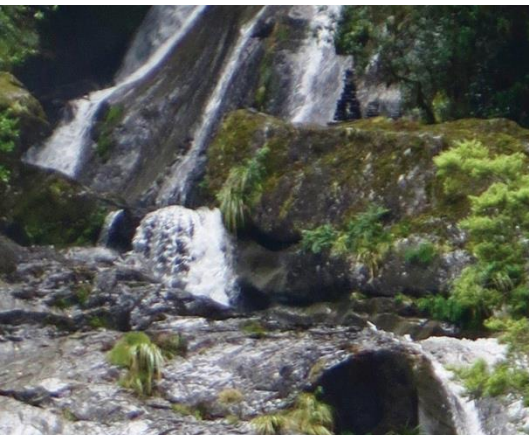






Kāpiti Flood Hazard Management

Methodology Report – Final Draft for Review

1/07/2024



Kāpiti Flood Hazard Management

Methodology Report

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Notation

Term or Abbreviation	Explanation
KCDC	Kāpiti Coast District Council

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1. Introduction

In 2020 Kāpiti Coast District Council (KCDC) commissioned Awa Environmental Limited (AWA) to rebuild their base urban stormwater hydraulic models.

Figure 1 shows the extents of the modelled catchments. Overall seven urban models have been developed under this specification.

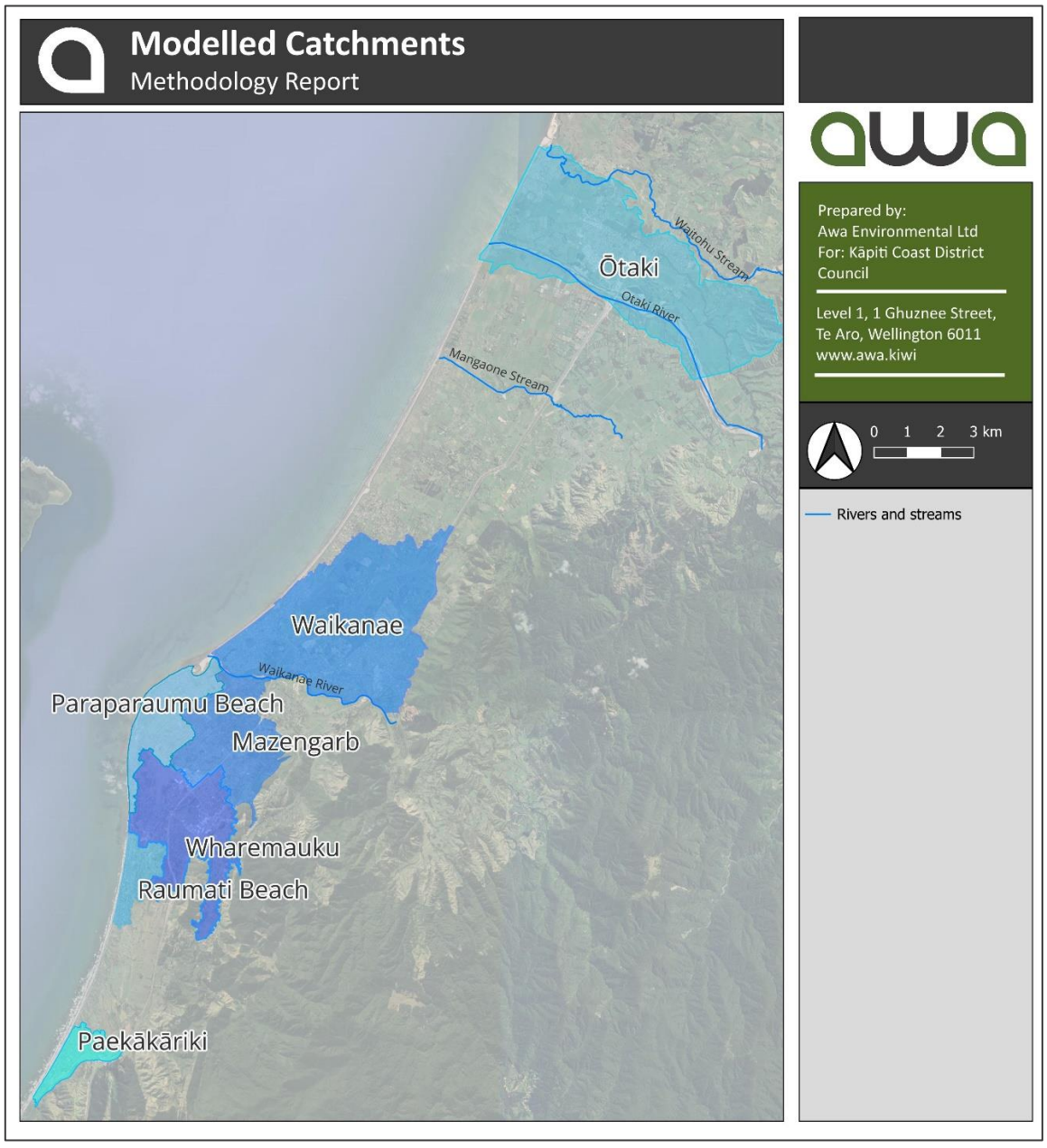


Figure 1. Modelled Catchments in Kāpiti Coast.

1.1. Purpose

The previous generation of models owned by the Council were fit-for-purpose when they were built. Over time however, the demands on their use have changed, giving rise to a number of issues including:

- Long run-times;
- Numerical instabilities leading to simulation termination or unreliable results;
- The models are expensive to update especially the hydrology components;
- A narrow focus in model schematisation, leading to models that cannot be used for multiple purposes;
- Difficulties in interpreting and communicating simulation results resulting in a lack of transparency for end-users;
- A lack of maintenance procedures leading to a loss of model currency.

The model update described in this report has provided the opportunity to thoughtfully resolve these issues as far as practicable. Due to the significant investment updating the models, provision was made for potential future model needs at the model build stage, ensuring maximum value for money from the investment[1]. This process has ensured the Council was able to address a number of practical considerations including:

- A focus on faster model run-times;
- Providing a straightforward hydrological tool to streamline future model use;
- Shifting from adopting static groundwater levels as initial conditions to a dynamic approach to groundwater interactions, which better represents the relationship between flooding and groundwater, which is critical in the Kāpiti District;
- Updating allowances for climate change to facilitate resilience planning, including consideration for district-specific Vertical Land Movement (VLM).
- Making the models more accessible to users, with a clear platform and structure to ensure proper use.
- Incorporating the new State Highway 1 into the models.
- Updating planning for future growth post commissioning of Transmission Gully.
- Addressing updated standards for the management of flood risk including Greater Wellington Regional Council's new standard[2] and the IPCC's guidance[3].

1.2. Model Build Specification

KCDC provided a technical specification within the tender documents, and this is attached in APPENDIX A. While the technical specification document provides guidance on the model build approach, it does not contain a prescriptive model build methodology.

Following the review of model requirements through the early stages of the project a detailed methodology was developed to meet Council's needs. This has included incorporating the work being undertaken on modelling of coastal climate change impacts[4].

[1] KCDC Flood Hazard Management Plan (pending)
[2] Greater Wellington Regional Council, 2021. Flood Hazard Modelling Standard.
[3] Intergovernmental Panel for Climate Change, 2022. Sixth Assessment Report Impacts Adaptation and Vulnerability.
[4] Kāpiti Coast District Council, 2022. Kāpiti Coast Coastal Hazards Susceptibility and Vulnerability Assessment Volume 2: Results.

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As part of the initial tender the modelling platform was specified as TUFLOW, a transition from the historic models which were built using DHI's MIKE software suite. BMT, the developers of TUFLOW, were engaged as peer reviewers for the model build.

1.3. Report Scope

This report describes the overall process of building, calibrating, and validating the seven catchment models as outlined in Figure 2. Also captured in this report is an assessment of boundary condition joint probabilities, including a joint probability analysis of rainfall and tidal storm surges, and the curation of flood hazard outputs.

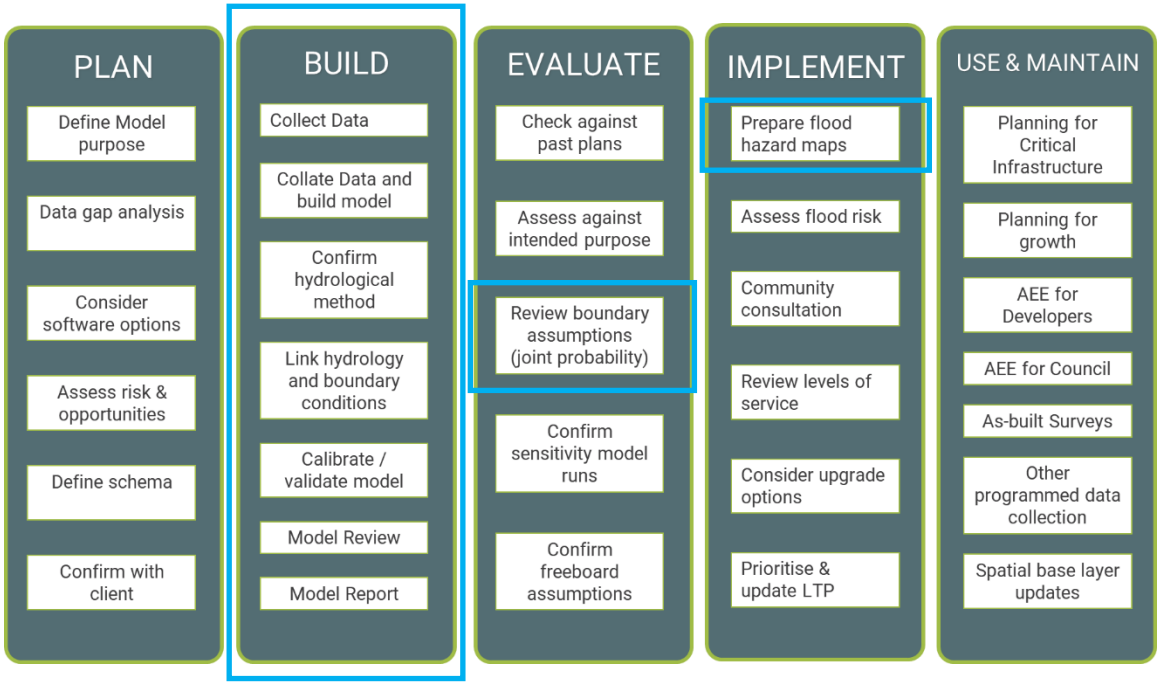


Figure 2. Scope of this report in relation to best practice flood modelling framework.

1.4. Report Outline

This report describes the methodology adopted for the model builds, covering:

- the key data inputs (section 2),
- base data sets (section 3),
- the hydrological modelling approach (section 4),
- allowances made for climate change (section 5),
- consideration of joint probability between rainfall, upstream catchment inflows and tides (section 6),
- hydraulic modelling methods (section 7),
- The processes of model calibration and validation (section 8),
- A description of the modelling outputs (section 9), and

It provides an overarching view of the modelling approach that guided the build process for the seven catchment models. Detailed descriptions of each of the respective model builds are provided in accompanying model build reports.

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2. Modelling Platform

The seven catchment flood models were built using the TUFLOW modelling platform. TUFLOW is a suite of advanced 1D/2D/3D computer simulation software for flooding, urban drainage, coastal hydraulics, sediment transport, particle tracking and water quality. TUFLOW was developed by BMT Commercial Australia Pty Ltd. from Brisbane Australia.

The flood models were built using TUFLOW 2023-03-AB Classic/HPC release, the most recent version at the time, as recommended by TUFLOW. The TUFLOW software offers a lower cost alternative to the MIKE software package that the KCDC flood models were previously built with. It also provides more flexibility for creating custom integrated tools for modellers, easy ways to access the results (through QGIS) can be accessed in 12D (engineering software), as well as faster and more stable run times.

TUFLOW has hardware requirements to run flood models. It can only run on NVIDIA GPUs[5] and requires specific CUDA core counts on the GPU depending on the model size. For example, the Ōtaki catchment model is recommended to have CUDA core count of around 10,400 or greater on the GPU (e.g. RTX 3090 or 4090) to have reasonable run-times (1-2 days).

A brief outline of setting up TUFLOW models is as follows.

- Base data sets are set up (e.g. Digital Elevation Model, pipe network, structures) as spatial layers.
- Material land-use (Manning's n roughness) and soil infiltration (interaction between groundwater and runoff) are added.
- Lastly, boundary conditions such as groundwater levels, rainfall conditions, and tides are added.
- Run-times vary between catchment/model size.

The base data sets are further explained in detail below (see Section 3)

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[5] NVIDIA Graphic Processing Unit.

3. Base Data Sets

Each model is built using 'layers' of spatial data to represent key features of the stormwater catchments being modelled.

3.1. Topographic Layer

Land terrain was defined using a Digital Elevation Model (DEM), a rasterised (cell based) representation of the topography of the District. The DEM was derived from LiDAR data captured for KCDC on the 5th and 9th of February 2017 by AAM Ltd (now Woolpert). KCDC provided the LiDAR data processed into a 2-metre DEM in Wellington Vertical Datum 1953 (WVD53). The technical specification of the LiDAR capture is provided in APPENDIX B.

3.1.1. Topographic Edits

Topographic edits have been made to the DEM to capture significant developments, correctly represent, or accentuate hydraulically significant features. The features edited are summarised in Table 1.

Table 1. Summary of topographic edits

FEATURE	DATA SOURCE	ELEMENTS
Peka Peka to Ōtaki (PP2O) SH1 expressway construction drawings	Construction Level	Polygons/DEM
	Swales	DEM
	Stormwater network	Points, lines, and polygons
	Pond design	Polygons
	Kerb and channels	DEM
	Open channels	Polygons, lines, and DEM
	Overflow paths	DEM
Mackays to Peka Peka (M2PP) SH1 Expressway	Swales	DEM
	Stormwater network	Points, lines, and polygons
	Kerb and channel	DEM
	Open Channels	Polygons, lines, and DEM
	Overflow paths	DEM
Kāpiti Coast District Council	Ponds or Storage Areas	DEM (set 500 mm below outlet level or water level)
	Minor Open Channels	Represented in the DEM and graded consistently from upstream outlet to downstream inlet where survey data does not exist

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FEATURE	DATA SOURCE	ELEMENTS
	Major Open Channels	Cross-section survey data from legacy MIKE FLOOD models. Represented at a 'finer' 0.5 m resolution sub-grid providing a higher level of detail.
	General topographic edits	cut/fill lines, points, and polygons to manipulate DEM

The TUFLOW 2D surface simulation engine allows DEM modifications, in the form of shape files, to be "stacked" allowing the modifications outlined above to be kept separate from the base DEM record and cataloguing modifications in a clear and consistent manner.

3.2. Soil Layer

A district wide spatially varying soil layer has been derived to represent the soil-runoff interaction in the initial/continuing loss in the TUFLOW infiltration model. The soil layer was derived from several data sources including:

- Soil Classification,
- Soil Compaction, and
- Landcover.

Spatial maps were developed and combined to produce a variety of combinations from each soil category.

3.2.1. Soil Classification

The soil classification layer has been derived from the data sources outlined below.

- Land Research Information Systems – Soil Maps (LRIS – SMAP) Soil Classification[6], as shown in Figure 3.
    - SMAP soil texture as shown in Figure 4.
    - SMAP soil drainage as shown in Figure 5.
    - SMAP soil depth as shown in Figure 6.

- LRIS – SMAP Soil Classification[7] - Ōtaki District, as shown in Figure 7.
- New Zealand Fundamental Soil Layers (NZ FSL) Soil Classification[8], as shown in Figure 8.
- An understanding of the various dune phases on the Kāpiti coastal margin[9], as shown in Figure 9.

[6] Landcare Research Manaaki Whenua, 2021. SMAP Soil Classification (Soil Order) Aug 2021.
[7] Landcare Research Manaaki Whenua, 2021. SMAP Soil Texture Aug 2021.
[8] Landcare Research Manaaki Whenua, 2021. SMAP Soil Drainage Aug 2021.
[9] Archaeology of the Wellington Conservancy: Kāpiti-Horowhenua, A prehistoric and palaeoenvironmental study, Bruce McFadgen, 1997

- New Zealand Geotechnical Database (NZGD)[10]

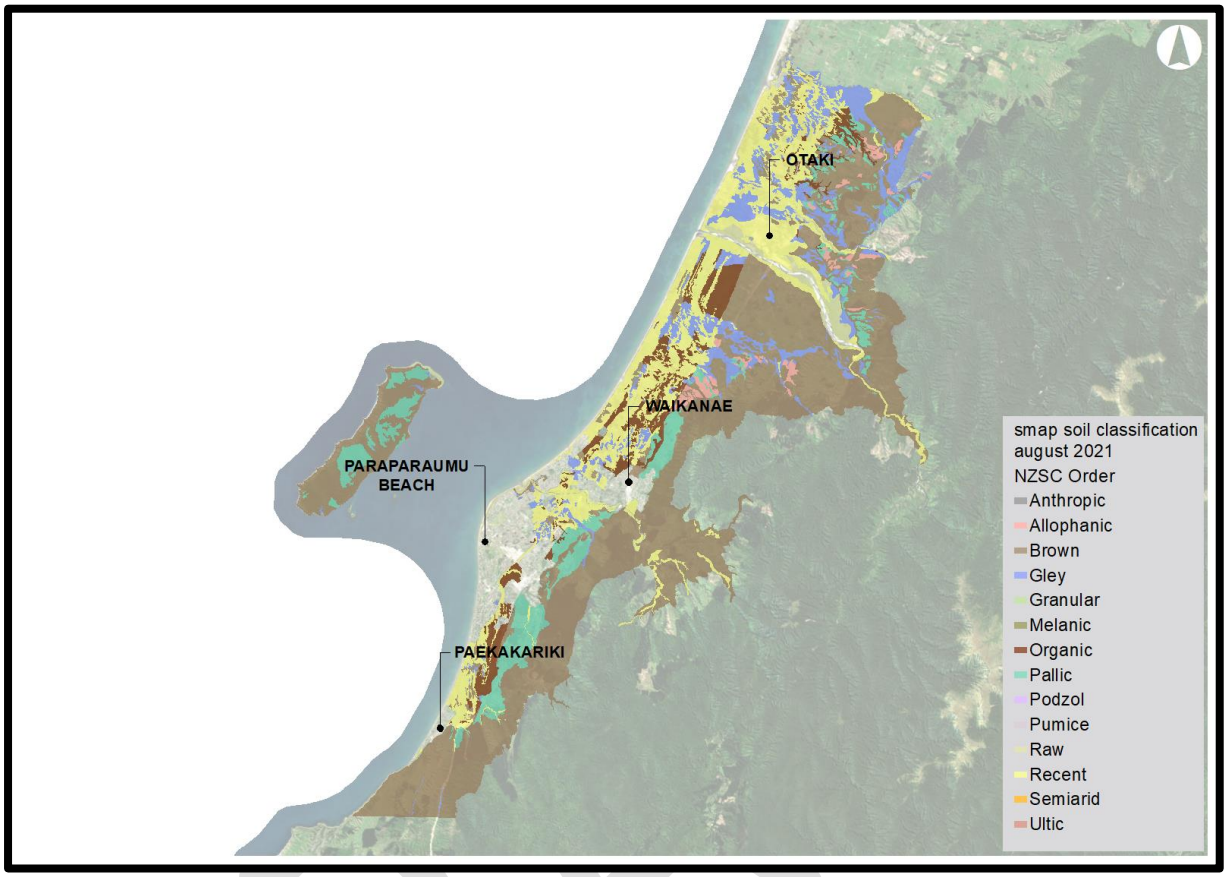


Figure 3. SMAP soil classification (August 2021). (Source: Landcare Research)

[10] The NZGD database is an online database recording geotechnical data collected throughout New Zealand. Bore log data has primarily been used to determine soil type in built-up areas.

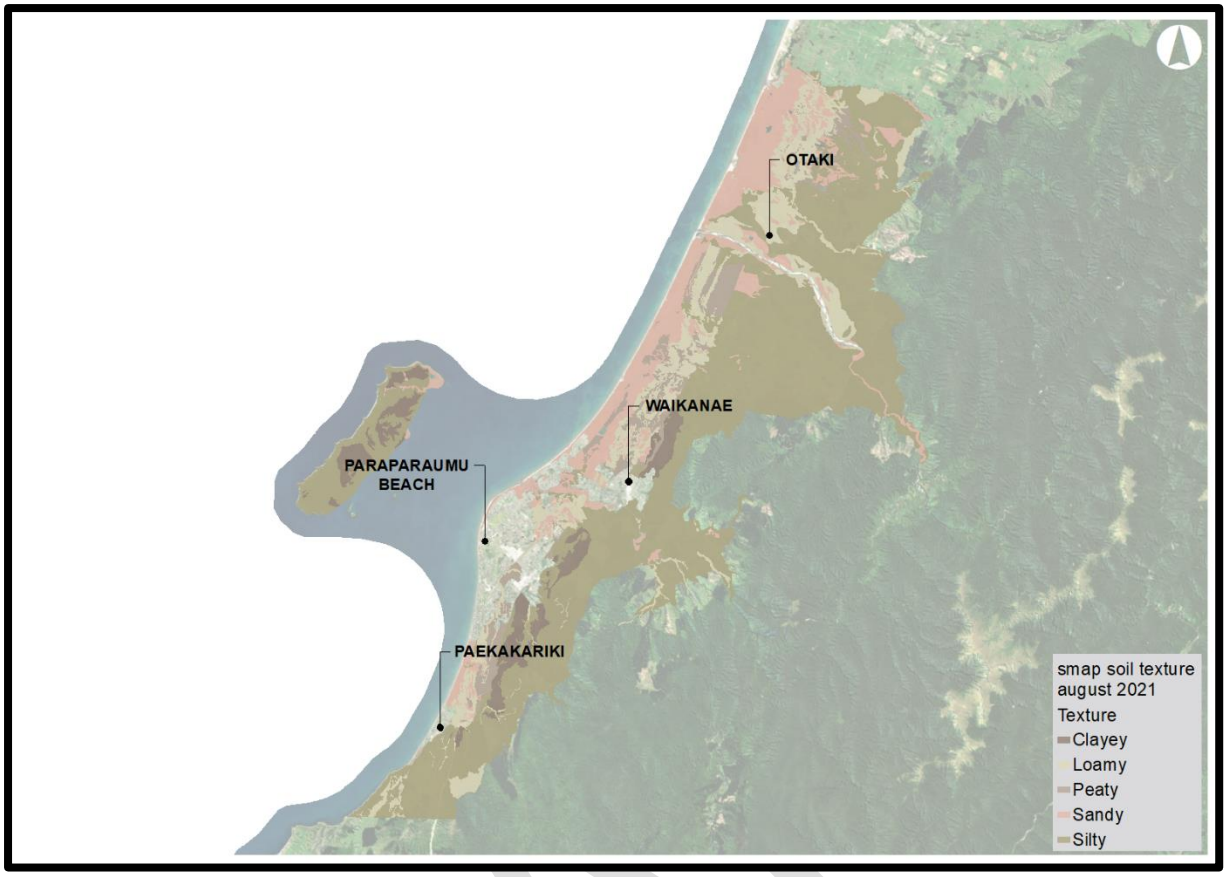


Figure 4. SMAP soil texture (August 2021). (Source: Landcare Research)

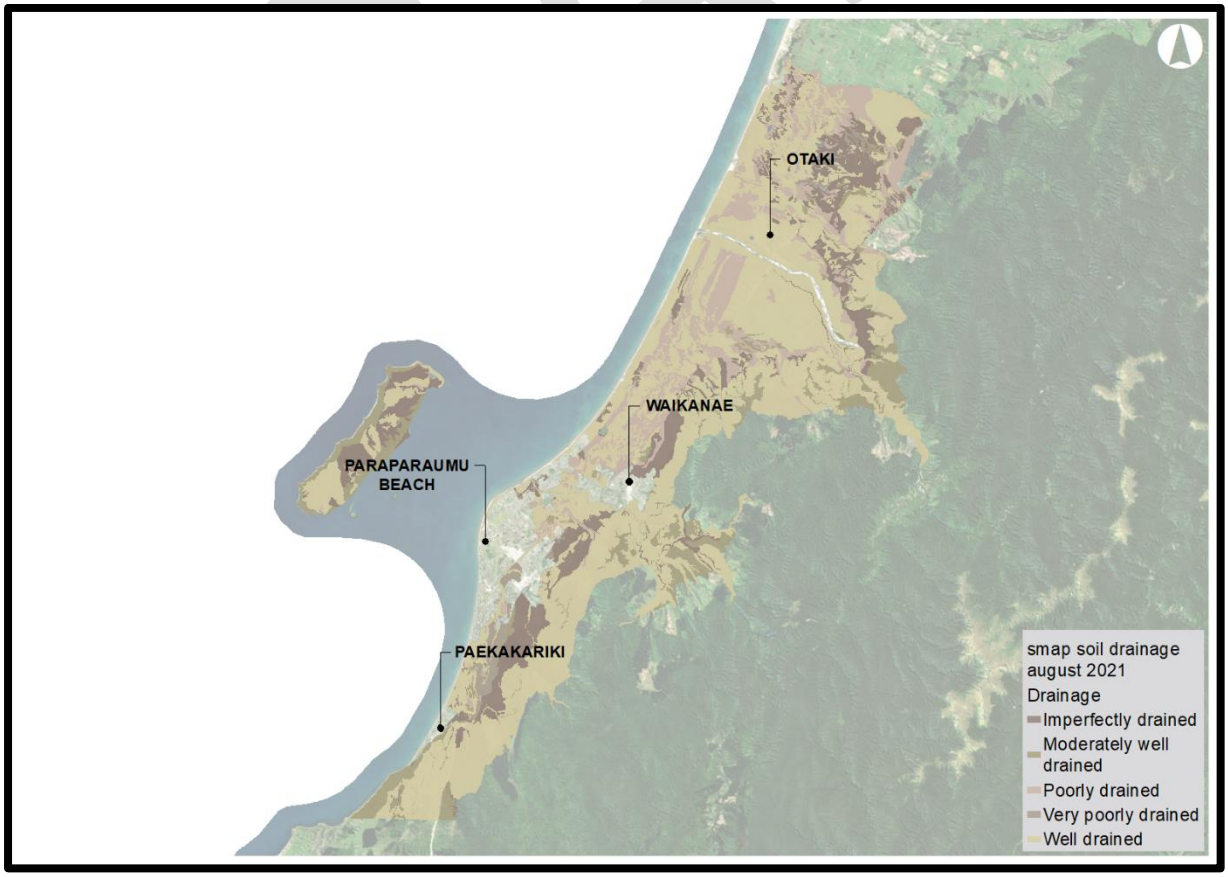


Figure 5. SMAP Soil drainage (August 2021). (Source: Landcare Research)

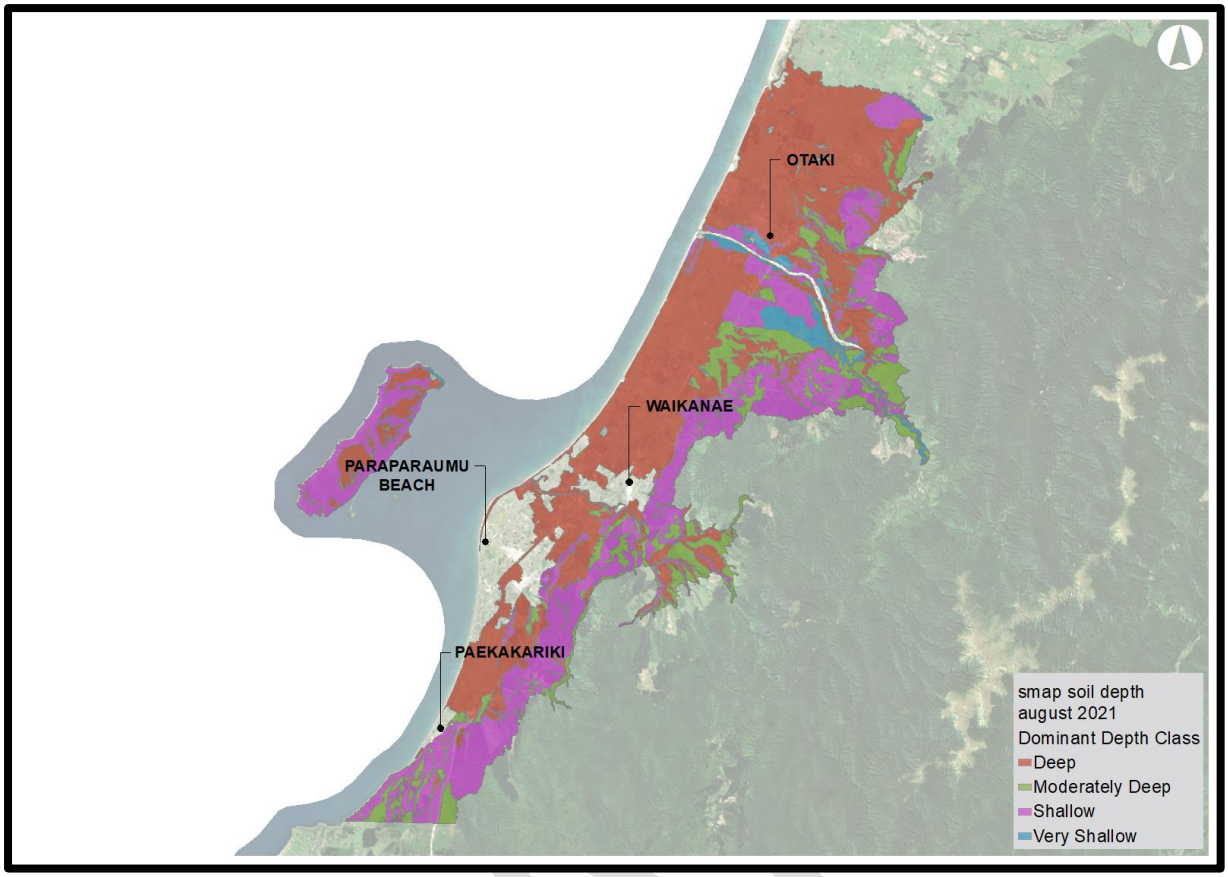


Figure 6. SMAP soil depth (August 2021). (Source: Landcare Research)

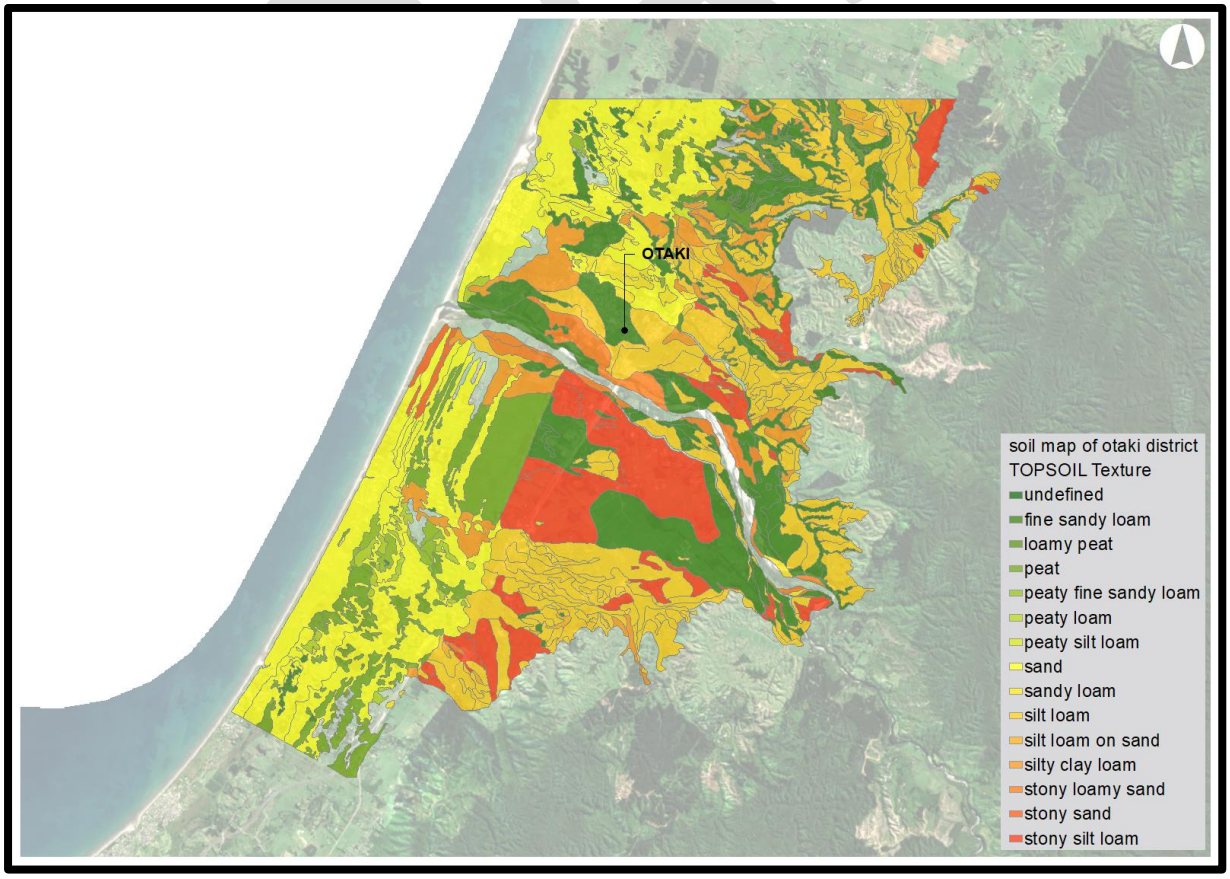


Figure 7. SMAP soil classification - Ōtaki District. (Source: Landcare Research)

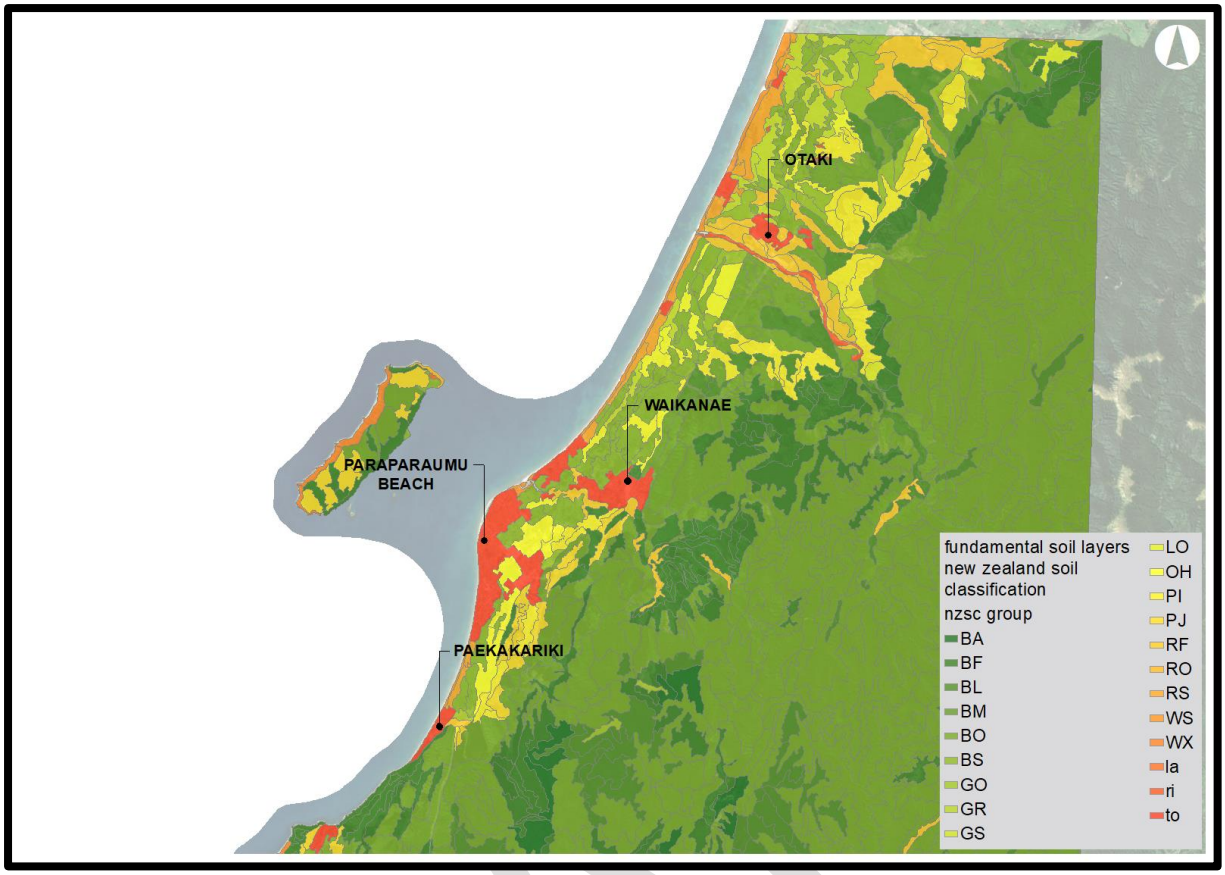


Figure 8. NZ fundamental soil layer soil classification. (Source: Landcare Research)

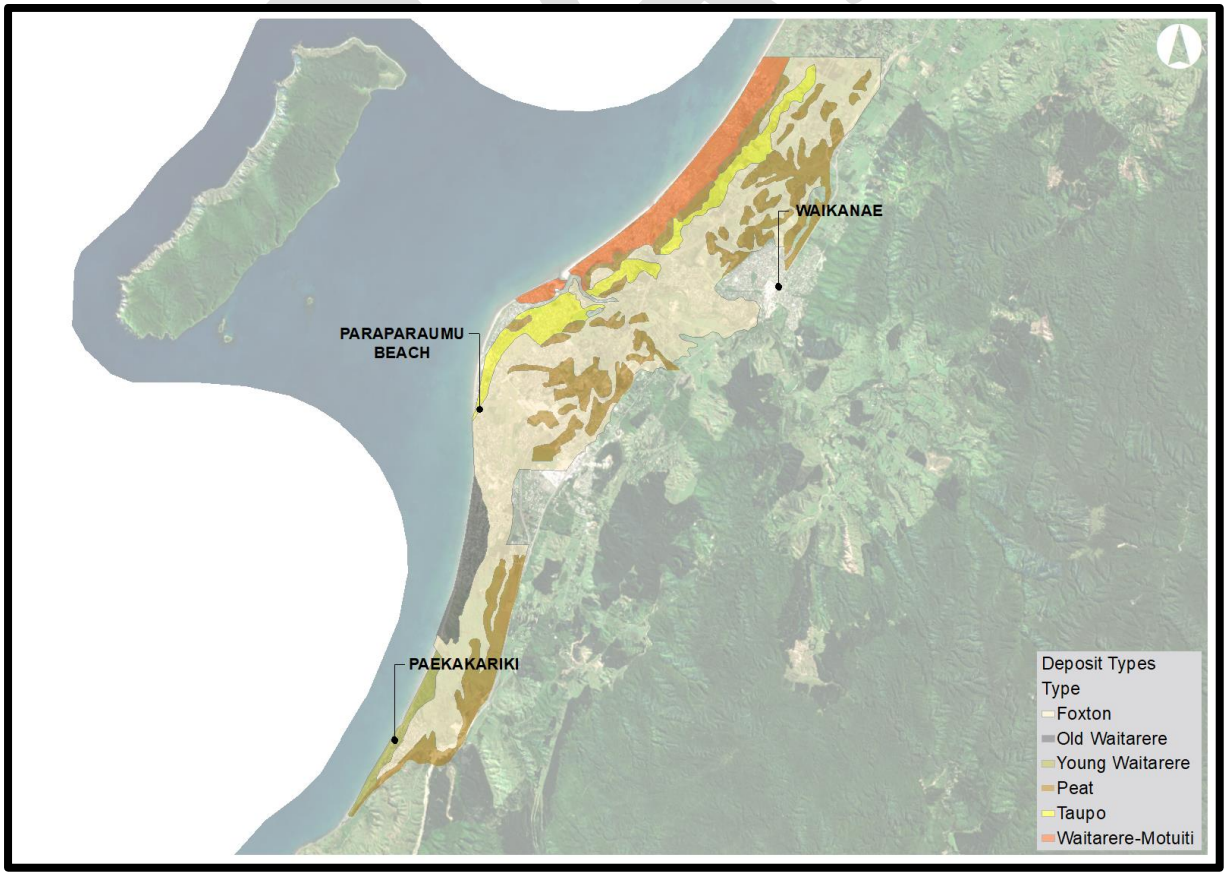


Figure 9. Archaeology of the Wellington Conservancy: Kāpiti-Horowhenua. (Source: McFadgen 1997)

Spatial analysis of the various datasets was undertaken to develop a consolidated soil classification across the district. This classification was distilled into the following six main soil groups defined by Awa, which are also depicted in Figure 10.

- Greywacke
- Landfill
- Loess
- Peat
- River Loam
- Sand

A limitation of both the LRIS – SMAP Soil Classification and NZ FSL Soil classification layers is the lack of soil classification information within built-up areas of the district. Where this occurs, Awa has used the NZGD database, topographic data, dune phase analysis, the results of our infiltration testing (refer to section 3.2.3) and historic knowledge to derive the soil classification.

The derived soils layer will become a base layer for all future modelling. It will be important that this layer is updated with new/additional information over time.

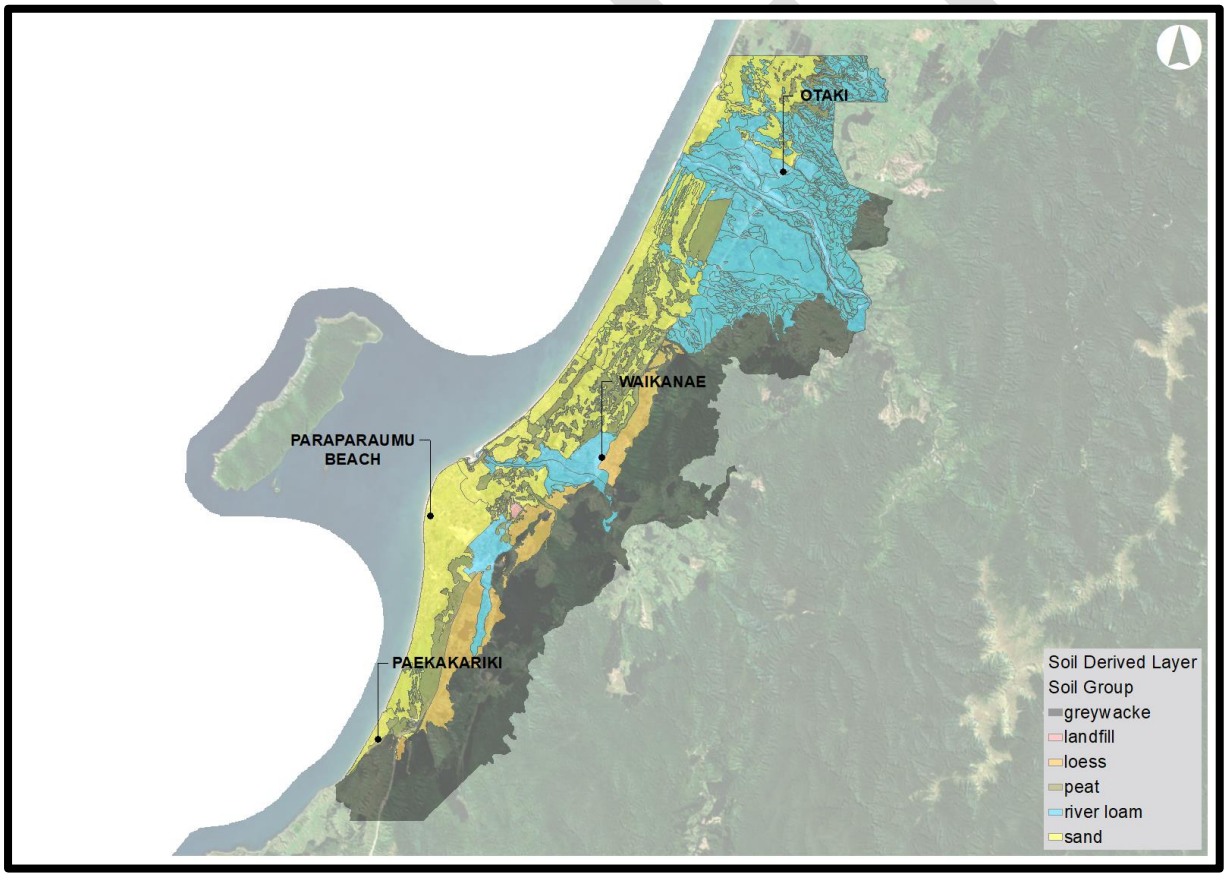


Figure 10. Final derived soil classifications.

3.2.2. Land Cover

District wide land cover identification has been captured from two main sources, depending on location. For built-up areas, a landcover classification prepared by Lynker Analytics using aerial

photography (flown in 2017) was adopted (see APPENDIX C). Elsewhere the Landcare Research Land Cover Database (LCDB)[11] was used.

The LCDB v5 dataset is a national dataset identifying landcover at a coarse resolution. While this is adequate for rural/natural landcover areas, the resolution in built-up areas was deemed inadequate for modelling purposes. Hence, the finer resolution Lynker landcover identification has been used within the built-up areas and the LCDB (v5) identification used outside of the built-up areas.

Landcover identification from Lynker Analytic's analysis of the 2017 aerial photography was classified into eight categories.

- Buildings
- Bushes and Trees
- Grass
- Other Impervious
- Road
- Rough Low Vegetation
- Sand, Gravel, Clay, Bare Earth
- Water

The technical specification of the landcover capture is provided in APPENDIX C.

Landcover identification from the LCDB (v5) dataset returned the 24 landcover categories listed in Table 2.

Table 2. LCDB (5) Landcover categories.

LCDB LANDCOVER CATEGORIES	
Broadleaved Indigenous Hardwoods	Low Producing Grassland
Built-up Area (settlement)	Manuka and/or Kanuka
Deciduous Hardwoods	Matagouri or Grey Scrub
Estuarine Open Water	Mixed Exotic Shrubland
Exotic Forest	Orchard, Vineyard, Perennial Crop
Fernland	River
Forest - Harvested	Sand or Gravel
Gorse and/or Broom	Short-rotation Cropland
Gravel or Rock	Surface Mine or Dump
Herbaceous Freshwater Vegetation	Transport Infrastructure
High Producing Exotic Grassland	Urban Parkland/Open Space
Indigenous Forest	Lake or Pond

A total of 32 distinct landcover categories have been identified. An example of the landcover classification is shown in Figure 11.

Analysis of landcover data should be updated every 5 years in communities with high levels of growth like Kāpiti. This would then be used to update model base layers.

[11] Landcover Database Version 5 (LCDB v5), available for download from lris.scinfo.org.nz, for remaining areas.

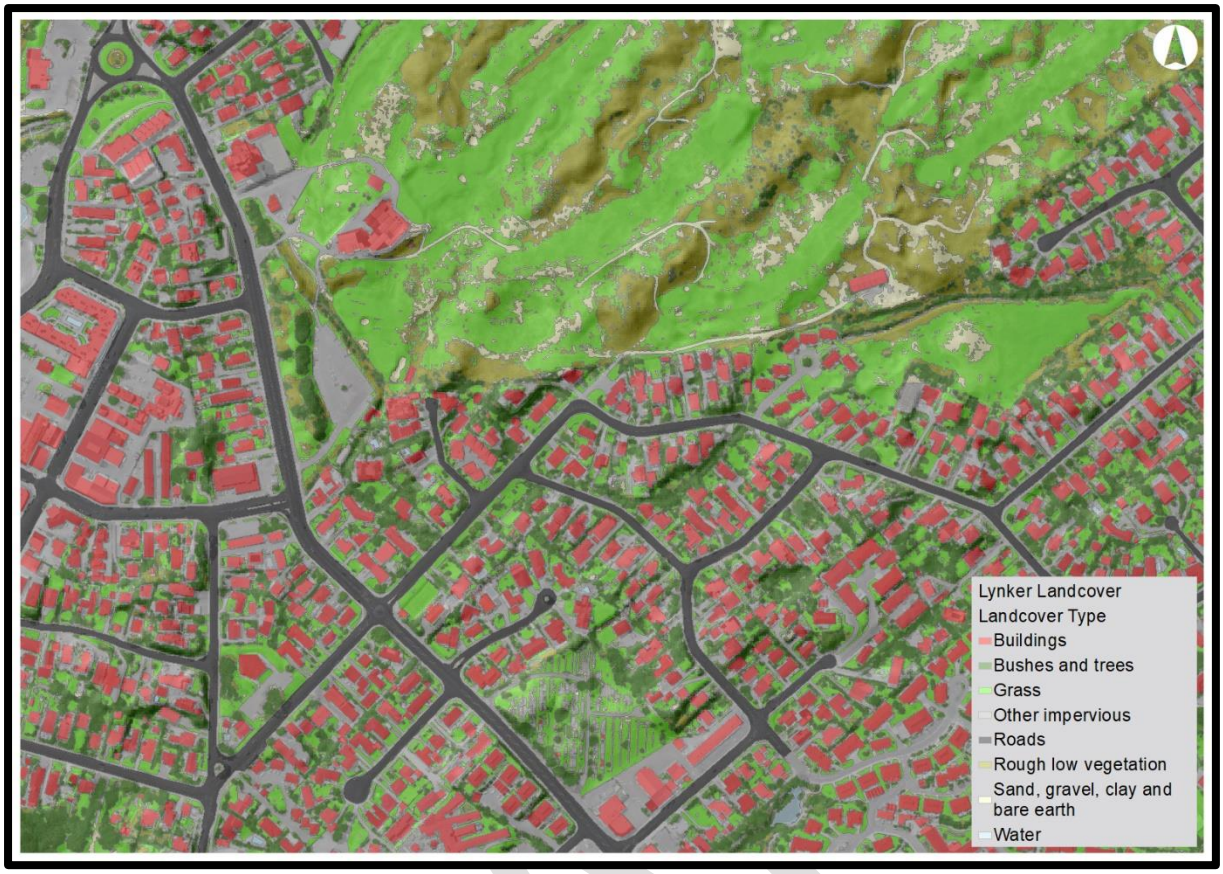


Figure 11. Lynker built-up area landcover identification.

3.2.3. Infiltration

Quantifying infiltration rates within the A-horizon of the soil profile is a critical piece of information for determining hydrologic losses in the model build. Historically, soil infiltration rates were assumed from literature and extrapolated over similar soil types in the Kāpiti Coast district. To obtain more regionally specific data, Awa undertook a comprehensive soil infiltration testing campaign in 2021-2022 (Awa Environmental Ltd. 2022). The study conducted testing across the various soil classifications, compaction states and land-use types to provide a comprehensive set of data.

In addition to confirming infiltration rate components for the model build, the outputs from the study will allow for better assessment of the impacts of future urban intensification and greenfield development in terms of increased run-off rates. The infiltration data collected in the field were distilled down to reflect several different soil types as shown in Figure 12. The full study report is provided in APPENDIX D.

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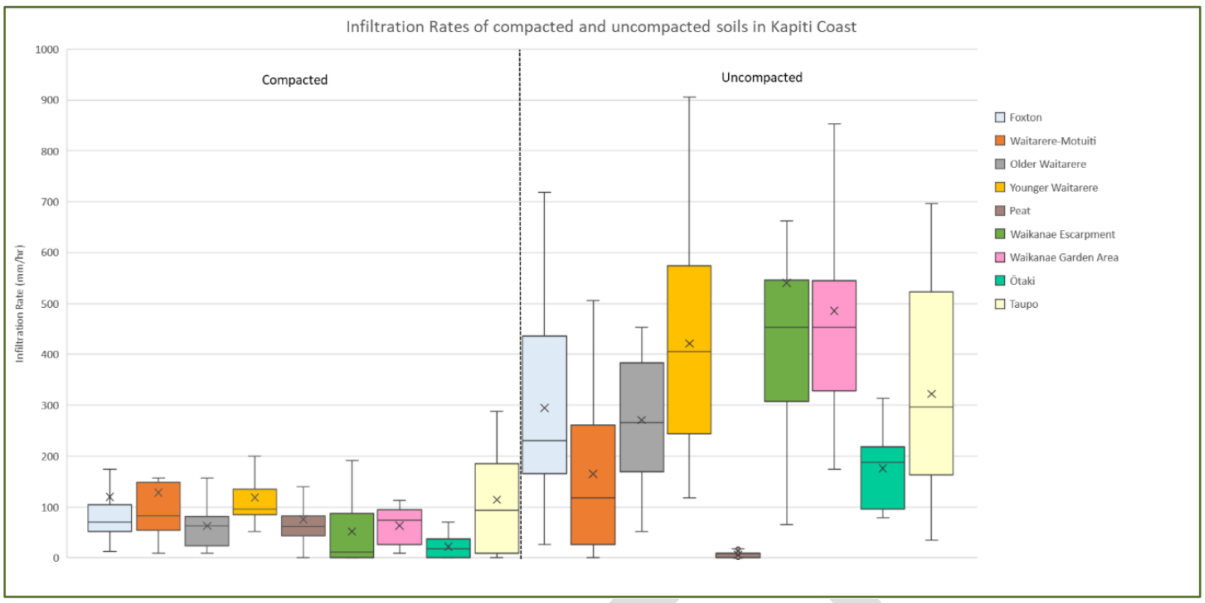


Figure 12. Infiltration test results from Kāpiti Coast District. Source: Awa (2021)

SOIL COMPACTION

The field study undertaken by Awa indicated a correlation between soil compaction and infiltration rates. Uncompacted soils have higher infiltration rates than compacted soils. Compaction also appears to be a more influential factor than dune phase age or composition. To capture this in the model build a soil compaction layer was developed.

The soil compaction layer was derived from Retro-lens historic aerial photography. Analysis of this photography shows a change in development methodology occurring in the late 1970's from pile and ring foundations built onto the dune form to a 'slab-on-grade' construction methodology. The latter construction methodology requires filling and compacting of the land to create building platforms.

Development pre-1977/1978 was thus assumed to be piled and or a ring foundation and development post-1978 assumed to be slab-on-grade. A sensibility check using Google Street View showed there was a mixture of development types in some locations, indicating a transition between development methodologies. More generally, however, this distinction was sound.

Additional filling and compacting were assumed to have occurred within the entire parcel extent of the airport, commercial areas, retirement villages, road and rail corridors, landfills, M2PP and PP2Ō expressway and associated storage areas and sports fields/school fields. The extent of compacted and uncompacted soil layer is shown in Figure 13.

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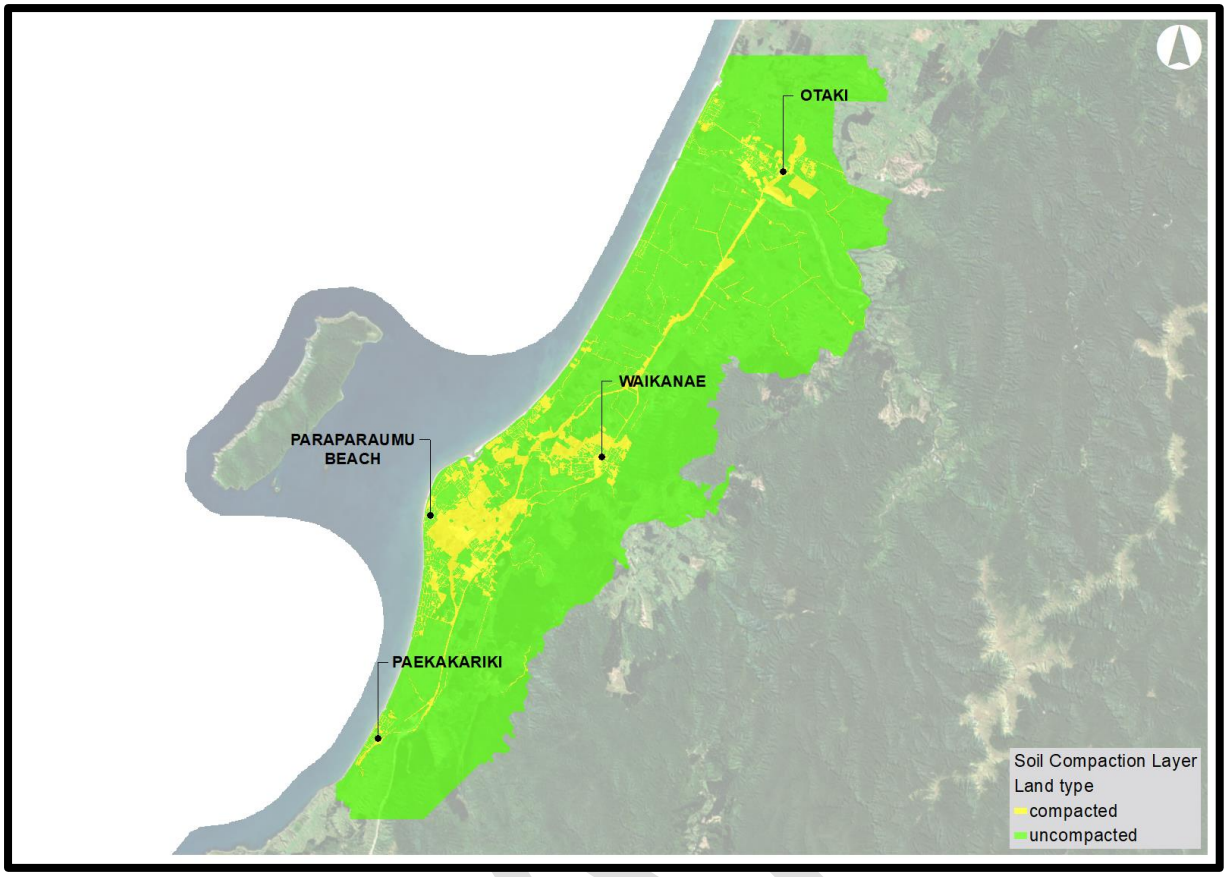


Figure 13. Compacted and uncompacted surfaces.

3.3. Hydrologic Data

3.3.1. Empirical Rainfall

SKM (2011) provides methods and hyetograph information for the 2008 and 2090 climate horizons. Since this work was carried out, two releases of the *High Intensity Rainfall Design System* (HIRDS) developed by NIWA have been made available, one in 2011 (HIRDS v3) and the second in 2018 (HIRDS v4). The most recent update includes analysis of rainfall data up to 2015 (NIWA, 2018a).

HIRDS v4 design rainfall depth estimates have been used for the Kāpiti model build. Figure 14 shows the locations of rainfall gauges used in deriving historical design rainfall depth estimates for HIRDS v4 in Kāpiti.

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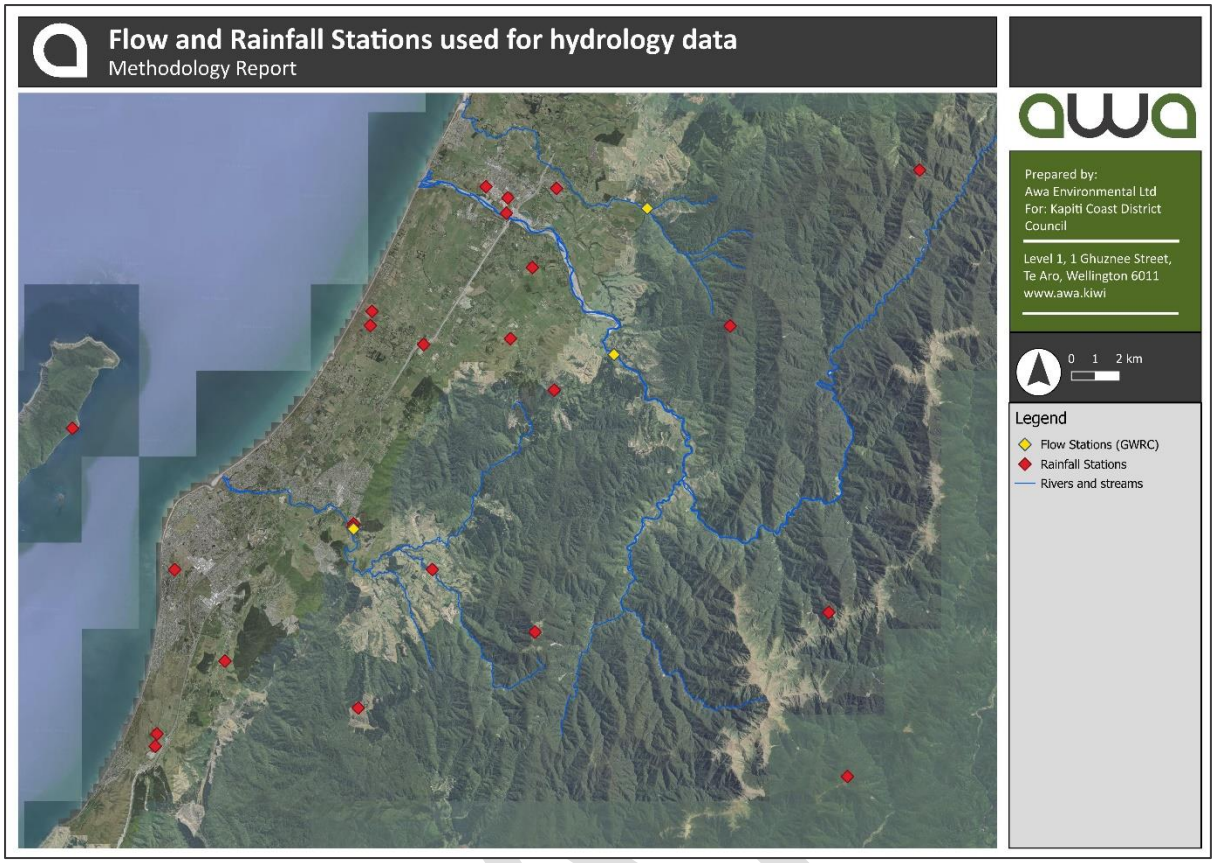


Figure 14. Rainfall gauges used in HIRDS v4 and flow gauges (GWRC) in the vicinity of Kāpiti. Source: NIWA and GWRC.

3.3.2. Flow Gauging

There are two catchments in the base models that utilise river/stream baseflows - the Ōtaki and Waikanae River catchments. GWRC records flows in these catchments at three gauging stations (see Figure 14). The flow data provided the base hydrology for the rivers and streams that were used as boundary conditions for the calibration/validation of the catchment models as per Table 3. For the flood scenarios, data was drawn from Greater Wellington Regional Council river flood models as described in section 4.5.

Table 3. Flow data sources used to inform hydrological boundary conditions.

FLOW SITES	CATCHMENT	SOURCE
Ōtaki River at Pukehinau	Ōtaki	Greater Wellington Regional Council
Waitohu Stream at Water Supply Intake	Ōtaki	Greater Wellington Regional Council
Waikanae River at Water Treatment Plant	Waikanae	Greater Wellington Regional Council

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3.3.3. Groundwater

Groundwater table surfaces were provided by Tonkin & Taylor in 2021 (Gyopari et al 2014) as summarised in Table 4 and depicted in Figure 15. Being based on RCP climate change scenarios, the gridded ground water surfaces were adjusted by an offset or interpolated using changes to sea level (as opposed to time) to give the desired time horizons and a more recent projection of climate change (SSP scenarios). These processes are explained in detail in section 4.7.

Table 4. Groundwater levels provided by Tonkin & Taylor.

CLIMATE CHANGE SCENARIO	GROUNDWATER LEVEL
2030 RCP 8.5M	0.29 mWVD (T&T_GWL_2030)
2130 RCP 8.5M	1.32 mWVD (T&T_GWL_2130_RCP8.5M)
2130 RCP8.5H+	1.66 mWVD (T&T_GWL_2130_RCP8.5H+)

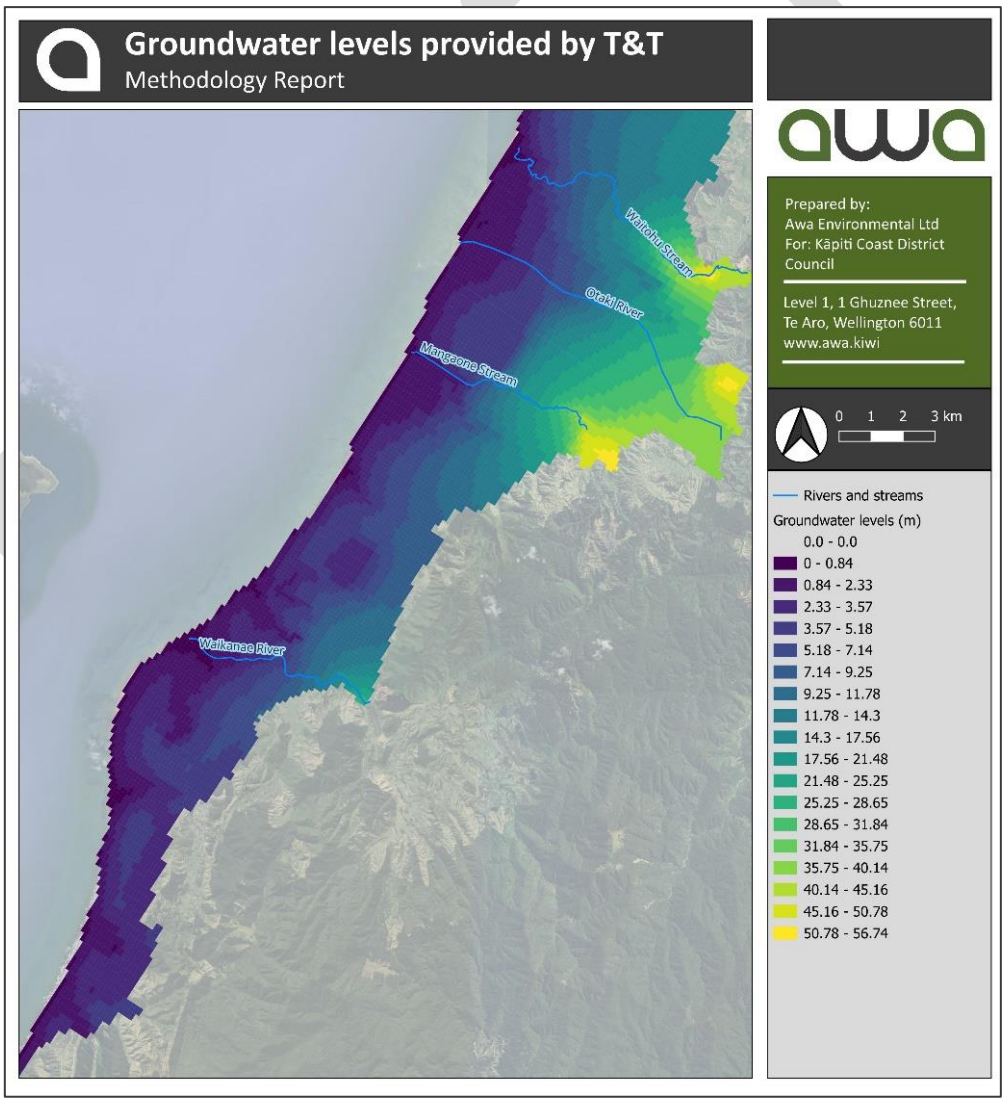


Figure 15. Groundwater levels provided by Tonkin & Taylor.

4. Hydrologic Methodology

4.1. Rainfall Inputs

As explained in section 3.3.1, HIRDS v4 has been used to generate the range of design storms for modelling purposes. Rainfall estimates for 2030 have been used for the baseline model scenarios. While conservative, 2030 has been adopted on the basis that it will take 3-5 years to implement the outcomes of this modelling work and embed funding for mitigating risk.

Future climate change scenario rainfall depths were generated by scaling baseline rainfall depth estimates with augmentation factors taken from NIWA (2018a) and temperature increase (anomaly) estimates from the appropriate RCP scenario and climate horizon.

Figure 16 shows the 1% AEP 6-hour storm design rainfall depth distribution map taken from HIRDS v4 for Kāpiti. The spatial resolution of the grid is 2 km. Note that the rainfall depths have been buffered by one grid cell to completely cover the coastline. This has been carried out as an inverse-distance-weighted average of neighbouring cells. The spatial variation in depth for each duration is accounted for in the rain-on-grid and sub-catchment based hydrological approaches described below.

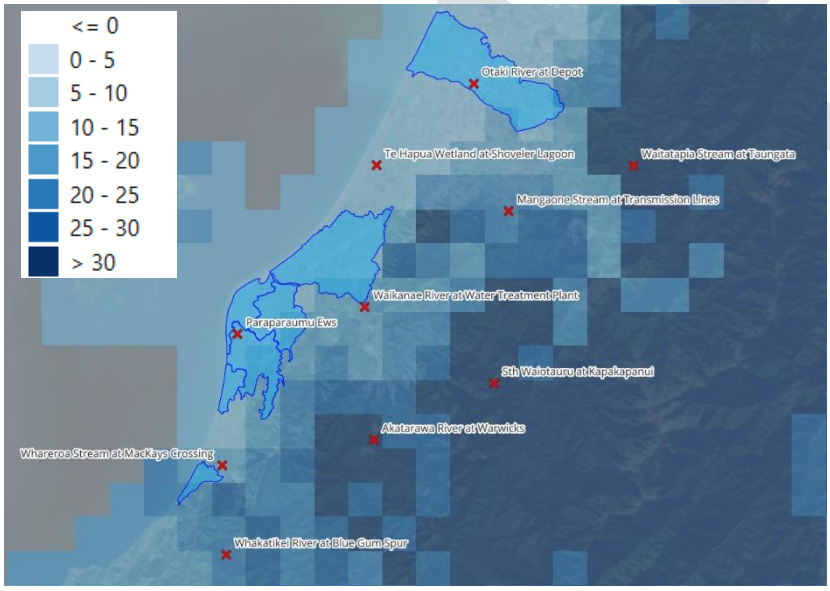


Figure 16. HIRDS v4 design rainfall depth distribution for the 1% AEP 2030 6-hour duration storm event.

4.2. Rainfall-Runoff Methodology

Two approaches were adopted for simulating rainfall-runoff - sub-catchment and rain-on-grid. The basis for selecting and applying the two methods is outlined in the following sections.

4.2.1. Sub-Catchment Based Hydrology

The historic sub-catchment hydrological approach applied in the Kāpiti legacy models is detailed in the report: Updated Isohyet Based Calculations of Design Peak-flows (SKM 2011) included as APPENDIX E. The approach includes the following aspects:

- Sub-catchment-based,
- Soil Conservation Service (SCS) Curve Number loss model,
- Clark Unit Hydrograph routing model, and
- Based on HEC hydrological modelling software.

This approach was developed through calibration of gauged catchments in Kāpiti and the wider Wellington Region (refer to section 8.1). In Kāpiti Coast, there is no calibration data (gauged streams) that represent coastal Holocene catchments. For this reason, the historic calibration reflects hill catchment characteristics and not those of the coastal flood plain.

4.2.2. Rain-on-Grid Based Hydrology

This method includes a distributed hydrological loss model and relies on hydrodynamic simulation to account for routing. In this method, rain "falls" on the ground and runoff is determined by spatially varying infiltration factors representative of Kāpiti's coastal soils.

The key advantage of the rain-on-grid approach to hydrology is the ability to approximate groundwater effects on soil infiltration capacity in a spatially variable manner allowing models to be used in investigations of the effects of elevated groundwater on flooding. More information on Rain-on-Grid methodology is available section 7.4.3 of the TUFLOW manual[12].

4.2.3. Method comparison

A summary of the advantages and disadvantages of the sub-catchment based, and rain-on-grid hydrology approaches, is given in Table 5 below.

[12] BMT 2018. *TUFLOW Classic/HPC User Manual, Build 2018-03-AD*. Software manual. Available at https://downloads.tuflow.com/TUFLOW/Releases/Latest/TUFLOW%20Manual.pdf

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Table 5. Advantages and disadvantages of sub-catchment based and rain-on-grid hydrological models.

HYDROLOGICAL MODELLING APPROACH	ADVANTAGES	DISADVANTAGES
Sub-catchment	• Traditional, familiar approach. • Multiple processes are combined in simple hydrological models, so they are quick to set up. • Sub-catchment models have been calibrated to produce reliable runoff estimates in Kāpiti's hill catchments. • Is computationally light and so simulations are quick. • Requires little input data. • 32-bit floating point precision is sufficient for hydrodynamic simulations: this means faster simulations and lighter computer hardware specifications.	• Developed for hand calculation and so requires judgement. • Parameters do not necessarily align with easily measurable catchment characteristics. • Relies on calibration for accuracy. • Calculation of parameters can be complicated and difficult to check. • Hydrological calculations cannot account for hydraulic effects such as back water. • Development requires sub-catchments to be split into numerous smaller sub-catchments to account for increased run-off.
Rain-on-grid	• Sub-catchment delineation is not required - less effort in defining model parameters where land-use changes. • Parameters correspond to measurable quantities. • Requires only a small number of simple parameters. • Is intuitive for lay people. • Groundwater interaction is possible.	• Requires more computational resources and slows down hydrodynamic simulations. • Cannot reproduce finest levels of detail that may be important to resolve runoff discharge for catchments. • Requires hi-fidelity data to produce realistic results: KCDC has data of sufficient quality. • 64-bit floating point precision is required for reliable simulation: this results in slower simulations and high-end computer hardware specifications

4.2.4. Hybrid approach

The traditional sub-catchment approach cannot satisfy some of the requirements set out for the model build, namely the level of detail of the models and the prospective use of the models for multiple purposes. It does, however, have benefits in managing model runtimes and resolving complex inter-relationships with simplified hydrologic inputs. Awa has therefore employed a hybrid hydrological modelling approach for flood modelling in Kāpiti:

- the rain-on-grid approach is applied to low-lying, flat land, particularly where there is the potential for ponding and where interaction with groundwater is likely,
- the sub-catchment-based approach is applied to steeper catchments where flow is channelised and insufficient data is available to simulate the fine-scale hydraulics.

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Table 6 summarises the rainfall-runoff method adopted for each catchment. Three catchments were modelled solely using the rain-on-grid approach, as they contained no steep uphill catchments. The other four catchments used a hybrid rain-on-grid and sub-catchment approach.

Table 6: Rainfall-Runoff method applied to each catchment.

ŌTAKI	RAIN-ON-GRID
Ōtaki	Rain-on-grid
Waikanae	Rain-on-grid and sub-catchment
Mazengarb	Rain-on-grid and sub-catchment
Paraparaumu Beach	Rain-on-grid
Wharemauku	Rain-on-grid and sub-catchment
Raumati South	Rain-on-grid
Paekākāriki	Rain-on-grid and sub-catchment

The portion of the 2D model domain where rain-on-grid approach is applied generally coincides with the areas of interest for flood mapping.

Runoff from hill sub-catchments were calculated using the following steps:

1. Delineation of upper hill sub-catchments using Urban Model Manager (Awa internal software).
2. Creation of hydrological discharges from rainfall based on the Clark Unit Hydrograph method.
3. Entering the discharge rates from the sub-catchment back to the 2D domain (Rain-on-Grid) as inflow boundary conditions.

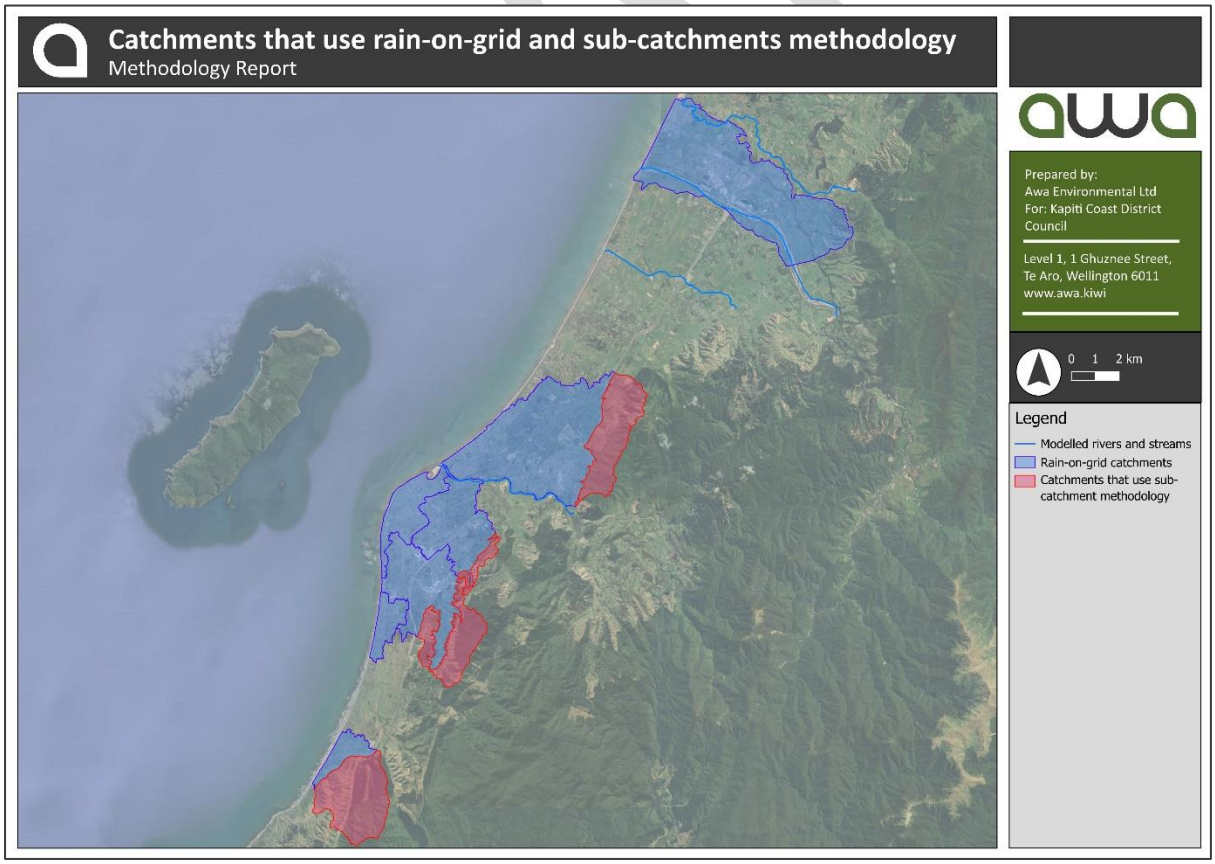


Figure 17. Catchment hydrology methodologies.

