Heathcote Phase 3

Report
Project No 44801472-02

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Prepared for Christchurch City Council

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Prepared for: Christchurch City Council
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1 Introduction

This report outlines the Phase 3 scope of work for the Heathcote catchment modelling completed for Christchurch City Council (CCC). This piece of work follows on from the Phase 2 work by DHI in /1/ which included model upgrades to include recent developments within the catchment, general model improvements and model calibration. The Phase 3 scope includes:

- Model updates for the Eastmans Basin to reflect the latest design and as-built data
- Model updates for the Waterloo Business Park
- General model updates and corrections
- Model adjustments to improve stability
- Update of the Upper Heathcote Storage – Active Management representation to reflect the latest functional descriptions
- Update of the CSNDC (comprehensive stormwater network discharge consent) 1991 model
- Run and processing of 74 design simulations

2 Model Build

2.1 Eastmans/Sutherlands/Hoonhay Basins

Figure 2-1: Eastmans Basin Location

The Eastmans area, Figure 2-1, was extensively updated based on both as-built data as well as design and preliminary design data. As such this area represents full future development rather than strictly the year 2022 as the rest of the model is. The mesh, roughness, infiltration, pipe network and open channel network were all updated in this location.

The mesh for the whole basin system has been updated and merged into the original mesh. Detail has been added to individual areas as discussed further below.

The model DEM data sources used for the MIKE 21 surface and in some cases the MIKE 11 cross sections are shown in Figure 2-2. The red areas represent the 0.25m DEM provided by WSP dated 11th May 2022. This covers the majority of the design surface area within the three basins. The surface also includes the Cashmere Stream Diversion and some of the smaller open channels within the Eastmans Basin. The green areas are where manual editing of the DEM was undertaken. The two green areas on the east and west were modified to represent a filled area for a development. These were filled to a level of RL 19.8m and then sloped towards the direction of drainage. The central green area, a level for the basin, and the fill on the south east side were provided via email, as outlines. The southern green area is a modification to the Hoon Hay West Basin design, where the area is lowered to match the area of the basin directly north of it. The remainder of the DEM area uses the 2020 LiDAR surface.

The land-use was updated taking into account that developed areas are residential with 50% imperviousness, i.e. halving the infiltration rates. The groundwater depth was updated based on the updated ground level data. The bed resistance was updated to account for new roads, and a combined resistance factor was used for the undefined developments. The 1-D roughness in the Cashmere Stream Diversion was lowered slightly (by 0.01) from the calibrated roughness, from the assessment by WSP in /2/. Otherwise the 1-D roughness values have been kept as per the calibration.

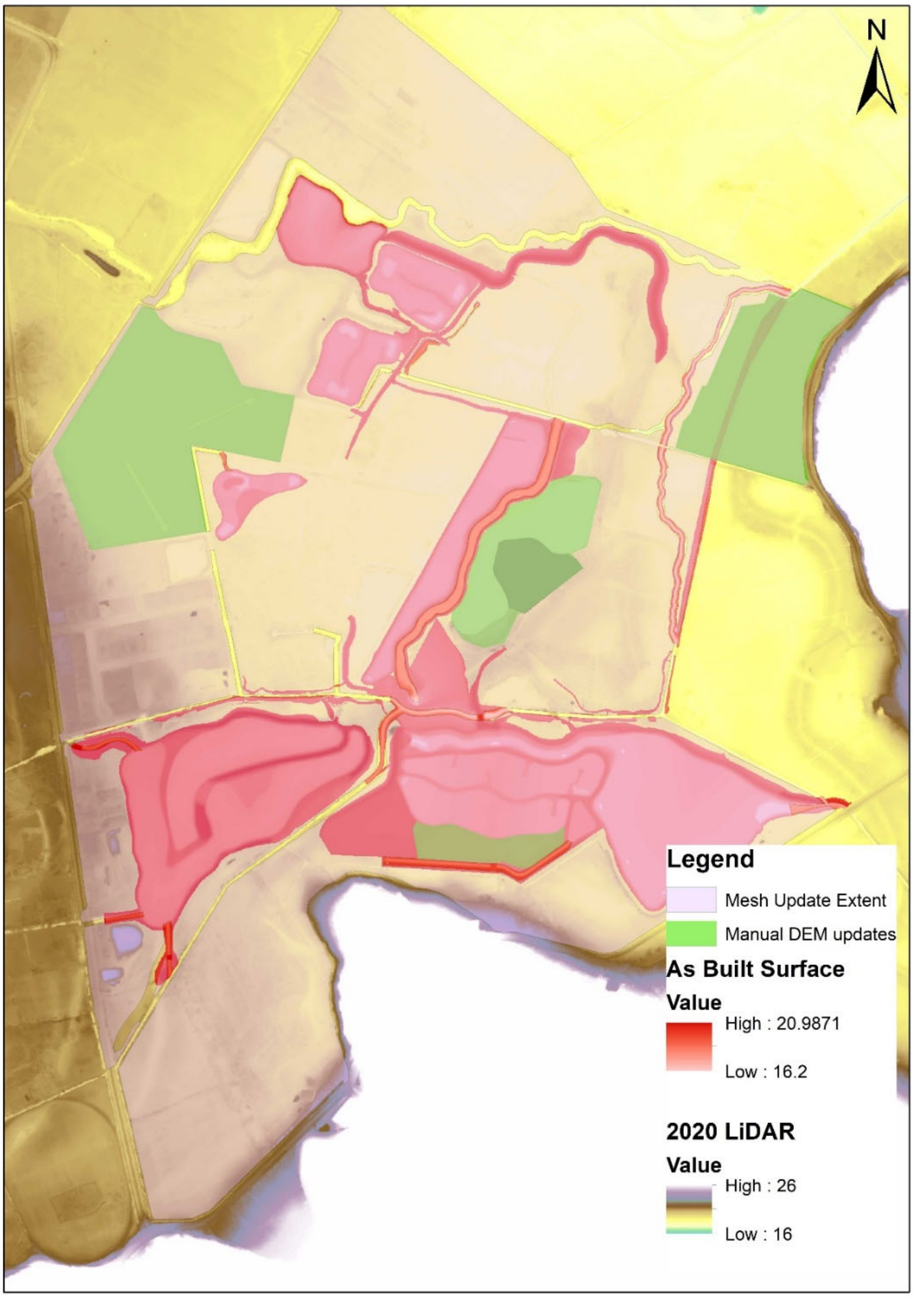

Figure 2-2: Model DEM data used

Figure 2-3 outlines the areas updated for the Eastmans/Sutherlands/Hoonhay basin area. It represents a combination of built infrastructure and some that is still under construction or design.

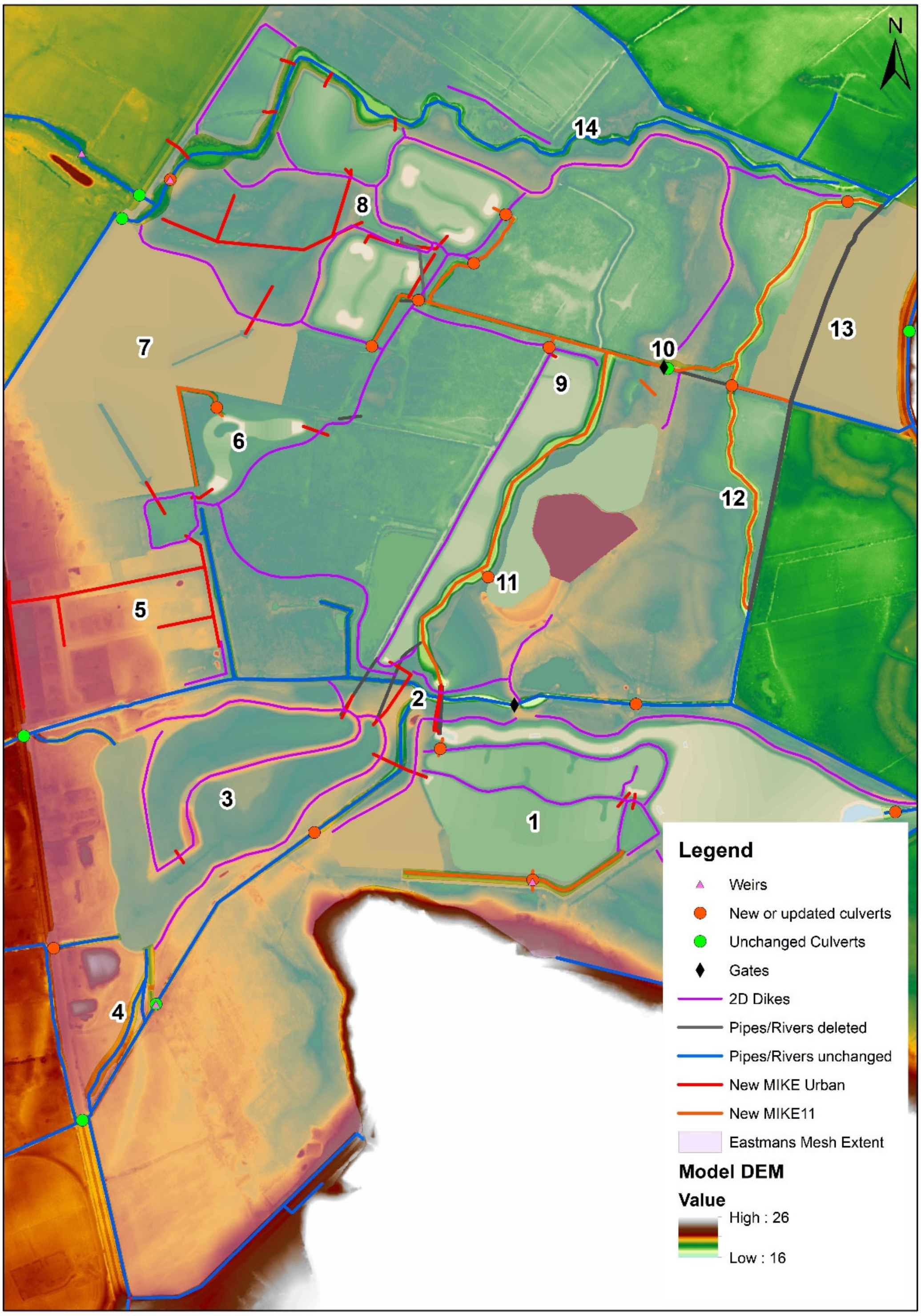

Figure 2-3: Eastman/Sutherlands/Hoonhay Basin Layout

The areas of update as marked in Figure 2-3 are detailed as follows:

2.1.1 Hoon Hay West Basin

Hoon Hay West Basin, the basin configuration has changed from previous iterations. Inter-basin structures were added as per the design drawings. Where these have scruffy dome inlets these have been modelled by setting the MIKE 21 level to the lip of the dome, and the MU-M21 inlet links to weir type. An inlet stream was added at the southern side of the basin, which also includes a spillway weir at right angles to the channel. Dikes have been added to represent all high edges of the basins, where these would not be picked up well in the 2-D mesh.

2.1.2 Inter-basin Syphons

Inter-basin syphons were shifted from the MIKE 11 model into the MIKE Urban model. This allows for improved calculation of losses along the pipelines. Inlet and outlet locations were shifted as needed. The west most syphon from Sutherlands Basin was only partially completed, and technically drains into the soil beneath Cashmere Stream. In the model we this syphon directly connected to the stream, with the invert levels adjusted to allow this to happen. For the syphons between Hoon Hay West and Eastmans the ground levels of the internal manholes have been raised to improve model stability. It is not expected that this will have any significant impact on the results.

2.1.3 Sutherlands Basin

Sutherlands Basin layout has been adjusted. For the model this required adjustment of the dike alignment and levels, and the addition of a new culvert structure between the first flush and attenuation basin.

2.1.4 Sutherlands Inlets

The Sutherlands Basin Inlet from the Quarry Rd Drain has been redesigned. The model is currently modelling both inlet designs however. This is because the new inlet is intended to drain a subdivision that does not yet exist. Thus this has just been added to the model as a placeholder at this stage. A culvert was also added to the south west inlet to Sutherlands Basin. This culvert was added from asset data, as it was noted as missing in previous model versions.

