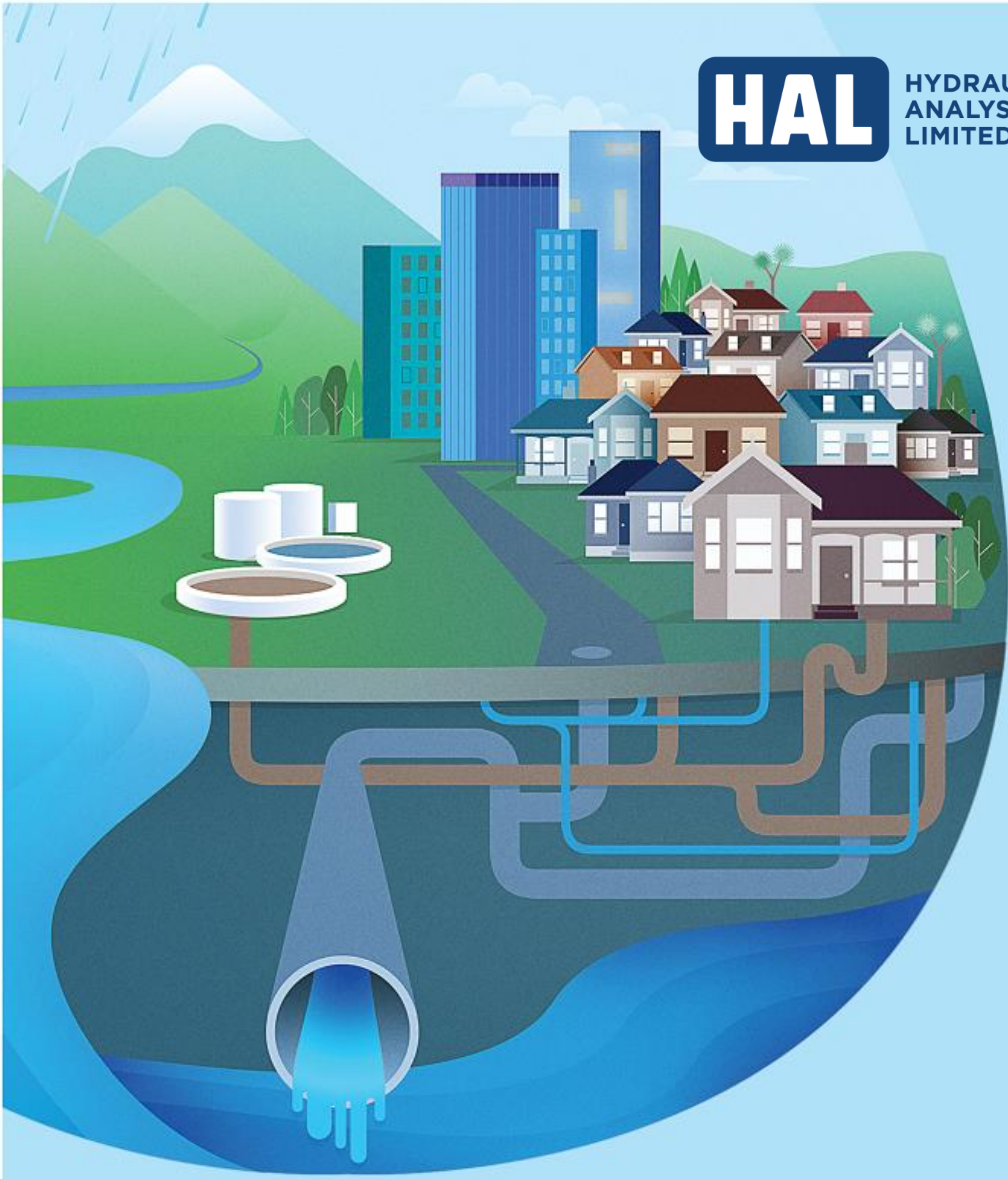

THAMES COROMANDEL DISTRICT COUNCIL

WHANGAMATA
MODEL BUILD AND SYSTEM PERFORMANCE

MAR 2021

Revision Schedule

Rev	Date	Description	Prepared by	Checked by	Reviewed by	Approved by
0	12/8/2020	Draft For Peer Review	RVL			
1	9/11/2020	Draft Including Peer Review Comments	RVL			
2	15/12/2020	Amended Appendix A	RVL			
3	22/3/2021	For Client Comment	RVL	RVL	MW	TL

Executive Summary

Study Objectives

The principal objective of this study is to assess the current level of service provided by the existing Whangamata catchment stormwater infrastructure, including the estimated frequency and extent of inundation within the catchment.

Catchment

The town of Whangamata has developed from a small gold mining and logging based settlement to a community consisting of permanent homes, holiday homes and camping grounds. Whangamata has increasingly become home to relatively few permanent residents, whilst over the summer months the population swells with absentee property owners and visitors holidaying.

The soil conditions of Whangamata vary from the flat sandy soils, which provides very good soakage, to clayey loam that has less soakage potential. The low lying flat main part of Whangamata township has limited drainage network installed and is susceptible to stormwater ponding / surface flooding.

Historically, the primary stormwater management approach has been via ground soakage. However, the increase of infill subdivision and construction of larger properties and infill development has increased hard stand areas (impermeable surfaces). This reduces the natural infiltration capacity and increases the stormwater runoff and the subsequent likelihood of ponding / flooding on private properties and road reserves.

Recent storm events have caused flooding and have raised concerns about the extent and capacity of the existing stormwater system, and the potential impacts of climate change need to be accounted for in TCDC's future planning.

Model Build

A detailed hydrological and hydraulic model of the Whangamata Catchment was developed in accordance with the Waikato Regional Council Stormwater Runoff Modelling Guideline (TR2018/02). InfoWorks ICM v12.0 (Dec 2020) software, has been used to develop the linked 1D-2D hydrological and hydraulic model, which integrates two-dimensional (2D) surface modelling with one-dimensional (1D) pipe and open channel flow.

The adopted hydrological method for generating and modelling the excess rainfall runoff is based on the SCS Unit Hydrograph Method[1] as per Waikato Regional Council (WRC) guidelines and applied as a combination of:

- Rain on Grid method for the developed lower lying catchments, where excess rainfall runoff (after deduction of initial abstraction and infiltration losses) is entered on the 2D surface and runoff routing is calculated within the hydraulic model component.
- Lumped catchment assessment for the Te Weiti and Waikiekie streams. For this method the catchment of the respective streams is identified including an assessment of the response time

[1] Soil Conservation Service Unit Hydrograph Method

(i.e. time of concentration). A runoff hydrograph is generated representing the runoff of the entire lumped catchment.

The 1D component of the hydraulic model comprises the piped network as derived from GIS data, survey data, design and as-built drawings, and site observations. Specific features are:

- Williamson Park Pond and Outlet
- Otahu Road Stormwater Pump Station
- Underground Storage and Soakage Systems
- Te Weiti and Waikiekie Culverts

The 2D surface is primarily based on LiDAR[1] survey data flown in 2013 in combination with 5m contour data in areas where no LiDAR data is available.

For the detailed hydrological model 24hr design rainfall data were obtained from NIWA[2] for various locations within the catchment, including allowances for the impact of climate change for future development scenarios (i.e. MPD[3]). Impervious areas have been assessed based on aerial photographs for the ED[4] scenario, and District Plan zoning limits for the MPD scenario.

A constant tailwater level has been assumed as downstream boundary condition for the model. The adopted level is based on the Mean High Water Spring level published by WRC[5]. Tailwater levels at outfalls along the Wentworth River have been adjusted following sensitivity analysis on the impact of elevated flood levels in that river. An allowance of 1.0m sea level rise has been added to all tailwater levels in the MPD scenario as per MfE[6] recommendations.

Limitations of the model are listed under Section 4.6.2. It is noted that the model has not been calibrated against existing storm events due to the lack of suitable data. As a result, the reported flood levels are estimates based on numerous uncertainties. As such these estimates should be treated as indicative for the purposes of determining flood levels, however the model can be utilised to assess the relative effects of potential option upgrades. Also note that modelling results represent computed flood inundation levels and exclude freeboard allowance.

Model validation includes the following validation / sensitivity runs:

- Te Weiti and Waikiekie Flow Validation
- Te Weiti and Waikiekie culvert flow validation
- Modelling catchpits
- Lowering Williamson Road Pond overflow level to 3.0mRL
- Storm duration
- Inconsistent GIS data near rugby field
- Elevated flood levels Wentworth River

[1] LiDAR (Light Detection And Ranging) - method for measuring ground surface levels
[2] NIWA HIRDS v4 – High Intensity Rainfall Design System – 2018
[3] MPD = Maximum Probable Development
[4] ED = Existing Development
[5] Waikato Regional Council Coastal Inundation Tool
[6] Coastal Hazards and Climate Change, Ministry for the Environment, Dec 2017

The model has been run for the scenarios and design storm events listed in Table 6-1. Flood maps have been prepared for the MPD scenario with ARI 10yr and 100yr 24-hour design storm event (refer Appendix C).

Findings

The findings from this study include:

- A hydrologic and hydraulic model has been developed of the Whangamata township and northern urban areas. This model has been used to complete a dynamic assessment of design rain storms for 2, 10 & 100yr ARIs for existing development (current climate conditions) and maximum probable development (including climate change allowances).
- The reported flows and levels are estimates based on numerous uncertainties which affect the confidence in this estimation such as soil infiltration rates, LiDAR data, rainfall, tide levels, dynamic blockages due debris and vegetation, localised obstructions, and so on. As such these estimates should be treated as indicative for the purposes of determining flood levels, however the model can be utilized to assess the relative effects of potential option upgrades.
- Validation activities for this model has found that:
  - Te Weiti and Waikiekie culverts are well represented in the model
  - Excluding individual catchpits from the model is acceptable
  - Lowering the Williamson Road Pond overflow level provides limited benefits
  - The flood maps in this report are based on simulation of the 24hr nested design storm event. For analysing flood mitigation options, 12hr simulation runs are acceptable.
  - The impact of elevated flood levels in the Wentworth River are small, but have been included in the model
- The Whangamata township is a flat low-lying catchment heavily relying on soakage infiltration for stormwater runoff. Public constructed soakholes are not included in the model (except for Otahu Road infiltration system and pump storage system) due to lack of information on these soakage systems. It is expected that there are more constructed public soakage systems, which could have a significant impact on modelled flood levels.
- The model predicts that flooding in Whangamata township under both existing and maximum probable development scenario is widespread over much of the township.
- Predicted ponding during heavy rainfall events is a normal occurrence and provides a fair volume of flood storage. However, it causes frequent nuisance flooding along many roads in the catchment especially in the areas lacking piped reticulation.
- Urban development and intensification increase rainfall runoff and reduces infiltration capacity which increases the risk of flooding.
- Reticulated drainage has limited application due to flat slopes and potential backwater affects particularly when sea level rise is considered.
- Properties at the northern end of the township (near the marina) with ground levels of approximately 1.5-2.0m above MSL are at risk of coastal inundation and particularly when sea level rise is included.

- Flood maps are presented in Appendix C for the 10yr and 100yr 24hr design storm event under MPD conditions. Presented levels are computed peak inundation levels and do not include freeboard to allow for:
    - physical processes that may not have been allowed for (like waves created by traffic)
    - uncertainties in the precision of the hydraulic modelling
    - uncertainties in the prediction of physical processes

Recommendations

The recommendations of this study are to:

- To improve the quality of the model and modelling results the following is recommended:
    - Identification and survey (if possible) of public soakage systems to better assess flood storage volume and soakage rates of these systems.
    - Survey of floor levels in critical areas to allow better estimates of current flood risk and quantification of flood mitigation benefits.
- Set minimum recommended building levels to ensure that new buildings and building extensions are constructed at a safe level to minimise risk of habitable floor flooding. It is recommended to apply a minimum freeboard to finished floor level of 300mm. A freeboard of 500mm could be considered along confined waterways and overland flow paths (i.e. non-flat catchment areas).
- Maximise ground infiltration by:
    - installing swales along the roads with designed infiltration trenches including prevention of siltation.
    - Requesting new developments to include soakage systems suitable to discharge runoff from a minimum 24hr 10yr ARI design storm including climate change allowance. Such system must include a well-designed filter systems to prevent siltation and blockage.
    - Implement a soakage maintenance plan for all private and public soakage systems.
- Maintain a record of all soakage systems including a maintenance database.
- Investigate and model stormwater upgrade options to reduce flooding.

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1 INTRODUCTION

1.1 Study Objectives

The principal objective of this study is to assess the current level of service provided by the existing Whangamata catchment stormwater infrastructure, including the estimated frequency and extent of inundation within the catchment.

1.2 Study Activities and Scope

The activities and scope of Model Build and System Performance stage of the study includes (refer respective sections of this report):

The activities and scope of Model Build and System Performance stage of the study includes (refer respective sections of this report):

- Section 2 - Catchment Description:
  An overview of the catchment, its extent and its main characteristics like topography, soils, district plan zoning limits, and key stormwater infrastructure features and flooding issues.
- Section 3 - Rapid Flood Hazard Assessment:
  A Rapid Flood Hazard Assessment (RFHA) has been undertaken to provide an initial assessment of the floodplain and its flood prone areas.
- Section 4 - Model Build:
  This section comprises the following key tasks of the Model Build process:
    - Review of existing data
    - Hydrological model
    - Hydraulic model
    - Boundary conditions
    - Modelling limitations and assumptions
- Section 5 - Model Validation
  Due to the lack of calibration data, a model validation was done for the flows and performance of the Te Weiti and Waikiekie streams including culvert performance. It also includes a range of model sensitivity tests.
- Section 6 - System Performance Assessment
  The performance of the system is presented on Flood Inundation Maps.
- Section 7 - Findings and Recommendations
  The report concludes with a summary of the key project findings and recommendations.

1.3 Previous Reports

The following reports are relevant to this study:

- Whangamata Stormwater Catchment Management Study – Updated Issues and Options Report, Draft – Version 2, Opus, Sep 2005.
- Whangamata Stormwater Model Build – Data Anomalies Report, Water Engineering Consultants, Aug 2006
- Williamson Road Stormwater Assessment, HAL Memorandum, 9 May 2018
- Whangamata Stormwater Master Plan – Proposal, HAL & Morphum Environmental, Nov 2018
- Whangamata Stormwater Master Plan – Strategic Context and Risks, Morphum Dec 2019

1.4 Projection and Vertical Datum

All data in the model and this report are in terms of:

- New Zealand Transverse Mercator 2000 (NZTM2000) horizontal projection, and
- Auckland 1946 (AKL1946) vertical datum.

2 CATCHMENT DESCRIPTION

2.1 Location

The town of Whangamata has developed from a small gold mining and logging based settlement to a community consisting of permanent homes, holiday homes and camping grounds. Whangamata has increasingly become home to a number of permanent residents, whilst over the summer months the population swells with absentee property owners and visitors holidaying.

The town is bordered by the Otahu River to the south, the Te Weiti Stream to the north, and the Whangamata Harbour and the sea to the east (Refer Figure 2-3 below). The urbanised area comprises:

- The main township on the flat grounds between the Whangamata Harbour and the Otahu River.
- The more undulated urban area north of the Moana Anu Anu Estuary,

The modelled main catchment areas are (from north to south):

- Te Weiti Catchment (215 ha)
- Waikiekie Catchment (664 ha)
- Township Catchment (440 ha)

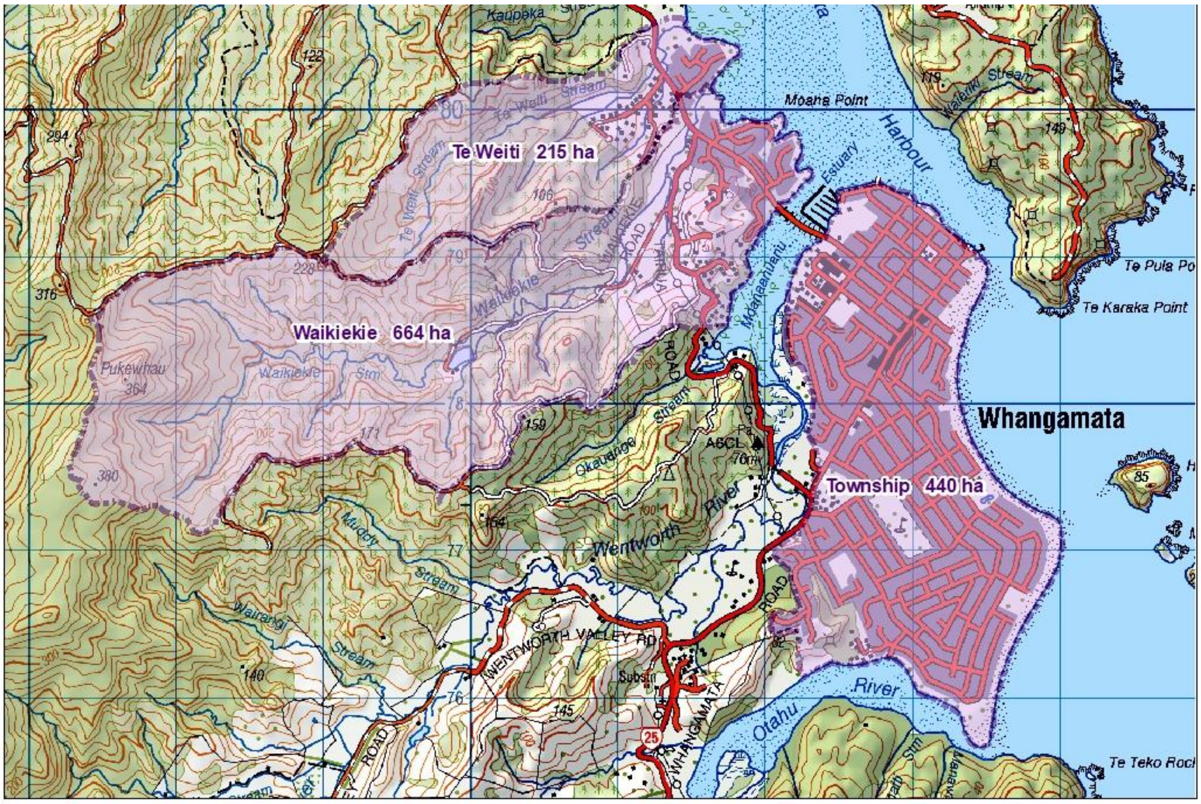

Figure 2-1 Whangamata Modelled Catchments

The Okauange Stream and Wentworth River catchments discharge into the Moana Anu Anu Estuary northwest of the town centre. These catchments and associated flood risk are excluded from the scope of this study. It is also noted that Waikato Regional Council does not have flood levels of this river that could be used as downstream boundary condition for discharges from the Whangamata Township catchment. However, sensitivity analysis is included in this study to estimate the effect of elevated flood levels in Wentworth River.