4.3. TUFLOW Infiltration Inputs

Infiltration is a key component of stormwater modelling as it governs the portion of rainfall that percolates into the soil instead of flowing overland. There are five infiltration methods available within TUFLOW:

- No Infiltration,
- Green Ampt methods 1 and 2,
- Horton, and
- Initial/Continuing Losses.

Armed with the data collected through the infiltration study outlined in section 3.2.3, Awa adopted the initial/continuing loss infiltration method. Table 7 presents the consolidated matrix of infiltration rate by soil type, compaction status, land use derived from the study findings that formed the basis of the TUFLOW input file.

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Table 7. Soil infiltration rates by land use, soil type and soil compaction

Land Use	Soil Infiltration Rate (mm/hr)									
	Compacted					Uncompacted				
	Greywacke	Loess	Peat	River loam	Sand	Greywacke	Loess	Peat	River loam	Sand
Broadleaved Indigenous Hardwoods	10	10	70	55	70	10	10	6	250	400
Buildings	0	0	0	0	0	0	0	0	0	0
Built-up Area (settlement)	-	-	28	22	28	10	-	2.4	100	120
Bushes and trees	10	10	70	55	70	10	10	6	250	400
Deciduous Hardwoods	10	10	70	55	70	10	10	6	250	400
Estuarine Open Water	-	-	-	-	-	-	-	-	0	0
Exotic Forest	10	10	70	55	70	10	10	6	250	400
Fernland	-	-	-	-	-	10	-	-	250	-
Forest - Harvested	-	5	-	-	-	5	5	6	250	300
Gorse and/or Broom	-	10	0	55	70	10	10	6	250	400
Grass	10	10	70	55	70	10	10	6	250	300
Gravel or Rock	-	-	70	55	70	10	0	0	250	300
Herbaceous Freshwater Vegetation	10	-	70	-	70	10	0	6	250	300
High Producing Exotic Grassland	10	10	70	55	70	10	0	6	250	300
Indigenous Forest	10	0	-	55	70	10	10	6	250	400
Lake or Pond	-	-	0	0	0	-	0	0	0	0
Landfill Buildings	0					-				
Landfill Bushes and trees	0					-				
Landfill Grass	0					-				
Landfill Other impervious	0					-				
Landfill Rough low vegetation	0					-				
Landfill Sand gravel clay and bare earth	0					-				
Landfill Water	0					-				
Low Producing Grassland	10	-	70	55	70	10	0	6	250	300
Manuka and/or Kanuka	10	-	-	55	70	10	0	6	250	400

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Matagouri or Grey Scrub	-	-	-	-	-	10	-	-	-	-
Mixed Exotic Shrubland	-	-	-	-	70	10	-	6	250	300
Orchard Vineyard or Other Perennial Crop		10	70	55		10	10	6	250	300
Other impervious	0	0	0	0	0	0	0	0	0	0
River				0				0	0	
Roads	0	0	0	0	0	0	0	0	0	0
Rough low vegetation	10	10	70	55	70	10	10	6	250	300
Sand gravel clay and bare earth	10	0	0	0	0	10	0	0	0	0
Sand or Gravel					70			6	250	300
Short-rotation Cropland			70	55	70	10		6	250	300
Surface Mine or Dump	0		0	0		0	0	0	0	0
Transport Infrastructure	0	0	0	0	0	0	0	0	0	0
Urban Parkland/Open Space				45	45		230	0	230	0
Water	0	0	0	0	0	0	0	0	0	0

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Porosity (or saturated moisture content), another infiltration characteristic, was approximated by specific yield as per Johnson (1967). Using specific yield is a conservatively low estimate of soil storage, which corresponds to the assumption that it has rained recently enough such that plants have not had time to draw down on specific retention moisture. It should also be noted that specific yield does not take soil horizons into account. Soil porosity values applied by soil type are summarised in Table 8.

Antecedent moisture conditions, expressed as the 'initial moisture' fraction, were assumed to be dry on account of the high infiltration and soakage rates.

Table 9 summarises the inputs to the look-up table that informed the TUFLOW soil input spatial layer.

Table 8. Soil porosity values adopted in the TUFLOW soil layer

Soil	Uncompacted	Compacted	Comments
greywacke	0.05	0.02	Upper end of range for shale to allow for some soil mixed in.
landfill	0.01	0.01	Compacted soil and impervious material.
loess	0.08	0.03	Silt. Compacted is low end of scale.
peat	0.01	0.21	Assuming that uncompacted peat is already highly saturated and compacted peat is replaced with fine sand.
river loam	0.25	0.21	Fine gravel. Compacted is low end of scale.
sand	0.26	0.2	Gravelly sand. Compacted is low end of the scale.

Table 9. TUFLOW Initial/Continuing Losses Infiltration Method Inputs

Initial/Continuing LossesParameter	Input File Column No.	Details
Soil ID	1	Unique identifier
ILCL	2	Selected methodology (initial/continuing loss)
Initial Loss (mm)	3	Approximations based on imperviousness, ranging between 5 - 20 mm for imperviousness through to rural.
Continuing Loss (mm/h)	4	Soil infiltration rates derived from the infiltration study outputs categorised by soil classification, landcover type & compaction (Table 7).
Porosity (Fraction)	5	Modelled porosity by soil classification and compaction based on specific yield (Table 8)
Initial Moisture (Fraction)	6	The fraction of the soil that is wet at the start of simulation is set to zero.

4.4. Soakage

Soakage to ground, in the form of soak pits connected to sumps, is widely used across the district for stormwater mitigation. While the soakage rates of public devices were not measured as part of the model build, a selection of soakage rates from across the district was provided by KCDC. These were analysed and assuming some degree of fouling of the device over time an average soakage rate of 300 mm/hr has been applied as the default soakage rate across public devices. The soakage discharge can be calculated as follows assuming flow only out of the bottom of the soakage device.

$$Q_{soakage} = \left(\frac{R_{soakage}}{1000*3600}\right) * A_{device}$$

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where:

$Q_{soakage}$ = soakage discharge in m^3/s,
$R_{soakage}$ = soakage rate in mm/hr, and
A_{device} = plan area of soakage device in m^2.

Soakage within private devices has not been accounted for and it is an assumption of the modelling that this will not have a significant effect on the flood hazard.

4.5. River Inflows

Only two catchments use river flow as a boundary condition: Ōtaki (Ōtaki River and Waitohu Stream) and Waikanae (Waikanae River). The ARI hydrographs used for these catchments were drawn from MIKE models from GWRC prepared by Wallace (2012 and 2013). 50-year and 100-year ARI hydrographs were already available for the Otaki and Waikanae rivers. For 10-, 5- and 2-year ARI storm scenarios, river hydrographs were adjusted in line with Otaki and Waikanae rating curves supplied by GWRC (2022), which had been generated in May 2021. Hydrographs for the Waitohu Stream had been included in the MIKE models for all storm scenarios. Figure 18 presents the hydrographs that were adopted for the Otaki River inflows.

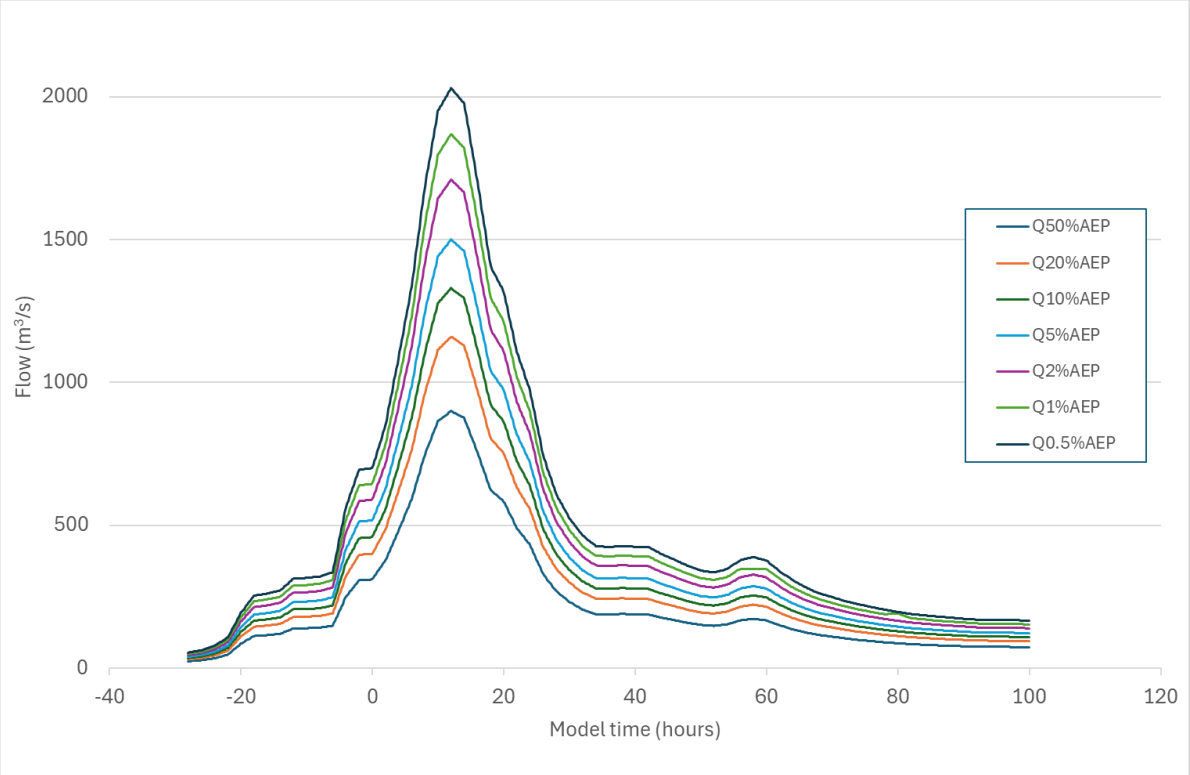


Figure 18 Hydrographs adopted for the Otaki River inflows boundary condition

4.6. Tidal Effects

The proximity of Kāpiti to the coastline, together with its topography make flooding prone to tidal influence. Hence one of the key boundary conditions on the KCDC flood models is the design tide. Design tide is the simulation of tidal conditions during a storm event.

Tides are composed of the elements listed below:

- Sea level rise (SLR) associated with climate change,
- Mean level of sea (MLoS) anomaly
- Astronomical tidal cycle, including daily cycles and spring tides;
- Storm surge (wind setup and atmospheric pressure effects) and
- Wave setup and wave run-up

The greatest risk of coastal flooding arises from the peaks of the elements coinciding to produce unusually high water levels. Thus the size, shape and timing of the elements must all be factored into building a design tide timeseries to model the interaction between catchment and coastal flooding. The approaches taken to each tidal component are described below.

4.6.1. Seal Level Rise

Global mean seal level is rising due to a combination of melt water from glaciers and ice sheets, and thermal expansion of seawater as it warms. To allow for the historical change in MLoS between when the Wellington 1953 local vertical datum (WVD53) was established and the launch year for 2005 climate change SLR projections (2005), an offset was applied to the Mean level of sea (MLoS). The offset of 0.188 m was derived from collation of a variety of sources that have analysed historical MLoS as summarised in Table 10.

Table 10. Historical Sea level rise

Year	Averaging Period	MLoS (m WVD53)	Source
1953		0.000	
1990	1980-1999	0.140	Stephens et al (2012)
1996	1986-2005	0.164	Bell R. G. and Allis M. J. (2021)
2008	2005-2011	0.196	Stephens et al (2012)
2010	2001-2019	0.245	Bell R. G. and Allis M. J. (2021)
2015	2012-2018	0.250	Todd et al (2021) [reproduced from Bosserelle and Lane (2019)]

Future SLR was compiled from New Zealand climate change projections available from the Takiwā platform as advised by MfE (2022). These SLR scenarios use a 2005 baseline so were offset to the WVD53 datum as described above. Table 11 gives the WVD53-adjusted SLR levels for the various climate change projections. The base flood models the SSP5-8.5M scenario SLR projection. Sensitivity runs of higher SLR projections (SSP5-8.5H+) were also used to support other uses of the flood maps (i.e. sensitivity runs for greenfield development) which are consistent with MfE (2022) standards.

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Table 11. SLR Scenarios from the Takiwā platform adjusted to Wellington MLoS WVD53.

YEAR	ADJUSTED MLoS (mWVD53)				
	SSP1-2.6M	SSP2-4.5M	SSP-7.0M	SSP5-8.5M	SSP5-8.5H+
2005	0.188	0.188	0.188	0.188	0.188
2020	0.248	0.248	0.248	0.248	0.278
2030	0.298	0.298	0.298	0.298	0.328
2040	0.338	0.348	0.348	0.358	0.408
2050	0.388	0.408	0.428	0.448	0.508
2060	0.428	0.468	0.498	0.528	0.618
2070	0.478	0.538	0.588	0.628	0.758
2080	0.528	0.608	0.688	0.748	0.908
2090	0.568	0.678	0.798	0.878	1.088
2100	0.618	0.758	0.928	1.028	1.278
2110	0.688	0.848	1.028	1.138	1.478
2120	0.728	0.928	1.148	1.278	1.678
2130	0.778	0.998	1.268	1.408	1.858
2140	0.818	1.068	1.388	1.538	2.038
2150	0.858	1.148	1.498	1.658	2.218

4.6.2. Astronomical Tide

Astronomical tide is the fluctuation of sea level over a lunar cycle (excluding storm effects). Over the course of a month the range of the astronomical tide varies with the orbit of the moon, causing 'spring' and 'neap' tides. Astronomical tide data were sourced from the NIWA Tide Forecaster (https://tides.niwa.co.nz/) and were used to provide the base sinusoidal 'shape' of the tidal cycle. As per GWRC (2021), a high water spring (3 January 2022 to 5 January 2022) was selected as the representative tide for the five locations of interest: Paekākāriki, Raumati Beach, Paraparaumu Beach, Waikanae Beach, and Ōtaki North. The locations of the astronomical tide data sources are shown in Figure 19. The astronomical tide timeseries from the different sites are shown in Figure 20.

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Figure 19. Site locations of astronomical tides sources.

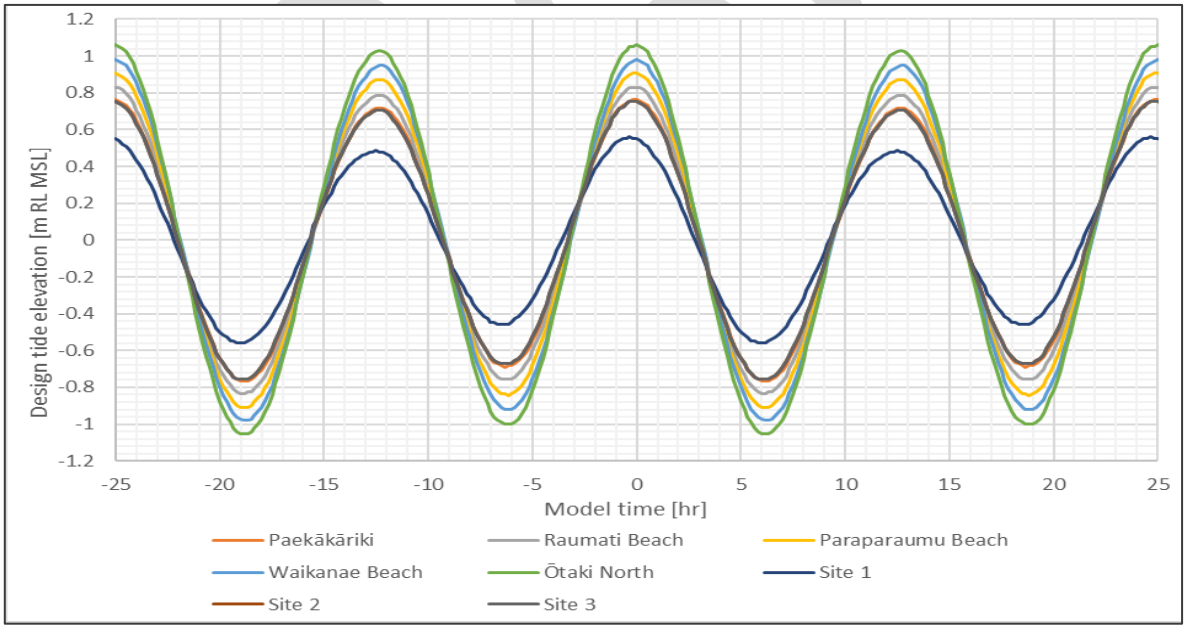


Figure 20. Astronomical tide timeseries from different sites in Kāpiti.

Spring tide (astronomical tide) boundary conditions, excluding any storm effects (storm surge, wave setup, wave run up), have been made available to model users for all decadal climate horizons up to 2150 for the five SSP's of interest.

4.6.3. Storm Surge

Storm surge is the temporary increase in sea water levels caused by storm conditions that last from hours to days. There are two weather processes that cause storm surge (Lane et al 2012):

- Barometric effect caused by low air pressure that allows increased sea levels. There is approximately a 1-cm rise in sea level for ever 1hPa drop in barometric pressure below normal conditions ;
- Strong, persistent onshore winds pushing water against the coast and raising water levels.

Storm surges were taken to have a bell-shaped amplification profile and to last 3 days as shown in Figure 21. Extreme storm tide levels for a range of AEPs were drawn from Stephens et al. (2012). The 1% AEP storm tide was drawn from Lane et al. (2012).

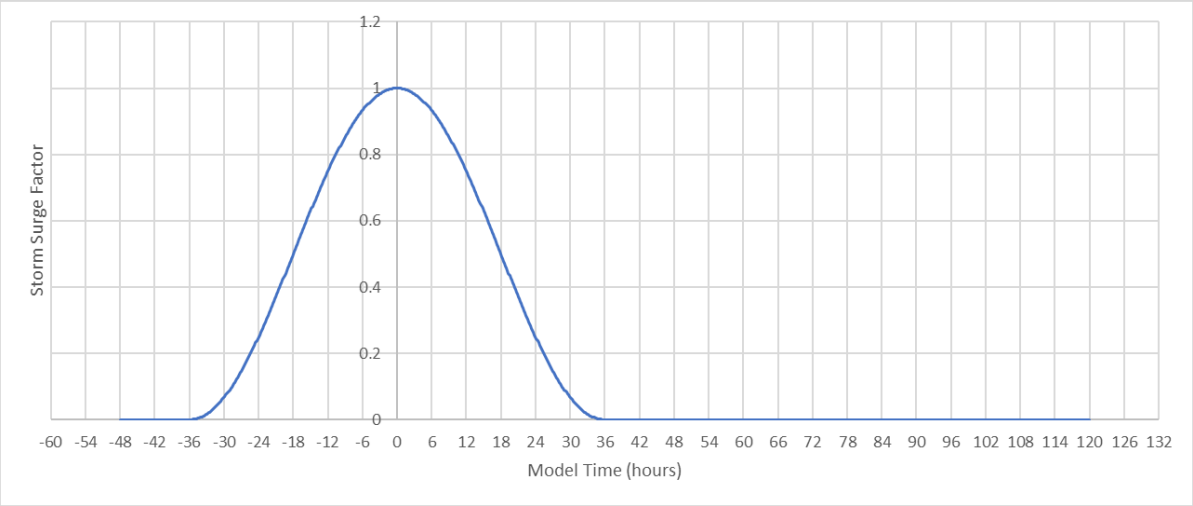


Figure 21. Storm surge profile

4.6.4. Wave Setup and Wave Run-Up

Wave setup is the increase in mean sea level at the coast as waves push through the surf zone and break in shallower waters (Lane et al. 2012). Stockdon et al. (2006) describes an empirical formulation of wave setup which was used for scaling the difference between peak storm tide and wave setup for multiple AEPs. The correlation between significant wave height and wave length was approximated (99th percentile of prediction interval) using a hindcast from the NIWA NZ Wave Data Tool. The joint probability of storm tide and significant wave height was drawn from Stephens et al (2012). The 1% AEP wave setup estimate was taken from Lane et al. (2012).

Wave run-up - the maximum rise above the instantaneous storm-tide water level caused by the impact of waves on a beach, dune or structure - is another component of coastal flooding. However, the fluctuations in water level caused by wave run-up are very brief in comparison to wave setup, tidal and storm-surge time scales. So while of concern to coastal hazard management, wave run-up makes only a minor contribution to flooding and was therefore discounted from the flood model design tide.

4.6.5. Design Tide Composition

The approach taken to building design tides was developed in reference to the methodology outlined in Todd D., MacDonald K., and Debski D. (2021). Figure 22 depicts how the components explained above were combined to produce a design tide timeseries. SLR comprises both historical and future increases to MLoS. The forecast component of SLR is described in section 5.3. As noted earlier, wave run-up has been excluded from the design tide. Tsunami inundation has also not been accounted for in the modelling.

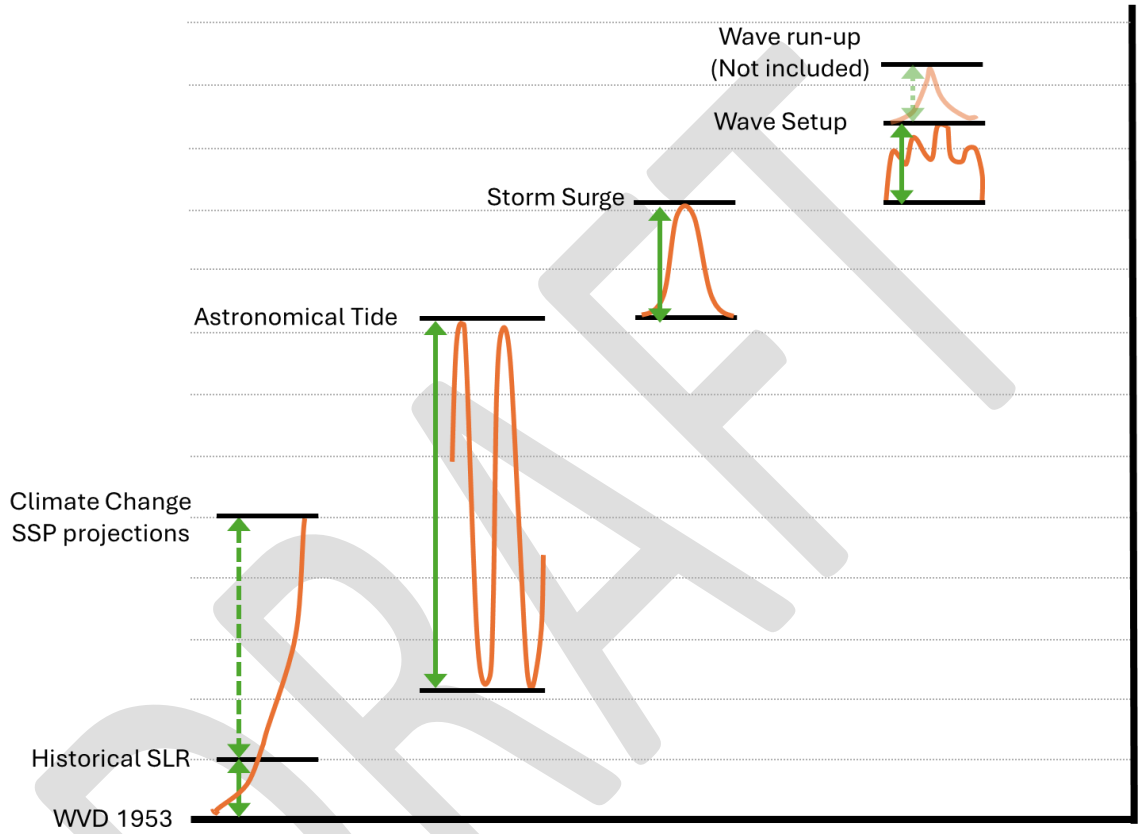


Figure 22. Design tide components.

The resulting tidal timeseries were applied along coastal boundaries and at river mouths in the TUFLOW hydraulic model. Tidal design timeseries, incorporating both storm tide and wave set up for a range of AEPs, are provided for model users in a series of CSV files. A separate file is available for each combination of location, Shared Socio-economic Pathway (SSPs) and climate scenario. A summary of the sources and treatment of each tidal component is provided in Table 12. Figure 23 plots the design tides that have been applied to the 1% AEP storm events across the five catchments with tidal boundaries.

Due to the complexity of rainfall and tidal interactions, there are uncertainties around the probabilities between different severities of rainfall or tidal events occurring. Accordingly, Awa commissioned Hydrology and Risk Consulting (HARC) to model the joint probabilities of rainfall and tidal events occurring for particular event horizons (e.g. 1%AEP at 2130 in Waikanae). A description of the joint probability analysis is presented in section 6.

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Table 12. The sources and descriptions of components of design sea level boundary timeseries.

COMPONENT	SOURCES	DESCRIPTION AND ASSUMPTIONS
Sea level rise (SLR)	Table 1 of MfE 2022 Climate change sea level rise estimates taken from Takiwā platform (MfE, 2022)	• MLOS adjustment of 0.188 m offset sourced from Bell R. G. and Allis M. J. (2021) and Stephens et. al. (2012). • Modelling of future climate change impacts on SLR is outlined in section 5.3.
Astronomical tide	Page 128 of LINZ (2021) NIWA Tide Forecaster	• Mean High Water Springs (MHWS) used as recommended by GWRC (2021). • Symmetric range of 1.9 m (LINZ 2021). • MHWS tidal profile for 3rd – 4th/1/2022.
Storm surge	• Extreme storm tide levels - Stephens et al (2012) • 1% AEP storm tide and wave set up estimates - Lane et al. (2012)	• Storm surge is caused by low pressure and wind stress. • Three-day, bell-shape surge profile. • Timed to peak at peak astronomical tide.
Wave setup	• Stockdon formula for the relationship between peak storm tide and wave set up Stockdon et al (2006) • 1% AEP storm tide and wave set up estimates - Lane et al. (2012)	• Wave setup is related to breaking waves in the surf zone • Distinct from wave runup, which was not included in the design tide.
Summary	Design Tide = SLR + Astronomical tide + Storm surge + Wave setup	• Final tidal boundary consists of the sea level rise, astronomical tide, storm surge and wave setup. • Astronomical tide forms the base shape of the tidal levels. • Sea level rise is implemented as a direct increase or offset on the whole tidal timeseries. • Storm surge and wave setup were subjected to joint probability analysis in combination with rainfall to recast design tide levels and AEPs (refer to section 6.1)

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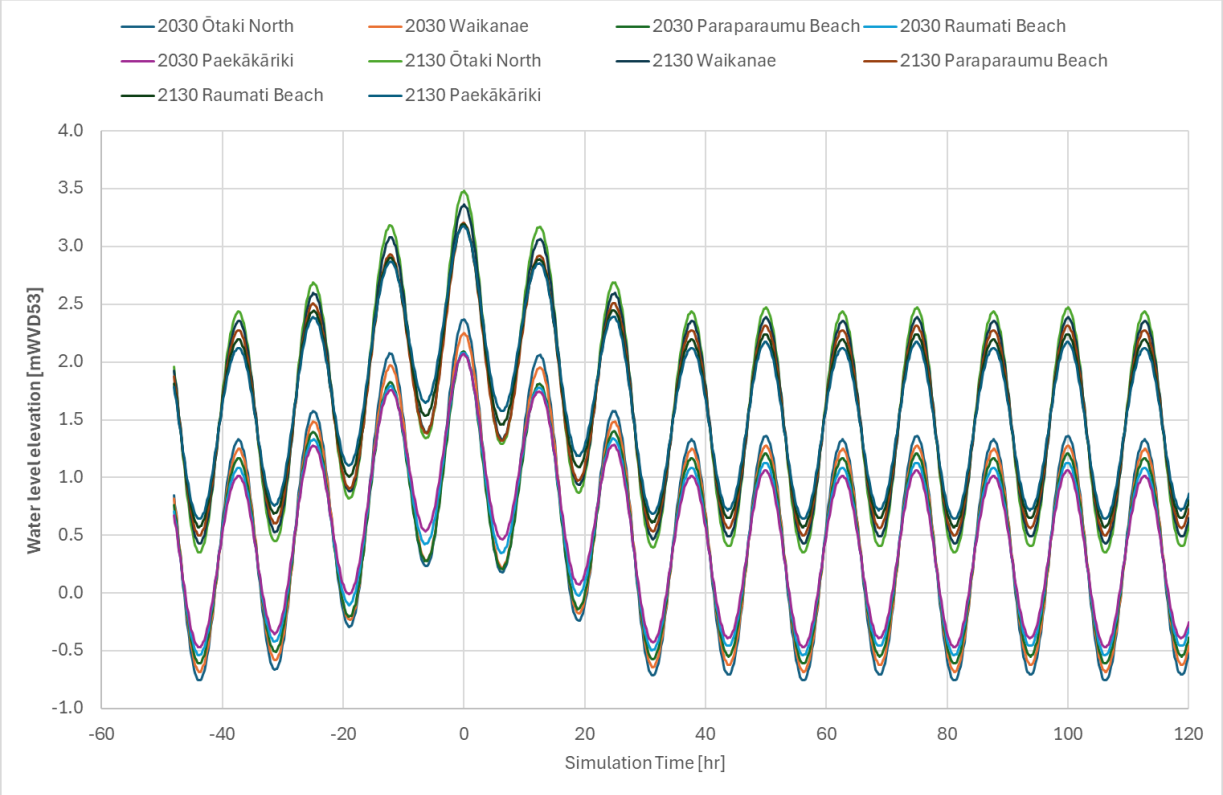


Figure 23. Design tide water levels for the 2030 and 2130 SSP5-8.5, 1% AEP rainfall event scenarios

4.7. Groundwater

Elevated groundwater levels have been observed following extended periods of wet weather in the district. Due to the predominance of low-lying sandy soils in the district, the interaction of infiltration rates and the groundwater table are key phenomena to be accounted for in flood modelling.

4.7.1. Groundwater Modelling

Groundwater levels were approximated from vector polygon layers provided by Tonkin & Taylor on 20 January 2022 from a project for GWRC (Gyopari et. al. (2014). These layers are based on the 50th percentile estimate of groundwater table elevation from 20-year MODFLOW[13] simulations with the sea level rise projections used as the Mean Level of the Sea.

The groundwater levels were adjusted by an offset or augmented against the sea level to reflect more recent projections of climate change effects using NZSeaRise estimates, which are expressed as SSPs instead of RCPs. Additionally, groundwater levels for 2070 were also interpolated from the 2030 and 2130 groundwater levels. Groundwater approximations based on NZSeaRise estimates are shown in Table 13. The associated calculations are summarised in APPENDIX F.

A polygon layer made from the MODFLOW output grid was rasterised in QGIS onto a 50m x 50m grid for sue in TUFLOW. Where the 2D model domains extended beyond the boundaries of the groundwater results, additional zones were added with levels based on those on the periphery of the polygon layer (i.e. Ōtaki and Wharemauku catchments). The 50th percentile groundwater table was adopted for all

[13] US Geological Survey modular finite-difference groundwater flow model.

scenarios as it is expected that variability within the mid-range groundwater table elevations is low. Note that joint-probability analysis to examine the correlation between rainfall events, storm surges and groundwater fluctuations has not been undertaken.

Table 13. Groundwater approximations.

YEAR HORIZON	CLIMATE CHANGE SCENARIO	GROUNDWATER TABLE SURFACE (HIGHEST VALUE AT 50TH PERCENTILE)
2020	NA	0.248 mWVD53 (GWL 2020)
2030	NA	0.298 mWVD53 (GWL 2030)
2070	SSP5-8.5M	0.628 mWVD53 (GWL 2070)
2130	SSP5-8.5M	1.408 mWVD53 (GWL 2130)
2130	SSP5-8.5H+	1.858 mWVD53 (GWL 2130H+)

4.7.2. Groundwater-Runoff Interaction

In the TUFLOW 2D model groundwater interacts with runoff via subsurface cells that fill via infiltration from the surface. The transformation of rainfall into runoff is mediated by land-use cover on the surface, infiltration rates, porosity, and base levels of groundwater according to the following rules.

- Groundwater and runoff interaction uses ILCL (initial loss/continuing loss) method (BMT 2018).
- If the groundwater rises to the surface, no added infiltration will occur.
- Only vertical filling of the groundwater table can occur.
- There is no horizontal movement of the groundwater.
- Groundwater levels do not reduce through seepage or discharge over time.

4.8. Design Storm Duration

Rain events that lead to substantial flooding in Kapiti tend to be long duration storms. Shorter events typically end before runoff concentrates. Longer events tend not contribute substantive additional flows beyond the 36-hour mark. The difference between a 24- and 48-hour storm is negligible, with the last 12 hours exerting minimal influence on flooding.

With the intent of the modelling being to test worst-case conditions that lead to flooding, 48-hour storm durations were adopted for the base models. Since it takes 12 or more hours for runoff to concentrate to a peak, simulations were initiated from around the 12-hour mark. This effectively reduces the simulation to a 36-hour event, saving considerable model run time, which for the larger catchments can span multiple days. Whilst the rain tapers off over last 12 hours of the storm event, the simulation continues on to the 48-hour mark to make sure the full potential for flooding is captured

4.9. Boundary Condition Timeseries Data Alignment

To reflect worst-case flooding scenarios, catchment-driven flood peaks need to coincide with tidal conditions that produce peak elevated tailwater levels and associated flooding. The design tide was

therefore located on the simulation time series so that the peak of the tidal storm surge coincided with the upstream catchment time of concentration (approximately hour zero in the simulation time series). This was a manually-driven process with results that varied between catchments. In addition, the design tide timeseries were configured so that peak astronomical tide coincides with the storm surge profile as recommended by GWRC (2021).

The start of the simulation period was then set to coincide with mid-(astronomical) tide so that the 2D catchment domain is not initialised with standing water on the land surface - in particular puddles in areas that are disconnected from the coast or that might be connected hydraulically, but are distant from the coast. This required some catchment models to commence before time zero and a slightly longer simulation period (>36 hours)

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5. Climate Change

Climate change will impact all five of the key boundary conditions for KCDC's flood models:

- rainfall,
- sea level,
- groundwater,
- river inflows and
- hill catchment runoff.

Rainfall and sea level boundary conditions were informed by guidelines produced by the Ministry for the Environment as outlined in Table 14 below.

Table 14. Guidelines used for climate change application on model boundary conditions.

PARAMETER	GUIDELINE	SCENARIOS USED
Rainfall	Ministry for the Environment. 2018. Climate Change Projections for New Zealand: Atmosphere Projections Based on Simulations from the IPCC Fifth Assessment, 2nd Edition.	RCP (Representative Concentration Pathways)
Sea level	Ministry for the Environment. 2022. Interim guidance on the use of new sea-level rise projections. Wellington: Ministry for the Environment.	SSP (Shared Socioeconomic Pathways)

Rainfall changes to the models were based on MfE (2018) temperature projections as they provide regional downscaling for a New Zealand context. MfE (2018) offers temperature projections to 2100 (3.0C) and 2120 (3.7C) specific to Wellington Region. The process and integration of the MfE (2018) projections into the flood models are outlined in section 5.1 below.

Regarding tidal boundaries, climate change effects were applied through increased sea level rise as recommended by MfE (2022). MfE (2022) uses SSP terminology (superseding RCP terminology) which considers the socioeconomic and political actions that may affect the overall climate change effects.

Note that new temperature projections for climate change were available on a global scale at the time of preparing this report. However, for the purposes of this model build, the existing downscaled NZ projections (Table 15) have been adopted in the absence of updated downscaled projections, as per MfE (2022) guidance[14].

Climate change impacts on groundwater and hill catchment run off are accounted for in the models through a combination of increased rainfall and sea level rise. The specifics of allowing for climate change impacts on each boundary condition type are described in the following sections.

In the absence of information on climate change impacts on river inflows, the hydrographs described in section 4.5 were retained for the 2130 simulations.

[14] Bodeker, G., Cullen, N., Katurji, M., McDonald, A., Morgenstern, O., Noone, D., Renwick, J., Revell, L. and Tait, A. (2022). Aotearoa New Zealand climate change projections guidance: Interpreting the latest IPCC WG1 report findings. Prepared for the Ministry for the Environment, Report number CR 501, 51p.

Table 15. Climate change temperature increase estimates for RCPs and climate horizons of interest.

CLIMATE HORIZON	TEMPERATURE INCREASE FOR RCP8.5
2030	0.4 °C
2130	3.7 °C

5.1. Rainfall

Climate change effects on rainfall are implemented through a process that incorporates NIWA HIRDS v4 (High Intensity Rainfall Design System) projections (2018) and augmentation using temperatures from MfE (2018).

The base rainfall was sourced from NIWA HIRDS v4, which provides gridded distribution of rainfall based on historical data (1986-2005) from existing rainfall sites. Drawing on the empirical data, HIRDS v4 provides a base distribution and distributions for design storm annual return intervals and durations.

Awa applied climate change effects on the rainfall distributions sourced from HIRDS v4 by augmenting rainfall depths based on temperature changes as per the approach described in the NIWA HIRDS v4 report (Carey-Smith et al. 2018) (see Figure 24). It projects rainfall depths across different durations and ARI into future scenarios based on the temperature changes summarised in Figure 24 (i.e. percent rainfall depth change per 1C change in temperature).

DURATION/ARI	2 YR	5 YR	10 YR	20 YR	30 YR	40 YR	50 YR	60 YR	80 YR	100 YR
1 HOUR	12.2	12.8	13.1	13.3	13.4	13.4	13.5	13.5	13.6	13.6
2 HOURS	11.7	12.3	12.6	12.8	12.9	12.9	13.0	13.0	13.1	13.1
6 HOURS	9.8	10.5	10.8	11.1	11.2	11.3	11.3	11.4	11.4	11.5
12 HOURS	8.5	9.2	9.5	9.7	9.8	9.9	9.9	10.0	10.0	10.1
24 HOURS	7.2	7.8	8.1	8.2	8.3	8.4	8.4	8.5	8.5	8.6
48 HOURS	6.1	6.7	7.0	7.2	7.3	7.3	7.4	7.4	7.5	7.5
72 HOURS	5.5	6.2	6.5	6.6	6.7	6.8	6.8	6.9	6.9	6.9
96 HOURS	5.1	5.7	6.0	6.2	6.3	6.3	6.4	6.4	6.4	6.5
120 HOURS	4.8	5.4	5.7	5.8	5.9	6.0	6.0	6.0	6.1	6.1

Figure 24. Percentage change factors to project rainfall depths derived from the current climate to a future climate that is 1 degree warmer (NIWA 2018).

The base flood models use 2030 and 2130 year horizons. Awa adopted the 2120 projected temperature from MfE (2018) for the 2130 rainfall augmentation calculation (refer to Table 16). This sits within the 2.8 – 5.1 °C range of uncertainty for the 2120 value.

Additionally, Awa ran sensitivity scenarios where a higher climate change scenario was used (RCP 8.5H+): 4°C increase at 2130. These extrapolated temperatures were used to produce a gridded distribution of rainfall that accounts for climate change in future scenarios (see Figure 25).

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Table 16. Projected temperature changes based on statistical downscaling for NZ context (MfE, 2018)

AREA	SOURCE	CLIMATE CHANGE SCENARIO	SUMMER (°C)	AUTUMN (°C)	WINTER (°C)	SPRING (°C)	ANNUAL (°C)
Wellington in **2030**	Awa extrapolation	Base					0.4
Wellington in **2100**	MfE 2018	RCP 8.5	3.1	3.1	3.2	2.7	3.0
Wellington in **2120**	MfE 2018	RCP 8.5	4.0	3.9	3.7	3.3	3.7
Wellington in **2130**	Awa extrapolation	RCP 8.5M					3.7
Wellington in **2130**	Awa sensitivity analysis	RCP8.5H+					4.0

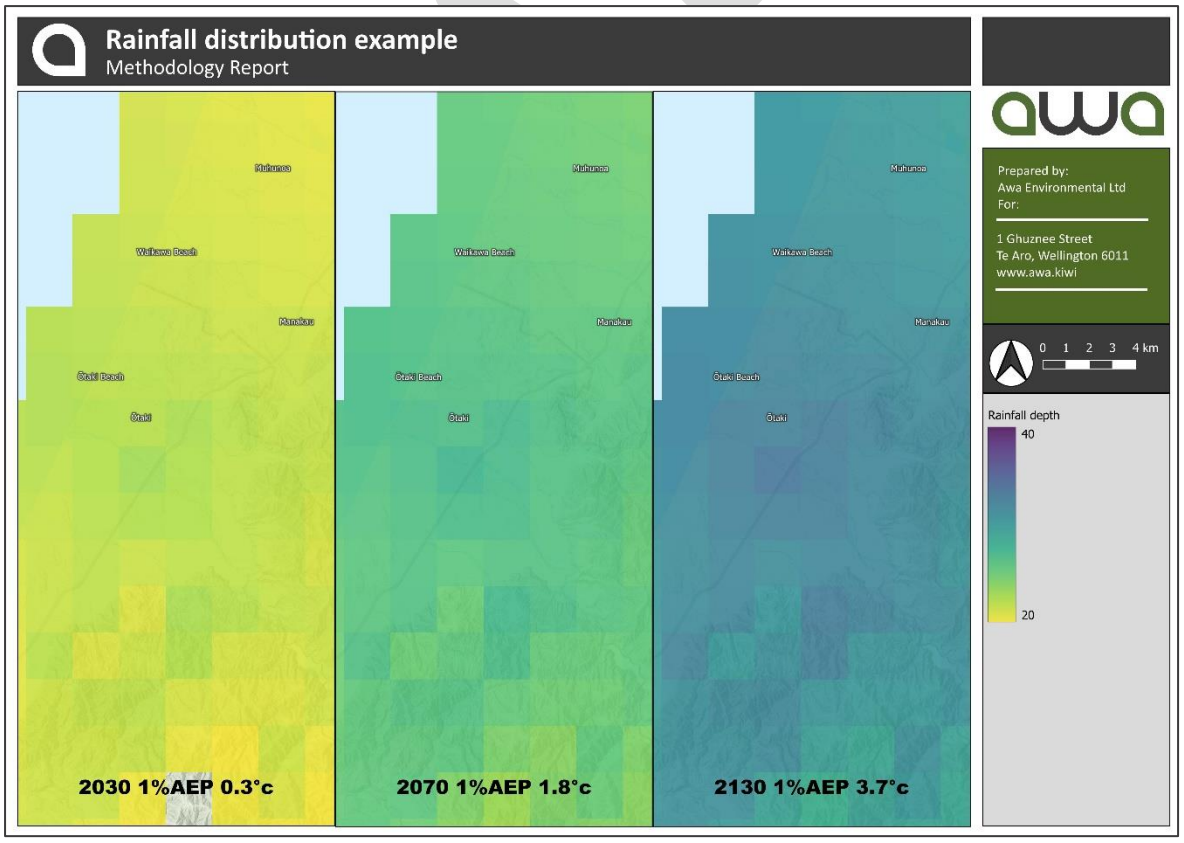


Figure 25. Rainfall distributions over Ōtaki using different year horizons and associated climate change temperature changes.

5.2. Hill Catchment runoff

Similar to river inflows, climate change effects are implemented on the hill catchment runoff through the increase of rainfall depths. Rainfall increase (as mentioned in section 5.1) will increase the runoff volumes on the hill catchments.

5.3. Tides

Climate change impacts on tides were modelled by adjusting the sea level rise (SLR) as described in section 4.6.1. While decreases in pressure and increases in wind stress caused by climate change could affect storm surge, NIWA (2012) state that the impact should be minor; hence storm surge was not adjusted for climate change. Wave set-up was also left unaltered for reasons outlined by Todd et al. (2021).

5.4. Groundwater

Groundwater is affected by climate change through both amplified sea level rise and rainfall depths. Changes to base groundwater levels are dictated by the sea level rise projections from MfE (2022) as explained in section 4.7.1. Soil water capacity, the parameter that governs the interaction between the groundwater and surface runoff, will be affected by the increase in rainfall depths described in section 5.1.

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6. Joint Probability Assessment

Joint probability refers to a statistical measure that estimates the likelihood of two events occurring at the same point in time. Flooding in a catchment may be predominantly fluvial/stormwater or tidal in nature, or, as in the case of the Kāpiti Coast, a combination of both. As described in section 4.5, tidal flooding is a function of astronomical tide, storm surge, wave setup and SLR. While astronomical tides and SLR are independent of weather, storm surges and wave setup can be influence by the same weather patterns that determine catchment flooding.

Research undertaken in Australia has shown that the probability of a heavy rainfall event occurring on catchments near the coastline in conjunction with elevated storm tide levels is higher than would be the case if those two phenomena were assumed to be independent from one another (HARC 2024). On the Kāpiti Coast, orographic rainfall associated with the Tararua Range that leads to flooding of urbanised catchments can be linked to weather systems emanating from the Tasman Sea and South Taranaki Bight. Hence an assessment of the joint dependence between extreme rainfall and storm surge events ought to be considered when modelling flooding in the region.

Joint probability assessment examines the likelihood of coincident low-probability boundary conditions to determine appropriate combinations of Average Exceedance Probabilities (AEPs) to use in flood modelling scenarios. The predictors of interested are:

- Rainfall,
- Tide (storm surge and wave set up),
- Groundwater levels, and
- Antecedent moisture conditions.

In this case joint probability was examined for two of these boundary conditions: rainfall and storm surge. Note that while runoff from upper hill catchments is connected as boundary inflows to the 2D domain of the TUFLOW model, the generation of these flows from rainfall is considered part of the overall model.

The inter-dependency between flooding and groundwater is less well understood but is expected to change with climate. In the absence of official guidance or precedent approaches on the matter, the 50th percentile groundwater table was applied across all modelling scenarios. Importantly, however, climate change effects on the groundwater table, through sea level rise, were considered as explained in section 5.4.

The joint dependence between Antecedent Moisture Conditions (AMC) and flood-causing rainfall can disrupt the exceedance probability neutrality between rainfall and runoff Nathan et al. (2016). Pathiraja et al. (2012) showed that the magnitude of flooding in heavy storm events tends to be underestimated when the AMC-rainfall joint dependence is not accounted for. This has implications for using design storms to predict flooding, particularly on rarer events and climate change scenarios. However in the case of the Kāpiti Coast, the combined effects of unusually high infiltration rates and elevated starting groundwater levels would render AMC a relatively benign influence on flood magnitude. In addition, model validation (see section 8) was performed using a large (1% AEP) storm event, which further negates the influence of AMC. Given the above, it was deemed that any uncertainty surrounding the potential effects of high AMC could be dealt with through a high rainfall, high groundwater (75th percentile) sensitivity analysis, which was to become part of the flood hazard management planning that follows the model build process.

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6.1. Joint Probability Analysis

Awa commissioned Hydrology and Risk Consulting (HARC) to analyse the joint dependence between rainfall and storm surge in relation to coastal- and catchment-driven flooding at three estuarine areas along the Kāpiti Coast: Wharemauku, Mazengarb, and Ōtaki. The analysis involved two discrete steps:

1. Estimation of joint dependence parameters using the methodology described in Australian Rainfall and Runoff (Westra *et al.*, 2019);
2. Application of the joint dependence parameters to produce the joint dependence AEPs and tidal levels for 50%, 20%, 10%, 2% and 1% AEP rainfall events using the method developed in ARR Project 18 for Joint Probability Modelling in Estuarine Regions (ARR, 2023)

Step 1 derived storm surge data from the difference between historical tidal data recorded at a GWRC-operated tide gauge in Porirua Harbour and NIWA (astronomical) tide forecaster. These were analysed together with rainfall data from the GWRC Tawa Pool gauge to determine joint dependence parameter values between 0 and 1 (0 indicating complete dependence and 1 indicating complete independence). The resulting values of 0.75, for 1h and 2h duration storms and 0.76 for the remining durations indicate a weak but nonetheless influential joint dependence[15], as anticipated based on the location, land form and weather patterns of the area.

For Step 2, Awa provided HARC with outputs from 36 model runs for each of the estuarine catchments comprising the matrix of combined rainfall and storm tide scenarios presented in Table 17. The selected scenarios spanned the range of interactions that were to be examined in the joint probability analysis, which meant that the analysis would be interpolating within the range of the hydraulic modelling rather than extrapolating outside that range. Table 18 also gives the inflow scenarios for the Ōtaki River and Waitoho Stream in the Ōtaki catchment, and the Waikanae River in the Mazengarb catchment. Flows from the upper catchment feeding the Wharemauku Stream were already integrated into the hydraulic model as explained in 4.2.1.

Table 17. Rainfall and storm surge scenarios used in applying the joint dependence parameter

Rainfall Scenario	Storm Duration (hours)	Storm surge scenario	Inflow Scenario
2020, 63.3% AEP, RCP8.5	48	2030, 63% AEP, SSP1-2.6M	Q50% AEP
2030, 50.0% AEP, RCP8.5	48	2030, 1% AEP, SSP5-8.5M	Q50% AEP
2030, 10.0% AEP, RCP8.5	48	2080, 1% AEP, SSP5-8.5M	Q50% AEP
2030, 2.0% AEP, RCP8.5	48	2100, 1% AEP, SSP5-8.5M	Q20% AEP
2130, 1.0% AEP, RCP8.5	48	2130, 1% AEP, SSP5-8.5M	Q2% AEP
2130, 0.4% AEP, RCP8.5	48	2130, 1% AEP, SSP5-8.5H+	Q1% AEP

HARC extracted water levels from the model outputs at key (longitudinal) locations along the main flow paths within each catchment waterway to calculate joint probability levels for a range of AEP flood events (≥1% and ≤100%). Application of the adopted joint probability parameter value of 0.75 produced

[15] For reference, joint dependence values around the entire Australian coastline range between 0.90 and 0.98

AEPs and tidal levels for 50%, 20%, 10%, 2% and 1% AEP rainfall events for each catchment. More detail is provided in the HARC report, APPENDIX G[16]

6.2. Design Event Boundary Conditions

The base model boundary conditions derived from the joint probability analysis for each grouping of catchments associated with the three estuarine areas are provided in Table 18, Table 19 and Table 20 below.

Table 18. Base model boundary conditions for Ōtaki Catchment.

YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)	TIDE LEVEL (MWVD)
2030	1%	5%	2.37
2030	2%	10%	2.32
2030	10%	15%	2.28
2030	20%	40%	2.18
2030	50%	80%	2.09
2130	1%	5%	3.49
2130	2%	10%	3.43
2130	10%	25%	3.34
2130	20%	40%	3.29
2130	50%	60%	3.25

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[16] Hydrology and Risk Consulting (HARC), (2023). *Joint probability analysis of coastal and catchment flooding*. Prepared for Awa Environmental and Kapiti Coast District Council.

Table 19. Base model boundary conditions for the Waikanae and Mazengarb Catchments.

YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)	TIDE LEVEL (MWVD)
2030	1%	1%	2.36
2030	2%	2%	2.31
2030	10%	10%	2.21
2030	20%	25%	2.12
2030	50%	50%	2.04
2130	1%	1%	3.47
2130	2%	2%	3.42
2130	10%	15%	3.28
2130	20%	30%	3.21
2130	50%	60%	3.13

Table 20. Base model boundary conditions for Paraparaumu Beach, Raumati, Wharemauku, and Paekakariki Catchments.

YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)	TIDE LEVEL (MWVD)
2030	1%	10%	2.03
2030	2%	10%	2.03
2030	10%	50%	1.86
2030	20%	50%	1.86
2030	50%	50%	1.86
2130	1%	30%	3.03
2130	2%	40%	2.99
2130	10%	50%	2.97
2130	20%	75%	2.91
2130	50%	75%	2.91

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6.2.1. Design Tide Components

The joint probability analysis produced overall peak tide levels. These were broken back down into component parts by subtracting astronomical and SLR components to give combined storm surge and wave setup as per the example in Table 21. This allowed the design tide time series to be recomposited for application in the final base models.

Table 21. Example of storm surge values in Ōtaki Catchment at 2130 from HARC tide levels.

RAINFALL AEP%	TIDAL AEP%	DESIGN TIDE (mWVD)	STORM SURGE + WAVE SETUP (mWVD)
1	5	3.49	2.08
2	10	3.43	2.02
10	25	3.34	1.93
25	40	3.29	1.88
50	60	3.25	1.84

6.3. Summary

Design tide comprises of sea level rise, astronomical tide, and a combination of storm surge and wave set up. HARC provided final tide AEP% and tide levels from a joint probability analysis of storm rainfall and storm surge, which were used to generate the tidal timeseries in the final base flood models.

The intent of the joint probability analysis was to ensure that an appropriate level of conservatism was adopted in the coastal-catchment flooding interface, rather than to simply assume the worst-case scenario across the two domains (strong joint dependence). The analysis resulted in changes to peak tide levels of ±5%, the direction of the change depending on the catchment. Peak tide levels for the Paraparaumu catchment came out slightly lower (milder joint dependence). Waikanae catchment ended up with slightly higher peak tide levels, while the effect on Ōtaki catchment being largely neutral. The differences observed between the catchments is likely to be a function of the coastline geometry. Example design tide timeseries pre- and post-adjustment for joint probability are shown in Figure 26. A summary of tidal boundary conditions and sources is given in Table 22.

Table 22. Final base model tidal boundary conditions and sources.

TIDAL BOUNDARY CONDITIONS AND SOURCES
• Sea level rise projections (Takiwā Platform and MfE, 2022); • Astronomical tidal profiles for a spring tide (2/1/2022 - 4/1/2022) from https://tides.niwa.co.nz/; • Storm surge and wave set up were calculated back from the final tide levels provided by HARC Joint Probability Assessment (2023).

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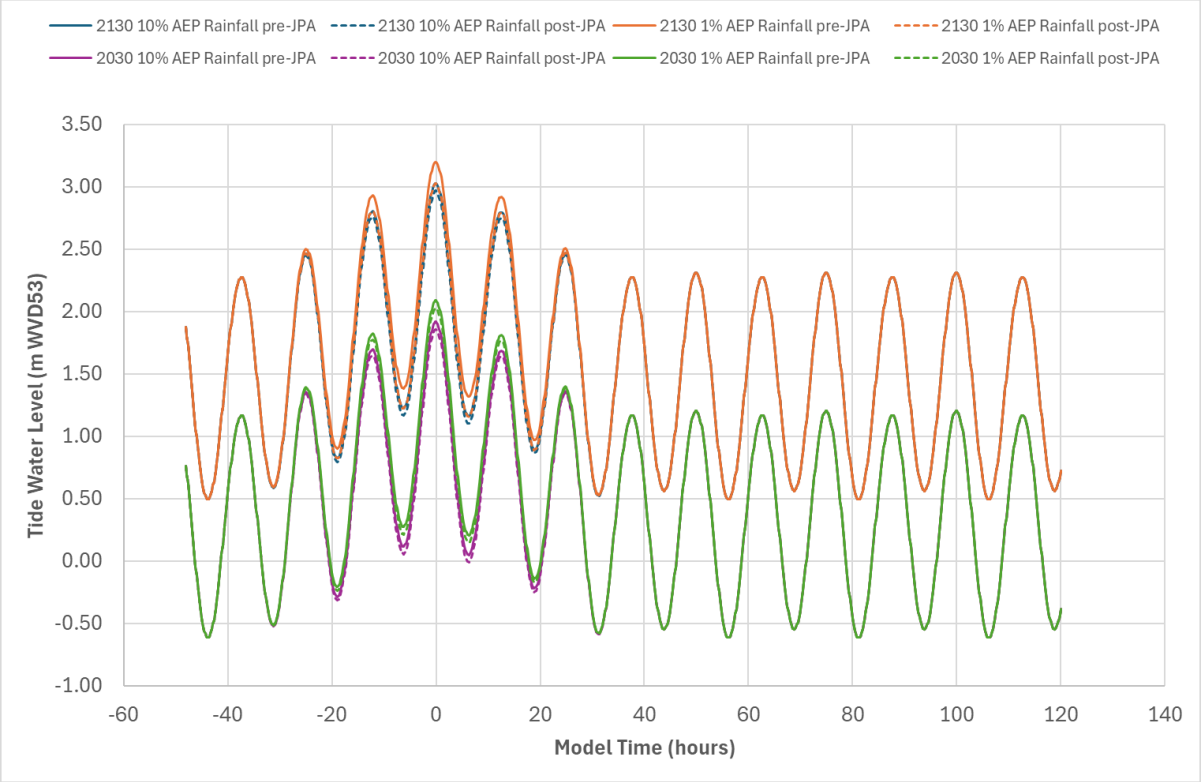


Figure 26. Paraparaumu Beach 1%AEP, SSP5-8.5M design water level boundary timeseries for 2030 and 2130 time horizons.

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7. Hydraulic Methodology

The TUFLOW hydraulic models include 1D networks, 2D networks, and climatic inputs. 1D network models pertain to the stormwater pipe network that simulate transfer and storage of stormwater on a one-dimensional scale. 2D network models simulate overland flow in two dimensions. These two networks are connected together to simulate the movement of stormwater from one place to another (e.g. from roads to pipes to rivers). The manner in which these integrated model components work is comprehensively described in BMT (2018).

The application of the 1D and 2D models to represent the Kāpiti Coast urban catchments, and their integration, are elaborated in the sections below. Details on the model build for each catchment are documented in their respective model build reports, which are provided in APPENDIX H.

7.1. 1D Network

The 1D network model is made up of points representing manholes, catch pits, sump, inlets/outlets and other network nodes, and lines that represent connecting pipes and culverts. Points are nodes that act as a connector or a storage feature such as a pit (sump), manhole, or outlet. Lines operate as a pipe and are represented in the model by pipes or culverts.

The 1D network data were sourced from KCDC's Geographic Information System (GIS) database, which was provided in 2021. An example of a 1D stormwater network from the Wharemauku catchment model is presented in Figure 27. Gaps in the network data were addressed by:

1. Site surveys,
2. TUFLOW's automatic network generation process,
3. Engineering judgement, and
4. Analysis using Awa's Urban Model Manager (UMM).

The UMM tool applies basic rules to ensure that a model network is contiguous.

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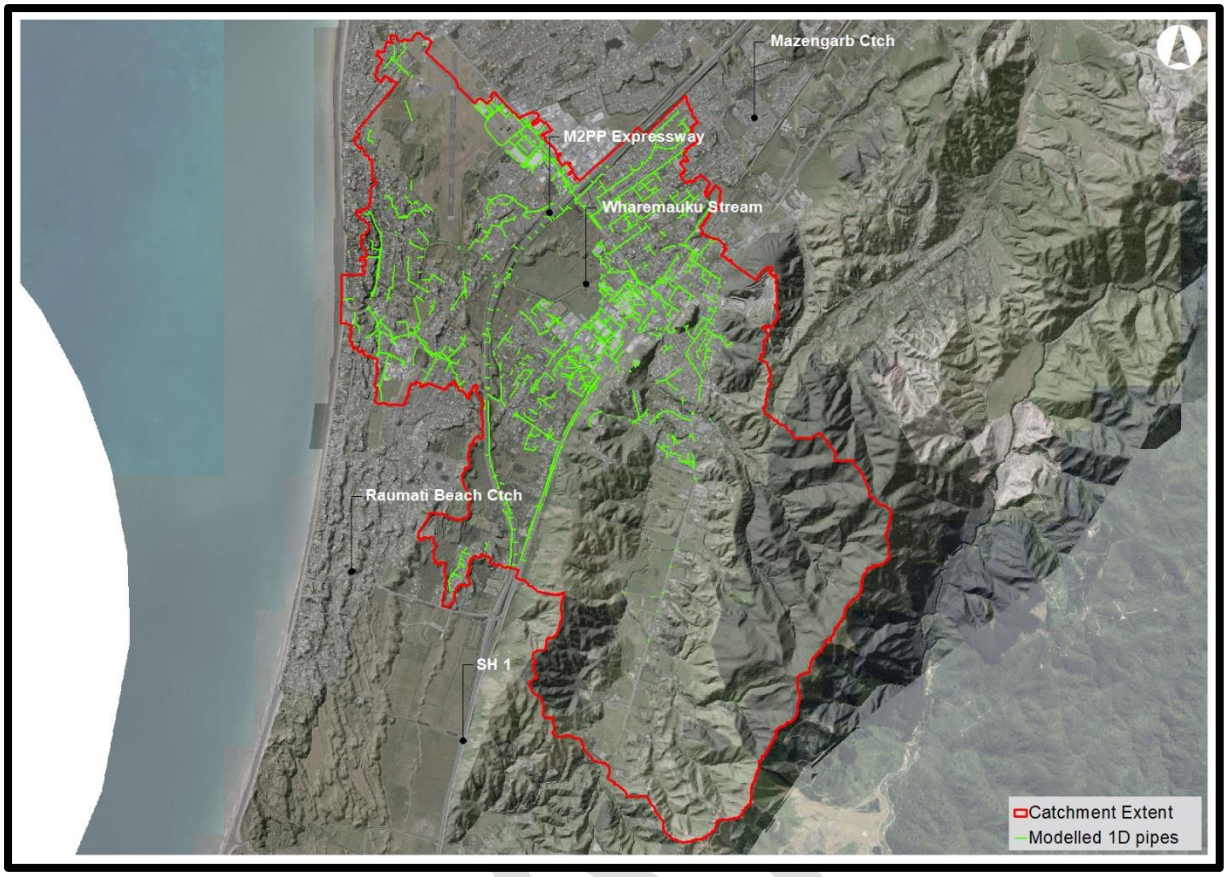


Figure 27. Example 1D network - Wharemauku catchment

7.1.1. Points

Points are nodes that allow for storage and connections between the 1D network components or between 1D and 2D networks. For example, connecting roads to pipes via sumps, or pipes to pipes via manholes. These points and nodes were sourced from KCDC's GIS and uploaded into TUFLOW. Or where data was missing, TUFLOW's automated network generation process was used to create nodes between pipe network features. Attributes such as type or storage capacity were added manually to the auto-generated points. The main types of points used in the Kāpiti models are described below.

MANHOLES

Aside from providing connectivity between pipe features, manholes in a 1D network model need to simulate the expansion/contraction of flow, and direction and level changes inside the structure and associated energy dissipation. The Engelund loss approach to simulating manhole losses was adopted across all of the Kāpiti Coast network models. Manhole lid levels are snapped to the surface of the model, which means that the chamber is not connected to the surface and is therefore unable to overflow/surcharge into the 2D model.

PITS

Pits (catchpits) represent connections between the model 1D network and 2D network (refer to section 7.2 below). Pits use water level inflow relationships to determine the type of pit used and the maximum inflow. The connected pipes were generally positioned halfway up the catch pit chamber unless

specified in KCDC's database. Where GIS and survey data were unavailable, application of the UMM tool assumed the depth of the pit to be 1.5 m below ground level and the connecting outlet is halfway up the chamber.

INLETS AND OUTLETS

Inlets and outlets are nodes connect to the end or start of a pipe/culvert that allows the 1D Network to take an inflow from the 2D network or to discharge flow into the 2D network. These features do not regulate flow; rather inflows/outflows are determined by the pipe/culvert characteristics.

NODE AREA (STORAGE NODES)

Node areas are storages that have the specific function of holding stormwater. For example, a pump station chamber would be represented in the model as a single node area with specific stormwater capacity. This information was sourced from KCDC and stored in CSV (comma separated variable) input files comprising rating tables of depth vs surface area.

SOAK PITS

Soak pits, a common drainage feature within the Kāpiti Coast region, drain water out of the network by allowing it to permeate into the groundwater table. In the TUFLOW models they were handled as boundary conditions assigned to nodes. All soak pits were classed as 1050-mm manholes with an underlying soakage rate of 300 mm/hr as recommended by KCDC.

7.1.2. Lines

Lines are 1D network features that allow for transfer of stormwater from one point to another. Three different types of lines were implemented within the 1D network models, with varying capacities and roughness values as explained below.

PIPES AND CULVERTS

Pipes and culverts function as a 1D line that transport stormwater based on the slope between the inlet and outlet. Pipe and culvert information were sourced from the KCDC GIS and surveys, with engineering judgement employed to fill any gaps.

Pipes and culverts are assigned attributes including size, inlet and outlet invert elevations, roughness (based on material), and bend, entrance and exits losses The network ID from the KCDC GIS was assigned to each pipe and culvert in the TUFLOW models for identification and cross referencing. Invert levels were taken from KCDC's database, and wherever there were gaps in the network data, a continuous grade was created using the UMM tool. Where negative grade pipes were specified in KCDC's data, the feature would be edited manually based on engineering judgement.

Pipe and culvert diameters and materials were also obtained from KCDC's GIS data. Where a pipe diameter data point was missing, the upstream and downstream network was used as guides to fill in the missing value. Concrete was adopted as pipe material wherever pipe material was not recorded. Pipes fitted with one-way outlet valves were designated as one-way flow direction in TUFLOW.

There are 3 types of culverts used in the models – Circular (C), Rectangular (R), and Irregular (I). Each culvert is assigned a storage volume based on diameter, and a length drawn from the GIS or survey

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data. Where culvert length is not available, TUFLOW automatically infers the length from the line geometry.

All pipes/culverts were assigned standard square entry and exit coefficients. Roughness, expressed as Manning's n coefficients, were assigned based on depending on the material type as per Table 23.

Table 23. Pipe and culvert materials and associated roughness.

PIPE MATERIAL	MANNING'S N
Concrete (smooth), copper	0.0005
Stone, asbestos cement, PVC	0.001
Concrete (normal), fibreglass	0.0015
Ceramics, iron (cast), stormboss, nexus	0.0025
Concrete (rough), timber, field tile,	0.003
Iron (wrought), corrugated steel, corrugated iron, corrugated aluminium, other corrugated metals, galvanised iron or steel	0.0035

Culverts in the Wharemauku Stream were modelled as 'irregular' (I) shaped culvert as the complex shapes could not be modelled in the shapefile using the base and width rows. The cross-section area was specified as a height-width relationship in a CSV file to be read into TUFLOW. The information has been taken from KCDC's Legacy MIKE model unless the structure had been surveyed.

1D OPEN CHANNELS

The Waikanae River in the Waikanae and Paraparaumu Beach catchments has been represented as a 1D open (inertial) channel line to improve efficiency in the modelling process since the tailwater boundary is heavily affected by the tides.

7.2. 2D Network

The 2D network is the plane over which water can move on two dimensions. This means lateral movement of water on a surface rather than within a pipe where it moves directly from one point to the other. The 2D network applies to most open surfaces including the ground surface, open channels (e.g. rivers), roads, carparks and storage areas (e.g. ponds). An example of a 2D model domain taken from the Wharemauku catchment model is given in Figure 28.

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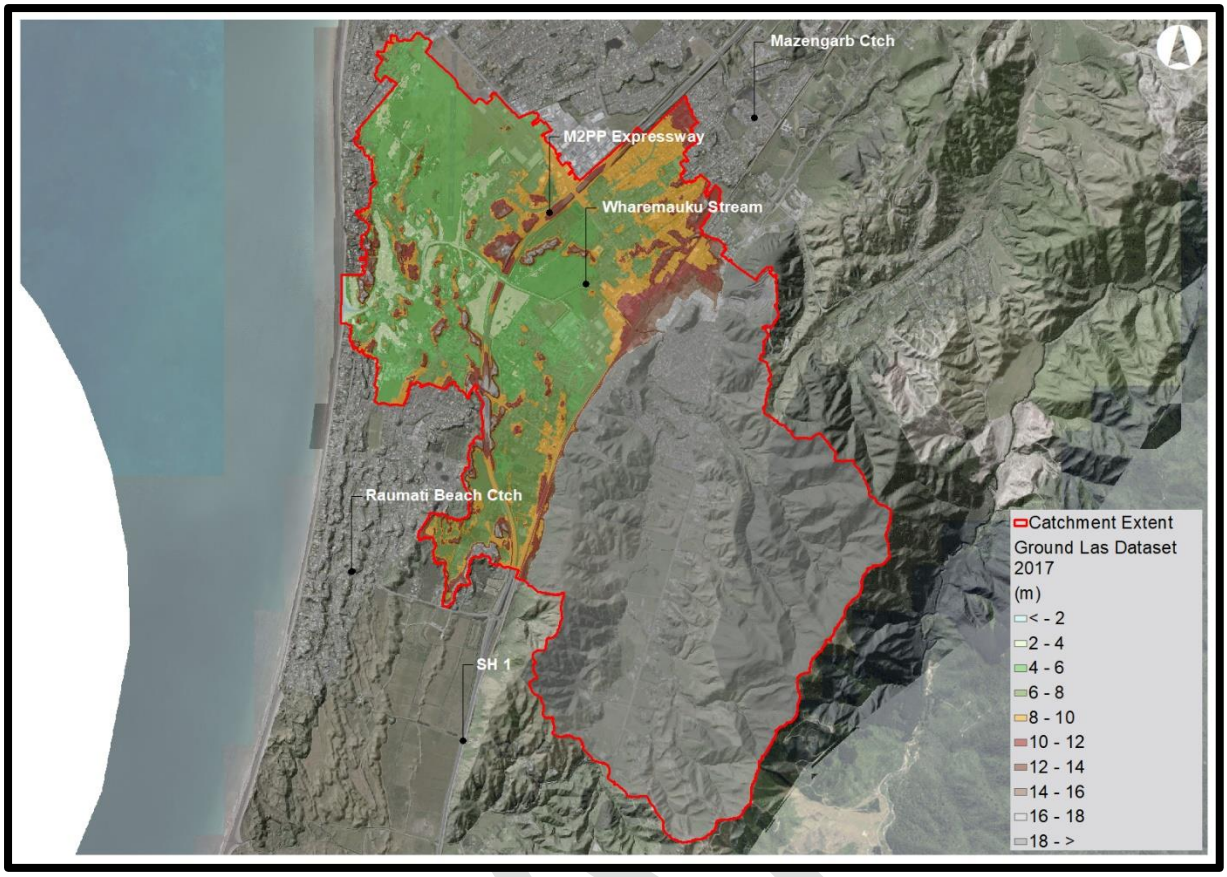


Figure 28. Wharemauku catchment 2D model domain.

Water movement over a 2D network, or overland flow, is dictated by flow resistance, a combination of friction and drag energy losses, which is simulated by applying Manning's roughness coefficients (n) to the shallow-water wave momentum equation that is used to solve for 2D flow. The higher the Manning's n value, the 'rougher' the surface and the greater the resistance to flow.

7.2.1. 2D Network Forms

OPEN CHANNELS

Open channels in TUFLOW are the rivers, streams and engineered channels which operate on the two-dimensional scale. This means that the water can travel and spread across the 2D-plane depending on the topography and depth. They are represented using a surface mesh generated from a raster file. Table 24 lists the water bodies that have been represented as 2D open channels.

Table 24. Open channels represented as 2D network features in the models.

CATCHMENT	RIVERS AND STREAMS IN 2D
Otaki	Ōtaki River, Waitohu Stream, Mangapouri Stream
Waikanae*	Waimeha Stream, Ngārara Stream
Paraparaumu	Tikotu Stream
Mazengarb	Mazengarb Stream
Raumati	None
Wharemaukū	Wharemaukū Stream
Paekakariki	Wainui Stream, Te Puka Stream

*Waikanae River is represented as 1D in the Waikanae and Paraparaumu Beach catchments as explained in section 7.1.2.

GROUND SURFACES (OVERLAND FLOW)

Overland flow is simulated over all ground level surface areas within catchment boundaries such as carparks, roads, paddocks, open areas, and parks. Ground surfaces are informed by the DEM (section 3.1) and sub-grid sampling from TUFLOW (see Section 7.2.1).

The movement of runoff or overland flow on ground surfaces are determined by elevation and 2D overland flow roughness which are elaborated further in Section 7.2.2 below.

NETWORK STORAGE NETWORKS

Storage features that are represented within the 2D network include stormwater ponds, wetlands, depressions and raingardens. These features allow for storage of stormwater and operate in 2D where the stormwater can move and spill over in the nearest and lowest elevation cell.

7.2.2. Sub-grid sampling

Sub-Grid Sampling (SGS) refers to the processing of the DEM within TUFLOW to provide a higher resolution. SGS provides interpolation within the 2 by 2 m grid of the DEM to produce a 1 by 1 m hydraulic grid within the TUFLOW model. This allows for a more representative surface in the TUFLOW model as shown in Figure 29. By utilising the underlying sub-grid scale topography, it is possible to more accurately represent the storage and conveyance that is possible within the system being modelled, particularly around open channels.

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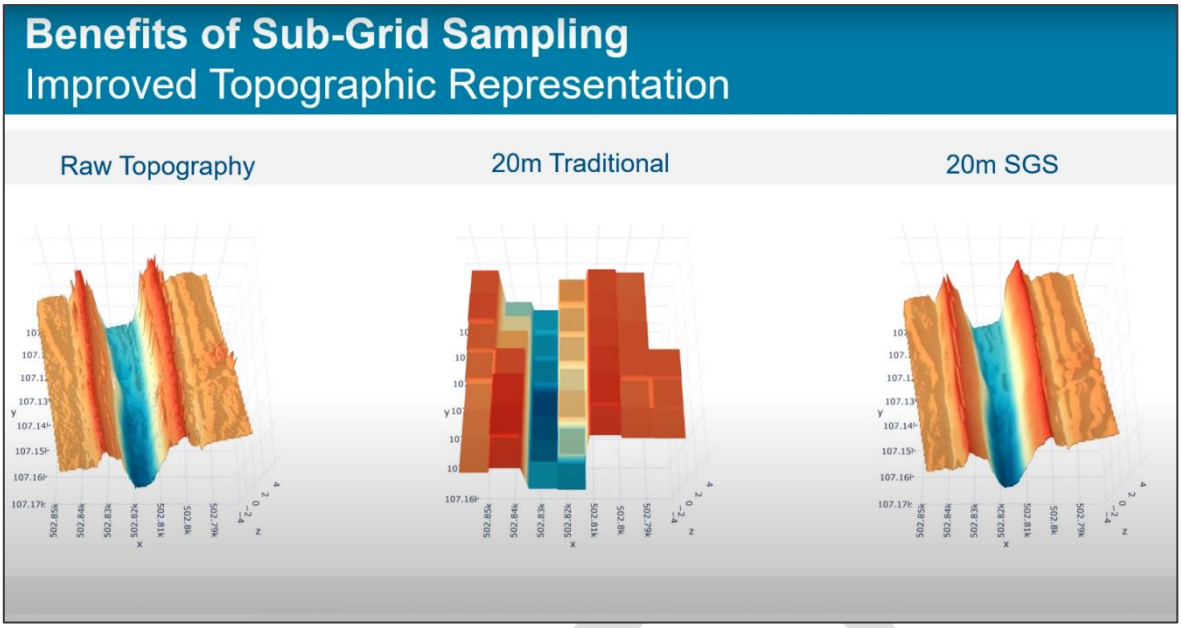


Figure 29. Sub-grid sampling concept example. Source: BMT

7.2.3. 2D Overland Flow Resistance

Overland flow resistance, a combination of friction and drag energy losses, is simulated by applying Manning's roughness coefficients (n) to the shallow-water wave momentum equation used to solve for 2D flow. The higher the Manning's n value, the 'rougher' the surface and the greater the resistance to flow.

Overland flow roughness coefficients are based on the 2021 land cover layer generated from Lynker, and Land Cover Database v5 (LCDB v5) data as described in section 3.2.2. Manning's (n) roughness coefficients have been applied to the landcover classification as per Table 25 to produce a spatial distribution for the model build. Importantly, the Manning's n value for buildings was changed from 5 to 0.02 as building perimeters have been used to simulate building flow interference instead of roughness (see section 7.4.1). Figure 30 shows the spatial application of roughness values to the Wharemauku catchment.

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Table 25. Roughness values applied to land cover classifications.

ROUGHNESS CLASS	MANNING'S n	SOURCE
Forest	0.1	WWL, 2017
Paved	0.02	WWL, 2017
Scrub	0.2	Arcement and Schneider, 1989
Residential obstructions	0.1	WWL, 2017
Maintained grass	0.03	Awa
Channel bed	0.025	Awa
Rural grass	0.04	Awa
Crops	0.2	Arcement and Schneider, 1989
Gravel	0.03	TCC, 2017
Building	0.02*	Awa

*Buildings modelled as objects with external surface roughness rather than using roughness coefficient applied to the ground level.

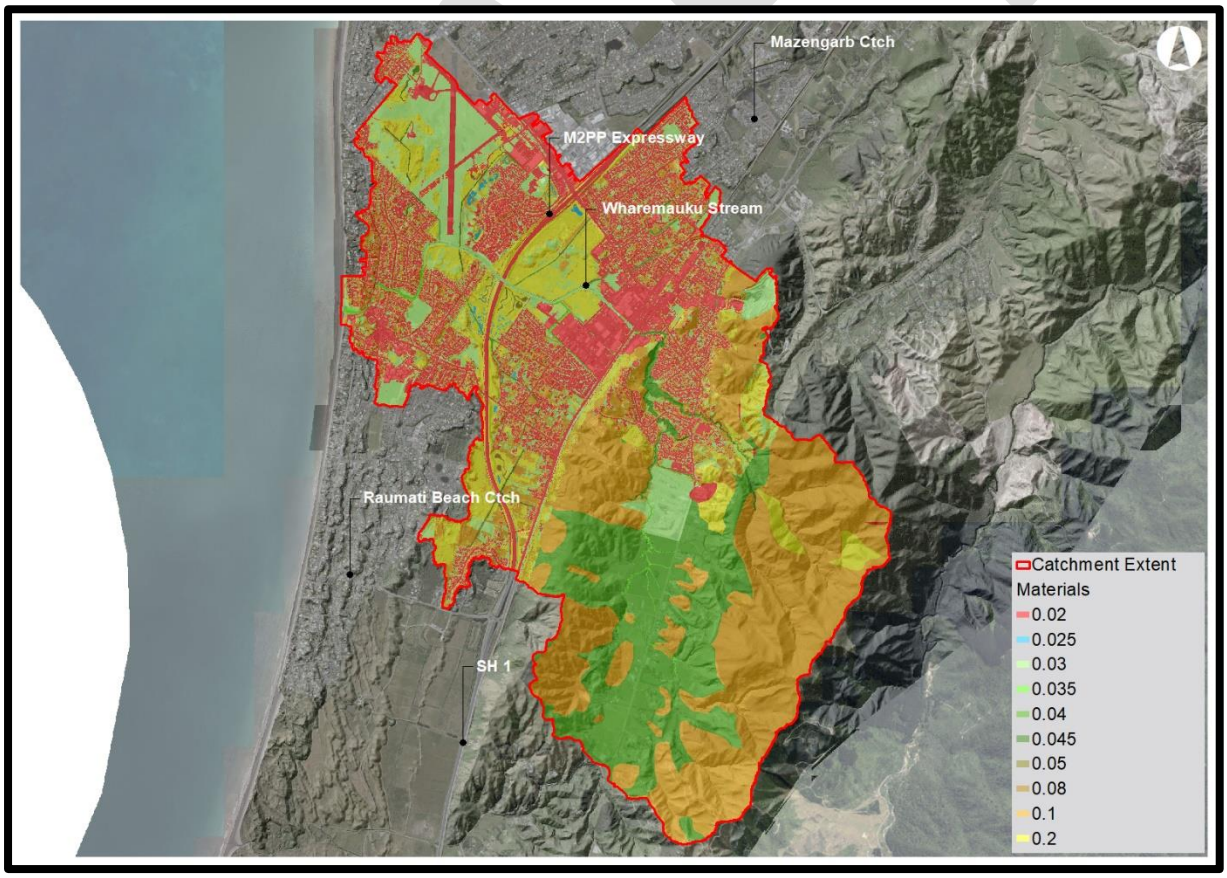


Figure 30. Mannings overland flow resistance parameters applied to the Wharemauku catchment model

7.2.4. 2D Open Channel Roughness

Roughness in open channels is digitised as polygons within a spatial layer that cover certain areas of open channels in accordance with the varying ground cover conditions. The initial Manning's n values adopted were largely based on guidance from GWRC (2021). However, the values were subjected to adjustment through the calibration and validation processes (section 8). The final open channel roughness values adopted in the model are shown in Table 26.

