

## Official Height Standard Change

From 1 July 2024, Auckland Council adopts the official height standard for New Zealand called New Zealand Vertical Datum 2016 (NZVD2016).

This model was carried out prior to the height standard change.

**All levels included in this modelling report are in Auckland Vertical Datum 1946 (AUK1946/AVD1946).**

Levels in this report can be transformed from Auckland Vertical Datum 1946 into New Zealand Vertical Datum 2016 by applying an offset value of **0.277 m**.

For example:

$$H_{NZVD2016} = H_{AVD1946} - \text{Offset Value}$$

A single offset value for the catchment has been taken from the Land Information New Zealand (LINZ) Auckland 1946 to NZVD2016 Conversion Raster therefore this offset should be taken as an approximation only for the catchment.

A more accurate height transformation value can be derived by downloading the conversion raster available on the LINZ website below:

<https://data.linz.govt.nz/layer/103953-auckland-1946-to-nzvd2016-conversion-raster/>

# **OTEHA VALLEY CATCHMENT STORMWATER MODELLING**

## **MODEL BUILD AND SYSTEM PERFORMANCE REPORT**

**IMPORTANT!!**

**This document consists of three PDFs:**

- 1. PDF – Main Report and Figures**
- 2. PDF – Appendices A to G**
- 3. PDF – Appendices H to J**

**1. PDF – Main Report and Figures**

# OTEHA VALLEY CATCHMENT STORMWATER MODELLING

## MODEL BUILD AND SYSTEM PERFORMANCE REPORT

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## GLOSSARY

1D	One dimensional means only one spatial dimension is considered i.e. the horizontal direction of flow.
2D	Two dimensional means two spatial dimensions are considered i.e. the horizontal and lateral directions of flow.
ARC	Auckland Regional Council.
Annual Exceedence Probability (AEP)	The probability that a given rainfall event or flow rate will be exceeded in a single year.
Average Recurrence Interval (ARI)	Average period of time between rainfall events or flow rates which exceed a certain magnitude.
Catchment	An area of land draining by force of gravity into a stream or watercourse at a given location.
Climate Change	Climate change resulting from global warming due to greenhouse gas emissions.
CRS	MIKE URBAN modelling feature to assign link cross-sections.
Curve Number (CN)	Defines the shape of the rainfall-runoff relationship and varies from 0 (no runoff) to 100 (complete rainfall).
Design Storm	The rainfall event calculated from historical record that can be expected for a specific AEP or ARI.
Design Flows	The flows estimated from various design storms, selected as a basis for the design of works in watercourses and catchments.
Drainage System	The network of pipes, streams, open watercourses and secondary overland flow paths which carry flow within a catchment.
Existing Development (ED)	The current land use development within the catchment.
Energy Grade Line (EGL)	The total energy of flow at a given location, it is the sum of the elevation head, the pressure head, and the velocity head.
Energy Loss	Energy or head loss occurs due to frictional resistance, contraction and expansion at entrance and exit, change in flow direction, change in elevation and change in cross-section.
Floodplain	The plan extent of flooding in a given AEP or ARI storm.
Flood Sensitive Area	The plan extent of flooding for 500mm (freeboard) above the 100-year ARI flood levels.
Freeboard	Design margin to allow for factors omitted in the overall design (e.g. uncertainties in flood level estimation, wave action, and localised water level variations).
GIS	Geographical Information System.

Hydraulic Grade Line (HGL)	A line coinciding with the level of flowing water in an open channel. In a closed conduit flowing under pressure, the HGL is the level to which water would rise in a vertical tube at any point along the pipe. It is equal to the energy grade line (EGL) elevation minus the velocity head.
Hydrograph	A graph illustrating the variation of flow with time.
Hydrological Soil Group	Soil classification (A, B, C, or D) according to infiltration rate, where A is very high infiltration and D is very poor infiltration.
Initial Abstraction (Ia)	Rainfall losses occurring before runoff begins, includes water retained in surface depressions, intercepted by vegetation, evaporation, and infiltration.
LiDAR	Light Detection and Ranging survey.
Link	Link represents stormwater drainage pipes, culverts, bridges, stream channel reaches or overland flow paths.
Manning's "n"	Manning's roughness coefficient to account for energy losses due to frictional resistance to flow.
Maximum Probable Development (MPD)	The ultimate future land use development which will proceed up to the maximum permitted under the current District Plan.
Node	Node represents the drainage system attributes such as manholes, inlets, outlets, junction between open channels, ponds.
Overland Flow	Stormwater runoff travelling downhill over the surface of the ground along the path of least resistance towards streams and watercourses or the sea.
Primary Drainage System	The pipes, stream networks and open watercourses that carry the main, frequent stormwater within a catchment.
Runoff	The fraction of rainfall which runs off the land surface to the drainage system.
Secondary Drainage System	The overland flow path that carry the excess stormwater when the capacity of the primary drainage system is exceeded.
Subcatchment	A smaller sub-area of the catchment draining to a watercourse.
Time of Concentration	Time for a water particle to travel from the hydraulically most distant point of a catchment to the outlet.
Topography	MIKE URBAN modelling feature to define natural channels which allows for variable cross-sections, Manning's roughness and slope for a channel between nodes.
TP108	Auckland Regional Council Technical Publication 108.
Unit Hydrograph	Hydrograph produced by a unit depth of rainfall excess falling uniformly in time and space over a unit area catchment.

## EXECUTIVE SUMMARY

### Objectives

The principal objectives of the Oteha Valley Catchment stormwater modelling study are to:

- Determine the design flows and flood levels for nominated design storm events and land use / future rainfall scenarios.
- Determine where the existing infrastructure does not meet Auckland Council level of service requirements.
- Determine properties with habitable floors at risk of flooding.
- Determine 10-year and 100-year ARI floodplain maps for the Oteha Valley Catchment based on the most recent topological, network and rainfall data.

### Activities and Scope

The activities and scope of the present modelling study are:

- Development of a hydrological and hydraulic model in accordance with the Auckland Council stormwater flood modelling specifications.
- Validation of the developed model with the measured gauge data and historical flood event information.
- System performance assessment of the stormwater drainage network.
- Floodplain mapping for the 10-year and 100-year ARI storm events under the maximum probable development (MPD) land use with future rainfall scenario.

### Model Development

A hydrological and hydraulic model of the stormwater drainage network system in Oteha Valley Catchment has been developed using MIKE FLOOD modelling software based on ARC TP108 rainfall-runoff modelling methods and 1-D and 2-D free surface gradually varied unsteady flow equations.

The model gauge validation results indicate that in general the model has been validated within reasonable agreement with the measured gauge data considering this large catchment with complex stormwater drainage system.

### System Performance Assessment

Model simulations were carried out for the 6 nominated design storm events under ED and MPD landuse with future rainfall scenarios. It has been found that under free full flow conditions 32% of the modelled stormwater drainage pipes within the Oteha Valley Catchment do not have the capacity to contain the 10-year ARI MPD storm event without

exceeding the soffit of the pipe. However, due to backwater or downstream effects 79% of the modelled pipes would be under surcharge conditions during the 10-year ARI MPD storm event. Considering both free full flow and backwater constraints 85% of the modelled pipes would be under surcharge conditions during the 10-year ARI MPD storm event.

It has been found majority of culverts within the Oteha Valley Catchment do not meet Auckland Council level of service, too small to pass the 10-year ARI design flow without surcharging at the inlet.

This modelling investigation has identified a total of 597 flood affected properties within the Oteha Valley Catchment which are subject to either nuisance flooding or floor flooding during the 100-year ARI MPD storm event. A total of 39 buildings within the Catchment are predicted to be within the extents of the 100-year ARI MPD floodplain. Of these buildings a total of 6 residential and 4 business floors are predicted to be inundated, 17 residential and 2 business floors are within 500mm of the predicted peak 100-year ARI flood level.

The main areas affected by 100-year ARI storm flooding are located along the Oteha Stream due to overtopping of the stream bank. A total of 17 buildings were identified within the main Oteha Stream floodplain. No buildings were identified within the Alexandra Stream 100-year ARI floodplain.

# 1. INTRODUCTION

## 1.1 Background

The recent Catchment Management Plan (CMP) for the Oteha Valley Stormwater Catchment was developed by Beca Carter Hollings and Ferner Limited (BCHF) in 2000. This formed the basis of an application to the Auckland Regional Council (ARC) under the Resource Management Act for a Comprehensive Catchment Consent.

The 2000 CMP used a model called PRISM to determine peak flows and flood levels at various locations along the stormwater drainage network.

The Stormwater Hydraulic Modelling Team of Auckland Council (AC) has been engaged to develop a hydrological and hydraulic model for Oteha Valley Catchment which can be used confidently to identify habitable floors at risk of flooding and assess the performance of existing stormwater drainage network system.

## 1.2 Study Objectives

The principal objectives of the Oteha Valley Catchment stormwater modelling study are to:

- Determine the design flows and flood levels for nominated design storm events and land use / future rainfall scenarios.
- Determine where the existing infrastructure does not meet Auckland Council level of service requirements.
- Determine properties with habitable floors at risk of flooding.
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## 1.3 Activities and Scope

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## 2. CATCHMENT DESCRIPTION

### 2.1 Location

The Oteha Valley Catchment is located in the centre of North Shore City (Figure 2.1). The Oteha Valley Stream is 11.5 km long with a contributing catchment area of approximately 1311 ha. The Alexandra Stream joins the Oteha Valley Stream approximately halfway up the main channel. The Alexandra Stream is 5 km long with a contributing catchment area of approximately 270 ha.

The Catchment is bounded by Albany highway on the West, Sunset Road on the South, East Coast Road on the East, Spencer Road and a part of SH1 from Spencer Road to Albany Highway on the North. The Oteha Valley Catchment contains the Rosedale Landfill and the Rosedale Waste Water Treatment Plant (WWTP) to the south, the Massey University Albany Campus, and the Northern Motorway (State Highway 1) transects the Catchment from north to south.

The Oteha Valley Catchment is the second largest stormwater catchment in North Shore City, about 10% of city area (Figure 2.1).

### 2.2 Topography

The topography of the Oteha Valley Catchment is complex due to man-made influences. This includes the infilling of southern and eastern tributaries in order to create the Rosedale Treatment Pond and Rosedale Landfill. In general, the southern extremity of the catchment is gently sloping, climbing to approximately 80 metres. The northern extremity climbs more sharply with a steep ridgeline bisecting the lower catchment and contours at the head of the catchment climbing to a little over 90 metres in elevation. The eastern extremity of the catchment is highest in elevation and is bounded by a ridgeline, which delineates the eastern and western harbour drainages. This part of the catchment rises to approximately 100 metres in many places.

### 2.3 Geology and Soils

The surficial geology of the Oteha Valley Catchment comprises interbedded mudstones and sandstones of the Waitemata group (East Coast Bays Formation) and quaternary sediment deposits as shown in Figure 2.2. The quaternary deposits include variable soil types such as undifferentiated alluvium, sand and coastal sediments and have moderate permeability. The Waitemata group soils are characterised by low permeability with relatively high runoff potential. The mudstones and sandstones in this area are highly weathered, to depths of 10 metres to 12 metres, comprising soft to firm silts and sands with minor clay.

### 2.4 Existing and Future Land Use

The current land uses within the Oteha Valley Stream Catchment is shown in Figure 2.3 according to the 2002 North Shore City Proposed District Plan. The Catchment

encompasses a mixture of residential, commercial, and light industrial land uses, and a significant portion remains in rural use. The current land use in the Oteha Valley Stream Catchment comprises pasture 30%, low density residential 15%, commercial 14%, road 14%, (light) industrial 12%, open space 10%, bush 4%, and high density residential 0.26%.

The District Plan allows for future development of land currently in pastoral use, to provide for land use as follows: (light) industrial 27%, low density residential 27%, commercial 15%, road 14%, open space 10%, bush 4%, pasture 3%, high density residential 0.26%.

## 2.5 Stormwater Drainage System

Oteha Valley Catchment is drained via the reticulated pipe drainage system, or overland flow and several tributaries to the Oteha Stream (Figure 2.1). The main Oteha Stream now piped in the upper tributaries, has its headwaters in the Constellation Drive industrial development. The Alexandra Stream drains the new subdivision of Unsworth Heights, and flows through open stream from Upper Harbour Highway to its confluence with the Oteha Stream at Bush Road Bridge.

To the west of the Alexandra Stream Catchment stormwater is collected through a piped drainage system and discharges into two detention ponds upstream of Rosedale Road. These tributaries join the Oteha Stream upstream of the Massey University Campus. A conglomeration of small streams and piped drainage systems drains the land on the eastern side of the Catchment, from Rosedale Landfill through to Spencer Road.

The Oteha Stream and its tributaries have been highly modified in recent times with the rapid urbanisation of most of the previously rural catchment. Much of the upper stream channel has been modified, however the lower stream course retains more of its natural character. The Oteha Stream and its tributaries now vary in quality, from a concrete lined channel immediately upstream of Rosedale Road to an intertidal, tree lined stream at its discharge into Lucas Creek.

Several detention ponds were constructed in the upper reaches of the catchment, many of which have been constructed as development has occurred. These ponds were designed to retain flows greater than the 2-year storm event, limiting outflows downstream. There are numerous culverts along the stream which restrict channel flow resulting in water backing up and increased flood levels upstream. In the lower reaches of Oteha Stream north of Albany Highway, flood levels are affected by tidal influences.

At the centre of the Oteha Valley Catchment are two NSCC Rosedale Lakes, one on the west side of the motorway and the other on the east side. They receive effluent from the secondary stage of the wastewater treatment plant. These ponds currently discharge via a 3 km tunnel under East Coast Road to an outfall several hundred metres off the coastline between Campbells and Castor Bays. Flow first enters the western pond, then flows over a weir into an outlet structure and passes through a pipe under the motorway into the eastern pond. The rain which falls over their surface area is discharged along with the treated effluent, and does not contribute to the flows in the Oteha Stream.

Stormwater runoff from the upstream catchment is collected into a concrete lined cut-off drain which runs along the southern boundary of each Rosedale Lake. The pipe network discharges stormwater into five detention ponds a short distance upstream (south and east) of the Rosedale Lakes. These ponds in turn discharge into the cut-off drains which flow to a central point to the east of the motorway where stormwater enters a pipe flowing north and eventually discharges into Oteha Stream.

## 2.6 Reported Flooding Issues

A number of flooding issues were identified within the Oteha Valley Catchment as reported in the Property Information Register (PIR) database. A summary of the reported flooding issues is presented below in Table 2.1. The major source of flooding was reported to be overland flow from upstream catchment. A total of 4 habitable floors, 1 non-habitable floor, and 30 properties were reported as flooding within the catchment.

Table 2.1 Summary of reported flooding issues

Property Address	Flooding Issues
2/ 39 Rifleman Rise, Albany	Flooding of habitable floors
2/ 37 Rifleman Rise, Albany	Flooding of habitable floors
1/ 35 Rosedale Road, Albany	Flooding of habitable floors
16 Malin Place, Albany	Flooding of habitable floors
8 Owl Court, Albany	Flooding of non-habitable floors
46 Ballymore Drive, Albany	Flooding from overland flow
48 Ballymore Drive, Albany	Flooding from overland flow
51 Ballymore Drive, Albany	Flooding from overland flow
53 Ballymore Drive, Albany	Flooding from overland flow
25 Kristin Lane, Albany	Flooding from overland flow
R 26 Kristin Lane, Albany	Flooding from overland flow
221 Albany Highway, Albany	Flooding from overland flow
30 Apollo Drive, Albany	Flooding from overland flow
26 Tyburn Lane, Albany	Flooding from overland flow
28 Tyburn Lane, Albany	Flooding from overland flow
36 Goldfinch Rise, Albany	Flooding from overland flow
27 Kristin Lane, Albany	Flooding from overland flow
41 Kristin Lane, Albany	Flooding from overland flow
225 Rosedale Road, Albany	Flooding from overland flow
229 Rosedale Road, Albany	Flooding from overland flow
235 Rosedale Road, Albany	Flooding from overland flow
2 Kanuka Way, Albany	Flooding from overland flow

<b>Property Address</b>	<b>Flooding Issues</b>
3 Kanuka Way, Albany	Flooding from overland flow
14 Sunset Road, Glenfield	Flooding from overland flow
303 Albany Highway, Albany	Flooding from overland flow
307 Albany Highway, Albany	Flooding from overland flow
309A Albany Highway, Albany	Flooding from overland flow
311 Albany Highway, Albany	Flooding from overland flow
309 Albany Highway, Albany	Flooding from overland flow
135 Albany Highway, Albany	Flooding from overland flow
6 Landsberg Way, Albany	Flooding from overland flow
11 Hauraki Crescent, Albany	Flooding from overland flow
700 East Coast Road, Albany	Flooding from overland flow
4 Kanuka Way, Albany	Flooding from property landscaping
88 Caribbean Drive, Albany	Flooding from property landscaping

Where possible the model will be used to mitigate the reported flooding issues during the options assessment phase. For the system performance phase the data above will be used to validate the model.

### 3. MODEL BUILD

The hydrological and hydraulic model of the Oteha Valley Catchment was developed in accordance with the AC standard stormwater hydraulic modelling specifications. Details of this modelling process are outlined below.

#### 3.1 Modelling Software

MIKE FLOOD (Version 2011) modelling software package developed by Danish Hydraulic Institute (DHI) has been used to develop the hydrological and hydraulic model of the Oteha Valley Catchment. MIKE FLOOD modelling software integrates the one-dimensional model (MIKE11) and the two-dimensional model MIKE 21 into a single, dynamically coupled modelling system. The coupled approach enables the best features of both the one-dimensional and the two-dimensional models to be utilised.

MIKE URBAN is an advanced, powerful, and comprehensive surface runoff, open channel flow, pipe flow, water quality and sediment transport modelling package utilised for urban drainage systems, stormwater sewers and sanitary sewers. MIKE11 is a 1D hydrodynamic modelling system for river and channel networks. MIKE 21 is a modelling system for 2D free-surface flows and simulates water level variations and flows in response to a variety of forcing functions in rivers, lakes, estuaries and coastal regions.

