Rotorua Lakes Council

Catchment 4 Stormwater Model Build and System Performance Report

OPUS

Rotorua Lakes Council

Catchment 4

Stormwater Model Build and System Performance Report

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Date: August 2017
Reference: 0201-MB01-3c142304
Status: Final

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1 Introduction

Rotorua Lakes Council (RLC) has commissioned Opus International Consultants (Opus) to produce an assessment of the performance of Stormwater Catchment 4 within the Rotorua township. As part of the project, a hydraulic stormwater model for the area has been built that will provide inputs to understanding the issues across the catchment solely.

The desired outcome of the model build is to develop a model that can be used to:

- Identify key flooding issues;
- Identify critical infrastructure and failure risks;
- Provide inputs for master planning; and
- Assist operation and maintenance.

The scope of this project involves the following stages:

Stage 1 - Data Review and Acquisition

Stage 2 - Model Build and Sensibility Checks

Stage 3 – Stormwater System Capacity Review

Stage 4 – Development of high-level options

This report represents the deliverables for Stages 1-3. Stage 4 will be addressed in a separate memorandum detailing the proposed high-level options.

2 Catchment Description

2.1 Catchment Extent

Catchment 4 is a stormwater catchment on the southern end of Lake Rotorua, east of the Rotorua central town area. The catchment covers an upstream rural area draining through the urban area of Owhata to the south-east corner of the lake. The urban area is predominantly residential and covers approximately 190 ha to the north of Basley Road and on either side of Te Ngae Road. The rural area, which is predominantly grassland, is 255 ha and is located to the south-east of the urban area. A large proportion of the rural area is due to be developed for residential housing as part of the Wharenui Road development. Figure 2-1 shows the location of the two areas comprising Catchment 4.

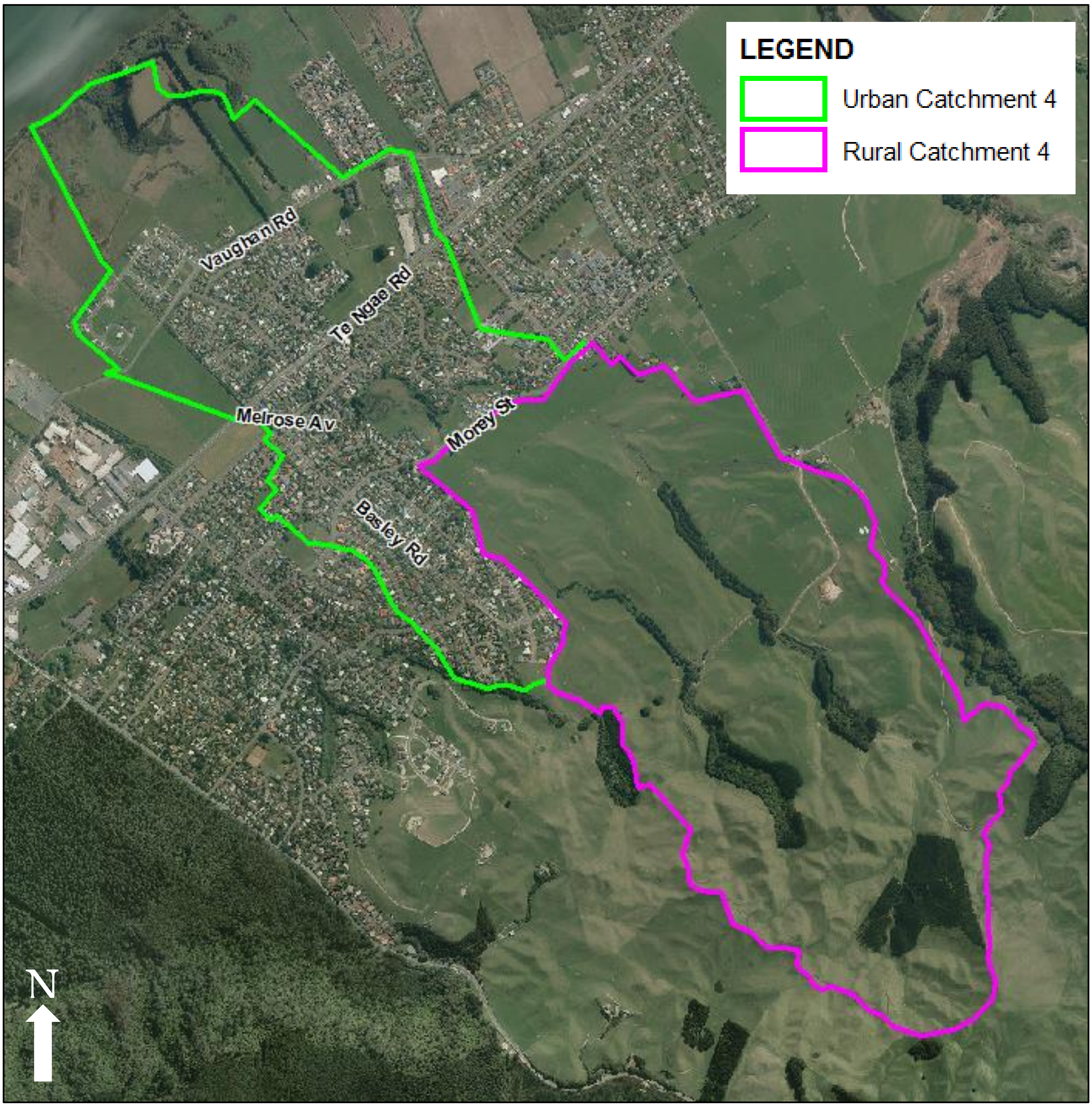

Figure 2-1: Catchment Extent

For simplicity, throughout the remainder of the report, "Catchment 4" will describe both the urban and rural catchments as one. The project is focussed on quantifying and understanding the flows and velocities in the downstream reaches of the open channel running through the catchment. No specific flooding problems have been identified by RLC.

2.2 Topography

The rural area in the south features a series of hills and valleys draining towards the urban area, with elevations between 500 m AD and 320 m AD, before levelling off to around 300 m AD. The developed area is much flatter, with elevations between 300 m AD and 280 m AD. Figure 2-2 shows the elevations across the catchment.

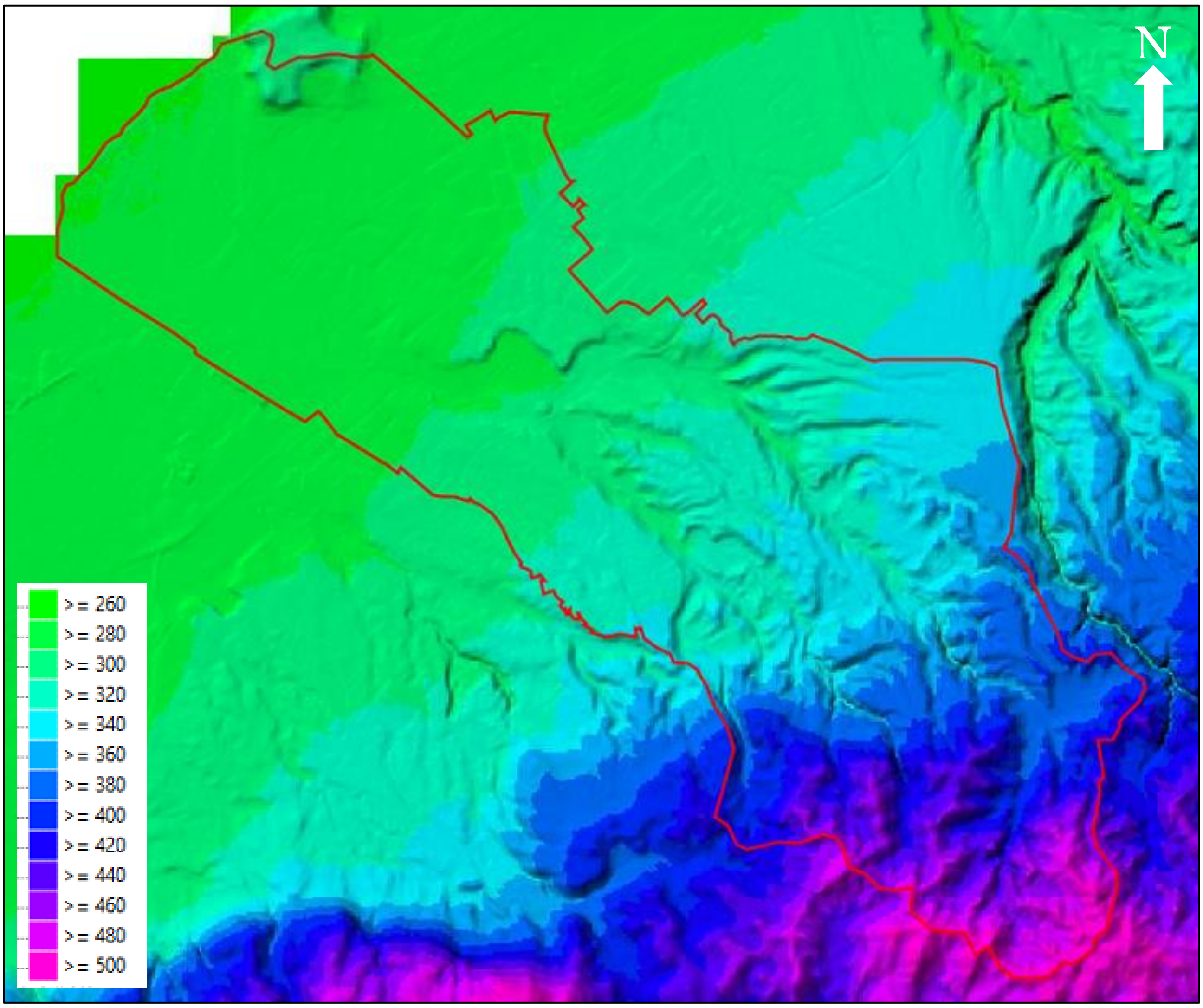

Figure 2-2: Catchment 4 Topography

2.3 Geology and Soils

The catchment is situated on the shores of Lake Rotorua where the geologic setting consists of late Quaternary alluvium, colluvium lake deposits, more commonly known as Zealandia Megasequence Terrestrial, and Shallow Marine Sedimentary Rocks. Soils in the area are generally formed from Tarawera Lapilli and rhyolitic tephra. Figure 2-3 shows the distribution of soil types across the catchment. As can be seen, the predominant soil types are F6.1a (Well-drained, low fertility soils) in the urban area and H2.2a (well-drained, moderate fertility soils) in the rural area.

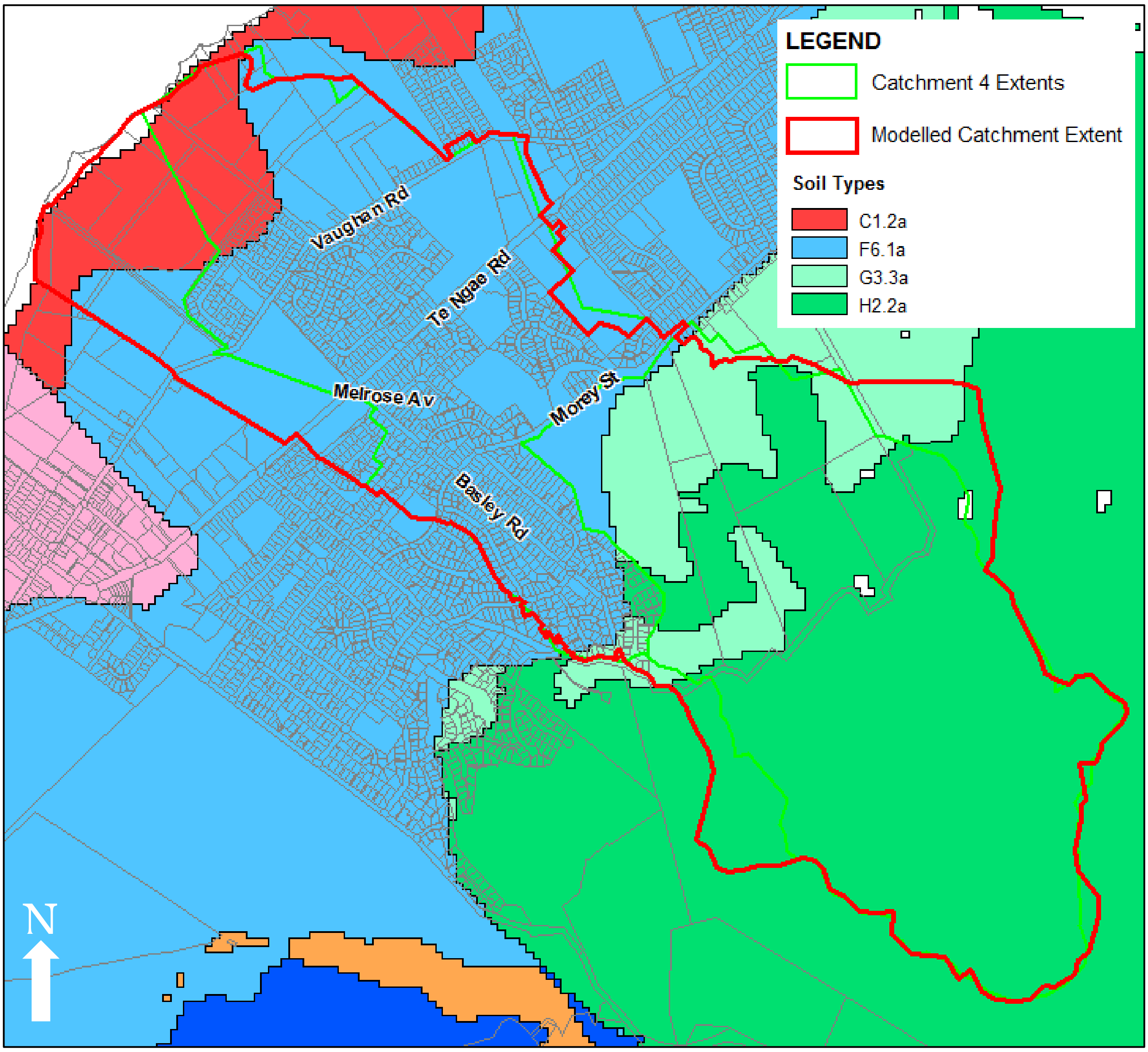

Figure 2-3: Soil Types (Landcare Research New Zealand, 2002)

Table 2-1 details the soil types within the catchment.

