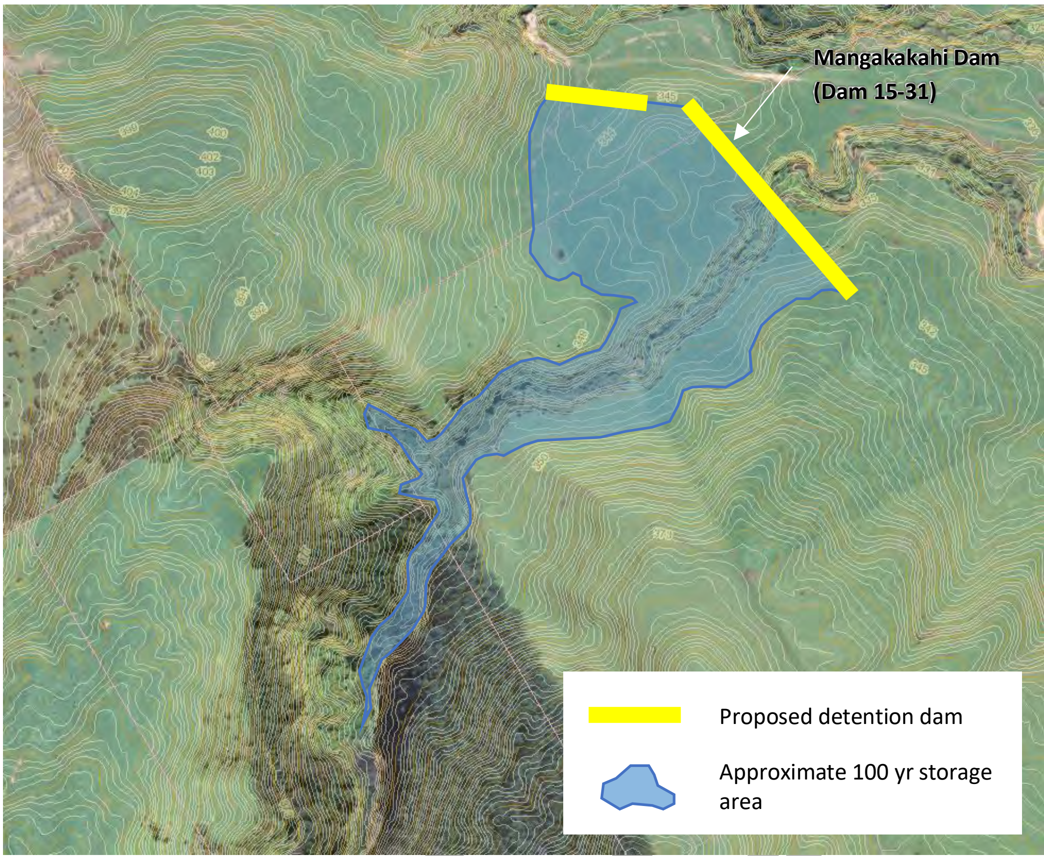


*Figure 5.26: Proposed location of Mangakakahi Dam (base image sourced from RLC's Geyserview)*

### 5.5.2 Recommendation

Table 5.10 below summarises the qualitative options assessment that has been undertaken for the detention based mitigation solution for the Utuhina Catchment. A purely conveyance based option is not considered suitable for the Utuhina Catchment given the existing downstream issues and the difficultly in upgrading a very long length of downstream network.

The TUFLOW modelling used to assess the proposed detention storage dams showed that potentially all three of the dams would be required to mitigate for the Pukehangi Heights Development. However, the draft provisions of the plan change decision for Pukehangi states that stormwater effects for Pukehangi Heights development must be "managed solely within the…development area without the need to reply on upstream or downstream mitigation options"[16]. Therefore, the detention dam options presented in this section are intended to provide an alternative to the onsite detention concept, and/or provide additional capacity to allow for other considerations like downstream flooding, intensification, and climate change.

On this basis we recommend adopting a detention based approach for the Utuhina Catchment that is a combination of onsite detention (as proposed by WSP/Opus for Pukehangi Heights) and offsite detention (a combination of the three dam sites discussed above). This will require further discussion with RLC on what their wider flood management objectives for the Utuhina Catchment are beyond just catering for growth, and further investigation into whether mitigation for existing

[16] Objective 2a from the draft plan change provisions (Appendix 2). Report accessed from https://letstalk.rotorualakescouncil.nz/District-Plan-Plan-Change-2-Pukehangi-Heights on 19 January 2020

flooding issues, climate change and/or intensification can be achieved with a combination of onsite and offsite detention. We note that, depending on what flood management objectives are required the size of these dams may be able to be decreased in subsequent design stages or all of the dams may not be required. Significant opportunity exists for optimisation of the design of these dams due to the interdependencies in their design.

We recommend that collaboration with BOPRC occurs at an early stage to get acceptance in principle for the proposed approach and to confirm which model should be used to assess the preliminary design (the Greater Utuhina Catchment Model developed by BOPRC or the TUFLOW model developed by T+T).

We recommend that the technical feasibility and consentability of the four potential detention dams is assessed.

**Table 5.10: Summary of qualitative options assessment for Utuhina Catchment**

| | |
|---|---|
| Rough order cost estimate (including professional fees) | • Wright Park Dam - $5.7 to $8.3 million<br>• Linton Park and Edmund Road Dams - $12.4 to 17.9 million<br>• Mangakakahi Dam - $9.7 to $14 million<br>• These cost estimates assume that the dams are constructed to the maximum heights with all excavation required. We note that, depending on what flood management objectives are required the size of these dams may be able to be decreased in subsequent design stages or all of the dams may not be required. |
| Ability to mitigate flooding | • Can provide mitigation for growth<br>• Potentially can address existing downstream flooding, climate change and intensification if used in conjunction with onsite detention within Pukehangi Heights |
| Ability to address other stormwater objectives (e.g. water quality, erosion etc.) | • Extended detention and water quality treatment will likely have to be achieved in separate stormwater management devices |
| Speed of implementation | • Design and consenting process required for each dam prior to construction<br>• Coordination with BOPRC required which may be a lengthy process |
| Constructability and maintenance | • Constructability of dams in the urban area be difficult because of limited access/laydown areas, interaction with public parks, proximity to dwellings and because of ecological value in the gullies. Ongoing maintenance and dam safety management for up to 4 dams |
| Potential risks (engineering, consenting etc.) | • Consentability constraints introduced in the 2020 Freshwater legislation<br>• Slope instability risk to nearby properties<br>• Suitability of the underlying soils for construction of an earth embankment<br>• Safety in design should be considered to mitigate health and safety risk of deep water near dwellings and in public reserve areas<br>• Dam break risk to downstream properties |
| Social/cultural/environmental impacts | • Some loss of ecological value in gullies for the Wright Park Dam and Mangakakahi Dam (however gullies are not permanently inundated)<br>• Function of flood detention dam near residential dwellings would have to be carefully communicated to community |
| Land purchase required? | • Landowner consultation and approvals required for Mangakakahi Dam |

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| | |
|---|---|
| | • Landowner consultation and approvals required for Wright Park Dam if inundation extends onto private property. Potentially land purchase required |
| Other considerations | • Capacity of pipe network and overland flowpaths between the development area and dams needs to be assessed. Upgrades may be required. |

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# 6 Summary and next steps

The preferred flood mitigation approaches and recommendations for next steps are summarised below:

**Subcatchments 1 and 2:**

- Given the current uncertainty in the hydrology and hydraulics associated with these catchments, it is prudent at this stage to assume that detention storage within the future urban growth areas in these catchments will be required. This assumption should be reviewed following collection of further data as recommended below.
- We recommend that further work be undertaken to determine the conveyance capacity of the Waingaehe Stream and bridges from Te Ngae Road to the lake. This work will need to include a survey of the Te Ngae Road bridge, and the Waingaehe Stream downstream of the bridge and detailed hydraulic modelling.
- We recommend that further work be undertaken to confirm the hydrological parameters for the eastern catchments. Collection of site specific rainfall and stream flow data in the eastern catchments will allow a significant reduction in the uncertainty that is inherent in the flood analyses to date.
- Once the existing capacity of the Waingaehe Stream is known and there is more certainty around catchment hydrology, a decision can be made on the preferred approach for flood management. This includes confirming an appropriate balance between provision of detention storage in the growth cells, and utilisation of conveyance capacity, and whether it is possible to divert flow from Subcatchment 1. The optimal result may well be a combination of all three options.
- If upgrade of conveyance capacity downstream of Te Ngae Rd is part of the preferred approach, collaboration with BOPRC occuring at an early stage in the project will be required to get acceptance in principle for the proposed approach due to the Waingaehe Stream being part of a rated flood scheme.

**Subcatchment 3:**

- For Subcatchment 3 we recommend adopting a pass flows forward approach (i.e. no detention of post-developed flows) with conveyance of primary flows in a new trunk main under Galvin Road and utilising the road network (including the existing open channel if retained) as an overland flowpath.
- We recommend that further work be undertaken to confirm the hydrological parameters that should be used for the eastern catchments as mentioned for Subcatchments 1 & 2.
- Further investigations should be undertaken by RLC around land purchase or easement creation to secure an outlet to the lake at the end of Galvin Road. If the option of using the existing paper road is preferred the effect of this on the Waingaehe Stream should be assessed as part of the future work recommended for Subcatchments 1 and 2.
- The mitigation approach for the top of Subcatchment 3 should be further investigated as part of the plan change structure plan for the Ngati Whakaue Lands.

**Subcatchment 4:**

- For Subcatchment 4 we recommend adopting a pass flows forward approach (no detention of post-developed flows) for development in the lower catchment (Owhatiura South and Vaughan Road Developments) and providing centralised detention for the Wharenui Development upstream of Morey St (Option 4A).

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- This option would involve construction of two flood detention dams upstream of Morey St and upgrades to the existing Owhatiura Eastern Drain and Vaughan Road culvert and a new drain for the developments along Vaughan Road.
- We recommend that further work be undertaken to confirm the hydrological parameters for the eastern catchments as mentioned for Subcatchments 1 & 2.
- As part of subsequent engineering feasibility/preliminary design stage the following issues should be addressed for the preferred option:
  - Consideration of the proposed development layout to ensure that the road/pipe networks will convey 100 year flows to the channels/dams.
  - Geotechnical investigations and engineering feasibility assessments should be undertaken for the proposed dam sites.
  - Optimisation of the dam configuration and whether the storage can be increased to address existing flooding or mitigate the effects of climate change for the Melrose Ave area.
  - Refinement of the concept to ensure that the dams can act independently of one another and to confirm at what stage of development the second dam is required.
  - Slope instability risk to neighbouring properties.
  - A dambreak assessment should be undertaken to determine the required PIC rating of the proposed dams.
  - Confirmation of the flood management requirements that BOPRC will expect to see met in a resource consent application including whether the attenuation to 80% of pre-development flows is required.
  - More detailed hydraulic modelling of the downstream channel should be undertaken to confirm the scope of the required conveyance upgrades.
- We recommend early engagement with Waka Kotahi to discuss conveyance upgrades across Te Ngae Road.

**Subcatchment 5:**

- For Subcatchment 5 we recommend progressing a decentralised detention approach (Option 5A) to engineering feasibility/preliminary design based on RLC's direction that a coordinated mitigation approach between the landowners will not be feasible.
- This option involves construction of two separate flood detention dams - one dam would be located on private property within the existing stream gully near Link Road while the other dam would be located within the council-owned stream reserve near Shane Place.
- We recommend that further work be undertaken to confirm the hydrological parameters that should be used for the eastern catchments as mentioned for Subcatchments 1 & 2.
- As part of subsequent engineering feasibility/preliminary design stage the following issues should be addressed for the preferred option:
  - Consideration of the proposed development layout and the assessment of the ability of the existing road/pipe network to convey flow to the dam sites.
  - Assess consentability of all proposed dams given the recent National Policy Statement for Freshwater Management 2020
  - Geotechnical investigations and engineering feasibility assessments should be undertaken for the proposed dam sites.
  - Whether the size of Dam 5-1 can be increased to address existing flooding or mitigate the effects of climate change in the reach downstream of the dam.

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- Slope instability risk to neighbouring properties.
- A dambreak assessment should be undertaken to determine the required PIC rating of the proposed dams.
- Confirmation of the flood management requirements that BOPRC will expect to see met in a resource consent application including whether attenuation to 80% of pre-development flows is required.

**Utuhina Catchment:**

- For the Utuhina Catchment we recommend adopting a detention based approach that is a combination of onsite detention (as proposed by WSP/Opus for Pukehangi Heights) and offsite detention (the three dam sites discussed in Section 5.5.1).
- Depending on what flood management objectives are, the required the size of these dams may be able to be decreased in subsequent design stages or all of the dams may not be required. Significant opportunity exists for optimisation of the design of these dams due to the interdependencies in their design.
- As part of subsequent engineering feasibility/preliminary design stage the following issues should be addressed for the preferred option:
  - Investigate whether mitigation for existing flooding issues, climate change and/or intensification can be achieved with a combination of onsite and offsite detention. This will require clarification from RLC on what their wider flood management objectives for the Utuhina Catchment are beyond just catering for growth.
  - Assess consentability of all proposed dams given the recent National Policy Statement for Freshwater Management 2020
  - Geotechnical investigations and engineering feasibility assessments should be undertaken for the proposed dam sites.
  - Slope instability risk to neighbouring properties.
  - A dambreak assessment should be undertaken to determine the required PIC rating of the proposed dams.
  - Confirmation of the flood management requirements that BOPRC will expect to see met in a resource consent application including whether the requirement for attenuation to 80% of pre-development flows is required.
- We recommend that collaboration with BOPRC occurs at an early stage to get acceptance in principle for the proposed approach and to confirm which model should be used to assess the preliminary design (the Greater Utuhina Catchment Model developed by BOPRC or the TUFLOW model developed by T+T).

**This report has been prepared for the exclusive use of our client RLC, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement. The mitigation options presented within this report are concept level only and their feasibility requires further investigation and design. Some mitigation options are located within private property, but we note no consultation with landowners have been undertaken as part of this study. The hydrological parameters/methods and the sizing or location of the proposed mitigations should not be relied on for any other context or purpose and this report should not be used as an engineering reference for stormwater/flood management for any proposed development without further consultation with RLC.**

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# 7 Limitations

The following limitations of this study should be noted:

- The mitigation solutions presented within this report have only been developed to a concept level. Further site investigations and design is required to assess site specific constraints and confirm feasibility and sizing of the proposed solutions.
- The indicative sizing and location of the proposed mitigation solutions are based on current topography within development areas (i.e. earthworks not accounted for). We note that earthworks associated with development and proposed road networks have the potential to change discharge locations and the balance of flows within subcatchments and this will have an effect on the design of flood management devices.
- The cost estimates presented within this report are rough order estimates only. These cost estimates have been produced from estimates of quantities from conceptual layouts of the mitigation solutions. In order to produce cost estimates for such a large number of concepts a number of simplifying assumptions have been applied. A contingency of 40% has been applied to the physical works costs due to the high-level nature of the estimate.
- We note that there is significant uncertainty in the hydrology for the Eastern Catchments due the inability to verify model results with calibration. This uncertainty has been mitigated for the purposes of this work by using multiple methods and adopting a conservative approach to peak flow estimates. However, it is recommended that calibration of rainfall/runoff parameters for the eastern catchments be investigated further for subsequent design stages.
- While the hydrological methods used are considered appropriate for the purposes of this study (i.e. identification of the preferred flood management approach), the hydrological method and analysis tools used to design the preferred option should be reviewed in subsequent design stages as they may not be suitable for more detailed design.

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# 8 Applicability

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

The construction rates utilised for this high level cost estimate are based on assumed design concepts, estimated quantities and a combination of recently submitted tender rates for similar projects within the regional area along with the latest available rates from QV Cost Builder database (formerly Rawlinsons). Consequently a significant margin of uncertainty exists on the cost estimate and the contingency we have allowed should be considered as part of the cost rather than a potential add on.

No allowance has been included for cost escalation beyond 2019.

**COVID-19 impacts:** The derived rates are based on information and data obtained prior to COVID-19 being declared a pandemic by the World Health Organisation. New Zealand subsequently entering COVID-19 Alert Level 4 "lockdown" plus the global economic impacts of COVID-19 will have an impact on the construction industry in at least the immediate and medium term future. The significance and extent of COVID-19 impacts is uncertain at this time but likely to impact both labour and materials rates.

We have not made any attempt to allow for the impact of COVID-19 in this estimate and recommend you seek specialist economic advice on what budgetary allowances you should make for escalation and changed construction costs post COVID-19.

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Tonkin & Taylor Ltd

Report prepared by: Authorised for Tonkin & Taylor Ltd by:

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# Appendix A: Model build report

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REPORT

Tonkin+Taylor

# Rotorua Stormwater Masterplan

## TUFLOW Model Build Report

**Prepared for**
Rotorua Lakes Council

**Prepared by**
Tonkin & Taylor Ltd

**Date**
April 2021

**Job Number**
1010988.1000.v2

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## Document Control

| **Title: Rotorua Stormwater Masterplan – TUFLOW Model Build report** | | | | | |
|---|---|---|---|---|---|
| **Date** | **Version** | **Description** | **Prepared by:** | **Reviewed by:** | **Authorised by:** |
| 27/01/2021 | 1 | Draft for client review | J Hodson | M Pennington | D Bouma |
| 08/04/2021 | 2 | Final draft | J Hodson | M Pennington | D Bouma |
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**Distribution:**

Rotorua Lakes Council 1 electronic copy

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## Table of contents



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# 1 Introduction

Tonkin & Taylor Ltd (T+T) has been engaged by the Rotorua Lakes Council (RLC) to prepare a stormwater masterplan for a selection of catchments where urban growth is expected. The Masterplan is founded on a series of simulation results from a direct rainfall driven 2-Dimensional (2D) stormwater model. The stormwater masterplan is intended to identify, design and construct integrated flood management solutions that will facilitate planned future urban growth in parts of the city with the greatest potential and demand for growth.

Significant effort has gone into development of the right kind of model to support the masterplanning process. A major challenge with this modelling work is the sheer scale of the assessment required. It was seen that a city-wide scale model was required, to ensure that cross-catchment effects could be adequately captured. It was also desirable to keep model run-times within reasonable limits, to ensure that iterative simulations required as part of the masterplan development would not be unduly delayed. Another key challenge with what is largely an urban catchment modelling exercise, is the interaction of the urban stormwater systems with several major riverine waterways, the most significant of which is the Utuhina Stream.

A key element of the modelling work was to quantify likely changes in rainfall-runoff behaviour that result from proposed land development, and to use the model to seek, test and prove measures to mitigate adverse flooding effects that may result. This requires a comparative assessment where the flood response from one land development state is compared to that of another, with the intention of adequately dealing with adverse flooding effects.

We undertook a model planning exercise, considering the required outputs, data requirements to support these outputs, potential issues likely to be encountered and then model type selection. This process can be summarised in the model-development generic tick-box list shown in Figure 1.1.

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| Required model outputs | Data requirements | Potential issues | Model types |
|---|---|---|---|
| | | | ☐ |
| ☑ Flood extents - extreme events | ☑ DEM coverage | ☑ DEM availability | ☐ 1D model |
| ☐ Flood extents - regular events | ☐ Detailed levels (structures) | ☑ DEM accuracy | ☐ 2D only |
| ☐ Flood levels - extreme events | ☐ Bathymetry survey | ☑ Grid size | ☑ 2D+ |
| ☐ Flood levels - regular events | ☐ Pipes and pits (GIS data) | ☐ On-ground survey | ☐ 1D-2D coupled |
| ☐ Flood risk assessment | ☐ Floor level survey | ☐ Mobile bed changes | ☐ Finite volume |
| ☑ Relative (effects) assessment | ☑ Land use spatial data | ☐ Accuracy of land use data | ☐ Rainfall-runoff only |
| ☑ Optioneering | ☑ Roughness applied to land use | ☐ Currency of DEM | ☑ Direct rainfall |
| ☐ District plan maps | ☑ Soils data | ☐ Currency of land use data | ☑ Lumped catchments |
| ☐ District plan rules | ☑ Hydrological losses | ☑ Rainfall data - nested, HIRDS, etc | ☐ Nested rainfall |
| ☐ Detailed design | ☑ Calibration data | ☐ Multiple event simulation | ☐ Fuzzy maps |
| ☐ Scour/sedimentation assessment | ☐ PSD data | ☐ Grid size convergence | ☐ 1D morphological model |
| ☐ Resource consent application or compliance | ☐ Depth-damage curves | ☐ Momentum vs energy | ☐ 2D morphological model |
| ☐ Structure plan development | ☐ Joint probability methodology | ☐ False pumps | ☐ Wave model |
| ☑ Overland flow path determination | ☑ Future climate data | ☐ PSD data spatial coverage | ☐ 2D advection/dispersion |
| ☐ Flood damage assessment | ☐ Sea level frequency data | ☐ Groundwater influence | |
| ☑ Assessment influenced by large river | ☑ Rainfall data | ☐ Future development conceptualisation | |
| ☐ Assessment influenced by coastal effects | ☐ Hydraulic structures | ☐ Event timing | |
| ☐ Assessment influenced by groundwater | ☑ Existing consents | ☐ Storm surge | |
| ☑ Present-day or future time horizons | ☐ Land ownership | ☑ Future climate horizon and RCP | |
| ☐ Level of detail - property specific | ☑ Future development data | ☐ Bare earth DEM or include above-ground | |
| ☐ Level of detail - neighbourhood specific | ☐ Bathymetric change with time | ☐ Bridge modelling approach | |
| ☑ Level of detail - regional focus | ☐ Groundwater level data | ☐ Debris allowance for blockage | |
| ☐ Bridge assessment | ☐ Baseflow data | ☑ Availability of calibration data - spatial and temporal | |
| ☐ Weirs | | ☐ Form, friction and turbulence losses | |
| ☐ Sensitivity assessment | | ☐ Sensitivity assessment | |
| ☐ Calibration | | ☐ Flap gate performance | |
| ☐ Legal defence | | ☐ Hearing evidence | |
| ☐ Pipe network capacity | | | |
| ☑ Land use change | | | |
| ☑ Peak flow estimation | | | |

*Figure 1.1: Model selection check-list process applied to model selection for the Masterplan*

As can be seen from the above, the principal model outputs (left column) were first identified to match the anticipated needs for this stage of the masterplanning process. The end result (right column) was identification of the type of model required to meet the needs.

The identified 2D rain-on-grid model has been developed in TUFLOW's HPC environment, and its validity has been supported by testing against flow gauge records and extensive sensitivity analysis on critical hydrological and hydraulic parameters.

In this model development report the model development, model validity testing, sensitivity analysis, and simulation results of design events is presented.

It should be noted that if the required model outputs are changed, then it may emerge that changes to the modelling approach are required which may result in a different model type being selected. In particular, the type of model developed to support the Masterplan may not totally support other purposes, such as detailed engineering design, resource consent application and individual property scale flood level prediction.

## 1.1 Background

The Rotorua Lakes Council has conducted numerous flood studies and flood models for localised developments over the past decade. In recent years, a surge in proposed developments spread over the city has required that RLC develops a set of transparent and efficient work processes to facilitate

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safe and sustainable urban growth. A comprehensive stormwater masterplan focussed on catchments where growth of urban development is expected, and brings together prior flood studies can be applied extensively to inform and provide initial guidance for the consent approval process, prioritised infrastructure upgrades, and effective flood management regimes.

## 1.2 Previous modelling

Several stormwater runoff and river flood models have been developed for different drainage basins within Rotorua over the years for various purposes. These models have been developed for flood management purposes or for specific site stormwater management plans by private consultants on behalf of developers. These reports have been reviewed by T+T and their conclusions have been considered for model development and parametrisation.

One of the most recent and notable flood models is the Utuhina flood model developed by the Bay of Plenty Regional Council (BOPRC). The BOPRC's Utuhina Stream flood model was developed using MIKE 11 to assess flood scheme performance during a range of design storms. The Utuhina flood model is a 1D hydrodynamic model where point source inflows from tributary catchments have been applied to the model along the stream branches across the entire Utuhina Catchment. Each of the inflows to the flood model was estimated with a lumped hydrological model. The design storm adopted for this model is the nested 100 year Average Recurrence Interval (ARI) rainfall event of 72 hour duration under climate change condition of RCP 8.5 for the period of 2080 – 2100.

## 1.3 Report outline and scope

This report includes descriptions of the general modelling approach in Section 2 and the resultant model structure and the key hydrological and geospatial inputs in Section 3. Ideally a model of this extent would be calibrated to several points, spaced within the model domain. However, there is only a single calibration point, this being the Depot Street location on the Utuhina Stream, that has a suitably long and reliable flow record to be used in calibration.

In recognition of this, rigorous model performance testing via simulating real storm events (refer Section 4.1) and sensitivity analysis of key model processes and parameters (refer Section 3.4 has been carried out.

The finalised model has been used to simulate the design storms of 1 hour, 6 hour, 12 hour, and 24 hour durations under the climate change scenario of RCP 8.5 projected for the period of 2080 – 2100. The simulated flood depths, water levels, and peak flows extracted at critical cross-sections of main flood conveyances are presented and discussed in Section 5. Lastly, Section 6 discusses the limitations of the Rotorua TUFLOW model and as a result the applicability of the results of the simulated design storms.