Lower part of the Oteha Stream is modelled using a 1D-2D dynamically coupled modelling approach i.e. Oteha Stream main channel is modelled in 1D MIKE11 model and overbank/floodplain areas are modelled in MIKE 21 2D model. The stormwater drainage pipe networks, culverts, ponds, other tributaries and streams are modelled in MIKE URBAN one-dimensional model.

#### 3.2 Review of Existing Data

##### 3.2.1 Asset Data

As part of the current modelling study, asset data surveys were carried out to collect the missing asset data for the existing stormwater drainage network system within the Oteha Valley Catchment. These included surveying of 98 manhole depths and 23 invert levels for culvert inlets and outlets scattered throughout the catchment. Manhole invert levels were computed from manhole lid levels generated from LiDAR survey data and manhole depths. In addition manhole data were extracted from as-built plans from archives and the digital plan library for 195 manholes.

TCB Surveyors surveyed a total of 104 invert levels for pipe outlets and culvert inlets and outlets during the stream cross-section survey. A total of 30 dry and wet ponds were identified within the Oteha Valley Catchment. The detailed information of these ponds was collected from pond design reports, project files, as-built plans, LiDAR data, and site visits. The information regarding the Rosedale Lakes including the linking pipes and weirs, cut-off channels, and outfall pipes were sourced from the existing Rosedale Lakes Outfall Model.

A total of 136 missing manhole invert levels were estimated in the model on the basis of upstream and downstream pipe slopes. A number of missing outlet invert levels were estimated on the basis of upstream pipe slope. Missing manhole lid levels were estimated in the model based on ground level data from LiDAR survey. Missing manhole diameters were estimated on the basis of NSCC Infrastructure Design Manual.

Missing pipe diameters were estimated on the basis of upstream and downstream pipe diameters. A number of pipe diameters in NSCC corporate GIS database appeared to decrease in downstream direction, these pipe diameters were updated in the model on the basis of upstream and downstream pipe diameters.

A summary of various types of asset data used in the model, including their source, is given below in Table 3.1 and presented in Figure 3.1.

As part of the present study, several field visits were carried out to collect information regarding pipe connectivity, depth of manholes, culverts and pipes diameter, and pond outlets configuration. A series of photographs taken during field visits are presented in Appendix A.

Table 3.1 Summary of asset data sources

Asset Data Type	No.	Data Sources
Manholes	98	Survey
	195	As-built plans from archives and digital library
	136	Estimated based on upstream & downstream pipe slopes
	914	NSCC GIS database
	7	Existing Rosedale Lakes Outfall Model
Dry and Wet Ponds	30	Survey, as-built plans, design reports, project files, LiDAR data, and site visits
Culvert / pipe inlets and outlets	127	Survey
Pipes	32	Estimated based on upstream & downstream pipe diameters
	1193	NSCC GIS database
	7	Existing Rosedale Lakes Outfall Model

### 3.2.2 Hydrometric Data

In September 2005 AWT New Zealand installed a total of thirteen flow gauges and five rain gauges within the Oteha Valley Catchment to monitor rainfall, flow depth, velocity and discharge data for the purposes of providing baseline data for model calibration of

stormwater system. The data capture period started at 17th October 2005, and ended at 9th January 2006, which comprises a total of 12 weeks of data capture. Detailed assessment of monitoring data is presented in Appendix B.

In general the data received from the rain gauges is good quality data. During the 12-week monitoring period, two storm events greater than 20mm total rainfall were captured. However, these events did not meet the minimum required intensity criteria of 10mm/hr over 10 minutes duration. These storm events are small which are not useful for model calibration of stormwater drainage system. Moreover monitoring data from five flow gauges was found unreliable – flow gauge 8OV008 (weir type) has an unusual wide scatter of depth-flow data, gauge 8OV015 (HVQ type) provides more runoff than rainfall and indicates 'negative' mass balance with the downstream gauge 8OV008, gauges 8OV004 (weir type), 8OV002 (HVQ type) and 8OV006 (weir type) provide more runoff than rainfall.

Two long-term permanent gauges are available within the Oteha Valley Catchment – ARC gauge on Oteha Stream at Days Bridge on Albany Highway (7811) and NIWA gauge on Alexandra Stream at Rosedale Road (7834). These flow gauges are used for model validation of Oteha Valley Catchment stormwater drainage system.

### **3.2.3 Topographical Data**

The model was developed from two separate topographical data sources – topographical survey and 0.25m interval contours/grid data based on LiDAR survey. As part of the current modelling study, TCB surveyors surveyed a total of 90 stream cross-sections within the Oteha Valley Catchment. In addition a number of spot levels were surveyed around culvert inlets/outlets, bridges, and pond outlets.

A Light Detection and Ranging (LiDAR) survey of the entire North Shore City was commissioned in 2004. LiDAR is an efficient survey technique, which produces a dense scattering of points with ground elevation (1m grid). LiDAR data were used to generate stream cross-sections, missing manhole lid levels and pond storage volume data. NSCC corporate GIS depression area shape file based on LiDAR data was used to identify the local depression areas within the Catchment.

### **3.2.4 Operational Data**

Auckland Council issues register database and Property Information Register (PIR) database were used to identify the reported flooding issues within the Oteha valley Catchment. Historical flood incident data was collected through document search from Dataworks database and meeting with AC stormwater operations staff.

## 3.3 Hydrological Model

### 3.3.1 Method Used

Stormwater catchment runoff was modelled following the guidelines outlined in the ARC Technical Publication No. 108 document (ARC, 1999). The key features of the ARC TP108 rainfall-runoff model are:

- A standard 24 hour temporal rainfall pattern, having peak rainfall intensity at mid-duration. Shorter duration rainfall bursts with a range of durations from 10 minutes to 24 hours are nested within the 24 hour temporal pattern,
- Runoff depth calculated using SCS (Soil Conservation Services) rainfall-runoff curves, with curve numbers determined from the SCS guidelines according to classifications assigned to Auckland soil types,
- Runoff hydrograph calculated using the standard SCS synthetic unit hydrograph,
- Time of concentration estimated using an empirical lag equation derived from a regression analysis of data from the Auckland Region.

### 3.3.2 Sub-Catchments

The catchment and sub-catchment boundaries were delineated in ArcGIS software based on the 0.25m interval LiDAR contours, aerial photographs, overland flow paths (generated from the DEM based on LiDAR data), cadastral property boundaries, and the location of stormwater collection pipes. Sub-catchment length and slopes were computed using an in-house developed tool in ArcGIS from manually digitised flow lines based on the equal area method as specified in ARC TP108 document (ARC, 1999).

A total of 685 sub-catchments were delineated for Oteha Valley Catchment (refer to Figure 3.2). Sub-catchment area ranges from 0.42 ha to 39.8 ha with an average value of 1.92 ha. The large sub-catchments are related to the green field areas and large Rosedale Lakes.

Sub-catchment pervious and impervious areas were modelled separately resulting in a total of 1370 pervious and impervious sub-catchments used in the model representing the Oteha Valley Catchment. These sub-catchments are connected to 685 inflow nodes within the hydraulic model network linking the hydrological model to hydraulic model.

### 3.3.3 Hydrological Parameters

The TP108 methodology uses two rainfall loss parameters i.e. SCS Curve Number (*CN*) and Initial Abstraction (*I<sub>a</sub>*) to describe rainfall losses and runoff timing parameter i.e. lag time or time of concentration to describe runoff routing process which provides attenuation and lag within the catchment. TP108 has derived *CN* and *I<sub>a</sub>* values for use in the Auckland Region. These are tabulated as a function of soil type and land use.

Oteha Valley Catchment comprises Group C hydrological soil type that relates to sandstone and siltstones of the Waitemata Formation having low infiltration losses and Group B hydrological soil type that relate to quaternary sediment deposits having moderate infiltration losses. For Group B soil, a SCS *CN* of 61 was used for undeveloped pervious areas and 74 for developed pervious areas in the model. For Group C soil, a SCS *CN* of 74 was used for developed and undeveloped pervious areas. A SCS *CN* of 98 was used for impervious areas in the model. An initial abstraction loss of 5mm for pervious area and 0mm for impervious area was used in the model.

The time of concentration for each sub-catchment was estimated using the empirical lag equation given in ARC TP108, derived from a regression analysis of data from the Auckland Region.

The existing development impervious coverage areas for the sub-catchments were determined using the 2007 impervious surface GIS shape files based the 2006 aerial photographs. The imperviousness of the Oteha Valley Catchment is currently estimated at 44.8%.

It has been assumed under the MPD scenario (the ultimate future development) development will proceed up to the maximum permitted under the current District Plan. The current District Plan allows for a maximum permitted impervious area of 70% for residential-zoned land and 80-100% for business-zoned land. Under the MPD scenario the imperviousness of the Catchment is estimated at 74.3%.

A summary of various hydrological model components used in the model are given in Table 3.2 below.

A detail list of all sub-catchments information including node attachments and characteristics required in the model are presented in Appendix C.

Table 3.2 Summary of hydrological model components

Hydrological Model Components	Values
Number of sub-catchments	685
Range of sub-catchment size (in Hectares)	0.42 to 39.8
SCS curve number for pervious area	61 and 74
SCS curve number for impervious area	98
Initial loss for pervious area (mm)	5
Initial loss for impervious area (mm)	0
Existing development imperviousness %	44.8
Maximum probable development imperviousness %	74.3

## 3.4 Hydraulic Model

### 3.4.1 Method Used

The hydraulic model of the study area was developed incorporating the existing stormwater pipe network, open channels, culverts, bridges, overland flow paths and storage basins. The hydraulic computation in 1D model (MIKE URBAN, MIKE11) is based on an implicit, finite difference numerical solution of basic 1-D free surface gradually varied unsteady flow equations (Saint Venant).

The hydrodynamic computation in MIKE 21 model is based on an Alternating Direction Implicit (ADI) finite difference numerical solution of the two-dimensional shallow water equations - the depth-integrated incompressible Reynolds averaged Navier-Stokes equations. The model consists of continuity and momentum equations. It simulates unsteady two-dimensional flows in one layer (vertically homogeneous) fluids.

### 3.4.2 Hydraulic Model Network

The hydraulic model network is made up of two main hydraulic components; the primary drainage system, comprising the formal stormwater system made up of the pipe and open channel network, and the secondary drainage system, comprising the overland flow paths. These two systems are linked by a number of weirs/orifices, as described below.

The primary and secondary stormwater drainage network systems were represented in the model by nodes (i.e. manholes, inlets, outlets, junctions between open channels or between pipe and open channel, ponds) and links as connecting pipes or stream channel reaches or overland flow paths.

A summary of various hydraulic model components are given in Table 3.3, presented in Figure 3.4 and briefly described below.

Table 3.3 Summary of hydraulic model components

Hydraulic Model Components	Values
MIKE URBAN 1D Model	
Total number of stormwater network system nodes	2433
Total number of basin nodes	40
Total number of links	2268
Total number of stormwater network system pipes	1225
Total number of weirs	1340
Total number of orifices	87
MIKE11 1D Model	
Total number of braches	10
Total number of computational points	689

Total number of bridge structures	3
Total number of weirs	1
MIKE21 2D Model	
Total number of 2D model grid	1201 x 1050 = 1261050

### **1D Model Nodes**

MIKE URBAN model nodes were utilised to represent the stormwater drainage network system attributes such as manholes, inlets, outlets, junction between open channels, ponds. The nodes were defined by X-Y co-ordinates, ground levels, invert levels and manhole diameters. Node inverts and ground levels were obtained from the AC GIS asset database, survey data, and LiDAR contour information.

The secondary overland flow node invert levels were set using the ground level of the associated manhole and an arbitrary ground level of 5 meters above the invert level of the overland flow node. Overland flow node diameters were normally set to 1m. Basin nodes were added to the model to represent dry and wet ponds and local depression storage areas.

### **1D Model Links**

MIKE URBAN model links were utilised to represent stormwater drainage pipes, culverts, bridges, stream channel reaches or overland flow paths. The pipe data input to the model comprised of diameter, upstream and downstream inverts and connecting nodes based on the AC GIS asset database. The stream channel cross-section configuration was developed from survey data and LiDAR data to replicate channel conveyance and storage.

AC corporate GIS overland flow path shape files were used to schematise the overland flow path in the model. In general standard default cross-sections were used for overland flow paths through reserves or properties, however, in cases where overland flow paths have contributing area of 10ha or more, cross-sections derived from LiDAR data were used in the model.

Two road bridges (Albany Highway and Bush Road), and one foot bridge (upstream of Albany Highway) were modelled in MIKE11 as bridge structure.

### **1D Ponding and Storage**

There are significant ponding areas within the Oteha Valley Catchment stormwater drainage network. A total of 30 dry/wet ponds and 9 local depression storage basins were modelled. The LiDAR contour information was used to generate elevation-storage volume tabular data for ponds and local depression areas. Pond outlet configuration, permanent water level and spillway/overflow level data were collected from pond design reports, project files, as-built plans and site visits. Ponding areas and volumes were represented in the model using representative nodal cross-section and basin node configurations. A list of all ponds and local depression basins included in the MIKE URBAN model are presented in Appendix D.

### **1D Model Weirs/Orifices**

Artificial weirs were modelled at manholes that spill during large storm events. The weir crest height was set at the manhole lid level and connected to the stormwater overland flow paths. In general a 2m weir width was used in the model for the weirs connecting spilling manholes to the overland flow paths. However, for manholes that do not spill during large storm events but stormwater returns back into the manholes from overland flow path, artificial orifices were modelled. A 2m wide by 0.1m high rectangular orifice was used in the model connecting manholes to the overland flow paths.

Weirs were also modelled for channel overflow, culvert spilling over the road, storage basin overflow and linking stormwater pipe outlets to open channels or basins. Wider overflow weirs were applied for linking to basin nodes or spilling over the road or channel overflow. The weir overflow levels in the model were updated with the LiDAR data.

### **2D Model Bathymetry**

The bathymetry consists of the 2D ground surface (Digital Elevation Model) created from the LiDAR data (refer to Figure 3.2). The dimension of the 2D ground surface (the grid size) is an important parameter as it sets the spatial resolution of the resultant floodplain. The 2D model grid used for lower part of Oteha Stream is 1201 by 1050 in size with a grid cell size of 2m.

### **2D Model Flooding & Drying Depth**

The flooding and drying depths used in the MIKE21 model were 0.03m and 0.02m respectively.

### **1D-2D Models Coupling**

Lower part of the Oteha Stream is modelled using a 1D-2D dynamically coupled modelling approach i.e. Oteha Stream main channel is modelled in 1D MIKE11 model and overbank/floodplain areas are modelled in MIKE 21 2D model.

Links between MIKE 11 and MIKE 21 are modelled as lateral and standard links which allows the dynamic exchange of flow between Oteha Stream (1D) and overbank/floodplain areas (2D). The lateral link define the movement of water along the stream banks and standard link define the movement of water at the connection point between end of MIKE 11 stream branch and 2D grid cells on the surface. In addition river/urban links are used to model exchange of flow between MIKE URBAN drainage networks and Oteha Stream branches.

## **3.4.3 Energy Losses**

### **Energy Losses due to Surface Friction**

Friction factors were assigned to the links as a Manning's "n" value of 0.014 for the stormwater pipes and culverts. Friction factors for Oteha Stream and tributaries were assigned as a Manning's "n" value of 0.08 for a composite section including floodplain with dense vegetation. Friction factors for overland flow paths through property were assigned as a Manning's "n" value of 0.10 and overland flow paths along roads were assigned as a Manning's "n" value of 0.02. Table 3.4 below shows the summary of friction factors for various types of links used in the model.

Manning's n values ranging from 0.05 to 0.078 were assigned for main channels in MIKE11 model for various Oteha Stream reaches considering considering surface material, vegetation, irregularities and stream channel sinuosity. The hydraulic roughness over the 2D model domain is expected to be variable based on various land uses as given in Table 3.4.

Table 3.4 Summary of friction factors used in the model

Link Type	Manning's "n" Value Used	MIKE URBAN Link Material Type
Pipe and Culvert	0.014	"Normal Concrete"
Natural Open Channel (MIKE URBAN)	0.08	"Other"
Natural Open Channel (MIKE11)	0.05 to 0.078	-
Unlined Open Channel	0.05	"Iron"
Concrete Lined Open Channel	0.02	"Ceramics"
Overland Flow Path Through Property	0.10	"Stone"
Overland Flow Path Along Road	0.02	"Ceramics"
2D Model Domains for various Land uses: Buildings, Grass, Roads, Bush	0.33, 0.05, 0.02, 0.125	-

### ***Energy Losses due to Turbulence***

Head losses through manholes, basins, inlets and outlets are calculated in MIKE URBAN by assigning an 'outlet shape' to each node. The "Mean Energy Approach" was used as manhole outlet shape to model all manhole head losses with a  $K_m$  factor of 0.30. Basin node head losses were modelled with a "Sharp Edged Outlet" with a  $K_m$  factor of 0.50. Overland flow path and stream channel nodes were assigned as the default "No Cross-Sectional Changes" i.e. no head loss through these nodes.

Pipe, bridge and culvert inlet head losses were modelled with a "Mean Energy Approach" with a  $K_m$  factor of 0.30 and pipe, bridge and culvert outlet head losses were modelled with a "Sharp Edged Outlet" with a  $K_m$  factor of 0.50. Table 3.5 below shows the summary of node head loss parameters used in the model.

A flow regulation function based on inlet controlled stage-discharge (H-Q) rating table was used at the culvert and pipe inlets. The inlet controlled stage-discharge rating table was generated using the free culvert analysis software HY-8 (version 7.0) by US Federal Highways Administration.

Due to the assigned flow regulation function, MIKE URBAN generates upstream water levels at the inlet equal to that in the stage-discharge rating table. However, the assigned flow regulation function is drowned out when the culvert becomes backwater controlled i.e. when the culvert is backwater controlled MIKE URBAN head loss calculations through the conduit take over and produce the correct outlet controlled upstream water level.

MIKE URBAN modelling software has inherent difficulties in accurately modelling energy losses at manholes when the outlet pipe from the manhole is inlet control, which results

in high velocity in the pipe. In case of steep inlet controlled pipes with high velocity (>6m/s), a Q-H relationship for inlet controlled pipe was derived from weir/orifice formulation as described in HEC22 manual which was input into the model to simulate inlet control energy losses.