2.1.5 Halswell Downs Development

A new development was added in this location to the model, using pipe and node data from the latest CCC asset dataset. Note that inlets were missing, so the inlet locations were derived from the location of the sump leads, and verified using the aerial photography. The network drains into the Halswell Downs basins. A low bund was added at the south-east corner of the development. Because this area was not included in the 2018 LiDAR dataset, the roads were not included in the GHD road adjusted surface. Thus the standard LiDAR levels for the road have been used here instead. These levels will need to be updated when the latest road adjusted surface is generated.

2.1.6 Halswell Downs Basins

The Halswell Downs basin system was added to the model, using the as-built DEM data provided and the connecting pipe network as per the drawing files provided. The system drains the developments at 5 and 7. The first flush basin has an overflow to the east which has been included as a dike structure at RL

19.4m. The outlet of the first flush goes directly into the main basin. Note that there was a blocked off pipe to the south that could divert water, but this has been removed from the model as it will not be used in the current design. The main basin has a small open channel and an outlet structure, which outflows into the main Eastmans Basin. The main Eastman Basin in this area is bunded off with a bund at RL 19.75m.

2.1.7 New North West Development

This area represents a development that is not yet constructed. Dummy outlet pipes have been included to drain the area into either the Halswell Downs Basins, or the Milns Basins. The pipes have been set with a 600mm diameter.

2.1.8 Milns Basins

The Milns Basins were updated to add the two basins on the east side and the connecting pipe network. The eastern basins drain into the main Eastmans Basin via the open channel, with the detail provided in the DEM data. The internal pipe network has a blocked off pipe going to the south east, this has been removed completely to prevent stability issues.

2.1.9 Eastmans Storage

The Eastman Storage, which runs alongside the Eastman Low Flow Channel has had an outlet structure added to the north side, which feeds into the open channels and towards the Eastmans Control Gate.

2.1.10 Eastmans Control Gate

The Eastman Control Gate logic has been updated to represent the latest functional descriptions. Note a dummy branch is included in this area for storing variable data, and should generally be ignored when looking at the results. The outlet channel from the control structure has been adjusted to align with the realigned Cashmere Stream.

Note the Cashmere gate control logic has also been updated but no other updates were made in this area of the gate.

2.1.11 East Eastmans Storage

On the right bank of the Eastmans low flow channel another storage area has been added, along with a filled area. The data for these was less complete and the culvert outlet structure has just been assumed as a 300mm diameter, as agreed with CCC.

2.1.12 Cashmere Stream diversion

Cashmere Stream was diverted in this location. Cross sections for the 1D channel were extracted from the DEM provided, and structures added as per design drawings and information provided. The old Cashmere Stream alignment has been filled in, in the DEM provided.

2.1.13 New Eastern Development

The Eastern Development also did not have design or as-built data. It has been assumed as fill and sloped towards the Cashmere Stream.

2.1.14 Milns Diversion Stream

The cross sections along the Milns Diversion Drain have been updated further to better match the as-built design surface. Bunds were added in the locations specified by CCC to divert water from the northern portion of the basin area.

2.1.15 General updates

A number of additional culvert structures were added to the model with dimensions taken from design drawings. These are indicated as new structures in Figure 2-3.

2.2 Waterloo Business Park

Waterloo Business Park, Figure 2-4, was updated in the north-west area of the model. In order to include the full development the mesh was extended to the south-west as shown in Figure 2-5. Note that only the area between Islington Ave and State Highway 1, indicated by number 2 in the plot, overlaps with the Halswell model. Pipes were added as indicated in red, including any associated inlets. Ponds were included at areas 1, 3 and 4, using detail from as-built drawings provided by CCC. The DEM was updated to use the 2022 LiDAR surface except for the roads which used the 2018 GHD road modified surface. The basin infiltration rates are as shown in Figure 2-4 where the higher rates represent rapid soakage basins. These rates were derived from the basin operation manuals.

Figure 2-4: Waterloo Business Park Location

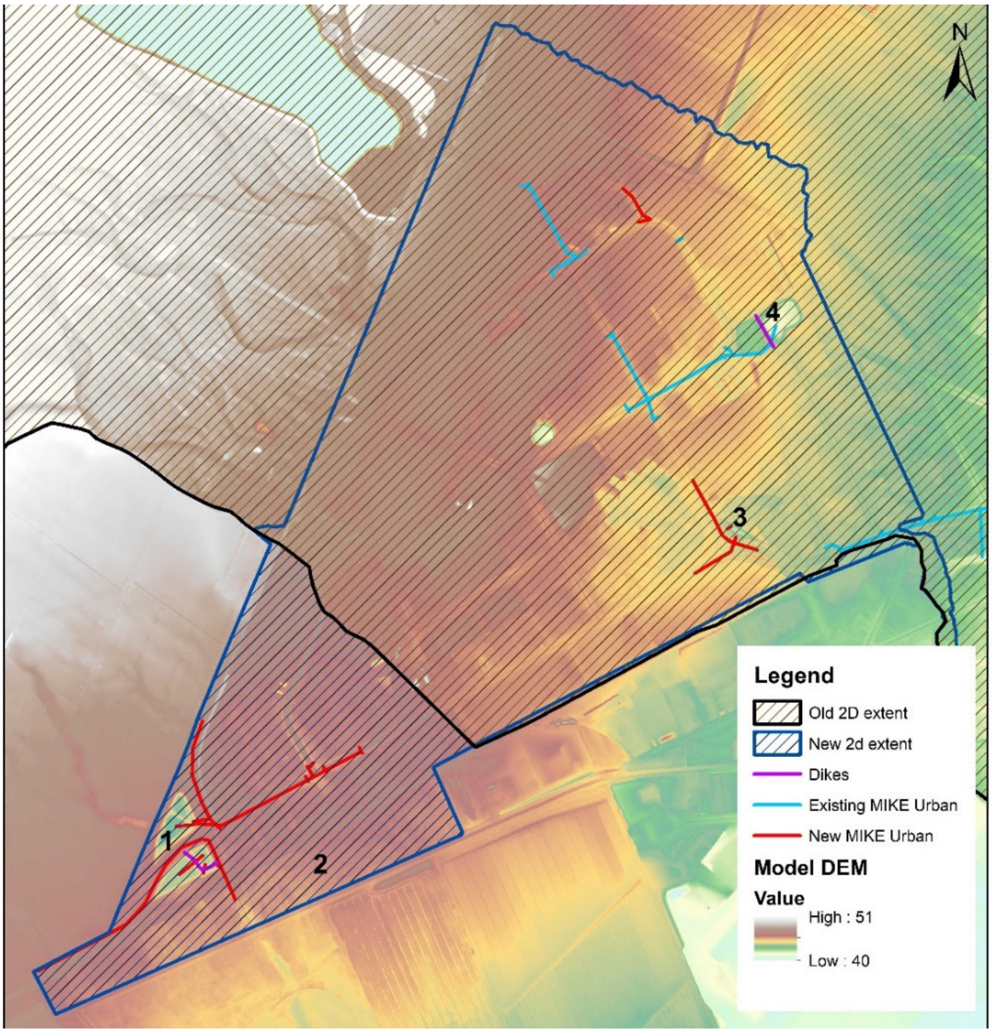

Figure 2-5: Waterloo Business Park

2.3 Cashmere Park

The Cashmere Park development, Figure 2-6, was added to the model using the latest pipe asset data from CCC and the 2020 LiDAR surface. Inlet data was missing from the asset data, but inlet names were inferred from the to and from node references in the pipe shapefile. Where these were missing nodes were inserted. The ponds at 1 were included based on the LiDAR surface, and the asset data, as no design drawings were made available for this site.

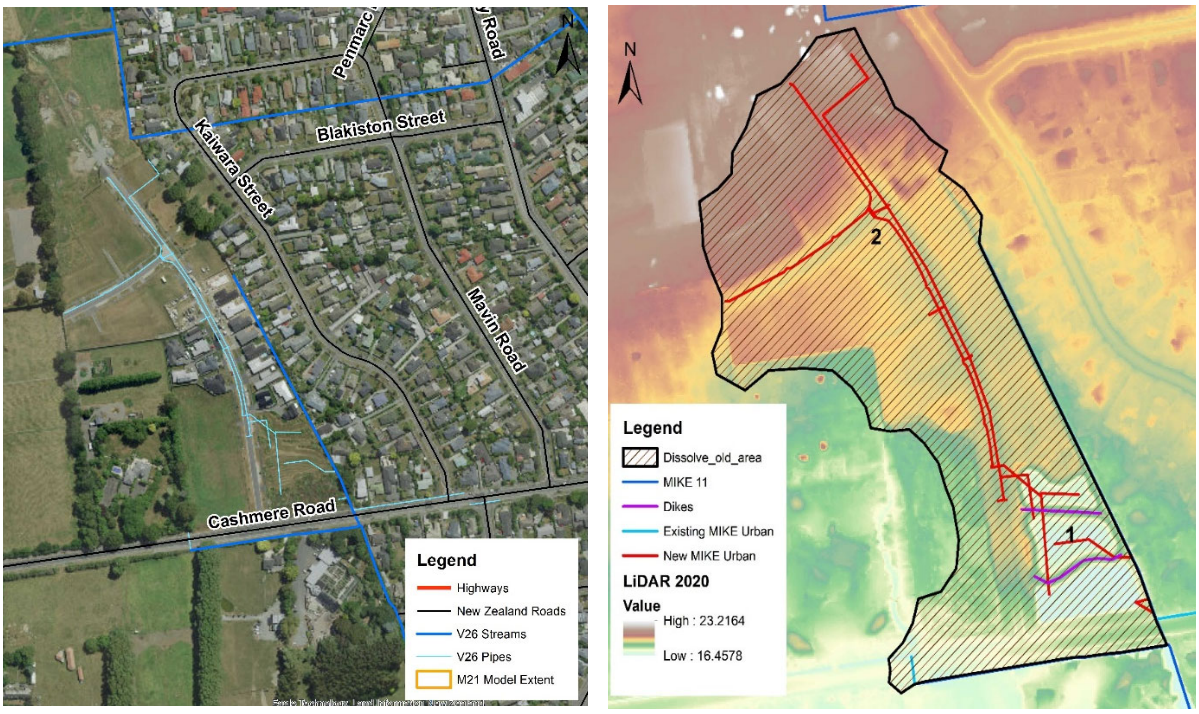

Figure 2-6: Cashmere Park Development

2.4 Miscellaneous Model updates

Mesh levels were updated at the Richardson Pump Station, this was a small change which raised the levels slightly in this area.

The area to the east of Maltworks was identified in a model review that the detail was not sufficient to resolve an overland flow path alongside the railway. This area was updated in the mesh to increase the resolution.

2.4.1 Filling in roads

A general issue with the City Wide models was identified where the mesh blockout for the MIKE 11 channels would continue along road crossings. This was setup as per the City Wide model schematisation. However, in cases where an overland flow path occurs along the road corridor, the river blockout will block this flow path creating ponding as shown in Figure 2-7. For the Heathcote model, the v16 200 year and 10 year max of max results were analysed to identify areas where water appeared to be ponding beside these mesh blockouts alongside the road. This analysis was done visually by systematically following the open channels and checking all of the road crossings against the 2D results. A total of 21 incidences of these blockages were identified using this method.