The soil conditions of Whangamata vary from the flat sandy soils, which provides very good soakage, to clayey loam that has less soakage potential. The low lying flat main part of Whangamata township has limited drainage network installed and is susceptible to stormwater ponding / surface flooding.

Historically, the primary stormwater management approach has been via ground soakage. However, the increase of infill subdivision and construction of larger properties and infill development has increased hard stand areas (impermeable surfaces). This reduces the natural infiltration capacity and increases the stormwater runoff and the subsequent likelihood of ponding / flooding on private properties and road reserves.

Recent storm events have caused flooding and have raised concerns about the extent and capacity of the existing stormwater system, and the potential impacts of climate change need to be accounted for in TCDC's future planning.

2.2 Topography

The majority of the Whangamata township is located on flat alluvial sand with small sand dunes along the coastline to the east and steep hills to the west. A number of streams/rivers flow from the hills eastwards to the sea.

The total catchment area of the modelled catchments is approximately 1320 ha. The urban development is primarily on the main flat land and ground levels closer to the coast. Ground levels generally vary here between 4 and 6 m above MSL, except for the northern end of the peninsula with property ground levels as low as 1.5 m above MSL. This area is shown in the forefront of Figure 2-2 below. North of the Moana Anu Anu Estuary the topography is more elevated. The hills to the west of the township are typically under forestry and rural land-uses.

The DEM (Digital Elevation Model) used for the hydraulic model is based on LiDAR flown in 2013.

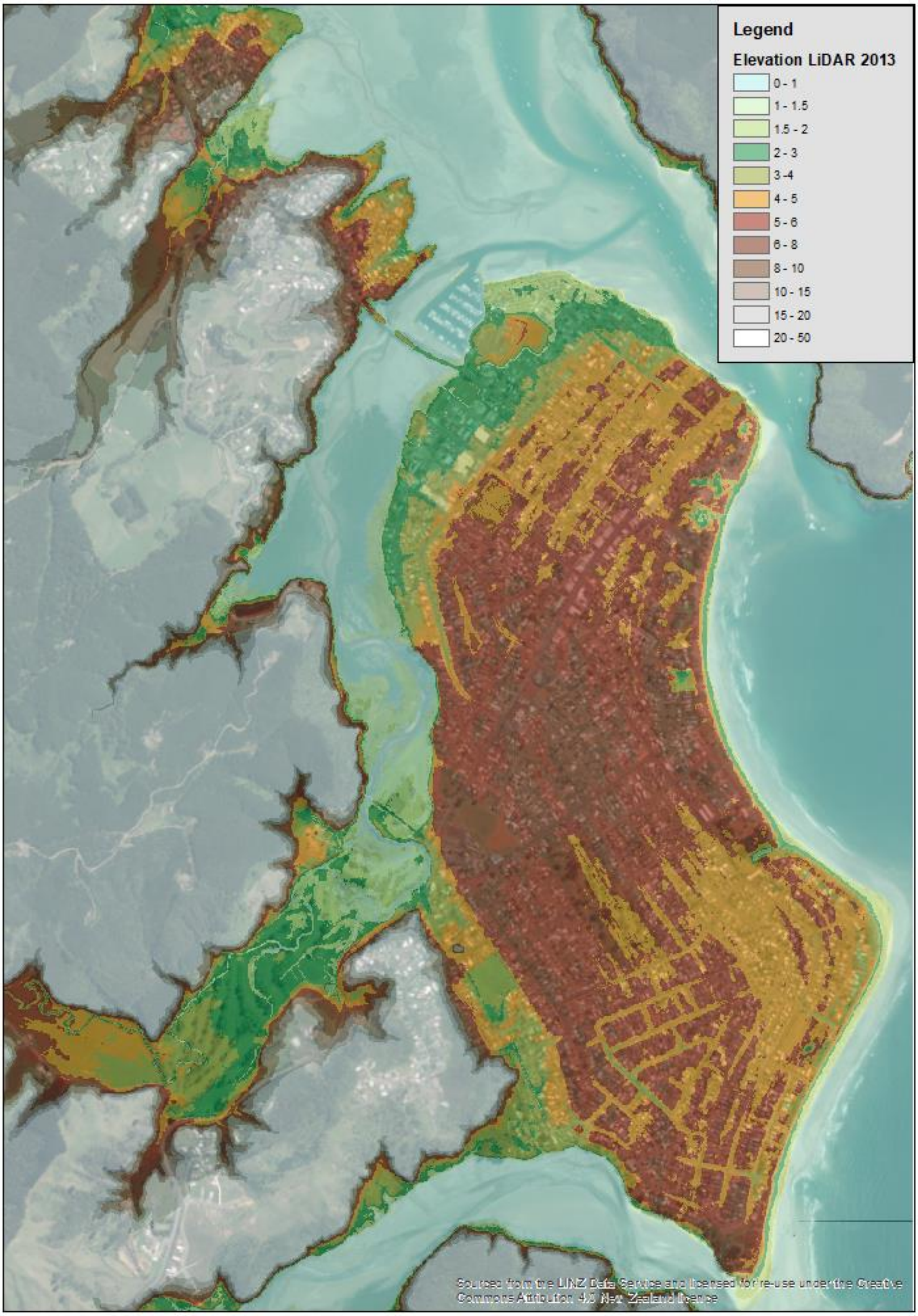

Figure 2-2 DEM levels from LiDAR in Whangamata township

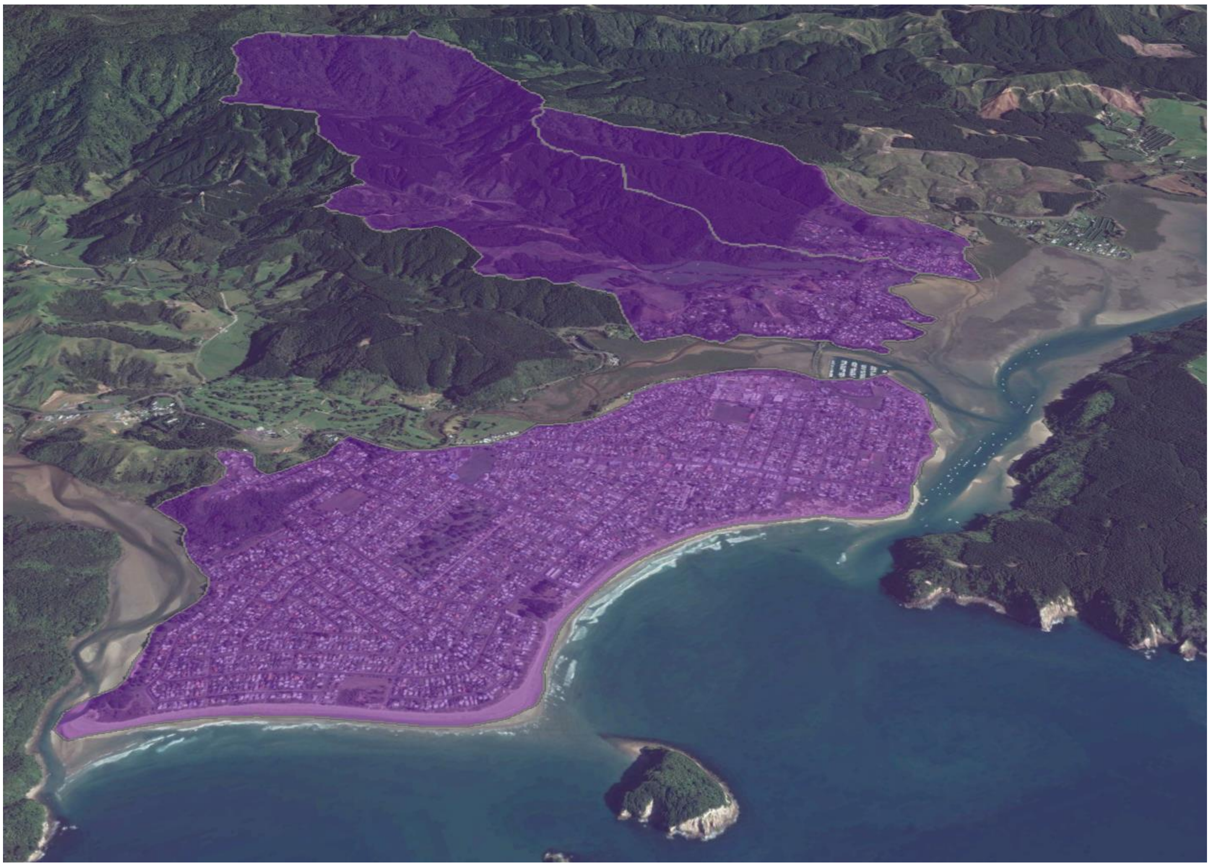

Figure 2-3 Whangamata township and hillside catchments

2.3 Geology and Soils

Soil maps have been obtained from Landcare Research soil maps and used to classify the infiltration capacity of the soils using the Hydrological Soil Group is specified in the Waikato Stormwater Runoff Modelling Guidelines (Refer WRC 2018). The allocation of the various soil groups as defined in these soil maps are shown in Figure 2-4 below. The Whangamata urban catchment comprises primarily of sandy or sandy loam soils (Soil Group A). Lower infiltrating soils are typically found in the valleys and along watercourses such as Wentworth River (mainly Soil Group B) and Waikiekie Stream (Soil Group C/D), which consist of clayey loam and peaty loam.

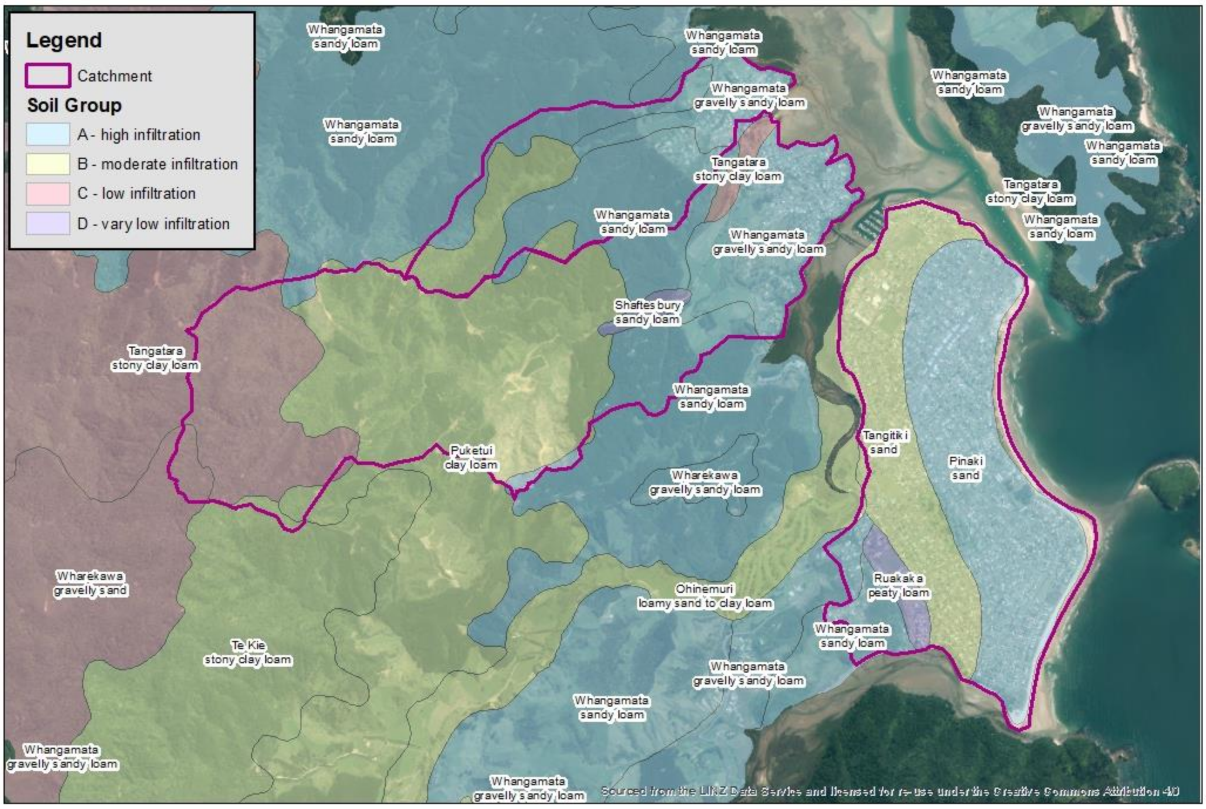

Figure 2-4 Model domain Hydrologic Soil Groups

The infiltration characteristics and impact on excess stormwater runoff for each of the soil groups is represented in the rainfall timeseries. The infiltration losses have been calculated and the net excess runoff is modelled using the rain on grid method (refer Section 3.2 and 4.3).

2.4 District Plan Zoning

The current land uses within the Whangamata catchment are shown in Figure 2-5 according to the TCDC District Plan online zone GIS maps.

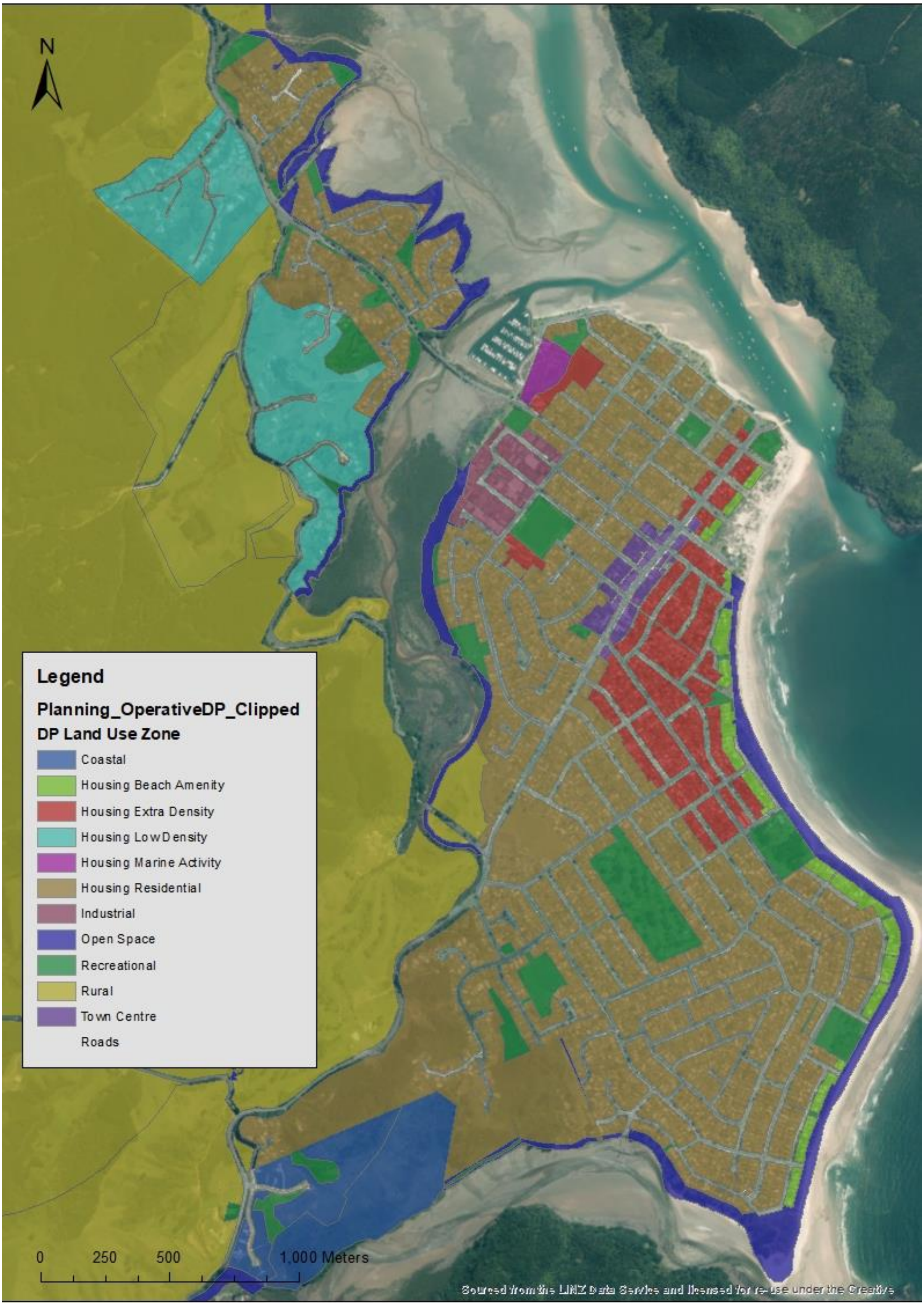

Figure 2-5 Model Domain District Plan Zones (Source: TCDC)

Most of the urban area is zoned residential, with extra density residential areas near the marina and between the town centre and the beach. South of the marina is an industrial area. The northern urban area is predominantly residential with low density housing further up the hills.

Detailed data on adopted percentage impervious areas are presented in Section 4.2.6.

2.5 Stormwater Drainage System

Stormwater drainage in the main township area comprises a mixture of gravity piped stormwater drains and natural and artificial soakage (e.g. constructed soakholes).

The piped stormwater network is presented in Figure B1 & B2 in Appendix C. The main piped network outlets (i.e. outlets 600mm diameter and larger) are summarised in Table 3-1:

Table 2-1 Main stormwater outlets (600mm and up)

Location	Outlet Size	Receiving Environment
Hetherington Road between the main bridge and the marina.	675 mm dia	Moana Anu Anu Estuary
Various outlets south of Hetherington Road bridge, including:		Moana Anu Anu Estuary and Wentworth River
• Casement Road	Up to 600 mm dia discharging into open channel	
• Lindsay Road	600 mm dia	
• Wattle Place	600 mm dia	
• Sharyn Place	675 mm dia	
• Mayfair Ave	750 mm dia	
Achilles Avenue	825 mm dia	Wentworth River
Kotuku Street	900 & 1000 mm dia	Otahu River
Williamson Park Pond	900 & 1050 mm dia	Coast
Beach Road	1050 mm dia	Whangamata Harbour

No detailed information has been provided on soakage systems and crude assumptions have been made in terms of available soakage infiltration capacity (Refer Section 4.2.6).

An open concrete lined v-shaped drain runs through Park Avenue Reserve (about 1km southwest of the town centre) and continues along McKellar Place Walkway to the south. It discharges the local runoff and runoff from the hills further to the west into the Otahu River (including some culverts / piped sections). The Park Avenue Reserve provides for some but limited flood storage, in the order of 0.5-1m depth (refer Figure 2-6).

Figure 2-6 Concrete lined open drain at Park Avenue Reserve

A pump station is located at the eastern end of Otahu Road, which includes an artificial underground storage area. The pump station discharges into Otahu River and has a high-level overflow pipe discharging onto the beach.

