Feilding Stormwater Modelling – Precinct 4

Manawatu District Council

Revision 3

Contact Details

Name: Louise Algeo

Opus International Consultants Ltd
Wellington Office
Level 10, The Majestic Centre, 100 Willis Street
PO Box 12003, Wellington 6011
New Zealand

Telephone: +64 4 471 7030
Mobile: +64 027 837 4726

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Prepared by:

Louise Algeo | Senior Hydraulic Modeller
Samwell Warren | Hydrologist

Reviewed by:

Franciscus Maas | Senior Hydraulic Engineer

Approved for Release by:

Andrea Harris | Project Manager

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Executive Summary

Opus International Consultants (Opus) has provided Manawatu District Council (MDC) with a GIS tool to review the minimum floor level requirements within Precinct 4. At each of the development sections or 'lots' provided on the Structure Plan (MDC, 2016) within the study area, maximum water levels are shown. The report accompanies the provided GIS tool and provides information on its development and recommendations for use. Figure 0-1 provides the extent of this information and the mapbook pages on which it is shown on.

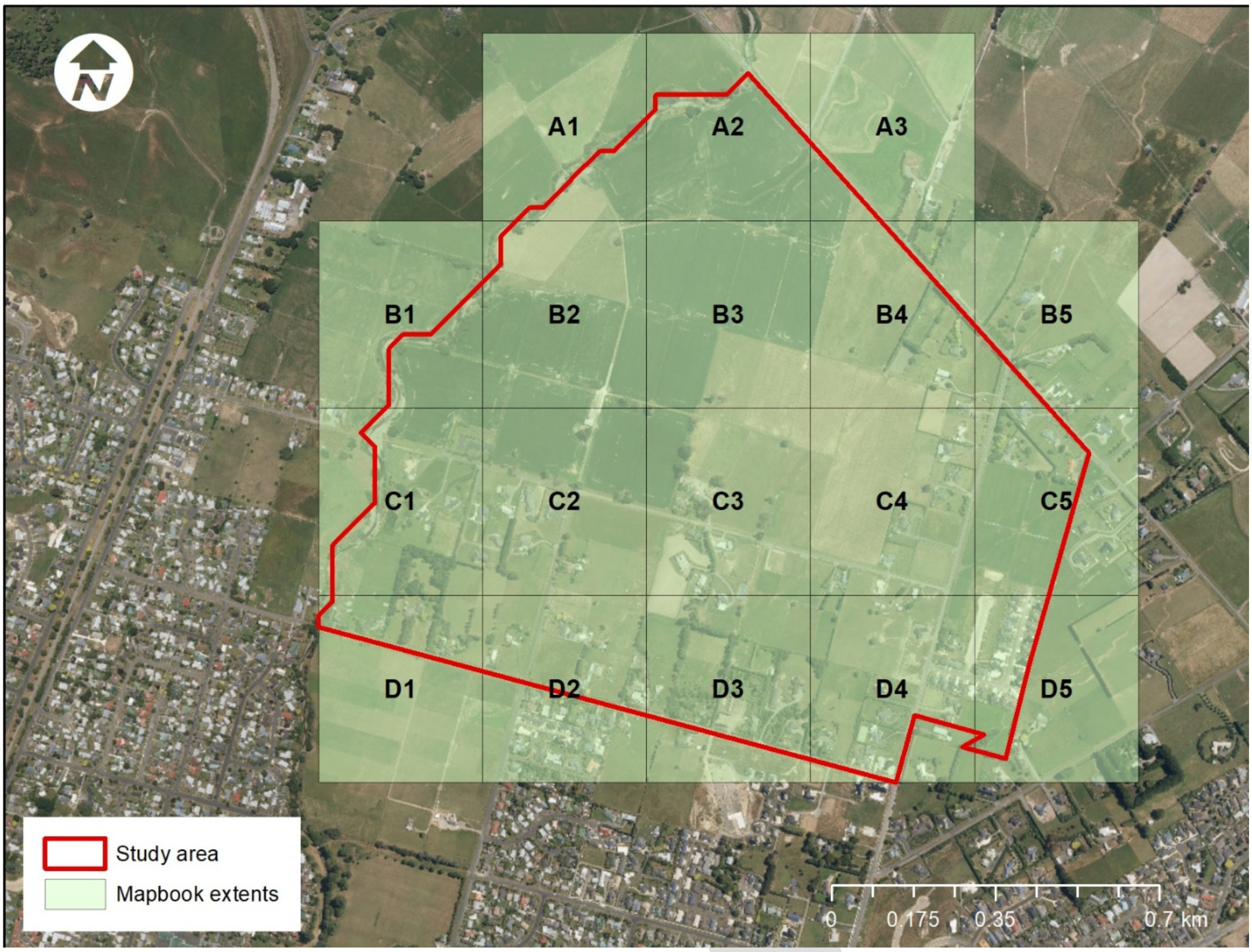

Figure 0-1: Extent of mapbook and study area

Defining the flood risk to proposed residential developments is complex due to the staged nature of the development of sections and the uptake of lots. Because the environment is ever changing, flood risk should not only be considered on the fully developed case. It is appropriate to review flood risk from an undeveloped, or current scenario, all the way through the many phases of semi developed to a fully developed site.

The large new development area of Precinct 4 offers further challenges to establishing requirements for minimum floor levels with changes to drainage infrastructure and overland flowpaths; the reliance on Horizons Regional Council (Horizons) to operate the Makino flood diversion scheme; and also the existing surface water flood hazard at the site.

Opus has developed a two-dimensional (2-D) hydraulic model of Precinct 4 using InfoWorks ICM v6.5. The model is bound by the Makino River to the West, the Makino Diversion to the North, The Oroua and Kiwitea Rivers to the East and Feilding (specifically North Road) to the South.

This 2-D model has provided water depth, level and velocity outputs for the 0.5% AEP design events simulated for the 'Existing' current level of development and the 'Proposed' fully developed situation, plus a number of alternative flow scenarios for the 'Proposed' scenario. These are provided in Wellington Vertical Datum 1953 and exclude freeboard.

To better represent the results within the area of interest, the boundaries of the model have not been applied at the edge of the study area. Therefore while results are also available outside of Precinct 4, they have not been extensively verified as they are only to ensure appropriate flow away from the study area.

A number of observations have been made from the results of the modelling analysis, in addition to the results output information that has determined the proposed minimum floor levels. Key observations are as follows:

1. The model indicates that some of the runoff from Precinct 4 does not arrive at the road or stormwater infrastructure, and instead ponds on adjacent land. The model outputs show areas where additional infrastructure may be required to drain these areas and therefore reduce the flood levels (at the junction of Roots Street and Proposed Road 3 for example).
2. The modelling analysis has shown that the proposed stormwater pipe network is functioning as designed, with the roads acting as a flowpath. There are some locations where the results do not show conveyance along the roads however it should be noted that the results only show depths greater than 50mm. The 2-D model has therefore provided further confidence that designed pipe sizes are sufficient to convey large flows.
3. The results of this analysis show that proposed changes to the drainage as part of the Precinct 4 development provide benefits to Feilding (to the south of the proposed development) by reducing the flood hazard. Surface water runoff is predicted to be intercepted and diverted to the Makino River rather than continuing overland to the township.
4. In some areas within Precinct 4, flood levels increase as a result of the development and in some cases they decrease. The results show the areas that are predicted to experience changes as a result of the development.

The model has been able to provide further confidence in the stormwater infrastructure design for Precinct 4. While the results outside Precinct 4 have not been extensively checked, they indicatively show wider benefits to Feilding. To quantify this in more detail further modelling analysis should be undertaken for the design level of service. To ensure the model remains suitable for use in future development studies, the invert levels and pipe sizes included in the model should be confirmed against the as built information of the pipes once installed.

The modelling results provided will ultimately be used to determine minimum floor levels, however Opus recommends using the information provided for the additional flow scenarios to deal with the uncertainty associated with the gradual and unknown staging of the development, the manual operation of the Makino flood diversion scheme and the hydrological conditions of the overall site.

Considering the differences in results between these alternative flow scenarios such as Makino Diversion culverts open during the 0.5% AEP event, and the wet antecedent condition of simulating a 1hr 0.5% AEP event 3 hours after the termination of a 12 hour 0.5% AEP event and the June 2015 event may provide a reasonable justification for the chosen allowance for freeboard.

The current MDC freeboard value of 350mm may be appropriate to consider but should be applied to the flood level and not the ground level.

This information has been provided in flood maps in Appendix A and in a GIS ‘map book’ accompanying the report. This will allow the user to click on any point in the catchment to find predicted water levels, depths, and ground level information.

1. Background

1.1. Context

Significant and immediate development is projected for Feilding and Precinct 4 has been identified as a primary growth area. A 2-d model held by Horizons Regional Council has provided predicted flooding information for the surrounding areas however, MDC and Horizons are not confident in the results for in this area as modelled results were inconsistent with anecdotal evidence. Additionally, changes to the topography and land use due to development that has already occurred in the area meant that the model likely was not representing the current terrain in this area.

Opus International Consultants Limited (Opus) have been commissioned by Manawatu District Council (MDC) to undertake two-dimensional (2-d) flood modelling of Precinct 4. The purpose of this is to further understand the flooding hazard in an area to the north of Feilding for which extensive development is planned; and to provide information to inform minimum floor levels for buildings in this area.

MDC is primarily interested in the area proposed for new development known as Precinct 4, shown in Figure 1-1, located in the north-eastern quadrant of the township.

This study therefore aims to build a site-specific 2-D hydraulic model using a 'Rain on grid' approach with the primary purpose of understanding the minimum floor level requirements for the new developments within this area. Specifically regarding the flooding impact in a for the 0.5% Annual Exceedance Probability (AEP) or 1 in 200 year Average Recurrence Interval (ARI) design scenario, which Horizons Regional Council equate to the 1% AEP or 1 in 100 year ARI event with the predicted impacts of climate change to 2090.

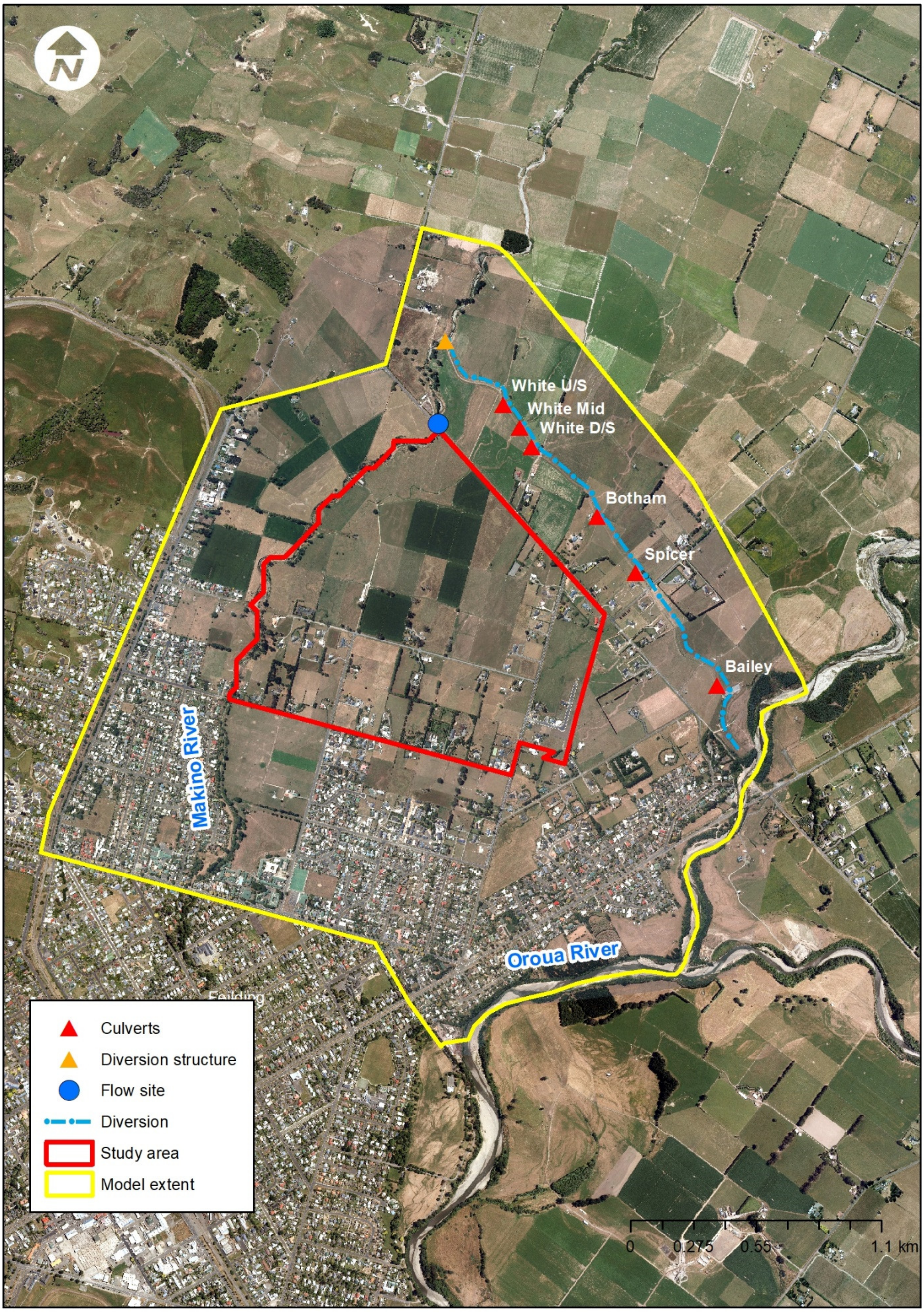

Figure 1-1: Feilding Township and the study area of interest.