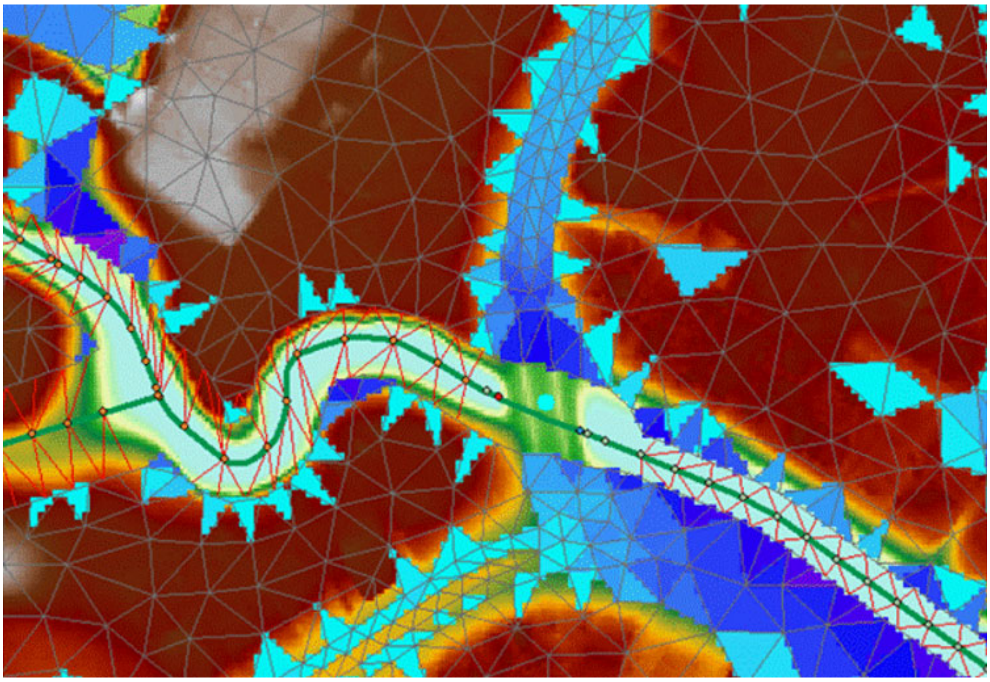

Figure 2-7: Old results, showing water ponding against river blockout at the road

Where the blockages were identified the mesh was manually modified to insert mesh elements where the road crosses the open channel. This insertion was done by using the mesh merge tool (MIKE SDK tool). Where possible the road gutter and centrelines were also updated in the connecting elements to produce a more continuous road pathway. In some areas the definition around the roads had been simplified due to the proximity of the open channel, with the mesh giving the open channel elements more priority. The new areas of mesh were updated using the 2018 DEM with road levels updated – as provided by GHD.

Figure 2-8 show some examples of how the mesh was updated. As can be seen from the figures the mesh alignment is changed not just within the blockout but also for the nearby elements. Blue shows the updated mesh, while the red wireframe shows the original mesh alignment.

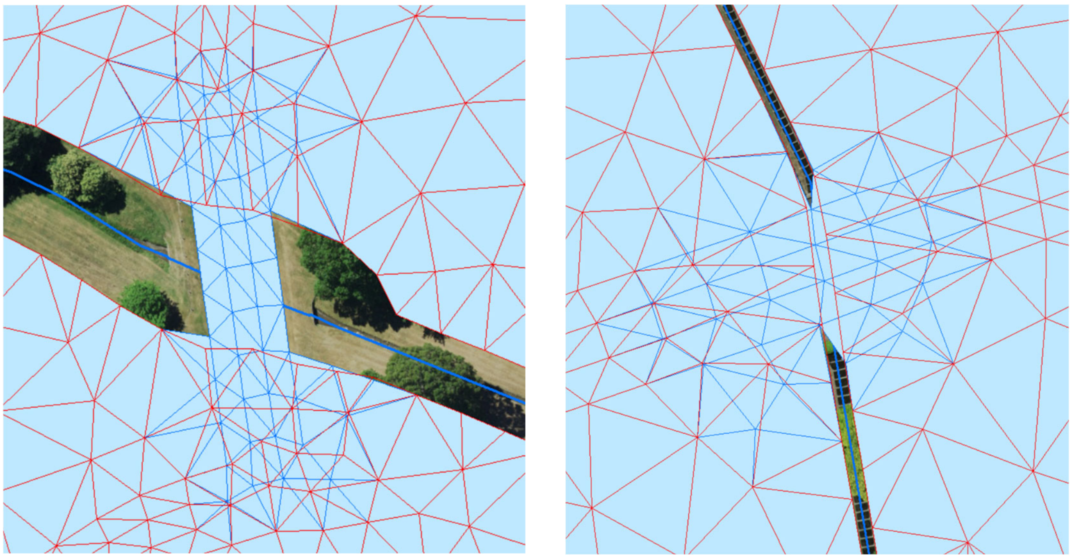

Figure 2-8: Mesh triangle updates around roads

Additional checks on the model setup were needed to ensure it would run correctly with these mesh changes, these were:

- Check that the lateral links were still on the mesh side of the MIKE 11 blockout. In some cases, the mesh edge was moved slightly in the process of re-meshing.
- Splitting lateral links where the new internal mesh is included. In several instances the links were not already split at the road crossing.
- Check that the 2018 road levels DEM was not picking up the river levels where the road crossed the river. In all of these cases the roads appeared to be picked up correctly in the DEM.

The results from the updated model show an improvement of the flow conditions along the road pathways, Figure 2-9 and Figure 2-10. These compare the v22 model (Phase 2 model) with the v26 model (latest version). The only difference in this area between the two models is the road blockout fix. In the pictured case the water level has decreased on the north side (upstream) by 1m for the left point and 250mm for the right point.

Figure 2-9: V22 results around Paparua Stream

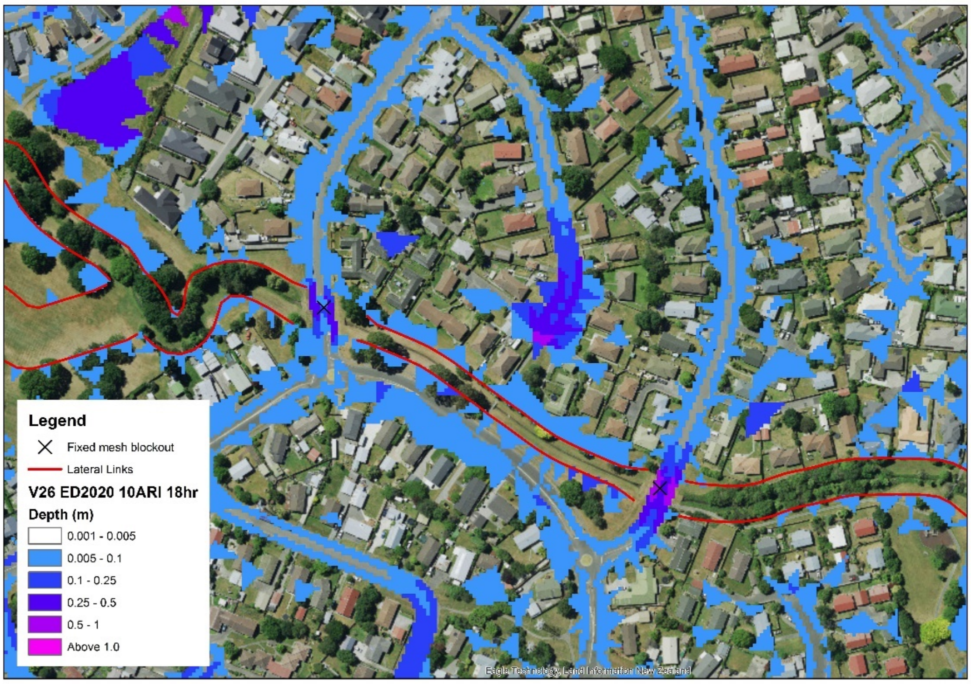

Figure 2-10: V26 results around Paparua Stream

2.4.2 Adjusting link spill levels

Some areas have been identified by CCC staff where water is ponding against the lateral links. The water is ponding in these locations because the link bank level that is being used is too high and not representative of the actual bank level. This is occurring in some links which have the HGH link level setting. This setting chooses the highest of the MIKE 11 or the MIKE 21 bank level. In areas where the MIKE 21 bank level is changing rapidly, like on a steep slope, the MIKE 11 level may not be representative along the full reach of the river.

The two main areas for adjusting the links from HGH to MIKE 21 were at:

- Ensors Road, along Jacksons Creek. An issue was identified around 20 Ensors Road where water appeared to be ponding at the banks. The bank levels were changed to use MIKE 21, this had a local impact of lowering the west bank water level by around 500mm in the 10-year 18 hour design rainfall event.
- Landsdowne Terrace where the MIKE 21 water levels were appearing perched above the MIKE 11 levels at the links. In this area the links were changed to HGH, which resulted in, lowering of the 2-D water levels along the banks (by around 200mm in the 10-year 18 hour design rainfall event), and more similar levels in MIKE 11 and MIKE 21.

Because the HGH issue is made worse in the steeper areas of the model all river branches on the hill catchments were changed to use MIKE 21 levels. This included Sibleys, Scotts, Victory and Popes Drains. The MFlateral file was also checked as part of this change to ensure that using the MIKE 21 bank levels will not cause the bank level to go below the MIKE 11 cross section level. This issue can occur in steep branches where the cross sections are not well defined. In this case however this issue did not occur, so no further changes were necessary.

2.4.3 Flapgates

It was found that a number of flapgates were missing from the model setup in the downstream area of the model. The CCC asset data and swValve shapefile, was used to check where these may be missing. Where there was a missing flapgate this has been added into the model as a non-return valve, these were added to either the MIKE urban or MIKE 11 models as appropriate. Figure 2-11 shows in red the flapgates that were added to the model, and in purple those that already existed. No flapgates were removed.

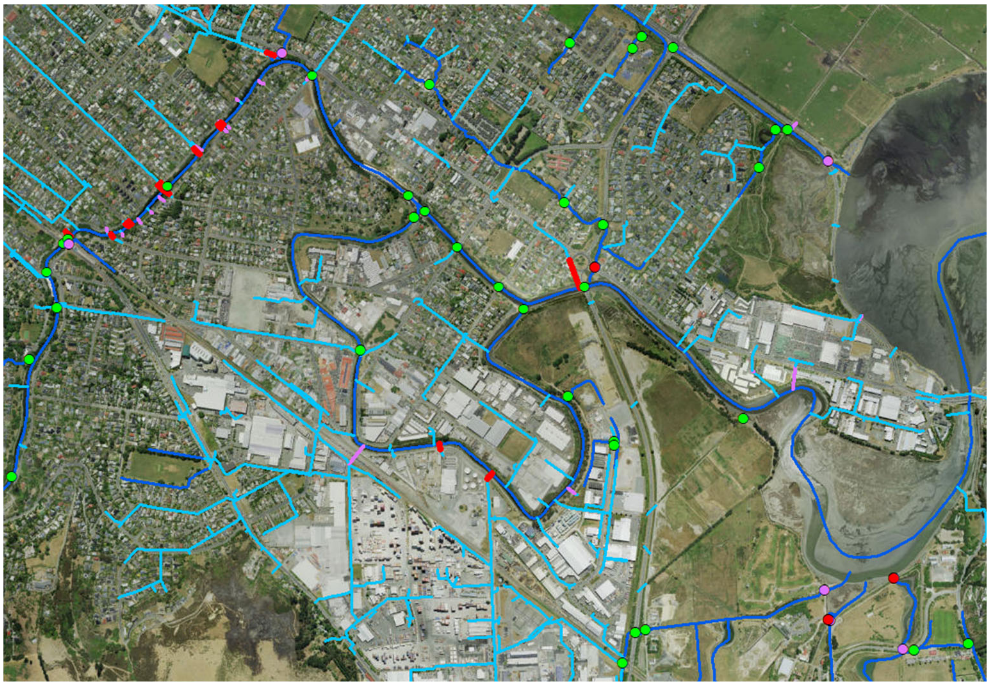

Figure 2-11: Location of flapgates in downstream area

2.4.4 Added dikes to tidal area

In the 1m sea level rise scenario the tidal area was checked to see if any bunding was being overtopped, that was not already well represented in the mesh, or with a dike structure. Two areas were identified where the high points were not being well represented. At these locations dike structures were added to the 2D model. In the existing climate models, the tide level was low enough that the peak 2Dwater level did not overtop the 2D banks, so the inclusion of dikes was not necessary. However, once the sea level rise is applied, these areas are overtopped and a more accurate representation of the spill levels was necessary. Figure 2-12 shows the areas where the two dikes have been added.

