REPORT

Henderson catchment FHM

Model Build and System Performance

Prepared for
Auckland Council

Prepared by
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Exceptional thinking together

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Executive summary

The Henderson catchment is approximately 11 km^2 and is located in West Auckland and drains to the Waitemata Harbour. It is located downstream of the Opanuku and Oratia sub-catchments, resulting in a total drainage area of approximately 55 km^2. Land use varies around the Henderson catchment with predominately residential settlements in the upper and eastern catchment, and industrial areas to the west of Henderson Creek.

This report summarises the methodology, data sources and assumptions that have been adopted to build the hydraulic and hydrological models, undertake system performance assessment and develop flood maps and flood hazard maps. The Opanuku and Oratia components of the total catchment are not covered within this report unless explicitly stated.

The report provides a detailed overview of the catchment topography and land use including both the existing and future development scenarios. The existing scenario is based on 2013 LiDAR, aerial photography from 2015-2016 and asset and watercourse surveys carried out between December 2015 and January 2016. As-built drawings from 2014 were used to implement the recently constructed Te Atatū State Highway 16 interchange into the existing scenario. The existing scenario therefore does not represent catchment changes that have occurred since these dates. Representing these changes is part of ongoing work with Auckland Council and outside the scope of this report.

A Mike Flood model comprising Mike 11, Mike 21 and Mike Urban was constructed for the FHM in accordance with Auckland Council Modelling Specification (November, 2011). The hydrological and hydraulic model build is detailed in this report.

There is no data available/provided to calibrate and validate the hydraulic model. Additional confidence in the model's ability to represent flows and water levels would be gained by hydrometric monitoring of future large storm events followed by model calibration and additional validation.

A sensitivity analysis showed that the catchment has a relatively low sensitivity to changes in land use with changes to both hydraulic roughness and hydrological curve numbers by +/- 20% resulting in only minor alterations to the flood extent.

The model results indicate that only 13% of the existing pipe network performs to a 10 year Average Recurrence Interval (ARI) design standard under the Maximum Probable Development (MPD) scenario. Pipe capacity catchment overview figures are provided in the report that highlight the areas where additional flows can be passed through the pipe network.

A floor level survey was carried out and model results show that there are 177 residential and 37 commercial buildings at risk from flooding above floor level in the 100 year ARI MPD event across the entire catchment. In the 100 year ARI Existing Development (ED) event 142 residential and 29 commercial buildings were identified as being at risk from flooding above floor level.

The floor level survey was used to predict average annual damage to buildings. The predicted average annual damage to buildings in the catchment as a result of flooding is approximately \$6,665,443 for Existing Development (ED), which increases to approximately \$9,024,510 under the Maximum Probable Development (MPD) scenario (with climate change). The net present value of the buildings affected by flooding (50 year period, 8% discount rate) is approximately \$81.5 M and \$110.4 M for ED and MPD scenarios respectively. It should be noted that non-building damage is not included in the damage assessment.

1 Introduction

Tonkin + Taylor (T+T) were appointed by Auckland Council (AC) to carry out the Henderson catchment Flood Hazard Mapping (FHM) study.

The overall objectives of the Henderson FHM study are to:

- Develop hydraulic and hydrological models for the Henderson catchment
- Undertake system performance assessment
- Complete floodplain and flood hazard mapping
- Undertake a flood damage assessment for the catchment.

The services provided by T+T under the contract are to:

- Undertake Rapid Flood Hazard Mapping for the catchment
- Assess asset data from AC and identify surveys required to obtain sufficient quality asset data for model build
- Develop hydraulic and hydrological models of the catchment using the most up to date versions of appropriate modelling software
- Validate models to gauged rainfall and stream flow data and historical flood incidents where available
- Assess performance of the stormwater drainage system in the catchment and identify where levels of service requirements are not, or will not be, met
- Identify significant flood hazard areas including properties with habitable and non-habitable floors at risk of flooding within the catchment and develop maps for flood plain, flood hazard and flood sensitive areas
- Undertake flood damage assessment for specified design storm events.

1.1 Activities and scope

This Model Build and System Performance Report has been prepared for AC, as required under the Stormwater Flood Modelling Specification (SWFMS) (November 2011) and the terms of the project contract (Auckland Council contract ACPN_13248).

The report structure follows Section 9.4 of the SWFMS.

2 Catchment description

2.1 Location and topography

The Henderson catchment is located in West Auckland and drains to the Waitemata Harbour. A map of the Henderson sub-catchment can be seen in Figure A1, Appendix A. It is located downstream of the Opanuku and Oratia sub-catchments, resulting in a total drainage area of approximately 55 km^2. The Henderson component of the entire catchment is approximately 11 km^2. This report relates only to the 11 km^2 Henderson catchment unless explicitly stated.

The Henderson catchment terrain consists of steep slopes around the Te Atatū Peninsula and upstream of Henderson creek. This is accompanied by gentler slopes to the west and relatively flat plains inland. The majority of the sub-catchments drain to Henderson Creek, before flows pass downstream to the Waitemata Harbour. Sub-catchments on the eastern side of the Te Atatū Peninsula drain directly eastwards into the Waitemata Harbour.

2.2 Geology and soils

The surface geology in the catchment is dominated by alluvial soils, with sections of Waitemata residual soils in the North, South, and Eastern borders.

Figure A2 in Appendix A shows the surface geology in the catchment, overlaid with the current property parcel boundaries.

2.3 Existing and future land use

Land use varies around the catchment with primarily residential settlements in the upper catchment and to the east of Henderson Creek in the lower catchment, with industrial areas to the west of Henderson Creek.

State Highway 16 intersects the catchment from East to West with a Bridge crossing over the Creek.

Figure A3 in Appendix A shows the Existing Development (ED) scenario catchment land use (sourced from the Ministry for the Environment New Zealand Land Cover Database Version 4).

Figure A4 in Appendix A shows the Maximum Probable Development (MPD) scenario catchment land use, derived from the Auckland Unitary Plan (AUP, dated 2014). Changes to the Unitary Plan layer since 2014 are discussed in section 4.2.5

There is more information provided in Section 3.3 regarding the representation of existing and future land use.

2.4 Stormwater drainage system

The surface water drainage system and the pipe network can be seen in Figure A1 in Appendix A.

A review of the stormwater drainage system was undertaken in the Model Extent and Data Assessment Report (MEDAR) (T+T, February 2015) and a comprehensive asset survey and topographic survey was undertaken by Cardno between December 2015 and January 2016. Data sources for the model build are discussed further in Section 3.2.

2.5 Reported flooding issues

2.5.1 Drainage operational issues

No operational issues reports were provided by Auckland Council during the Model Extent and Data Assessment Report (MEDAR) (T+T, February 2015).

2.5.2 Reported flooding issues

Catchment Management Plans (CMP's) were provided by Auckland Council during the project MEDAR. These reports included an indication of flood frequency reported by property owners, however the information was over 10 years old and insufficiently detailed for use in a model validation process as there are no dates of flooding. Therefore, the information has not been used as part of the FHM process.

Flooding was reported within the catchment during a rainfall event on the 29th June 2016 however insufficient data was provided to validate the model for this event.

There are no flow gauges located within the Henderson Catchment.

3 Model build

3.1 Modelling software

The software used to model the Henderson catchment model is shown in Table 3.1.

Table 3.1: Modelling software

Software Type	Mike 11 by DHI	Mike 21FM by DHI	Mike Urban by DHI
Software Version	Version 2014	Version 2014	Version 2014
Service Pack	Service Pack 3	Service Pack 3	Service Pack 3
Method used for Runoff	n/a	n/a	UHM

3.2 Review of existing data

3.2.1 Hydrometric data

A detailed review of the hydrometric data within the Henderson and surrounding area was undertaken for the MEDAR (T+T, February 2015).

In summary:

1 There are five rain gauges located in the area surrounding the Henderson catchment and one rain gauge within the catchment itself (Te Pai Park). These rain gauge locations and records are as follows:
- Forrest Hill Road, Waiatarua (1999 – now),
- Oratia Cemetery, Oratia (1999 – now),
- Candia Road, Opanuku (1999 – now),
- Te Pai Park, Henderson (1999 – now),
- Keeling Road, Utilitech Training Centre (1990 – now),
- Pooks Road, Swanson (1985 – 2013).

2 There are five flow gauges located in the area surrounding the Henderson catchment but none within the catchment itself. These flow gauge locations are as follows:
- Universal Drive, Paremuka Stream (2008 – 2013),
- Candia Road Bridge, Opanuku Stream (2006 – now),
- Millbrook Road, Oratia (1999 – now),
- Vintage Reserve, Opanuku (1999 – now),
- Woodside Reserve, Swanson Stream (1994 – now).

3 There are no tide gauges located near the Te Atatū Peninsula. The closest gauge is located at the Port of Auckland

4 A detailed model validation is not possible due to the following reasons:
- No historical flood incident records relating to specific events,
- No flow records within the catchment.

3.2.2 Topographical data

A detailed review of the asset and topographic data of the catchment was completed by T+T in the MEDAR (T+T, February 2015). The report was used to inform the detailed catchment survey, which was undertaken by Cardno between December 2015 and January 2016. T+T project managed the survey scope of works, which was awarded to Cardno under AC contract number ACPN 19209.

This section refers to the topographical data and assumptions used to represent two scenarios in the catchment model:

1. Existing development (ED)
2. Maximum probable development (MPD).

The two scenarios are discussed under each topographical information sub-section discussed below.

3.2.2.1 Topography outside watercourse

2013 LiDAR data was used for the entire model DEM. Manual adjustments were made to the model DEM at the following locations:

1 Te Atatū State Highway 16 interchange (2014 as built drawings)
2 Ponds – modifications were made to ensure that storage volumes within ponds were represented by the 2013 LiDAR and the 2D model grid resolution.

The location of the pond features is shown in Figure B1 in Appendix B.

The same LiDAR data set was used to represent ED and MPD. The schematisation for the ED and MPD scenarios can be seen in Figure B3 in Appendix B.

3.2.2.2 Watercourses

A cross section survey of the watercourses was undertaken between Cardno between December 2015 and January 2016. For some areas of the catchment, LiDAR data was used to derive cross sections. Cross sections surveyed by GHD in 2005 are also used on Henderson Creek to improve the spacing resolution.

Figure B1 in Appendix B shows the location of open watercourses in Henderson catchment and the locations of cross section surveys.

3.2.3 Asset data

The data sources for the watercourses, ponds, culverts and bridges can be seen in Figure B1, Appendix B. The data sources for other stormwater assets (e.g. manhole and pipes) can be seen in Figure B2, Appendix B.

3.2.3.1 Culverts and bridges

Existing culverts and bridges within open channel extents are modelled using Mike 11 1D structures. The location of the 4 modelled culverts and 8 modelled bridges can be seen in Figure B1, Appendix B.

Tables B1 and B2 in Appendix B describes the locations, characteristics and source of information for all culverts and bridges modelled in the catchment.

3.2.3.2 Weirs

There are 12 weirs modelled within the catchment (Mike 11 1D structures). The location of these weirs can be seen in Figure B1, Appendix B. All 12 weir structures represent overtopping at culvert or bridge structures where flows remain within the 1D model extent.

Table B3 in Appendix B describes the locations, characteristics and source of information for all weirs modelled in the catchment.

3.2.3.3 Ponds

Existing ponds within the catchment are modelled within the 2D (Mike 21) model extent. The location of the 16 modelled ponds can be seen in Figure B1, Appendix B.

Table B3 in Appendix B describes the location, pond ID and source of information used to represent the ponds in the model.

3.2.3.4 Manholes, pipes, inlets, outlets

The dimensions of the manholes, pipes, inlets and outlets were determined from four sources:

1. Council records
2. Asset survey of manholes, pipes, inlets, outlets, culverts and bridges (Cardno Survey from December 2015 to January 2016)
3. Te Atatū State Highway 16 interchange as-builts (2014)
4. Incomplete data sets have required some assumptions based on best available information.

The locations of all the manholes, pipes, inlets and outlets and their data sources can be seen in Figure B2 in Appendix B. Where the data source is identified as 'assumed' this means that there was at least one dimension which has been based on a desktop assumption.