A stormwater pond is located at the northeastern end of Williamson Road. In 2019, a large stormwater upgrade has been installed comprising duplication of the main pipe section between Williamson Road / Ocean Road intersection and the pond (refer Figure 2-7 below). Runoff discharged into the pond is stored and slowly infiltrates into the ground. A weir overflow comprising gabion baskets and a concrete nib (refer Figure *2-8* below) allows for runoff to discharge onto the beach during times of high stormwater runoff and elevated pond levels.

Figure 2-7 Recent duplication of SW outlet into Williamson Park pond

Figure 2-8 Gabion basket and concrete nib overflow from Williamson Park pond onto beach

There are a couple of stormwater drains that discharge into beach dune depression areas like Island View Road and Hunt Road. No details have been provided on the design principles of these systems and if they include artificial underground soakage systems. It is expected that these systems rely on natural soakage into the well-draining beach sands.

North of the Moana Anu Anu Estuary two large culverts allow runoff of the Te Weiti and Waikiekie Stream to cross the State Highway No. 25 (refer Figure 2-9 & Figure 2-10 below). South of Herbert Drive is a series of small retention ponds installed as part of Moana Park development. These ponds discharge directly upstream of the Te Weiti SH25 culvert.

Figure 2-9 SH25 Culvert at Te Weiti Stream

Figure 2-10 SH25 Culvert at Waikiekie Stream

2.6 Reported Flooding Issues

Flood incidents reported by residents have been obtained from TCDC's flood incident database, which contains incidents from 2009 to 2019. The location of reported incidents are presented in Figure 2-11 below. The figure shows:

- Reported Flood Incidents, which shows the location of incidents related to observed flooding of roads, properties, and buildings.
- Reported Maintenance Issues, which shows the location of maintenance issues related to flooding and drainage
- Flood Incident Heatmap, which shows the areas with higher or lower volume of reported flood incidents. It is noted that the heatmap is based on the number of flood incidents only and excludes maintenance issues.

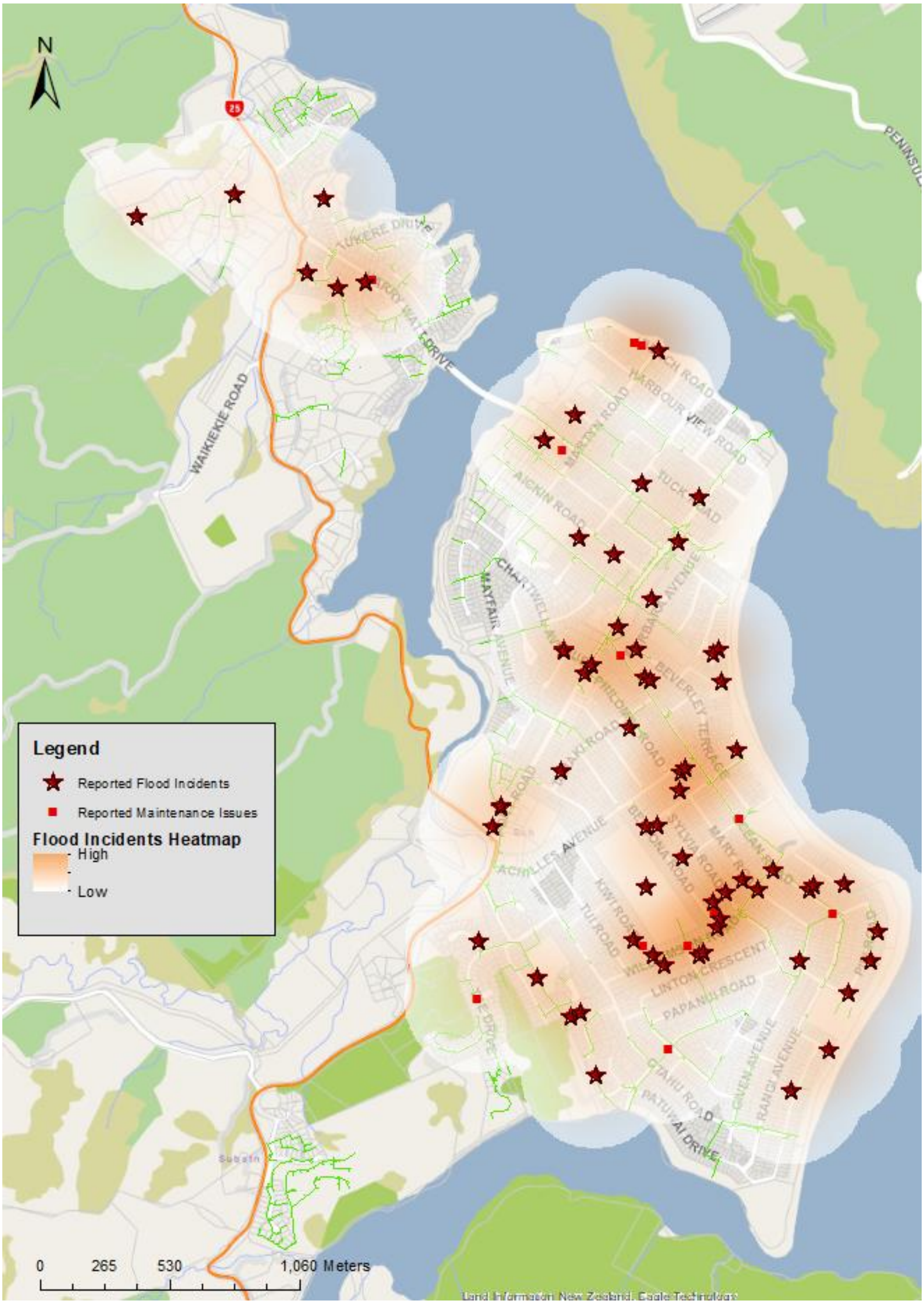

Figure 2-11 Location of flood incidents

3 RAPID FLOOD HAZARD ASSESSMENT

A Rapid Flood Hazard Assessment (RFHA) has been undertaken to provide an initial assessment of the floodplain. The assessment is based on the methodology specified in the Rapid Flood Hazard Assessments Modelling Specification, Auckland Council, Aug 2012 (AC 2012). The assessment assumes no pipe network is available, which allows for a rapid assessment of the flood extend to be undertaken. A digital terrain model is developed from LiDAR and contour data of the entire catchment area and then the rain on grid approach is used to produce the 100-year RFHA result.

3.1 Digital Elevation Model for RFHA

The following topographical data sets are available:

- 2006 LiDAR, which covers the township and some of the hills to the east.
- 2013 LiDAR, which has a larger coverage of the catchment, but exclude some of the higher areas within the catchment.
- 5m contour data that covers the entire area.

The 2013 LiDAR is the most accurate and up to date data set available and therefore preferred to be used for the model. However, it lacks data at elevated areas (typically above RL 30 to 40m) which are within the area of interest for assessing flood risk.

It is also noted that at the boundary of the 2013 LiDAR there are significant variances in elevation (in the order of meters) between the three data sets. This creates issues along the boundaries when data sets are merged. To minimise inaccuracies and resolve ground profile consistency (i.e. preventing artificial ponding areas), the following has been adopted:

- Limit to two datasets being:
    - the most accurate and up to date 2013 LiDAR
    - and the 5m contour data for the remaining missing areas and the higher elevations
- Smoothen the 5m contour data along the boundary of the two datasets.

The RFHA assumes the pipe network is fully blocked and is therefore not included in the assessment. However, the DEM has been cut for the RFHA at two locations to represent these large two culverts under SH25 crossing the Te Weiti and the Waikiekie streams. It is considered unlikely that these culverts will become blocked.

Local depressions and storage areas have been filled in to provide a conservative assessment of the flood risks.

No specific buildings have been identified that could significantly obstruct the flows.

A triangular mesh is used for the modelling. Three different meshing zones have been created with varying mesh cell sizes. The large rural areas are represented in a relatively large mesh, whilst the urban areas and the rural streams have a much smaller mesh size. This allows for more detailed (and more accurate) model results in the areas of interest, whilst not excessively increasing the computation times.

The adopted mesh parameters are presented in Table 3-1 below.

Table 3-1 Mesh Parameters

Mesh Zone	Min Element Area	Max Triangle Area	Roughness
Rural Catchments	50 m^2	200 m^2	0.1
Streams	2 m^2	10 m^2	0.1
Urban Catchments	2 m^2	5 m^2	0.1

3.2 Rainfall Data

The RFHA is based on a rain on grid rainfall approach, where infiltration losses are calculated depending on soil type and impervious footprint. The resulting excess runoff is then used as boundary condition and connected to the 2D surface creating surface runoff.

The rainfall data in the catchment has been derived from HIRDSv4 (Ref NIWA 2018). The variance in the rainfall over the catchment is shown in Figure 3-1 below. This figure shows the 24 hour 100yr ARI (Average Recurrence Interval) rainfall depths for 8 locations in the catchment. The approximate rainfall depths are:

- 320mm in the township,
- 350mm in the northern urban area and the lower rural areas, and
- 425mm in the upper rural areas

For the RFHA the rainfall over the catchment has been assumed uniformly based on the methodology specified in the RFHA Modelling Specification (AC 2012). This methodology is a conservative assumption as shown in the below calculation:

$$Rain = Rain_{min} + (Rain_{max} - Rain_{min}) * 0.8$$

Where:

$Rain_{min}$ = 320 mm

$Rain_{max}$ = 425 mm

Rain = 320 + (425 – 320) * 0.8 = 404 mm

This is for the existing climate conditions. For the RFHA a climate change allowance in accordance with Ministry for the Environment guidelines (MfE 2008) assuming 2.1°C temperature rise by 2090 has been included, which results in the following 24h design rainfall depth:

Rain_24h_2090: 472 mm

The rainfall temporal pattern is as per TP108 (ARC 1999) and TR2018/02 (WRC, 2018). Note that this rainfall pattern was superseded by subsequent model build as outlined in Section 4.2.4 below, adopting instead a temporal pattern derived from HIRDSv4.

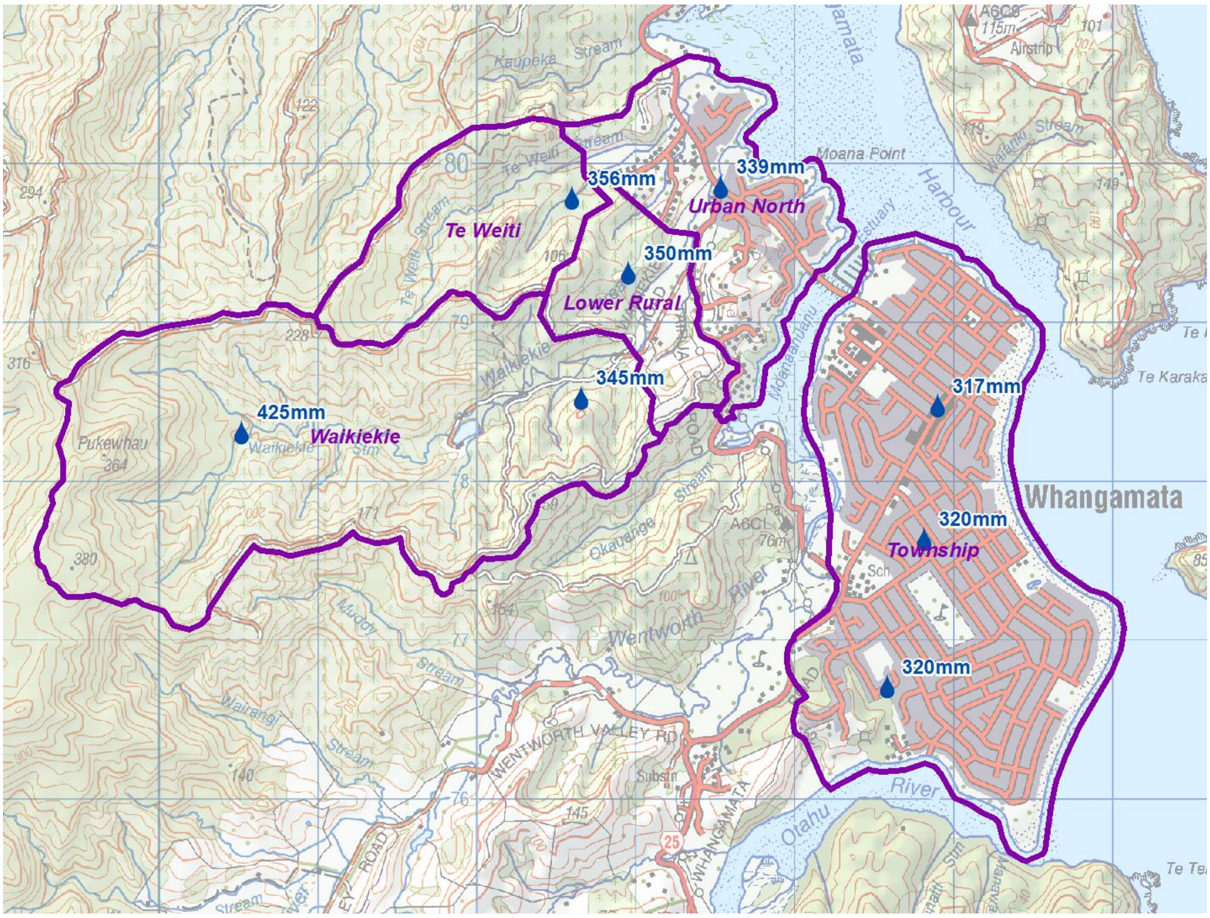

Figure 3-1 Rainfall Profile Areas

3.3 Rainfall Runoff Modelling

For the RFHA the hydrological model runoff is calculated using the SCS[1] Runoff Curve Number (CN) method as is specified in TR2018/02[2]. This method is also known as the SCS Unit Hydrograph Method (SCS-UHM) and is specified in TP-108 (ARC-1999). The curve number represent the infiltration characteristics of the catchment depending on:

- Cover Type, which describes the land usage, like open space, bush, or impervious area.
- Hydrologic Condition, which is the condition of the vegetation (i.e. poor, fair, or good).
- Hydrological Soil Group (HSG), which depends on the soil type and respective infiltration rate. There are four HSG's defined labelled Type A to D.

CN values have been derived from Table 5-2 in TR2018/02. Relevant values for the Whangamata catchment are shown in Table 3-2 below for the various cover or land use types in the catchment.

[1] Refer NRCS 1986 – Urban Hydrology for Small Watersheds, TR55
[2] TR2018/02 – Waikato stormwater runoff modelling guidelines, 2018

Table 3-2 Curve Numbers adopted for surfaces

Cover Type	Cover Description	HSG - Hydrologic Soil Group			
		A	B	C	D
Urban Pervious Areas	Open Space - Fair Hydrologic Condition (grass cover 50% - 75%)	49	69	79	84
Rural Pervious Areas	Bush – Good Hydrologic Condition	30	55	70	77
Impervious Areas		98	98	98	98

Note: HSG – Hydrologic Soil Group

For the RFHA, timeseries of excess rainfall runoff have been generated using HEC-HMS software. The catchment has been split in 2 separate areas, the area northwest of the Moana Anu Anu Estuary and the township, to allow for the large difference in impervious footprint. The assumed hydrological parameters to calculate the infiltration losses are presented in the below Table 3-3. It is noted that the TR2018/02 method uses a formula for the Initial Abstraction that has been modified from the equation in the original SCS version.

Table 3-3 Hydrological Parameters RFHA Catchments

Catchment	Predominant Soil Type	Cover	CN Pervious Area	% Imp Area	Weighted CN	Initial Abstraction
Urban Catchments	A	Open Space / Fair Condition	49	70%	83	2.6 mm
Rural Catchments	B	Bush / Good Condition	55	0%	55	10.4 mm

The RFHA method specifies to use a uniform percentage impervious area across the catchment of 70%. This is a conservative percentage and typically suitable for urbanised catchments. The catchment northwest of Moana Anu Anu Estuary has primarily bush and rural land use and the developed area is about 50% low density and 50% normal housing density. It would therefore not be realistic to model the runoff based on 70% impervious footprint. Also note that the adopted rainfall of 404 mm (excl CC) is conservative. For simplicity reasons a 0% impervious footprint has been assumed for rural catchments. A 70% impervious footprint has been assumed for the township catchment (refer Table 3-3 above).

ICM software is used to model the runoff based on the rain-on-grid method, where the generated excess runoff is entered onto the triangular mesh elements representing the topography of the catchment.

4 MODEL BUILD

Following the RFHA to quickly assess the likely flood prone or sensitive areas, a more detailed hydrological and hydraulic model of the Whangamata Catchment was developed in accordance with the Waikato Regional Council Stormwater Runoff Modelling Guideline (TR2018/02). This involved refinement of the hydrology and the topographical surface and including the 1D piped network and structures. Details of this modelling process are outlined below.

4.1 Modelling Software

InfoWorks ICM v12.0 (Dec 2020) software, developed by Innovyze, has been used to develop the linked 1D-2D hydrological and hydraulic model of the Whangamata catchment. ICM is software that integrates two-dimensional (2D) surface modelling with one-dimensional pipe and open channel flow.

Hydrological modelling was also supplemented by HEC-HMS v4.3 software developed by the U.S. Army Corps of Engineers.

4.2 Review of Existing Data

4.2.1 Topographical Data

The DEM for the hydraulic model is based on the same topographical data as used for the RFHA (refer Section 3.1 above), which is a merge of the following data sets:

- 2013 LiDAR, which has a larger coverage of the catchment, but is limited to levels below approximately 40 to 50m.
- 5m contour data that covers the entire area.
- The extent of the 2D surface has significantly been reduced as the upper catchments are now represented as lumped catchments instead of rain-on-grid catchments. Other modifications to the DEM were required to ensure a mathematically stable connection between the piped network and the 2D surface. Modifications to the DEM are described in Section 4.4.2 further below.

4.2.2 LiDAR data versus GIS data

A comparison has been made between the 2013 LiDAR data and manhole lid levels presented in TCDC GIS. The 2013 LiDAR ground levels of 283 manholes have been compared with their Lid Level as specified in the TCDC GIS asset data system. It is noted that the LiDAR data is presented in Auckland Vertical Datum 1946, while no reference is provided in the GIS data to what vertical datum the levels are referenced to.

Figure 4-1 below shows both the LiDAR and GIS levels for the respective manholes. Note that only the levels below 20mRL are shown. The figure shows that there seem to be a structural variance between the two data sets with a median value of +0.84m. Figure 4-2 shows an histogram of the same data set, which shows that 87% of the manholes have a LiDAR level more than 0.5m above the GIS level.

As a result, the GIS ground level data is not considered suitable for modelling purposes and the modelled ground levels have been based on the 2013 LiDAR data (i.e. AVD-46). Manhole invert levels are primarily based on the existing 1D model. A review of the long section profiles showed that those levels were generally providing consistent gradients and looked suitable for modelling purposes.