2. Hydraulic modelling results

A 2-D hydraulic model has been constructed using InfoWorks ICM v6.5. The construction and analysis of this is detailed in Appendix B, with the resulting outputs shown in Appendix A.

The model has been set up with rainfall applied directly to the grid (Rain on grid). Therefore depth results less than 0.05m have been omitted from all results and Figures.

2.1. Scenarios

The maps in Appendix A are based on 2 scenarios at this stage: the 'Existing' current level of development situation, and the 'Proposed' fully developed situation with flood depths for a 0.5% AEP event shown in each case. These are provided in Wellington Vertical Datum 1953 and exclude freeboard. A summary of the model inputs is shown in Table 2-1.

Table 2-1: Summary of modelled scenarios

Model	Scenario	Diversion culverts	Rainfall (0.5% AEP)	Makino Diversion	Makino River
Existing (road network layout)	Existing 0.5% AEP	Closed	Design events: 1-hr, 2-hr, 6-hr, 12-htr duration.	Assumed Maximum peak of 50cumecs	Design inflow of 70cumecs (assumes 120m3/s - 50cumecs in diversion)
Proposed (road network layout)	Proposed 0.5% AEP	Closed	Design events: 1-hr, 2-hr, 6-hr, 12-htr duration.	Assumed Maximum peak of 50cumecs	Design inflow of 70cumecs (assumes 120m3/s - 50cumecs in diversion)

Further information is included in Appendix B.

2.2. Results

The results output depth, water level and velocity at every point within the catchment for every rainfall event duration that has been run. The results have been aggregated and the maximum water level, and depth and every point has been determined and reported. This has allowed detailed flood hazard information throughout Precinct 4.

Figure 2-1 shows the water depth results for the 'Existing' 0.5% AEP event.

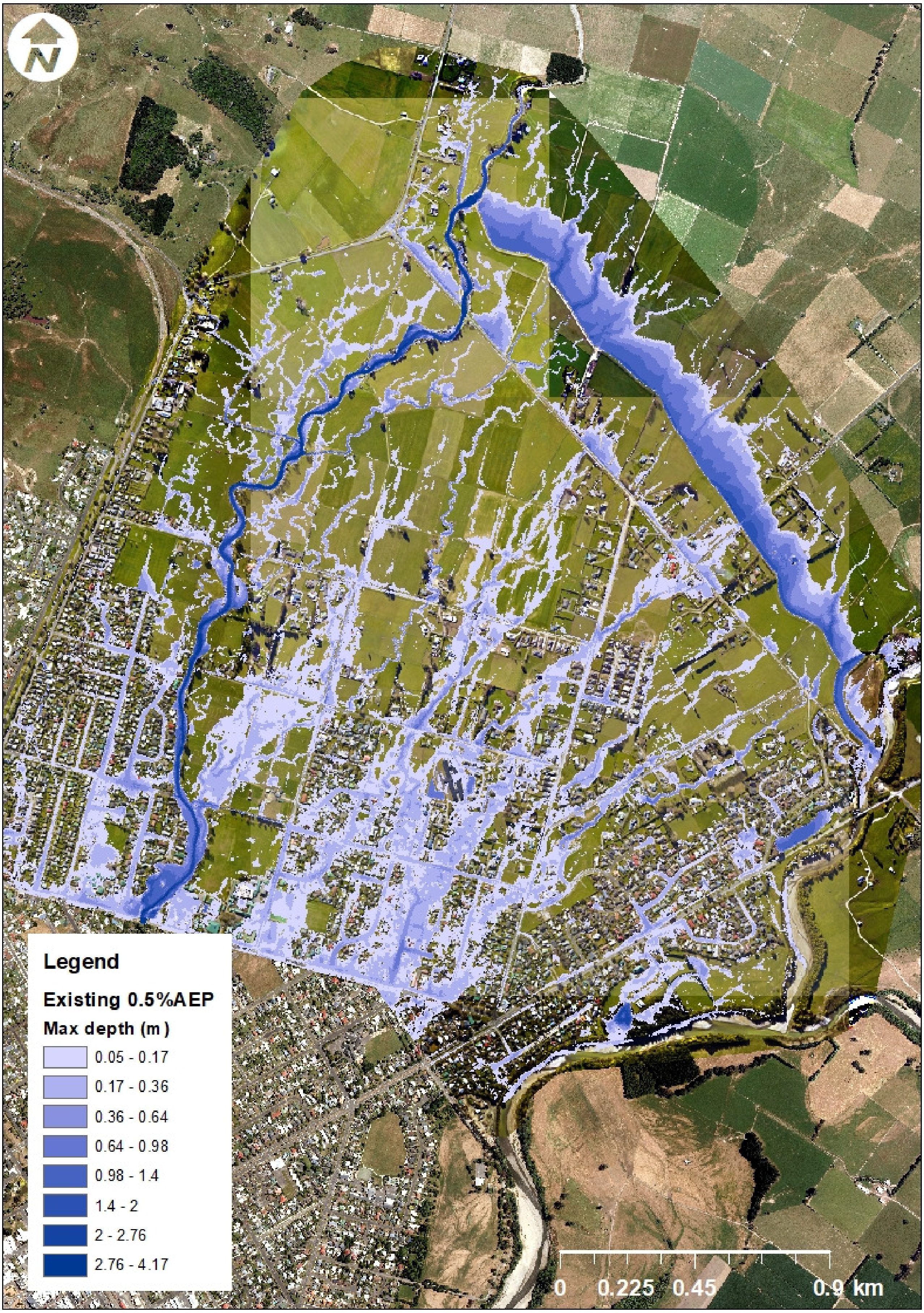

Figure 2-1: 0.5% AEP event for 'Existing' undeveloped scenario

Figure 2-2 shows the water depth results for the 'Proposed 0.5% AEP event.

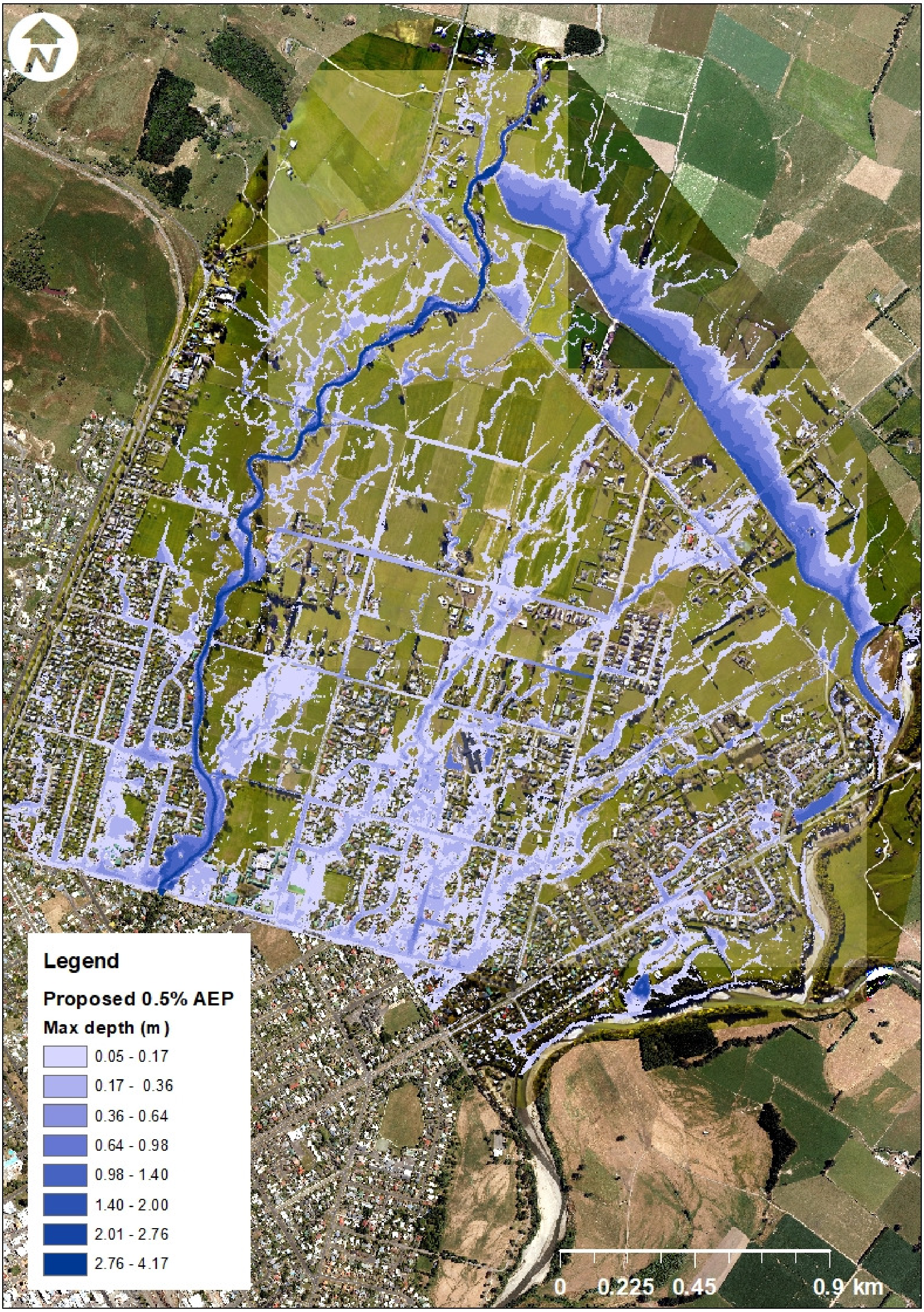

Figure 2-2: 0.5% AEP event for 'Proposed' fully developed scenario

3. Conclusions

3.1. Precinct 4 minimum floor heights

It is reasonable to consider that the water levels obtained from the design rainfall events under normal Makino diversion operating conditions are suitable for use in setting minimum flood levels.

The maps in Appendix A show the water levels predicted by the model for the 'Existing' and 'Proposed' scenarios throughout Precinct 4. These are provided in Wellington Vertical Datum 1953 and exclude freeboard.

3.2. User guide

To allow this information to be easily used, Opus has provided plans presenting minimum floor height recommendations in Appendix A and also as a 'map book' to accompany this report in GIS.

On the maps, the water depth for the corresponding scenario is shown, along with a maximum water level predicted by the 2-D hydraulic model for each individual 'lot'. This value has been derived from the maximum of each of the four durations for every point within Precinct 4, and then sampling the highest water level spatially across the 'lot'. These water levels are provided in Wellington Vertical Datum 1953 and exclude freeboard. Figure 3-1 shows an example of the information provided.

It is recommended that freeboard is applied to this water level, and not the ground level. This is discussed in more detail in Section 3.4.