This model build report focuses on the model development process and the various steps that have been undertaken to advance understanding of uncertainties and improve confidence in model performance. The report presents the simulated results of design storms but does not address any optioneering simulations.

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# 2 Model Approach

## 2.1 Model domains

Given the large modelling domain, the Rotorua TUFLOW model has been split into two discrete rain-on-grid models, these being the eastern and western domains. The extent of the two rain-on-grid models is the shaded areas in Figure 2.1, However, the model has only been refined for the catchments that are the focus of the masterplan project (catchments shaded in red).

The split between the two occurs along the Puarenga Stream, as shown in Figure 2.1. The eastern catchment has been modelled entirely with the rain-on-grid model, whereas the western catchment is represented by the rain-on-grid model with a point-source inflow from the undeveloped upper Utuhina watershed.

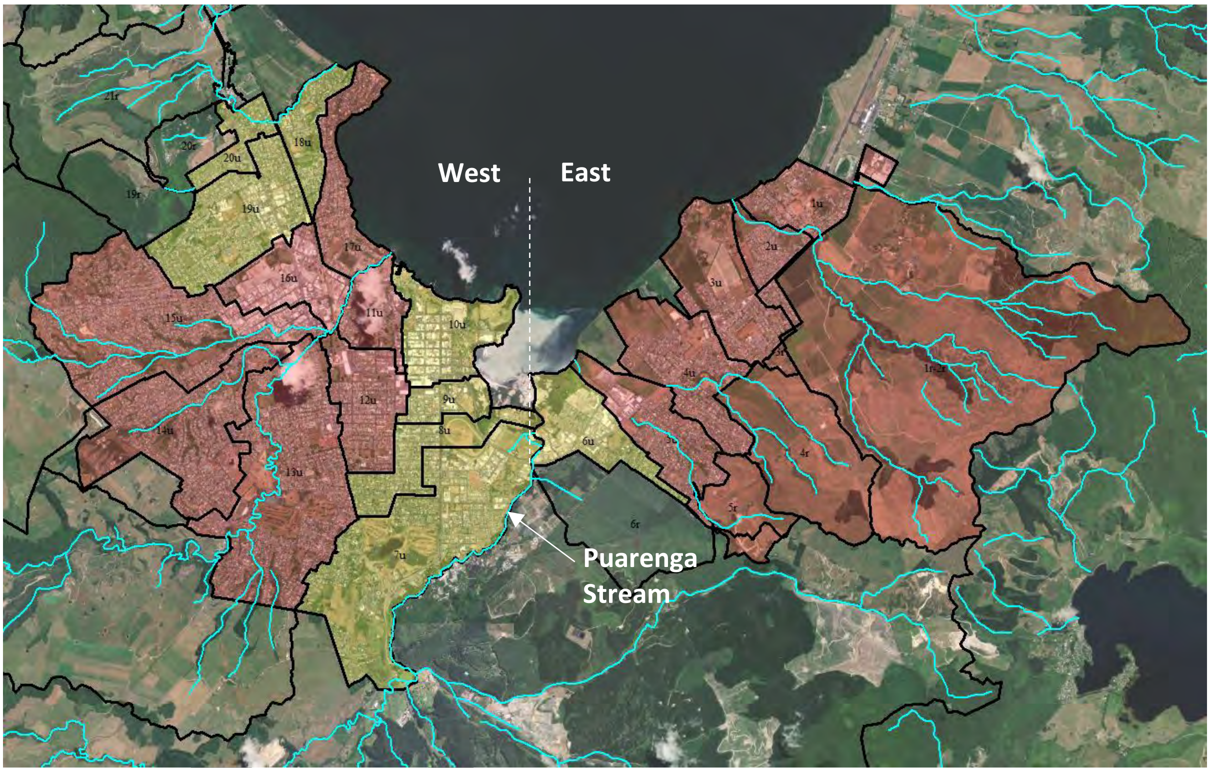


*Figure 2.1: Split between eastern and western model domains*

## 2.2 Rainfall approach

In the Rotorua TUFLOW model, excess rainfall (the total amount of rainfall minus all abstractions within a catchment) is directly applied to a 2D computation grid that solves the 2D shallow water equations and provides spatially distributed estimates of hydraulic parameters such as depth, velocity and discharge. This type of modelling offers the unique advantage of assessing the combined flood hazards from direct runoff and overspilling from main channelised conveyances. In urbanised areas where the hydrological processes are dominated by direct runoff from gently sloped impervious surfaces, rain-on-grid models are effective tools for councils in addressing storm-driven floods.

## 2.3 Stormwater infrastructure

Stormwater infrastructure in the Rotorua TUFLOW model is primarily represented in 2D domain and key hydraulic crossing structures represented by 1D elements. The existing stormwater pipes have been incorporated in the model with 2D+ approach, whereby the hydraulic effect of the significant below ground infrastructure is represented by a network of open channels, preserving both volume and conveyance. In addition to this 2D+ approach, several culverts have been represented in the more traditional approach as 1D elements. This has been done where hydraulic structures are seen to potentially significantly impede flow conveyance or create pressurised flow conditions.

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# 3 TUFLOW Model Development

The Rotorua TUFLOW 2D+ rain-on-grid model requires two main categories of inputs: the spatial data and the hydrological data. This section describes the various model inputs and the model structure.

## 3.1 Spatial input

### 3.1.1 Projection and Datum

The model projection system is NZTM 2000 and the datum is Moturiki 1953. All spatial data and levels presented in this report reference the above projection system and vertical datum unless otherwise specified.

### 3.1.2 Digital Elevation Model

Two sets of LiDAR survey data, flown in 2011 and 2018 by the Bay of Plenty Regional Council (BOPRC), have been made available for the RLC flood model. Both 2011 and 2018 LiDAR data have been processed by RLC's GIS department to generate bare earth Digital Terrain Models (DEMs) with all buildings and above ground features removed. This is a common approach for spatially large assessments, but it does ignore localised effects at the individual property scale (e.g. by excluding above-ground features, overland flow is sometimes simulated to pass through buildings, which may not be realistic at the property scale, but at a macro scale the overall effect is well captured). As referenced in Section 1.1, this approach was deemed appropriate for the intended model outputs.

The 2018 DEM is of 1 m by 1 m resolution and only covers the urbanised sub catchments within RLC whereas the 2011 DEM extends beyond the upper rural watersheds. The 2 m grid 2011 DEM has been adopted to supplement the more recent and refined 2018 DEM.

Prior to adopting the two sets of DEMs, their compatibility was tested, and was confirmed to be suitable. Inspection of the resulting model surfaces was undertaken, and the DEM was locally modified in the catchments of interest (i.e. where growth is planned) for the following reasons:

- Remove blockages in channel due to major bridge and culvert crossings;
- Enforce channel conveyance at locations of poor LiDAR data coverage; and
- Smooth abrupt and artificial grade changes in topography from triangulation. processes.

### 3.1.3 Soil types and infiltration

The Fundamental Soil Layers (FSL) is a set of spatial representation of different soil groups categorised by a range of physical and chemical properties of soils developed by the New Zealand Land Care Research. The spatial distribution and the drainage properties of the soil groups within the modelling domain of the TUFLOW models have been adopted for modelling the infiltration processes.

The FSL categorised the various soil groups into five drainage classes. Within the RLC catchment, most soil groups have fallen under the "well drained" category where infiltration characteristics resemble pumiceous soil. Based on the porous and absorbent soil drainage properties and through iterative model calibration processes, an initial abstraction of 0 mm and a constant infiltration rate of 30 mm/hour for pervious areas was adopted for the model over these soils. The infiltration parameters were applied directly to rainfall intensity data to generate excess rainfall time-series for the Rotorua TUFLOW model. This infiltration rate is prorated based on the percentage

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imperviousness of the different land cover types in the model domain (e.g. an area with an average imperviousness of 50% would have a constant infiltration rate of 15 mm/hr applied to the rainfall).

#### 3.1.4 Land cover and spatially distributed Manning's roughness 'n'

Roughness is a hydraulic calibration parameter, which should be derived by matching model performance to observed performance. In the absence of suitable calibration data, roughness is often estimated using "look-up" tables that document ranges of calibrated roughness that have been found on other river systems. Having applied "look-up" roughness values, and in the absence of calibration data, it is necessary to assess the sensitivity in results to the applied roughness values.

In the Rotorua TUFLOW model, land use is used to set the hydraulic roughness that is applied to the DEM. Land use is also the key factor in setting percentage of impervious cover of different parts of the modelled catchments.

Manning's 'n' roughness coefficients were applied to the model based on land use characteristics defined by the Land Cover Database Version 5 (LCDBv5)[1] sourced from Landcare Research. The LCDBv5 layer provides a relatively coarse representation of land use, which is sufficient for areas of the floodplain and for the modelling approach adopted.

Table 3.1 shows the Manning's 'n' roughness coefficients applied to each land use zone. The values applied to the model are consistent with 2D roughness values recommended in Australian Rainfall and Runoff ARR[2] (2019) and are within close range of those applied to other recent modelling undertaken for RLC.

Roughness values were adjusted within the ranges of the tabled values as part of the model calibration process.

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**Table 3.1: 2D roughness (taken from Table 6.2.2 in Ball et al, 2019)**

| Land Use Type | Manning 'n' |
|---|---|
| Residential areas – high density | 0.2 – 0.5 |
| Residential areas – low density | 0.1 – 0.2 |
| Industrial/commercial | 0.2 – 0.5 |
| Open pervious areas, minimal vegetation (grassed) | 0.03 – 0.05 |
| Open pervious areas, moderate vegetation (shrubs) | 0.05 – 0.07 |
| Open pervious areas, thick vegetation (trees) | 0.07 – 0.12 |

| Land Use Type | Manning 'n' |
|---|---|
| Waterways/channels – minimal vegetation | 0.02 - 0.04 |
| Waterways/channels – vegetated | 0.04 – 0.1 |
| Concrete lined channels | 0.015 – 0.02 |
| Paved roads/car park/driveways | 0.02 – 0.03 |
| Lakes (no emergent vegetation) | 0.015 – 0.35 |
| Wetlands (emergent vegetation) | 0.05 – 0.08 |
| Estuaries/Oceans | 0.02 – 0.04 |

The land cover type affects the roughness and perviousness of the surfaces, and therefore changes the direct runoff volume and overland flood routing. The spatial distributions of various land cover

[1] Source available online at https://lris.scinfo.org.nz/layer/104400-lcdb-v50-land-cover-database-version-50-mainland-new-zealand/

[2] Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors) Australian Rainfall and Runoff: A Guide to Flood Estimation, © Commonwealth of Australia (Geoscience Australia), 2019.

for both pre-development and post-development scenarios has been created with the LCDB version 5.0 (LCDBv5.0) and various land cover layers provided by RLC.

For the pre-development and post-development scenarios the percent imperviousness assumed for each land use zone was based on the maximum permitted development (MPD) allowed in the District Plan. However, for calibration model runs the percent imperviousness was based on estimates of current day imperviousness made by sampling aerial imagery.

In Figure 3.1 and Figure 3.2 the existing and proposed land use assessed as part of this process for the western and eastern model domains (respectively) are shown.

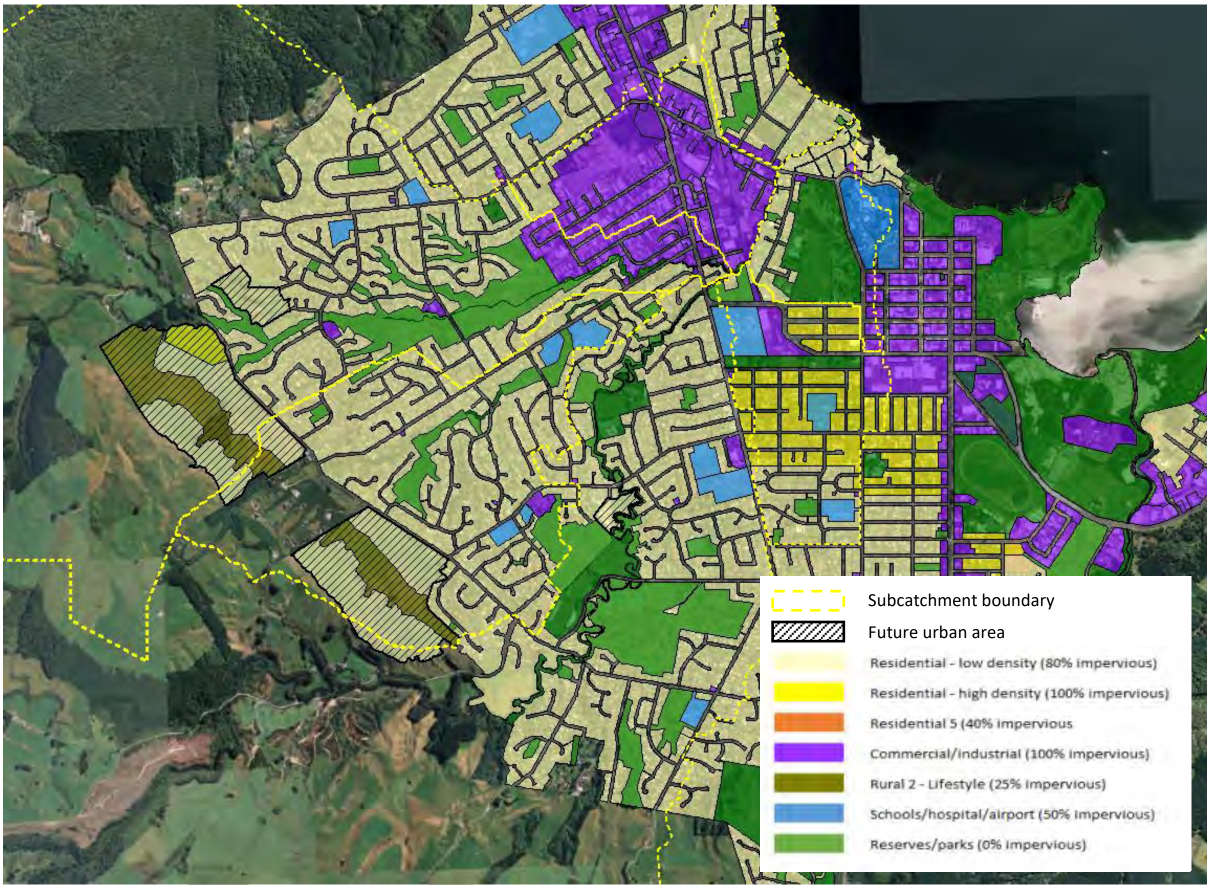


*Figure 3.1: Existing and proposed land use in the western model domain*

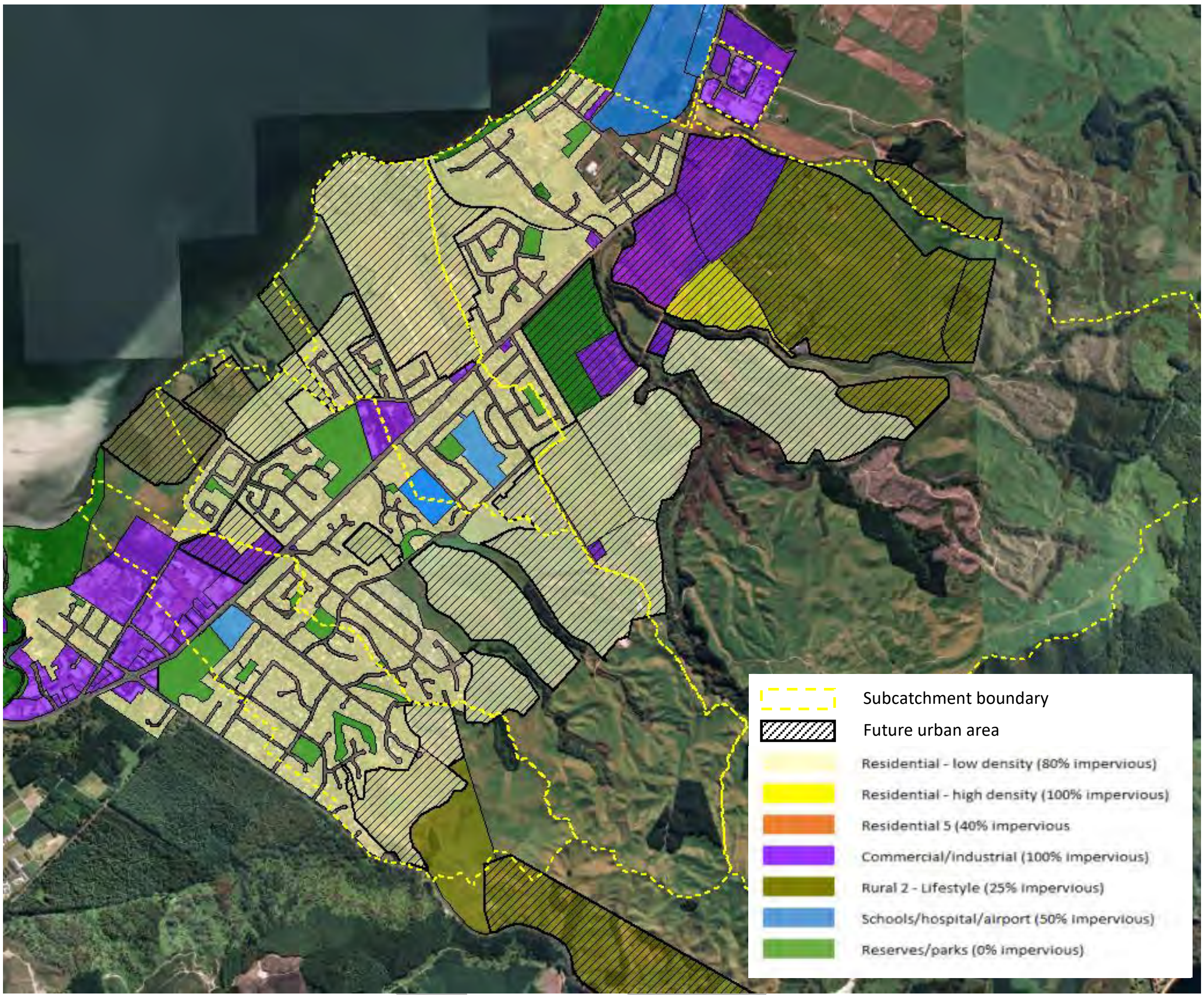


*Figure 3.2: Existing and proposed land use in the eastern model domain*

Important to note is that, within urban areas, separate delineation of buildings from lawns and grassed areas has not been made. This is consistent with the model scale and level of detail. Should individual property scale model results be required, then this will need to be further refined from the current approach.

## 3.2 Hydraulic Infrastructure

### 3.2.1 Stormwater pipe network

The spatial layout and specifications of stormwater pipes has been provided by RLC in shapefile format. The major stormwater network has been represented using the 2D+ approach, as explained in Section 2.3.

### 3.2.2 Streams and open channels

Streams and open channels are represented in the 2D domain of the model with stream geometry and invert levels based on the LiDAR DEM. Riparian areas associated with the main streams were assigned a manning's roughness specific for waterways.

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### 3.2.3 Culverts and bridges

In total, 16 culverts in the western and 9 culverts in eastern catchments have been represented as 1D elements. The culvert structure data such as the inlet and outlet invert elevations, diameters, materials, and barrel layouts have been extracted from stormwater infrastructure inventory provided by RLC. The details of these culvert specifications are assumed to be accurate. The model DEMs have been modified near the culvert inlets and outlets where necessary.

Bridges have been represented as openings of channels in the 2011 and 2018 DEMs. As bridge survey data is generally not available, bridge decks are shown as channel blockages in the DEMs provided by RLC. These blockages have been manually removed by referencing satellite images and the channel conveyance at these locations has been restored by interpolating between the upstream and downstream cross-sections. This approach, therefore, is based on the assumption that flow through bridges does not pressurise under design event conditions. Where accurate flood level predictions are required at an individual property level, a more detailed model including as-built details of bridges and culverts will be required.

## 3.3 Hydrological input

The primary driver for flood hazards is rainfall, its intensity temporal and spatial distribution. The following sections discuss the concurrent precipitation data used for simulating historical high flow events and the statistically developed design storm rainfall data under future climate conditions.

### 3.3.1 Design events

#### 3.3.1.1 Design rainfall

Rainfall depths generated using NIWA’s High Intensity Rainfall Design System (HIRDS) version 4. This data was used to create HIRDSV4 temporal rainfall profiles for a number of rainfall events and durations. To ensure that the different response times of all catchments contributing to flooding in the model domain are captured, 1 hour, 6 hour, 12 hour and 24 hour rainfall hyetographs have been simulated within the model for the 100 year ARI event. Given the above adopted critical duration analysis, HIRDSV4 temporal rainfall hyetographs are applied to the model instead of the fully-nested rainfall profile that had been adopted by the BOPRC’s MIKE 11 flood model of the Utuhina Stream.

#### 3.3.1.2 Lake levels

Lake levels affect flood levels in the lower parts of the catchments where flood levels in the streams are influenced by lake level. Lake levels of various return periods under the present-day climate conditions have been provided by RLC. The 20 year high lake level has been paired with the 100 year design rainfall.

### 3.3.2 Calibration events

The most reliable rainfall gauge used in calibration is that located at Whakarewarewa. The discharge gauge used is at Depot Street on the Utuhina Stream. It should be noted that the rain gauge is not actually located within the catchment whose response is gauged at Depot Street. This rainfall is used as “representative”, and may be somewhat different in the actual catchment. The differences could be in depth, duration and timing of the event.

Utilising the gauge record for Waingaehe Stream as a second calibration location has been investigated. However the rainfall-runoff response in the data for this catchment appears inconsistent and so is not considered reliable, in part because there is no rain gauge located in the catchment. This, combined with the lack of rainfall data in the Utuhina Catchment, means there is some uncertainty in the model results. We recommend that additional monitoring equipment is



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installed to record rainfall-runoff response in the Waingaehe Catchment for a period of time to reduce the uncertainty in the model parameters that have been adopted.

Analysis of the largest annual events (assessed on peak discharge attained) recorded at Depot Street over the past few years is shown in Table 3.2. From this it can be seen that the three highest flows were recorded from events in 2011, 2014 and 2018 (bold in Table 3.2).

**Table 3.2: Annual Maxima: Utuhina at Depot Street**

| Year | Peak discharge ($m^3/s$) | Rainfall Depth (mm) | Rainfall Duration (hours) |
|---|---|---|---|
| 2005 | 13.20 | 58 | 19 |
| 2006 | 20.54 | 22.5 | 7 |
| 2007 | 14.34 | 45 | 22 |
| 2008 | 19.09 | 119.5 | 48 |
| 2009 | 14.55 | 43 | 2 |
| 2010 | 12.33 | 20.7 | 5 |
| 2011 | **27.94** | **117** | **6** |
| 2012 | 20.28 | 61.5 | 14 |
| 2013 | 11.86 | 13 | 11 |
| 2014 | **28.21** | **118.6** | **6** |
| 2015 | 11.42 | 31.1 | 12 |
| 2016 | 18.69 | 52.7 | 10 |
| 2017 | 24.10 | 159.1 | 48 |
| 2018 | **28.48** | **121.3** | **3** |
| 2019 | 20.42 | 49.5 | 9 |

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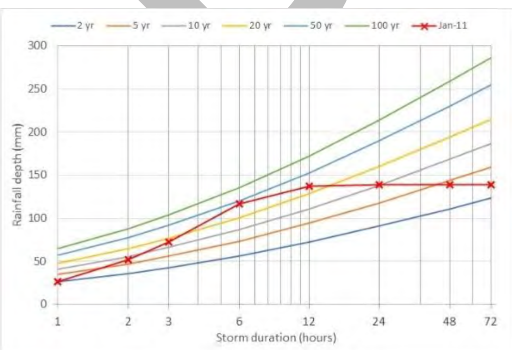
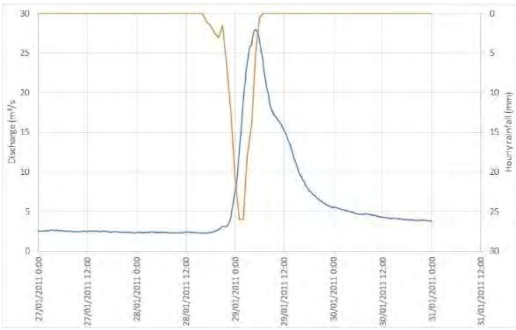
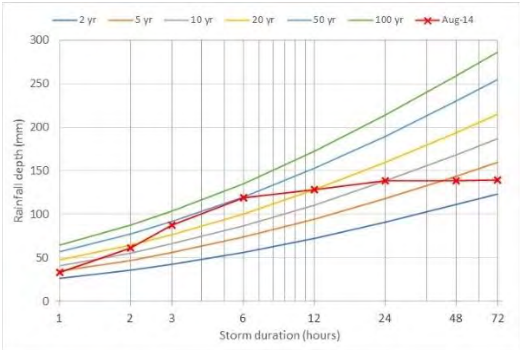
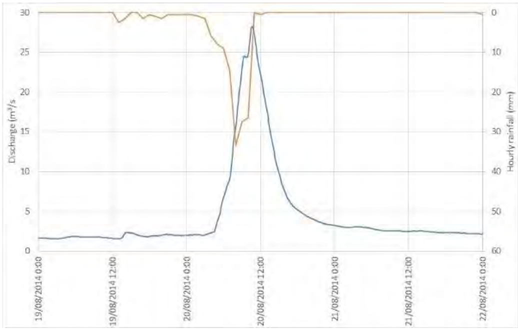
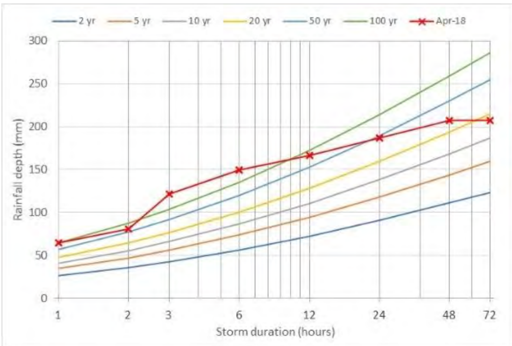
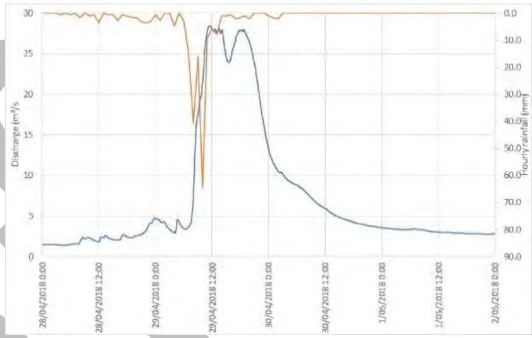
Analysis of the events causing the three largest flows is shown in Table 3.3.