Table 2-1: LENZ Soil Types (Landcare Research New Zealand, 2002)

Level III Classification	Landform	Soils	Level IV Characteristics
C1.2	Gently undulating plains	Poorly-drained peat soils of low fertility with some alluvium	a) Warm temperatures, high solar radiation, slight annual water deficits
F6.1	Undulating hills	Well-drained, low fertility soils from mid-age rhyolitic tephra	a) Warmer temperatures c) Cooler temperatures
G3.3	Very gently undulating flood plains	Recent, well-drained soils of low fertility from mixed alluvium	Warm temperatures, high solar radiation, moderate vapour pressure deficits, low annual water deficits
H2.2	Easy rolling hills	Recent, well-drained soils of moderate fertility from Tarawera lapilli and rhyolitic tephra	a) rolling hills, low fertility

2.4 Stormwater Network Overview

The stormwater system is shown in Figure 2-4 and consists of a combination of piped networks and natural and man-made waterways (open channels). The piped network intercepts and conveys stormwater flows from the road corridor and property connections to the open channel. Stormwater collected from the road corridor consists of both road run-off and property discharges to the kerb and channel.

The predominant open channel in the catchment starts on the northern side of Morey Street and runs west behind properties on Melrose Avenue and Basley Road, ultimately discharging at the lake. There are three culverts on the channel, beneath road crossings at Te Ngae Road, Vaughan Road and Carroll Place.

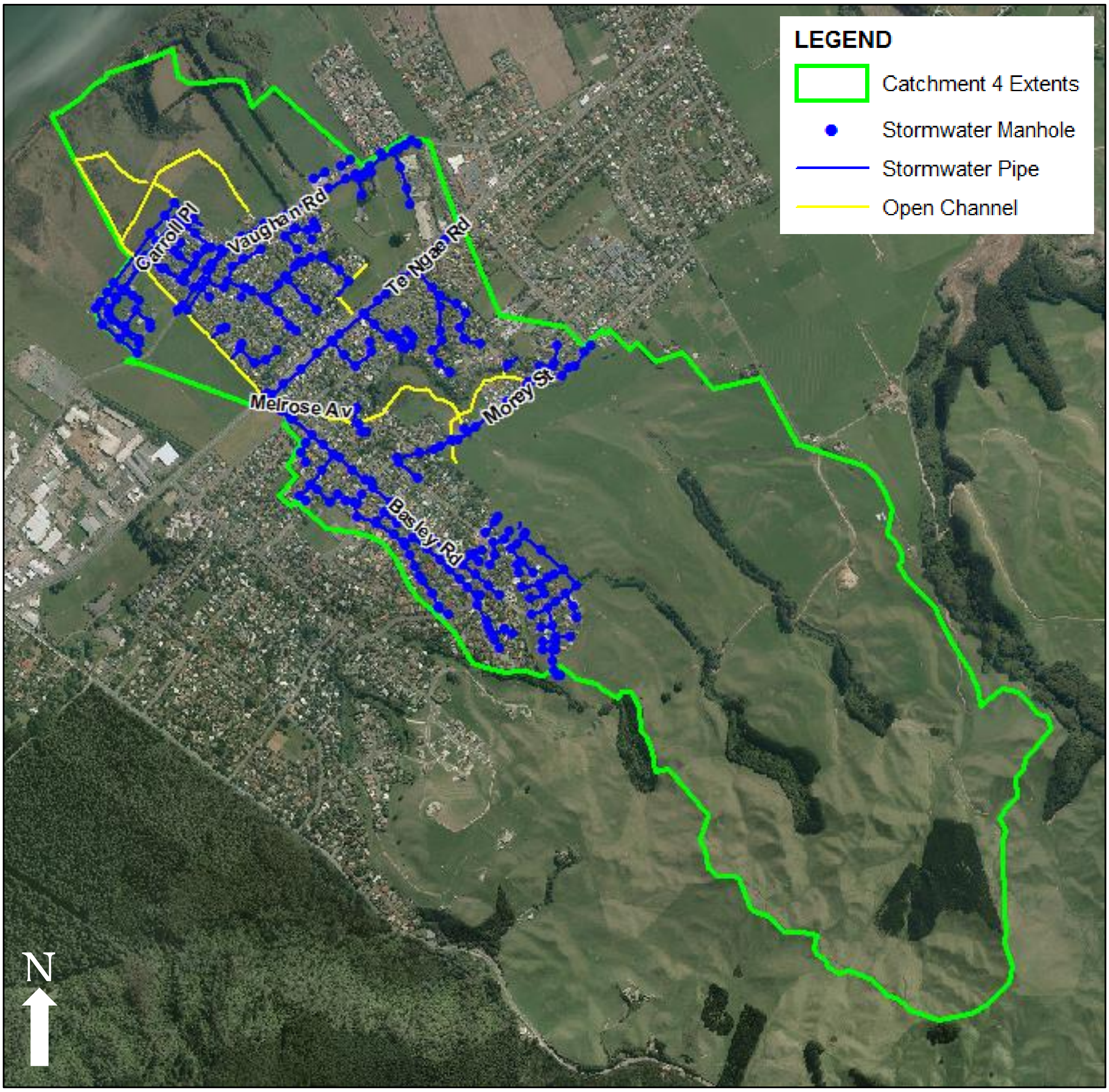

Figure 2-4: Catchment 4 Stormwater Network

2.5 Land Use

The upstream end of the catchment contains a significant area of rural land encompassing approximately 60% of the catchment, while the remainder of the catchment is urban residential. Land uses within the catchment are presented in Figure 2-5 (Rotorua Lakes Council, 2016).

This plan also shows the proposed extent of the Wharenui Development, which is designated residential, and is the land south of Morey Street.

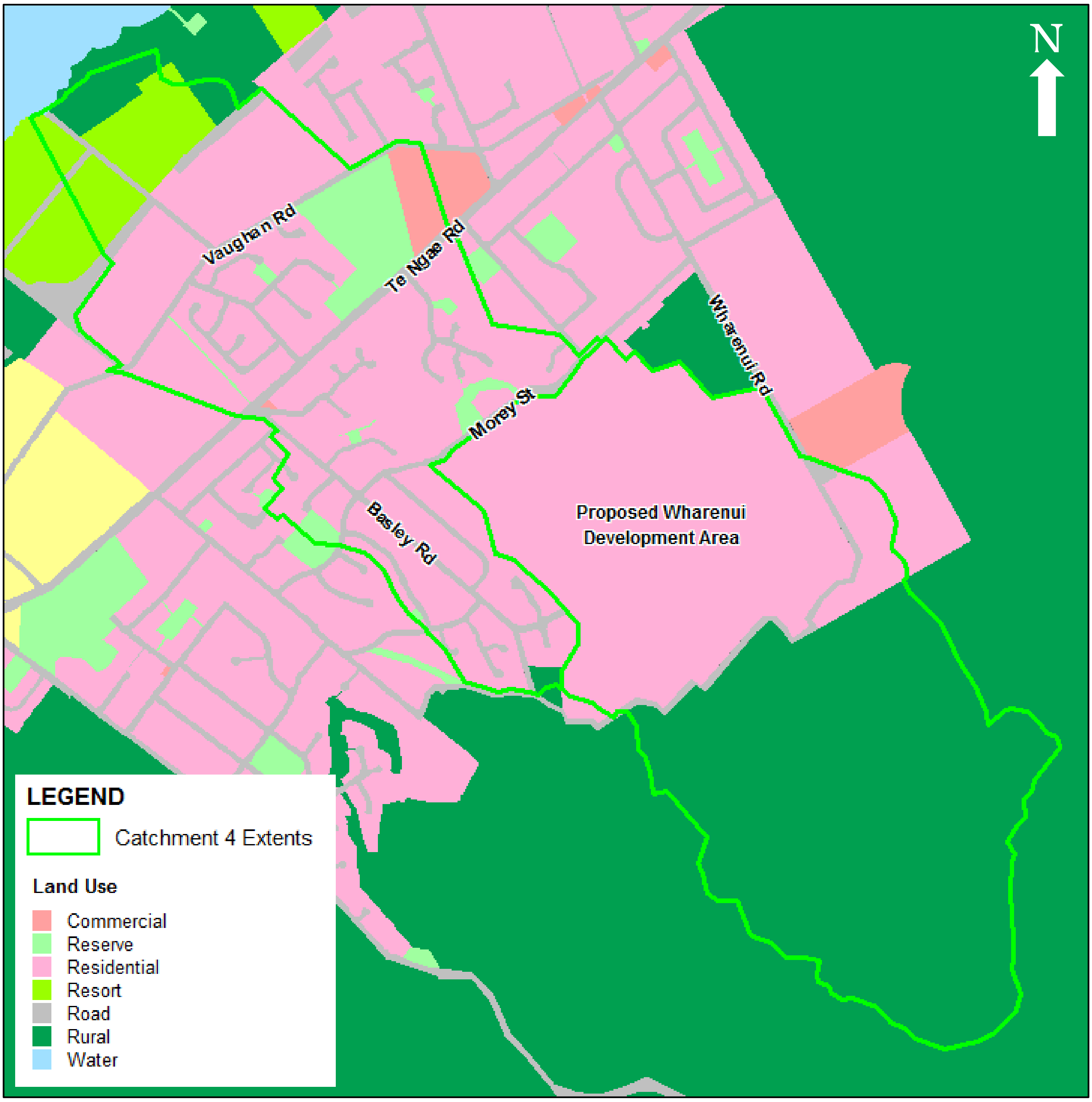

Figure 2-5: Land Use

2.6 Stormwater Issues

The predominant stormwater issue within the catchment, and the main driver for this investigation, are the high velocities reported in the downstream reach of the open channel, between Vaughan Road and Carroll Place. The high velocities are causing scouring issues of the channel and banks.

A secondary issue is the potential impact of the Wharenui Road Development on the existing stormwater flows in the catchment.

3 Model Build

3.1 Hydraulic Model

RLC provided Opus with the stormwater layout in MapInfo TAB file format. This was imported into InfoWorks ICM v7.0 as a 1D hydraulic model, where the data cleansing and 1D model build was undertaken. Once the 1D model was built, it was converted to a linked 1D-2D hydraulic model, incorporating a 2D surface based on LiDAR.

The coordinate system used was the New Zealand Geodetic Datum 2000 (NZGD2000) using the New Zealand Transverse Mercator (NZTM) projection. Levels are in terms of the Moturiki Mean Sea Level Datum 1953.

3.2 Asset Data

Asset and survey data acquired for development of the hydraulic model included:

- GIS data;
- DTM ground level information;
- Aerial imagery;
- Site inspections; and
- Level surveys for selected inlets/outlets and cross sectional surveys of the open channel.

Table 3-1 lists the flags that have been used to identify the various data sources in the model.

Table 3-1: Data Flags Used

Flag	Description
#A	Asset Data
#D	System Default
AS	Data assumed based on engineering judgement
DU	Dummy parameter
EJ	Engineering Judgement
FIX	Modelling Fix
GIS	From RLC GIS Datasets
HY	Hydraulic calculation
IF	Data inferred by InfoWorks automated process
LD	Inferred from LiDAR
PHO	Data from photos, aerials, Google Street view
RPTD	Report – Technical Document e.g. Culvert Design Guide, TP108
SD4	Survey data – RLC Survey
ST	RLC Standards

3.2.1 Survey

Key infrastructure was surveyed by RLC in March 2017. The following data was collected:

- Inlet and outlet levels on open channels; and
- Multiple cross sections along culverts and open channels.

3.2.2 Nodes

The nodes were named as per the original GIS dataset provided by RLC. A few additional nodes were added to the model for connectivity purposes, and where required to model complex structures such as culverts beneath Te Ngae Road. Flood types used in the model are summarised in Table 3-2.

Table 3-2: Flood types used to represent point assets

ICM Flood Type	Objects	Description
Sealed	Junctions	Water levels can rise indefinitely, pressurising the system.
2D	Manholes	Stormwater can flow to and from the 2D surface. The weir equation is used to control flow.
2D Outfall	Outfall	Used at the downstream ends of small networks that discharge into a stream and for inlets and outlets to sections of culvert outside the river reaches. Flow exiting these outfalls flows onto the 2D mesh.
2D Gully	Gully / inlet	Used to model sumps as identified by RLC's GIS "inlet" table. A head discharge curve is used to control flow. Assumed to be single sumps unless identified as other.
Outfall	Outfall	Stormwater is lost from the system (in conjunction with a boundary level condition) - used at the end of the stormwater system.
Break	Nodes	Used to connect the piped network to modelled river reaches.

All manhole data was calculated using the InfoWorks ICM defaults, including the node chamber area.

Figure 3-1 shows how this is calculated.

$$A = \frac{\pi}{4} \times (W + 0.762)^2$$

where:
A = default area
W = width of widest link incoming or outgoing

Figure 3-1: Calculation of chamber area in ICM (Innovyze, 2014)

3.2.2.1 Sumps

Gullies / catch pits / sumps were modelled using a head discharge profile based on empirical flow curves developed through laboratory testing. Unless specific details are provided, they have been assumed to be single catch-pits with back entry, referred to as a "combination 13 inlet" (James C Y Guo, 2009).

3.2.2.2 Soak Holes

There are no known soak holes in the network. There may be some private on-site soakage, however no initial losses to account for these storage devices have been included in the model.

3.2.3 Pipes

Approximately 9% (32 pipes) of the RLC GIS "SWMainLine" dataset was missing an upstream, downstream or both invert levels.