Table 26. Open channel roughness model inputs.

FEATURE	MATERIAL ID	MANNING'S N
River (channel bed)	5	0.03
Stream channel grass (maintained grass)	33	0.035
Stream channel grass (rough grass)	34	0.04
Stream channel grass (dense grass)	35	0.045
Stream channel grass (overgrown)	36	0.05 – 0.75
Pond or wetland	37	0.08

7.3. 1D and 2D Integration

Linkages between the 1D and 2D networks were implemented using the techniques described in the TUFLOW manual (BMT 2018). The integrations have been reviewed by BMT TUFLOW for quality assurance purposes.

Flows from a 1D network into a 2D network were determined by head, slope, pipe roughness, and outlet diameter. For example, in the case of a stormwater pipe connecting to an outlet that discharges into a stream, flow is determined by the head pressure, slope (difference in elevation between the intake and outlet), pipe roughness, and diameter of the pipe. This discharge is applied at a singular grid cell or group of cells in the stream from which the discharging stormwater carries its momentum and then moves towards the adjacent cell with the lowest water level.

Flow from a 2D network into a 1D network were governed by the flow-head relationship. Flow-head relationship were determined based on the head pressure on the cell in which the 2D network connects to the 1D network. An example of this is a sump polygon with a scruffy dome which connects overland flow to the stormwater pipe network. During a rainfall event, stormwater will accumulate on the surface of the sump while the rainfall exceeds the infiltration rate of the sump area. The head pressure over the cell of the scruffy dome, along with the sump capacity, determine the movement of water from the sump (2D network) as it moves to the stormwater pipes (1D network). The flowrate will also be influenced by the pipe characteristics (e.g. pipe diameter, material).

The main forms of 1D-2D connections were applied in the catchment models:

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- Points (2D_BC_P) – 1D feature connected to a singular grid cell
- 2D boundary connection lines (2D_BC_L) – connects a 1D point/line to multiple 2D grid cells
- 2D Polygons (2D_BC_R) – connecting large 1D feature to multiple 2D grid cells defined by a polygon
- Sub-catchment storage area discharge polygons (2D_SA_R)

Examples of 1D-2D integrations are given in Figure 31. For more information, please see the TUFLOW manual (BMT 2018).

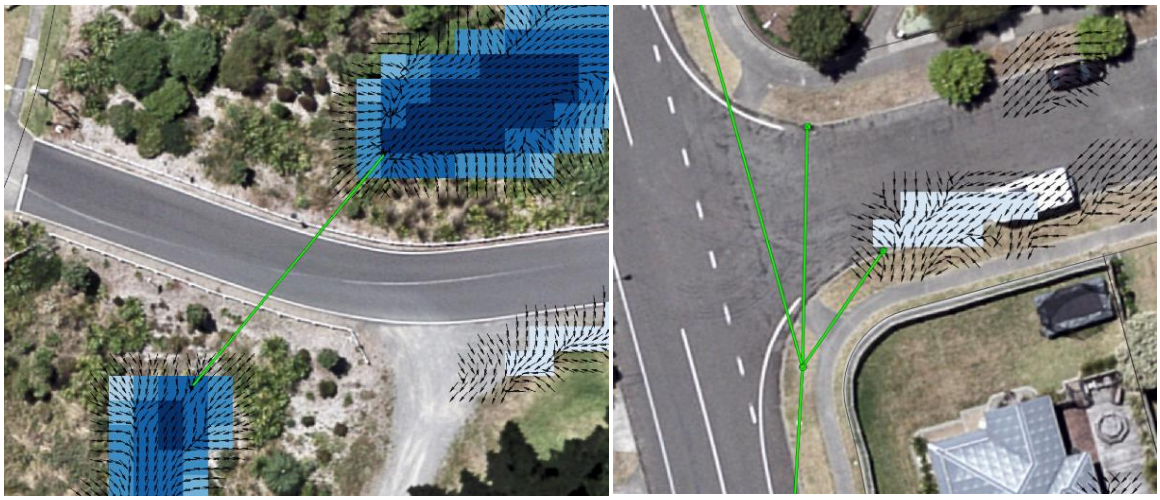


Figure 31. 1D-2D model integration examples: Water flowing into a 2D_BC_P and entering a 1D pipe then connecting back to the 2D surface through a 2D_BC_P (left) and water flowing into a 1D pit to enter the 1d network (right)

7.4. Overland Flow Obstructions

Obstructions such as buildings, stop-banks, and pump stations can exert significant impacts on overland flow by creating constrictions that accelerate flow or blockages that cause water to pond.

7.4.1. Buildings

In the absence of building downpipe information and to simplify the model build process, buildings were integrated into the 2D surface and assumed to not drain directly to stormwater reticulation. Incident rain runs off the sides of buildings to either collect on or run from the property. Initially building footprints had been raised 300 mm from highest point in the DEM within the footprint and assigned a roughness of 5. This caused the model timesteps to compress resulting in slow processing time and perverse outputs. Integrating buildings into the 2D surface reduces the model runtime and allows possible rainwater tank connections in the future if needed.

Buildings falling into the following ePlan Zones (classifications are taken from the KCDC Operative District Plan) were considered *commercial/industrial*:

- General Industrial Zone,
- Local Centre Zone,
- Town Centre Zone,
- Hospital Zone,

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- Metropolitan Centre Zone, and
- Airport Zone.

All other buildings were considered *residential*.

The approach to modelling restrictions associated with residential, commercial, and industrial buildings is summarised in Table 27.

Table 27. Method for representing buildings.

BUILDING TYPE	ASSUMPTIONS AND INTENTIONS	MODELLING APPROACHES
Residential	All stormwater spills onto the ground around the building, approximating blocked gutters. • No distinction is made between construction types. • It is assumed that buildings are solid and water cannot enter through doors or windows. • Consequently, buildings do not provide floodplain storage.	• Buildings with slab-on-grade foundations on compacted soil were set to a uniform elevation equal to 100 mm above the maximum DEM elevation within the footprint (with future flood damage analysis in mind). • Buildings on pile foundations on uncompacted soil were set to a uniform elevation equal to 400 mm above the maximum DEM elevation within the footprint (with future flood damage analysis in mind). • All residential buildings were then assigned a height of 2.5 m. • Roughness on all building surfaces was set to a uniform Manning's n of 0.02. • The building footprint is completely impervious.
Commercial/Industrial		• Buildings were set to a uniform elevation equal to 3.0 m above the maximum DEM elevation within their footprint. • Roughness is set to a Manning's n of 0.02. • The building footprint is completely impervious.

7.4.2. Stop-Banks

Stop-banks were included directly in the topography through the DEM as they are generally wider than the grid resolution. The crest levels have been refined within the topography with surveyed levels where the LiDAR is inadequate. The following stop-banks were integrated into the DEM (see also Figure 32).

- Ōtaki Catchment – Chrystal's
- Waikanae River – El Rancho
- Wharemaukū Stream – Titoki, Kiwi Rd, Ihakara St/Rimu Rd
- Whareroa Stream (Paekākāriki) – Tilley Rd

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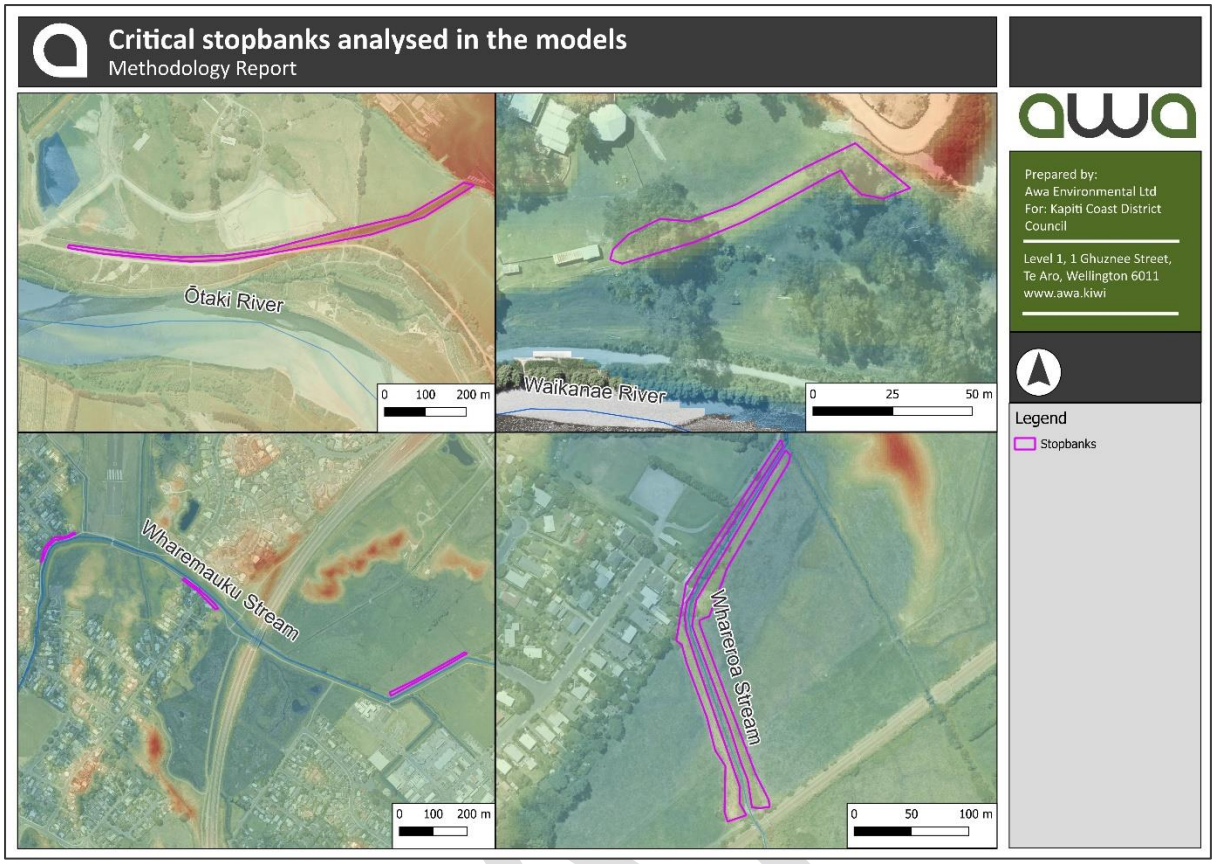


Figure 32. Stopbanks incorporated into the flood models.

7.4.3. Flood Walls

Initially, flood walls were considered as separate structures with zero thickness and crest levels to allow for overtopping in the flood models. However, simulation instabilities and the option to use the 2D sub-grid sampling technique in the 2D model domain meant that incorporating the structures into the DEM was a superior approach. UMM was used to make the adjustments, which were then entered into TUFLOW using the ZSH layer.

7.4.4. Pump Stations

Pump stations are used to mitigate flooding by conveying stormwater away from a flooded area. KCDC has several operating stormwater pump stations in the district, which have been incorporated into the flood models. The modelled pump stations – locations shown in Figure 33 - were set to operate at specified rates in accordance with data provided by KCDC (i.e. volume or level triggers).

Note that only KCDC pump stations were included in the modelling; private pump stations were not considered as asset data was not available.

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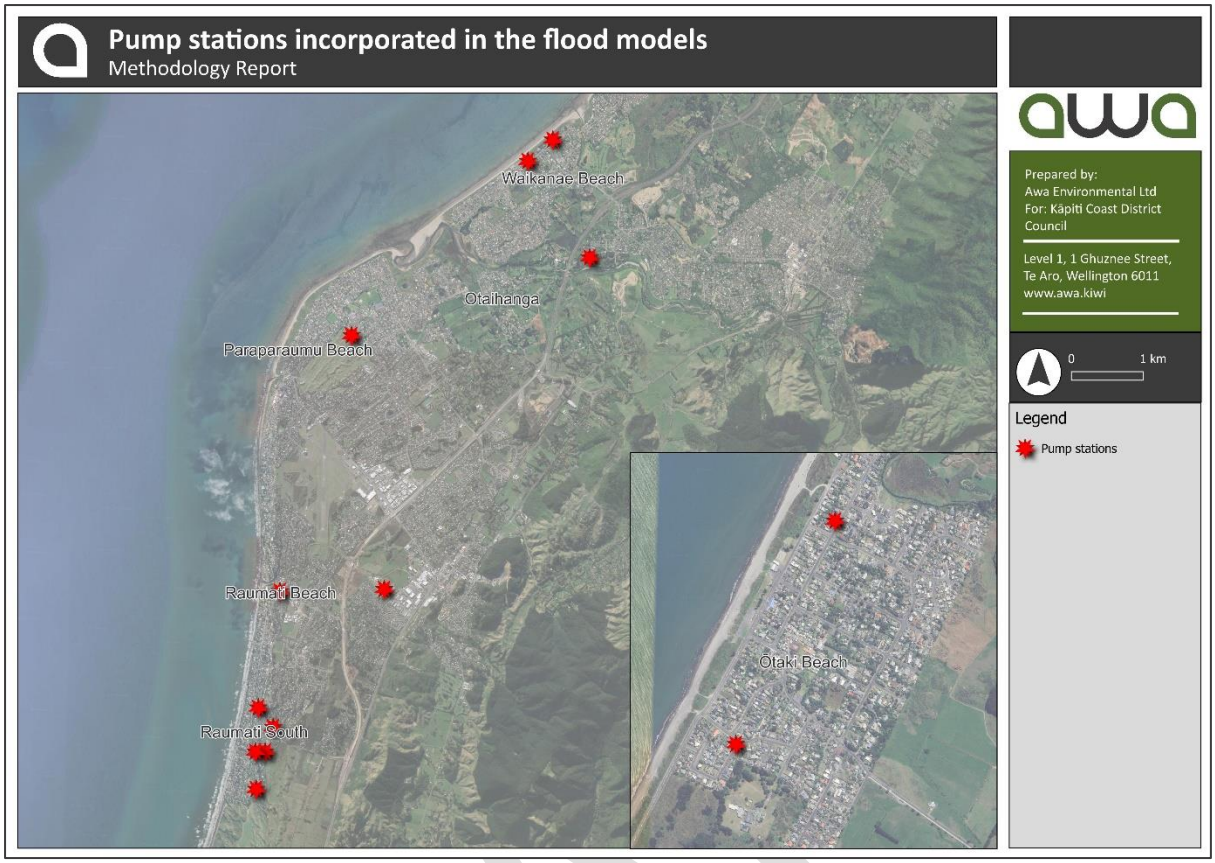


Figure 33. KCDC pump stations in the flood models.

7.5. Quality Assurance

Quality assurance processes were conducted with BMT (TUFLOW Developers) and KCDC.

Initially, a pilot study was applied to the Paraparaumu catchment for KCDC review in 2021. This study detailed the flood modelling approach, which was developed in consultation with BMT, and sought input from KCDC on the hydrologic and hydraulic methodologies employed in the models. A copy of the report is provided in APPENDIX I.

Subsequent early model builds were provided to BMT TUFLOW for peer review. Reviews canvassed model setup and hydraulic checks. Additionally, a draft Model Build Report for the Mazengarb Catchment was submitted to BMT in accompaniment to the Mazengarb model. An updated report incorporating feedback from BMT was then used as the reference report for the other model build reports.

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8. Model Calibration and Validation

The Kāpiti Coast comprises two main types of (sub-)catchments – steep hill catchments with minimal urban development and urbanised lower lying catchments. As outlined in section 4.2, two hydrological approaches were taken to modelling the different catchment types. The upper hill catchments were modelled using a standard sub-catchment rainfall-runoff methodology while the lower lying and coastal plain catchments were modelled using the TUFLOW rain-on-grid methodology. Where broader catchments comprised sub-catchments of both types, a hybrid approach combining the two methodologies was adopted.

The Kāpiti district contains only one local gauging station that is suitable for catchment model calibration. Located on the Wharemauku Stream just upstream of the Coastlands shopping centre, the station has been used to calibrate runoff parameters for a mode of the upper Wharemauku catchment. Whilst there are gauging stations on other major water courses in the Kāpiti district, namely the Waitohu Stream, Otaki River, Mangaone Stream and Waikanae River, they all sit at the upper reaches of the urbanised catchments and are fed by large mountain range catchments that have not been mapped to a DEM. Hence data from those gauging stations were used as boundary conditions only.

In the absence of gauging stations to calibrate the remaining (sub-)catchments, a validation process was applied to each model whereby flood extent outputs were compared with historical flooding records. This allowed refinement of certain model parameters to better reflect the flooding characteristics of the catchments in question. It also provided a check on the construction of the model as it can highlight areas where flooding may have been recorded but it is not represented by the model and vice versa.

8.1. Hill Sub-Catchment Calibration

Model calibration was performed on the upper portion of the Wharemauku catchment that lies upstream of the Wharemauku Stream gauging station as shown in Figure 34. The gauged portion of the Wharemauku Catchment comprises steep hilly and valley floor sub-catchments. Accordingly, the hybrid approach of combining sub-catchment and rain-on-grid methodologies was adopted for the model of the gauged catchment depicted in Figure 44.

For the upper hills sub-catchments, data on land use, land cover and soil type from the Land Cover Database were combined with empirical rainfall to build a Clark Unit Hydrograph using Awa's THUNDER software[17]. The unit hydrographs were fed into a TUFLOW model of the valley basin immediately upstream of the gauging station, which merged them with the runoff generated from the rain-on-grid outputs. This produced stream level and flow outputs at the gauging station location that could be compared with the observed data for corresponding storm events.

Calibrated models of the Wharemauku catchment have previously been produced by SKM (2009) and by Connell Wagner (2001). The outcomes of both past calibrations have informed the current calibration. Additionally, the previous calibration was compared to other available Wellington steep hill

[17] Based on the HEC-HMS hydrological modelling software, incorporating the Soil Conservation Service (SCS) Curve Number loss model and the Clark Unit Hydrograph routing model.

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catchments to ensure consistency. Most recently calibration was undertaken on a number of gauged hill catchments including the Horokiri, Pauatahanui, and Porirua Streams as part of the consenting of Transmission Gully (Fountain et al. 2011).

The process of calibration and validation of the upper Wharemauku model is outlined below. Full details on the process and outcomes are provided in the model build report for the broader Wharemauku Catchment (Awa Environmental 2024) available in APPENDIX H.

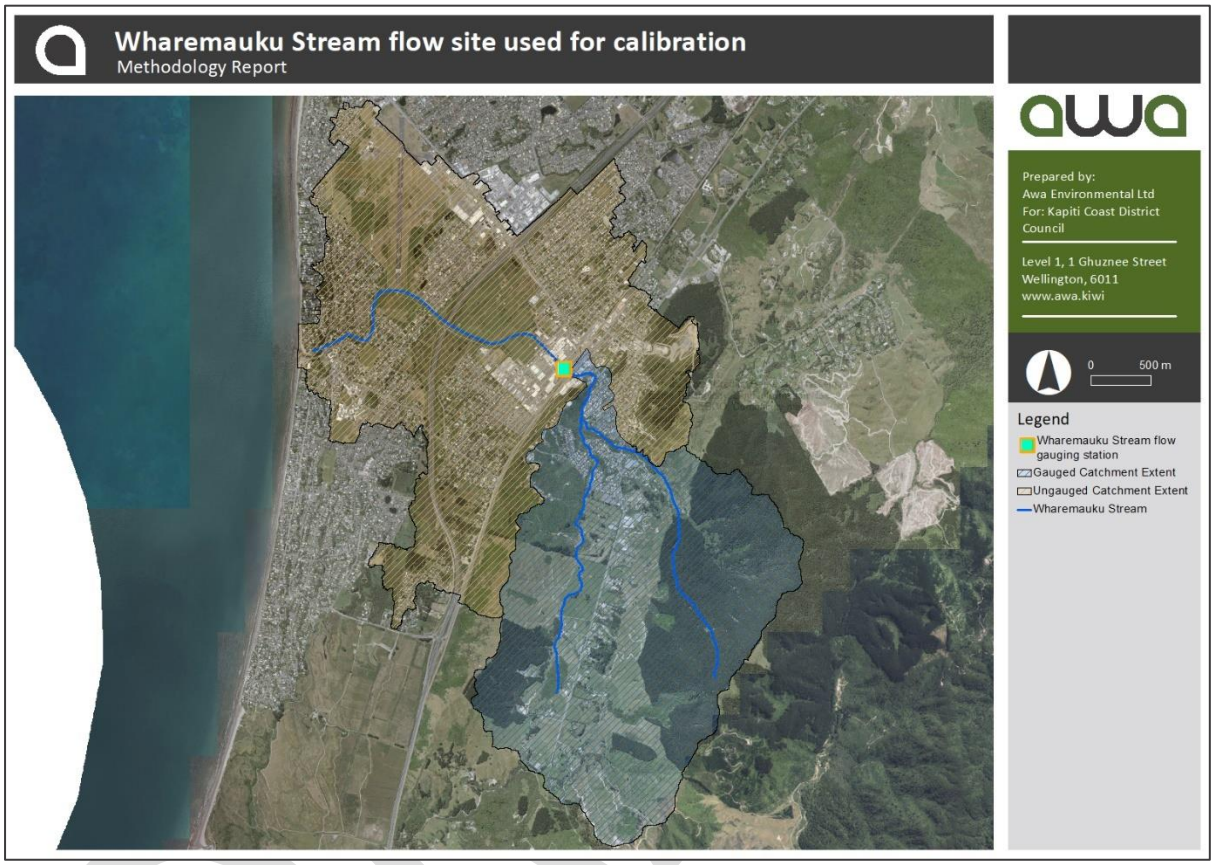


Figure 34. The Wharemauku Catchment and stream gauging station. The blue-hatched area indicates the gauged portion of the catchment that was subject to calibration

8.1.1. Calibration process

The following variables were adjusted to calibrate the upper hill sub-catchment model components:

- Rainfall scaling factor to account for orographic effects associated with the Tararua range based on SKM (2011);
- Initial abstraction (loss);
- SCS curve number;
- Connected impervious area;
- Time of concentration;
- Ratio split of 'slow' and 'fast' catchment areas.

For the rain-on-grid valley floor sub-catchment 2D model domain, bed resistance in the form of Manning's n values and infiltration rates (initial and continuing losses) were adjusted in the respective GIS layers.

Calibration of model outputs was performed first against observed flow recorded at the Wharemauku gauging station (shown in Figure 35 and Figure 36 below) for a large storm event that occurred on 14 May 2015.

Further adjustments were made through calibrating to a storm on 6 December 2021.

The calibration considered peak flow, the shape of the broader flow time series and total runoff volume. Rainfall from the two storm events measured at the various rain gauges in the district are plotted in Figure 37 and Figure 38.



Figure 35. Wharemauku Stream gauging station location (structure highlighted in yellow)



Figure 36. Wharemauku Stream gauging station

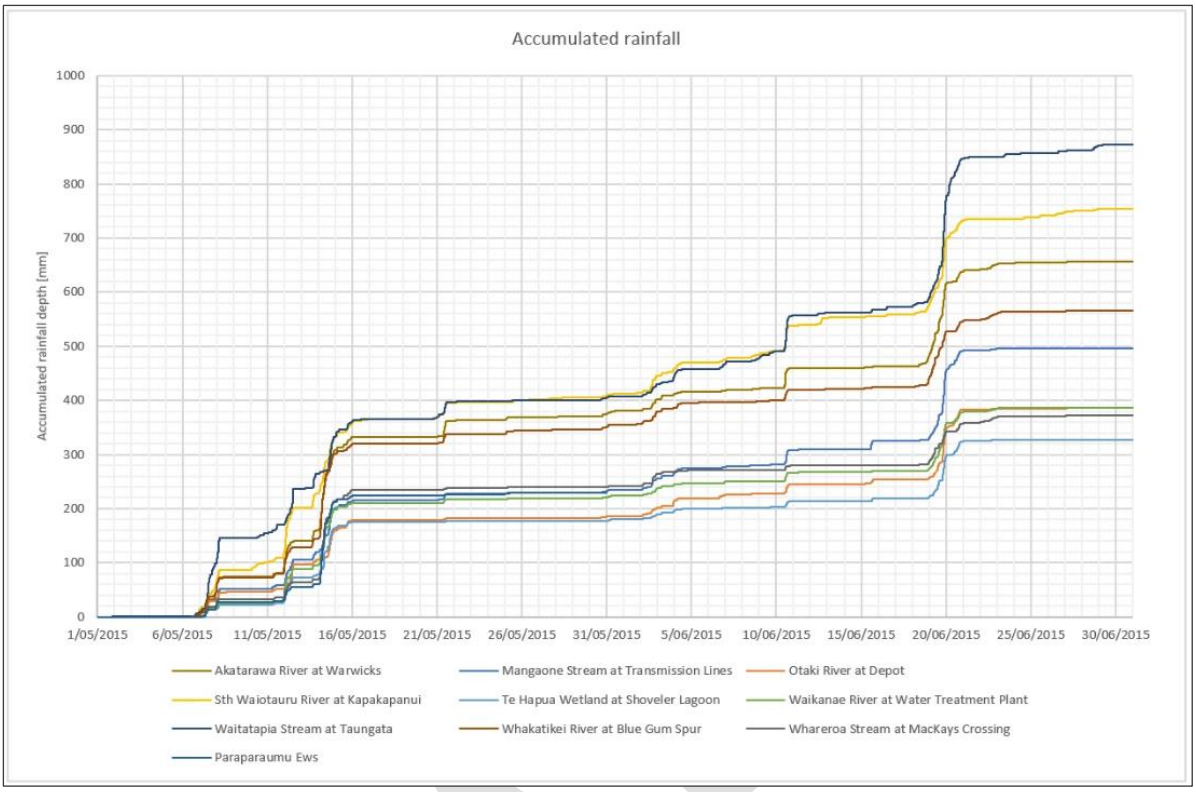


Figure 37. May 2015 Storm event.

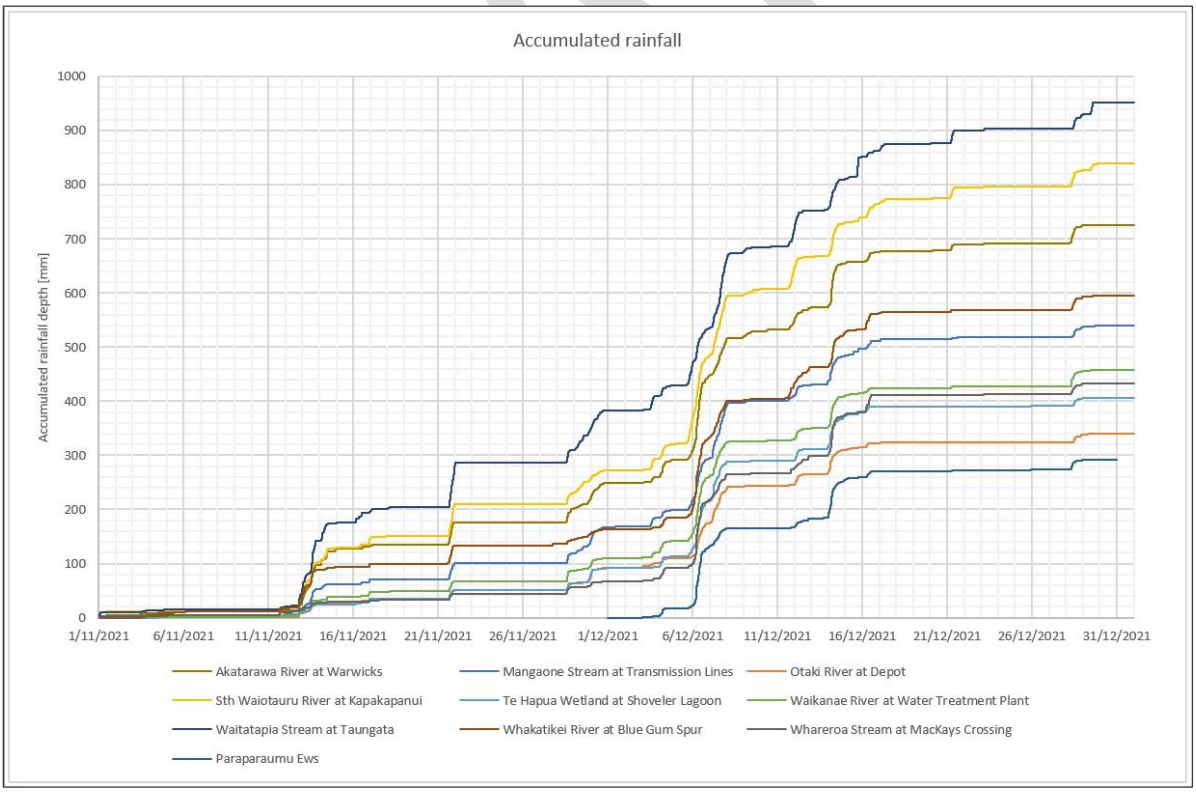


Figure 38. December 2021 Storm event.

8.1.2. Boundary Conditions

Boundary conditions for the historic calibration events were based on field measurements:

- rainfall gauge recordings,
- tide gauge recordings, and
- borehole recordings.

Gauged rainfall data were distributed across the Wharemauku catchment using Thiessen polygons.

Table 29 summarises the data used in deriving the historic event boundary conditions. Additional detail on the tides and groundwater levels is provided in the following sub-sections.

Table 28. Data referenced for deriving historic event boundary conditions.

DATA TYPE	DESCRIPTION	INTERVALS	COMMENT	SOURCE
Rainfall	Measured rainfall data (mm) extracted from NIWA rainfall stations around Kāpiti.	Fixed 5-min, 10-min, 30-min, 1-hr, and 24-hr intervals.	Not all stations had complete records.	Cliflo (NIWA)[18]
	Measured rainfall data (mm) extracted from GWRC rainfall stations around Kāpiti.	Fixed 1-hr and 24-hr intervals.		GWRC Environmental Monitoring Data[19]
	Measured rainfall data for the last 20 years from specific sites.	5-min intervals	Requested data from GWRC	GWRC Environmental Monitoring Data
Tide Level	Measured tide level (mm) from the tide station at Porirua Harbour.	5-minute interval. Received in 1-minute intervals.	There is no tide station in Kāpiti Coast. The closest one is in Porirua Harbour.	GWRC Environmental Monitoring Data
Groundwater	Measured ground water levels (mm) from Kāpiti Coast boreholes.	All recordings are on 15-min intervals apart from one 5-min.		GWRC Environmental Monitoring Data
Stage and Discharge	Measured stream and river levels (mm), flow (m^3s^{-1})	5-minute intervals	There was no data for a station on Waitohu Stream.	GWRC Environmental Monitoring Data

[18] https://cliflo.niwa.co.nz/pls/niwp/wgenf.genform1
[19] http://graphs.gw.govt.nz/

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TIDES

Unlike modelling of future flood scenarios in which design tides are applied to the coastal boundary, the calibration simulation required a sea water level timeseries representative of the tidal conditions at the time of the calibration storm events. To develop this, outputs from the NIWA Tide Forecaster were compared with observation drawn from the Queens Wharf tide gauge in Wellington Harbour and the at the Mana Cruising Club tide gauge in Porirua Harbour. The Porirua Harbour gauge was found to be approximately 1 hour behind the forecasted tides while the Wellington Harbour gauge was around 3 hours out of phase. The Porirua Harbour gauge also had a reduced range compared to the Forecaster tide and is known to register tidal fluctuations 25 minutes behind the harbour mouth on account of the shape of the harbour. Tidal predictions from the Forecaster were deemed most suitable basis for generating boundary condition for the calibration as they provided the most appropriate timing and likely range experienced on the open coastline of Kāpiti.

The Forecaster was used to generate a tidal timeseries for Paraparaumu Beach, which was considered applicable to all catchments south of the Waikanae River mouth, and therefore the Wharemauku model calibration. Tidal predictions were also generated for the Waikanae and Otaki River mouths to be used in the validation of the seven full catchment models (refer to 8.2). The timeseries data were adjusted to match the model datum (WVD 1953) and sea level rise as per section 4.5. The calibration/validation tidal timeseries data are presented in Figure 39.

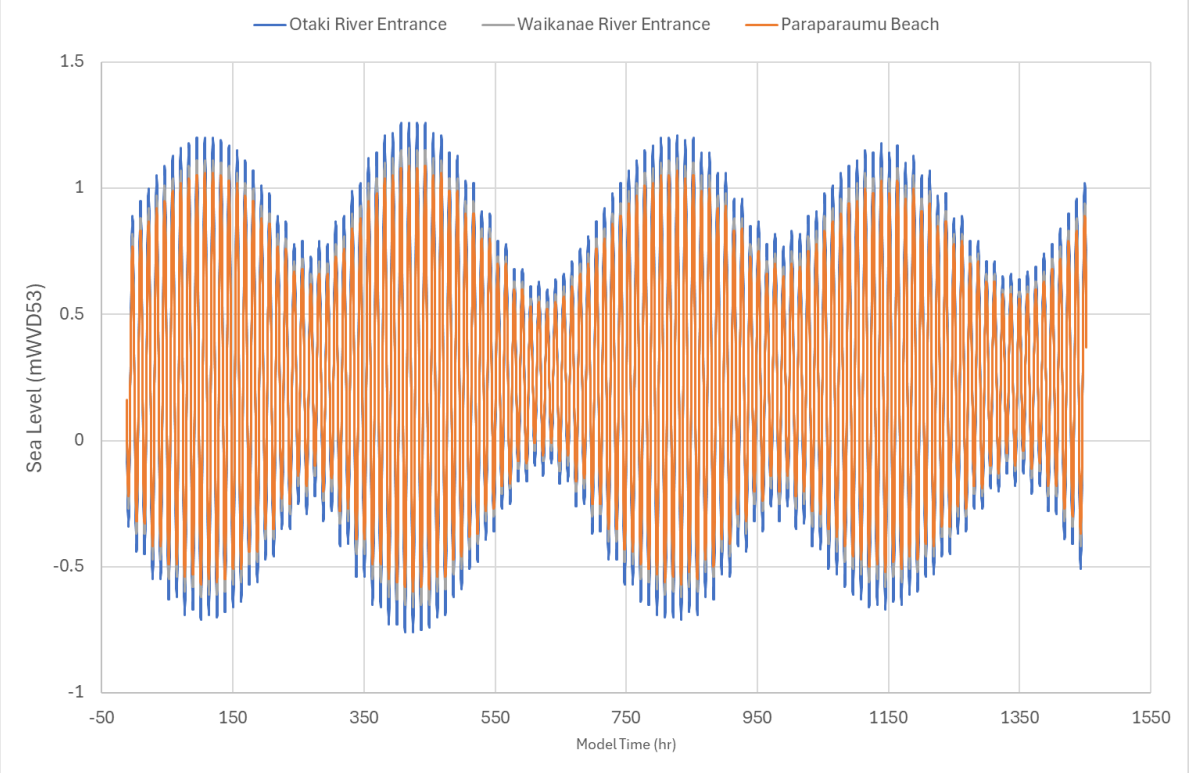


Figure 39. Model calibration and validation tidal boundary timeseries

GROUNDWATER

Much of the flooding experienced on the Kāpiti Coast is groundwater related. This is particularly the case in unusually wet years such as 2004 and 2021-22. Groundwater levels recorded at Rangihiroa Street, Waikanae Beach (R26/6287) and Larch Grove, Paraparaumu (R26/6831) recorded 3 distinct groundwater level peaks in 2015, 2021 and 2022, as shown in Figure 40. These peaks correspond with elevated rainfall in those years, as shown in Figure 41.

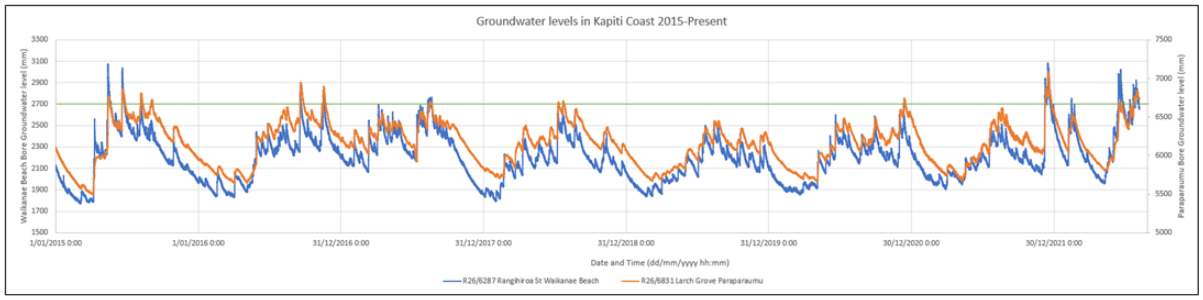


Figure 40. Groundwater levels 2015 - 2022.

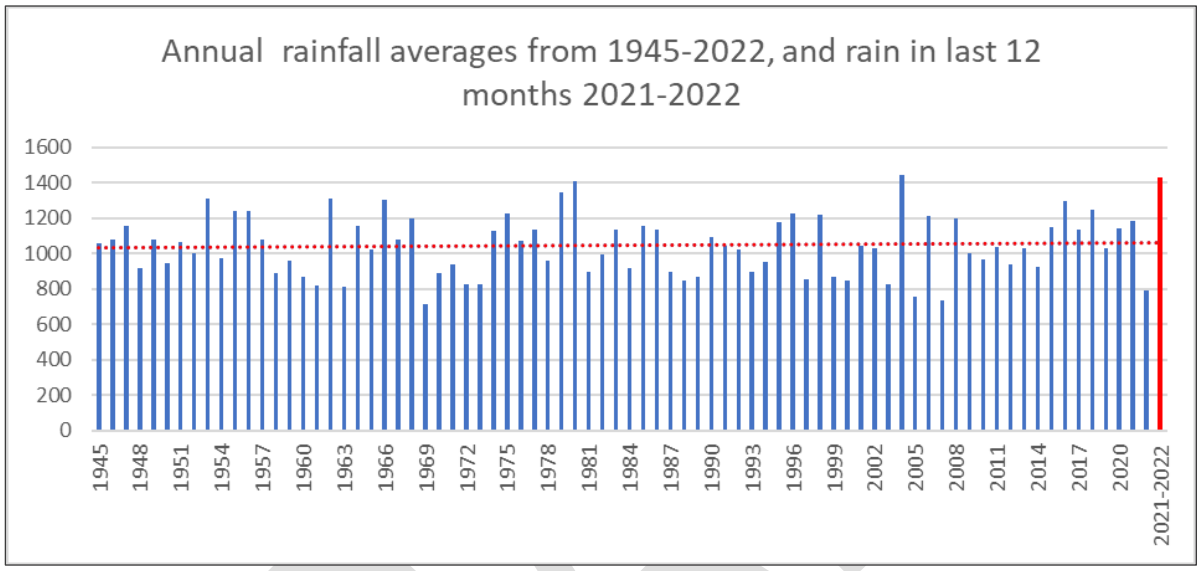


Figure 41. Observed annual and average rainfall 1945-2022

The interface between groundwater and surface water has been incorporated into flood modelling for the first time. However on account of there being a limited number of continuously monitored bores, historical observations could not be used to set the initial groundwater conditions across the 2D model domains. An attempt was made to formulate an adjustment to the modelled 50th percentile groundwater surface (see section 4.7) to align it with conditions preceding the 14 May 2015 and 6 December 2021 floods by comparing modelled levels to monitoring bore data. However differences between the model data and corresponding bore logs (shown in Figure 42 and Figure 43) varied significantly across the district and did not reveal a clear trend.

Consequently, 50th percentile groundwater surface was adopted as the initial groundwater conditions for both calibration events.

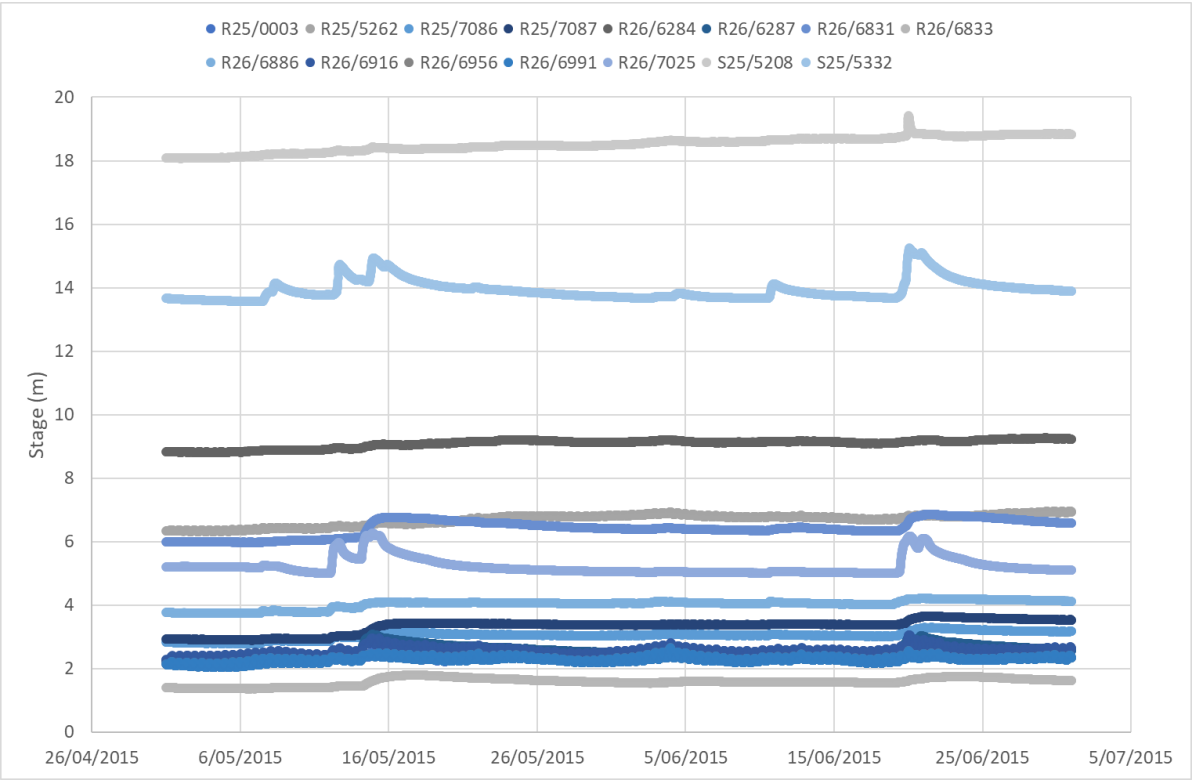


Figure 42. May-June 2015 groundwater stage records (source GWRC)

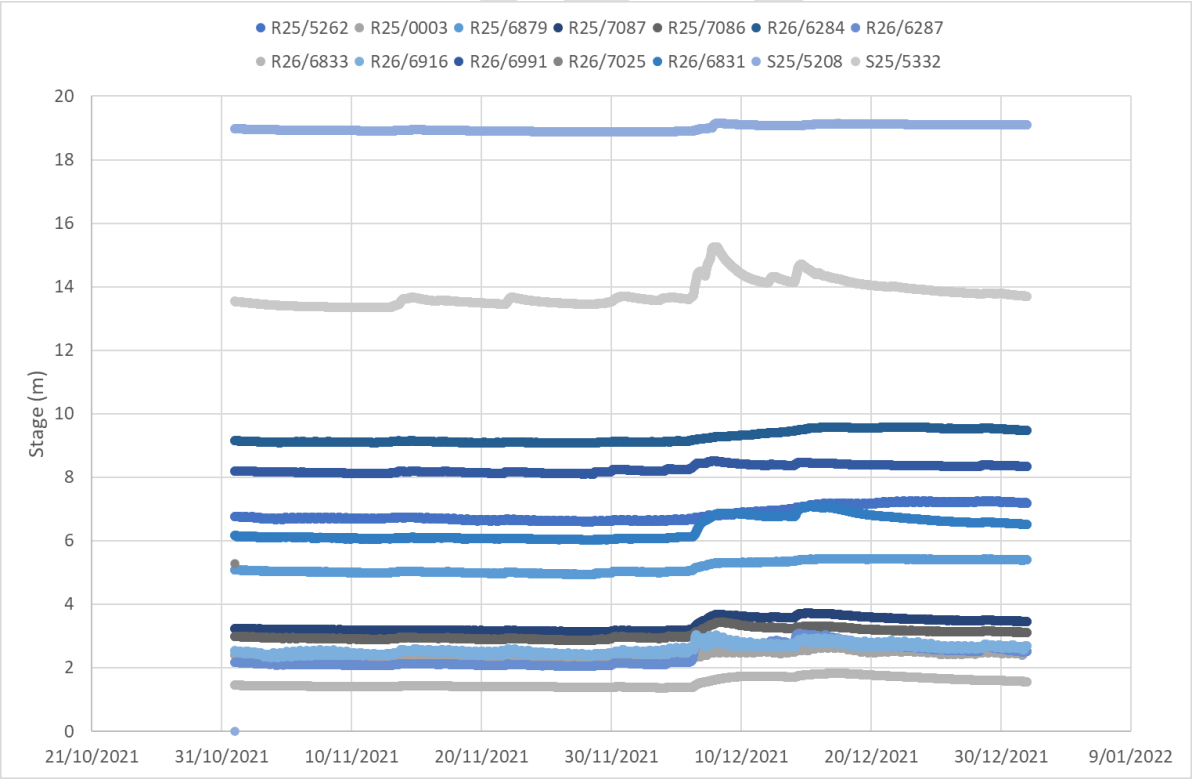


Figure 43. November-December groundwater timeseries (source GWRC)

8.1.3. Validation

The calibrated model was also subjected to its own validation exercise whereby model outputs were checked against observed data from a further four historical storm events: 20 October 1998, 28 October 1998, 17 June 2002 and 30 April 2008. The approach and outcomes are detailed in the Wharemauku Model Build Report (Awa Environmental 2024) provided in APPENDIX H.

8.1.4. Propagation to other sub-catchment models

Other hill sub-catchments with similar characteristics were not calibrated in the same manner due to lack of DEMs, and soil and land use data. The calibration of the upper Wharemauku catchment was therefore used to characterise the rainfall-runoff relationships for the other three hill sub-catchments that contribute to urban streams - Mazengarb, Paekākāriki and Waikanae. The subsequent validation of the full catchment models described below provided a sense check on the application of the parameters derived from the upper Wharemauku calibration.

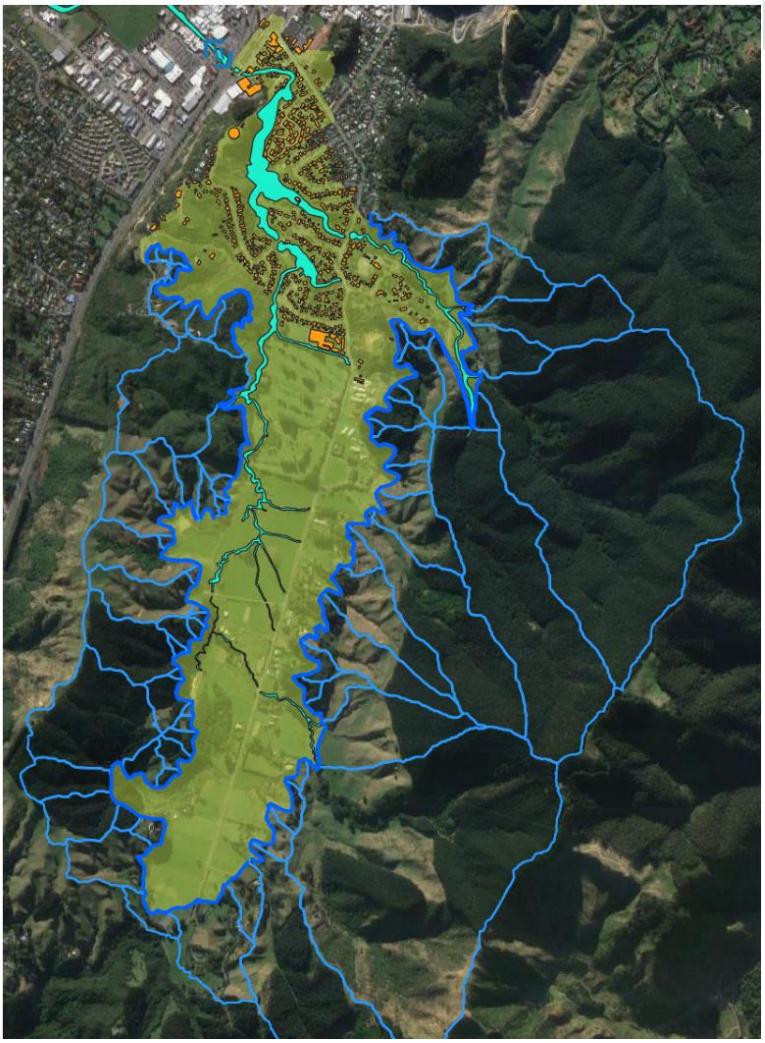


Figure 44. Wharemauku upper catchment extent and partitioning into hill sub-catchments (light blue outlines) and the valley floor rain-on-grid hydrodynamic domain (yellow area).

8.2. Validation of full Catchment Models

The Kāpiti coastal urban stormwater catchments could not be subjected to calibration due to lack of discharge data (i.e. a gauge station located in the lower part of a catchment) against which to match model outputs. It is not unusual for urban stormwater networks to have limited datasets for calibration. For this reason, historic flood records are often used as a proxy for flow in urban environments.

In Kāpiti there is a degree of public expectation in relation to flooding during heavy rain in low lying areas, and high soil soakage rates mean flooding is typically short-lived for all but the lowest lying areas. It is only the largest events, therefore, for which Council receives significant flooding complaints. These complaints were used to validate the integrated 1D-2D TUFLOW flood models of each of the seven catchments.

8.2.1. Storm events

Entries from KCDC's flood incidence database were compiled with additional photographic evidence, public reports and drone footage to perform validation against two significant storm events:

- 14-15 May 2015
- 5-8 December 2021.

These events were selected on the basis of size and available data. The May 2015 flood was a high-intensity event with 6-, 12-, and 24-hour peak rainfall depths, all larger than the corresponding 100-year ARI events for the Paraparaumu Aerodrome. The storm lasted approximately 28 hours, registering a total depth of 157mm (at the Aerodrome), and was significant enough to generate numerous flooding complaints to KCDC. The complaint records were, however, descriptive and photographic evidence was limited.

The December was less intense, occurring over several days after a long period of wet weather that had created high antecedent moisture conditions and elevated groundwater levels. The storm lasted 52 hours and registered143 mm rain at Paraparaumu Aerodrome. While not as significant as the 2015 event, the 2021 event had enough impact to generate flooding complaints to the Council. In addition, time stamped photographic evidence was collected by Awa and KCDC contractors. The photographic evidence collected was not exhaustive, nor was it collected at the peak of the storm. Nonetheless it provides a useful accompaniment to the complaint records and a sound basis for validation of the models.

8.2.2. Boundary Conditions

Boundary conditions for the validation process were derived in the same manner as for the hill-sub-catchment calibration. Empirical rainfall data from rain gauges were distributed spatially using Theissen polygons. For the tidal boundaries, the three catchments south of the Waikanae River mouth were assigned the Paraparaumu Beach tidal timeseries. Catchments north of and including the Waikanae River mouth were assigned the Waikanae River mouth tidal timeseries, while the Ōtaki River mouth timeseries was applied to the Ōtaki catchment model. The modelled 50th percentile spatial groundwater level distribution was adopted for the start of the validation simulations.

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8.2.3. Validation Process

Validation of the catchment models was an iterative process that involved aligning model results with recorded flooding observations. Adjustments were made to model parameters to achieve agreement between observed and predicted flooding as summarised in Table 29.

Table 29. Model parameter adjustments made in the catchment model validation process

PARAMETER	ADJUSTMENT
Roughness	The roughness of the stream corridors was updated from a constant roughness value of 0.03 across all stream corridors to classified roughness values for each individual stream corridor ranging from 0.025 to 0.08. This update was applied to better reflect the unique characteristics of each stream reach and was based on site walkover and aerial data.
Network Improvements	In some areas the modelled networks were not functioning as expected and/or had missing data. These areas were investigated and resolved and then re-tested in the model.
Digital Elevation Model	The DEM was updated to reflect earthworks had been undertaken since the last version of the LiDAR (2017) was secured by the Council.

Recorded complaints in the KCDC database have varying levels of information attached to them, requiring different approaches to validation between different sites and storm events. An example of validation performed on an affected property with only a descriptive complaint record is shown in Figure 45. Figure 46 presents an example of an affected property that had both descriptive and photographic records that allowed validation of the modelled peak flood level.

The Mazengarb was the first catchment to be validated in the modelling programme and many learnings from this validation were carried through to other models. The Mazengarb catchment the model went through five iterations with the following changes made to the model through this process. Figure 47 gives an example comparison between results from an initial Mazengarb catchment model iteration and the final iteration utilising photographic evidence from the December 2021.

The validation effort proved essential to refining models, with particular benefit around stream roughness. The model validation process for each catchment is outlined in the individual catchment model build reports provided in APPENDIX H.

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REPORTED FLOOD INCIDENT	
Storm event	May 2015
Affected properties	14 and 16 Georgia Grove
Caller description	14 and 12 maybe 10 Georgia Grove Paraparaumu require sandbags. Right away is just below knee deep and starting to come into properties. Water getting closer to house - less than a meter away to coming into the garage which is attached to the house. Home by self with 10-year-old. Any possibility for sandbags at this property.
Description of modelled flooding	The model shows flooding, areas affected are situated in low lying areas.
MODEL RESULT	

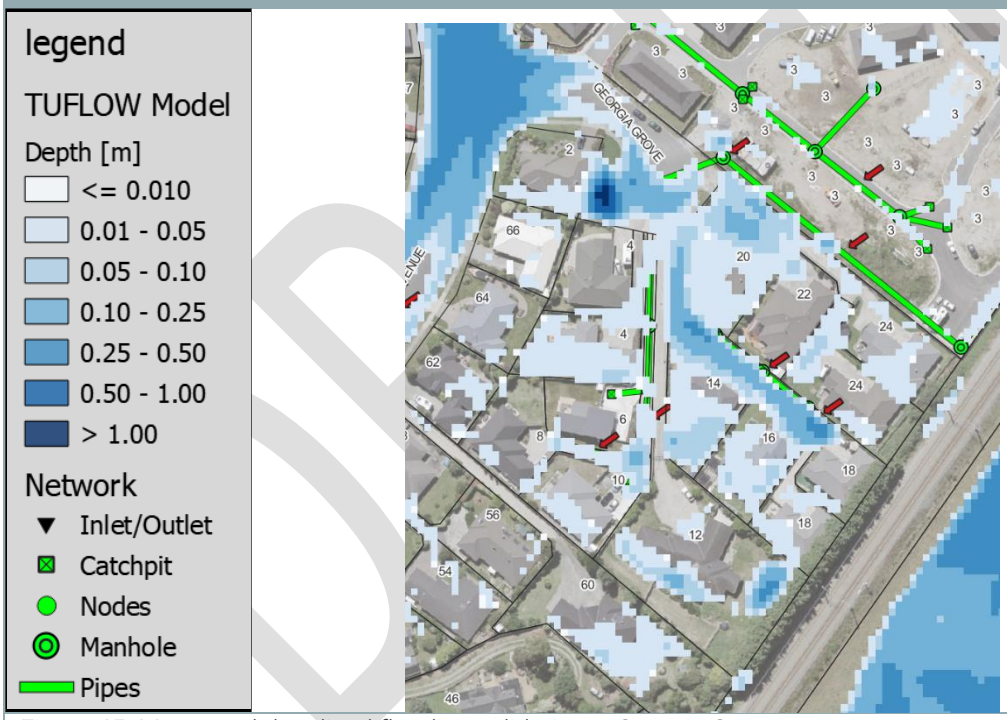
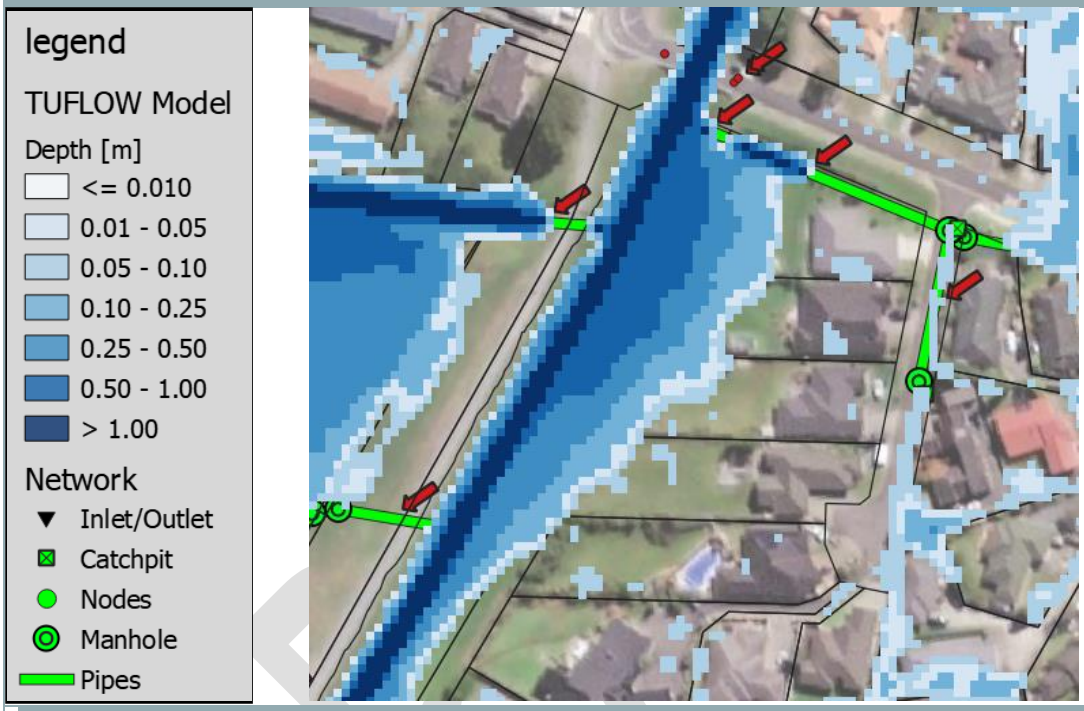


Figure 45. Mazengarb localised flooding validation – Georgia Grove

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REPORTED FLOOD INCIDENT	
Storm event	December 2021
Affected properties	Wood Leigh
Description of modelled flooding	Flooding from open channel filling low lying area.

MODEL RESULT



PHOTOGRAPHIC EVIDENCE



Figure 46. Mazengarb Stream flooding validation – Wood Leigh



Figure 47. Guildford Drive model validation – comparison between initial model results and final validated model results

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9. Model Outputs

The model build described in this report produced the 'base' models for each of the seven urban catchments in the Kāpiti District. These base models reflect the physical state of the catchments and networks as of approximately 2019 and the latest available boundary condition data.

This section outlines the outputs that are generated from running simulations using the base models, and the post-processing of those outputs that has been applied to produce flood maps.

9.1. Modelled Scenarios

The base models were used to simulate flooding arising from 1%, 2%, 10%, 20% and 50% AEP design storms, for the years 2030 and 2130. The storm duration for all model runs was 48 hours, representing the worst-case flooding scenario. However, the first twelve hours of the storm events were omitted from the simulations to minimise model run time (refer to section 4.8). These ten scenarios, multiplied across seven catchments, amounted to a total of 70 simulations and associated model outputs.

9.2. Raw results

9.2.1. Standard outputs

TUFLOW raw results produce several maps and time series data. Depth and velocity maps are standardised outputs of TUFLOW, while extents and hazard maps are produced in through post-processing the TUFLOW results in accordance with GWRC (2021) guidance. The full range of outputs available from the TUFLOW models is summarised in Table 30.

Table 30. Full suite of outputs from TUFLOW model simulations.

OUTPUTS	TYPE	DESCRIPTION
Bed Elevation	2D Map	The sub-grid sampled surface. This layer does not change over time and is a snapshot from the moment the timeseries has started recording.
Depth	2D Map	Difference between water level and ground level throughout the simulation. Changes with time.
Depth to groundwater	2D Map	Distance from the surface elevation to the groundwater table. Changes with time.
Minimum dt	2D Map	Allows visual display of the model which shows areas causing the slowest or fastest calculations. Useful for troubleshooting models to make them as efficient as possible. Changes with time.
Velocity vectors	2D Map	A map layer that shows the direction and magnitude of velocity of water. Changes with time.
Velocity	2D Map	Shows the velocities throughout the simulation. Changes with time.

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Water Level	2D Map	Water level relative to the datum used in the model. Changes with time.
Time of peak water level	2D Map	A static grid that shows the time when the highest water level occurred.
Level	Time series	Water level going through a 1d_po line.
Flow regime	Time series	A flow regime refers to the pattern or configuration of flow in a system or channel. It is typically categorised by properties such as velocity, pressure, density, and viscosity, and can vary from laminar (smooth, ordered flow) to turbulent (chaotic, random flow).
Velocity	Time series	Velocities going through the 1d structure.
Flow	Time series	Discharge going through the 1d structure.
Losses	Time series	Total losses associated from the 1d structure.
US Levels	Time series	Water levels upstream of the 1d structure.
DS Levels	Time series	Water levels downstream of the 1d structure.
Water Level	Time series	Water level inside of the 1d structure
Max Water Level	Time series	Highest water level inside of the 1d structure
Time of Max H	Time series	Time when max water level occurred inside the 1d structure.
Bed Level	Time series	Invert level along the 1d structure.
Culverts and Pipes	Time series	Side view of a selected 1d pipe and draws connecting downstream network.
Left bank obvert	Time series	Inside roof of the structure (left) applies to 1d open channels.
Right bank obvert	Time series	Inside roof of the structure (right) applies to 1d open channels.
Pit Ground Levels (if any)	Time series	Marks the ground level of the pits connected to 1d network

9.2.2. Naming conventions

There are two naming conventions used for the raw results. While GWRC (2021) provides a standardised approach for naming conventions, it does not capture all elements of the model runs (i.e. year horizon). Thus, Awa created its own custom naming convention to reflect the varied time horizons, in addition to providing a naming convention consistent with the GWRC (2021) standard. An example is provided in Table 31.

Table 31. Example TUFLOW model run naming convention

BOUNDARY CONDITION	GWRC (2021) NAMING CONVENTION	AWA NAMING CONVENTION (DEFAULT TUFLOW OUTPUT)	AWA MODEL CODE
Ōtaki Catchment at 2130 with 1%AEP Rainfall and 5%AEP Tide using SSP5-8.5M climate change	ŌTAKI_W1PCAEPCC_F	Ot_2130_M_R100yr_StormTide-2130_SSP5-8.5M_T20yr_Q100yr-OR_Q100yr-6hr-WS	BM-OT6

Additionally, Awa has created name codes for boundary conditions as the current default TUFLOW output are too long, though informative. The base model boundary conditions for each catchment are available in APPENDIX J.

9.3. Post-processing

The TUFLOW base models permit a high level of detail to be supplied to the simulation as both raster inputs and geospatial poly-shapes. The standard grid size used in these models was 2 m. In conjunction with sub-grid sampling and quadtree-type methods (Ōtaki catchment), highly detailed results can be produced with good accuracy; for example, kerbside pooling can often be visible.

This level of accuracy also has disadvantages with regard to interpreting the model outputs, specifically:

1. Many small puddles are visible due to minor fluctuations in the digital elevation model (DEM). A large subset of these will be removed when a basic 0.05 m filter is applied to the depth result layer. The remaining ones are either:
    - completely isolated, in which case they do not pose a flood risk and can be removed, or
    - clustered, in which case they should be joined together into a single contiguous pond covering a larger area.
2. Water flows around the buildings, which have been modelled as an impervious layer. This results in flooded areas with large gaps where the buildings are.
3. For advisory purposes, the level of detail is a significant departure from previous model results with larger elements and/or hand-drawn maps.

Model outputs were therefore subjected to a range of refinements using post-processing techniques implemented in the FME geospatial, transformation and load platform. Initial processing used filtration of the depth maps by using a logical mask to remove depth less or equal to 50mm. This was applied on all base models as advised by TUFLOW for noise reduction. Further refinements included:

1. Filtering out small free-standing puddles and merging puddles lying in close proximity to one another.
2. Voids in flooded areas created by buildings were filled in by capturing neighbouring wetted cells for each building, then creating concave hulls and converting back to cells with depth set to the maximum depth of the neighbouring cells.
3. Filling in voids in flooded areas created by the presence of buildings
4. Blending variable depths in flooded cell groups to create a smoother representation of water levels
5. Flood water boundaries were smoothed by converting raster cells into polygons.

For a description of the steps followed and functions used in FME see APPENDIX K.

9.4. Mapping

Three main forms of maps were produced from the modelling results:

- Flood depth maps showing the depth of flood waters at a resolution of 0.05 – 0.5 m.
- Flood hazard maps produced from post-processing of flood depths and velocities in accordance with the classification recommended by GWRC (2021) – refer to Figure 48.
- Flood extent maps showing flooded areas layered by modelling scenario as per GWRC (2021)

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These maps are all available through the Awa Webviewer

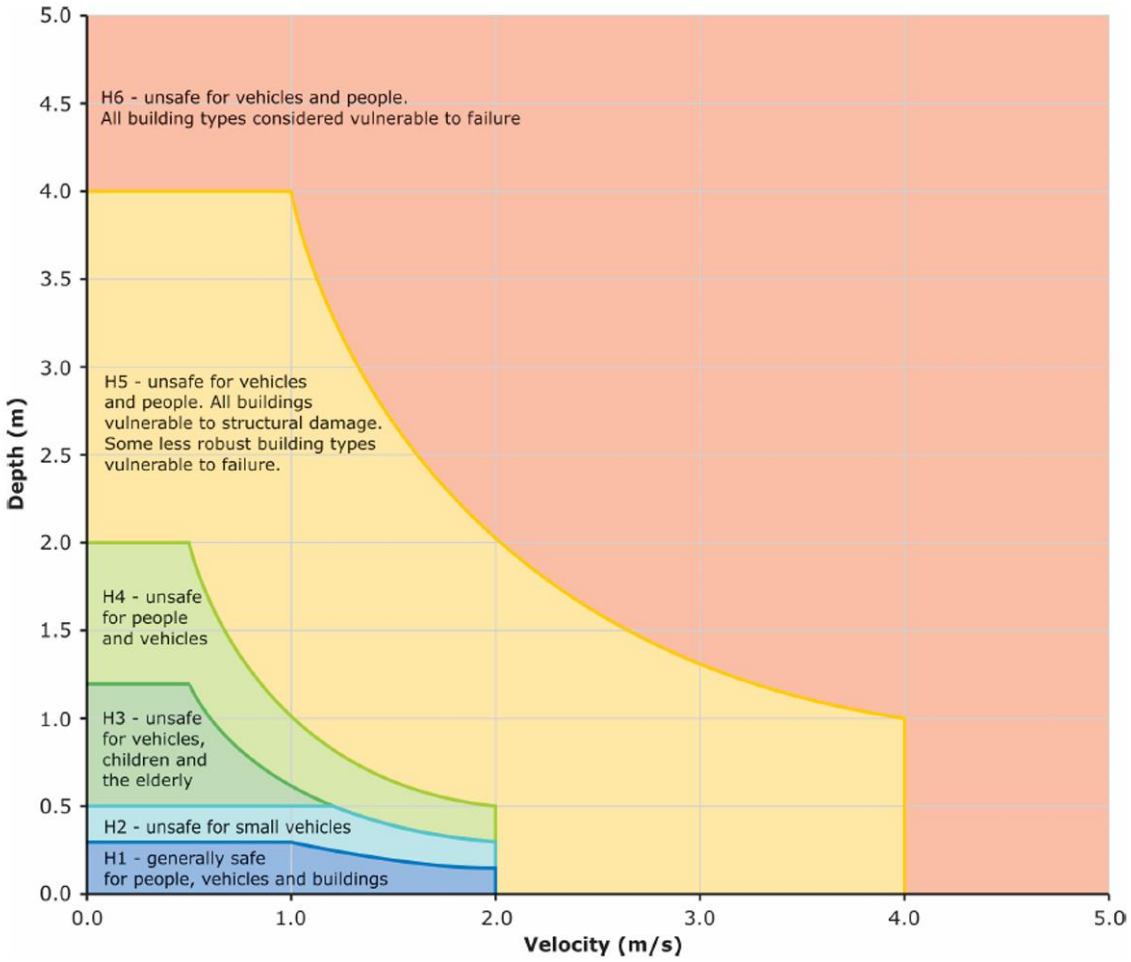


Figure 48. flood hazard classification adopted for flood hazard mapping. Source GWRC (2021)

9.5. Products

In addition to the models themselves, the rebuilt KCDC base urban stormwater hydraulic models have been used to produce a range of flood map outputs. All modelling results are currently accessible on Awa Environmental's ArcGIS Online portal. Table 32 summarises the final outputs produced from the modelling from one example catchment. Example maps are outlined below:

- Flood depth
    a. Refer to Figure 49 for an example of a flood depth map generated from the raw results with a simple 0.05 m noise filter applied. This is a raster layer in geotiff (TIF) format with a 2 m grid.
    b. Refer to Figure 50 for an example of a depth map after refinement post-processing. The results are a polygon layer in shapefile (SHP) format.
- Flood hazard
    a. Refer to Figure 51 for an example of a hazard map generated from the raw results, filtered by a mask created from a simple 0.05 m filter applied to the depth. This is a raster layer in geotiff (TIF) format with a 2 m grid.
    b. Refer to Figure 52 for an example of a hazard map after refinement post-processing. The results are a polygon layer in shapefile (SHP) format.
- Flood extent

a. Refer to Figure 53 for an example of a flood extent map using the raw data filtered by a mask created from a simple 0.05 m filter applied to the depth. This is a raster layer in geotiff (TIF) format with a 2 m grid.
b. Refer to Figure 54 for an example of an extent map after refinement processing. The results are a polygon layer in shapefile (SHP) format.

Table 32. Map outputs for Waikanae Catchment.

AWA REF	YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)	MAIN OUTPUTS		
				FLOOD EXTENT	DEPTH MAP	HAZARD MAP
BM 1- WK1	2030	1%	10%	✓	✓	✓
BM-WK2	2030	2%	10%	✓	✓	✓
BM-WK3	2030	10%	50%	✓	✓	✓
BM-WK4	2030	20%	50%	✓	✓	✓
BM-WK5	2030	50%	50%	✓	✓	✓
BM-WK6	2130	1%	30%	✓	✓	✓
BM-WK7	2130	2%	40%	✓	✓	✓
BM-WK8	2130	10%	50%	✓	✓	✓
BM-WK9	2130	20%	75%	✓	✓	✓
BM-WK10	2130	50%	75%	✓	✓	✓

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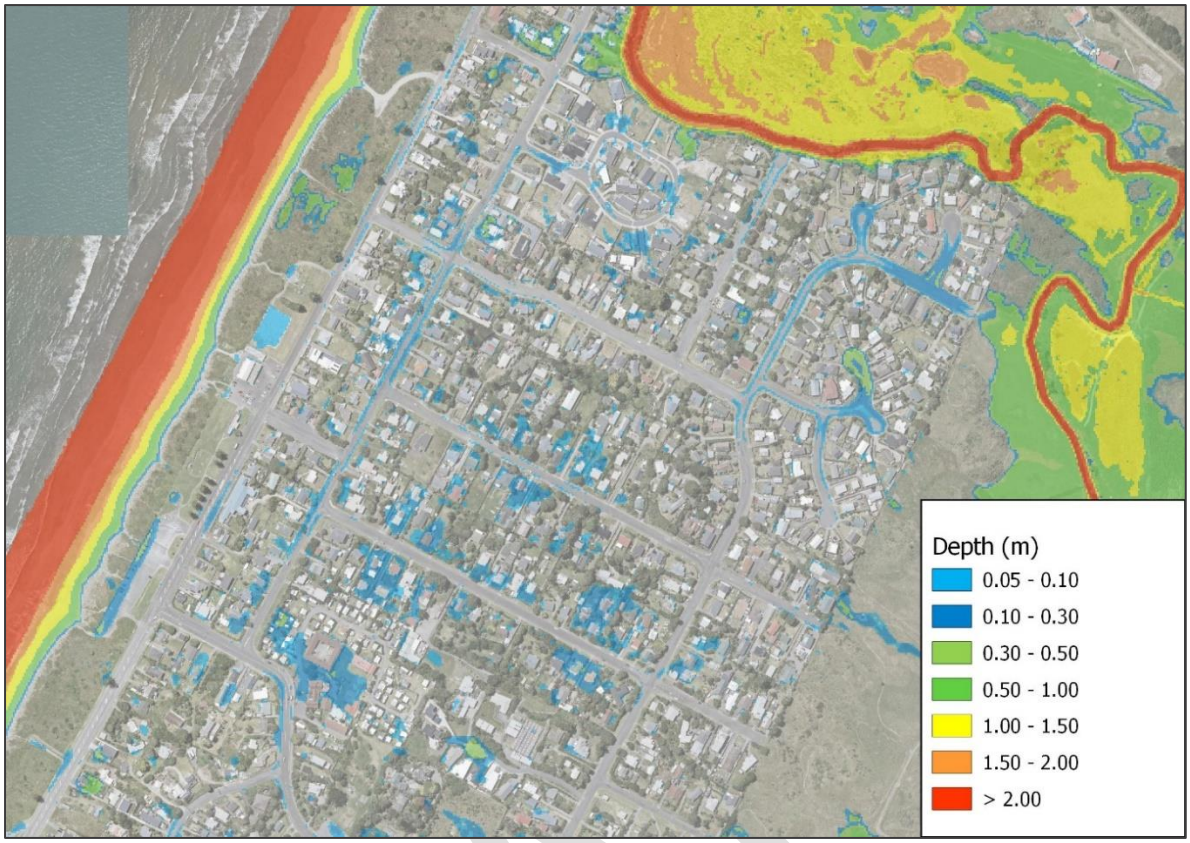


Figure 49. Flood depth map taken for a 2130 100-year rainfall event with a simple filter applied.

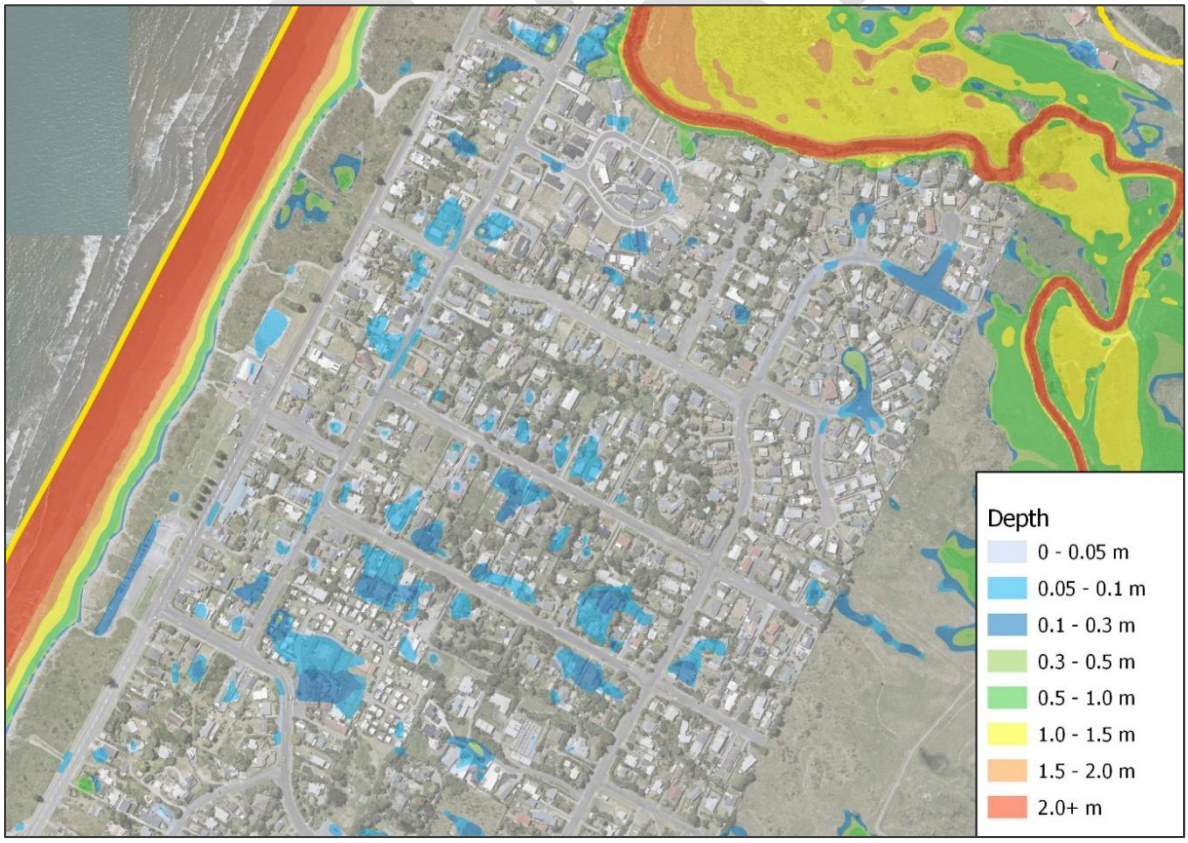


Figure 50. Flood depth map after refinement processing for a 2130 100-year rainfall event.