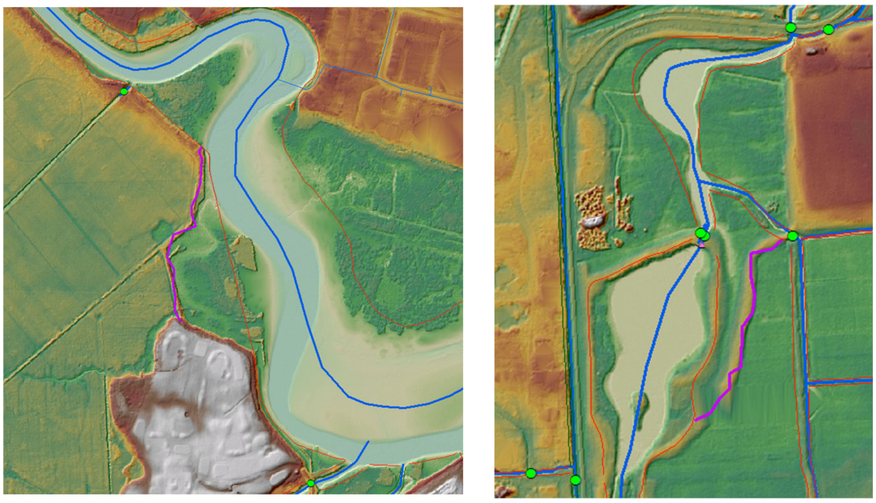

Figure 2-12: Dikes added to Saltmarsh (left), and Matuku ponds (right)

2.4.5 Underpass

An underpass, near Mowbray and Thackery Street intersections, was identified as missing from the model setup. This underpass was added to the MIKE 11 model as a culvert structure, with cross sections extracted from LiDAR. The dimensions of the underpass were estimated by CCC based on site observations. The underpass was estimated at 2.4 m high x 2 .0 m wide. The location of the underpass is located in the SwStation asset dataset.

A small number of abandoned pipes in the vicinity of the underpass were removed, these were overlooked in an earlier update of the pipe network. The removed pipes are shown in Figure 2-13. Note that where the underpass is located a 300mm diameter pipe was removed.

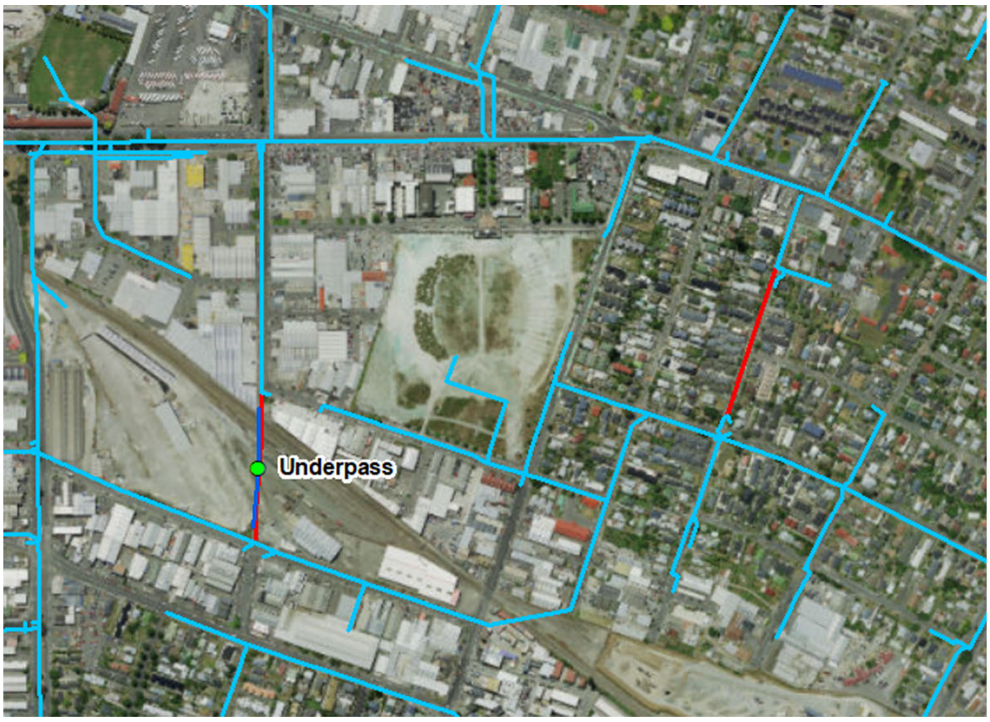

Figure 2-13: Underpass and removed pipes (red)

2.4.6 Bridge in Paparua Stream

A bridge was added to Paparua Stream, where the Runway crosses the stream. The bridge data was provided as an as-built pdf. The bridge was modelled as a culvert, 3.73m wide by 1.87m high, with a soffit level of RL 27.93m.

3 Active Management

The active management extends across four main basin areas in the Upper Heathcote catchment. These basins have control gates which activate based on water levels at sensors either within the basins or at specific locations in the catchment. The basins where active management is used are:

- Wigram Basin
- Curletts Basin
- Cashmere Dam (upper) and Cashmere Worsleys Basin (lower)
- Eastman and Hoonhay basins

The active management gate logic has been setup to reflect the latest Functional Description document /3/ (R2 was produced by Jacobs for CCC in May 2022) which represents the logic from the full catchment perspective. Local basin functional descriptions, were also considered, which resulted in some variations to the full catchment logic. These differences mainly relate to using local triggers. In some cases the logic does still differ slightly from the Functional Descriptions, and/or the logic of the Functional Descriptions is not adequately formulated to achieve the desired outcomes. It is expected that further work may follow to optimise the logic, however the current logic used does achieve the purpose of the control system, i.e. reducing overflows, reducing discharges during the peak of the storm and allowing the ponds to return to their normal operating levels.

The basin logic is generally split into three modes. **Storage Activation**, where the basin starts storage mode. This is usually triggered when Buxton Terrace water levels are high. **Storage Management**, this mode occurs directly after the storage activation with the purpose of preventing the local basin spillway overflow if possible, while holding back as much water as it can. **Storage Recovery**, this mode activates when the downstream flood levels have dropped back to an acceptable level and the basins can start to empty at a faster rate. Generally, storage recovery should not be delayed too long or vegetation in the basins can die off.

For each basin the control logic has been outlined, and example result shown for the ED 2020 50 year 12hr design rainfall flood event. This simulation was run for 6 days to allow for observations of the storage recovery, but in some cases it may occur after this time. It is expected that further optimisation of the Active Management may be undertaken at a later date however, the current model results will reflect a conservative result.

3.1 Wigram Basin

Wigram Basin operation is based on the sensors listed in Table 3-1. The system operates two gates, Gate 1 from the wetpond, and Gate 3 from the wetland.

Table 3-1: Wigram Sensors

Sensors	Chainage	Trigger Level (RL m)
Buxton Terrace	Heth.Hethcote 14904.1	11.6
Lincoln WL	Heth.Hethcote 6207.81	21.3
Downstream of Wigram Outlet - Heathcote	Heth.Hethcote 4875	22.5 (22.3 for recovery)
Wigram Wetpond	Heth.WigPnd_Outlet 0	25.5 (25.6 for emergency)

3.1.1 Wigram Pond Outlet (Gate 1)

Gate 1 activates on either the water level downstream of the outlet, on the Heathcote River, based on the local functional description, or on the trigger level at Lincoln Road. To reduce gate hunting and to allow the storage recovery to activate earlier the local reset level was not used, and once the Lincoln level is above RL 21.3m the local trigger will not be used. This assumption may need to be reviewed in the future. Storage management and emergency control are combined using the Table A which allows for 50% flow up to the emergency spill level and then 100% flow after this. Storage recovery initiates once water levels in the catchment have lowered, opening the gate and scaling flow via the scale factor.

Storage management + Emergency control

1. If the Wigram pond WL > 25.5 then apply 50% scaling to the outlet Qh relationship – Table A.

Note when Table A reaches 25.6m, the Emergency control activates, which opens the gate fully.

Storage activation

2. If Heathcote - Downstream of Wigram gate >22.5 & Lincoln trigger = 0 then close the gate - stops this priority from working after the flood activation, so it won't occur during recovery
3. If Lincoln > 21.3 then close the gate

Storage Recovery

4. If Lincoln <21.3 and Wigram WL < 25.5 AND Buxton <11.6 and then Multiply the Wigram Scale Factor by the Qh relationship for this outlet, Table B.
5. Else Unchanged

Table 3-2: Table A and B, QH relationships for Gate 1

	Flow (m3/s)	
Level	Table A	Table B
24.5	0	0
24.55	0.05	0.1
24.6	0.1	0.2
24.65	0.15	0.3
24.7	0.25	0.5
24.75	0.35	0.7
24.8	0.45	0.9
24.85	0.55	1.1
24.9	0.65	1.3
24.95	0.75	1.5
25	0.9	1.8
25.02	0.9	1.8
25.17	0.935	1.87
25.24	0.95	1.9
25.48	1	2
25.59	1.025	
25.6	2.05	
25.7	2.09	2.09
35	2.09	2.09

Wigram Dummy gate - Scale Factor – This gate controls a scale factor which is used for the main gate logic. Gate level values 0-1 represent the scaling factor.

1. Unchanged – only initiate a change in the gate level every 15 minutes
2. If Lincoln WL > 21.25, reduce scale factor by 5%
3. If Lincoln WL < 21.2, increase scale factor by 5%
4. Else Unchanged

Lincoln Trigger

Initial value = 0

1. If Lincoln WL > 21.3 then value = 1
2. Else value = 1

This trigger will only activate once, the first time Lincoln rises above 21.3m.

Discussion

The gate operation is illustrated in Figure 3-1, for the 50 year 12 hour design rainfall event. As Lincoln rises above the trigger level, the storage activation is triggered and the gate is closed, then once the water level in the wetpond rises above the wetpond trigger the storage management mode kicks in, setting the gate discharge to ~1m3/s. For a very short period the emergency management kicks in, which can be seen by the small blip on the discharge curve during the storage management stage. Storage recovery in this case occurs once Buxton Terrace and the Wetpond drop below their trigger water levels, as Lincoln is already low by this point. During storage recovery the discharge starts at around 2m3/s and reduces based on the QH relationship setup for the gate.