The assumptions were made for a variety of reasons including:

1. Minimising survey extent to save unnecessary costs without compromising quality; for example:
   - Manhole diameter unknown, but reasonable to assume diameter based on incoming pipe sizes
   - Assuming dimensions and elevations based on surrounding network for non-critical assets, for example:
     - Interpolating invert level based on upstream and downstream levels
     - Assuming pipe diameter based on upstream and downstream diameters.
2. Surveyors unable to find the asset
3. Ground level was calculated from LiDAR topography.

3.3 Hydrological model

3.3.1 Method used

In accordance with the Auckland Council Stormwater Flood Modelling Specification (SFMS, November 2011), the Soil Conservation Services (SCS) hydrological method was used to transform rainfall into runoff. The 24 hour rainfall depths were determined from TP108. The hydrological processes were modelled using Unit Hydrograph Method (UHM) from Mike Urban.

3.3.2 Hydrological model extents and sub-catchment delineation

The model schematisation, sub-catchment delineation and sub-catchment loading for both the ED and MPD scenarios is shown in Figure B3 in Appendix B.

Sub-catchments were defined to meet the requirements of the SFMS, taking into consideration topography, stormwater networks, and other obstructions such as roads and railways. The sub-catchments were delineated using the 2013 LiDAR dataset. The range of sub-catchment sizes is shown in Figure C1 in Appendix C.

The sub-catchment delineation for ED and MPD scenarios are the same. Overall sub-catchment imperviousness for the ED scenario is illustrated in Figure C2 in Appendix C while Figure C3 illustrates the overall sub-catchment imperviousness for the MPD scenario.

The hydrological models are linked to the hydraulic models in three different ways:

1 Sub-catchments draining directly to open channels have hydrology modelled in Mike Urban, linked to dummy nodes that are coupled to Mike 21. The dummy nodes lie on overland flowpaths so that runoff drains into the Mike 11 model
2 Sub-catchments draining to floodplain without reticulation have hydrology modelled in Mike Urban, linked to dummy nodes that are coupled to Mike 21
3 Reticulated sub-catchments have hydrology modelled in Mike Urban, which are linked directly to the Mike Urban hydraulic model.

Figure B3 in Appendix B show the sub-catchment delineation using the three hydrological methods for existing and maximum probable development scenarios. The catchment boundary was determined as part of the MEDAR (T+T, February 2015) to include runoff from all areas draining to Henderson.

Table 3.2 provides a summary of the hydrological sub-catchments.

Table 3.2: Summary of hydrological sub-catchments

Hydrological Model Components	ED and Future MPD
Number of sub-catchments	1538
Range of sub-catchment size (ha)	0.15 to 7.30

3.3.3 Hydrological model parameters

The catchment characteristics are defined using the following:

1 Curve number
2 Lag time
3 Initial abstraction.

The pervious and impervious catchments were represented independently for each sub-catchment. The impervious catchment was represented using a curve number of 98. The curve number for the pervious component of the sub-catchment was based on a weighted average of curve numbers within the sub-catchment (such as lawns, open spaces, and forests).

The curve number was determined following an assessment of hydrological soil type, impervious coverage and land use. The hydrological soil type for the catchment can be seen in Figure A2 in Appendix A. The surface soil type was determined from a “Soil Type.shp” layer provided by AC, which was considered appropriate for use based on a small number of observations made during the site walkover and reflected general trends seen in the sub-surface geological maps.

The land use for the MPD was determined from the Auckland Council Unitary Plan. Table 3.3 shows the impervious percentage assumptions for different land use types within the Henderson Catchment. Table C1 in Appendix C gives a complete list of land use types and the impervious percentage assumptions. Impervious percentage assumptions are based on AC recommendations.

Table 3.3: Impervious percentage assumptions for different land use types

Zone Description	MPD Percentage Impervious
Business Park	80
Countryside living	10
Future Urban	70
Heavy Industry	80
Local Centre	80
Terrace Housing and Apartment	70
Metropolitan Centre	80
Mixed Rural	10
Mixed Use	70
Rural Conservation	10
Rural Production	5
Light Industry	80
Mixed Housing Suburban	70
Single House	70
Rural and Coastal Settlement	10
Town Centre	80
Large Lot	10
Water	100
Strategic Transport Corridor	100
Road	90
General Coastal marine	10
Public Open Space	10
City Centre	100
Minor Port	100
Defence	100
Marina	100
Mooring	100
Hauraki Gulf Islands	0
Neighbourhood Centre	80
Ferry Terminal	100
Rural Coastal	10
General Business	80
Special Purpose	80
Coastal Transition	70
Mixed Housing	70
Public Open Space	10

Lag time and initial abstraction was determined in accordance with the TP108 procedure, where lag time equals two-thirds of the time of concentration, and initial abstraction varies between 0 mm and 5 mm.

The lag time was determined based on TP108 methodology, where the longest flow path for each sub-catchment was determined from LiDAR and an overland flowpath assessment. Catchment length and slope (by equal area method) were calculated based on identified longest flow path. The lag time for each sub-catchment was calculated based on catchment length, slope, curve number and channelisation factor. A channelisation factor of 0.6 was used for impervious sub-catchments, and 1.0 for pervious sub-catchments.

Figures C2 and C3 in Appendix C show impervious surface percentage and catchment delineation for the ED and MPD scenarios respectively.

The catchment characteristics for each of the sub-catchments, including peak flows for the 100 year Average Recurrence Interval (ARI) design storm event, can be found in Tables C2 and C3 in Appendix C for the ED and MPD scenarios respectively.

3.3.4 TP108 validation

A TP108 assessment of peak flows for each of the sub-catchments was carried out to review the outputs from the hydrological model. A comparison of the hydrologically modelled peak flows and the TP108 peak flows can be seen in Tables C2 and C3 in Appendix C for ED and MPD scenarios respectively. The results show that the model produces similar results to the expected peak flows using the TP108 graphical method. Note that this process should not be considered a model validation, which is discussed in Section 4.

3.4 Hydraulic model

3.4.1 Method used

A comprehensive hydraulic model was developed using DHI's Mike Flood software. The Mike Flood software dynamically couples three hydraulic models so that flows can pass from one model to another. The three models within the Mike Flood model are indicated below:

- In channel flow (Mike 11 model)
- Floodplains and overland flowpaths (Mike 21 Flexible-Mesh model)
- Stormwater reticulation (Mike Urban model).

3.4.1.1 Hydraulic model extents

Figure B3 in Appendix B shows the model schematisation (i.e. which areas are represented with which model) for the ED and MPD scenarios in relation to the Henderson catchment extent.

3.4.1.2 Modelled manholes, inlets and outlets

All manholes, inlets and outlets lying within the catchment boundary associated with pipes greater than 300 mm diameter that are located downstream of a hydrological loading node are included in the Mike Urban model. There are also a small number of pipes smaller than 300 mm that are included in the model to ensure connectivity between pipes.

Figure B3 in Appendix B identifies the stormwater assets that have been included in the model. Figure B2 shows the data source for the assets, including those assets which have required assumptions to build the model. Table 3.4 provides a summary of modelled manholes, inlets and outlets.

Table 3.4: Summary of modelled manholes, inlets and outlets

Hydraulic Model Components	ED and MPD
Total number of stormwater network system manholes (Mike Urban)	1,601
Total number of weirs (for loading nodes) (Mike Urban)	12
Total number of basins (conceptual ponds) (Mike Urban)	0
Total number of inlets and outlets (Mike Urban)	81

Note: This table does not include dummy manholes created for the purpose of distributing flows onto the Mike 21 mesh.

3.4.1.3 Modelled pipes, culverts and channels

All pipes and culverts located within the catchment boundary that are greater than 300 mm diameter and located downstream of a hydrological loading node are included in the Mike Urban model. There are also a small number of pipes smaller than 300 mm that are included in the model to ensure connectivity between pipes.

Figure B3 in Appendix B identifies the stormwater assets that have been included in the model. Figure B2 shows the data source for the assets, including those assets which have required assumptions to build the model.

Table 3.5 provides a summary of modelled pipes, culverts and channels.

Table 3.5: Summary of modelled pipes, culverts and channels

Hydraulic Model Components	ED and MPD
Total number of stormwater network system links (Mike Urban)	1,565
Total number culverts (Mike 11)	4
Total number bridges (Mike 11)	8
Total number open channels (Mike 11)	55

Note: This table does not include dummy pipes created for the purpose of distributing flows onto the Mike 21 mesh.

3.4.1.4 Modelled ponds, wetlands and other storage areas

The 16 ponds modelled are shown in Figure B1 in Appendix B and detailed in Table B3 in Appendix B. Checks were made to ensure the storage volumes within ponds were represented by the 2013 LiDAR and the 2D model grid resolution.

3.4.1.5 Modelled control structures

There are no control structures represented in the model.

3.4.2 Energy losses

Manning's roughness coefficients 'n' values were selected to represent energy losses due to surface friction. Manning's roughness values for the Mike 11 model were calculated in accordance with the SFMS by taking surface material, vegetation and channel sinuosity into consideration.

Roughness values were applied to branches in the Mike 11 HD file. Where roughness is not specified, a global Manning's n value of 0.03 has been applied in the Mike 11 HD file.

Table 3.6 shows the branches where roughness values differ from the global Manning's n value. A full list of the roughness values used in the Mike 11 network is provided in Table D1 in Appendix D. The locations of the network branches can be seen in Figure D1 in Appendix D. Manning's roughness values assigned to each culvert modelled in the Mike 11 network is detailed in Table B1 in Appendix B.

Table 3.6: Manning's roughness values used in Mike 11 model

Branch name (Model)	Chainage	Manning's n
CPDBridge	0 - 40.5	0.05
DwnhvnDr1	0 - 25.5	0.085
DwnhvnDr2	0 - 40.5	0.085
HamPl1	0 - 65	0.085
HamPl2	0 - 92.5	0.065
HamPl3	0 -58	0.03
HC1	0 - 1013	0.03
HC2	0 - 1352	0.03
HC3	0 - 5482	0.03
HCTR1	0 - 82	0.03
HCTr10	0 - 143.5	0.03
HCTr11	0 - 63	0.03
HCTr12	0 - 357	0.03
HCTr12Cul	0 -19	0.03
HCTr12Sub1	0 - 17.5	0.03
HCTr13	0 - 9.86	0.03
HCTr13	9.86	0.065
HCTr13	9.86 - 108.5	0.03
HCTr2	0 - 18.5	0.03
HCTr2	18.5	0.04
HCTr2	18.5 - 37	0.052
HCTr2	37 - 52.25	0.04
HCTr2	52.25 - 74.75	0.07475
HCTr2	74.75 - 87.5	0.04
HCTr2	87.75 - 119.5	0.0575
HCTr2	119.5 - 312.5	0.065
HCTr3	0 - 84	0.05
HCTr3	84 - 196.6	0.065
HCTr3	196.6 - 228.5	0.05
HCTr3Sub1	0 - 26.5	0.05
HCTr4	0 - 41	0.05
HCTr5	0 -130.5	0.065
HCTr6	0 - 29.5	0.05
HCTr6Cul	0 - 7.5	0.085
HCTr6Sub1	0 - 52.5	0.085
HCTr7	0 - 41.75	0.05
HCTr7	41.75 - 61.25	0.0575
HCTr7	61.25 - 71	0.025
HCTr8	0 - 112.5	0.07475

Branch name (Model)	Chainage	Manning's n
HCTr8	112.5 - 161	0.045
HCTr8Br	0 - 8	0.05
HCTr8Sub1	0 - 8.5	0.05
HCTr8Sub1Br	0 - 8	0.05
HCTr8Sub2	0 - 15	0.065
HCTr8Sub3	0 - 18	0.065
HCTr9	0 - 289	0.085
HCTr9	289 - 451	0.05
HCTr9	451 - 834	0.046
HCTr9Sub1	0 - 73	0.085
McLeodRd	0 - 56.5	0.065
Oratia1	0 - 95	0.0575
Oratia2	0 - 365	0.065
OratiaFB	0 - 7	0.065
OratiaTr	0 -189	0.085
OrPark	0 - 225	0.045
OrPark	225 - 290	0.052
OrPark	290 - 534.5	0.045
OrParkTr1	0 - 221	0.085
OruPt	0 - 149	0.085

The Mike Flood model applies roughness as a hydraulic control in the lateral link between the Mike 11 and Mike 21 Flexible-Mesh model. The default Mike Flood roughness (where n=0.05) was used for these links.