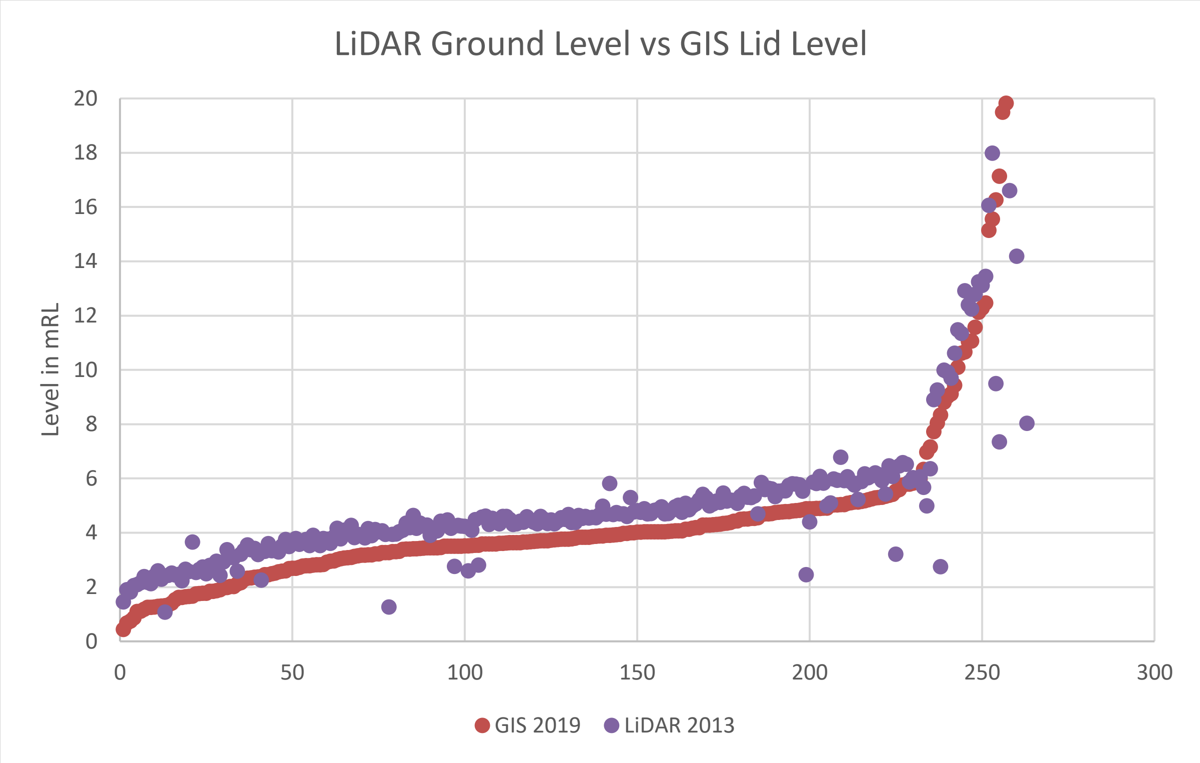

Figure 4-1 LiDAR Ground Level versus GIS Lid Level

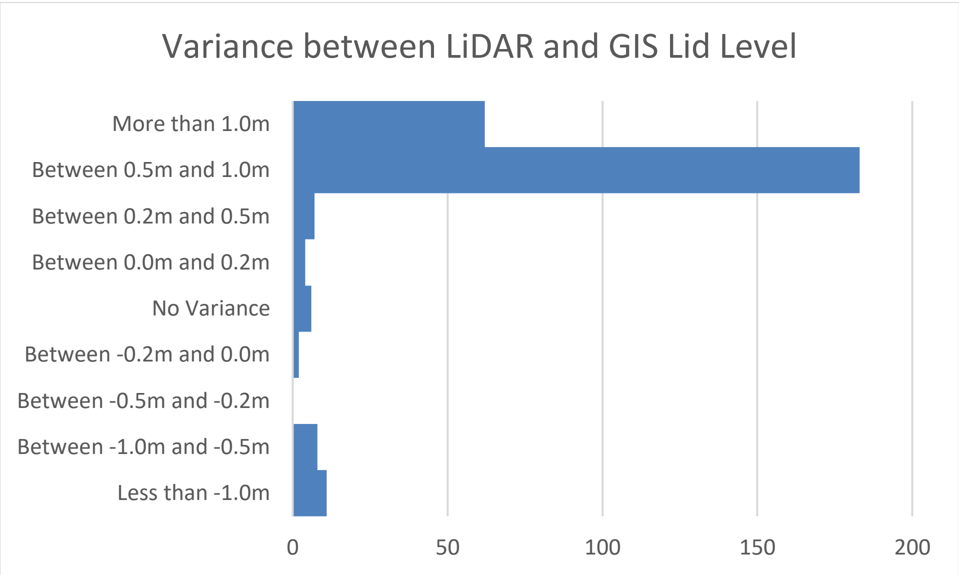

Figure 4-2 Variance between LiDAR and GIS Lid Level

4.2.3 Asset Data

The bulk of model asset data was sourced from:

- the existing 1D XP-SWMM model developed by WEC in 2006
- combined with TCDC's GIS database providing data for areas excluded from the 1D model (e.g. north of Moana Anu Anu Estuary) and new drainage networks
- asset data surveys (i.e. in 2007 and July 2011)
- design and as-built plans of recent upgrades and developments (e.g. Williamson Park, Otahu Road pump station, and Moana Park)
- observations and approximate measurements undertaken during a site visit in July 2019 (e.g. drainage network around Williamson Park and Te Weiti and Waikiekie culverts crossing SH25 north of Moana Anu Anu Estuary).

Most of the data was obtained from the existing 1D XP-SWMM model. TCDC's GIS data was lacking detail (i.e. many levels were missing) and lacking confirmation of vertical datum references (Refer Section · above). WEC had gone through a thorough process of asset data review and analysis during their model build process. Assumptions had been made and the model was checked to ensure continuity of the network and gravity drainage. The modelled network was therefore considered to be of better quality than TCDC's GIS Asset Data database. No additional asset survey pick-up was considered required.

For locations with missing asset data the following assumptions were typically applied:

- Apart from surveyed structures, all manhole lid levels were estimated from LiDAR to correspond with the modelled 2D ground surface.
- All pipes <100mm diameter and subsoil drains were excluded from the model.
- Missing invert data was assumed 0.7m below the GIS lid level, or LiDAR level if lid level is also missing.
- Missing outlet invert levels were estimated from LiDAR ground levels.
- All manhole diameters were estimated using ICM-software default assumptions, which is based on the size and number of connecting conduits.
- Inverts for manholes with negative depths were overwritten by GIS measured depth below LiDAR ground surface, or 0.7m depth where no GIS depth data exists.

All data sources and assumptions are flagged in the model.

4.2.4 Rainfall

For the detailed hydrological model 24hr design rainfall depths were obtained from HIRDSv4 (NIWA-2018) for various locations within the catchment. Based on the data the following rainfall zones were identified (refer Figure 3-1 above) and the respective 24hr rainfall depth for various probability events are presented in Table 4-1 below.

An allowance for the impact of climate change on rainfall intensities have been included in the model in accordance with TR2018/02. The allowance is based on MfE 2008 guidelines, which adopts an increase up to 16.8% assuming 2.1ºC average temperature rise. It is noted that MfE published updated Climate Change Projections for New Zealand (MfE 2018), which are more conservative. The 2018 climate change projections have not been included in WRC's TR2018/02.

Table 4-1 24hr Design Rainfall Depth for various locations

Probability	24hr Rainfall Depth Existing Climate (in mm)			24hr Rainfall Depth Including Climate Change (in mm)		
	Township	Urban North & Lower Rural	Upper Rural	Township	Urban North & Lower Rural	Upper Rural
2YR ARI	131	144	176	143	157	192
10YR ARI	205	225	273	232	255	309
100YR ARI	320	350	425	374	409	495

Source: HIRDS-v4 NIWA 2018

Notes: Climate change allowance in accordance with MfE-2008 guidelines

Temporal rainfall patterns are generated from the local HIRDS data as per TR2018/02 (WRC 2018) guidelines. Two standard patterns are used for the entire catchment, one for current climate and including the impact of climate change (refer Table 4-2 and Figure 4-3 below). 24hr Nested rainfall profile are generated from these patterns and used as timeseries in the hydrological HEC-HMS model (refer Section 4.3 below).

Table 4-2 Temporal Pattern derived from local HIRDS Rainfall Data

Time		Normalised Intensity (I/I24)	
From	To	Current Climate	Incl Climate Change
0:00	6:00	0.46	0.41
6:00	9:00	0.79	0.75
9:00	11:00	1.43	1.45
11:00	11:30	2.52	2.71
11:30	11:40	3.62	4.01
11:40	11:50	3.62	4.01
11:50	12:00	6.06	6.71
12:00	12:10	11.44	12.69
12:10	12:20	4.81	5.30
12:20	12:30	3.62	4.01
12:30	13:00	2.52	2.71
13:00	15:00	1.43	1.45
15:00	18:00	0.79	0.75
18:00	0:00	0.46	0.41

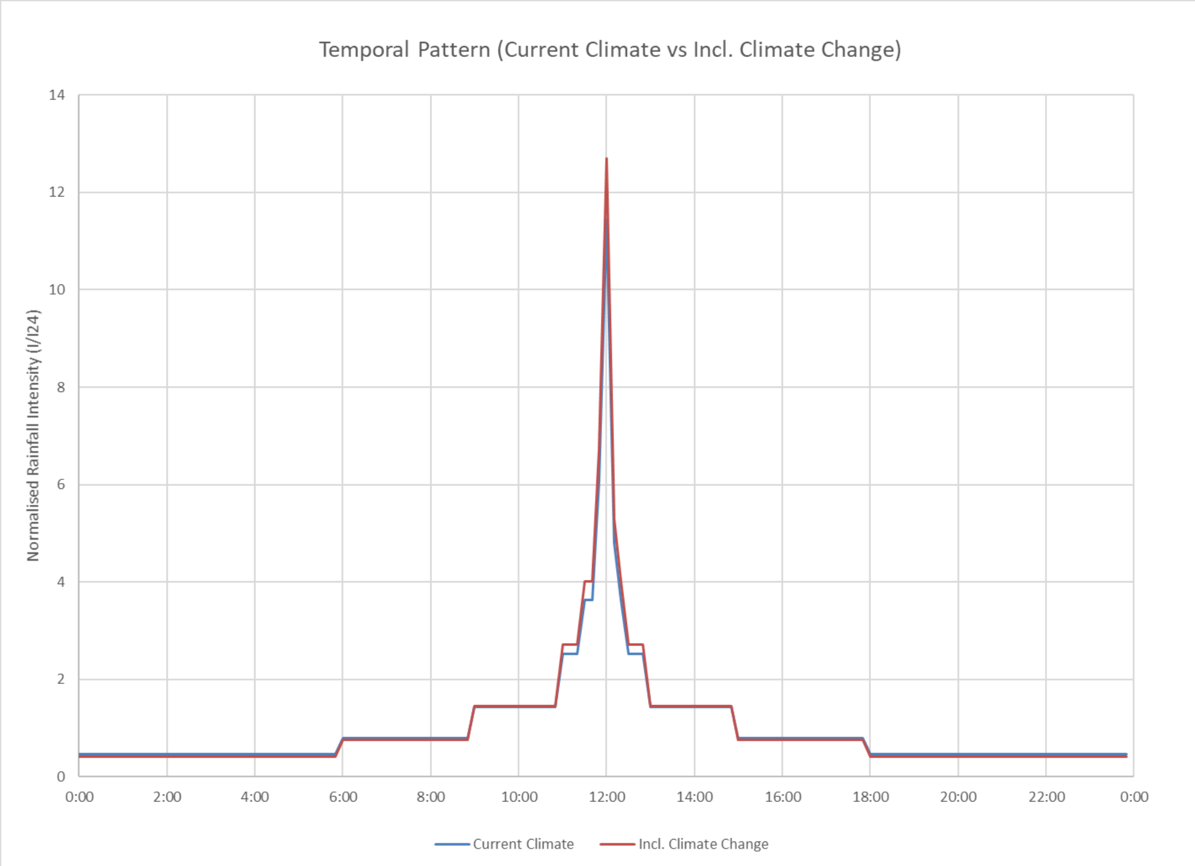

Figure 4-3 Temporal Pattern 24-hour Design Rainfall Event

4.2.5 Hydrometric Data

No long-term flow gauge is available in the Whangamata catchment. Hence model validation against gauging data was not undertaken.

4.2.6 Impervious Area

For the hydrological model, the catchment has been split in 51 different "hydrological" zones, depending on:

- TCDC District Plan Zones and Policy Areas,
- Rainfall,
- Soil type (Hydrological Soil Group)

For each of these hydrological zones a representative impervious area coverage has been assessed.

The respective zones are shown in Figure C1 & C2 Appendix C. It is noted that the following simplifications have been made to limit the number of hydrological zones in the model:

- Roads have been included with adjacent zoned land, rather than having a separate zone for each housing block.
- Housing Zone Beach Amenity has been incorporated wit adjacent housing zones.
- Very narrow sections (primarily open space) have been removed.
- Industrial and Service Industrial have been combined into Industrial.
- Town Centre and Pedestrian Frontage have been combined into Town Centre.

The resulting simplified zoning plan including the respective hydrological zone is shown in Figure 4-4 below (refer also Figure C1 & C2 in Appendix C).

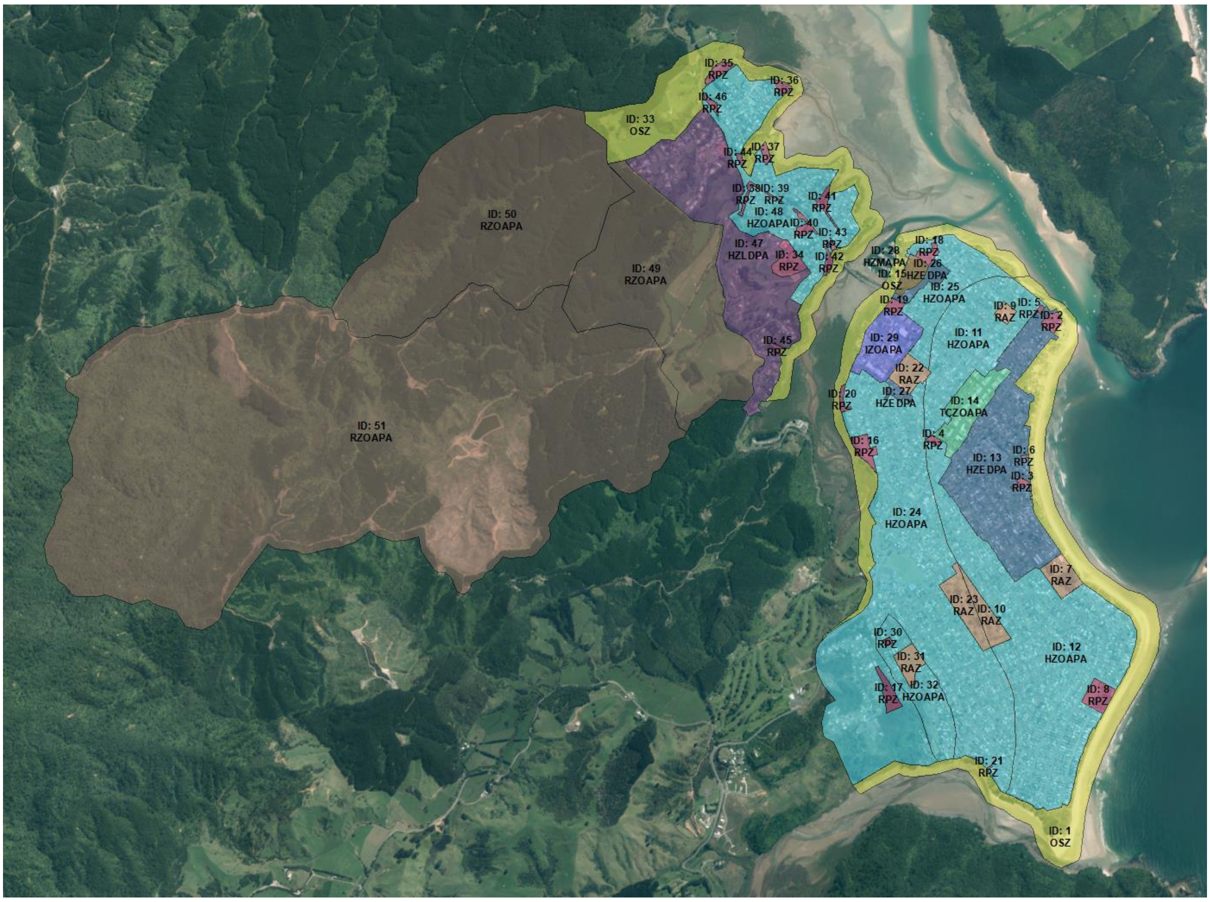

Figure 4-4 Simplified District Plan zoning and respective hydrological zone ID.

Existing Development

For the Existing Development scenario, the impervious area has been based on:

- TCDC GIS Building Footprint data
- Aerial Image

TCDC has data for building footprints but not for other impervious surfaces such as driveways, road carriageways and footpaths. Percentage impervious areas has been assessed using a combination of the building footprint layer and aerial images. For this assessment each hydrological zone has been assessed individually. This involved a combination of:

- Calculating the building footprint area and the road reserve area using the respective GIS layers
- Manually identifying the other hard stand areas (like driveways, garages, sheds, etc.)

For manual assessment of the other hard stand areas (especially the residential areas) a representative ratio between building footprint and the other hard stand areas was calculated using a 10ha sample area (refer Figure 4-5 below).

Buildings have been separated from other impervious areas as the runoff of residential buildings is assumed to be discharged by means of soakage. It is noted that commercial and industrial buildings are assumed to be connected to a reticulated network. Also, buildings in Type D soils are not expected to discharge into soakage due to the poor soakage characteristics of the soil. There are no buildings on Type C soils in Whangamata.

Figure 4-5 Residential sample area to calculate impervious footprint of other hard stand impervious areas

Table 4-3 below provides a summary of the assessed percentages impervious area for the various land use types (as per District Plan). Within each of the District Plan land use zones the assessed impervious coverage may vary. A full list of the 51 hydrological zones that have been identified is provided in Appendix A. The overall coverage is also presented in Figure C1 & C2 in Appendix C.

Table 4-3 Impervious coverage assumptions Existing Development

District Plan ID	District Plan Land Use Description	Building roofs connected to soakage*	Total Modelled Impervious Coverage
HZBPA	Housing Zone Beach Amenity [1]	N/A	N/A
HZEDPA	Housing Zone Extra Density	5 – 31%	17 – 53%
HZLDPA	Housing Zone Low Density	6%	12%
HZMAPA	Housing Zone Marine Activity	2%	54%
HZOAPA	Housing Zone	17%	40%
IZOAPA	Industrial Zone	23%	70%
IZSIPA	Industrial Zone Service[2]	N/A	N/A
OSZ	Open Space	0%	0%

[1] Beach Amenity zoned land has been combined with adjacent housing zone.
[2] Industrial Zone Service has been combined with Industrial Zone.