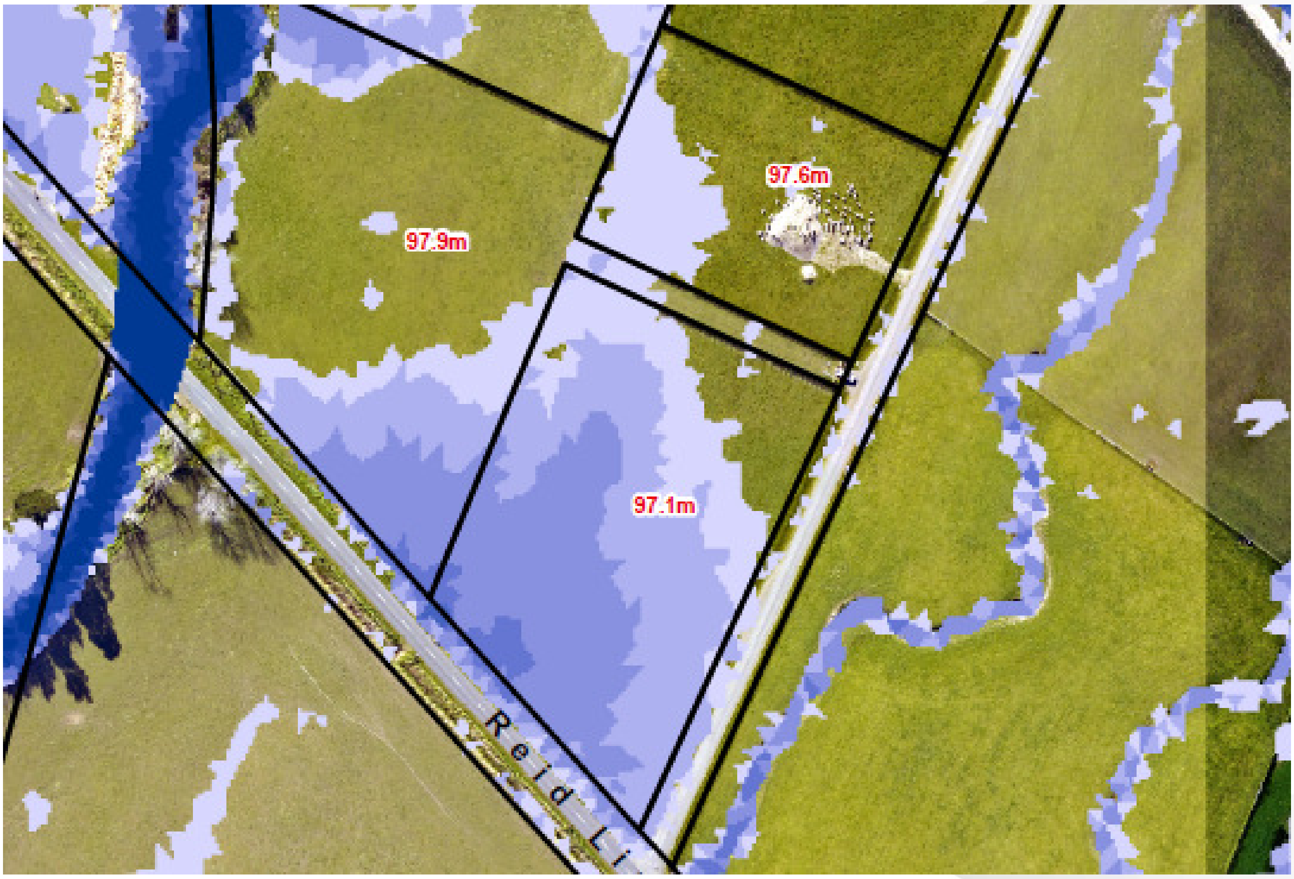

Figure 3-1: Example of potential minimum floor levels shown on maps in Appendix A

3.3. Consideration of other scenarios

Although the Horizons operated Makino flood diversion scheme is not in the control of MDC, this study has identified a number of features that MDC should be aware of, and consider when assessing a worst case scenario for flooding of the study area. These are discussed below.

3.3.1.1. Diversion gates

The gates that set the flow rate in the Makino River and consequently the flow in the Makino diversion are operated using a manual process. This is therefore subject to the potential for error and increased flood risk as a result of increased flow in the Makino.

3.3.1.2. Diversion channel bank culverts

Anecdotal evidence indicates that the Makino Flood Diversion culverts are not always closed for the entire operation of the diversion. The culverts in the diversion should be closed when a flooding event occurs, however as this is manual process and not logged, this cannot be confirmed for every flood. It is understood that property owners may not close the culverts or open them prematurely to reduce flooding on their properties. The impact of this on the water levels in the study area should therefore be considered when setting minimum floor heights.

3.3.1.3. Pharazyn Road flood gates

There are flood gates on Pharazyn Road as the road crossing the diversion channel is lower than the surrounding banks of the diversion channel. Not only is this a manual flood gate and relies on an operator to correctly close the gate at the appropriate time, there is anecdotal evidence that it is not working effectively and allowing some flow under the gate during flooding events.

3.3.1.1. Makino River blockages

There are a number of bridges along the Makino River and there is the potential for blockages with debris from the upper catchment or various items from residential or commercial properties. Therefore a blockage scenario at the downstream end of the model on the Makino River has also been modelled.

3.4. Freeboard

An allowance for freeboard is typically applied on top of modelled water levels when designing stopbanks, bridge deck levels and setting minimum floor levels to account for differences such as local waves or flow variations due to blockages or data errors.

Building code E1 states that the level of the floor should be set at the height of the secondary flow path plus an allowance for freeboard (MBIE, 2016). This allowance should be 500mm for areas where the water depth is 100mm or greater and 150mm elsewhere.

However as the results vary spatially, it may not be appropriate to apply a blanket value across the flooded areas. This is because it may result in properties adjacent to each other where it is predicted to be ‘dry’ and the other has 100mm of flooding predicted with a larger freeboard added on top.

MDC have historically adopted a blanket approach to provide freeboard of 350mm above the ground level and therefore this problem with neighbouring properties is not an issue. However it does present the question of whether in fact the freeboard may not be the difference between the flood level and the floor height, and if these levels are sufficient.

This study will allow MDC to establish site specific minimum floor requirements based on flooding information throughout Precinct 4. These can be based on any of the information provided and ultimately it is MDC’s decision to define the requirements.

In addition to the 0.5% AEP event simulated for the proposed situation, a number of alternative flow scenarios have been simulated. The outputs of these have been aggregated and the maximum water levels of all of the scenarios at each point within the model have been determined. The flow scenarios that should at least be considered are the ‘Existing’ scenario with 0.5% AEP event to provide a starting point for the staged development, Makino Diversion culverts open during the 0.5% AEP event, and the wet antecedent condition of simulating a 1hr 0.5% AEP event 3 hours after the termination of a 12 hour 0.5% AEP event.

MDC should review these water levels in addition to the water level provided for the ‘Proposed’ situation 0.5% AEP design rainfall event and take a conservative maximum and consider this the ‘base value.

In addition to the base value, an allowance for freeboard should be applied. The current MDC value of 350mm may be appropriate to consider but should be applied to the flood level and not the ground level.

3.5. Key observations

A number of observations have been made from the results of the modelling analysis, in addition to the results output information that has determined the proposed minimum floor levels. Key observations are as follows:

1. The model indicates that some of the runoff from Precinct 4 does not arrive at the road or stormwater infrastructure, and instead ponds on adjacent land. The model outputs show areas where additional infrastructure may be required to drain these areas and therefore reduce the flood levels (at the junction of Roots Street and Proposed Road 3 for example).
2. The modelling analysis has shown that the proposed stormwater pipe network is functioning as designed, with the roads acting as a flowpath. There are some locations where the results do not show conveyance along the roads however it should be noted that the results only show depths

greater than 50mm. The 2-D model has therefore provided further confidence that designed pipe sizes are sufficient to convey large flows.

3. The results of this analysis show that proposed changes to the drainage as part of the Precinct 4 development provide benefits to Feilding (to the south of the proposed development) by reducing the flood hazard. Surface water runoff is predicted to be intercepted and diverted to the Makino River rather than continuing overland to the township.
4. In some areas within Precinct 4, flood levels increase as a result of the development and in some cases they decrease. The results show the areas that are impacted.

The model has been able to provide further confidence in the stormwater infrastructure design for Precinct 4 and show wider benefits to Feilding. To quantify this in more detail further modelling analysis should be undertaken for the design level of service. To ensure the model remains suitable for use in future development studies, the invert levels and pipe sizes included in the model should be confirmed against the as built information of the pipes once installed.

3.6. Additional model uses

This area is changing rapidly and the flood hazard at every stage is derived from extremes of undeveloped to fully-developed. The flood risk to areas with early uptake should not have to be more vulnerable to flooding during the time it takes to develop the rest of the area and therefore for some site specific locations, further information may be required. The model has been developed to be adaptable to changes to the development process and this includes staging if required.

This modelling exercise has also provided a check for the stormwater pipe network sizing. Although assumptions have been made with the representation of the pipes, the model shows that the pipes have the capacity to convey the flow arriving at the stormwater pipes in a 0.5% AEP event. The modelling indicates that the flowpaths to convey the water to the pipe or the designed overland flowpath within the road reserve may be the limitation.

There are some areas with larger depths of flooding. Opus recommends that for areas with depths predicted of over 500mm, further investigation is undertaken to review additional drainage installation to ensure that these levels will be reduced.

3.7. Map book update

If required the maps can show water level instead of depth, or results of a different, more appropriate simulation. The polygons for the proposed ‘lots’ can also be updated as further information becomes available. The ‘lots’ are currently based on information provided on 12 January 2018.

4. Recommendations

The following recommendations should be considered by MDC before finalising and utilising this information.

- MDC should review the results for June 2015 and confirm that it is a reasonable representation of what happened in the areas that are still undeveloped in 2018.
- MDC should review their application of freeboard and apply a level to the modelled water levels across Precinct 4 and not the ground level.
- Opus recommends using the proposed fully developed case as a 'base value' for minimum floor heights and considering the scenarios to determine contingency for freeboard requirements.
- MDC should review the flooding information to determine if additional drainage is required in some areas. The model has been able to provide further confidence in the stormwater infrastructure design for Precinct 4 and show wider benefits to Feilding. To quantify this in more detail further modelling analysis should be undertaken for the design level of service.
- To ensure the model remains suitable for use in future development studies, the invert levels and pipe sizes included in the model should be confirmed against the as built information of the pipes once installed.

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Appendix A
Recommended minimum floor height maps for Precinct 4

Appendix B
Hydraulic model information

1. Hydraulic model information

To construct a 2d hydraulic model the following data and information for both the existing and proposed situation is required: topographical information; land use information; drainage connectivity information; hydrological input data (stream flow and rainfall); boundary conditions (physical or hydrological).

1.1. Topographical data

A Digital Elevation Model (DEM) has been created using the point elevation data captured during the LiDAR survey in August 2017. This data was supplied by Horizons and confirmed to be Wellington Vertical Datum (WVD) 1953. Consequently, any development in the vicinity of the development area following this data capture may affect the findings from this study. All terrain data used was spatially projected in NZGD 2000.

An 8m DEM obtained from LINZ for the surrounding areas was also reviewed, however is not deemed suitable for this study due to the coarse resolution and therefore the model extent has been limited to the provided 2017 LiDAR extent.

1.1.1. Topographic adaptations

When LiDAR data is captured, ground levels sheltered from the laser are not collected. Therefore areas with thick vegetation, underneath bridges or underneath the water level are not obtained and post processing the DEM is required to make it better represent the topography.

For this DEM, the following adjustments have been made:

- Power lines have been removed from the elevation data to remove barriers to flow that are not there in reality.
- The Makino River has a number of bridges. The bridge decks have been removed to ensure hydraulic connectivity throughout the length of the study area. If the flooding impact from the Makino River directly was required to be understood then the bridges would have to be appropriately represented.
- Immediately upstream of the diversion flow input location, the DEM has been adjusted to ensure that all flow is conveyed along the diversion and not into the Makino.

Existing buildings captured in the LiDAR have been included in the DEM to allow for better representation of the flowpaths.

For the ‘Proposed’ version of the model, changes to the topography along the proposed roads were based on the drawings provided by GHD 2017a.

1.1.2. Mesh size

The underlying terrain is represented using a triangular mesh with a triangle size of 9-20m^2. Additional detail has been added for all existing and proposed roads, and the Makino Stream represented by a refined mesh size of 5-9m^2.

1.2. Land use and roughness

The spatial difference in land use has been determined from a review of aerial photographs and a site visit to the area in November 2017. The Manning’s n hydraulic roughness values listed in Table B-1-1 have been determined for each land use.

Table B-1-1: Hydraulic roughness (Manning’s n) values

Land use	Hydraulic roughness (Manning’s n value)
Open grassland	0.045
River channel	0.04
Urban areas	0.10
Roads	0.022

1.3. Stormwater infrastructure

1.3.1. Existing and proposed stormwater pipe network

Large stormwater pipes within Precinct 4 have been included in the model. For the 'Existing' case, pipes larger than 600mm have been included. This is to balance between a worst case situation where the flow does not enter the pipes, and a realistic situation where excess ponding is removed where pipes would convey the water. A lower limit of 600mm was chosen due to available invert level information, and therefore to limit the assumptions associated with including the pipes, and recognition of blockages to inlets is more likely to occur with smaller pipes. Information was provided in GIS format by MDC and was cross checked with proposal drawings (GHD, 2017). These are shown on Figure B-1-0-1.