Table 3.5 Summary of node head loss parameters used in the model

Link Type	Node Head Loss Parameter
Manhole Nodes	"Mean Energy Approach" With $K_m = 0.30$
Pipe & Culvert Inlet Nodes	"Mean Energy Approach" With $K_m = 0.30$
Pipe & Culvert Outlet Nodes	"Sharp Edged Outlet" With $K_m = 0.50$
Basin Nodes	"Sharp Edged Outlet" With $K_m = 0.50$
Open Channel & Overland Flow Path Nodes	"No Cross-Sectional Changes"

## 3.5 Boundary Conditions

### 3.5.1 Rainfall Data

Four permanent rainfall gauges (NSSC02, NSCC04, NSCC06, and NSCC08) were used for model validation against measured data at two long-term permanent gauges. Three storm events were used for model validation of permanent flow gauges. The existing and future 24-hour temporal design rainfall profiles were used for performance assessment of the Oteha Valley Catchment stormwater drainage system.

The standard TP108 nested 24-hour duration temporal rainfall profile was adopted and utilised following the rainfall derivation procedure outlined within the TP108 guidelines (ARC, 1999). The 24-hour design rainfall depth for Oteha Valley Catchment was estimated from the design rainfall contour maps for a range of average recurrence intervals (ARIs) given in the TP108 document (ARC, 1999). The temporal rainfall profiles were developed for the nominated 6 design storm events (2, 5, 10, 20, 50, and 100 year ARI) based on the 24-hour design rainfall depth and the standard TP108 normalised 24-hour temporal design storm profile.

The future 24-hour design rainfall depth due to future climate change has been estimated in accordance to the guideline provided by the Ministry for the Environment (2008). The guideline provides a table of percentage increase in rainfall per degree Celsius of warming for a range of ARIs and durations. The projected average increase in annual mean temperature for Auckland Region is 2.1°C for the period from 1990 to 2090. This is considered for estimating the future 24-hour design rainfall depth and developing the future normalised 24-hour temporal design storm profile due to future climate change.

### 3.5.2 Tidal Data

The model has three outlets, two outlets discharging into the Lucas Creek and one outlet from the eastern Rosedale Lake discharging into the Hauraki Gulf. The present modelling study has used a constant downstream tidal boundary conditions of 0.5m RL.

## 3.6 Model Limitations and Assumptions

### 3.6.1 Limitations

- Explicit calibration of the Oteha Valley Catchment model was not undertaken. The present modelling is limited by the ARC TP108 rainfall-runoff model which is expected to be within  $\pm 25\%$  at a confidence level of 90 percent for 2 to 100-year ARI storm events (ARC, 1999).
- The model accuracy for historical flood events will be dependent on the antecedent ground conditions and spatial rainfall variation. Antecedent ground conditions are variable, depending on the season and the timing of the storm within the sequence of storms. The TP108 runoff model is limited to the average antecedent moisture condition.
- The modelled overland flow paths were based on the LiDAR data, no onsite overland flow path inspections were carried out as part of the modelling exercise. Overland flow paths that pass through properties can have fences, vegetation and walls that alter flow path routes and dimensions.
- No specific survey was conducted for flood extent mapping. Therefore the accuracy of the flood extent maps depends on the compound effects of uncertainties in the TP108 rainfall-runoff model, uncertainties in the hydraulic model parameters, and the accuracy of the LiDAR data.

### 3.6.2 Hydrological Model Assumptions

- The sub-catchment length was measured from the most distant point of the catchment to the inflow node.
- The sub-catchment slopes were calculated from the 1m grid raster dataset based on LiDAR data according to the Equal Area Method as outlined in ARC TP108 document.
- The 2, 5, 10, 20, 50 and 100-year 24-hour rainfall profiles used in the model are based on ARC TP108 runoff modelling guidelines.

### 3.6.3 Hydraulic Model Assumptions

- All sub-catchment runoff was assumed to enter freely into the reticulation system i.e. catch pit inlet control was not modelled for stormwater reticulation. The model effectively assumes catch pit inlet capacity is equal to or greater than the modelled pipe capacity.

- No blockage has been assumed in catch pits, manholes, pipes, culverts and entry points into the stormwater network system.
- No sedimentation has been allowed for in the pipes, i.e. all pipes are capable of performing at full capacity.
- Private rainwater tanks were not considered in this modelling study.

### 3.7 Initial Model Testing

As part of the initial model testing, sensibility analysis was carried out prior to carrying out any model validation to ensure that the model results are 'sensible' i.e. within reasonably expected values. As part of the sensibility test of the hydraulic model, a comparison of peak flows and runoff volumes at critical locations with those estimated by the TP108 graphical method was undertaken. The results of the sensibility analysis are given in Table 3.6 below. It shows that the hydraulic model predicted significantly (49%) lower peak flow at the Catchment outlet compared to that estimated by the TP108 graphical method. However, for reticulated catchment the difference in peak flow is small (5%). The runoff volume difference is negligible. The large differences in peak flows are due to the large storage volume within the Catchment through a large number of ponds/ basins (39) and channels.

The mass continuity balance is used as a measure of system water volume balance error due to water generated within the model for various situations e.g. sharp changes in surface width or cross-section shape or surface area of basins with water depth. The continuity balance is calculated as percentage of total inflow volume. Values below 5% are considered acceptable. The continuity balance of the Oteha Valley Catchment model ranged from 0.5% to 2.0% of the total inflow volume. No major instabilities were noticed within the model results.

Table 3.6 Comparison of peak flows and runoff volumes

Location	Catch. Area (ha)	TP108 Graphical Method (100-yr ARI MPD)		Hydraulic Model (100-yr ARI MPD)		Differences	
		Peak Flow (m <sup>3</sup> /s)	Runoff Volume (m <sup>3</sup> )	Peak Flow (m <sup>3</sup> /s)	Runoff Volume (m <sup>3</sup> )	Peak Flow	Runoff Volume
Pipe Outlet at 45 Hugh Green Drive	47	19.0	112,000	18	113,000	5%	0.9%
Oteha Stream at Apollo Drive	46	14.8	108,000	12	107,000	19%	0.9%
Alexandra Stream at Rosedale Rd	260	62.9	619,000	38.9	608,000	36%	1.8%

Catchment Outlet	1311	321	3,157,000	163.2	3,123,000	49%	1.1%
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### 3.8 Quality Assurance and Quality Checks

The Oteha Valley Catchment stormwater drainage network model has been internally quality assured and checked.

It is considered that the Oteha Valley Catchment model is 'fit for purpose' given acceptable modelling uncertainty and reliability.

## 4. MODEL VALIDATION

The hydrological and hydraulic model of the Oteha Valley Catchment was validated against measured data at two long-term permanent gauges. The model was also validated using the historical flood incident information. Moreover sensitivity analysis was carried out to identify the effect of model parameters on predicted flows and flood levels. The present model results were compared with the previous model results and MIKE 21 2D model results. Details of this model validation process are outlined below.

### 4.1 Model Gauge Validation

A total of thirteen flow gauges and five rain gauges were monitored to provide baseline data for model calibration of stormwater system. During the 12-week monitoring period, two storm events greater than 20mm total rainfall were captured. However, these events did not meet the minimum required intensity criteria of 10mm/hr over 10 minutes duration. These storm events are small which are not useful for model calibration of stormwater drainage system because stormwater events of interest are the 10 to 100 year events. Models calibrated for small storm events cannot be used for extreme storm events due to variable antecedent conditions and stochastic nature of the rainfall-runoff process.

Moreover as discussed in Section 3.2.2 and Appendix B, monitoring data from five flow gauges was found to be unreliable. In addition two long-term permanent gauges are available within the Catchment – ARC gauge on Oteha Stream at Albany Highway (7811) and NIWA gauge on Alexandra Stream at Rosedale Road (7834). However, these gauges are located at the lower part of the Catchment and are not suitable for hydrological model calibration. As such a detailed calibration of the hydrological and hydraulic model was not carried out. However, a validation of model results was carried out by comparing the model results with the measured gauge data without changing the hydrological and hydraulic model parameter values.

The model results were compared against measured data at the two long-term permanent gauges for three storm events (2 to 10-year ARI). Flow monitoring data at the thirteen temporary gauges were not used for model validation as the storms during the monitoring period were less than 1-year ARI storm event.

Three permanent rainfall gauges (NSSC02, NSCC06, and NSCC08) were used for model validation of permanent flow gauges. The following three storm events were used for the two long-term permanent gauges:

- 28 – 29 June 2000, total rainfall of 96mm over 24 hours and maximum intensity of 64mm/hr, 5 to 10-year ARI storm event;
- 1 – 2 February 2004, total rainfall of 94mm over 24 hours and maximum intensity of 55mm/hr, 2 to 5-year ARI storm event;
- 1 – 2 October 2006, total rainfall of 124mm over 24 hours and maximum intensity of 61mm/hr, 2-year ARI storm event.

A summary of model gauge validation results are presented in Table 4.1 below. Model gauge validation plots are presented in Appendix E (Figures E1 to E12).

Table 4.1 Summary of model gauge validation results

Storm Events	Rainfall ARI	Monitoring Gauges	Volume Difference (%)	Peak Flow Difference (%)	Peak Depth Difference % (m)	Correlation Coefficient (r <sup>2</sup> )	Within Acceptable Criteria
28-29 June 2000	5 to 10-Year ARI	ARC 7811	-7.7%	+7.8%	+1.1% (+0.03m)	0.96	Yes
		NIWA 7834	-15.4%	-25.1%	-6.9% (-0.15m)	0.98	No
1-2 February 2004	2 to 5-Year ARI	ARC 7811	+13.5%	+26.9%	+10.5% (+0.28m)	0.95	No
		NIWA 7834	+13.8%	+2.7%	+13.4% (+0.26m)	0.97	Yes
1-2 October 2006	2-Year ARI	ARC 7811	+17.7%	+5.4%	+11.2% (+0.19m)	0.91	Yes
		NIWA 7834	+16.4%	+38.0%	+31.1% (+0.30m)	0.90	No

Notes: + indicates model over prediction and - model under prediction

According to NSCC standard stormwater hydraulic modelling specifications, the acceptable criteria for the model gauge validation are:

- Runoff volume error: +20% to -10%
- Peak flow error: +25% to -15%
- Peak depth error: +15% to -15%
- Correlation coefficient: 0.6 to 1.

The correlation coefficients of all the three events were between 0.9 and 0.98, representing a good match between the shape of the modelled and measured hydrographs. Model gauge validation results indicate the predicted peak flow, runoff volume and peak depth at the ARC gauge on Oteha Stream were validated within the acceptable criteria for the two storm events. However, at the NIWA gauge on Alexandra Stream the model was validated within the acceptable criteria for only one storm event (February 2004).

In general the model has been validated within reasonable agreement with the measured gauge data considering this large catchment with complex stormwater drainage system. As the permanent gauges are located at the lower part of the Catchment and there are several detention ponds above the gauges which make it difficult to identify the causes

for the discrepancies in model results. Moreover, as shown in the next section the rainfall variability over the Oteha Valley Catchment has significant effect on the model results, making it difficult to replicate the actual situation.

The differences between the model results and measured data could be due to several issues or missing features that are not accounted for within the model such as catch pit/inlet blockages, missing ponding/storage features or pond outlet characteristics during the historical storm events.

## 4.2 Sensitivity Analysis

Sensitivity analysis entails varying model parameters to identify what type of impact the changes have on the model outcomes. Sensitivity analysis identifies the model parameters that have significant effect on model output. These parameters require special attention to reduce uncertainty either through accurate characterisation or through calibration. The hydrological rainfall-runoff model is based on the prescriptive ARC TP108 method and the hydrological model parameters SCS *CN*, initial abstract  $I_a$ , and time of concentration are selected/computed in accordance to prescriptive tables or empirical equations. As such sensitivity analysis of the hydrological model parameters other than SCS *CN* was not undertaken.

A sensitivity analysis of SCS *CN* was carried out to assess the effect of *CN* on predicted flows and flood levels. Model simulations were carried out with SCS *CN* values of 61, 70 and 74 for pervious areas. The model simulation results indicate reduction of *CN* from 74 to 70 for pervious areas has a minor effect on peak flows and flood levels. The peak flows and flood depths were reduced less than 3%. The reduction of *CN* from 74 to 61 resulted in reduction of up to 13% in peak flows and up to 7% in flood depths.

An analysis of the sensitivity of the hydraulic model parameter Manning's roughness coefficient "n" for natural streams and overland flow paths through properties was undertaken. Manning's "n" values equal to 0.06, 0.08, and 0.10 for natural streams and 0.05, 0.10, and 0.20 for overland flow paths through properties were input in the model and the corresponding changes in peak flows and flood levels at various locations along the main stream were assessed.

The differences in flood levels and peak flows were computed relative to that used a Manning's "n" value of 0.08 for natural streams and 0.10 for overland flow paths. The results of the sensitivity analysis are given in Table 4.2 and Table 4.3.

Sensitivity analysis indicated the Manning's roughness "n" value for overland flow paths through properties has very minor effects on the predicted peak flows and flood levels. However, Manning's roughness "n" value for natural streams has significant effect on the predicted peak flows and flood levels. The maximum effects were at the lower reaches of the main stream. A 25% decrease in channel roughness provides a maximum of 9.6% increase in peak flows and 390mm decrease in flood levels. This increase in peak flows at the lower reaches was due to less flow attenuation upstream as a result of reduction in flood levels. Although flow is increased but the water level is decreased, indicating the water level is controlled by the local Manning's "n" value. Similarly a 25% increase in

channel roughness provides a maximum of 8.5% decrease in peak flows and 330mm increase in flood levels, which is within the recommended 500mm freeboard above the predicted 100-year ARI flood level.

Table 4.2 Impact of Manning's "n" value on water levels

Location	100-Year ARI Water Level			Differences Relative to n=0.08	100-Year ARI Water Level			Differences Relative to n=0.10
	Channel n = 0.06	Channel n = 0.08	Channel n = 0.10		OFF n = 0.05	OFF n = 0.10	OFF n = 0.20	
Upper Harbour Culvert	32.96	32.96	32.95	-0.01m to 0.0m	32.96	32.96	32.95	-0.01m to 0.0m
Tawa Drive Culvert	23.02	22.98	23.06	-0.04m to +0.08m	22.99	22.98	22.97	-0.01m to +0.01m
Bush Road Bridge	13.68	13.90	14.08	-0.22m to +0.18m	13.90	13.90	13.89	-0.01m to 0.0m
Albany Hwy Bridge	6.77	7.16	7.49	-0.39m to +0.33m	7.16	7.16	7.16	0.0m

Table 4.3 Impact of Manning's "n" value on peak flows

Location	100-Year ARI Peak Flow			Differences Relative to n=0.08	100-Year ARI Peak Flow			Differences Relative to n=0.10
	Channel n = 0.06	Channel n = 0.08	Channel n = 0.10		OFF n = 0.05	OFF n = 0.10	OFF n = 0.20	
Upper Harbour Culvert	25.5	25.3	25.4	+0.4% to +0.8%	25.3	25.3	25.0	-1.2% to 0.0%
Tawa Drive Culvert	64.4	63.9	63.2	-1.1% to +0.8%	64.2	63.9	63.7	-0.3% to +0.5%
Bush Road Bridge	109.1	102.4	96.8	-5.5% to +6.5%	102.5	102.4	102.1	-0.3% to +0.1%
Albany Hwy Bridge	133.2	121.5	111.1	-8.5% to +9.6%	121.5	121.5	121.5	0.0%

A sensitivity analysis of the rainfall variability over the Oteha Valley Catchment was undertaken. Model simulations were carried out for two storm events and for each storm event three model runs were undertaken using the high rainfall gauge, the low rainfall gauge and the available rainfall gauges (three rainfall gauges for 2000 storm event and four rainfall gauges for 2004 storm event). Model results are presented in Table 4.4. Comparisons of model results at the two long-term permanent flow gauges are presented in Appendix E (Figures E13 to E16). For 2000 storm event, the 24 hour rainfall varies over the Catchment from 102mm at high rainfall gauge to 94mm at low rainfall gauge.

For 2004 storm event, the 24 hour rainfall varies from 95mm at high rainfall gauge to 92mm at low rainfall gauge. However, the maximum variation in rainfall intensity over the Catchment in both storm events is about 10mm/hr.

Model results indicate rainfall variability over the Oteha Valley Catchment has significant effect on the predicted flows. For example, high rainfall gauge provides 26.8% higher peak flow and 14.8% higher peak depth compared to the measured peak flow and depth at ARC gauge on Oteha Stream, whereas low rainfall gauge provides only 5.4% higher peak flow and 3.1% higher peak depth compared to the measured peak flow and depth.

Table 4.4 Impact of catchment rainfall variability on peak flows and depths

Storm Events	Monitoring Gauges	Peak Flow Difference (%) Compared to Measured Peak Flow			Peak Depth Difference (%) Compared to Measured Peak Depth		
		High Rainfall Gauge	Low Rainfall Gauge	Available Rainfall Gauges	High Rainfall Gauge	Low Rainfall Gauge	Available Rainfall Gauges
28-29 June 2000	ARC 7811	+26.8%	+5.4%	+5.8%	+14.8%	+3.1%	+3.2%
	NIWA 7834	-9.9%	-25.1%	-25.2%	+6.6%	-4.9%	-5.1%
1-2 February 2004	ARC 7811	+24.9%	+4.4%	+23.4%	+14.5%	+2.6%	+14.2%
	NIWA 7834	+2.8%	-15.3%	+2.3%	+15.3%	+2.5%	+14.9%

Notes: + indicates model over prediction and - model under prediction

### 4.3 MIKE21 2D Model Results Validation

A 2D hydraulic model, MIKE 21, has been used for a number of catchments within the North Shore City to develop rapid flood hazard maps. Figure 4.1 shows the 100-year ARI MPD flood depth map of Oteha Valley Catchment based on MIKE 21 model. The MIKE21 modelling uses a 2m grid 2D ground surface generated from LiDAR data, buffers the existing floodplain, assumes the primary drainage system is blocked, and applies ARC TP108 stormwater inflow hydrographs to the upstream of each sub-catchment. The 2D model provides a conservative floodplain estimation which defines the level of worst risk of property flooding within the Catchment.