No level or diameter information was provided for the sump leads, identified from the "SWLead" dataset. This is common for stormwater GIS datasets. In general, these have been assumed to be DN225, and levels have been assumed using RLC's standard drainage drawings (Rotorua Lakes Council, 2004). If a sump was identified as a double sump, the sump lead has been modelled as a DN300.

The manhole survey provided invert levels for key outlets and culverts and the remaining levels were inferred either directly from their connecting manhole or by straight line interpolation. The interpolated results were then checked to ensure that the long section looked sensible when compared to the LiDAR. Outlet pipes were assumed to have their invert level at ground level at the point of outfall based on LiDAR or survey where available.

In some instances, it was necessary to use engineering judgement to set invert levels for terminal manholes on a pipeline or for pipes where the straight line interpolation put the pipeline above ground. In general, a minimum pipe cover based on the levels of surrounding manholes and DTM data was assumed.

Surface friction is applied to the piped network using typical Colebrook-White roughness coefficients depending on pipe material (range: 0.6-30).

Transitional head losses at the manholes have been inferred in ICM and applied to the pipes. The transitional head losses are based on the manhole approach and exit angles, pipe grade and approach velocity. The "Normal" head loss curve was used which is appropriate for well-constructed manholes.

Pipe gradients were calculated using InfoWorks ICM. Where gradients greater than 0.1m/m were calculated, the associated structure energy loss was set to "None", to reduce model instabilities, as is recommended by Innovyze.

Service connections / private laterals have been excluded from the modelling.

3.2.4 Culverts

Turbulence losses associated with the entry and exit of culverts between river reaches have been represented using culvert inlet and outlet links. Entry losses have been modelled using values recommended in Table A1.3 of the Culvert Design Manual (CIRIA, 2010).

3.2.5 Open Channel

There is one predominant open channel in the catchment that forms an integral part of the stormwater network. Its upper reaches, by Morey Street, are fairly open, with a shallow, swale-like system to the east, and more overgrown sections to the west.

The channel becomes a much narrower, timber-lined channel when it runs along the rear of properties between Melrose Avenue and Alastair Avenue. Here, it is crossed by several fences and footpaths that could create flow restrictions during larger storm events. At this stage, these potential flow restrictions within the timber-lined section of the channel have not been accounted for in the model. It is recommended that a sensitivity test is undertaken to identify the potential impact on the network if a blockage were to occur in this location.

The channel drops approximately 2m into an open concrete chamber at the junction of Melrose Avenue and Te Ngae Road.

For its remaining open length, it is a well-defined channel on the south-western edge of new residential developments between Te Ngae Road and Carroll Place.

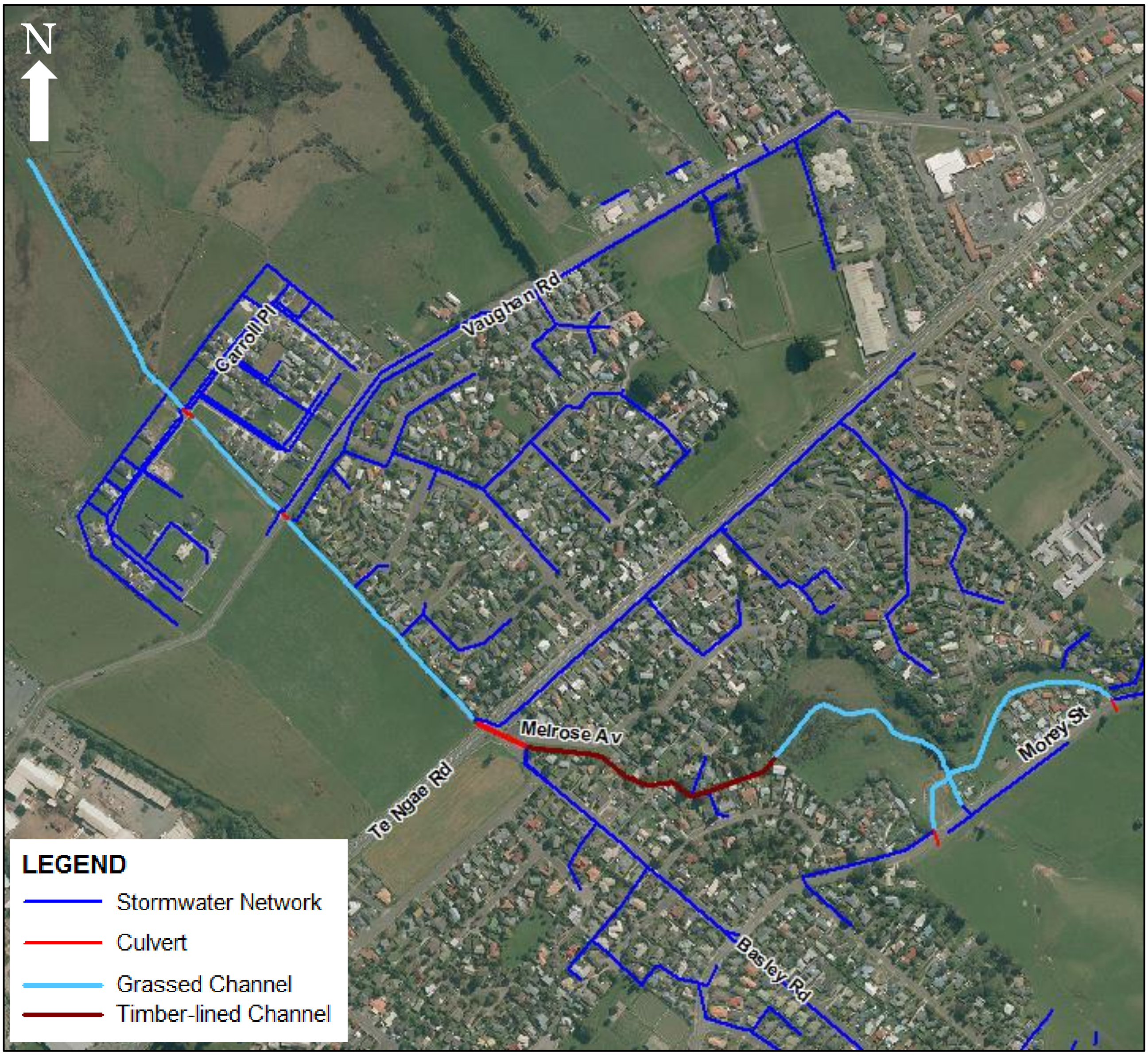

Figure 3-2: Open Channel Route

The channel was surveyed at multiple locations to capture its geometry via cross sections. Photos of the various channel profiles, key structures and potential flow restrictions are provided in Appendix A.

The cross-section data was used to create river reaches that were linked to the 2D surface via bank lines to permit lateral flow. The bank lines for grassed channels have been modelled using a discharge coefficient of 1 with a modular ratio of 0.7. The discharge coefficient for the timber-lined section have been reduced to 0.8, due to the presence of fences along the bank lines. Manning's roughness values have been applied based on the channel profiles shown in aerial photos.

3.2.6 Detention Ponds

There is one detention pond within the catchment. This is located in the rural area north of Devoy Drive and receives flows from the recently developed subdivision at the end of Basley Road. The pond itself has been modelled using the LiDAR surface, and its DN300 diameter discharge pipe has been modelled based on survey data.

3.3 Hydrological Model

Initially, Horton's runoff parameters were proposed for use. However, due to this runoff method's incompatibility with the nested storms used for flood map generation, the SCS runoff model has been used instead. This is because the SCS method has an infiltration loss proportional to intensity, whereas Horton uses a fixed infiltration rate. This can lead to higher runoff values versus the SCS approach.

The SCS runoff model is a well-established approach suited to both rural and urban catchments but uses a combined runoff model for pervious and impervious surfaces referred to as a 'CN' curve. The CN curve number is based on soil characteristics, plant cover, level of impervious area and surface storage. Values presented in this report are derived from the Urban Hydrology for Small Watersheds TR-55 Document (United States Department of Agriculture, 1986).

3.3.1 Sub-catchments

Model sub-catchments were digitised in InfoWorks ICM. The sub-catchment boundaries align with either parcel boundaries or ground contours and were attributed to a node based on the ground contours and the road and reticulation layout. A GIS layer showing stormwater service connections was available for the catchment and this information was used to allocate sub-catchments in this area. Sub-catchments have also been digitised to include only one land use type, which are based on the zoning information supplied by RLC and an inspection of aerial imagery.

3.3.2 Hydrologic soil group

The SCS approach uses four soil group categories; A, B, C and D, which range from low to high runoff potential. Catchment 4 has dominant soil types of F6.1a and H2.2a (Figure 2-3), both of which are characterised to have good drainage potential. All curve numbers were therefore based on the hydrological soil group A.

3.3.3 Cover type

Cover type was determined by undertaking a desktop assessment of aerial photography. Four cover types were identified in line with TR-55 classifications:

- Open Spaces;
- Residential: lot size 1000 m^2 (average for catchment 4 is approximately 800 m^2);
- Commercial; and
- Streets/roads: sealed.

The assigned sub-catchment cover types and their corresponding curve numbers are detailed in Table 3-3.

Table 3-3: Curve numbers for sub-catchments.

Cover description	Average Impervious Area (%)	Curve Number
Open Spaces	0	39
Residential: lot size 1000 m²	38	61
Commercial and business	85	89
Streets/roads: sealed	98	98

3.3.4 Hydrologic Condition

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

3.3.5 Antecedent rainfall condition

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

3.3.6 2D Surface

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

- Min triangle - 25 m^2
- Max triangle - 100 m^2
- Default surface roughness - 0.1
- Boundary condition – Normal hydraulic condition (where no boundary condition has been applied)

The triangle sizes for Catchment 4 have been increased from the previous Catchment 5 modelling, which used a minimum triangle size of 5 m^2 and a maximum of 20 m^2. The values were amended as a result of 'flow limiting' due to significant flow rates at the two Morey Street culverts. Flow limiting occurs when the volume of the 2D flow exceeds the triangle's perimeter length by a certain ratio creating an artificially steep hydraulic gradient. This technical limitation of 2D surface hydraulics is resolved through the use of a larger triangle size which increases perimeter length. This avoids an unstable model and erroneous results.

3.3.7 Surface Roughness

Table 3-4 shows the range of Manning's 'n' surface values for differing cover type based on industry guidance.

Table 3-4: Typical 2D Manning's 'n' Roughness Values

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

For Catchment 4, a standard Manning's 'n' surface roughness of 0.1 has been used. This value represents roughness values appropriate for urban residential parcels. Road parcels have been imported into the network as roughness zones and assigned a roughness of 0.013. Further roughness zones have been digitised manually covering open spaces, reserves and the upstream rural area, and assigned a roughness of 0.07.

3.4 Boundary Conditions

The channel has been modelled with a boundary level of 281.18 m AD at the downstream extent of the model. This level was provided by the RLC GIS team. It is taken from drawing number 10383-05 and is contained within the District Plan GIS layers as the peak flood level for the lake during a 1 in 50 year ARI (2% AEP) flood. The red area in Figure 3-3 shows the area below this level, based on the provided LiDAR.

Figure 3-3: Extent of Catchment 4 lower than the 1 in 50 Year ARI Level

3.5 Summary of Modelled Objects

Table 3-5 summarises all the modelled objects.

Table 3-5: Modelled Objects

Modelled Object	Number
Number of Nodes	747
Number of Manholes Modelled	356
Number of Sumps Modelled	356
Number of River Reach (Break) Nodes	25
Number of Outfall Nodes	1
Number of 2D Outfall Nodes	9
Number of Modelled Pipes	688
Total Modelled Pipe Length (m)	16,631
Pipe / Culvert Size (mm)	100 – 2900
Number of River Reaches	21
Number of Sub-Catchments	469
Total Sub-Catchment Area (ha)	132
Average Sub-Catchment Size (ha)	0.281

3.6 Data Issues

The following data issues were identified and resolved during the model build process:

- Approximately 9 % of the pipe invert levels were missing from the GIS dataset. The missing invert levels were interpolated, inferred, or assumed.
- The inlets and connecting sump leads had no information held within the GIS datasets. Ground levels have been calculated using LiDAR data, pipe diameters have been assumed to be DN225, and invert levels have been assumed based on adjacent invert levels.

3.7 Assumptions

A number of assumptions were agreed with RLC in order to simplify the model build process. The impact of these assumptions on the model outputs are discussed below.

3.7.1 Culvert Inlet Losses

Where possible, inlet losses have been based on survey photos.

3.7.2 Pervious Runoff

All sub-catchment CN values have been set based on SCS CN curve guidance.

3.7.3 Baseflow

Baseflow has not been added to any sub-catchments as it is likely to be insignificant when compared to stormwater runoff during significant rainfall events.

3.7.4 Soak holes

Soak holes / soak pits have not been modelled.

4 Model Sensibility Checks

4.1 Sensibility Checks

The model will not be calibrated against observed river levels, flow or rainfall data under the current scope of works, as no available gauge information exists within this catchment.

However, sensibility checks have been undertaken to ensure that the model data appears appropriate and is suitable for the intended purpose. The model outputs were checked against the following:

- Audit / visual review of model asset data;
- Observed and anecdotal evidence at Carroll Place and Morey Street for the August 2014 storm event;
- Mass balance checks; and
- Rational Method runoff checks.

4.2 Storm Event Validation

The model reliability has been tested by comparing the predicted stream levels against observed levels for a recent storm event with known rainfall. From Tuesday 19th through to Wednesday 20th August 2014, Rotorua experienced heavy rainfall causing levels within the main channel to almost overtop its banks at Carroll Place at the downstream end of the network.

Photos of the stream level by the Carroll Place culvert were provided by RLC, along with anecdotal evidence of the operation of the Morey Street culverts. The flood map shown in Appendix B shows the extent of the predicted flooding for this event.

As a result of this exercise, the percentage rainfall applied to the 2D zone was reduced from an initial 60% (as was used for Catchment 5) to 30%. This value relates to the curve number for Meadow (United States Department of Agriculture, 1986) and is deemed more appropriate for the rural area.

