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Report

Mangaheka Integrated Catchment Management Plan - Stormwater 1D Modelling Report

Prepared for Hamilton City Council

Prepared by CH2M Beca Ltd

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Revision History

Revision Nº	Prepared By	Description	Date
1	**Angela Pratt**	Modelling Report	8/12/16
2	**Angela Pratt**	Updated based on Client comments	16/2/17
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Document Acceptance

Action	Name	Signed	Date
Prepared by	**Angela Pratt**		6/6/17
Reviewed by	**Michael Law**		6/6/17
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on behalf of	CH2M Beca Ltd		

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1 Introduction

CH2M Beca Ltd has been engaged by Hamilton City Council (HCC) to undertake 1D hydraulic modelling of the Mangaheka Stream catchment in the north-west part of Hamilton. This catchment sits across the boundary of Hamilton City and Waikato District. The section of catchment within the Hamilton City boundary (upper part of the catchment and model) has been designated for future development and is currently underway in the Partly Operative District Plan. To support this development, HCC is preparing an Integrated Catchment Management Plan (ICMP).

The modelling undertaken has sought to assess the effects of development on stream water levels, peak flows and flooding duration. It has also sought to determine what size attenuation devices would be needed to mitigate these effects.

The modelling has been undertaken using the HEC-RAS modelling package. The basis of the modelling is an existing model developed by Lysaght Consultants Ltd during the development of the Hamilton Joint Venture Development and Porters Development in 2012. These developments are located in the upper reaches of the catchment. During design of these developments, AECOM also developed a Mike model as part of the detailed design of these developments.

The Lysaght HEC-RAS model has been updated to include the existing development (ED) and flood mitigation devices that have been installed as part of these developments. In addition, since 2012 the 4 Guys car yard and Z Energy petrol station have also been developed at the upper end of the catchment. This development includes an attenuation pond which has been included in the updated model.

Existing Development (ED) and unmitigated Maximum Probable Development (MPD) scenarios have been tested during 10-year and 100-year average recurrence interval (ARI) events, as well as an MPD with mitigation scenario. The effects of climate change have also been assessed.

2 Model Objectives

Objective 1: The primary objective of the modelling is to assess the impacts of future developments in the catchment on peak water levels and flows downstream, and to confirm what is required to mitigate these effects.

In addition, the following objectives were incorporated:

Objective 2: Update the Lysaght 1D model, to take account of currently-consented development (Porters/HJV/4 Guys developments) – this formed the Existing Development (ED) model in the current study.

Objective 3: Confirm the conclusions of the Lysaght/AECOM modelling, i.e. that to maintain flood levels downstream, mitigation is needed to reduce peak flows to 70% of pre-development.

Objective 4: For the Maximum Probable Development (MPD) scenario, confirm sizing of mitigation required to maintain current flood levels downstream

Objective 5: Confirm that the mitigation does not result in overbank flooding which is longer than 72 hours in duration. This is a requirement of Land Drainage Board managed by the Waikato Regional Council in relation to flooding of farmland. When flooded longer than 72 hours, grass die-off can occur which is a problem for livestock farmers.

Note that, in relation to the objectives, the focus of the modelling has been on the 100-year ARI event as this governs the overall size of attenuation devices and the design of the device outlet structures. We have also run 10-year ARI scenarios, however we have not specifically designed the outlet structures in terms of achieving the above objectives in this event. During detailed design, outlet structures will need to be optimised to achieve objectives in the 10-year ARI event.

3 Catchment Overview

The Mangaheka catchment is approximately 2100 ha in size with approximately 10% existing imperviousness, mainly focussed in the industrial areas in the upper catchment. At MPD it is likely that this will increase to approximately 15%.

There is a variety of roads dissecting the catchment, with the most major ones being the Te Rapa Bypass motorway in the upper catchment and Ngaruawahia Rd in the lower and western parts of the catchment. When Te Rapa Bypass was constructed, Koura Drive was also constructed to provide an on-off ramp system connecting the two sides of the motorway.

In terms of topography, the catchment is primarily flat, rural farmland, however the lower parts of the Mangaheka stream are quite incised and densely vegetated. The north-eastern boundary of the catchment is bounded by an area of higher rolling hills.

The above features are shown in Appendix A.

4 Model Inputs

4.1 Model Description

As described earlier, the modelling has been based on an existing HEC-RAS model developed by Lysaght Ltd. The hydrological inputs (hydrographs) to the HEC-RAS model were originally developed by Lysaght Ltd using Drains, an Australian hydrological modelling program. For our work, we have converted this Drains model to a HEC-HMS model as this directly links to HEC-RAS. This was considered a more efficient and accurate way to update the HEC-RAS model given you are not likely to have data transfer errors between two programs that don't have a direct interface, as was potentially the case with Drains.

4.2 Catchment Characteristics

The catchment has been broken down into sub-catchments for the purposes of determining runoff. These catchments are shown in Appendix B. Parameters used to determine the flow generated by each catchment are presented as Appendix C. These characteristics include:

- Catchment area
- Imperviousness
- Curve number
- Time of concentration

Note that as far as possible, sub-catchment divisions are consistent with those used in previous modelling (Lysaght, 2012). Some minor adjustments have been made to account for recent developments as well as the construction of the Te Rapa Bypass motorway.

4.3 Stream Channel Cross-sections

The bulk of the stream channel cross-sections have been taken directly from the previous Lysaght model. During a site visit it was noted that some of these cross-sections should be refined. Surveying in a number of locations has provided additional catchment boundary information and channel cross-section changes.

4.4 Mannings Roughness

The mannings roughness values used were as per the original Lysaght HEC-RAS model. We have however reviewed a selection of cross-sections through the model to check appropriateness of the values. Table 2 below shows the values used for the various surfaces in the model.

Table 1: Mannings Roughness Values

Surface Type	Mannings Roughness
Pipes	0.012
Channel bed	0.030
Channel sides	0.030
Floodplain	0.045

4.5 Downstream Water Levels

The HEC-RAS model extends to the downstream confluence with the Waipa River. At this location, a boundary condition in the form of a fixed water level has been applied. These have been taken from the existing Lysaght model, those being:

- 10-year ARI water level = 14.33 m RL
- 100-year ARI water level = 16.07 m RL

4.6 Rainfall Intensities and Storm Shape

Rainfall intensities and storm shape have been taken from the HCC Standard Stormwater Modelling Methodology (HCC, 2013).

4.7 Time of Concentration

Time of concentration for each catchment has been calculated using the method described in TP108. Whilst TP108 is an Auckland specific flow calculation method, in terms of determining the time of concentration, this aspect of TP108 is widely used outside of Auckland.

5 Model Scenarios

Table 2 below shows the scenarios that have been modelled. Note that this includes scenarios with and without climate change. Climate change adjustments are provided for in HCC, 2013 which incorporates a 2.08 degree increase in temperature.

Table 2: Model Scenarios

Scenario	10 year ARI	100 year ARI
Existing Development	✓	✓
Existing Development with Climate Change (CC)	✓	✓
MPD without mitigation	✓	✓
MPD without mitigation (with CC)	✓	✓
MPD with mitigation (with CC)	✓	✓

6 Reporting Locations

In assessing the effects of the proposed development, we have determined a number of key locations where effects have been compared in terms of water elevation, flow rate and drain-down duration data. Appendix D shows the twelve locations selected. Appendix D also has a table describing each location as well as their HEC-RAS model chainage. Note that locations 5 and 7 are the sites of proposed offline detention basins, while location 4 is downstream of a proposed inline basin. Results for these locations are therefore only important for the MPD with mitigation scenarios.

7 Model Results – Without Mitigation

7.1 Overview

To be able to determine what mitigation might be required, we have first assessed the impact of development. In accordance with HCC requirements, attenuation also takes account of the effects of climate change (i.e. increases in rainfall intensity)

Our assessment of effects is based on comparisons between ED and MPD water levels and peak flows.

7.2 100-year ARI Results

Table 3 below lists peak water level at each reporting location for 100-year ARI model scenarios without mitigation.

Table 3: Maximum water levels for 100-year ARI scenarios

Location Label	ED 100 yr	ED 100 yr with CC	MPD 100 yr	MPD 100 yr with CC	Difference: MPD 100 yr CC – ED 100 yr
	mRL	mRL	mRL	mRL	m
1	32.26	32.42	32.90	32.95	0.69
2	32.15	32.29	32.36	32.61	0.46
3	31.51	31.62	31.51	31.61	0.1
4	30.63	30.86	31.03	31.21	0.58
5	n/a	n/a	n/a	n/a	n/a
6	30.46	30.58	30.86	31.03	0.57
7	n/a	n/a	n/a	n/a	n/a
8	29.34	29.46	29.40	29.49	0.15
9	26.61	26.67	26.63	26.68	0.07
10	22.68	22.86	22.71	22.87	0.19
11	16.34	16.46	16.34	16.48	0.14
12	16.19	16.22	16.19	16.22	0.03

Table 4 shows the maximum flow rates for the 100-year ARI event without mitigation.