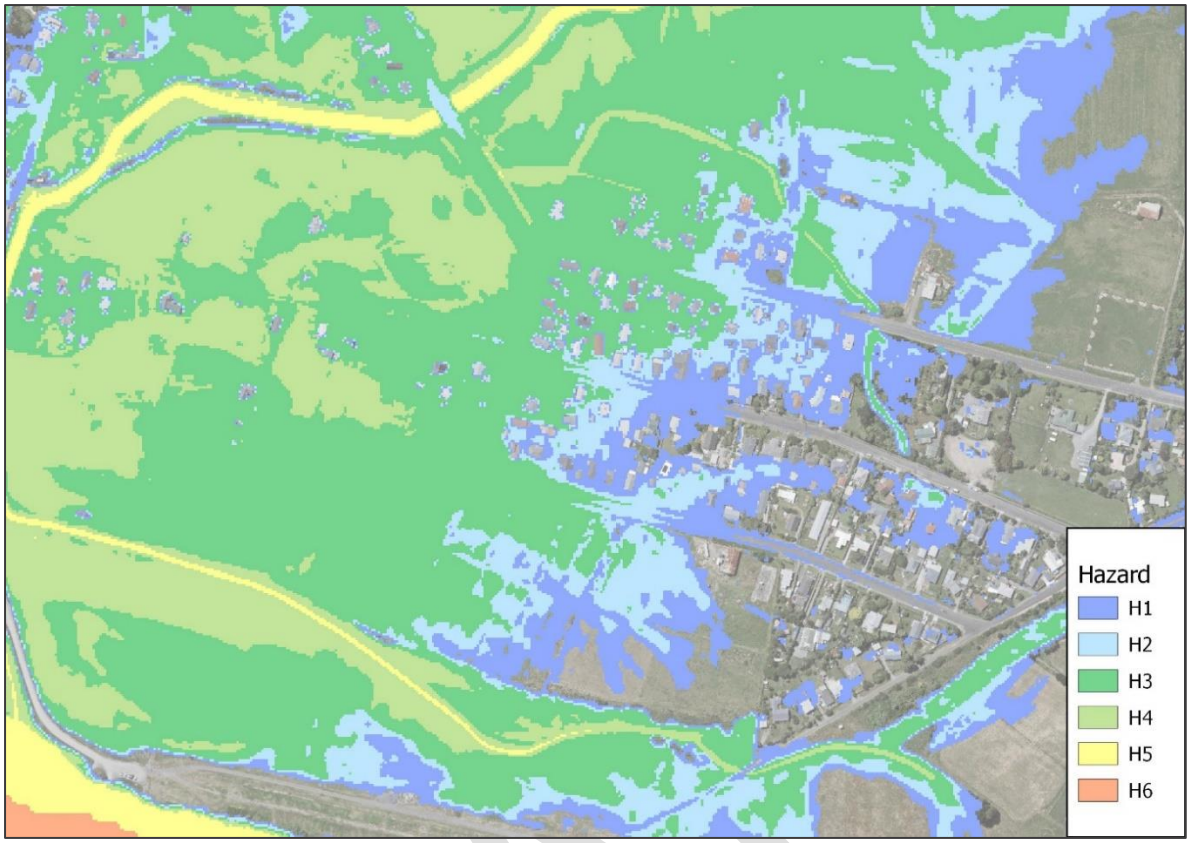


Figure 51. Flood hazard map for a 2130 100-year rainfall.

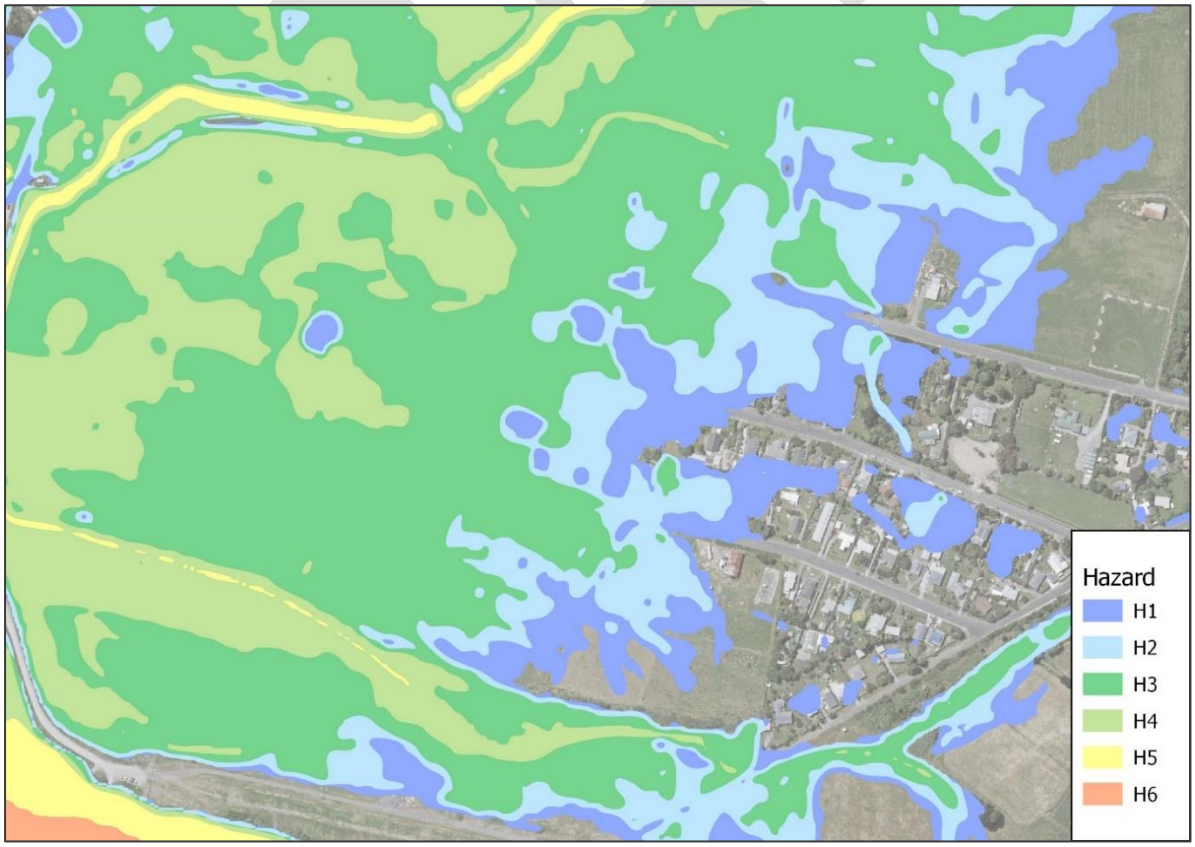


Figure 52. Flood hazard map after refinement processing for a 2130 100-year rainfall event

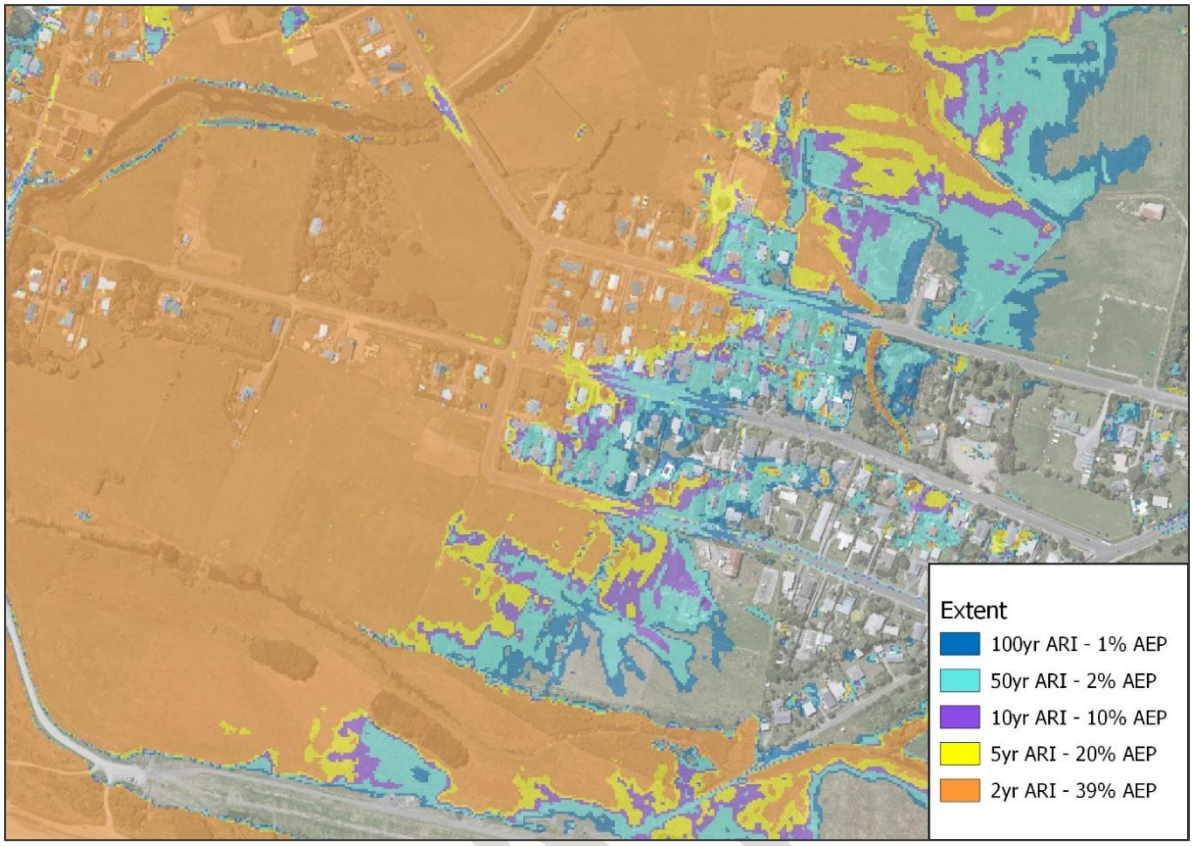


Figure 53. Flood extent map for rainfall events at the 2130 horizon.

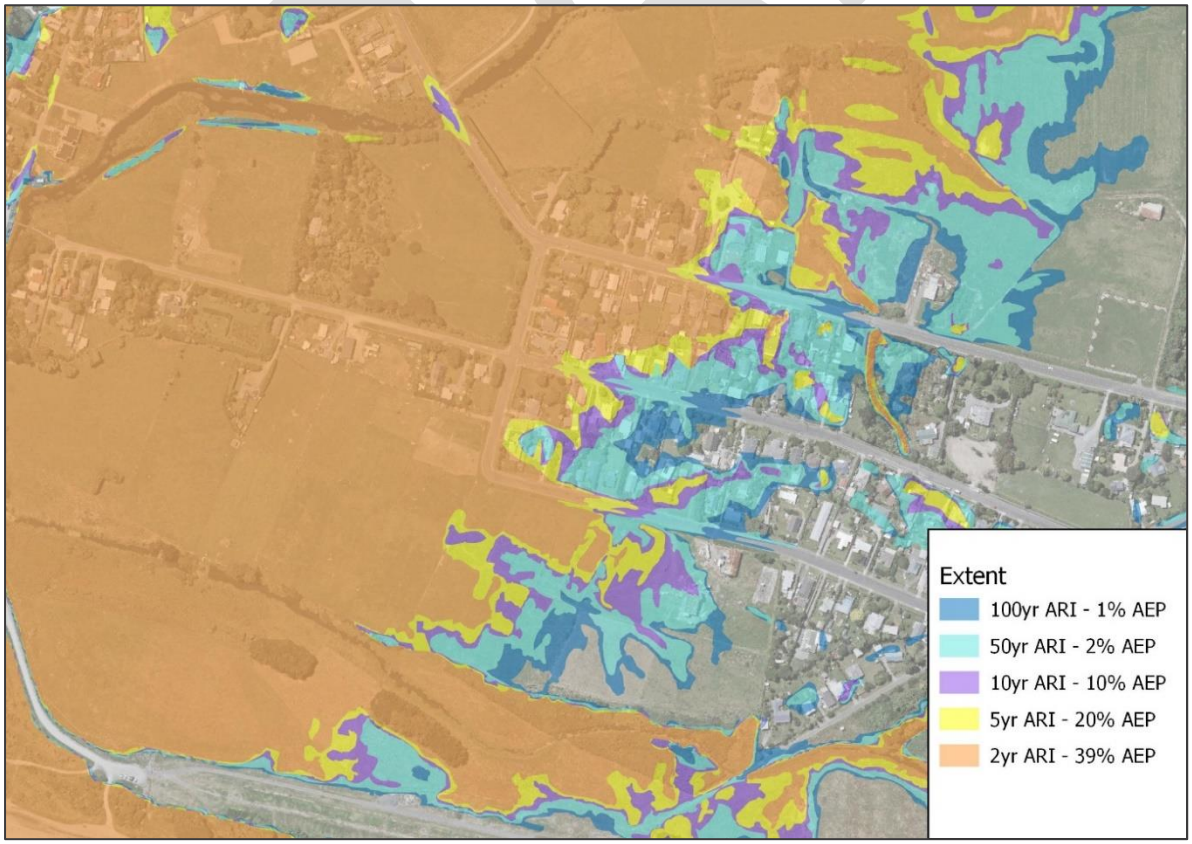


Figure 54. Flood extent map after refinement processing for a 2130 100-year rainfall event

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11. Appendices

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APPENDIX A KCDC TENDER Technical Specification

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Technical Specifications

Attachment 3

Glossary

The following terms and abbreviations are used in this document:

Term	Description
1D	One dimensional means only one spatial dimension is considered, i.e. the horizontal direction of flow
2D	Two dimensional means two spatial dimensions are considered i.e. the horizontal and lateral (x and y) directions of flow.
2D Surface	The surface which is used in the model for 2D flow computations
Annual Exceedence Probability [AEP]	The probability that a given rainfall event or flow rate will be exceeded in a single year.
Average Recurrence Interval (ARI)	Average period of time between rainfall events or flow rates which exceed a certain magnitude.
catchment	An area of land draining by force of gravity to a given location.
Climate change	Climate change resulting from global warming due to greenhouse gas emissions
DEM	Digital Elevation Model representing the surface elevation of the catchment
Design Storm	The rainfall event calculated from historical record that can be expected for a specific AEP or ARI.
Design Flows	The flow estimated from various design storms, selected as a basis for the design of works in watercourses and catchments
Existing development [ED]	The land development within the catchment at the time of model development
Energy grade line [EGL]	The total energy of flow at a given location, it is the sum of the elevation head, the pressure head, and the velocity head.
Energy loss	Energy or head loss occurs due to frictional resistance, contraction and expansion at entrance and exit, change in flow direction, change in elevation and change in cross-section.
Floodplain	The plan extent of flooding in a given AEP or ARI storm.
Freeboard	An allowance to take account of the uncertainties associated with: • Precision of the hydraulic modelling; • Physical processes that have not been allowed for in the design water level e.g. debris build-up, wave action, changes in bed level; • Uncertainty in the prediction of physical processes (e.g. rainfall patterns) that affect the design water level. The inclusion of a freeboard allowance provides an upper confidence that ensures that the water levels will have a high degree of certainty of not being exceeded.
GIS	Geographical Information System
GWRC	Greater Wellington Regional Council

Term	Description
HEC-1 / HEC HMS software	Unit hydrograph based approach used to estimate catchment runoff
Horton's Decay Exponent (k)	Determines the dynamics of the infiltration rate reduction over time during a rainfall event.
Hydraulic Grade Line (HGL)	A line coinciding with the level of flowing water in an open channel. In a closed conduit flowing under pressure, the HGL is the level to which water would rise in a vertical tube at any point along the pipe. It is equal to the energy grade line (EGL) elevation minus the velocity head
hydrograph	A graph illustrating the variation of flow with time.
Initial loss	A one-off loss which accounts for wetting of the catchment surface and defines the precipitation depth required for filling the depressions on the catchment surface before runoff can occur.
KCDC	Kāpiti Coast District Council
LiDAR	Light Detection and Ranging (Aerial Laser Survey)
Link	Link represents stormwater drainage pipes, culverts, bridges, stream channel reaches or overland flow paths.
Manning's "n"	Manning's coefficient is a lumped energy loss parameter representing all forms of loss in the flow between the sections used to define the water level slope. The Manning's number (M) is the inverse of Manning's "n" (i.e. $M = 1/n$)
Mesh	The mesh defines the 2D surface. Mesh elements shall be square when using TUFLOW
Node	Node represents the drainage system attributes such as manholes, inlets, outlets, junction between open channels, ponds.
Overland flow	Stormwater runoff travelling downhill over the surface of the ground along the path of least resistance towards streams and watercourses or the sea.
Primary Drainage System	The pipes, stream networks and open watercourses that carry the main, frequent stormwater within a catchment.
Runoff	The fraction of the rainfall which runs off the land surface to the drainage system
Secondary Drainage System	The overland flow path that carry the excess stormwater when the capacity of the primary drainage system is exceeded.
Sub-catchment	A smaller sub-area of the catchment, defined depending on a number of factors and depending on the purpose for defining them.
Storage loss	A one-off loss which defines the precipitation depth required for filling the depressions on the catchment surface before runoff can occur.
Wetting loss	A one-off loss, that accounts for wetting of the catchment surface.

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Table 3-1: Flood Hazard Categorisation

Table 4-1: Report Delivery Milestones

1 Introduction

1.1 Background

Kāpiti Coast District Council [KCDC] currently have 7 flood models constructed using a combination of MIKE Urban, MIKE 11 and MIKE Flood. The models cover the Districts urban open channel and piped network, with Greater Wellington Regional Council responsible for modelling the two major river systems that transect the District, i.e. the Waikanae and Otaki Rivers [as well as a number of the smaller rural catchments].

The current suite of models was built in 2010-2012 off earlier uncoupled models developed for KCDC by Jacobs (then SKM) between 2006 and 2009, for the District Plan and flood hazard mapping. These models are not dynamically coupled to the stormwater network and do not consistently reflect the urban development which has taken place in Kāpiti over the past 7 years since the models were built, including the effects of the MacKay's Crossing to PekaPeka [M2PP] Expressway which is now complete. In 2013, some of the models were updated to include the expressway for the M2PP AEE and design phases, and to include dynamic links between the stormwater pipe network and the surface water components of the model.

A summary of the models held by Jacobs for KCDC is attached as Attachment 3-1 ['Stormwater Models Built by Jacobs for KCDC', dated 1 August 2018]. Attachment 3-2 [hydrologic and hydraulic review of Waikanae Catchment] and Attachment 3-3 [Hydraulic Model Scoping Studies for each existing model catchment] provides an assessment of the quality of existing data, open channels, and catchment delineation for all seven stormwater hydraulic models.

The existing models need re-building to include recent development which has occurred within the District. There are also, now opportunities to improve the models by taking advantage of updated software capability, improved computing power, improved topographic data [LiDAR] and improved information on the drainage network, updated climate change predictions, and to merge model catchments as appropriate. These changes will simplify and rationalize the configuration; and improve accuracy, run times and behaviour [stability].

It is expected that these changes will also allow the models to be more agile and transparent within the wider statutory framework for managing hazards, particularly for the assessment [and communication] of risk implications for a range of future sea-level rise and climate change scenarios. Models need to be a sufficient quality and detail to ensure outputs are robust, transparent and defendable.

This specification has particularly built on the stormwater modelling approaches developed by Wellington Water Limited.

Model outcomes can be broadly classified into themes. Understanding the potential end-users and their outcome needs enables recommendations to be made regarding approach and methodology to satisfy the outcome needs in the most cost effective and efficient manner. Table 1.1 provides a summary of themes and potential end users of the model outputs.

Table 1.1: Summary of Themes and Potential End Users of the Model Outputs

Themes	Area of Interest	End User
land use planning	• Zoning and urban design • Growth projections – determine and plan for effects of future development • District Plan provisions and s106 assessment [RMA] • Develop flood hazard maps and identify secondary overflow paths	• Strategic decision makers within Council and Government • Community • Property owners [present and future] • Iwi • Mayor and Councillors • Other users or departments within Council, such as resource consent planners, building consent vetting officers, stormwater / drainage engineers, policy / strategic planners / development control engineers / asset planners and managers • Flood risk management professionals • Developers [& engineers involved in developing land; and designing, constructing, maintaining and operating infrastructure] • Emergency management professionals -– WEMO • Hydrologists and meteorologists involved in flood protection and forecasting • Other Councils in the catchment • Insurers • Hydrologists, other scientists and engineers • Land developers [present and future]
building controls	• Minimum floor levels • Structural resilience • S71-74 assessment [BA]	
civil works / land development	• Engineering design • Assessment of effects [RMA]	
infrastructure	• Catchment management planning • Flood plain management planning • Infrastructure design • Renewal / upgrade programme • Quantify deficiencies in level of surface • Investigate and compare options to manage flooding issues	
landscape and environment	• Riparian and wetland management • Strategic vegetation management • Stormwater management strategy	
education and information	• Hazard awareness • Understanding impact / vulnerability • LIM	
emergency management	• Evacuation and recovery plan • Lifeline planning	
insurance	• Premium calculations • Insurance coverage • Understanding impact / vulnerability	
Coastal management	• Storm surge • Coastal inundation – coincidental	

In addition to this document, the relevant software manuals, guides and on-line resources should also be referred to during the rebuild of the models. Kāpiti Coast District Council encourages modellers to identify opportunities for enhancement, efficiency and innovation. The modeller is to review this specification and identify any issues or contradictions with other relevant standards and guidance documents.

The modeller's work in part or all may be subject to peer review by Kāpiti Coast District Council or an independent third party.

Intersection with other guidance and reports – This scope sits within a framework of existing guidance, policies and legislation which are described in more detail below:

- Coastal Hazards and Climate Change – Guidance for Local Government; Ministry for the Environment, December 2017. This guidance is a major revision of the 2008 edition and includes the findings and projections of the latest Fifth Assessment Report produced by the Intergovernmental Panel on Climate Change [IPCC]. It also emphasises the importance of working collaboratively with stakeholders and affected communities throughout the decision-making process, and the need for good vulnerability assessment to underpin collaborative processes when assessing the evaluating response options.
- Climate Change and variability – Wellington Region; prepared for Greater Wellington Regional Council by NIWA, June 2017. This report describes changes that may occur over this century in the climate of the Wellington Region, and outlines some possible impacts and implications of these changes.
- Assessing the storm inundation hazard for coastal margins around the Wellington region, prepared for Greater Wellington Regional Council, Kāpiti Coast District Council and Wellington City Council, by NIWA, May 2012. This report provides an assessment of the exposure to inundation from storm-tide and wave hazards for coastal margins in the Wellington Region.
- Sea-level variability and trends: Wellington Region, prepared for Greater Wellington Regional Council by NIWA, June 2012. This report provides a synthesis of historic sea-level variability and trends in the Wellington regions and how these relate to sea level variability in national and global context.
- Coastal Storm Surge Inundation Maps for the Kāpiti Coast; prepared for Greater Wellington Regional Council by NIWA, May 2019. This report completes the assessment of the storm inundation hazard for coastal margins around the Wellington region for Peka Peka to Otaki.
- Determining an objective land-sea boundary for the Kāpiti Coast, prepared for Greater Wellington Regional Council and Kāpiti Coast District Council by NIWA, April 2019. This report determines the land-sea boundary for the Kāpiti Coast in order to define the coastal marine area (CMA) and delineate the jurisdictional boundary.
- Potential areas of Elevated Groundwater Levels Vulnerable to Climate Change Impacts, prepared for Kāpiti Coast District Council by Sinclair Knight Merz, 2013, draft for client review.

1.2 Specification Structure

This specification sets out Kāpiti Coast District Council's proposed methodology for hydraulic modelling of stormwater.

The consultant, if being unable to comply fully with the specification, should comply with the intent of the specification. If any doubt exists as to the intent of the specifications, clarification should be sought from Kāpiti Coast District Council.

See 'Request for Tender' document for details.

2 Project Programming and Framework

2.1 Project Milestones

The modelling process is split into 4 main stages: scoping, model build, mapping and on-going model management. Although on-going model management is beyond the current scope of works, the development of protocols and procedures which facilitate its on-going management are included. Model build to proceed over the next 2-3 years in line with Long Term Plan budgets.

The following are significant milestones;

1. Project Inception
2. Model Extents and Data Assessment Report (draft and final)
3. Model Build Report (drafts and final)
4. Maps (draft and final)
5. Project completion, including data handover.

Figure 2-1 presents a schematic of the hydraulic modelling process.

Following the inception meeting, the successful tenderer will prepare, in consultation with KCDC and its peer reviewer, a work programme outlining key dates for each hold point and milestone.

2.2 Preferred Software

KCDC currently have 7 flood models constructed using a combination of MIKE Urban, MIKE 11 and MIKE Flood. However, the preferred software package for the stormwater hydraulic model rebuild is TUFLOW HPC, utilising the GPU module for simulation acceleration, in ESRI Arc GIS compatible format [including check files].

The preferred software package for hydrological modelling is the Hydrologic Engineering Centre – Hydrological Modelling System [HEC-HMS], version 4.2. The unit hydrograph based approach is modelled on the US Soil Conservation Service software HEC-1 and is used to estimate catchment runoff. The methodology and parameters documented in 'Appendix 1 – Updated Isohyet Based Calculation of Design Peakflows' [Jacobs 2011] of KCDC's Subdivision and Development Principles and Requirements [2012] [Appendix 3D] should be followed for the design events used in modelling, except for climate change. The topography is to be updated utilizing the most recent LiDAR information. Contour and elevation data [LiDAR] for use in defining sub-catchment and channel slope shall be obtained from KCDC.

Models must use KCDC's specified software version and service pack.

2.3 Ownership

All models developed and all associate model build data is the asset and intellectual property of KCDC. The model and any associated data must be delivered in such a manner that allows all scenarios to be simulated, and that allows maintenance, updating and further development of the model by KCDC or its consultant(s).

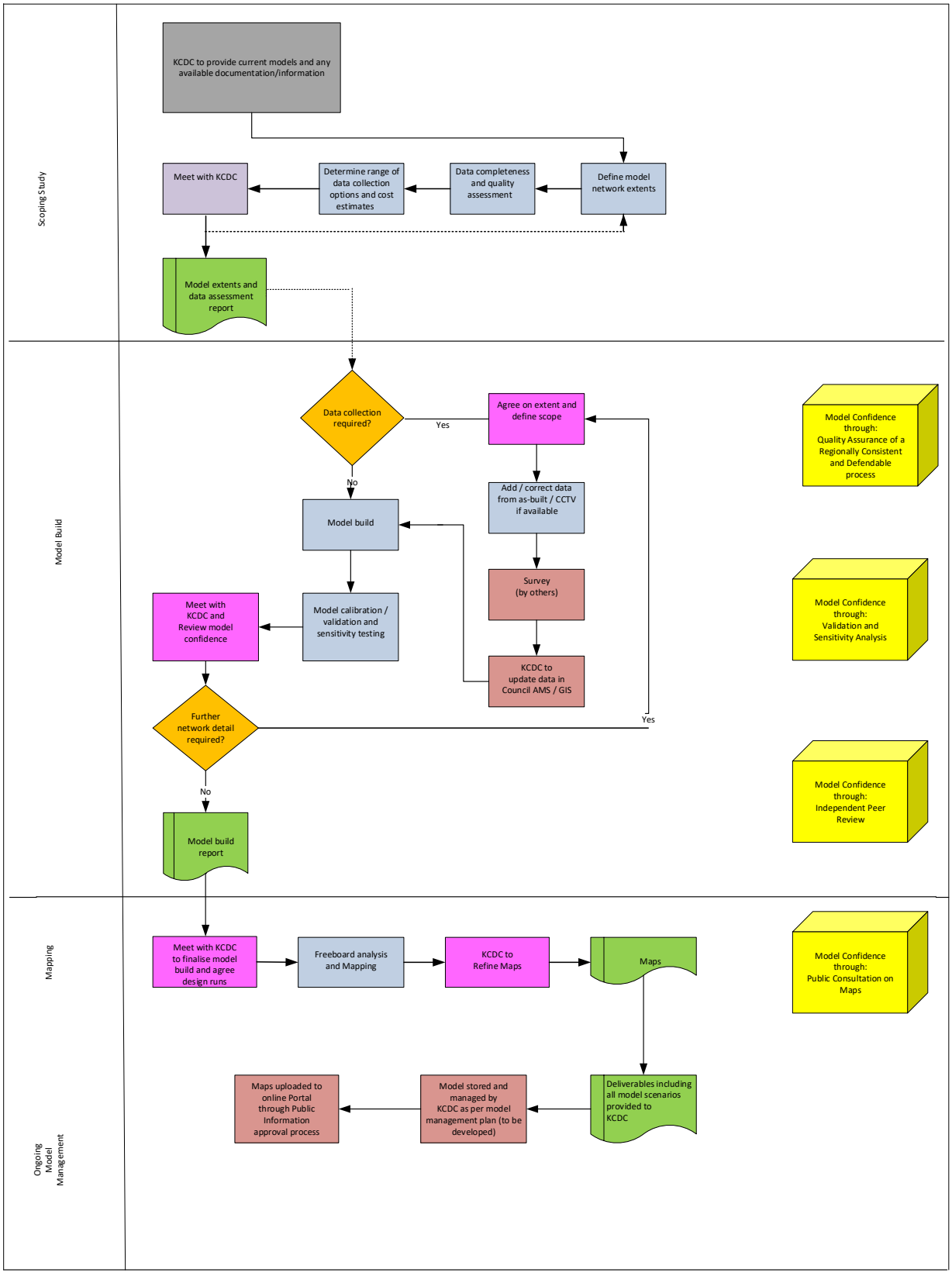


Figure 2-1 Hydraulic Modelling Process Schematic

2.4 Quality Systems

The work of building a flood model often involves large amounts of data. A quality system should be adopted in order to provide a good framework for controlling and documenting the flow of data. Efficient quality systems for modelling processes will provide systematic tracking of the source of all model build data and a basis for model updates, validation and justification for model parameters used.

At every phase of the modelling project quality systems shall be established and followed to ensure confidence in the resulting model.

2.5 Naming Convention

Prior to work commencing, the modeller shall develop a folder and file naming convention. Once approved, the naming convention is to be strictly followed throughout the project and continue to be relevant to its on-going management.

2.6 Coordinate System

The coordinate system for all models and associated data shall be in terms of New Zealand Geodetic Datum 2000 [NZGD2000] and using the New Zealand transverse Mercator [NZTM] projection. Levels shall be in terms of 1953 Wellington Local Datum.

3 Methodology

3.1 Stage 1: Model Extents and Data Assessment Report

The purpose of the scoping study is to ensure sufficient detail is available which appropriately defines the model extents and schematization, and identifies the gaps in the required information and how to fill the gaps and plan the model build(s). The elements of the scoping study are outlined in the following sections.

3.1.1 Collation of Historical Flooding Records

Understanding the locations prone to flooding provides guidance on the current performance of the network and areas of special interest. Photos, historical investigations and customer complaints can provide valuable catchment information.

3.1.2 Previous Model Review

At the commencement of each model build, any previous model build and/or calibration work already carried out in the catchment shall be reviewed [this should include GWRC models and hydrological assessments]. The extent to which this previous modelling can be used to meet the objectives of the current study has been established by Robert Bird Group's review of the hydrologic and hydraulic models for the Waikanae area.

There have been significant changes to the District and in the modelling world, that warrant an update of the hydraulic modelling previously undertaken. Differences in model setup may mean that parameters from previous models may not be directly re-useable.

KCDC commissioned Robert Bird Group to review the hydrologic [HEC-HMS] and hydraulic [Mike Flood] models for the Waikanae area, on the assumption that key findings and recommendations would apply across all of the District's flood models. The findings are attached as Attachment 3-2: 'Stage 1 – Hydrologic and Hydraulic Review, Waikanae River and Stormwater Modelling', Robert Bird Group, September 2018.

The hydrological methodology adopted for the previous studies [HEC-HMS] is still appropriate as the situation is unchanged.

Greater Wellington Regional Council's current programme for updating its flood hazard maps is as follows:

- Waikanae River model update already done, new mapping to be finalized this financial year;
- Mangaone Stream model update to be completed 2019-20;
- Waitohu Stream is being surveyed this summer [2018-19] with model update following 2020-21;
- Possibility of residual flood hazard mapping from breach scenarios for the Otaki River.

Each of the 'Roads of National Significance' projects (RONS) have modelled its impact on the flood plain.

A revision of existing hydraulic models and output appropriateness for use should be undertaken. KCDC can facilitate access to any models not held by KCDC. Consideration could be given to combining the KCDC and GWRC models and running then as a single dynamically linked model if this simplifies the process of mapping the complete flood hazard in the urban area.

3.1.3 Determination of Model Extents

Before commencement of model build, a general understanding of the structure and layout of the stormwater drainage system is required. The stormwater catchment boundaries are represented in present models and stored in KCDC's GIS database, these can be considered as indicative as to the extent of the model being developed.

The model extent should be primarily defined as being large enough to cover the area of interest and should be based on watersheds and the development extent allowed for in the District Plan / Growth Strategy [Attachment 3-5 – Growth Areas for Inclusion in Model Extent]. The location of boundaries (tidal, overland flows and piped flow in and out of the catchment] should also be considered and the model extent set so boundary effects do not influence the model within the area of interest.

The model should include all publicly owned stormwater assets [including sumps and sump leads] and any significant private assets. It is not usually necessary to include privately owned lateral connections.

All network intakes shall be represented in the model as well as 50m of the open channels upstream of the intakes. Intakes are often the area where flooding occurs and including in the model is important for capturing overland flow paths.

Consideration should be given to combining [as appropriate] any of the existing models under one model extent reduce the number of models and enable better management. Where it is necessary to split the District into several zones, then flood maps for different zones need to be merged into one seamless flood map. The modeller should confirm there will be no stitching issues.

3.1.4 Topographical Data

Typically, topographical data will consist of contours and spot levels based on LiDAR [2017] or other survey techniques. A topographic surface may exist as an 'as-built' drawing where recent development has altered the land surface. KCDC will supply all available electronic topographic data, on request, to the successful tenderer.

3.1.5 Tidal and River Level Data

Stormwater runoff from each catchment finally discharges into a river or the sea. The hydraulic condition of the drainage network system may be affected by the downstream river or sea level. Water level data for modelling and validation runs can be obtained from NIWA or GWRC. The datum of this data should be checked and converted to 1953 Wellington Local Datum as required. The consultant to confirm the tidal and river level datasets to be used in the model. The dataset to be provided to the peer reviewer and KCDC for feedback and comment. Following incorporation of any feedback or comments, the dataset to be used in the model build.

3.1.6 Update to Current Development Conditions

It will be necessary to amend the model to include development that has happened since 2012. Key developments include:

1. Residential and commercial development;
2. Off ramps, road upgrades;
3. M2PP, Transmission Gully, PP2O expressway projects [RONS];
4. Dams and detention basins;
5. Pump stations – Raumati beach, Otaki beach;

6. Regional mitigation strategies.

3.1.7 Operational Data

A meeting is to be arranged with relevant KCDC operations staff to ensure that all relevant operational information is incorporated into the model build including:

- Network information that is at variance with that shown on GIS and recorded in the Asset Register.
- Operational issues that have a significant impact on stormwater drainage system performance.
- Any flood incidents, other than those caused by temporary blockages, that have been observed by, or reported to, operational staff.

Operations staff can also provide valuable on-site validation.

Reported flood incidents should be analysed to determine the location and frequency of reported flooding in the catchment. These will be used for model validation.

Information on blockage incidents should be obtained for the period of any flow survey or where they are associated with flooding.

All information provided by operational staff that is used in the model, including corrections to the drainage system data, should be documented in the model report.

Corrections made to drainage system data should be separately reported to KCDC.

3.1.8 Review and Quality Assessment of Existing Data

Existing asset and hydrometric data that will be used to build and run the hydrological and hydraulic models is to be reviewed and assessed for its adequacy and accuracy against the requirements of the project. An assessment of data completeness and quality is to be prepared.

The dataset required to develop hydrological and hydraulic models is generally a composite set of existing data, from various sources, and additional data that needs to be collected in a project specific survey. The consultant shall provide and implement a method for the:

- Acquisition, review, frame-works and ‘scrubbing’ of existing data;
- Assessment of the quality and adequacy, of existing data; and
- Provision of a detailed schedule for additional data needed to be collected for the development of hydrological and hydraulic models that will comply with this specification.

It is expected the review of existing asset and hydrometric data will include a review of all the structures within the existing models to ensure all relevant crossings, structures and cross sections are adequately represented in the model rebuild.

3.1.9 Finalise Model Extents and Model Schematisation

Meetings are to be held with KCDC to confirm model extents and an appropriate hydraulic model methodology schematic. Outputs from this process are to include a plan detailing the model components, boundary conditions, hydrological inputs and associated linkages. A review of available data quality should be used to agree on where data gaps are critical and survey to obtain missing data is required.

3.1.10 Model Extents and Data Quality Assessment Report

The Model Extents and Data Quality Assessment stage is to be completed with a Model Extents and Data Quality Assessment Report [refer Section 4.2.1].

3.2 Stage 2: Model Build

This section details the model build process based on the data provided, collected and the agreed approach to data estimation where good quality data are not available. The model's coordinate system will be NZGD2000 and using the NZTM projection. Levels shall be in terms of 1953 Wellington Local Datum. The model will be tested for sensibility, stability and other accuracy requirements.

3.2.1 Model Build Data Source Flagging

Flags shall be used during the model development process to document the source of all important model build data. Comments are also required where the flag does not give sufficient detail [e.g. 'as-built' flag should be complemented with a plan number]. Data sources and all assumptions should be clearly documented.

The model build data tagging information shall become the metadata which will be prepared in a spreadsheet or geodatabase and accompany the model upon delivery. The consultant shall prepare detailed procedures for preparing metadata.

The model build metadata shall include at least the following attributes:

- Sub-catchments – Curve Number, Initial Abstraction, Lag Time, Area
- Nodes – invert levels, ground levels, diameter
- Links – diameter, width/height, cross-section, roughness material, upstream and downstream invert levels; and.
- Structures – invert levels, width, cross-section, passive flow regulations. Possible sources of metadata could include council GIS, survey, site visits, derived from other data [e.g. LiDAR contours or raster grid, derived from interpolation, extrapolation, assumption, parameter values assigned from literature, KCDC modelling specifications, as-built drawings.]

Note: the majority of the information associated with the Nodes, Links and Structures is available in the default TUFLOW simulation check files.

3.2.2 Hydrologic Model (HEC-HMS)

KCDC commissioned Robert Bird Group to review the hydrologic [HEC-HMS] and hydraulic [Mike Flood] models for the Waikanae area, on the assumption that key findings and recommendations would apply across all of the District's flood models. The findings are attached as Attachment 3-2: 'Stage 1 – Hydrologic and Hydraulic Review, Waikanae River and Stormwater Modelling', Robert Bird Group, September 2018.

The hydrological methodology adopted for the previous studies [HEC-HMS] is still appropriate as the situation is unchanged. Based on this finding the existing HEC-HMS models will form the basis of this project.

Consideration could be given to the use of design rainfall totals obtained from the High Intensity Rainfall Design System [HIRDS] that has been developed by NIWA. If this were to be used, the successful tender would be required to undertake a comparison between the two. The program can be accessed at http://hirds.niwa.co.nz/.

3.2.2.1 Sub-catchment Delineation

Sub catchment delineation is required to identify hydrology catchments that will discharge to the hydraulic model. Fine resolution delineation will be required in urban areas. If the sub catchments used are too large the discharge hydrograph from the sub catchments may not adequately replicate reality, instead concentrating flow and potentially artificially exceeding the capacity of the network at the nodes the sub catchments are connected to. Note, this does not imply that a sub-catchment boundary needs to be delineated for each manhole/inlet and open channel reach. Coarser delineation can be used if appropriate. TUFLOW's 2D Source Area (2D_SA) and 1D Boundary Condition (1D_BC) polygon regions can be used to apply and distribute the hydrology model discharges into the hydraulic model. For example, if a 1D_BC polygon encompasses multiple 1D manhole/inlets, the hydrology model discharge will be proportioned equally between the 1D features.

Contour and elevation data (LiDAR) for use in defining sub-catchment and channel slope shall be obtained from KCDC.

3.2.2.2 Hydrologic Calculation Method

The unit hydrograph based approach is to be used to estimate catchment runoff. The 'Updated Isohyet Based Calculation of Design Peakflows' [Jacobs, 2011] [Attachment 3-4] provides guidance and examples on how to produce design hydrographs using both HEC-1 and HEC-HMS software. The modeller is to use this method [updated to reflect latest predictions for climate change to 2130] to create a hydrology model. The consultant will provide a provisional item in the proposal costing for updating the '*Isohyet Based Calculation of Design Peakflows Report*' to ensure consistency with the parameters used in the hydrology model. The updated report provided to the peer reviewer and Council for feedback and comment. Following incorporation of any feedback or comments, the report to be re-issued to the peer reviewer and Council.

3.2.2.3 Calibration Events, Design Rainfall and Climate Change

Events to be modelled are:

1. Three calibration/verification historic events, to be identified by the successful tenderer in consultation with KCDC.
2. Design rainfall events: 2% and 1% AEP using current climate change predictions of 16% increase in rainfall as well as 0.8m of sea level rise and Q_{20} storm surge, or as otherwise confirmed with the successful tenderer.

   For the purposes of understanding and managing any differences in the flood risk attributed to changing model platforms
3. Design rainfall events: 20%, 10%, 2% and 1% AEP
4. Climate Change: 2% and 1% AEP.

   For the purposes of understanding and managing the flood risk for new residential buildings Kāpiti District Council's Standard design scenario includes 20% increase in rainfall as well as 1 m of sea level rise. It is however recommended that prior to commencing modelling work that the MfE website: (http://www.mfe.govt.nz/climate-change/how-climate-change-affects-nz/climate-change-impacts and http://www.mfe.govt.nz/climate-change/adapting-climate-change/adapting-sea-level-rise) is checked to understand the latest guidance.

The consultant shall provide a provisional item in the proposal costing for scenario different rainfall data, or boundary condition data, within the hydrology / hydraulic model for the above events.

3.2.3 Hydraulic Model (TUFLOW)

3.2.3.1 Pipe Network

The piped drainage network shall be reviewed and rationalised where appropriate for inclusion in the hydraulic model as 1D features. The full publicly owned piped system shall be included (for the avoidance of doubt, including sumps and sump leads). Inlets can be lumped to simplify the model. Privately owned features should also be included in the model if they are part of a significant drainage network.

All inlet structures shall be included, including 50m of open channel upstream of the intakes.

3.2.3.2 Open Channel Network Schematisation

Watercourses and open channels should be incorporated in 1D where the channels are too small to be adequately represented in the 2D. Exceptions are flows in the road side kerb and channel and rivers where simulation of super elevation at bends needs to be accounted for.

1D open channels should be dynamically linked with the 1D piped network and/or the 2D surface model.

The Greater Wellington Regional Council [GWRC] have developed a regional flood models for the Waikanae and Otaki Rivers, and the Mangaone and Waitohu Streams. It will be necessary to consult with GWRC to ensure consistency between the models and to ensure a match in flood levels adjacent to river catchments. Consideration could be given to combining the KCDC and GWRC models and running then as a single dynamically linked model if this simplifies the process of mapping the complete flood hazard in the urban area. One seamless 1% AEP flood extent layer which combines the best available flood information for the District would provide greater legibility for the community. The feasibility and cost implications of doing this can be explored by the successful tenderer in the initial stages of the model re-build.

Modellers can rely on the cross-sections used in the current models unless these have been superseded by the construction of any new in-channel structure or any change to the channel profile, or if determined the cross sections are not of sufficient accuracy. Some cross sections in the existing models include unusually high and irregular roughness values [see Appendix 3B for details]. These are to be reviewed and reasonable roughness values adopted, as appropriate.

3.2.3.3 Bridge Structure Representation

Bridge structures can be included in TUFLOW as 1D elements if it is part of a 1D open channel or pipe network, or within the 2D domain using TUFLOW layered form loss approach. As with all 2D models it may be advisable to verify the bridge losses using another method, such as a local HECRAS model, where bridge losses are important.

3.2.3.4 2D Topography

The 2D model topography shall utilize the most recent LiDAR information, supplied by KCDC.

The consultant shall provide a provisional item in the proposal costing to create a digital terrain model (DTM).

Overbank flow in floodplains, overland flow, secondary overland flow paths, and flow in the kerb and channels should be represented in the 2D hydraulic model. Larger streams may be modelled in the 2D hydraulic model if it is necessary to model super elevation at bends. Wide flat shallow rivers or beaches between channels in braided rivers may also be modelled in the 2D hydraulic model.

Under certain conditions the following topographic features should be included in TUFLOW by enforcing 3 dimensional strings and levels using TUFLOW's inbuilt line and polygon (Z Shape) features:

- Road crests
- Road kerbs and channels
- Channel / river inverts
- Levees, embankments, retaining walls, flood barriers

This approach should be considered when the grid size is not sufficiently fine to define geometric properties that influence flow. For example, if there is a flow over the road which acts as a levee / hydraulic control it is important to define the crest level accurately.

Stormwater ponds, basins and other above ground storage areas should be modelled in the 2D hydraulic model. The inlet and outlet control structures are typically best represented in the 1D model.

3.2.3.5 Ponds

Pond outlet control structures such as culverts, orifices should be included in the 1D hydraulic model, while the pond storage volume should in included in the 2D model.

3.2.3.6 Soakpits

Modelling soakage is often difficult as soak pits are not gauged and, in many cases, not properly maintained. Where better information is unavailable a conservatively low soakage rate should be used. Even with detailed information available, there will still be a high level of uncertainty around the soakage rates and sensitivity testing should be carried out to determine the effect soak pits have on the model results. Privately owned soak pits should be accounted for in the sub-catchment hydrology.

Since 2005, KCDC requires all development to be hydraulically neutral and proposes different performance standards based on the availability of secondary overflow paths [10% or 1% AEP]. This is achieved through the deployment of soakpits and/or attenuation tanks on individual properties. Development within a catchment management framework is required to be hydraulically neutral in all design events [Q_{100}, Q_{50}, Q_{20}, Q_{10} and Q_5].

Council has completed a district-wide survey of all private stormwater connections, which will be made available to the successful tenderer as and when required, together with any other relevant information. A report which identifies areas where soakage may be feasible based on the rock type, permeability and drainage class is attached as Attachment 3-8.

In 2013, Council investigated potential areas of elevated groundwater levels vulnerable to climate change impacts. The report remains a '*draft for client review*' but can be made available to the successful tenderer if required.

3.2.3.7 Pump Stations

Where possible pump capacity should be based on manufacturer's capacity curves and the pumps performance should, where possible, be confirmed on site by drawdown tests. Pumps can be modelled in 1D or 2D. 1D representation will be required if the pump is located in a 1D open channel or within the pipe network.

3.2.3.8 Sumps

All publicly owned sumps should be included in the model. Privately owned sumps should also be included where they are required to drain significant areas, such as large car parks. The sump inlet capacity QdH curves for various sump configurations shown in Attachment 3-6 are to be used where the hydrology catchments discharge to the mesh. Where hydrology catchments discharge directly to the sumps, the inlet capacity QdH curves shown in Attachment 3-7 shall be used.

Key attributes that describe the sump barrel are inlet level, ground level, location and plan view area. There these attributes are missing or are incorrect in the available GIS / RAMM data and as-built plans, they should be determined through survey. Survey may not be required if the attributes can be reasonably estimated as follows:

- Lid levels may be taken from the DEM;
- Invert levels may be estimated from ground level and sump depth where this is known. If the depth is not known, the depth can be based on the appropriate sump type depths shown in Attachment 3-6.

Sump leads shall also be included in the pipe network model.

3.2.3.9 Additional Network and 2D Topography Updates

The model shall include the latest significant developments since 2012, including;

- Off ramps, road upgrades;
- M2PP, Transmission Gully, PP2O expressway projects [RONS];
- Dams and detention basins;
- Pump stations – Raumati beach, Otaki beach;
- Regional mitigation strategies.

3.2.3.10 2D Model Resolution

A 2m grid cell size shall be adopted [unless model runtimes become unacceptable].

When developing the model, the following should be considered:

- Check that topography of the linear features such as roads and railways have not been lost or degraded during the 2D surface interpolation process. Manual editing, or the use of TUFLOW's inbuilt line and polygon features, may be required to fix the problem areas. A site visit is generally recommended to investigate whether kerb lines and driveways are accurately represented, especially in areas where overland flow may occur. Any changes made to the base DEM or 2D surface must be documented.
- Large buildings that are likely to block overland flow paths should be excluded from the mesh. This is likely to include all commercial and industrial buildings and larger multi-unit residential developments.
- Physical site inspection of drainage structures may need to be undertaken and photographic evidence of all important features should be collected and checked to ensure good alignment with 2D model grid.
- In the case of modelling an open channel in 2D, the cell size should be fine enough to adequately define the channel cross-section in sufficient detail.

3.2.3.11 Boundaries

The boundaries should be located so as to minimize the impact of boundary effects such as 'artificial' backwater from outflow (downstream) and jetting of flows at inflow (upstream) boundary. It is important that the extent of the 2D surface is large enough to accommodate maximum flood extent, i.e. the flood waters should not touch the boundary.

Inflows to the hydraulic model from the hydrologic model can be applied using TUFLOW's 2D Boundary Condition (2d_BC) and Source Area (2d_SA) Flow vs Time (HT) type input features.

Where the catchment discharges to a tidal area an oscillating downstream tidal boundary should be used. Ideally the intertidal zone should be included in the 2D model with the tide boundary applied using a 2D Boundary Condition (2d_BC) Head vs Time (HT) type. Pipe outlets to the tidal zone and streams/rivers that discharge to the tidal zone should be connected to the intertidal area modelled in 2D. This allows the tide boundary to be changed in a single location in the model and also allows the interaction between tide levels and overland flows to be captured.

Consideration should be given to the upstream catchment areas such that they differentiate between stream catchments and overland flow catchments. Open channels up to 50m upstream of all network intakes shall be represented in the model.

3.2.4 Model Quality Review

Once the hydrologic and hydraulic model has been substantially built, it must be subject to initial testing.

3.2.4.1 Layout Representation

Importing a geo-positioned background map into the modelled network is often required to validate the model setup and connectivity.

3.2.4.2 Instability Tests

Initial testing shall be undertaken in order to gain confidence that the model is performing correctly, running at an appropriate time step without mass errors and numerical instabilities. Once confidence has been established, pre-defined routine checks can be used later to ensure that the model remains stable. To test the stability of the model, the model must be run with events of differing magnitude. The following events are suggested:

- 10% AEP design storm;
- 1% AEP design storm;

When resolving model instabilities, it is important to try and balance the necessary model adjustments with the models ability to accurately represent the study area.

3.2.4.3 Sensibility Tests

Sensibility testing will be undertaken prior to carrying out any model validation to ensure that the model results are 'sensible' i.e. within reasonably expected values. As part of the sensibility test of the hydraulic model, a comparison of peak flows and runoff volumes at critical locations with those estimated by other hydrological methods available shall be undertaken for a 1% AEP storm event.

One of the sensibility checks is to check model outputs against the output from Council's current, calibrated models.

Local energy losses at manholes, hydraulic structures, transitions or bends should be checked to see if modelled energy losses are reasonable and may require cross-checking with manual calculations or other software.

3.2.4.4 Mass Balance Checks

Mass balance checks shall be carried out to ensure that the total water mass unaccounted for (continuity balance) is small compared to the total inflow volume to the model. Usually a continuity error less than about 1.0% is acceptable.

3.2.4.5 Sensitivity Analysis

Sensitivity analysis is used to understand the impact on the model results of altering the model input assumptions of parameters. Typically, this involves identifying model inputs with the greatest uncertainty, and running the model with these adjusted to an extreme of their reasonable values.

Each catchment will have specific parameters or issues that need to be tested through sensitivity analysis. Typical issues include:

- Boundary Conditions
    - Rainfall depth
    - Rainfall profile
    - Downstream / tidal water levels [refer to Section 3.2.5.2]
    - Inflow boundaries
- Blockages
    - Blockage of key inlets, culverts/pipes.

3.2.5 Model Simulations

3.2.5.1 Calibration / Verification Simulation

Calibration of the model against suitable flow or level gauged data should be undertaken where possible. Typically for urban flood risk there is limited available monitoring data and calibration is not possible, therefore verification should be undertaken against available historical flood records such as photographs and public incident reports.

A series of meetings to be arranged with relevant Kāpiti Coast District Council to ensure that all relevant information is incorporated into the verification of the model. Historic events to be used should be identified by the successful tenderer in consultation with Council.

Kāpiti Coast District Plan will provide historic flood incident reports to the successful tenderer. These should be cross referenced with rainfall records to identify events suitable for use in model validation. Ideally at least three events, with a number of reported flood incidents and a high rainfall return period should be used for model validation.

The validation rainfall events should be simulated using corresponding boundary conditions and the results should be compared to reported flooding incidents. A summary of model validation results should be presented in the Model Build Report. Explanations should be provided in case the model could not replicate the reported flood incidents.

Consideration should be given to how conditions on the floodplain have changed since the time of the calibration / validation event. It is noted that drainage upgrades, or changes in topographic

features or structures, may have occurred following significant historic rainfall events. This should be recognised and considered when validating results using rainfall events prior to drainage upgrade works.

If model calibration or validation identifies changes to the model that are outside the parameters or methods within this specification these should be recorded in the model reporting along with detailed justification. The calibration / validation process should guide the final model configuration, as well as the selection of hydraulic grid cell resolution and in setting freeboard values.

3.2.5.2 Design Event Simulation

Simulation runs are required for the following Annual Exceedance Probabilities (AEP) events: 20%, 10%, 2%, 1% AEP present climate and the 2% and 1% AEP climate change event.

Climate Change

The climate change simulation shall be defined as the 2130 design rainfall profile, translating to a 20% increase in rainfall intensity, in combination with:

- A 1m increase in sea level, reflecting 2115 (to comply with the New Zealand Coastal Policy Statement 2010) in situations when the downstream boundary is coastal and is affected by tide.
- 2100 tail water level in situations where the downstream boundary is associated with a larger stream or river (i.e. not the coast).

Tail water level

When the catchment runoff discharges directly to the coast or a stream/river affected by the tide, an oscillating tidal boundary condition should be applied as a downstream boundary that equates to a storm tide with an exceedance probability of 5% AEP. The storm tides are typically made up of MLOS + tide + storm surge and the exceedance probability is for a combination of all three.

The peak of the tide is to coincide with the peak of the catchment runoff at the outlet. The peak tide levels to be modelled are to be confirmed with the successful tenderer, as detailed in section 3.1.5.

In situations where the downstream boundary is associated with a larger stream or river (i.e. not the coast) an appropriate water level has to be assumed. If a hydraulic model of the stream exists, a fixed or varying water level representing the river flood level under the respective storm probability and duration will normally be adopted from the modelling results. The methodology should be provided to the peer reviewer and KCDC for feedback and comment. Following incorporation of any feedback or comments, the method to be used in the model build.

Note: additional sensitivity runs may be required when statistical probability of the receiving environment does not necessarily correlate with the local catchment runoff. This may be required when:

- the catchment discharge and flooding is affected by the tide.
- the catchment response time of the receiving environment is significantly different from the modelled local catchment.

3.2.6 Model Build and Confidence Report

The Model Build and Confidence stage is to be completed with a Model Build and Confidence Report [section 4.2.2]

3.3 Network Performance Scenarios

3.3.1 Network Performance Scenario testing

The model shall be capable of scenario testing network performance.

Network performance identifies the level of performance provided by the stormwater drainage network including open channels and overland flow paths. The system performance consists of the following aspects:

- The capacity of the existing stormwater drainage system and identification of key bottle necks and system constraints.
- An assessment of the level of service provided compared to the desired level of service, and the identification of network sections that do not provide the necessary level of service.
- Floodplain and flood hazard mapping to identify significant flood hazard areas.
- Design flows and water levels for nominated design storm events and land use development / future rainfall scenarios.

The capacity of the existing pipe system shall be assessed. Its capacity should be assessed for the flow rate when the pipe is full under the 20% AEP and 10% AEP design event scenario conditions. Thematic maps should be developed for the 20% AEP, 10% AEP, 2% AEP and 1% AEP; as well as 2% AEP and 1% AEP + predicted impacts of climate change, and appended to an accompanying report.

The capacity of the existing culverts and bridges shall also be assessed. A summary of the capacity assessment of the culverts and bridges within the catchment shall be tabulated in an accompanying report. The report shall also tabulate a summary of the maximum water levels upstream of culverts, pipe inlets and bridges compared to overtopping level.

3.4 Stage 3: Mapping

3.4.1 Floodplain Maps

Interpreting output from the model is one of the most important aspects of the entire modelling process and an essential communication and planning tool for a wide-range of end–users. The outputs from the model are to be used for the following applications:

- Flood hazard identification;
- Kāpiti Coast District Council GIS layer for internal and public information;
- Land Information Memorandums [LIMS];
- Determination of whether level of inundation means the threshold for trivial or minimal under the Building Act.

It is therefore necessary to ensure that outputs sufficiently fulfil the requirements of all applications while maintaining consistency in presentation and extractable information.

Maps of the following model outputs are required for all the existing and future climate events listed in Section 3.2.3.11: maximum depth, maximum water surface elevation, maximum velocity and flood hazard.

Flood hazard classification shall be based on Smith et al [2014][1], shown in Figure 3-1 and Table 3-1.

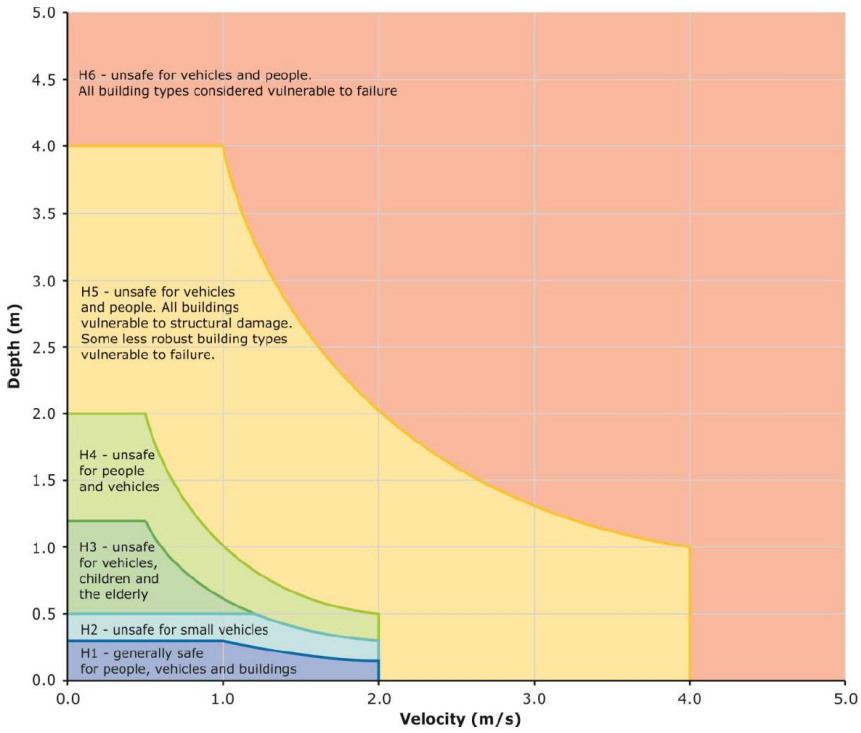


Figure 3-1 Flood Hazard Categorisation

Table 3-1 Flood Hazard Categorisation

Hazard Vulnerability Classification	Classification Limit [D and V in combination]	Limiting Still Water Depth [D]	Limiting Velocity [V]
H1	D*V ≤ 0.3	0.3	2.0
H2	D*V ≤ 0.6	0.5	2.0
H3	D*V ≤ 0.6	1.2	2.0
H4	D*V ≤ 0.1	2.0	2.0
H5	D*V ≤ 0.4	4.0	4.0
H6	D*V ≤ 0.4	-	-

The above hazard category is represented by Map Output Type “ZAEM1” in TUFLOW. By default TUFLOW tracks the maximum ZAEM1 hazard over the duration of a simulation on a computation time step basis. The maximum ZAEM1 hazard shall be mapped in 2D areas for each event. Open channels modelled in 1D are by their nature likely to be classified as a significant hazard area and may be mapped as ZAEM1 classification H6.

3.4.2 Flood Planning Level Maps [Application of Freeboard]

Before flood levels can be used for design purposes, an allowance for freeboard must be made. This allows for uncertainties in model data and for phenomena such as waves or culvert blockages that cannot be modelled.

[1] Smith GP, Davey EK and Cox RJ [September 2014]; Flood Hazard; WRL Technical Report 2014/07

Freeboard values shall be set through sensitivity analysis undertaken as part of initial model testing phase.

Applying freeboard to floodplain inundation predictions extends the flood surface beyond the extent predicted by the model. The consultant to provide a method for the application of freeboard to be used in the model to the peer reviewer and KCDC for feedback and comment. Following incorporation of any feedback or comments, the method shall be applied.

To avoid vertical discontinuities of the flood surface at the margins of the flooding, freeboard is added to the top of all peak flood levels and a simulation run to let the water spill over the floodplain. The peak levels and extent reached on the floodplain in that simulation then becomes the final 'with freeboard' flood surface.

Maps including freeboard allowance shall be developed for the 2% and 1% AEP climate changes.

3.4.3 Map Creation Advice

Consideration of the end-user at the map concept stage will ensure that the map has an appropriate design and includes sufficient content of the intended use. Maps should be free ambiguity and readily interpretable, and where necessary, accompanied by a comprehensive [jargon free] explanatory note.

All flood maps should, as a minimum include the following:

- Title of map, making clear reference to the map content including flood event size in AEP and mapped data type [flood extent, peak level, peak velocity, etc.]
- Location of the map as part of the catchment
- North arrow and scale bar [preferably using a scale bar to allow for changes in page size]
- Author / creator of the map and any limitations regarding its currency or application
- Base data for the map and date of publication.

The development of flood maps should also consider how to convey uncertainty in the mapping to ensure that the user can make well-informed decision, and whether the map will be accessible by people with colour blindness, and if the maps will be legible if printed in black and white.

Readers with colour blindness may have difficulty recognising different colour bands, particularly where adjacent mapped colours are of a similar lightness, and where red and green are mapped in adjacent regions. When developing maps, some modifications can be made to the map style to improve the accessibility of maps to readers with colour blindness, including:

- Providing a distinct contrast between foreground and background colours, e.g. between the flood results and aerial photography
- Using thicker contour borders which are labelled directly and do not rely solely on the polygon fill colour for interpretation
- Varying the lightness of colours used in the map, e.g. high contrast colours are more easily interpreted than colours of a similar contrast, even if the hues are different

Using a colour scheme recommended by the colour brewer website as suitable for colour blind use also provides an option for 'photocopy safe' colours, which will increase the legibility of flood maps when viewed in black and white.

3.4.4 Reporting

The floodplain, flood hazard and flood planning level maps will be appended to the Model Build and System Performance Report as a series of A3 floodplain maps at Kāpiti Coast District Council approved scales. The report contact is listed in section 4.2.3.

4 Project Deliverables

4.1 Reporting Schedule

Adequate documentation should be maintained and provided with the model in order to ensure that the maximum use can be obtained from the models, to give confidence in the model and results and to ensure that new users can easily understand the models. The reports in Table 4-1 shall be submitted:

Table 4-1 Report Delivery Milestones

Deliverable	Report Title	Due Date
1	Model Extents and Data Quality Assessment	As agreed with KCDC at the Pre Commencement Meeting
2	Hydrology Model and updated Isohyet based calculation of Design Peak flows	As agreed with KCDC at the Pre Commencement Meeting
4	Model Build and Confidence	As agreed with KCDC at the Pre Commencement Meeting
6	System Performance Report	As agreed with KCDC at the Pre Commencement Meeting
7	Model Build Report (including maps)	As agreed with KCDC at the Pre Commencement Meeting

All the above reports should be prepared and presented electronically in both PDF and Microsoft Word [2010] document format.

KCDC will review and provide formal comment on the work at any stage throughout the process. KCDC may also arrange for the model / reports to be peer reviewed by a third party, depending on the criticality of the project outputs. This will most likely (not necessarily) be at one or more of the following key milestones:

- Model extents and data assessment report;
- Initial model build;
- Validation / calibration; and
- System performance.

The outcomes of any reviews are to be included in the work and final deliverables.

4.2 Reporting Deliverables

4.2.1 Model Extents and Data Assessment Report (Deliverable 1)

A Model Extents and Data Assessment Report shall be prepared, submitted and finalised at the early stages of the Project which will include details of hydrological and hydraulic model extents including the basis for their selection, model build data availability and assessments, and data collection and survey requirements.

The report aims to define the model extents and schematization, and identify the gaps in the required information and how to fill the gaps and plan the model build(s). The Model Extents and Data Assessment Report shall include the following:

- The modelling study objectives, activities and scope.
- Review of historic flood issue records
- Summary review of any previous relevant modelling carried out in the catchment.
- The results of a detailed assessment of the adequacy and reliability of available data in the context of the agreed hydraulic model schematic.
    - list of all available data including their sources and date of data supply – manholes, pipes, culverts, ponds/ basins, topographical, operational, hydrologic (infiltration capacity) and hydrometric data.
    - Identification of, and specification for, additional data needing to be obtained to enable the models to be built to the agreed schematic and to Kāpiti Coast District Council specification.
    - A representation of any of the missing data priority.
    - Identification of locations where good quality data is not available but where it has been agreed that estimation will not impact significantly on the outputs of the model.
    - Recommended methodology for collection of additional data.
- Description and presentation of the agreed Model Extent and Model Schematic.

It is important that the sources of all data that is to be used in the model build are referenced in the Model Extents and Data Quality Assessment Report.

4.2.2 Model Build and Confidence Report (Deliverable 4)

This report shall contain a description of the catchment and its drainage system, the work involved in building the model and explanation of why the reader can have confidence that the model is fit for the purposes it was built for. Detail of the content of the report is included below:

<u>Introduction</u>

- A brief description of the background to the modelling study i.e. the purpose for which the model has been built, the date it was built, and the software used;
- The general objectives of stormwater catchment modelling including the activities and scope of the modelling project.
- Key limitations of the model and summary of confidence/usability of the model

<u>Catchment Description</u>

- A description of the catchment including the location in the region;

- Description of the existing stormwater drainage system, geology, soils, topography, existing and future land use;
- A discussion on the degree of homogeneity in the catchment;
- A brief discussion of reported flooding issues within the catchment.

Model Build and Confidence

- Modelling Software: A description of the modelling software used for hydrological and hydraulic modelling of the catchment, including model version and service packs (or any hot fixes) installed.
- Review of Exiting Data: A description of asset, hydrometric and topographical data used for model construction including its sources (including date of data supply); summary of rainfall and historic flood data
- Hydrological Model:
    - A description of the schematisation / modelling method used for catchment rainfall-runoff process;
    - A description of sub-catchment delineation, impervious areas and CN numbers;
- Hydraulic Model:
    - A description of the schematisation / modelling method used for hydraulic drainage network;
    - Details of 1D and 2D hydraulic model components (e.g. pipes, open channels, control structures, pump stations).
- Boundary Conditions:
    - Summary of existing and future design rainfall depth and temporal profiles estimates;
    - Summary of downstream tidal boundary used in the model.
- Model Confidence: Provide details of the processes and analysis undertaken to provide confidence in the model. This includes:
    - Quality assurance
    - Sensibility and stability checks
    - Calibration\Validation
    - Sensitivity analysis

  The model has to be internally quality assured and checked and has to be stated as “fit for purpose” in this section;
- Model Limitations and Assumptions: A description of limitations in the modelling and how they influence the applicability of the results
- Model Scenarios and Simulations: A description of the approach to developing various design event and climate change model scenarios;
- Summary of the system performance assessment (if required;
- References: A comprehensive list of all references used in the modelling study shall be included.

4.2.3 Model Build Report Including Maps (Deliverable 7)

The floodplain, flood hazard and flood planning level maps will be appended to the Model Build and System Performance Report as a series of A3 floodplain maps at Kāpiti Coast District Council approved scales. Documentation associated with the mapping shall include:

- List of significant flood risk areas including the causes of flooding;
- A description of methodology for identification of flood risk areas and floodplain mapping.
- Discussion of predicted flood risk areas in relation to the reported flooding issues identified from issues register/questionnaire response.
- Detailed description of the significant flood risk areas including duration, causes of flooding and plans at an appropriate scale
- A description of methodology for identification of flood risk areas and floodplain mapping, as described by on Smith et al [2014][2].
- Discussion of predicted significant flood hazard areas in relation to depth and/or velocity to pose a possible safety hazard to vehicles and pedestrians, including duration of hazard.

4.3 Digital Deliverables

In addition to the above reports and maps, all necessary electronic files, including but not limited to all model simulation input and result files, GIS files of maximum floodplain extents, depth, velocity and hazard, system data files and databases, metadata, model log-book, spreadsheets, QA checks, PDF files, MS Word document files and other associated files generated shall be submitted.

- All source data (including the original asset data) must be electronically stored, clearly labelled (including date of data download) and presented with the project upon completion. This ensures the source data can be compared to any updates made in the future. The source data and source date of all input data must be recorded.
- Metadata outlining the source of all important model build data must be maintained in order for KCDC to assess the accuracy of the modelled asset data any time in the future. The modeller to develop detailed procedures for preparing metadata in consultation with peer review. The metadata shall be included using the model data flags and comments and also duplicated in a spreadsheet or geo database, electronically stored and accompany the model upon delivery.
- It is good practice to maintain a record of model changes within a modelling log. All day-to-day assumptions, major decisions made and the basis behind them, and details of parameter adjustments should be documented in a model log-book. The model log-book shall be prepared in spreadsheet, electronically stored and accompany the model upon delivery.
- All spatially oriented data shall be presented in terms of NZGD2000 and using the NZTM projection. Levels shall be in terms of 1953 Wellington Local Datum. If data is presented in a coordinate system other than the above, this must have been previously approved by KCDC and each dataset must be clearly referenced with a coordinate system. This applies to spatial data stored inside and outside of GIS systems

[2] Smith GP, Davey EK and Cox RJ [September 2014]; Flood Hazard; WRL Technical Report 2014/07

- Quality assurance is an on-going process throughout the modelling project. At the completion of the model build phase, the model shall be reviewed internally by the modeller to confirm it is built in accordance with the specification and the brief and to assure it is 'fit for purpose'.

  At the beginning of each stage, the modeller to provide a standard template for model review (QA/QC) to the peer reviewer and KCDC for feedback and comment. Following incorporation of any feedback or comments, the template should be used to review the model, and be included with the model deliverables.

4.4 Public Workshops/Meetings

Community / stakeholder engagement is vital during the development and adoption of the flood hazard maps. All community / stakeholder engagement to be led by KCDC, with the consultant providing technical support for, and attendance at, a number of public workshops and/or Councillor and Community Board briefings.

In addition to attending public workshops and briefings, the consultant may also be required to assist KCDC with responding to queries that may arise as part of the model re-build.

4.5 Project Meetings

Throughout the contract period, liaison with KCDC and its peer reviewer(s), and attendance at monthly progress meetings as / if required. Iwi Groups, GWRC and various other interested parties may also be liaised with on an as-required basis.

APPENDIX B LiDAR technical specification

DRAFT



KAPITI COAST DISTRICT COUNCIL

KAPITI COAST DC LIDAR 2016-17

VOLUME 11334B01NOK

Summary

Project

AAM was engaged by Kapiti Coast DC to undertake an airborne LiDAR survey over part of the district in 2016-17. The data was captured on 5th and 9th February 2017.

LiDAR data and other products supplied in this volume are as follows:

1. Classified point cloud as LAS v1.2
2. 0.5m Contours as ESRI Shapefile
3. Ancillary Files:
   - Trajectories as ESRI Shapefile
   - Tile Index Metadata as ESRI Shapefile
   - Project Extent as ESRI Shapefile

The project specification for point cloud accuracy is vertical <= 0.10m (90% CL), and horizontal <= 50cm. This dataset is supplied on NZTM map projection and Wellington 1953 (via NZVD2016) vertical datum.

(Ref: PWNZ 11334B, PW 27440B)

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6 Ossian Street, Napier, New Zealand info@aamgroup.com

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1. Project Report....3
2. Data Installation....4
3. Additional Services....5
4. Metadata....6
5. Accuracy....7
6. Conditions Of Supply....8
7. Validation Plots....9

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1. PROJECT REPORT

Acquisition: AAM was engaged by Kapiti Coast DC to undertake an airborne LiDAR survey over part of the district in 2016-17. The data was captured on 5th and 9th February 2017. The acquisition design was planned on recording a minimum 4 pulses per square metre (NPS) for each flightline swath. An earlier flight, flown on 11 Dec 2016 was not required.

Ground Support: GPS base station support was sourced from LINZ CORS. The ground check points acquired by Sound Surveying Ltd allowed an assessment of the accuracy of the ALS point cloud data.

Data Processing: Reduction of the ALS data proceeded without any significant problems. Laser strikes were classified as follows:

- Automatic classification of all collected data (ICSM Level 1); Default, Ground, Low/high points.
- Manual checking and editing of the data classification to improve the quality of the ground definition - to achieve 98% accuracy levels for ground data classification.

The classified point cloud product contains the following classes: Default, Ground. The focus of the Level 1 classification was on a creating a clean ground point cloud, free of atmospheric points and noisy return data.

Further Processing: The contours supplied on this volume are designated “Engineering Contours”. They are compiled from a rigorous triangulation of the supplied data. No cartographic licence has been applied. They are intended to provide a visual representation of the terrain data, and are not suitable to be used as a cartographic map.

Data Presentation: The data provided on this volume has been supplied in accordance with a specification agreed with the primary client. Subsequent users experiencing difficulties in handling the data should please contact AAM to arrange a more appropriate data presentation.

Further Issues: There are no further issues to report.

Project Contacts:

Client
Wayne Maxwell

Company
Kapiti Coast DC

AAM Account Manager
Chris Worts c.worts@aamgroup.com

AAM Project Manager
Lorraine Claydon l.claydon@aamgroup.com

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2. DATA INSTALLATION

Data formats	:	LAS 1.2, ESRI Shapefile
Information files on media	:	Readme_11334B01NOK.pdf
Data formatted on	:	26.05.2017
Disk volume	:	11334B01NOK

README FILE

This document (Readme_11334B01NOK.pdf) is provided as an Acrobat file in this volume.

FILE SIZES AND NAMES

Discrete point data sets are provided in NZTopo50 1:500 tiles with the following file naming convention:

KCDC_Topo50Prime_Scale_tile_[ProductID].[format]

[ProductID] – CL (Classified LAS), CONT (Contour)
[format] - *.las, *.shp,

Separate folders are used to separate the products.

LAS file point classifications levels are formatted to comply with ASPRS Standard LiDAR Point Classes.

1 Default
2 Ground

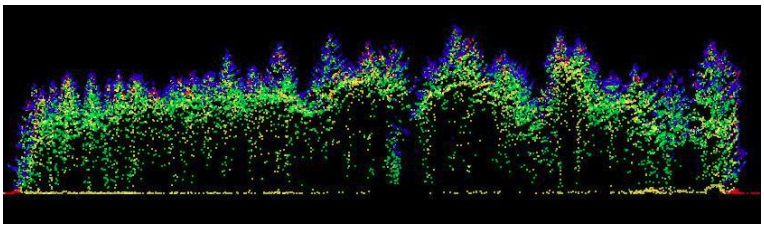
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3. ADDITIONAL SERVICES

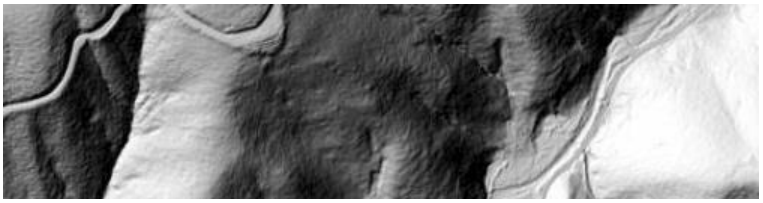
Product Generation

AAM can perform the following additional services on the data contained on this volume if required:

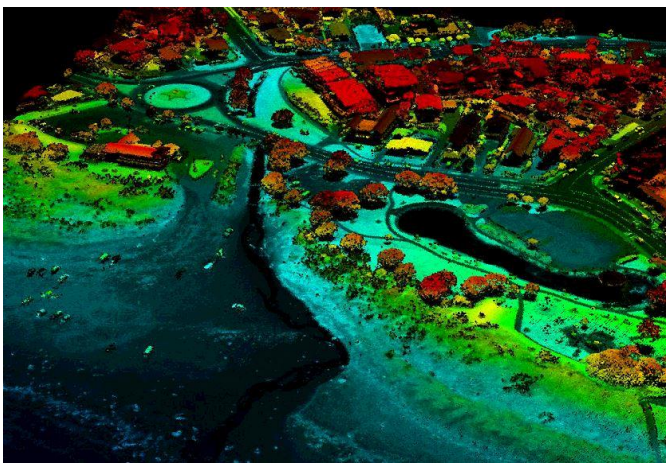
Change horizontal datum	:	to another local grid
Alter geoid modeling	:	by transforming ALS data to fit orthometric survey heights
Improve data classification	:	by tailoring parameters to suit regional variations
Further classification	:	assist building identification by further classifying non-ground strikes
Further point cloud manipulation		by normalising point cloud to terrain, creating canopy height models etc



Data thinning	:	to remove superfluous points not adding to the terrain definition
Data subset	:	by dividing the data into different tiles or polygons
Data presentation	:	by creating additional contours, hill shade reliefs profiles, perspectives, flythrough, colour-coded height plots etc.