4.2.1 Rainfall

Rainfall for the event was taken from records for the Kaituna at Whakarewarewa rain gauge. This rain gauge is located approximately 2 km from Catchment 4, and due to potential spatial variation in rainfall intensity, can only provide an approximate rainfall profile to input into the model. The event occurred on the 20th August 2014, peaking at 09:10am with a recorded peak intensity of 84 mm/hr and a total depth of 23 mm. Figure 4-1 shows the recorded rainfall for the event.

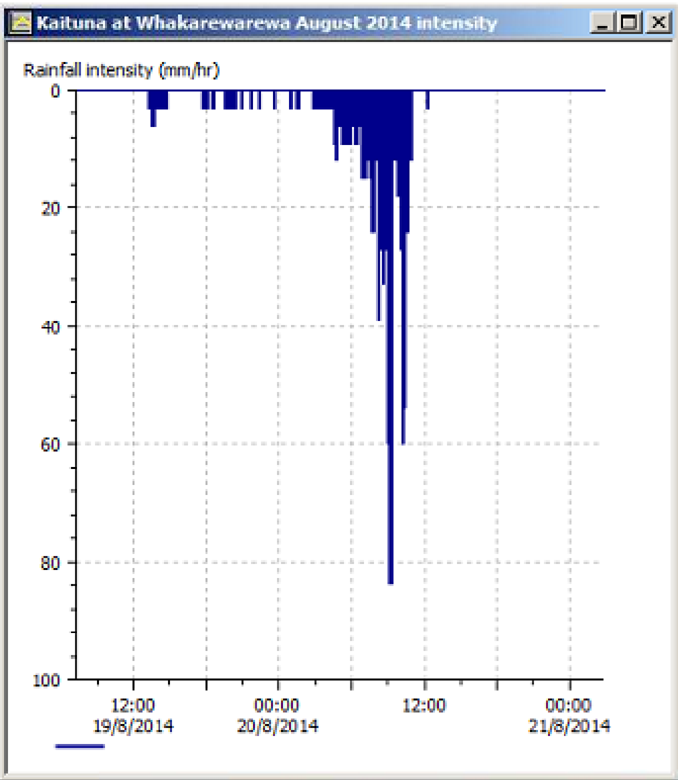

Figure 4-1: Observed rainfall between 19/08/14 and 21/08/14 used for model runs.

4.2.2 Carroll Place Culvert

Figure 4-2 and Figure 4-3 provide a comparison between the observed stream levels and those predicted in the hydraulic model. With the reduced, yet more appropriate, applied rainfall percentage, a comparison between the model and photographs suggests that the model may be slightly underestimating flows in this area, although without recorded water levels, there is some degree of uncertainty.

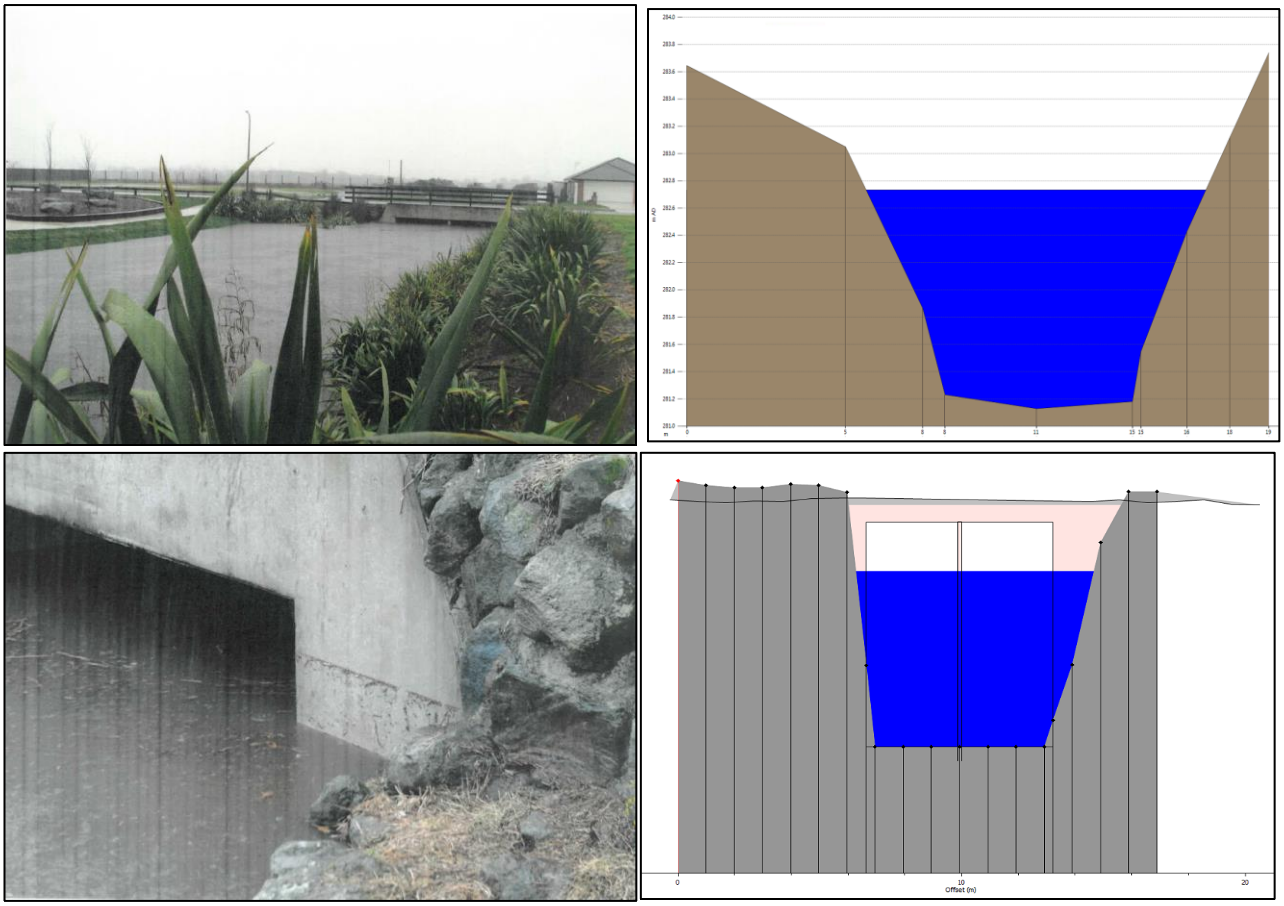

Figure 4-2: Comparison of Stream Levels at the Carroll Place Culvert

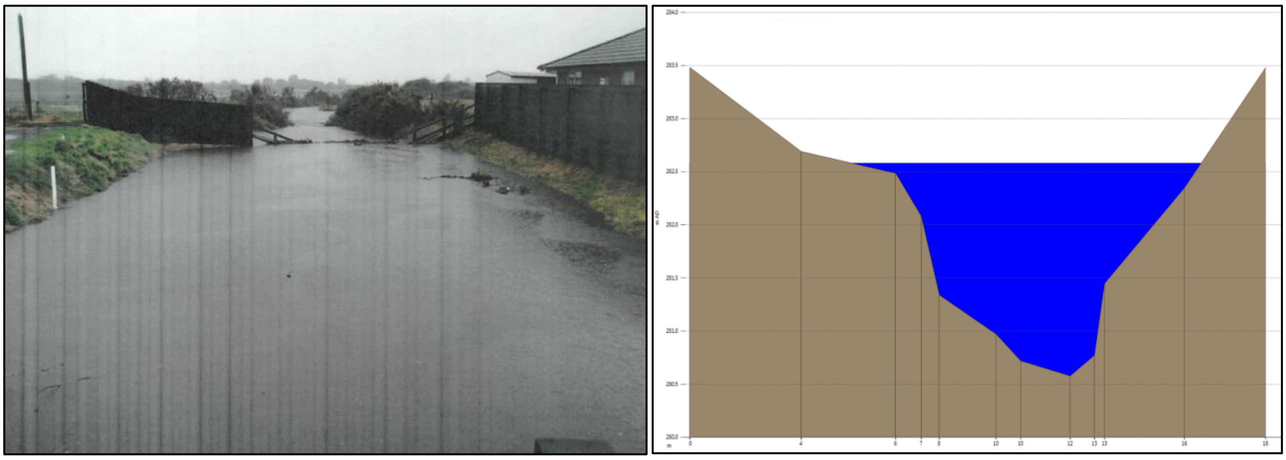

Figure 4-3: Comparison of Stream Levels downstream of Carroll Place

4.2.3 Morey Street Culverts

Anecdotal evidence provided by RLC suggested that both the Morey Street culverts were coping with the rural overland flow during this event. The maximum predicted flow depths through the two culverts for the storm event support this as there was still freeboard available at peak water levels. The northern culvert had 0.46 m freeboard at peak water level and the southern had 0.26 m freeboard. Both freeboard levels are at the culvert outlets. Predicted ponding upstream of the culverts is shown in Appendix B.

4.3 Mass Balance Checks

Cumulative mass balance checks are automatically undertaken by ICM's software engine at each simulation time step. If the cumulative mass balance error exceeds 0.01 m^3 at any time step, the simulation is automatically terminated. Thereby, any completed simulation can be considered to have passed this check.

Following the successful completion of the simulation, the simulation log file identifies the volume balance for each node within the network, and as a total for the whole simulation. The volume balances for the 24-hour duration nested storm simulations are shown in Table 4-1.

Table 4-1: Summary of % Volume Balance

Design Storm Event (AEP)	1D Volume Balance		2D Volume Balance	
	m^3	%	Mass Error Balance (%)	Total Mass Error (m3)
10%	-2.67	0.03	0	0
2%	-2.38	0.009	0	0

4.4 Rational Method Runoff Checks

The runoff predicted during the 10% and 2% AEP nested storms were compared against the runoff generated using the Rational Method as a manual check of the model hydrology. A sample of six sub-catchments were chosen, covering residential, open spaces and road areas.

The results showed a reasonable correlation between the predicted and calculated runoff. The results are shown in Table 4-2.

Table 4-2: Summary of Rational Method Check

Sub-catchment ID	10% AEP Difference	20% AEP Difference
DI26457	-2%	14%
DI125207	-4%	12%
DH000751	-3%	13%
DH000775	-2%	14%
DI125529	-2%	-2%
DI000600!	-19%	10%
Area-Weighted Average – Difference	**-5%**	**10%**

5 System Performance

System performance was assessed for both the 10% and 2% AEP 24-hour nested storms with climate change, see Figure 5-1. The nested storms have been generated using HIRDS v3 rainfall data (NIWA, n.d.). Climate change has been accounted for using a temperature increase of 2.1ºC.

All system performance maps can be found in Appendix C; these include flood depth, parcels with flood depths greater than 300 mm, and flood hazard maps.

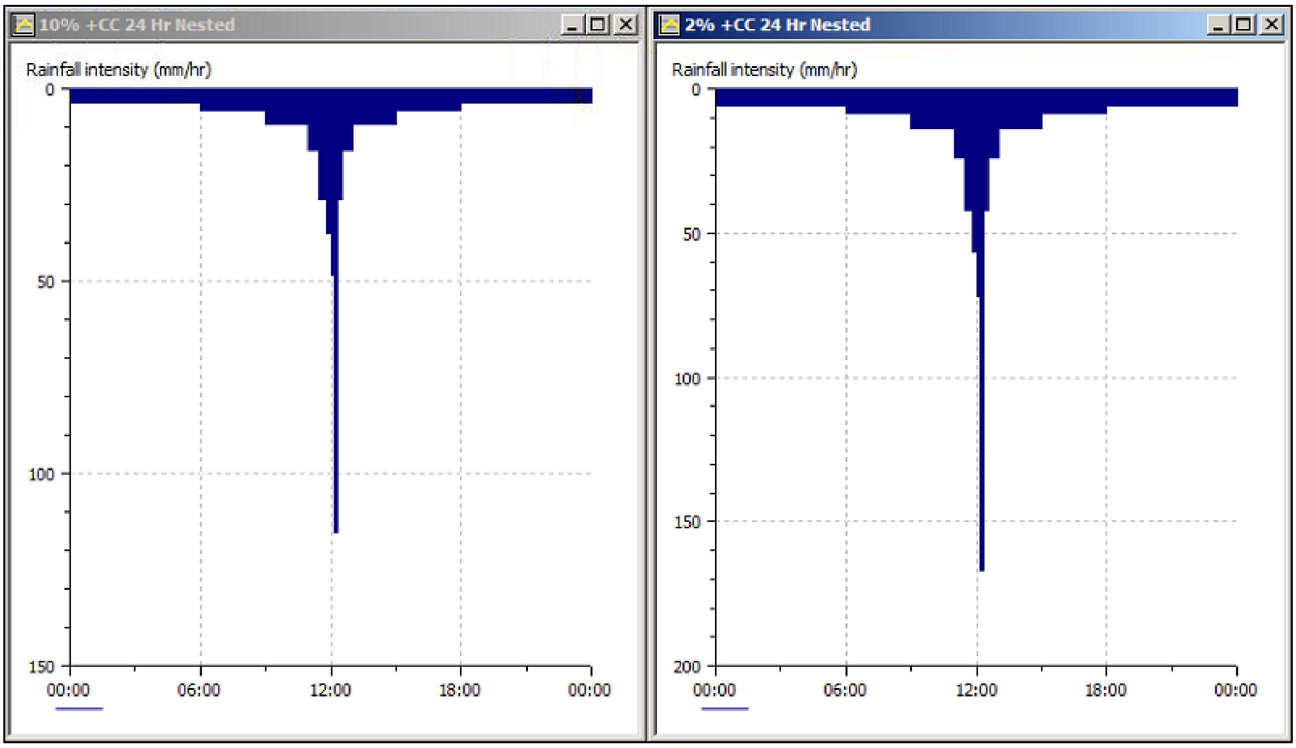

Figure 5-1: 24 hour nested storm with climate change (left 10% AEP, right 2% AEP)

5.1 Predicted Flood Depths

Table 5-1 summarises the total ponding areas by depth of ponding excluding those areas that have been modelled as river reaches. Flood maps in Appendix C show these ponding locations spatially. In these maps, flows within the open channels modelled as river reaches are assumed to be greater than 300 mm deep. For reference, the total modelled catchment size is 527 ha.