Table 4: Maximum flow rates for 100-year ARI scenarios

Location Label	ED 100 yr	ED 100 yr with CC	MPD 100 yr	MPD 100 yr with CC	Difference: MPD 100 yr CC – ED 100 yr
	m³/s	m³/s	m³/s	m³/s	
1	0.26	0.25	1.04	1.55	496%
2	0.55	0.58	0.60	0.66	20%
3	1.45	3.10	1.47	3.01	108%
4	3.70	4.40	4.83	5.33	44%
5	n/a	n/a	n/a	n/a	
6	3.48	5.52	3.70	3.93	13%
7	n/a	n/a	n/a	n/a	
8	5.84	6.87	7.90	8.64	48%
9	14.77	18.11	15.50	18.60	26%
10	21.05	32.90	22.87	34.32	63%
11	43.68	58.34	44.25	59.89	37%
12	60.65	78.53	61.26	80.08	32%

Table 3 and Table 4 above show that in comparing MPD 100 year (with CC) with ED 100 year, there is an increase in both water level and peak flows at all locations. This is the effect that we have then sought to mitigate by including a number of proposed attenuation devices in the model. Comparing ED 100 year and MPD 100 year indicates the effect of only the MPD development i.e. without considering climate change. The only location where (close to) no increase is seen in MPD 100 year is Location 3. This location is at the outlet of the HJV pond, which has already been designed to take account of climate change and also where the catchment area will reduce at MPD. At MPD the catchment south of Te Rapa Bypass (the Shark-fin) will flow into the Rotokauri catchment, rather than the Mangaheka catchment.

Long-sections of water levels in the catchment are shown in Section 9.1.1.

7.3 10-year ARI results

Table 5 below lists peak water level at each reporting location for 10-year model scenarios without mitigation.

Table 5: Maximum water levels for 10-year ARI scenarios

Location Label	ED 10 yr	ED 10 yr with CC	MPD 10 yr	MPD 10 yr with CC	Difference: MPD 10 yr CC – ED 10 yr
	mRL	mRL	mRL	mRL	m
1	31.92	32.03	32.52	32.70	+0.78
2	31.77	31.87	31.79	31.86	+0.09
3	31.15	31.30	31.14	31.29	+0.14
4	30.24	30.36	30.61	30.76	+0.52
5	n/a	n/a	n/a	n/a	n/a
6	30.04	30.20	30.49	30.63	+0.59
7	n/a	n/a	n/a	n/a	n/a
8	29.23	29.31	29.36	29.41	+0.18
9	26.22	26.43	26.34	26.47	+0.25
10	21.38	21.84	21.49	22.01	+0.63
11	15.26	15.44	15.27	15.45	+0.19
12	15.15	15.30	15.16	15.31	+0.16

Table 6 shows the maximum flow rates for the 10 year event without mitigation.

Table 6: Maximum flow rates for 10-year ARI scenarios

Location Label	ED 10 yr	ED 10 yr with CC	MPD 10 yr	MPD 10 yr with CC	Difference: MPD 10 yr CC – ED 10 yr
	m^3/s	m^3/s	m^3/s	m^3/s	%
1	0.18	0.20	0.41	0.48	267
2	0.39	0.47	0.41	0.46	118
3	0.70	0.85	0.68	0.82	117
4	2.05	2.66	3.58	3.97	194
5	n/a	n/a	n/a	n/a	n/a
6	2.53	2.81	3.18	3.37	133
7	n/a	n/a	n/a	n/a	n/a
8	3.62	4.18	5.40	6.05	167
9	7.62	10.23	8.94	11.06	145
10	10.20	11.84	10.61	12.37	121
11	21.27	26.84	21.82	27.45	129
12	29.28	36.64	29.77	37.22	127

Tables 5 and 6 above show that in comparing MPD 10 year (with CC) with ED 10 year, there is an increase in both water level and peak flows at all locations except location 3. Comparing ED 10 year and MPD 10 year also indicates an increase at all locations except location 3. As described earlier, location 3 is at the outlet of the HJV pond, which has already been designed to take account of climate change and also where the catchment area will reduce at MPD. This means that the target was ED 10 year

8 Proposed Mitigation

8.1 Overview

As shown in Table 7, a number of attenuation devices are proposed to mitigate the water level and peak flow increases as a result of development (MPD compared to ED) as well as climate change. The locations of these devices are depicted in Appendix E. Pond configurations were based on discussions with HCC. A summary of these discussions is attached as Appendix F.

Table 7: Existing and Proposed Flow Mitigation Devices

Device	Existing or New	Type	Mitigates for development in
Porters Pond	Existing	Inline	Catchment F
HJV Pond	Existing	Inline	Catchment B
4 Guys Pond	Existing, to be modified	Inline	Catchment A (MPD)
Device 7	Proposed	Inline	Catchment C
Device 6	Proposed	Offline	Catchment D*
Device 5	Proposed	Offline	Catchments G, H and E**

*Flows into this device only come from the south-west side of the stream, however the device attenuates for the whole catchment i.e it over attenuates the flows which reach the basin to account for the parts of the catchment that won’t reach the basin.

**This device over attenuates flows from catchments E and H, thus also providing attenuation for catchment G.

In accordance with HCC requirements, attenuation has been sized to mitigate the effects of development as well as climate change. The criteria for achieving this is shown in Section 8.2.

8.2 Device Design Targets and Constraints

In designing the modifications to the existing 4 Guys pond and the proposed new devices, a number of constraints and design targets needed to be met/achieved. In terms of the constraints, we have sought to not increase the water levels in the existing devices, such that existing flood levels upstream are not increased.

Table 8 lists flow and water level design constraints of the existing device that is to be modified (4 Guys Pond) and design criteria for the proposed devices. Depending on the location, either the water level or flow rate governed. Table 13: details which governed for each of model reporting locations.

In sizing devices, we began with the previous modelling target of reducing peak flows to 70% of predevelopment and then assessed whether this is still appropriate for MPD. This is discussed further as part of the results in Section 9.1.5.

Table 8: Mitigation design targets/constraints

Device	Targets/Constraints
4 Guys Pond	■ Water level in 4 Guys Pond (location 1) must be ≤ 32.31 mRL (‘ED 100 yr’ scenario) ■ Water level downstream of 4 Guys Pond (location 2) must be ≤ 32.17 mRL (‘ED 100 yr’ scenario)
Device 7	■ Water level in HJV pond (location 3) must be ≤ 31.7 mRL (‘ED 100 yr with CC’ scenario) ■ Outflow from Device 7 (location 4) must be ≤ 5.7 m^3/s (equivalent to 70% contribution of 'ED 100 yr' flow from catchment C plus flow coming from upstream i.e. HJV pond outlet) ■ Water level downstream of Device 7 bund (location 4) must be ≤ 30.76 mRL (‘ED 100 yr’ scenario)
Device 6	■ Water level in Device 6 basin (location 5) must be ≤ 31.2 mRL (minimum ground elevation in nominal basin location) ■ Outflow from Device 6 (location 5) must be ≤ 0.93 m^3/s (equivalent to 70% of ‘ED 100 yr’ flow from Catchment D). ■ Water level downstream of Device 6 (location 6) must be ≤ 30.53 mRL (‘ED 100 yr’ scenario)
Device 5	■ Water level in Device 5 basin (location 7) must be ≤ 29.7 mRL (300 mm lower than ground surface on the eastern side of this pond) ■ Combined total outflow from Device 5 (location 7) must be ≤ 3.46 m^3/s (equivalent to 70% of ‘ED 100 yr’ flow from catchments E, H and G) ■ Water level downstream of confluence of Porters Drain and Mangaheka Stream (location 8) must be ≤ 29.37 mRL (‘ED 100 yr’ scenario’)

8.3 Proposed Device Characteristics

Table 9 lists device sizes required to achieve the above targets. Note that basins have been included in the model as having flat invert and 1:4 batter slopes. Basins will need to be refined further during detailed design. Refer to Appendix B for catchment locations.

Table 9: Proposed Device sizes and characteristics

Device	Catchment	Catchment Area	Type	Stored volume	Basin invert	Max water surface elevation	Max depth	Water surface area	Outlet diameter	Outlet invert (u/s end)	Outlet grade
		Ha		m^3	m RL	m RL	m	ha	mm	m RL	
4 Guys	A	7.0	Inline	5000	31.3	32.31	1.0	0.5	700	31.3	1 in 277
Basin 7	C	38.4	Inline	26000*	30.0†	31.7	1.7	3.4	1050	29.4	1 in 62
Basin 6	D	49.4	Offline	36000	28.6	31.2	2.6	1.7	560	28.6	1 in 5
Basin 5	E, G, H	40.7	Offline	44000	28.3	29.7	1.4	3.2	375	28.3	1 in 5

** Maximum volume retained between downstream outlet/embankment and HJV pond in MPD 100 yr with CC and mitigation scenario.*
† Excluding low flow channel

Device 7, which is an inline pond, has been sized against its own design targets (as per Table 8), with the assumption that Devices 5 and 6 do not exist. Building these latter ponds will further reduce peak water level downstream of the Device 7 bund.