Ground truthing	:	by comparing the ALS terrain model with extra independent height data
Data gridding	:	to convert the measured spot heights into a regular grid
Intensity Image	:	greyscale image created from laser's intensity returns
3d Perspectives	:	image draping /slope models



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4. METADATA

SOURCE DATA

Item	Source	Description	Ref No	Date
Laser System	AAM	Riegl Q1560	11334B	05, 09.02.2017
Base Stn Coords	LINZ	CORS	11334B	"
Field Survey Data	Sounds Surveying Ltd	RTK GPS	11334B	11-15.12.2016

LASER DATA CHARACTERISTICS

Characteristic	Description
Device Name:	Riegl Q1560 – 2 laser channel system
Half Scan Angle:	30 degrees
Laser Pulse Rate:	350 kHz
Laser Pulse Mode:	Multipulse
Laser Return Types:	1^{st}, 2^{nd}, 3^{rd} … 7^{th} and last
File Format:	ESRI Shapefile, LAS 1.2
Contour Interval:	0.5m
Horizontal Datum:	NZGD2000
Vertical Datum:	NZVD2016
Map Projection:	NZTM
Vertical Accuracy Specification:	±0.06m RMS
Horizontal Accuracy Specification:	±0.30m RMS

REFERENCE SYSTEMS

	Horizontal	Vertical
Discrete Data		
Datum Projection Geoid Model	NZGD2000 NZTM N/A	Wgtn 1953 N/A NZGeoid16
Primary Reference Station	OTAK, PAEK	OTAK, PAEK

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5. ACCURACY

PROJECT DESIGN ACCURACY

Project specifications and technical processes were designed to achieve data accuracies as follows:

	Measured Point	Basis of Estimation
Vertical data Horizontal data	<= 0.06 m <= 0.3 m	Project Design Project Design
Test points	0.05m	Survey methodology used

Notes on Expected Accuracy

- Accuracy estimates for terrain modeling refer to the terrain definition on clear ground. Ground definition in vegetated terrain may contain localized areas with systematic errors or outliers which fall outside this accuracy estimate.
- Laser strikes have been classified into “ground” and “non-ground”, based upon algorithms tailored for major terrain/vegetation combinations existing in the project area. The definition of the ground may be less accurate in isolated pockets of dissimilar terrain/vegetation combinations.

LIMITATIONS OF DATA

- The definition of the ground under trees may be less accurate.

DATA VALIDATION

- Ground data in this volume has been compared to 378 test points obtained by field survey and assumed to be error-free. Comparison of the field test points with elevations interpolated from measured data resulted in:

Ref Point Site	No. of Points	Mean Difference (m)	Std Deviation (m)	RMS (m)
Kapiti	378	0.019	0.06	0.06

- Data classification has been manually checked and edited against any available imagery.

USE OF DATA

- Intended use : Preliminary Design subject to final survey
- Intended use : Planning, Conceptual Design
- Intended scale of use : 1:500

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6. CONDITIONS OF SUPPLY

The data in this volume has been commissioned by **KAPITI COAST DISTRICT COUNCIL**.

The data in this volume is provided by AAM NZ Limited (AAM) to **KAPITI COAST DISTRICT COUNCIL** under **MQM021_Terms of Engagement_NZ Version 1 (Amended for WAGGIS Imagery Project 2016)**, subject to the following conditions:

1. This file (Readme_11334B01NOK.PDF) is always stored with the unaltered data contained in this volume.

2. This volume of data is not altered in any way without the approval of AAM. The data may be copied from this file to another.

Any breach of these conditions will result in the immediate termination of the license issued by AAM, and KAPITI COAST DISTRICT COUNCIL will indemnify AAM from all resulting liabilities.

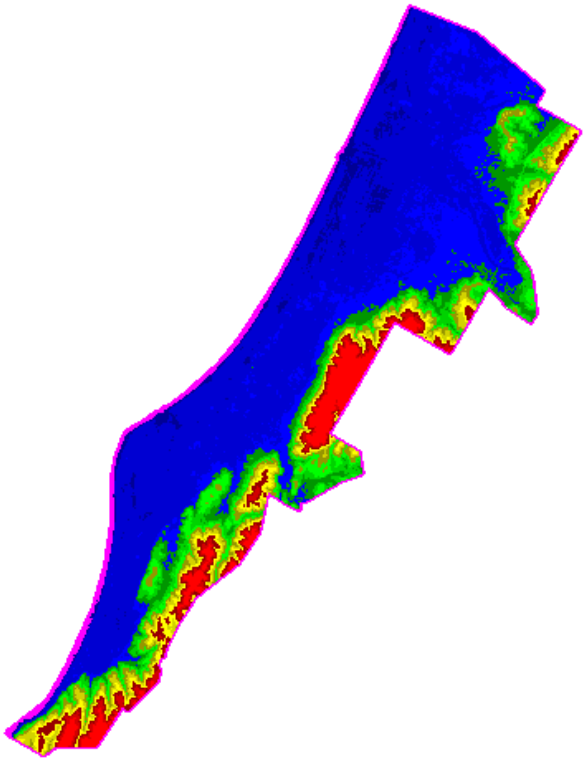
Any problems associated with the information in the data files contained in this volume should be reported to AAM NZ Limited. A list of project related contacts is listed on page 3 under the Project Report heading.

6 Ossian St,
NAPIER
New Zealand
Telephone +64 27 502 6900
Email info@aamgroup.com
Web www.aamgroup.com

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7. VALIDATION PLOTS

Project area coloured by elevation



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APPENDIX C Land Cover Technical Specification

DRAFT

Data Quality and Accuracy Report

Kapiti Coast District Council (KCDC)
Machine Learning Land Cover



March 5th 2021

Lynker Analytics Ltd ©2021

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1 Purpose 3
2 Methodology 4
3 Land Cover Classes 5
3.1. Roads 7
3.2. Buildings 8
4 Data Accuracy 10
4.2. Results 12
5 Post processing 14
6 Data Definition 14

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1 Purpose

This report explains the methodology, land cover class definitions, data definitions, data validation and accuracy and the final data generated and supplied from this project.

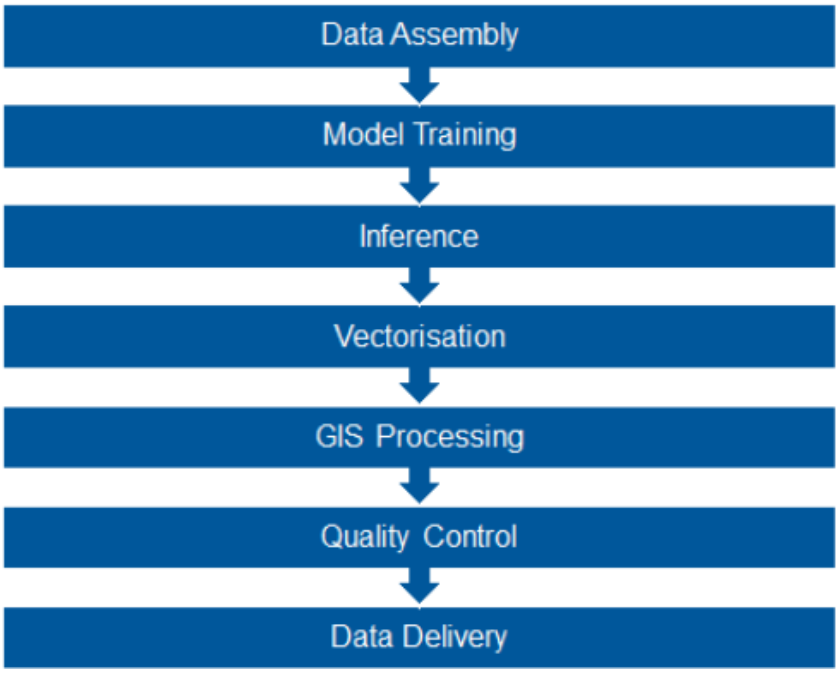
This Document has been prepared by Matt Lythe, David Knox & Michael Stanley, Lynker Analytics Ltd.

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2 Methodology

Lynker Analytics was commissioned by Kapiti Coast District Council (KCDC) to produce a set of land surface classes from 2017 10cm RGB imagery mosaiced and clipped to the built up area within the Kapiti Coast region. The data set is designed to be used to inform hydrological imperviousness estimates and hydraulic roughness for overland flow for the build/update of the city's 2D stormwater models.

The project encompassed seven phases which were run in a serial process to generate all the output data.



Stages of production

We used Python, Tensorflow and Keras to build neural network architectures to annotate, classify, segment and achieve object localization from the imagery. Our approach used an amalgamation of the provided data and machine learning to classify each pixel in the KC region.

An inference cycle was run using Python and Tensorflow/Keras. The ML Models are mathematical models that generate predictions on these target features using the patterns extracted from training data sets.

Vectorisation and comprehensive GIS post processing then occurs in ArcGIS. This includes clipping, geometry size checks, vertex counts, criteria-based dissolve and eliminate tasks and full data re-assembly.

Finally, quality checks and validation exercises are run to ensure all features are representative and topologically correct. The models are calibrated using a separate hold out set of ground truth data carefully selected and not used for training.

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3 Land Cover Classes

Eight Land cover classes were classified including:

1. Buildings,
2. Roads,
3. Vegetation (trees and shrubs),
4. Scrub (rough grass, rushes, low profile vegetation often around wetlands),
5. Grass (open space without trees and shrubs),
6. Water,
7. Sand/Gravel/Bare Earth,
8. Other impervious (driveways, carparks etc).

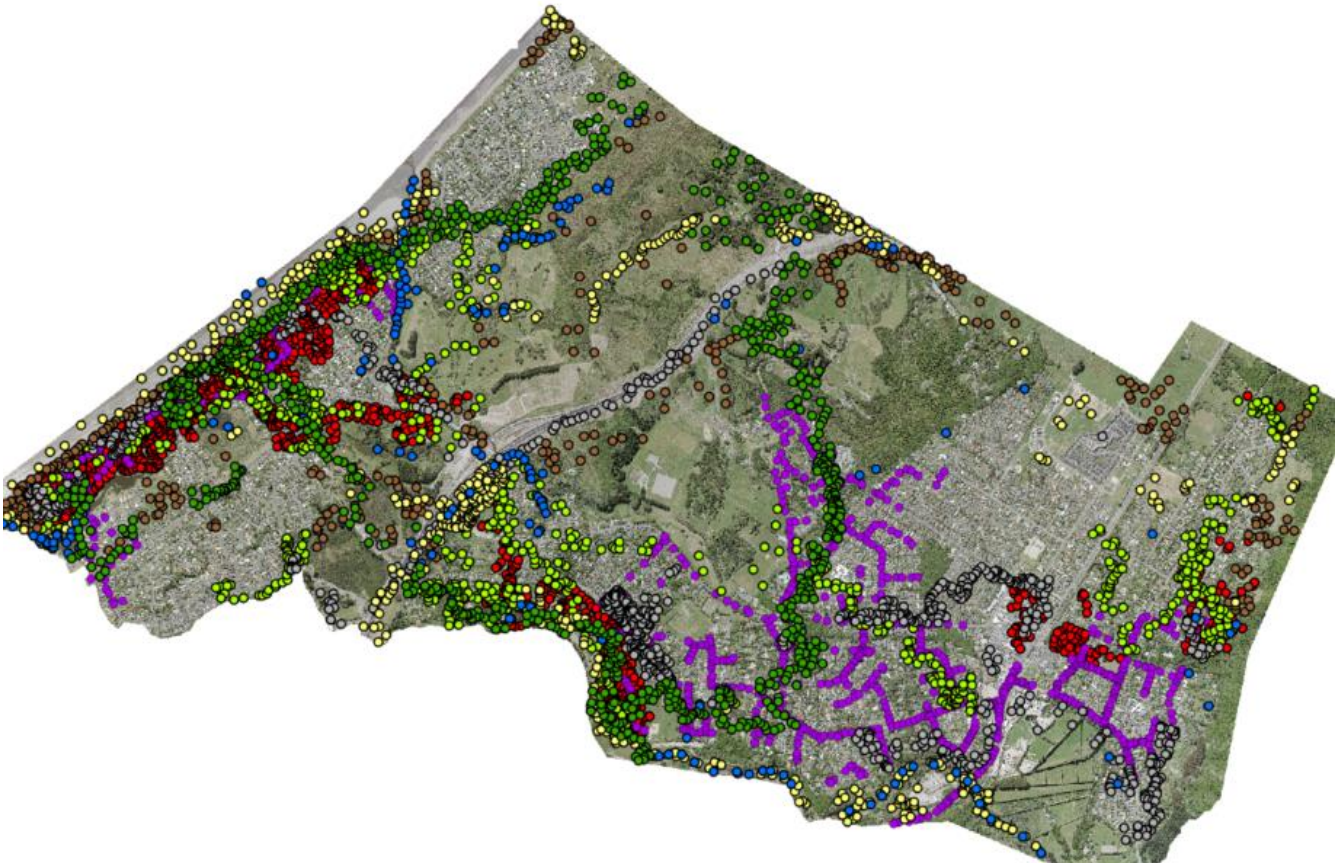
The study area is shown below.



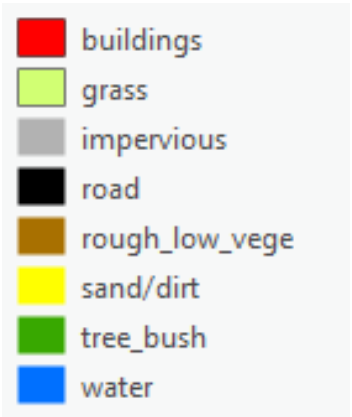
Modelling area.

Existing Roads and Buildings from KCDC were used. Machine Learning was then used to generate additional features in these classes (Roads, Buildings) where present in the ortho photography. The training data for these classes was generated by Lynker Analytics using both point and polygons annotations.

Training the initial model was focused on the Waikanae region shown here.



Training annotations in Waikanae area



Manual point annotations were generated to streamline the data creation process.

6,000-point annotations were created over all classes except roads, for which 500 polygons were created.

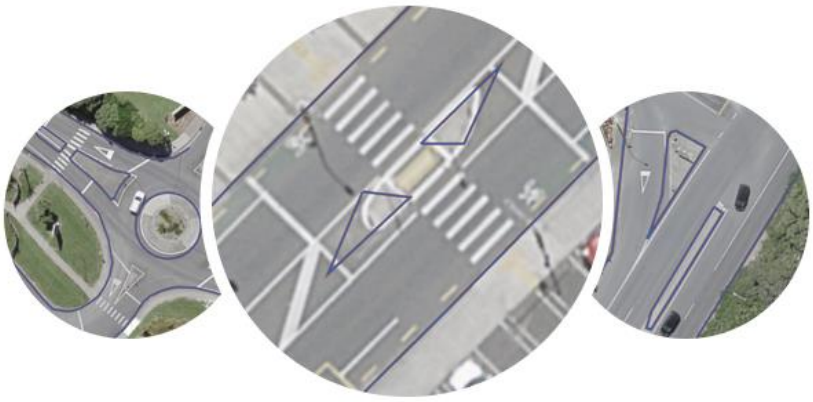
These point annotations were roughly balanced between the remaining classes. The legend for the annotations shown above are contained in this legend.

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3.1. Roads

Existing Roads supplied by KCDC were used as a foundational layer. The Kerb data was in a polyline format and was aligned to both sides of roads. After ensuring that all the ends of these lines were joined from one side of the road to the other the Feature to Polygon tool was used to create polygons over the roads for the entire Kerb dataset.

Although this provided us with a large quantity of polygons for the roads class many of these polygons were for objects that had kerb lines to specifically separate them from surrounding roads. Examples of such objects were roundabouts, pedestrian crossing islands and islands to separate traffic these are shown below.



By selecting polygons by attributes where shape area / shape length < 1.2 we were able to remove many such instances.

The next step was to identify the roads that were missing or had changed since the kerb dataset was created. By using the machine learning model's output new roads are added to our road polygons dataset by using a split to separate the desired road from the rest of the inference and then appending this to the roads attribute table.

Roads that were missed by both the kerb data and the inferences were added manually.

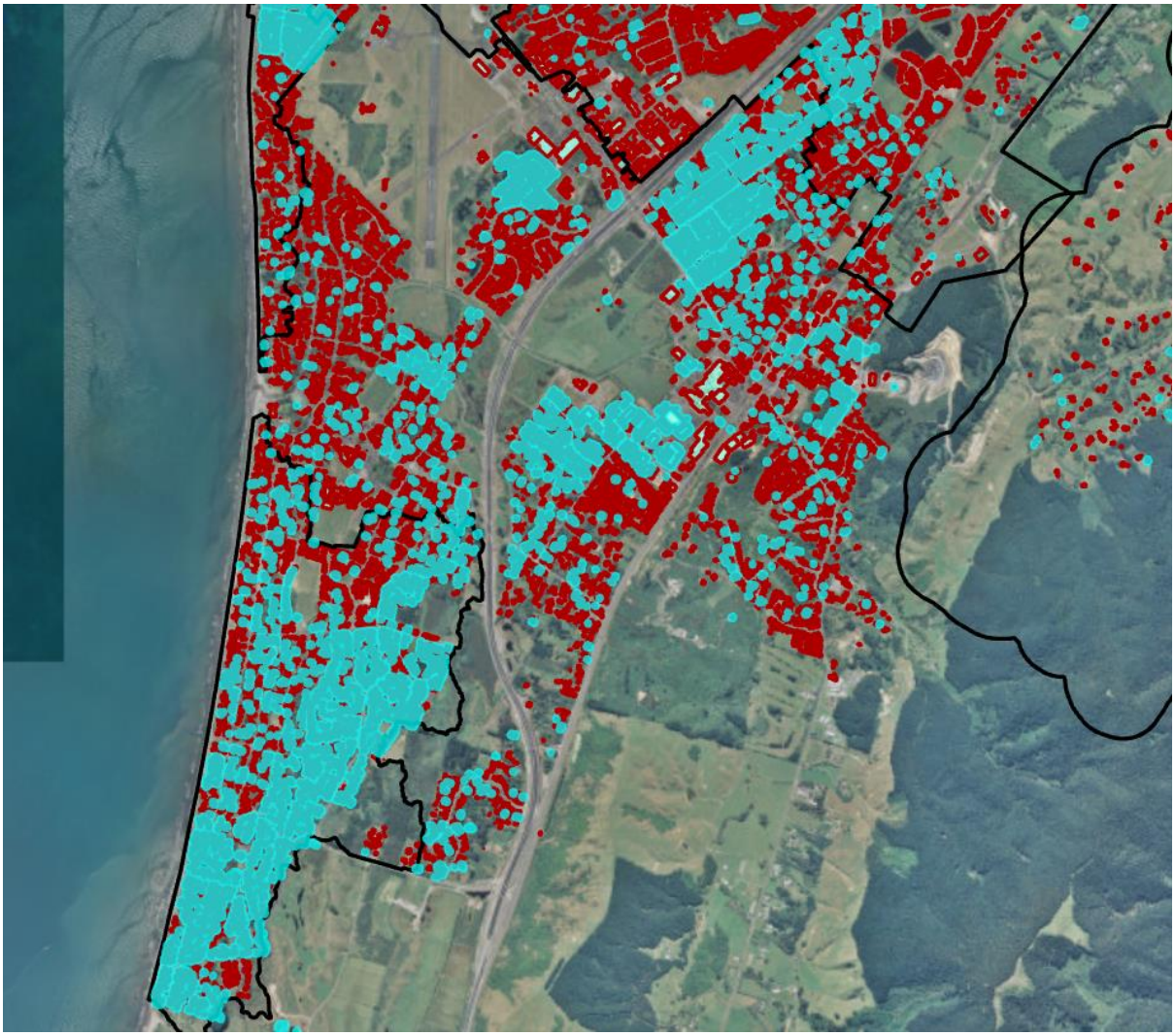
In the image to the right the kerb data is shown by the blue lines while the model's inference is shown by the yellow polygons.



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3.2. Buildings

We found the current LINZ building outlines to be good in general but with some gaps and some misalignments. The buildings are particularly good in Waikanae and parts of Paraparaumu. However, the alignment issues were significant in Otaki, Paekakariki and Raumati South. Where alignment issues and gaps were noticed, they were corrected by either human edits or automated ML feature extraction. This screen shot of Raumati and parts of Paraparaumu gives a qualitative view of where edits and additions occurred:



Existing Building Outlines in red and additions in blue

Edits and additions were made using both human edit/addition and ML automated feature extraction.

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The criteria used for automated replacement of existing LINZ buildings with buildings from the Lynker Analytics land cover model was:

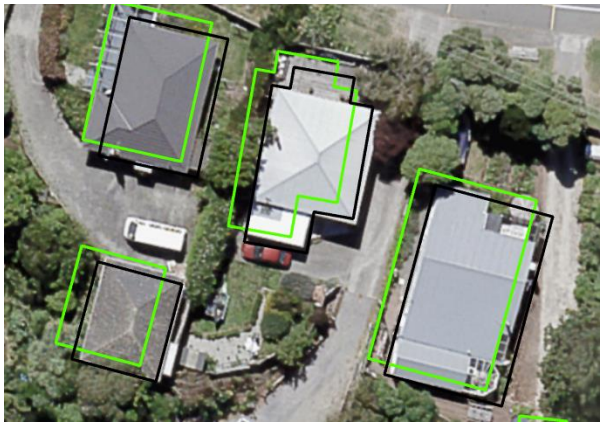
1) New and old shapes intersect
2) New shape centroid within old shape
3) Old shape centroid within new shape
4) New shape area within 20% of old shape area
5) Area of intersection of old and new shape less than 80% of area of old shape

In Total,

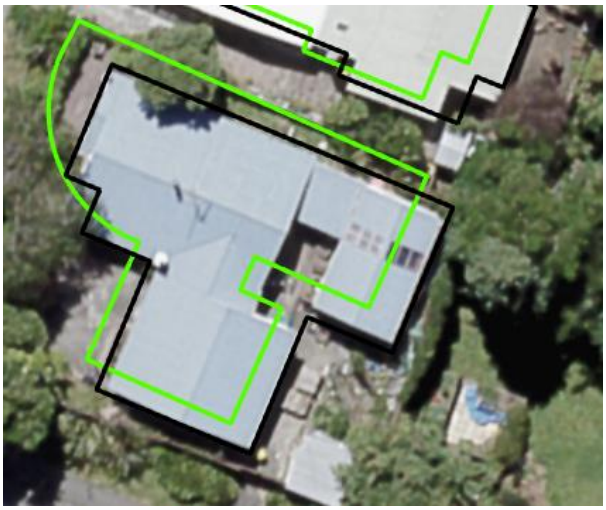
Building Source	Count
LINZ	27595
Lynker Analytics model	3572
Realigned to imagery	1731
Human Addition	97

Please note that the level of processing and QA/QC for this project was not focused on building outlines as a standalone product and the quality achieved is not that of a dedicated buildings outlines project. However, the building areas found should be an improvement over the existing areas, in general, as applicable to hydrologic models.

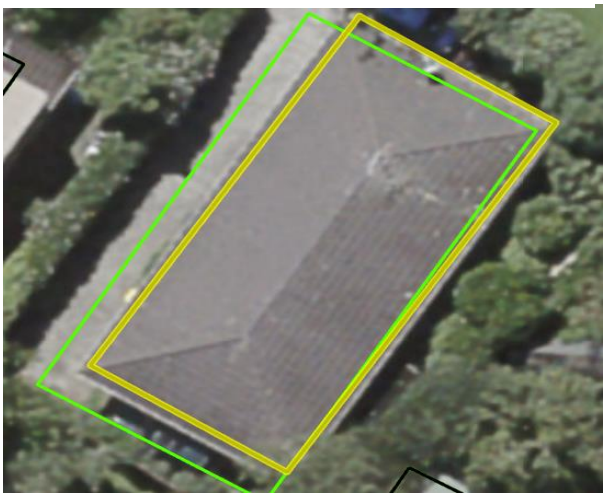
Move Edit



Edit Vertices



Machine Learning Inference Edit



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4 Data Accuracy

The following tables and discussion examine the confusion and accuracy of the machine learning model predictions relative to hand labelled validation data points within the area of interest.

The validation data was not used to train the machine learning models but was captured at the same time and using the same process as the training data. The per pixel statistics are calculated on the raw machine learning output and are before the GIS post processing which includes thresholding, vectorisation and simplification.

The data below is based on the raw output of the land cover machine learning model. As such, it will include points belonging to small polygons that are removed in subsequent GIS post processing. Particularly small grass, water, sand-gravel-clay-earth polygons abutting or within impervious polygons were removed and the area dissolved into the dominant adjacent area.

The Confusion Matrix explains the prediction at a pixel scale of each of the classes.

	Building	Grass	Imperv.	Road	Rough	Bare	Vegetn.	Water
Building	180	0	2	0	0	1	0	1
Grass	0	134	0	0	3	0	1	1
Impervious	3	2	133	6	0	6	0	0
Road	0	0	1	192	0	0	1	0
Rough	0	4	0	0	76	2	1	0
Bare/sand	0	1	2	0	0	89	0	0
Vegetation	0	3	2	0	2	0	144	1
Water	0	1	0	0	0	1	1	27

Confusion Matrix

The following table presents several additional statistical parameters including:

- True Positive (TP) = the model correctly predicts the positive class
- False Positive (FP) = the model incorrectly predicts the positive class
- True Negative (TN) = the model correctly picks the negative class
- False Negative (FN) = the model incorrectly predicts the negative class
- Precision = TP / (TP + FP)
- Recall = TP / (TP + FN)
- F1 = 2 X ((Precision X Recall) / (Precision + Recall))

Precision, Recall and F1 scores provide additional measures of accuracy by class.

- Precision shows how precise/accurate the model is out of those predicted to be positive.
- Recall calculates how many of the actual positives the model capture through labeling it as True Positive.

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- F1 conveys the balance between the precision and the recall. An F1 score reaches its best value at 1 and worst value at 0. A low F1 score is an indication of both poor precision and poor recall.

	precision	recall	f1-score	count
Buildings	0.98	0.97	0.98	188
Grass	0.91	0.96	0.93	134
Impervious	0.92	0.90	0.91	150
Roads	0.97	0.98	0.98	190
Rough	0.91	0.91	0.91	89
Bare/sand	0.92	0.98	0.95	89
Trees	0.97	0.92	0.94	154
Water	0.90	0.87	0.88	30
Accuracy			0.95	1024
Weighted avg	0.95	0.95	0.95	1024

The weighted average F1 score of **0.95** showing us that both the precision and recall were high. F1-score is a better indicator of the models performance than the accuracy as imbalanced classes are present, i.e. as the segmentation was performed on-land the water class will be less present than vegetation.

Precision for buildings tells us that 98% of pixels that are classified as buildings are buildings and Recall tells us that 97% of actual buildings' pixels from the validation set are correctly classified.

Overall, the model performed well across all classes. However, the recall for the impervious class was significantly lower than the other classes, though still very good, this may indicate that some impervious surfaces were mis classified by the model.

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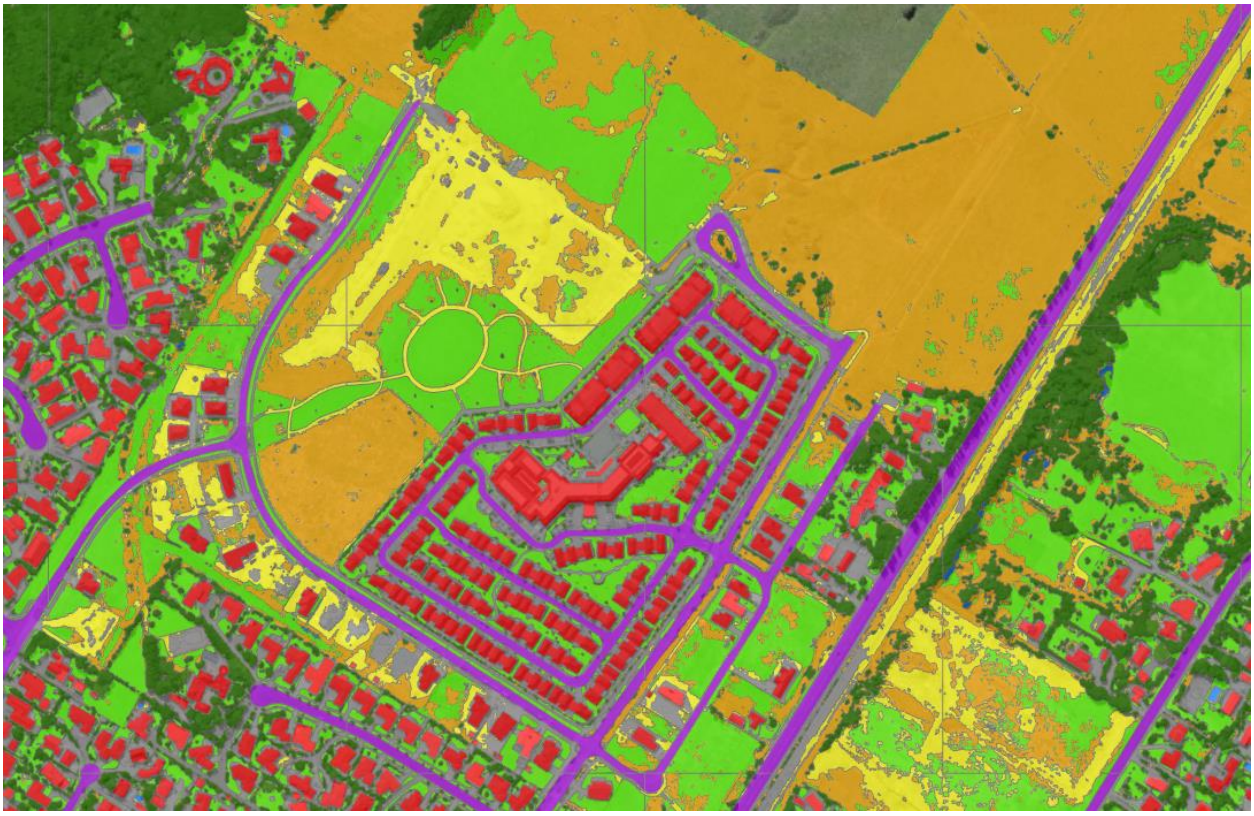
4.2. Results

Examples of the final data set are shown below.



Residential area

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New subdivision



Industrial area

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5 Post processing

The machine learning process produces raster tiles depicting the classes as predicted by the models. We then apply a systematic GIS post process which involved the following steps.

I. Vectorise inference rasters
II. Merge KCDC Roads and Buildings (prioritise these layers)
III. Merge Lynker Analytics Roads and Buildings (prioritise these layers)
IV. Clip to Area of Interest (KCDC Supplied) – retain tolerances
V. Fill any holes introduced through these steps using original layer step IV.
VI. Visual inspection and clean-up including
    a. Correct outdated Kerb data and convert to polygons
    b. Reclassify water false positives in sand dune shadows to vegetation
    c. Check and reduce excess Scrub/Rough outside of appropriate areas (saline herbaceous, bogs, coastal grasses)
    d. Reduce any remaining sand/gravel in industrial areas except for prominent areas
    e. Tidy up new roads and remove old construction roads
VII. Dissolve entire data set on gridcode
VIII. Repair geometries and topology verification.

The final data set includes 753,877 polygons and is approximately 350Mb. The polygons depict the land cover as identified in the RGB (optical) bands in the 0.10m GSD ortho imagery.

6 Data Definition

The final data is delivered in Esri File Geodatabase (FGDB) format with a single polygon feature class called “Kapiti Coast Land Cover”. The table and category schemas are as follows:

Attribute	Description
OBJECTID	polygon identifier
Shape	geometry type
Gridcode	class
Landcover	Land cover class

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Shape_Length	polygon length
Shape_Area	polygon area

Data is provided in the NZGD 2000 New Zealand Transverse Mercator coordinate system. Other key parameters include:

Parameter	Value
Linear Unit	Metres
XY Resolution	0.0001 M
XY Tolerance	0.001 M
Attribute Index	OBJECTID

Note the overall spatial resolution of the land cover data is 0.10m. The resolution and tolerance stated in the table are the defaults held in the file geodatabase.

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APPENDIX D Soil infiltration investigation

DRAFT

KĀPITI COAST SOIL INFILTRATION RATE STUDY

Summary Report

Project Purpose

The Kāpiti Coast Whaitua Committee will set out to develop long-term water quality objectives for the Kāpiti Coast District. One of the key risks to water quality in this District is the ongoing development, largely residential, of rural and peri-urban land. With recent changes introduced through the National Policy Statement for Urban Development, residential intensification also needs careful consideration. Kāpiti Coast is dominated by highly transmissive sandy soils and gravel loams. It has long been understood that modern development practices (e.g. compaction of the soil for concrete slab-on-grade construction) reduces the permeability of the soil. However, there has been no work done to quantify this impact which will materially influence runoff and hence water quality. As Kāpiti Coast has naturally high infiltration rates, changes are likely to have an impact on both surface and groundwater hydrology.

This study is focussed on the soil infiltration field measurements of the A-horizon or top layer of the soil around Kāpiti Coast District. This test aims to replace current overseas data that is used to understand runoff with site specific date. Ideally, this in-situ soil infiltration dataset will be able to represent the soil permeability more accurately in the stormwater modelling of the District. This new area-specific, district-wide representation will be available to the Whaitua Committee as they consider the impacts of proposed development on water quality.

Quantifying soil infiltration rates will be a key piece of data for:

- water quality modelling,
- understanding the benefits and appropriateness of low impact stormwater devices,
- interaction between rainfall and groundwater recharge,
- providing an insight into potential wetlands in the dune dip landscape,
- soil and vegetation relationship impacts on infiltration,
- understanding the influence of infiltration on flows rates,
- understanding the influence of infiltration on contaminate loads and concentration,
- quantifying the impact of ongoing land development on overall water quality and quantity, and
- impact of development practices on infiltration rates (i.e. traditional land compaction vs low earthworks solutions).

Kāpiti Coast District Council partially funded the Greater Wellington project. The results from the infiltration testing will inform the updated flood hazard maps via new infiltration rates in the new stormwater models. Current flood hazard maps are based on models which used infiltration rates from international standards. With new infiltration rates, the models and therefore new flood hazard maps, will better contextualise the hydraulic connectivity between the runoff and subsurface in the District.

Introduction

Aotearoa New Zealand is under pressure to build new homes. The shortage of available housing coupled with high prices have amplified the effects of a long running housing crisis across the country (Hunter, 2019). This has put housing development pressures in urban areas, surrounding suburbs and townships. This has resulted in an increased need for adequate infrastructure to support development growth.



Figure 1. Kapiti Coast district with Kapiti Island in the background. (Source: https://images.squarespace-cdn.com/)

Increasingly rapid housing development can be observed in the Kāpiti Coast District, with pressure mounting to create new housing for the last few decades. The development pressures from the housing shortage in the urban centres of Wellington City and Porirua have transferred north to the nearby low-lying, coastal district. Furthermore, the recent opening of Transmission Gully motorway, connecting the Kāpiti Coast district to Porirua and Wellington City with four lanes, will prove to increase the number of Wellington residences willing to commute, thus increasing the housing development pressures in the Kāpiti District.

A major risk factor for any city or township that is rapidly growing is how the stormwater network is developed in kind with that growth. If the stormwater system isn't developed with the entire system's capacity in mind, environmental and human health can be reduced by way of contaminants and floods.

Due to the topography of the District, the low-lying areas are common to flooding. Historically, the high groundwater levels have also influenced flooding in the region (Sinclair Knight Merz, 2006; Kāpiti Coast District Council, 2021). Further development will require engineering and practice reviews to create a process to enable sustainable growth and provide housing, while achieving the environmental responsibilities of a territorial authority (TA) in the District.

Robust scientific base-knowledge can provide the insight into meaningful and informed decision-making. Stormwater models illustrate potential areas of liabilities such as flooding and climate change effects (sea level rise) and highlight opportunities for mitigation such as appropriate stormwater infrastructure. All things considered, informed forward thinking and dynamic solutions for stormwater strategy is needed to accommodate the challenges of the development growth in the Kāpiti Coast District.

Background

Kāpiti Coast Geomorphology

The soil structure and topography of the District has been shaped by the interaction of aeolian and fluvial process resulting in several dune phases, with interdune areas of peat, and extensive floodplain gravels (McFadgen, 1997; SKM, 2006). The dune sands are derived mainly from alluvial materials and deposited on the Coast. Over time, prevailing northwesterlies aerially distribute sands inland creating dunes and foredunes (McFadgen, 1997; SKM, 2006). The oldest sands, the Foxton dunes, began accumulating about 6500 years BP (McFadgen, 1997). There are four phases of dune-building younger than the Foxton dunes named, in order of decreasing age, Taupo, Waitarere-Motuiti, Older Waitarere, and Younger Waitarere.

Both Waikanae and Ōtaki township are characterised by alluvial gravel loams that underlay the area. These areas are known to have high natural soil permeability and soakage. Though studies have shown that recent housing development have affected stormwater managementt by increasing flooding on alluvial soils through increased runoff volume and peak flows (SKM, 2006).

Development and Flooding History

Kāpiti Coast has traditionally had high growth demands on its low-lying coastal plains which are dominated by dune-dip geomorphology with high soakage sands separated by interdune swamp areas. Early housing development predominantly used piles-based foundations in this area.

During this time, aside from high volume high intensity rain events, groundwater ponding was a major issue in the Kāpiti Coast. Elevated groundwater during rainfall events causes ponding and flooding in the low-lying areas of the District, particularly on peat soils (SKM, 2006).

New housing developments have been linked to more flooding in the area. Building of new housing essentially increased the area of impervious ground and subsequentially decrease the area available to absorb water during rainfall events.

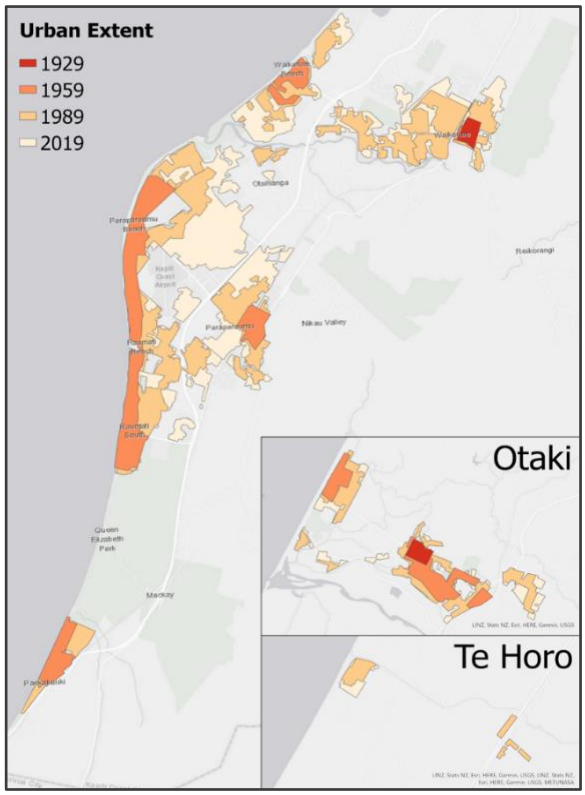


Figure 2. The timing of historic development on the Kāpiti Coast.

Furthermore, recent industrial compaction of new development sites has been linked in increased runoff and flooding (SKM, 2006). In the early 2000s, KCDC adopted the NZS:4404 Earthworks Standards as a requirement for developers applying for housing consents (KCDC, 2021). This requirement continues to cause new housing development to industrially compact housing lots to obtain consents. Housing developments built on slab-on-grade (i.e. cement platform) foundation has become common in housing projects, particularly in large areas, resulting in mass compaction of greenfield developments in the District.

Excessive runoff can cause flooding and can lead to infrastructure and environmental damage. Contaminants carried in runoff and overland flow also raises environmental concerns. (Boardman et al. 2003). Government agencies managing resource consents are generally aware of and concerned about how changes in soil infiltration capacity under changing land-use can affect flooding and water quality, particularly in Kāpiti District.

Stormwater challenges

Kāpiti Coast District Council (KCDC), following flooding in 1998, was one of the first councils to build detailed hydraulic models of their entire stormwater network. Modelling work was completed in 2002, (with a major update between 2007-2012). The models were used extensively for future planning and managing the Council's programme of upgrades and renewals.

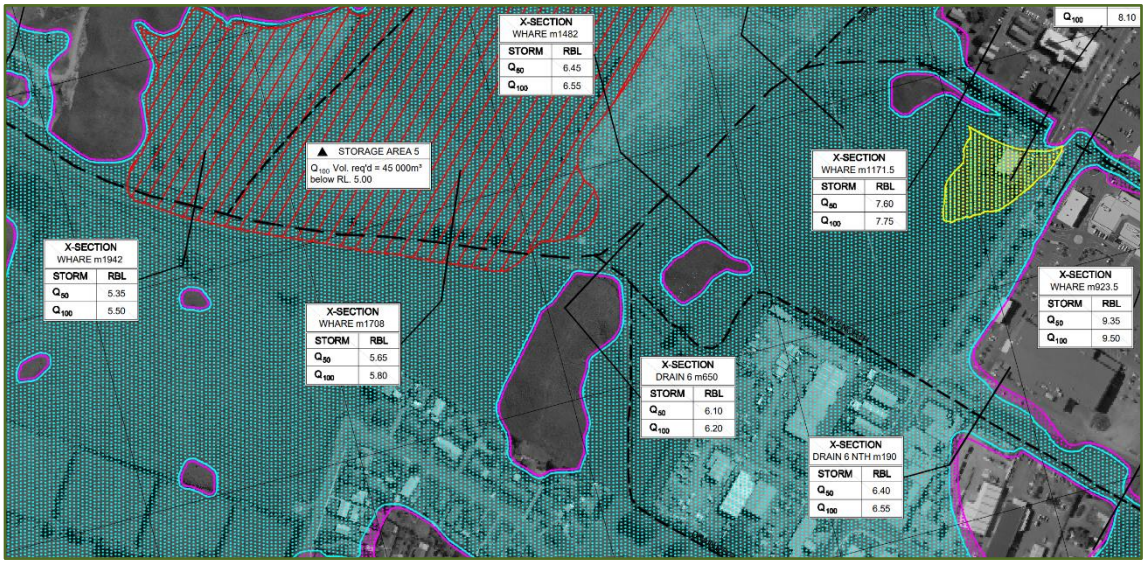


Figure 3. Initial flood hazard management plans for Kāpiti's urban zones. *(Source: KCDC)*

While the original hydraulic models have served the Council well, several factors of flooding are not accurately represented in the current models. These factors include the impact of groundwater, urban intensification, and climate change (particularly regarding coastal inundation, erosion, and retreat).

To better account for the effects of climate change and allow for more accurate assessment of development impacts, some updates have been made KCDC's flood modelling methodology. For example, incorporating the results of this study as an in-situ infiltration data set from Kapiti to better quantify the impact of infiltration rates on the interactions between runoff, high permeability dune and alluvial soils, and groundwater.

In other words, the results from this study (i.e. the infiltration tests) will be combined with other data to form an updated localized infiltration data layer. This data layer will be an input to the updated models which will in turn produce more accurate apportioning of water between surface water runoff and groundwater losses.

Infiltration Rate

Infiltration rate is the rate at which water passes through the top layer of soil. This top layer is often referred to as the A-horizon. The top layer can either be homogenous within its sub-layer or crusted, which is the more common composition. This crust is formed by either vegetation or compaction. The presence or absence of this layer, together with its properties, controls the amount of water that can percolate to the sub-layers and the amount of runoff on the surface.

An infiltration rate is used in stormwater modelling to determine the amount of surface water that contributes to overall runoff. Essentially, if rainfall intensity exceeds the infiltration rate, water will accumulate on the surface and flow overland as runoff. The infiltration rate of different soil types and land-use are therefore central to simulating overland water flowing and the accuracy of stormwater and flood modelling.

Infiltration rates can be affected by plant matter or compaction on the top layer. For example, if vegetation with a root system occupies the top layer, the roots act as preferential percolation flow paths which increases the infiltration rate. Conversely, high levels of organic matter and compaction reduce the porosity of surface soil, reducing its infiltration rate.



Figure 4. Persistent "groundwater" flooding on the Paraparaumu golf course after a wet December in 2021.

Other soil water movement

At this juncture it helps to make some important distinctions in relation to different forms of water movement in soil.

Infiltration specifically refers to the process of water penetrating the surface soil and is typically measured in units of mm per hour. Soakage, on the other hand, refers to water percolating into the underlying soil layers within the unsaturated zone where soil is not sitting in water, or 'soaked'. The rate of soakage, also termed permeability, refers to the speed at which water moves through and into the subsurface layers and has the same units as infiltration rate (mm/hour). Coarse soils with larger grains tend to have higher soakage rates as there are larger voids for the water to pass through and fill.

Typically, soakage rates are higher than infiltration rates on account of compaction of surface soils and they are unaffected by plant matter and compaction. In Kapiti, the high soakage rates allow stormwater to dissipate quickly into the subsurface which is a common natural stormwater solution. For example, soakage pits in Ōtaki township connect the surface runoff directly to the subsurface via engineered pits that bypass the top layer of the soil that would otherwise slow the movement of water into the ground. However under high, prolonged rainfall and rising groundwater, sub-soils can eventually become saturated, stopping further infiltration and soakage and thereby inducing surface flooding in low-lying areas (e.g. groundwater ponding in Figure 4).

Below the unsaturated zone is the groundwater table or aquifer. The rate at which groundwater moves through the saturated soil is typically characterized by transmissivity (m^2/d). The various forms of soil water movement described above are depicted in Figure 5 below.

With high soakage inputs, the water table can rise, which amounts to unsaturated soil layers becoming saturated. The ability of an underlying aquifer to convey soakage water away from a particular area will dictate the extent to which overlying soils become saturated and lead to flooding. In Kapiti, groundwater is both a significant stormwater contributor and sink to flooding, and is often incorporated into water sensitive stormwater design through soakage pits.

In the context of rapid urban development and associated transport and stormwater infrastructure, understanding the infiltration rates is vital to assessing stormwater flow requirements in a model. Since infiltration is the limiting factor in the transfer of water into the ground, there is not the same need to collect local data on soakage rates.

This aim of this study was therefore to measure infiltration rates around Kāpiti to provide regionally-specific data for stormwater modelling, the outputs from which provide better understanding of stormwater and flooding issues, and mitigation strategies and solutions.

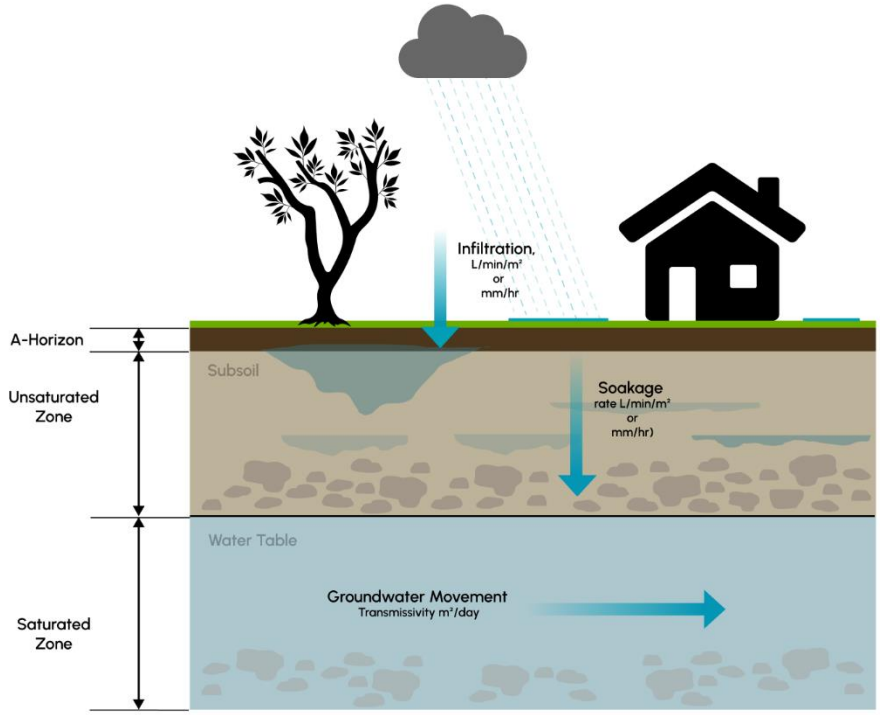


Figure 5 Macro scale soil water movement

Methodology

Literature Review

Two methods of measuring infiltration in the field were considered for this study: single-ring and double-ring infiltrometers. The single-ring infiltrometer method comprises a short length of 150-mm diameter tube that is pushed into the ground to a depth that penetrates beyond the crusted surface. Water is poured into the ring and the flow rate is tracked over time. The double-ring method simply adds an outer concentric ring to ensure that water flow moves in a downward direction (not laterally).

Walsh and McDonnell (2012) compared single ring and double ring infiltrometers and found no statistical difference. On the other hand, Chowdary et al (2006) reported infiltration rates between single and double ring infiltrometers were similar up to the 50-minute mark, with divergence between the measurements observed beyond this duration. This suggest that lateral flow in the single ring infiltrometer is taking place. Walsh and McDonnell (2012) showed that the single ring infiltrometers can have water and labour savings compared to double ring infiltrometers, but recommended double-ring method as the default method of infiltration testing, noting the time savings and accuracy it has over single ring.

A recent study by Lewis et. al. (2021) investigated the difference between a Cornell sprinkle infiltrometer (CSI) (uses single-ring) and double-ring infiltrometer for measuring steady-state infiltration rate. Though there are differences in results, the study states that the results strongly correlate and recommends either method depending on the availability of equipment.

Recent New Zealand infiltration literature with field measurements is scarce. Environment Waikato (2008) compared in infiltration rates between grazing land and forestry using double-ring infiltration method to avoid lateral flow.

With this in mind, this study employed the double-ring infiltrometer method because of its availability at the time and the opportunity to get a custom-made ring to suit. The double-ring also provided reassurance of mitigating lateral flow in comparison to a single-ring method.

Constant Head vs Falling Head

Gregory et. al. (2005) compared falling head and constant head infiltration tests and concluded that a double-ring method using constant head is most appropriate for sandy soils in North Central Florida.

Additionally, Environment Waikato (2008) used constant head methodology for infiltration testing in Upper Waikato

The study proceeded with a constant head infiltration test to avoid the head pressure effect and minimise physical efforts during the testing as 3 simultaneous tests were planned per site.

Test Duration

Studies (Hemmat et. al., 2007; Lai and Ren, 2007) used 90-minute durations and deemed it adequate to represent stable infiltration rate. Bagarello and Sgroi (2004) performed their test with a 3-hour duration and reported it no significant change after 120 minutes. Walsh and McDonnell (2012) expressed the same observation: that there was no significant change to the infiltration rate after 120 minutes.

The test duration chosen for this study was 90 minutes after initial tests were conducted and showed no significant change after 90 minutes. Furthermore, the 90-minute duration allowed for two sites sampled per day while providing time for unexpected changes or delays during testing period.

Sampling Approach

Soil classification

Kāpiti's geomorphology consists of sandy dune phases, low-lying peat, and alluvial gravel fan. The approach is to get a representative amount of infiltration tests on a range of different soil classifications derived from McFadgen (1997). The study consists of *six* tests for each of the nine soil classifications and each of the five vegetation/compaction classes.

Layers identified by McFadgen (1997) were digitally mapped and extrapolated on ARCGIS. Additionally, layers from Landcare Research were used to identify the soil typologies on the northern side of the District, between Te Horo and Ōtaki. Nine different soil classifications were chosen in the District based on soil typologies along with area-based classification. The dune phases include Foxton, Taupo, Waitarere-Motuiti, Older Waitarere, and Younger Waitarere.



Figure 6. Common topography, Kāpiti Coast. Strips of sand dunes and peat lands with interdunal wetlands. (Source: GNS Lloyd Homer)

Other soil typologies include peat, Waikanae Garden Area, Waikanae Escarpment, and Ōtaki Gravel. The Waikanae Garden Area and Escarpment are differentiated by location. Waikanae Escarpment pertains to the area on the east side of Waikanae town characterised by raised elevation from the eastern hills. The Waikanae Garden Area refers to the area that is underlaid by a gravel fan bed and mainly situated on the flood plain on the eastern side of Waikanae town. The Waikanae and Ōtaki classification based on location is partly due to lack of data pertaining to classification of soil types in the areas (Waikanae) and due to complicated high-resolution studies (Ōtaki).

While some soil typologies may share similar soil properties, such as the three dominant dune phases, separating the typologies initially will give confidence should there be no variance between dune phases. If the dune phases or gravel bed typologies show correlation initially, then combining some soil typologies may occur.

In summary the following soil classifications were sampled:

- Ōtaki
- Waitarere-Motuiti

- Foxton
- Peat
- Waikanae Garden Area
- Waikanae Escarpment
- Taupo
- Older Waitarere
- Younger Waitarere

The distribution of tests carried out by soil types is plotted in Figure **7**.

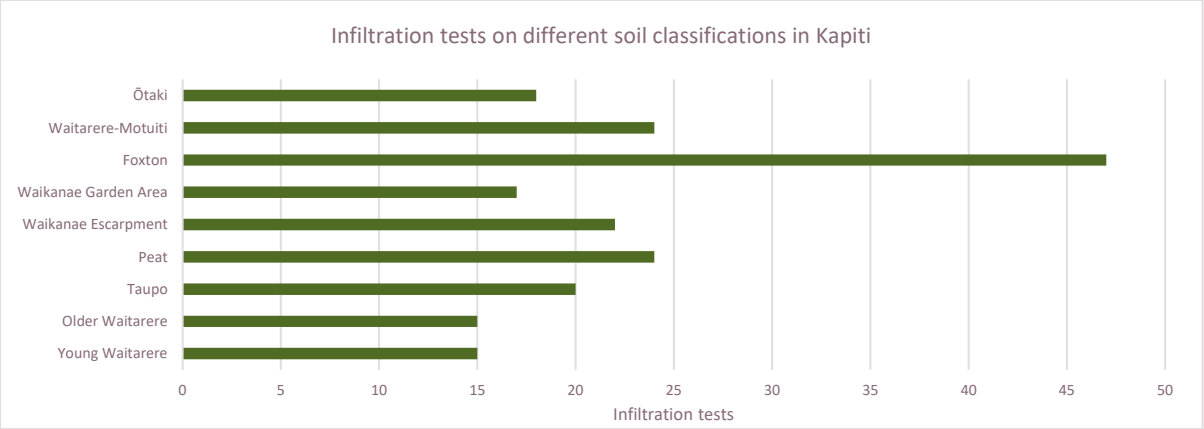


Figure 7 Infiltration tests carried out on different soils in Kāpiti Coast

Vegetation and compaction classification

Awa set out to sample the following soil land-use typologies:

TYPOLOGY	DESCRIPTION
Rough Low Vegetation	scrub, young trees on unaltered soils, typically on coastal dunes.
Coastal Forests	mature coastal forests on unaltered, uncompacted soils.
Reserves	a mix of compacted and uncompacted Reserves e.g., parks, recreational Reserves.
Uncompacted Residential Grass	grass patches around uncompacted residential properties identified with external steps or raised houses which are consistent with piles-based foundations and do not require industrial compaction. This is typical of older houses in the District.
Compacted Residential Grass	grass patches around properties identified with zero elevation difference to the surrounding land, indicative of slab-on-grade foundations and require industrial compaction. This is common in properties built from the 1980s.

These classifications aimed to reflect the different land-use classes common in the Kāpiti Coast District, particularly the residential areas which has the highest vulnerability in terms of asset value. The distribution of infiltration tests conducted by vegetation/compaction classification is given in Figure 8.

Intensification of residential development is allowed under the District Plan; Awa wanted to understand the impact of modern development techniques on surface infiltration. It is assumed that all rural areas are uncompacted in the use of the permeability results.

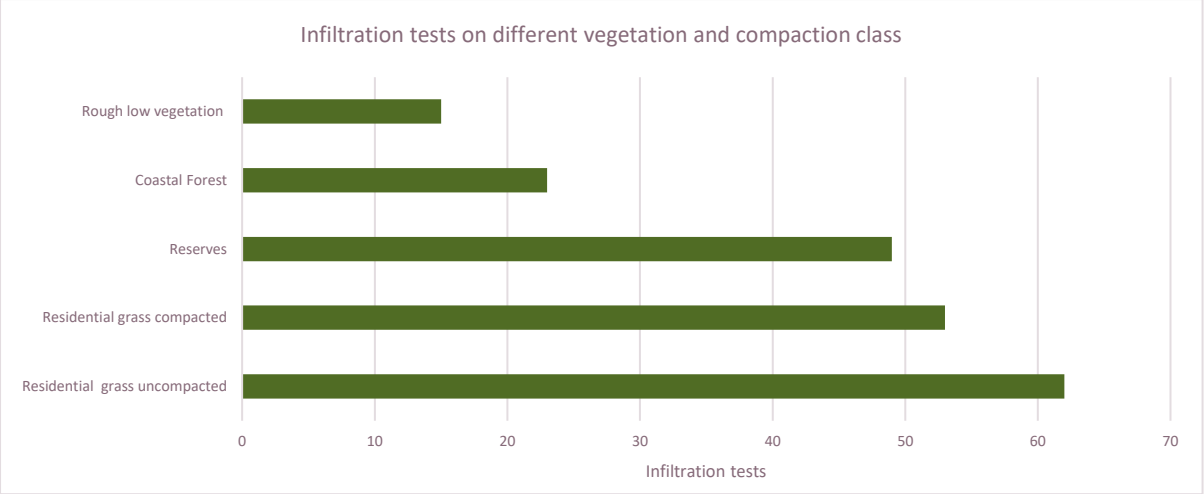


Figure 8. Infiltration tests on different vegetation cover and compaction type in Kāpiti Coast.

Testing Procedure

Double Ring Infiltrometer (DRI)

Based on the literature and current practice, double-ring infiltrometer (DRI) was decided as the method of choice. Though studies (Reynolds et. al., 2002) suggest that the presence of an outer ring does not statistically affect potential lateral flow, the Kāpiti Coast is known for its high soakage sands. Regardless of the additional labour and resources that a second ring brings to the experiment, the outer ring offers a reliable barrier to avoid any risk of three-dimensional flow that could potentially be caused by high soakage sands.



Figure 9. Double-ring soil infiltration test. Brass ball cock mechanism regulating the water flow into the inner ring. Outer ring is directly fed by another container. (Source: Awa Environmental Ltd.)

This study used the guidelines advice by Auckland Council (2017) regarding double ring infiltrometers. An inner ring to test for infiltration and an outer ring to maintain vertical flow of water beneath the rings.

Three simultaneous tests were run per site using custom built rings. The inner rings had a diameter of 150mm diameter and outer rings had a diameter of 300mm.

At each site, the rings were placed no more than 10 metres apart from each other. During ring placement, no disturbance of topsoil or vegetation root systems was allowed. Each set of rings were put on the ground at around 20-40mm depth or until no visible leakage was observed during soaking for soil saturation. Water was then poured into the inner and outer rings for approximately 10 - 15 minutes prior to the infiltration test.

A team of two scientists conducted three DRI tests at each site, and a total of 191 tests across the Kāpiti Coast. Set up requirements and tank capacity (1,000 L) limited the number of tests that could be conducted per day



Figure 10. *Three simultaneous infiltration tests occurring. Transport vehicle in photo carrying 1000L water container. (Source: Awa Environmental Ltd.)*

Constant-head method

New Zealand guidelines (Auckland Council, 2017) and other studies (Walsh and McDonnell, 2012) are consistent in recommending constant-head method when test sands for infiltration rate. This method is measuring the infiltration rate within a cylinder while maintaining a constant level, avoiding the effect of head pressure changes. Therefore, a constant-head method is appropriate for the sand and gravel dominated soils of Kāpiti Coast District. Furthermore, constant head reduces areas of uncertainty, particularly since head pressure influence on flow rate is isolated. A constant head with an automated mechanism also allowed for easier maintenance of the equipment and water storage during test, that it enabled three tests to be carried out simultaneously.

The constant head was maintained by a brass ball cock mechanism with a Styrofoam float automating water flow into the inner ring and keep the inner ring at a constant height. The height (between 25mm and 150mm as per guidelines) (Auckland Council, 2017) were maintained between 50-70mm in both inner and outer rings. The orifice of the mechanism opens, letting water into the inner ring, when the float drops. The orifice then closes when there is enough water in the inner ring pushing the float up. At equilibrium, the orifice opens at the optimum amount to match the infiltration occurring on the topsoil.

The rate of water flow from a square water container to the inner ring through the mechanism described above represents the water loss and therefore infiltration rate.

Test Duration

Depending on the soil type, literature suggests up to 4 hours of test duration. Initial testing was run for extended timeframes but proved that shorter duration tests were appropriate for soil types in Kāpiti as results typically normalised after as little as 1 hour.

Calculations

The final infiltration rates were calculated from the mean in the last 30 minutes of the test. During this time, the soil is at its highest saturation, and therefore best represents the stable infiltration rate in wetted soils.

The sites were selected to examine how urban soil compaction might have affected infiltration rates, as has been observed in other studies (Alaoui et. al., 2017; Parnas et. al. 2021). As such, residential areas were a focus for this study. Though Kāpiti Coast is underlaid by high soakage sands, the effects of crusting through land-use development will be captured by this test.

Due to accessibility, identification, and permission issues the following two soil types were not tested as frequently: Rough Low Vegetation, and Coastal Forests (Figure 9). Rough low vegetation is typically situated on top of unaltered dunes near the beach. This presented accessibility issues for the vehicle transporting a 1000kg water container on the back. Additionally, there are a few Coastal Forest sites in the region and the time available to the study did not allow the process needed to gain permission to enter these areas.

Results

Infiltration rates – Soil Type

The results contained in the figure below displays all the data collected from the 191 sites tested and grouped by soil types. The graph has each set of data graphed by soil type and segmented into compacted (left) versus uncompacted (right) sites.

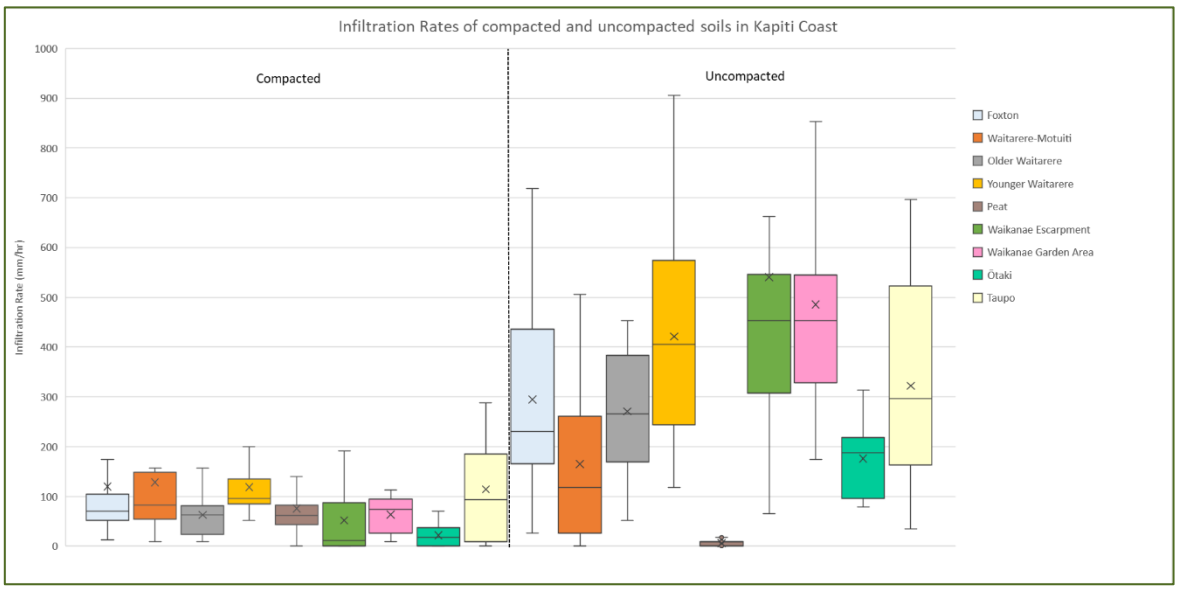


Figure 11. Infiltration rates of compacted and uncompacted soils in Kāpiti Coast district. Left side shows the compacted soils while the right side shows the uncompacted soils. The [X] in the graph refers to the mean while the box show the 25th-75th percentile range, and the line in the box refers to the median.

The following are the conclusions drawn from this data:

1. The results show no clear relationship between the vegetation types within each soil classification.
2. There appears to be a correlation between compacted and uncompacted Residential Grass.
3. There is a negative relationship between compaction and infiltration rates.
    a. Compacted Residential Grass across all soil types ranged around 1-300mm/hr.
    b. Uncompacted Residential Grass, apart from peat, ranged around 1-900mm/hr across all soil types.
    c. Uncompacted Residential Grass had a much larger variation in data.
4. The limited samples of Rough Low Vegetation and Coastal Forest, (unaltered and uncompacted sites) show high infiltration rates from both classes ranging from 200-800mm/hr.
5. The highest distribution and largest variation are observed from Reserves at 21-800mm/hr, which are a mix of compacted and uncompacted areas.
6. Peat infiltration rates from the previous models correlate with newer rates from the current infiltration results.
7. Other soil classes showed much higher rate than current models.

Infiltration rates – Dune Phase

The following figure plots the data by dune phase and segments the compacted sites (left) from the uncompacted sites (right).

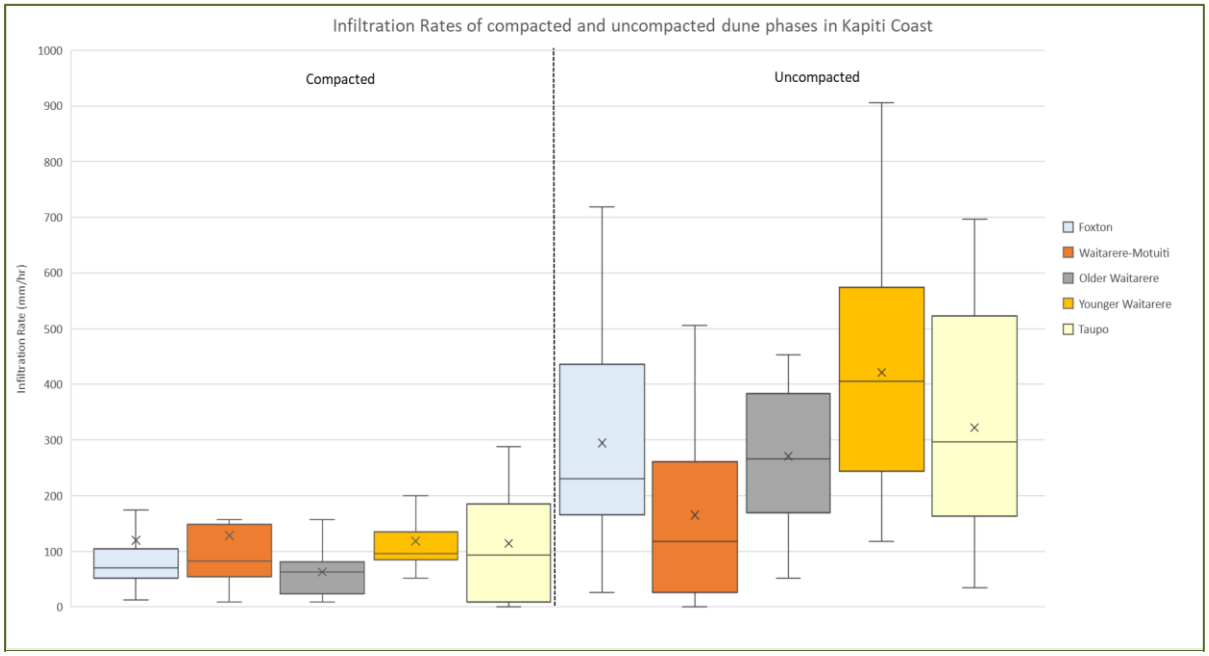


Figure 12. Infiltration rates of compacted and uncompacted dune phases in Kāpiti Coast district.

The following conclusions are drawn from this data:

1. There were no significant differences between the range of infiltration rates between the dune phases and the results show relatively close and overlapping ranges (Figure 9).
2. Notably, the compacted Taupo and uncompacted Waitarere-Motuiti infiltration rates have inconsistent results which are explained further in the Discussion section below.

Infiltration rates – Foxton Dune Phase Soils

The following graph examines the infiltration rates of Foxton Soils alone.

Foxton soil is the most widespread dune phase and soil type in the District. Figure 12 shows the ranges of the different compaction types. The main conclusion drawn from this data is that:

1. Reserves show lower rates than Uncompacted Residential Grass highlighting the mixed compaction of the reserve sites (i.e. compacted parks and Reserves versus untouched and uncompacted parks and Reserves).

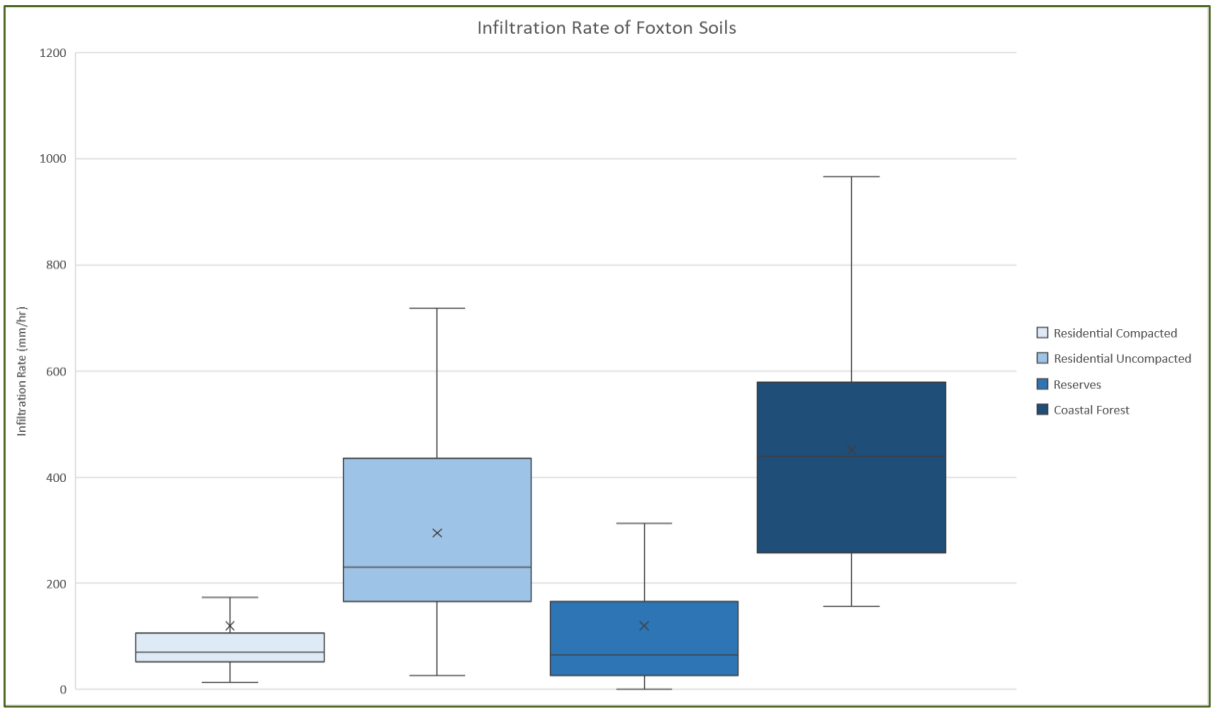


Figure 13. Infiltration rate in Foxton soils.

Compaction Investigations

Findings revealing compaction as the main determinant of soil infiltration highlights the significance of accurate compaction classification. Two examples are outlined below showing investigations that were observed as outliers initially and were recategorised after investigation.

A residential house on Gray St in Paraparaumu which was on Foxton sands was initially classified in this study as Compacted Residential Grass. Contrary to this classification, the results showed high infiltration rates for around 250-300 mm/hr, around 2.5-3 times more than the average Compacted Residential Grass rate.

Upon investigating Digital Elevation Model (DEM) of the location, the model showed a generally flat elevation of the area, suggesting wide-spread compaction when the houses were built, and hence it was categorised as a Compacted Residential Grass.

This led to further investigation as to whether the foundation of the house had been compacted or not. Google images and test photographs showed that the house was raised, common to houses that were built with piles-based foundations which does not need area-wide compaction, like would be required of concrete slab-on-grade foundations that are common to modern developments. Historic aerial photographs from 1966 as shown in Figure 8, reveal gradual development (as opposed to white patches on the map suggesting area wide compaction) which are consistent with piles-based foundations. This finding resulted in recategorising the Gray Avenue house from Compacted to Uncompacted Residential Grass.



Figure 14. Aerial photograph of Paraparaumu from 1976 showing gradual development on Gray Avenue (circled). Source: Retrolens NZ

Another example is the forest patch on the north of Haruatai Park in Ōtaki. This site was categorised as a Coastal Forest, but infiltration results showed that notable low infiltration rates. Upon investigation, it was revealed that the site was previously part of the park, which was compacted, and was planted in the early 2000s by children from Ōtaki as part of a community project. As a result of this work, the site was later identified as the Millennium Forest and this data point was categorised from Coastal Forest to Reserves.



Figure 15 Re-vegetated forest area adjacent to Haruatai Park with historically compacted soil.

Flood Hazard Maps

The two maps included here (Figures 14 and 15) are the Awa simplified versions of the maps that accompany the infiltration rates results. The results will be used to produce an infiltration layer that will be used as an input into the new flood hazard models.

The map of soil typology is shown in Figure 14 (right). Awa recognises similar infiltration rates between different dune phases (i.e. Foxton and Younger Waitarere) and river deposits (i.e. Ōtaki and Waikanae Garden Area), and combined them as sand and river loams, respectively.

Loess soil typology has been classified as the main soil typology for the high permeability soils on the Tararua foothills. Peat soil typology was classified to represent the low-lying historic

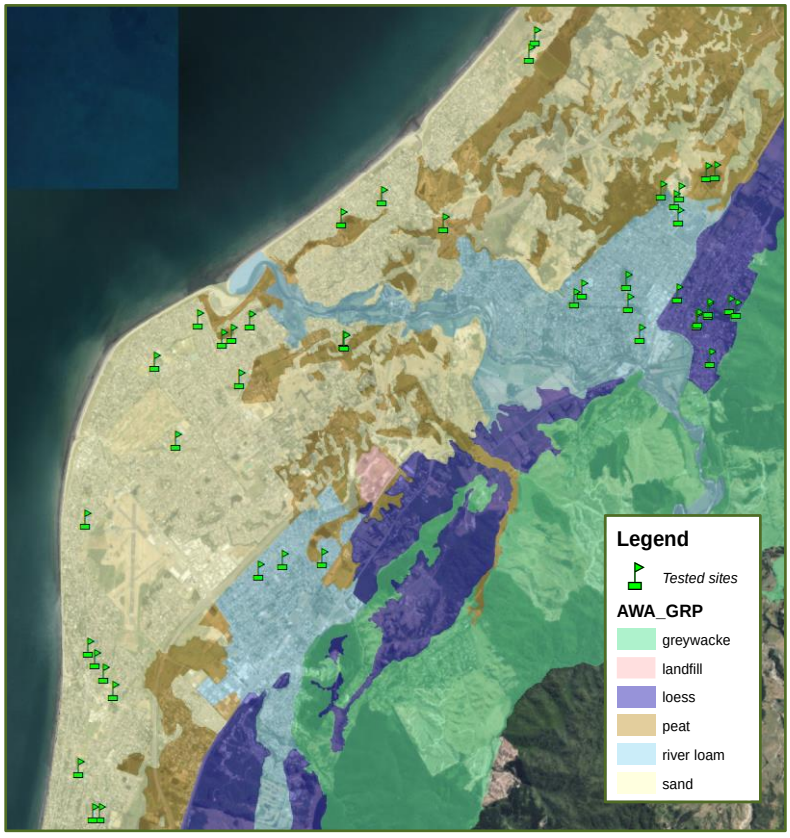


Figure 16. Simplified soil classifications on Waikanae, Waikanae Beach, Paraparaumu and Raumati Beach townships.

and current wetlands and peatlands in the District. Lastly, greywacke soil typology is the main typology for the Tararuas ranges.

Upon recognising the influence of compaction, Awa created a layer based on foundation methods during construction. Uncompacted sites were identified as properties with piles-based foundation, while properties that had slab-on-grade foundation were identified as compacted. Awa used the late 1970s popularisation of slab-on-grade compaction to create a temporal cut-off for extrapolating district-wide compaction layers. Figure 15 (left) shows the compacted (pre-1977 development) and uncompacted (development from 1978 to present) areas in the District. Notably, roads, commercial areas, the airport, and railway has been identified as compacted areas.

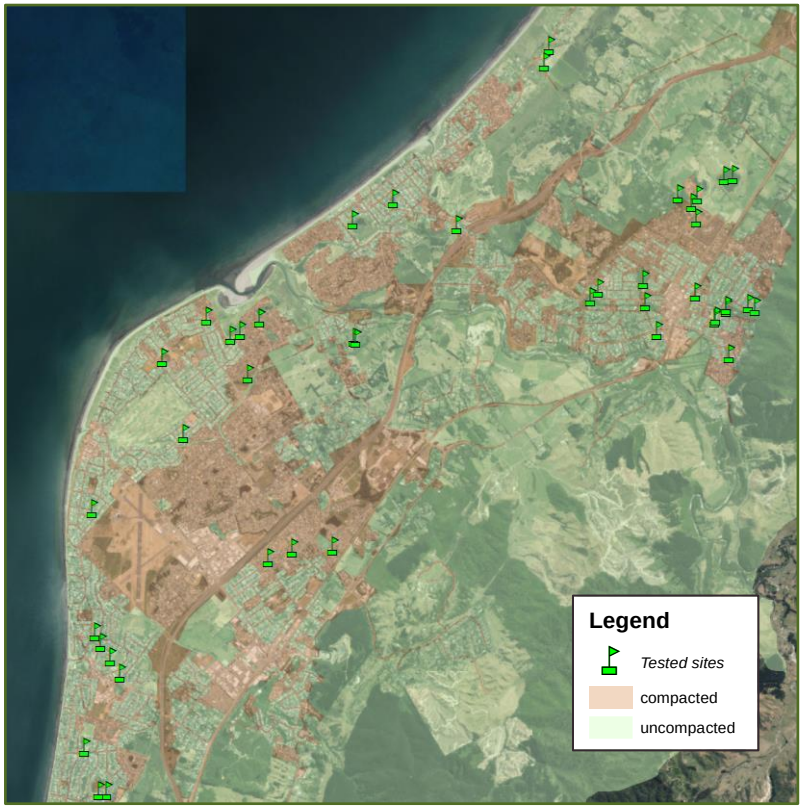


Figure 17. Compacted and uncompacted areas based on a 1977 cut-off. Sites sampled are also shown on the map.