Figure B-1-0-1: Existing stormwater infrastructure within Precinct 4 included in the 2-D model

The ‘Proposed’ model included all stormwater pipes within Precinct 4 labelled on drawings 51-33090-01 SK008A, B and C (GHD 2017). Invert levels have been assumed from provided invert levels and the provided grade of the pipe. All pipes are larger than 375mm and have been included as less assumptions have been made regarding invert levels and the fully developed situation will have less farmland and vegetation for potential blockages. These are shown on Figure B-1-0-2

Figure B-1-0-2: Proposed stormwater infrastructure within Precinct 4 included in the 2-D model

1.3.2. Road and farm drainage

Four culverts under Reid's Line have been included in both the 'Existing' and 'Proposed' models. Dimension information about the culverts was obtained from Philpot (2016a). Invert levels have been determined from the ground level information provided in the LiDAR. These culverts were chosen after preliminary modelling showed the requirement for hydraulic conductivity in these locations and allowed for more realistic flooding information downstream of Reid's line within the development area.

Open drains are included within the DEM. The model results show that the triangular mesh size is sufficiently small to represent the flowpaths and therefore it is not recommended further detail is included at this stage.

1.4. Rainfall inputs

It is common practice to consider a range of durations of typical rainfall events and the hydrological analysis provided a suite of rainfall depths ranging from 10 minutes to 72 hours. An assessment of the rainfall and the catchment concluded that the rainfall event durations in the area are typically less than 24hours. The size of the catchment area indicates times of concentration of less than 12 hours therefore four design durations were chosen: 1-hour, 2-hour, 6-hour and 12-hour. Table B-1-2shows the 0.5% AEP design rainfall depths for these durations.

Table B-1-2: 1% AEP design rainfall depths adjusted for climate change (0.5% AEP design rainfall depths).

AEP (%)	ARI (yrs)	Duration (hours)			
		1	2	6	12
0.5	**200**	59.2	74.2	106.3	133.4

1.5. Boundary conditions

The development area is bounded by a number of physical features: the Makino River to the West; the Makino Diversion structure to the North, the Kiwetea and Oroua Rivers to the East and Feilding township (specifically North Street) to the South.

The Makino River water levels provide a boundary condition for the majority of the drainage from Precinct 4. To determine water levels along the Makino, flow for the 0.5% AEP were derived from the 1% AEP provided by Horizons. This is detailed in Appenidx B. This flow was input as a point source to the 2-D hydraulic model and resulting water levels provided dynamically.

The Makino Flood Diversion Scheme operation is discussed in Appendix C. Larger flows are controlled through the Makino Diversion and the peak flow limit is reported to be the same for the 0.5% AEP as for the 1%AEP event and therefore the 1% AEP profile provided by Horizons was utilised in the study.

The Kiwetea and Oroua Rivers are the receiving water bodies for the Makino Diversion. They are located at a lower elevation than the Makino River ad the area of interest and therefore can be ignored. A comparison of peak flows in the Makino River and the Oroua River indicates that rainfall in the two catchments do not coincide and therefore the stormwater network along Pharazyn Street that drains to the Oroua River is assumed to be a free outfall.

The downstream catchment and Feilding township has been included as a boundary condition assuming flow under normal conditions i.e. will continue to flow downstream of the modelled extent with the same slope immediately upstream of the boundary. The Feilding stormwater network is not expected to back up to limit flow from Precinct 4 and sufficient area between the boundary and the area of interest has been included in the model.

1.6. Summary

Two versions of the hydraulic model have been set up: the 'Existing' case which represents the current situation; and the "Proposed' case which represents the fully developed situation.

Table B-1-3 summarises the differences between the two versions.

Table B-1-3: Summary of model elements in the two versions

Aspect	Existing	Proposed
Terrain	LiDAR data from August 2017 in Wellington Vertical Datum 1953 provided by Horizons. Adjustments to terrain for power lines exclusion and to ensure no additional flow into the Makino from the Makino Diversion.	As Existing, with additional changes to road elevations for proposed roads based on GHD (2017a).
Model mesh	Underlying terrain triangle size of 9-20m^2. More detailed mesh zones with triangle size of 5-9m^2 for roads, Makino Stream, and any terrain adaptations in culverts.	As Existing, with additional smaller triangular mesh areas for the proposed roads.
Roughness (Manning's n value)	Roads: 0.022 (11.7ha) Urban area: 0.1 (209.3ha) River: 0.04 (29.3ha) Farmland: 0.045 (438.5ha)	Roads: 0.022 (19.1ha) Urban area: 0.1 (450.4ha) River: 0.04 (29.3ha) Farmland: 0.045 (190ha)
Stormwater network	Existing network with diameter>600mm	Existing network with diameter>600mm. Proposed network with diameter>375mm
Reid's Line culverts	Four culverts as specified in Figure 1-1	No changes as proposed diversion channels are not clear
Makino River	Removed bridges to allow flow along the river.	No changes

2. Hydraulic modelling analysis

2.1. Conceptualisation

The 2-D hydraulic model physically represents the catchment, however the simulations also need to represent the hydrological characteristics of the rainfall and flows in the catchment. Therefore analysis was undertaken to confirm the conceptualisation.

2.1.1. Coincidence of flow and rainfall peaks

A comparison of the rainfall recorded at Holcombe Road and the flow in the Makino River at Reid's Line during the June 2015 event is presented in Figure B-2-1.

The peak rainfall depth during the design events will be applied to the model to coincide with the peak flow applied to the Makino River to produce a 'worst case' scenario.

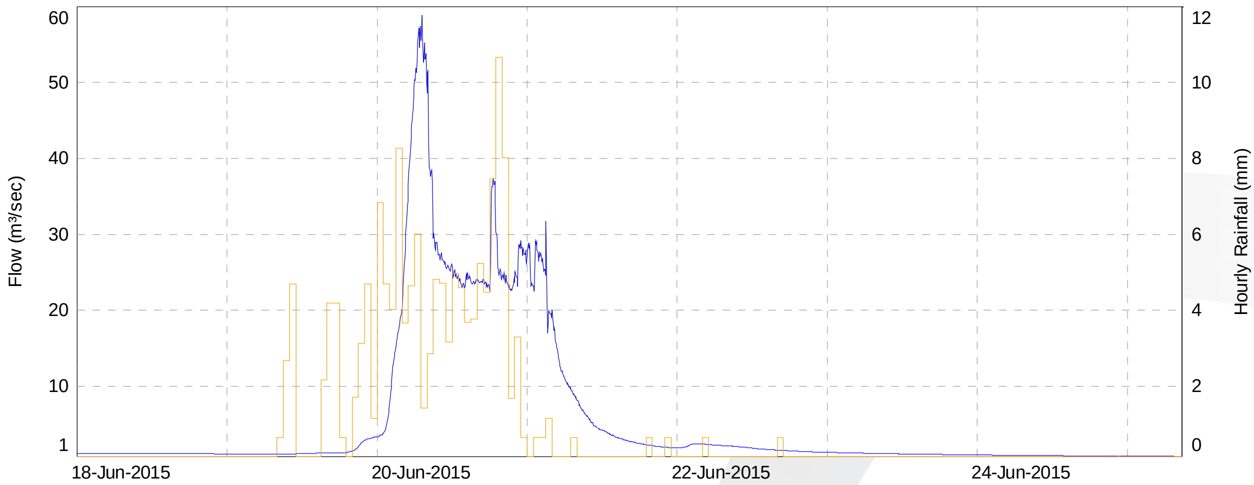

Figure B-2-1: Rainfall recorded at Holcombe Road, and flows in the Makino River during the June 2015 event.

It should be noted that the design event used in the hydrologic modelling is based on rainfall over the area, and not the actual runoff. While a 1% AEP design rainfall event is relatively easy to define, a 1% AEP runoff event will be affected by the temporal distribution of the rainfall, antecedent conditions, and the exact rainfall-runoff relationship operating during a particular event.

2.1.2. Makino diversion capacity

While Horizons own and operate the diversion scheme which limits diverted flows during operation to 50m³/s, it is considered important to confirm that 50m³/s plus the 0.5% runoff from the area upstream of the culverts will not overtop the stop bank and inundate the upper area of the development site.

To validate this, a model run was conducted that applied a 50m³/s flow through the diversion structure with a peak that that coincides with a 0.5% AEP peak discharge at the upstream sub-catchment of each culvert. Table B-2-1 shows the flows from each sub-catchment.

Table B-2-1: Sub-catchment flow upstream of culverts at Makino diversion bank.

Culvert	"30-yr ARI" peak Discharge event (upstream of culvert) (m³/s)	1% AEP peak flow (m³/s)	1% AEP Peak flow adjusted for Climate Change (0.5% AEP) (m³/s)
Terry Clare *	0.53	0.7	0.85
White U/S	2.76	3.7	4.43
White Mid	5.85	7.8	9.39
White D/S	5.42	7.2	8.70
Botham	1.88	2.5	3.02
Spicer	2.52	3.4	4.04
Bailey	2.24	3.0	3.60

The 1% AEP flows in Table B-2-1 have been determined from scaling the '30-year' flows from Philpott (2016) using the ratio between AEP flows at Makino at Reids Line. This has then been adjusted for climate change to determine the 0.5% AEP flows.

The results of this validation exercise conclude that the capacity of the Makino Diversion is sufficient to convey a 50m³/s diverted flow and a 0.5% AEP flows from the catchment upstream of the diversion and therefore there will be no direct impact on the study area from the Makino diversion as long as it is operated as planned, with the culverts closed.

2.1.3. Makino River flooding

The impact of the Makino River on flood levels in the development area can result from: direct flooding from the Makino River; and high water levels impeding drainage from the area.

The impact of high water levels on drainage will be determined using a test of the model sensitivity to water levels.

To ascertain the impact of flooding from the Makino River, a 0.5% AEP (total 120 m^3/s including 50 m^3/s in the Makino Diversion and 70 m^3/s in the Makino River), and 1% AEP (total of 100 m^3/s including 50 m^3/s in the diversion and 50 m^3/s in the River) have been simulated in the model without rainfall applied to the study area.

The results indicate that the Makino River can convey the required flow for a 0.5% AEP event assuming the diversion is operational. Therefore the study area is not predicted to be affected directly from the Makino River.

2.2. Validation - historical flood events

The April 2015 and June 2015 storm events caused widespread flooding in the Manawatu region. No site specific historical flooding information has been provided for Precinct 4 and although a validation would increase confidence in the model results it should be noted that there may be differences due to the timing of the data provided. The 'Existing' model is based on information as of August 2017, and significant changes to the catchment have been undertaken since the flooding events in 2015.

However Opus would welcome additional information to support the findings in this report.

This information is still useful for understanding potential flood risk under different scenarios.

2.2.1. April 2015

The April 2015 event did not translate into significant flow in the Makino River. However rainfall over the period was significant from a surface flooding and drainage perspective.

The flow in the Makino River was checked for the April 2015 event and the flow found to be less than 10% of the 1% predicted flow. Therefore the flow in the Makino River was ignored for this simulation. The rain gauge data used was from Holcombe Road

2.2.2. June 2015

The June 2015 event produced a significant flood in the Makino River that required the operation of the Makino Diversion spillway. The peak flow for Makino at Reid's Line was 58m^3/s.

Horizons have not been able to confirm whether the culverts were closed during this event and believe they may have been opened during the event, or not closed in the first instance. Therefore an additional scenario with the Makino flow diversion culverts open has been simulated for this event.

Figure B-2-2 shows the flow and rainfall inputs for the scenario where the Makino diversion scheme operational procedures were followed. Figure B-2-3 shows the additional inflows at downstream of the Makino diversion bank to represent a scenario in which they were not followed and all culverts remained open.