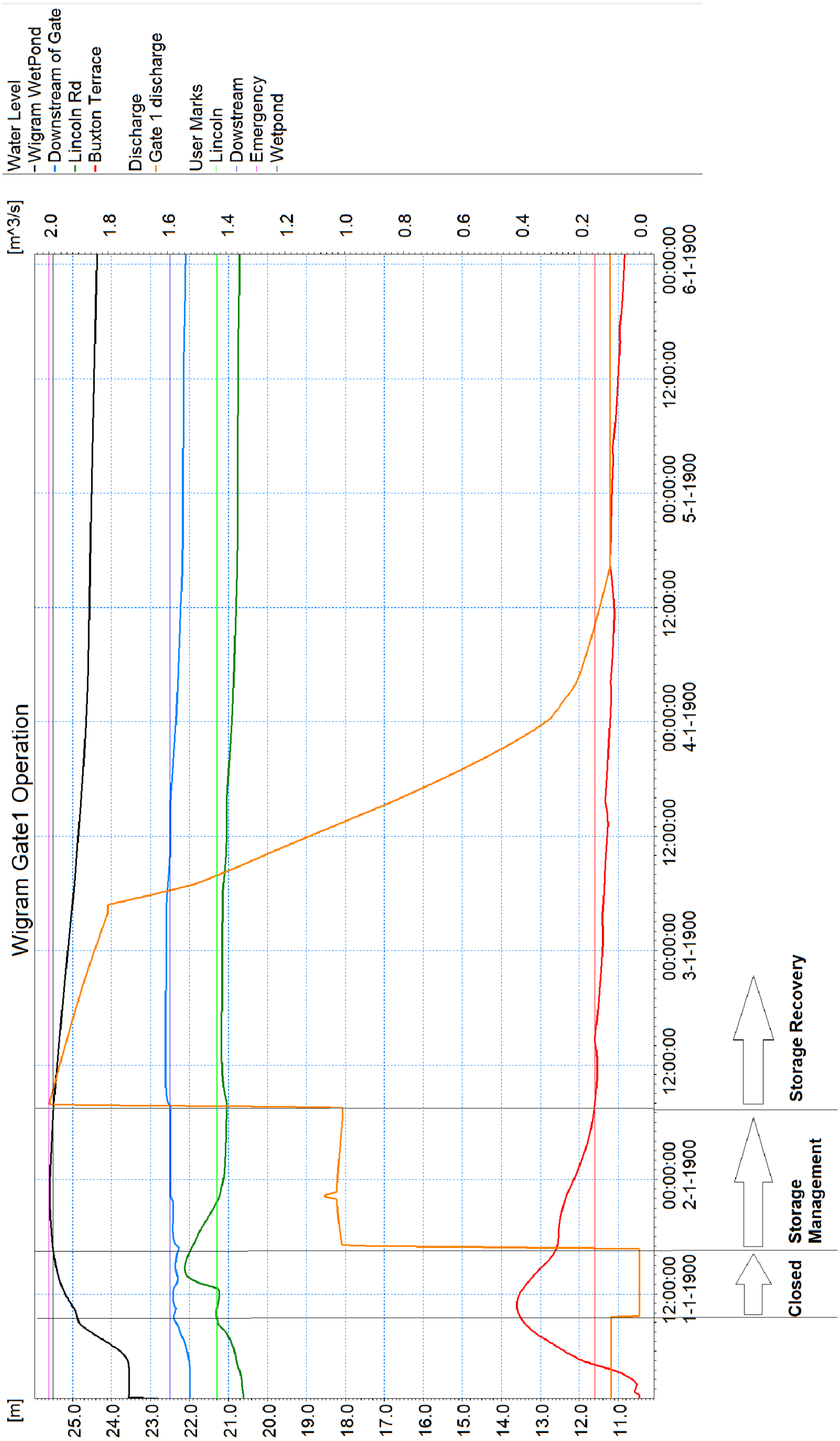

Figure 3-1: Wigram Gate 1 operation

3.1.2 Wigram Wetland outlet (Gate 3)

The Wigram wetland outlet has a similar but simpler operation logic than gate 1. Gate activation is based on the Lincoln and the level downstream of the gates. There is however no storage management mode, so the gate will just remain closed after activation, until the emergency control level is activated. Once the Lincoln, Buxton and downstream levels are low, then the storage recovery begins.

Emergency Control

1. If Wetpond > 25.6 gate is fully open based on Table C

Storage Activation

2. If Heathcote - DS of Wig gate >22.5 then close the gate
3. If Lincoln > 21.3 gate is closed

Storage Recovery

4. If Lincoln < 21.3 AND Buxton < 11.6 and DS of Wig gate <22.3 then open based on Table C
5. Else open based on Table C

Table 3-3: QH table C for Wigram Gate 3

	Flow (m3/s)			Flow (m3/s)
Level	Table C		Level	Table C
0	0		23.85905	0.29
23.5	0		23.96602	0.31
23.52	0.006109		24.08004	0.33
23.54	0.01728		24.20111	0.35
23.56	0.031745		24.32921	0.37
23.58	0.048875		24.46436	0.39
23.6	0.068305		24.60656	0.41
23.62	0.08979		24.75579	0.43
23.64	0.113148		24.91207	0.45
23.66	0.13824		25.07539	0.47
23.68	0.164954		25.24576	0.49
23.7	0.193196		25.42316	0.51
23.72	0.222889		25.60761	0.53
23.74	0.253963		25.79911	0.55
23.76871	0.27		25.99765	0.57
23.77845	0.27		26.20323	0.59
23.78827	0.28		26.41585	0.61
23.79816	0.28		26.63658	0.63
23.80811	0.28		26.86332	0.65
23.81814	0.28		27.09711	0.67
23.81814	0.28		27.33794	0.69
23.82824	0.28		27.58581	0.71
23.8385	0.29		27.84073	0.73
23.84874	0.29		28.10269	0.75

Discussion

From the model results, Figure 3-2, we can see that the storage activation is triggered twice as the Lincoln Road level shifts around the trigger level, Figure 3-2. Note that Gate 1 doesn't do this because its default position for the gate is unchanged, while the Gate 3 default position is open. While Lincoln Road is high the gate remains closed, and then once the pond water level exceeds the emergency trigger the gate opens fully. While the pond water level is still above its trigger the gate is switching between open and closed, this is due to the downstream water level sitting around the trigger level (and the default gate position being set to open). As the event progresses the gate closes again due to the downstream level remaining above the trigger level, this level is high because during this period Gate 1 is in the storage recovery mode. By keeping Gate 3 closed here the local downstream impact is being reduced. Once this local downstream level reduces then Gate 3 goes into storage recovery model.

The gate default open mode may need to be revised, as well as how the activation on the local downstream level works with both gates. The general operation of the gates does appear to be working well however.

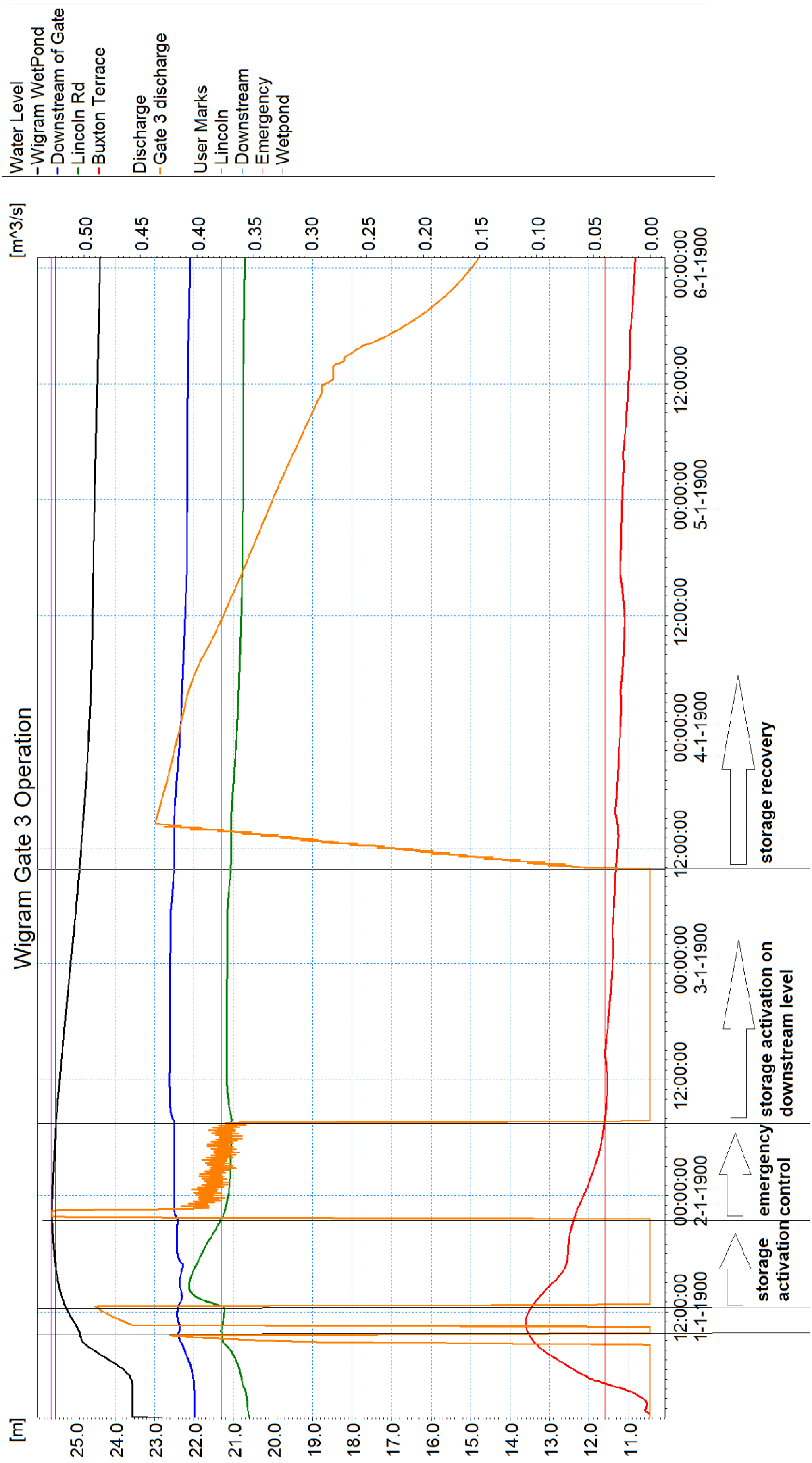

Figure 3-2: Wigram Gate 3 operation

3.2 Curletts Basin

The Curletts Basin consists of a first flush basin and a wetland, along with a bypass channel which connects the first flush basin directly to Curletts Stream, effectively bypassing the wetland. The bypass channel has a bypass structure which sits upstream of where the wetland outfalls into Curletts Stream. The wetland can also overtop into Curletts Stream via the spillway weir at a level of RL 23.9m. The idea behind the control structure at this location is to control the level in the first flush basin and also indirectly the wetland, to optimise the storage and to allow for drainage once the flood has past. The gate operation is based on the sensor levels in Table 3-4.

Table 3-4: Curletts Sensors

Sensors	Chainage	Trigger Level (RL m)
Buxton Terrace	Heth.Hethcote 14904.1	11.6
Lincoln WL	Heth.Hethcote 6207.81	21.3
First Flush Basin (FFB)	heth.curlet_FFtoWtlnd -5	23.7
Wetland	heth.curlet_wtlnd.outlet 0	22.25

The control logic on the bypass gate is as follows:

1. Five minute stand down between gate level changes

Storage management

2. If First Flush basin > 23.7 then Use Table D

Storage Activation

3. If Lincoln Rd > 21.3m, Set gate level equal to 24.01m

Storage Recovery

4. If Buxton WL < 11.6 AND Lincoln WL <21.3 AND Wetland >22.25 use Table E

Managed Bypass Mode

5. If Buxton WL < 11.6, Lincoln WL <21.3 AND if FFB Var = 1 (i.e. high WL in FFB outside of flood) use Table F

Wetland Protection Mode

6. If Buxton WL < 11.6 AND Lincoln WL <21.3 AND Wetland > 22.25 for at least 48hrs use Table F (uses Wetland Variable to keep track)

Set gate to default position outside of flood when all levels are low

7. If Buxton WL < 11.6 AND Lincoln WL <21.3 AND if FFB Var = 2 (i.e. low WL in FFB outside of flood) then Gate level = 23.59 (default position)
8. Unchanged

Variables

Two variables are used, via dummy gates, in this area to keep track of when the flood is active or not. This helps with setting the wetland protection and managed bypass modes.

First Flush - Variable 1

Value will be 2 outside of the flood and 1 during the flood.

Initial value = 1

If FFB level < 23.35 and This Variable >0.9 then value = 2

If FFB level < 23.65 the value = 1

Else Unchanged

Wetland - Variable 2

Used in Priority 6 to allow for tracking how long the wetland level has been greater than the trigger level.

Initial value = 1

If Wetland level > 22.25 then value = 2

Else value = 1

Tables

Table 3-5: Curletts Table D

FF basin WL	Gate level - Table D
23.6	24.01
23.7	23.69
23.8	23.59
23.9	23.01
24	22.135

Table 3-6: Curletts Table E

Level Upstream	Gate level – Table E		Level Upstream	Gate level – Table E
0.0	21.9		23.1	23.0
22.0	21.9		23.3	23.2
22.1	22.0		23.4	23.3
22.2	22.1		23.5	23.4
22.3	22.2		23.6	23.5
22.4	22.3		23.7	23.6
22.5	22.4		23.8	23.7
22.6	22.5		23.9	23.8
22.7	22.6		24.0	23.9
22.8	22.7		24.1	24.0
22.9	22.8		30.0	24.0
23.0	22.9			

Table 3-7: Curletts Table F

Level Upstream	Gate level – Table F
1.0	23.3
23.25	23.15
23.4	23.3
23.5	23.4
23.6	23.5
23.7	23.6
23.8	23.7
23.9	23.8
24.0	23.9
24.1	24.0
30.0	24.0

Discussion

In this scenario, Figure 3-3, the storage is activated on the first flush basin level, where the bypass gate then switches into storage management mode, using Table D. The wetland spillway is still activated as the wetland level raises above the spillway level of RL 23.9m. Once the first flush basin level and Buxton Terrace levels drop below their triggers the storage recovery mode is activated. This drops the gate discharge down significantly and the gate level is set to 100mm below the upstream water level. The managed bypass and the wetland protection modes are not activated before the end of the simulation due to the slow emptying of the basin.