District Plan ID	District Plan Land Use Description	Building roofs connected to soakage*	Total Modelled Impervious Coverage
PF	Pedestrian Frontage Town Centre[1]	N/A	N/A
RAZ	Recreational Active Zone	0 – 10%	0 – 30%
RPZ	Recreational Passive Zone	0 – 5%	0 – 23%
RZFDPA	Rural Zone Future Development	0%	0%
RZOAPA	Rural Zone	0%	0%
TCZOAPA	Town Centre Zone	28%	75%

Note: Soakage only assumed for catchments with HSG Type A or B.

An overall detailed table specifying the adopted percentages for each hydrological zone is presented in Appendix A and graphically represented in Figure C.1.

Maximum Probable Development

For the MPD scenario the impervious footprint is based on the District Plan development restrictions for the respective land use specified (refer TCDC District Plan Portal[2]). Additional percentage impervious area has been included to allow for road impervious footprints and hardstand areas. The in the model adopted percentages are presented in

Table 4-4 below.

Table 4-4 Impervious coverage assumptions Maximum Probable Development

Zone	Zone Description	DP Max Site Coverage	Building roofs connected to soakage*	Total Impervious Coverage
HZBPA	Housing Zone Beach Amenity	N/A	N/A	N/A
HZEDPA	Housing Zone Extra Density	45%	30%	60%
HZLDPA	Housing Zone Low Density	15%	10%	20%
HZMAPA	Housing Zone Marine Activity	60%	40%	60%
HZOAPA	Housing Zone	35%	33%	50%
IZOAPA	Industrial Zone	70%	0%	70%
IZSIPA	Industrial Zone Service	N/A	N/A	N/A
OSZ	Open Space Zone[3]	1%	0%	0%
PF	Pedestrian Frontage Town Centre	N/A	N/A	N/A
RAZ	Recreational Active Zone[4]	60%	0%	15%
RPZ	Recreational Passive Zone	15%	0%	15%
RZFDPA	Rural Zone Future Development	10%	0%	10%
RZOAPA	Rural Zone	10%	0%	0%
TCZOAPA	Town Centre Zone	0%	0%	80%

Notes:

- Three areas have an ED %imp Area that is larger than the MPD maximum. The higher ED %Imp Area has been adopted.
- The maximum site coverage for Recreation Active Zone is 60%, which is considered too high and unrealistic. This has been reduced to 15%, being the same as Recreation Passive Zone.

[1] Pedestrian Frontage Town Centre zoned land has been combined with Town Centre Zone.
[2] TCDC District Plan Portal Part VIII – Zone Rules (https://eplan.tcdc.govt.nz/pages/plan/Book.aspx?exhibit=TCDC_Plans_External)
[3] Open Space Zone max site coverage is 1%, but for simplicity reasons this has been set to 0%.
[4] Recreational Active Zone Max Site Coverage percentage is not realistic and has been reduced to 15%, unless existing coverage is larger.

- The 1% maximum site coverage for Open Space Zoned land has been reduced to 0% for model simplification reasons, as it is not expected to have a significant impact on the results due to the relatively small increase.
- Residential housing zones include additional allowance for road impervious area (5% for Low Density and 15% for all other).
- Building roof soakage is only assumed for non-commercial land use zones with HSG Type A or Type B.
- Building roof areas are assumed to be 2/3 of the impervious footprint. Building roof areas have been reduced for three areas to avoid MPD runoff being less than ED runoff due to high soakage assumptions.
- Industrial Service Zone has been combined with Industrial Zone.

4.3 Hydrological Model

4.3.1 Method Used

Modelling of the excess runoff is based on the guidelines outlined in the Waikato Regional Council Stormwater Runoff Modelling Guideline (TR2018/02). The key features of the TR2018/02 rainfall-runoff model are:

- A standard 24-hour temporal rainfall pattern derived from HIRDS local data, 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,
- Excess runoff depth calculated using SCS runoff curve number method, with curve numbers determined from the TR2018/02 guidelines according to classifications assigned to soil types or Hydraulic Soil Groups (HSGs) obtained from Landcare Research[1] soil maps.

  It is noted that for urbanised areas the allocated soil group has been altered to account for soil disruption and compaction following development of the land. A residential or commercial zoned catchment with a group A soil has been allocated the CN value for a group B soil (similarly a HSG B becomes HSG C).

The adopted hydrological method for generating and modelling the excess rainfall runoff is a combination of:

- Rain on Grid method for the developed lower lying catchments. This is the same method that has been used for the RFHA, where excess rainfall runoff (after deduction of initial abstraction and infiltration losses) is entered on the DEM surface and runoff is calculated within the hydraulic model component.
- Lumped catchment assessment for the Te Weiti and Waikiekie streams. For this method the catchment of the respective streams is identified including an assessment of the response time (i.e. time of concentration). A runoff hydrograph is generated representing the runoff of the entire lumped catchment. This runoff is coupled to a location on the DEM where the respective stream enters the so called 2D-Zone (i.e. a zone that represents the extent of the modelled surface).

The Rain on Grid method requires that the entire catchment is represented by a DEM. This may result in large computation times, which is the downside for large catchment models. The lumped catchment approach is much faster but can be complicated to model if the catchments do not have clear boundaries or specific discharge points. It is also noted that excluding catchments outside the area of interest provides a large potential for reduction of the DEM extent and therefore may result in a significant reduction in computation times. The Rain on Grid method is therefore ideally suited for flat

[1] Landcare Research S-Map Online, https://smap.landcareresearch.co.nz/

non-confined catchments (like the urbanised areas), while lumped catchments are ideal for modelling runoff of large confined catchments (like the large stream catchments).

4.3.2 Rain on Grid Catchments

For the rain on grid catchment areas the excess rainfall runoff has been calculated for each of the land use zones listed under Appendix A. The resulting timeseries are applied directly to each mesh element of the modelled ground surface in the respective land use zones. The subsequent routing of the runoff is modelled in 2D using the DEM and triangular mesh (refer Section 4.4 below).

For this method timeseries were developed representing the excess rainfall runoff for each land use zone depending on:

- Soil Type (i.e. Hydrological Soil Group) (refer Section 2.3 above).
- Rainfall obtained from HIRDSv4 for various locations in the catchment (refer 4.2.4 above).
- Percentage impervious area based on District Plan zoning (refer Section 4.2.6 above).

For each land use zone, the following pervious and impervious areas were identified, and excess runoff calculated:

- Roof Soakage:

  For non-commercial buildings, roof runoff located in areas with well-draining soils (i.e. HSG A or B) is assumed to be discharged by means of on-site soakage systems with a discharge capacity equal to the 2-year peak rainfall.

- Other Impervious Areas:

  For other impervious areas it is assumed that 100% of the rainfall runs off, which is slightly more conservative than the using a CN of 98.

- Pervious Areas:

  Runoff from pervious areas has been calculated for each land use type and rainfall zone using HEC-HMS software. Refer to Table 3-2 for adopted CN values.

The calculated excess runoff depth has been included in the model as a Rainfall time series boundary. Each of the identified rain-on-grid catchment zones, the respective net excess runoff is represented by a unique profile, of which there are 49 in total (plus the two rainfall profiles for the lumped catchments).

An example of the rainfall and excess runoff for a selection of three land use types is provided in Figure 4-6 below. It shows the cumulative rainfall for the 24hr 100yr design storm event including allowance for climate change versus the calculated excess runoff for the land use zones listed under Table 4-5 below for soil type A.

Table 4-5 Example Land Use zones

ID	DP Description	2D Zone	HSG	Perc Imp Area - MPD		
				Roof Soakage	Other Imp Area	Perv Area
3	Recreation Passive Zone	Township	A	0%	15%	85%
12	Residential Zone	Time Step	B	33%	17%	50%
14	Commercial Zone	Time Step	B	0%	80%	20%

Figure 4-6 shows that the runoff of residential zoned land is similar to the runoff of recreational land. This is due to the assumption that roof runoff is discharged through soakage with a capacity of the 2yr rainfall event. As a result, the contribution of runoff from the roofs of buildings occurs only during the peak half hour of the storm event.

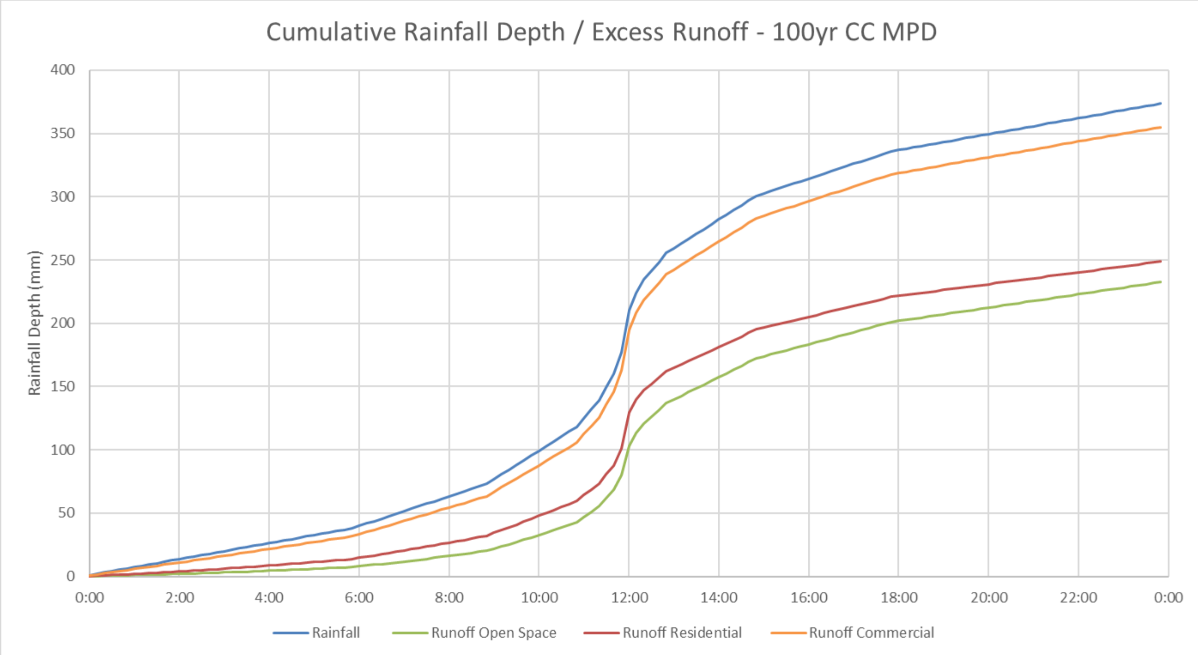

Figure 4-6 Cumulative Rainfall Runoff Depth for various land use types

4.3.3 Lumped Catchments

For the lumped catchments of the Te Weiti and Waikiekie streams, the catchment boundaries were delineated in GIS software based on primarily the 5m contour data as the LiDAR only covers the lower extent of the catchments. The catchment boundaries are shown in Figure A of Appendix C.

Infiltration characteristics are based on the respective soil type classification as shown in Section 2.3. The weighted CN value has been calculated using GIS. The impervious area has been assumed 0% for both ED and MPD scenarios. Sub-catchment lengths and slopes were computed using data from manually digitised flow lines in GIS based on the equal area method as specified in ARC TP108 document (ARC, 1999). The hydrological parameters for the three catchments are presented in Table 4-6 below, adopted for both existing (ED) and future (MPD) development scenarios.

Table 4-6 Hydrologic data for lumped sub-catchments

Catchment	Total Area (Ha)	Weighted CN	Initial Abstraction (mm)	Flow Length (m)	Slope (Equal Area)	Time of Concentration (min)
Te Weiti North	102	42	17.9	2200	4.1%	75
Te Weiti South	40	34	24.3	1100	8.0%	45
Waikiekie	511	56	9.8	5300	3.2%	120[1]

[1] The Time of Concentration for the Waikiekie stream has been adjusted to 180min following flow validation (refer Section 5.1 below)

It is noted that following validation the Time of Concentration for Waikiekie has been adjusted from 120min to 180min (refer Section 5.1 below).

The above catchment characteristics are included in the ICM model as sub-catchments.

The following rainfall time series have been set up representing the rainfall in the catchments (Refer Table 4-1 for details):

- Te Weiti North and South: Urban North & Lower Rural (Time Series Profile 50)
- Waikiekie: Upper Rural (Time Series Profile 51)

4.4 Hydraulic Model

4.4.1 Model set up

The hydraulic model adopted the RFHA model as a base and added the 1D piped network.

4.4.2 DEM

The hydraulic model of the study area was developed incorporating the 2D digital elevation model (DEM) used for the RFHA model including some modifications. This DEM is a combination of the 2013 LiDAR data and the 5m contour data in areas where no 2013 LiDAR data is available (refer Section 3.1).

Modifications were required to adequately model the inlet and outlet structures that are linked to the 2D surface. LiDAR data often does not adequately pick up the low points of streams and channels due to vegetation and water surface light reflection. For model stability it is essential that invert levels of linked 1D-2D structures have the same level. It is therefore required to adjust the DEM.

This is done by using Mesh Level Zones. For most linked structures a small area of the 2D surface is lowered to match the invert level of the respective structure. For some locations this was considered better to adjust longer sections of the stream to ensure positive gradient and ultimately a better mathematical computation. These areas are:

- Sections of both Te Weiti and Waikiekie Stream, which involved lowering the channel upstream and downstream of the State Highway culverts. Streambed levels were assumed to be 200 to 300 mm below water surface level (based on observations during site walk over).
- The concrete drains at Park Avenue Reserve and McKellar Place Walkway to ensure that the low points of the concrete channel adequately represented in the model.

4.4.3 Hydraulic Model Extents

The 2D hydraulic model extent is defined by the 2D Zone as shown in Figure 4-7 below. This covers an area of 269 ha north of the Wentworth River and 494 ha south of it. The total catchment area (including the lumped catchments outside the 2D model extent) is approximately 1400ha large.

The primary stormwater drainage network system is represented in the model by nodes (i.e. manholes, inlets, outlets, catchpits) and conduits as connecting pipes. A summary of various hydraulic model components are given in Table 4-7 , and briefly described below.

Table 4-7 Summary of hydraulic model components

Hydraulic Model Components	Values
1D Model Components	
Total number of stormwater network system nodes	577
Number of manholes / sumps	492
Number of outfalls	74
Number of dummy nodes	10
Number of storage nodes	1
Total number of conduits	498
2D Model Components	
Total area of model domain	763 ha
Number of mesh vertices	845,193
Number of mesh triangles	1,689,926
Number of mesh elements	1,662,423

Two different mesh sizes have been adopted as shown in Table 4-8 below. Although the meshing has been done with a Minimum Element Area of 2m^2, the generated minimum element size is 1.7m^2.

Table 4-8 Mesh parameters

Location	Minimum Element Area	Maximum Triangle Area
Within main areas of interest	2 m^2	5 m^2
Outside main areas of interest	20 m^2	100 m^2

1D Components

Model nodes are utilised to represent the stormwater drainage network system attributes such as manholes, inlets, outlets, and catchpits. Catchpits are generally not included in the model to reduce model complexity and the level of detail.

To provide the exchange of flow between the 2D surface and the piped network, manholes are modelled as nodes with "2D" Flood Type, with water exchange occurring at manhole lid level. Following peer review, it was recommended to use nodes with "Gully 2D" Flood Type to prevent unrealistically high inflows from the 2D surface. This caused strange results with large jumps in the node's 2D results. In coordination with the peer reviewer and TCDC it was decided to use the traditional "2D" Flood Type, except for locations were the inflow causes unrealistic impact on the model results. For those locations individual catchpits are included in the model, but with a flow restriction of 100L/s maximum (using orifice structures with limiting discharge) for the following three locations:

- 125 Lorraine Place
- 106 Apperly Street
- 104 Kotuku Street

Two types of outfalls have been used, 'Outfalls' and 'Outfalls 2D', where runoff from "Outfalls" are lost from the model, while 'Outfalls 2D' discharge their runoff onto the 2D surface. In general, "Outfalls" were used when flows ware discharged into the coastal area, not affecting any areas or structures downstream. A constant water level boundary is allocated to the outfall representing the sea level. The 'Outfalls 2D' are typically used for culvert and pipe outlets discharging onto land within the catchment.

Model conduits were utilised to represent stormwater drainage pipes. The pipe data input to the model comprised of diameter, upstream and downstream inverts and connecting nodes based on the TCDC GIS asset database or survey information.

Six dummy manholes and three dummy conduits were inserted to model the downstream connection of the three lumped hill sub-catchments.

Streams are represented and modelled using the 2D DEM surface. It is common that the streambed is not or not well represented in the LiDAR and DEM, due to:

- Dense vegetation blocking the penetration of LiDAR to the actual ground surface,
- LiDAR typically picking up water surface level instead of stream bed level,
- Loss of detail during the conversion from LiDAR points to DEM triangular surface.

To ensure that the streambed is properly represented in the 2D model, mesh level zones have been specified. These represent the low flow streambed. The base width of the mesh level zone is set at 2m. The streambed levels have been estimated at approximately 3 locations for each stream section. This estimate is based on observations during the site visit in July 2018, using aerial images, and LiDAR.

2D Components

The 2D mesh is bounded by the 2D Zones, indicated by the orange line in Figure 4-7 below. A coarse mesh definition is applied for this zone, with the parameters Maximum Triangle Area set at 100 m^2, and the Minimum Element Area at 20 m^2. Note that rain-on-grid is applied, with a Manning's roughness of 0.02. Within the 2D zone, several "Mesh Zones" define areas of finer detail as indicated by the purple line in Figure 4-7 above. For the Mesh Zones the Maximum Triangle Area is $5m^2$ and the Minimum Element Area is set at $2m^2$.

In addition to the mesh zones, are "Mesh Level Zones" (refer red areas in Figure 4-7 below), which are areas of modification of the terrain surface. These include (refer also Section 4.4.2 above):

- Te Weiti and Waikiekie streams, both downstream and upstream of the SH25 culverts.
- Open channel drains at
- Culvert inlet and outlet structures coupled to the 2D surface
- Linkage of dummy manholes used for coupling the inflow hydrographs of the streams onto the 2D surface.

This is to better represent the 1D features and to ensure no level difference between the 1D invert level and the 2D surface level at 1D-2D coupled structures.