**Table 3.3: Event summary**

| Year | Rainfall | Hydrograph: Utuhina at Depot Street |
|---|---|---|
| 2011 |   |   |

| Year | Rainfall | Hydrograph: Utuhina at Depot Street |
|---|---|---|
| 2014 |  |  |
| 2018 |  |  |

From the above, it can be seen that the events of 2011 and 2014 exhibit the following:

- High intensity rainfall of about 6 hours in duration.
- Single-peaked flood response in Utuhina Stream.
- Rainfall event maximum ARI about 44 years.

Conversely, the event in 2018 showed double-peak in both maximum rainfall intensity and in resulting discharge. The event was more severe in terms of rainfall return period, yet did not result in correspondingly high peak discharge – probably due to the total event discharge occurring over two peaks instead of one. The total volume of the 2018 event was significantly greater than that of the other two events.

From the above, the events of 2011 and 2014 were selected as more suitable for model calibration purposes.

## 3.4 Model grid size convergence

TUFLOW (Release version 2020AB) allows the model mesh to sample the underlying DEM at a finer resolution than the mesh grid size. With the sub-grid sampling function turned on, the event that occurred on 27 August 2017 was simulated using a range of grid sizes, from 20 m to 2 m. The purpose of this was to establish whether or not model computation grid size had an effect on model results. Key findings are summarised below:

- Model results from mesh with 20 m and 10 m cell sizes showed flows that were substantially higher than the rest of the test simulations (cell sizes 5 m to 2 m), as shown in Figure 3.3. This is true not only in channelised flow in the lower catchment, but throughout the entire simulation domain. The flood inundation extent and flow connectivity also varied greatly in comparison to the rest of the test simulations.

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- Simulated flows, depths, and flood extents are within a relatively tight range from simulations using 5 m, 4 m, 3 m, and 2 m grid cells.

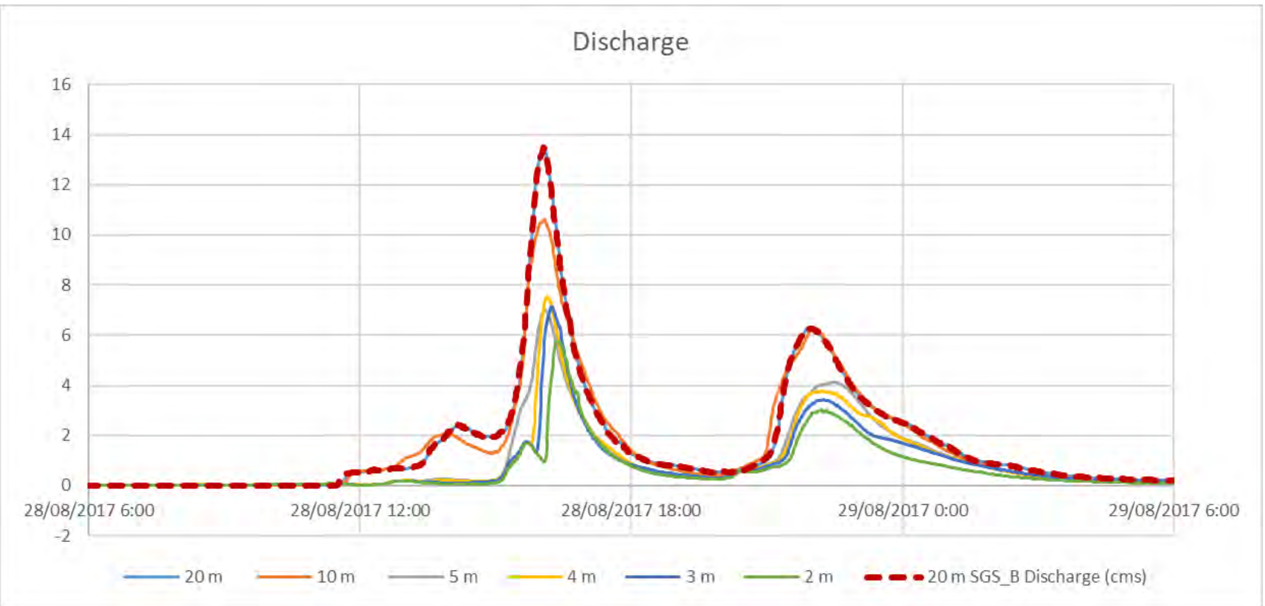


*Figure 3.3: Model grid size convergence test results*

Given that the smaller the grid size, the more computation points there are, and the longer simulations would take to run, there is a driver to use the largest grid size that will still yield suitable accuracy in results. From the above analyses, it was obvious that cell sizes of 10 m and 20 m were too large.

This resulted in selection of 5 m x 5 m as the optimal grid size to be used in meeting the overall intentions of the modelling work undertaken. A smaller grid size may be required for local scale assessments.

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# 4 Model calibration results

The primary purpose of the model is to provide RLC with an overview of ongoing flood hazards and serve as a live tool to investigate future flood hazards and inform flood hazard mitigation plans for potential development schemes on a community scale. The performance of the model has therefore been evaluated through simulating real storm events and sensitivity tests.

## 4.1 Model simulation of recorded storms

As described in Section 3.3.2, the simulation of observed flood events focused on the two single-peak events in 2011 and 2014.

### 4.1.1 20 January 2011

The model was calibrated by adjusting roughness and hydrological losses. The resulting performance is shown in Figure 4.1. Evident from the model performance is that baseflow was missing. The model produced a peak discharge of about 24 $m^3/s$, in comparison to the gauged 28 $m^3/s$.

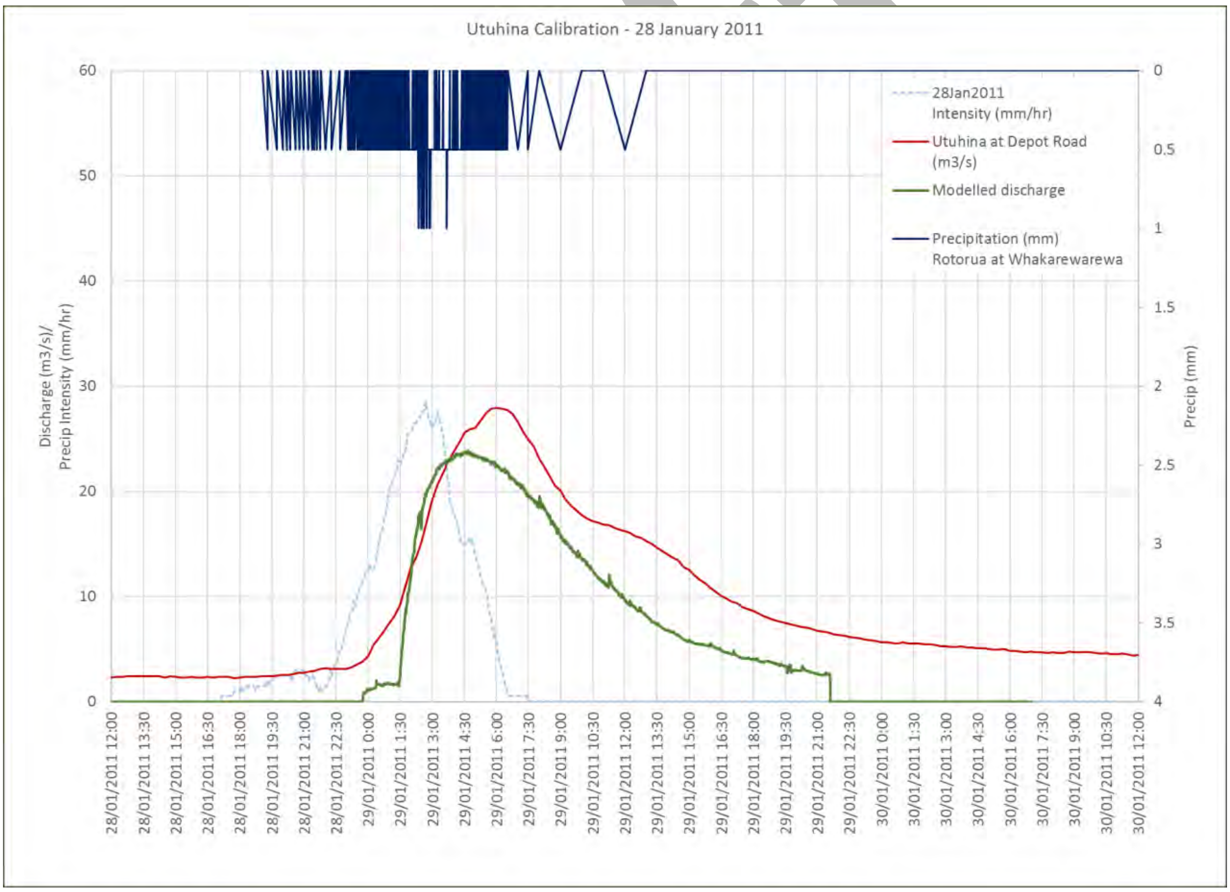


*Figure 4.1: Model calibration: January 2011*

### 4.1.2 20 August 2014

Gauged and modelled responses for this event are shown in Figure 4.2.

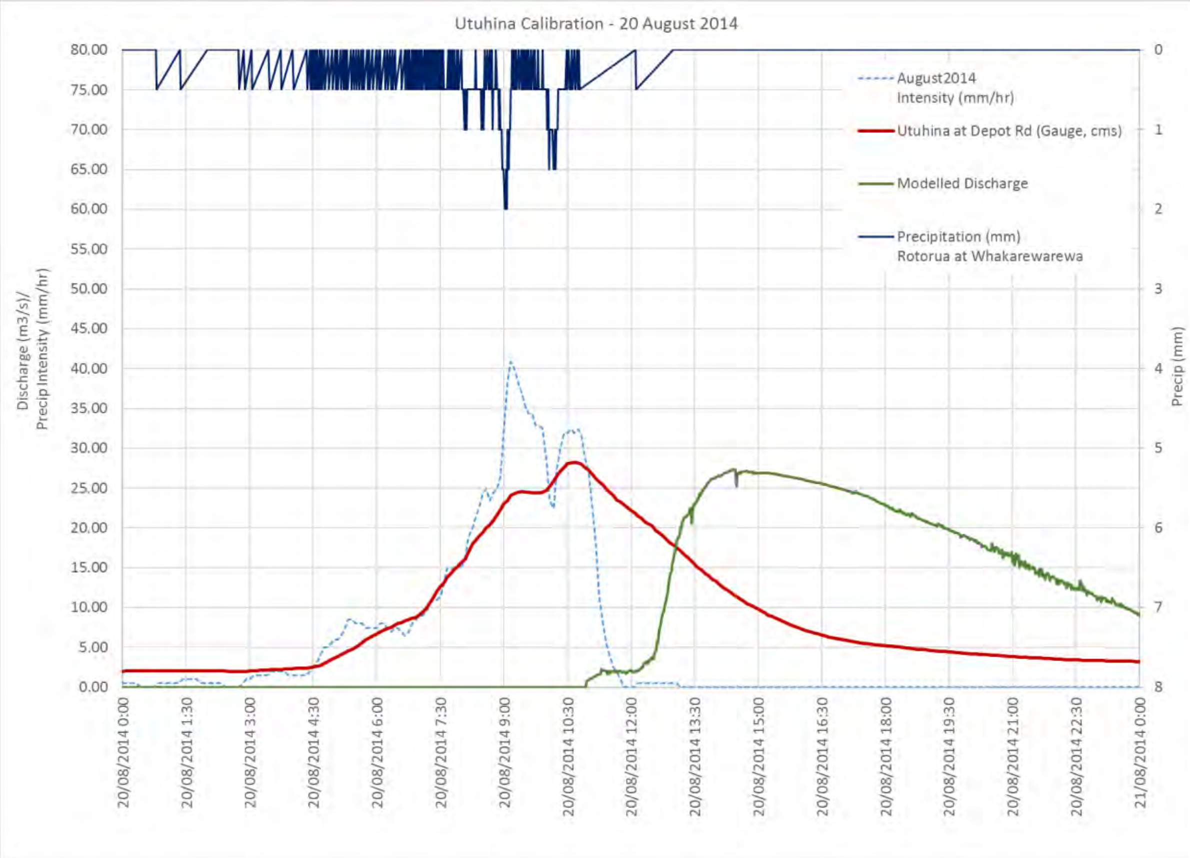


*Figure 4.2: Model calibration: August 2014*

For this event, the model produced similar peak discharge to the gauge (27 $m^3/s$ modelled, 28 $m^3/s$ gauged). The results show that the modelled peak occurred approximately three hours after the peak in the gauged data. This is likely due to the fact that the rain gauge is not located within the catchment and therefore the timing of the rainfall hyetograph relative to the stream gauge response is dependent on the direction the rain storm tracks across an area. This is inferred by the fact that there is no lag between the hyetograph and hydrograph shapes which would be expected when there is a "lag time" between catchment rainfall and catchment runoff response.

### 4.1.3 Calibration summary

It is noted that the calibration events occurred at different times of the year, with potentially different antecedent conditions. The model performance seemed to match observations within reasonable limits, particularly given the limitations of recorded data outlined in Section 3.3.2. However, an exact calibration was not possible given the lack of rainfall data (including spatial variation) in the catchments being modelled. That said, the calibration achieved is considered acceptable for the purposes of the masterplan work.

The lack of rainfall data in the Utuhina Catchment and a rainfall/runoff data in the eastern catchments means there is some uncertainty in the model results. We recommend that additional monitoring equipment is installed to record rainfall-runoff response in the Waingaehe Catchment for a period of time to reduce the uncertainty in the model parameters that have been adopted.

# 5 Model outputs

A sample model output is shown in Figure 5.1 (eastern and western models overlapped). Detailed assessment of model results and the overall masterplan work is presented in the masterplan report. Model extent can be seen as covering all of urban Rotorua, with predicted inundation depths shown in differing colour bands.

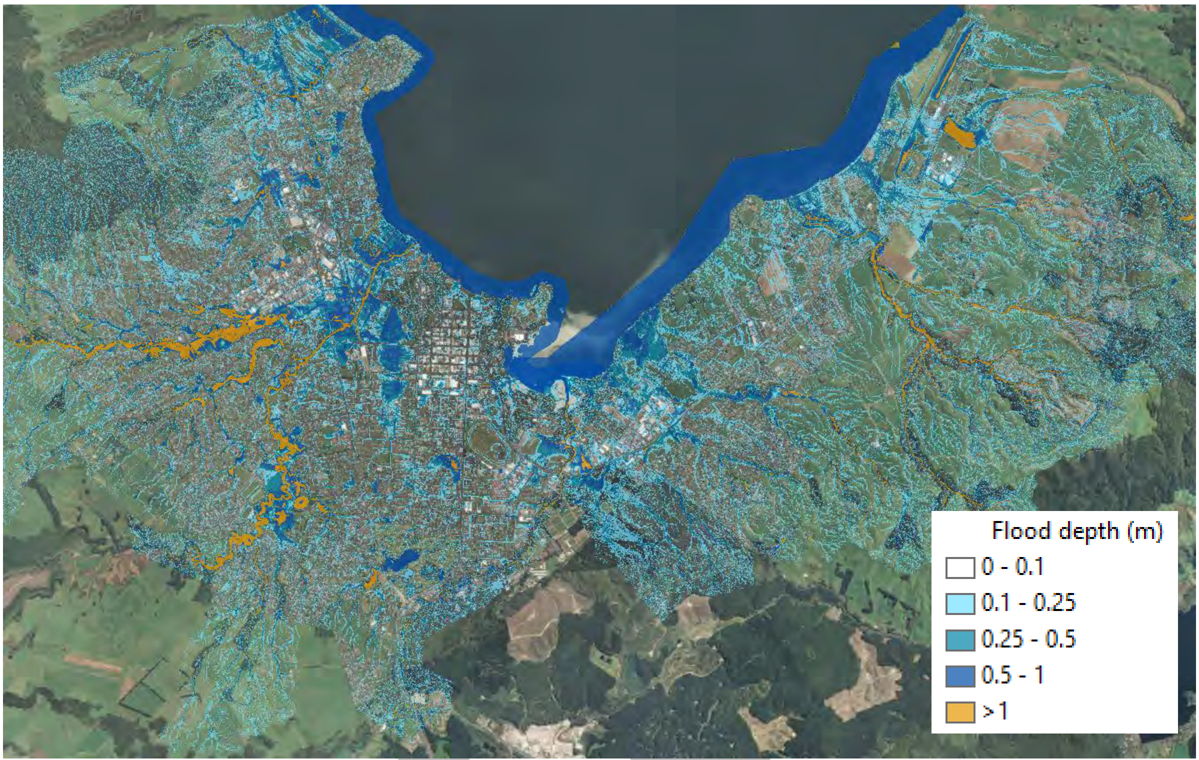


*Figure 5.1: Sample model output (1 hr duration design event)*

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# 6 Limitations

The purpose of this model build was to provide a better understanding of the flood hazard within areas where there is currently limited, or out-of-date, flood hazard information available. It is understood that RLC intends to use the model outputs for flood mapping purposes and to identify areas of potential hazard. The primary purpose of this model was to support the masterplan process in the selected catchments. While the model build methodology described in this report is appropriate for these purposes, caution should be taken when using model outputs for other means beyond the scope of this work.

A 2D 'rain-on-grid' model was created for the model domains using TUFLOW software. This model build uses a 2D+ methodology which consists of a simplified representation of many hydraulic structures within the model. For example, the majority of stormwater pipes were not explicitly represented in 1D within the model, and instead the DEM was modified to represent pipes as open channels with equivalent hydraulic performance, in accordance with the agreed methodology. Further discussion on this approach is given in Section 3.2. A notable limitation of this approach is in the lack of ability for representation of detailed 1D hydraulic structures, however for the purposes of this flood hazard assessment it was deemed adequate. It is recognised that should the intended purpose of these models change in the future, further consideration of this limitation will be made. It is possible to add detail to these models where and when needed in the future.

The rainfall gauge used for calibration is that located at Whakarewarewa. This is approximately 10 km away from the centroid of the Utuhina Catchment and 5 km away from the centroid of the Waingaehe Catchment. In addition to this the rainfall-runoff response in the data for the Waingaehe Stream appears inconsistent so was not able to be used as a second calibration. This means there is some uncertainty in the model results. We recommend that additional monitoring equipment is installed to record rainfall-runoff response in the Waingaehe Catchment for a period of time to reduce the uncertainty in the model parameters that have been adopted.

The modelling undertaken has been based on remotely sensed ground levels (LiDAR survey) and on design future rainfall events, both of which have accuracy limitations. The model results have generally been presented only to show flooding where maximum depth in excess of 100 mm has been estimated. Furthermore, a direct rainfall approach has been applied, which can highlight accuracy deficiencies in input data by showing small "puddles" in predicted flooding. It is usual with flood depth results from this kind of modelling approach that the results be "cleaned" by removing puddles before publication. T+T has presented raw model results in this report, in anticipation of further "cleaning" of model result before publication and use for design purposes.

In modelling of wide areas such as these reported on in this document, accuracy is limited, and field verification of flood predictions is strongly recommended. The modelling approach adopted has been aimed at determination of flood hazard extent for the events considered and does not necessarily deliver design flood levels to be applied. We recommend that if flood levels are to be considered, that this field verification be undertaken. This is particularly the case where a minor degree of flooding (both in depth and in extent) has been predicted on a parcel which, given accuracy limitations in the approach, may or may not be real.

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# 7 Applicability

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

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# Appendix B: HEC-HMS model results

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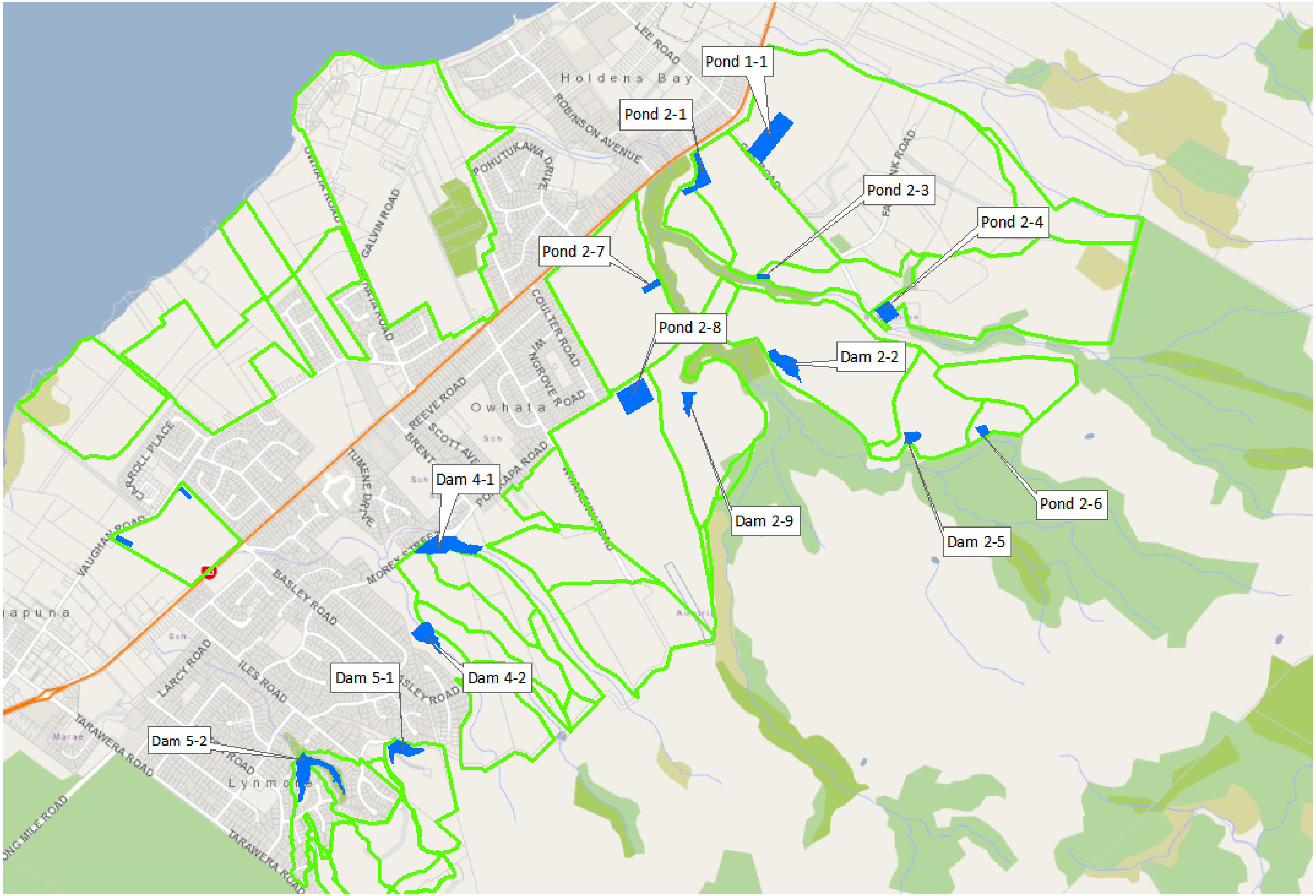
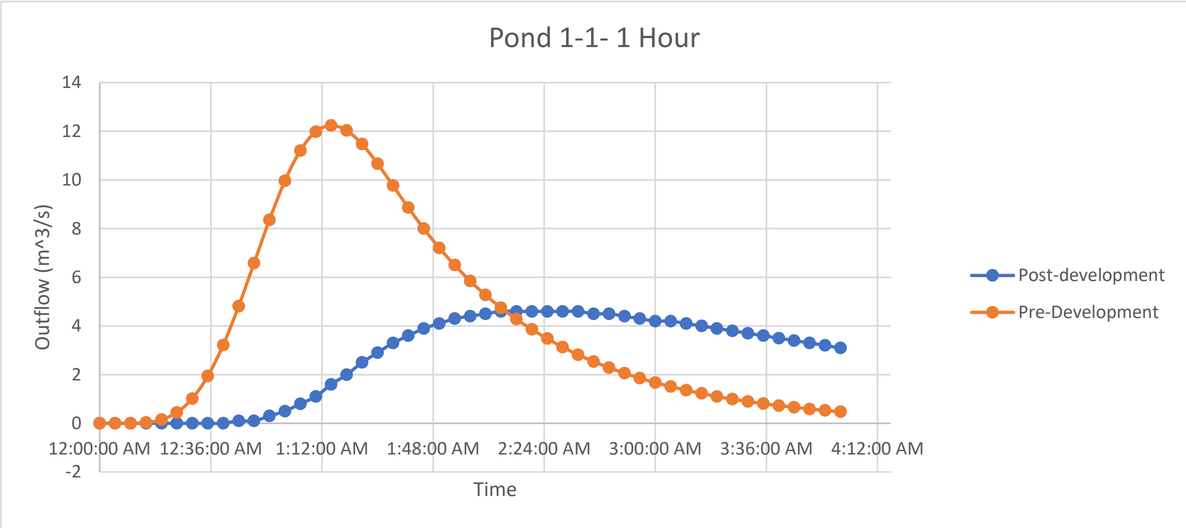
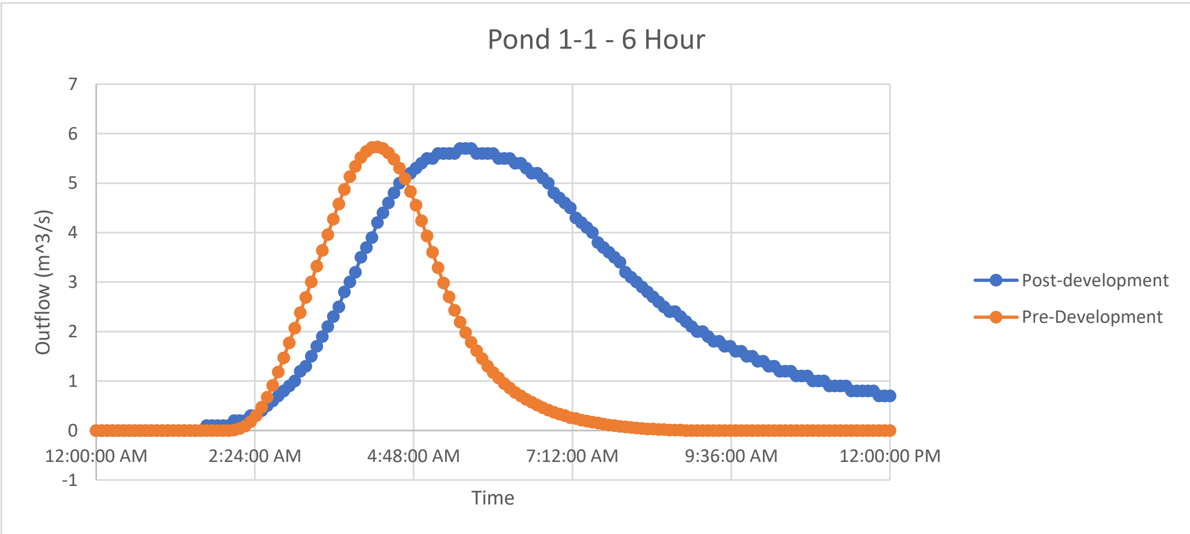
# East Optioneering - Basin Names