The 2D model floodplain provides a valuable tool during the development of detailed 1D Oteha Valley Catchment model. The 1D model floodplain was compared with the 2D model floodplain and identified the areas where 1D floodplain differs greatly compared to the 2D floodplain. The 1D model was updated accordingly for accurate schematisation of secondary overland flow paths along the Constellation Drive and William Pickering Drive, accurate modelling of flow diversions at the drop inlet of the Greville Road Culvert.

## 4.4 Model Historical Flood Incident Validation

The habitable floor of the dwelling at 16 Malin Place was reported as being inundated during the 2 February 2004 storm event. The house was inundated up to approximately 50mm above the floor level (RL 52.93m). This information was correlated with the model performance and the results are presented in Table 4.4.

The 1-2 February 2004 storm event was used for validation of the model. To assess the model performance with respect to this storm event a model simulation run was undertaken using rainfall data from four rain gauges around the catchment over a simulation period of 24 hours from 1 February 2004 14:00 to 2 February 2004 14:00.

The property of 16 Malin Place is located in a low-lying area and the house is adjacent to a designated overland flow path. A 525mm diameter pipe passes through the rear of the property that conveys upstream stormwater runoff and discharges into the Hugh Green Drive detention pond. During severe storm events the pipe overflows and overland flow passes through the property.

The model simulation results for the 2 February 2004 storm event indicate that there is no habitable floor flooding occurred at 16 Malin Place. The existing 525mm pipe overflows and overland flow passes through the property but the predicted flood level is below the habitable floor level.

The model is not replicating the reported habitable floor flooding. It is assumed that this could be due to blockage of the catch pits or inadequate capacity of the existing catch pits or lack of adequate catch pits within the designated overland flow path to capture the surface runoff.

## 4.5 Previous Model Results Comparison

The 2000 CMP used the PRISM model to determine peak flows at various locations along the drainage network system within the Oteha Valley Catchment. A comparison of these peak flows with those obtained for model simulations undertaken in the present study is presented in Table 4.5.

Table 4.5 Comparison of modelled peak flows

Location	100-year ARI Flows (m <sup>3</sup> /s)		
	2000 CMP (MPD Scenario)	Present Study (ED Scenario)	Present Study (MPD Scenario)
Rosedale Road Culvert	15.4	31.9	38.9
Tawa Drive Culvert	23.4	47.9	63.6
Bush Road Bridge	51.2	95.5	121.9
Days Bridge at Albany Highway	81.3	122.1	152.4
Oteha Stream Outlet to Lucas Creek	84.7	130.8	163.2

Note: The 2000 CMP used 40% imperviousness for the fully developed catchment whereas the present study used 44.8% & 74.3% imperviousness for ED & MPD scenario respectively.

The present modelling study provides significantly higher peak flows compared to the 2000 CMP model results. Differences in the peak flows between the modelling studies could be expected due to several reasons e.g. differences in the modelling software, modelling methods, design rainfall depth and temporal profile, and land use assumptions.

## 5. SYSTEM PERFORMANCE ASSESSMENT

System performance identifies the level of performance provided by the stormwater drainage system. The system performance consists of the following aspects;

- The capacity of the existing stormwater drainage system.
- An assessment of the level of service provided compared to the desired level of service, and the identification of network sections that do not provide the necessary level of service.
- Floodplain mapping to identify significant flooding areas and buildings at risk of flooding.
- Design flows and water levels for the nominated design storm/land use development scenarios.

### 5.1 Model Scenarios and Simulations

#### 5.1.1 Design Storm Model Scenarios

System performance assessment was carried out for the nominated 6 design storm events (2, 5, 10, 20, 50, and 100 year ARI). The existing and future 24-hour temporal design rainfall profiles were used for performance assessment of the Oteha Valley Catchment stormwater drainage system. The total rainfall depth used in the model for various design storm events is given in Table 5.1 below.

Table 5.1 24-hour design rainfall depths for various design storm events

Design Storm Event	Existing 24-hour Design Rainfall Depth (mm)	Future 24-hour Design Rainfall Depth (mm)
2-Year ARI	80	87.2
5-Year ARI	120	133.6
10-Year ARI	145	164.2
20-Year ARI	170	195.7
50-Year ARI	205	239.4
100-Year ARI	225	262.8

The basic assumption in the design storm approach is the rainfall of a given ARI results in runoff of the same ARI.

### 5.1.2 Land Use Development Model Scenarios

System performance was carried out for Existing Development (ED) and Maximum Probable Development (MPD) land use scenarios. The ED scenario used the existing design rainfall profile and the MPD scenario used the future design rainfall profile which takes into account future climate change.

The imperviousness of the Oteha Valley Catchment is currently estimated at 44.8%. It has been assumed under the MPD scenario (the ultimate future development) development will proceed up to the maximum permitted under the current District Plan.

The current District Plan allows for a maximum permitted impervious area of 70% for residential-zoned land and 80-100% for business-zoned land. Under the MPD scenario the imperviousness of the Catchment is estimated at 74.3%.

### 5.1.3 Simulation Matrix

Table 5.2 below summarises the system performance simulations that were run. Model simulation file details are included in Appendix C. The present modelling study has used a constant downstream tidal boundary conditions of 0.5m RL.

Table 5.2 Model simulation matrix for system performance assessment

Simulation	Land Use	Design Storm Event	Boundary	
			Rainfall	Tide Level for 2100 (m RL)
1	ED	2-year ARI	Existing 2-year	0.5
2	MPD	2-year ARI	Future 2-year	0.5
3	ED	5-year ARI	Existing 5-year	0.5
4	MPD	5-year ARI	Future 5-year	0.5
5	ED	10-year ARI	Existing 10-year	0.5
6	MPD	10-year ARI	Future 10-year	0.5
7	ED	20-year ARI	Existing 20-year	0.5
8	MPD	20-year ARI	Future 20-year	0.5
9	ED	50-year ARI	Existing 50-year	0.5
10	MPD	50-year ARI	Future 50-year	0.5
11	ED	100-year ARI	Existing 100-year	0.5
12	MPD	100-year ARI	Future 100-year	0.5

## 5.2 Water Balance of the Catchment

Table 5.3 below summarises the water balance of the Oteha Valley Catchment for the 6 design storm events under MPD land use scenarios.

Table 5.3 Summary of water balance within the Oteha Valley Catchment

Model Scenario	Catchment Rainfall Volume (m <sup>3</sup> )	Catchment Runoff Volume (m <sup>3</sup> )	Catchment Runoff Volume in %
2 year ED	1,049,136	666,419	64%
5 year ED	1,573,704	1,107,712	70%
10 year ED	1,901,559	1,396,594	73%
20 year ED	2,229,414	1,692,221	76%
50 year ED	2,688,411	2,114,593	79%
100 year ED	2,950,695	2,358,941	80%
2 year MPD	1,143,558	923,936	81%
5 year MPD	1,752,057	1,491,066	85%
10 year MPD	2,153,352	1,872,504	87%
20 year MPD	2,566,449	2,269,575	88%
50 year MPD	3,139,539	2,825,271	90%
100 year MPD	3,446,412	3,123,654	91%

A maximum 91% of catchment rainfall volume was predicted as catchment runoff for 100-year ARI storm event under MPD landuse scenario which is acceptable for the future maximum 74.3% imperviousness of the catchment. During a 100-year ARI storm event the rainfall volume is increased by about 16.8% for the MPD scenario compared to ED scenario and the corresponding runoff volume is increased by about 32.4%.

## 5.3 Capacity of the Existing Primary Pipe System

The capacity of the existing primary pipe system was determined and an assessment of the current level of service provided by the existing stormwater drainage pipe system was undertaken and compared with the AC's legacy Infrastructure Design Standards for new development as summarised below:

- Stormwater reticulation in residential areas – 10% AEP storm event
- Stormwater reticulation in commercial / industrial areas – 5% AEP storm event

The AC's legacy Infrastructure Design Standard also states the hydraulic design of stormwater pipelines must cater for the design peak flow without surcharge in the ultimate future development scenario.

The capacity of the existing primary pipe reticulation system has been assessed for two basic flow conditions: free full flow and backwater effect. Under gravity full flow condition, the flow is controlled by the frictional resistance of the total circumference of the pipe and the free full flow capacity of the pipe conduit was determined using Manning's Equation assuming

- flow is steady-state,
- the pipe is flowing full under gravity,
- the pipe is free from sediment and blockage, and
- pipe friction slope equals the pipe invert slope.

The pipe conduit under backwater effect was assessed by pipe filling which is calculated as the downstream maximum depth divided by the pipe diameter or height, e.g. if the pipe is running under pressure the ratio will be above 1.0.

The capacity of the existing pipe system has also been assessed based on the maximum system flow through the pipe considering no upstream flow restriction or attenuation. This provides the level of flow attenuation due to engineering within the catchment.

A summary of the performance of the existing modelled pipe reticulation system is presented in Table 5.2 for various flow conditions. Modelled peak flows and water levels along the stormwater drainage network for various design storm events under ED and MPD scenarios are provided in Appendix G.

Thematic maps showing the pipe capacity of the existing primary stormwater drainage pipe reticulation system under MPD scenario are presented in Appendix H. Figure H.1, Figure H.2 and Figure H.3 in Appendix H show the capacity of various modelled pipes under free full flow condition, backwater or downstream effect, and free full flow condition based on the maximum system flow through the pipe considering no upstream flow restriction or attenuation respectively.

Table 5.2 Performance of the existing modelled stormwater pipe system

Capacity Assessment Basis	Flow Condition	Scenario	Percentage of Total Number of Pipes with Capacity					
			> 2yr ARI	> 5yr ARI	> 10yr ARI	> 20yr ARI	> 50yr ARI	> 100 yr ARI
Maximum pipe flow compared to pipe full flow capacity	Free full flow	ED	94%	85%	76%	69%	62%	60%
		MPD	91%	77%	68%	62%	57%	55%
Pipe downstream maximum depth compared to pipe diameter or height	Backwater effect	ED	59%	37%	27%	22%	17%	15%
		MPD	47%	27%	21%	17%	12%	10%
Total pipe capacity considering full flow & backwater effect	Worst of above 2 conditions	ED	56%	33%	22%	17%	12%	9%
		MPD	44%	22%	15%	11%	7%	6%
Maximum system pipe flow compared to pipe full flow capacity	Free full flow - No upstream restriction	ED	92%	76%	63%	50%	38%	32%
		MPD	86%	61%	47%	37%	26%	20%

Under free full flow MPD conditions, 32% of the modelled stormwater drainage pipes within the Oteha Valley Catchment do not have the capacity to contain the 10-year ARI MPD storm event without exceeding the soffit of the pipe. However, due to backwater or downstream effects 79% of the modelled pipes would be under surcharge conditions during the 10-year ARI MPD storm event. Considering both free full flow and backwater constraints 85% of the modelled pipes would be under surcharge conditions during the 10-year ARI MPD storm event.

A large percentage of stormwater pipes within the catchment are under the AC's design standard for new reticulation, but these pipes do not necessarily require upgrade. Only those pipelines causing flooding of habitable floor need to be considered for upgrade works, these will be discussed in Options Assessment Report.

## 5.4 Capacity of the Existing Culverts and Bridges

The required level of service for primary drainage system is set out in AC's legacy Infrastructure Design Standards for new development which states that culverts and bridges must be designed to convey 100 year ARI design flows.

The full flow, upstream control capacities and design peak flows for the culverts and bridges within the Oteha Valley Catchment are presented in Table 5.3 below.

Table 5.3 Capacity and design flow for road culverts and bridges

ID	Culvert/Bridge Location	Link ID	Culvert Capacity		MPD Design Flow (m <sup>3</sup> /s)		10-Year Capacity Available
			Full Flow (m <sup>3</sup> /s)	Inlet Control (m <sup>3</sup> /s)	10-year ARI	100-Year ARI	
1	R 99 Apollo Drive	1548082	15.1	4.5	8.0	11.2	No
2	7-9 Apollo Drive	1548974, 1548975 & 297856	21.4	12.3	4.9	12.3	Yes
3	R 2 Centorian Drive	297963	13.4	5.5	7.1	10.5	No
4	R 52 Unsworth Drive	295108	3.4	1.4	5.7	9.4	No
5	R 74 Barbados Drive	1549094 & 335051	10.3	5.4	19.0	35.1	No
6	Upper Harbour Highway	1546119	17.8	11.2	18.9	34.0	No
7	Paul Matthews Road	1546524	22.0	11.2	22.1	29.9	No
8	Condor Place	1524009	7.8	1.4	1.7	2.2	No
9	Princeton Parade	1547274 & 312197	5.3	4.0	9.7	14.4	No
10	R 320 Rosedale Road	1552290	85.6	22.2	26.6	39.9	No
11	R 15 Tawa Drive	1546116	128.3	54.0	41.1	63.7	Yes
12	Road in Massey Uni Campus	1546113	18.4	8.0	4.0	6.5	Yes
13	Road in Massey Uni Campus	1546107	40.0	8.0	5.9	9.8	Yes
14	Albany Highway/ Library Lane	459079	9.0	2.7	8.1	9.9	No
15	Albany Highway Days Bridge	AlbanyRd_Bridge	182.7	-	99.5	152.4	Yes
16	Bush Road Bridge	BushRd_Bridge	74.9	-	80.2	121.9	No

- Notes:
- Full flow capacity assumes pipe/bridge flow full-barrel with the water surface slope equal to the bed slope and Manning n values of;
    - 0.014 for culverts,
    - 0.050 for bridges with a natural invert
  - Inlet control capacity (due to culvert inlet configuration) is the flow that can enter the culvert inlet with a headwater level at the upstream culvert soffit.

With the exception of the triple culverts at Apollo Drive (No. 2), culverts at Tawa Drive (No. 11) and Massey University Campus (No. 12 & 13); all culverts are too small to pass the 10-year ARI design flow without surcharging at the inlet.

The Albany Highway Bridge has the capacity to convey the 100-year ARI capacity without surcharge whereas the Bush Road Bridge does not have the capacity to convey even the 10-year ARI flow.

Based on the above analysis, 10 culverts and 1 bridge are under-capacity and would require upgrading. However, the need for culvert/bridge upgrading works is not necessarily required unless an existing habitable floor is at risk of flooding or a road is overtopped too frequently. Some surcharge upstream of under-sized culverts can be acceptable in order to provide the necessary head to push the attenuated design flow through the culvert. The required upgrade works will be discussed in Options Assessment Report.

## 5.5 Flood Risk Areas and Floodplain Mapping

Flood risk areas were assessed in relation to the AC's legacy Infrastructure Design Standards for new development as summarised below:

- Living areas of buildings – 1% AEP flood level with a minimum of 500mm freeboard
- Non living areas of buildings / garages – 1% AEP flood level with no freeboard provision

The results of the hydraulic model were used to identify the extent of floodplains and potentially significant overland flow paths. Floodplain extent maps were developed for the 10-year and 100-year ARI storm events under maximum probable land use development with future rainfall scenario. The flood extent mapping for overland flow path was carried out for those with peak flow greater than 2.0 m<sup>3</sup>/s.

In addition to the flood extent mapping, major overland flow paths where overland flow exceeds 0.5 m<sup>3</sup>/s during the 100 year ARI MPD storm event are identified in the floodplain extent maps.

The floodplain extent maps for the 10-year and 100-year ARI MPD storm events including flood sensitive area (extent of flooding for 500mm plus 100-year ARI MPD floodplain) are presented in Appendix F.

Stormwater flooding can be caused by several circumstances e.g. flooding from rising streams and channels, flooding from water backing up from some downstream restriction to flows, flooding by overland flows passing through properties located on flow paths, flooding from water spilling from channels or flow paths located adjacent to properties.

A number of significant flood hazard areas where residential or commercial buildings are located within the extent of floodplain were identified within the Oteha Valley Catchment. The list of flood risk areas including the number of residential and business floor flooding and the causes of flooding is presented in Table 5.4.

The floodplain extent maps for the 100-year ARI MPD storm event are presented in Appendix I. The major overland flow paths where overland flow exceeds  $0.5 \text{ m}^3/\text{s}$  during the 100 year ARI MPD storm event are identified in the floodplain extent maps.

System performance modelling identified a total of 597 flood affected properties within the Oteha Valley Catchment which are subject to either nuisance flooding or floor flooding during the 100-year ARI MPD storm event.

A total of 39 buildings within the Catchment are predicted to be within the extents of the 100-year ARI MPD floodplain. The floor levels of these 39 buildings at risk of flooding have been surveyed. Of these buildings a total of 6 residential and 4 business floors are predicted to be inundated, 17 residential and 2 business floors are within 500mm of the predicted peak 100-year ARI flood levels and 7 residential and 3 business floors are above 500mm of the predicted peak 100-year ARI flood levels. Lists of properties and buildings at risk of flooding are presented in Appendix J.

Several flooding issues were identified within the Oteha Valley Catchment as reported in the Property Issues Register (PIR) database. The major source of flooding was reported to be overland flow from upstream catchment. A total of 4 habitable floors, 1 non-habitable floor, and 30 properties were reported as flooding. Out of these 6 properties are within the predicted 100-year ARI MPD floodplain. The majority of these flooding issues are localised problems related to deficiencies within individual private drainage systems.

The main areas affected by flooding in the Oteha Valley Stream Catchment are located along the main Oteha Stream due to spilling over stream bank. A total of 17 buildings were identified within the main Oteha Stream floodplain (refer to Figures 5.1 & 5.2). No buildings were identified within the Alexandra Stream floodplain. Details of the significant flood hazard areas are discussed below.

Building floor flooding issues associated with overland flow paths resulting from localised drainage issues are addressed in a separate study.