Each device should be built as their corresponding catchments are developed (see “Mitigates for development” column in Table 7).

9 Model Results with Mitigation

9.1 100-year ARI

9.1.1 Peak Water Levels

Table 10 lists maximum water levels for the ‘MPD 100-yr ARI with CC and mitigation’ scenario against the ‘ED 100 yr’ scenario. All corresponding mitigation targets given in Table 8 are satisfied, with the exception of the criteria on water level downstream of the confluence between Porters Drain and Mangaheka Stream (location 8). In this case, ‘MPD 100 yr with CC and mitigation’ water level is 10 mm greater than the target of 29.37 mRL. This is considered to be within the modelling margin of error.

Note that:

- ‘MPD 100 yr with CC and mitigation’ water levels at locations 9 to 12 are expected to be higher than equivalent ‘ED 100 yr’ water levels. This is because of increased inflows from catchments 7 to 21 (i.e. rural areas) due to climate change effects, for which mitigation is not proposed or expected.
- Water levels at locations 11 and 12 are influenced by the water level boundary condition set at Waipa River of 16.07 for all 100-year ARI scenarios.
- Locations 9 and 10 are immediately upstream of surcharged culverts, which have a similar effect to the boundary condition affecting locations 11 and 12.

As stated in Table 8, the target water level in the HJV pond (location 3) was 31.7 mRL, as determined with respect to the ED 100 yr with CC scenario. This is because this device has already been designed for the effect of climate change. Therefore the comparison in this table with the ED 100 yr value is of only nominal interest.

Table 10: Maximum water levels for ‘MPD 100 yr with CC and mitigation’ against ‘ED 100 yr’.

Location Label	ED 100 yr	MPD 100 yr with CC and mitigation
	mRL	mRL
1	32.26	32.29*
2	32.15	32.10
3	31.51	31.62
4	30.63	30.62
5	n/a	31.08
6	30.46	30.43
7	n/a	29.57
8	29.34	29.35
9	26.61	26.67
10	22.68	22.85
11	16.34	16.46
12	16.19	16.22

*Note that at location 1, the MPD 100 year with CC and mitigation value is 30mm higher than the ED100 year value. This is due to a minor error that was found whilst finalising this report. The 4 Guys pond will need to be slightly larger to meet the target value at this location, however this is within the bounds of normal modelling errors therefore it was not considered necessary to iterate the model. This does not affect the conclusions of this report. Other more minor increases at Location 3 and 10 and considered to be within normal modelling errors.

The below long sections show the water levels along the stream channels within the model. Three sections are presented, as illustrated in Figure 1:

1. Porters Drain (Figure 2)
2. Upper Mangaheka Stream above the Mangaheka stream/Porters drain confluence (Figure 3)
3. Lower Mangaheka Stream, from the Waipa River confluence to the Mangaheka stream/Porters drain confluence (Figure 4)