These two maps will be synthesised along with the infiltration rates from testing locations to create an infiltration layer. This layer will feed into the updated flood hazard maps and hydraulic models in the future.

Discussion

The infiltration data shows a distinct relationship between compaction and infiltration rates. Rough Low Vegetation and Coastal Forests showed expected high infiltration rates, while Reserves showed a large range due to its mixture of compacted (i.e. parks and sports grounds) and uncompacted (i.e. undisturbed dune) sites. A comparison of infiltration rates between different dune phases is shown on Figure 11. This shows that dune typology is not a determinant factor for infiltration rate.

Unpublished reports from Auckland Council in 2007, studying compaction effects on runoff, are consistent with the results. Topsoil is removed and subsoils deliberately highly compacted to create stable surfaces to support concrete slab-on grade house and road foundations.

Though the dunes are relatively consistent, two sites have been identified that may be skewing the data. A Taupo Compacted Residential Grass near Paraparaumu was identified as a compacted site due to the slab on grade foundation of the house. However, the test showed high infiltration rate (120-290mm/hr). Upon investigation, it was discovered that the residents were aware of the hydrophobicity of their grass and would periodically turn the sprinklers on to keep the soil moisture at a level that would be able to absorb runoff more efficiently.

Additionally, a site classified as a Waitarere-Motuiti Residential Grass in Ōtaki Beach was identified as a rolling hill near residential area with mature trees nearby. However, the infiltration at the site was significantly low (1 - 60mm/hr). It was later identified that the time of testing was a few days after the December 2021 rainfall event and may have affected the test with higher groundwater levels. Furthermore, groundwater ponding was also identified near the site indicating raised water table. This highlights the significance of current groundwater conditions to infiltration rate.

Implications on water quality

The infiltration tests will help produce an infiltration layer. This layer will significantly adjust how the model calculates the amount of runoff and contaminants flowing on the surface and permeating through the soils.

Obtaining information on the quantity of water and the interaction of surface and subsurface layers can be vital in identifying water quality, not only in the stormwater network (i.e. drains, pipes, waterways) but also how it affects the groundwater recharge and quality in Kāpiti. This is particularly important in Kāpiti since the subsurface have high permeability and the interactions between surface and subsurface are not clearly understood.

A more accurate in-situ representation of infiltration rates will help to better understand the water quality issues regarding stormwater ponds. Stormwater ponds act as a buffer during flooding events. However, ponds usually accumulate contaminants as storm events pass by. These are then flushed out when bigger rainfall events occur and released into the stormwater network causing a spike in contaminant concentration.

Implications on stormwater model

The infiltration results identified as a result of this study show larger rates compared to the infiltration rates that are currently being used in the existing model.

Currently the models use different soil infiltration rates based on land use categories and soil types A - D from USDA sources from overseas. The current soil classifications are significantly lower than what the soil infiltration tests show.

It is important to note that the new infiltration rates, while much higher than expected, are still subject to sensitivity testing which will likely result in a lower representative rate. While lower rates will still represent the new soil classifications, the new infiltration rates will still be higher than the current data and will provide a more accurate representation of the District.

In general, the results of this study imply that an update to the infiltration layer, having higher infiltration rates will produce larger ‘losses’ of surface water to the ground, meaning less runoff in some places

(TUFLOW, 2017). This increased infiltration into groundwater will have to be validated against groundwater levels in the model, especially as groundwater behaves in a dynamic nature, influencing water flow three dimensionally (i.e. laterally or back to the surface) (KCDC, 2008; TUFLOW, 2017).

Implications on Policy

Study results show that infiltration rates are higher than anticipated, particularly in uncompacted residential areas. The concern with existing policy is that required compaction of new developments will significantly change this and could result in increased runoff volumes (i.e. flooding), requiring engineered stormwater solutions.

NZS:4404 Earthworks Standards (Standards New Zealand, 2010) by KCDC under its District Plan requires new housing developments to be compacted to meet foundation requirements (KCDC, 2021). Although enhanced engineering solutions can be used to compensate for the potential effects of this policy, the result will be ongoing reduction of infiltration rates and increased runoff potentially resulting in an increase of flood frequency and intensity.

This compaction requirement also applies to brownfield developments meaning that intensification of current residential areas which are currently uncompacted will be subjected to compaction if subdivided. This could become more pronounced given the allowable nature of intensification under new District Plan rules (KCDC, 2021).

While partly driven by construction methodology (concrete slab-on-grade foundations) and is embedded as an engineering requirement,[1] the requirement will have an impact in Kāpiti on the amount of runoff and groundwater recharge. These effects have not been traditionally connected to these requirements and in order to address the unintended consequences of the requirements there may need to be considered changes to multiple guidance documents to reverse or allow for alternative foundation methodologies.

Implications on climate change adaptation and resilience

Sea level rise will increase the groundwater levels in the District over time (KCDC, 2008; SKM, 2006). The low-lying coastal area is already vulnerable to groundwater ponding and will be more at risk should climate change sea level rise influence a further increase groundwater levels (KCDC, 2008).

NIWA (2017) projects that the District is likely to see an increase of 5% annual rainfall by 2090. A more significant factor is that rainfall will become more intense. If infiltration rates are to continue to be effected by development, then the newly lowered infiltration rates in soils are likely to have increased runoff and induce flooding during a high-intensity rainfall events. Though this can be mitigated by building stormwater infrastructure with larger capacity, there will be limits to what engineering can provide for and this further highlights the importance forward planning with regard to decision-making about compaction practices at present.

Conclusions

The Kāpiti Coast has been under pressure from housing development. This will only increase in the future as the District improves accessibility and mobility with other major urban areas in the region. To plan for environmental concerns while providing housing, the District Council has decided to update their stormwater models. The stormwater models will help locate areas in the District that are less susceptible to flooding and measure the effects of proposed housing developments on the current stormwater network.

The District Council aims to address climate change and groundwater effects in its models by creating an in-situ soil infiltration data set from this research. Results have shown that with the exception of peat soil typologies, there is a major distinction between compacted and uncompacted soils in terms of soil infiltration. The evidence suggests that compaction is the primary driver of reduced soil infiltration rates.

[1] NZS:4404 under section 2.2.4 of Standards NZ, 2010

The update of the infiltration rates layer will be observed in the updated modelling, resulting in changes to the volume of runoff and modelled flooding.

Investigations have revealed that there are policy regulations that require formal compaction on new developments and intensification developments. Though an increase in rainfall and sea level rise can theoretically be mitigated by further enhancement of stormwater infrastructure, it can be costly. Therefore, a review of Kāpiti Coast regulations is justified (i.e. regulations associated with developments requiring soil compaction for foundations). Climate change is an important factor that territorial authorities must consider when making decisions on whether to maintain or amend existing policy. It is not clear if engineering solutions will suffice long-term.

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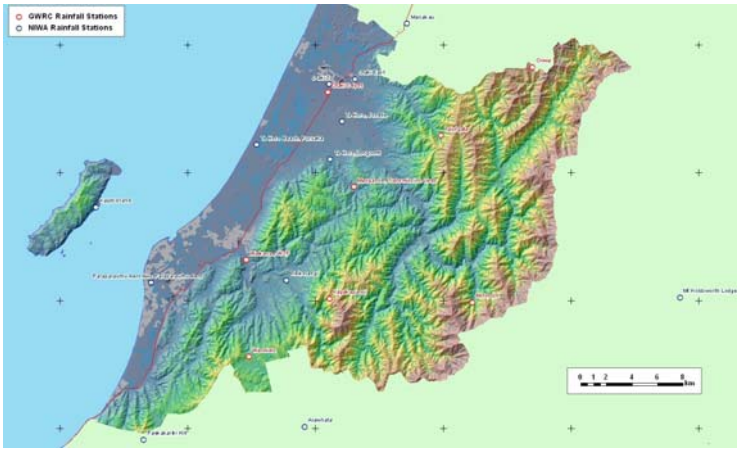
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APPENDIX E Updated Isohyet Based Calculations of Design Peak-flows, SKM (2011)

DRAFT

Appendix 1 - Updated Isohyet Based Calculation of Design Peakflows



UPDATED ISOHYET BASED CALCULATION OF DESIGN PEAKFLOWS

Final

1) 11 October 2011

Updated Isohyet Based Calculation of Design Peakflows

UPDATED ISOHYET BASED CALCULATION OF DESIGN PEAKFLOWS

Final

11 October 2011

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Introduction

As part of Kāpiti Coast District Council's subdivisional and development requirements, Sinclair Knight Merz have been engaged by the Council to prepare a standard for the calculation of stormwater design flows on the Kāpiti Coast.

This report is an update of the 2003 Sinclair Knight Merz report titled "*Isohyet Based Calculation of Design Peakflows"*. While the methodology remains unchanged this update includes the revised isohyets plans that were developed in the report "*Update of Kāpiti Coast Hydrometric Analysis*, 2008, SKM".

These plans incorporate the predicted impacts of climate change into the Kāpiti Coast hydrometric analysis. The predicted 2090 mid-range temperature scenario was used to estimate the climate change rainfall growth factor to 2090. This growth factor was used to produce a second set of isohyet maps. In 2008 the predicted effects of climate change on the Kāpiti Coast District are an increase in rainfall by between 8 and 16% over the 2 to 100 year ARI events.

Table 1-1. Growth factor for Paraparaumu Airport rainfall based on NIWA's 2090 mid-range temperature climate change scenario.

Average Recurrence Interval (years)	2	*5*	10	20	50	100
2090 Climate Change Growth Factor	1.08	*1.10*	1.12	1.14	1.16	1.16

Note: The 5 year result was not included in the NIWA report, so has been interpolated from the data.

In this report the unit hydrograph based approach is modelled on the US Soil Conservation Service software HEC-1. This approach has been internationally recognised and is widely used throughout the world. Haestad's HEC-1 and HEC-HMS are unsupported 'freeware' versions available through the US Army Corps of Engineers website. Support is available to subscribers through the Bentley software provider.

These guidelines provide examples on how to produce design hydrographs using both of the aforementioned versions of the software.

Unit hydrograph modelling is widely used for the development of floodplain management plans, and has been used for this purpose in Kāpiti since the early 1990's. This method of developing runoff design flows is also being widely used throughout New Zealand and is the basis of the Auckland Regional Councils rainfall-runoff guidelines 'TP-108'.

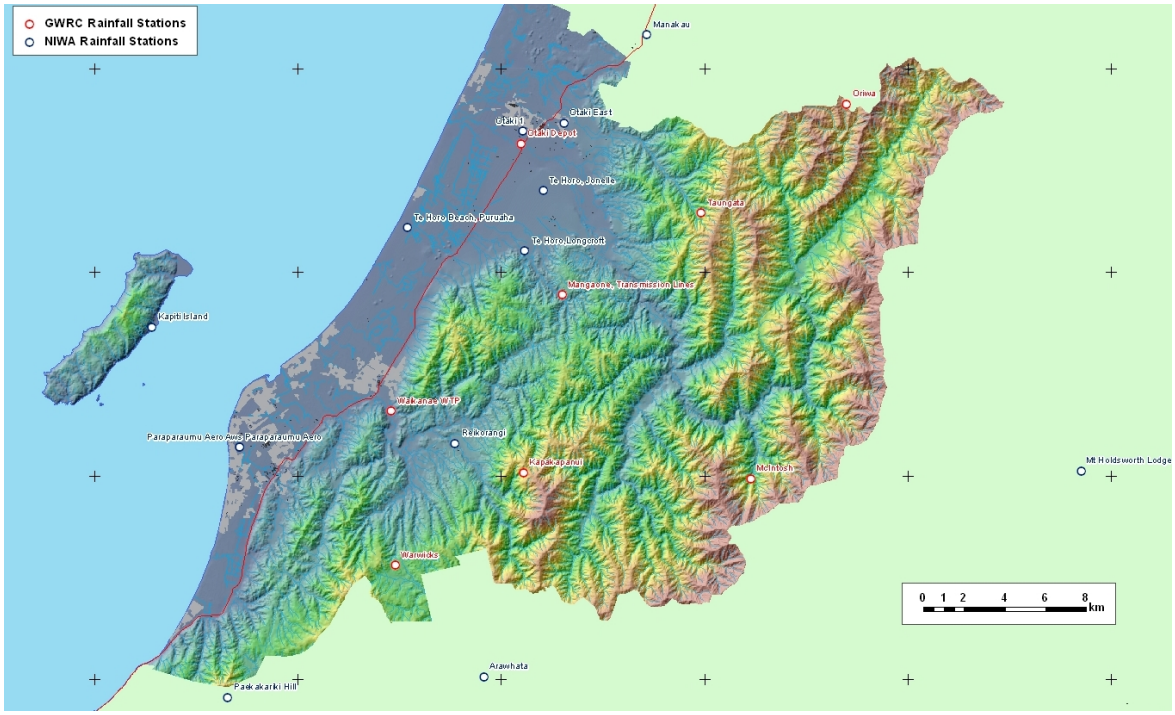
Background

24 Hour Rainfall Isohyet Plans

The intention of this report is to provide a system of assessing design storm runoff for peak flows and storage volumes to allow for consistent design of low impact structures. Design storm isohyets have been developed for the 2, 5, 10, 20, 50 and 100 year annual recurrence interval events as well as for the 2, 5, 10, 20, 50 and 100 year annual recurrence interval events assuming the 2090 mid scenario climate change predictions.

Annual maxima daily rainfall totals were extracted for 22 stations in the Kāpiti Coast region. This represents a combined total of 549 years of data. The stations that were used in the analysis are shown in Figure 2-1 below. The South Waiotauru site was excluded from the 2008 update due to consistently lower rainfall depths than other surrounding sites. Further investigation would have to be undertaken on this site before it was to be included within the study.

Figure 0-1 : Annual Maxima Rainfall Station



The methodology employed in this study for determining the frequency distribution of annual maximum storm rainfalls for the Kāpiti region involves a regional frequency analysis technique using the method of L-Moments. The approach involves identification of the most appropriate distribution for the region, followed by estimation of the regional parameters for that distribution.

The parameter estimates are used to calculate the rainfall quantiles for each locality (station) within the region.

Further discussion of this analysis is covered by the document "*Update of Kāpiti Coast Hydrometric Analysis*, 2008, SKM". A generalised logistic distribution was adopted for the Regional Analysis.

Rainfall recurrence isohyet maps have been generated for the 2, 5, 10, 20, 50 and 100 year ARI rainfall depths and for the predicted impacts of climate change, using kriging geostatistical techniques. The respective maps are appended at the rear of this report as Appendix A.

Clark Unit Hydrograph using the SCS Curve Numbers

The unit hydrograph method of flood estimation was first proposed by Sherman and has since found wide application for both design and estimation of actual floods where a hydrograph and reasonable accuracy are required (Maidment, 1992).

Unit Hydrographs are defined by the runoff resulting from uniform units of rainfall depth over an entire catchment. The difference between rainfall volumes and runoff volumes in the unit hydrograph model is expressed as losses to the system, which can relate to a variety of forms such as evapo-transpiration, and storage within vegetation, the soil, and undrained depressions.

The key components for the development of a unit hydrograph model are therefore rainfall depth relationships, rainfall losses, and catchment characteristics. Each of these items have been separately discussed through the remainder of this chapter, and are covered considerably more comprehensively in Hoggan (1996).

Rainfall Depth Relationships

A 24 hour balanced storm has been proposed for general use. A long balanced storm of this nature is commonly used for floodplain management based work and allows for storage volumes to be more accurately assessed for low impact design storage based solutions.

The balanced storm approach 'nests' high intensity rainfall events within the 24 hour storm profile. This allows for peak estimates of flow to be accurate for small catchments while also providing an appropriate assessment of larger storage volume issues.

A symmetrical distribution has been applied in this case so that the peak rainfall intensities will fall across the midpoint of the storm.

24 hour rainfall depths for any catchment can be estimated for the 2, 5, 10, 20, 50 and 100 year annual recurrence interval (ARI) events and for the predicted impacts of climate change using the

24 hour isohyets maps attached as Appendix A. For larger catchments weighted averages can be calculated, or the catchment can be split up and modelled as a series of sub-catchments.

Once 24 hour rainfall depths have been defined, these have to be converted into some form that allows a balanced storm to be developed in Graphical HEC, or HEC-HMS. For both of these software packages a normalised depth-duration-frequency relationship can be multiplied by the 24 hour total depths to provide duration-depth data. The normalised rainfall depth-duration relationship is shown in table 2-1.

Table 0-1 : Normalised Depth-Duration Relationship for 24-hour Rainfall

Duration	Normalised Rainfall Depth (I/I_{24})
5 Mins	0.08
15 Mins	0.14
1 Hour	0.26
2 Hour	0.38
3 Hour	0.46
6 Hour	0.60
12 Hour	0.81
24 Hour	1

Computed duration depths can be entered directly into Graphical HEC or HEC-HMS as a balanced storm as will be explained in chapter 3 and 4.

The normalised rainfall depth relationship has been developed using the Paraparaumu Aerodrome rainfall record. This provides the longest record of continuous (as opposed to daily read) data in the region with 48 years record.

Rainfall-Runoff Losses

One of the advantages of the SCS method of unit hydrograph modelling is that it provides a system for the delineation of rainfall losses over the period of the storm based on a catchments soil, and land-use characteristics. These curves, unlike linear losses across a storm, allow for antecedent storage to impact the shape of the storm profile. This reflects reality where greater rainfall losses would be expected through the earlier portions of the storm.

The SCS loss method is defined by the following equations; (Hoggan, 1996).

$$Q = \frac{(P - I_a)^2}{P - I_a + S}$$

$$S = \frac{1000}{CN} - 10$$

Where[1];

Q = accumulated Runoff (mm)

P = accumulated Rainfall (mm)

I_a = initial Abstraction

S = potential maximum retention after runoff begins (mm)

CN = curve number, percent of runoff

Initial Abstraction

Initial abstraction is approximated by the empirical equation $I_a = 0.2S$ as the default parameter of the SCS model. Initial abstraction has been found to be less than this in studies completed within the Kāpiti Coast however. Work completed for a Masters thesis confirmed that for storms in the order of an annual flood, I_a values fluctuated between 0 and 4mm (Watts, 2002). This corroborates the results of TP108 (Beca Carter, 1999), which suggests initial abstraction values of 5mm for pervious areas and 0mm for impervious.

Our suggestion is that for fully urbanised catchments I_a values of 0mm be used, and small rural catchments 5mm is used. No data is available for larger catchments, but this is one area where further work could be done with available data.

Delineation of Curve Numbers

Curve numbers (CN's) define the volume of storage losses for any modelled rainfall event. These values can be delineated from a standard set of runoff curve tables that were developed by the US Soil Conservation Service. These tables, (Appendix B), allow for a variety of differing land uses, including urban land, for a range of differing soil types.

As these tables allow for the analysis of a wide variety of land uses and soil types previous work completed in the Kāpiti Coast District has identified base CN values that encompass most of the soil types typically found in the region. A summary table of typical CN values (based on Connell Wagner, 2001) is as follows;

1) [1] TP-108, ARC 1999

Soil Type	Curve Number
1. **Loose Dune Sands**	
Assumed soil type A.	45
2. **Gravel Silt Loams**	
Pasture	69
Urban Gardens*	61
Bush	48
Assumes soil type B. In some cases testing will show gravel soils to have higher infiltration capacities than this in which case soil type A should be assumed.	
3. **Residential Inland Dune sands**	
Assumes soil type B and accounts for construction compactions.	61
4. **Greywacke Argillite Steepland Soils**	
Pasture	79
Urban*	74
Bush	65

* Excludes connected impervious areas as covered in section 2.2.3.

This table is intended as guidance and should not substitute formal ground investigation in cases of uncertain soil conditions.

Catchment Characteristics

A variety of catchment characteristics need to be defined for the development of the Clarke's unit hydrograph. These include

An assessment of soils for development of the Curve Numbers covered above.

An assessment of Connected Impervious Areas.

The nature of catchment Storage.

Calculating the Time of Concentration

Soils Categories

CN tables, as covered in Appendix B, identify the substantial impact of soils properties on total runoff volumes. Soils are assessed under four categories (TP108, 1999) as follows:

Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission (greater than 8mm/h).

Loose dune sands would typically fall within the Group A soil category, and well drained gravels would also fall within this category.

Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well drained to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (4-8mm/h).

Inland compacted sands, less well drained gravel loams and valley based gravel loams would typically fall within this soil group.

Group C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (1-4mm/h).

Most of the steepland argillite, greywacke and loess based soils would fall into this category.

Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils and a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-1mm/h). Existing peat bogs may fall into this category.

Connected Impervious Areas

Connected impervious areas are recorded as a percentage of the residential zone that is impervious, and directly connected via formal drainage systems to the receiving waterway. A previous assessment of current land-use (Connell Wagner, 2001), gave Connected impervious values of;

Residential A – older lots 800m2 average	38% CIA
Residential B – newer lots 600m2 average	55% CIA
Residential C – retirement villages	65% CIA
Industrial	72% CIA
Commercial	85% CIA
Road Designation	36% CIA
Town Centre (Paraparaumu)	50% CIA
Educational	72% CIA
Hospital	72% CIA
Open Space/Rural Zone	0% CIA

Storage Values

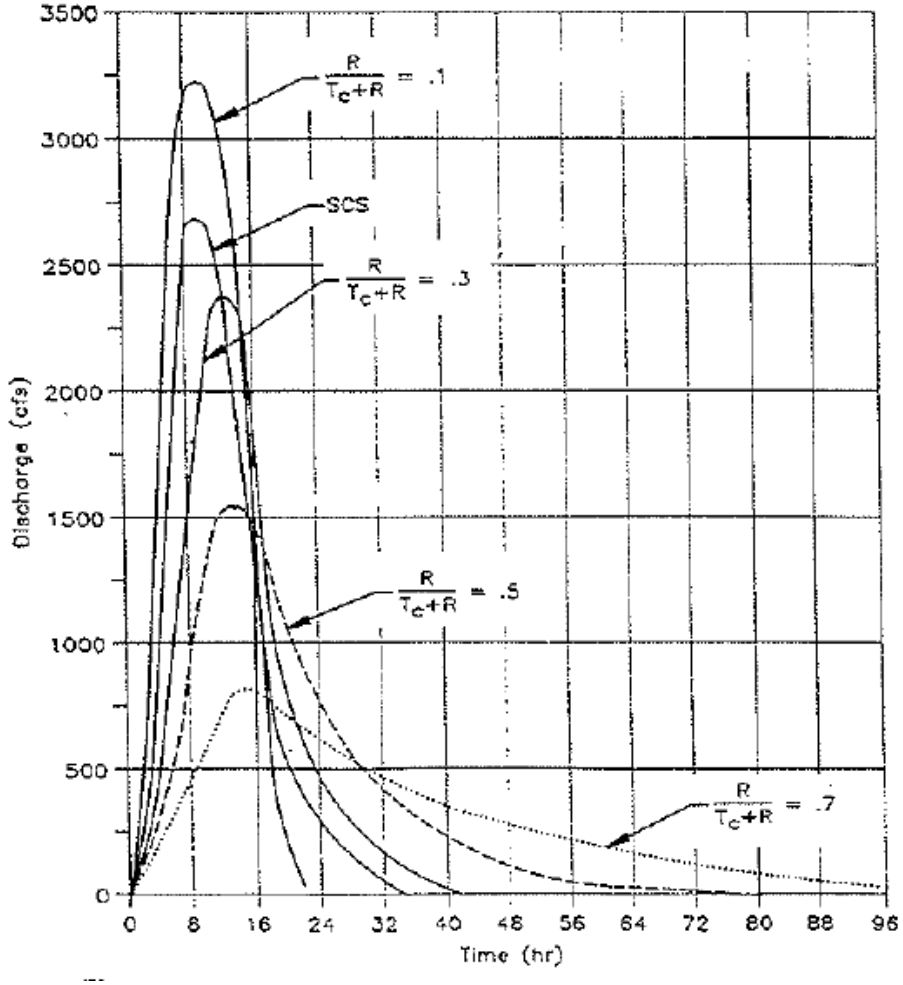
Storage volumes for any one event are defined by the given CN. The shape of the volume outflow in the Clarke's unit hydrograph is determined by the storage coefficient R. The value R is interdependent of the concentration time and can be given as;

R = Tc*Ratio/(1-Ratio) Where the ratio represents the shape of a given hydrograph as shown in figure 2-2.

In this situation the concentration time becomes the control of R given a set ratio of runoff. This is seen as a distinct advantage as although some work has been undertaken in an attempt to quantify R in the Kāpiti Region, (Watts, 2001, Connell Wagner, 1999), this work has not been conclusive, and has provided a wide variety of results.

Using the runoff ratio approach, ratios can be set for differing landuse types to control the runoff hydrograph shape. These can be applied for any given time of concentration to provide a value for R that can be entered into the hydrograph equation.

Figure 0-2 : Storage/Concentration Time Ratio Curves (Hoggan, 1996)



From previous work undertaken by Connell Wagner, (2001), appropriate ratios for R have been defined as;

0.25 for highly developed industrial commercial catchments;

0.30 for most residential catchments;

0.35 for high storage residential catchments;

0.45 for rural steepland catchments.

0.60 for rural flatland catchments.

A more detailed discussion of the calculation of storage coefficients, and indeed all of the above SCS parameters, is covered in Hoggan, 1996. This is an excellent background text for the engineering hydrologist.

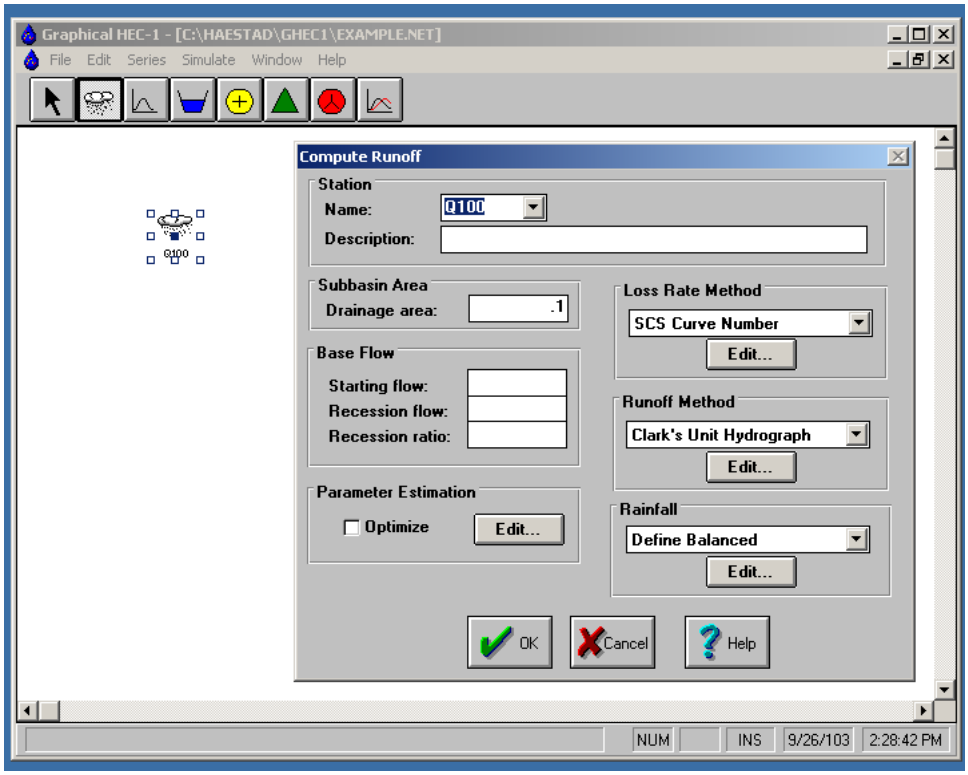
Concentration Times

Times of concentration should be calculated in accordance with the requirements for subdivision and development. An example calculation of concentration times is covered in section 6.4 of this report.

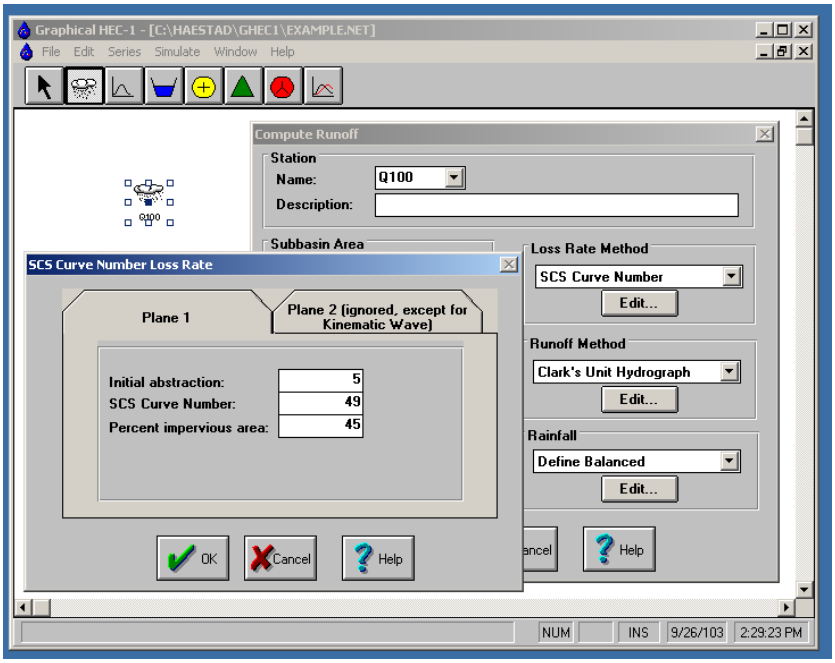
Haestad's Graphical Hec-1

Haestad's are an American software company that have taken the proprietary HEC 'freeware' and developed a windows front end that is both easy to use, and backed up by full software support currently provided by Bentley.

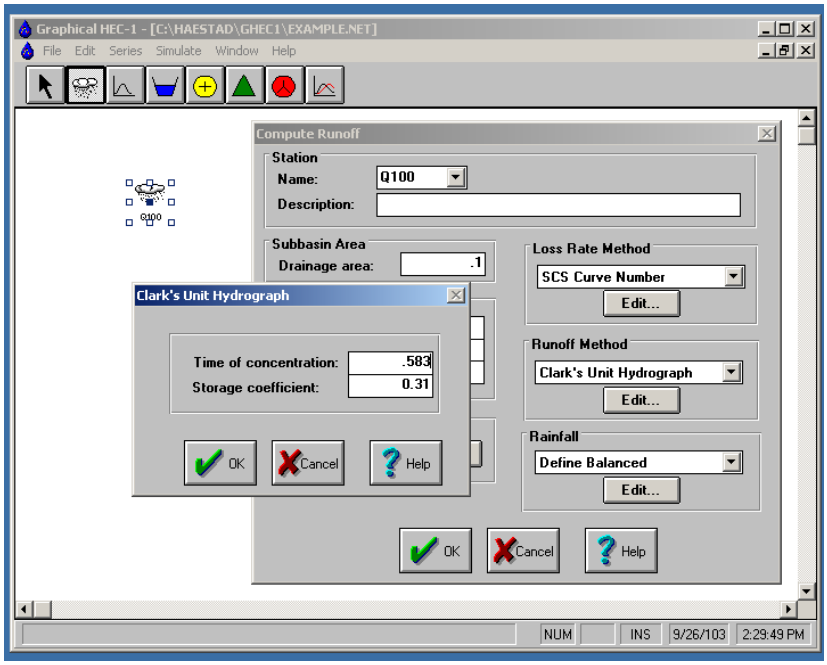
To develop an SCS hydrograph using the information from the previous chapter, the design engineer or hydrologist will need to follow the following steps.



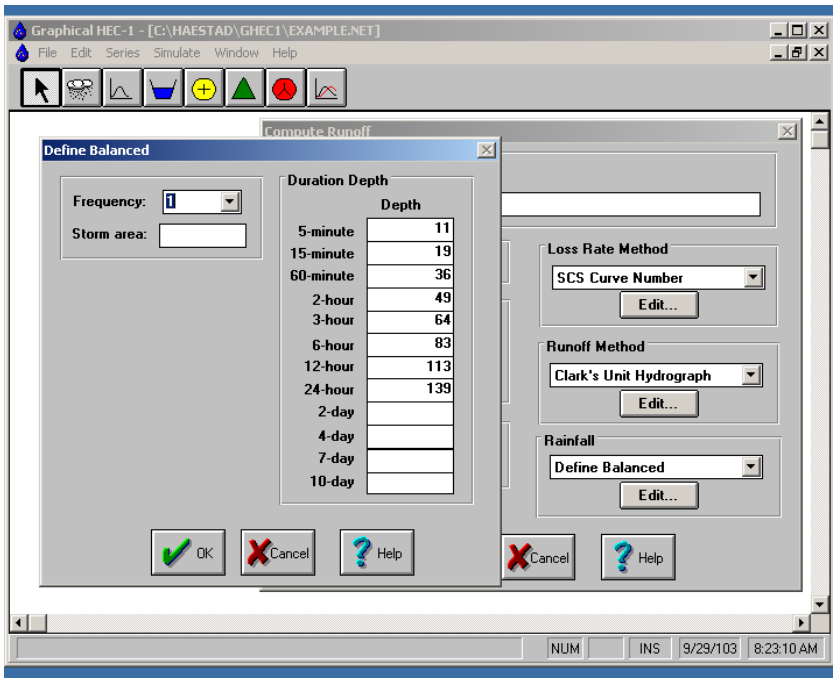
Step 1 – Select the storm item to open a new basin model and enter catchment size under drainage area.



Step 2 – Select the SCS curve number item under the loss rate method and enter the initial abstraction, Curve Number (CN), and Percent impervious area (CIA) in the boxes as shown. These items should be worked through as discussed in the previous chapter.

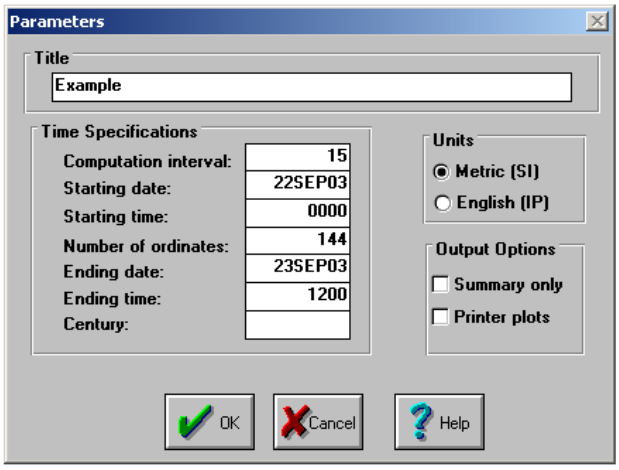


Step 3 – Calculate the time of concentration and enter the time, (in hours), and the assessed storage coefficient under the Clarkes Unit Hydrograph runoff method.

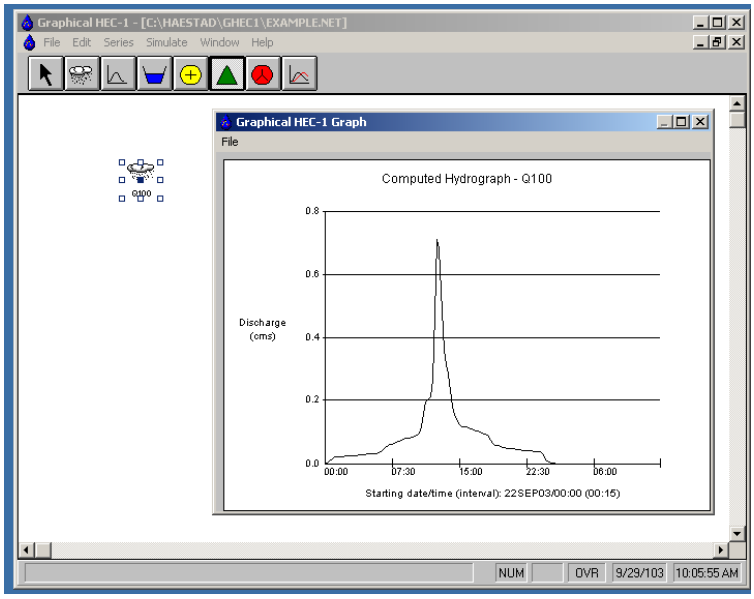


Step 4 – Add in the balanced rain event under 'Define Balanced' in the rainfall portion of the model. These values would be calculated as per sections 2.2.1.

This is all the data that is required to run the model. Typically the model would be run for the full 24 hour storm but in some special circumstances it could be run for shorter periods of time. The hydrograph can be reduced to a twelve hour storm simply by leaving the 24 hour rainfall depth out of the rainfall duration depth series. The model will automatically adjust to a twelve hour storm.

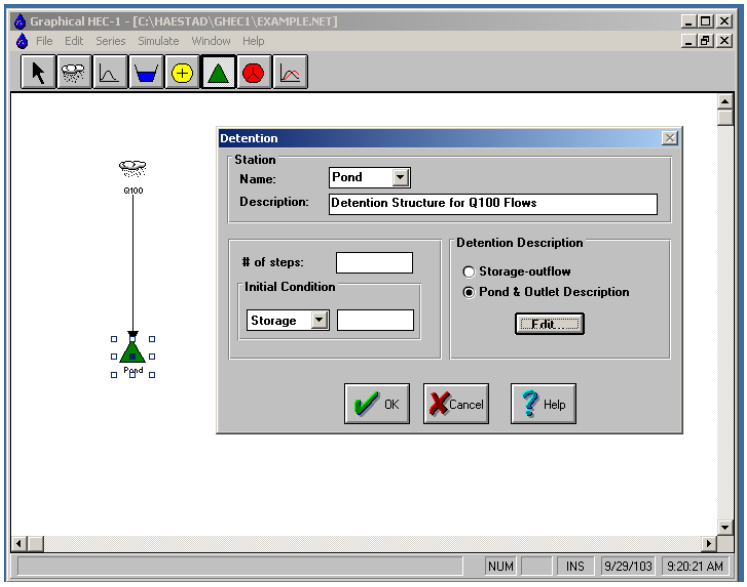


Step 5 – From the *simulation* menu the model parameters need to be entered to allow the programme to be run. Typically for a 24 hour storm the model is run for 36 hours as shown above to allow for the full hydrograph to be represented. In situations where detention is being modelled this may need to be further extended to allow for slow drawdown from detention structures. The final results of the analysis can be represented in tabular form or graphically as shown below.



Storage Structures

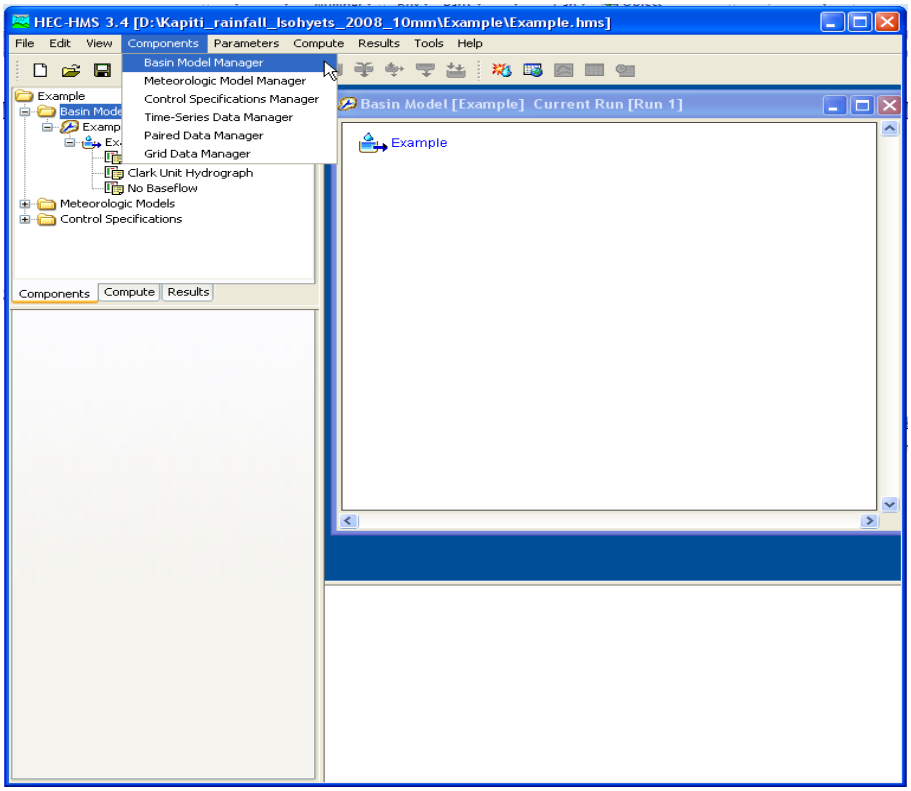
Storage ponds and other structures can be easily modelled within the HEC system by running the newly calculated unit hydrograph through a stage-storage-discharge relationship. Pond and outlet relationships are defined from first principles and entered into the detention basin. The hydrograph is attached to the basin, and the model run as before.



US Army Corps of Engineers HEC-HMS

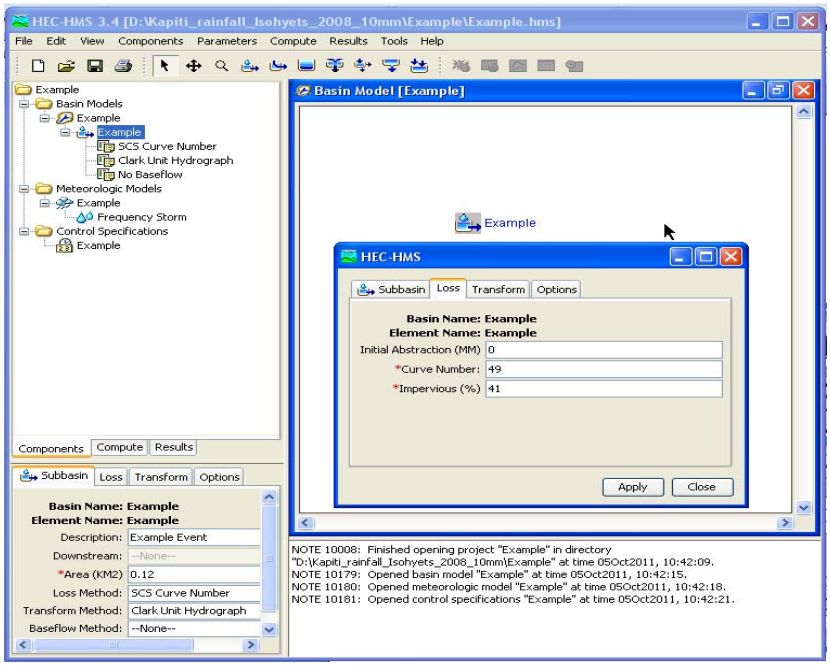
The freeware version of the unit hydrograph modelling software is available off the internet as HEC-HMS. Also available for download with this software are full software user manuals. To compute a unit hydrograph the following steps need to be completed.

For each hydrological 'project' there are three separate 'model' components. These are the basin model, the meteorological model, and the control specifications. Each of these components can be created under the *component* menu in the project screen.

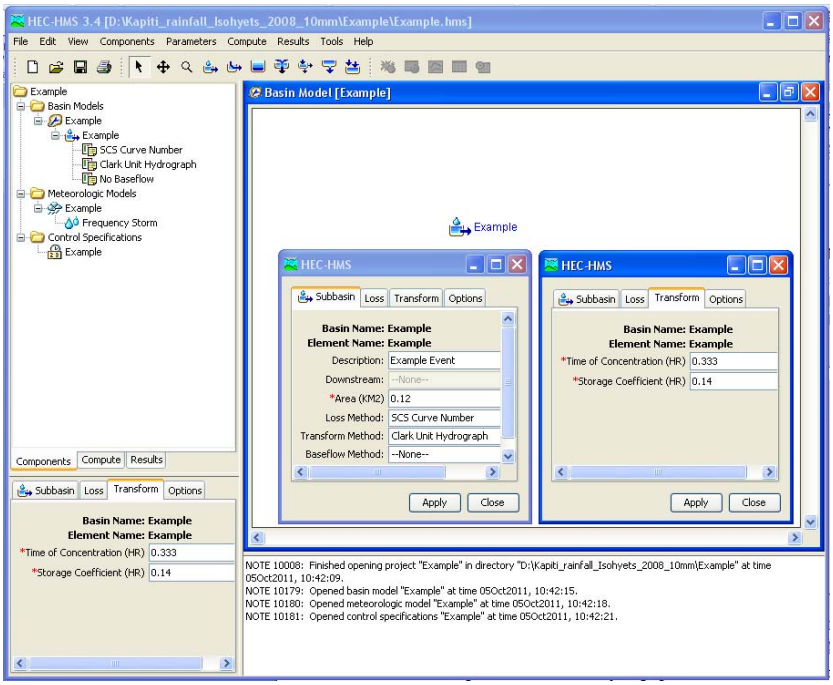


Step 1 – Create a new basin model manager and open a sub-basin element using the sub-basin creation tool. This icon is displayed on the toolbar. Once this element has been created the catchment characteristics can be entered into the model.

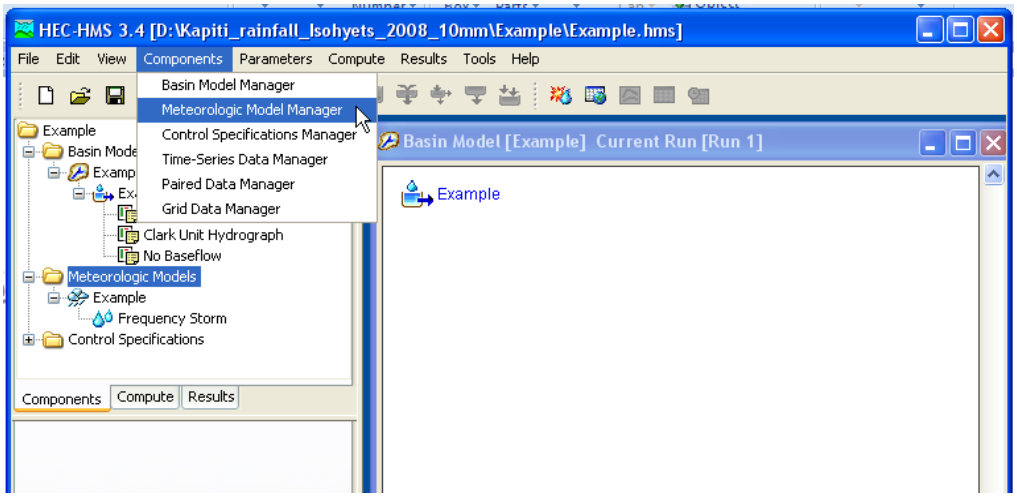




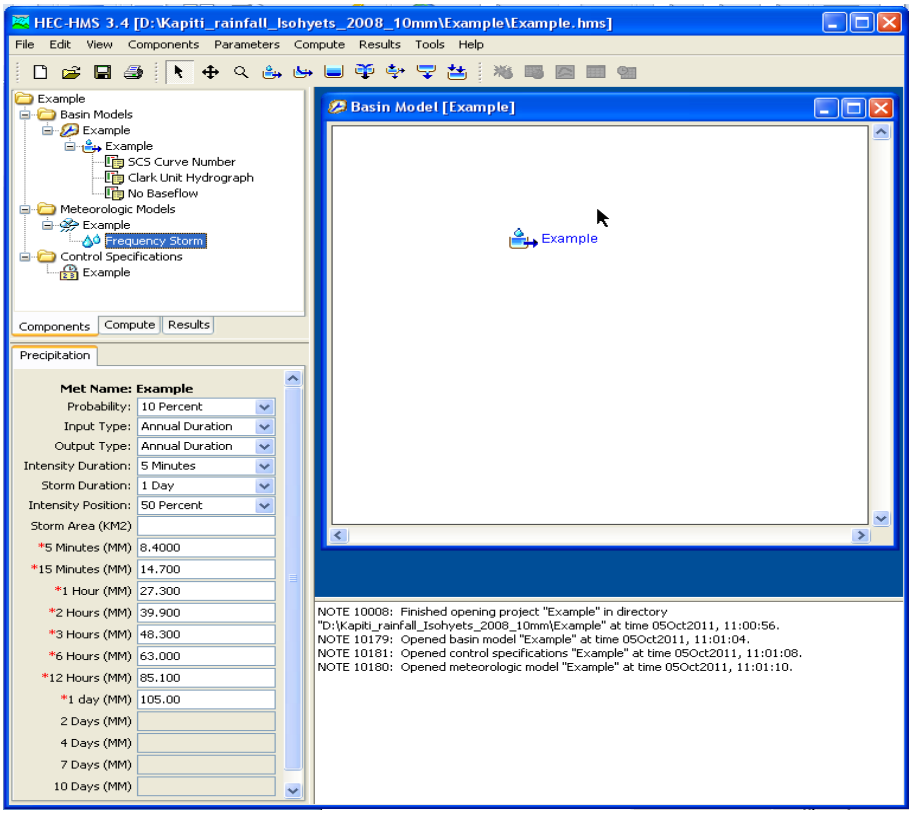
Step 2 – Select the loss rate method as the SCS Curve Number, and enter the initial abstraction, loss rate, and connected impervious area data into the model as outlined in chapter 2.



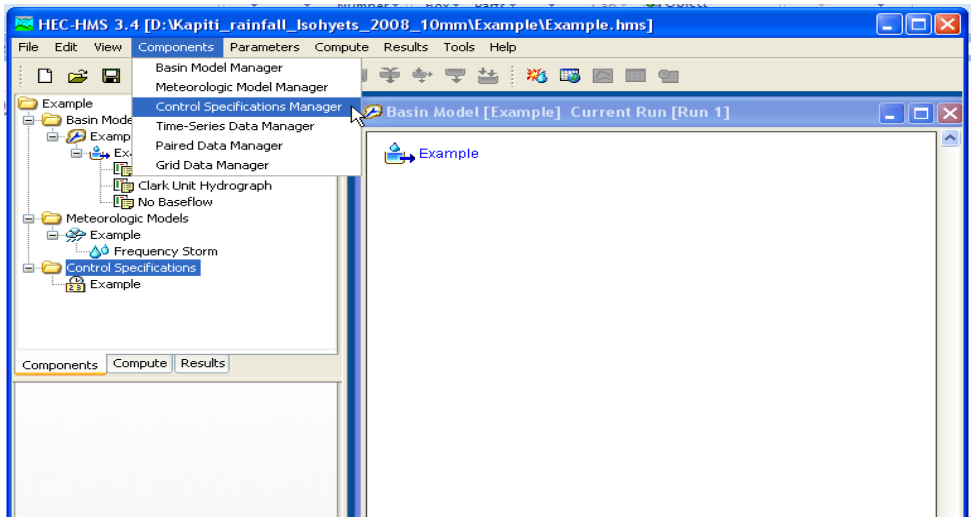
Step 3 – Select the unit hydrograph method as the 'Clark' unit hydrograph and enter time of concentration and storage ratio into the model. Baseflow should be set to 'none' unless some good data is available and the baseflow is going to be significant comparative to the peakflow and total storage volumes.



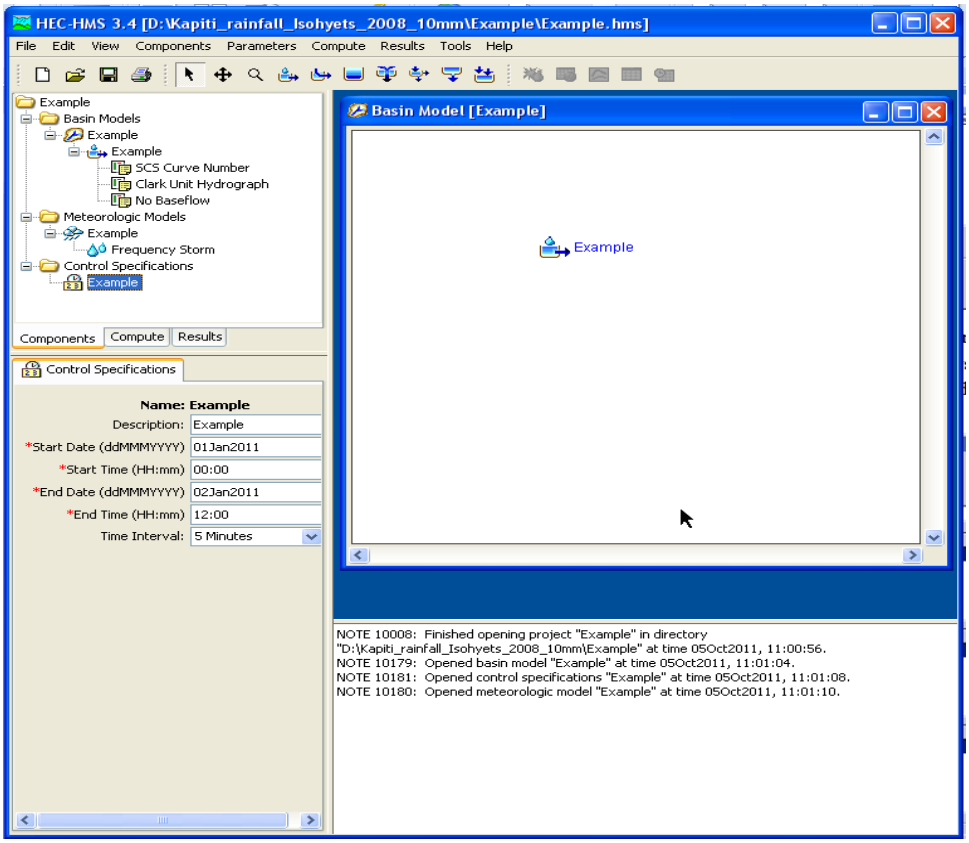
Step 4 – A meteorological model needs to be created under the *component* menu of the project. Once this is defined it can be opened to allow entry of design rainfall information as shown below.



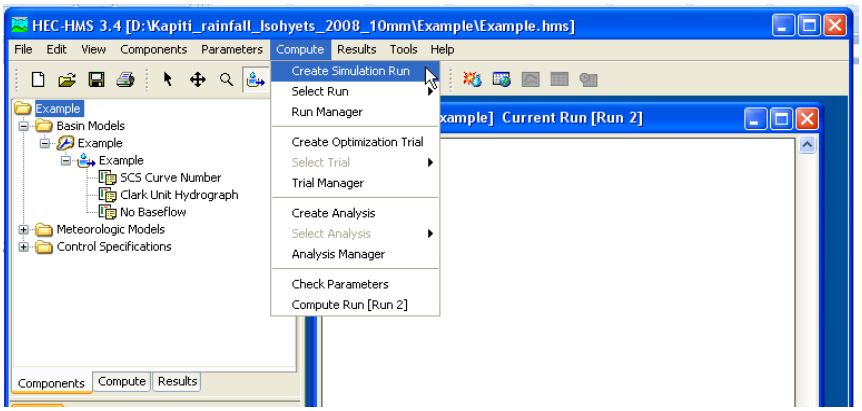
Step 5 – Selecting the 'frequency storm' method from the menu, the balanced rainfall data can be entered into the meteorological model.



Step 6 – A control specification needs to be created under the *components* menu of the project. Once this is defined it can be opened to allow entry of information as shown below



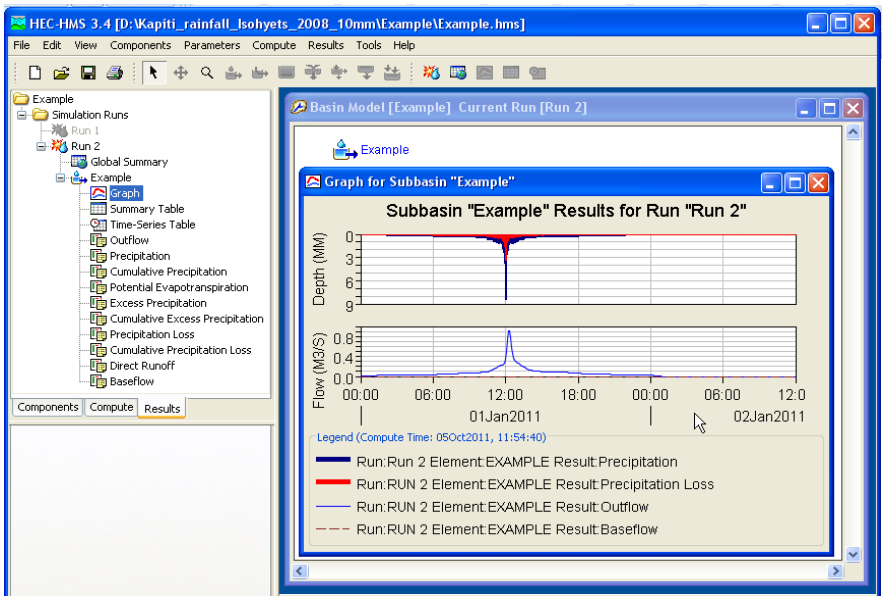
Step 7 - The control specifications are essentially the project time and date running parameters. Again the models should be run for 36 hours as in the previous chapter.



Step 8 – Once the control specifications have been set, the project model run is set up by selecting *Create Run Simulation* under the *Compute* menu. Here the run is given a name and the Basin Model, Metrologic Data and Control Specifications are selected.

The model is run by selecting *Compute Run* under the *Compute* menu or using the icon on the toolbar.





Results can be viewed by selecting the Results Tab. From here the results can be viewed as graphs as well as time series and general tabular data.

Storage Structures

Storage structures in HEC-HMS are developed as a detention structure within the basin model. Again, pond and outlet relationships are defined from first principles and entered into the detention basin. The stage storage and discharge relationship will be based on specific outlet conditions that would be engineered for each specific situation.

Model Limitations.

Rainfall Isohyet Plans

Statistically the frequency analysis undertaken on the available rain gauge sites is sound. Development of the regional isohyets brings these records together in a broad way however, and will tend to ignore small local differences. It should also be noted that most of the upper catchment gauges (Taungata, Oriwa, McIntosh and Kapakapanui) have the shortest rainfall records and may therefore be subject to change in the longer term when a more substantial record is available.

Calibration of Empirical Assumptions

Calibration of empirical assumptions for the development of extreme event storms, covered in this report, is limited. Previous work undertaken by Watts (2002), while providing good data on Initial Abstraction, lacked the large storm events required to meaningfully assess general losses (CN's) and storage functions (R). Some work optimising Hec-1 for the large event storms was undertaken following the storms of October 1998, which were recorded at the Coastlands weir, but a lack of other large events at this gauging station is again a limitation to these results.

The data that has been analysed has typically been from catchments of less than 7 km^2. The isohyet based approach to analysing catchment runoff should not typically be applied to catchments with a greater area than this prior to further calibration work being completed.

For the designer it is important to recognise that due to this a lack of existing hydrometric data the approach encompassed in this report is essentially an uncalibrated empirical process and should be treated as such.

Further Work

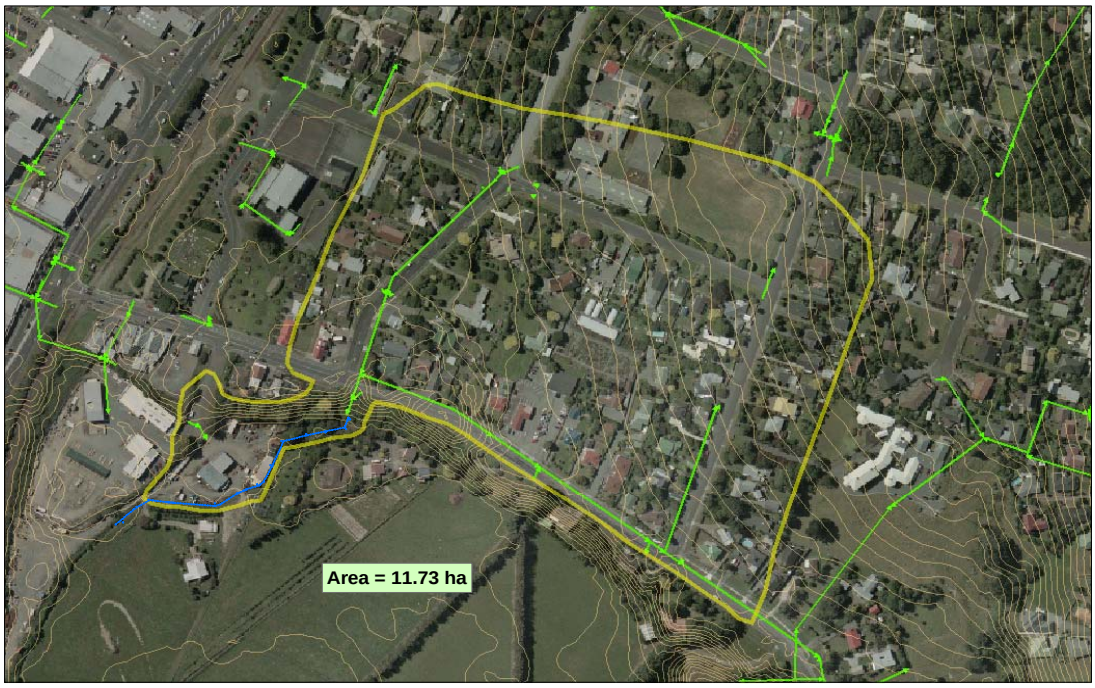
- Some additional local rain gauges through urbanising areas may, in the long term, add useful detail to the rainfall isohyet plans.
- Further analysis of I_a could be developed from the data for the Waikanae and Ōtaki Rivers. This would give some feel for large catchment initial abstraction values.
- New data collected at the Wharemauku, Mazengarb, Mangaone and Waitahu flow gauges could be analysed for frequency, and then used to calibrate for storage runoff ratios and general catchment losses.
- Gauging of additional smaller urban streams, such as the Muaupoko and Wainui, would add to the existing knowledge base and allow for more conclusive analysis in the long term.

Worked Example

Introduction

The following example has been developed to assist the engineer in applying this standard. The example is based on a small residential/commercial catchment in Waikanae that discharges via an open channel to the Waikanae River. The area was defined as being 11.73 hectares and is drained via a formal stormwater piped network as shown below in Figure 6-1.

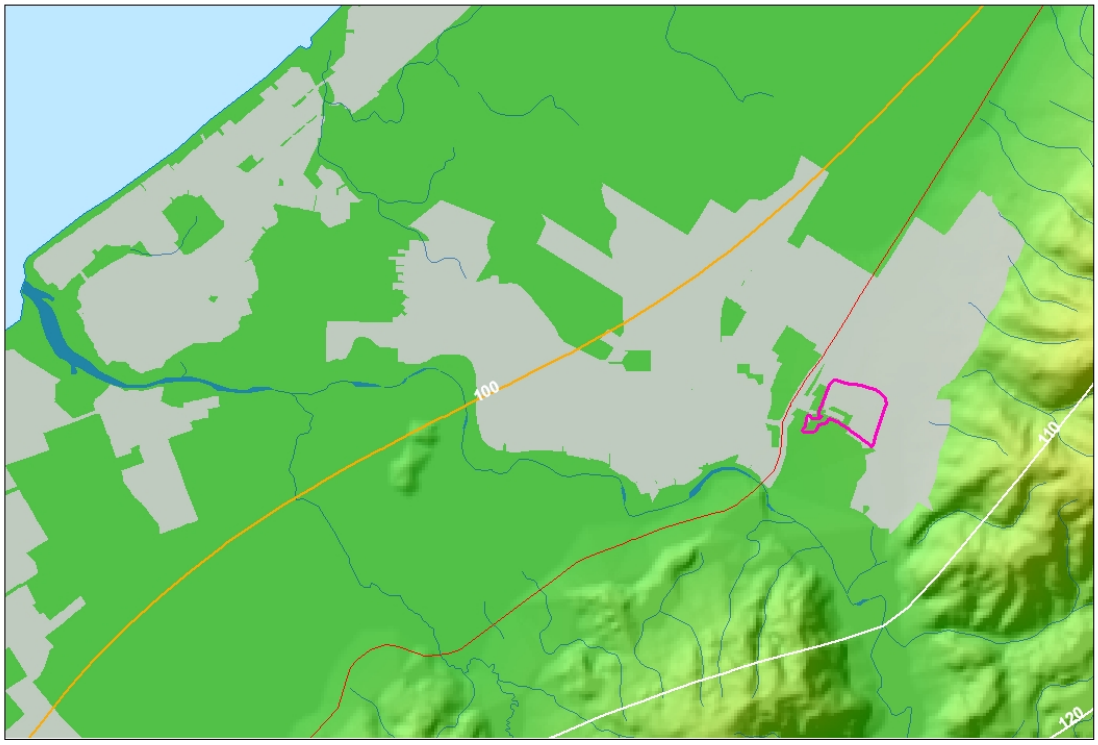
Figure 6-1. Site Plan of Example Catchment.



Rainfall

Rainfall depths are taken from the 10 year isohyets plan for the catchment area identified. Figure 6-2 locates the catchment on the isohyets plan.

Figure 6-2. Catchment Location on the 10 Year Isohyet Plan.



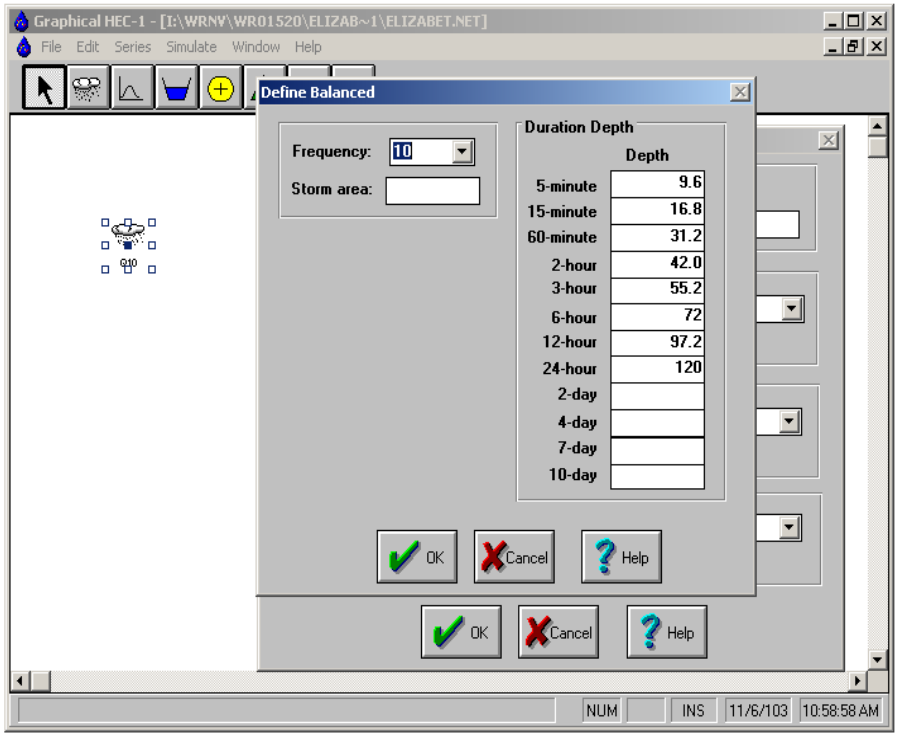
Average rainfall depths for this catchment are estimated at 105mm off the plan. This total depth is then applied to the normalised depth-duration relationship, (section 2.2.1), as shown in table 6-1.

Table 6-1: Normalised Depth-Duration Relationship for 24-hour Rainfall

Duration	Normalised Rainfall Depth (I/I_{24})	Normalised 10 Year Rainfall
5 Mins	0.08	**8.4**
15 Mins	0.14	**14.7**
1 Hour	0.26	**27.3**
2 Hour	0.38	**39.9**
3 Hour	0.46	**48.3**
6 Hour	0.60	**63.0**
12 Hour	0.81	**85.1**
24 Hour	1	**105**

These normalised 10 year rainfall depths can then be entered, with the catchment area, into the Hydrological model as outlined in chapter 3 and 4 and shown in Figure 6-3 using Graphical Hec.

Figure 6-3. Rainfall depths applied for the 10 Year Example



Catchment Characteristics

Initial Abstraction

Initial abstraction in a fully urbanised catchment would be set at **0mm** as outlined in section 2.2.2.1.

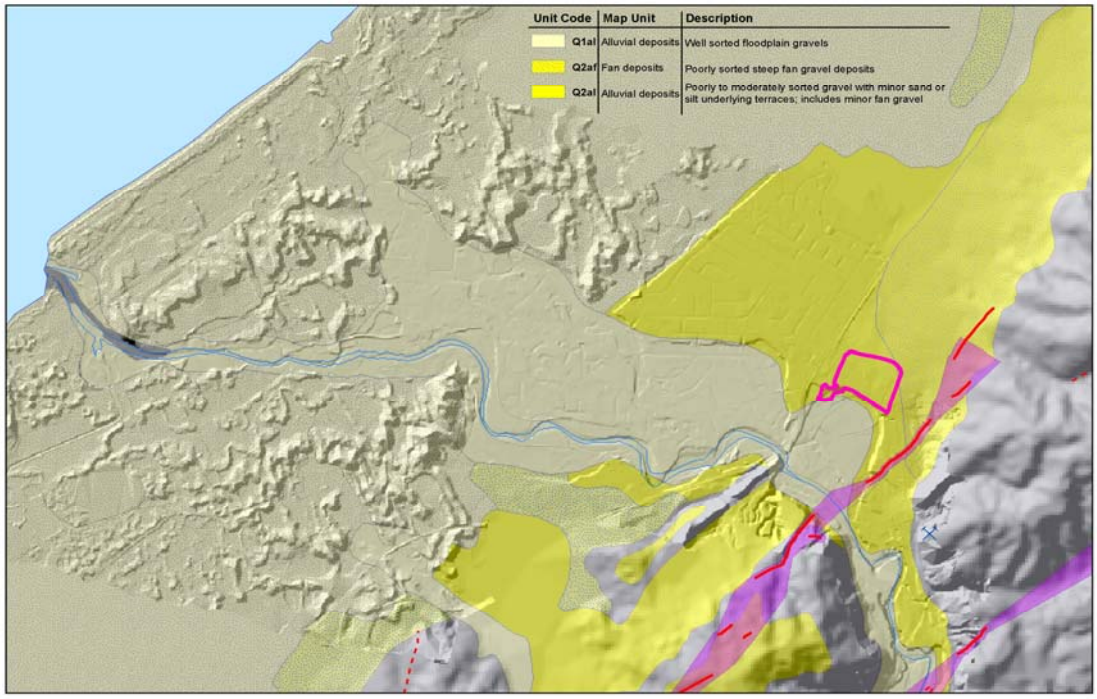
Curve Numbers (CN) and Connected Impervious area.

Defining soils parameters can be taken from Soil Bureau Land-Use Inventory Maps in all but urban areas. These maps are difficult to source however and may need to be supplemented by larger geological plans which also cover urban areas.

In either case site an initial site walkover should confirm the nature of the soils with ground investigation if necessary.

For the example case we have used the larger geology maps for the urban based area (figure 6-4). The geology shows a mixture of alluvial and outwash gravels that are well to moderately well drained.

Figure 6-4. Geology of the Given Catchment Area



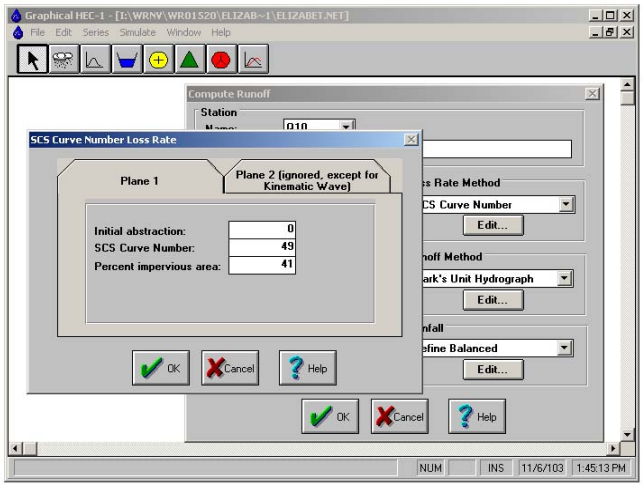
These alluvial soils are well drained and there is no record of flooding in this catchment on Councils flood incidence records. In addition previous soakage testing in this area have shown high soakage rates. From Appendix B it has been assumed that these soils would fall into category A, and that the general ground cover (lawns gardens etc) would be in fair (average) condition. This gives a **CN of 49.**

To this value the impervious portion of the catchment needs to be added. This has been estimated as 38% under section 2.2.3.2 for the Residential portion of the catchment (92%), and 72% for the remaining industrial area. This gives a weighted connected impervious area of;

Industrial @ 72%*.08	5.8
Residential @ 38%*.92	35.0
Estimated Total Impervious Area	**41%**

These loss rate characteristics are entered into the model under the loss rate method as discussed in chapter 3 and shown in figure 6-5.

Figure 6-5. Loss Rate Factors Applied for the 10 Year Example.



Time *of Concentration*

Calculation of the time of concentration under Kāpiti District Councils new sub-divisional and development requirements is as follows.

The first step in the calculation of rainfall intensity is to determine the "time of concentration" for the catchment, or the time taken for water to travel from the remotest part of the catchment to the head of the section of the drain or culvert in question.

Tc = time of flow to design point = (overland flow + open channel flow + kerb and channel flow + pipe flow), where:

a) Time of overland flow for most urban drainage systems can be obtained from "Chart for Overland Flow" Appendix C.

If the natural surface is longer than 1000 metres the Empirical Bransby-Williams formula shall be used:

$$tc = \frac{FL}{A^{0.1}\ S^{0.2}}$$

where

tc = time of concentration in minutes

F = 59.5 when area in square metres

= 92.7 when area in hectares
L = main channel length (km)
A = catchment area
S = main channel slope (m/km)

b) Time of channel flow can be obtained using Mannings formula.

c) Time of kerb and channel flow can be obtained using Mannings formula for n 0.018 as an average value.

d) Time of pipe flow (the example assumes a 300mm diameter) can be obtained by from the "Chart for Pipe Flow" Appendix C.

Time of concentration shall not be taken less than 10 minutes in all areas:
The outcome for the Time of Concentration calculations for the example catchment would therefore be;

Overland Flow is 20m @ 3% grade (Refer Appendix C) 10.5min

Gutter Flow is 150m @ 4.8% grade (Refer Appendix C) 2.0min

Pipe Flow is 600m @ 2% grade @ 1.8m/s (Refer Appendix C) 5.5min

Concentration time in Minutes. **Say 20min**.

Storage Value (R)

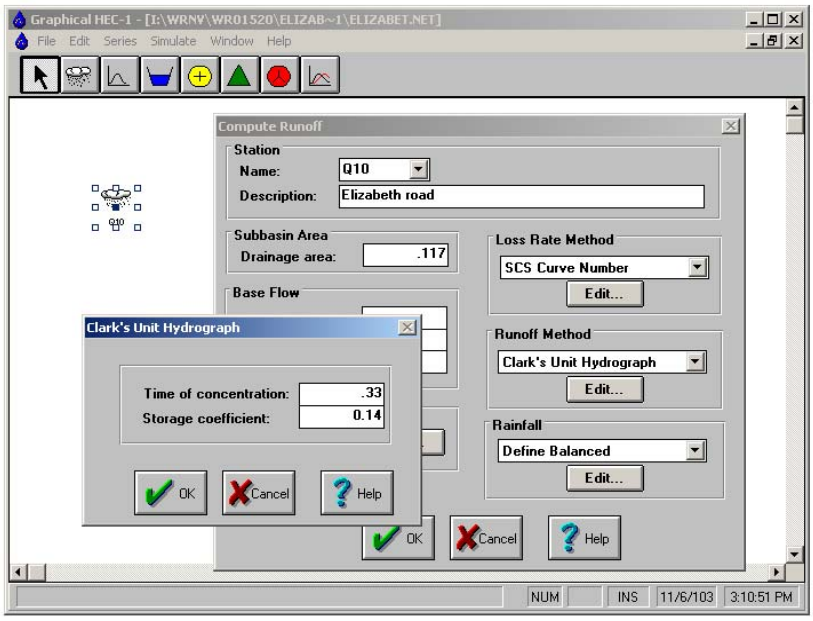
From section 2.2.2.3 the ratio for R has been defined as 0.30 for standard residential catchments. To determine R from this ratio the function R = Tc*Ratio/(1-Ratio) is applied.

In this case this would give an R value of

$$R = 0.33*.30/(1-.30) = \mathbf{0.14}$$

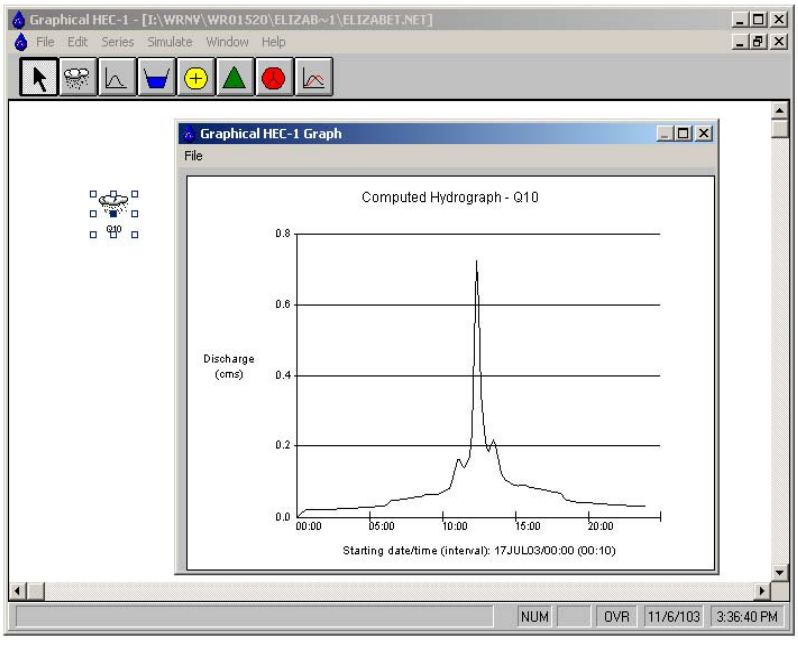
The time of concentration and R values are the final parameters to be entered into the hydraulic model as given in figure 6-6.