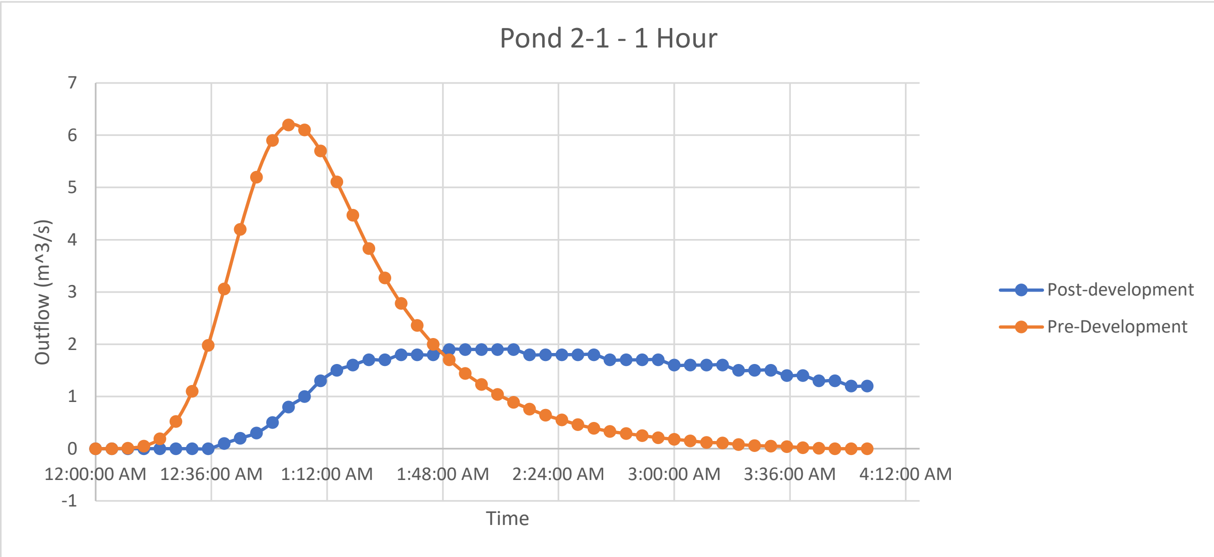
# Summary Graphs

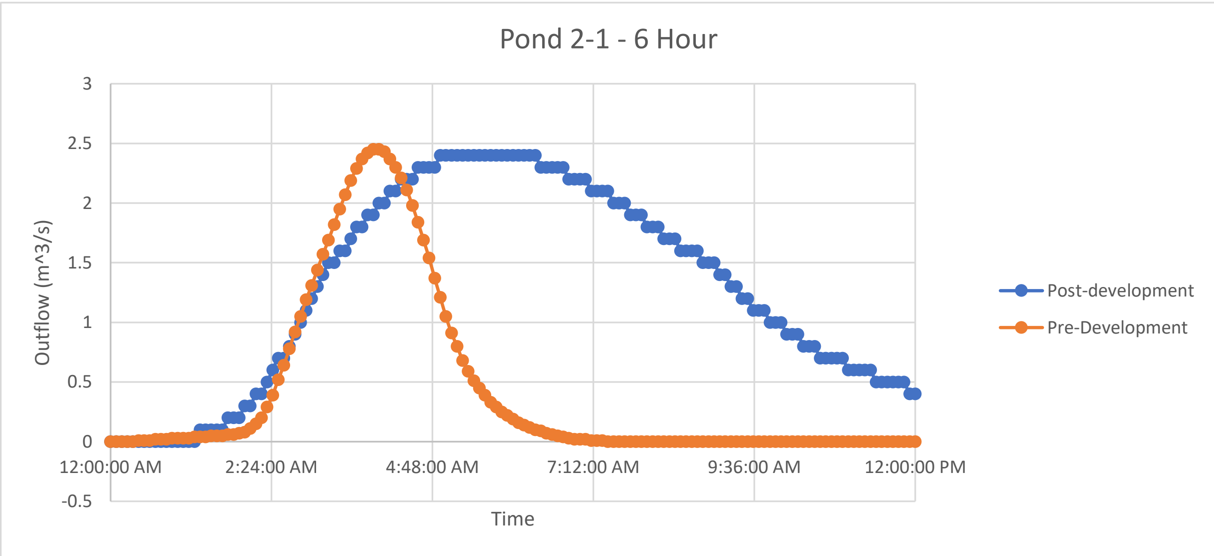
## Pond 1-1



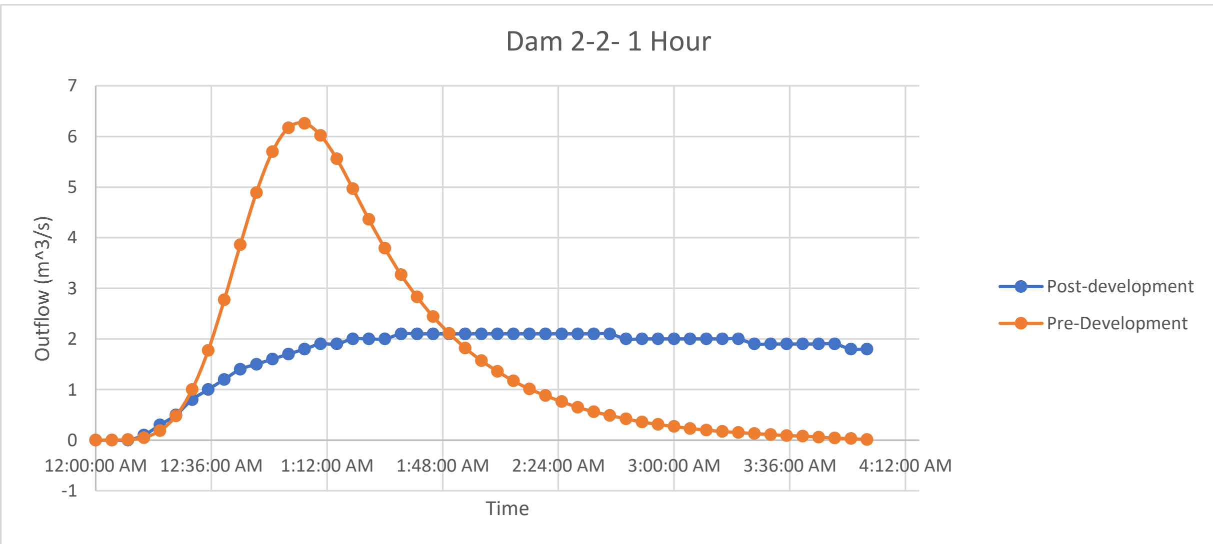



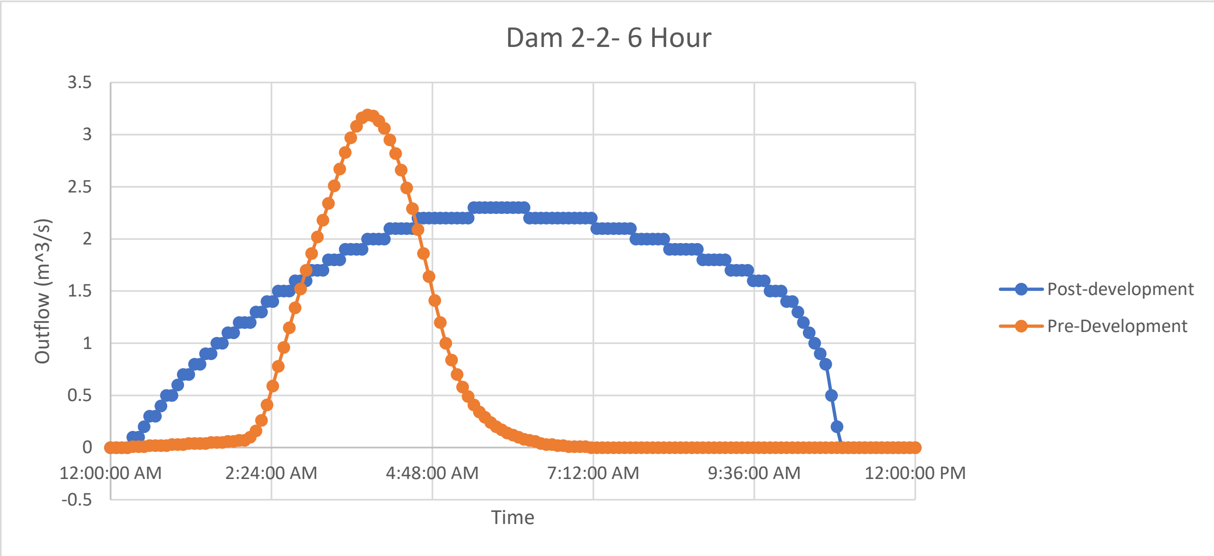
## Pond 2-1



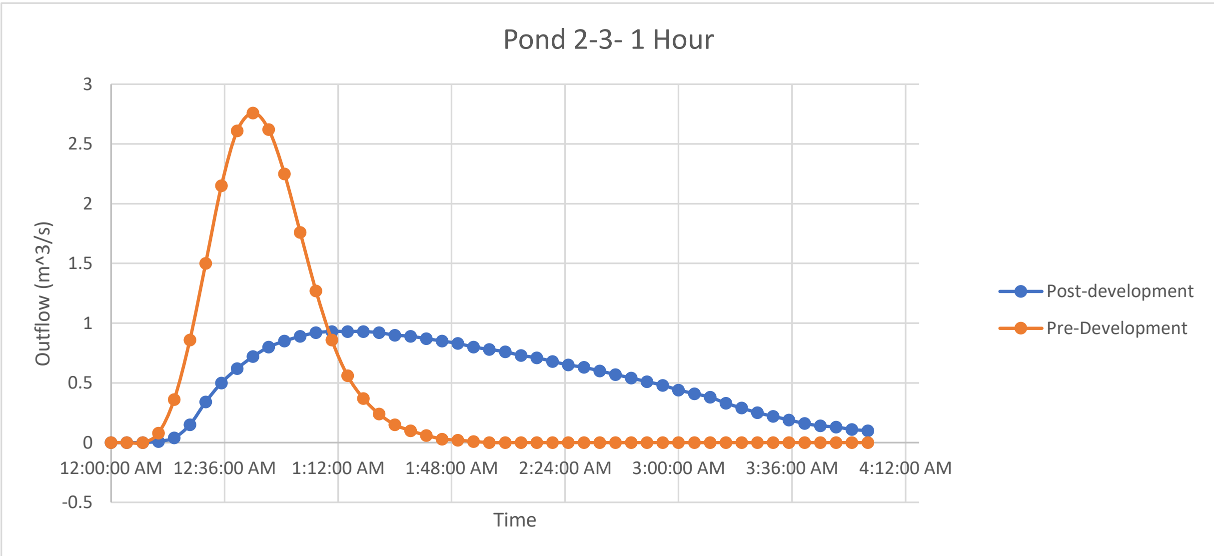



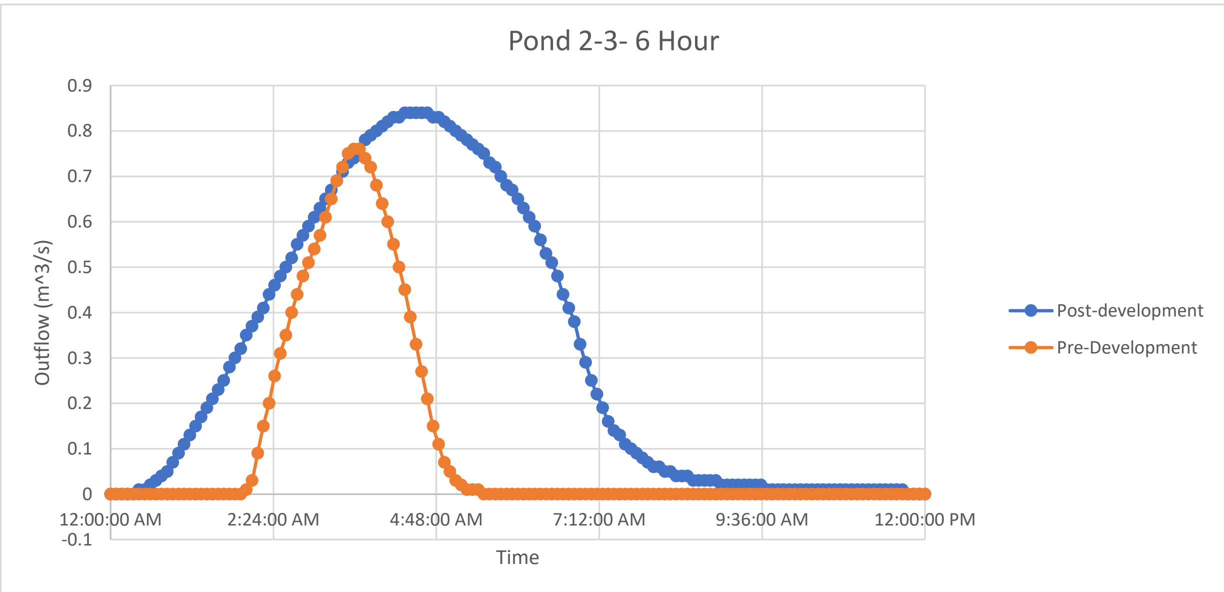

## Dam 2-2



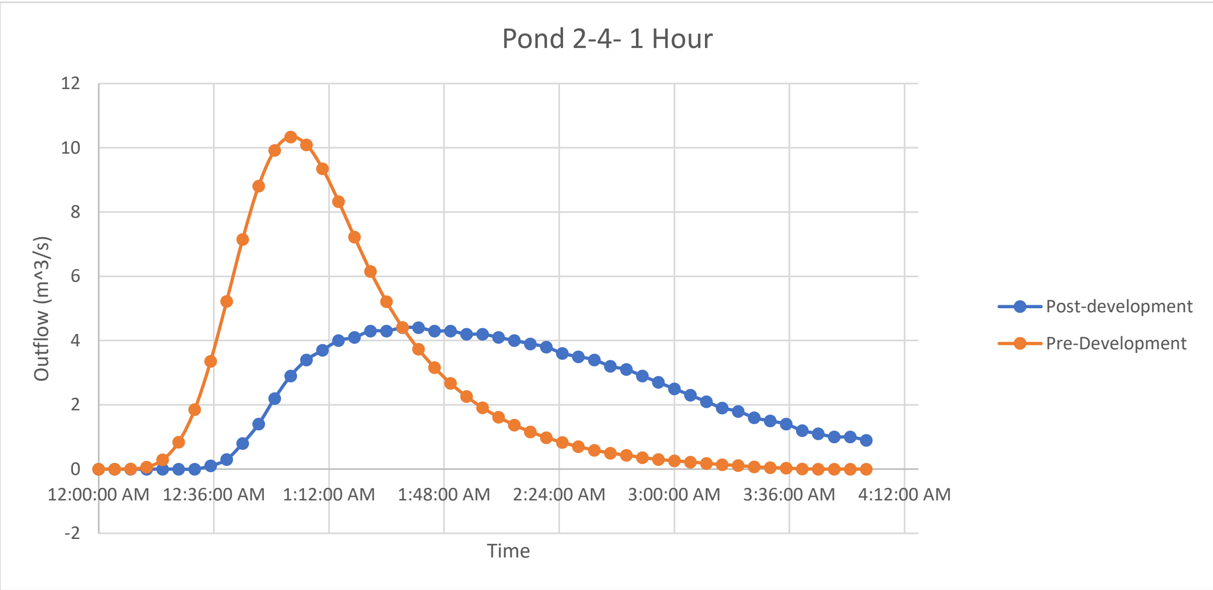

Dam 2-2- 6 Hour
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
Outflow (m^3/s)
12:00:00 AM
2:24:00 AM
4:48:00 AM
7:12:00 AM
9:36:00 AM
12:00:00 PM
Time
Post-development
Pre-Development

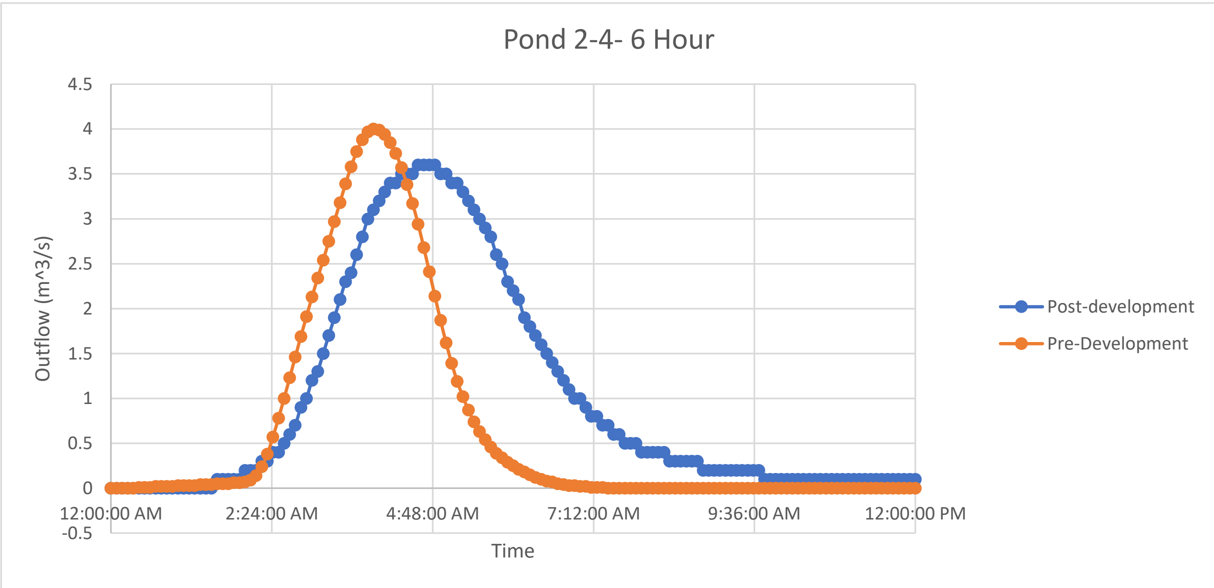
## Pond 2-3


Pond 2-3- 1 Hour
3
2.5
2
1.5
1
0.5
0
-0.5
Outflow (m^3/s)
12:00:00 AM
12:36:00 AM
1:12:00 AM
1:48:00 AM
2:24:00 AM
3:00:00 AM
3:36:00 AM
4:12:00 AM
Time
Post-development
Pre-Development


Pond 2-3- 6 Hour
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
Outflow (m^3/s)
12:00:00 AM
2:24:00 AM
4:48:00 AM
7:12:00 AM
9:36:00 AM
12:00:00 PM
Time
Post-development
Pre-Development