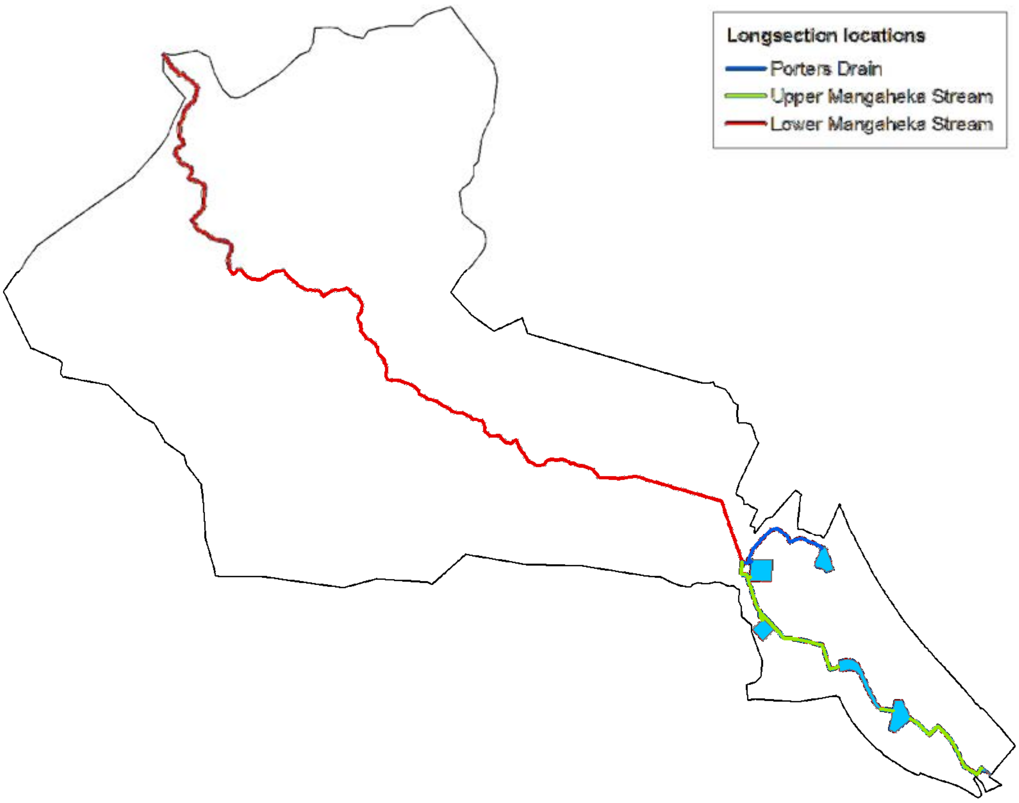


Figure 1: Plan View of Channel Long Sections

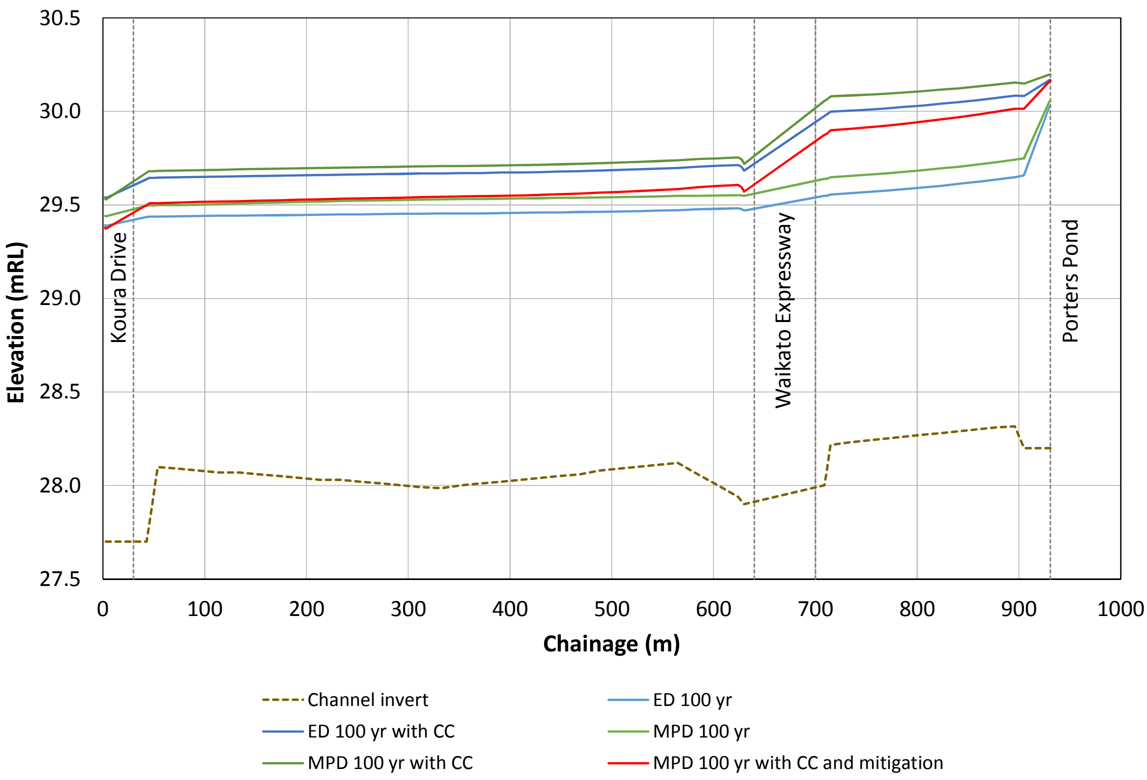


Figure 2: Water Levels - Porters Drain

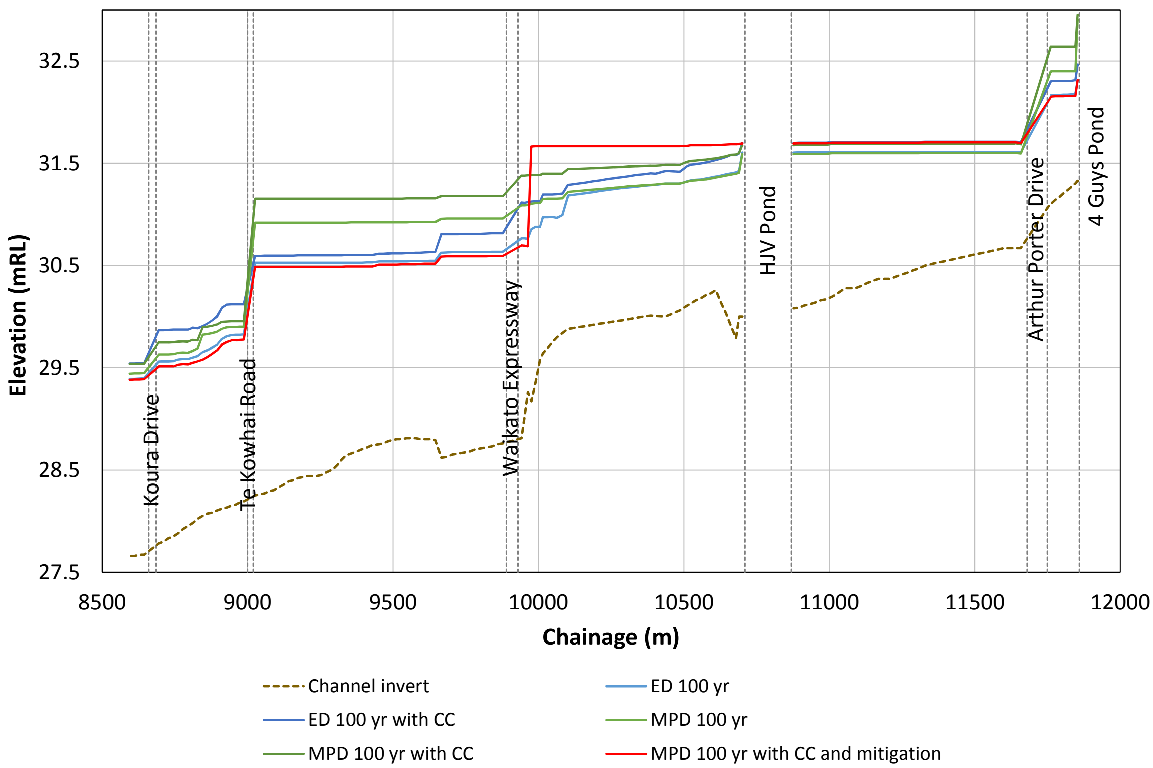


Figure 3: Water Levels - Upper Mangaheka Stream, from Koura Drive to 4 Guys Pond

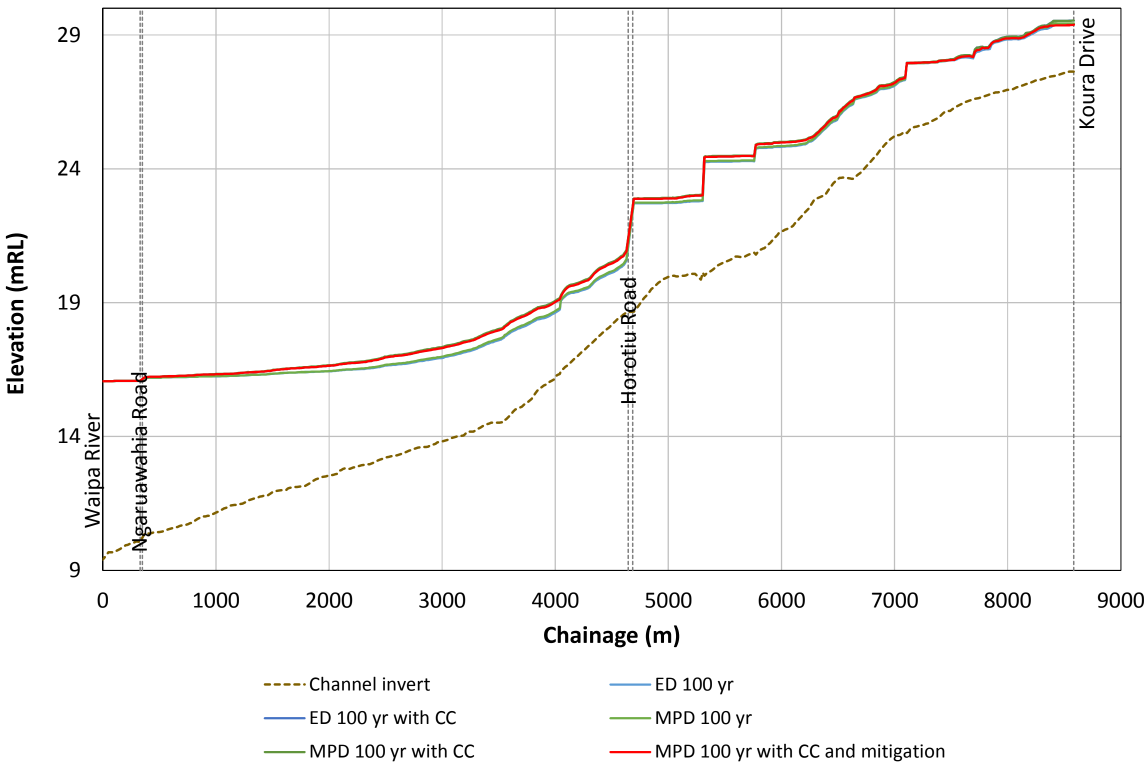


Figure 4: Water levels - Lower Mangaheka Stream, from Waipa River to Koura Drive

The above long sections show that the MPD 100 year with CC and mitigation water levels are at or below the ED 100 year water levels, other than in Porters Drain. In this location, water levels are higher as flows from the future development catchment (G) are mitigated by over-attenuation in Device 5. It is worth nothing that the elevated water levels are within the stream banks and are therefore not considered an issue.

9.1.2 Peak Flows

Table 11 lists maximum flow rates for the 'MPD 100 yr with CC and mitigation' scenario against the 'ED 100 yr' scenario as well as velocities. All corresponding mitigation targets given in Table 8 are satisfied. Whilst peak flow and water level are the main drivers, it is also helpful to understand velocities at each location as this is a key factor in erosion potential.

Table 11: Maximum flow rates and velocities for 'MPD 100 yr with CC and mitigation' against 'ED 100 yr'

Location Label	ED 100 yr		MPD 100 yr with CC and mitigation	
	Flow (m^3/s)	Velocity (m/s)	Flow (m^3/s)	Velocity (m/s)
1	0.26	0.03	0.59	0.06
2	0.55	0.30	0.53	0.30
3	1.45	0.02	3.03	0.05
4	3.70	1.43	2.85	1.03
5	n/a	n/a	0.88	
6	3.48	0.43	3.41	0.47
7	n/a	n/a	0.22	
8	5.84	1.06	5.53	0.96
9	14.77	0.96	17.78	1.09
10	21.05	0.73	32.63	0.72
11	43.68	0.58	58.27	0.74
12	60.65	0.31	78.47	0.40

Note that:

- Whilst the peak flow at location 1 for the 'MPD 100 yr with CC and mitigation' scenario is greater than for either ED scenario, water level criteria at this location are satisfied, as seen in Table 10. Velocities at this location are significantly lower than 0.1 m/s and therefore unlikely to lead to erosion.
- The HJV pond, which has already been built, is expected to mitigate peak outflow rates to pre-development levels. ED scenarios in this current study include this development, and therefore it is appropriate that peak pond outflow rates (location 3) in the 'MPD 100 yr with CC and mitigation' scenario are no greater than in the 'ED 100 yr with CC' scenario. A comparison between Table 9 and Table 4 shows this to be the case.
- 'MPD 100 yr with CC and mitigation' flow rates as well as velocities at locations 9 to 12 are expected to be higher than equivalent 'ED 100 yr' flow rates. This is because of increased inflows from catchments 7 to 21 (i.e. rural areas) due to climate change effects.

9.1.3 Drain down times

In assessing drain down times, we have determined the length of time that water levels have increased above bank levels at each reporting location. Bank levels have been assessed using aerial photography compared against the cross-section level data. Table 12 shows the drain down times for each of the 100 year scenarios.

Table 12: Drain down times

Location Label	Reference elevation	ED 100 yr	ED 100 yr with CC	MPD 100 yr	MPD 100 yr with CC	MPD 100 yr with CC and mitigation
	mRL	hours	hours	hours	hours	hours
2	32.11	1.0	1.7	2.5	3.1	1.6
4	30.5	1.6	5.0	6.1	9.1	5.8
6	29.5	9.6	12.6	12.4	14.1	16.1
8	29.5	0	0	0	0	0
9	25.7	13.6	15.0	14.7	16.9	18.2
10	21.1	7.8	10.9	8.9	12.1	10.2
11	20.5	0	0	0	0	0

Location 12 has been excluded as this is flooded throughout the whole event due to the fixed downstream water level control at the discharge point to the Waipa River.