Roughness values (Manning's n) for the Mike 21 model are detailed in Table 3.7. These are applied using a spatially varying resistance dfs2 file in the Mike 21 model and are consistent with the approach recommended in the SFMS. Figure D2 in Appendix D shows the spatially varying resistance values applied for the catchment.

Table 3.7: Manning's roughness values used in Mike 21 model

Land use	**Manning's n**
Building footprints	0.345
Road	0.020
Built-up Area (settlement)	0.100
Herbaceous Saline Vegetation	0.110
Orchard, Vineyard or other perennial crop	0.050
Exotic Forest	0.149
Lake or Pond	0.043
Mangrove	0.053
Urban Parkland/Open Space	0.021

A Manning's n of 0.013 was selected for all pipes that were modelled in the Mike Urban model.

3.4.3 Energy losses due to turbulence

The constant eddy formulation was used in Mike 21 to represent the energy losses due to turbulence in Mike 21 (viscosity = 0.4 m^2/s).

3.4.4 Energy losses at manholes, inlets and outlets

The parameters used in Mike Urban to represent the head losses are listed in Table 3.8.

Table 3.8: Node head loss parameters

<table>
<tr><th colspan="2">Link Type</th><th>Mike Urban Node Head Loss Parameter</th></tr>
<tr><td rowspan="3">Manhole Nodes</td><td>Full Benching</td><td>"Mean Energy Approach" with K_m = 0.10 [1]</td></tr>
<tr><td>Half Benching</td><td>"Mean Energy Approach" with K_m = 0.30 [1]</td></tr>
<tr><td>No Benching</td><td>"Mean Energy Approach" with K_m = 0.50 [1]</td></tr>
<tr><td colspan="2">Culvert Inlet Nodes</td><td>Specified QH Relations [2]</td></tr>
<tr><td colspan="2">Culvert Outlet Nodes</td><td>"Mean Energy Approach" with K_m = 0.3 [1]</td></tr>
</table>

Notes:

1. These values are specified in SFMS, and are considered appropriate for catchment scale flood mapping purposes.
2. A QH relation was developed for each inlet based on the nomographs from the Culvert Manual (Ministry of Works and Development, 1978). The relationship was applied to the pipe.

3.5 Linkage between models

3.5.1 Linkage between Mike 11 and Mike 21

Mesh triangles in Mike 21 along both sides of the open channels were linked with Mike 11 branches. The link type used is “lateral link”, the structure method is “cell to cell”, the structure type is “weir”, and the structure source is “M21”. The default depth tolerance (0.1), weir coefficient (1.838) and friction (n = 0.05) were maintained.

3.5.2 Linkage between Mike 11 and Mike Urban

The link types used for discharge from Mike Urban to Mike 11 and vice versa are shown in Table 3.9.

Table 3.9: Linkage between Mike 11 and Mike Urban

Linkage	Link type
Mike Urban to Mike 11	Mike Urban outlets to Mike 11
Mike 11 to Mike Urban	Mike 11 water level boundary

3.5.3 Linkage between Mike 21 and Mike Urban

Manholes modelled in Mike Urban were divided into four categories:

1. Loading nodes - Network manholes that have catchments loaded
2. Dummy loading nodes - Dummy manholes setup to load catchments where there are no network manholes
3. Inlets and outlets
4. Manholes - Remaining network manholes that are neither loading nodes nor inlet/outlets.

All 'loading nodes' and 'dummy loading nodes' are sealed manholes which were not coupled to the Mike 21 FM grid. A dummy weir was created for each node, sharing the same ID as the sealed manhole. All dummy weirs were linked to Mike 21 FM using the link type 'weir to M21'. The dummy weir allows flows to spill onto the Mike 21 FM grid and prevents flows returning through the loading nodes back into the Mike Urban network.

Manholes are linked with Mike 21 FM, by using the link type 'inlet to M21'. The maximum inflow and outflow between the manhole and Mike 21 FM grid was set to 0.1 m^3/s (i.e. $Q_{max} = 0.1\ m^3/s$). For flows less than 0.1 m^3/s, flows are calculated using the orifice equation. Dummy manholes were used to transfer flows generated using the UHM in Mike Urban directly onto the Mike 21 FM grid by setting the maximum inflow and outflow to 0 m^3/s (i.e. $Q_{max} = 0\ m^3/s$) in the coupling file.

Inlets and outlets are linked with Mike 21 FM, by using the link type 'outlet to M21'.

3.6 Boundary conditions

3.6.1 Rainfall data

The 24 hour rainfall depths for 2 year ARI, 10 year ARI, 20 year ARI, 50 year ARI and 100 year ARI design storms were obtained from the AC TP108's design rainfall isohyets graphs, based on the centroid of the catchment. To incorporate climate change to 2090, the rainfall depth was then increased by respective percentages based on guidance from the SFMS.

Table 3.10 shows the 24 hour rainfall depths.

Table 3.10: 24-hour rainfall depths

Average Recurrence Interval (ARI)	24-hour rainfall depth (mm)	Percentage increase due to Climate Change	24-hour rainfall depth with Climate Change (mm)
2 year	81.5	9.0%	88.8
5 year	110	11%	122.1
10 year	130	13%	146.9
20 year	150	15%	172.5
50 year	180	16%	210.6
100 year	200	16%	234

The temporal rainfall patterns for the design storms was based on the SFMS, where the existing rainfall temporal pattern is the same as TP108.

The 24 hour rainfall depth with climate change was used to represent the MPD scenario.

3.6.2 Upstream boundary condition

The Henderson catchment commences downstream of the confluence of the Opanuku and Oratia streams. Auckland Council provided T+T with the twin stream Opanuku model flows at the Henderson catchment boundary in June 2017. Interpolation and smoothing of the inflow hydrographs were undertaken to remove instability spikes and extend the low flows either side of the 12 hour hydrographs to the 24 hour runtime used in the Henderson catchment model.

3.6.3 Downstream boundary condition

The downstream boundary condition was set at a constant tailwater level for the simulated storm event. Table 3.11 details the tide levels applied in the model.

For the Existing Development scenarios, the tidal level was based on the Mean High Water Spring (MHWS) for Henderson Creek, as specified in the SWFMS. For the Maximum Probable Development (MPD) scenarios, the Mean High Water Spring 10 percentile (MHWS10%ile) tidal level at Henderson Creek was used. The 10%ile level was derived from Appendix A of the NIWA report, Development of an updated Coastal Marine Area boundary for the Auckland Region (July 2012). 1.0 m was added to this level for the MPD scenario to take into account sea level rise due to future climate change, as specified in the SWFMS. The downstream boundary water levels are shown in Table 3.11.

Table 3.11: Downstream Boundary Tidal Level

Model Scenario	Tidal Boundary
ED – no sea level rise	1.74 m R.L
MPD with sea level rise	2.74 m R.L

3.7 Model limitations and assumptions

3.7.1 Hydrological model assumptions

During the modelling process, assumptions consistent with the SFMS and rainfall-runoff modelling in the Auckland region (as per TP108) were applied in order to represent the hydrology. Further information on the rainfall-runoff methodology can be found in section 5.2.1 of the SFMS. Specific assumptions related to the hydrological modelling are detailed in the relevant sections of this report (e.g. impervious percentage assumptions for different land use can be found in section 3.3.3).

The design rainfall events used in the modelling have ARIs of 2, 5, 10, 20, 50 and 100 years, as per the SFMS. Storms of greater intensity and duration than those modelled, or with a more adverse rainfall profile, may occur and may give rise to greater flooding than modelled.

The MPD imperviousness was based on the land use zoning in the AC Unitary Plan. This excludes areas where impervious coverage limits have already been exceeded. In such areas, the existing impervious is also used for the MPD scenario.

3.7.2 Hydraulic model assumptions

All assumptions specified in section 5.3 and 5.4 of the SFMS apply (e.g. relating to assumptions regarding the Saint Venant Equations).

The ponds, culverts and bridges are based on catchment understanding following survey carried out by Cardno between December 2015 and January 2016 and as-built drawings of the Te Atatū State Highway 16 interchange from 2014. Due to the rate of development in the catchment modifications to the ponds, culverts and bridges since this time may not be represented in the model.

No blockage has been assumed in manholes, pipes, culverts and catchpits in the stormwater system.

No sedimentation has been allowed for in the pipes, i.e. it is assumed that all pipes are capable of performing at full capacity.

No topographical changes, natural or otherwise have been allowed for in the modelling, including but not exclusive to geomorphological changes, volcanic activity and landslides.

The potential for change in asset condition over time is not represented.

Screens, orifice plates, control gates, valves, backflow preventers, choke points and other such obstructions and hydraulic controls are not modelled unless this data has been provided.

The bathymetry for modelling was developed using ground contours which were derived from LiDAR survey data. In urban areas the LiDAR data is stated to have a vertical accuracy of ± 0.2 m with a 95% confidence interval and a horizontal accuracy of ± 0.2 m with a 95% confidence interval.

Inlets were modelled as outlets in Mike Urban, with a QH relation assigned to each inlet based on nomographs from Culvert Manual (1978) by Ministry of Works and Development.

No account has been taken of the execution of any operations and maintenance works that may affect system performance (i.e. regular pipe cleaning may indicate a serious deficiency in the network affecting hydraulic conditions).

The asset data that was not captured or verified in the field as part of this project is assumed to be correct.

3.8 Initial model testing

Initial model testing of the catchment model was carried out on the Mike 11, Mike 21 and Mike Urban models separately, and then collectively for the Mike Flood model. Instability tests on the model included comparisons of flows with TP108 estimates, velocity checks in pipes and mass balance checks.

No local changes were made to the models for stabilisation. The mass balance of the model was assessed for the 100 year ARI model run and deemed to be appropriate (-2.6%).

4 Model validation and sensitivity

4.1 Model validation

There are five flow gauges surrounding the Henderson Catchment. However, due to these gauges being outside the catchment the flow gauge sites are not suitable for model validation.

CMP reports were provided by Auckland Council during the project MEDAR. These reports included an indication of flood frequency reported by property owners, however the information was over 10 years old and insufficiently detailed for use in a model validation process as there are no exact dates of flooding. Therefore, the information has not been used as part of the FHM process.

Flooding was reported within the catchment during a rainfall event on the 29th June 2016 however insufficient data was gathered to validate the model against this event.

Overall there is no data available/provided to calibrate and validate the hydraulic model. Additional future confidence in the models ability to represent flows and water levels would be gained if hydrometric monitoring of future large storm events is carried out followed by model calibration and additional validation.

4.2 Sensitivity analysis

A sensitivity analysis was undertaken on both the floodplain roughness and the hydrological inflows to the model in order to establish areas of uncertainty, areas sensitive to changes in land use and to support the lack of model validation.

Table 4.1 shows the four sensitivity runs that were simulated by adjusting the 100 year ARI MPD baseline scenario.

Overall the model showed a relatively low sensitivity to changes in land use, both in terms of hydraulic roughness and hydrological ground conditions, as described in this section.

Table 4.1: Sensitivity simulation matrix

Model component adjusted	Sensitivity run 1	Sensitivity run 2	Sensitivity run 3	Sensitivity run 4
	Increased roughness	Decreased roughness	Increased hydrological curve numbers	Decreased hydrological curve numbers
Mike21	Baseline Manning's n roughness values increased by 20%	Baseline Manning's n roughness values increased by 20%	-	-
Mike11	-	-	-	-
MikeUrban	-	-	Sub-catchment pervious area curve number values increased by 20% (capped at a maximum value of 98)	Sub-catchment pervious area curve number values decreased by 20%

4.2.1 Increased roughness

There were some increases to the modelled flood extent but these changes were minor and localised. Increases to the maximum flood depths were <0.15 m.

Localised increases to the flood extent occur near Yeovil Road (Figure 4.1) and Waipani Road (Figure 4.2), however these increases to the flood extent are largely outside of the 100 year ARI MPD floodplain in overland flowpaths.

At Stokes Avenue there is a localised increase to the flood extent and flood depths of up to 0.12 m at 9 Stokes Avenue, as shown in Figure 4.3.