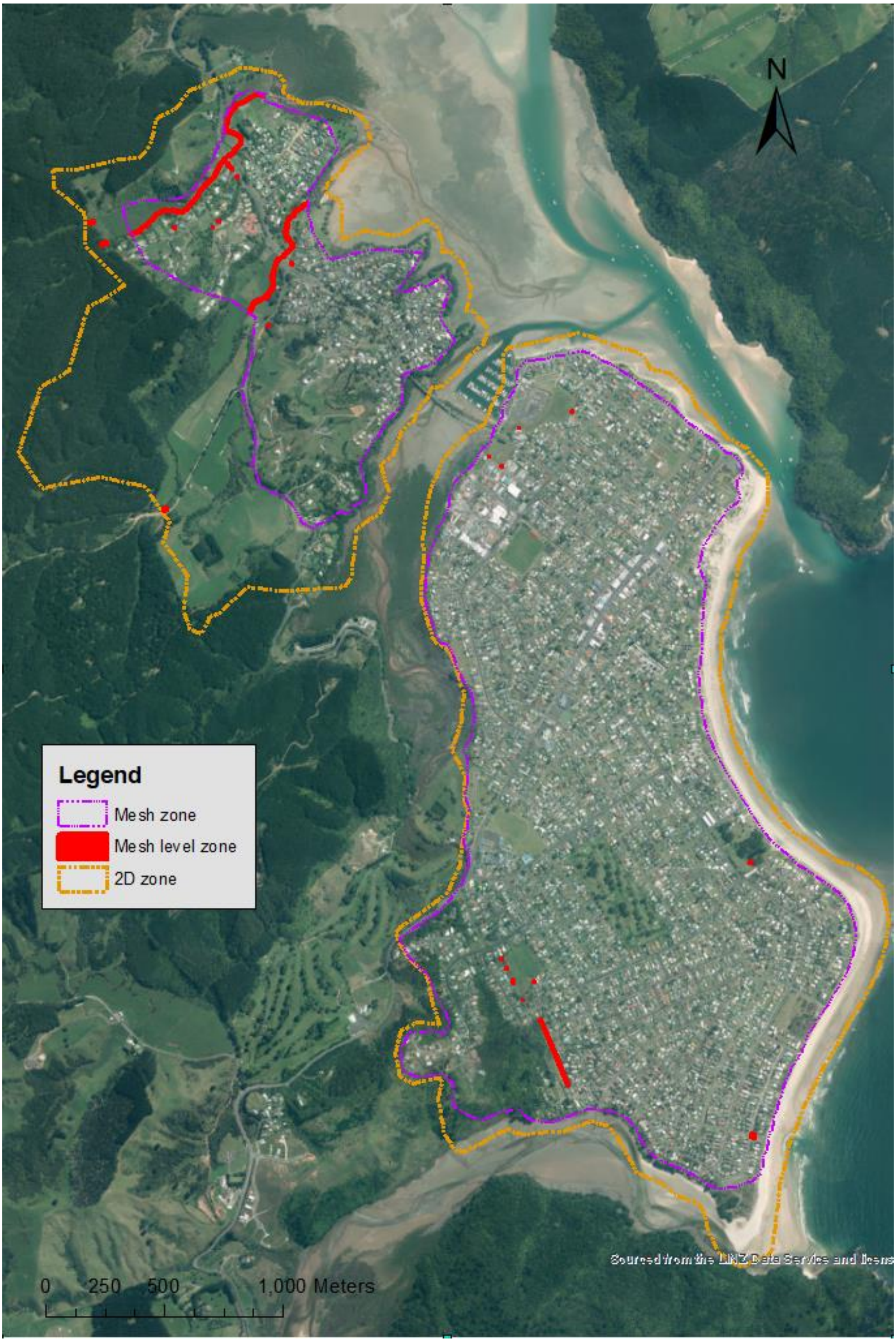

Figure 4-7 "2D Zone", "Mesh Zone", and "Mesh Level Zone" extents

4.4.4 Energy Losses

1D Network Head Losses

Friction factors were assigned to the conduits as a Manning's n roughness of 0.013. Default values were adopted for upstream and downstream head loss, with "Normal" head loss type appropriate for "well-constructed manholes on pipe systems" the ICM inference tool was used to infer pipe entrance head loss coefficients. Nodes were modelled assuming "full benching".

For the two SH25 culverts at Te Weiti and Waikiekie streams a roughness of 0.02 was adopted.

2D Surface Friction losses

Roughness Zones have been specified to represent the following areas:

- Developed areas (both residential and commercial/industrial: n = 0.35
- Rural, Open Space and Recreational areas: n = 0.05
- Road Reserve: n = 0.02

These values are based on Auckland Council's "Stormwater Modelling Specification" (Nov 2011) in combination with the Australian Rainfall Runoff (ARR 2012).

Building footprints were not specifically modelled or blocked out but are represented by the relatively high roughness value for the entire property.

It is noted that for most of this catchment (i.e. Township area) the roughness coefficient is not considered a critical model parameter due to the flatness of the terrain and subsequently low flow velocities.

4.4.5 Specific drainage features

Details on a number of specific drainage structures / features are provided below:

Williamson Park Pond and Outlet

Williamson Park Pond receives stormwater runoff from the southwestern part of the township catchment. The piped network to the pond has recently been upgraded (Refer TCDC Williamson Park Stormwater Outlet Duplication Project, WSP/OPUS 2019). The upgrade comprises a new outlet from the Ocean Road / Williamson Road intersection, where the Williamson Road stormwater pipe is separated from the existing 900mm dia outlet and discharges now through a new 1050mm dia pipe into the pond.

The pond has no piped outlet. The primary means of water discharge is through soakage. It has an overflow onto the beach, consisting of a gabion basket structure with an 8m wide concrete level spreader on top of it. During the site visit in July 2019, some of the overflow was covered by dunes and the length of the overflow was observed to be 6m (Refer Figure *2-8* in Section 2.5 and Figure 4-8below).

Figure 4-8 Overflow path between Williamson Park Pond and the beach

The level of this level spreader is modelled at 3.5m AVD-46. The level was surveyed in July 2018 by TCDC at an unknown RL of 2.56m. A conversion is made to AVD-46 based on the available data sources (i.e. TCDC GIS, OPUS 2018 survey, WSP/OPUS Design drawings and 2013 LiDAR). As there is a discrepancy between the 2013 LiDAR and the WSP/OPUS design the level is expected between 3.29m and 3.53m AVD-46. The modelled level of 3.5m AVD-46 is a conservative assumption.

It is noted that maintenance to the overflow occurred after the 2013 LiDAR as dune sand had built up over time. The LiDAR levels near the outfall are therefore too high. In the model this has been adjusted by specifying a 6m wide mesh level zone between the pond and the beach at the 3.5m AVD-46 overflow level.

Otahu Road Stormwater Pump Station

The model includes a stormwater pump station at the eastern end of Otahu Road (Refer Figure 4-9). The capacity of the pump station, underground flood storage and piped network is based on as-built drawings and design data from Thames Civil Engineering Ltd (Thames-2012). The modelled capacity of the pump station is 40L/s and the rising main discharges the runoff into a manhole at Otahu Road and Given Avenue intersection, which is about 200m to the west. A 375mm dia gravity pipe discharges the runoff into Otahu River at 149 Patuwai Drive.

Figure 4-9 Otahu Road Pump Station

The pump station wet well has a high-level overflow pipe to the beach. Based on site observations the diameter of this pipe is 525mm and the length is 45m. The overflow level was measured to be at 1m below ground level.

Figure 4-10 Underground storage and soakage near Otahu Road Pump Station

Underground Storage and Soakage Systems

Additional underground storage directly south of the pump station is included in the model based on as-built drawings by RMS Surveyors (Thames - 2012). The storage consists of Atlantis Flo-tanks providing storage and soakage infiltration (refer Figure 4-10 below). The assumed infiltration rate is 10mm/hr (refer Appendix A, TR-55) over the surface area of the storage units.

Further to the west (between the pump station and Marie Crescent) two rows of Triton Drainage Cells also provide underground storage and soakage. This has been included in the model based on the RMS Surveyors as-built drawings (Thames - 2012). The width of the units is assumed 1.4m and the height 0.86m. The infiltration rate is assumed to be 10mm/hr.

It is understood that there are other underground soakage systems within the township area. These have not been included in the model, due to lack of data on these systems.

Te Weiti and Waikiekie Culverts

Two culverts crossing SH25 have been included in the model. The modelled culverts are based on dimensions taken during the site visit, which are:

- Te Weiti Culvert 2.5m wide x 1.2m high
- Waikiekie Culvert 5.5m wide x 2.1m high

More details on these culverts and their performance are provided in Section 5.2.

4.5 Boundary Conditions

4.5.1 Rainfall Data

Rainfall data have been derived from HIRDS v4 as per WRC TR2018/02. A nested 24-hour duration temporal rainfall profile was developed from the HIRDS rainfall data. The rainfall depths used for the model are presented in Section 4.2.4 above.

4.5.2 Tidal Data

A constant tailwater level has been assumed as downstream boundary condition for the model. The adopted level is based on the Mean High Water Spring (MHWS) level published by the Waikato Regional Council Coastal Inundation Tool (Refer Waikato Regional Council web-site, https://www.waikatoregion.govt.nz/services/regional-services/regional-hazards-and-emergency-management/coastal-hazards/coastal-flooding/coastal-inundation-tool/). The MHWS levels for various climate change scenarios are presented in Table 4-9 below.

Note that the levels presented in the above tool refer to Moturiki Vertical Datum 1953 (MVD-53), while the model and this report refer to Auckland Vertical Datum 1946 (AVD-46). Based on LINZ data (https://www.linz.govt.nz/data/geodetic-system/datums-projections-and-heights/vertical-datums/vertical-datum-relationship-grids) the difference between the two datums is 26mm (AVD-46 = MVD-53 + 0.026m), which has been used for the conversion of the levels.

Tailwater levels at outfalls along the Wentworth River have been adjusted following sensitivity analysis on the impact of elevated flood levels in that river (refer Section 5.7 for details).

Table 4-9 Tidal Conditions

Climate Change Scenario	Model Scenario	MHWS (MVD-53)	MHWS (AVD-46)
Present Day	ED	1.07mRL	1.10mRL
Future Projection 0.5m Sea Level Rise	n/a	1.57mRL	1.60mRL
Future Projection 1.0m Sea Level Rise	MPD	2.07mRL	2.10mRL

The Existing Development (ED) model is based on the present day sea level conditions, while for the Maximum Probable Development (MPD) scenario the future projection with 1.0m sea level rise has been adopted. This is based on the recommendations provided in Coastal Hazards and Climate Change guidance for local government (MfE 2017) as shown in Table 4-10 below assuming Category C is the most relevant for Whangamata and TCDC.

The impact of sea level rise along low-lying properties on the northern end of the peninsula is shown in Figure 4-11 below. The image shows the extent of the sea at MHWS assuming 1.0m SLR. It is noted that this excludes the impact of low barometric pressure, storm surge, wave run-up, and runoff from rainfall, which result in a further increase in flood levels.

Table 4-10 Minimum transitional New Zealand-wide SLR allowances and scenarios for use in planning instruments where a single value is required at local/district scale while in transition towards adaptive pathways using the New Zealand-wide SLR scenarios

Category	Description	Transitional response
A	Coastal subdividion, greenfield developments and major new infrastructure	Avoid hazard risk by using sea-level rise over more than 100 years and the H+ scenario
B	Changes in land use and redevelopment (intensification)	Adapt to hazards by conducting a risk assessment using the range of scenarios and using the pathways approach
C	Land-use planning controls for existing coastal development and assets planning. Use of single values at local/district scale transitional until dynamic adaptive pathways planning is undertaken	1.0 m SLR
D	Non-habitable short-lived assets with a functional need to be at the coast, and either low-consequences or readily adaptable (including services)	0.65 m SLR

Source: Table 12 of Coastal Hazards and Climate Change, Ministry for the Environment, Dec 2017 (MfE 2017)

Figure 4-11 Coastal inundation MHWS including 1.0m SLR[1]

4.6 Model Limitations and Assumptions

4.6.1 Network Model Assumptions

17 pipes have been identified to be decreasing in diameter in the downstream direction. These pipes are listed in Table 4-11 below. A number of them have been confirmed, while others are likely to have been designed as such (i.e. providing storage and soakage). One section has been adjusted in the model based on engineer's judgement (Conduit SWMH_302020.1) and is discussed in Section 5.6. Two pipe sections have been identified TBC (To Be Confirmed) and are recommended to be considered for survey.

Table 4-11Stormwater pipes with decreasing diameter in downstream direction

Conduit ID	Node ID	Location	Description
SWCP_207755.1	SW_Storage_553054	801 Otahu Rd	Soakage System at Otahu Rd pump station (confirmed by drawings)
SWMH_201689.1	SWMH_551785	1000 Port Rd	525 mm Ø into 375 mm Ø Likely some storage/soakage system
SWMH_201692.1	SWMH_301110	804 Port Rd	525 mm Ø into 375 mm Ø Likely some storage/soakage system

[1] Source: Waikato Regional Council Coastal Inundation Tool (Refer Waikato Regional Council website, https://coastalinundation.waikatoregion.govt.nz/).

SWMH_201694.1	SWMH_551785	906 Port Rd	525 mm Ø into 375 mm Ø Likely some storage/soakage system
SWMH_201695.1	SWMH_201696	1006 Port Rd	525 mm Ø into 225 mm Ø Likely some storage/soakage system
SWMH_201795.1	SWMH_203338	322 Williamson Rd	450 mm Ø into 300 mm Ø No significant impact expected
SWMH_203397.1	SWMH_203398	100 Ocean Rd	375 mm Ø into 300 mm Ø No significant impact expected
SWMH_204155.1	SWMH_550935	620 Port Rd	525 mm Ø into 450 mm Ø No significant impact expected
SWMH_204516.1	SWMH_203248	Near parking area behind 103 Winifred Ave	300 mm Ø into 225 mm Ø No significant impact expected
SWMH_301102.1	SWMH_301101	103 Winifred Avenue	600 mm Ø to 450 mm Ø Confirmed by survey
SWMH_301111.1	SWMH_301117	329 Port Rd	Flow split 375 mm Ø into 375 & 300 mm Ø
SWMH_302020.1	SWMH_301085	212 Martyn Rd (playground near golf club)	675 mm Ø into 450 mm Ø This has been modelled as a 600mm dia continuous pipe (refer Section 5.6)
SWMH_302105.1	SWMH_550421	300 Hetherington Rd	675 mm Ø to 600 mm Ø To be confirmed (refer Figure 4-12 below)
SWMH_302876.1	SWMH_301099	100 Hetherington Rd	450 mm Ø into 375 mm Ø Confirmed by survey
SWMH_303404.1	SWMH_303405	123 Seabreeze Ln	375 mm Ø into 300 mm Ø To be confirmed (refer Figure 4-13 below)
SWMH_303779.1	SWMH_303778	108 Casement Rd	300 mm Ø into 225 mm Ø No significant impact expected
SWMH_553141.1	SWMH_203249	Near parking area behind 103 Winifred Ave	300 mm Ø into 225 mm Ø No significant impact expected

Figure 4-12 *Reduction pipe diameter @ Hetherington Road*

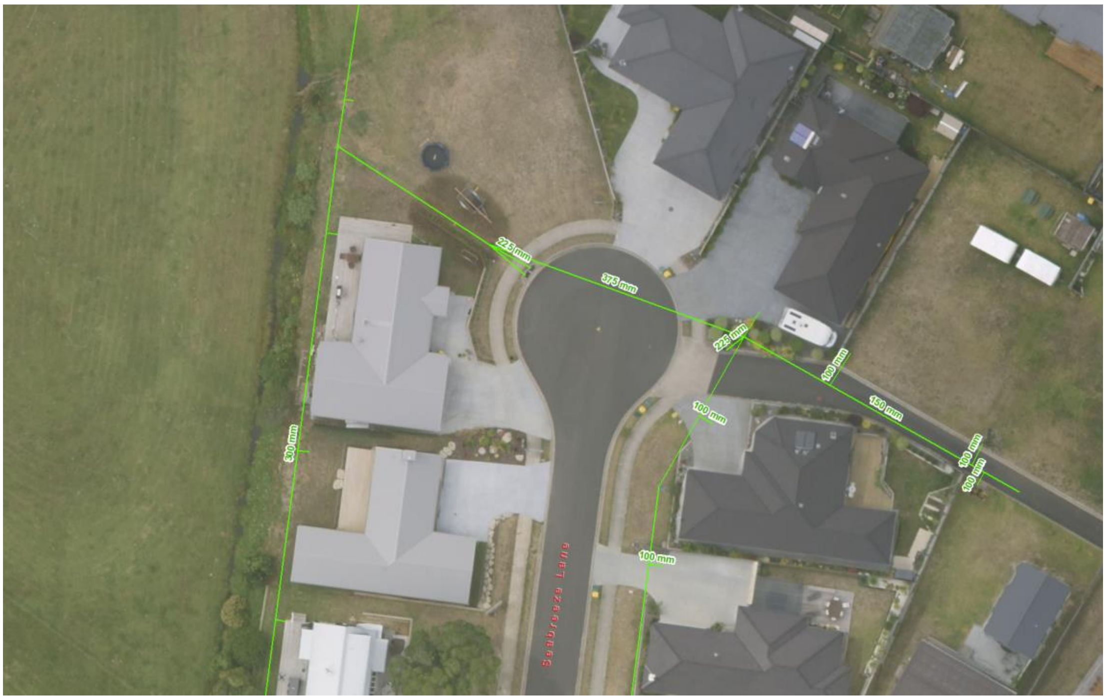

Figure 4-13 *Reduction pipe diameter @ Seabreeze Lane*

4.6.2 Model Limitations

The following constraints apply to this model analysis:

- The present modelling adopts the Waikato Regional Council stormwater runoff model (TR2018/02), the assumptions and limitation from this methodology should also be read in conjunction with this report.
- General model assumptions (like soil infiltration rates, percentage impervious area, surface roughness, etc.) are averaged over wider areas and do not represent localised variations.
- The ground surface is represented as a triangular mesh with element size of 2 to 5 m^2. Each mesh element has a ground level allocated being the average level based on LiDAR data. Level variances within the element are not represented.
- The Wentworth River and the Otahu River are not part of the flood model and no information has been provided on flood levels in these rivers. Elevated levels in these rivers can have a backwater or flooding affect in the Whangamata township. Sensitivity analysis is included in this study to estimate the effect of elevated flood levels in Wentworth River.
- No calibration of the Whangamata Catchment model has been undertaken, with hydraulic and hydrologic parameters developed from guidance documentation and engineering judgement. These adopted parameters may vary from actual catchment conditions, which could also vary in time.
- 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 runoff model is limited to the average antecedent moisture condition.
- The modelled overland flow paths are based on the LiDAR information. The extent of the flow paths may vary due to simplified model assumptions. Overland flow paths that pass-through properties can have fences, vegetation and walls that alter flow path routes and may result in localised variances in flood levels.
- The extent of floodplains and ponding areas were mapped based on LiDAR ground contours. 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 contour model.
- Large areas of the Whangamata catchment and especially the township rely on soakage as primary means of stormwater drainage. The uncertainties related to the performance of soakage systems are:
    - Infiltration rates can vary significantly depending on the location.
    - Soakage systems are at risk of clogging up by small sediments and subsequently reduce infiltration capacity and performance. The actual capacity depends on design (i.e. provision to prevent clogging up) and maintenance.
    - Infiltration can be affected due to elevated groundwater levels.
    - Often soakage systems are designed for a combination of storage and infiltration assuming a certain design rainfall profile. This means that for long duration storms the storage component may have reached maximum volume whilst inflow is more than infiltration and consequently flooding occurs.
- It is also very complicated to assess the actual capacity of a soakage system due to:
    - Lack of information on the location and design of specifically build systems. No information has been received on the design (and/or construction) such as the

    - underground storage volume, site specific infiltration rates, and infiltration surface area except for the underground storage and soakage at Otahu Road.
    - Lack of information on provisions to prevent silting up of the system, the subsequent condition, and how this affects the infiltration capacity.
- For the model it is assumed that:
    - Industrial and commercial buildings are connected to a piped network system.
    - Roofs of residential housing are connected to an on-site soakage system with the assumption that the system has the capacity equal to the current climate peak rainfall intensity for the 2yr ARI rainfall event.
    - No soakage system has been assumed for roads and catchpits.
    - No soakage system has been assumed in areas with low infiltration capacity (i.e. Type C and D Hydrological Soil Groups) except for the normal infiltration that can be expected for respective soils.
- The soakage is modelled by reducing the rainfall with the assumed soakage rate. The respective excess runoff (i.e. rainfall – infiltration loss) is entered onto the 2D surface. No further infiltration is assumed, and consequently ponded water in depression areas remain in the model (i.e. will not soak away once rainfall recedes). This results in ever increasing water levels (until it reaches a natural overflow pathway) when storm durations increase.
- Reported flooded properties are based on flood extent data only. It does not consider the level of habitable floors. Floor levels have not been surveyed.
- No freeboard is included in the presented modelling results to allow for physical processes that may not have been allowed for and uncertainties in the precision of the hydraulic modelling and the prediction of physical processes.