Table 12 above show that at all locations, the drain down times are less than the required 72 hours.

9.1.4 Attenuation Target Achievement

As described earlier, the attenuation target that governed (flow rate or water level) differed for each reporting location. Table 13: provides the results against the governing target for each location.

Table 13: Attenuation targets at each reporting location.

Location Label	Target	Target basis	MPD 100 yr with CC and mitigation value
1	32.31 m RL	ED 100 yr water level	32.29
2	32.17 m RL	ED 100 yr water level	32.10
3	31.7 m RL	ED 100 yr with CC water level	31.62
4	30.76 m RL	ED 100 yr water level	30.62
4	5.7 m^3/s	70% contribution of 'ED 100 yr' flow from catchment C plus flow coming from upstream	2.85*
5	31.2 m RL	Minimum ground elevation in nominal basin location	31.08
5	0.93 m^3/s	70% of 'ED 100 yr' flow from Catchment D	0.88
6	30.53 m RL	ED 100 yr water level	30.43
7	29.7 m RL	300 mm lower than ground surface on the eastern side of this pond	29.57
7	3.46 m^3/s	70% of 'ED 100 yr' flow from catchments E, H and G	0.22*
8	29.37 m RL	ED 100 yr water level	29.35

* For these reporting locations, reducing the peak flow to at or below the target was not enough to also achieve the water level target, hence the water level target governed.

9.1.5 Attenuation Requirements for Developers

One of the overall objectives of this modelling is to confirm that flood levels are not raised by future development. A common method to do this—and one previously recommended by AECOM (2013)—is to reduce peak flows in order to mitigate water level increases. However due to the flat nature of the catchment (the upper catchment in particular), peak flows do not directly correlate with water levels and therefore it is the water levels that have directly governed the device sizing. This includes the effect that coincidence of flows have on water levels. This has meant that attenuation requirements (in terms of peak flow reduction) are different for each of the devices. Table 14 below outlines the attenuation requirements for each proposed device in terms of peak discharge from the catchment(s) served and also in terms of what flow would be expected downstream of each device, if they are designed and built correctly.

Table 14: Attenuation Requirements

	Catchment Served	Peak discharge from catchment as % of ED	Peak flow downstream of device as % of ED
Device 5	E, H, G	9	101
Device 6	D[1]	70	96
Device 7	C	-27	73
4 Guys	A	76	96

Note that pond sizing has been carried out in the model assuming all development (MPD) and devices are present and working together to achieve appropriate mitigation across the whole catchment. If one sub-catchment was developed in isolation, further modelling would be needed to determine interim mitigation requirements.

In relation to Device 5, this is located in a very flat part of the catchment. On the basis of comparing peak flows generated by this catchment in isolation, the percentage reduction in peak flows is very high. This however should be considered against the results in Table 10 (location 8), which shows that water levels downstream of the device at MPD (with CC and mitigation) match ED (without CC).

Note also that MPD peak flows need to be less than ED peak flows for Device 7 in order that water levels downstream are not higher than ED. This is because Device 7 is an inline pond, and is therefore affected by both upstream (which are higher as a result of climate change) and downstream water levels. A very high level of mitigation is therefore required in terms of managing MPD flows from the local catchment draining to Device 7. When compared to Table 10 (location 4), the water level is slightly lower at MPD (with mitigation and CC) as compared to ED. This is why the % change in Table 12 is negative for this device. If Device 7 were considered in isolation i.e. no other development occurred, it is possible that less mitigation would be required (smaller pond).

9.2 10-year ARI

9.2.1 Peak Water Levels

Table 15 lists maximum water levels for the 'MPD 10 yr with CC and mitigation' scenario against the 'ED 10 yr' scenario. Whilst mitigation targets (listed in Table 8) applied only to 100-year scenarios, equivalent

[1] Whilst only the southern side of catchment D drains into Device 6, this device over-attenuates for runoff from the remaining portion of this catchment. "Peak discharge from catchment as % of ED" for this device refers to total catchment flow; i.e. both sides of catchment D.

comparisons can be made, particularly with respect to water levels in the ‘ED 10 yr’ scenario for locations 1, 2, 4, 6, and 8.

Note that like the 100 year event, water levels in the lower parts of the catchment are affected by un-mitigated climate change in the rural catchments, and by the water level boundary condition set at Waipa River of 14.33 for all 10-year ARI scenarios.

Table 15: Maximum water levels for ‘MPD 10 yr with CC and mitigation’ against ‘ED 10 yr’

Location Label	ED 10 yr	MPD 10 yr with CC and mitigation
	mRL	mRL
1	31.92	32.01
2	31.77	31.84
3	31.15	31.27
4	30.24	30.27
5	n/a	30.35
6	30.04	30.02
7	n/a	29.24
8	29.23	29.19
9	26.22	26.35
10	21.38	21.76
11	15.26	15.44
12	15.15	15.30

Long sections in Figures 6 to 8 show the water levels along the stream channels within the model.