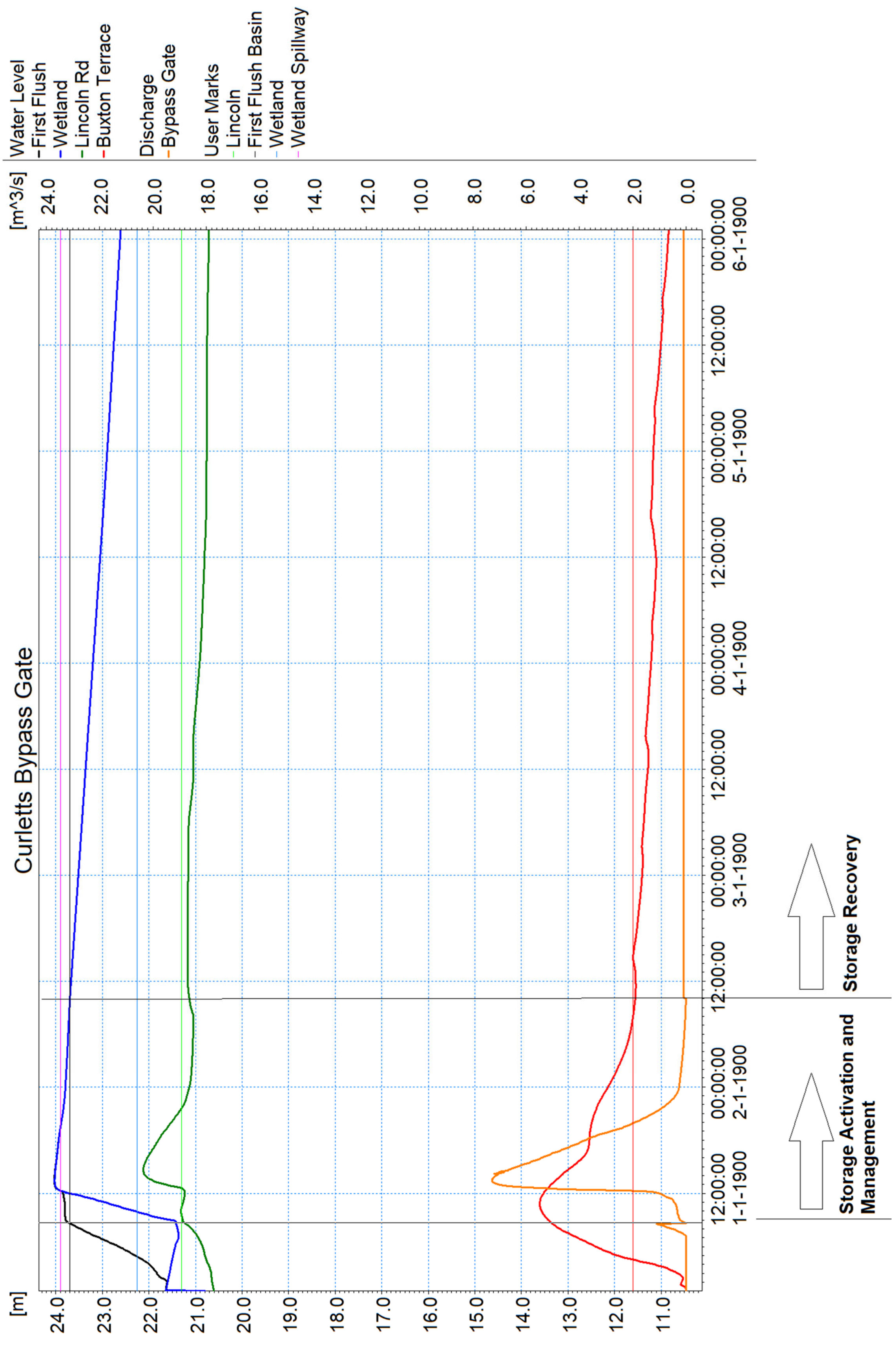

Figure 3-3: Curletts Bypass Gate Operation

3.3 Cashmere Stream

The Cashmere Stream only operates on local levels, in the Eastman and Hoon Hay basins. The gate is generally closed during the flood as there is no storage management mode. The gate operation is based on the sensors in Table 3-8.

Table 3-8: Cashmere Stream Sensors

Sensors	Chainage	Trigger Level (RL m)
Hoon Hay Basin (HHB)	heth.HHBW_03	19.3
Eastman Basin (ESTM)	heth.eastlow 0	19.0

Storage Activation

1. If HHB > 19.3 AND ESTM > 19.0 then close gate

Storage Recovery

2. If HHB < 19.3 AND ESTM < 19 AND gate is closed then
   Gate level = 17.15 (i.e. move directly to the 150mm opening)

Storage Recovery – once upstream and downstream levels have equalised

3. If HHB < 19.3 AND ESTM < 19 AND DH gate -.25 < 0.25 Fully open gate

4. Else unchanged

Discussion

The Cashmere Gate operation is reasonably simple, Figure 3-4. Once the water level in the Hoon Hay Basin reaches the trigger level, the storage activation occurs, and the gate fully closes. Once both the water level in Eastmans and Hoon Hay are low then the gate first opens by 150mm, and then fully, once the water level upstream and downstream becomes similar. Note that the discharge is reasonably low even though priority 3 is active for the remainder of the simulation, this is due to virtually no difference between the levels upstream and downstream of the gate. The small water level fluctuations that can be seen in the Eastmans level and the downstream Cashmere Stream level are caused by the Eastmans gate operation.

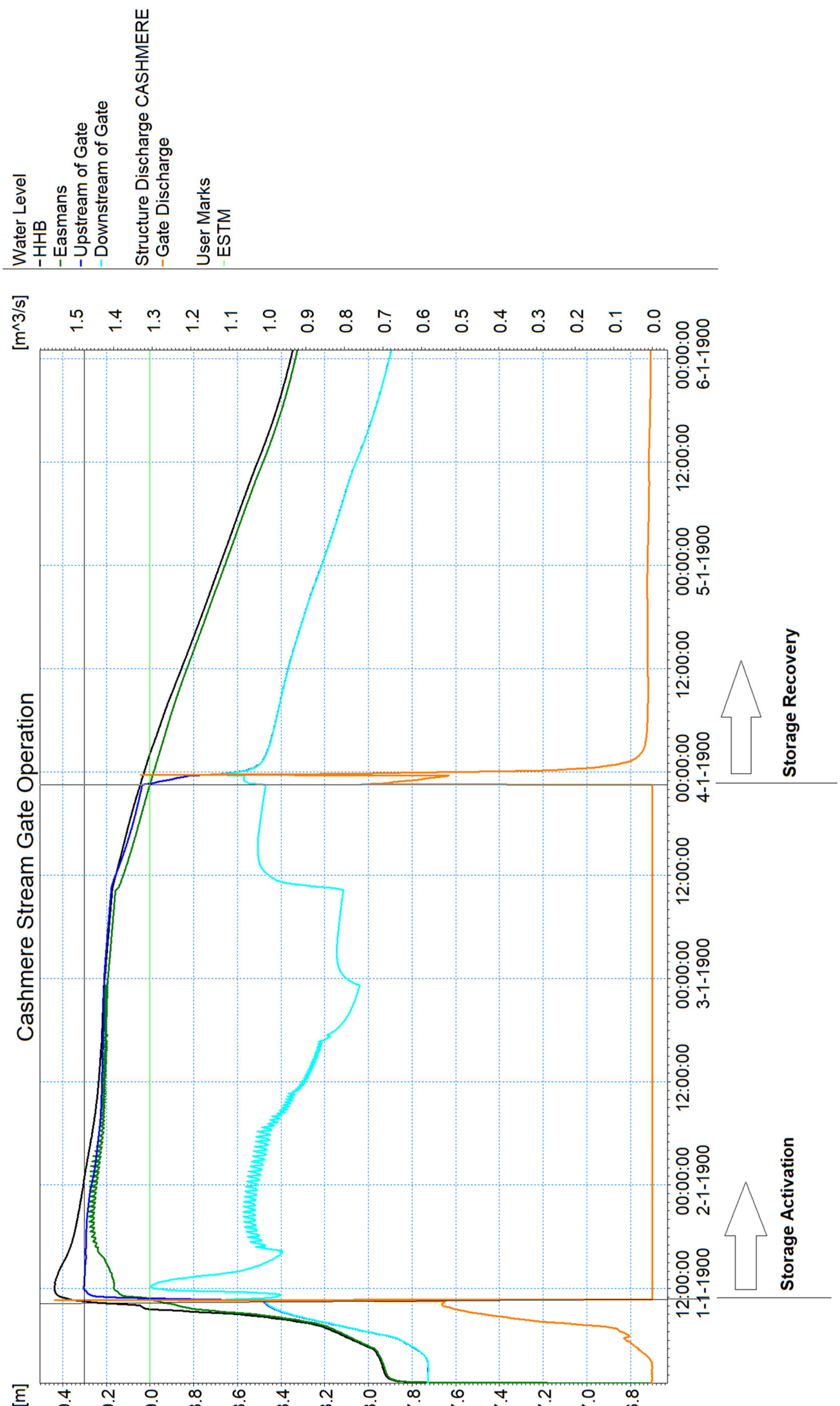

Figure 3-4: Cashmere Stream Gate Operaton

3.4 Eastmans Basin

The Eastmans Basin control structure controls the flow out of the basin, and aims to keep local levels below the spillway, which operates in the MIKE 21 model, as a dike at a level of RL 19.3m. The gate uses the upstream and downstream levels to control the flow through the gate, and references the level at Buxton Terrace and Ferniehurst, Table 3-9.

Table 3-9: Eastman Gate Sensors

Sensors	Chainage	Trigger Level (RL m)
Buxton Terrace	Heth.Hethcote 14904.1	11.6
Ferniehurst	Heth.Hethcote 9929.79	16.6
Downstream WL	heth.cash.diversion 391	18.4
Upstream WL - Basin	Heth.Eastlow 590	19.2

The control rules are as follows:

1. Delay timer for 5 minutes

Storage Management

1. If Eastmans basin (Upstream level) is > 19.2 AND if gate more than 150mm from fully open (based on Table G)
   Then open (raise) by 150mm
2. If Eastmans basin (Upstream level) is > 19.2 AND if the gate is within 150mm of being fully open
   Then set gate level to Table G – Max Opening

Storage Activation

3. If Buxton > 11.6 AND Cashmere Stream Level (Downstream level) > 18.4 then close gate

Storage Recovery

4. If Fernihurst > 16.6 then close (lower) gate
5. If Fernihurst < 16.5 AND if gate more than 150mm from fully open (based on Table G)
   Then open (raise) gate by 150mm
6. If Fernihurst < 16.5 AND if the gate is within 150mm of being fully open
   Then set gate level to Table G – Max Opening

7. Else unchanged

Table 3-10: Table G - Eastmans gate

Head differential (m)	Maximum Gate Level (RL m)
-5	19
0	17.65
0.1	17.8
0.15	17.95
0.25	18.1
0.35	18.25
0.5	18.4
0.75	19
0.9	19
5	17.2

Discussion

The Eastman Gate results, Figure 3-5, show that there is an early storage activation on the High Ferniehurst level, priority 4, before this would activate once the downstream of gate level reaches its trigger. This will be preventing some early discharge through the gate. Once the upstream level reaches the trigger the storage activation mode initiates. This mode causes some oscillations in the water levels due to the gate opening and closing by small amounts. Using the current logic it is difficult to remove these oscillations completely due to the flat hydraulic grade of the system. The logic would need to be adjusted to remove these oscillations. Once the water level at Ferniehurst drops down past the low trigger level the storage recovery mode is activated.