Table 5-1: 2D ponding depths

Event	Ponding depth areas (ha)			
	≥ 50 mm	≥ 150 mm	≥ 300 mm	Total
10% AEP	26.5	40.7	32.4	67.2 (12%)
2% AEP	39	52.2	37.3	91.2 (17%)

5.2 Parcels with Predicted Ponding

Parcels that intersected any ponding greater than 300 mm were identified. This excluded open channels modelled as river reaches unless there was out-of-bank flow causing significant ponding. The number of parcels with ponding greater than 300 mm are shown in Table 5-2. For reference, the total number of parcels within or intersecting the Catchment 4 boundary is 1,263. The locations of these parcels are shown in Appendix C together with the extent of ponding.

Table 5-2: Parcels with ponding greater than 300 mm

Event	Parcels with Ponding greater than 300 mm	Percentage of total Parcels (%)	Developed Parcels with Ponding greater than 300 mm
10% AEP	44	3.5	9
2% AEP	72	5.7	30

The plans show that while there is a considerable area of predicted ponding, in the urban area the ponding is mainly either in areas of open space, or contained within the road networks, with few developed parcels predicted to have habitable floor flooding (>300 mm depth on parcel) during the 10% AEP storm event.

5.3 10% AEP Predicted Flooding

While RLC note that there are no reported flooding or ponding complaints within Catchment 4, the model does predict flooding within developed parcels. The following section provides detail on the mechanisms of the predicted flooding within the developed parcels. It should be noted that only flooded parcels are identified, not flooded properties, as floor levels are not known at this point in time. In addition, any potential impediments to overland flow, such as property boundary walls or fences have not been included in the model at this stage.

5.3.1 Melrose Avenue

During the 10% AEP storm event, eight of the nine developed parcels with predicted ponding depths greater than 300 mm are adjacent to the narrower, timber-lined section of the stream between Melrose Avenue and Alastair Avenue, as shown in Figure 5-2.

The ponding at this location is predominantly due to a lack of capacity within the stream itself, but flooding depths are exacerbated by overland flows running down Alastair Avenue, which in turn originate from the surcharged stormwater system on McKenzie Road.

In order to model the existence of the property fences on this stretch of the stream, the discharge coefficient of the modelled bank lines has been reduced from 1 to 0.8, based on advice from Innovyze. It is recommended that some sensitivity testing of the effect of further reducing this coefficient is undertaken. It may also be prudent to test the application of porous walls close to the bank lines to further account for the presence of the property fences.

Figure 5-2: Predicted Ponding at Melrose Avenue (10% AEP)

5.3.2 Warwick Drive

The remaining parcel with predicted ponding is a rear parcel on Warwick Drive (Figure 5-3). The ponding here is initially due to surcharge within the local stormwater network causing flooding to the rear of properties on Stanley Drive and on Warwick Drive itself. This occurs at the peak of the nested storm. Ponding is later exacerbated by overland flow from the surcharged stormwater system on Basley Road.

The surcharge within the stormwater system in this area is predominantly due to pipe size reductions. On Lynwood place a DN900 reduces down to a DN675 and at the junction between Lynwood Place and Basley Road a DN900 reduces down to a DN 700. In addition, there is increased headloss through the two DN900 pipes due to their flat gradient. These network issues are all based on received GIS data. It is recommended that the pipe sizes and inverts on this line are confirmed.

Figure 5-3: Predicted Ponding at Warwick Drive (10% AEP)

5.4 Flood Hazard

Flood hazard maps have been produced for emergency planning purposes (see Appendix C), and are intended to provide an indication of the severity of flooding during both the 10% and 2% AEP events. These maps utilise a Hazard Rating (HR) to quantify the flood risk to the public during such an event. The Hazard Rating calculated in ICM is based on the flood flow velocity, depth of flow, and a debris factor, according to the following formula:

$$HR = d \times (v + 0.5) + DF$$

Where:

d = depth of flooding (m)

v = velocity of flood waters (m/s)

DF = debris factor

The full methodology applied is described in “Supplementary Note on Flood Hazard Ratings and Thresholds for Development Planning and Control Purpose” (Surendran et. al. 2008).

Table 5-3 defines the flood hazard ratings used in the emergency planning maps. The relationship between flood hazard rating, flow depth and velocity is illustrated in Figure 5-4.

River reaches were set to extreme hazard which is appropriate for an open channel in flood.

Table 5-3: Flood Hazard Rating Criteria

Thresholds for Flood Hazard Rating	Degree of Flood Hazard	Flood Hazard Description
< 0.75	Low	Caution - flood zone with shallow flowing water or deep standing water
0.75 - 1.25	Moderate	Dangerous for some (i.e. children) - flood zone with deep (< 250 mm) or fast flowing water
1.25 - 2.0	Significant	Dangerous for most people - flood zone with deep (250 mm – 500 mm), fast flowing water
> 2.0	Extreme	Dangerous for all - flood zone with deep (500 mm or greater), fast flowing water

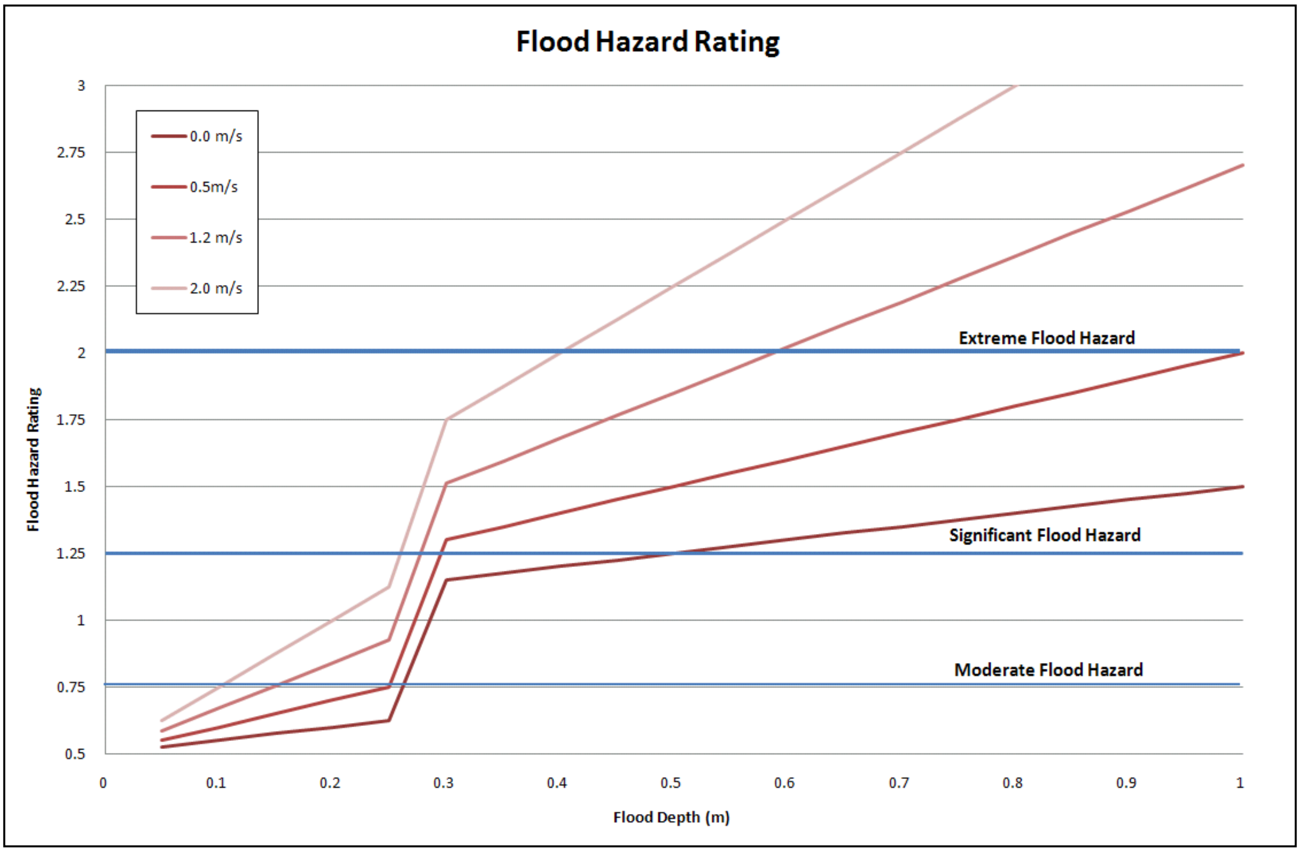

Figure 5-4: Flood Hazard Rating details

5.5 Stream Velocities

High velocities in the downstream reaches of the open channel, which had caused eroding of the river banks, was identified by RLC as a priority issue within Catchment 4.

Predicted velocities in the stream between the Vaughan Road and Carroll Place culverts are generally between 1 and 2 m/s which, dependant on the bed and bank geology and vegetation, could cause erosion issues. The model also predicts higher velocities locally at the exit of the Vaughan Road culvert, and on the upstream side of the culvert where there is a concrete ramp structure. Figure 5-5 shows the predicted velocities within the modelled river reach for a 2% AEP storm. Note that higher velocities could occur during more frequent events where less of a backwater condition from the lake is present or the flow remains in-channel. Localised effects may also be causing erosion, for example obstructions into the channel causing localised scour.

Figure 5-5: Predicted Velocities in Modelled Stream

5.6 Wharenui Road Development Area

There is a large area of currently undeveloped rural land in the south-east of the catchment, bordered by Morey Street, Basley Road and Wharenui Road. This area is zoned for development with around 880 dwellings proposed, plus a small commercial area.

The Wharenui Road Development Plan (Rotorua Lakes Council, 2016) states that “all new subdivisions shall be designed for attenuation of the 2% AEP flood peak flows from individual sub-catchments to be less than or equal to pre-development peaks”. There is no specification as to the duration of that peak flow, so it is assumed it should cover all durations including the critical duration for peak flow and attenuated volume. The use of in- or off-line attenuation for storm runoff may limit the peak flows, but will cause periods of high flows for longer and could worsen any existing erosion issues.

The proposed development area has fairly steep topography with two separate valleys conveying overland flow towards two culverts beneath Morey Street. Figure 5-6 shows the proposed development layout overlaid with the predicted pre-development 2% AEP nested storm flood depths.

This shows that the overland flow paths, in general, follow the areas designated as green space, although there are some minor flow paths through the areas to be developed. In addition, overtopping of Morey Street at both culverts is predicted, thus any additional flows from the development will exacerbate this.

Any attenuation systems located within these flow paths would need to cater for the upstream flows, and preferably they would be constructed off-line of the flow paths.

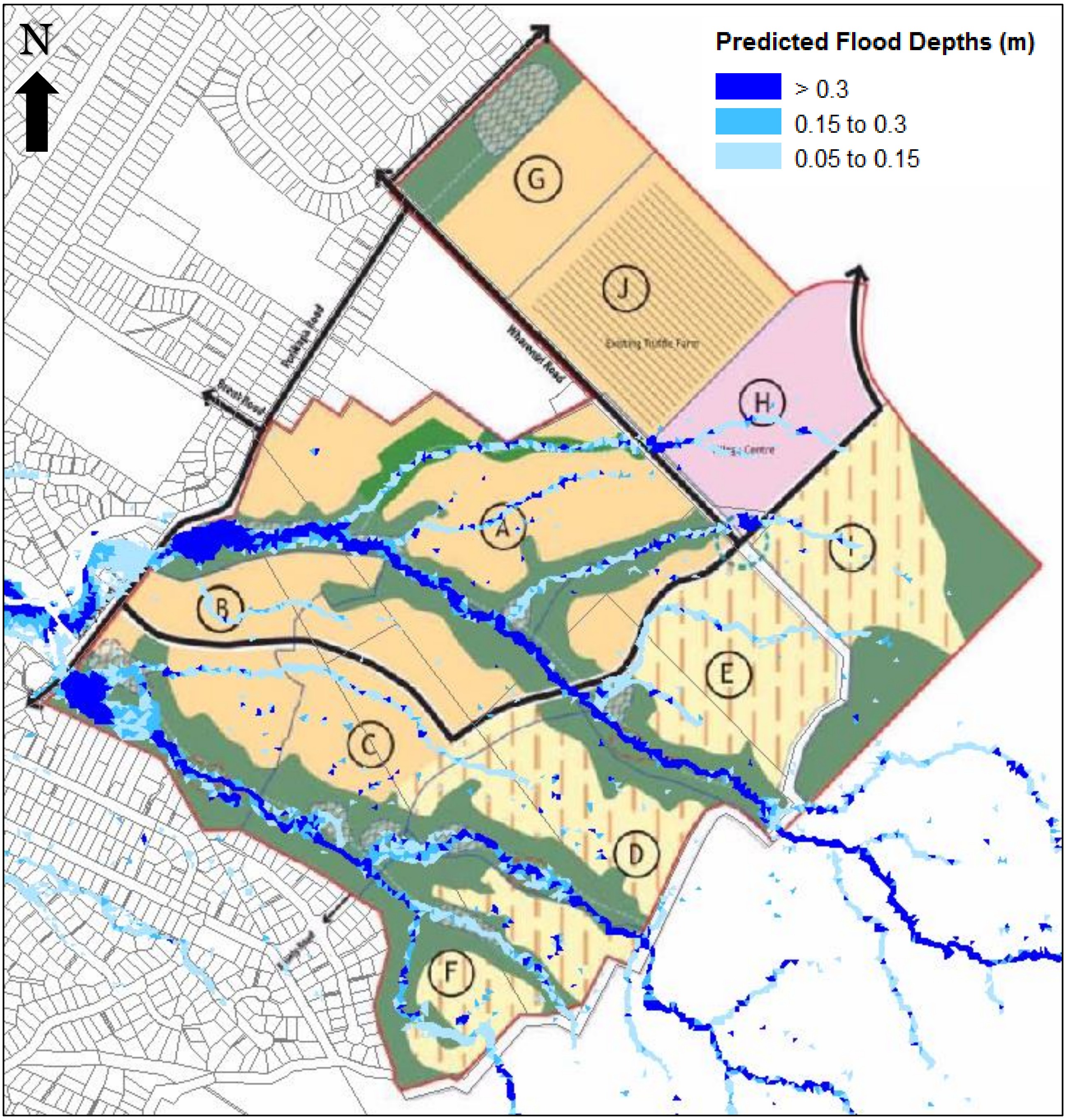

Figure 5-6: Development Plan Overlaid with Predicted 2% AEP Flood Depths

6 Model Confidence and Recommendations

6.1 Model Confidence

Based on the quality of the survey data available to build the model and a reasonable correlation between predicted and observed levels within the stream, the model is thought to be the best tool currently available to RLC to provide inputs to the options assessment against the overarching Levels of Service identified within the Infrastructure Strategy 2015-2045.