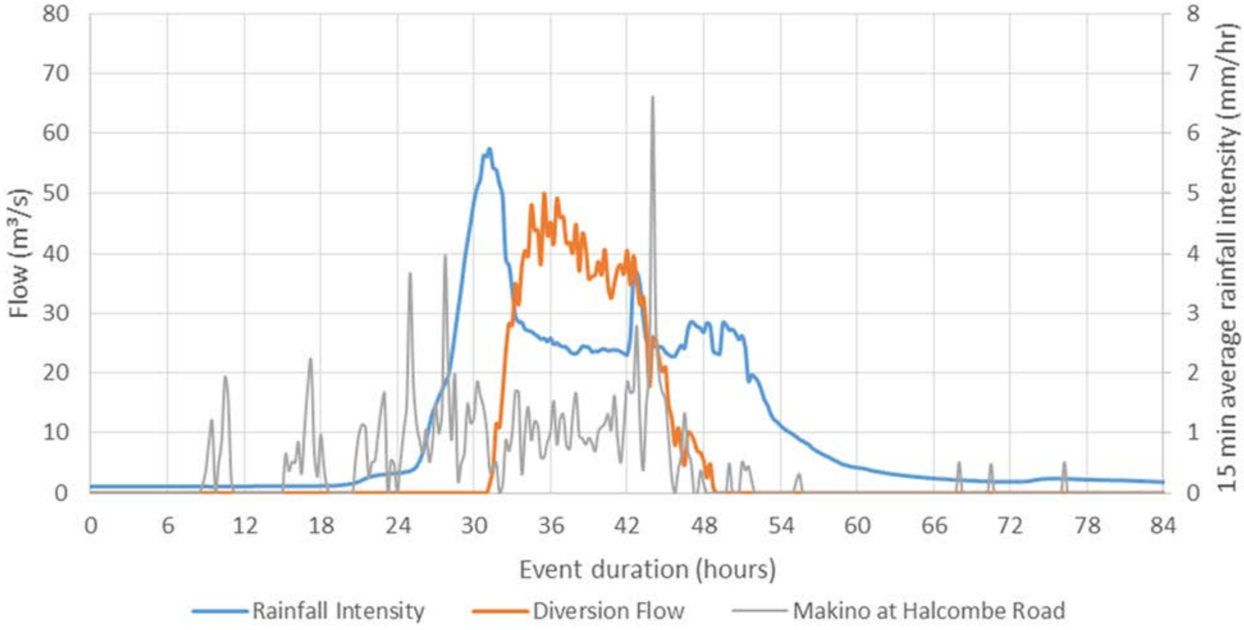

Figure B-2-2: June 2015 validation model inputs - operational procedures followed

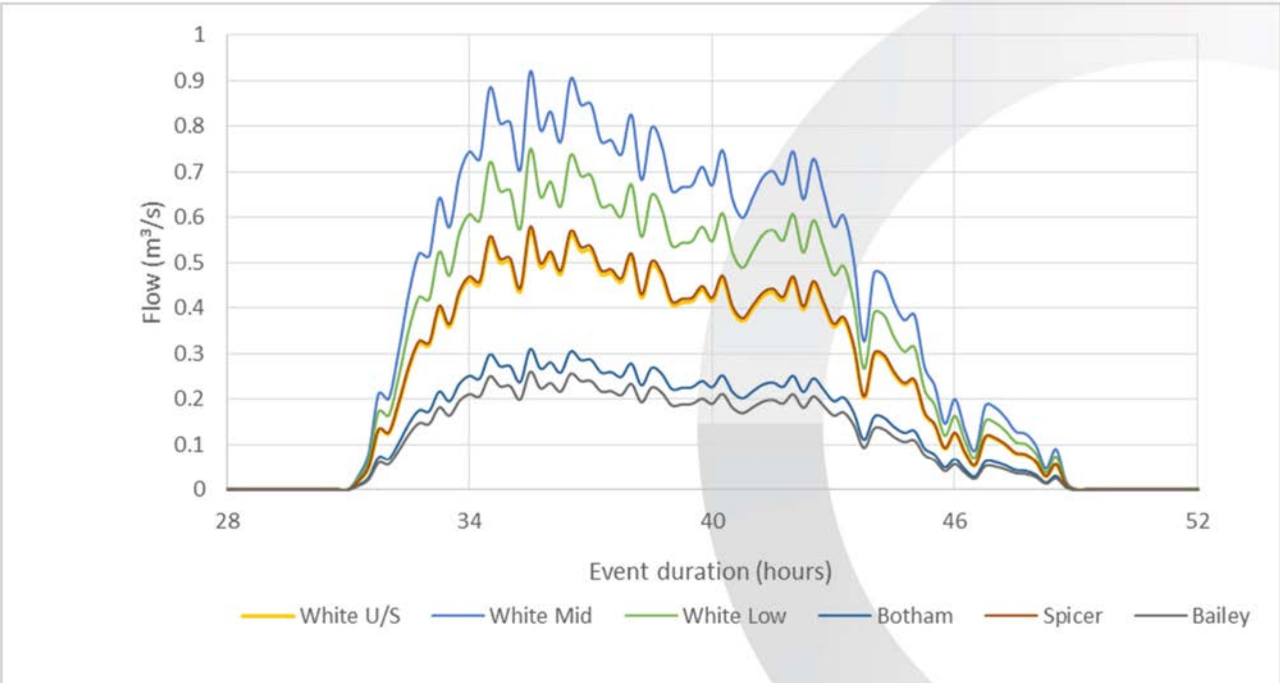

Figure B-2-3: Culvert flow profile estimated from diversion profile.

2.3. Scenarios

To help to account for the uncertainty, a number of scenarios have been modelled in this study to further understand the flood hazard at each point across Precinct 4. Table B-2-2 summarises the scenarios simulated.

Table B-2-2: Summary of modelled scenarios

Scenario	Model	Diversion culverts	Rainfall (0.5% AEP)	Makino Diversion	Makino River
Proposed 0.5% AEP	Proposed (road network layout)	Closed	Design events: 1-hr, 2-hr, 6-hr, 12-htr duration.	Assumed Maximum peak of 50cumecs	Design inflow of 70cumecs (assumes 120m3/s - 50cumecs in diversion)
Existing 0.5% AEP	Existing	Closed	Design events: 1-hr, 2-hr, 6-hr, 12-htr duration.	Assumed Maximum peak of 50cumecs	Design inflow of 70cumecs (assumes 120m3/s - 50cumecs in diversion)
June 2015 Operational procedures followed	Existing	Closed	Holcombe Road	Peak at 50m³ - hydrograph based of Figure C-1-9	Gauged flow for Makino at Reids Line
June 2015 operational procedures not followed	Existing	Open (attenuated flow provided by Philpott 2016a)	Holcombe Road	Peak at 50m³ - hydrograph based of Figure C-1-9	Gauged flow for Makino at Reids Line
April 2015	Existing	Closed	Holcombe Road	Not used	Gauged flow for Makino at Reids Line
Proposed with culvert flow 0.5% AEP	Proposed with culvert flow	Open (attenuated flow provided by Philpott 2016a)	Design events: 1-hr, 2-hr, 6-hr, 12-htr duration.	Assumed Maximum peak of 50cumecs	Design inflow of 70cumecs (assumes 120m3/s - 50cumecs in diversion)
Proposed with downstream blockage on Makino River 0.5% AEP	Proposed with DS blockage	Closed	Design events: 1-hr, 2-hr, 6-hr, 12-htr duration.	Assumed Maximum peak of 50cumecs	Design inflow of 70cumecs (assumes 120m3/s - 50cumecs in diversion)
Proposed with increase in roughness 0.5% AEP	Proposed with roughness increase	Closed	Design events: 1-hr, 2-hr, 6-hr, 12-htr duration.	Assumed Maximum peak of 50cumecs	Design inflow of 70cumecs (assumes 120m3/s - 50cumecs in diversion)
Proposed with decrease in roughness 0.5% AEP	Proposed with roughness decrease	Closed	Design events: 1-hr, 2-hr, 6-hr, 12-htr duration.	Assumed Maximum peak of 50cumecs	Design inflow of 70cumecs (assumes 120m3/s - 50cumecs in diversion)
Proposed with wet antecedent conditions	Proposed	Closed	Design events: 12-hour followed by 1-hr.	Assumed Maximum peak of 50cumecs	Design inflow of 70cumecs (assumes 120m3/s - 50cumecs in diversion)

3. Results

3.1. Validation event – June 2015

No site specific historical flooding information has been provided for Precinct 4 and although a validation would increase confidence in the model results it should be noted that there may be differences due to the timing of the data provided. Figure B-3-1 shows the predicted modelled water depths for June 2015.

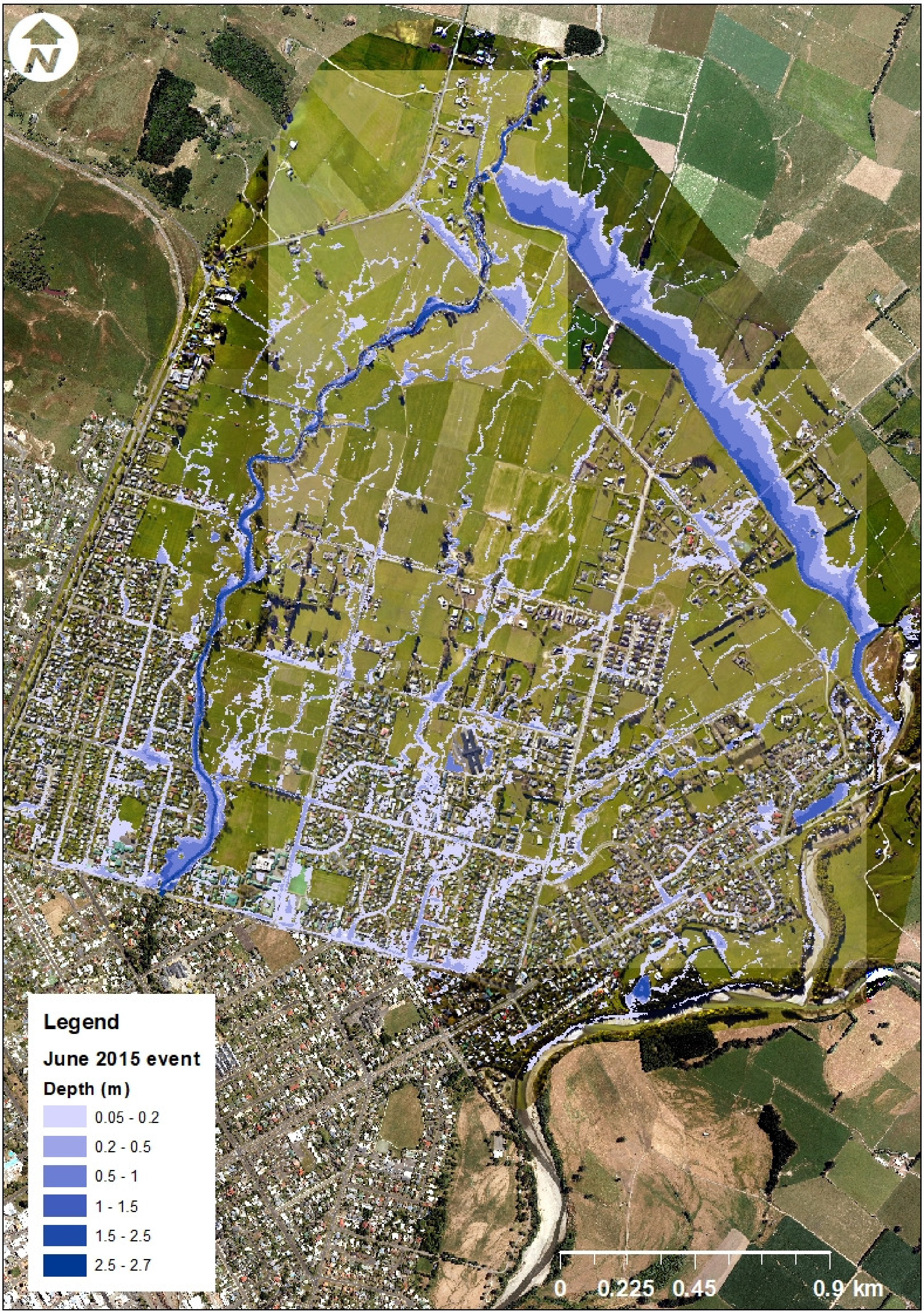

Figure B-3-1: June 2015 flooding event for the Existing undeveloped scenario

The 0.5% AEP event has also been simulated for the Existing situation in addition to the historical flood events to establish if there is greater flood risk during earlier stages of development.

3.2. Existing and proposed results

The ‘Existing’ and ‘Proposed’ 0.5% AEP results are shown in the main report.

Historical records have shown that this catchment can experience rainfall events in succession and therefore a simulation to consider wet antecedent conditions was undertaken. These results are also shown in the main report.

In some areas within Precinct 4, flood levels increase as a result of the development and in some cases they decrease. Capacity in the proposed pipes indicates that surface water can be collected to reduce flood levels in some areas without impacts others.

The 2-D model result predict that the flood hazard in Feilding township is reduced as part of the Precinct 4 development.

3.3. Sensitivity analysis

Additional simulations have been undertaken to determine the model sensitivity to hydraulic roughness. As each pointing he catchment is a key location of interest for this study, the impact on the extent has been reviewed, what is the additional impact across Precinct 4.

Roughness is represented with a Manning’s n value and this was increased and decreased by 20% in two separate sensitivity scenarios.

The analysis has shown that the model is not sensitive to increasing the roughness and the flood extent differences where minor. However when comparing the flood extents with the scenario is which the roughness value was decreased by 20%, the flood extents reduced noticeably. Given the uncertainty it is recommended that the conservative approach is adopted and roughness remain as proposed unless further attempts to validate the model are undertaken.