Table 5.4 Flood risk areas with summary of building floor flooding

ID	Flood Hazard Area Location	100 Year ARI Storm (MPD)						Land Use	Reason for Flooding	Reported Flooding Issues	Appendix I Figure Reference
		Floor Flooding		Floor within 500mm		Floor above 500mm					
		Residential	Business	Residential	Business	Residential	Business				
1	Vanderbilt Parade	2	-	1	-	-	-	Residential	Flooding from Stream Bank Spilling	None	Figure I-C2
2	Pepperdine Place	1	-	7	-	-	-	Residential	Flooding from Stream Bank Spilling	None	Figure I-C2
3	Villanova Place	-	-	-	-	2	-	Residential	Flooding from Stream Bank Spilling	None	Figure I-C2 & Figure I-C3
4	Kristin Lane	1	-	2	-	-	-	Residential	Flooding from Stream Bank Spilling	Yes	Figure I-C4
5	249 Rosedale Road	-	-	-	-	-	1	Business	Flooding from Stream Bank Spilling	None	Figure I-C3
6	Notre Dame Way	1	-	-	-	3	-	Residential	Flooding from D/S Culvert Restriction	None	Figure I-C2
7	117 & 121 Rosedale Road	-	1	-	-	-	1	Business	Flooding from D/S Culvert Restriction	None	Figure I-C5
8	Centorian Drive	-	-	1	-	2	-	Residential	Flooding from D/S Culvert Restriction	None	Figure I-D7
9	Tawa Drive	-	-	-	1	-	-	Business	Flooding from Overland Flow	None	Figure I-C4
10	Apollo Drive	-	2	-	-	-	-	Business	Flooding from Overland Flow	None	Figure I-D6
11	Hugh Green Drive	-	-	4	-	-	-	Residential	Flooding from Overland Flow	None	Figure I-A5 & Figure I-B5
12	Te Hoe Grove	1	-	1	-	-	-	Residential	Flooding from Overland Flow	None	Figure I-A5
13	Arirang Rise	-	-	1	-	-	-	Residential	Flooding from Overland Flow	None	Figure I-A5
14	Constellation Drive	-	1	-	-	-	-	Business	Flooding from Overland Flow	None	Figure I-D6
15	Meadowood Drive	-	-	-	1	-	1	Residential	Flooding from Overland Flow	None	Figure I-E5
Total		6	4	17	2	7	3	= Total 39 buildings within the 100-year ARI floodplain			

### *Oteha Stream Floodplain*

A total of 13 buildings along the left bank of the Oteha Stream from downstream of its confluence with the Alexandra Stream at Bush Road to Stanford Street are predicted within the 100-year ARI MPD floodplain (refer to Figure 5.1). Of these 13 buildings, a total of 3 residential floors are predicted to be inundated and 8 residential floors are within 500mm of the predicted peak 100-year ARI MPD flood levels, and 2 residential floors are above 500mm of the predicted peak 100-year ARI MPD flood level (Table 5.4: No. 1 to 3).

A total of 4 buildings along the left bank of the Oteha Stream from downstream of Tawa Drive to its confluence with the Alexandra Stream at Bush Road are predicted within the 100-year ARI MPD floodplain (refer to Figure 5.2). Of these 4 buildings, a total of 1 residential floor is predicted to be inundated and 2 residential floors are within 500mm of the predicted peak 100-year ARI MPD flood levels and 1 business floor is above 500mm of the predicted peak 100-year ARI MPD flood level (Table 5.4: No. 4 to 5).

The Oteha Stream at these locations has insufficient capacity to pass the 100-year ARI design flow resulting in increased extent of flooding due to spilling of left banks in the low lying area along the left bank of the Oteha Stream. Photos along the Oteha Stream left bank floodplain are shown below (Photos 5.1 to 5.6).



Photo 5.1 Oteha Stream left bank floodplain behind Vanderbilt Parade



Photo 5.2 Oteha Stream tributary floodplain behind Vanderbilt Parade



Photo 5.3 Oteha Stream left bank floodplain behind Pepperdine Place



Photo 5.4 Oteha Stream left bank floodplain behind Pepperdine Place



Photo 5.5 Oteha Stream left bank floodplain behind Kristin Lane



Photo 5.6 Oteha Stream left bank floodplain behind Kristin Lane (NSCC Pump Station 16, 249 Rosedale Road)

### *Flooding from Downstream Culvert Restriction*

#### *Notre Dame Way*

A total of 4 buildings around Notre Dame Way are predicted within the Oteha Stream left bank tributary floodplain during the 100-year ARI MPD storm event. The existing 1200mm diameter twin culverts under Princeton Parade have insufficient capacity to pass the 100-year ARI design flow resulting in significant ponding with a maximum depth of 6.1m upstream of the twin culverts (refer to Figure 5.1 and Photo 5.7). One residential floor is predicted to be inundated (Table 5.4: No. 6).

#### *Rosedale Road*

Significant ponding is predicted to occur in the low-lying area of 117 Rosedale Road during the 100-year ARI MPD storm event (refer to Figure 5.2 and Photo 5.8). The existing 2400mm diameter pipeline under Rosedale Road and State Highway 1 restricts upstream overland and stream flows resulting in inundating the areas to a maximum depth of 2.5m during the 100-year ARI MPD storm event. A total of 2 buildings are predicted within the ponding area and 1 business floor is predicted to be inundated (Table 5.4: No. 7).



Photo 5.7 Oteha Stream left bank tributary floodplain behind Princeton Parade



Photo 5.8 Low lying area at 117 Rosedale Road

### *Centorian Drive*

The existing 750mm diameter culvert under the dam upstream of the cut-off drain along the eastern Rosedale Lake restricts upstream stream flows resulting in significant ponding with a maximum depth of 7.5m upstream of the culvert during the 100-year ARI MPD design storm event (refer to Figure 5.4). Once the ponding reaches the dam crest the stream overtops the dam and discharges into the cut-off drain with a maximum flow of 22.3 m<sup>3</sup>/s. Due to downstream restriction water backs up to the Centorian Drive resulting in inundating of a number of buildings along the left bank of the Oteha Stream Tributary. A total of 3 buildings are predicted within the 100-year ARI MPD floodplain with 1 residential floor within 500mm of the predicted peak 100-year ARI MPD flood levels (refer to Table 5.4: No. 8 and Photo 5.9).



Photo 5.9 27 Centorian Drive along the Oteha Stream Tributary left bank floodplain

### *Overland Flow Path Floodplain*

A total of 13 buildings are predicted within the floodplain extent of overland flow paths during the 100-year ARI MPD storm event (refer to Figures 5.2 to 5.4). One residential and 3 business floors are predicted to be inundated, 6 residential and 2 business floors are within 500mm of the predicted peak 100-year ARI MPD flood levels, and 1 business floor is above 500mm of the predicted peak 100-year ARI MPD flood level (Table 5.4: No. 13 to 19). The overland flow along these flood hazard areas varies from 2.8 m<sup>3</sup>/s in Malin Place to a maximum of 5.7 m<sup>3</sup>/s in Apollo Drive. Photos of buildings at risk of flooding within the overland flow path floodplain are shown below (Photos 5.10 to 5.18).



Photo 5.10 2 Te Hoe Groove within the overland flow path floodplain



Photo 5.11 20 Arirang Rise within the overland flow path floodplain



Photo 5.12 17 Te Hoe Groove on overland flow path floodplain



Photo 5.13 55 Meadowood Drive within the overland flow path floodplain



Photo 5.14 28 Hugh Green Drive within the overland flow path floodplain



Photo 5.15 30 Hugh Green Drive within the overland flow path floodplain



Photo 5.16 122 Hugh Green Drive within the overland flow path floodplain



Photo 5.17 101 Hugh Green Drive within the overland flow path floodplain



Photo 5.18 8 Apollo Drive within the overland flow path floodplain

## 5.6 Previous Floodplain Comparison

A comparison of present model predicted 100-year ARI MPD floodplain with the previous 100-year ARI MPD floodplain is shown in Figure 5.5. The previous floodplain identified a total of 8 buildings within the extents of the 100-year ARI MPD floodplain whereas the present floodplain identified a total of 39 buildings. As discussed in Section 4.5 the present modelling study predicted significantly higher peak flows compared to the previous 2000 CMP model results. The present floodplain covers an area of about 62 ha (excluding Rosedale Lakes) compared to about 16 ha covered by the previous floodplain.

## 6. CONCLUSIONS and RECOMMENDATIONS

- A hydrological and hydraulic model of the stormwater drainage network system in Oteha Valley Catchment has been developed using MIKE FLOOD modelling software based on ARC TP108 rainfall-runoff modelling methods and 1-D and 2-D free surface gradually varied unsteady flow equations.
- The model gauge validation results indicate that in general the model has been validated within reasonable agreement with the measured gauge data considering this large catchment with complex stormwater drainage system.
- It has been found that under free full flow conditions 32% of the modelled stormwater drainage pipes within the Oteha Valley Catchment do not have the capacity to contain the 10-year ARI MPD storm event without exceeding the soffit of the pipe. However, due to backwater or downstream effects 79% of the modelled pipes would be under surcharge conditions during the 10-year ARI MPD storm event. Considering both free full flow and backwater constraints 85% of the modelled pipes would be under surcharge conditions during the 10-year ARI MPD storm event.
- It has been found majority of culverts within the Oteha Valley Catchment do not meet Auckland Council level of service, too small to pass the 10-year ARI design flow without surcharging at the inlet.
- This modelling investigation has identified a total of 597 flood affected properties within the Oteha Valley Catchment which are subject to either nuisance flooding or floor flooding during the 100-year ARI MPD storm event.
- A total of 39 buildings within the catchment are predicted to be within the extents of the 100-year ARI MPD floodplain. Of these buildings a total of 6 residential and 4 business floors are predicted to be inundated and 17 residential and 2 business floors are within 500mm of the predicted peak 100-year ARI flood levels.
- It has been found that the main areas affected by 100-year ARI storm flooding are located along the main Oteha Stream due to overtopping of the stream bank. A total of 17 buildings were identified within the main Oteha Stream floodplain. No buildings were identified within the Alexandra Stream floodplain.
- It is recommended that outcomes of this system performance assessment be used as the basis for the subsequent system improvement works options development.

## 7. REFERENCES

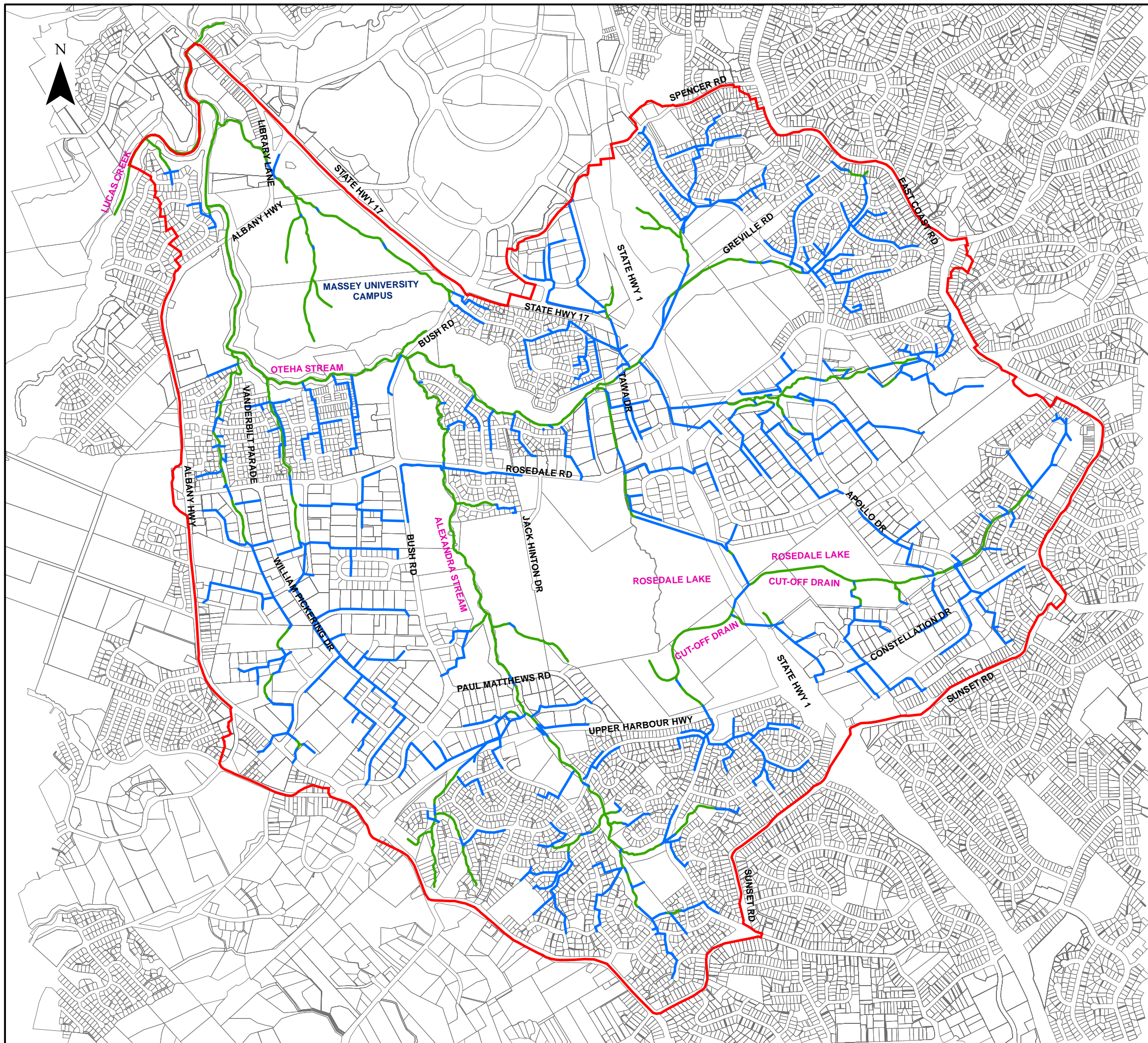
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Ministry for the Environment (2008). *Climate Change Effects and Impacts Assessment, A Guidance Manual for Local Government in New Zealand*. 2<sup>nd</sup> Edition, Ministry for the Environment, Wellington.

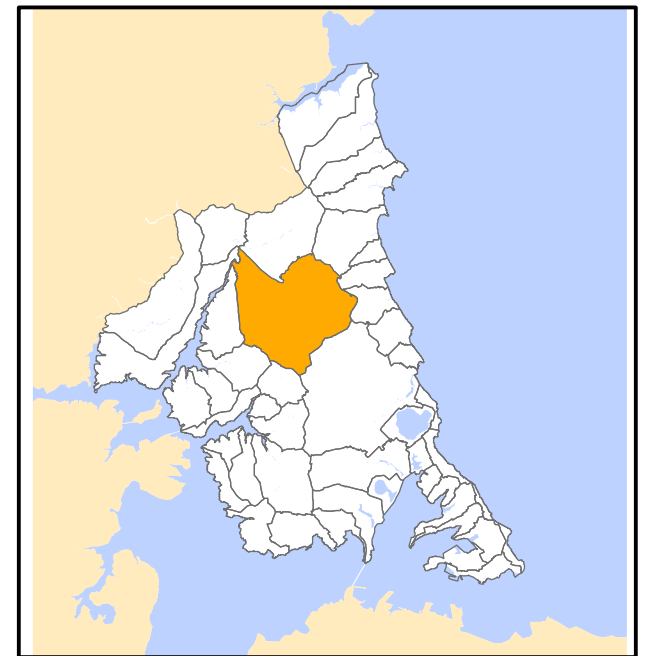
## **Figures**

- Figure 2-1    Modelled Stormwater Drainage System**
- Figure 2-2    Oteha Catchment Surficial Soil Type**
- Figure 2-3    Oteha Catchment Land Use**
- Figure 3-1    A1-Asset Data Source**  
**A2-Asset Data Source**  
**B1-Asset Data Source**  
**B2-Asset Data Source**
- Figure 3-2    Oteha Model SubCatchments**
- Figure 4-1    100 Year ARI MPD Flood Depth Based on MIKE 21 Model**
- Figure 5-1    100 Year ARI MPD Flooded Buildings**
- Figure 5-2    100 Year ARI MPD Flooded Buildings**
- Figure 5-3    100 Year ARI MPD Flooded Buildings**
- Figure 5-4    100 Year ARI MPD Flooded Buildings**
- Figure 5-5    Comparison of Present and Previous 100 Year ARI MPD Flood Plain**