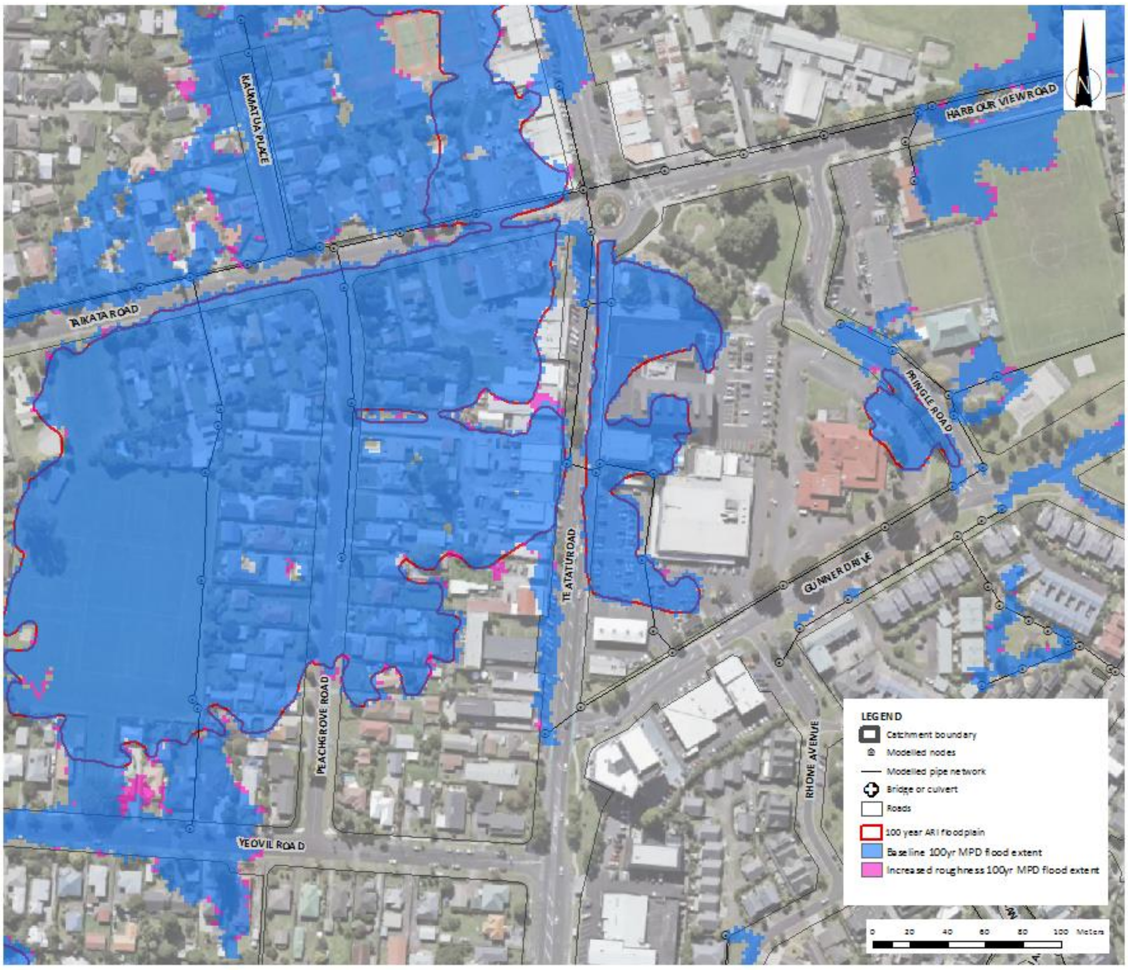

Figure 4.1: Increased roughness sensitivity flood extent: Yeovil Road

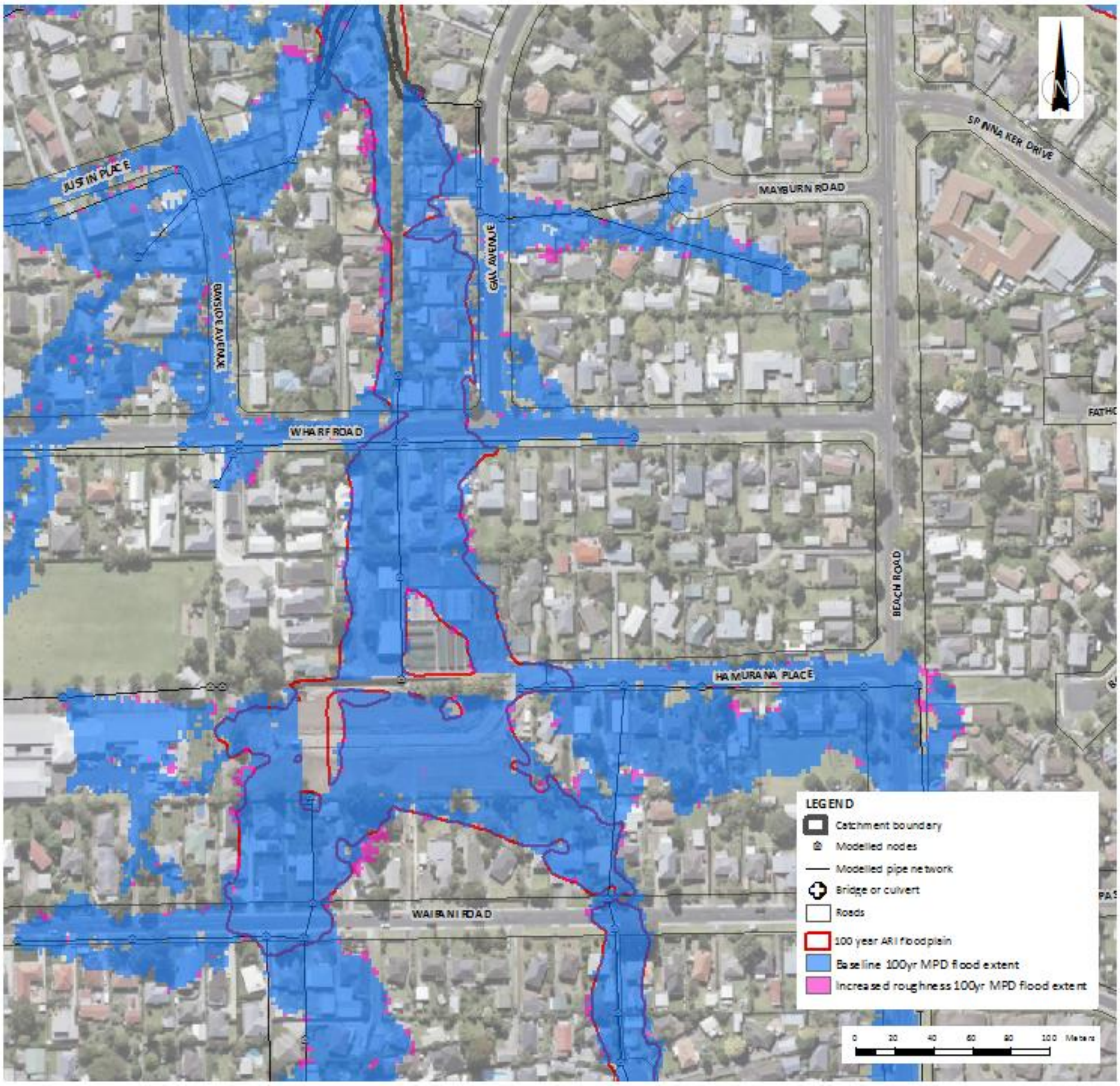

Figure 4.2: Increased roughness sensitivity flood extent: Waipani Road

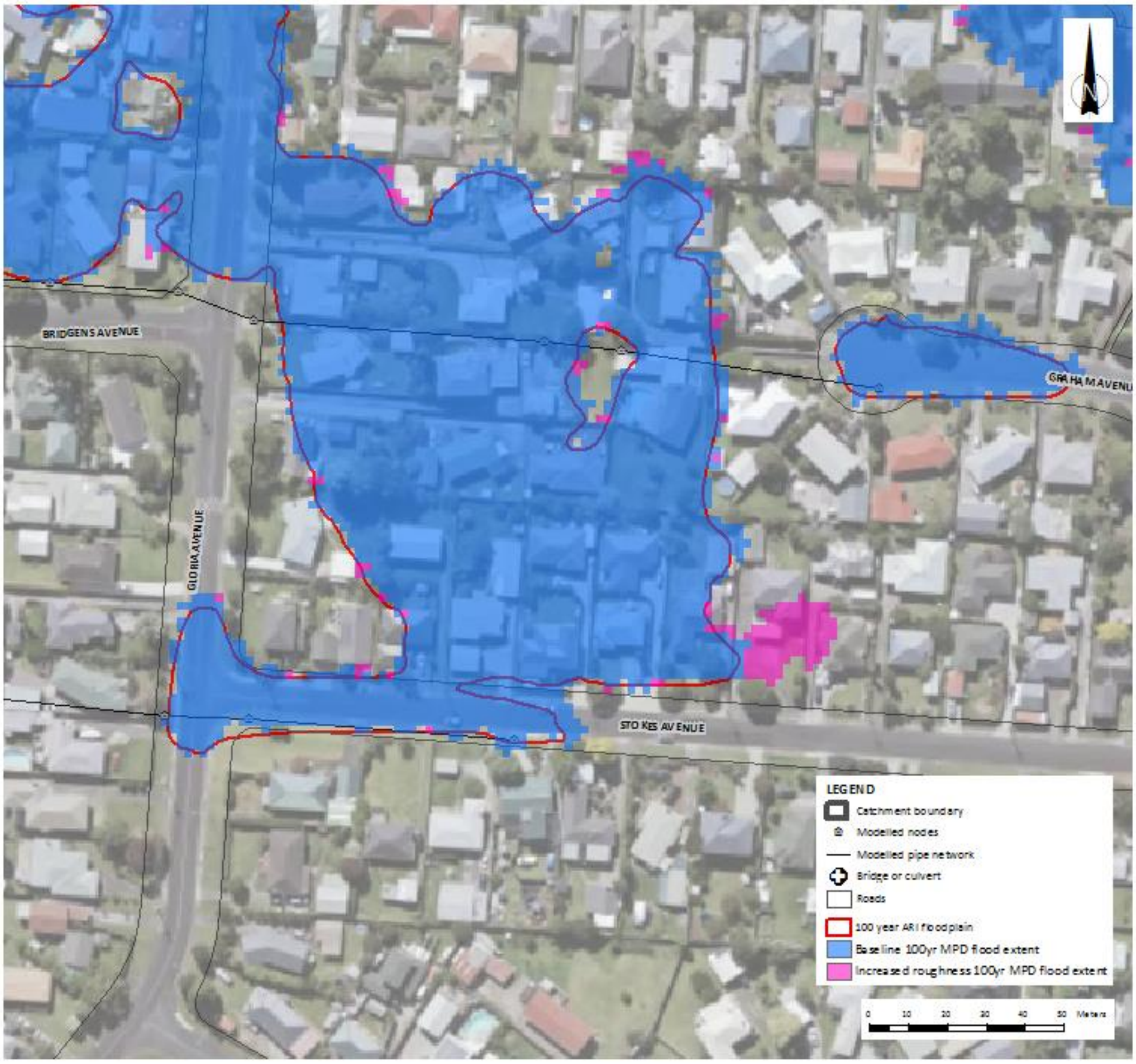

Figure 4.3: Increased roughness sensitivity flood extent: Stokes Avenue

4.2.2 Decreased roughness

Decreases to the overall flood extent were very minor and outside of the 100 year ARI MPD floodplain.

The only noticeable change to the flood extent within the floodplain occurred at The Concourse Industrial Park where maximum flood depths were reduced by up to 0.16 m. This did not remove any of the industrial buildings from the flood extent however, as shown in Figure 4.4.

Figure 4.4: Decreased roughness sensitivity flood extent: The Concourse

4.2.3 Increased hydrological curve numbers

Increases to the flood extent as a result of increased CN were minor and generally negligible or outside of the 100 year ARI MPD floodplain.

There were localised increases to the flood extent at 4 Hereford Street which was inundated with a maximum flood depth of 0.13 m, as shown in Figure 4.5.

There were also increases to the flood extent along minor overland flowpaths across properties at Enderby Drive (as shown in Figure 4.6) and Mcleod Road (as shown in Figure 4.7). These changes in the flood extent do not affect the mapping of the floodplains.