3.4. Residual uncertainty

There is uncertainty associated with 2-D hydraulic modelling due to input data (terrain and hydrological data) and analysis and assumptions to simplify reality to construct a model. This study has attempted to reduce the uncertainty by simulating historical flooding events to validate the model; by undertaking a sensitivity analysis on model parameters such as hydraulic roughness; and simulating alternative scenarios to establish whether additional contingency should be included in recommending minimum floor heights.

However there are still limitations to this approach. This includes:

- The results have been averaged across a mesh triangle and therefore although the resolution of the mesh is relatively fine (<20m^2), terrain and therefore modelled depths can also changes across this area.
- Channels are not explicitly defined in the mesh and therefore although all flowpaths have been reviewed and appear to be flowing, the conveyance may differ in reality. The mesh size of the Makino River is also smaller to ensure a better representation.
- Many invert levels have been assumed based on the location on plans. Once installed it may be beneficial for MDC to check the location, levels and pipe sizes to ensure that these are appropriately represented in the model.
- The exact locations of buildings has not been taken into account. Once development is underway it may be appropriate to revisit the conceptualisation of these buildings in the model and include specific polygons.

Appendix C
Hydrology

1. Hydrology

An assessment of the temporal pattern of rainfall and local overland flows in the vicinity of Precinct 4 are required to set boundary conditions for the Feilding development model. Of particular interest are the flows and rainfalls during two significant flooding events (April and June 2015) to validate the flood model.

1.1. Rainfall

1.1.1. Data review

The following long-term rainfall records were received from Horizons Regional Council (Horizons) (Table C-1-1).

The two rain gauges close to Feilding are Makino at Holcombe Road and Makino at Cheltenham. The Cheltenham rain gauge is approximately 10.5km to the north east of the development area, on a relatively flat area with similar characteristics to the low-lying areas of Feilding. The Holcombe Road rain gauge is 2.8km from the development area, on top of a hill to the west of Feilding (Figure C-1-1). Both rain gauges are operated by Horizons and provide high resolution (6-min) rainfall data which covers both calibration events.

Figure C-1-1: Rainfall and flow sites in close proximity to the development area.

Table C-1-1: Summary of available rain gauges in the vicinity of Feilding (Horizons Regional Council)

Site Name	Type	Start	End	Easting (NZMG)	Northing (NZMG)	Altitude (MSL)
Makino at Cheltenham	6-min	28-Jul-1998	04-Aug-2017	2734639	6117740	226
Makino at Holcombe Road	6-min	16-Dec-1998	06-Jul-2017	2725516	6109311	114.1

Cumulative rainfall analysis of these two sites indicates that the Cheltenham gauge typically receives more rainfall than Holcombe Road (Figure C-1-2). This is likely a result of the elevation of the rain gauge (226m) and distance from the Manawatu Plain. Comparison of the cumulative rainfall over the calibration events (April 2015 and June 2015) demonstrates similar temporal patterns and total rainfall depths (i.e. within 6%) (Figure C-1-3 and Figure C-1-4).

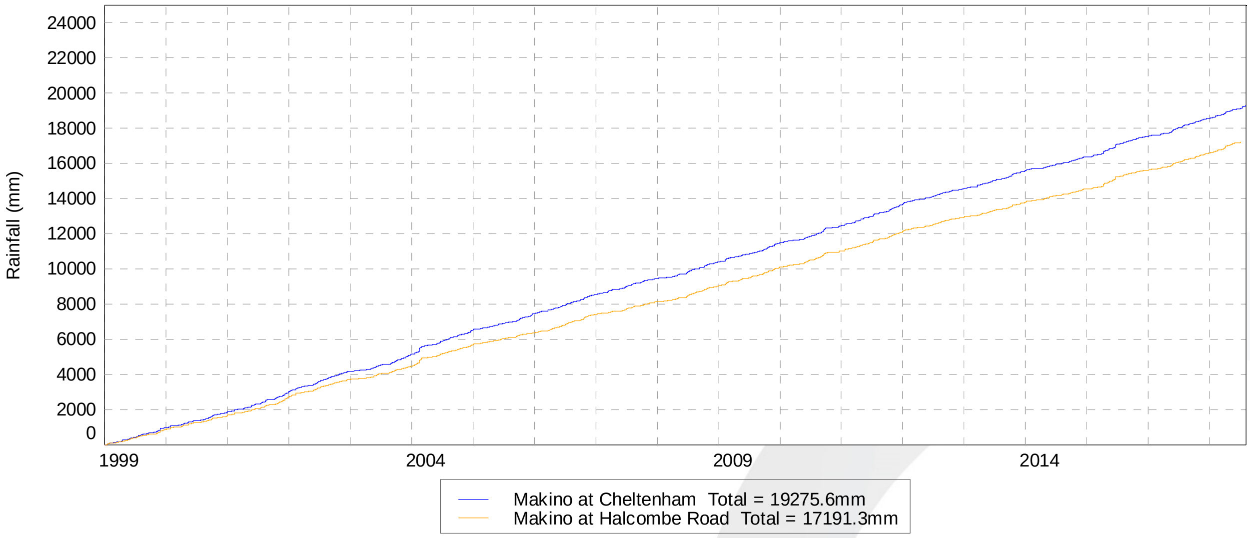

Figure C-1-2: A comparison of Holcombe Road and Cheltenham cumulative rainfalls since 1999.

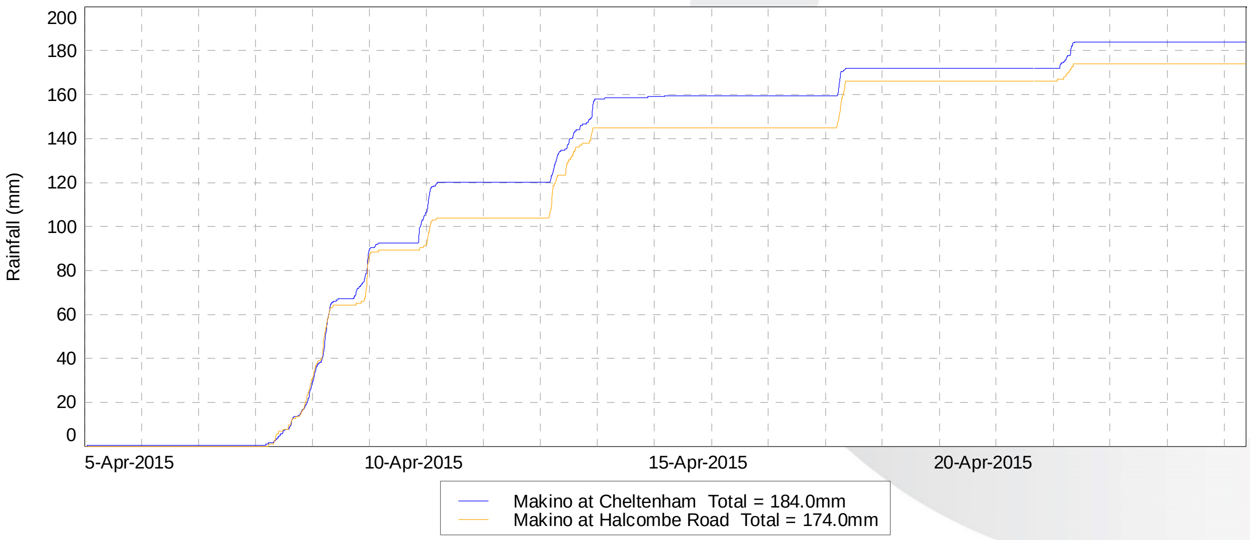

Figure C-1-3: Cumulative rainfall at the Cheltenham and Holcombe Road gauges during the April 2015 rainfall event.

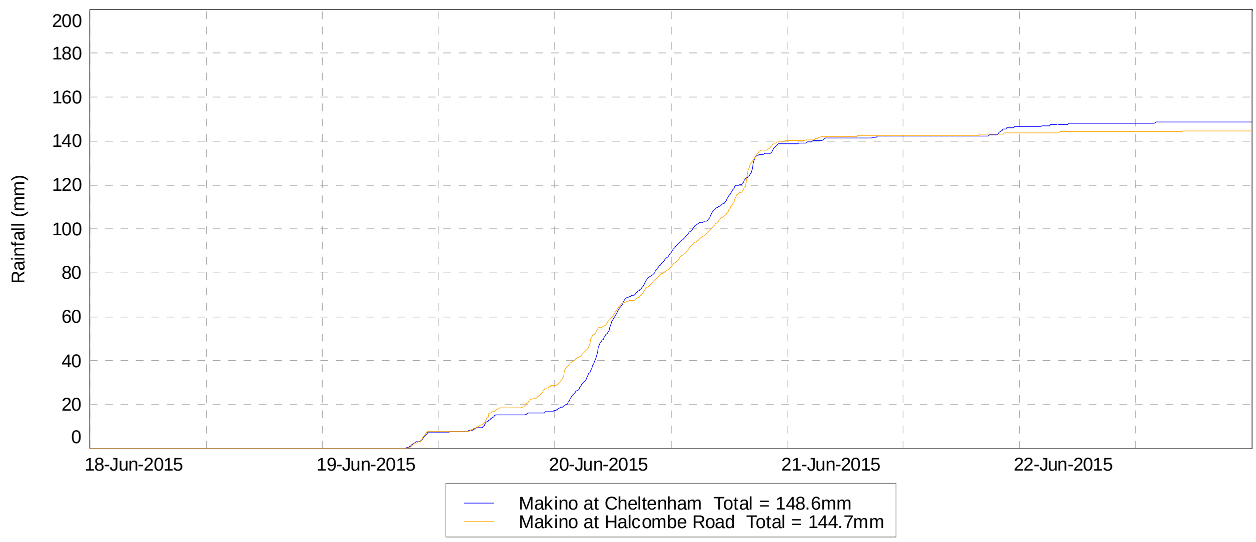

Figure C-1-4: Cumulative rainfall at the Cheltenham and Holcombe Road gauges during the June 2015 rainfall event.

1.1.2. Temporal rainfall pattern

It is considered that the Holcombe Road gauge is both spatially appropriate and likely to be more representative of the temporal distribution of rainfall over the proposed development area. than Cheltenham Street. Further analysis is therefore based on the rainfall record at Holcombe Road.

The temporal pattern of rainfall during the five largest events; over each of a range of storm durations (1-hour to 72-hours) were used to derive an average temporal pattern for each duration. These average temporal distributions have been compared with the 6 and 12-hours PMP from Tables and 7.4 and 9.4 in Tomlinson and Thompson (1992) (Figure C-1-5).

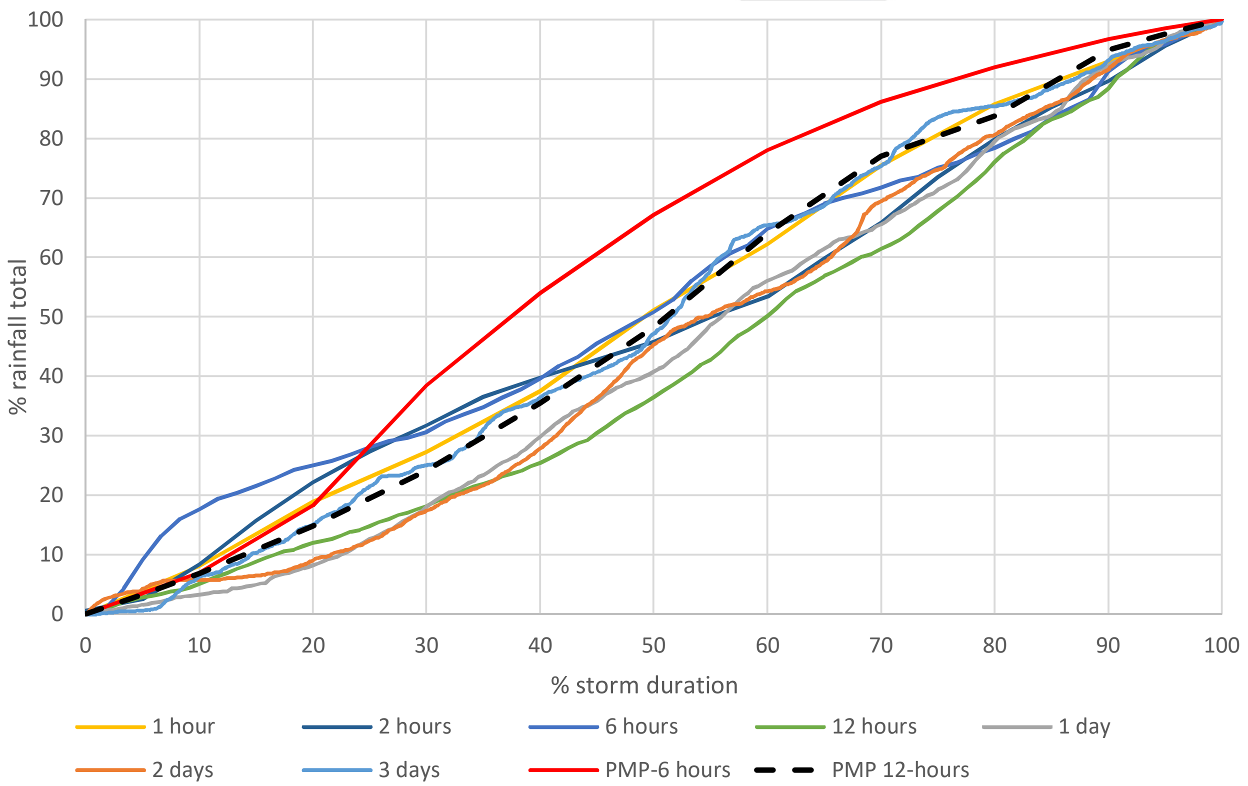

Figure C-1-5: The average temporal pattern of rainfall recorded at Holcombe Road over a range of storm duration events.