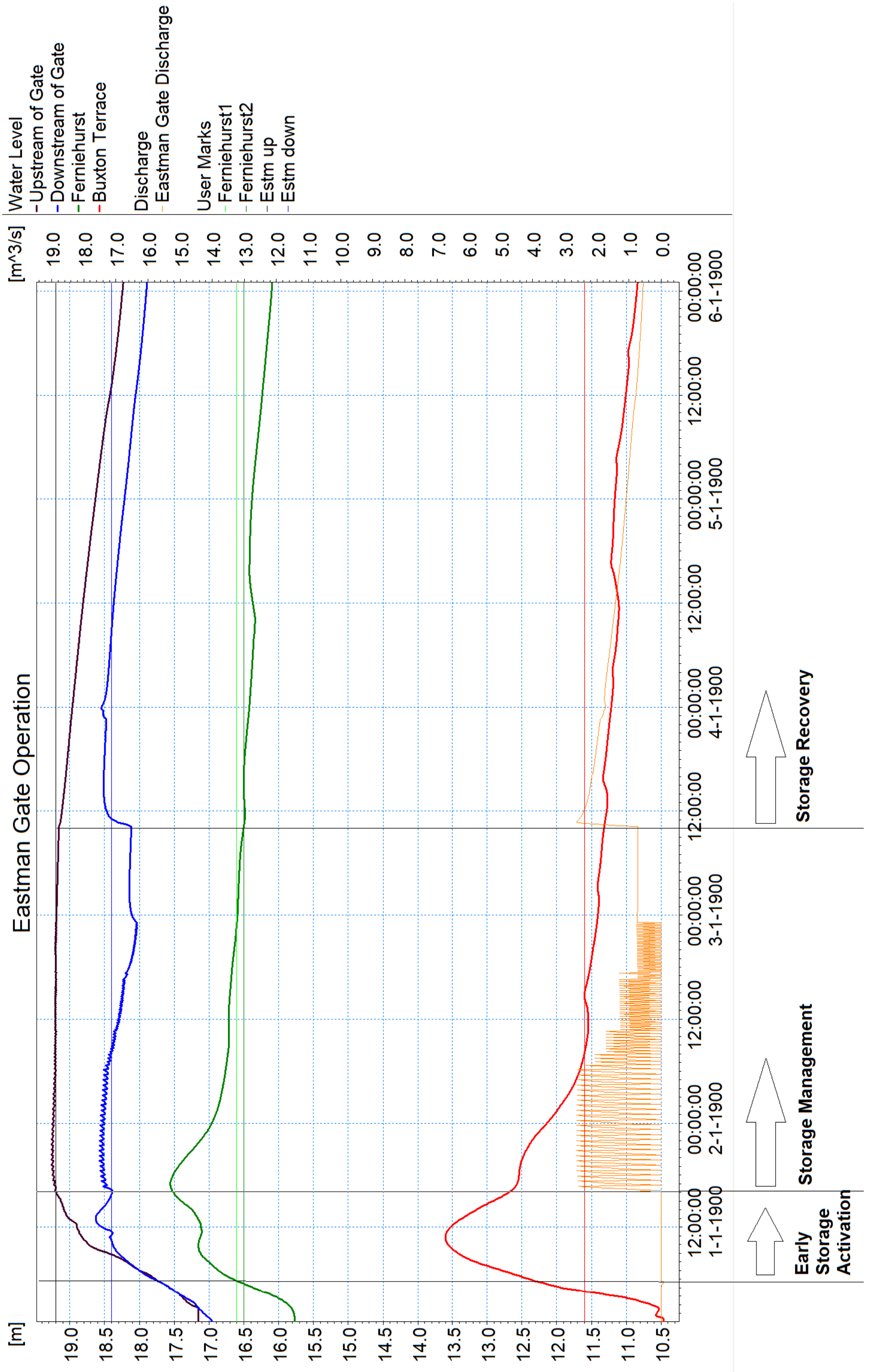

Figure 3-5: Eastman Gate Operation

3.5 Cashmere Worsleys Valley and Upper Dam

The Cashmere Dam sits upstream of the Cashmere Worsleys Valley storage basin. Both of these storages are controlled by gates. The Cashmere Dam gate generally holds back water until the lower storage has reduced in level, while the lower storage operation is related to the levels at Ferniehurst and Buxton. The lower basin storage needs to be kept below the downstream road level, at around RL 18.9m, so that the road is not overtopped and downstream properties are not flooded. The Lower Cashmere sensors are listed in Table 3-11.

Table 3-11: Cashmere Worsley Valley Sensors

Sensors	Chainage	Trigger Level (RL m)
Buxton Terrace	Heth.Hethcote 14904.1	11.6
Ferniehurst	Heth.Hethcote 9929.79	16.5/16.55
Downstream WL	heth.cashvadr 4460	17.0
Lower Storage WL	heth.cashvadr 4440	17.8

The control system logic operates as follows:

Gate 1 - duty

1. Unchanged - 15 min delay timer

Storage Activation

2. Downstream Level > 17.0 AND StorageTrigger =0 close the gate
3. Lower basin level <17.8 and Buxton > 11.6 then close the gate

Storage Management

4. Lower basin level >17.8 then set the gate level based on Table H

Storage recovery

5. Unchanged – 45min delay timer (added to the first delay this is 1 hour)
6. If Ferniehurst < 16.5 & Buxton <11.6, open the gate by 5%
7. If Ferniehurst > 16.55, close the gate by 5%
8. Unchanged

Gate 2 - assist

1. Unchanged - 15 min delay timer

Storage Activation

2. Downstream Level > 17.0 close the gate AND StorageTrigger =0
3. Lower basin level <17.8 and Buxton > 11.6 then close the gate

Storage Management

4. Gate 1 is fully open and Lower basin level >17.8 and Ferniehurst > 16.6, then set the gate level based on Table H

Storage recovery

5. Unchanged - 15 min (added to the first delay this is 30 minutes)
6. If Ferniehurst < 16.5, Buxton <11.6, and Gate 1 is fully open, open the gate by 10%
7. Unchanged

Variables

Cashmere Valley Storage Trigger Variable

This value will change to one the first time storage is activated, then it will not change back. This allows the system to not retrigger storage activation given two different conditions can trigger activation.

Initial value = 0

If the lower storage WL > 17.8 then value =1

Else unchanged

Tables

Table 3-12: Cashmere Dam storage management - Table H

Basin level	Gate level	Note
17.8	18.91	Fully closed
17.87	17.62	
17.94	17.74	
18.01	17.80	
18.08	17.62	
18.15	17.35	
18.22	17.08	
18.29	16.81	
18.36	16.54	
18.43	16.27	
18.5	16.1	Fully open

Discussion

The Cashmere Worsleys gates activate based on the Cashmere valley water level rising above the basin trigger level, Figure 3-6. Once this occurs the Storage Management mode is active and the Duty Gate opens to prevent overflow of the basin. The Assist Gate is not used, as this only opens once the Duty gate is fully open. Storage recovery can only occur once the valley water level drops below the trigger level so this is not activated during this simulation. A suggestion was made that to force storage recovery then a condition for Buxton Terrace to be above the trigger level should be added to priority 4 (storage management), however if this is done this allows the basin level to raise above 17.8m due to the draining of the upper dam, so this was not included. Some adjustment may be necessary to adjust the logic of these gates and the upper dam gate to allow the move into recovery mode earlier without threatening overflow of the road, while keeping downstream levels low.

Any changes to the storage recovery are unlikely to impact on peak water levels in the rest of the catchment, especially since any outflow will still be limited by the other discharge criteria.

After day 4 the flow from the upper dam can be seen entering the lower dam which increases the discharge outflow from the lower dam.

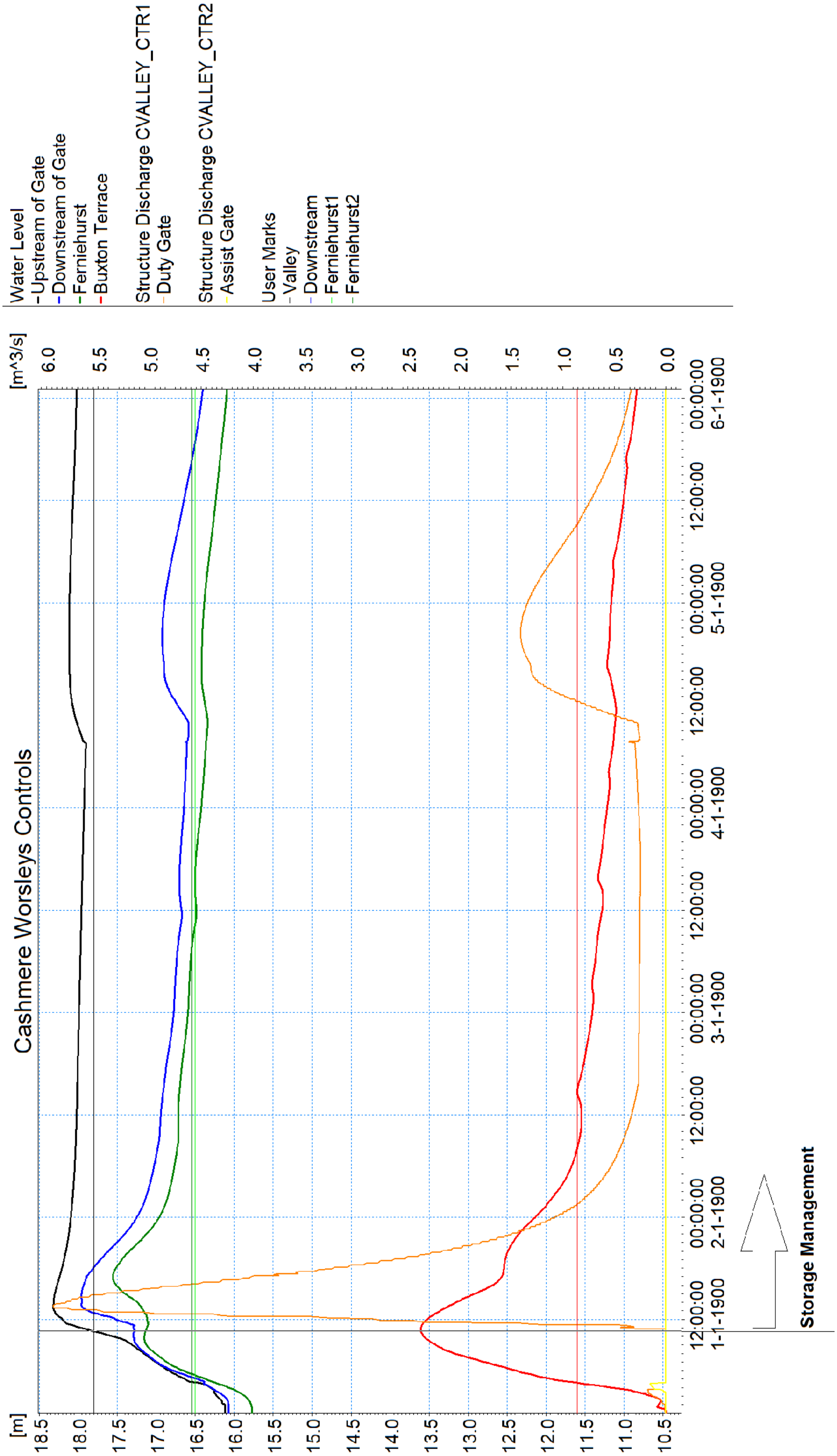

Figure 3-6: Cashmere Worsleys Controls

Upper Dam

The upper dam sensors are listed in Table 3-13. The dam has a simpler control system than the lower valley, and it generally remains closed during the flood period and opens once the lower basin levels have dropped.