Figure 4.5: Increased hydrological curve number sensitivity flood extent: Hereford Street

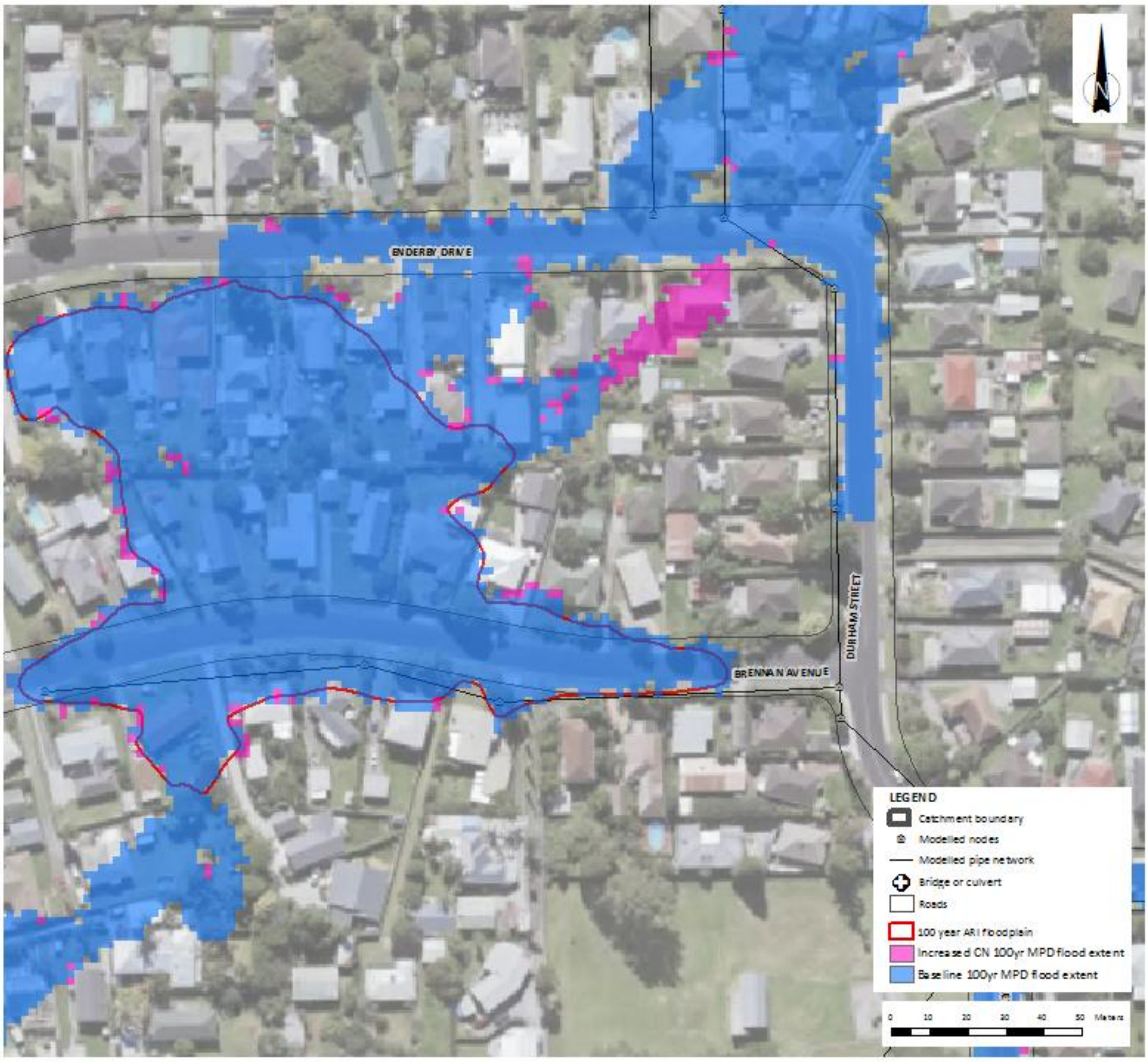

Figure 4.6: Increased hydrological curve number sensitivity flood extent: Enderby Drive

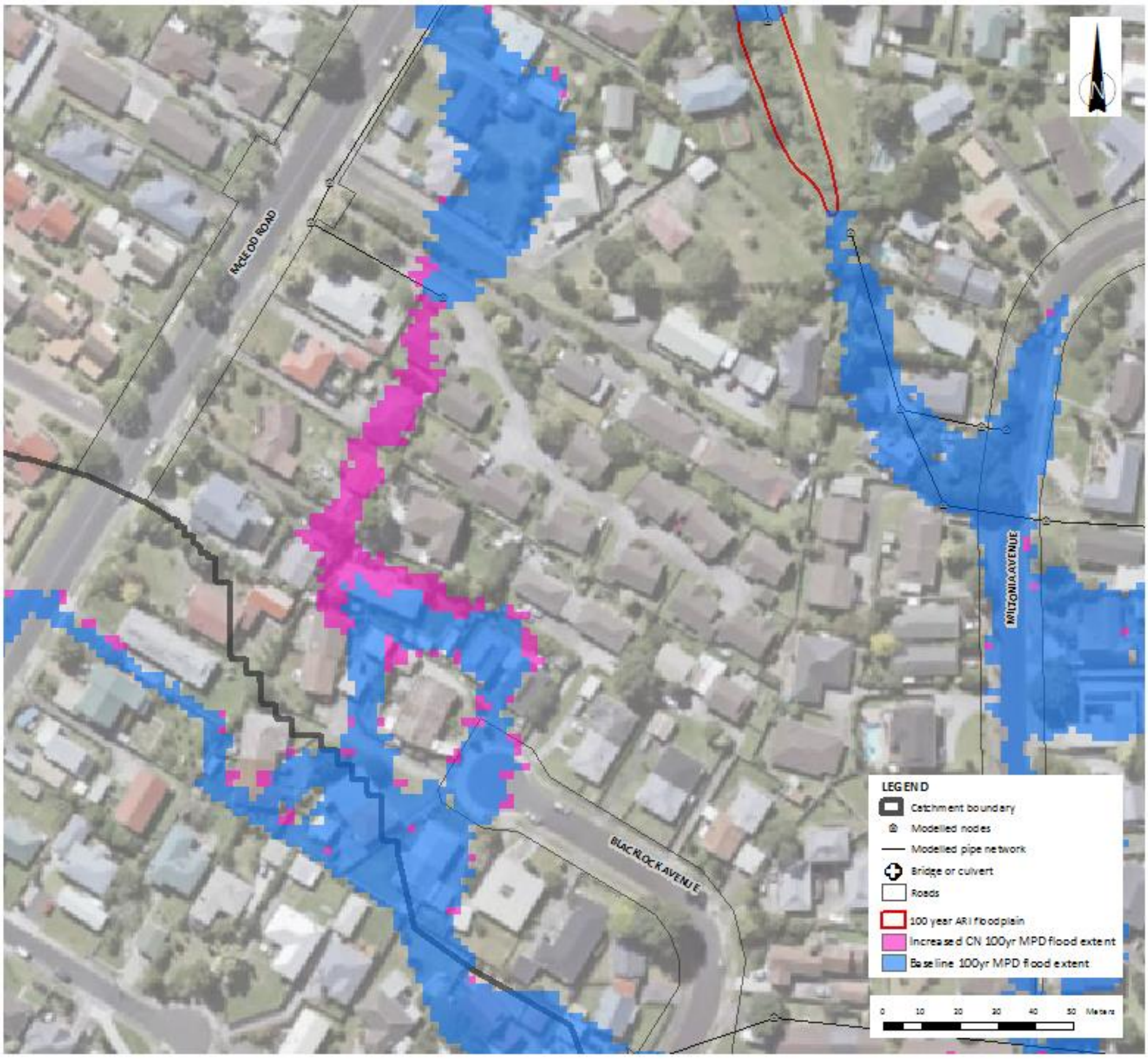

Figure 4.7: Increased hydrological curve number sensitivity flood extent: Mcleod Road

4.2.4 Decreased hydrological curve numbers

Decreases to the flood extent were minor and all outside of the 100 year ARI MPD floodplain. Overland flowpath flooding was reduced at properties on Blacklock Avenue, with depth decreases of up to 0.06 m (as shown in Figure 4.8).

Figure 4.8: Decreased hydrological curve number sensitivity flood extent: Blacklock Road

4.2.5 Changes to the Auckland Unitary Plan

The Maximum Probable Development (MPD) scenario catchment land use was derived from the Auckland Unitary Plan (AUP, dated 2014) which was provided during the model build stage. There are localised differences within the Henderson catchment between the Unitary Plan layer dated 2014 and the current Unitary Plan layer on Auckland Council Geomaps (October 2018).
The effect of these differences on the resulting impervious assumptions for the MPD scenario are shown in Appendix A, Figure A5.

The figure shows that in some areas (shown in green) the imperviousness in the MPD scenario is overestimated when compared with the latest Unitary Plan. There are also small localised areas near the Te Atatū State Highway 16 interchange (shown in red) where the MPD scenario imperviousness is underestimated when compared with the latest Unitary Plan.

The hydrological curve number sensitivity analysis results showed either negligible or minor changes to the flood extent (outside of the 100 year ARI MPD floodplain) at the areas where the differences between the Unitary Plan layers are identified. Figure 4.9 shows the hydrological curve number sensitivity analysis results at the Te Atatū State Highway 16 interchange, where the biggest difference in the unitary plan layer was observed.

Updating the MPD scenario to reflect imperviousness assumptions in the latest Unitary Plan layer is therefore unlikely to effect the 100 year MPD floodplain.

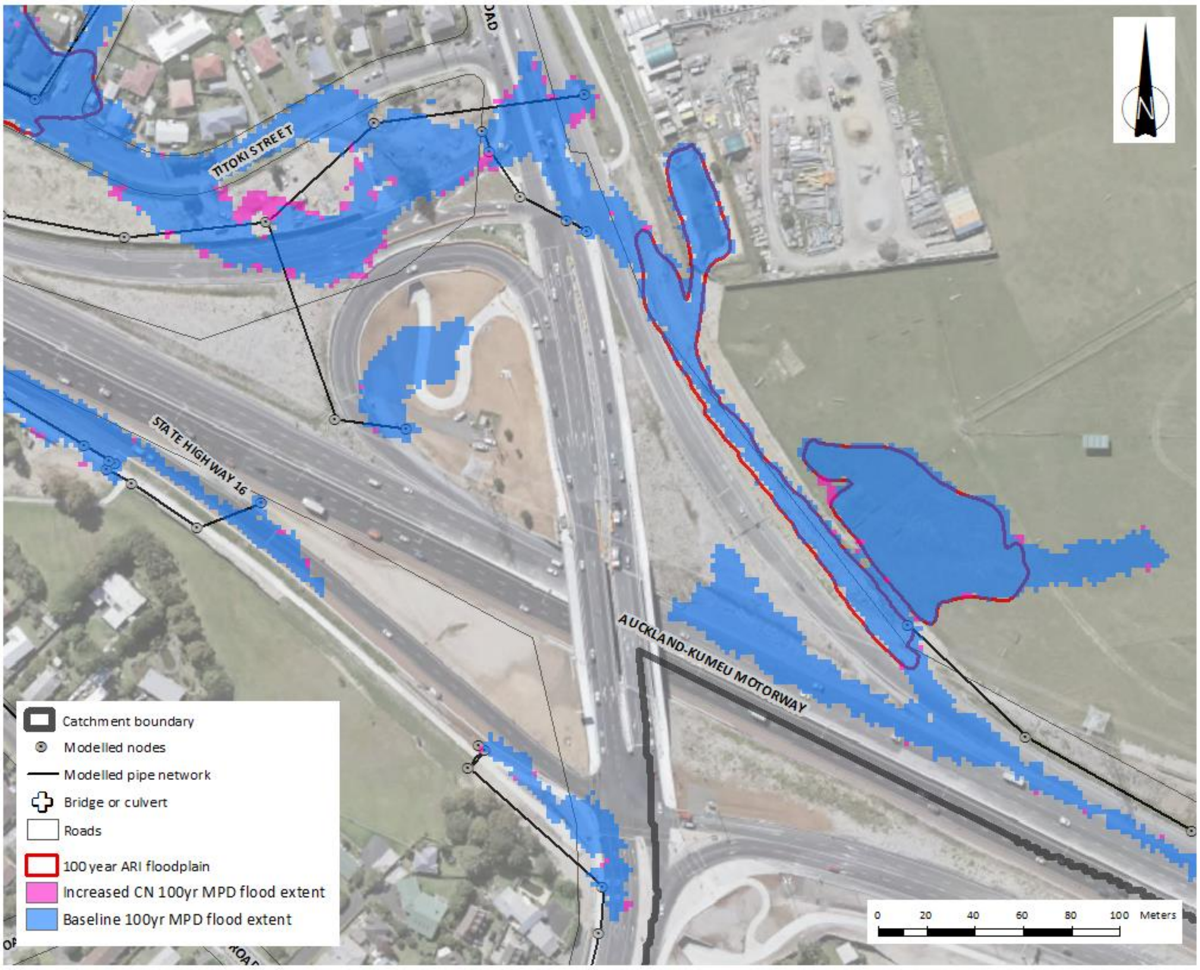

Figure 4.9: Increased hydrological curve number sensitivity flood extent: Te Atatū State Highway 16 interchange

5 System performance assessment

The system performance identifies the level of performance provided by the stormwater system. The system performance consists of the following aspects:

- The capacity of the existing stormwater drainage system
- An assessment of the level of service provided comparisons with the desired level of service, and identified network sections that do not provide the desired level of service
- Floodplain, flood hazard and flood sensitive area mapping to identify significant flood hazard areas and habitable floors at risk of flooding
- Design flows and water levels for nominated design storm events and land use development/future rainfall scenarios.