Analysis of the temporal patterns of rainfall during events from 1-hour to 72-hours in durations indicates a relatively even distribution of rainfall over the storm. These distributions compare well against the 12-hour North Island PMP distribution, especially during the first 50% of the storm event.

Rainfall depths for 1% and 0.5% AEP events over a range of storm durations were therefore scaled using the 12-hour PMP NZ temporal rainfall pattern.

1.1.3. HIRDS comparison

HIRDS is an acronym for High Intensity Rainfall Design System. It is a generalised procedure to obtain spatially and temporally consistent depth-duration-frequency design rainfalls for New Zealand. In the absence of a site-specific empirical data to produce a design rainfall table, generalised design rainfalls can be obtained from HIRDS. While design rainfalls derived from actual site-specific measurements are more reliable than those from HIRDS, the location and elevation of the Holcombe Street and Cheltenham rain gauges, may render design rainfall tables that are overly conservative.

It is useful to test, where practicable, the validity of data from HIRDS where these overlap those from measured high intensity rainfall data. This helps to confirm the validity of applying HIRDS data to locations where there are no empirical rainfall data.

Data were therefore obtained from HIRDS for the locations of the two gauges at Holcombe Road and Cheltenham. These data were then compared to those derived from the empirically-derived design rainfalls at each site to determine the appropriateness of HIRDS in this area.

The percentage difference between the design rainfalls obtained from the two sources (i.e. from HIRDS and empirically-derived) are displayed in Table C-1-2 for Holcombe Road. Positive percentages indicate that the design rainfalls from HIRDS are greater than those from the empirical data i.e. HIRDS are conservative.

Table C-1-2: Comparison of HIRDS design rainfall and empirical design rainfall for Holcombe Road for 1% AEP durations.

Rainfall	Duration								
	10-m	30-m	1-hr	2-hr	6-hr	12-hr	24-hr	48-hr	72-hr
HIRDS	21.6	35.8	49.3	61.7	87.9	109.9	137.4	162.6	179.5
Empirical	13.4	28.3	35.5	38.9	59.8	85.7	127.8	159.3	168.7
Difference	38%	21%	28%	37%	32%	22%	7%	2%	6%

In general, HIRDS appears to over-estimate the design rainfalls significantly for the shorter duration (i.e. 10min – 12-hr), rainfall events at both locations. It provides reasonable estimates of design rainfalls for the longer duration (i.e. 24-72hr) events.

The results suggest that the use of HIRDS provides more conservative estimates of rainfall depth and intensity at the development site. The use of these data ensures conservative design.

Design rainfall data was therefore obtained from HIRDS for the development site (Table C-1-3).

Table C-1-3: HIRDS design rainfall depths centred on the development area.

AEP (%)	ARI (yrs.)	Duration								
		10-m	30-m	1-hr	2-hr	6-hr	12-hr	24-hr	48-hr	72-hr
50	**2.33**	7.5	12.4	17.1	22.7	35.5	47.1	62.6	74.6	82.7
20	**5**	10	16.6	22.9	29.9	45.6	59.6	77.8	92.8	102.9
10	**10**	12.2	20.2	27.7	35.8	53.8	69.6	89.9	107.2	118.8
5	**20**	14.6	24.2	33.3	42.7	63.1	80.7	103.3	123.1	136.5
2	**50**	18.6	30.8	42.4	53.5	77.4	97.8	123.5	147.3	163.3
1	**100**	22.2	36.8	50.7	63.4	90.3	113	141.3	168.5	186.7

1.1.4. Climate Change

Horizons Regional Council consider that a 0.5% AEP rainfall event is equivalent to the 1% AEP event including the predicted impacts of climate change to 2090.

In 2008 MfE produced *"Climate change effects and impacts assessment – A guidance manual for Local Government in New Zealand – 2nd Edition"*. Different climate change scenarios are provided for the various regions throughout the country. Estimates of the expected increase in temperatures out to the 2040s and 2090s are also provided.

The mean annual temperature for the Manawatu-Wanganui region is predicted to increase by between a lower limit of 0.6°C and an upper limit of 5.5°C by the 2090s; with an average of 2.1°C (Ministry for the Environment, 2010).

The MfE methodology recommends a percentage adjustment per degree of warming that should be applied to high intensity rainfall to account for the effect of global warming. For example, during any event with an AEP less than about 3%, rainfall is expected to increase by 8 percent per degree of warming. Consequently, rainfalls within the Feilding catchment might be expected to increase by an average of 16.8% by the 2090s (i.e. 2.1ºC x 8%).

To allow for the potential effects of global warming over the next 100 years i.e. to 2117 rather than 2090, the design rainfall depths have been increased by 20%; rather than 16.8%. The 1% AEP and 0.5% AEP (assumed to be 1% AEP adjusted for the effects of climate change) design rainfall depths are shown in Table C-1-4 for various durations.

Table C-1-4: 1% AEP design rainfall depths adjusted for climate change (0.5% AEP design rainfall depths).

AEP (%)	ARI (yrs.)	Duration								
		10-m	30-m	1-hr	2-hr	6-hr	12-hr	24-hr	48-hr	72-hr
1	100	22	36.8	50.7	63.4	90.3	113	141.3	168.5	186.7
0.5	200	26.6	44.2	60.8	76.1	108.4	135.6	169.9	202.2	224.0

1.2. Flows

Precinct 4 is bounded by a number of hydrologically features: the Makino River to the west; the Makino Diversion structure to the north, the confluence of the Kiwitea and Oroua Rivers to the east, and Feilding (specifically North Street) to the south.

1.2.1. The Oroua River

The Oroua River floodplain to the east of Precinct 4 flows at a lower elevation than the historic terrace on which Precinct 4 is located. Consequently this floodplain will have no influence on flooding on the site directly, or on the outfall of the Makino diversion that may affect the site. Flows in the Oroua River have therefore been omitted in this study and the boundary conditions set to allow runoff from the model.

1.2.2. The Makino River

Horizons operate a number of flow sites on the Makino River. These sites are primarily to monitor flooding, and are not considered reliable at recording flows less than 10m³/s.

The Makino River at Reids Line flow site is located approximately 400m downstream of the Makino River diversion gates (Figure C-1-6), and has a record spanning 29 years (Figure C-1-7). The diversion gates were installed in 2009. Subsequently, the derivation of design flows must be conducted on data prior to the installation of the gates.

Table C-1-5: Flow sites operated by Horizons on the Makino River.

Site Name	Start	End	Length of record (years)
Makino at Boness Road	19-Dec-1991	21-Sep-2017	26
Makino at Rata Street	23-Dec-1986	21-Sep-2017	31
Makino at Reids Line	24-July-1988	21-Sep-2017	29

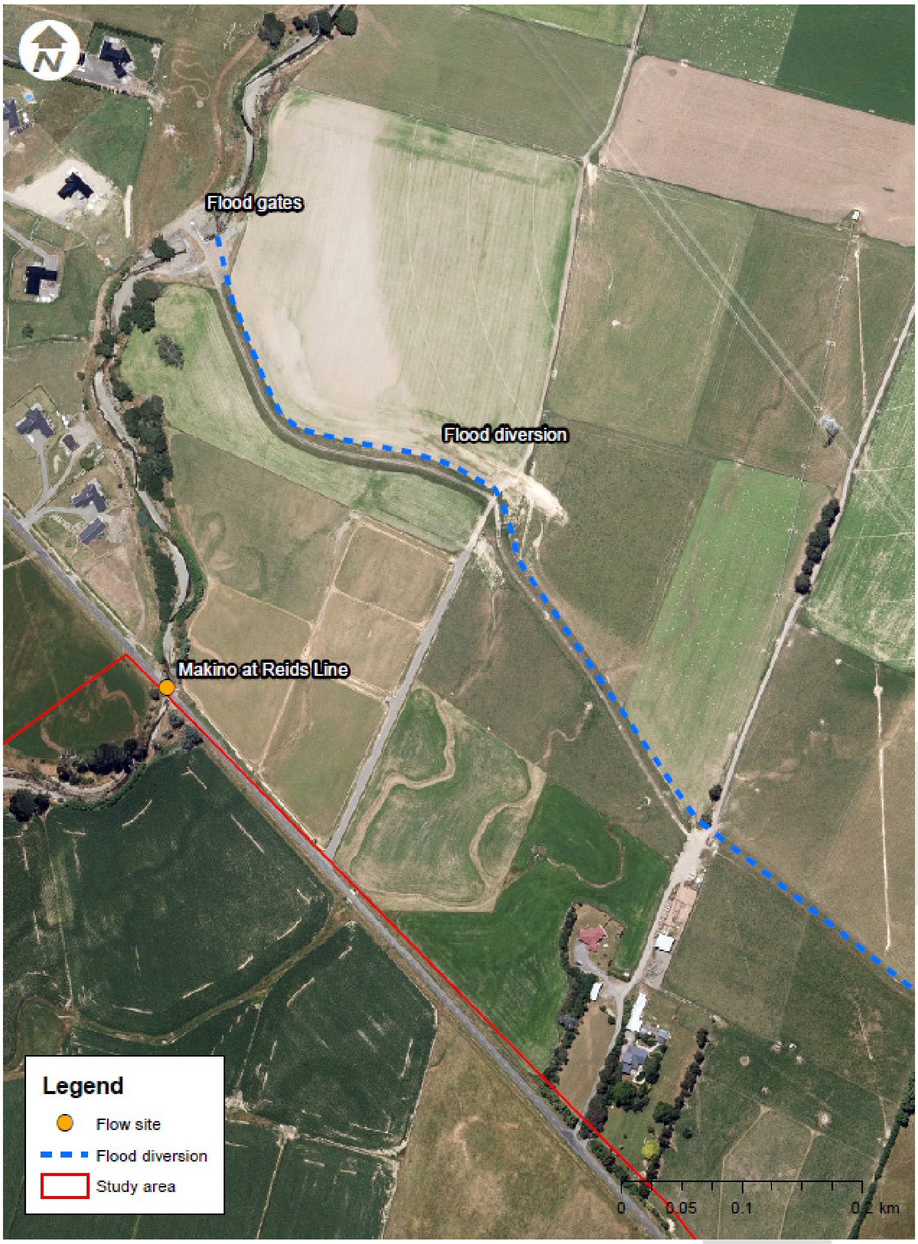

Figure C-1-6: The Makino River flood control gates, diversion and flow monitoring site at Reid's Line.

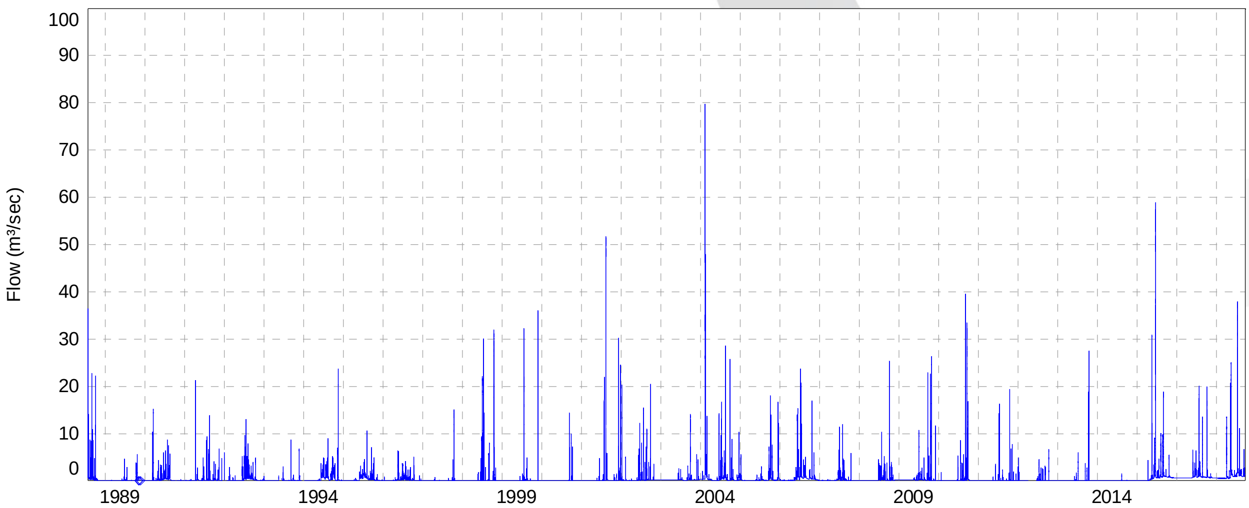

Figure C-1-7: The Makino River at Reids Line flow record (1988-2017).