However, the model results are likely to be highly dependent on factors such as antecedent rainfall (catchment wetness). Further sensitivity analysis could be used to confirm areas within the model where the results are largely independent of parameter changes. The model can then be used in these areas with higher certainty for planning purposes and decision making.

Improving model confidence in areas with lower certainty could be targeted and improved at a later date by calibrating the model against recorded flow and rainfall data. This would provide greater confidence in the pervious, baseflow, soakage and hydraulic assumptions.

6.2 Recommendations

The following additional model enhancements or investigations are recommended:

- **Confirmation of Pipe Diameters on Warwick Drive to Basley Road line** – significant surcharge is predicted in this line due to flat gradients and two reductions in size from DN900 to DN675 and DN900 to DN750. It is recommended that these are confirmed prior to any flood alleviation works.
- **Joining of Catchment 4 with Catchment 5** – Overland flow from Catchment 4 is predicted to enter the stream between Te Ngae Road and Vaughan Road, on the southern boundary between the two catchments. This stream is piped for a short section south-west along Vaughan Road as part of the Catchment 5 network where significant ponding was predicted by the Catchment 5 modelling study. It is recommended that simulations are undertaken with the two catchments joined together, to fully determine the interactions between the catchments.
- **Sensitivity testing on Sump Lead size** – It has been assumed that sump leads are of DN225 diameter, unless identified as a double sump, in which case they have been modelled as DN300. Sensitivity testing could be undertaken by upsizing all sump leads to DN300 as this would produce more conservative trunk main flows.
- **Sensitivity Testing on Obstructions** – The topographical survey identified several fences over the timber-lined section of the stream, upstream of the Te Ngae / Melrose culvert. While under normal operating conditions, these are unlikely to represent any flow restrictions, however there is the potential for them to collect debris and create a localised headloss / flow restriction. A sensitivity test to see the effect on the predicted flood extents as a result of a blockage is recommended. Photos of the fences over the stream are provided in Figure 9-5 in Appendix A.
- **Sensitivity Testing of Bank Line Discharge Co-efficient** – the bank line discharge coefficient on the timber-lined section has been reduced from the standard value of 1 to 0.8 to account for the presence of property fences on this section of the stream. It is recommended that further sensitivity testing is undertaken to determine the effect of further reducing this value.

- **Inclusion of Wharenui Road Development** – Once the proposed stormwater network layout is available, it is recommended that this is incorporated in the hydraulic model. This should also include all proposed attenuation basins and any swales / streams.

In addition, the following sensitivity analyses could be undertaken:

- **Hydrology** – The SCS curve applied to pervious areas could be set higher or lower and initial loss / antecedent condition sensitivity tested;
- **Manhole Headloss** – Set the headloss curve to High instead of Normal;
- **Roughness** – Surface roughness can be increased or decreased;
- **Boundary Conditions** – Check impact of varying lake levels on the catchment; this could affect predicted channel velocity in the lower reaches.

These analyses will indicate how sensitive the model results are to changes in the model parameters.

7 High-level Options for Consideration

As part of the next stage of the project scope for Catchment 4, three key issue areas have been identified where remediation options can be assessed. Following a workshop with RLC to discuss the viability and suitability of these options, a maximum of two high-level options will be implemented in the model to assess their performance on the identified issues. A brief memorandum will be produced to present the options and their predicted performance.

The locations where high-level options should be considered for Catchment 4 are shown in Figure 7-1. Some preliminary ideas for possible high-level options to resolve the issues are presented in the sections below.

Figure 7-1: Locations of Potential Options

7.1 High Stream Velocities

This section between Vaughan Road and Carroll Place has issues with high velocities that are causing erosion of the channel. Potential options would require a solution that will both increase the channel capacity and slow down flows, i.e. channel widening.

7.2 Timber-lined Stream Section

This section involves a narrow timber-lined channel in between residential properties. The section also involves multiple fences across the channel which could cause hydraulic restrictions during high flows. A number of properties are predicted to flood along this narrow section of stream. A remediation option may consider providing additional capacity on-line or storage in the open space upstream of this area.

7.3 Attenuation of Rural Flows

In order to resolve issues downstream in the urban area, an option may consider attenuation of the rural flows from the catchment upstream of the culverts between Morey Street in formal attenuation structures.

8 References

CIRIA. (2010). *Culvert Design Manual.*

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NIWA. (n.d.). *High Intensity Rainfall System v3.* Retrieved from www.hirds.niwa.co.nz

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Surendran, S., Gibbs, G., Wade, S., & Udale-Clarke, H. (May 2008). *Supplementary Note on Flood Hazard Ratings and Thresholds for Development Planning and Control Purpose, Clarification of the Table 13.1 of FD2320/TR2 and Figure 3.2 of FD2321/TR1.*

United States Department of Agriculture. (1986). *Urban Hydology for Small Watersheds (TR-55).*

9 Appendices

Appendix A – March 2017 Survey Photos

Appendix B – Model Validation Map

Appendix C – System Performance Maps

Appendix A – Channel Survey Photos

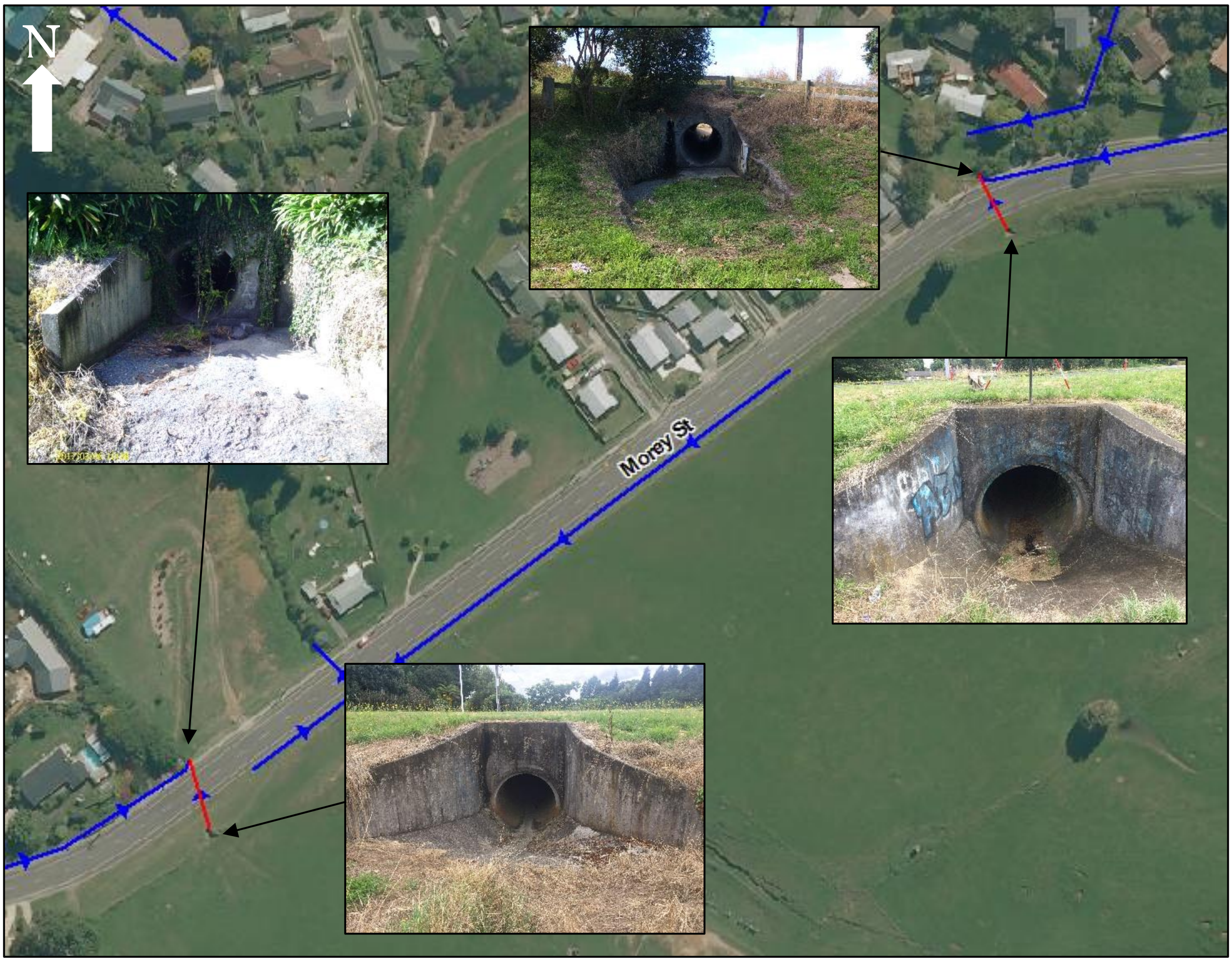


Figure 9-1: Morey Street Culverts

Figure 9-2: Timber-lined Section

Figure 9-3: Te Ngae/Melrose Culvert

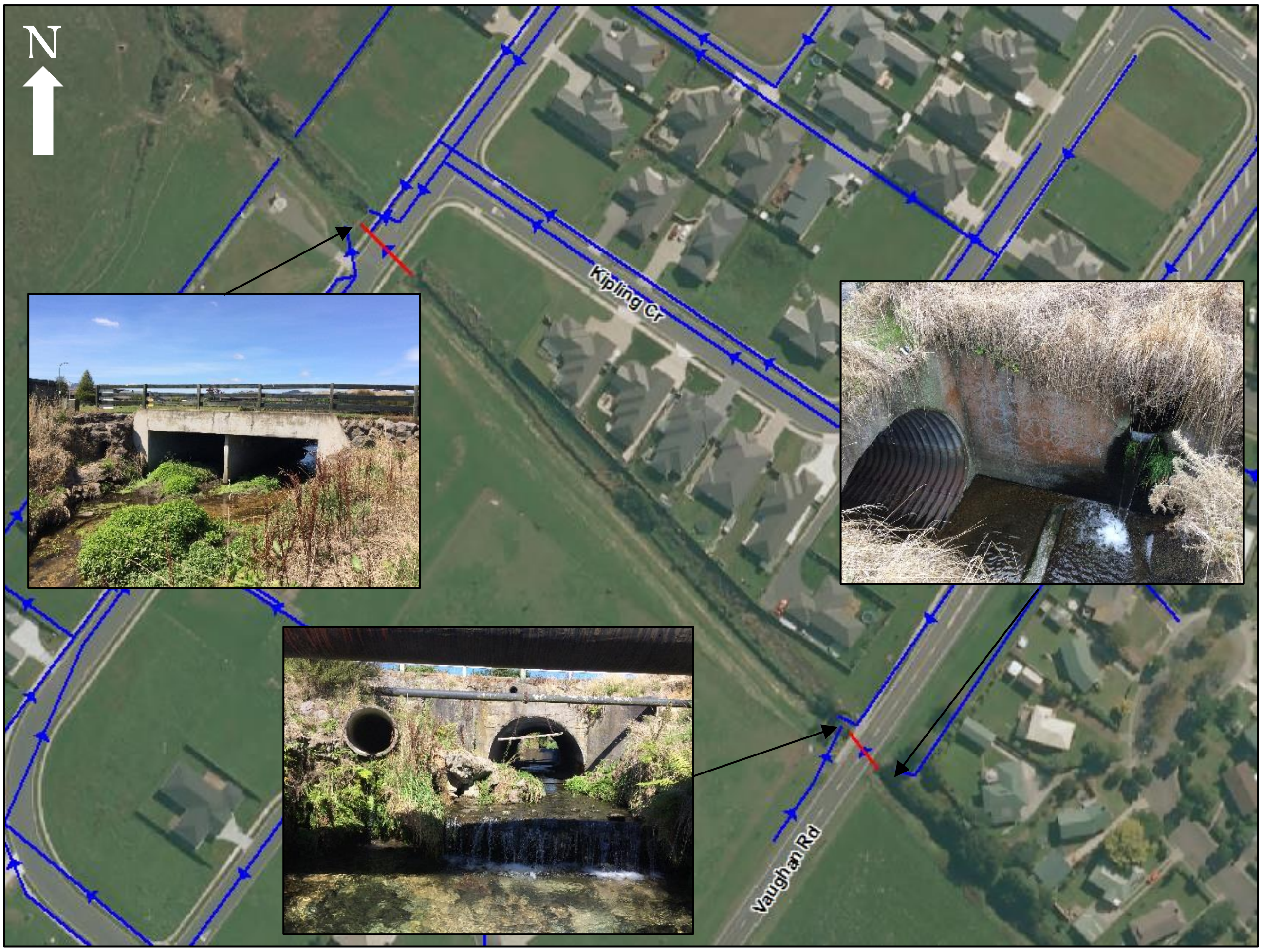
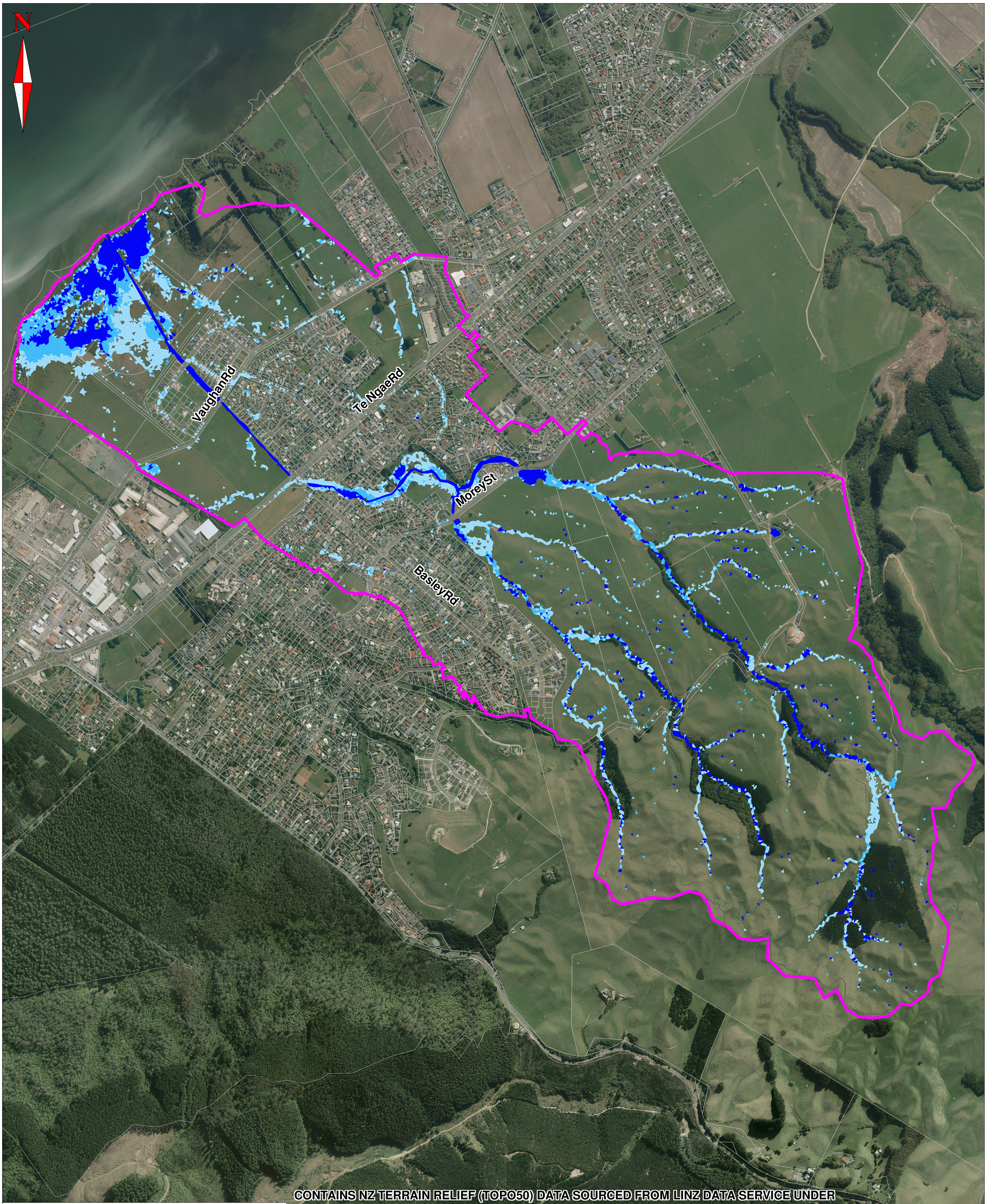