In summary the reported flows and levels are estimates based on numerous uncertainties, which affect the confidence in this estimation such as floor levels, tide levels, rainfall, soil infiltration rates, LiDAR data, interpolation between surveyed stream cross-sections, dynamic blockages due to debris and vegetation, and so on. As such these estimates should be treated as indicative for the purposes of determining flood levels, however the model can be utilised to assess the relative effects of potential option upgrades.

4.6.3 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, 10, and 100-year 24-hour rainfall profiles used in the model are based on WRC TR2018/02 runoff modelling guidelines.

4.6.4 Hydraulic Model Assumptions

- 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 can perform at full capacity.
- No specific underground soakage system has been included in the model, except for the underground storage at Otahu Road stormwater pump station, which has been based on as-built data.

5 MODEL VALIDATION

5.1 Te Weiti and Waikiekie Flow Validation

Te Weiti and Waikiekie streams are modelled as lumped catchments as described in Section 4.3.3 above. The runoff peak flow has been validated with other hydrological methods typically used in New Zealand, being:

- Rational Method[1]
- Flood Frequency Method[2]

Catchment data for both methods can be obtained from the NIWA New Zealand River Flood Statistics website. For the Rational Method a Runoff Coefficient of 0.25 has been adopted for all three stream catchments assuming medium soakage soil types with bush and scrub cover (Refer Table 1 of MBIE-2016).

The computed peak flows for three rainfall probability events (assuming existing climate conditions for a 24-hour nested storm) is presented in Table 5-1 and Figure 5-1 to Figure 5-3 below.

Te Weiti Flow Validation

The validation shows that the SCS UHM generated flows do reasonably match the flows generated using the Rational or Flood Frequency method for the Te Weiti stream (both northern and southern branch). For the 10yr and 100yr ARI events the SCS UHM flows are higher, while for the 2yr ARI event the flows are slightly lower.

Waikiekie Flow Validation

For the Waikiekie stream the SCS UHM peak flows are about twice as large as the other assessments. It is therefore decided to increase the time of concentration from 120 min to 180 min to get a better match. With the increased time of concentration, the flows adopted in the model are about 50% above the other assessments.

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[1] Refer MBIE-2016, New Zealand Building Code – E1 Surface Water

[2] Refer McKerchar-1989, Flood Frequency in New Zealand

Table 5-1 Validation Peak Flow from stream catchments (in m^3/s)

Probability	SCS UHM	Rational Method	Flood Frequency Method
Te Weiti North			
2yr ARI	2.4 m^3/s	2.2 m^3/s	3.1 m^3/s
10yr ARI	5.8 m^3/s	4.1 m^3/s	5.8 m^3/s
100yr ARI	12.4 m^3/s	7.3 m^3/s	9.1 m^3/s
Te Weiti South			
2yr ARI	0.8 m^3/s	1.1 m^3/s	0.9 m^3/s
10yr ARI	2.1 m^3/s	2.1 m^3/s	1.7 m^3/s
100yr ARI	4.9 m^3/s	3.8 m^3/s	2.7 m^3/s
Waikiekie Original (Tc=120 min)			
2yr ARI	22.5 m^3/s	7.7 m^3/s	12.8 m^3/s
10yr ARI	45.4 m^3/s	14.2 m^3/s	24.1 m^3/s
100yr ARI	85.7 m^3/s	25.3 m^3/s	38.2 m^3/s
Waikiekie Adjusted (Tc = 180 min)			
2yr ARI	17.9 m^3/s	7.7 m^3/s	12.8 m^3/s
10yr ARI	36.1 m^3/s	14.2 m^3/s	24.1 m^3/s
100yr ARI	68.1 m^3/s	25.3 m^3/s	38.2 m^3/s

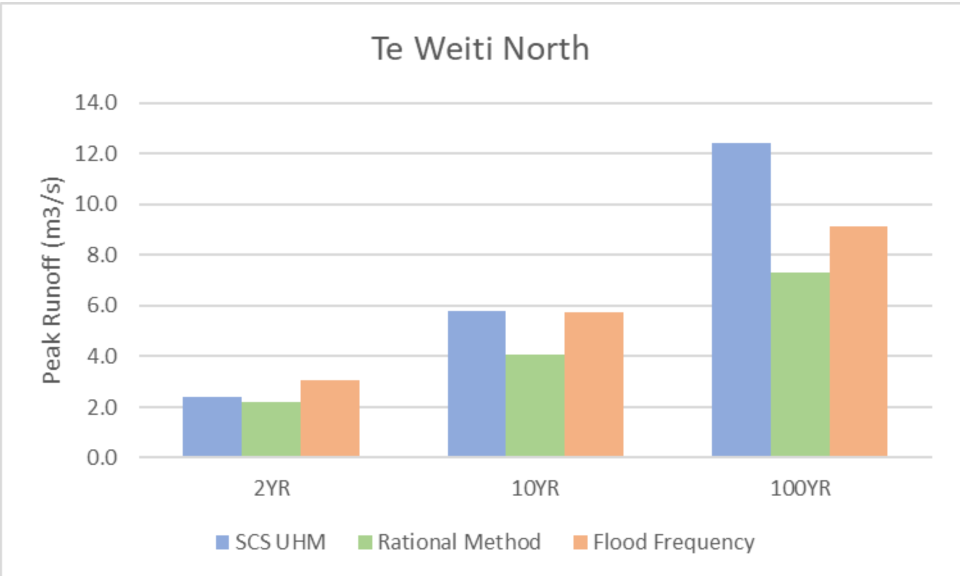

Figure 5-1 Te Weiti North Peak Flow Validation

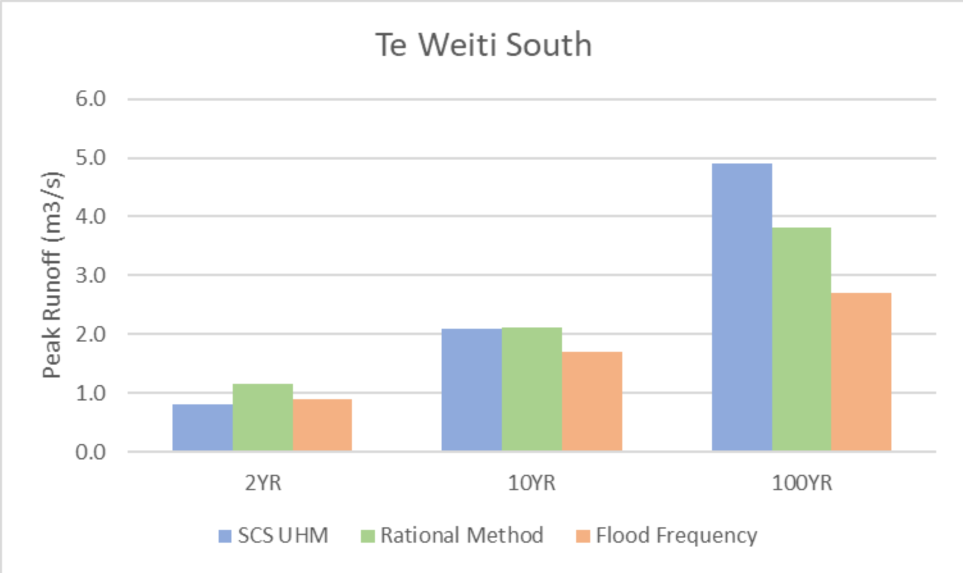

Figure 5-2 Te Weiti South Peak Flow Validation

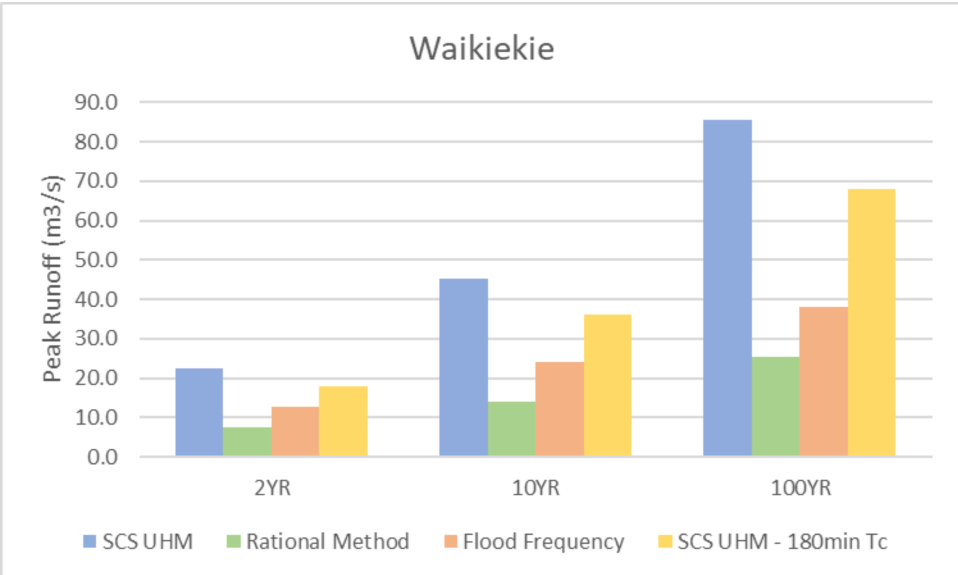

Figure 5-3 Waikiekie Peak Flow Validation[1]

[1] Note that the SCS UHM with a time of concentration of 180 min has been adopted in the model.

5.2 Te Weiti and Waikiekie culvert flow validation

The modelled hydraulic performance of the two culverts under SH25 at Te Weiti and Waikiekie streams has been validated against culvert performance computations using HY-8[1] software. Following the initial results of this validation test, culvert parameters and modelling method have been modified to achieve better resemblance.

For the Baseline scenario the culverts were modelled as "Conduit 2D" type structures, while for the sensitivity runs the "Conduit" type was used. The results are presented in Table 5-2 for the Te Weiti Culvert and Table 5-3 for the Waikiekie Culvert. For the comparison with HY-8 software ICM modelled peak flow and tailwater level was used as boundary conditions. The computed upstream water levels (and head loss dH) are compared to assess the performance for both methods. The flow used for the validation are the peak flows modelled under the Baseline scenario.

The following parameters have been adopted for the conduits:

- Bottom roughness culvert n = 0.020
- Top roughness culvert n = 0.015
- Culvert inlet head loss coefficient k = 0.5
- Culvert outlet head loss coefficient k = 1.0

The Te Weiti Culvert (refer Table 5-2 below) has for the baseline scenario a 0.6m head loss over the culvert for the ICM Baseline scenario, compared with a head loss of 0.23m computed using HY-8 software. When using the Conduit method, the head loss is 0.20m for the same flow, which is similar to the head loss modelled using the HY-8 software.

Table 5-2 Te Weiti Culvert Validation

Parameter	ICM Baseline – Conduit 2D	ICM Sensitivity Run - Conduit	HY-8
Width		2500 mm	
Height		1200 mm	
IL US		1.80 mRL	
IL DS		1.70 mRL	
Flow		4.5 m^3/s	
WL US	4.05 mRL	**3.70 mRL**	**3.68 mRL**
WL DS	3.45 mRL	**3.50 mRL**	**3.45 mRL**
Head Loss dH	0.6 m	**0.20 m**	**0.23 m**

For the Waikiekie Culvert the head loss is 0.44m for the Conduit 2D Baseline scenario and 0.55m for the Conduit Sensitivity scenario. The variance in modelled head loss compared with the HY-8 computation is similar (i.e. 50 to 60mm variance). The more conservative Conduit method is favourable as it is more conservative. (refer Table 5-3 below).

Following this analysis, it is concluded that the Sensitivity scenario (using the Conduit methodology to model culvert flows), provides a better resemblance with the HY-8 computed head losses and has therefore been adopted for both culverts.

[1] HY-8 Culvert Hydraulic Analysis Program, US Department of Transportation, Federal Highway Administration, https://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/

Table 5-3 Waikiekie Culvert Validation

Parameter	ICM Baseline – Conduit 2D	ICM Sensitivity Run - Conduit	HY-8
Width		5500 mm	
Height		2100 mm	
IL US		0.90 mRL	
IL DS		0.80 mRL	
Flow		30.9 m^3/s	
WL US	4.30 mRL	**4.04 mRL**	**4.35 mRL**
WL DS	3.86 mRL	**3.49 mRL**	**3.86 mRL**
Head Loss dH	0.44 m	**0.55 m**	**0.49 m**

5.3 Modelling catchpits

The representation of the piped network is limited to pipes of 225mm diameter or larger and manholes. Catchpits and their leads are generally not included in the model. The impact of this model assumption has been tested by running a sensitivity scenario that includes all the stormwater drainage pipes and the catchpits in Williamson Road (refer Figure 5-4 below and Figure F1 in Appendix C). The modelled baseline stormwater network is shown as black lines, while the pipes added to the modelled sensitivity scenario are presented in red. The sensitivity test is based on the MPD scenario for the 100yr 6hr design storm event including climate change allowance.

The impact of including the catchpits and leads into the model is less than 5mm reduction (light yellow-green areas) in flood levels with some areas showing a small increase in flood levels (orange areas). This increase is likely due to the increased flow into the piped network under Williamson Road creating a backwater affect for the flows from the Ocean Road network. As the impact is within the +/-5mm range, it is considered acceptable to use a simplified model that does not include all the sumps and their leads, which results in slightly conservative flood levels.

Figure 5-4 Sensitivity Run - Modelling Catchpits

5.4 Lowering Williamson Road Pond overflow level to 3.0mRL

The impact of lowering the overflow level from Williamson Road Pond to the sea from approximately 3.5mRL down to 3.0mRL has been modelled and analysed. The change in flood elevation is shown in Figure 5-5 below (and Figure F2 in Appendix C).

As expected, the results show a drop in peak flood levels within the pond of approximately 450mm, which is slightly less than the lowering of the overflow level. Flood levels at the intersection Ocean Road / Williamson Road reduced by 30 to 40mm, while further away from the pond the change is less than 30mm.

This shows that lowering of the overflow level of the pond has very limited impact on flood levels outside the reserve area.

Figure 5-5 Sensitivity Run - Lowering Pond Overflow Level

5.5 Storm duration

Waikato Regional Council requires flood risk assessments to be modelled using a 24-hour nested design storm. Considering the long computation time (approximately 10 hours for the 24-hour storm) the sensitivity of the storm duration on flood risk has been undertaken in order to justify using 12hr simulation runs for options analysis.

For the analysis simulation runs have been undertaken and compared for 6hr, 12hr, and 24hr storm durations (for the MPD 100yr ARI + CC design storm). The differences in modelled peak flood elevation are presented in two difference maps (Refer Appendix C):

- Figure F3.1 Showing the difference between the 12hr simulation versus the 6hr.
- Figure F3.2 Showing the difference between the 24hr simulation versus the 12hr

The modelled variances can broadly be summarised as shown in Table 5-4 below.

Table 5-4 Impact Modelled Storm Duration on Peak Flood Depth

Description / Location	Difference in modelled peak elevation (100yr MPD + CC)	
	12hr vs 6hr simulation	**24hr vs 12hr simulation**
General – majority of the flood plain	0 - 50 mm	0 – 20 mm
Main street near shops	0 – 10 mm	0 – 5 mm
Ponding / soakage areas near the dunes	100 -150mm	80 – 130 mm
Waikiekie	50 – 100 mm	30 – 50 mm
Te Weiti	50 - 100 mm	30 – 50 mm

The increase in peak flood levels between the 24hr and 12hr simulation run is about half of the increase between the 12hr and 6hr simulation. And for the general flooding in the catchment (i.e. excluding the streams and ponding areas along the dunes) the increase is less than 20mm.

It is also noted that the increase in flooding in the two streams is relatively small compared to the flow depth (i.e. in the order of 2m). It is also not really affecting many properties.

For the ponding areas along in the dunes it is expected that the model is likely conservative as it does not account for ongoing infiltration / soakage of ponded water in those areas. Only the direct loss to infiltration when rain falls onto the surface is accounted for in the model.

Based on the above it is considered acceptable to adopt 12hr simulation runs for the model when analysing flood mitigation options. For assessing peak flood levels for design and planning purposes, the 24hr storm duration will be adopted. The flood maps as presented in Appendix C are for the 24hr storm duration.