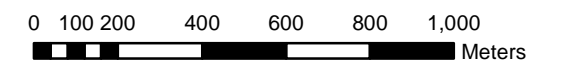
**LEGEND**

- OPEN WATERCOURSE
- MODELLED SW PIPE
- CATCHMENT BOUNDARY
- PROPERTY BOUNDARY



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CHECKED: NB

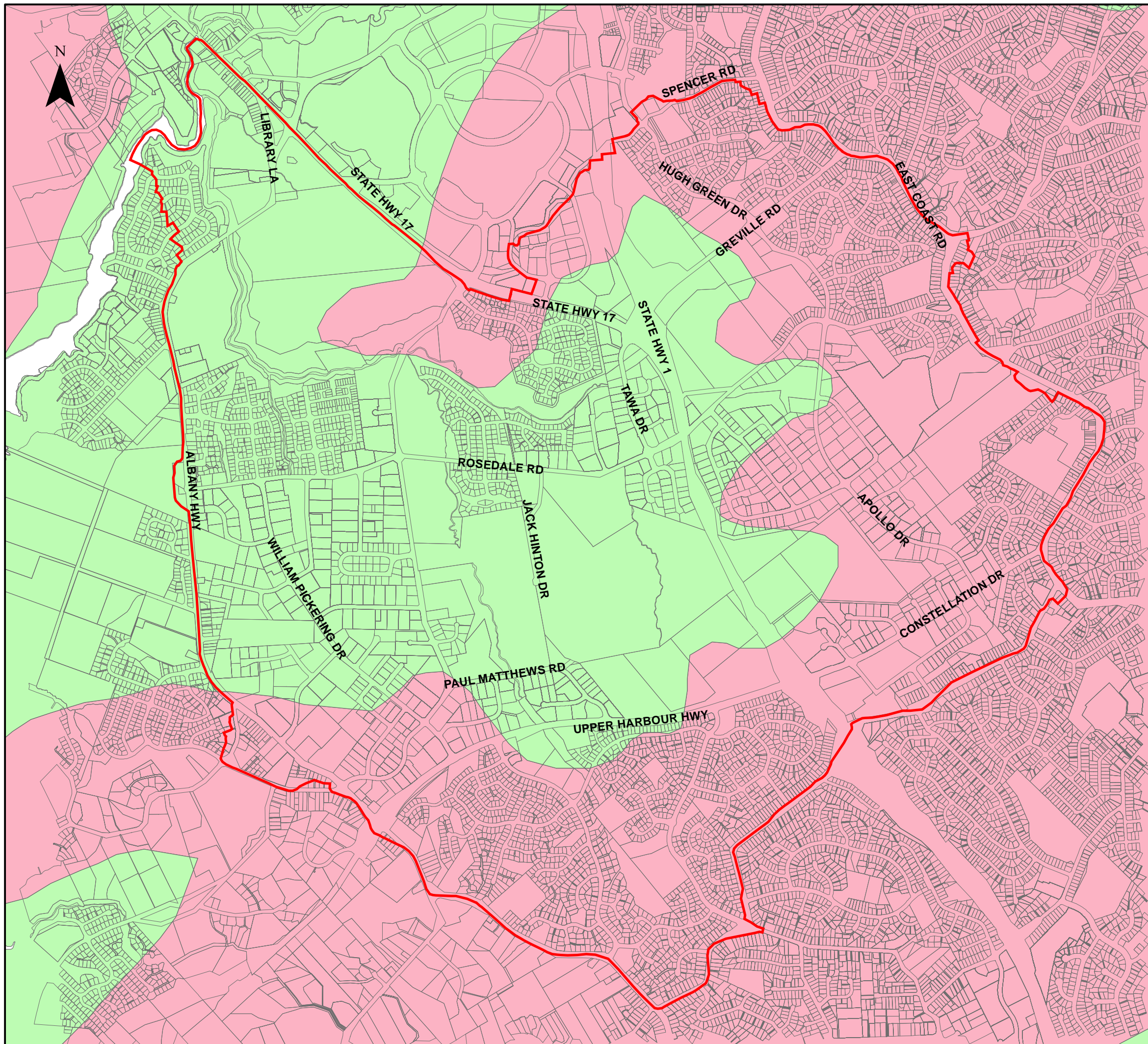


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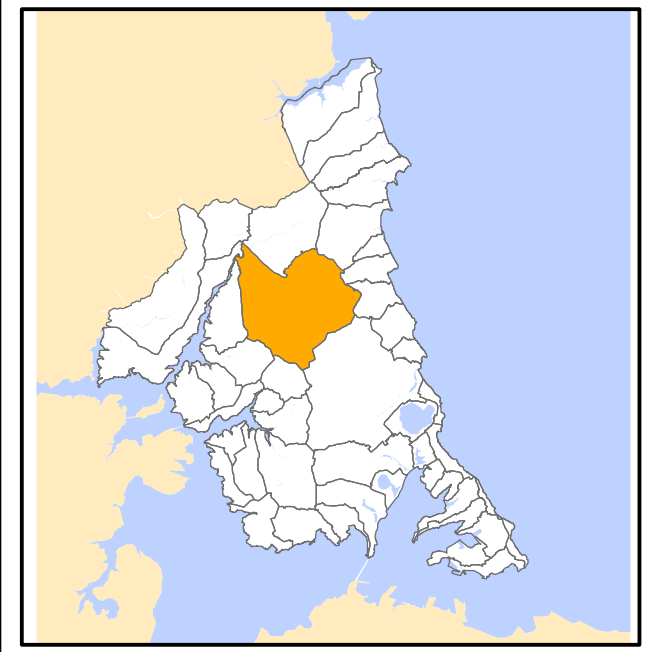
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Figure 2.1 Modelled Stormwater Drainage System



**LEGEND**

- SURFICIAL SOIL TYPE**
- WAITEMATA GROUP
  - QUATERNARY SEDIMENTS
- CATCHMENT BOUNDARY**
- 
- PROPERTY BOUNDARY**
- 



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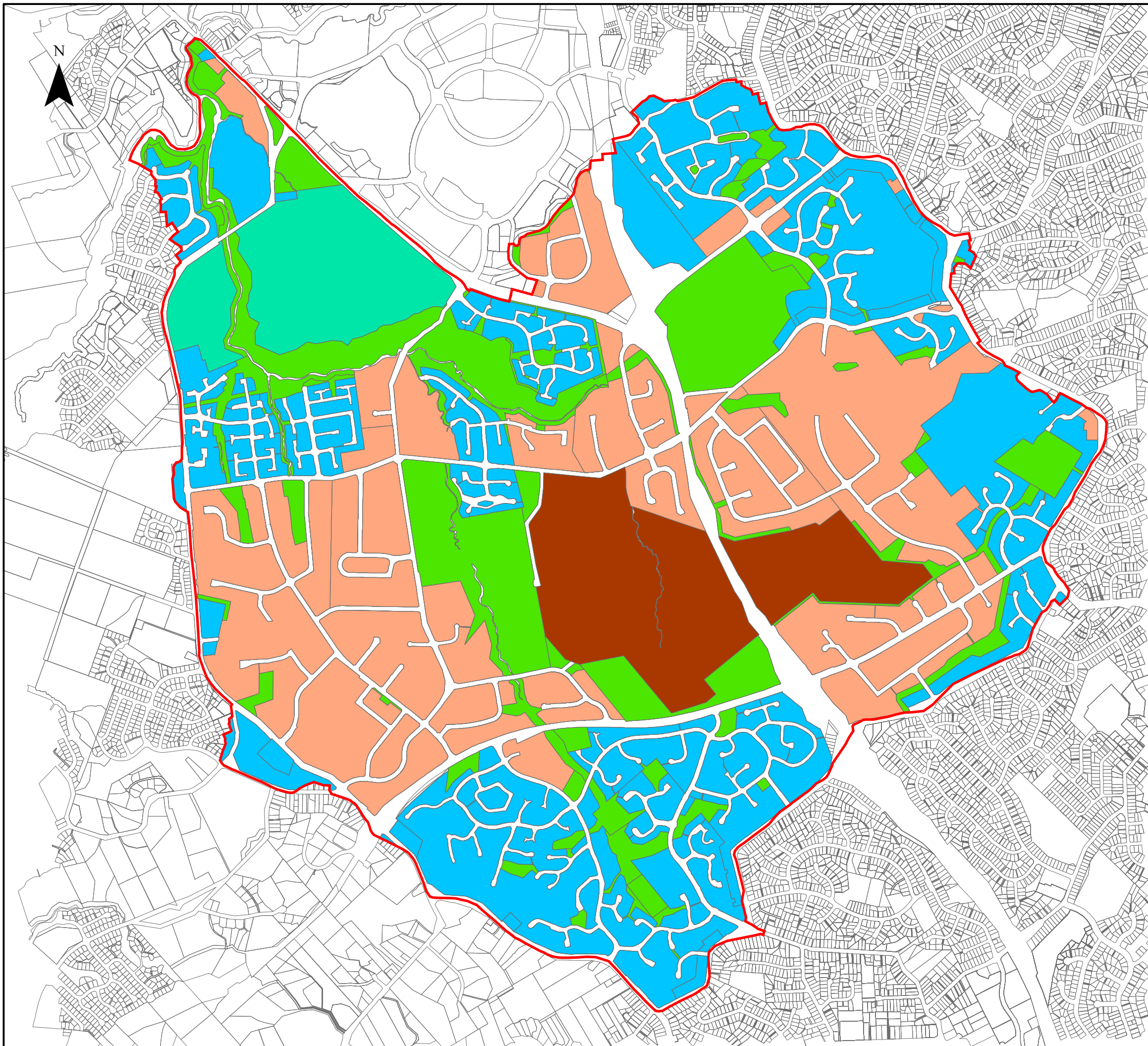
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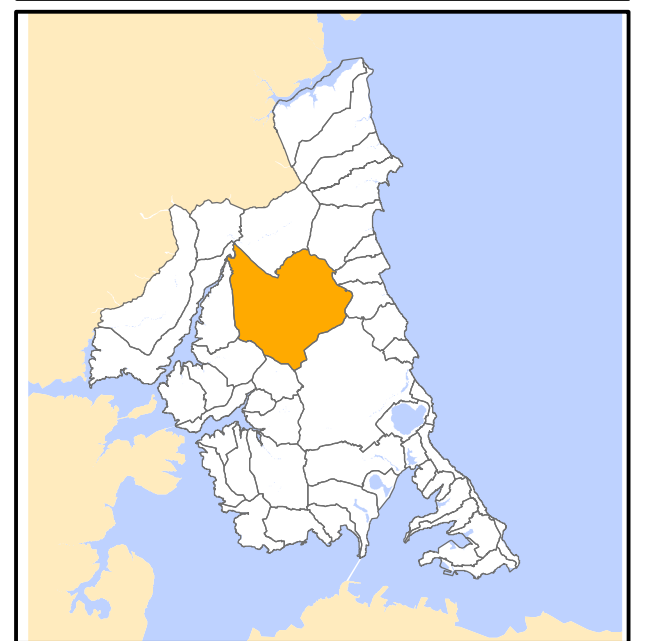
Figure 2.2 Oteha Catchment Surficial Soil Type



**LEGEND**

**OTEHA LAND USE**

- Residential
- Business
- Recreation
- SP-Education
- SP-Wastewater Treatment Plant
- Road
- CATCHMENT BOUNDARY



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CHECKED: NB

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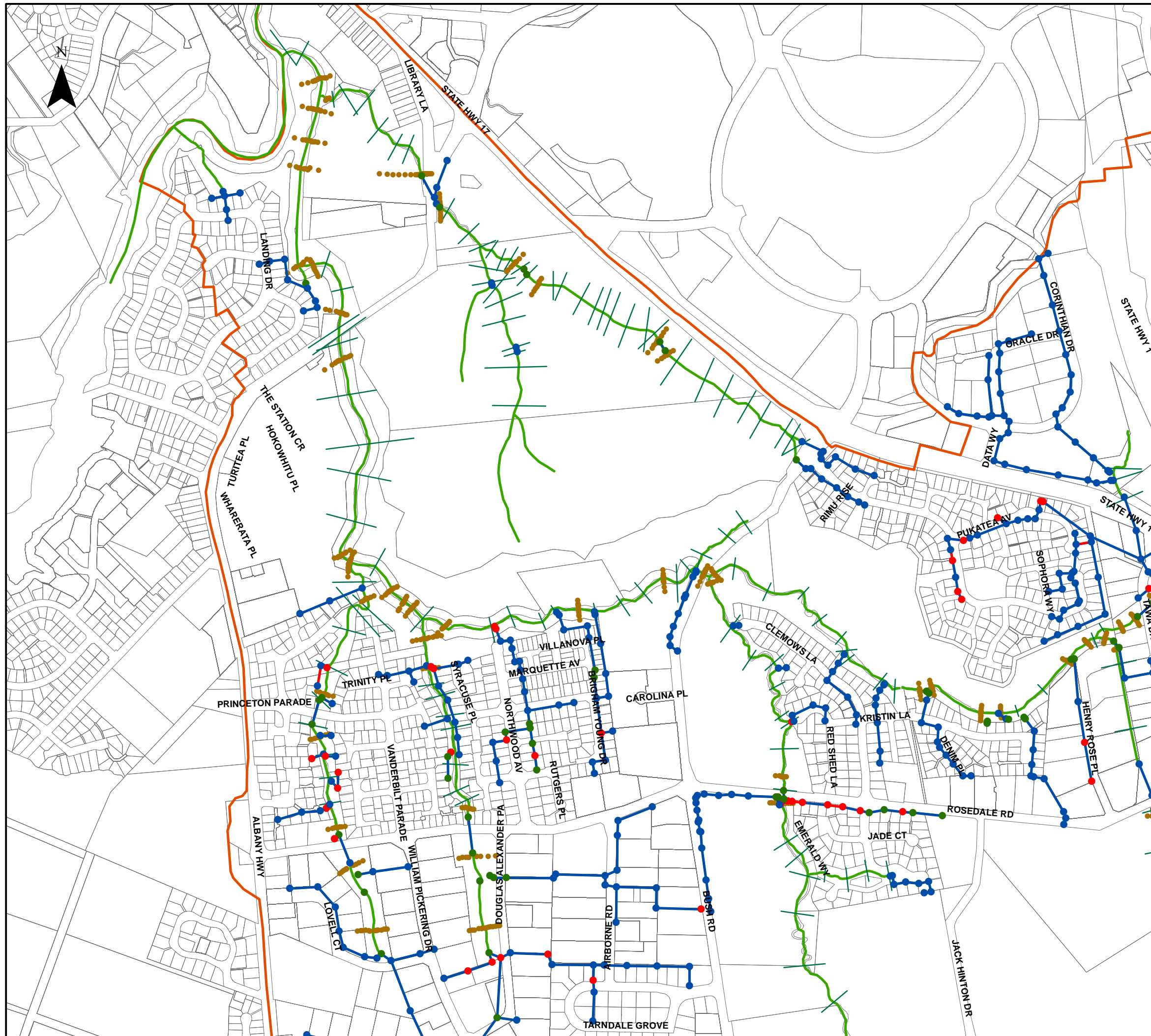
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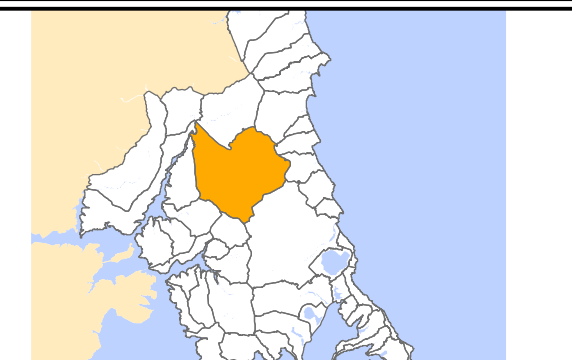
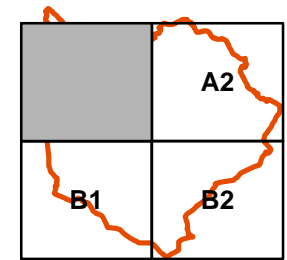
Figure 2.3 Oteha Catchment Land Use



**LEGEND**

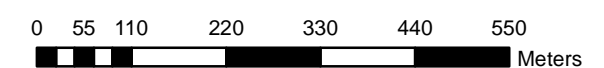
**ASSET DATA SOURCE**

- MANHOLES - GIS
- MANHOLES - SURVEY
- MANHOLES - ESTIMATED
- PIPES - GIS
- PIPES - ESTIMATED
- LIDAR CROSS-SECTIONS
- SURVEY CROSS-SECTIONS
- OPEN WATERCOURSE



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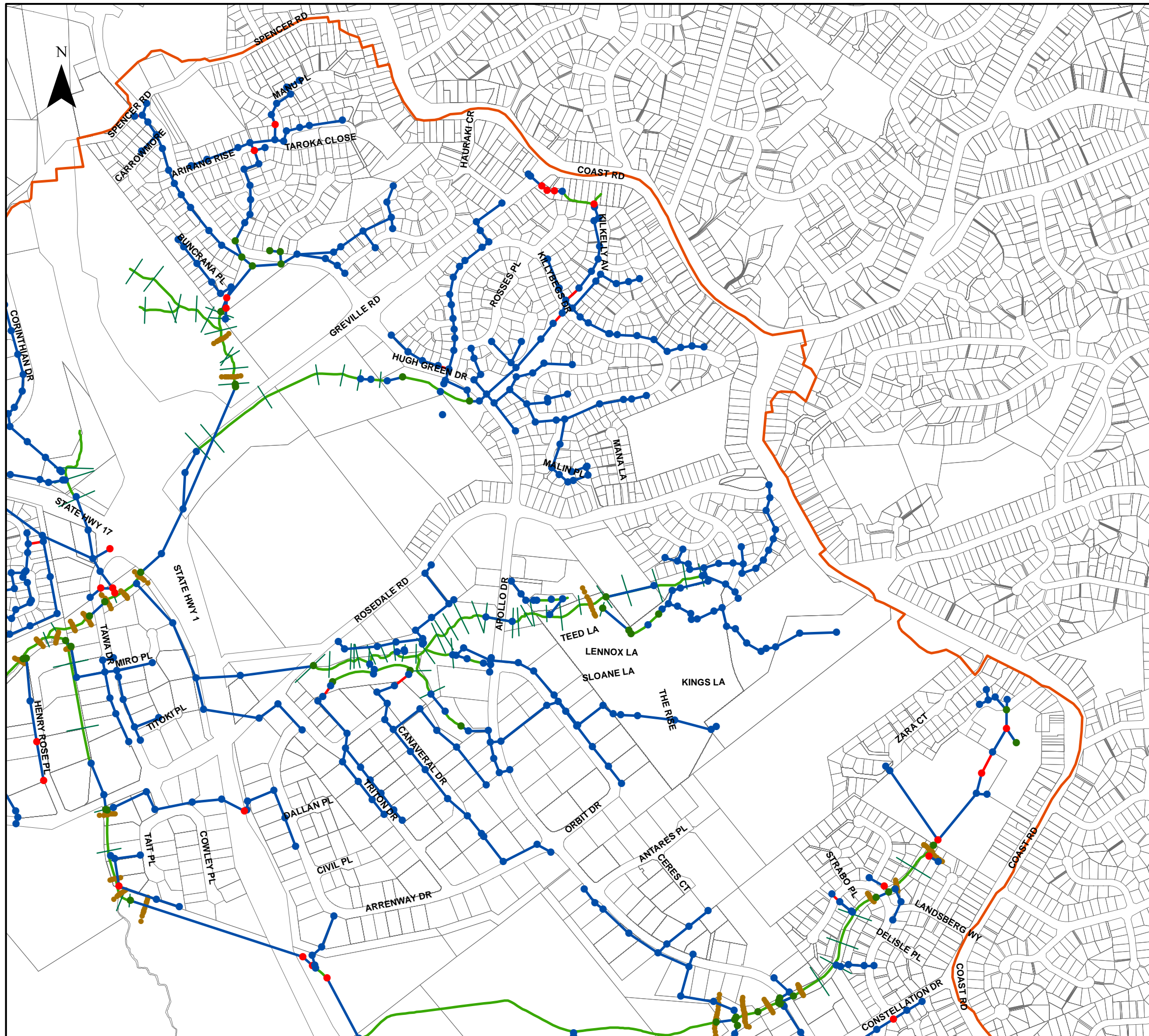