## Pond 2-4

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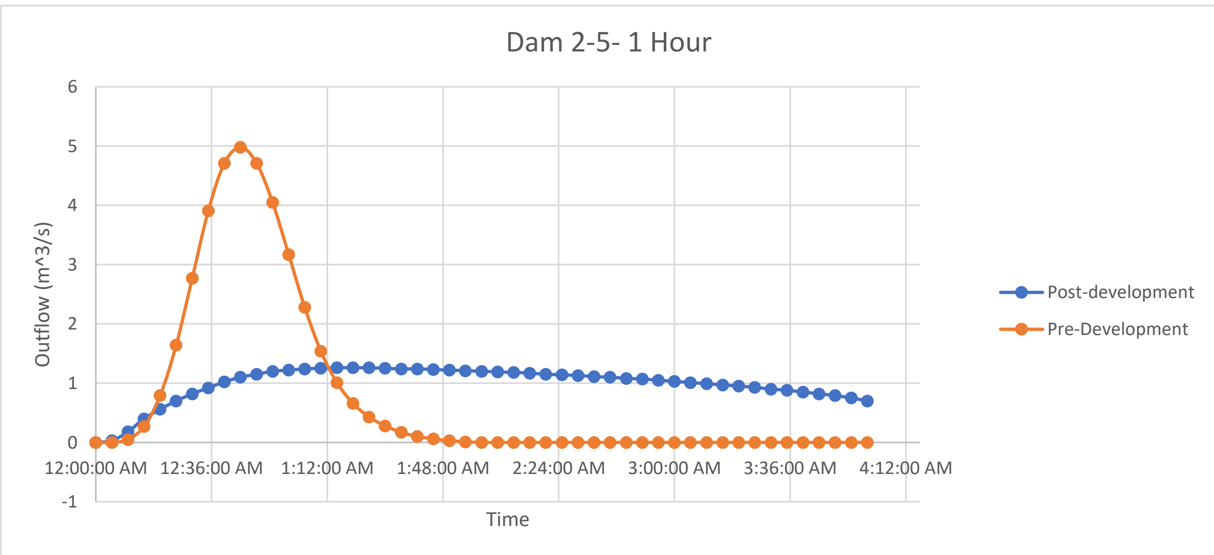
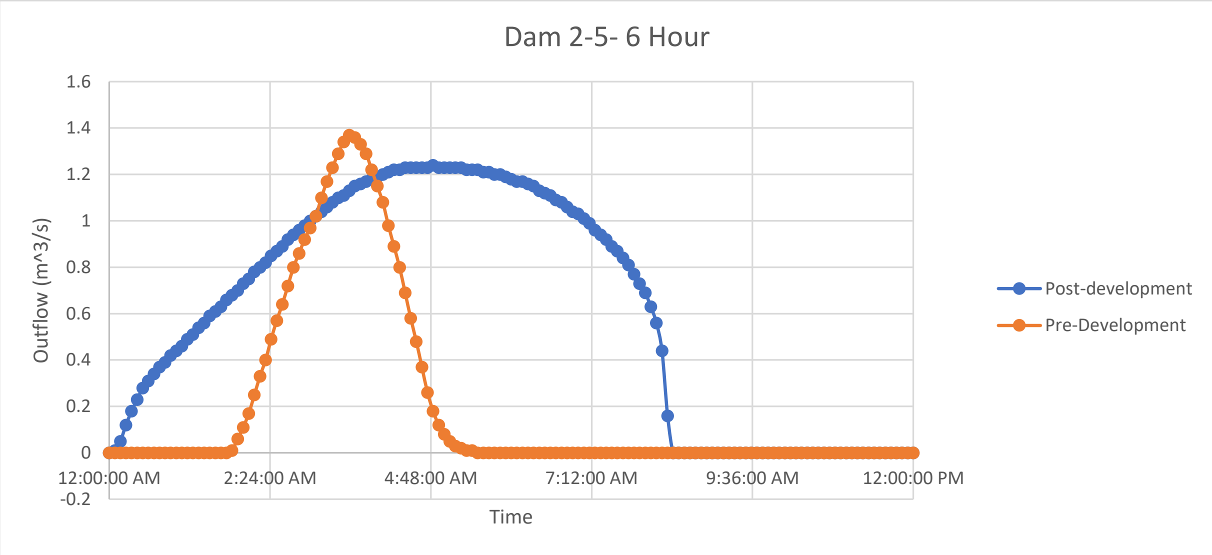
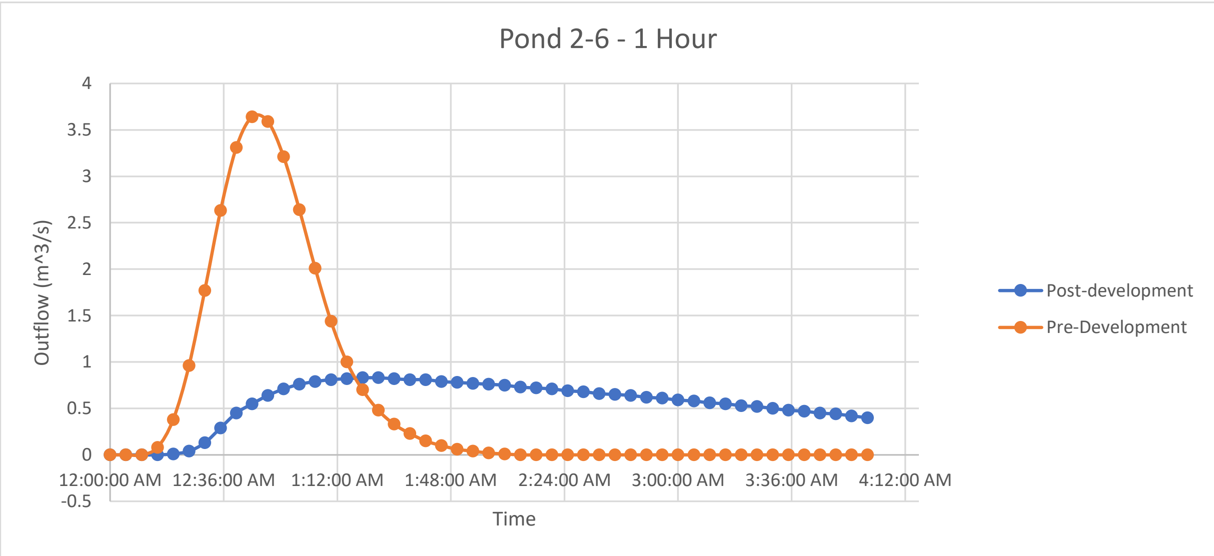
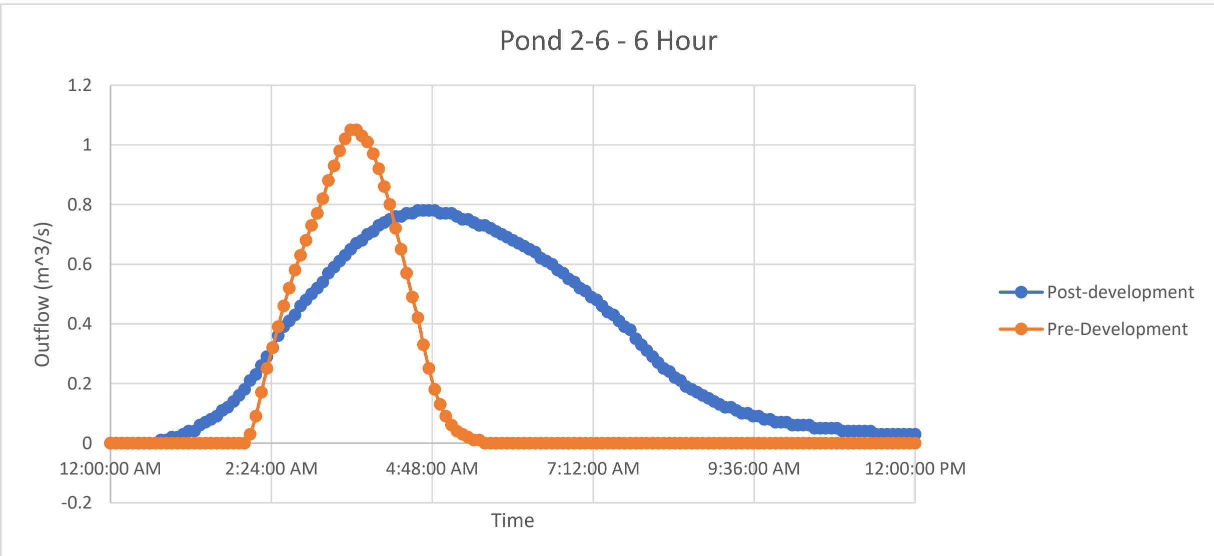


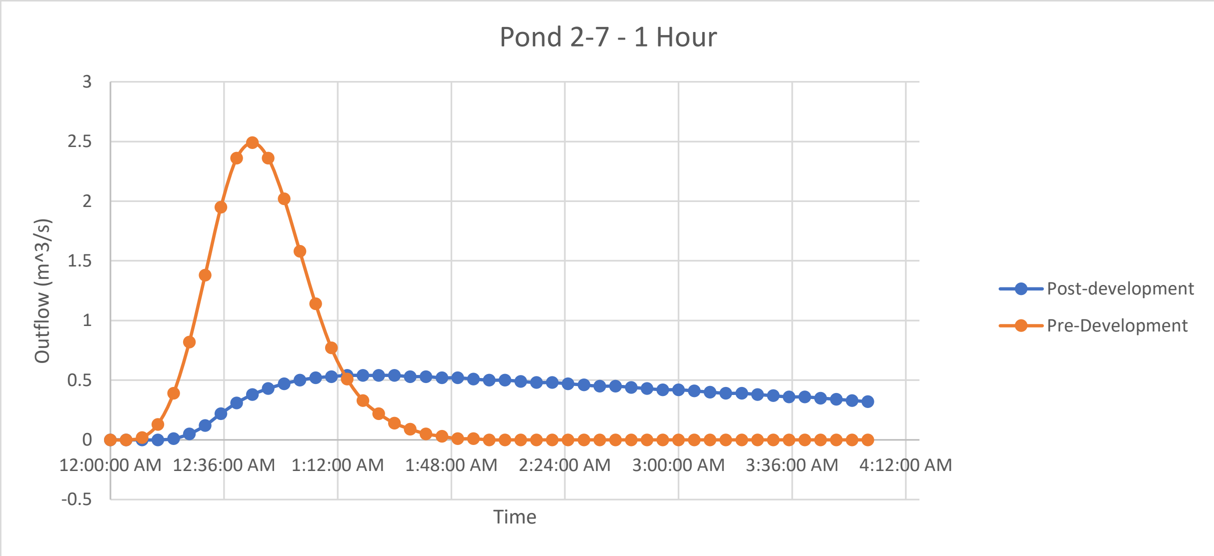

Dam 2-5- 6 Hour
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
Outflow (m^3/s)
12:00:00 AM
2:24:00 AM
4:48:00 AM
7:12:00 AM
9:36:00 AM
12:00:00 PM
Time
Post-development
Pre-Development

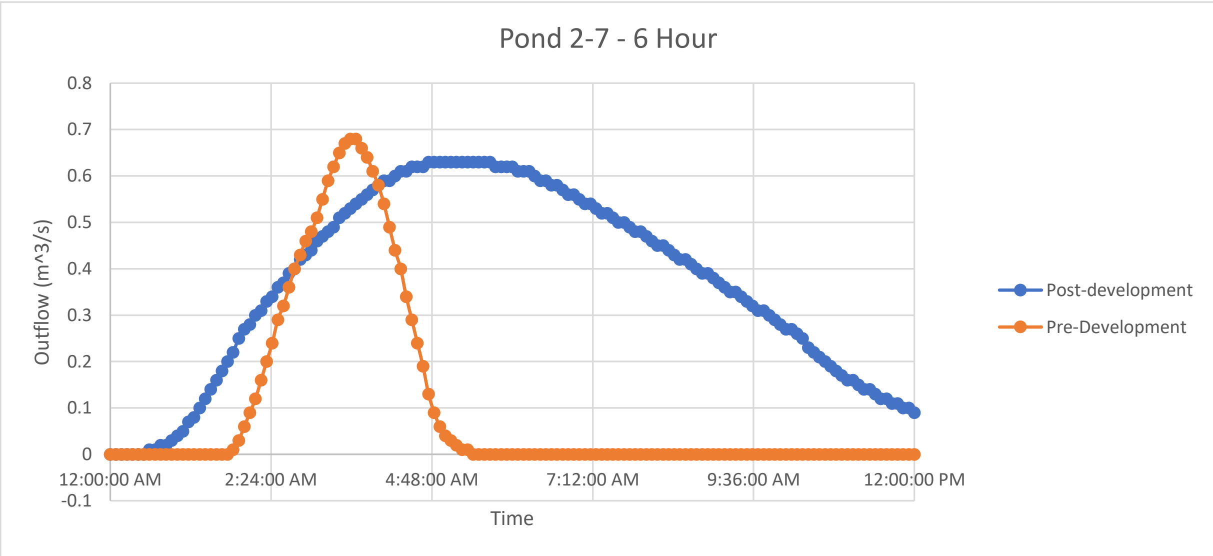
## Pond 2-6


Pond 2-6 - 1 Hour
4
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
Outflow (m^3/s)
12:00:00 AM
12:36:00 AM
1:12:00 AM
1:48:00 AM
2:24:00 AM
3:00:00 AM
3:36:00 AM
4:12:00 AM
Time
Post-development
Pre-Development

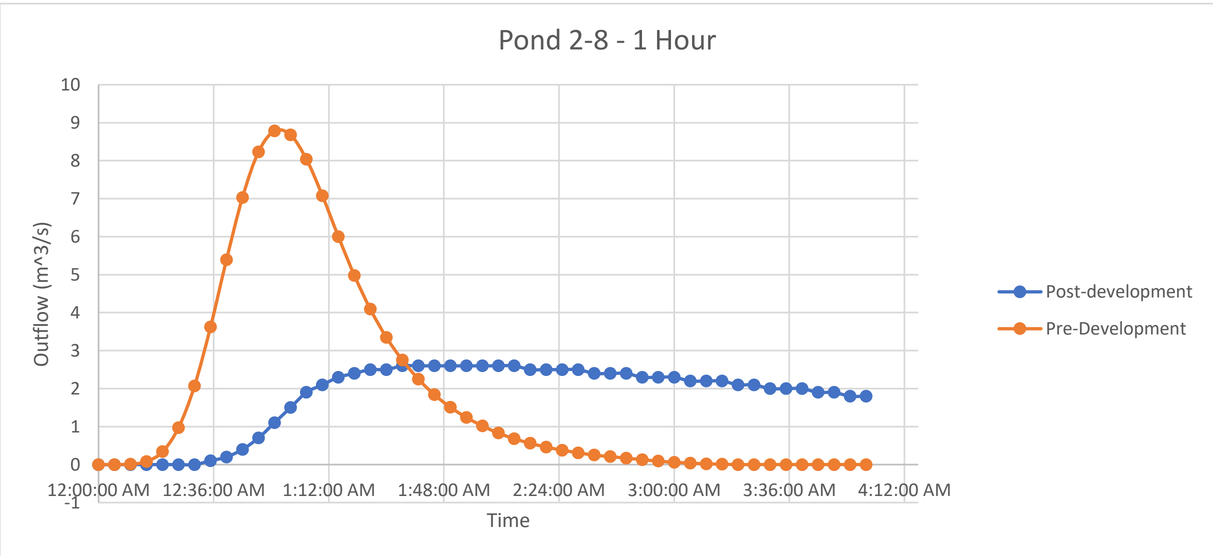

Pond 2-6 - 6 Hour
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
Outflow (m^3/s)
12:00:00 AM
2:24:00 AM
4:48:00 AM
7:12:00 AM
9:36:00 AM
12:00:00 PM
Time
Post-development
Pre-Development

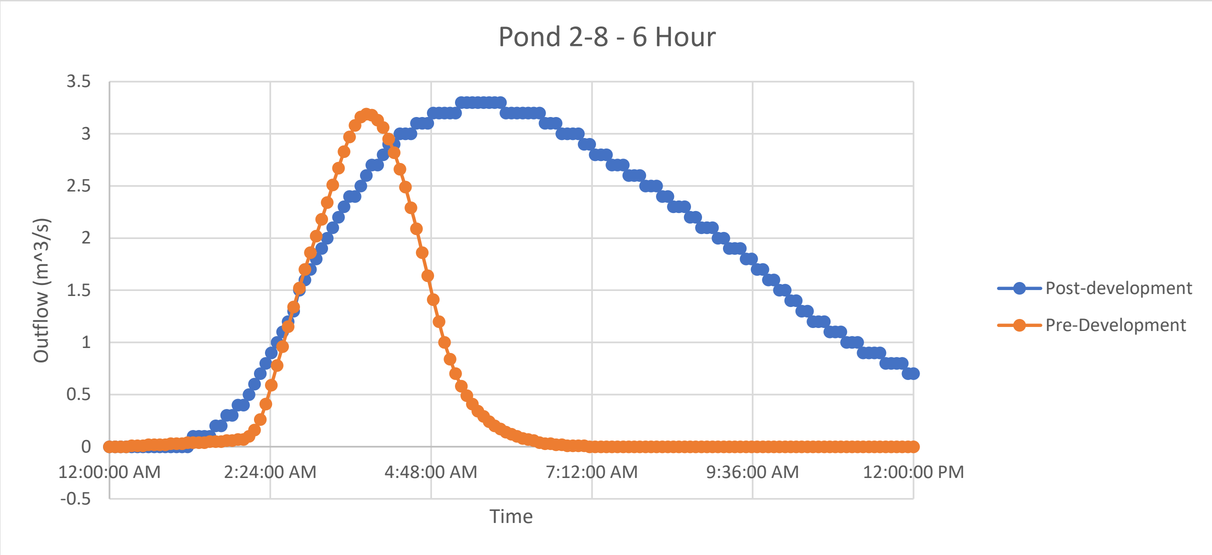
## Pond 2-7



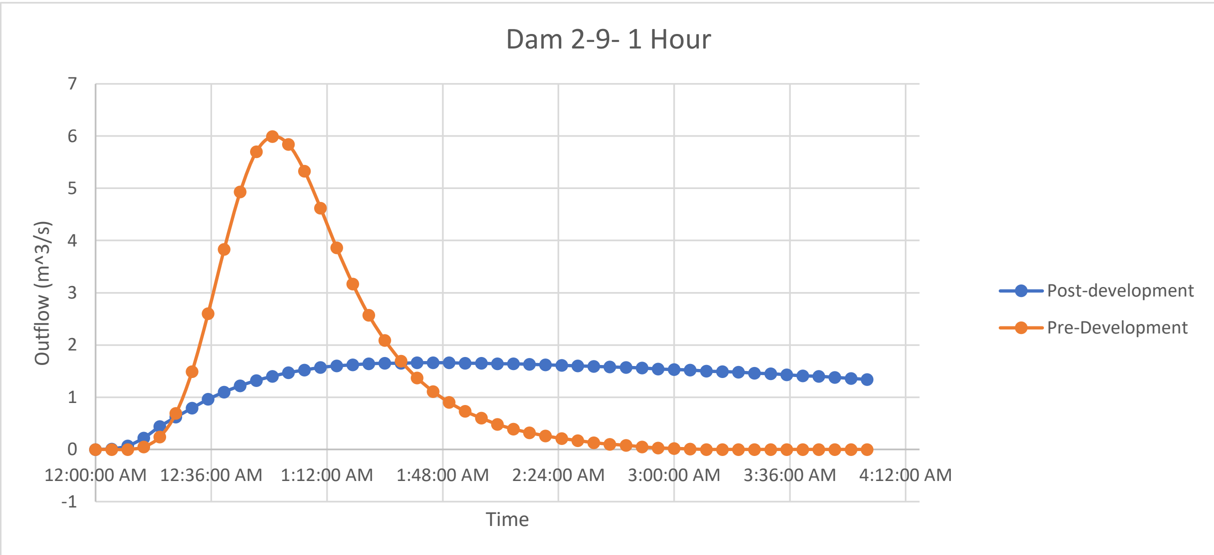



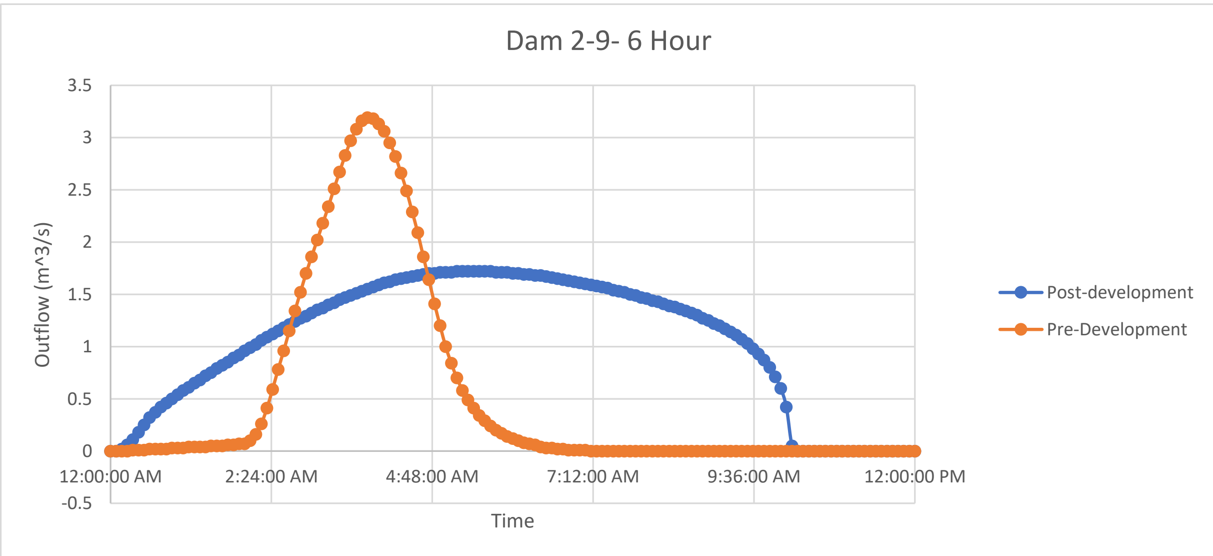

## Pond 2-8



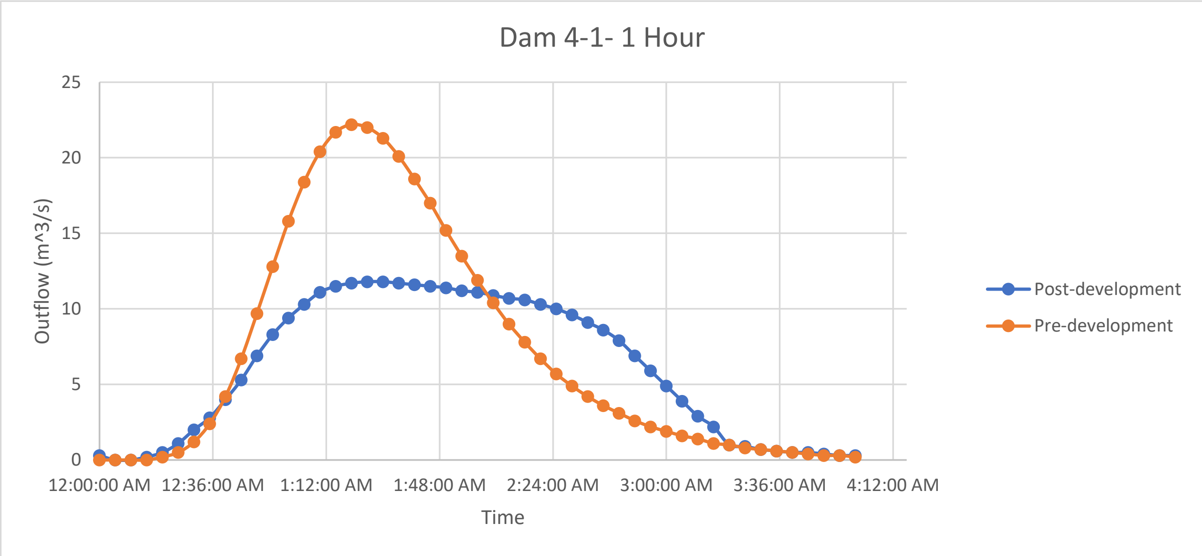

Pond 2-8 - 6 Hour
Outflow (m^3/s)
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
12:00:00 AM
2:24:00 AM
4:48:00 AM
7:12:00 AM
9:36:00 AM
12:00:00 PM
Time
Post-development
Pre-Development

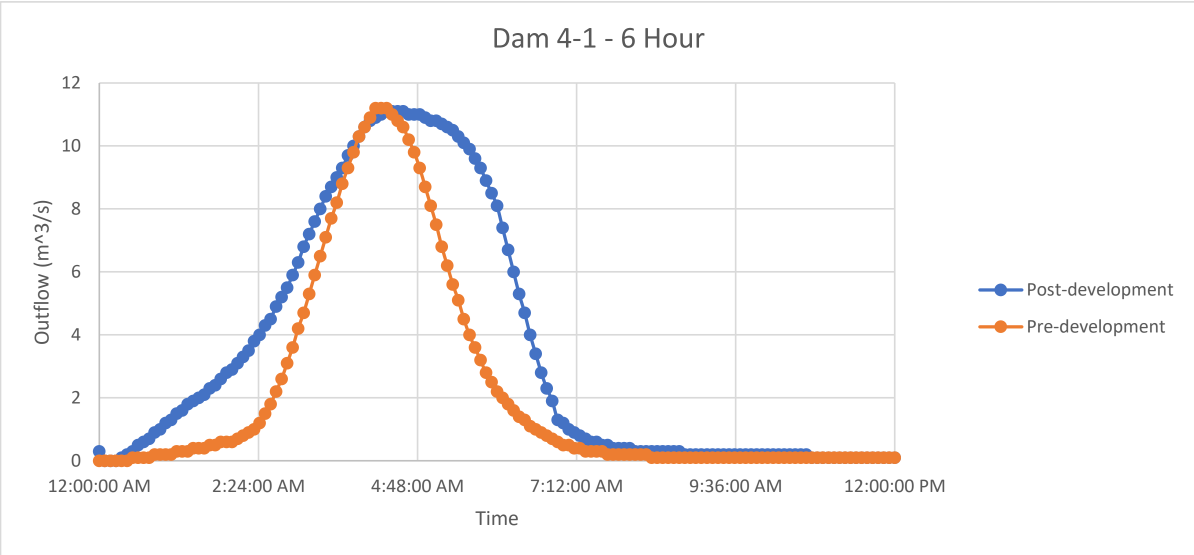
## Dam 2-9


Dam 2-9- 1 Hour
Outflow (m^3/s)
7
6
5
4
3
2
1
0
-1
12:00:00 AM
12:36:00 AM
1:12:00 AM
1:48:00 AM
2:24:00 AM
3:00:00 AM
3:36:00 AM
4:12:00 AM
Time
Post-development
Pre-Development

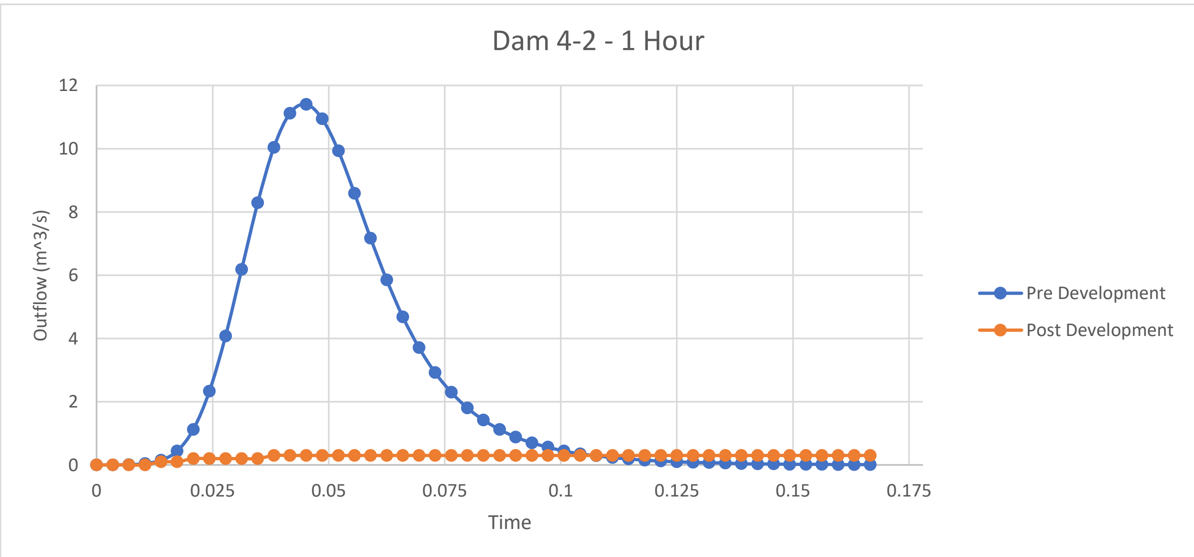

Dam 2-9- 6 Hour
Outflow (m^3/s)
3.5
3
2.5
2
1.5
1
0.5
0
-0.5
12:00:00 AM
2:24:00 AM
4:48:00 AM
7:12:00 AM
9:36:00 AM
12:00:00 PM
Time
Post-development
Pre-Development