5.6 Inconsistent GIS data near rugby field

A main stormwater line discharges runoff from the township centre and runs under Lincoln Road, the rugby field and Lindsay Road where it discharges into the Wentworth River. The TCDC GIS Asset Data shows the following pipe dimensions (refer highlighted section in Figure 5-6 below):

- 525mm dia under Lincoln Road
- 675mm dia under the rugby field
- 450mm dia under Lindsay Road
- 600mm dia outfall into Wentworth River

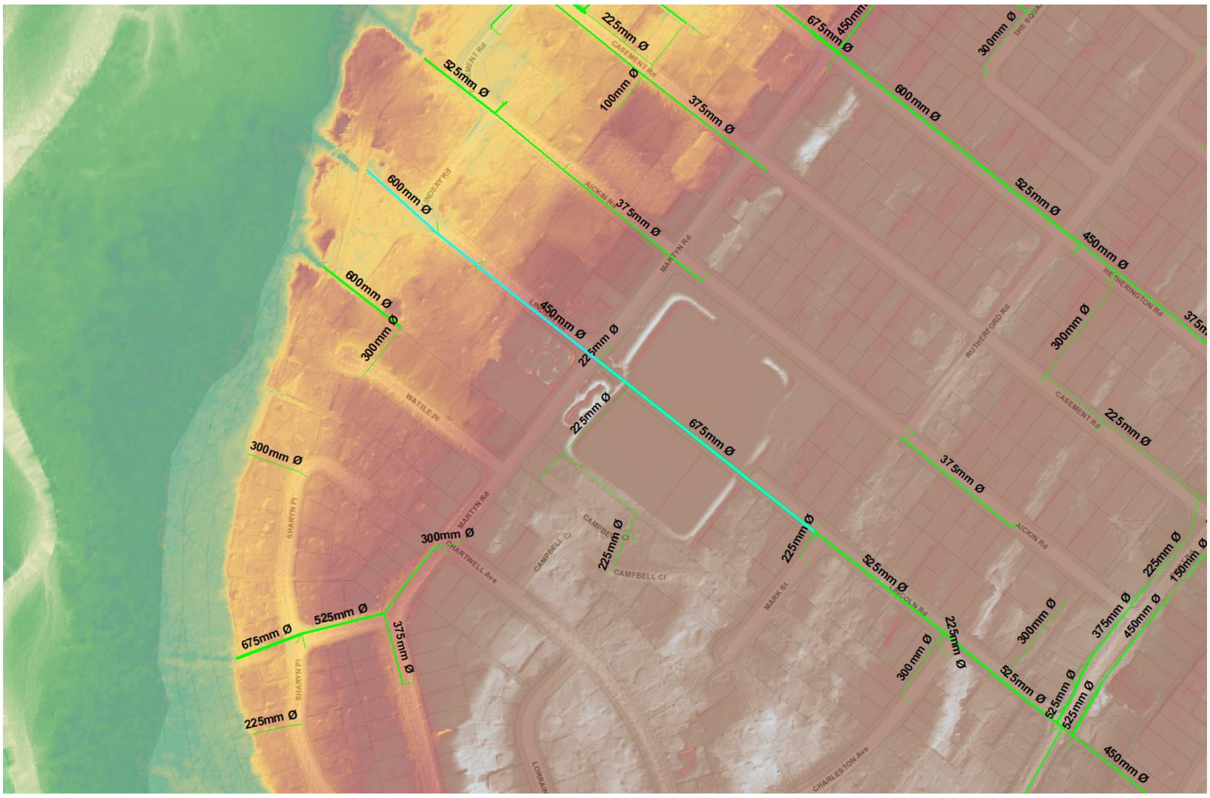

Figure 5-6 Main stormwater drain under rugby field

Note that the pipe diameter reduces in size from 675mm down to 450mm at Martyn Road (i.e. northwest of the rugby field). The diameter of the outfall has been confirmed by survey.

Based on the information available, the historic 1D model assumed that the 450mm dia and the 675mm dia are all 600mm dia in line with the outfall, as this would be the most logical from a stormwater design perspective (i.e. no reduction in pipe dimensions going downstream). However, it is very well possible that the GIS data is correct.

A sensitivity run has been done to assess the impact of modelling the system as per GIS asset data. The sensitivity run is based on the 6-hour MPD 100-year (incl. climate change) scenario. The variance in flood levels within the catchment is shown in Figure 5-7 below (and Figure F4 in Appendix C).

The results show an increase in flood levels upstream (southeast) of the rugby field up to 20mm. Near the outfall the flood levels are reduced by up the 10mm.

Figure 5-7 Sensitivity Run – Inconsistent GIS data rugby field

5.7 Elevated flood levels Wentworth River

The Wentworth River (2400 ha catchment) flows to the west of the township and discharges into the Whangamata Harbour north of the township. Under heavy rain conditions, elevated water levels in the river may affect the stormwater runoff in low-lying catchments discharging into the river. The baseline model adopts a static tail water level at all stormwater outlets set to the MHWS (Mean High Water Spring) level (Refer Section 4.5.2 above). An indicative assessment has been undertaken to set more realistic tail water level conditions and its effect on flood risk within the township.

A separate 2D model has been set up to model the Wentworth River from approximately 1.5km upstream of the SH25 bridge down to the sea (approximately 300m south of the Port Road jetty). The modelled flood levels near the locations of outfalls have been adopted as boundary condition within the catchment model. The sensitivity to flood levels within the township has been visualised.

Assumptions:

- Design flows have been derived from NIWA New Zealand River Flood Statistics (refer Table below). The adopted flows are based on the Flood Frequency Method. To allow for the impact of climate change, the design flows have been increased by 17% representing 2.1°C temperature rise.

Table 5-5 Wentworth River Design Flows

River Flow Probability	Peak Flow Existing Climate	Peak Flow incl Climate Change
MAF (Mean Annual Flood)	63 m^3/s	74 m^3/s
ARI 10 year	119 m^3/s	139 m^3/s
ARI 100 year	188 m^3/s	220 m^3/s

- Peak river flood levels have been computed using a DEM of the river based on LiDAR data. This does not represent the underwater flow area and is therefore considered conservative.
- Riverbed roughness has been assumed 0.045 based on WRC TR-20-07 Waikato Stormwater Management Guideline – May 2020 Table 14.1 (Minor Streams, irregular section, with pools, slight channel meander).
- The modelled peak flood levels in the Wentworth River has been adopted as a constant tailwater level for outfalls discharging along the Wentworth River.
- Wentworth River levels have been modelled with a constant MHWS (Mean High Water Spring) level at the coast.
- As the response time of the Wentworth River catchment (2400ha) is considered much longer than the Whangamata township catchment (500ha) the following joint probability assumptions have been made:

Table 5-6 Joint probability scenarios

Probability Event	Rainfall Probability	Wentworth River Level Probability
ARI 2 year	ARI 2 year	MHWS level
ARI 10 year	ARI 10 year	Mean Annual Flood
ARI 100 year	ARI 100 year	ARI 10 year

For the sensitivity test the 6-hour MPD 100-year (incl. climate change) scenario was run with the modelled Wentworth River flood levels as boundary condition for the outfalls. The variance in flood levels within the catchment is in two areas (Refer Figure F5 in Appendix C):

- North (Refer Figure 5-8 below):
  From Mako Road near the marina up to Sharyn Place / Mayfair Avenue with increases up to 40mm at the industrial area around Lindsay Road.
- South (Refer Figure 5-9 Figure 5-8 below):
  An increase of up to 30mm around the intersection of Waihi Whangamata Road with Hilton Drive and Achilles Avenue.

Figure 5-8 Sensitivity Run – Elevated flood levels Wentworth River (north)

Although the variances in flood levels are small (i.e. up to 40mm) it is proposed to include the elevated Wentworth River levels as boundary conditions in the baseline model.

Figure 5-9 Sensitivity Run – Elevated flood levels Wentworth River (south)

6 SYSTEM PERFORMANCE ASSESSMENT

The model has been run for the scenarios and design storm events listed in Table 6-1 below. Flood inundation maps have been prepared for the MPD scenario with ARI 10yr and 100yr 24 hour design storm event (refer Figure E1-E16 in Appendix C). Note that these maps represent the computed flood levels and do not include freeboard to allow for:

- physical processes that may not have been allowed for
- uncertainties in the precision of the hydraulic modelling
- uncertainties in the prediction of physical processes.

Table 6-1 Model simulation matrix

Simulation	Figure	Land Use	Design Storm Event	Boundary	
				Rainfall	Tide Level (m RL)
1	n.a.	ED	2-year	ARI 2yr	1.10
2	n.a.	ED	10-year	ARI 10yr	1.10
3	n.a.	ED	100-year	ARI 100yr	1.10
4	n.a.	MPD	2-year	ARI 2yr + CC	2.10
5	E1-E16	MPD	10-year	ARI 10yr + CC	2.10
6	E1-E16	MPD	100-year	ARI 100yr + CC	2.10

7 FINDINGS AND RECOMMENDATIONS

7.1 Summary of findings

The findings from this study include:

- A hydrologic and hydraulic model has been developed of the Whangamata township and northern urban areas. This model has been used to complete a dynamic assessment of design rain storms for 2, 10 & 100yr ARIs for existing development (current climate conditions) and maximum probable development (including climate change allowances).
- The reported flows and levels are estimates based on numerous uncertainties which affect the confidence in this estimation such as soil infiltration rates, LiDAR data, rainfall, tide levels, dynamic blockages due debris and vegetation, localised obstructions, and so on. As such these estimates should be treated as indicative for the purposes of determining flood levels, however the model can be utilized to assess the relative effects of potential option upgrades.
- Validation activities for this model has found that:
    - Te Weiti and Waikiekie culverts are well represented in the model
    - Excluding individual catchpits from the model is acceptable
    - Lowering the Williamson Road Pond overflow level provides limited benefits
    - The flood maps in this report are based on simulation of the 24hr nested design storm event. For analysing flood mitigation options, 12hr simulation runs are acceptable.
    - The impact of elevated flood levels in the Wentworth River are small, but have been included in the model
- The Whangamata township is a flat low-lying catchment heavily relying on soakage infiltration for stormwater runoff. Public constructed soakholes are not included in the model (except for Otahu Road infiltration system and pump storage system) due to lack of information on these soakage systems. It is expected that there are more constructed public soakage systems, which could have a significant impact on modelled flood levels.
- The model predicts flooding in Whangamata township under both existing and maximum probable development scenario is widespread over much of the township.
- Predicted ponding during heavy rainfall events is a normal occurrence and provides a fair volume of flood storage. However, it causes frequent nuisance flooding along many roads in the catchment especially in the areas lacking piped reticulation.
- Urban development and intensification increase rainfall runoff and reduces infiltration capacity which increases the risk of flooding.
- Reticulated drainage has limited application due to flat slopes and potential backwater affects particularly when sea level rise is considered.
- Properties at the northern end of the township (near the marina) with ground levels of approximately 1.5-2.0m above MSL are at risk of coastal inundation and particularly when sea level rise is included.
- Flood maps are presented in Appendix C for the 10yr and 100yr 24hr design storm event under MPD conditions.

7.2 Recommendations

The recommendations of this study are to:

- To improve the quality of the model and modelling results the following is recommended:
  - Identification and survey (if possible) of public soakage systems to better assess flood storage volume and soakage rates of these systems
  - Survey of floor levels in critical areas to allow better estimates of current flood risk and quantification of flood mitigation benefits
- Set minimum recommended building levels to ensure that new buildings and building extensions are constructed at a safe level to minimise risk of habitable floor flooding. . It is recommended to apply a minimum freeboard to finished floor level of 300mm. A freeboard of 500mm could be considered along confined waterways and overland flow paths (i.e. non-flat catchment areas).
- Maximise ground infiltration by:
  - installing swales along the roads with designed infiltration trenches including prevention of siltation.
  - Requesting new developments to include soakage systems suitable to discharge runoff from a minimum 24hr 10yr ARI design storm including climate change allowance. Such system must include a well-designed filter systems to prevent siltation and blockage
  - Implement a soakage maintenance plan for all private and public soakage systems
- Maintain a record of all soakage systems including a maintenance database
- Investigate and model stormwater upgrade options to reduce flooding

8 References

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ARC 1999	TP108 Guidelines for stormwater runoff modelling in the Auckland Region, Technical Procedure no. 108, Auckland Regional Council, April 1999
ARR 2012	Australian Rainfall Runoff – Project 15 Two Dimensional Modelling in Urban and Rural Floodplains – Nov 2012.
MBIE-2016	New Zealand Building Code - E1 Surface Water, Ministry of Business, Innovation and Employment, 2016
McKerchar-1989	Flood Frequency in New Zealand, Publication No. 20, Hydrology Centre, McKerchar and Pearson, 1989
MfE 2008	Climate change effects and impacts assessment: A guidance manual for local government in New Zealand, 2nd Edition, Ministry for the Environment, May 2008
MfE 2017	Coastal Hazards and Climate Change, Ministry for the Environment, Dec 2017
MfE 2018	Climate Change Projections for New Zealand, Atmospheric projections based on simulations undertaken for the IPCC 5th Assessment, 2nd Edition, Ministry for the Environment Sep 2018
NIWA 2018	High Intensity Rainfall Design System version 4 (HIRDSv4), NIWA, 2018, https://hirds.niwa.co.nz/
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Thames 2012	Otahu Road stormwater catchment upgrade, Project Completion Report, Thames Civil Engineering Ltd, Aug 2012
NRCS 1986	TR-55 Urban Hydrology for Small Watersheds, Technical Release 55, United States Department of Agriculture, National Resources Conservation Services, June 1986
WRC	Coastal Inundation Tool, Waikato Regional Council, https://www.waikatoregion.govt.nz/services/regional-services/regional-hazards-and-emergency-management/coastal-hazards/coastal-flooding/coastal-inundation-tool/
WRC 2018	Waikato stormwater runoff modelling guideline, Technical Report 2018/2, Waikato Regional Council, June 2018
WSP|OPUS 2019	TCDC Williamson Park Whangamata, Stormwater Outlet Duplication Project, Design Drawings, WSP|OPUS, June 2019

APPENDIX A

Hydrological Parameters

Hydrological Parameters for each catchment.

Zone ID[1]	DP Description	2D Zone	HSG[2]	CN	Perc Imp Area - ED			Perc Imp Area - MPD[3]		
					Roof Soakage	Other Imp Area	Perv Area	Roof Soakage	Other Imp Area	Perv Area
1	Open Space Zone	Township	A	49	0%	0%	100%	0%	0%	100%
2	Recreation Passive Zone	Township	A	49	5%	16%	79%	5%	16%	79%
3	Recreation Passive Zone	Township	A	49	0%	0%	100%	0%	15%	85%
4	Recreation Passive Zone	Township	A	49	0%	6%	94%	0%	15%	85%
5	Recreation Passive Zone	Township	A	49	0%	0%	100%	0%	15%	85%
6	Recreation Passive Zone	Township	A	49	0%	0%	100%	0%	15%	85%
7	Recreation Active Zone	Township	A	49	2%	0%	98%	0%	15%	85%
8	Recreation Passive Zone	Township	A	49	0%	2%	98%	0%	15%	85%
9	Recreation Active Zone	Township	A	49	10%	20%	70%	10%	20%	70%
10	Recreation Active Zone	Township	A	49	1%	5%	94%	0%	15%	85%
11	Residential Zone	Township	B*	69	17%	23%	60%	33%	17%	50%
12	Residential Zone	Township	B*	69	17%	23%	60%	33%	17%	50%
13	Extra Density Residential Zone	Township	B*	69	18%	24%	58%	33%	27%	40%
14	Commercial Zone	Township	B*	69	10%	65%	25%	0%	80%	20%
15	Open Space Zone	Township	B	69	0%	0%	100%	0%	0%	100%
16	Recreation Passive Zone	Township	B	69	1%	0%	99%	0%	15%	85%
17	Recreation Passive Zone	Township	B	69	0%	0%	100%	0%	15%	85%
18	Recreation Passive Zone	Township	B	69	0%	23%	77%	0%	23%	77%
19	Recreation Passive Zone	Township	B	69	0%	8%	92%	0%	15%	85%
20	Recreation Passive Zone	Township	B	69	0%	0%	100%	0%	15%	85%
21	Recreation Passive Zone	Township	B	69	0%	0%	100%	0%	15%	85%
22	Recreation Active Zone	Township	B	69	2%	4%	94%	0%	15%	85%
23	Recreation Active Zone	Township	B	69	1%	5%	94%	0%	15%	85%
24	Residential Zone	Township	C*	79	17%	23%	60%	33%	17%	50%
25	Residential Zone	Township	C*	79	17%	23%	60%	33%	17%	50%
26	Extra Density Residential Zone	Township	C*	79	5%	12%	83%	28%	32%	40%
27	Extra Density Residential Zone	Township	C*	79	31%	22%	47%	40%	20%	40%
28	Marine Service Zone	Township	C*	79	2%	52%	46%	20%	40%	40%
29	Industrial Zone	Township	C*	79	0%	70%	30%	0%	70%	30%
30	Recreation Passive Zone	Township	D	84	0%	0%	100%	0%	15%	85%
31	Recreation Active Zone	Township	D	84	0%	0%	100%	0%	15%	85%
32	Residential Zone	Township	D	84	0%	40%	60%	0%	50%	50%

[1] Refer Figure C1 & C2 Appendix C for location of respective zones

[2] HSG = Hydraulic Soil Group

[3] Highlighted %Imp Area have been adjusted to ensure %Imp Area for MPD is not smaller than ED or Highlighted %Roof Soakage have been adjusted to ensure runoff volume for MPD is not smaller than ED

Zone ID[1]	DP Description	2D Zone	HSG[2]	CN	Perc Imp Area - ED			Perc Imp Area - MPD[3]		
					Roof Soakage	Other Imp Area	Perv Area	Roof Soakage	Other Imp Area	Perv Area
33	Open Space Zone	Urban North	A	49	0%	0%	100%	0%	0%	100%
34	Recreation Passive Zone	Urban North	A	49	0%	15%	85%	0%	15%	85%
35	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
36	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
37	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
38	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
39	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
40	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
41	Recreation Passive Zone	Urban North	A	49	0%	2%	98%	0%	15%	85%
42	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
43	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
44	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
45	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
46	Recreation Passive Zone	Urban North	A	49	0%	0%	100%	0%	15%	85%
47	Low Density Residential Zone	Urban North	B*	69	6%	6%	88%	10%	10%	80%
48	Residential Zone	Urban North	B*	69	17%	23%	60%	33%	17%	50%
49	Rural Lower Zone	Rural Lower	A	30	0%	0%	100%	0%	10%	90%
50	Te Weiti Catchment[1]	Rural Lower	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
51	Waikiekie Catchment[2]	Rural Upper	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a

B*/C* Hydraulic Soil Group classification within Residential and Commercial zones have been increased one level to allow for reduced infiltration due to compaction as per WRC TR201802

[1] Te Weiti Catchment is modelled as a lumped catchment in ICM. See Table 4-5 for hydrological parameters.
[2] Waikiekie Catchment is modelled as a lumped catchment in ICM. See Table 4-5 for hydrological parameters.

APPENDIX B

Site Observations

APPENDIX C

Figures

Figure A	**Catchment Overview**
Figure B	**Depression Areas**
Figure C1	**Impervious Area ED**
Figure C2	**Impervious Area MPD**
Figure D	**Index Map Grid**
Figure E1-16	**Flood Inundation Maps MPD**