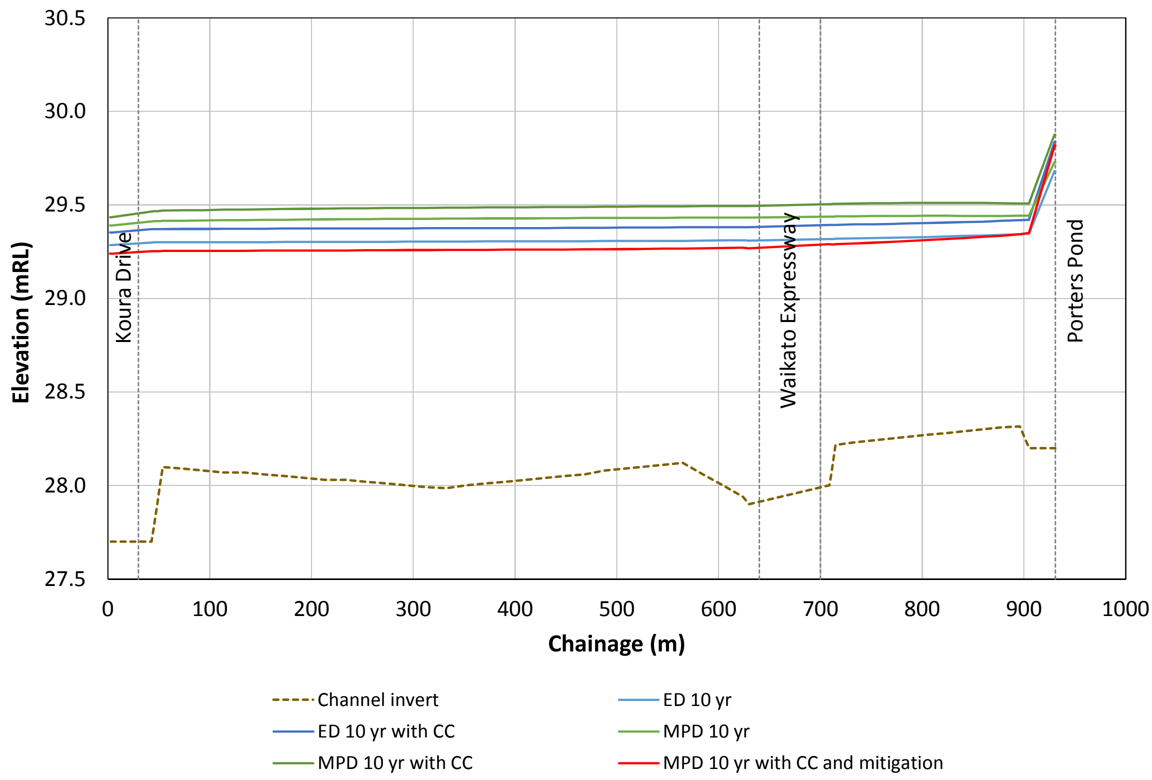


Figure 5: Water Levels - Porters Drain

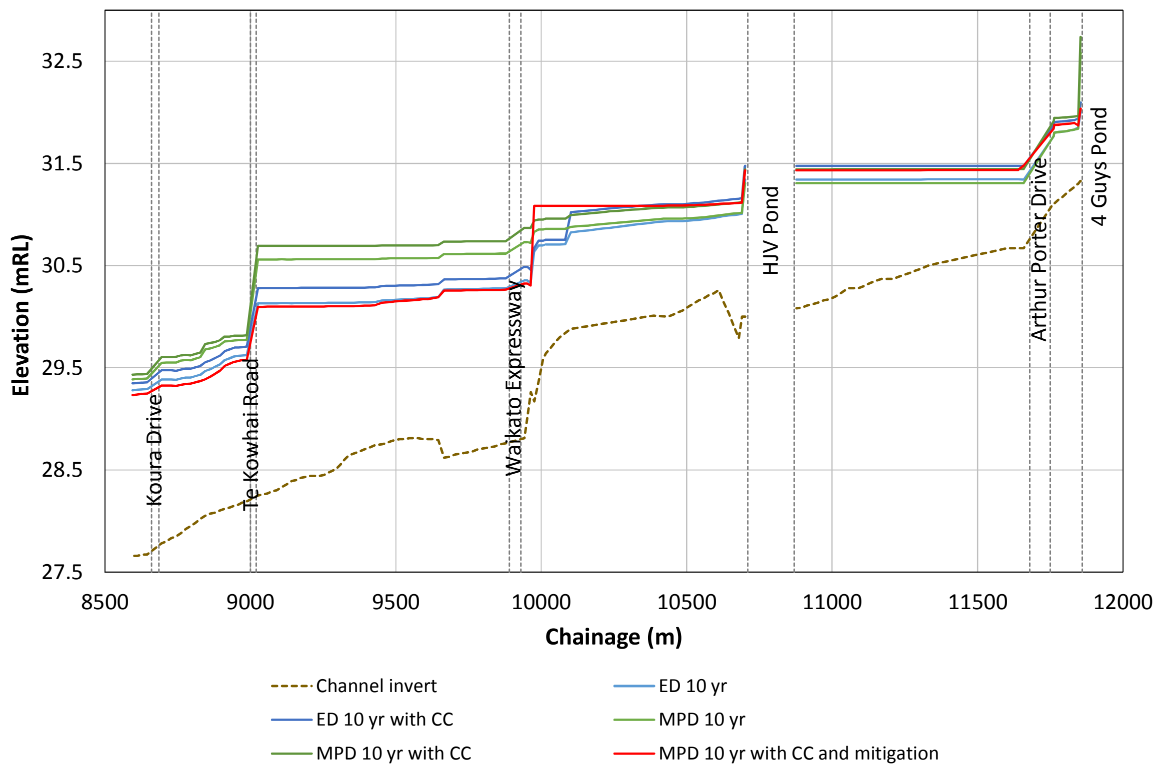


Figure 6: Water Levels - Upper Mangaheka Stream, from Koura Drive to 4 Guys Pond

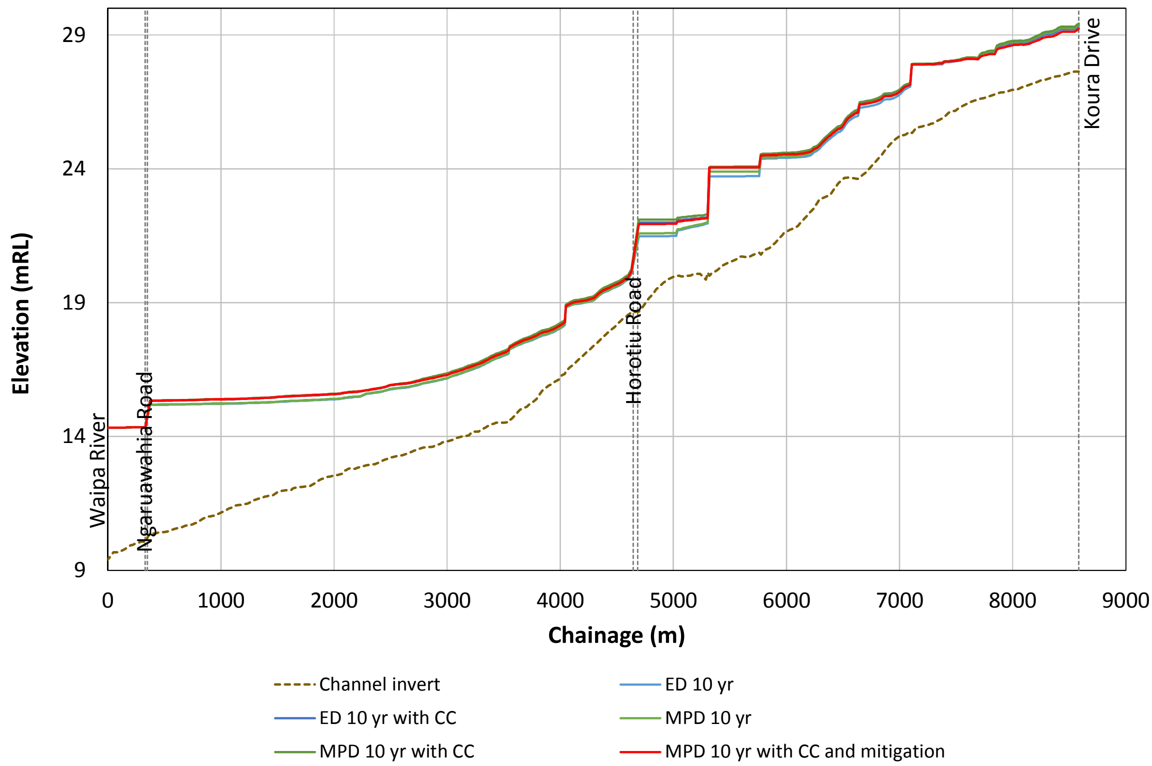


Figure 7: Water levels - Lower Mangaheka Stream, from Waipa River to Koura Drive

The above long sections show that the MPD 10 year with CC and mitigation water levels are at or below the ED 10 year water levels.

9.2.2 Peak Flows

Table 16 lists maximum flow rates for the 'MPD 10 yr with CC and mitigation' scenario against the 'ED 100 yr' scenario.

Note that:

- The HJV pond, which has already been built, is expected to mitigate peak outflow rates to pre-development levels. ED scenarios in this current study include this development, and therefore it is appropriate that peak pond outflow rates (location 3) in the 'MPD 10 yr with CC and mitigation' scenario are no greater than in the 'ED 10 yr with CC' scenario. A comparison between Table 12 and Table 4 shows this to be the case.
- 'MPD 10 yr with CC and mitigation' flow rates at locations 9 to 12 are expected to be higher than equivalent 'ED 10 yr' flow rates. This is because of increased inflows from catchments 7 to 21 (i.e. rural areas) due to climate change effects.

Table 16: Maximum flow rates for 'MPD 10 yr with CC and mitigation' against 'ED 10 yr'

Location Label	ED 10 yr	MPD 10 yr with CC and mitigation
	m³/s	m³/s
1	0.18	0.46
2	0.39	0.45
3	0.70	0.89
4	2.05	2.11
5	n/a	0.56
6	2.53	2.55
7	n/a	0.18
8	3.62	3.27
9	7.62	9.13
10	10.20	11.59
11	21.27	27.09
12	29.28	36.85

9.2.3 Drain down times

Table 17 below shows the drain down times for each reporting location.

Table 17: Drain down times

Location Label	Reference elevation	ED 10 yr	ED 10 yr with CC	MPD 10 yr	MPD 10 yr with CC	MPD 10 yr with CC and mitigation
	mRL	hours	hours	hours	hours	hours
2	32.11	0	0	0	0	0
4	30.5	0	0	1.5	2.9	0
6	29.5	3.8	6.2	6.8	8.4	10.7
8	29.5	0	0	0	0	0
9	25.7	7.7	9.9	9.6	11.9	13.2
10	21.1	3.5	5.4	4.5	6.2	6.2
11	20.5	0	0	0	0	0

Location 12 has been excluded as this is flooded throughout the whole event due to the fixed downstream water level control at the discharge point to the Waipa River.

Table 17 above shows that at all locations, the drain down times are less than the required 72 hours.

10 Flood Maps

10.1 Overview

Flood maps have been produced by interpolating flood extents from the HEC-RAS cross-sections and then overlaying these on an aerial photograph of the catchment. An alternative methodology is to drape the flood extents over a LiDAR surface. This however has not been possible given the lack of recent, accurate, high-resolution LiDAR data. Flood maps for all scenarios are presented as Appendices F (100 year) and G (10 year).

10.2 100-year ARI flood maps

The following observations can be made:

- Flood maps for ED 100-yr CC and MPD non-mitigation scenarios show evidence of ponding in the area where the Device 7 inline basin is proposed. That is, Device 7 would increase ponding levels and spatial extent in an area that is already subject to flooding.
- MPD non-mitigation maps show increased flooding along Mangaheka Stream between Te Rapa Bypass and Te Kowhai Road when compared against ED maps
- Both ED 100-yr CC and MPD 100-yr CC maps show increased flooding extents around the Porters Drain / Mangaheka Stream junction when compared with their non-CC versions. This includes a narrow ‘sliver’ of flooding extending 300 m towards the south-west, where water depths are approx. 25 mm. Cross-section elevations are constant over this extent, which is unlikely to be true in reality.
- The map for MPD 100-yr CC with mitigation depicts flooding extents that are very similar to those seen in the ED 100-yr scenario, with the exception of intentionally-increased ponding at Device 7 and within Porters Drain.