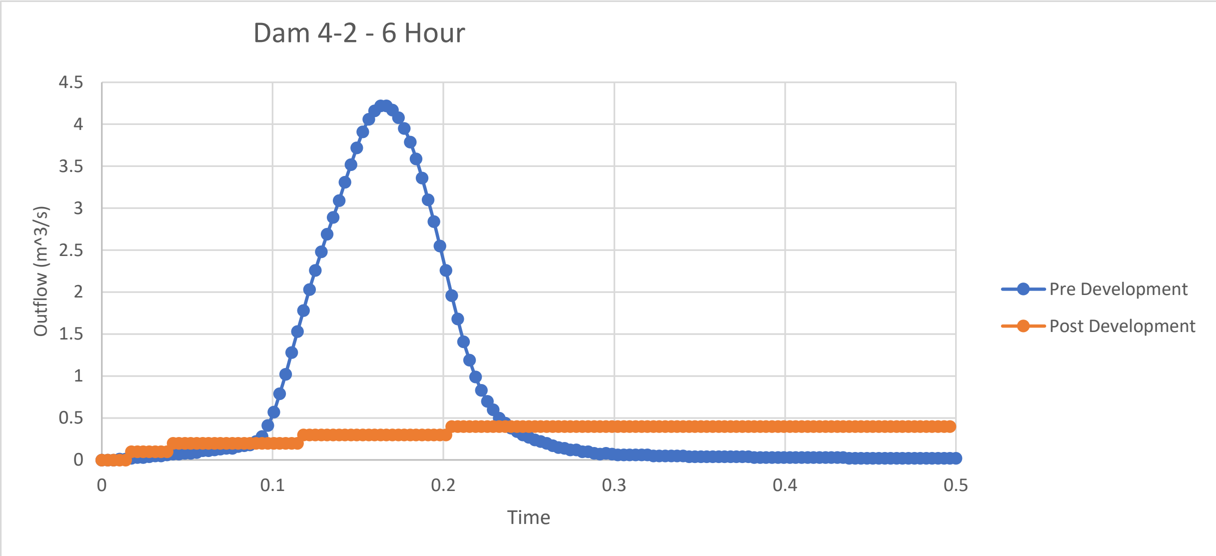
## Dam 4-1


Dam 4-1- 1 Hour
Outflow (m^3/s)
25
20
15
10
5
0
12:00:00 AM
12:36:00 AM
1:12:00 AM
1:48:00 AM
2:24:00 AM
3:00:00 AM
3:36:00 AM
4:12:00 AM
Time
Post-development
Pre-development

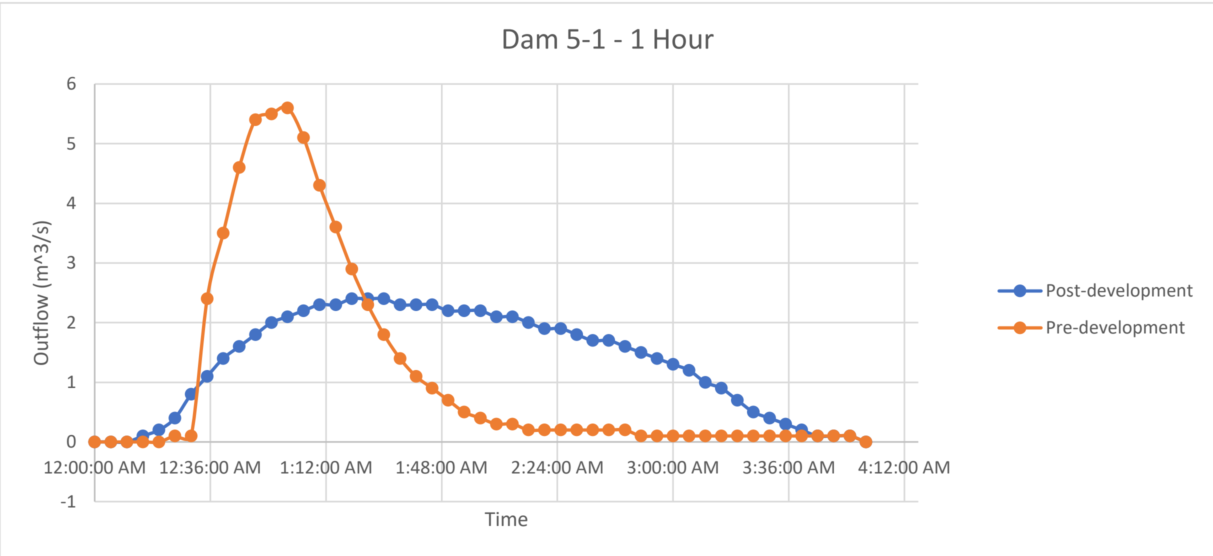

Dam 4-1 - 6 Hour
Outflow (m^3/s)
12
10
8
6
4
2
0
12:00:00 AM
2:24:00 AM
4:48:00 AM
7:12:00 AM
9:36:00 AM
12:00:00 PM
Time
Post-development
Pre-development

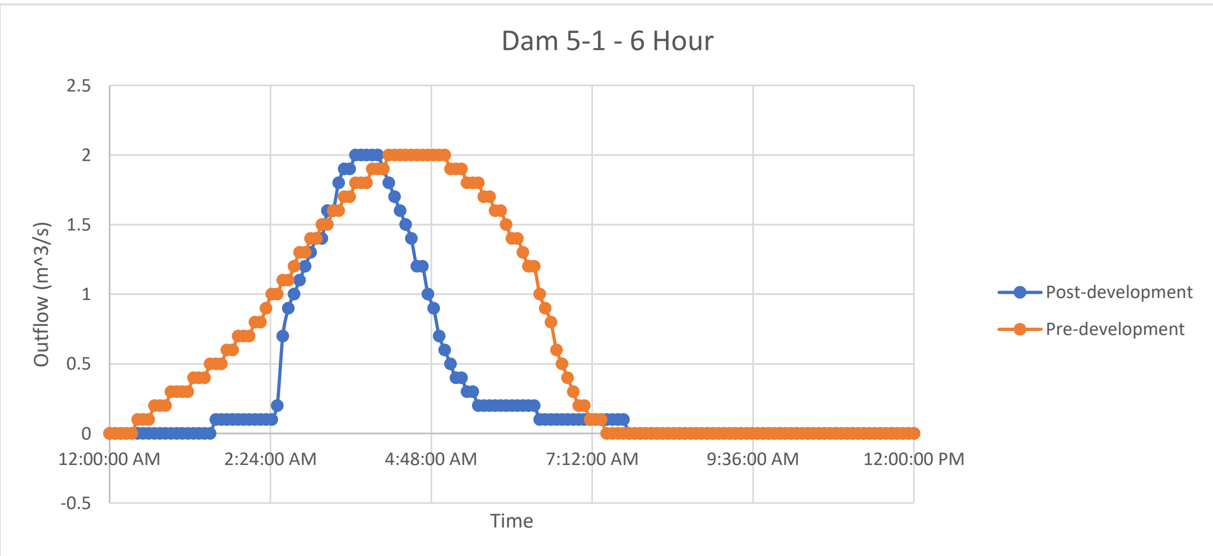
## Dam 4-2


Dam 4-2 - 1 Hour
Outflow (m^3/s)
12
10
8
6
4
2
0
0
0.025
0.05
0.075
0.1
0.125
0.15
0.175
Time
Pre Development
Post Development

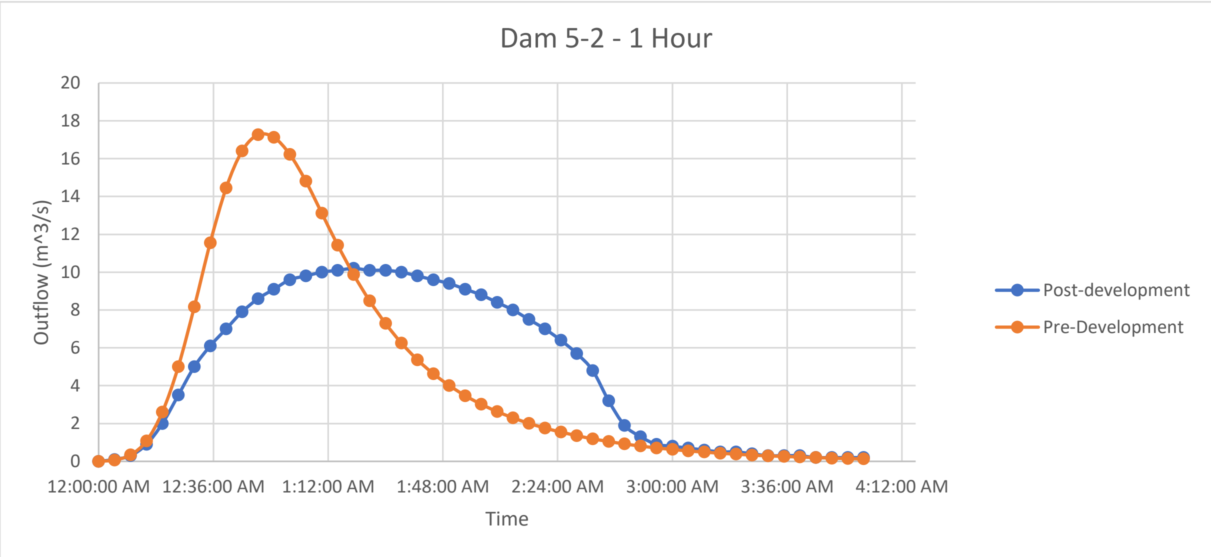

Dam 4-2 - 6 Hour
Outflow (m^3/s)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
0.1
0.2
0.3
0.4
0.5
Time
Pre Development
Post Development

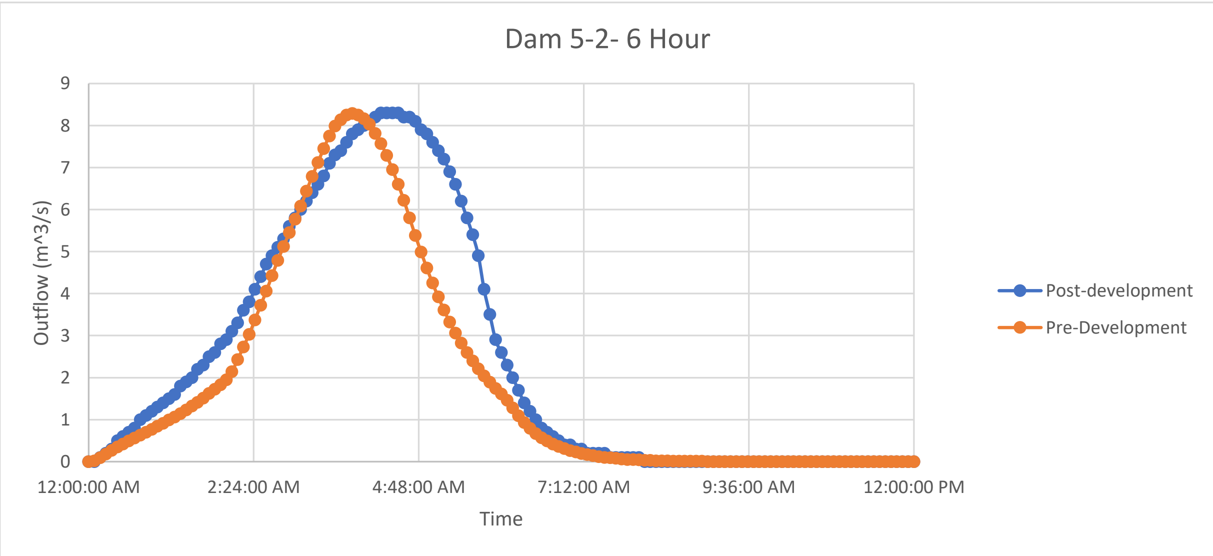
## Dam 5-1


Dam 5-1 - 1 Hour
Outflow (m^3/s)
6
5
4
3
2
1
0
-1
12:00:00 AM
12:36:00 AM
1:12:00 AM
1:48:00 AM
2:24:00 AM
3:00:00 AM
3:36:00 AM
4:12:00 AM
Time
Post-development
Pre-development


Dam 5-1 - 6 Hour
Outflow (m^3/s)
2.5
2
1.5
1
0.5
0
-0.5
12:00:00 AM
2:24:00 AM
4:48:00 AM
7:12:00 AM
9:36:00 AM
12:00:00 PM
Time
Post-development
Pre-development

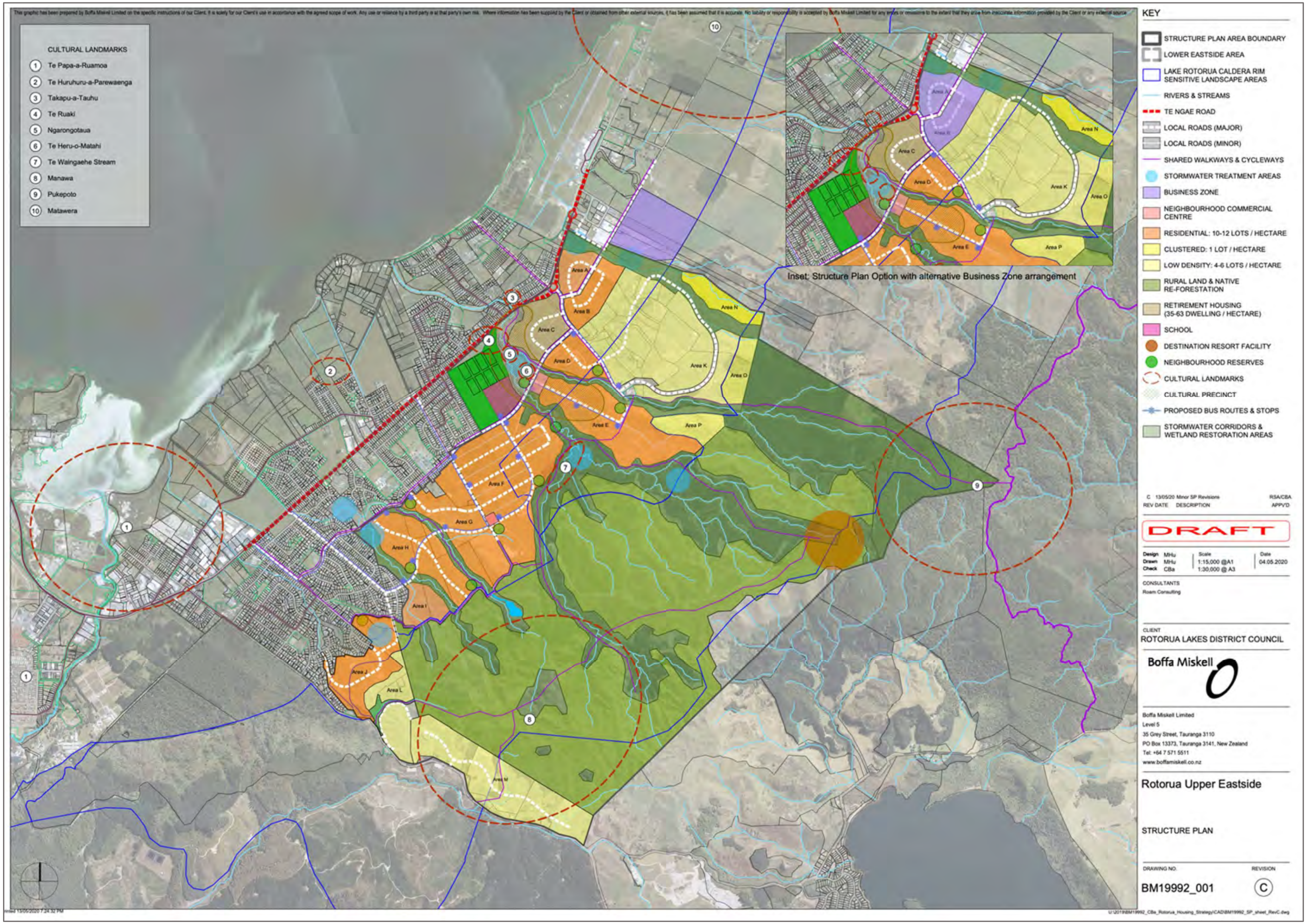
## Dam 5-2


Dam 5-2 - 1 Hour
Outflow (m^3/s)
0
2
4
6
8
10
12
14
16
18
20
12:00:00 AM
12:36:00 AM
1:12:00 AM
1:48:00 AM
2:24:00 AM
3:00:00 AM
3:36:00 AM
4:12:00 AM
Time
Post-development
Pre-Development


Dam 5-2- 6 Hour
Outflow (m^3/s)
0
1
2
3
4
5
6
7
8
9
12:00:00 AM
2:24:00 AM
4:48:00 AM
7:12:00 AM
9:36:00 AM
12:00:00 PM
Time
Post-development
Pre-Development

# Appendix C: Proposed development plans

Draft



CULTURAL LANDMARKS
1 Te Papa-a-Ruamoa
2 Te Huruhuru-a-Parewaenga
3 Takapu-a-Tauhu
4 Te Ruaki
5 Ngarongotaua
6 Te Heru-o-Matahi
7 Te Waingaehe Stream
8 Manawa
9 Pukepoto
10 Matawera
Inset: Structure Plan Option with alternative Business Zone arrangement
KEY
STRUCTURE PLAN AREA BOUNDARY
LOWER EASTSIDE AREA
LAKE ROTORUA CALDERA RIM SENSITIVE LANDSCAPE AREAS
RIVERS & STREAMS
TE NGAE ROAD
LOCAL ROADS (MAJOR)
LOCAL ROADS (MINOR)
SHARED WALKWAYS & CYCLEWAYS
STORMWATER TREATMENT AREAS
BUSINESS ZONE
NEIGHBOURHOOD COMMERCIAL CENTRE
RESIDENTIAL: 10-12 LOTS / HECTARE
CLUSTERED: 1 LOT / HECTARE
LOW DENSITY: 4-6 LOTS / HECTARE
RURAL LAND & NATIVE RE-FORESTATION
RETIREMENT HOUSING (35-63 DWELLING / HECTARE)
SCHOOL
DESTINATION RESORT FACILITY
NEIGHBOURHOOD RESERVES
CULTURAL LANDMARKS
CULTURAL PRECINCT
PROPOSED BUS ROUTES & STOPS
STORMWATER CORRIDORS & WETLAND RESTORATION AREAS
C 13/05/20 Minor SP Revisions RSA/CBA
REV DATE DESCRIPTION APPV'D
DRAFT
Design MHu
Drawn MHu
Check CBa
Scale 1:15,000 @A1 1:30,000 @ A3
Date 04.05.2020
CONSULTANTS
Roam Consulting
CLIENT
ROTORUA LAKES DISTRICT COUNCIL
Boffa Miskell
Boffa Miskell Limited
Level 5
35 Grey Street, Tauranga 3110
PO Box 13373, Tauranga 3141, New Zealand
Tel: +64 7 571 5511
www.boffamiskell.co.nz
Rotorua Upper Eastside
STRUCTURE PLAN
DRAWING NO.
BM19992_001
REVISION
C

KEY

- Lower Terrace
- Mid-site Escarpment
- Upper Terrace
- Upper Escarpment
- Pukehangi Southern Slopes
- Escarpment Transition Area 1
- Escarpment Transition Area 2
- Extent of Areas A and B
- RL 385 Contour Line
- Archeological site
- Water Reservoir
- Pukehangi Pa

**Indicative Structure Plan Features:**

- Stormwater / Recreation Areas
- Medium Density Residential Development Area
- Community, Retail & Commercial Area
- Primary Roads
- Additional Primary Road Connection
- Overland Flow Paths (Indicative width 10-15m to accommodate 0.2% AEP event )
- Walkway
- Future Walkway (Not formed until and unless legal access over Lot 36 DP 367989 and Lot DP 367989 is created)
- Cycleway
- BS Bus Stop


Boffa Miskell
www.boffamiskell.co.nz

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0 150 300 m

1:10,000 @ A3

*Data Sources:* Land Information New Zealand (LINZ)
*Projection:* NZGD2000 N.Z. Transverse Mercator

**Pukehangi Heights Development Area**

## Proposed Structure Plan

**Date:** 21.04.2020 | **Revision:** K

Plan prepared for RLC by Boffa Miskell Limited

*Project Manager:* **craig.batchelar@boffamiskell.co.nz** | *Drawn:* TKe | *Checked:* MHu


**OPTION 3.0 "TRADE CENTRE"**

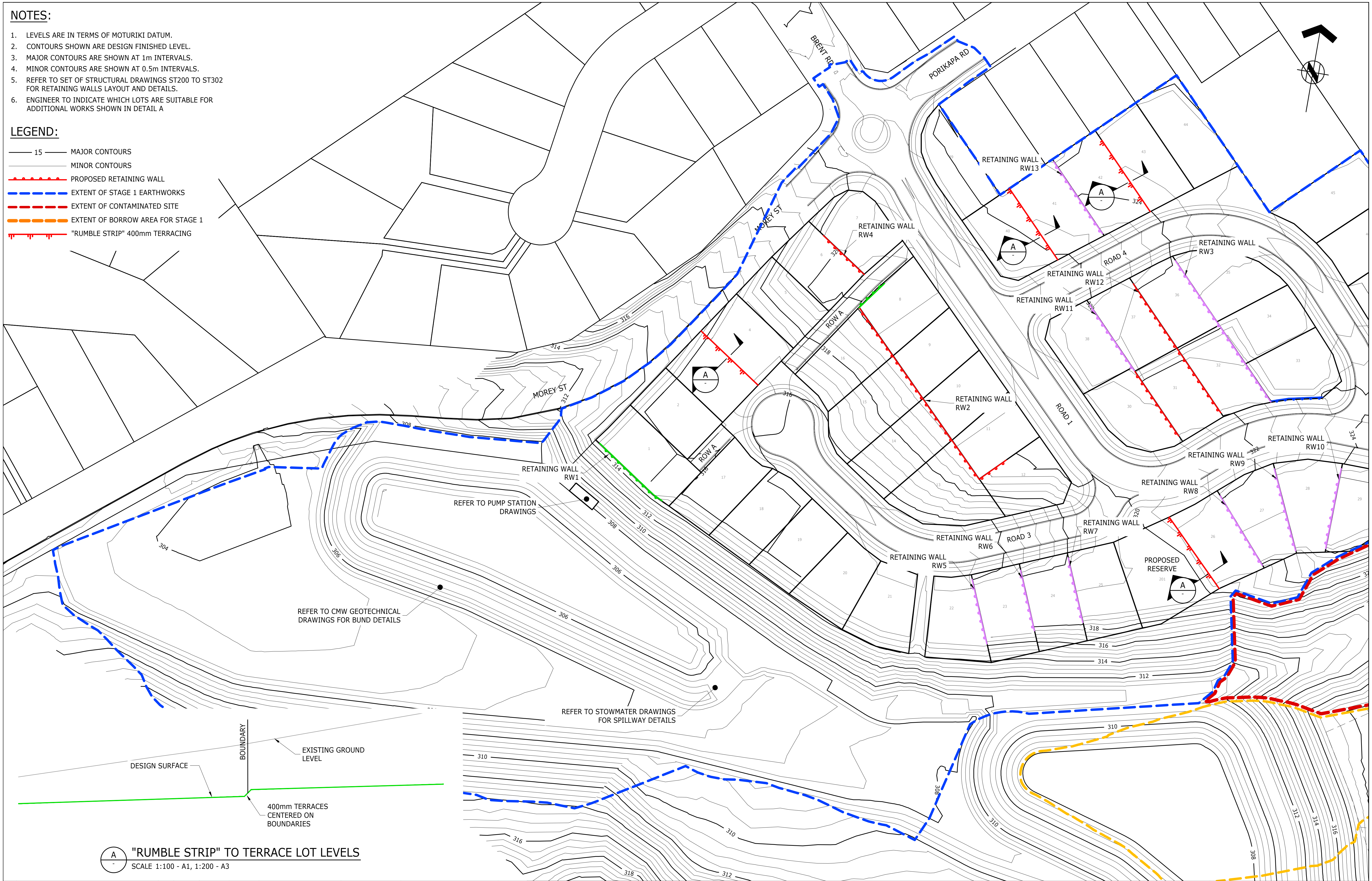
OWHATIURA
TE NGAE RD. ROTORUA.
1:2500 @ A4.