Figure 6-6. Concentration Times and Storage Values Applied for the 10 Year Example



This is the last of the parameters required for the calculation of the unit hydrograph which can now be computed to provide the output hydrograph. This output is given below as figure 6-7.

Figure 6-7. Output Hydrograph from 10 Year Example.

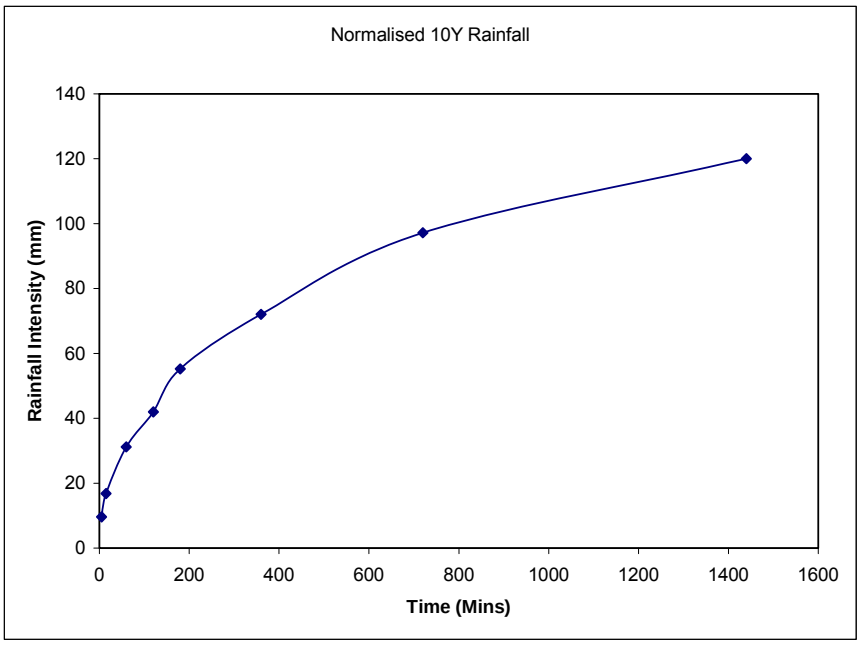


Rational Formula

The Rational Formula has been traditionally used to estimate catchment peak flows in urban areas. It is not the intention of the updated sub-divisional and development requirements to exclude the use of the rational formula for this purpose. It will typically not be useful for calculating volume based solutions however and will not be encouraged for use in this area of design.

The isohyets plans that have been developed can be used to provide all the rainfall intensity data required under this method. This can be achieved by plotting the normalised rainfall results from table 6-1 against time as shown below in figure 6-8.

Figure 6-8. 10 Year Rainfall Intensity Curve for the Example Catchment



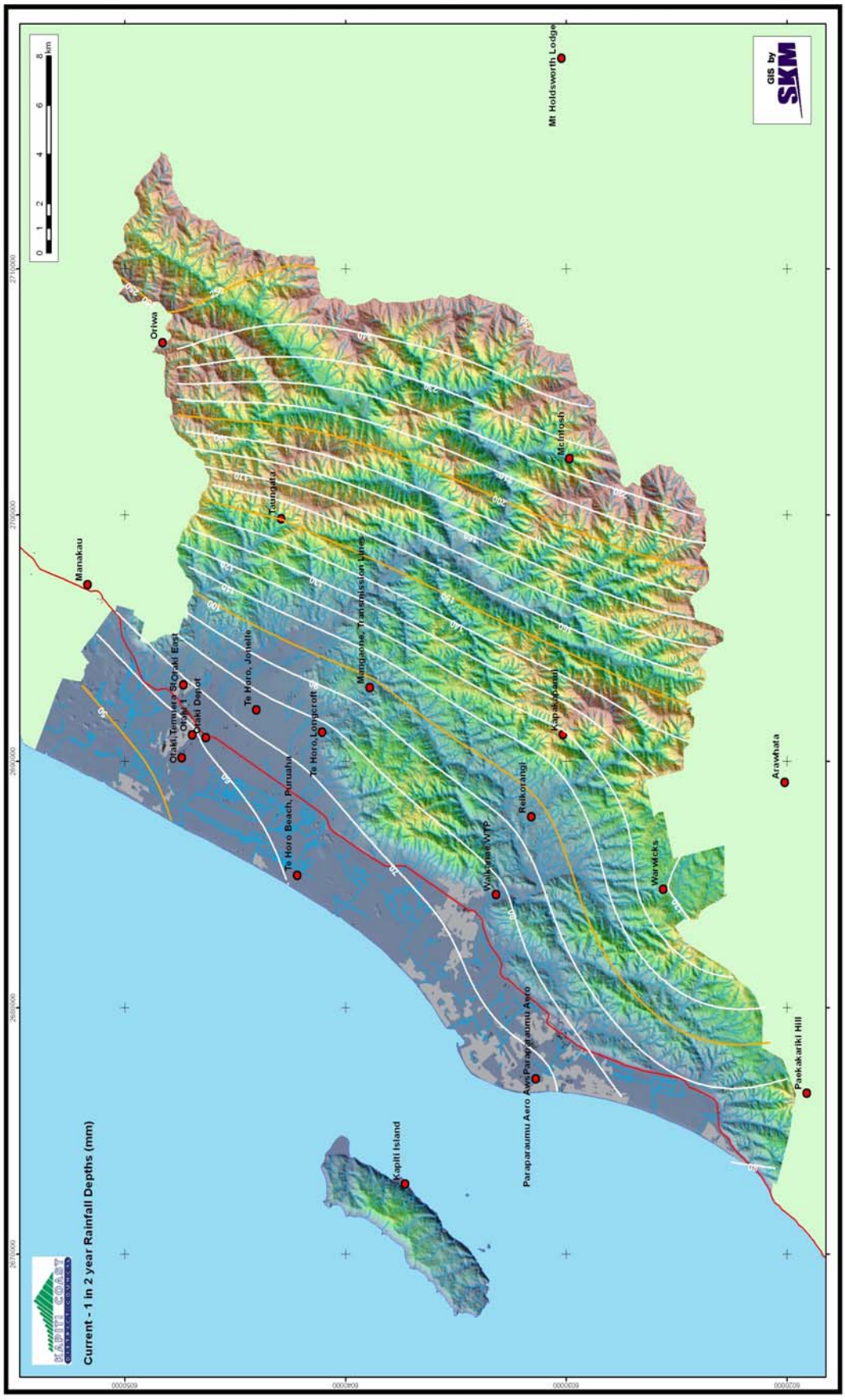
Importantly it should be remembered that intensity under the rational formula is measured in hours so ensure that intensities for shorter or longer periods are factored up or down respectively.

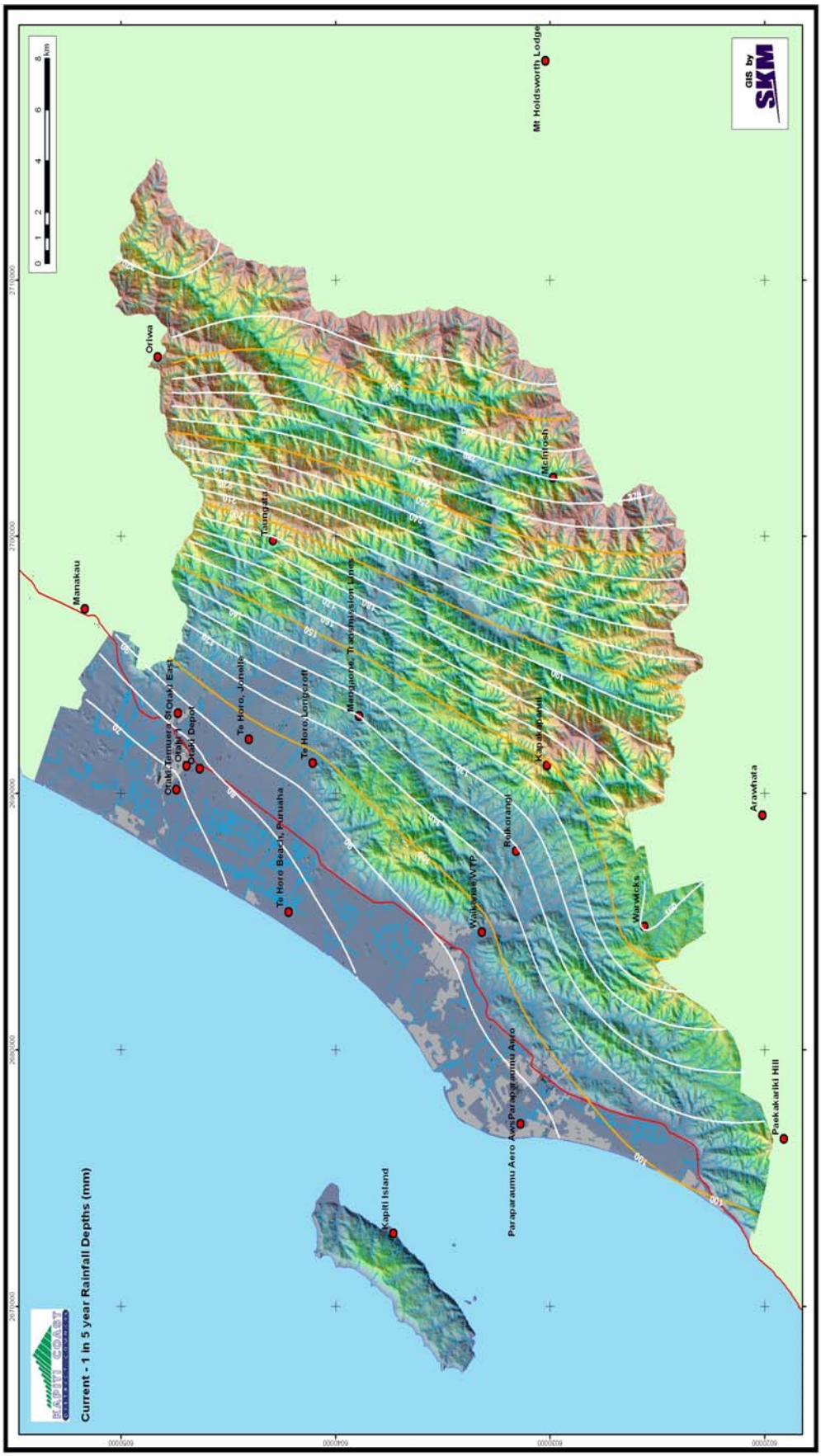
References

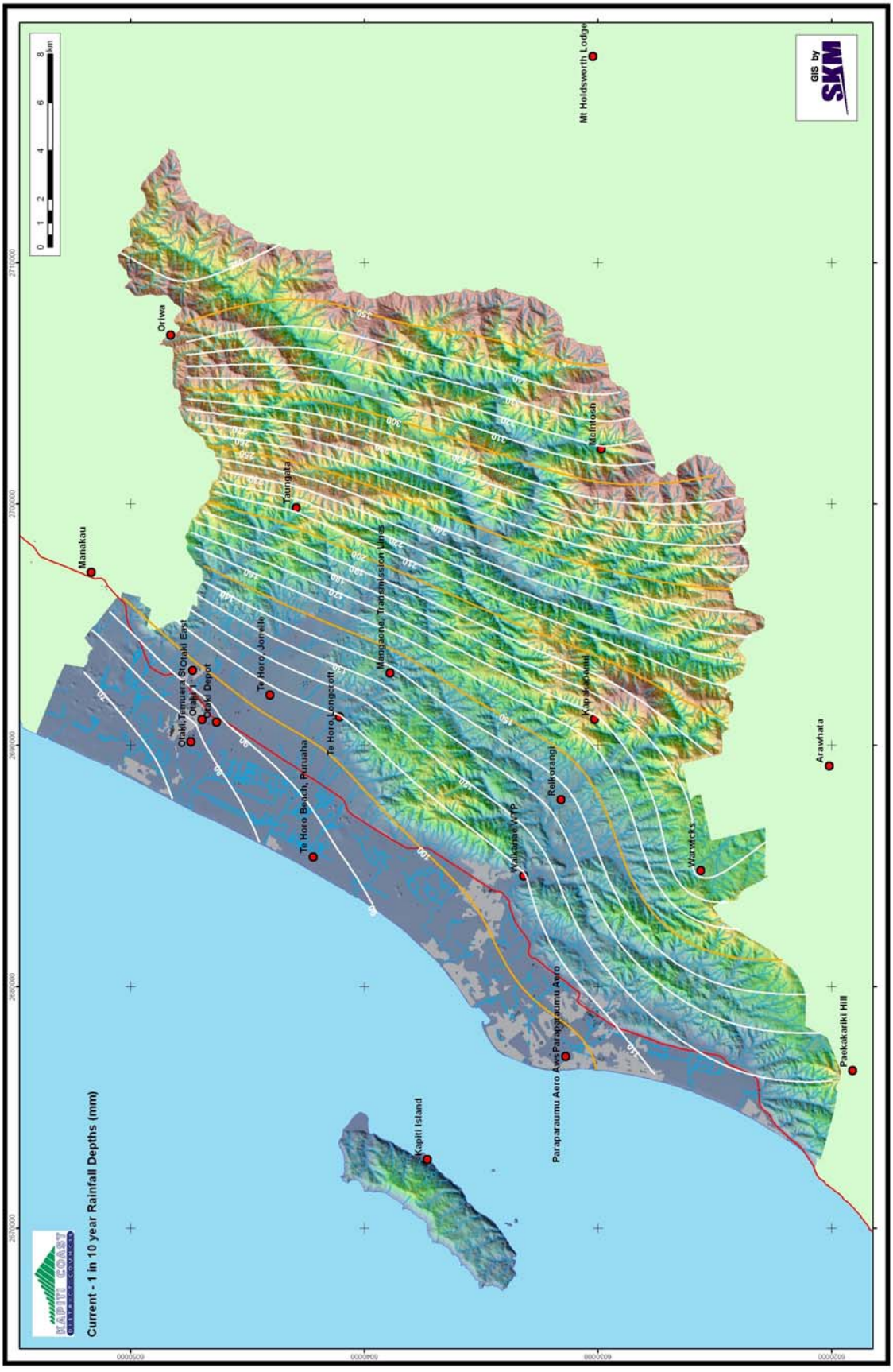
Maidment, D (ed.), 1992	Handbook of Hydrology, McGraw Hill.
Watts, L, 2002	Hydrologic Response and Runoff Model Parameters in the New Zealand Coastal Zone. Unpublished Masters Thesis, Victoria University of Wellington
Auckland Regional Council, 1999	Guidelines for Stormwater Runoff Modelling in the Auckland Region. Technical Publication N° 108, Auckland Regional Council
Hoggan, D, 1996	Computer Assisted Floodplain Hydrology and Hydraulics, Second Edition. Department of Civil and Environmental Engineering, Utah State University, McGraw-Hill.
Connell Wagner, 1999	Wharemauku Stream Hydrometric Analysis of October 20-21st Storm, Connell Wagner Ltd
Connell Wagner, 2001	Wharemauku Stream Stormwater Runoff and Floodplain Assessment.
USACE, 2000	Hydrologic Modelling System HEC-HMC Technical Reference Manual – US Army Corps of Engineers, Hydrologic Engineering Centre
SKM, 2008	Update of Kāpiti Coast Hydrometric Analysis

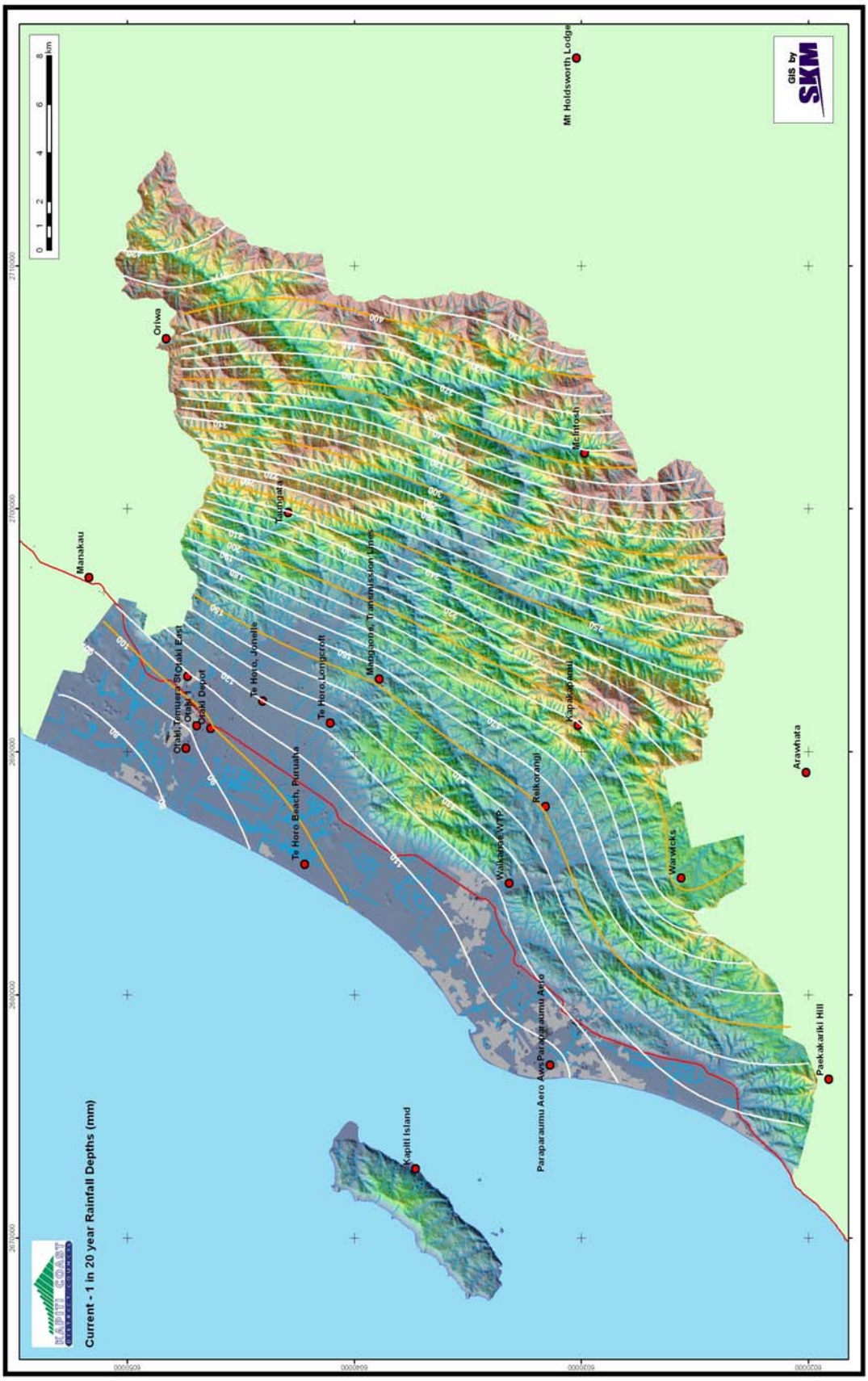
Appendix A Rainfall Recurrence Isohyet Maps

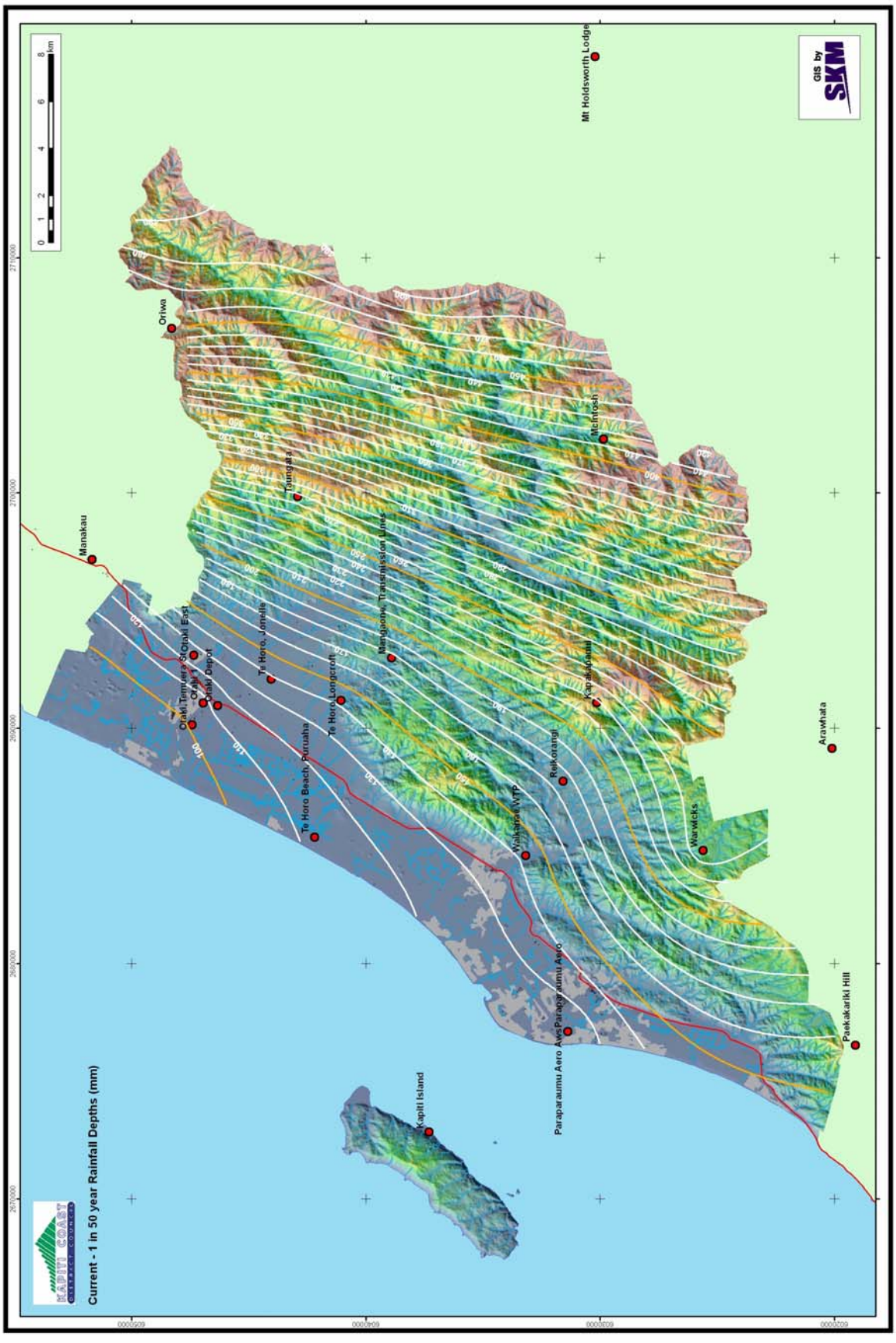
Current Rainfall Recurrence Isohyet Maps

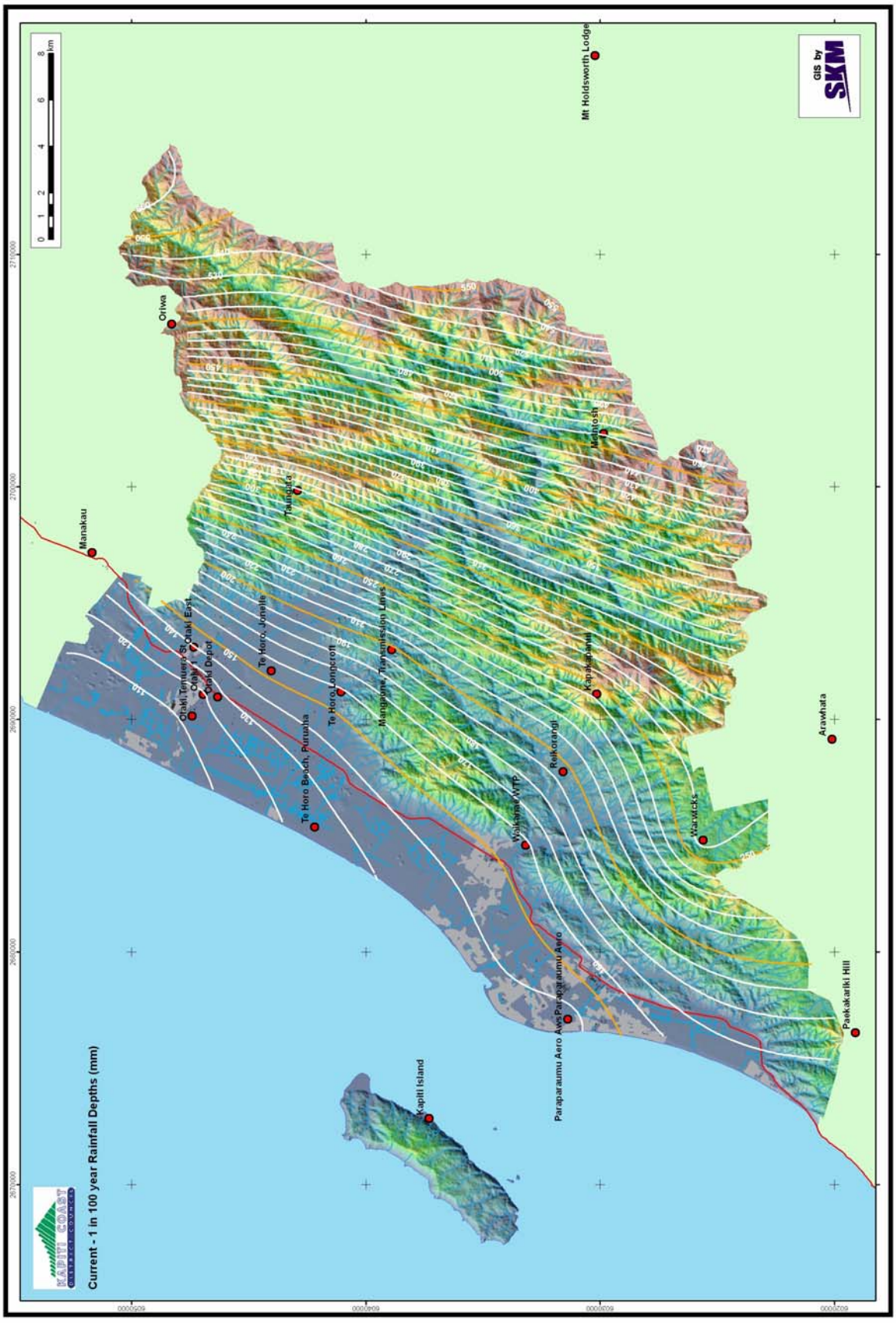

Mt Holdsworth Lodge
Otiwa
Manakau
Tararua
Melrose
Te Horo, Joiselle
Te Horo, Longcroft
Te Horo Beach, Puruahu
Reikorangi
Waikanae WTP
Waikanae
Paraparaumu Aero
Paekakariki Hill
Kapiti Island
Current - 1 in 2 year Rainfall Depths (mm)
GIS by SKM

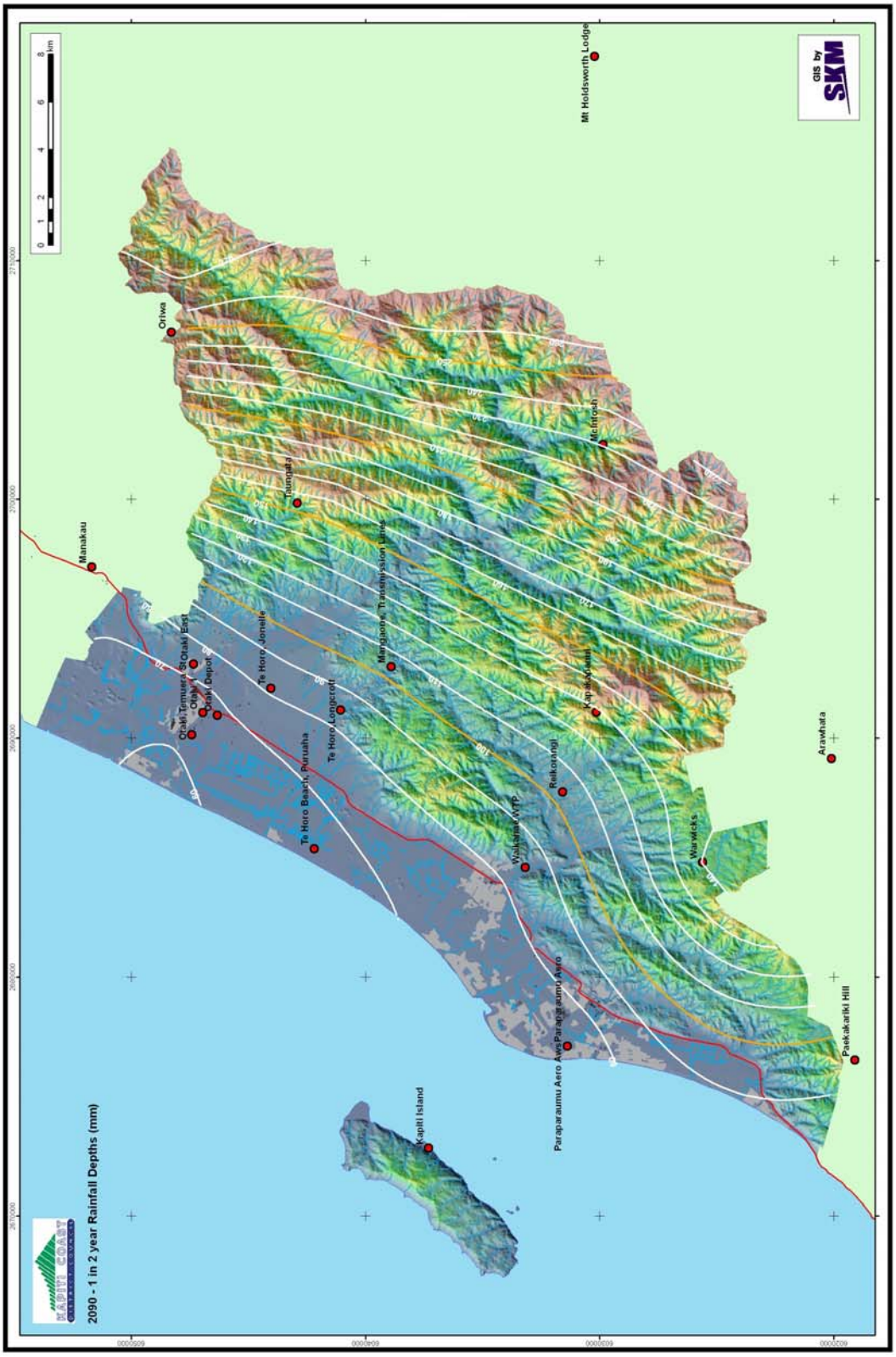

Current - 1 in 5 year Rainfall Depths (mm)
GIS by SKM
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Tararua
Melrose
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Te Horo, Lombardt
Te Horo Beach, Puruaha
Otaki, Tamuera St
Otaki East
Otaki
Otaki Depot
Kapakapanui
Arawhata
Reikorangi
Waikanae WTP
Waikanae
Paekakariki Hill
Paraparaumu Aero Aws
Paraparaumu Aero
Kapiti Island

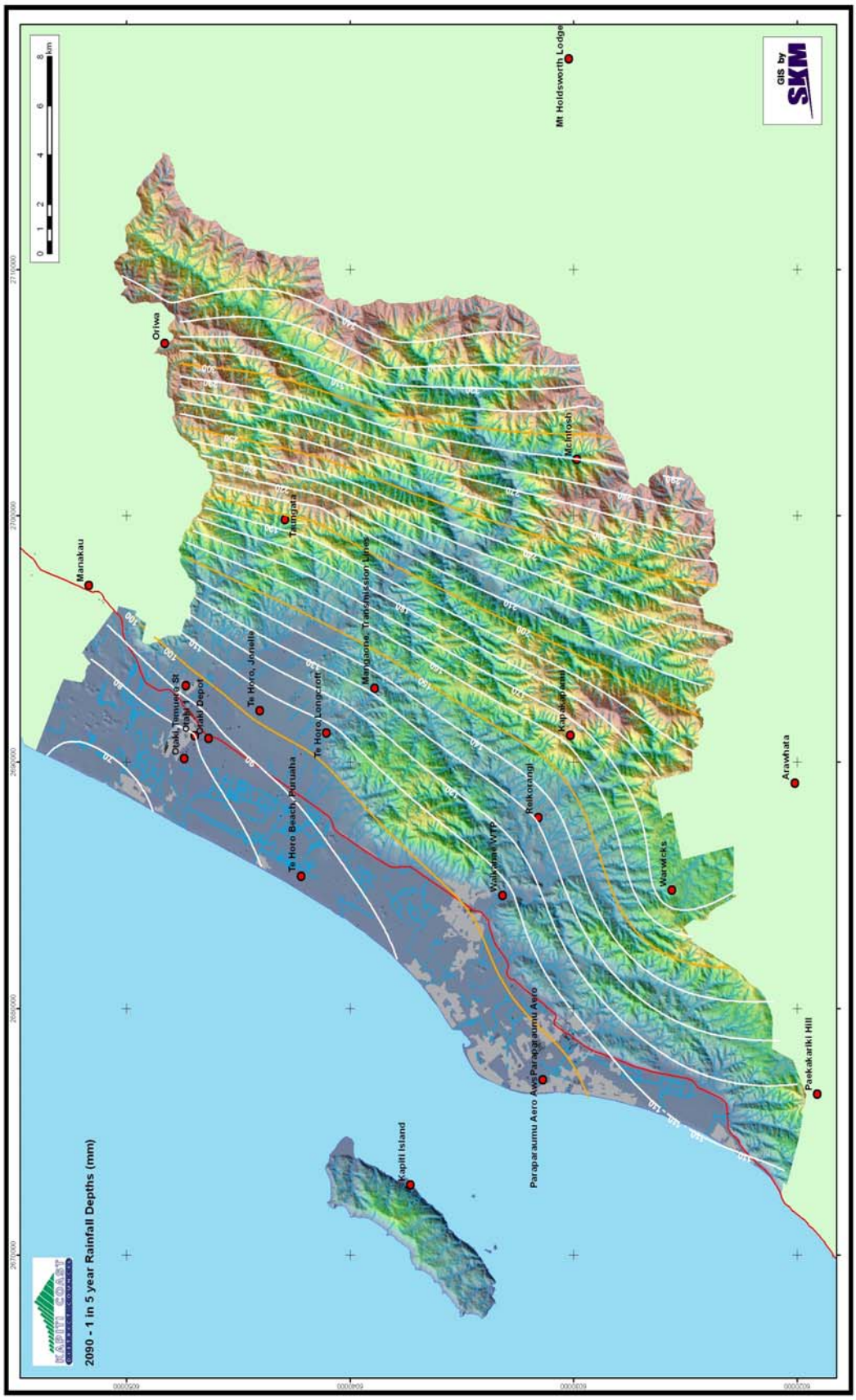

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Manakau
Tararua
Melrose
Te Horo, Jonelle
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Kapakapanui
Arawhata
Reikorangi
Waikanae WTP
Warwicks
Paraparaumu Aero Ave
Paraparaumu Aero
Paekakariki Hill
Kapiti Island
Current - 1 in 10 year Rainfall Depths (mm)
GIS by SKM

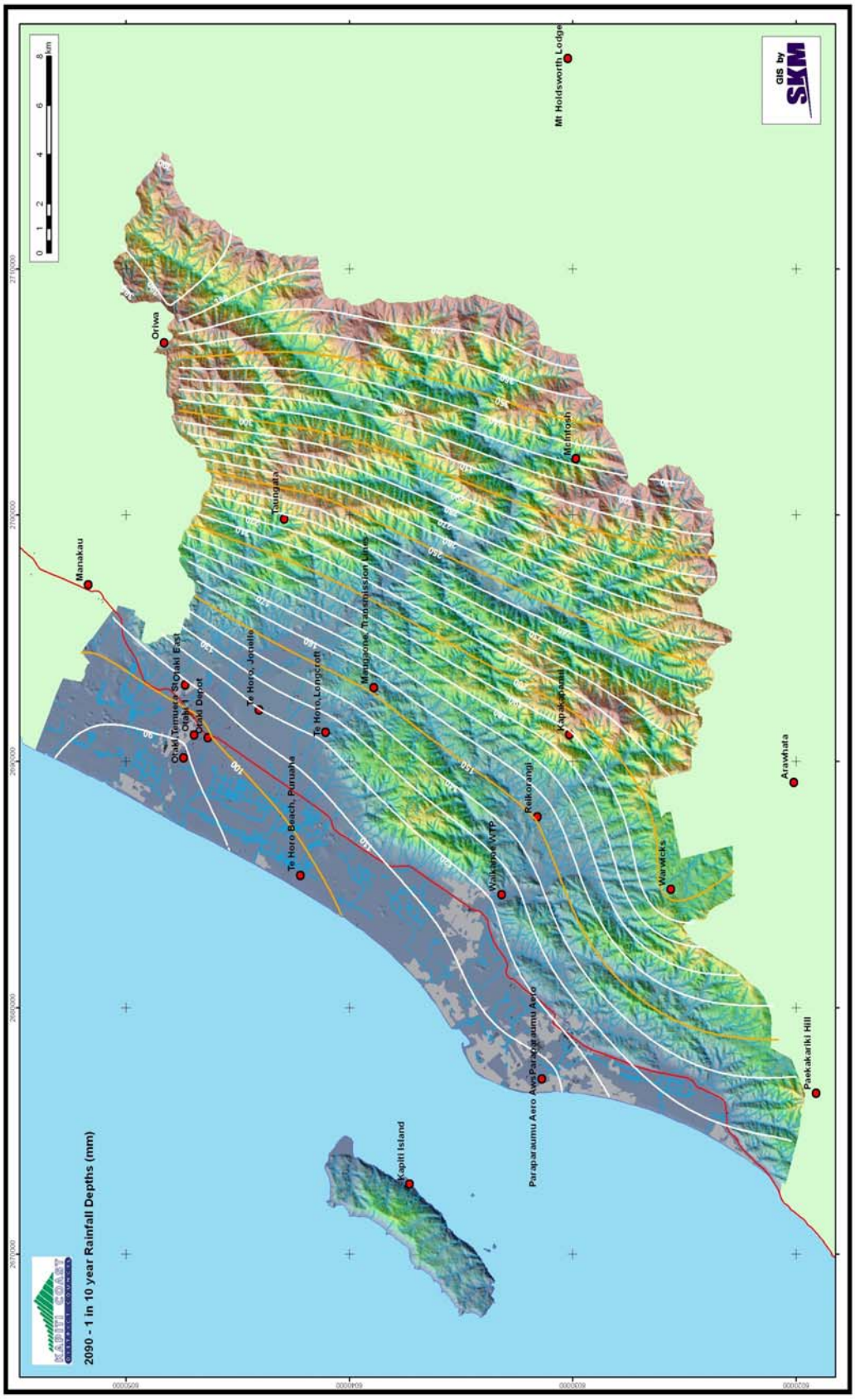

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Te Horo, Longcroft
Te Horo Beach, Putuaha
Otaki, Tamatea St
Otaki East
Otaki, 1
Otaki Depot
Mangaroa Transmission Lines
Kapakapanui
Raikorangi
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Waikanae WTP
Warwicks
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Paraparaumu Aero Aws
Paekakariki Hill
Kapiti Island

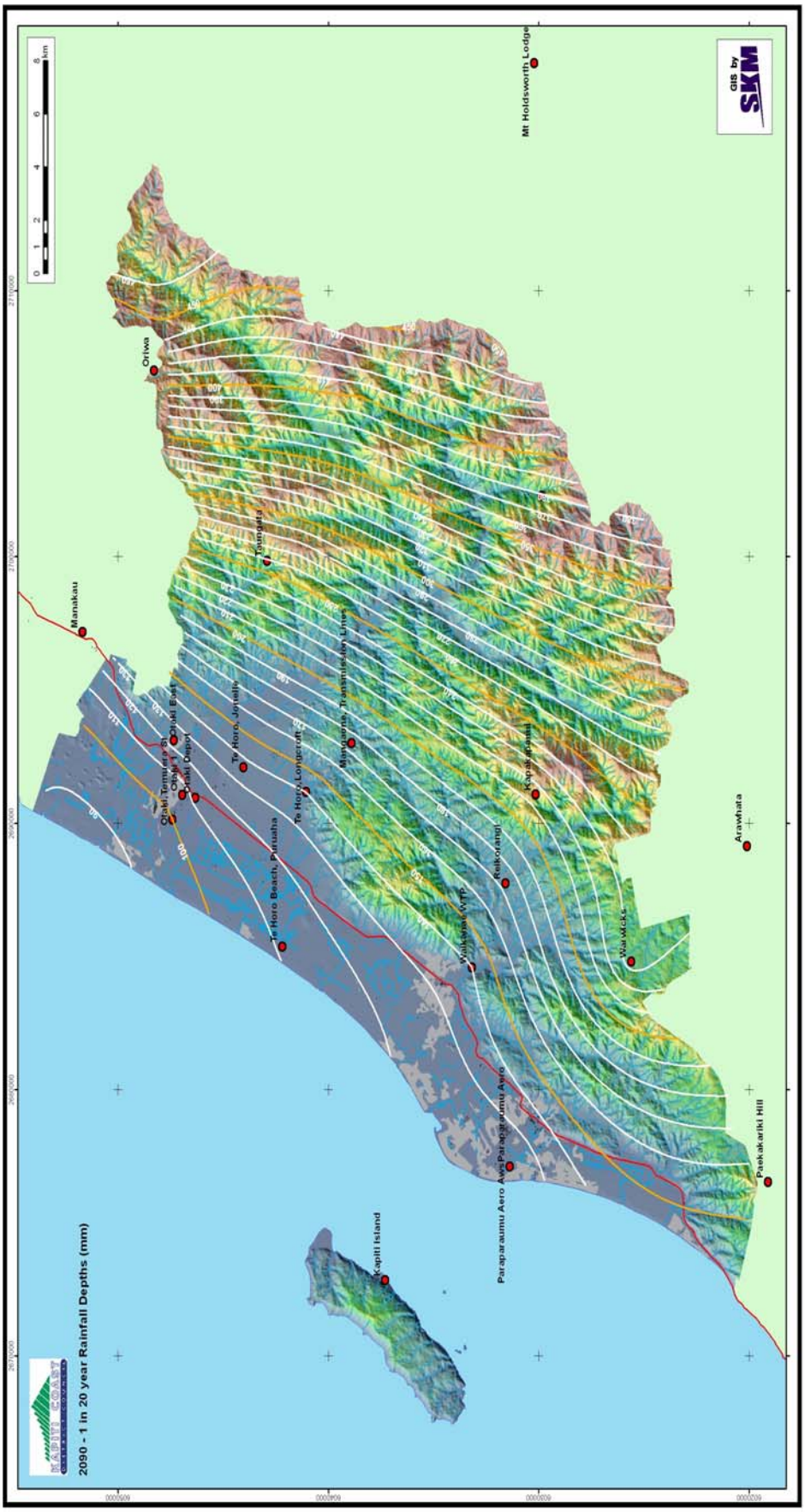

Current - 1 in 50 year Rainfall Depths (mm)
GIS by SKM
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McIntosh
Tolangata
Manakau
Mangaone Transmission Line
Te Horo, Jomelle
Te Horo, Longcroft
Te Horo Beach, Poruaha
Kapakapanui
Raikonangi
Aravhata
Warwicks
Waikanae WTP
Paraparaumu Aero Aws
Paraparaumu Aero
Paekakariki Hill
Kapiti Island
Oraki Termiera St
Oraki East
Oraki 1
Oraki Depot
KAPITI COAST

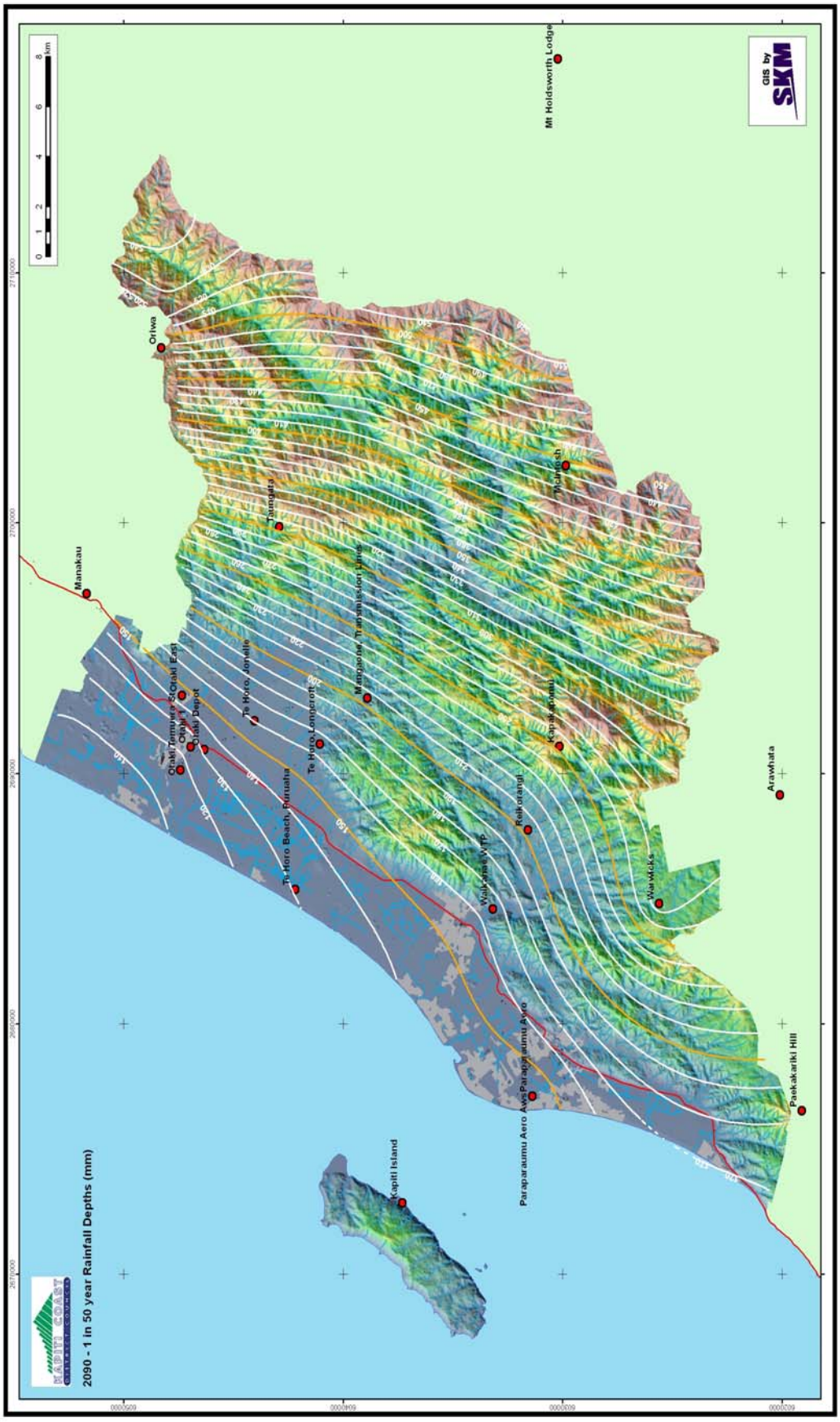

Current - 1 in 100 year Rainfall Depths (mm)
GIS by SKM
KAPITI COAST DISTRICT COUNCIL
Mt Holdsworth Lodge
Oriwa
Manakau
Tauhara
McIntosh
Te Horo, Jomelle
Te Horo, Longcroft
Te Horo Beach, Putuniha
Otaki Tennent St
Otaki East
Otaki 1
Otaki Depot
Mangaone, Transmission Lines
Kapakapanui
Raikotanga
Waikanae WTP
Warwicks
Arawhata
Paraparaumu Aero Aws
Paraparaumu Aero
Kapiti Island
Paekakariki Hill

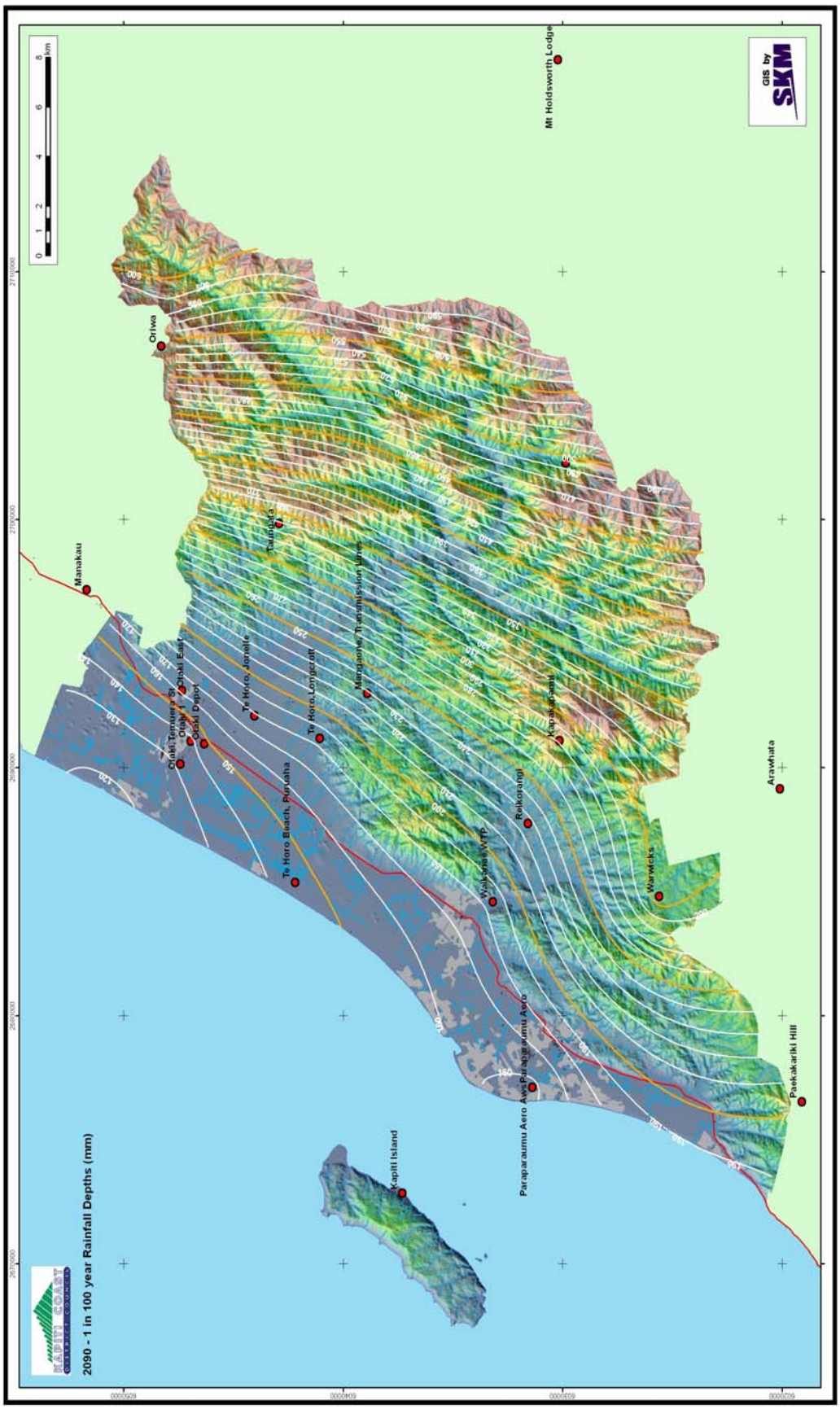

Mt Holdsworth Lodge
Oriwa
Manakau
Otaki East
Otaki Depot
Te Horo Beach Road
Te Horo Beach, Pukuaha
Te Horo, Longcroft
Mangaone Transmission Lines
Kapakapanui
Reikotangi
Waikanae WTP
Waikanae
Paraparaumu Aero
Kapiti Island
Paekakariki Hill
Arawhata
GIS by SKM
2090 - 1 in 2 year Rainfall Depths (mm)


Mt Holdsworth Lodge
Orlwa
McIntosh
Tauhara
Manakau
Te Horo Jonelle
Te Horo Longcroft
Mangaone Transmission Lines
Kapakapanui
Aorangi
Arawhata
Te Horo Beach, Rotuaha
Waikanae WTP
Waikanae
Warwicks
Paraparaumu Aero
Paekakariki Hill
Kapiti Island
2090 - 1 in 5 year Rainfall Depths (mm)


Mt Holdsworth Lodge
Oriwa
Mangaroa
Tolangata
Manakau
Otaki Tramway Otaki East
Otaki 4
Otaki Depot
Te Horo, Jonelle
Te Horo, Longsloth
Mangaone Transmission Lines
Kapakapanui
Arawhata
Reikorangi
Te Horo Beach, Pukuaha
Waikanae WTP
Waiwiri
Kapiti Island
Paraparaumu Aero
Paraparaumu Aero Aws
Paekakariki Hill
GIS by SKM
2090 - 1 in 10 year Rainfall Depths (mm)


2090 - 1 in 20 year Rainfall Depths (mm)
Mt Holdsworth Lodge
Oriwa
Manakau
Tongariro
Te Horo, Jintelle
Te Horo, Longcroft
Te Horo Beach, Pukuaha
Kapakapanui
Kelvinside
Waiwiri
Arawhata
Paekakariki Hill
Paraparaumu Aero
Kapiti Island
GIS by SKM


2090 - 1 in 50 year Rainfall Depths (mm)
Mt Holdsworth Lodge
Otaki
Tararua
Manakau
Waikanae WTP
Paraparaumu Aero
Kapiti Island
Paekakariki Hill
Arawhata
Waimeka
Reikorangi
Te Horo Beach Rurualia
Te Horo Longcroft
Te Horo Jonelle
Otaki Depot
Otaki 1
Otaki Tamure Otaki East
Matapoua Transmission Lines
Kapakapanui
Tauhara
Mangaroa
Oriwa


Mt Holdsworth Lodge
Oriwa
Manakau
Tolaga
Te Horo, Jorelle
Te Horo, Longcroft
Te Horo Beach, Ponaha
Otaki Tarmac
Otaki East
Otaki Depot
Otaki WWTP
Waikanae WTP
Paraparaumu Aero Aws
Paraparaumu Aero
Kapiti Island
Raikorafapi
Warwicks
Arawhata
Paekakariki Hill
2090 - 1 in 100 year Rainfall Depths (mm)

Appendix B Curve Number Delineation Tables

(Sourced from USACE, 2000)

Table 2-2a – Runoff curve numbers for urban areas[1] (SCS, 1986)

Cover Description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	**Average percent impervious area[2]**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, gold courses, cemeteries etc)[3]					
Poor condition (Grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c)					

[1] Average runoff condition, and Ia = 0.2S.

[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: Impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 and 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2b – Runoff curve numbers cultivated agricultural lands[7] (SCS, 1986)

Cover Description			Curve numbers for hydrologic soil group			
Cover type	**Treatment[8]**	**Hydrologic condition[9]**	**A**	**B**	**C**	**D**
Fallow	Bare soil		77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or	SR	Poor	66	77	85	89
Broadcast		Good	58	72	81	85
Legumes or	C	Poor	64	75	83	85
Rotation		Good	55	69	78	83
Meadow	C&T	Poor	63	73	80	83
		Good	51	67	76	80

[7] Average runoff condition, and Ia = 0.2S.

Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

[9] Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes in rotations, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tender to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Table 2-2c – Runoff curve numbers for other agricultural lands[10] (SCS, 1986)

Cover Description		Curve numbers for hydrologic soil group			
Cover Type	**Hydrologic Condition**	**A**	**B**	**C**	**D**
Pasture, grassland, or range-continuous forage for Grazing.[11]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
1) Meadow-continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush-brush-weed-grass mixture with brush the major element[12]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	[13]30	48	65	73
Woods-grass combination (orchard or tree farm)[14]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods.[15]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	[4]30	55	70	77
Farmsteads-buildings, lanes, driveways, and surrounding lot.	.....	59	74	82	86

[10] Average runoff condition, and Ia = 0.2S.

[11] *Poor: 5* 0% ground cover or heavily grazed with no mulch
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.

[12] *Poor:* 50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

[13] Actual curve number is less than 30; use CN = 30 for runoff computations.

[14] CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture

[15] *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are graced but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil

SCS TR-55 Table 2-2d – Runoff curve numbers for arid and semi-arid rangelands[16]

Cover Description		Curve numbers for hydrologic soil group			
Cover Type	**Hydrologic Condition[17]**	**A[18]**	**B**	**C**	**D**
Herbaceous – mixture of grass, weeds and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen – mountain brush mixture of oak brush, 2) aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper – pinyon, juniper or both; grass understorey	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub – major plants include saltbrush, Greasewood, creosotebush, black brush, bursage, Palo verde, mesquite and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

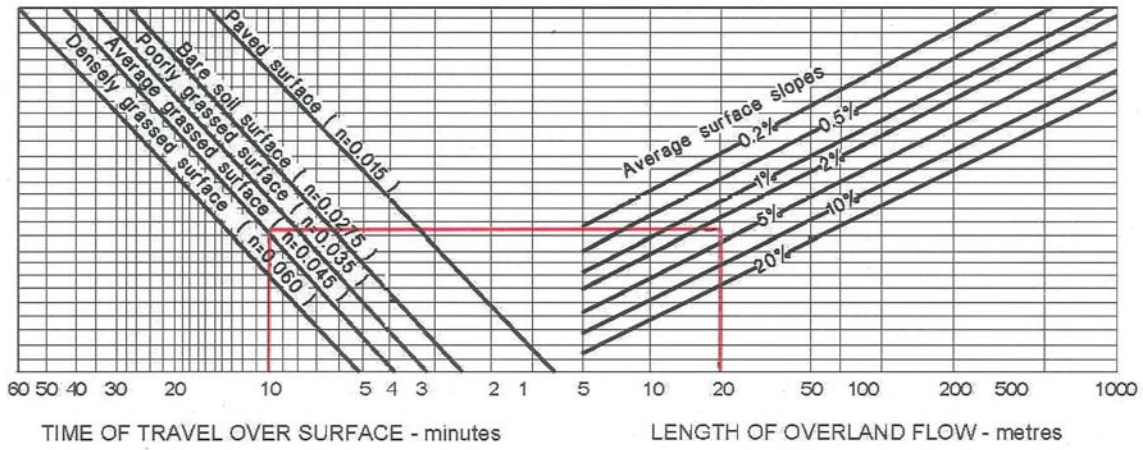
[16] Average runoff condition, and $I_a = 0.2S$.

[17] *Poor :* <30% ground cover (litter, grass and brush overstory).
Fair: 30 to 70% ground cover
Good: >705 ground cover

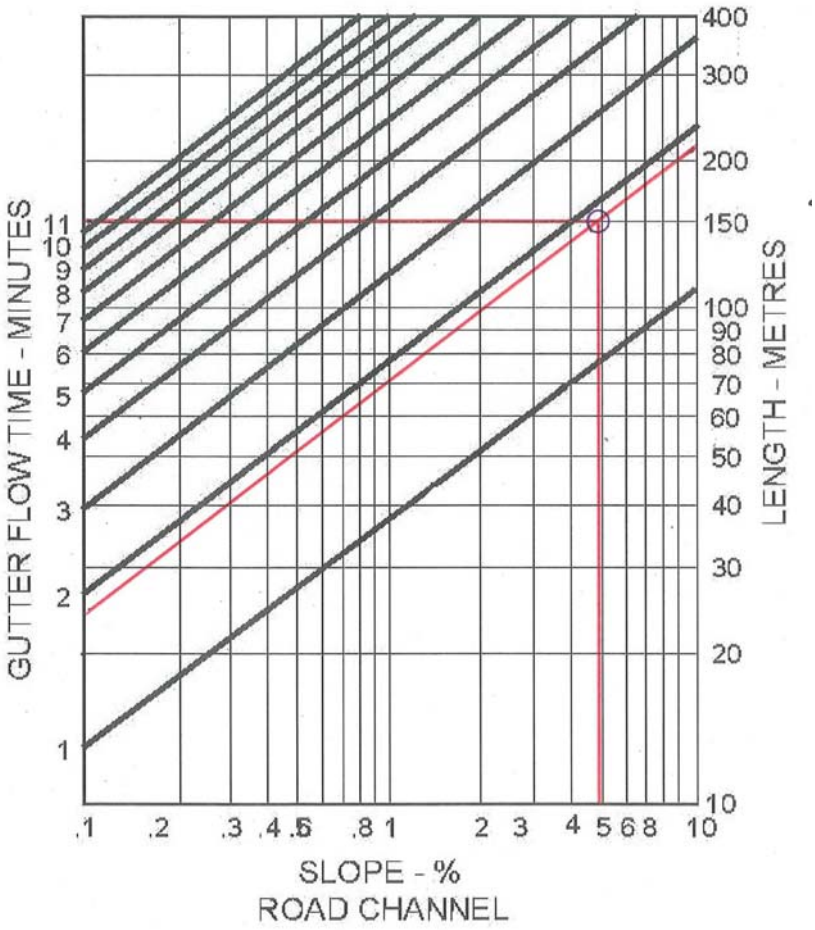
[18] Curve numbers for group A have been developed only for desert shrub

Appendix C Time of Concentration Calculations

Overland Flow Calculation



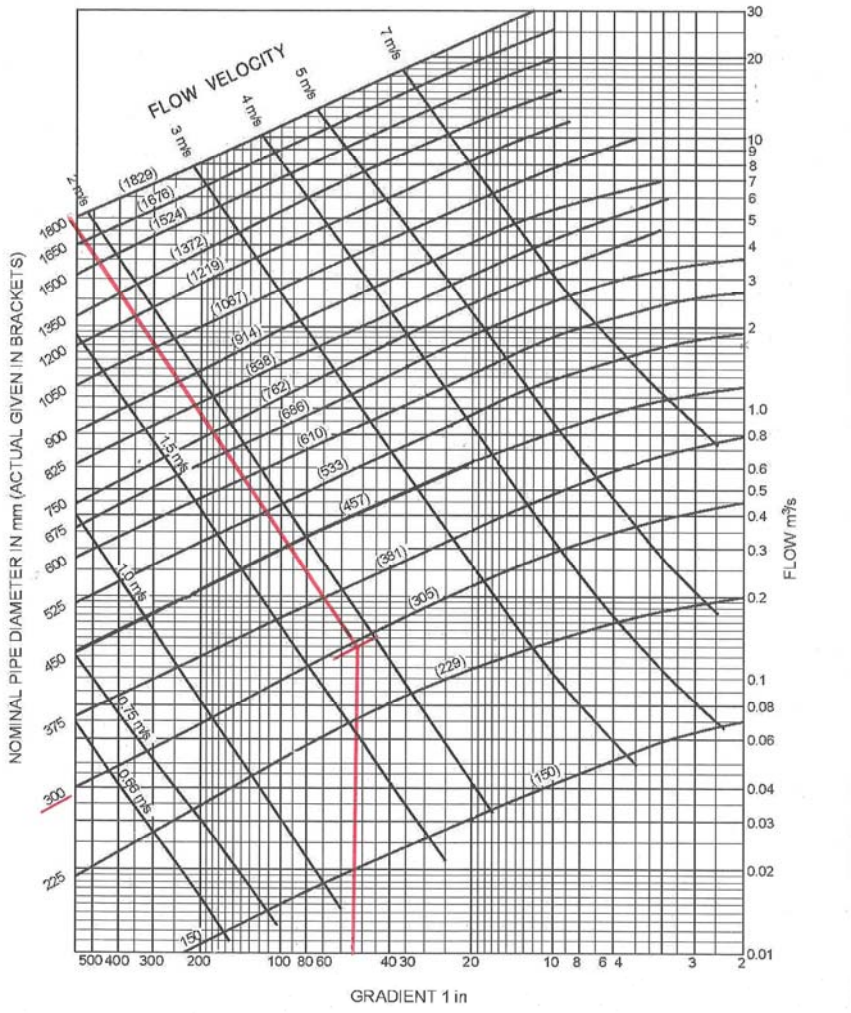
Source: Compliance Document for
New Zealand Building Code
Clause E1
Surface Water



Source: Compliance Document for
New Zealand Building Code
Clause E1
Surface Water

Pipe Flow Calculation

(note: chart relates to concrete pipes)



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Clause E1
Surface Water

APPENDIX F Groundwater Calculations

YEAR HORIZON	CLIMATE CHANGE SCENARIO	GROUNDWATER TABLE SURFACE (HIGHEST VALUE AT 50TH PERCENTILE)	CALCULATIONS
2020	NA	0.248 mWVD53 (GWL 2020)	GWL_2020 = T&T_GWL_2030 - 0.042 = 0.248mWVD53
2030	NA	0.298 mWVD53 (GWL 2030)	T&T_GWL_2030 levels: GWL_2030 = T&T_GWL_2030
2070	SSP5-8.5M	0.628 mWVD53 (GWL 2070)	GWL_2070 = 0.7 * T&T_GWL_2030 + 0.3 * (0.74 * T&T_GWL_2130_RCP8.5M + 0.26 * T&T_GWL_2130_RCP8.5H+) = 0.7 * T&T_GWL_2030 + 0.222 * T&T_GWL_2130_RCP8.5M + 0.078 * T&T_GWL_2130_RCP8.5H+
2130	SSP5-8.5M	1.408 mWVD53 (GWL 2130)	GWL_2130 = T&T_GWL_2030 + (T&T_GWL_2130 - T&T_GWL_2030) * (1.408 - 0.298) / (1.32 – 0.298)
2130	SSP5-8.5H+	1.858 mWVD53 (GWL 2130H+)	GWL_2130H+ =T&T_GWL_2130_RCP8.5M + (T&T_GWL_2130_RCP8.5H+ - T&T_GWL_2130_RCP8.5M) * (1.858 - 1.32) / (1.66 - 1.32)

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APPENDIX G Joint Probability Report

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Kāpiti Coast District Council Flood and Stormwater Study

Joint probability analysis of coastal and catchment flooding

Version 1

May 2024

Document status

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Project manager	Phillip Jordan
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Project number	AWA00004

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Draft B	12/02/2024	Philip Jordan	Phillip Jordan	Awa Environmental	Addition of results for Mazengarb, Ōtaki and Wharemauku catchments
Version 1	31/05/2024	Philip Jordan	Phillip Jordan	Awa Environmental	Addition of joint probability levels results for Mazengarb and Wharemauku catchments.

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1. Introduction

Jacobs (2021) produced estimated inundation extents along the Kāpiti Coast. These estimates considered the combined influences of storm surge, astronomically driven tides, wave set up and projected sea level rise under climate change and continued land subsidence. The Jacobs (2021) projections of storm surge, tide and wave set up drew upon previous investigations by Met Ocean Solutions (2007) and NIWA (2012). Jacobs (2021) identified that the Met Ocean Solutions (2007) and NIWA (2012) studies produced relatively consistent estimates of the AEP design storm tide level for locations along the Kāpiti Coast, once appropriate corrections had been made for datums and sea level rise.

The Jacobs (2021) investigation assumed 'bath-tub' modelling of inundation and impacts, using the peak water level at the coast, for a given scenario, and then assuming all land below that horizontal plane would be inundated. Design water levels associated with storm tide (storm surge and tides) or the joint probability of storm tide and significant wave heights on the open coast are well-defined for the Kāpiti Coast. However, the 'bath tub' approach used by Jacobs (2021) does not provide a good representation of flood inundation for areas that are subject to the joint influences of coastal and fluvial flooding. Flood levels surrounding these estuaries and streams will be affected by the joint probability of inundation from storm tides and flooding generated from runoff in the catchments that drain to them.

Analyses in Australia have demonstrated that the probability of a heavy rainfall event occurring on catchment near the coastline, in conjunction with elevated storm tide levels, is much higher than would be the case if those two phenomena were assumed to be independent from one another. A case study for the Hawkesbury-Nepean catchment, to the north of Sydney, Australia, demonstrated that the joint probability between storm tide levels and riverine flooding had a significant influence on design flood levels for a significant portion of the estuary.

Floods in estuarine areas can be caused by runoff generated by an extreme rainfall event, an elevated ocean level generated by a storm surge and/or a high astronomical tide, or a combination of both processes occurring simultaneously or in close succession. Research in Australia (Zheng *et al.*, 2013, 2014; Westra *et al.*, 2019) and in the United Kingdom (Svensson and Jones, 2002, 2004; Hawkes and Svensson, 2003) has shown that extreme rainfall and storm surge processes are statistically dependent, and therefore their interaction needs to be considered for areas affected by both processes.

Flood levels in catchments along the Kāpiti Coast are likely to be influenced by the joint dependence between coastal and catchment driven flooding. Conceptually, this is shown by the illustration in Figure 1-1. There will be a region of each catchment where possible flood levels are enclosed in an envelope bounded by cases where flood events and ocean levels are completely dependent (upper curve) and completely independent (lower curve) (Westra *et al.*, 2019).

The dependence strength between extreme rainfall and storm surge in Australia was found to vary as a function of geographic location and the duration of the rainfall burst (Zheng et al., 2013; Zheng et al., 2014a). It is reasonable to expect that the dependence strength will also vary with rainfall burst duration for the Kāpiti Coast. Given the relatively strong influence of orographic rainfall on flooding from catchments draining from the Tararua Range to the Kāpiti Coast and the general movement of many weather systems from the Tasman Sea and South Taranaki Bight over the region, it would be

reasonable to expect that joint dependence could be stronger for the Kāpiti Coast region than is generally observed in Australia.

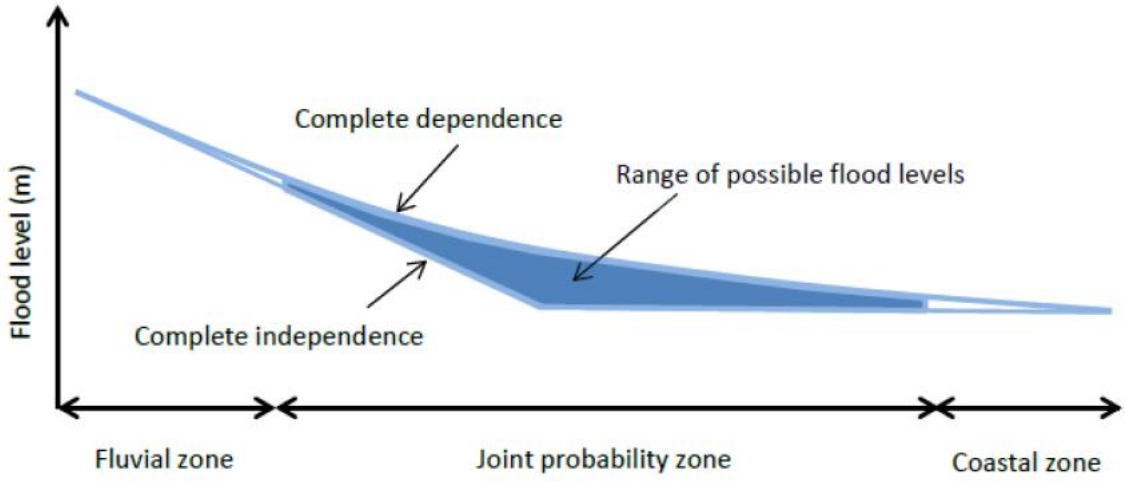


Figure 1-1: Schematic of a longitudinal section of an estuary, which shows two hypothetical water levels: the level obtained by assuming that fluvial floods will always coincide with storm tides of the same exceedance probability (upper curve); and the level assuming fluvial processes and ocean processes are completely independent and thus will almost never coincide (lower curve) (reproduced from Figure 6.5.1 of Westra et al., 2019)

The purpose of this study is to produce combinations of storm tide levels along the Kāpiti Coast that can be applied in conjunction with design rainfall and flood events, to produce peak design water level estimates along the estuaries of the study area that have defined AEP. This requires analysis of the joint probability of storm tide levels along the Kāpiti Coast and intense rainfall events in the catchments that drain to the Kāpiti Coast.

This report describes the methodology adopted to calculate the joint probability levels at Ōtaki and Waitohu catchments in the Kāpiti Coast and presents the peak design level estimated with the adopted methodology at several locations relevant to the Ōtaki and Mazengarb Stream.

2. Calculation of joint dependence parameter for rainfall and storm surge

A method for estimating the joint dependence of coastal and catchment flooding is set out in Australian Rainfall and Runoff (Westra *et al.*, 2019). The approach adopted therein was developed in (Zheng *et al.*, 2014). They found that the point-process method was the most suitable approach for modelling coastal and catchment flooding and that the most appropriate distribution for characterising the joint dependence between them was the logistic probability distribution.

For Kāpiti Coast, the logistic model was also chosen as the parametric model adopted to characterize the bivariate extreme value distribution, where α measures the dependence between two random variables at extreme levels. Complete dependence is represented by $\alpha \to 0$, and $\alpha = 1$ represents complete independence.

$$G(x,y)=\exp\left\{-(x^{-1/\alpha}+y^{-1/\alpha})^{\alpha}\right\} \quad 0<\alpha\leq 1, x>0, y>0$$

The joint dependence parameter for rainfall and storm surge was calculated using the Point Process Method as per Zheng (2014). In the Point Process Method, the full dataset must be transformed to have standard Fréchet margins by applying the following set of equations:

$$\tilde{z}=\begin{cases} -\left(\log\left\{1-\hat{\zeta}_{u_z}\left[1+\frac{\hat{\xi}_z(z-u_z)}{\hat{\sigma}_z}\right]^{-1/\hat{\xi}_z}\right\}\right)^{-1}, & z>u_z, \hat{\xi}_z\neq 0 \\ -\left(\log\left\{1-\hat{\zeta}_{u_z}\exp\left(-\frac{z-u_z}{\hat{\sigma}_z}\right)\right\}\right)^{-1}, & z>u_z, \hat{\xi}_z=0 \\ -\{\log\hat{F}(z_i)\}^{-1}, & z\leq u_z \end{cases}$$

Where σ is the scale, and ξ the shape parameter obtained from a Generalised Pareto Distribution fitted to the largest daily values in each of the datasets. The threshold adopted to filter the extreme event is represented by u_z,(eg. 1% highest values). $\hat{F}(z_i)=i/(n+1)$ is the empirical distribution of z, i is the rank of z_i , and n is the total number of data points.

Using the radial (r) and the angular (w) components of the Fréchet transformed data, a joint threshold (r_0) is selected as the lowest value such as the histogram of $w|r>r_0$ remains constant as r increases. The dependence parameter is then calculated for the selected data.

The steps underaken for application of the Point Process Method to determine the depencence parameter for the Ōtaki and Mazengarb catchments region are described below.

2.1 Preparation of rainfall and storm surge input data

The astronomical tide series was obtained from the NIWA Tide Forecaster at Porirua Harbor from 09/02/2009 to 13/11/2023. The recorded tide values were obtained from a tide gauge in Porirua Harbour at Mana Cruising Club, operated by the Greater Wellington Regional Council (GWRC) from

09/02/2009 to date (just over 14 years). The storm surge series was then calculated by subtracting the maximum daily astronomical tide from the maximum daily recorded tide values.

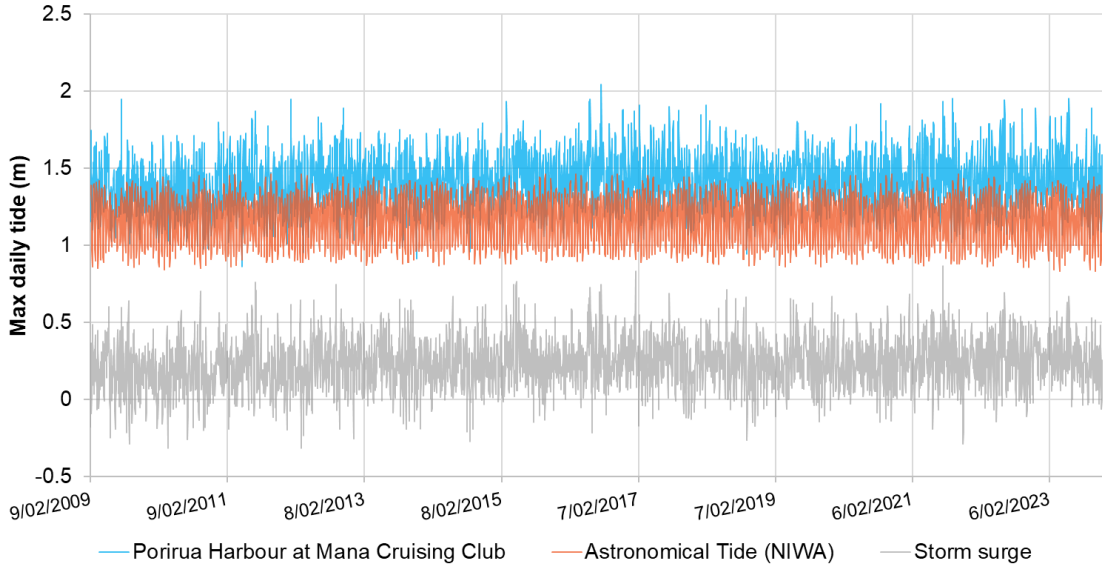


Figure 2-1: Storm surge, recorded, and astronomical tides series adopted for Ōtaki and Mazengarb catchments shore

The maximum daily rainfall series of 1h, 2h, 6h, 12h and 24h duration was calculated from the recorded rainfall data from Stream at Tawa Pool gauge for the period between 09/02/2009 and 13/11/2023.