10.3 10-year ARI flood maps

The following observations can be made:

- Non-mitigation flooding extents in the area occupied by the proposed Device 7 are much smaller than seen in equivalent 100-year runs.
- ED models predict flooding immediately downstream of the junction between Mangaheka Stream and Porters Drain, but not upstream of Koura Drive. However this area is flooded in MPD non-mitigation scenarios.
- Flood extents for MPD 10-yr CC with mitigation are very similar to those for ED 10-yr, with the exception of increased extents at a) Device 7, because this is an inline device and b) in an area 700 m downstream of the junction between Mangaheka Stream and Porters Drain. Here, water levels are higher because of effect of climate change on rural catchments which are not mitigated.

11 Conclusions

In terms of the primary modelling objective (**Objective 1**), the modelling carried out has shown that the effect on water levels resulting from MPD can be mitigated by using attenuation basins such that there is no more than minor downstream flooding effect. This mitigation also results in peak flows which are at or below ED water levels (except where increases have been deemed appropriate and acceptable).

In the lower catchment, if climate change occurs, water levels will increase as a result of the predicted increases in rainfall intensity. In this part of the catchment mitigation is not proposed as no development (beyond normal rural development) is proposed.

As discussed in section 9.1.5, water levels have driven the sizing of the attenuation devices. In terms of peak flows and the objective to confirm the conclusions of the Lysaghts/AECOM modelling (**Objective 3**), this modelling indicates that a different target will be required at each of the devices due to the differing constraints on each. Table 18 below provides details of the peak flow reductions required by each.

Table 18: Attenuation Percentages

Device	Catchment Served	Peak discharge from catchment as % of ED	Peak flow downstream of device as % of ED
Device 5	E, H, G	9 (a)	101
Device 6	D	70	96
Device 7	C	-27 (b)	73
4 Guys	A	76	96

In terms of **Objective 4**, to mitigate the increases in water levels associated with development in the upper catchment, attenuation devices will likely be required. These are shown in Figure 8 below (refer also Appendix E). Their details are shown in Table 19.

Table 19: Proposed device sizes and characteristics

Device	Catchment	Catchment Area	Type	Stored volume	Basin invert	Max water surface elevation	Max depth	Water surface area	Outlet diameter	Outlet invert (u/s end)	Outlet grade
		ha		m^3	m RL	m RL	m	ha	mm	m RL	
4 Guys	A	7.0	Inline	5000	31.3	32.31	1.0	0.5	700	31.3	1 in 277
Basin 7	C	38.4	Inline	26000*	30.0†	31.7	1.7	3.4	1050	29.4	1 in 62
Basin 6	D	49.4	Offline	36000	28.6	31.2	2.6	1.7	560	28.6	1 in 5
Basin 5	E, G, H	40.7	Offline	44000	28.3	29.7	1.4	3.2	375	28.3	1 in 5

** Maximum volume retained between downstream outlet/embankment and HJV pond in MPD 100 yr with CC and mitigation scenario*

† Excluding low flow channel

In terms of **Objective 5**, section 9.1.3 and 9.2.3 provide details of drain down times in the 100 year and 10 year events respectively. These sections indicate that the requirement that mitigation does not result in overbank flooding which is longer than 72 hours in duration is met.

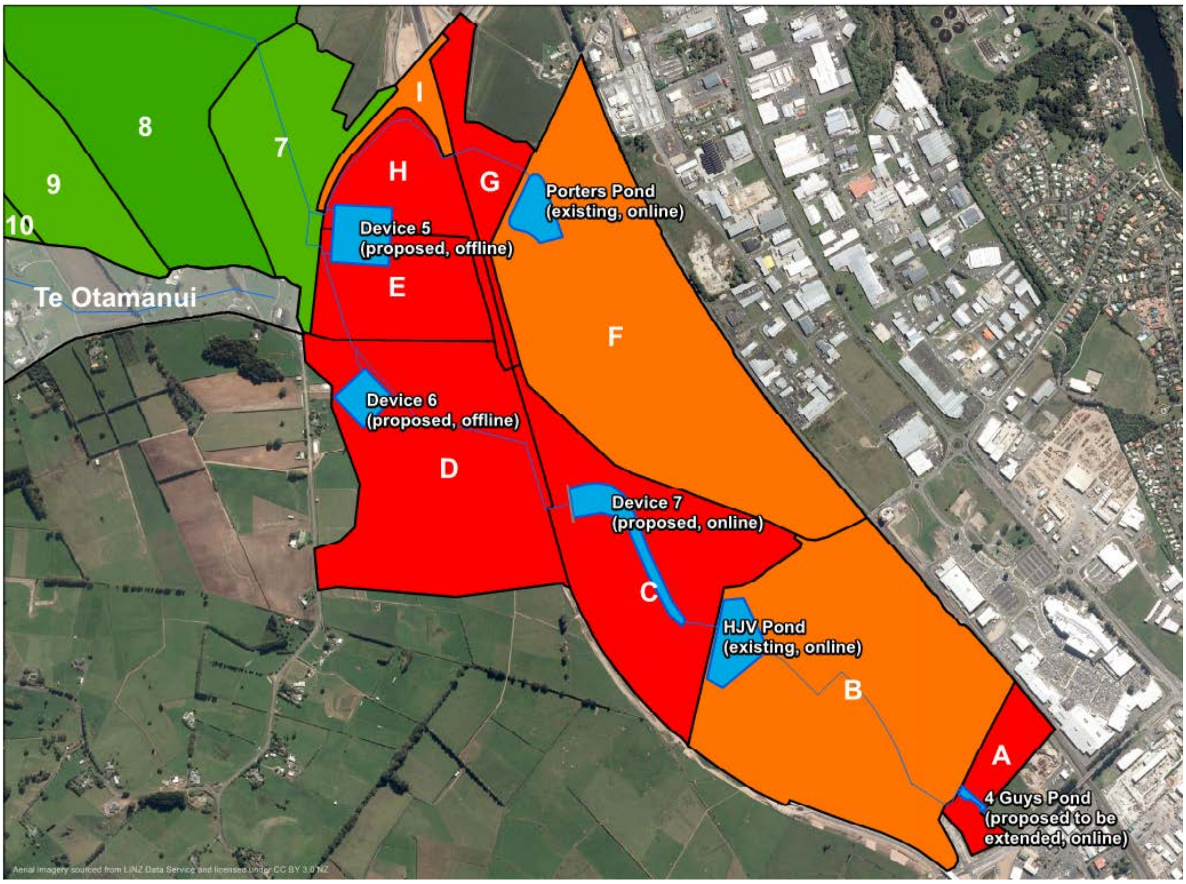


Figure 8: Proposed and Existing Device Locations

12 Further Work

It is possible that discharging stormwater into the Te Otamanui catchment from Mangaheka Stream will reduce or remove the attenuation requirements of MPD. A fatal flaw assessment has been carried out and it has been determined that there are not likely to be any fatal flaws to such discharge. The full assessment can be read in Beca, 2017. Investigations into this are being carried out separately.

13 Variations from HCC Modelling Specification

HEC-RAS has been used for 1-D modelling, instead of the recommended MIKE software. HEC-RAS has been used as it was seen as beneficial to adapt the existing Lysaghts model rather than developing a new model. This approach has been agreed with HCC and Morphum.

14 Peer Review

A peer review of this modelling report has been carried out by Morphum Ltd. Peer review comments and our responses can be found in Appendix I. A number of changes have been made to this report to reflex the peer review.

15 Assumptions and Exclusions

15.1 Assumptions

- Existing attenuation device dimensions, outlets and the channels within the existing developments have been taken from the various supplied modelling reports. Whilst these devices have been inspected on site, and it appeared that these were built as per the plans, the exact details were not measured and confirmed on site. It is therefore assumed that the as-built devices are as per the modelling reports supplied.
- Surveying has been carried out in areas where catchment boundaries were unclear from our site visit and to provide further definition of channel cross-sections. It has been assumed that the cross-section data is representative of the channel in locations between the surveyed cross-sections.
- The vast majority of the existing model cross-sections and elevations from the Lysaghts HEC-RAS model have been retained. It has been assumed that these are accurate and appropriate for this modelling. Some modifications have been made by way of adding cross-sections and adjusting levees and ineffective flow areas during our modelling.
- It has been assumed that the existing culvert and bridge deck levels and dimensions are accurate.
- The downstream boundary condition in the form of a fixed outlet level at the discharge point to the Waipa River has been used. These (100 year and 10 year levels) have been taken from the existing Lysaght model.
- Device initial water levels were set at the Extended Detention level in the 10 year event and empty in the 100 year events. This is similar to the modelling carried out by Aecom.
- A range of more minor assumptions have been made but not included here. These can be provided upon request.