The following sub sections are discussed in accordance with the template provided in SFMS (November, 2011), noting that there are some repeats with previous sections.

5.1 Model scenarios and simulations

Table 5.1 shows the simulation matrix for Existing Development (ED) and Maximum Probable Development (MPD). There are 12 scenarios in total that have been modelled. Table E1 in Appendix E details the modelling files for the design storm scenarios described in Table 5.1.

The land use and rainfall is discussed in Sections 2 and 3, respectively.

Table 5.1: Simulation matrix

Simulation	Land Use	Design Storm Event	Rainfall	Hydraulic Structure
1	ED	2 year ARI	Existing 2 year	Existing
2	ED	5 year ARI	Existing 5 year	Existing
3	ED	10 year ARI	Existing 10 year	Existing
4	ED	20 year ARI	Existing 20 year	Existing
5	ED	50 year ARI	Existing 50 year	Existing
6	ED	100 year ARI	Existing 100 year	Existing
7	MPD	2 year ARI	Future 2 year	Existing
8	MPD	5 year ARI	Future 5 year	Existing
9	MPD	10 year ARI	Future 10 year	Existing
10	MPD	20 year ARI	Future 20 year	Existing
11	MPD	50 year ARI	Future 50 year	Existing
12	MPD	100 year ARI	Future 100 year	Existing

5.2 Water balance of the catchment

The water balance of rainfall volume versus catchment runoff is compared in Table 5.2.

Table 5.2: Water balance of the catchment (ED and MPD)

Model Scenario	Catchment Rainfall Volume (m^3)	Catchment Runoff Volume (m^3)	Catchment Runoff Volume in %
2 year ED	902,588	624,395	69%
5 year ED	1,218,218	884,878	73%
10 year ED	1,439,712	1,072,940	75%
20 year ED	1,661,206	1,264,211	76%
50 year ED	1,993,447	1,555,956	78%
100 year ED	2,214,941	1,752,998	79%
2 year MPD	983,434	688,148	70%
5 year MPD	1,352,221	997,991	74%
10 year MPD	1,626,874	1,232,803	76%
20 year MPD	1,910,387	1,479,236	77%
50 year MPD	2,332,333	1,848,644	79%
100 year MPD	2,591,481	2,081,052	80%

5.3 Capacity of the existing primary pipe system

Table 5.3 summarises the performance of the existing stormwater pipe system for the different modelled scenarios. The results for each Mike Urban node and link can be seen in Tables E2 and E3 in Appendix E for the two scenarios (ED, MPD). The maximum system pipe flow results (ED and MPD) can be seen in Table E4.

Table 5.3: Summary of the performance of the existing stormwater pipe system

<table>
<tr><th>Capacity Assessment Basis</th><th>Flow Condition</th><th>Scenario</th><th>> 2yr ARI</th><th>> 5yr ARI</th><th>> 10yr ARI</th><th>> 20yr ARI</th><th>> 50yr ARI</th><th>> 100yr ARI</th></tr>
<tr><td rowspan="2">Maximum pipe flow compared to pipe full flow capacity. Percentage of total catchment pipes with capacity available under each ARI event</td><td rowspan="2">Free full flow</td><td>ED</td><td>78%</td><td>70%</td><td>67%</td><td>66%</td><td>63%</td><td>62%</td></tr>
<tr><td>MPD</td><td>75%</td><td>68%</td><td>67%</td><td>64%</td><td>63%</td><td>62%</td></tr>
<tr><td rowspan="2">Pipe downstream maximum depth compared to pipe diameter or height. Percentage of total catchment pipes with capacity available under each ARI event</td><td rowspan="2">Backwater effect</td><td>ED</td><td>34%</td><td>25%</td><td>21%</td><td>18%</td><td>15%</td><td>14%</td></tr>
<tr><td>MPD</td><td>30%</td><td>20%</td><td>18%</td><td>15%</td><td>12%</td><td>11%</td></tr>
<tr><td>Total pipe capacity considering full flow & back water effect.</td><td>Worst of above 2 conditions</td><td>ED</td><td>30%</td><td>20%</td><td>16%</td><td>14%</td><td>11%</td><td>9%</td></tr>
</table>

Capacity Assessment Basis	Flow Condition	Scenario	> 2yr ARI	> 5yr ARI	> 10yr ARI	> 20yr ARI	> 50yr ARI	> 100yr ARI
Percentage of total catchment pipes with capacity available under each ARI event	('Total Capacity')	MPD	25%	16%	13%	11%	8%	7%
Maximum system pipe flow compared to pipe full flow capacity. Percentage of total catchment pipes with capacity available under each ARI event	Free full flow - No upstream restriction	ED	66%	54%	45%	39%	30%	27%
		MPD	61%	46%	38%	31%	25%	22%
Frequency of Manholes overflowing	Percentage of Manholes overtopping for each event	ED	22%	29%	36%	40%	46%	49%
		MPD	25%	35%	41%	46%	52%	54%

Thematic maps for the MPD pipe capacity assessments described in Table 5.3 can be seen in Appendix F.

5.4 Capacity of the existing culverts and bridges

The results of the capacity assessment of culverts and bridges are summarised in Table 5.4 for the two scenarios (ED, MPD) using a 10 year ARI and 100 year ARI design standard.

The culverts and bridges were assessed in two ways:

1. The flow that causes headwater to exceed soffit level in 10 year ARI
2. The flow that causes overtopping of the road (or other asset) above the culvert in 100 year ARI.

The detailed results are provided in Tables E7 and E8 in Appendix E. A summary of culvert and bridge capacity before overtopping occurs in provided in Table 5.4.

Table 5.4: Summary of capacity assessment of existing culverts and bridges

Scenario	Percentage meeting standard		Culvert or bridge not meeting standard	
	10 year ARI	100 year ARI	10 year ARI	100 year ARI
ED	42%	42%	• Jack Colvin Park – (SAP ID: 2000601411, Model Branch: HCTr9Cul, Chainage: 3.5) • Sherwood Park (SAP ID: 2000016616, Model Branch: HCTr6Cul, Chainage: 3.5) • Coletta Esplanade Bridge Model Branch: HCTr3, Chainage: 124.5)	• Jack Colvin Park – (SAP ID: 2000601411, Model Branch: HCTr9Cul, Chainage: 3.5) • Sherwood Park (SAP ID: 2000016616, Model Branch: HCTr6Cul, Chainage: 3.5) • Coletta Esplanade Bridge Model Branch: HCTr3, Chainage: 124.5)

Scenario	Percentage meeting standard		Culvert or bridge not meeting standard	
	10 year ARI	100 year ARI	10 year ARI	100 year ARI
			• Bridge near Edmonton Road 1 (Model Branch: TASthPBr, Chainage: 4) • Bridge near Edmonton Road 2 (Model Branch: TASthPBr, Chainage: 21)	• Bridge near Edmonton Road 1 (Model Branch: TASthPBr, Chainage: 4) • Bridge near Edmonton Road 2 (Model Branch: TASthPBr, Chainage: 21)
MPD	42%	58%	• Jack Colvin Park – (SAP ID: 2000601411, Model Branch: HCTr9Cul, Chainage: 3.5) • Sherwood Park (SAP ID: 2000016616, Model Branch: HCTr6Cul, Chainage: 3.5) • Coletta Esplanade Bridge Model Branch: HCTr3, Chainage: 124.5) • Bridge near Edmonton Road 1 (Model Branch: TASthPBr, Chainage: 4) • Bridge near Edmonton Road 2 (Model Branch: TASthPBr, Chainage: 21)	• Jack Colvin Park – (SAP ID: 2000601411, Model Branch: HCTr9Cul, Chainage: 3.5) • Sherwood Park (SAP ID: 2000016616, Model Branch: HCTr6Cul, Chainage: 3.5) • Coletta Esplanade Bridge Model Branch: HCTr3, Chainage: 124.5) • Bridge near Edmonton Road 1 (Model Branch: TASthPBr, Chainage: 4) • Bridge near Edmonton Road 2 (Model Branch: TASthPBr, Chainage: 21) • Cranwell Park Footbridge (Model Branch: OratiaFB, Chainage 3.5) • Sherwood Park Footbridge (SAP ID: 2000805130, Model Branch: HCTr8Sub1Br, Chainage 4)

5.5 Floodplain mapping

Floodplain maps for the MPD scenario can be seen in Figure G1 in Appendix G. The figure shows the 10 and 100 year ARI for the following:

- Floodplains:
    1. Areas where overland flow is greater than 2 m^3/s or
    2. Flood prone areas (depressions with a surface area greater than 500 m^2, a maximum depth greater than 0.3 m and a volume greater than 50 m^2)
    3. Areas downstream of criteria 1 and 2, where the overland flow remains above 0.5 m^3/s.
- Overland flow paths:
    1. Major flowpath; flow is greater than 0.5 m^3/s and less than 2 m^3/s during a 100 year ARI MPD scenario
    2. Minor flowpath; flow is greater than 0 m^3/s and less than 0.5 m^3/s during a 100 year ARI MPD scenario.

5.6 Flood hazard mapping

The flood hazard map for the MPD scenario can be seen in Figure G2 in Appendix G. The flood hazard is classified for the 100 year ARI scenario.

The flood hazard classification is based on SFMS, where:

1 Potential Hazard, 0.05 m < depth < 0.1 m

2 Minor Hazard, 0.1 m ≤ depth < 0.3 m, and velocity < 2.0 m/s

3 Significant Hazard, depth ≥ 0.3 m, and depth ≥ 0.1 m and velocity ≥ 2.0 m/s.

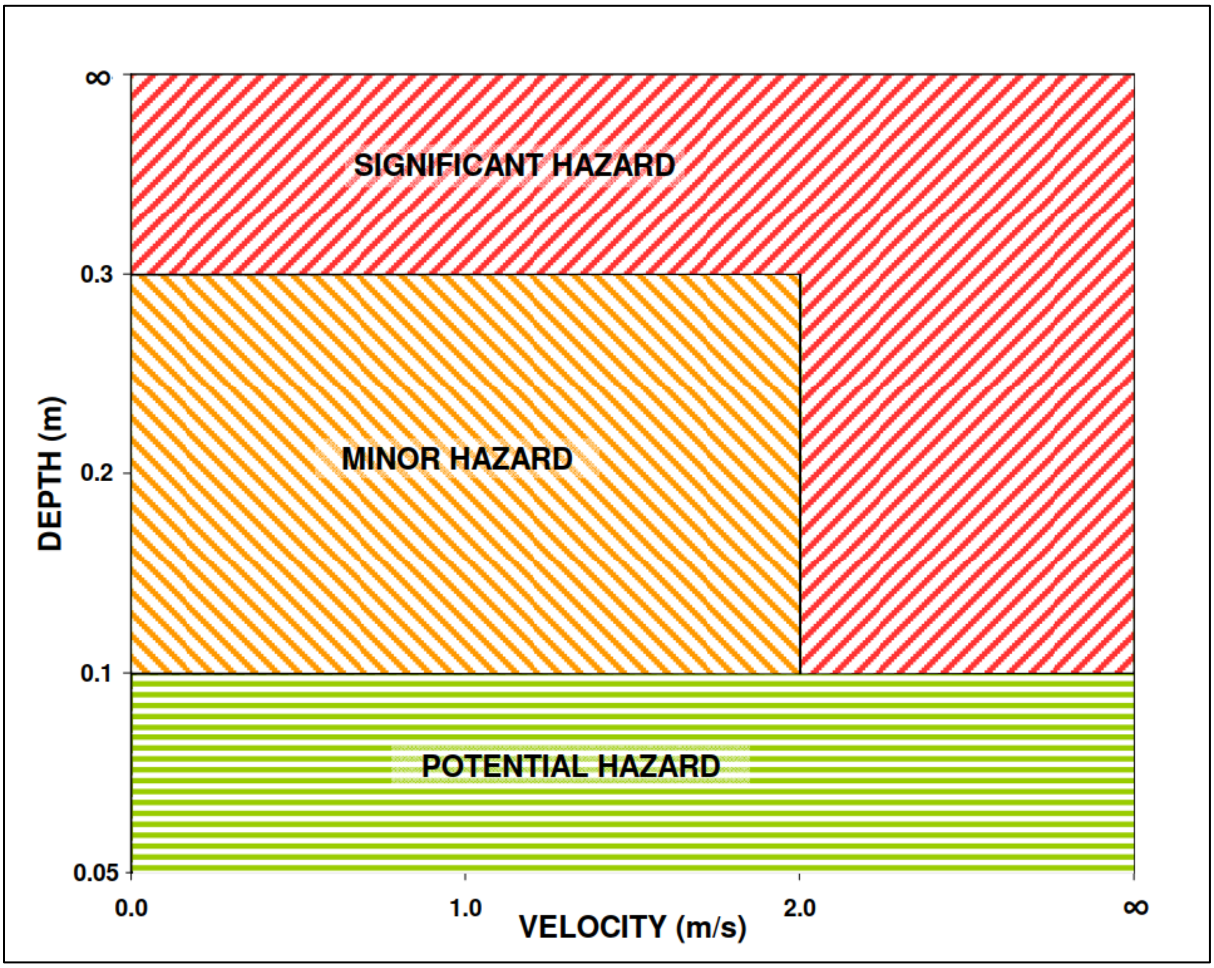

Figure 5.1: Depth – Velocity criteria for hazard classification in the Auckland SFMS (2011)

6 Flood damage assessment

6.1 Results

The Flood Damage Assessment (FDA) was undertaken in accordance with Metrowater's FDA Guideline Report (November 2008).

There were 862 residential buildings and 127 commercial buildings surveyed by Cardno between February and May 2018. Surveyed floor levels and flood levels for each building can be seen in Appendix H. All building floor levels were surveyed where the building was located within the 100 year ARI MPD floodplain. It should be noted that this assessment therefore excludes buildings that lie within an overland flowpath.

There were 14 residential buildings that were unable to be accessed for survey, primarily due to owner refusal. For the purposes of the flood damage assessment, the floor levels at these properties have been estimated by adding 150 mm (minimum building regulation floor level) to the maximum ground level within the footprint. Buildings with assumed floor levels are identified in the figures in Appendix G and the tables in Appendix H.

The results presented in Figure 6.1 are based on residential damage curves and commercial damage curves for retail business buildings. Out of the 862 residential buildings surveyed, 197 buildings were identified as being non-habitable. These buildings have not been included in the flood damage assessment but their frequency of flooding is shown in Figure G1 in Appendix G.

Model results show that there are 177 residential and 37 commercial buildings at risk from flooding above floor level in the 100 year ARI MPD event across the entire catchment.

Figure 6.1 shows the Expected Annual Flood Damage Curves for the 2, 5, 10, 20, 50 and 100 year ARI (or 50%, 20%, 10%, 5%, 2% and 1% Annual Exceedance Probability (AEP) respectively) rainfall events for ED and MPD scenarios. A linear interpolation is assumed between the 2 year ARI (50% AEP) damage and the zero damage amount at 100% AEP.

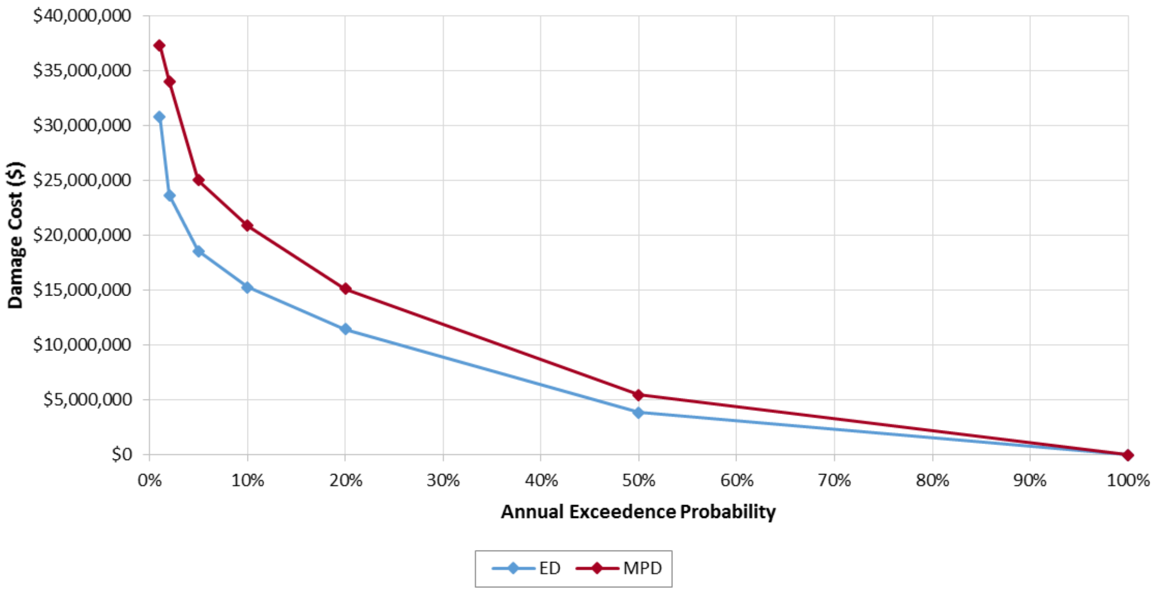

Figure 6.1: Expected Annual Flood Damage Curves for ED and MPD

Table 6.1 and Table 6.2 tabulate the detail flood damage values for each scenario in terms of residential and commercial buildings.

Table 6.1: Summary of predicted flood damage for all ED design storm events

	Residential		Commercial		Total Damage	
Return Period	**Values ($)**	**No. of Buildings**	**Values ($)**	**No. of Buildings**	**Values ($)**	**No. of Buildings**
2 year	3,461,500	24	423,000	5	3,884,500	29
5 year	6,623,500	50	4,829,000	10	11,452,500	60
10 year	10,220,000	70	5,028,000	11	15,248,000	81
20 year	12,923,500	87	5,630,000	15	18,553,500	102
50 year	15,873,000	112	7,785,500	25	23,658,500	137
100 year	20,735,500	142	10,080,000	29	30,815,500	171

Table 6.2: Summary of predicted flood damage for all MPD design storm events

	Residential		Commercial		Total Damage	
Return Period	**Values ($)**	**No. of Buildings**	**Values ($)**	**No. of Buildings**	**Values ($)**	**No. of Buildings**
2 year	4,955,000	36	518,500	5	5,473,500	41
5 year	10,296,000	74	4,846,000	12	15,142,000	86
10 year	15,031,000	104	5,838,500	17	20,869,500	121
20 year	17,837,500	126	7,185,500	23	25,023,000	149
50 year	23,448,500	161	10,590,000	33	34,038,500	194
100 year	26,375,000	177	10,945,500	37	37,320,500	214

The Average Annual Damage (AAD) and Net Present Value (NPV) were calculated as per Metrowater's FDA Guidelines for both scenarios Table 6.3.

Table 6.3: Predicted Average Annual Damage

Scenario	Average Annual Damage	Net Present Value
ED	$6,665,443	$81,541,500
MPD	$9,024,510	$110,401,000

The table above also shows the present value of estimated flood damage over a period of 50 years. This rate represents the 'opportunity cost' of not having to spend the expected annual damage and assumed an interest rate of 8%.

6.2 Assumptions and limitations

The maximum flood level within both the floodplain and the building footprints has been compared to the surveyed or assumed floor level to determine to depth of inundation used in the analysis. This assumption is accurate in flooding areas where the water level is relatively flat and the depth of inundation is consistent over the building footprint. However, where the water level varies over the building footprint, the predicted level of inundation relative to the floor level may be conservative.

7 Flooding Issues

Within the Henderson catchment flooding from the model results is widespread with 42 building floor levels being below the maximum 2 year ARI MPD water level.

Only 25% of the pipe network has capacity in the 2 year ARI MPD scenario, primarily due to backwater effects, which suggests that the impacts of downstream conveyance issues are being propagated upstream throughout the catchment.

18 hydraulic issue areas are identified located across the Te Atatū Peninsula, Te Atatū south and Henderson suburbs. These issue areas are mapped in Figure G3 in Appendix G.

Table 7.1 provides a summary of the total number of commercial and habitable buildings at risk within each issue area. The flooding mechanism for each individual area and the number buildings at risk for each ARI MPD event is described in sections 7.1, 7.2 and 7.3, which relate to the Te Atatū peninsula, Te Atatū south and Henderson areas respectively.

Although the Peachgrove Road and Te Atatū Road area has the largest number of buildings within the 100 year MPD floodplain, the majority of these buildings have floor levels above the peak modelled water level. The Coletta/Edmonton Road and Orukuwai Point Creek areas contain the largest number of commercial and habitable floor level flooding in the 100 year ARI MPD event.

Table 7.1: Flood Issue areas – summary of buildings at risk

Suburb	Flood Issue Area	Number of Commercial and Habitable buildings		
		Within 100 year MPD Floodplain	100 year MPD floor level flooding	Floor level within 500mm of 100 year MPD peak level
Te Atatū Peninsula	1 - Orukuwai Point Creek	68	25	27
	2 - Wharf Road area	17	3	8
	3 - Brennan Avenue and Clinton Avenue	34	8	9
	4 - Capstan Court	5	5	0
	5 - Kervil Avenue/Matipo Road	68	16	30
	6 – Peachgrove Road and Te Atatū Road (542-633)	124	12	72
	7 – Barberry Lane and Bridgens Avenue	60	10	24
	8 – Graham Avenue	22	6	4
	9 – Kotuku Street	21	8	7
	10 – Noall Street area	11	5	3
Te Atatū south	11 – Area around State Highway 16 Te Atatū interchange	16	2	5
	12 - Sylvan crescent	9	2	4
	13 - Coletta Road and Edmonton Road area	59	29	10
	14 – Mcleod Road to Central Park Drive	55	22	23
	15 – Chilcott Road	37	21	5

Suburb	Flood Issue Area	Number of Commercial and Habitable buildings		
		Within 100 year MPD Floodplain	100 year MPD floor level flooding	Floor level within 500mm of 100 year MPD peak level
Henderson	16 – Lincoln Road/Concourse Industrial Park	60	14	28
	17 – Waipareira Avenue	2	1	1
	18 – Buscomb Avenue and Epping Road	21	15	4

7.1 Te Atatū Peninsula suburb

7.1.1 Area 1 - Orukuwai Point Creek

There are a total of 68 commercial and habitable and 17 non-habitable building footprints within the 100 year ARI MPD floodplain within this area. Table 7.2 shows the number of commercial and habitable building floor levels at risk for each ARI MPD event in this area.

The system performance assessment in this area showed that in the 2 year ARI MPD event, 44% of the stormwater pipes are over capacity in free full flow and 96% are over capacity from backwater effects.

The small stream downstream of Wharf Road (Mike 11 1D channel branch: OruPt) is also over capacity with flood levels up to 0.5 m above the banks in the 100 year ARI MPD event.

Table 7.2: Buildings at risk of flooding - Area 1 - Orukuwai Point Creek

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							1
Habitable	10	6	4	1	3	1	26

7.1.2 Area 2 - Wharf Road area

The key stormwater pipes draining this area are over capacity in the 2 year ARI MPD event, primarily due to backwater effect. Table 7.3 shows the number of commercial and habitable building floor levels at risk for each ARI MPD event in this area.