2.2 Calculation of the Fréchet transformed series

The Fréchet trasformed series were calculed with an R script using the function fbvpot from the extRemes library. The function was applied for the 1% highest values of rainfall and storm surge pairs, for each rainfall durantion. Figure 2-2 shows plots of the original data, Fréchet trasformed data, and histogram of the angular components of the Fréchet trasformed data for each rainfall duration.

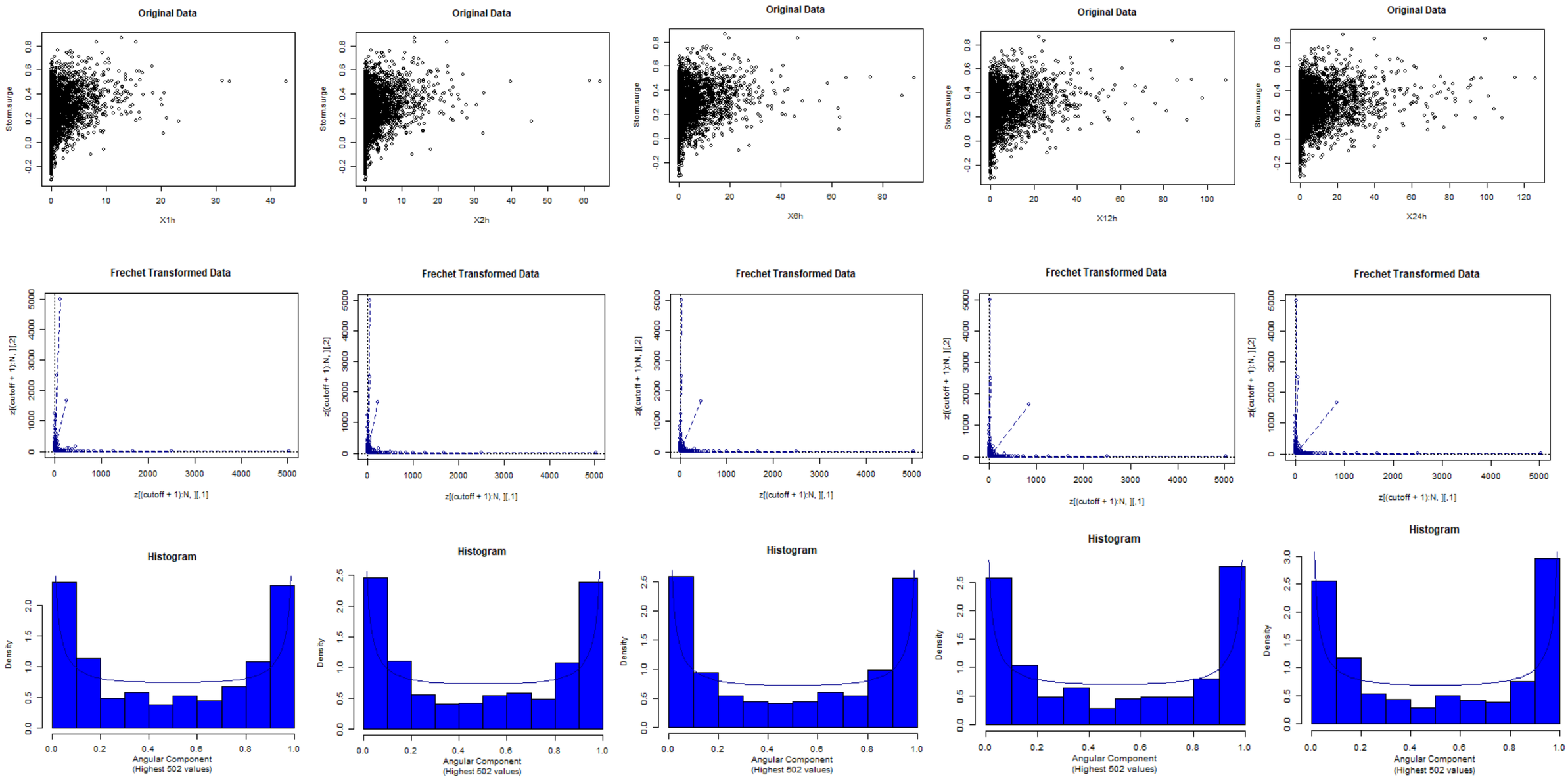


Figure 2-2: Storm surge versus rainfall (top), Fréchet transformed data (middle), Fréchet angular component histogram (bottom), for 1h, 2h, 6h, 12h and 24h rainfall (left to right)

2.3 Determination of the critical radius (r_0)

The intent of the analysis of joint dependence is to concentrate the analysis on events that are sufficiently rare (in terms of storm surge, rainfall intensity or a combination of both), to provide an understanding of the joint dependence of large events. Calculation of the dependence parameter, α, involves a trade-off between setting a threshold that is sufficiently large to limit the analysis to events that are sufficiently rare to provide an indication of how design floods will behave but sufficiently low to provide a suitable number of events to obtain a robust estimate of the parameter of the bi-logistic distribution.

Mathematically, the threshold used to analyse joint dependence is the critical radius, r_0, which is the minimum value of Fréchet transformed values used to define joint dependence. Reducing the r_0 value results in more events above the threshold used to analyse joint dependence (which provides more individual events to produce a value of α with less uncertainty) but increasing the r_0 value limits the analysis to rarer events, which are more relevant to design flood estimation. To determine the critical radius for all the rainfall durantion and storm surge paried series, the histogram of $w|r>r_0$ was plotted for 81 r_0 values determined by the series of values between 10% and 90% exceedance radial values quantiles on a 1% interval. It was then observed that the angular component histogram obtained for all the r_0 values such as $w|r>r_0$ presented similar shape. This indicates that the joint dependence relationship does not change appreciably with the value of the critical radius across the relatively wide range of r_0 values that were investigated.

Figure 2-3 presents the dependece parameter obtained for each differented critical radius adopted. The curves shows that rainfall events are generally weekly dependent from storm surge events with values ranging from 0.69 to 0.78. It is also noticed that rainfall events of longer duration are slightly less dependent with storm surge than the shorter durantion ones.

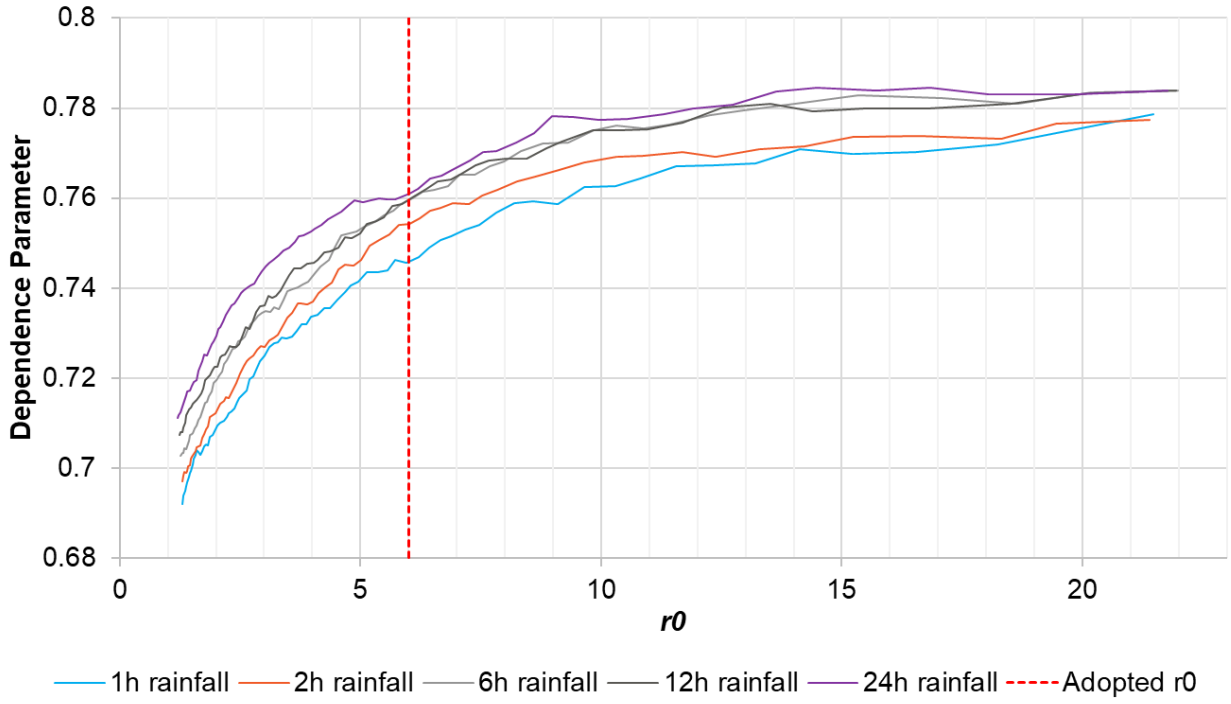


Figure 2-3: Series of dependence parameters calculated for different critical radius for each rainfall duration

For the purposes of this study, given that the angular component histograms obtained for all the r_0 values such as $w|r>r_0$ presented similar shape, a critical radius of 6 was deemed suitable. The dependence parameter values obtained for this threshold are 0.75, for 1h and 2h duration, and 0.76, for the remining durations (Figure 2-5).

The joint dependence parameter values for the Kāpiti Coast, of 0.75 and 0.76, were lower than the equivalent joint dependence parameter values that are adopted in Australia for rainfall event durations of less than 24 hours. Figure 2-4 shows that for rainfall event durations of 12 hours and shorter, α values for the Australian coastline range between 0.90 and 0.98. The joint dependence between coastal and catchment flooding is stronger along the Kāpiti Coast than for any part of the Australian coastline. This would be an expected outcome, as low pressure systems in the Tasman Sea and the Taranaki Bight with winds causing storm surge along the Kāpiti Coast would also have a reasonably high likelihood of causing intense rainfall in the catchments draining from the North-Western side of the Tararua Range, when moving towards and over the land. There is generally lower relief along most of the Australian coastline and lower probability of joint occurrence of coastal and catchment flooding than along the Kāpiti Coast.

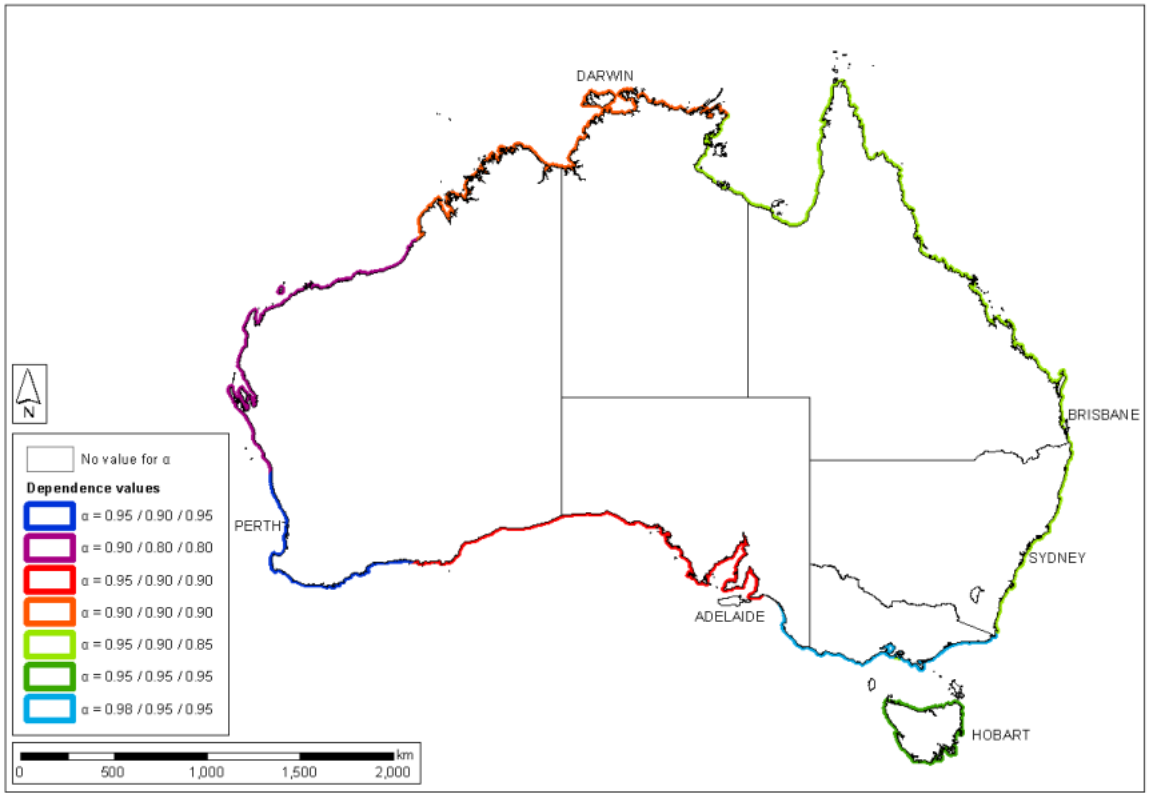


Figure 2-4 Joint dependence parameter, α, map for interaction of coastal and catchment flooding adopted for design flood estimation in Australia (Australian Rainfall and Runoff, Figure 6.5.13, Westra et al., 2019). For each section of coastline, the three values provided are for rainfall event durations of shorter than 12 hours, 12 to 48 hours and longer than 48 hours.

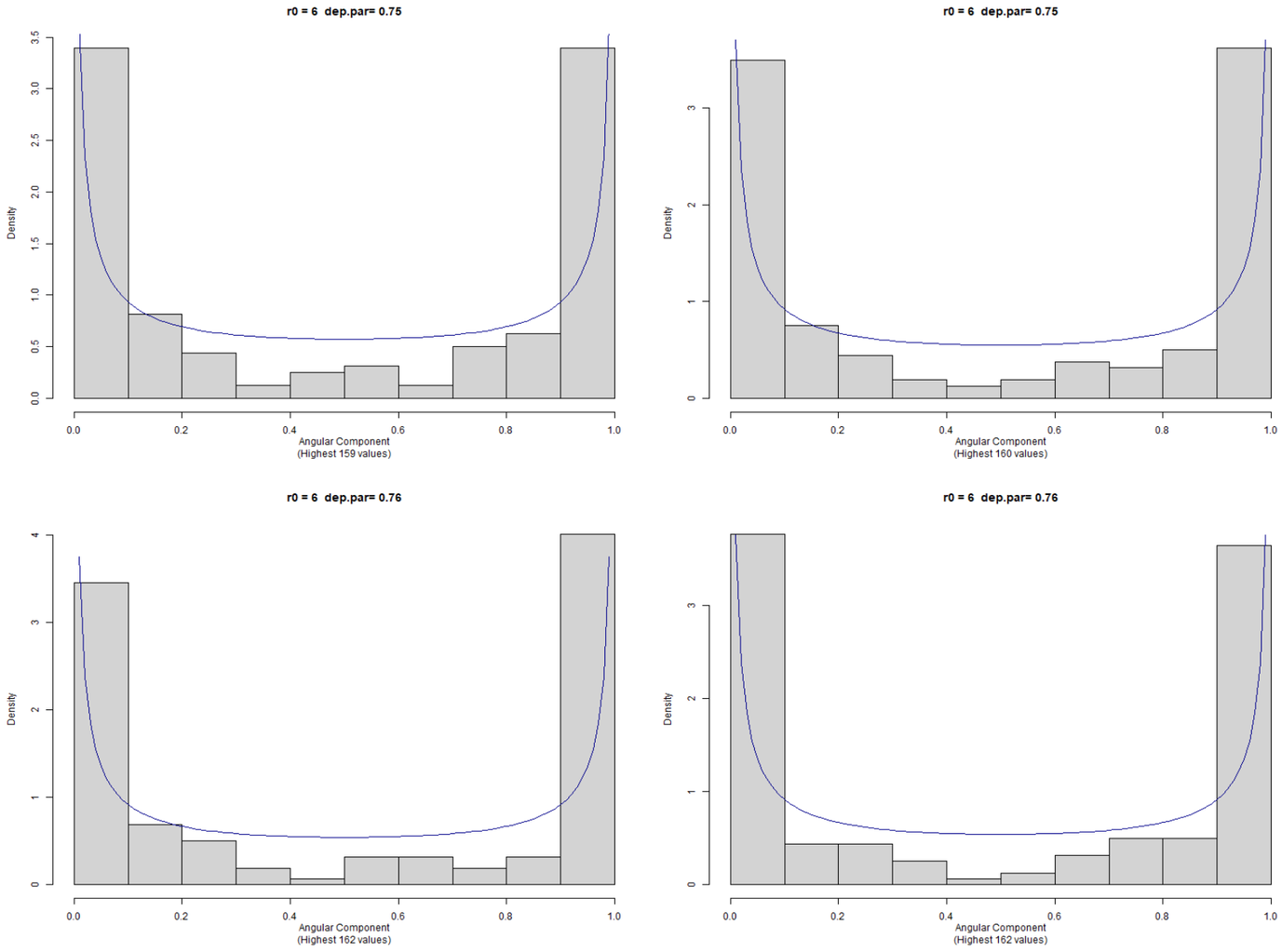


Figure 2-5: Angular component histograms for 1h, 2h, 6h, 12h and 24h rainfall duration (left to right, top to bottom)

3. Application of the joint dependence parameter

3.1 Calculation of joint dependency levels at different locations within the catchments

The application of the joint dependence parameter consisted of the following steps:

1. Selection of key locations along the main flow path(s) within the catchment
2. Extraction of the levels from 36 hydraulic model outputs at each location
3. Calculation of the joint probability levels for a range of AEP flood events (≥1% and ≤100%) by applying the method developed in ARR Project 18 for Joint Probability Modelling in Estuarine Regions (ARR, 2023)
4. Calculation of the joint dependence AEPs and tidal levels for 50%, 20%, 10%, 2% and 1% AEP rainfall events for each catchment

AWA Environmental provided HARC with a set of 36 hydraulic model outputs for Wharemauku, Ōtaki and Mazengarb catchments, composed of the combination of the rainfall and storm surge scenarios such as it spans to the most extreme scenario levels.

The hydraulic runs results for each scenario were then extracted at different locations along the main flow path within each catchment. The locations are shown in Figure 3-1 for Wharemauku, Figure 3-2 for Mazengarb, and Figure 3-3 for Ōtaki catchment.



Figure 3-1: Locations chosen for application of the joint probability analysis in Wharemauku catchment



Figure 3-2: Locations chosen for application of the joint probability analysis in Mazengarb catchment



Figure 3-3: Locations chosen for application of the joint probability analysis in Ōtaki catchment

The probability levels for different AEP flood events were calculated at each of those locations by applying the method developed in ARR Project 18 for Joint Probability Modelling in Estuarine Regions (ARR, 2023), using the dependence parameter of 0.75 (Section 2).

The tidal AEP and levels were then calculated for climate change projection scenarios SSP5-8.5M 2030 and 2013 (m WVD53) for 50%, 20%, 10%, 2% and 1% AEP rainfall events. Equivalent tide levels are provided in Table 3-1 for the Wharemauku catchment, Table 3-2 for the Mazengarb catchment and Table 3-3 for the Ōtaki River catchment.

Table 3-1 Tide levels (m) that provide for equivalent design flood event peak levels in the Wharemauku catchment for 1%, 2%, 10%, and 50% rainfall AEPs

Climate Change Scenario	Rainfall AEP (%)	Tidal AEP (%)	Tide Level (m)
SSP5-8.5M 2130	**1%**	30%	3.03
	2%	40%	2.99
	10%	50%	2.97
	20%	75%	2.91
	50%	75%	2.91
SSP5-8.5M 2030	**1%**	10%	2.03
	2%	10%	2.03
	10%	50%	1.86
	20%	50%	1.86
	50%	50%	1.86

Table 3-2 Tide levels (m) that provide for equivalent design flood event peak levels in the Mazengarb catchment for 1%, 2%, 10%, and 50% rainfall AEPs

Climate Change Scenario	Rainfall AEP (%)	Tidal AEP (%)	Tide Level (m)
SSP5-8.5M 2130	**1%**	1%	3.47
	2%	2%	3.42
	10%	15%	3.28
	20%	30%	3.21
	50%	60%	3.13
SSP5-8.5M 2030	**1%**	1.0%	2.36
	2%	2.0%	2.31
	10%	10%	2.21
	20%	25%	2.12
	50%	50%	2.04

Table 3-3 Tide levels (m) that provide for equivalent design flood event peak levels in the Ōtaki River catchment for 1%, 2%, 10%, and 50% rainfall AEPs

Climate Change Scenario	Rainfall AEP (%)	Tidal AEP (%)	Tide Level (m)
SSP5-8.5M 2130	**1%**	5%	3.49
	2%	10%	3.43
	10%	25%	3.34
	20%	40%	3.29
	50%	60%	3.25
SSP5-8.5M 2030	**1%**	5%	2.37
	2%	10%	2.32
	10%	15%	2.28
	20%	40%	2.18
	50%	80%	2.09

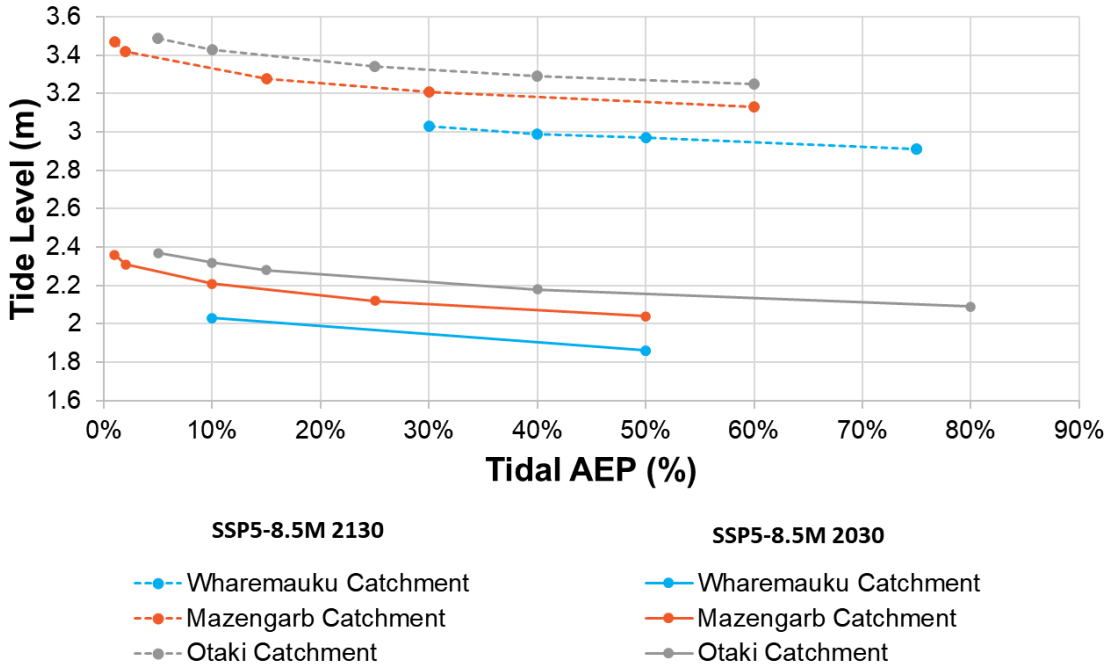


Figure 3-4: Tidal levels and AEPs plot

3.2 Calculation of joint dependency levels within the catchments on a grid basis

A series of scripts was developed so that the approach described in session 3.1 could be applied such as a joint probability level could be estimated at each grid cell within each of the catchments. In a nutshell, the scripts performed the following tasks:

1. Extraction of the levels from 36 hydraulic model outputs at each grid cell
2. Preparation of the input as per the requirements for executing the ARR R function (ARR, 2023)
3. Execution of the Joint Probability calculation using the ARR R script (ARR, 2023)

4. Creation of the output grids containing the water levels calculated in step 3 for 50%, 20%, 10%, 2% and 1% AEP, for the 2030 and 2130 SSP5-8.5M climate change scenarios.

Steps 1 to 4 generated the flood levels within the joint probability zone within each catchment (refer to Figure 1-1). To estimate equivalent flood levels within the coastal and fluvial zones of each catchment, water levels were interpolated from the 1% AEP, 2130, SSP5-8.5H+ downstream tidal scenarios grids provided by AWA. This was undertaken with the use of a different R script prepared for this task. The results were then merged with the extents previously obtained by steps 1 to 4, and a final water level flood extent was created to each AEP for the 2030 and 2130 climate change scenarios.

The flood depths obtained for the 100 years ARI, 2130 climate change projection for Mazengarb and Wharemauku catchments are shown in Figure 3-5, Figure 3-6 and Figure 3-7.

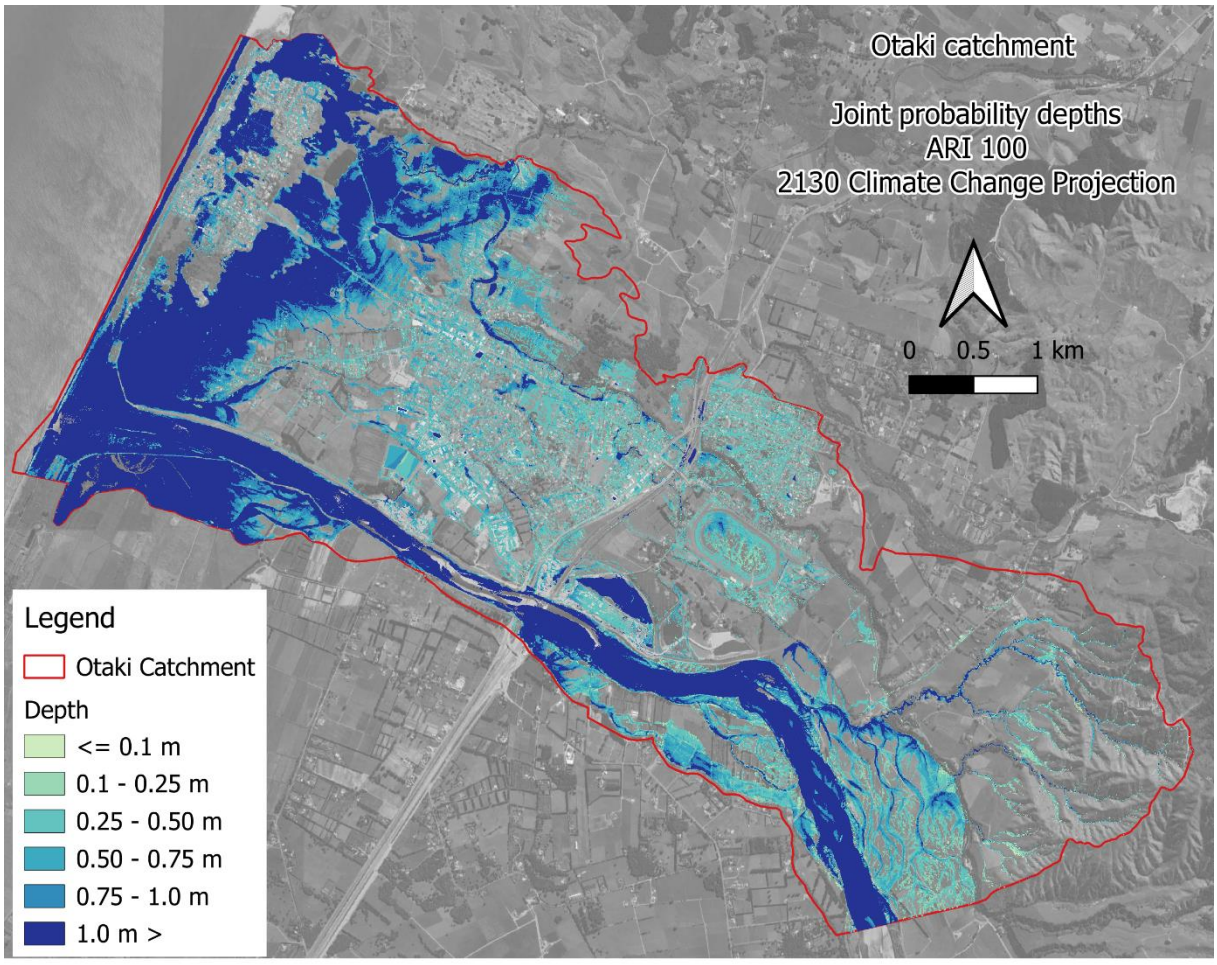


Figure 3-5: Otaki catchment joint probability depths for ARI 100 2130 RCP 8.5 M climate change projection

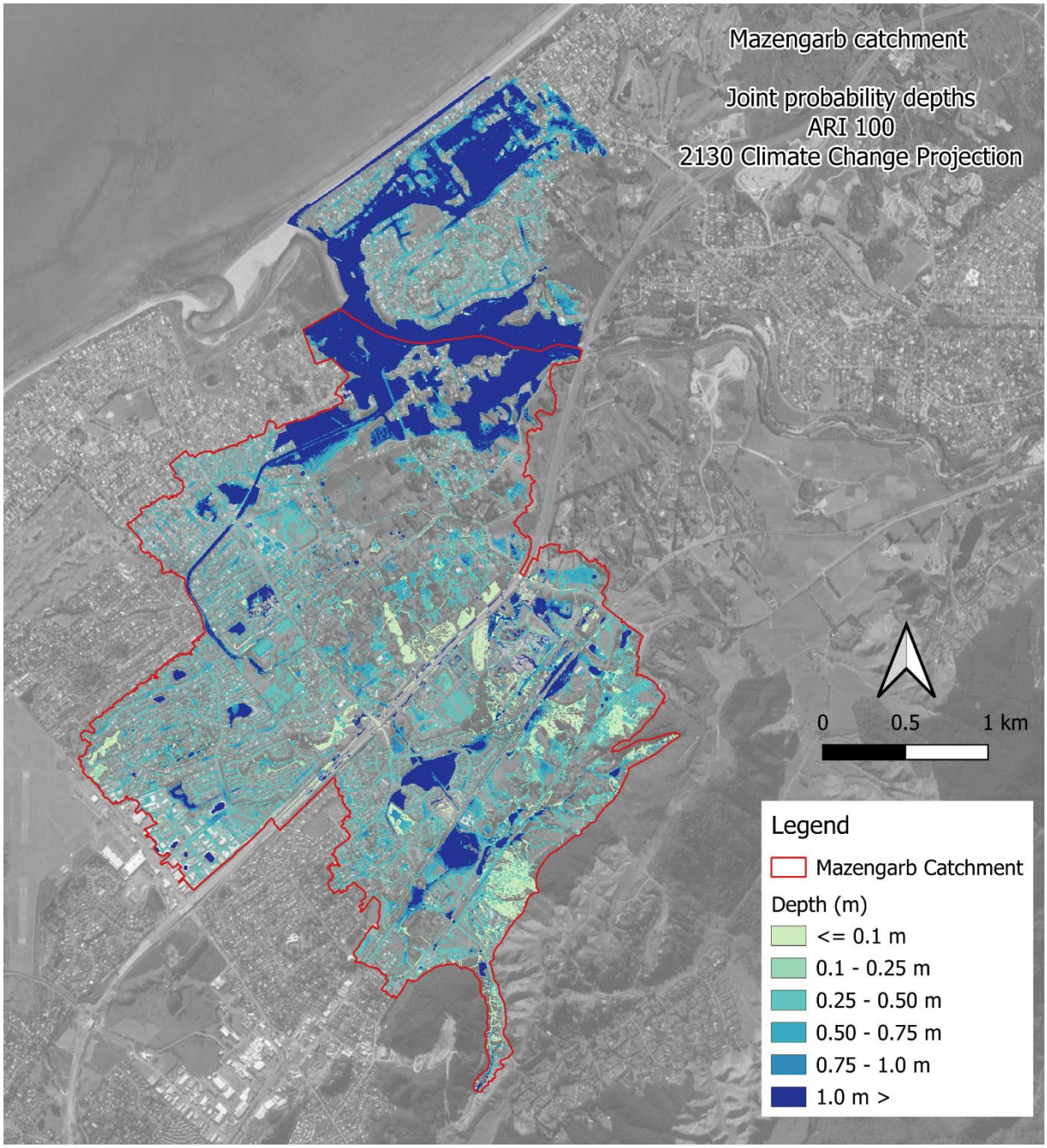


Figure 3-6: Mazengarb catchment joint probability depths for ARI 100 2130 RCP 8.5 M climate change projection

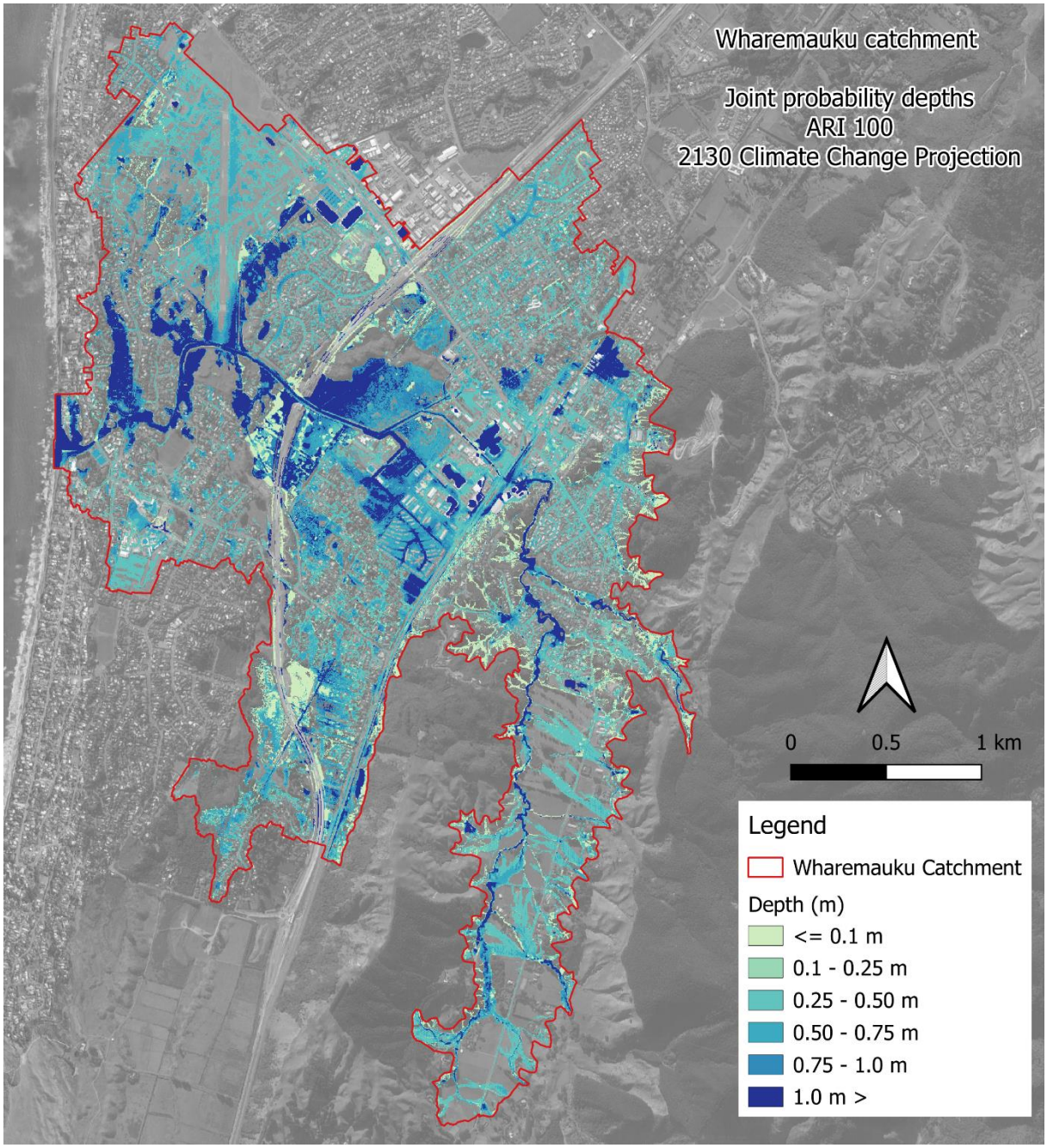


Figure 3-7: Wharemauku catchment joint probability depths for ARI 100 2130 RCP 8.5 M climate change projection

4. Discussion

The joint dependence parameter of 0.75 estimated catchments indicates that there is low dependence between rainfall and storm surge in the area. The design levels calculated for the chosen locations shows that the for the same AEP, the water levels for the completely dependent, independent and dependency parameter of 0.75 are close together.

For the same water level, the results show that the ARI for the independent, dependent and dependence parameters of 0.75 aligns, but in a smaller extent, to analyses in Australia, that have demonstrated that the probability of a heavy rainfall event occurring on catchment near the coastline, in conjunction with elevated storm tide levels, is higher than would be the case if those two phenomena were assumed to be independent from one another.

5. References

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Zheng F, Westra S, Sisson SA (2013) Quantifying the dependence between extreme rainfall and storm surge in the coastal zone. *Journal of hydrology* **505**: 172–187

APPENDIX H Model Build Reports

Refer model build reports available separately from this document.

MazengarbModelBuildReport_Final_Draft_Delivery.pdf

OtakiModelBuildReport_Final_Draft_Delivery.pdf

PaekakarikiModelBuildReport_Final_Draft_Delivery.pdf

ParaparaumuBeachModelBuildReport_Final_Draft_Delivery.pdf

RaumatiBeachModelBuildReport_Final_Draft_Delivery.pdf

WaikanaeModelBuildReport_Final_Draft_Delivery.pdf

WharemaukuModelBuildReport_Final_Draft_Delivery.pdf

DRAFT

APPENDIX I Paraparaumu Beach Pilot Study

DRAFT

MEMO

TO:	Rita O'Brien	**DATE:**	10th January 2021
FROM:	Carl Johnson	**PROJECT NO.:**	J000294
COPY:	Craig Martell, Tony Trueman		
SUBJECT:	Paraparaumu Beach Pilot Study		

INTRODUCTION

Kāpiti Coast District Council (KCDC) engaged Awa Environmental Ltd. (Awa) to undertake a pilot study to investigate the use of TUFLOW for flood modelling in Kāpiti. The investigation was to compare the performance and results of the TUFLOW simulation engine against those of the MIKE FLOOD simulation engine, as well as test TUFLOW's *Sub-Grid Sampling* (SGS) method for resolving open channels and the *rain-on-grid* hydrological approach. KCDC currently has many MIKE FLOOD models that are out-of-date but are in active use across the district. This memo summarises the legacy flood modelling in Kāpiti and provides context for the testing of new modelling methodologies, parameter sets and software (TUFLOW); results from the comparisons between existing and proposed methods and software outputs are then presented.

LEGACY FLOOD MODELS

KCDC currently has seven flood models that are in active use across the district. All these models are currently out-of-date due to the age of the data they are based on. In addition, new modelling approaches have been developed since these models were schematised. The section below summarises the origins of all KCDC's flood models, and the subsequent section summarises parameter sets, and methodology commonly used in these models.

SUMMARY OF MODEL DEVELOPMENT

KCDC has commissioned consultants to build and upgrade models to support flood hazard mapping since 2005; some models were developed earlier, but these were not comprehensive and did not include two-dimensional (2D) representation of flood plains. The Kāpiti Coast has been divided into seven model domains that cover most settlement areas within KCDC's jurisdiction. A summary of each model's development is given in Table 1. MIKE FLOOD models were built initially in 2006-2009 and were built upon in 2010-2012. Further upgrades were made to some project models in connection with work on the MacKay's Crossing to Peka Peka (M2PP) assessment of environmental impacts (Jacobs, 2017). None of the legacy MIKE FLOOD models include as-built data for the M2PP Expressway, Peka Peka to Otaki and Otaki to Levin road upgrades, or Transmission Gully works. Jacobs (2017) provides detail on the development of the legacy models for each location and Table 1 summarises pertinent points.

Table 1. Summary of legacy MIKE FLOOD models built for KCDC.

LOCATION	YEAR	COMMENT
Gaps North	2012	SKM built a MIKE FLOOD model of the open channels and flood plain. This model is loaded by surcharge timeseries from a separate pipe network model.
Gaps South	2012	SKM built a MIKE FLOOD model of the open channels and flood plain. This model is loaded by surcharge timeseries from a separate pipe network model.
Mazengarb	1991	Modelled using HEC-1 (river basin modelling software) and HEC-2 (water profile calculation software).
	2003	Connell Wagner converted the HEC models to a fully-dynamic hydraulic model of the open channels (MIKE 11).
	2007	SKM built a MIKE FLOOD model of the open channels and flood plain.
	2012	SKM and River Edge Consulting upgraded the MIKE FLOOD model to include pipe network (MIKE URBAN) and updated channel survey. This model is loaded by surcharge timeseries from a separate pipe network model.
	2013	SKM upgraded the MIKE FLOOD model to include pipe network (MIKE URBAN).
	2014	Jacobs upgraded the model to include the M2PP Expressway, however this version of the model was not used for flood hazard mapping.
Otaki Local Catchments	2008	SKM built a MIKE FLOOD model of the open channels and flood plain. This model is loaded by surcharge timeseries from a separate pipe network model.
	2011	The model was upgraded.
Paekakariki	2006-2009	SKM built a MIKE FLOOD model of the open channels and flood plain. This model is loaded by surcharge timeseries from a separate pipe network model.
	2006 and 2008	SKM modelled stormwater network, catchments and overland flow.
	2012	SKM built a MIKE FLOOD model of the open channels and flood plain. This model is loaded by surcharge timeseries from a separate pipe network model.

LOCATION	YEAR	COMMENT
Waikanae Local Catchments	1999-2010	SKM, Connell Wagner and Greater Wellington Regional Council (GWRC) built models of subcatchments within the Waikanae model extent.
	2010	SKM built a MIKE FLOOD model of the open channels and flood plain. This model is loaded by surcharge timeseries from a separate pipe network model. The GWRC MIKE 11 model of the Waikanae River was included.
	2013	SKM upgraded the MIKE FLOOD model to include pipe network (MIKE URBAN). This version of the model was not used for flood hazard mapping.
Wharemauku	2001	Connell Wagner built an open channel model (MIKE 11).
	2009	SKM built a MIKE FLOOD model of the open channels and flood plain. This model is loaded by surcharge timeseries from a separate pipe network model.
	2013	SKM upgraded the MIKE FLOOD model to include pipe network (MIKE URBAN).

Of the legacy MIKE FLOOD models, the following include dynamic interaction between open channels (MIKE 11), pipe network (MIKE URBAN/MOUSE), and flood plain (MIKE 21 Classic). These models were used in M2PP Expressway assessments. No models include coastal inundation outside of backwater effects from piped outlet and river and stream mouths.

- Mazengarb.
- Waikanae local catchments.
- Wharemauku.

KEY INFLUENCES ON MODELLING METHODOLOGY

The following influences were important in shaping the methodology used in modelling floods in Kāpiti when the existing models were originally schematised and subsequently upgraded.

- Limitations in modelling software made it impossible or very difficult to connect both open channels (MIKE 11) and pipe network (MIKE URBAN/MOUSE) to the flood plain (MIKE 21 Classic).
- Hardware and software limitations prevented comprehensive, high-spatial-resolution model simulations from completing within practical timeframes.
- Traditionally lumped-subcatchments had been used exclusively in engineering analysis to generate runoff from rainfall. This approach had been applied to countless catchments of a variety of types and so flood modelling practitioners had confidence in the runoff estimates produced.

- Commercially viable 2D flood modelling was a relatively new development in the engineering field at the time and the available software was not robust and so a cautious approach was applied, particularly when it came to model size and complexity.
- Remote sensing datasets such as aerial photography or building footprints were not commonly available and national datasets did not exist or were of low quality. The extraction of land use from photography was not commercially viable. Spatial datasets were generally of low resolution.

MODELLING PARAMETERS AND METHODOLOGY

Existing modelling methodology included the following key features and parameter sets. The design of the methodology was driven by key influences described in the *Key influences on modelling methodology* section above.

- Models include dynamic coupling between 2D flood plain overland flow domain and 1D open channel domains, but do not dynamically couple the open channels or flood plain to the pipe networks, except in later versions of the Mazengarb, Waikanae and Wharemauku models.
- Lumped subcatchment runoff timeseries are generated by HEC-HMS according to SKM (2011) for all subcatchments across the entire catchment. Sub catchments are delineated using ARCGIS hydrology tools. Subcatchment parameters were generated by area-weighted averages of Curve Number (CN), Connected Impervious Area (CIA), Initial Abstraction (IA) and Storage Values (R).
- Subcatchment runoff timeseries are connected to stormwater main manholes or inlet structures. A separate pipe network simulation is run and overflows from the pipe network are stored as spilling discharge timeseries. Road-side and private stormwater inlets are not included in the network.
- Pipe network spilling discharge timeseries are used as source inflows across the 2D model domain to simulate flow that cannot be conveyed by the pipe network. As the pipe network is in a separate model, flow cannot return to the pipe network from the surface or from open channels.
- The 2D overland flow computations are discretised onto a uniform 5 m-resolution square grid and simulated using MIKE 21 Classic. The extent of the 2D domain does not include the coastline and is delineated to only encompass flooding from rainfall.
- Overland flow resistance estimates are based on available Land Cover Database (LCDB) layers.
- Significant open channels were surveyed and included in the MIKE FLOOD model as MIKE 11 branches. Open channel resistance estimates are based on aerial photography.
- The footprint of the channels is blocked out of the 2D domain and the open channels and overland flow domains are connected via MIKE FLOOD lateral links, which consist of a string of weirs that exchange flow between the 1D and 2D domains.
- Outputs from the hydraulic simulations were stored at 1/2 min intervals. These outputs were then processed to produce flood maps of maximum water depth, maximum surface elevation, and maximum flow velocity magnitude.
- Design simulation boundary condition scenarios include existing case (approximate 2008 climate) and “CC” or the climate change scenario predicted for 2090.
- Validation of the flood models was carried out by simulating the 28 October 1998, 17-18 June 2002, 3-4 October 2003, 5-6 January 2025 and 29-30 April 2008 historical storm events and comparing surveyed levels and anecdotal evidence to model results. Flow gauging is only available in river channels upstream of the lower catchments where flooding occurred. The quality of the validations were considered to be acceptable although it highlighted the

wide variability of runoff response that can occur from any given rainfall event. The rainfall depths estimate is likely to be the single greatest variable and in the future the analysis would benefit from a rainfall gauge in the upper catchment.

- For the most extreme events however the runoff factors assumed appear not to have the conservatism they may require to reflect the high runoff that can occur when a catchment is previously saturated, or in situations of high intensity rainfall.
- The CN values should be adjusted dependant on the return period of the event being modelled.

SOFTWARE INVESTIGATION

The focus of this investigation was on comparing the capabilities of the TUFLOW and MIKE FLOOD simulation engines. The outputs were analysed and where discrepancies arose these have been investigated so that a fair comparison of the technical capabilities can be made.

This section does not document the entire model-build process, but it does contain a summary of the key features in the models that have been built for the purpose of software investigation, using simplified methodology. This methodology includes much greater geometric detail than models built to the existing methodology but lacks resolution of parameters sets as these were in development during the pilot model study.

Both single- and double-precision simulation engines for MIKE FLOOD release 2019 Update 1 and TUFLOW 2020-10-AA were used for comparison. The precision of a simulation engine dictates its capacity to accurately resolve floating-point calculations involving numbers that differ in magnitude. BMT, the providers of TUFLOW, recommends using the single-precision variant of the engine as it is faster (smaller run times) than the double-precision variant and the latter does not provide any discernible advantage for most applications. However, though testing and communications with TUFLOW support we have determined that the double-precision variant of the engine will be required for rain-on-grid simulations that include distributed infiltration, and so this engine variant has been included in the investigations.

PILOT CATCHMENT MODEL

KCDC's Paraparaumu Beach stormwater catchment (formerly *Gaps North*) was selected as the pilot catchment for testing modelling software for the following reasons:

- it does not receive flow from upstream hill catchments;
- it includes significant open channels (Tikotu Stream and Kena Kena Drain);
- it includes interaction with the coast;
- it includes interaction with a significant river (the Waikanae River); and
- it is one of Kāpiti's smaller stormwater catchments.

APPROACH

The investigation followed the steps below.

1. Build a MIKE FLOOD model using MIKE 21 Classic (MIKE FLOOD Classic) of the Paraparaumu Beach catchment.
2. Copy the model and convert it to use MIKE 21 Flexible Mesh (MIKE FLOOD FM), instead of MIKE 21 Classic.
3. Use the MIKE FLOOD model components to create a TUFLOW model that resembles the MIKE FLOOD model as closely as possible. A major exception to this is the treatment of the open

channels, which are resolved using TUFLOW's SGS feature, whereas these channels are represented as 1D branches in the MIKE FLOOD models.

4. Run the MIKE FLOOD Classic and MIKE FLOOD FM variants of the MIKE FLOOD engine with a range of simulation configurations to test performance.
5. Run the High-Performance Computing (HPC) variants of the TUFLOW engine with a range of parameters to test performance.
6. Compare simulation performance.
7. Compare outputs.

The construction of both models generally followed the approaches applied in Tauranga, Wellington and Kāpiti, but will not align completely with KCDC modelling standards as these were still under development and some parameter datasets, particularly for hydrology and overland flow resistance were not available at the time of the model build in 2021.

KEY MODEL FEATURES

Figure 1 shows the extent of the model domain used in all models and includes the extent of the Waikanae River branch (in dark blue). Table 2 summarises key features of both the MIKE FLOOD models and the TUFLOW model side-by-side for comparison. The Percival pump station has been approximated for the purposes of this investigation.



Figure 1. Boundary of model domain (light blue zone) and extent of modelled Waikanae River reach (dark blue line).

Table 2. Key features of both MIKE FLOOD and TUFLOW models.

FEATURE	MIKE FLOOD	TUFLOW
Overland flow	• The 2D model domain is discretised into 1,445,288 2 m x 2 m square grid cells. • 2017 "bare-earth" LiDAR is used as the default surface estimate. • MIKE 21 Classic and FM variants use the same discretisation. • A uniform overland flow resistance Manning's n of 0.048. • Building footprints are not accounted for.	• Same as the MIKE FLOOD model with the following exceptions. • The 2D model domain is discretised into 1,451,839 2 m x 2 m square grid cells. • *Source over Area* polygons are used to distribute runoff onto the 2D surface.
Open channel	• Open channels are represented as 1D branches in MIKE 11. • Channel footprint is removed from the 2D domain by using "land" values or removing elements from the domain. • MIKE FLOOD *Lateral links* are used to connect open channels to adjacent flood plain. • Transitions from and to piped reaches are either represented by MIKE 11 structures (culverts) or by a variety of different linking arrangements that have been finalised after debugging. • The Waikanae River is represented as a MIKE 11 1D channel extending from the Mazengarb Stream confluence to the coast and is connected to the 2D domain by *Lateral links*.	• Open channel bed geometry is resolved by a 500 mm resolution grid that is used by TUFLOW's SGS functionality to better describe sub-grid scale features. This approach eliminates the need for 1D channels and lateral linking. • The 500 mm grid is interpolated in a curvilinear manner along the channels between cross sections. This interpolation was produced by the MIKE 11 model. • Transitions to piped reaches are simulated by *SX* links. • The Waikanae River is represented by an ESTRY 1D channel extending from the Mazengarb Stream confluence to the coast and is connected to the 2D domain by *HX* links.
Pipe network	• MIKE URBAN is used to represent pipe, manholes and sumps. The simulation engine is called MOUSE and this integrates with MIKE 21 FM and MIKE 11 via MIKE FLOOD links. • The MIKE URBAN model includes all pipes, manholes and sumps in the KCDC asset GIS dataset except those that cross the model boundary, which have simply been removed. • Additional dummy networks, consisting of two nodes and one connecting pipe, are used to direct flow onto the 2D domain.	• ESTRY is used to represent pipes, manholes and sumps/pits. The pipe network is connected to the 2D domain via pits and *SX* links. • The ESTRY model includes all pipes, manholes and sumps/pits that are included in the MIKE URBAN model. • Pipe friction and Manhole head losses are applied in a similar method to that employed by MOUSE. • Pipe flow accounts for a range of flow regimes. • The duty and stand-by pumps are simulated using the same Q-H relation as in the MIKE URBAN

FEATURE	MIKE FLOOD	TUFLOW
	• Pipe friction losses are represented by Manning's M numbers based on material type. • Manhole head losses are based on the *Engelund* approach with *full-flow-area* scaling of the contraction coefficient (set to 0.25). • The Percival Road duty and stand-by pumps are modelled by two standard MIKE URBAN pumps with a Q-H relation pump curve for a *HOMA M3448-E47* pump. The rising main is not accounted for. • The *IMPLICIT_PUMP* switch in the dhiapp.ini file must be turned on to stabilize the pump discharge. • The pipe network is connected to the 2D domain at sumps (inlets) and at outlets via MIKE FLOOD *Urban links*.	model. The pump operation is described by TUFLOW's operational rules.
Boundary	• 2D model domain extent is based on surface drainage analysis and is buffered by 20 m along boundaries shared with neighbouring catchments. No flow conditions apply. • Tidal boundary applied to boundary along beach. For the MIKE 21 Classic model this requires a workaround: a laterally-linked MIKE 11 branch is used to approximate the coastal boundary condition. • The tidal boundary condition is applied at the coastal end of the Waikanae River branch. • A flow boundary is applied at the upstream end of the Waikanae River reach near the Mazengarb confluence. This has been taken from an existing Greater Wellington Regional Council Waikanae River model.	• Same as the MIKE FLOOD model.
Hydrology	• A runoff hydrograph is generated for each subcatchment using HEC-HMS. • SCS CN loss method (SKM, 2011) with a catchment-wide uniform CN of 45. • Clark Unit Hydrograph routing method (SKM, 2011). • If the rain-on-grid hydrological method is applied, no infiltration loss is applied.	• Same as the MIKE FLOOD model.

Stabilisation is a key part of a numerical model build and can result in significant deviations to the default methodology. The following steps were required to stabilise each of the models.

For the MIKE FLOOD Classic model, instabilities were severe and appeared in most of the open channels, particularly in the vicinity of transitions between MIKE 11 and MIKE URBAN. The MIKE FLOOD FM model is based directly on the debugged MIKE FLOOD Classic model and so includes the following stabilisation workarounds. The MIKE FLOOD FM model has not been further tuned after the conversion and the results contain minor, non-terminal instabilities throughout most of the open channels, which are not present in the MIKE FLOOD Classic version.

1. The *Max dx* (maximum H-point) spacing has been set to 10 m as at 5 m the open channels were unstable.
2. MIKE 11 *Delta* set to 0.85.
3. Exponential smoothing is set to 0.2 for all MIKE FLOOD links except at the downstream end of the Tikotu open channel where this has been set to 0.1 (even further increased smoothing).
4. Severe instabilities are located at the transitions from open channels (MIKE 11) to closed pipes (MIKE URBAN). In many locations the large-shallow-manhole workaround has been used to improve stability.
5. Stabilisation of the Oakley Road long culvert was very difficult and required complicated work arounds to produce sufficient stability (MIKE 11 connecting to MIKE 21 and then MIKE URBAN connecting to MIKE 21). It was particularly difficult to incorporate inlet control at the upstream end when using MIKE FLOOD *Urban outlet* links.
6. MIKE 11 to MIKE URBAN links have been modified extensively in all drains and no one workaround has been proven effective across all locations. The debug process was slow and iterative. In locations where MIKE URBAN inlets at culvert entries have been replaced with outlets, fictitious pumps generate small amounts of energy when flow enters the outlet.
7. The coastal boundary branch has been modified with increased roughness and a graded bed to reduce terminal instabilities. The coastal branch is removed in the MIKE FLOOD FM version of this model and is replaced with a boundary condition.

For the TUFLOW model instabilities mostly occurred at inlets and outlets of large culverts on the Tikotu Stream and Kena Kena Drain. The same approach to stabilisation has been applied in all locations where there are issues.

1. Increase in the storage area multiplier in nodes upstream and downstream of the culverts.
2. Flattening of channel bed to match culvert invert in vicinity of inlet or outlet.
3. Increase in number of cells connected to the inlets or outlets via SX connections. In one instance 13 cells have been connected.

DATA

Key datasets used in the construction of the models developed in this study are identified in Table 3.

Table 3. Key datasets used in the construction of the MIKE FLOOD and TUFLOW models.

DATASET	SOURCE	DESCRIPTION
Digital Elevation Model (DEM)	KCDC	Provided as a 2 m resolution grid based on LiDAR survey.
Land use	Lynker/LCDB	Lynker land use data classification from aerial imagery (2017) in built up areas and LCDB for rural areas.
Infiltration	AWA	Derived from district wide soakage testing.
Soils	AWA	Derived from soil maps (LRIS-SMAP), (NZFSL)
Waikanae River model	GWRC	Legacy MIKE 11 Waikanae River model informed the river channel and inflow boundary conditions.
Stormwater assets	KCDC	Point and line GIS files including locations and attributes for a range of assets including pipes, manholes, intake structures and outlets.
Open channel survey	KCDC's *Gaps North* MIKE FLOOD model	The cross-section information (2004-2008) from the existing *Gaps North* legacy model of the catchment was used.
Sump and intake Q-H relations	Wellington Water Ltd. modelling guidelines	Flow vs. depth curves for a range of stormwater inlet types.
Percival Road pump station pump curve	Wallace Pumps (NZ agents for HOMA pumps)	M3448-E(U)74 pump performance curves.
Pipe roughness	Wellington Water Ltd. modelling guidelines	Pipe roughness estimates.

SIMULATION CONFIGURATIONS

The model and simulation configurations provided in Table 4 were used in the comparison of the MIKE FLOOD and TUFLOW simulation engines. These solver configurations were chosen to be the most representative of settings used in flood modelling. An explanation of "precision" is provided at the start of the *For the* most extreme events however the runoff factors assumed appear not to have the

conservatism they may require to reflect the high runoff that can occur when a catchment is previously saturated, or in situations of high intensity rainfall.

- The CN values should be adjusted dependant on the return period of the event being modelled.

Software Investigation section.

Table 4. Test models and simulation solution details.

ID	ENGINE	PRECISION	SOLUTION SCHEME
CD-3167	MIKE FLOOD Classic	Double	Alternating Direction Implicit solver with a fixed timestep size.
CD-3325	MIKE FLOOD FM	Double	Explicit Roe solver with 2nd order spatial (flux-limiting) scheme and a 2nd order temporal scheme.
CD-3142	TUFLOW HPC (with SGS)	Single	An explicit solver using a 2nd order spatial (flux-limiting) scheme and the 4th order Runge-Kutta temporal scheme.
CD-3344	TUFLOW HPC (with SGS)	Double	”

An additional five other simulations were run to test various simulation parameters, but these are not reported here as the results are not necessary to draw the central conclusions of this investigation. Some reference will be made to these simulations when pertinent, however.

A 100 year-ARI (1% AEP) rainfall event paired with a 20 year-ARI (5% AEP) storm surge event in a 2090 climate was used for all simulations. A simulation period of 20 hours was chosen to correspond to the available Waikanae River inflow boundary condition timeseries; this period captures peak flood levels.

COMPARISON OF PERFORMANCE

All simulations were carried out on a single computer with the specifications and software specified in Table 5. Although both MIKE FLOOD FM and TUFLOW can use multiple GPU’s in a single simulation, this avenue has not been explored for the following reasons:

- the test model is relatively small, so it is likely that inter-process communication overhead will be significant;
- a machine with dual GPU’s suitable for testing was not available; and
- it is common that single GPU machines are used for flood modelling and so excluding this option is not highly detrimental to the value of the study.

Table 5. Specifications of computer used for comparisons.

SPECIFICATION	DESCRIPTION
Operating System Name	Microsoft Windows 10 Pro
System Type	X64-based PC
Processor	Intel(R) Core(TM) i7-6700K CPU @ 4.00GHz, 4001 Mhz, 4 Core(s), 8 Logical Processor(s)
Installed Physical Memory (RAM)	64.0 GB
Graphical Processing Unit Name	NVIDIA GeForce RTX 3090

The run time performance of the four engine variants for a 20-hour simulation period is presented in Table 6. Explanation of the reported statistics is as follows.

- *Total run time* is the elapsed time from the launching of the executable until the successful completion and termination of the simulation process as logged: this includes simulation initiation and the writing out of results to hard disk and corresponds to what is commonly understood to be the duration of a simulation. Smaller run times are generated by faster simulation engines.
- *Average timestep size (2D)* is the average of the varying timestep size used by the 2D hydraulic solution component to solve overland flooding. The 2D solution is generally the slowest part of a flood simulation and so this metric is the more relevant than that of the 1D component for performance. Larger time steps indicate that the simulation engine is doing less work to simulate the same time period.
- *Run time / simulated time* is the ratio, expressed in percentage, of the time taken to simulate an equivalent simulated period. A percentage of 100% indicates that simulations run at the same speed as a clock, whereas percentages less than 100% indicate that the simulation will complete before equivalent period elapses in real time. This will vary throughout the simulation based on hydrodynamic parameters, but the values presented here is based on totals for the whole simulation. Smaller percentages are generated by faster simulation engines.

Table 6. Simulation engine performance for a simulated 20-hour period.

ENGINE	TOTAL RUN TIME	AVERAGE TIMESTEP SIZE (2D)	RUN TIME / SIMULATED TIME
MIKE FLOOD Classic	62.2 hr	0.25*	311%
MIKE FLOOD FM	6.4 hr	0.125 s	32%
TUFLOW HPC single-precision	1.5 hr	0.423 s	7%
TUFLOW HPC double-precision	2.2 hr	0.424 s	11%

* The MIKE 21 Classic solver uses a fixed timestep size.

Some points to note with regard to the results presented in Table 6 include the following.

- Each engine spends different amounts of time initiating:
    - MIKE FLOOD Classic – 23 s;
    - MIKE FLOOD FM – 75 s;
    - TUFLOW single-precision – 47 s; and
    - TUFLOW double-precision – 47 s.

  Time spent initiating each simulation is negligible compared to the overall run time.
- TUFLOW creates cached representations of datasets used in the simulation initiation process, which are only rebuilt when the corresponding dataset is modified (updated modified date time). A known bug prevents the optimisation of the code for generation of the computational grid in the double-precision engine, with the consequence that the first time a double-precision simulation is run with an updated grid, this process takes approximately 20 hours; the single-precision engine can generate the grid in minutes. In successive simulations this generation process is not necessary and so the initialisation process is identical for both engine variants. The double-precision engine grid generation time is not included in the performance statistics.
- The TUFLOW (2D) and ESTRY (1D) solvers are closely integrated and share adaptive timesteps using a variety of stability criteria. For MIKE FLOOD Classic all three components, MIKE 11, MIKE URBAN and MIKE 21 Classic use the same fixed timestep. In contrast the 1D components, MIKE 11 and MIKE URBAN, of MIKE FLOOD FM use a fixed time step size and exchange flow at each fixed timestep with the MIKE 21 FM engine, which uses adaptive sub timesteps in order to maintain stability (Courant-Friedrichs-Lewy (CFL) Condition). The consequences of this will be discussed in a later section.
- A single-precision version of the MIKE FLOOD FM simulation was carried out and the resulting run time came to 10.1 hours, which is larger than that of the double-precision engine. The reason for the extended runtime is the erroneous behaviour of a single cell at a subcatchment runoff loading point in the Kotuku Park Pond. Water enters this cell but cannot leave it resulting in an ever-rising water level. The level reaches more than 190 m RL by the end of the simulation. The configuration of the single- and double-precision models is identical.

COMPARISON OF OUTPUTS

The following are observations made while inspecting model results.

- Significant instabilities are apparent in the MIKE FLOOD simulation results, mostly around transitions between the MIKE URBAN and MIKE 11 1D models.

The MIKE FLOOD FM simulation produces an anomalous maximum water level upstream of the Oakley Court culvert. It appears that some kind of instability or limitation in the linking arrangement from the MIKE 11 open channel via the MIKE 21 FM model and then into the MIKE URBAN model, which was implemented to stabilise the MIKE FLOOD Classic model, has caused erroneous results in the MIKE FLOOD FM model. It may be possible to further debug this model, but this has not been undertaken at this point: stabilisation workarounds suitable for the MIKE FLOOD Classic variant are not necessarily suitable for the FM variant.

DISCUSSION

There are three key elements in the comparison of the TUFLOW and MIKE FLOOD simulation engines:

1. speed of simulation;

2. ease of use; and
3. reliability of results.

Statistics presented in Table 6 demonstrate that the TUFLOW simulation engines are capable of running simulations faster than MIKE FLOOD, for both the MIKE 2019 Update 1 and MIKE 2021 releases.

Overall TUFLOW provided a better model build experience, which cannot be attributed to the modeller's experience. The model was simpler to stabilise and a single strategy could be applied to all locations that were prone to instabilities, generally the link between culverts and open channels. The new SGS functionality largely removes the need for 1D open channel models, which are time-consuming to build and connect to the 2D domain and are a cause of instabilities.

A major weakness in the MIKE FLOOD software is the linking between MIKE URBAN and MIKE 11 models. This was the source of significant instabilities that were not possible to fully eliminate. The fixed exchange time interval also prevents MIKE FLOOD from running at larger timesteps when the Courant number and other factors would allow, as demonstrated by the comparatively low average timestep size in Table 6.

CONCLUSIONS

Overall, TUFLOW engines run faster than MIKE FLOOD, the models are easier to build and modify, and the results are more stable and reliable.

The following specific conclusions have been drawn from this study.

1. The TUFLOW simulation engines run faster than MIKE FLOOD simulation engines. This appears to be due to:
    a. The close integration of the 2D (TUFLOW) and 1D (ESTRY) solvers, which allows for the sharing of the adaptive timestep; and
    b. Better numerical stability in both the 2D and 1D components of the model.
2. The single- and double-precision, HPC and Quadtree TUFLOW engine variants demonstrate more consistency than the MIKE FLOOD FM variants do.
3. TUFLOW models are simpler to build than MIKE FLOOD models because:
    a. The 2D TUFLOW engine only integrates with a single 1D model (ESTRY), whereas MIKE FLOOD integrates MIKE 21 with MIKE 11 and MIKE URBAN;
    b. They are composed of common file formats (predominantly text and GIS formats) are used;
    c. Simulation initialisation logging is comprehensive, and error messages are generally useful for debugging models;
    d. Sophisticated geometry modification operations are available during the model initialisation, which negates the need for many pre-processing steps in other software;
    e. Plug-ins are available for commonly used GIS platforms such as QGIS and ArcGIS, which relieves the user of learning another software platform interface specifically designed for building TUFLOW models;
    f. Features for connecting hydrological inflows to the 2D domain are straight-forward to implement;
    g. Debugging unstable 1D to 2D connections generally involves a limited number of techniques; and
    h. Models can be structured to include multiple layers of overlapping and overriding geometry so that variations on a base model can be made without modifying any of the base datasets.

4. Comparison of TUFLOW simulation results has found that:
    a. They are more stable than MIKE FLOOD results while not demonstrating any significant numerical dampening;
    b. The SGS feature enables the definition of open channels such that results are consistent with 1D open channel representations at hydraulic structures and provides a better representation of flow (and momentum) exchange between open channels and adjacent flood plain; and
    c. TUFLOW's numerical scheme prevents spurious velocities from developing in shallow flows on steep terrain.
5. General disadvantages of TUFLOW when compared to MIKE FLOOD include the following:
    a. TUFLOW does not include hydrological modelling, except for the rain-on-grid with distributed infiltration approach.
    b. Results are output in a wide range of text and GIS files that can be difficult to navigate without using a GIS plug-in tool. MIKE FLOOD results are combined into binary files that provide better context to the results.
    c. 1D results are output with a fixed 3 decimal place precision, so it is not possible to accurately extract small flows from the results.
    d. The output and logging are very verbose and can include a wide range of warnings and "checks". It can be difficult to distinguish between "checks" that require attention and those that signal model features that are correct, but TUFLOW flagging does highlight issues.
    e. TUFLOW interpolates and assumes default values in many places throughout a model before beginning a simulation. This can become a problem when incorrect configuration leads to important model features being excluding without a terminal error.
    f. Maintaining a tidy model will require on-going diligence to ensure that files are located in correct folders and that the free-format control files are well and sparingly documented.

HYDROLOGICAL METHOD COMPARISON

RAIN-ON-GRID

This method includes a distributed hydrological loss model and relies on hydrodynamic simulation to account for routing. In this method, rain "falls" on the ground and runoff is determined by spatially varying infiltration factors representative of Kāpiti's coastal soils.

The key advantage of the rain-on-grid approach to hydrology is the ability to approximate groundwater effects on soil infiltration capacity in a spatially variable manner allowing models to be used in investigations of the effects of elevated groundwater on flooding.

SUB-CATCHMENT BASED

This method includes a lumped hydrological loss model where discharges are connected directly to a discharge point/location. Routing and losses are calculated externally and accounted for within the resultant discharge hydrograph.

PARAPARAUMU BEACH FLOOD MAP COMPARISON

The rain-on-grid vs the lumped sub catchment hydrological approaches are compared in the previous section, however there are differences in how the losses are calculated so a direct comparison is not possible. In general, the rain-on-grid methodology results in.

- A more distributed ponding pattern as rainfall is captured and retained within all low-lying areas within the bathymetry.
- TUFLOW produces more realistic flow patterns, and the Wu 3D approach maintains eddy viscosity where Smagorinsky does not.
- MIKE FLOOD Classic, MIKE FLOOD FM and TUFLOW produce very similar estimates for peak flood levels.
- The solution scheme (explicit vs. implicit) has a greater impact on the results than the specific implementation, i.e. MIKE FLOOD FM and TUFLOW results are more similar than those of MIKE FLOOD FM and Classic.
- Most differences are within 50mm with a small bias towards higher levels in Tuflow.

KEY INFLUENCES ON MODELLING METHODOLOGY

The proposed methodology and parameter sets are presented and discussed in a separate document Methodology Report – 1/07/2024.

The following influences are important in shaping the proposed approach for modelling floods in Kāpiti. The contrast in the influences on modelling methodology between current drivers and those of the existing models, schematised in 2006-2009 (see previous section), give rise to the many proposed changes to modelling approach.

The following key influences guided the development of the proposed modelling methodology.

- Recent developments in software capability.
- Successful implementation elsewhere internationally.
- Mixture of rain-on-grid and lumped sub catchment hydrology.
- Availability of high-resolution remote-sensing datasets for land cover.

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APPENDIX J Base model Boundary conditions per catchment

Note: All base models use RCP 8.5M rainfall, SSP5 8.5M tides, and 50th percentile SSP5-8.5M groundwater levels climate change scenarios.

Ōtaki Catchment base model boundary conditions.

AWA CODE	YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)	ŌTAKI RIVER (AEP%)	WAITOHU STREAM (AEP%)
BM-OT1	2030	1%	5%	1%	1%
BM-OT2	2030	2%	10%	2%	1%
BM-OT3	2030	10%	15%	10%	1%
BM-OT4	2030	20%	40%	20%	1%
BM-OT5	2030	50%	80%	50%	1%
BM-OT6	2130	1%	5%	1%	1%
BM-OT7	2130	2%	10%	2%	1%
BM-OT8	2130	10%	25%	10%	1%
BM-OT9	2130	20%	40%	20%	1%
BM-OT10	2130	50%	60%	50%	1%

Waikanae Catchment base model boundary conditions.

AWA CODE	YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)	WAIKANAE RIVER (AEP%)
BM-WK1	2030	1%	1%	1%
BM-WK2	2030	2%	2%	2%
BM-WK3	2030	10%	10%	10%
BM-WK4	2030	20%	25%	20%
BM-WK5	2030	50%	50%	50%
BM-WK6	2130	1%	1%	1%
BM-WK7	2130	2%	2%	2%
BM-WK8	2130	10%	15%	10%
BM-WK9	2130	20%	30%	20%
BM-WK10	2130	50%	60%	50%

DRAFT

Mazengarb boundary conditions.

AWA CODE	YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)
BM-MZ1	2030	1%	1%
BM-MZ2	2030	2%	2%
BM-MZ3	2030	10%	10%
BM-MZ4	2030	20%	25%
BM-MZ5	2030	50%	50%
BM-MZ6	2130	1%	1%
BM-MZ7	2130	2%	2%
BM-MZ8	2130	10%	15%
BM-MZ9	2130	20%	30%
BM-MZ10	2130	50%	60%

Paraparaumu Beach base model boundary conditions.

AWA CODE	YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)
BM-PB1	2030	1%	10%
BM-PB2	2030	2%	10%
BM-PB3	2030	10%	50%
BM-PB4	2030	20%	50%
BM-PB5	2030	50%	50%
BM-PB6	2130	1%	30%
BM-PB7	2130	2%	40%
BM-PB8	2130	10%	50%
BM-PB9	2130	20%	75%
BM-PB10	2130	50%	75%

DRAFT

Wharemauku Catchment base model boundary conditions.

AWA CODE	YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)
BM-WH1	2030	1%	10%
BM-WH2	2030	2%	10%
BM-WH3	2030	10%	50%
BM-WH4	2030	20%	50%
BM-WH5	2030	50%	50%
BM-WH6	2130	1%	30%
BM-WH7	2130	2%	40%
BM-WH8	2130	10%	50%
BM-WH9	2130	20%	75%
BM-WH10	2130	50%	75%

Raumati Catchment base model boundary conditions.

AWA CODE	YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)
BM-RB1	2030	1%	10%
BM-RB2	2030	2%	10%
BM-RB3	2030	10%	50%
BM-RB4	2030	20%	50%
BM-RB5	2030	50%	50%
BM-RB6	2130	1%	30%
BM-RB7	2130	2%	40%
BM-RB8	2130	10%	50%
BM-RB9	2130	20%	75%
BM-RB10	2130	50%	75%

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Paekakariki Catchment base model boundary conditions.

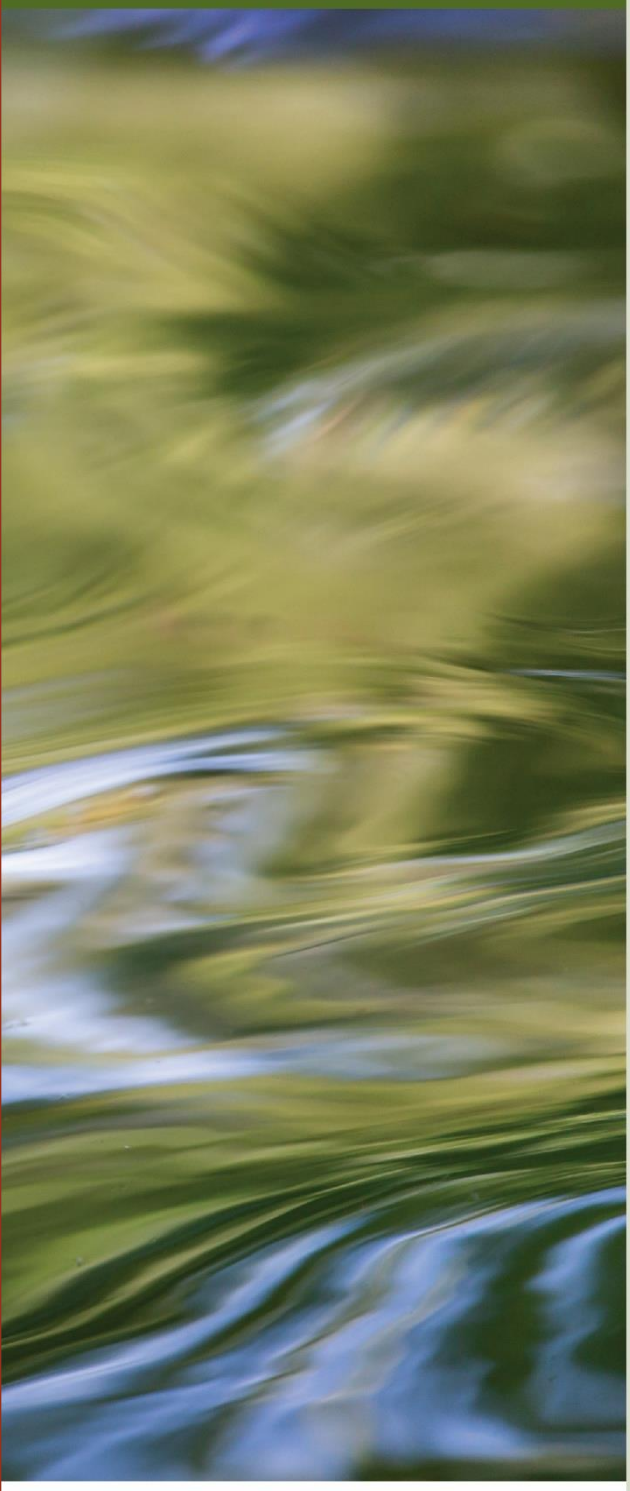
AWA CODE	YEAR HORIZON	RAINFALL (AEP%)	TIDAL (AEP%)
BM-PK1	2030	1%	10%
BM-PK2	2030	2%	10%
BM-PK3	2030	10%	50%
BM-PK4	2030	20%	50%
BM-PK5	2030	50%	50%
BM-PK6	2130	1%	30%
BM-PK7	2130	2%	40%
BM-PK8	2130	10%	50%
BM-PK9	2130	20%	75%
BM-PK10	2130	50%	75%

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APPENDIX K FME Post-Processing Steps

6. Coerce polygons from contiguous wetted cells, then split polygons into large ($> 4000\ m^2$) and small "ponds".
7. Small ponds are combined with other nearby small ponds (within $8m$) to form combined ponds, with the shape formed from creating concave hulls and converting back to cells with depth set to the mean depth of the original cells.
8. Building voids are filled in by capturing neighbouring wetted cells for each building, then creating concave hulls and converting back to cells with depth set to the maximum depth of the neighbouring cells.
9. The large ponds, combined small ponds, and filled building voids are combined to form a working depth raster.
10. A convolved raster is created from this raster using an unweighted mean at a 9x9 cell range and setting the minimum depth to 0.05 m.
11. Noise is then filtered out by removing all thin connectors holding groups of contiguous dry cells together as well as groups with area under a minimum value ($400\ m^2$), and repeat the same for groups of contiguous wetted cells.
12. This noise-removed raster is then applied as a mask over the convolved raster.
13. The raster is converted to contiguous polygons and edges are smoothed using a densify-smooth-generalize process.

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