15.2 Exclusions

- We have not determined drain down times for each of the proposed devices in terms of whether die-off of vegetation will occur. This should be assessed at detailed design. It is possible that wetland planting (which can handle extended periods of being wet) may be required if drain down times are longer than approximately 72 hours.
- No formal flood hazard assessment and mapping has been carried out. The attached flood maps are simply a flood extent laid over an aerial photograph. To carry out a flood hazard assessment and mapping exercise, a 2D model would be required.
- Plus exclusions noted in the IFS document dated 11/05/2016.

16 Future Actions

- Update model if LiDAR is flown in the future.

17 Glossary

ARI Average Recurrence Interval, or return period

ED Existing Development

MPD Maximum Probable Development

18 References

AECOM, 2013. Mangaheka Catchment Management Plan. 30 Aug 2013, Ref 60273984.

Beca, 2017. Te Otamanui Fatal Flaw Assessment, CH2M Beca Ltd. May 2017.

Hamilton City Council, 2013. Standard Stormwater Modelling Methodology. 1 May 2013, Ref D-974909.

Lysaght, 2012. Te Rapa North Industrial Development Stormwater Modelling – Discharge Consent. Lysaght Consultants Limited, 23 Nov 2012, Ref 112196.

Appendix A

Catchment Overview Plan

Appendix B

Catchment and Device Plans

Appendix C

Catchment Characteristics

ED Catchment Characteristics

Catchment	Area (km^2)	Weighted SCS Curve Number	Percent Impervious	Time of Concentration
A	0.0325	87.4	63.3%	13
A0	0.0377	69.0	0.0%	41
B	0.7499	85.7	57.7%	87
C	0.3835	73.4	15.0%	61
D	0.4936	71.4	5.0%	61
E	0.1730	71.9	10.0%	48
F	0.6990	89.6	70.9%	67
G	0.0951	80.6	40.0%	34
H	0.1391	70.5	5.0%	65
I	0.0411	89.3	70.0%	25
7	0.2349	70.5	5.0%	98
8	0.9553	71.4	5.0%	92
9	0.7650	72.2	5.0%	90
10	1.7631	72.5	5.0%	80
11	0.6866	76.5	5.0%	64
12	0.2195	71.1	5.0%	99
13	0.6087	75.4	5.0%	74
14	0.5629	75.6	5.0%	61
15	0.4046	70.5	5.0%	142
16	2.5500	70.7	5.0%	136
17	0.5928	71.6	5.0%	66
18	2.6593	76.4	5.0%	160
19	4.0947	70.9	5.0%	278
20	2.0190	77.0	5.0%	90
21	0.0672	70.5	5.0%	53

MPD Catchment Characteristics

Catchment	Area (km^2)	Weighted Curve Number	Percent Impervious	Time of Concentration
A	0.0702	95.1	90.0%	22
B	0.6674	88.6	67.6%	81
C	0.3836	95.4	91.0%	53
D	0.4936	95.5	91.0%	47
E	0.1730	95.4	91.0%	36
F	0.6990	89.6	70.9%	67
G	0.0951	95.4	91.0%	34
H	0.1391	95.4	91.0%	49
I	0.0411	89.3	70.0%	25
7	0.2349	70.5	5.0%	98
8	0.9553	71.4	5.0%	92
9	0.7650	72.2	5.0%	90
10	1.7631	72.5	5.0%	80
11	0.6866	76.5	5.0%	64
12	0.2195	71.1	5.0%	99
13	0.6087	75.4	5.0%	74
14	0.5629	75.6	5.0%	61
15	0.4046	70.5	5.0%	142
16	2.5500	70.7	5.0%	136
17	0.5928	71.6	5.0%	66
18	2.6593	76.4	5.0%	160
19	4.0947	70.9	5.0%	278
20	2.0190	77.0	5.0%	90
21	0.0672	70.5	5.0%	53

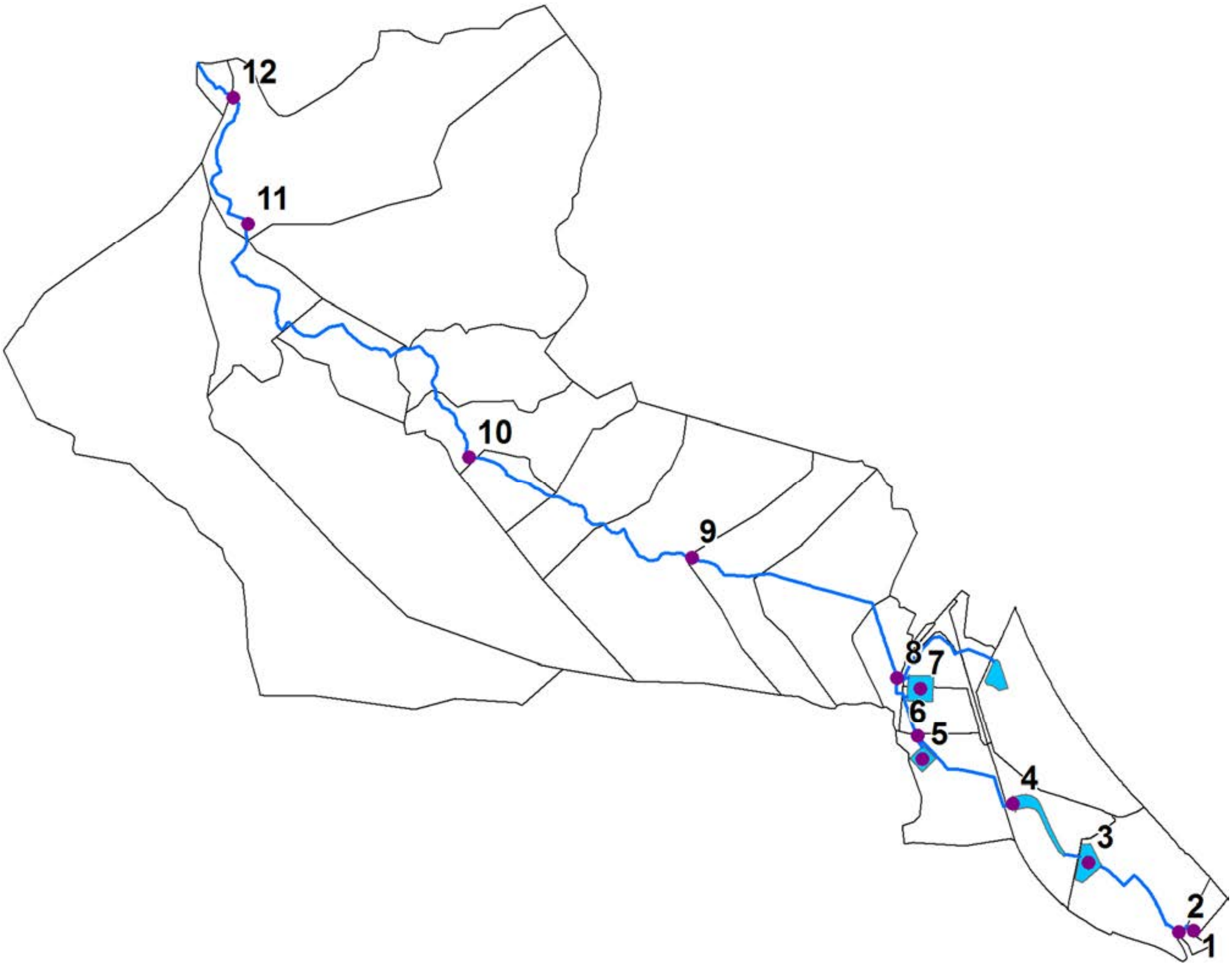
Appendix D

Reporting Locations

Reporting location descriptions

Location Label	Channel	Chainage	Description
1	n/a	n/a	4 Guys Pond (stage or outflow rate)
2	HJV Drain	11764	Immediately upstream of culvert under Arthur Porter Drive
3	n/a	n/a	HJV Pond (stage or outflow rate)
4	Mangaheka Stream	9963.79	Immediately downstream of Device 7 bund, but upstream of culvert under Waikato Expressway
5	n/a	n/a	Device 6 (stage or outflow rate)
6	Mangaheka Stream	9026.23	Immediately downstream of Device 6 outflow but upstream of culvert under Te Kowhai Road
7	n/a	n/a	Device 5 (stage or outflow rate)
8	Mangaheka Stream	8584	Immediately downstream of junction between Mangaheka Stream and Porters Drain
9	Mangaheka Stream	6662.78	Downstream end of catchment 9
10	Mangaheka Stream	4695.33	Immediately upstream of culvert under Horotiu Road
11	Mangaheka Stream	1524.27	Downstream of catchment 18 inflow hydrograph
12	Mangaheka Stream	373.54	Immediately upstream of culvert under Ngaruawahia Road

Reporting Locations



Blown up view of reporting locations

Appendix E

Proposed Mitigation Device Locations

Appendix F

Holdpoint email summarising pond configurations

Appendix G

Flood Maps – 100 year

Appendix H

Flood Maps – 10 year

Appendix I

Peer Review Comments and Responses