## NOTES:

1. LEVELS ARE IN TERMS OF MOTURIKI DATUM.
2. CONTOURS SHOWN ARE DESIGN FINISHED LEVEL.
3. MAJOR CONTOURS ARE SHOWN AT 1m INTERVALS.
4. MINOR CONTOURS ARE SHOWN AT 0.5m INTERVALS.
5. REFER TO SET OF STRUCTURAL DRAWINGS ST200 TO ST302 FOR RETAINING WALLS LAYOUT AND DETAILS.
6. ENGINEER TO INDICATE WHICH LOTS ARE SUITABLE FOR ADDITIONAL WORKS SHOWN IN DETAIL A

## LEGEND:

- 15 — MAJOR CONTOURS
- MINOR CONTOURS
- PROPOSED RETAINING WALL
- EXTENT OF STAGE 1 EARTHWORKS
- EXTENT OF CONTAMINATED SITE
- EXTENT OF BORROW AREA FOR STAGE 1
- "RUMBLE STRIP" 400mm TERRACING




A "RUMBLE STRIP" TO TERRACE LOT LEVELS

SCALE 1:100 - A1, 1:200 - A3

| REF | REVISIONS | BY | DATE |
|---|---|---|---|
| C | REVISED CONTOURS AROUND POND | GPR | 07.04.20 |
| B | REVISED CONTOURS & ADDITIONAL RETAINING WALLS | GPR | 30.01.20 |
| A | ISSUED FOR CONSTRUCTION | GPR | 22.11.19 |

| | | | |
|---|---|---|---|
| ORIGINATOR: PSJ | DATE: 13.06.19 | SIGNED: | PLOT BY: RJM |
| DRAWN: NXS | DATE: 13.06.19 | SIGNED: | PLOT DATE: 07.04.20 |
| CHECKED: GPR | DATE: 07.04.20 | SIGNED: | SURVEY BY: |
| APPROVED: RJM | DATE: 07.04.20 | SIGNED: | SURVEY DATE: |

ACENZ ASSOCIATION OF CONSULTING ENGINEERS NEW ZEALAND

ISO 9001 QUALITY ASSURED

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HG
TAURANGA OFFICE
LEVEL 1, 60 SPRING STREET
TAURANGA 3110
T +64 7 578 0023
W www.harrisongrierson.com

PROJECT: NGATI WHAKAUE BRENT BLOCK ROTORUA

TITLE: STAGE 1 CIVILS DESIGN CONTOURS SHEET 1

ISSUE STATUS: CONSTRUCTION

PROJECT No: 1530-144247-03

SCALES: 1: 625 - A1, 1: 1250 - A3

A1

DRAWING No: 144247-03-211

REV: C


cheal

www.cheal.co.nz

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Client Logo

Client Ref

| Rev | Date | Amendment | By | Chk | App |
|---|---|---|---|---|---|
| A | 11/11/19 | Information | CC | ND | AGM |

Project Title

Tuapuku Tautoko LP
Diamond Street
Rotorua

Drawing Title

Concept Plan
Proposed Subdivision of Lot 1
DPS 64449 and Lot 1 DPS 64748

| | | | |
|---|---|---|---|
| Surveyed | | | |
| Designed | A Moss | 8/11/19 | AGM |
| Drawn | C Chen | 8/11/19 | CC |
| Checked | N Davis | 11/11/19 | ND |
| Approved | A Moss | 11/11/19 | AGM |

Status INFORMATION

| Scale | | | |
|---|---|---|---|
| A1 | 1000 | | A1 |
| A3 | 2000 | | |

Drawing Number 19522- SW 1

Rev

AutoCAD


cheal

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| Rev | Date | Amendment | By | Chk | App |
|---|---|---|---|---|---|
| A | 20/03/20 | Information | AGM | JH | ND |

Project Title

Tuapuku Tautoko LP
Diamond Street
Rotorua

Drawing Title

Concept Plan Proposed Subdivision of Lot 1 DPS 64449, Lot 1 DPS 64748 & Lot 4 DP 541922

| | | | |
|---|---|---|---|
| Surveyed | | | |
| Designed | A Moss | 1803/20 | AGM |
| Drawn | A Moss | 18/03/20 | AGM |
| Checked | J Hallam | 18/03/20 | JH |
| Approved | N Davies | 20/03/20 | ND |

Status INFORMATION

Scale A1 1000 | A1
A3 2000

Drawing Number 19522- SW 2 | Rev

AutoCAD

# Appendix D: Cost estimate breakdowns

Draft

## Rotorua Stormwater Masterplan - Rough order cost estimate summary

| | Construction cost | Contingency | Professional fees (design, consenting, construction monitoring) | Property purchase | Total (Presented as a range -10% to +30% of total) |
|---|---|---|---|---|---|
| **Option 1/2A** | | | | | |
| Pond 1-1 | $ 3,950,000 | $ 1,580,000 | $ 580,000 | $ - | $ 5,500,000 to $ 7,900,000 |
| Pond 2-1 | $ 2,430,000 | $ 970,000 | $ 360,000 | $ - | $ 3,400,000 to $ 4,900,000 |
| Pond 2-2 | $ 570,000 | $ 230,000 | $ 80,000 | $ - | $ 800,000 to $ 1,100,000 |
| Pond 2-3 | $ 1,520,000 | $ 610,000 | $ 220,000 | $ - | $ 2,100,000 to $ 3,100,000 |
| Dam 2-4 | $ 3,280,000 | $ 1,310,000 | $ 960,000 | $ - | $ 5,000,000 to $ 7,200,000 |
| Dam 2-5 | $ 1,650,000 | $ 660,000 | $ 490,000 | $ - | $ 2,500,000 to $ 3,600,000 |
| Pond 2-6 | $ 840,000 | $ 340,000 | $ 120,000 | $ - | $ 1,200,000 to $ 1,700,000 |
| Pond 2-7 | $ 890,000 | $ 360,000 | $ 130,000 | $ - | $ 1,200,000 to $ 1,800,000 |
| Pond 2-8 | $ 3,380,000 | $ 1,350,000 | $ 500,000 | $ - | $ 4,700,000 to $ 6,800,000 |
| Dam 2-9 | $ 1,450,000 | $ 580,000 | $ 430,000 | $ - | $ 2,200,000 to $ 3,200,000 |
| **Option 1/2A total** | | | | | **$ 28,600,000 to $ 41,300,000** |
| **Option 4A** | | | | | |
| Owhatiura Drain East | $ 660,000 | $ 260,000 | $ 210,000 | $ - | $ 1,000,000 to $ 1,500,000 |
| Vaughan Rd culvert | $ 480,000 | $ 190,000 | $ 140,000 | $ - | $ 700,000 to $ 1,100,000 |
| Vaughan Road Drain | $ 670,000 | $ 270,000 | $ 210,000 | $ - | $ 1,000,000 to $ 1,500,000 |
| Dam 4-1 | $ 1,760,000 | $ 700,000 | $ 520,000 | $ - | $ 2,700,000 to $ 3,900,000 |
| Dam 4-2 | $ 1,670,000 | $ 670,000 | $ 490,000 | $ - | $ 2,500,000 to $ 3,700,000 |
| Morey St Culverts | $ 180,000 | $ 70,000 | $ 50,000 | $ - | $ 300,000 to $ 400,000 |
| **Option 4A total** | | | | | **$ 8,200,000 to $ 12,100,000** |
| **Option 4B** | | | | | |
| Owhatiura Drain East | $ 1,060,000 | $ 420,000 | $ 340,000 | $ - | $ 1,600,000 to $ 2,400,000 |
| Vaughan Rd culvert | $ 480,000 | $ 190,000 | $ 140,000 | $ - | $ 700,000 to $ 1,100,000 |
| Te Ngae Rd Culvert | $ 600,000 | $ 240,000 | $ 160,000 | $ - | $ 900,000 to $ 1,300,000 |
| Melrose Ave Drain | $ 1,240,000 | $ 490,000 | $ 400,000 | $ 3,500,000 | $ 5,100,000 to $ 7,300,000 |
| Vaughan Road Drain | $ 670,000 | $ 270,000 | $ 210,000 | $ - | $ 1,000,000 to $ 1,500,000 |
| Morey St Culverts | $ 540,000 | $ 220,000 | $ 140,000 | $ - | $ 800,000 to $ 1,200,000 |
| **Option 4B total** | | | | | **$ 10,100,000 to $ 14,800,000** |
| **Option 5A** | | | | | |
| Dam 5-1 | $ 990,000 | $ 400,000 | $ 290,000 | $ - | $ 1,500,000 to $ 2,200,000 |
| Dam 5-2 | $ 1,400,000 | $ 560,000 | $ 410,000 | $ - | $ 2,100,000 to $ 3,100,000 |
| **Option 5A total** | | | | | **$ 3,600,000 to $ 5,300,000** |
| **Option 6** | | | | | |
| Wright Park Dam | $ 3,760,000 | $ 1,500,000 | $ 1,100,000 | $ - | $ 5,700,000 to $ 8,300,000 |
| Linton Park West (Edmund Rd) | $ 2,020,000 | $ 810,000 | $ 590,000 | $ - | $ 3,100,000 to $ 4,400,000 |
| Linton Park East | $ 6,120,000 | $ 2,450,000 | $ 1,800,000 | $ - | $ 9,300,000 to $ 13,500,000 |
| Mangakakahi Dam | $ 6,380,000 | $ 2,550,000 | $ 1,870,000 | $ - | $ 9,700,000 to $ 14,000,000 |
| **Option 6 total** | | | | | **$ 27,800,000 to $ 40,200,000** |

**ROUGH ORDER COST ESTIMATE - Utuhina dams**

Project: Rotorua Stormwater Masterplan — Prepared: LIMI 3/12/2020
Description: High level cost estimates - Optioneering — Checked: JTIH 25/01/2021
Project No: 1010988.1000
File location: \\ttgroup.local\files\TGAProjects\1010988\1010988.1000\WorkingMaterial\08 Costing\[20210127 High Level Cost Estimate. limi.xlsx]Eastern Dams

| | Wright Park Dam | | Linton Park West (Edmund Rd) | | Linton Park East | | Mangakakahi Dam | |
|---|---|---|---|---|---|---|---|---|
| Haul road length: | 330 | m | n/a | m | n/a | m | 240 | m |
| Foundation area: | 3750 | sq.m | 2740 | sq.m | 5000 | sq.m | 6900 | sq.m |
| Foundation area +1.5m buffer: | 4270 | sq.m | 3130 | sq.m | 5700 | sq.m | 9180 | sq.m |
| Culvert through dam: | 60 | m | 35 | m | 15 | m | 100 | m |
| Chimney drain area: | 550 | sq.m | 370 | sq.m | 570 | sq.m | 1040 | sq.m |
| Toe blanket area: | 1550 | sq.m | n/a | sq.m | n/a | sq.m | 3550 | sq.m |
| Toe drain: | 105 | m | 260 | m | 475 | m | 240 | m |
| Fill volume: | 11300 | cu.m | 3020 | cu.m | 3920 | cu.m | 37000 | cu.m |
| Fill volume + 1.5m buffer area (1m below): | 15570 | cu.m | 6150 | cu.m | 9620 | cu.m | 46180 | cu.m |
| Impermeable fill: | | | 3960 | cu.m | | | | |

| Item # | Description | Unit | Rate | Quantity | Unit | Amount | Quantity | Unit | Amount | Quantity | Unit | Amount | Quantity | Unit | Amount |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Site Preparation** | | | | | | | | | | | | | | | |
| 1 | Build haul road for access | m | $ 110.00 | 330 | m | $ 36,300 | | | | | | | 240 | m | $ 26,400 |
| 2 | Remove large vegetation | sq.m | $ 25.00 | 8300 | sq.m | $ 207,500 | 1200 | sq.m | $ 30,000 | 11200 | sq.m | $ 280,000 | 3430 | sq.m | $ 85,750 |
| 3 | Foundation prep - strip topsoil and grass, assume sub-excavate by average 1m under entire footprint, dispose to landscape fill within development area (maybe on dam batters to create flatter than 1:3 slopes) | cu.m | $ 20.00 | 4270 | cu.m | $ 85,400 | 3130 | cu.m | $ 62,600 | 5700 | cu.m | $ 114,000 | 9180 | cu.m | $ 183,600 |
| **Primary spillway construction** | | | | | | | | | | | | | | | |
| 4 | Supply and install 2.5 x2.5 m box culvert through dam. | m | $ 6,000.00 | 60 | m | $ 360,000 | 35 | m | $ 210,000 | 15 | m | $ 90,000 | 100 | m | $ 600,000 |
| 5 | Supply and install precast wingwall and rip rap apron at upstream end of pipe | LS | $ 20,000.00 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 |
| 6 | Supply and install precast manhole complete with scruffy dome | LS | $ 20,000.00 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 |
| 7 | Supply and install custom wingwall to attach to 2.5 x 2.5 m box culvert and rip rap apron at downstream end of pipe | LS | $ 50,000.00 | 1 | LS | $ 50,000 | 1 | LS | $ 50,000 | 1 | LS | $ 50,000 | 1 | LS | $ 50,000 |
| 8 | Allow for reinforced concrete encasing upstream half of the box culvert. | cu.m | $ 1,000.00 | 70 | cu.m | $ 70,000 | 40 | cu.m | $ 40,000 | 20 | cu.m | $ 20,000 | 110 | cu.m | $ 110,000 |
| **Auxiliary spillway construction** | | | | | | | | | | | | | | | |
| 9 | Supply materials and construct reinforced grass lined spillway | LS | See right -> | 1 | LS | $ 250,000 | 1 | LS | $ 700,000 | 1 | LS | $ 600,000 | 1 | LS | $ 1,300,000 |
| **Dam construction** | | | | | | | | | | | | | | | |
| | **Supply and place imported graded sand filter** | | | | | | | | | | | | | | |
| 10 | Filter around downstream half of box culvert | cu.m | $ 100.00 | 70 | cu.m | $ 7,000 | 40 | cu.m | $ 4,000 | 20 | cu.m | $ 2,000 | 110 | cu.m | $ 11,000 |
| 11 | Chimney drain | cu.m | $ 100.00 | 330 | cu.m | $ 33,000 | 222 | cu.m | $ 22,200 | 342 | cu.m | $ 34,200 | 624 | cu.m | $ 62,400 |
| 12 | Blanket drain | cu.m | $ 100.00 | 930 | cu.m | $ 93,000 | | | | | | | 2130 | cu.m | $ 213,000 |
| 13 | Install toe drain at toe of dam. 110 dia. HIWAY drain | m | $ 150.00 | 105 | m | $ 15,750 | 260 | m | $ 39,000 | 475 | m | $ 71,250 | 240 | m | $ 36,000 |
| 14 | Bulk earthfill for dam including conditioning, testing, trimming (impermeable fill) | cu.m | $ 55.00 | 7785 | cu.m | $ 428,175 | 3960 | cu.m | $ 217,800 | 9620 | cu.m | $ 529,100 | 23090 | cu.m | $ 1,269,950 |
| 15 | Cut to fill (site won fill) downstream half of dam | cu.m | $ 30.00 | 7785 | cu.m | $ 233,550 | 2190 | cu.m | $ 65,700 | | | | 23090 | cu.m | $ 692,700 |
| **Reinstatement** | | | | | | | | | | | | | | | |
| 16 | Topsoil and grass the dam surface and any other cut areas | sq.m | $ 5.00 | 7420 | sq.m | $ 37,100 | 3130 | sq.m | $ 15,650 | 51000 | sq.m | $ 255,000 | 9180 | sq.m | $ 45,900 |
| **Excavation of reservoir area** | | | | | | | | | | | | | | | |
| 17 | Bulk earthworks - cut - separate to dam structure. Transported and disposed off site or at a location on site. | cu.m | $ 50.00 | 16800 | cu.m | $ 840,000 | | | | 49000 | cu.m | $ 2,450,000 | | | |
| | **Sub Total - base construction cost** | | | | | **$ 2,786,775** | | | **$ 1,496,950** | | | **$ 4,535,550** | | | **$ 4,726,700** |
| 18 | Contractor P&G | % | 25% | | % | $ 696,694 | | % | $ 374,238 | | % | $ 1,133,888 | | % | $ 1,181,675 |
| 19 | Unscheduled & minor items | % | 10% | | % | $ 278,678 | | % | $ 149,695 | | % | $ 453,555 | | % | $ 472,670 |
| | **Construction subtotal** | | | | | **$ 3,762,146** | | | **$ 2,020,883** | | | **$ 6,122,993** | | | **$ 6,381,045** |
| 20 | Contingency | % | 40% | | % | $ 1,504,859 | | % | $ 808,353 | | % | $ 2,449,197 | | % | $ 2,552,418 |
| 21 | Construction monitoring | % | 6% | | % | $ 225,729 | | % | $ 121,253 | | % | $ 367,380 | | % | $ 382,863 |
| | **Construction total** | | | | | **$ 5,492,734** | | | **$ 2,950,488** | | | **$ 8,939,569** | | | **$ 9,316,326** |
| 22 | Planning/consents/consultation/negotiating access | % | 4% | | % | $ 219,709 | | % | $ 118,020 | | % | $ 357,583 | | % | $ 372,653 |
| 23 | Design fees | % | 12% | | % | $ 659,128 | | % | $ 354,059 | | % | $ 1,072,748 | | % | $ 1,117,959 |
| | **TOTAL -10% Contingency** | | | **Wright Park Dam** | | **$ 6,370,000** | **ark West (Edmund Rd)** | | **$ 3,420,000** | **Linton Park East** | | **$ 10,370,000** | **Mangakakahi Dam** | | **$ 10,810,000** |

**Exclusions**

Land purchase (except where specified)
Estimate does not attempt to make any allowance for potential effects of COVID-19.
Government taxes
Rotorua Lakes District Council Project Management
Building consent
Public relations
Insurances during construction & commissioning
Project financing & other admin costs, incl legal fees
Bid construction cost volatility due to such reasons as: changes in costs fuel, and any other commodity subject fluctuations or commodity world demand;
Other costs not specifically referred to above

## ROUGH ORDER COST ESTIMATE - Eastern dams

Project: Rotorua Stormwater Masterplan
Description: High level cost estimates - Optioneering
Project No: 1010988.1000
File location: \\ttgroup.local\files\TGAProjects\1010988\1010988.1000\WorkingMaterial\08 Costing\[20210127 High Level Cost Estimate. limi.xlsx]Eastern Dams

Prepared: LIMI 20/01/2021
Checked: JTIH 25/01/2021

| | Dam 4-1 | | Dam 4-2 | | Dam 5-1 | | Dam 5-2 | | Dam 2-4 | | Dam 2-5 | | Dam 2-9 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Haul road length: | 0 | m | 290 | m | 0 | m | 140 | m | 0 | m | 120 | m | 380 | m |
| Foundation area: | 2600 | sq.m | 2610 | sq.m | 1320 | sq.m | 1470 | sq.m | 4380 | sq.m | 1975 | sq.m | 2456 | sq.m |
| Foundation area +1.5m buffer: | 3150 | sq.m | 2970 | sq.m | 1650 | sq.m | 1790 | sq.m | 4900 | sq.m | 2300 | sq.m | 2940 | sq.m |
| Culvert through dam: | 50 | m | 50 | m | 30 | m | 50 | m | 90 | m | 60 | m | 40 | m |
| Chimney drain area: | 400 | sq.m | 370 | sq.m | 160 | sq.m | 170 | sq.m | 710 | sq.m | 250 | sq.m | 150 | sq.m |
| Toe blanket area: | 1310 | sq.m | 1310 | sq.m | 600 | sq.m | 600 | sq.m | 2400 | sq.m | 1050 | sq.m | 600 | sq.m |
| Toe drain: | 140 | m | 100 | m | 50 | m | 60 | m | 100 | m | 65 | m | 50 | m |
| Fill area: | 4590 | cu.m | 7760 | cu.m | 3460 | cu.m | 3460 | cu.m | 22000 | cu.m | 5660 | cu.m | 2260 | cu.m |
| Fill area + 1.5m buffer area (1m below): | 7740 | cu.m | 10730 | cu.m | 5110 | cu.m | 5250 | cu.m | 26900 | cu.m | 7960 | cu.m | 5200 | cu.m |
| Length of spillway: | | m | | m | | m | | m | | m | | m | | m |

| Item # | Description | Unit | Rate | Dam 4-1 Quantity | Dam 4-1 Unit | Dam 4-1 Amount | Dam 4-2 Quantity | Dam 4-2 Unit | Dam 4-2 Amount | Dam 5-1 Quantity | Dam 5-1 Unit | Dam 5-1 Amount |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Site Preparation** | | | | | | | | | | | | |
| 1 | Build haul road for access | m | $ 110.00 | | | | 290 | m | $ 31,900 | | | |
| 2 | Remove large vegetation | sq.m | $ 25.00 | | | | | | | 270 | sq.m | $ 6,750 |
| 3 | Foundation prep - strip topsoil and grass, assume sub-excavate by average 1m under entire footprint, dispose to landscape fill within development area (maybe on dam batters to create flatter than 1:3 slopes) | cu.m | $ 20.00 | 3150 | cu.m | $ 63,000 | 2970 | cu.m | $ 59,400 | 1650 | cu.m | $ 33,000 |
| **Primary spillway construction** | | | | | | | | | | | | |
| 4 | Supply and install 2.5 x2.5 m box culvert through dam. | m | $ 6,000.00 | 50 | m | $ 300,000 | 50 | m | $ 300,000 | 30 | m | $ 180,000 |
| 5 | Supply and install precast wingwall and rip rap apron at upstream end of pipe | LS | $ 20,000.00 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 |
| 6 | Supply and install precast manhole complete with scruffy dome | LS | $ 20,000.00 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 |
| 7 | Supply and install custom wingwall to attach to 2.5 x 2.5 m box culvert and rip rap apron at downstream end of pipe | LS | $ 50,000.00 | 1 | LS | $ 50,000 | 1 | LS | $ 50,000 | 1 | LS | $ 50,000 |
| 8 | Allow for reinforced concrete encasing upstream half of the box culvert. | cu.m | $ 1,000.00 | 60 | cu.m | $ 60,000 | 60 | cu.m | $ 60,000 | 40 | cu.m | $ 40,000 |
| 9 | Grassed overflow spillway | cu.m | $ 50.00 | 3000 | cu.m | $ 150,000 | | | | | | |
| **Auxiliary spillway construction** | | | | | | | | | | | | |
| 10 | Supply materials and construct reinforced grass lined spillway | LS | See right -> | 1 | LS | $ 150,000 | 1 | LS | $ 100,000 | 1 | LS | $ 100,000 |
| **Dam construction** | | | | | | | | | | | | |
| | **Supply and place imported graded sand filter** | | | | | | | | | | | |
| 11 | Filter around downstream half of pipe | cu.m | $ 100.00 | 60 | cu.m | $ 6,000 | 60 | cu.m | $ 6,000 | 40 | cu.m | $ 4,000 |
| 12 | Chimney drain | cu.m | $ 100.00 | 240 | cu.m | $ 24,000 | 222 | cu.m | $ 22,200 | 96 | cu.m | $ 9,600 |
| 13 | Blanket drain | cu.m | $ 100.00 | 786 | cu.m | $ 78,600 | 786 | cu.m | $ 78,600 | 360 | cu.m | $ 36,000 |
| 14 | Install toe drain at toe of dam. 110 dia. HIWAY drain | m | $ 150.00 | 140 | m | $ 21,000 | 100 | m | $ 15,000 | 50 | m | $ 7,500 |
| 15 | Bulk earthfill for dam including conditioning, testing, trimming (impermeable fill) | cu.m | $ 55.00 | 3870 | cu.m | $ 212,850 | 5365 | cu.m | $ 295,075 | 2555 | cu.m | $ 140,525 |
| 16 | Cut to fill (site won fill) downstream half of dam | cu.m | $ 30.00 | 3870 | cu.m | $ 116,100 | 5365 | cu.m | $ 160,950 | 2555 | cu.m | $ 76,650 |
| **Reinstatement** | | | | | | | | | | | | |
| 17 | Topsoil and grass the dam surface and any other cut areas | sq.m | $ 5.00 | 6150 | sq.m | $ 30,750 | 2970 | sq.m | $ 14,850 | 1650 | sq.m | $ 8,250 |
| **Excavation of reservoir area** | | | | | | | | | | | | |
| 18 | Bulk earthworks - cut - separate to dam structure. Transported and disposed off site or at a location on site. | cu.m | $ 50.00 | | | | | | | | | |
| | **Sub Total - base construction cost** | | | | | **$ 1,302,300** | | | **$ 1,233,975** | | | **$ 732,275** |
| 19 | Contractor P&G | % | 25% | | % | $ 325,575 | | % | $ 308,494 | | % | $ 183,069 |
| 20 | Unscheduled & minor items | % | 10% | | % | $ 130,230 | | % | $ 123,398 | | % | $ 73,228 |
| | **Construction subtotal** | | | | | **$ 1,758,105** | | | **$ 1,665,866** | | | **$ 988,571** |
| 21 | Contingency | % | 40% | | % | $ 703,242 | | % | $ 666,347 | | % | $ 395,429 |
| 22 | Construction monitoring | % | 6% | | % | $ 105,486 | | % | $ 99,952 | | % | $ 59,314 |
| | **Construction total** | | | | | **$ 2,566,833** | | | **$ 2,432,165** | | | **$ 1,443,314** |
| 23 | Planning/consents/consultation/negotiating access | % | 4% | | % | $ 102,673 | | % | $ 97,287 | | % | $ 57,733 |
| 24 | Design fees | % | 12% | | % | $ 308,020 | | % | $ 291,860 | | % | $ 173,198 |
| | **TOTAL** | | | | **Dam 4-1** | **$ 2,980,000** | | **Dam 4-2** | **$ 2,820,000** | | **Dam 5-1** | **$ 1,670,000** |