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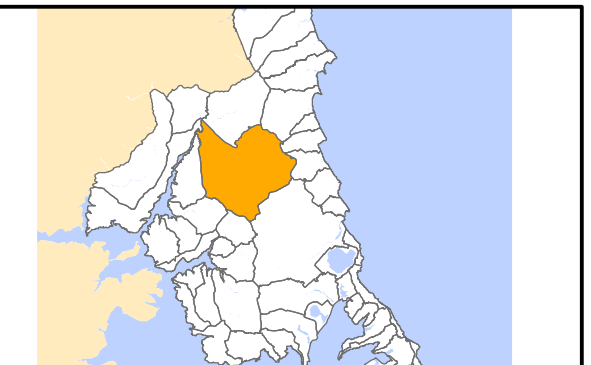
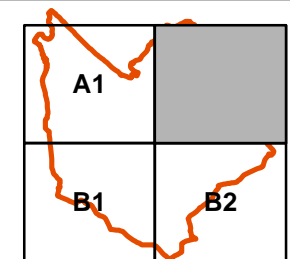
Figure 3.1-A1 Asset Data Source



**LEGEND**

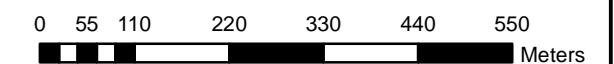
**ASSET DATA SOURCE**

- MANHOLES - GIS
- MANHOLES - SURVEY
- MANHOLES - ESTIMATED
- PIPES - GIS
- PIPES - ESTIMATED
- LIDAR CROSS-SECTIONS
- SURVEY CROSS-SECTIONS
- OPEN WATERCOURSE



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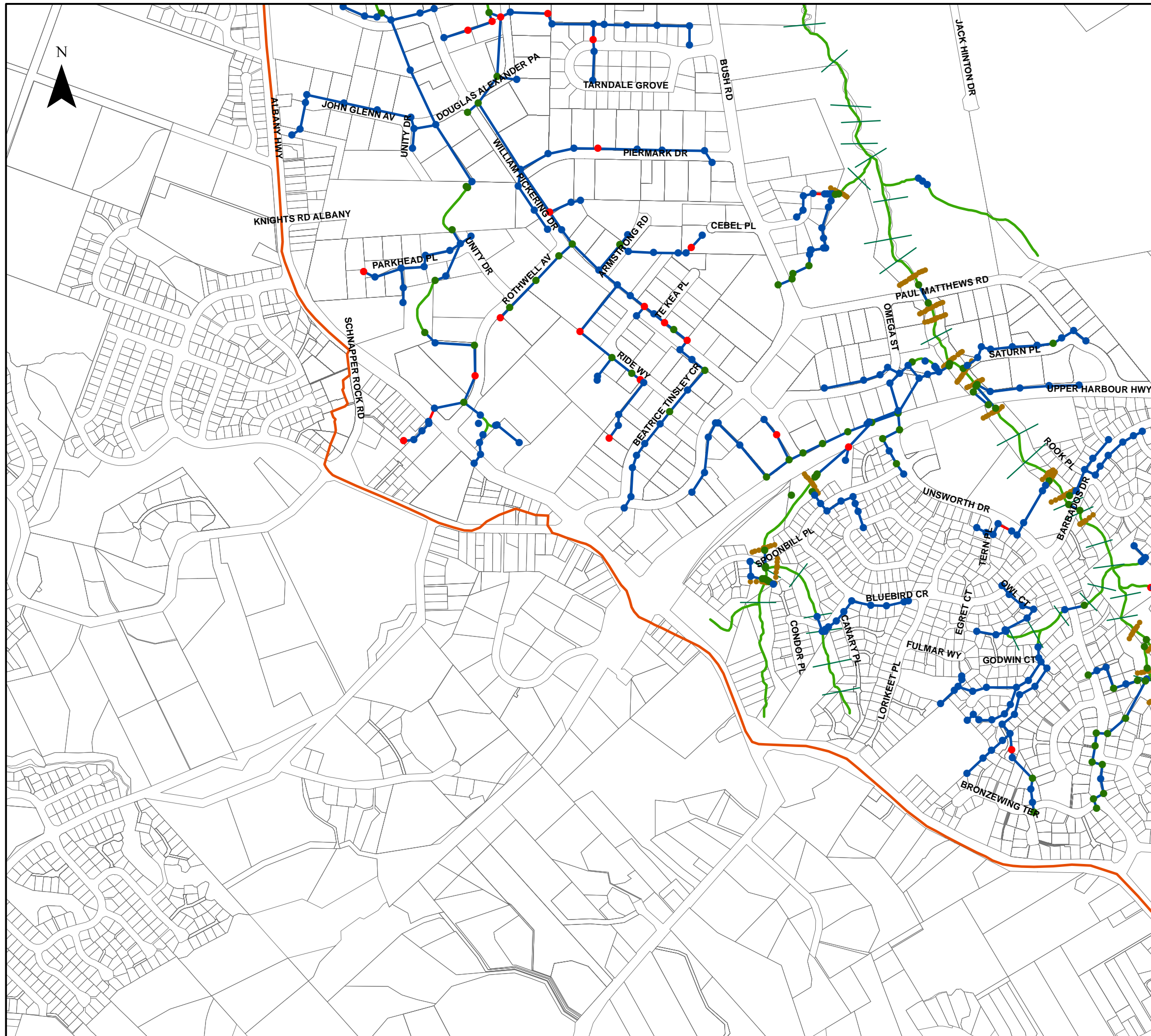


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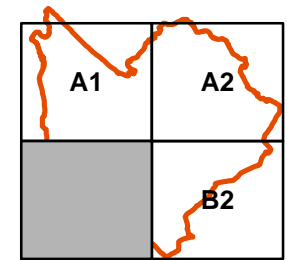
Figure 3.1-A2 Asset Data Source



**LEGEND**

**ASSET DATA SOURCE**

- MANHOLES - GIS
- MANHOLES - SURVEY
- MANHOLES - ESTIMATED
- PIPES - GIS
- PIPES - ESTIMATED
- LIDAR CROSS-SECTIONS
- SURVEY CROSS-SECTIONS
- OPEN WATERCOURSE



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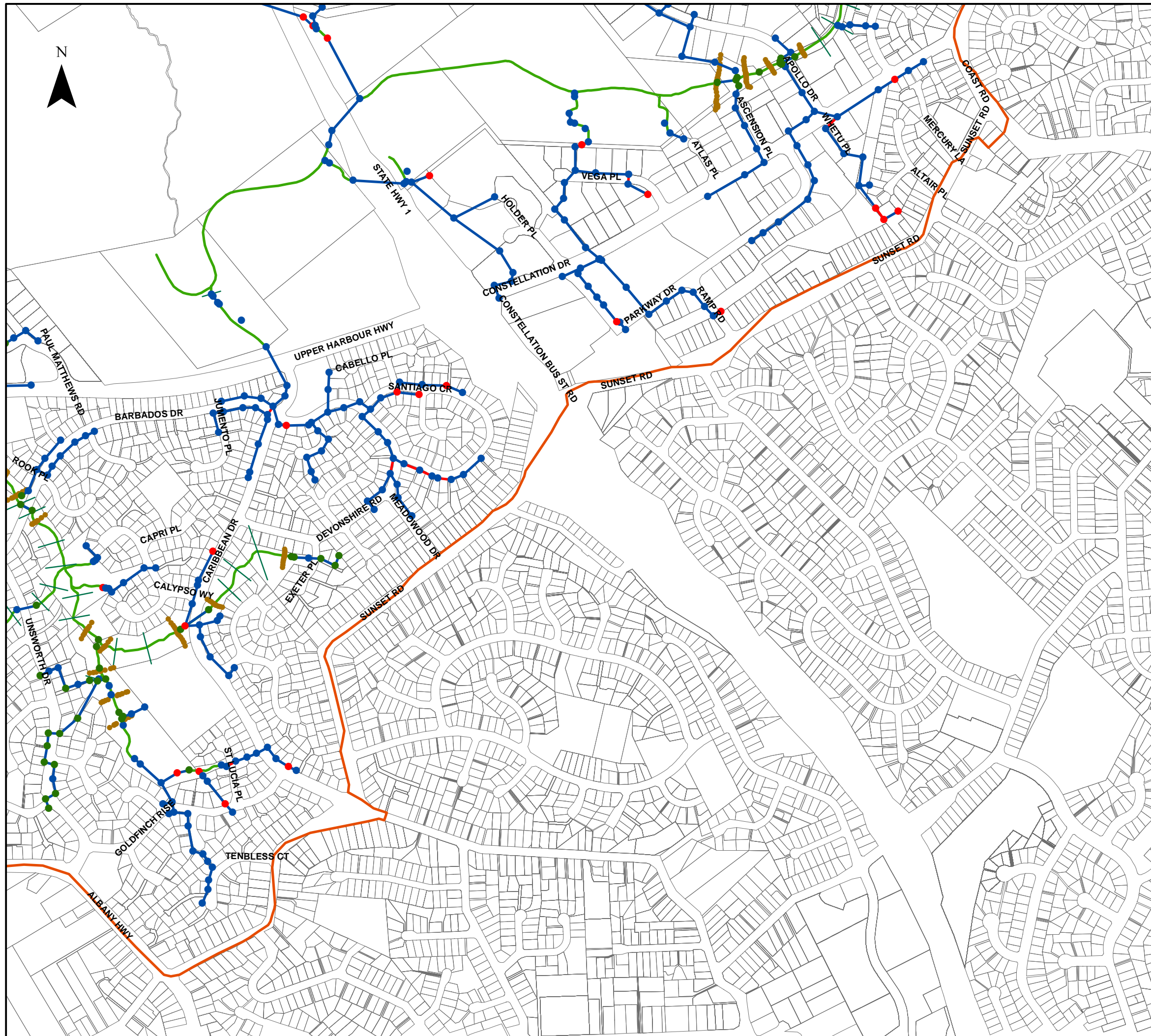
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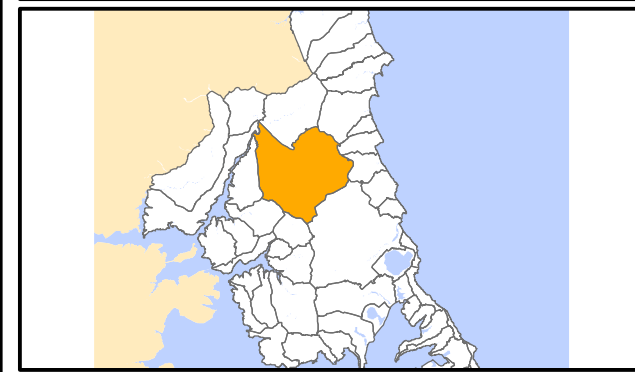
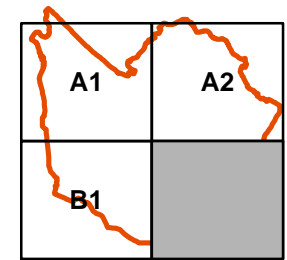
Figure 3.1-B1 Asset Data Source



**LEGEND**

**ASSET DATA SOURCE**

- MANHOLES - GIS
- MANHOLES - SURVEY
- MANHOLES - ESTIMATED
- PIPES - GIS
- PIPES - ESTIMATED
- LIDAR CROSS-SECTIONS
- SURVEY CROSS-SECTIONS
- OPEN WATERCOURSE



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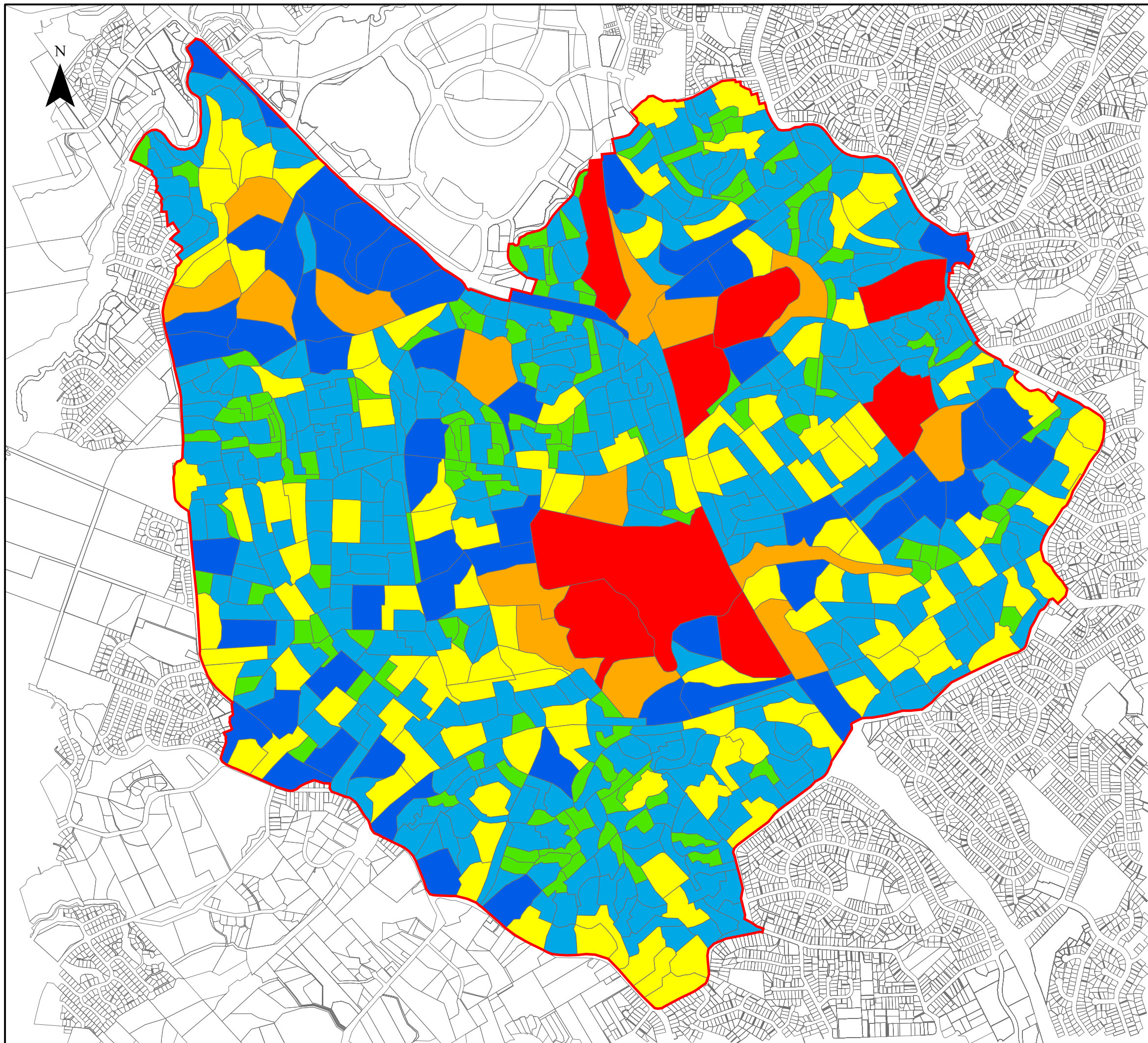
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Meters

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Figure 3.1-B2 Asset Data Source