1.2.2.1. Frequency Analysis

An analysis of the annual flood maxima series from the Makino River at Reids Line was undertaken to confirm that a previous 1% AEP design flow estimate of 100m³/s was realistic; and to allow extrapolation of more extreme events (i.e. 0.5% AEP).

Frequency analysis was undertaken on the annual flood maxima series derived from the flow record up to 2009. Three types of statistical distribution were assessed for how well they modelled the actual annual flood maxima series (i.e. Gumbel, Pearson 3 (PE3) and GEV). The distribution which provided the best fit to the annual flood maxima series was then used to estimate flows for storm events of specific AEPs.

As is standard practice, the frequency analyses were performed on a 12-month partition. That is, only the largest flood in each year was plotted, and the most appropriate statistical distribution fitted to those annual values. It is sometimes difficult to find a single statistical distribution that provides an excellent model of the annual maxima series. In these situations, some subjectivity is required in selecting the most appropriate model. The criteria adopted in this study were:

- The distribution that provided the best-fit through all the data points;
- The distribution with the most realistic shape; and
- The distribution that provides the closest approximation to the extreme values.

While this process may appear subjective, in most cases the choice of a specific statistical distribution for the annual maxima series results in relatively minor differences in the estimated flow-frequency table; at least for events out to 1% AEP (i.e. 100-year ARI).

The PE3 distribution was selected as the most appropriate statistical distribution to fit the annual flood maxima series. Design flows of various AEP's are presented in Table C-1-6.

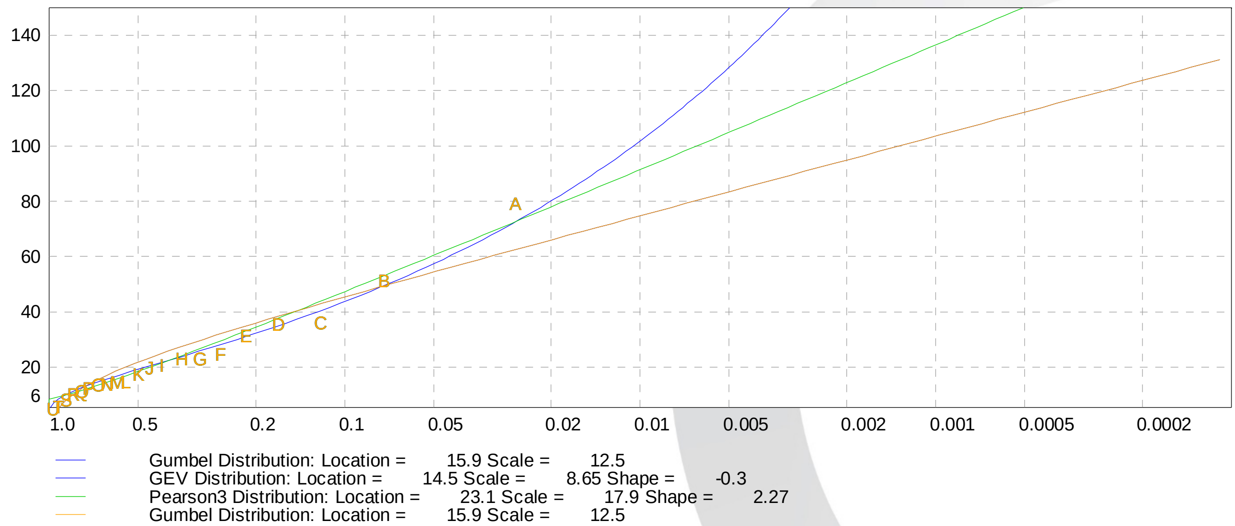

Figure C-1-8: Frequency analyse of the annual flow maxima series at Makino River at Reids Line, (1975-2017).

Table C-1-6: Design flows derived from the frequency analysis of the annual flow maxima series at Makino River at Reids Line.

ARI (Years)	AEP (%)	Design Flows (Horizons) (m³/s)	Design Flow Opus (m³/s)
10	10	53	46
20	5	66	59
50	2	85	77
100	1	100	90
200	0.5	120	108*

*Following the scaling approach outline in Section 1.1.4

Design flow estimates provided by Horizons are roughly 10-15% greater than those calculated using the above methodology. Therefore adoption of these design flow estimates will be conservative (i.e. estimate greater flows down the Makino).

1.2.3. Makino flood diversion scheme

When required, Horizons will operate the Makino flood diversion scheme. During high flow events Horizons station an Officer at the Duke Street Bridge, who assesses the level of the river and initiates use of the diversion. During operation, this Officer will coordinate the adjustment of the gates to account for runoff from the upstream area to ensure that the Makino River does not breach its banks at Duke Street. Horizons have indicated that the capacity of the Makino River to convey flow through Feilding is 50m³/s.

Since the capacity of the diversion channel is 50m³/s, Horizons cannot divert a higher flow. Subsequently, if the capacity of the diversion is reached, all remaining flow is passed down the Makino River. The scheme has been designed to convey a total flow of 100m³/s (50m³/s down the Makino River and 50m³/s down the diversion) which is equivalent to a 1% AEP flow in the Makino River (Table C-1-6).

There is no empirical data for the gate openings during operation; however, "design scenario hydrographs" during a 1% AEP flood event have been provided by Horizons (Figure C-1-9).

The 0.5% AEP flow for the Makino upstream of the diversion structure is 120m³/s. To account for the 0.5% AEP flow, the hydrographs have therefore been adjusted, considering the operating capacity of the diversion (Figure C-1-10).

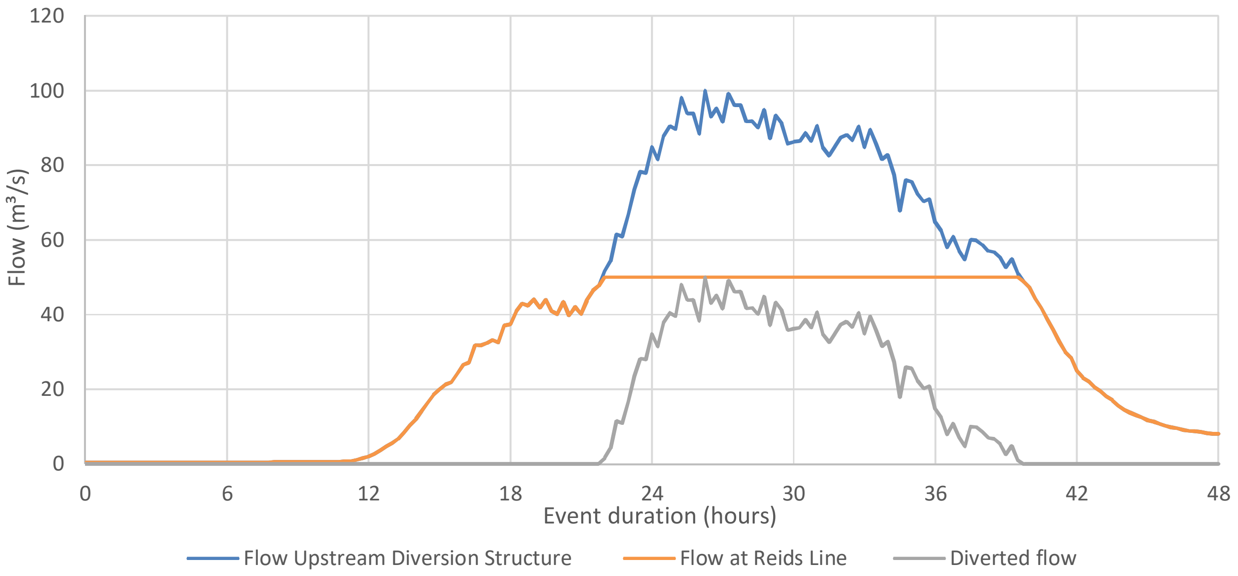

Figure C-1-9: Operation of the Makino River Flood Diversion during a 1% AEP flow event (100m³/s) (provided by Horizons)

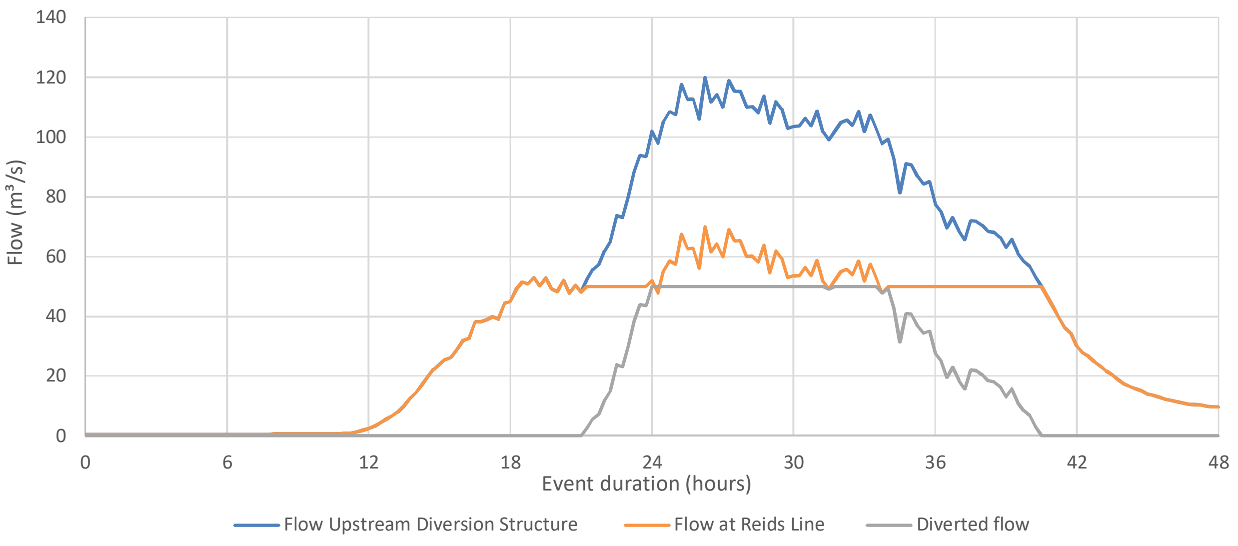

Figure C-1-10: Operation of the Makino River Flood Diversion during a 0.5% AEP flow event (120m³/s)

1.2.3.1. Flood Diversion Culverts

Seven culverts with control gates exist through the Makino diversion stop bank. Under normal conditions, these culverts are left open to allow conveyance of a number of natural channels through the diversion bank.

However, during flood events larger than the 30-year event or when the Makino diversion operates, the culvert control gates will be closed and no flow will pass downstream of the floodway bank.

The operation of the diversion culverts is controlled by local land owners. Anecdotal evidence suggests that during the operation of the Makino diversion, these gates are sometimes left open, or prematurely opened during a flooding event by property owners to reduce flooding upstream. For this reason, both validation models, and the design events will be run under a range of scenarios that will model these culverts being open and closed as it cannot be assumed that during a storm event that coincides with the operation of the diversion that these will be shut.

The capacities of these culverts in the 3% AEP design flows (as provided by Horizons) are presented in Table 1-7. A brief comparison of the attenuated discharges through the culverts by Opus, based on invert levels from LiDAR and the culvert diameters provided, has validated these attenuated discharges.

Table 1-7: Culvert details

Culvert	Diameter	Attenuated discharge (MDC)	Max discharge (Opus)	3% AEP discharge event (upstream of culvert)
Terry Clare *	300	0.19		0.53
White U/S	450	0.38	0.37	2.76
White Mid	750	0.92	0.96	5.85
White D/S	750	0.75	0.63	5.42
Botham	450	0.31	0.3	1.88
Spicer	450	0.58	0.54	2.52
Bailey	375**	0.26	0.29	2.24

* The Terry Clare culvert has been removed and the flow diverted along the upstream side of the floodway stop bank to pass through the White U/S culvert

** Two culvert diameters have been provided for this culvert (375mm and 300mm). Based on attenuated discharges, and to be conservative, the culvert is considered to have a diameter of 375mm.

OPUS