| Item # | Description | Dam 5-2 Quantity | Dam 5-2 Unit | Dam 5-2 Amount | Dam 2-4 Quantity | Dam 2-4 Unit | Dam 2-4 Amount | Dam 2-5 Quantity | Dam 2-5 Unit | Dam 2-5 Amount | Dam 2-9 Quantity | Dam 2-9 Unit | Dam 2-9 Amount |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Site Preparation** | | | | | | | | | | | | | |
| 1 | Build haul road for access | 140 | m | $ 15,400 | | | | 120 | m | $ 13,200 | 380 | m | $ 41,800 |
| 2 | Remove large vegetation | 1820 | sq.m | $ 45,500 | 1000 | sq.m | $ 25,000 | | | | | | |
| 3 | Foundation prep | 1790 | cu.m | $ 35,800 | 4900 | cu.m | $ 98,000 | 2300 | cu.m | $ 46,000 | 2940 | cu.m | $ 58,800 |
| **Primary spillway construction** | | | | | | | | | | | | | |
| 4 | Supply and install 2.5 x2.5 m box culvert through dam. | 50 | m | $ 300,000 | 90 | m | $ 540,000 | 60 | m | $ 360,000 | 40 | m | $ 240,000 |
| 5 | Supply and install precast wingwall and rip rap apron at upstream end of pipe | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 |
| 6 | Supply and install precast manhole complete with scruffy dome | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 | 1 | LS | $ 20,000 |
| 7 | Supply and install custom wingwall | 1 | LS | $ 50,000 | 1 | LS | $ 50,000 | 1 | LS | $ 50,000 | 1 | LS | $ 50,000 |
| 8 | Allow for reinforced concrete encasing upstream half of the box culvert. | 60 | cu.m | $ 60,000 | 100 | cu.m | $ 100,000 | 70 | cu.m | $ 70,000 | 50 | cu.m | $ 50,000 |
| 9 | Grassed overflow spillway | | | | | | | | | | | | |
| **Auxiliary spillway construction** | | | | | | | | | | | | | |
| 10 | Supply materials and construct reinforced grass lined spillway | 1 | LS | $ 200,000 | 1 | LS | $ 200,000 | 1 | LS | $ 200,000 | 1 | LS | $ 300,000 |
| **Dam construction** | | | | | | | | | | | | | |
| | **Supply and place imported graded sand filter** | | | | | | | | | | | | |
| 11 | Filter around downstream half of pipe | 60 | cu.m | $ 6,000 | 100 | cu.m | $ 10,000 | 70 | cu.m | $ 7,000 | 50 | cu.m | $ 5,000 |
| 12 | Chimney drain | 102 | cu.m | $ 10,200 | 426 | cu.m | $ 42,600 | 150 | cu.m | $ 15,000 | 90 | cu.m | $ 9,000 |
| 13 | Blanket drain | 360 | cu.m | $ 36,000 | 1440 | cu.m | $ 144,000 | 630 | cu.m | $ 63,000 | 360 | cu.m | $ 36,000 |
| 14 | Install toe drain at toe of dam. 110 dia. HIWAY drain | 60 | m | $ 9,000 | 100 | m | $ 15,000 | 65 | m | $ 9,750 | 50 | m | $ 7,500 |
| 15 | Bulk earthfill for dam including conditioning, testing, trimming (impermeable fill) | 2625 | cu.m | $ 144,375 | 13450 | cu.m | $ 739,750 | 3980 | cu.m | $ 218,900 | 2600 | cu.m | $ 143,000 |
| 16 | Cut to fill (site won fill) downstream half of dam | 2625 | cu.m | $ 78,750 | 13450 | cu.m | $ 403,500 | 3980 | cu.m | $ 119,400 | 2600 | cu.m | $ 78,000 |
| **Reinstatement** | | | | | | | | | | | | | |
| 17 | Topsoil and grass the dam surface and any other cut areas | 1820 | sq.m | $ 9,100 | 4900 | sq.m | $ 24,500 | 2300 | sq.m | $ 11,500 | 2940 | sq.m | $ 14,700 |
| **Excavation of reservoir area** | | | | | | | | | | | | | |
| 18 | Bulk earthworks - cut - separate to dam structure. | | | | | | | | | | | | |
| | **Sub Total - base construction cost** | | | **$ 1,040,125** | | | **$ 2,432,350** | | | **$ 1,223,750** | | | **$ 1,073,800** |
| 19 | Contractor P&G | | % | $ 260,031 | | % | $ 608,088 | | % | $ 305,938 | | % | $ 268,450 |
| 20 | Unscheduled & minor items | | % | $ 104,013 | | % | $ 243,235 | | % | $ 122,375 | | % | $ 107,380 |
| | **Construction subtotal** | | | **$ 1,404,169** | | | **$ 3,283,673** | | | **$ 1,652,063** | | | **$ 1,449,630** |
| 21 | Contingency | | % | $ 561,668 | | % | $ 1,313,469 | | % | $ 660,825 | | % | $ 579,852 |
| 22 | Construction monitoring | | % | $ 84,250 | | % | $ 197,020 | | % | $ 99,124 | | % | $ 86,978 |
| | **Construction total** | | | **$ 2,050,086** | | | **$ 4,794,162** | | | **$ 2,412,011** | | | **$ 2,116,460** |
| 23 | Planning/consents/consultation/negotiating access | | % | $ 82,003 | | % | $ 191,766 | | % | $ 96,480 | | % | $ 84,658 |
| 24 | Design fees | | % | $ 246,010 | | % | $ 575,299 | | % | $ 289,441 | | % | $ 253,975 |
| | **TOTAL** | | **Dam 5-2** | **$ 2,380,000** | | **Dam 2-4** | **$ 5,560,000** | | **Dam 2-5** | **$ 2,800,000** | | **Dam 2-9** | **$ 2,460,000** |

**Assumptions/clarifications:**

No allowance made for realignment of accessway to 15 Link Road for Dam 5-1

**Exclusions**

- Land purchase (except where specified)
- Estimate does not attempt to make any allowance for potential effects of COVID-19.
- Government taxes
- Rotorua Lakes District Council Project Management
- Building consent
- Public relations
- Insurances during construction & commissioning
- Project financing & other admin costs, incl legal fees
- Bid construction cost volatility due to such reasons as: changes in costs fuel, and any other commodity subject fluctuations or commodity world demand;
- Other costs not specifically referred to above

**ROUGH ORDER COST ESTIMATE - Ponds**

| | | | | |
|---|---|---|---|---|
| Project: | Rotorua Stormwater Masterplan | Prepared: | LIMI | 21/01/2021 |
| Description: | High level cost estimates - Optioneering | Checked: | JTIH | 25/01/2021 |
| Project No: | 1010988.1000 | | | |
| File location: | \\ttgroup.local\files\TGAProjects\1010988\1010988.1000\WorkingMaterial\08 Costing\[20210127 High Level Cost Estimate. limi.xlsx]Eastern Dams | | | |

| | | | | Pond 1-1 | | | Pond 2-1 | | | Pond 2-2 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Topsoil /GCL Area | | | **28000** | **sq.m** | | **16000** | **sq.m** | | **3000** | **sq.m** | |
| | Excavation volume | | | **76000** | **cu.m** | | **43000** | **cu.m** | | **6000** | **cu.m** | |
| | Pipe length (inlet + outlet) | | | **50** | **m** | | **50** | **m** | | **50** | **m** | |
| **Item #** | **Description** | **Unit** | **Rate** | **Quantity** | **Unit** | **Amount** | **Quantity** | **Unit** | **Amount** | **Quantity** | **Unit** | **Amount** |
| **Site Preparation** | | | | | | | | | | | | |
| 1 | Strip topsoil and grass and stockpile on site | sq.m | $ 6.00 | 28000 | sq.m | $ 168,000.00 | 16000 | sq.m | $ 96,000.00 | 3000 | sq.m | $ 18,000.00 |
| 2 | Excavate by average 0.3 m under entire footprint, stockpile, place on top of GCL once installed | cu.m | $ 30.00 | 8400 | cu.m | $ 252,000.00 | 4800 | cu.m | $ 144,000.00 | 900 | cu.m | $ 27,000.00 |
| **Pond construction** | | | | | | | | | | | | |
| 3 | Excavate volume of pond | cu.m | $ 20.00 | 76000 | cu.m | $ 1,520,000.00 | 43000 | cu.m | $ 860,000.00 | 6000 | cu.m | $ 120,000.00 |
| 4 | Prepare surface, supply and place GCL | sq.m | $ 30.00 | 28000 | sq.m | $ 840,000.00 | 16000 | sq.m | $ 480,000.00 | 3000 | sq.m | $ 90,000.00 |
| 5 | Supply and install inlet/outlet pipes | m | $ 2,000.00 | 50 | m | $ 100,000.00 | 50 | m | $ 100,000.00 | 50 | m | $ 100,000.00 |
| 6 | Supply and install precast MH with scruffy dome at upstream end of outlet pipe | LS | $ 20,000.00 | 1 | LS | $ 20,000.00 | 1 | LS | $ 20,000.00 | 1 | LS | $ 20,000.00 |
| 7 | Supply and install precast wingwall and rip rap apron at downstream end of outlet/inlet pipe | No. | $ 20,000.00 | 2 | No. | $ 40,000.00 | 2 | No. | $ 40,000.00 | 2 | No. | $ 40,000.00 |
| 8 | Connection between wingwalls and GCL | LS | $ 6,000.00 | 1 | LS | $ 6,000.00 | 1 | LS | $ 6,000.00 | 1 | LS | $ 6,000.00 |
| 9 | Load and cart topsoil from stockpile and place on site (25% of total area) | sq.m | $ 6.00 | 7000 | sq.m | $ 42,000.00 | 16000 | sq.m | $ 96,000.00 | 3000 | sq.m | $ 18,000.00 |
| 10 | Supply and place planting (assume 25% of total area planted) | sq.m | $ 25.00 | 7000 | sq.m | $ 175,000.00 | 4000 | sq.m | $ 100,000.00 | 750 | sq.m | $ 18,750.00 |
| | **Sub Total - base construction cost** | | | | | **$ 3,163,000** | | | **$ 1,942,000** | | | **$ 457,750** |
| 11 | Contractor P&G | % | 15% | | % | $ 474,450 | | % | $ 291,300 | | % | $ 68,663 |
| 12 | Unscheduled & minor items | % | 10% | | % | $ 316,300 | | % | $ 194,200 | | % | $ 45,775 |
| | **Construction subtotal** | | | | | **$ 3,953,750** | | | **$ 2,427,500** | | | **$ 572,188** |
| 13 | Contingency | % | 40% | | % | $ 1,581,500 | | % | $ 971,000 | | % | $ 228,875 |
| 14 | Construction monitoring | % | 6% | | % | $ 237,225 | | % | $ 145,650 | | % | $ 34,331 |
| | **Construction total** | | | | | **$ 5,772,475** | | | **$ 3,544,150** | | | **$ 835,394** |
| 15 | Planning/consents/consultation/negotiating access | % | 2% | | % | $ 115,450 | | % | $ 70,883 | | % | $ 16,708 |
| 16 | Design fees | % | 4% | | % | $ 230,899 | | % | $ 141,766 | | % | $ 33,416 |
| | **TOTAL** | | | | **Pond 1-1** | **$ 6,120,000** | | **Pond 2-1** | **$ 3,760,000** | | **Pond 2-2** | **$ 890,000** |

| | Pond 2-3 | | | Pond 2-6 | | | Pond 2-7 | | | Pond 2-8 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | **10000** | **sq.m** | | **5000** | **sq.m** | | **5000** | **sq.m** | | **22000** | **sq.m** | |
| | **24000** | **cu.m** | | **11000** | **cu.m** | | **13000** | **cu.m** | | **64000** | **cu.m** | |
| | **50** | **m** | | **50** | **m** | | **50** | **m** | | **50** | **m** | |
| **Item #** | **Quantity** | **Unit** | **Amount** | **Quantity** | **Unit** | **Amount** | **Quantity** | **Unit** | **Amount** | **Quantity** | **Unit** | **Amount** |
| **Site Preparation** | | | | | | | | | | | | |
| 1 | 10000 | sq.m | $ 60,000.00 | 5000 | sq.m | $ 30,000.00 | 5000 | sq.m | $ 30,000.00 | 22000 | sq.m | $ 132,000.00 |
| 2 | 3000 | cu.m | $ 90,000.00 | 1500 | cu.m | $ 45,000.00 | 1500 | cu.m | $ 45,000.00 | 6600 | cu.m | $ 198,000.00 |
| **Pond construction** | | | | | | | | | | | | |
| 3 | 24000 | cu.m | $ 480,000.00 | 11000 | cu.m | $ 220,000.00 | 13000 | cu.m | $ 260,000.00 | 64000 | cu.m | $ 1,280,000.00 |
| 4 | 10000 | sq.m | $ 300,000.00 | 5000 | sq.m | $ 150,000.00 | 5000 | sq.m | $ 150,000.00 | 22000 | sq.m | $ 660,000.00 |
| 5 | 50 | m | $ 100,000.00 | 50 | m | $ 100,000.00 | 50 | m | $ 100,000.00 | 50 | m | $ 100,000.00 |
| 6 | 1 | LS | $ 20,000.00 | 1 | LS | $ 20,000.00 | 1 | LS | $ 20,000.00 | 1 | LS | $ 20,000.00 |
| 7 | 2 | No. | $ 40,000.00 | 2 | No. | $ 40,000.00 | 2 | No. | $ 40,000.00 | 2 | No. | $ 40,000.00 |
| 8 | 1 | LS | $ 6,000.00 | 1 | LS | $ 6,000.00 | 1 | LS | $ 6,000.00 | 1 | LS | $ 6,000.00 |
| 9 | 10000 | sq.m | $ 60,000.00 | 5000 | sq.m | $ 30,000.00 | 5000 | sq.m | $ 30,000.00 | 22000 | sq.m | $ 132,000.00 |
| 10 | 2500 | sq.m | $ 62,500.00 | 1250 | sq.m | $ 31,250.00 | 1250 | sq.m | $ 31,250.00 | 5500 | sq.m | $ 137,500.00 |
| **Sub Total - base construction cost** | | | **$ 1,218,500** | | | **$ 672,250** | | | **$ 712,250** | | | **$ 2,705,500** |
| 11 | | % | $ 182,775 | | % | $ 100,838 | | % | $ 106,838 | | % | $ 405,825 |
| 12 | | % | $ 121,850 | | % | $ 67,225 | | % | $ 71,225 | | % | $ 270,550 |
| **Construction subtotal** | | | **$ 1,523,125** | | | **$ 840,313** | | | **$ 890,313** | | | **$ 3,381,875** |
| 13 | | % | $ 609,250 | | % | $ 336,125 | | % | $ 356,125 | | % | $ 1,352,750 |
| 14 | | % | $ 91,388 | | % | $ 50,419 | | % | $ 53,419 | | % | $ 202,913 |
| **Construction total** | | | **$ 2,223,763** | | | **$ 1,226,856** | | | **$ 1,299,856** | | | **$ 4,937,538** |
| 15 | | % | $ 44,475 | | % | $ 24,537 | | % | $ 25,997 | | % | $ 98,751 |
| 16 | | % | $ 88,951 | | % | $ 49,074 | | % | $ 51,994 | | % | $ 197,502 |
| **TOTAL** | | **Pond 2-3** | **$ 2,360,000** | | **Pond 2-6** | **$ 1,300,000** | | **Pond 2-7** | **$ 1,380,000** | | **Pond 2-8** | **$ 5,230,000** |

**Exclusions**

Land purchase (except where specified)
Estimate does not attempt to make any allowance for potential effects of COVID-19.
Government taxes
Rotorua Lakes District Council Project Management
Building consent
Public relations
Insurances during construction & commissioning
Project financing & other admin costs, incl legal fees
Bid construction cost volatility due to such reasons as: changes in costs fuel, and any other commodity subject fluctuations or commodity world demand;
Other costs not specifically referred to above

## ROUGH ORDER COST ESTIMATE - Pipes/culverts

| | | | | | |
|---|---|---|---|---|---|
| Project: | Rotorua Stormwater Masterplan | Prepared by | JTIH | 30/07/2020 | |
| Description: | High level cost estimates - Optioneering | Updated by | LIMI | 25/01/2021 | |
| Project No: | 1010988.1000 | Checked by | SJK/JTIH | 44221 | |
| File location: | \\ttgroup.local\files\TGAProjects\1010988\1010988.1000\WorkingMaterial\08 Costing\[20210127 High Level Cost Estimate. limi.xlsx]Eastern Dams | | | 4b | 4a |

| Project | | | Vaughan Rd culvert | | | Te Ngae Rd Culvert | | | Morey St Culverts (Option 4B) | | | Morey St Culverts (Option 4A) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Total length of pipe (m) | | 15 | | | 75 | | | 45 | | | 20 | | |
| | | | | | | | | | | | | | | |
| **Item #** | **Description** | **Unit** | **Rate** | **Quantity** | **Amount** | **Rate** | **Quantity** | **Amount** | **Rate** | **Quantity** | **Amount** | **Rate** | **Quantity** | **Amount** |
| 1 | Pipe/culvert | m | $ 20,000.00 | 15 | $ 300,000.00 | $ 6,000.00 | 75 | $ 450,000.00 | $ 6,000.00 | 45 | $ 270,000.00 | $ 3,000.00 | 20 | $ 60,000.00 |
| 2 | Wingwalls/inlet connections | LS | $ 15,000.00 | 2 | $ 30,000.00 | $ 15,000.00 | 2 | $ 30,000.00 | $ 15,000.00 | 4 | $ 60,000.00 | $ 15,000.00 | 2 | $ 30,000.00 |
| 3 | Reinstate road | LS | $ 50,000.00 | 1 | $ 50,000.00 | | | | $ 50,000.00 | 2 | $ 100,000.00 | $ 50,000.00 | 1 | $ 50,000.00 |
| | **Sub Total - base construction cost** | | | | **$ 380,000.00** | | | **$ 480,000.00** | | | **$ 430,000.00** | | | **$ 140,000.00** |
| 4 | Contractor Preliminary and general | % | 15% | | $ 57,000.00 | 15% | | $ 72,000.00 | 15% | | $ 64,500.00 | 15% | | $ 21,000.00 |
| 5 | Unscheduled & minor items | % | 10% | | $ 38,000.00 | 10% | | $ 48,000.00 | 10% | | $ 43,000.00 | 10% | | $ 14,000.00 |
| | **Construction subtotal** | | | | **$ 475,000** | | | **$ 600,000** | | | **$ 537,500** | | | **$ 175,000** |
| 6 | Contingency | % | 40% | | $ 190,000 | 40% | | $ 240,000 | 40% | | $ 215,000 | 40% | | $ 70,000 |
| 7 | Construction management | % | 6% | | $ 28,500 | 6% | | $ 36,000 | 6% | | $ 32,250 | 6% | | $ 10,500 |
| | **Construction total** | | | | **$ 693,500** | | | **$ 876,000** | | | **$ 784,750** | | | **$ 255,500** |
| 8 | Planning/consents | % | 4% | | $ 27,740 | 4% | | $ 35,040 | 4% | | $ 31,390 | 4% | | $ 10,220 |
| 9 | Professional fees (Detailed design and Construction monitoring) | % | 12% | | $ 83,220 | 10% | | $ 87,600 | 10% | | $ 78,475 | 10% | | $ 25,550 |
| | **Project total** | | | **Vaughan Rd culvert** | **$ 800,000** | | **Te Ngae Rd Culvert** | **$ 1,000,000** | | **Morey St Culverts** | **$ 890,000** | | **Morey St Culverts** | **$ 290,000** |

**Assumptions/clarifications:**

Assumes road upgrades for Te Ngae Road is covered in a separate budget

**Exclusions**

Land purchase (except where specified)

Estimate does not attempt to make any allowance for potential effects of COVID-19.
Government taxes
Rotorua Lakes District Council Project Management
Building consent
Public relations
Insurances during construction & commissioning
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Bid construction cost volatility due to such reasons as: changes in costs fuel, and any other commodity subject fluctuations or commodity world demand;
Other costs not specifically referred to above

## ROUGH ORDER COST ESTIMATE - Open channels

| | | | | |
|---|---|---|---|---|
| Project: | Rotorua Stormwater Masterplan | Prepared by | JTIH | 30/07/2020 |
| Descriptior | High level cost estimates - Optioneering | Updated by | LIMI | 25/01/2021 |
| Project No: | 1010988.1000 | Checked by | SJK/JTIH | 25/01/2021 |

File locatio \\ttgroup.local\files\TGAProjects\1010988\1010988.1000\WorkingMaterial\08 Costing\[20210127 High Level Cost Estimate. limi.xlsx]Eastern Dams

| **Project** | | | | **Owhatiura Drain East - Option 4a** | | **Owhatiura Drain East - Option 4b** | | **Melrose Ave Drain** | | **Vaughan Road Drain** | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Total length of channel (m) | | | 420 | | 605 | | 270 + 100 | | 730 | |
| | Channel corridor width (m) | | | 20 | | 22 | | 19 | | 14 | |
| **Item #** | **Description** | **Unit** | **Rate** | **Quantity** | **Amount** | **Quantity** | **Amount** | **Quantity** | **Amount** | **Quantity** | **Amount** |
| 1 | Demolish existing building | $m^2$ | $ 200.00 | | | | | 800 | $ 160,000.00 | | |
| 2 | Site clearance (including topsoil removal) | $m^2$ | $ 10.00 | 8400 | $ 84,000.00 | 13310 | $ 133,100.00 | 5130 | $ 51,300.00 | 10220 | $ 102,200.00 |
| 3 | Filling existing channel (using material excavated on site) | $m^3$ | $ 30.00 | | | | | 600 | $ 18,000.00 | | |
| 5 | Earthworks for new channel | $m^3$ | $ 20.00 | 9030 | $ 180,600.00 | 16033 | $ 320,650.00 | 2000 | $ 40,000.00 | 6250 | $ 125,000.00 |
| 6 | Form low flow channel | m | $ 30.00 | 420 | $ 12,600.00 | 605 | $ 18,150.00 | 370 | $ 11,100.00 | 730 | $ 21,900.00 |
| 7 | Extend connections to existing SW pipe network | LS | $ 10,000.00 | 2 | $ 20,000.00 | 2 | $ 20,000.00 | 2 | $ 20,000.00 | | |
| 8 | Additional work required to cross existing WW pipe | LS | $ 10,000.00 | | | | | 3 | $ 30,000.00 | | |
| 9 | Planting/surfacing | $m^2$ | $ 25.00 | 8400 | $ 210,000.00 | 13310 | $ 332,750.00 | 5130 | $ 128,250.00 | 10220 | $ 255,500.00 |
| 10 | Cycleway/maintenance acess (1.2m wide) | m | $ 40.00 | 420 | $ 16,800.00 | 605 | $ 24,200.00 | 270 | $ 10,800.00 | 730 | $ 29,200.00 |
| 11 | Realign powerlines, water line and WW pipe in road berm to make space for new concrete channel | LS | $ 20,000.00 | | | | | 1 | $ 20,000.00 | | |
| 12 | Timber lined drain with naturalised base (1m deep, 3m wide) - includes excavation | m | $ 5,000.00 | | | | | 100 | $ 500,000.00 | | |
| | **Sub Total - base construction cost** | | | | $ 524,000.00 | | $ 848,850.00 | | $ 989,450.00 | | $ 533,800.00 |
| **13** | Contractor Preliminary and general | % | 15% | | $ 78,600.00 | | $ 127,327.50 | | $ 148,417.50 | | $ 80,070.00 |
| 14 | Unscheduled & minor items | % | 10% | | $ 52,400.00 | | $ 84,885.00 | | $ 98,945.00 | | $ 53,380.00 |
| | **Construction subtotal** | | | | **$ 655,000** | | **$ 1,061,063** | | **$ 1,236,813** | | **$ 667,250** |
| **15** | Contingency | % | 40% | | $ 262,000 | | $ 424,425 | | $ 494,725 | | $ 266,900 |
| **16** | Construction management | % | 6% | | $ 55,020 | | $ 89,129 | | $ 103,892 | | $ 56,049 |
| | **Construction total** | | | | **$ 972,020** | | **$ 1,574,617** | | **$ 1,835,430** | | **$ 990,199** |
| **17** | Planning/consents | % | 4% | | $ 38,881 | | $ 62,985 | | $ 73,417 | | $ 39,608 |
| 18 | Professional fees (Detailed design and Construction monitoring) | % | 12% | | $ 116,642 | | $ 188,954 | | $ 220,252 | | $ 118,824 |
| 19 | Property purchase | LS | $ 500,000.00 | | | | | **7** | **$ 3,500,000.00** | | |
| | **Project total** | | | | **$ 1,130,000** | | **$ 1,830,000** | | **$ 5,630,000** | | **$ 1,150,000** |

**Assumptions/clarifications:**

Assumes no property purchase required for the Owhatiura Drain or Vaughan Rd drain

Some of the property purchase costs for Melrose Ave Drain can be recouped at the completion of the job. This is not factored into cost estimate

**Exclusions**

Land purchase (except where specified)

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Government taxes

Rotorua Lakes District Council Project Management

Building consent

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Other costs not specifically referred to above

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