Figure 9-4: Vaughan Road and Carroll Place Culverts

Figure 9-5: Potential Flow Restrictions in Timber-Lined Section

Appendix B – Model Validation Map



LEGEND

Modelled Boundary

Parcel Boundary

Flood Depth (m)

Greater than 0.3

0.15 to 0.3

0.05 to 0.15

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DRAWN	APPROVED	REV	REV DATE	PLAN
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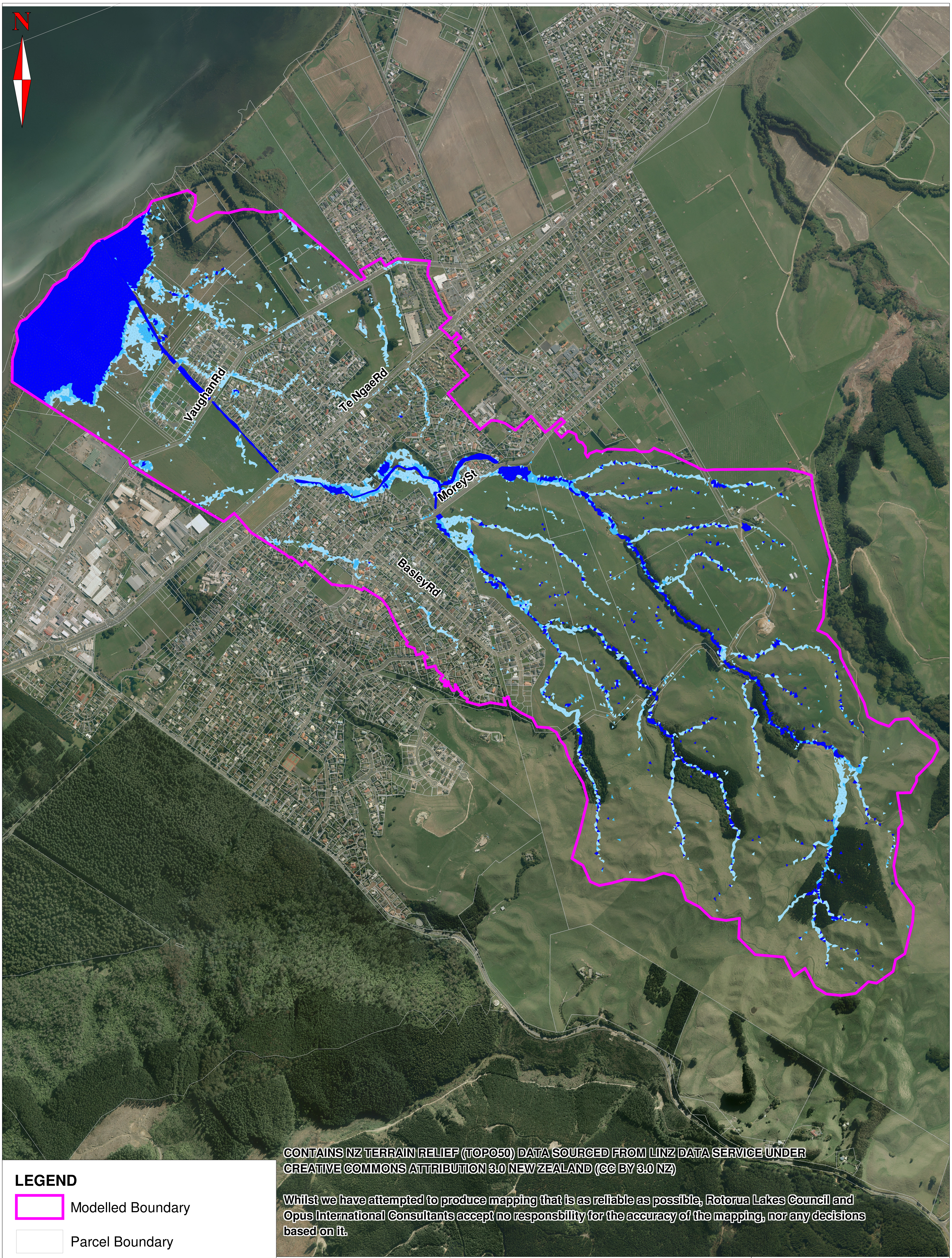
Appendix C – System Performance Maps



LEGEND

Modelled Boundary

Parcel Boundary

Flood Depth (m)

Greater than 0.3

0.15 to 0.3

0.05 to 0.15

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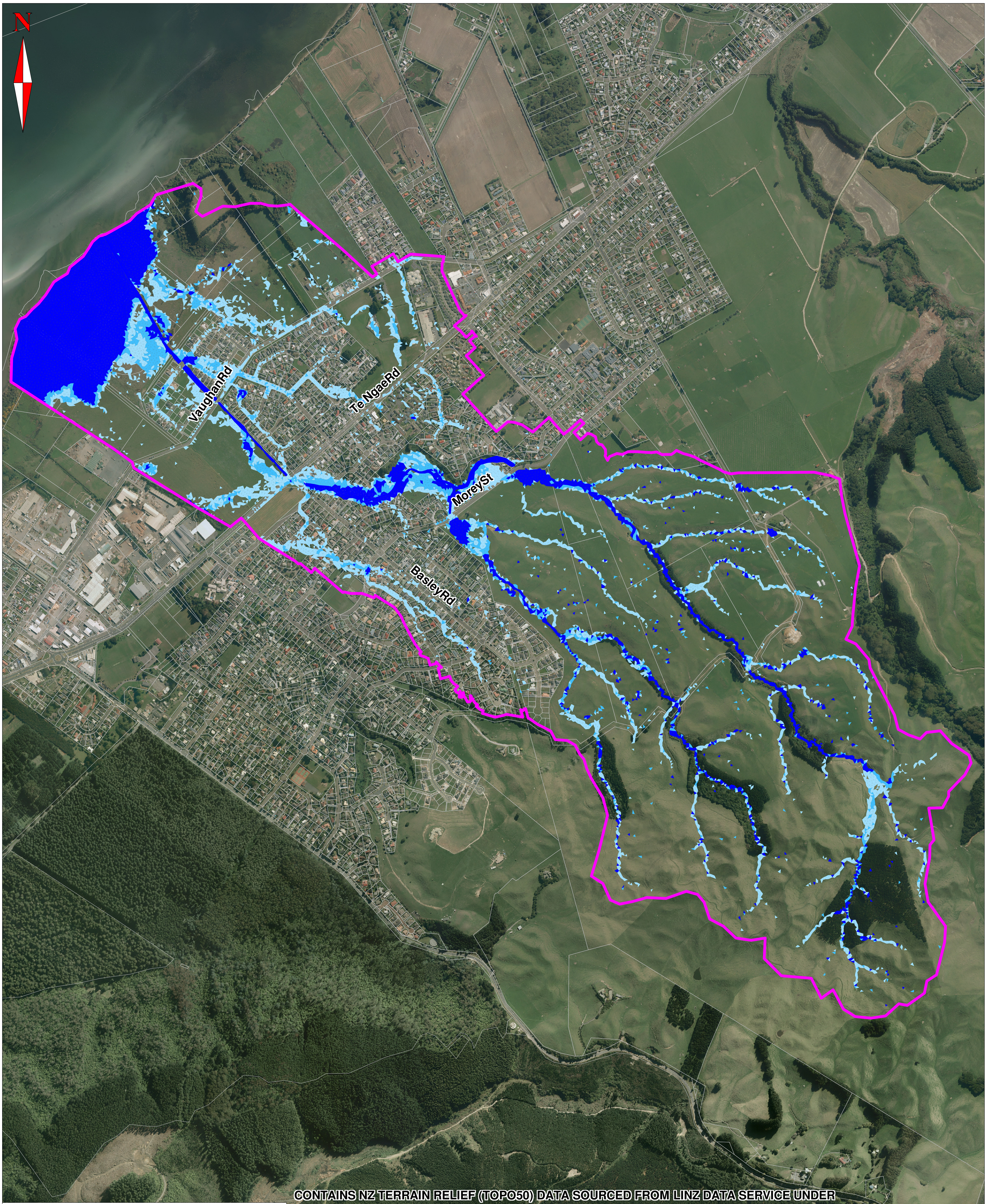

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LEGEND

Modelled Boundary

Parcel Boundary

Flood Depth (m)

Greater than 0.3

0.15 to 0.3

0.05 to 0.15

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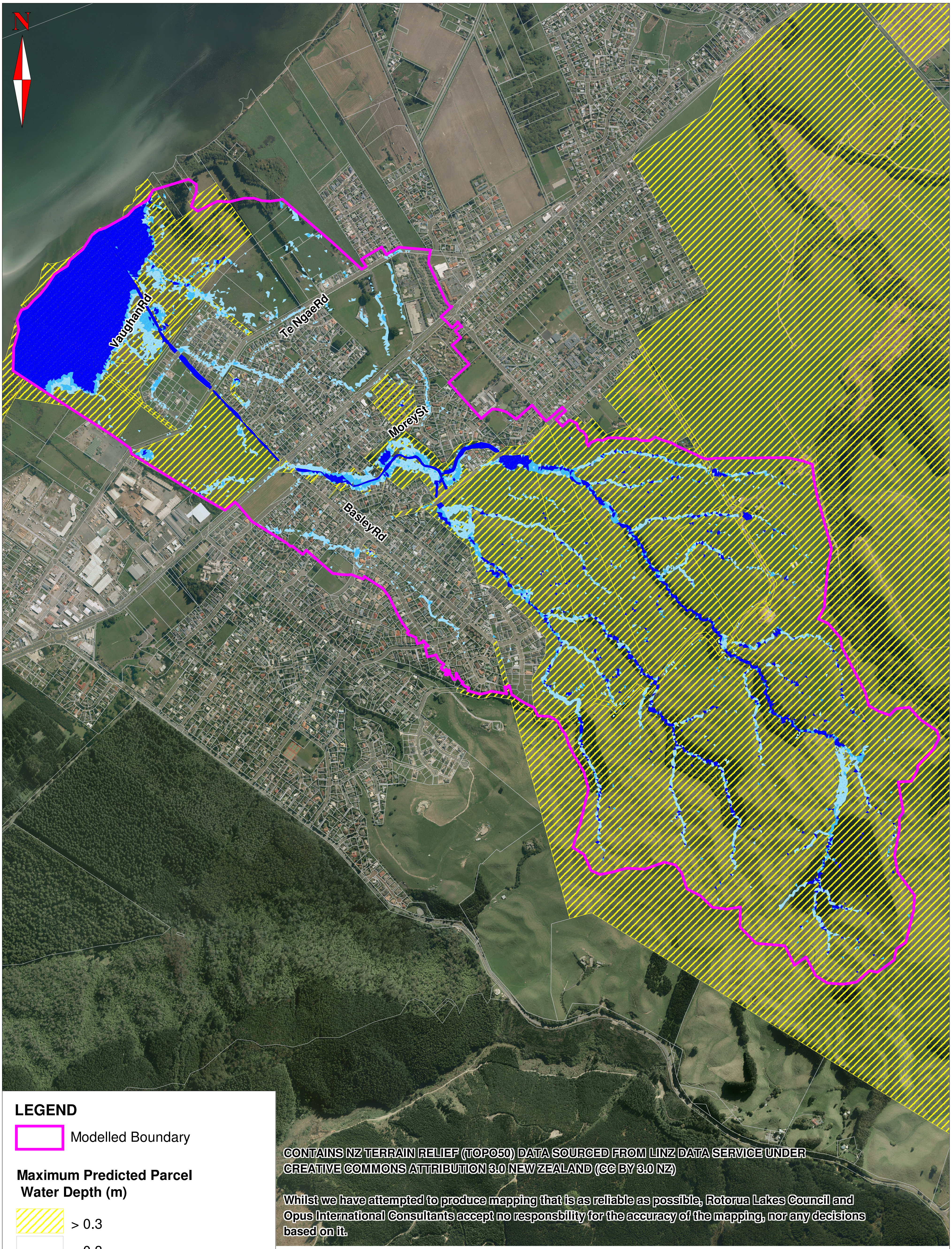