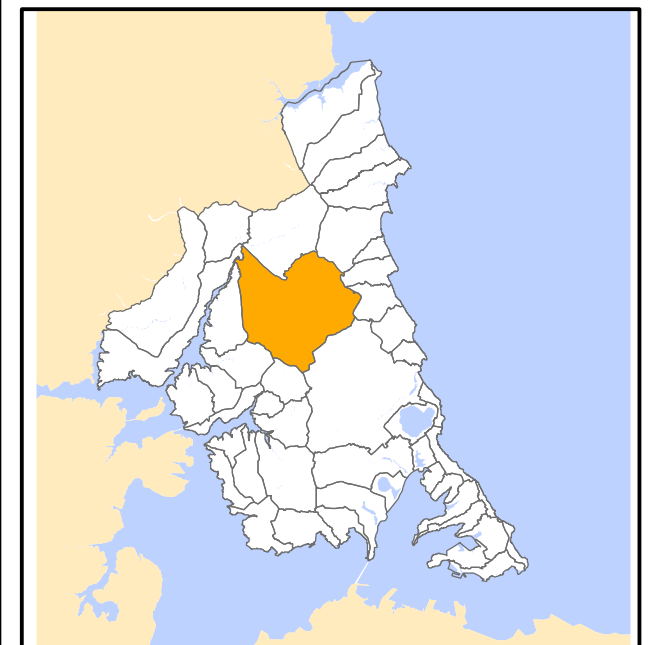


**LEGEND**

**MODEL SUBCATCHMENTS**

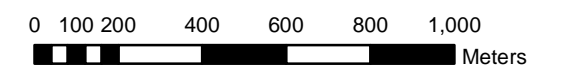
**Catchment Area (ha)**

- 0.4 - 1.0
- 1.0 - 2.0
- 2.0 - 3.0
- 3.0 - 5.0
- 5.0 - 8.0
- 8.0 - 40
- CATCHMENT BOUNDARY



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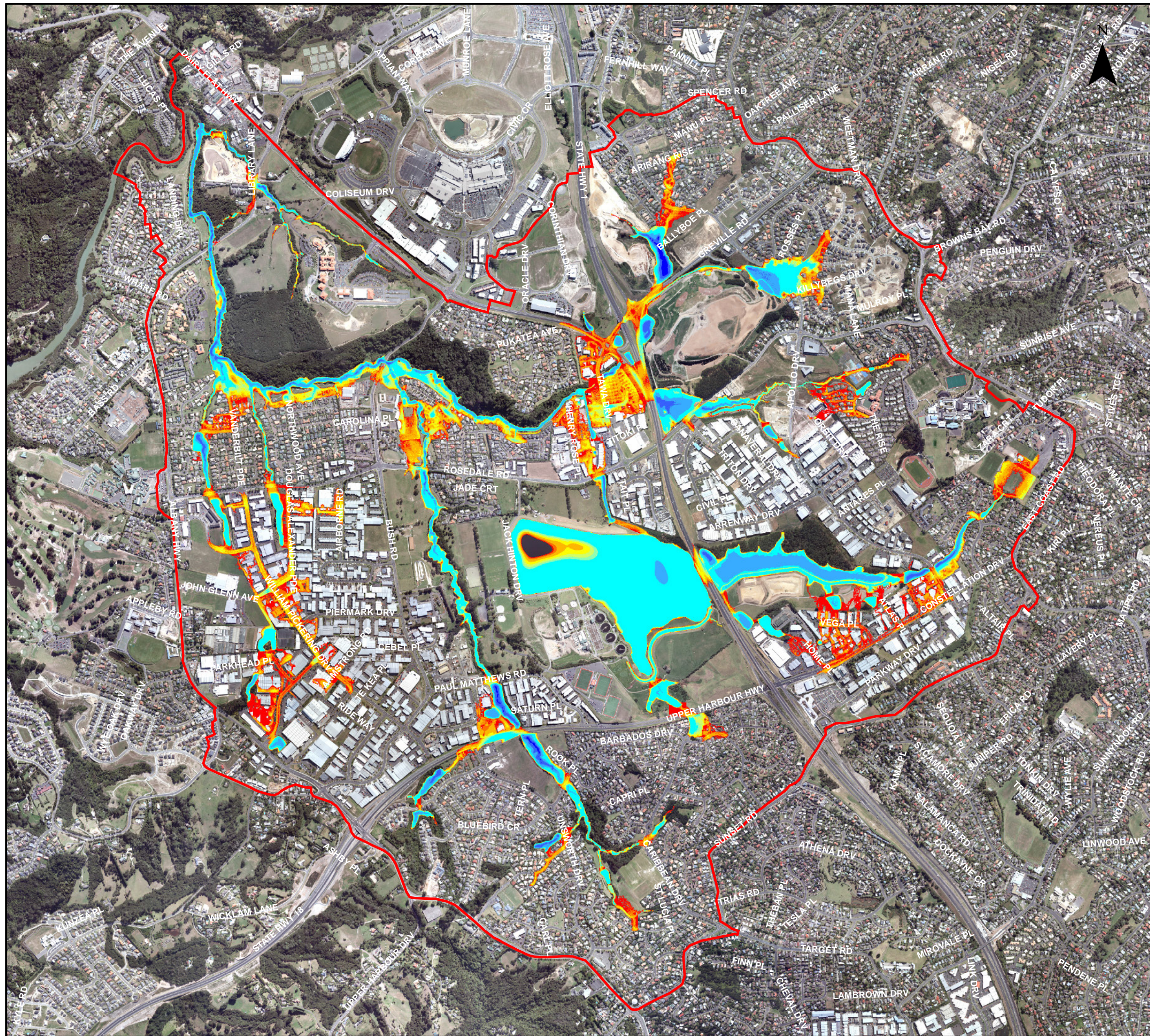


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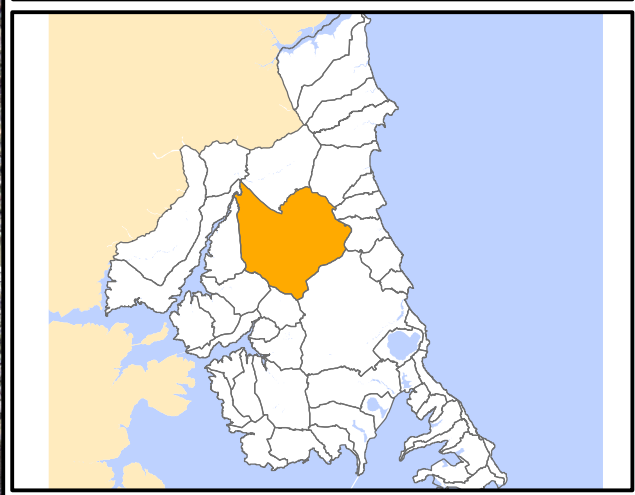
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Figure 3.2 Oteha Model SubCatchments



**LEGEND**

**2D Model Flood Depth**



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DRAWN: JI  
CHECKED: NB

0 120 240 480 720 960 1,200  
Meters

SCALE (A3): 1:18,000

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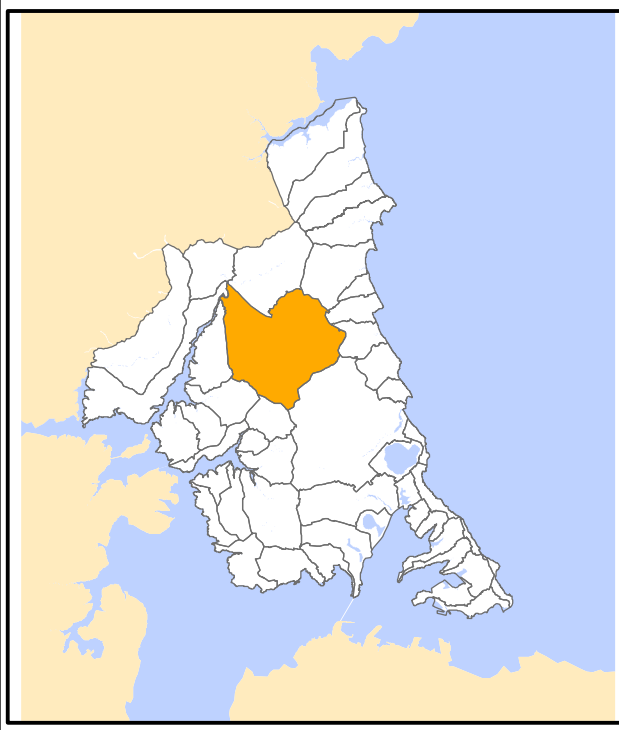
Figure 4.1 100 Year ARI MPD Flood Depth  
Based on MIKE 21 Model



**LEGEND**

**Floor Flooding Frequency**

- 10-YEAR ARI MPD
- 50-YEAR ARI MPD
- 100-YEAR ARI MPD
- > 100-YEAR ARI MPD
- MODELLED SW PIPE
- 100 YEAR ARI MPD FLOOD PLAIN



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CHECKED: NB

0 25 50 100 150 200 250  
Meters

SCALE (A3): 1:4,000

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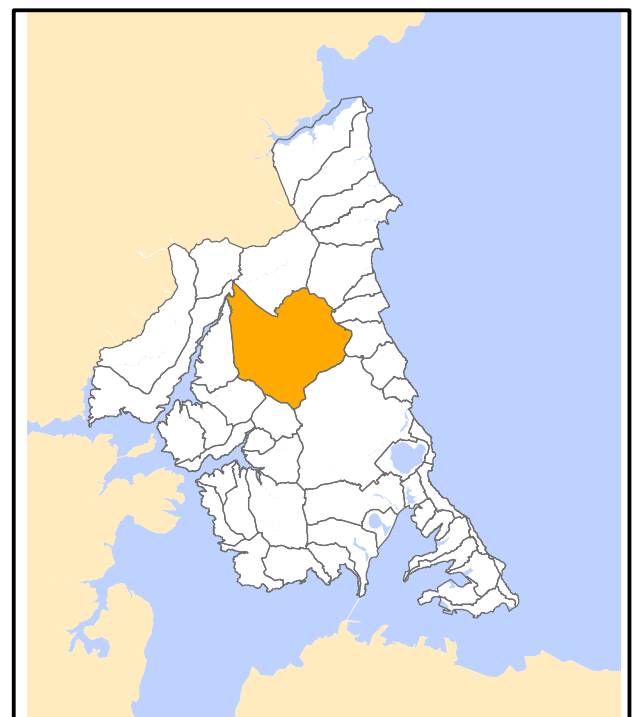
Figure 5.1 100 Year ARI MPD Flooded Buildings



**LEGEND**

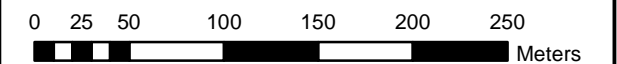
**Floor Flooding Frequency**

- 10-YEAR ARI MPD
- 50-YEAR ARI MPD
- 100-YEAR ARI MPD
- > 100-YEAR ARI MPD
- MODELLED SW PIPE
- 100 YEAR ARI MPD FLOOD PLAIN



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FINAL

DRAWN: JI  
CHECKED: NB



SCALE (A3): 1:4,000



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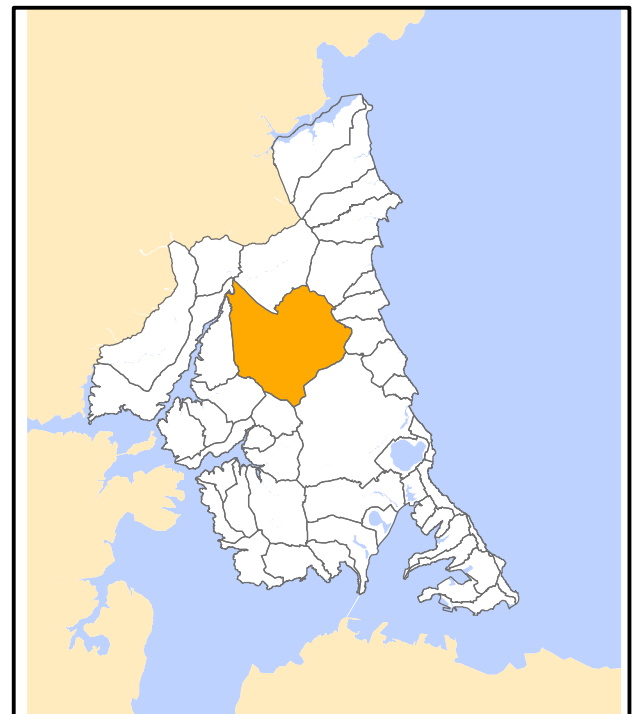
Figure 5.2 100 Year ARI MPD Flooded Buildings



**LEGEND**

**Floor Flooding Frequency**

- 10-YEAR ARI MPD
- 50-YEAR ARI MPD
- 100-YEAR ARI MPD
- > 100-YEAR ARI MPD
- MODELLED SW PIPE
- 100 YEAR ARI MPD FLOOD PLAIN



JULY 2013  
FINAL

DRAWN: JI  
CHECKED: NB

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Meters

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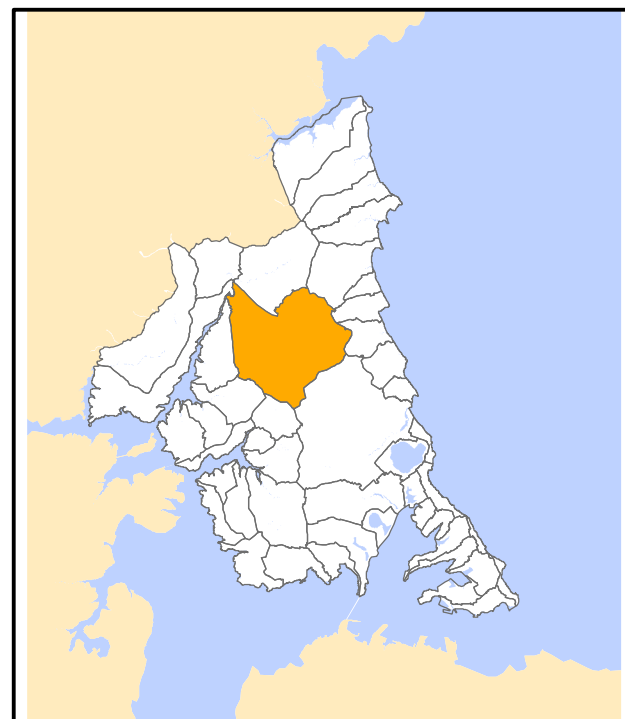
Figure 5.3 100 Year ARI MPD Flooded Buildings



**LEGEND**

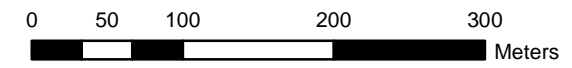
**Floor Flooding Frequency**

- 10-YEAR ARI MPD
- 50-YEAR ARI MPD
- 100-YEAR ARI MPD
- > 100-YEAR ARI MPD
- MODELLED SW PIPE
- 100 YEAR ARI MPD FLOOD PLAIN



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FINAL

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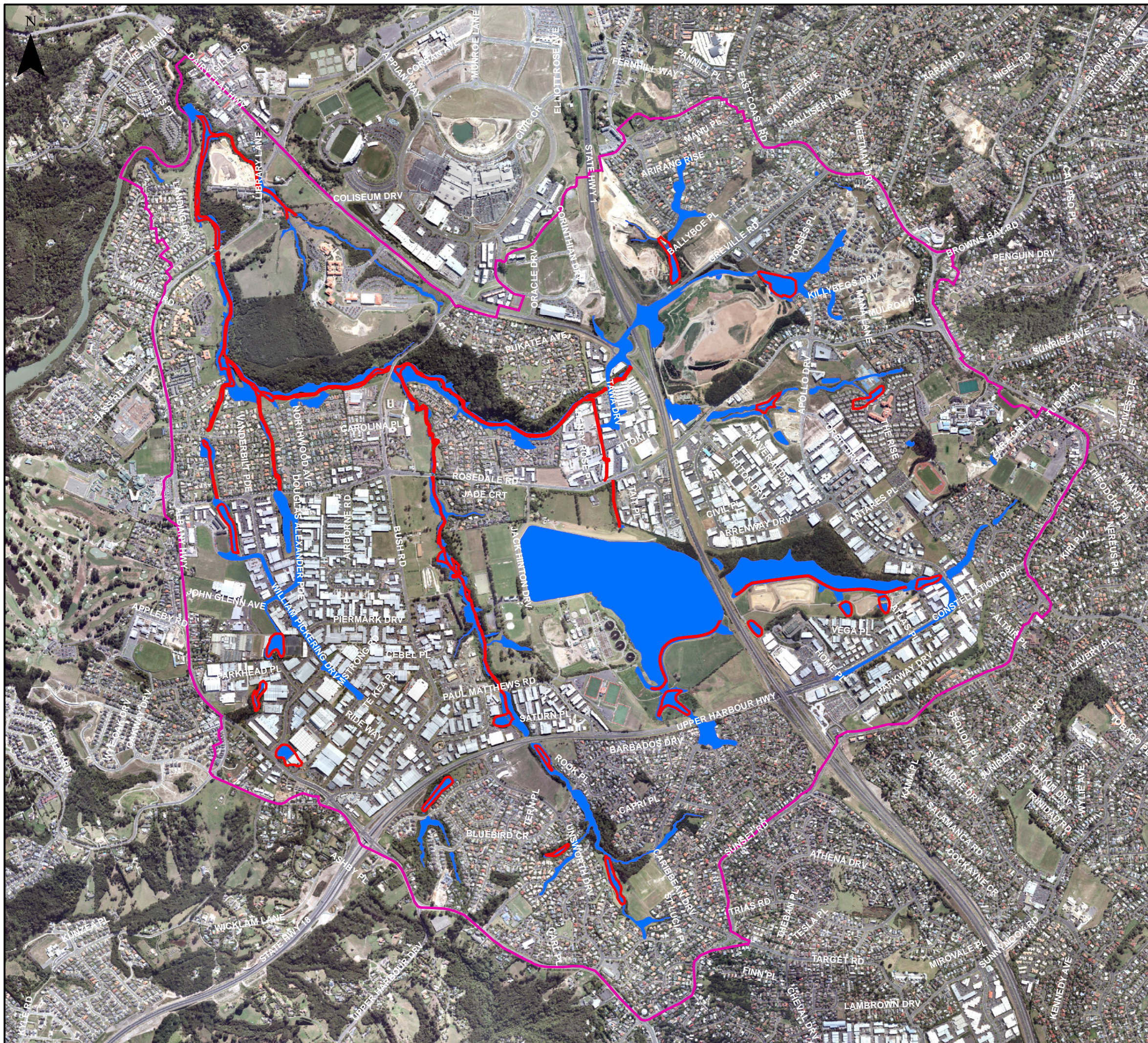


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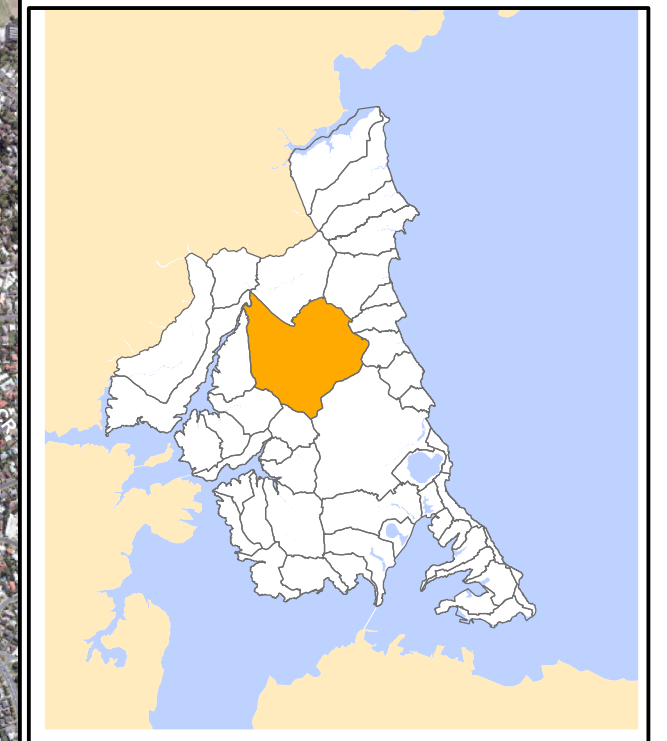
Figure 5.4 100 Year ARI MPD Flooded Buildings



**LEGEND**

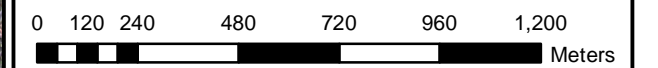
**100 YEAR ARI MPD FLOOD PLAIN**

- PRESENT FLOOD PLAIN
- PREVIOUS FLOOD PLAIN
- CATCHMENT BOUNDARY



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DRAWN: JI  
CHECKED: NB



SCALE (A3): 1:18,000



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Figure 5.5 Comparison of Present and Previous  
100 Year ARI MPD Flood Plain