Table 7.3: Buildings at risk of flooding - Area 2 - Wharf Road area

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial	1				1		1
Habitable		1					7

7.1.3 Area 3 - Brennan Avenue and Clinton Avenue

The key stormwater pipes draining this area are over capacity in the 2 year ARI MPD event, primarily due to backwater effect. Table 7.4 shows the number of commercial and habitable building floor levels at risk for each ARI MPD event in this area.

Table 7.4: Buildings at risk of flooding – Area 3 Brennan Avenue and Clinton Avenue

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							
Habitable		3	2		3		9

7.1.4 Area 4 - Capstan Court

Table 7.5 shows that there are 5 habitable building floor levels at risk from the 5 year ARI MPD event onwards in this area. The system performance assessment in this area showed that 72% of the stormwater pipes have a capacity of < 2 year ARI MPD due to backwater effects.

Table 7.5: Buildings at risk of flooding - Area 4 - Capstan Court

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							
Habitable		5					

7.1.5 Area 5 – Kervil Avenue/Matipo Road

The stormwater pipes are over capacity in this area with 85% having a pipe capacity < 2 year ARI MPD due to backwater. Table 7.6 shows the number of commercial and habitable building floor levels at risk for each ARI MPD event in this area.

Table 7.6: Buildings at risk of flooding - Area 5 – Kervil Avenue/Matipo Road

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							1
Habitable	9	2	2		2		29

7.1.6 Area 6 – Peachgrove Road and Te Atatū Road (542-633)

Table 7.7 shows that there are 84 properties at risk of floor level flooding in this area. In the 2 year ARI MPD event 100% of the stormwater pipes modelled are over capacity due to backwater effects and 47% are over capacity in in free full flow conditions.

Table 7.7: Buildings at risk of flooding - Area 6 – Peachgrove Road and Te Atatū Road (542-633)

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial		2		2	2		50
Habitable	2	1	1	1		1	22

7.1.7 Area 7 – Barberry Lane and Bridgens Avenue

There are three main stormwater pipelines that drain runoff from this area to Henderson Creek and all are over capacity from both free flow conditions and backwater in the 2 year ARI MPD event. Table 7.8 shows a number of properties are at risk from floor level flooding here.

Table 7.8: Buildings at risk of flooding - Area 7 – Barberry Lane and Bridgens Avenue

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							
Habitable	2	3	2	2	1		24

7.1.8 Area 8 – Graham Avenue

There are 10 habitable buildings at risk from floor level flooding in this area, as shown in Table 7.9. One of the key mechanisms of flooding here is due to the build-up of floodwater on the upstream side of Old Te Atatū road which is slightly embanked/raised. The stormwater pipeline beneath the road has a free flow capacity of 5 – 10 year ARI in the MPD scenario.

Table 7.9: Buildings at risk of flooding - Area 8 – Graham Avenue

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							
Habitable	3	1	2				4

7.1.9 Area 9 – Kotuku Street

The stormwater pipeline through this area has a capacity of < 2 year ARI in the MPD scenario, largely due to backwater effects. Table 7.10 shows that there are 7 commercial and 8 residential floor levels at risk here.

Table 7.10: Buildings at risk of flooding - Area 9 – Kotuku Street

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial	3		1	1			2
Habitable	2				1		5

7.1.10 Area 10 – Noall Street area

Table 7.11 shows the buildings at risk from floor level flooding in this area. The habitable property at risk in the 2 year ARI MPD event (41 Taipari Road) has a surveyed floor level of 2.89 m R.L which is only 150 mm above the Mean High Water Spring 10 percentile (MHWS10%ile) tidal level plus sea level rise which is 2.74 R.L. There are 15 stormwater pipes with downstream invert levels below 2.74 m R.L and the primary mechanism of flooding is therefore due to backwater effects.

Table 7.11: Buildings at risk of flooding - Area 10 – Noall Street area

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial			2	1	1		1
Habitable	1						2

7.2 Te Atatū South suburb

7.2.1 Area 11 – Area around State Highway 16 Te Atatū interchange

Table 7.12 shows that there are 2 habitable buildings at risk in the 5 year ARI MPD, which are both located on the southern side of the highway (McCormick Road and Milich Terrace) and there are 5 habitable buildings within 500mm of the peak 100 year ARI MPD water level on the northern side of the highway.

The system performance assessment shows that the stormwater pipes around the buildings at risk on the southern side of the highway have < 2 year ARI MPD capacity due to backwater effect.

On the Northern side of the highway the culvert in Jack Colvin Park – (SAP ID: 2000601411, Model Branch: HCTr9Cul, Chainage: 3.5) does not have capacity to take flows in the 10 year ARI MPD event. There are also stormwater pipes on the northern side of the highway with < 2 year ARI MPD capacity (both free full flow and backwater effect).

Table 7.12: Buildings at risk of flooding - Area 11 – Area around State Highway 16 Te Atatū interchange

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							
Habitable		2					5

7.2.2 Area 12 - Sylvan crescent

Table 7.13 shows the buildings at risk from floor level flooding in the MPD scenario at this location. The habitable floor level at 48 Sherwood Avenue has a 20 year ARI frequency of flooding in the MPD scenario. There is a stormwater pipe adjacent to the property with a capacity <2 year ARI MPD due to backwater and 10 to 20 year ARI MPD under free flow conditions.

The Sherwood Park Footbridge (SAP ID: 2000805130, Model Branch: HCTr8Sub1Br, Chainage 4) is over capacity in the 100 year MPD event, however there are no properties upstream of this footbridge within the flood extent.

The other properties identified as being at risk in this area are due to high water levels within Henderson Creek.

Table 7.13: Buildings at risk of flooding - Area 12 - Sylvan crescent

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							
Habitable				1		1	4

7.2.3 Area 13 - Coletta Road and Edmonton Road area

There is a total of 39 commercial or habitable buildings at risk from floor level flooding in this area, as shown in Table 7.14. The majority of the stormwater pipelines draining runoff from this area to Henderson Creek have < 2 year ARI MPD capacity due to backwater effects.

The culvert at the downstream end of the issue area (Coletta Esplanade Bridge Model Branch: HCTr3, Chainage: 124.5) does not have capacity to take flows in the 10 year ARI MPD event.

Table 7.14: Buildings at risk of flooding - Area 13 - Coletta Road and Edmonton Road are

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial					2	1	
Habitable	1	8	1	3	9	4	10

7.2.4 Area 14 – Mcleod Road to Central Park Drive

Table 7.15 shows that there are numerous buildings at risk from habitable floor level flooding in this area.

68% of the stormwater pipes modelled in this area are over capacity from backwater effects in the 2 year ARI MPD event and only 14% of the pipes perform above the 10 year ARI MPD.

There is also over 500 m of open channel through Te Atatū South Park and the two bridges through this reach (Model Branch: TASthPBr, Chainage: 4 and Model Branch: TASthPBr, Chainage: 21) overtop in the 10 year ED event.

Table 7.15: Buildings at risk of flooding - Area 14 – Mcleod Road to Central Park Drive

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							
Habitable		1	4	7	5	5	23

7.2.5 Area 15 – Chilcott Road

Table 7.16 shows the buildings at risk from floor level flooding in the MPD scenarios in this area. The primary flood mechanism in this area is from high water levels in the adjacent Henderson Creek.

Table 7.16: Buildings at risk of flooding - Area 15 – Chilcott Road

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial							
Habitable	1	2	3	8	4	2	5

7.3 Henderson suburb

7.4 Area 16 – Lincoln Road/Concourse Industrial Park

Table 7.17 shows the buildings at risk of flooding in this area which are predominantly commercial. The primary mechanism of flooding is due to the stormwater pipes being over capacity. In the 2 year ARI MPD event 57% of pipes are over capacity, predominantly due to backwater effects.

There are also some buildings at risk due to high water levels in the adjacent Henderson Creek. One of the buildings that floods in the 2 year ARI MPD event (50 The Concourse) has a surveyed floor level 0.05 m below the Mean High Water Spring 10 percentile (MHWS10%ile) tidal level plus sea level rise which is 2.74 R.L.

Table 7.17: Buildings at risk of flooding - Area 16 – Lincoln Road/Concourse Industrial Park

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	5 year	10 year	20 year	50 year	100 year	Within 500mm
Commercial	2	4	1	1	2	3	27
Habitable							1

7.5 Area 17 – Waipareira Avenue

There is a commercial building at risk in the 5 year ARI MPD event in this area, as shown in Table 7.18. The stormwater pipes adjacent to this building have a capacity of < 2 year ARI MPD due to backwater.

Table 7.18: Buildings at risk of flooding - Area 17 – Waipareira Avenue

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	**5 year**	**10 year**	**20 year**	**50 year**	**100 year**	**Within 500mm**
Commercial		1					1
Habitable							

7.6 Area 18 – Buscomb Avenue and Epping Road

Table 7.19 shows that there are 19 buildings at risk from floor level flooding in this area.

The flooding mechanism for buildings on Buscomb Avenue is primarily due to the stormwater pipes being over capacity in the 2 year ARI MPD event due to backwater and the flooding mechanism for buildings on Epping Road are primarily due to high water levels in the adjacent Henderson Creek.

Table 7.19: Buildings at risk of flooding - Area 18 – Buscomb Avenue and Epping Road

Number of buildings at risk of floor level flooding	MPD ARI event in which buildings first appear at risk						
	2 year	**5 year**	**10 year**	**20 year**	**50 year**	**100 year**	**Within 500mm**
Commercial							
Habitable	4	1	3		6	1	4

8 Conclusions and recommendations

This report provides the methodology and outcomes of the model build, system performance assessment and flood mapping. The flood plain, flood hazard and flood sensitive areas along with supporting information can be found in the Appendices of this report. The report also provides the results of a flood damage assessment which can be used to support project prioritisation and cost-benefit analysis related decisions.

There is no data available/provided to calibrate and validate the hydraulic model. Additional future confidence in the models ability to represent flows and water levels would be gained if hydrometric monitoring of future large storm events is carried out followed by model calibration and additional validation.

A sensitivity analysis showed that the catchment has a relatively low sensitivity to changes in land use with changes to both hydraulic roughness and hydrological curve numbers by +/- 20% resulting in only minor alterations to the flood extent.

The model results indicate that only 13% of the existing pipe network performs to a 10 year Average Recurrence Interval (ARI) design standard under the Maximum Probable Development (MPD) scenario. Pipe capacity catchment overview figures are provided in the report that highlight the areas where additional flows can be passed through the pipe network.

A floor level survey was carried out and model results show that there are 177 residential and 37 commercial buildings at risk from flooding above floor level in the 100 year ARI MPD event across the entire catchment. In the 100 year ARI Existing Development (ED) event 142 residential and 29 commercial buildings were identified as being at risk from flooding above floor level.

The floor level survey was used to predict average annual damage to buildings in accordance with Metrowater's FDA Guideline Report (November 2008). The predicted average annual damage to buildings in the catchment as a result of flooding is approximately $6,665,443 for Existing Development (ED), which increases to approximately $9,024,510 under the Maximum Probable Development (MPD) scenario (with climate change). The net present value of the buildings affected by flooding (50 year period, 8% discount rate) is approximately $81.5 M and $110.4 M for ED and MPD scenarios respectively which represents the 'opportunity cost' of not having to spend the expected annual damage. It should be noted that non-building damage is not included in the damage assessment.

The existing scenario represented in this report is based on 2013 LiDAR, aerial photography from 2015-2016 and asset and watercourse surveys carried out between December 2015 and January 2016. Te Atatū drawings from 2014 were used to implement the recent construction of the Te Atatū State Highway 16 interchange into the existing scenario. The existing scenario therefore does not represent catchment changes that have occurred since the topographical data were captured. It is therefore our recommendation that the model be kept up to date as changes to topography, land use and stormwater assets are made.

9 References

Auckland Council, 2011. Stormwater Flood Modelling Specification. November 2011.

Metrowater, November 2008. Flood Damage Assessment Guideline Report.

Ministry of Works and Development, 1978. Culvert Manual. Prepared by Civil Engineering Division. CDP 706/A.

10 Applicability

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

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Appendix E: File names and model results

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Appendix F: Thematic maps of pipe capacity

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Appendix G: Floodplain extent maps

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Appendix H: Properties at risk of flooding

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- **Table H3: List of commercial properties at risk of flooding (ED)**
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