LEGEND

Modelled Boundary

Maximum Predicted Parcel Water Depth (m)

- > 0.3
- < 0.3

Predicted Flood Depth (m)

- Greater than 0.3
- 0.15 to 0.3
- 0.05 to 0.15

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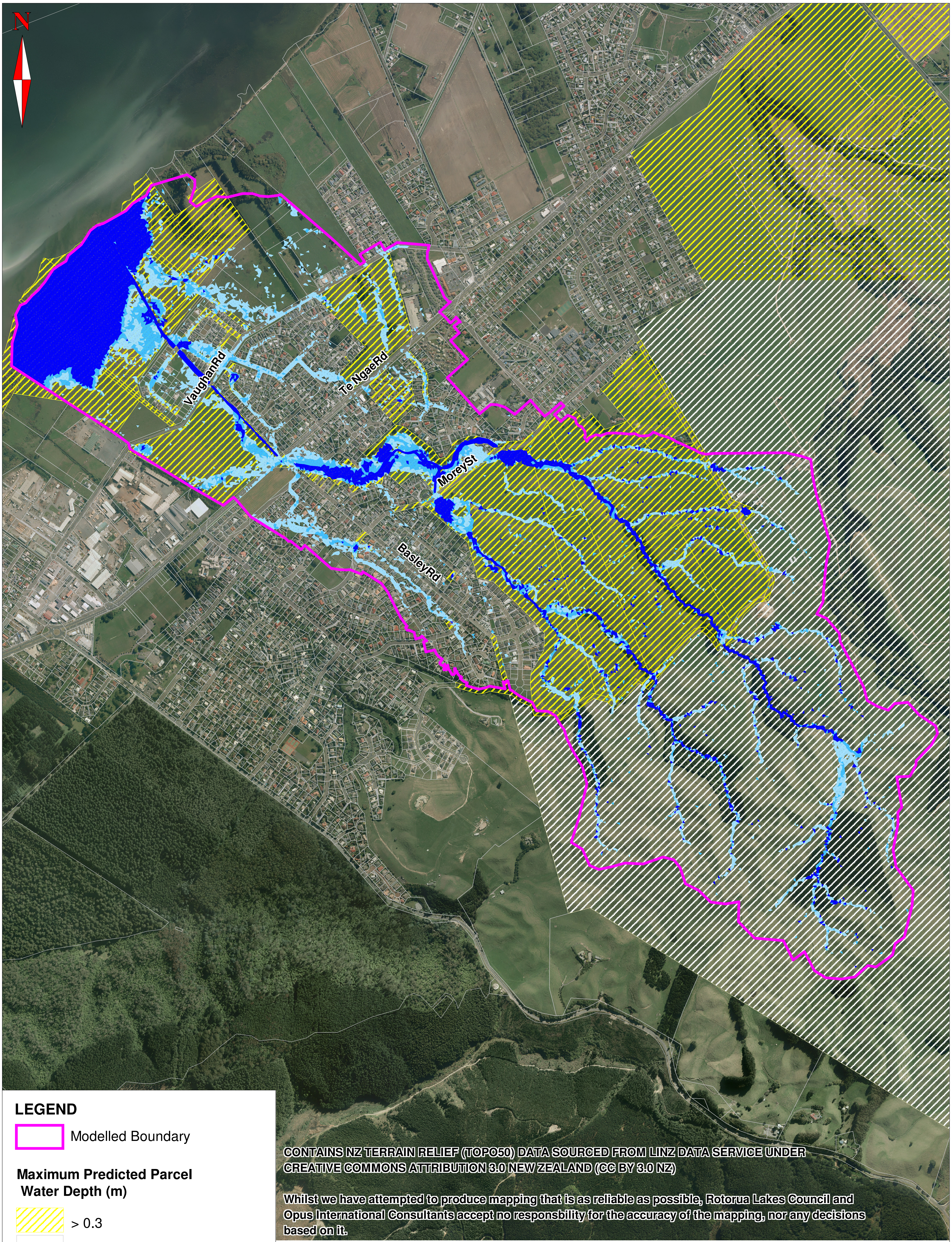

LEGEND

Modelled Boundary

Maximum Predicted Parcel Water Depth (m)

> 0.3

< 0.3

Predicted Flood Depth (m)

Greater than 0.3

0.15 to 0.3

0.05 to 0.15

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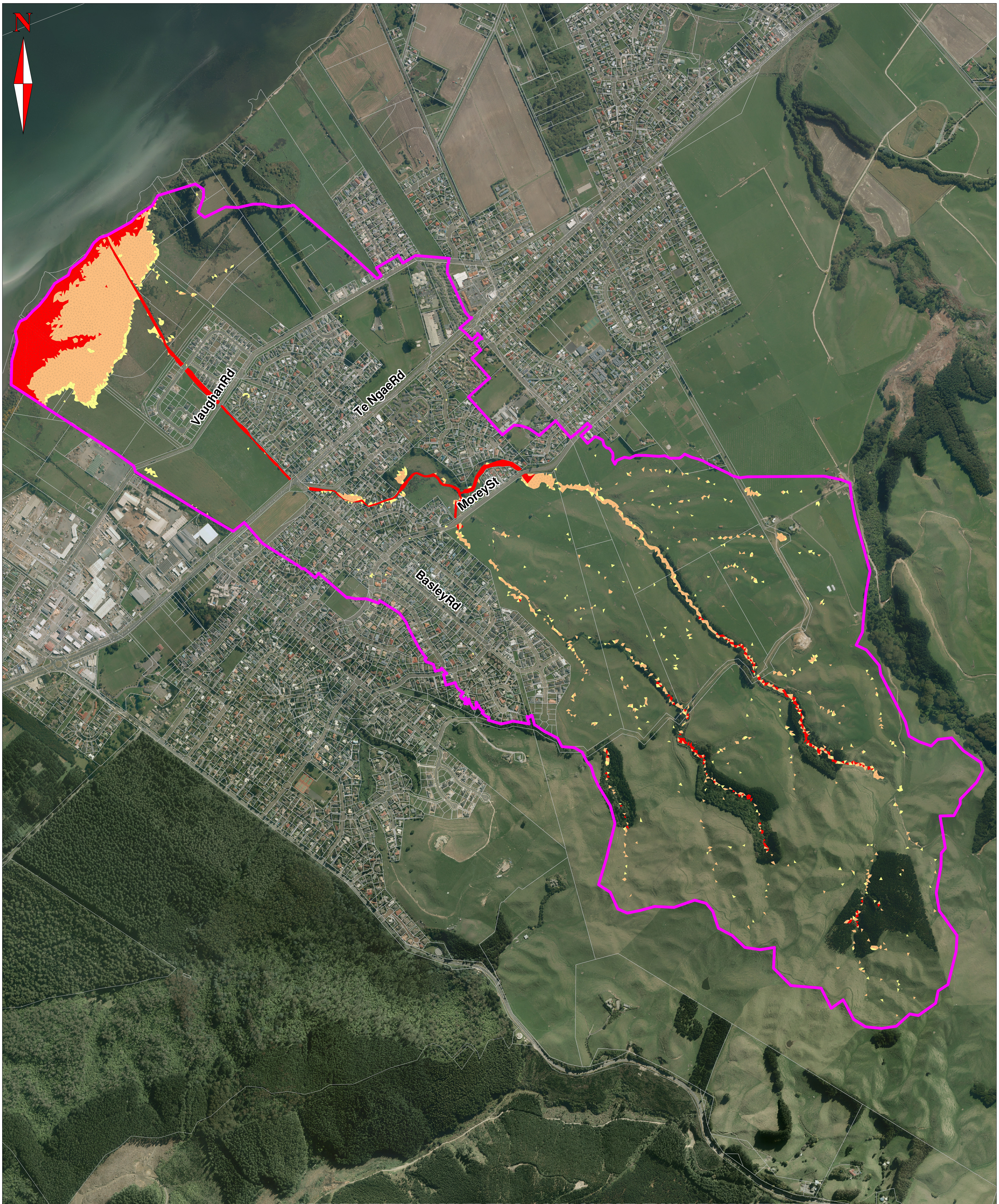

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LEGEND

Modelled Boundary

Parcel Boundary

Hazard Classification

> 2 (Extreme)

1.25 to 2 (Significant)

0.75 to 1.25 (Moderate)

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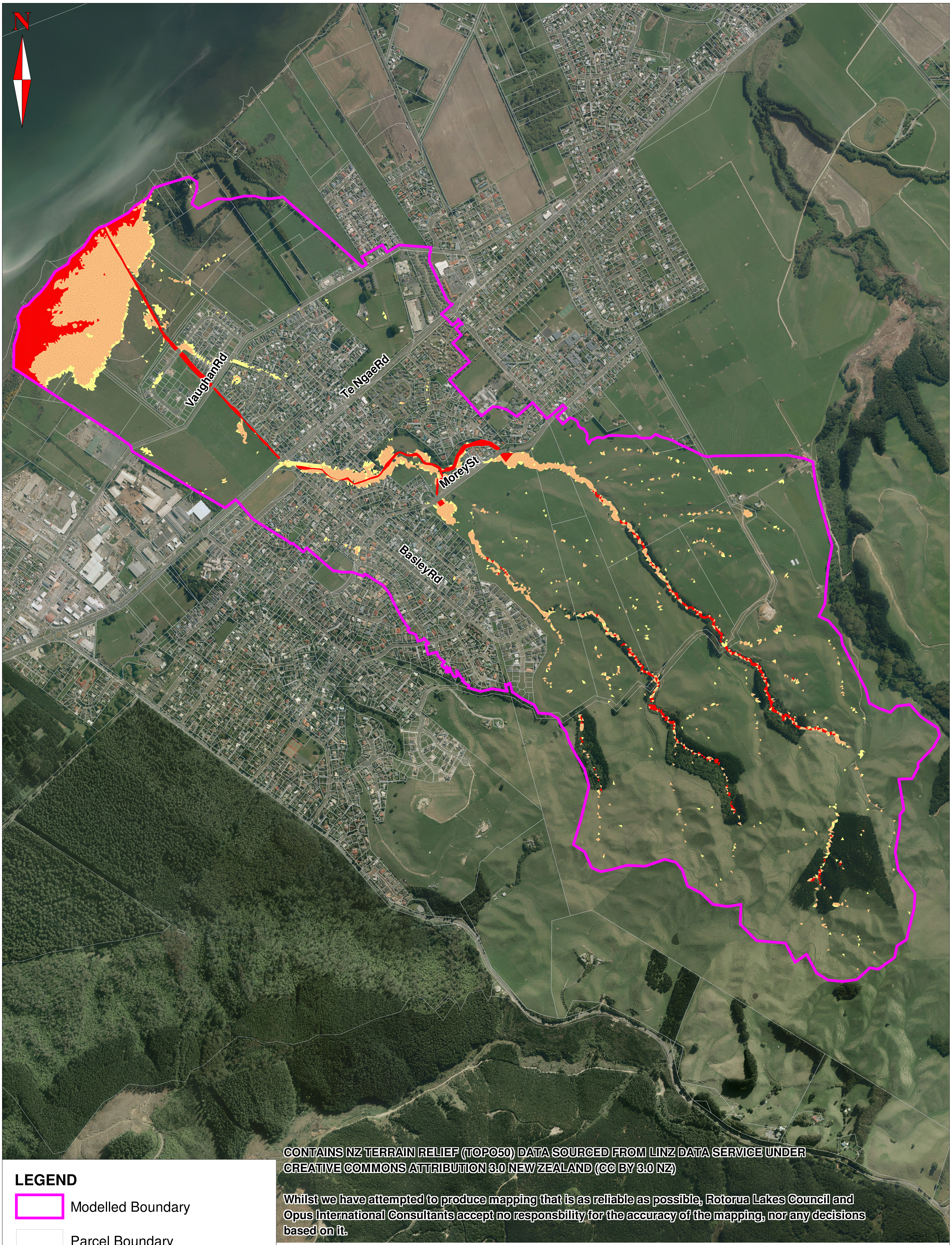

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LEGEND

Modelled Boundary

Parcel Boundary

Hazard Classification

> 2 (Extreme)

1.25 to 2 (Significant)

0.75 to 1.25 (Moderate)

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