Table 3-13: Cashmere Upper Dam Sensors

Sensors	Chainage	Trigger Level (RL m)
Buxton Terrace	Heth.Hethcote 14904.1	11.6
Lower Storage WL	heth.cashvadr 4376	17.9
Upper Dam WL	heth.cashvadr.DAM 3718	18.1/17.5

The control logic for the upper dam is as follows:

Storage Activation

1. If Buxton > 11.6 AND StorageTrigger =0, close
2. If upper basin WL > 18.1, AND StorageTrigger =0 then close

Storage Recovery

3. If lower basin WL <17.9 AND upper basin WL > 17.5 set Gate Level to 17.5 (mostly open)

Normal Operation

4. If upper basin WL < 17.5, fully open
5. Else unchanged

Variables

Cashmere Dam Storage Trigger Variable

This value will change to one the first time storage is activated, then it will not change back. This allows the system to not retrigger storage activation given two different conditions can trigger activation.

Initial value = 0

If the lower storage WL > 17.9 then value =1

Else unchanged

Discussion

The dam operates as per the logic, Figure 3-7. The dam closes early on as Buxton Terrace raises above its trigger level. The dam then begins to empty once the lower basin water levels have dropped below the 17.9 level. Note that because the upper dam does not consider the additional emptying logic of the lower basin the storage recovery of the upper dam has priority over that of the lower basin. This may be the intention of the logic, but it does mean that the lower basin will take longer to enter into storage recovery mode. Additional work may be needed to better optimise the two systems.

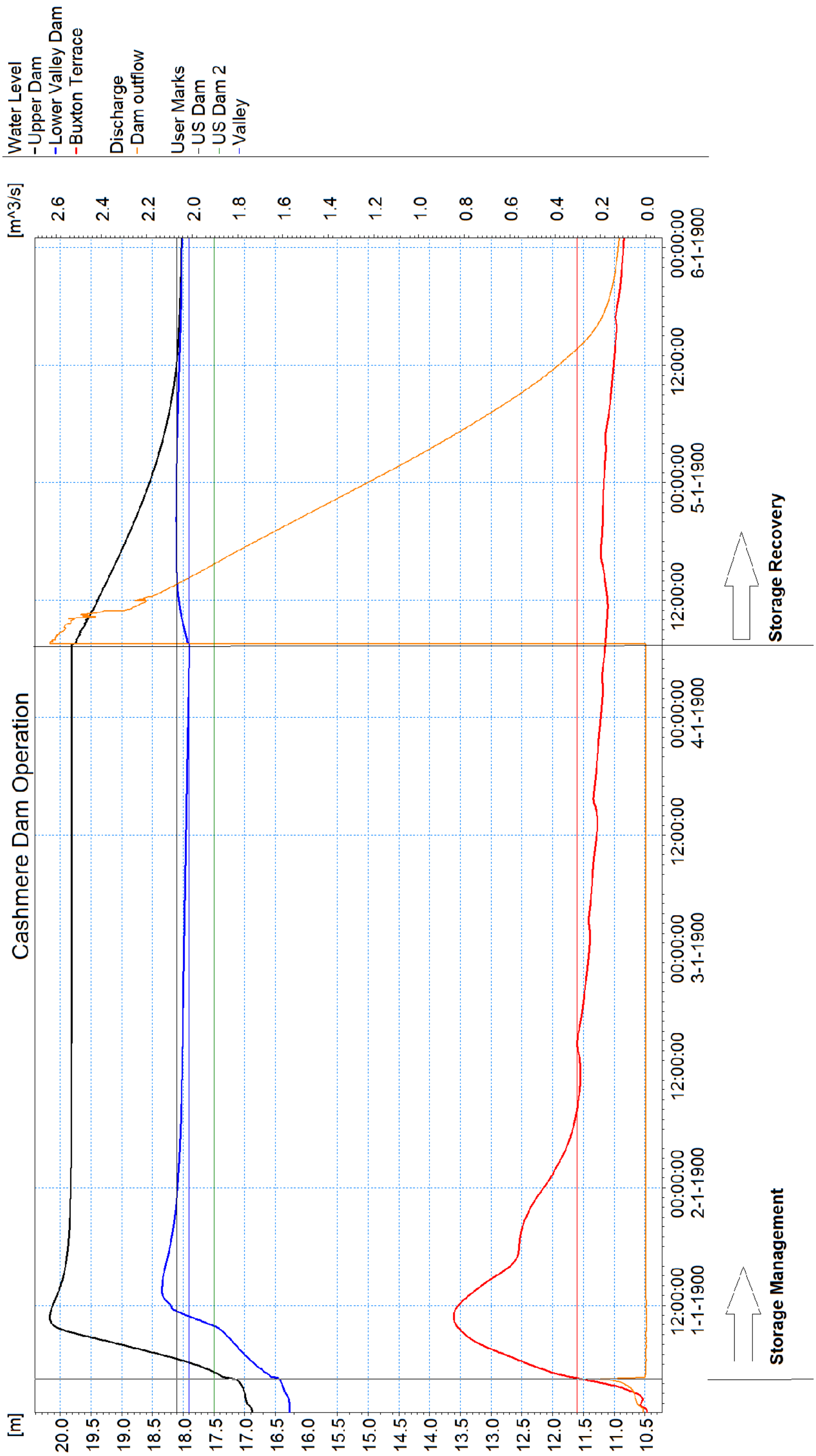

Figure 3-7: Cashmere Dam Operation

4 Specific Model Scenarios

4.1 No Basins Scenario

The v26 model was changed to revert the four upper Heathcote stormwater basins to a pre-development state. To do this the 2011 August/December LiDAR and/or AECOM v16 model were used. The following actions were undertaken to revert the basins:

- Streams realigned within the highlighted areas, Figure 4-1, in the MIKE 11 model, to reflect the original alignment. Cross sections used from the v16 model.
- New structures (bridges, culverts, weirs, control structures) removed and old ones reinstated to reflect the pre-development condition.
- Urban pipes removed within the highlighted areas.
- Terrain updated in the highlighted areas to reflect the pre-development condition, generally this meant using the v16 model levels or the 2011 LiDAR.
- Mesh structure updated to match pre-development stream alignment. The mesh structure was only changed around the streams and was kept as similar as possible in all other areas.
- MIKE Flood links updated to account for new stream alignment and changes in outlet levels (due to change in DEM).
- Dike added to represent the Cashmere Worsleys old road levels (as this is not included in the mesh update.
- Dikes around basins removed where appropriate to reflect the pre-development condition.
- Updated bed roughness values in the 1D and 2D models to be consistent with landuse and nearby values.
- Updated the infiltration and groundwater depths to reflect change in landuse and ground levels.

QA checks were done to ensure that flood links were not left disconnected by the update, which could potentially lead to water generation.

The no basins model was run for the 10 year and 50 year ARI, 24 hour duration storm events. The runs are based on current climate with the 2020 landuse in all areas not highlighted in Figure 4-1.

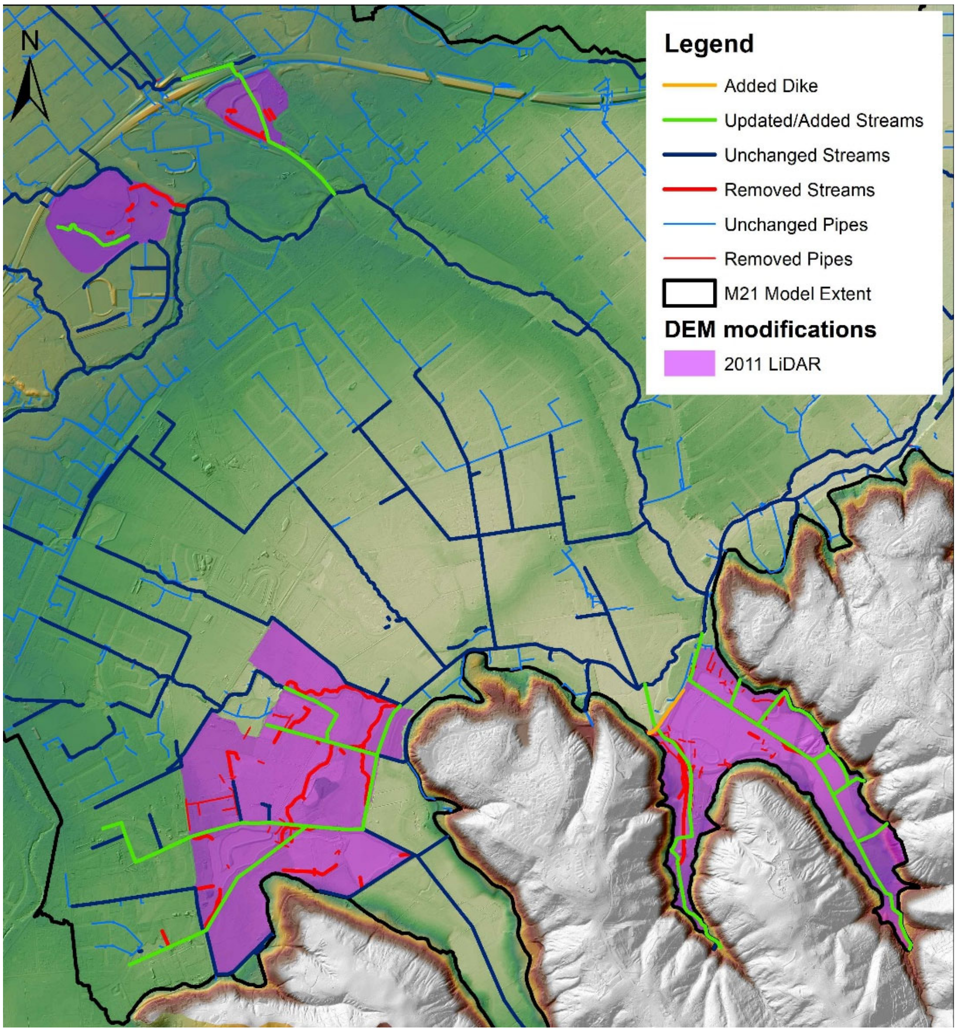

Figure 4-1: Areas reverted in the No Basins model

4.1.1 Results discussion

The no basins model run illustrates the direct impact of the 4 major stormwater basins and their active management control. The results show a significant impact along the Heathcote River which is consistent with modelling of the Basins prior to using the CWM. The no basins model was run for the 10-year and 50 year ARI, 24 hour storm duration events for the current climate, and the results compared to the equivalent base model simulation. The results of both simulations show a reduction in peak flood level along the length of the Heathcote River from 1km upstream of Wigram Basin to just beyond Opawa Road. Results from the simulations at key monitoring locations identified by CCC are shown in Table 4-1 below.

Table 4-1: Key water level results and differences - No basins modelling

	Water Level (m RL)				Difference (m)	
Location	50yr base	50yr no basins	10yr base	10yr no basins	Diff 50yr	Diff 10yr
Lodestar Avenue	27.84	27.84	27.28	27.28	-0.00	0.00
Templetons Road	22.88	23.37	22.57	22.65	-0.49	-0.08
Lincoln Road	21.83	22.06	20.91	21.43	-0.23	-0.52
Frankleigh Street/Sparks Road	19.70	20.04	19.06	19.39	-0.34	-0.33

Ferniehurst Street	17.69	17.91	16.82	17.35	-0.22	-0.53
Buxton Terrace	13.40	13.54	12.91	13.14	-0.14	-0.24
Opawa Road Bridge	11.74	11.93	11.18	11.28	-0.19	-0.10
Ferry Road upstream of Radley Street	11.23	11.29	10.96	10.99	-0.06	-0.03

Figure 4-2 and Figure 4-3 show the depth difference for the two scenarios comparing against the Existing Development model, i.e. Existing Development minus No Basins. The differences show that there is a consistent decrease in flood depth downstream of the basins in both flood events. The north-east section of Hendersons Basin has a depth decrease in the order of 220mm in both events, and the floodplain between Cashmere Road and Worsleys Road has decreased in level by 180mm in the 50 year event, and 270-300mm in the 10 year event.

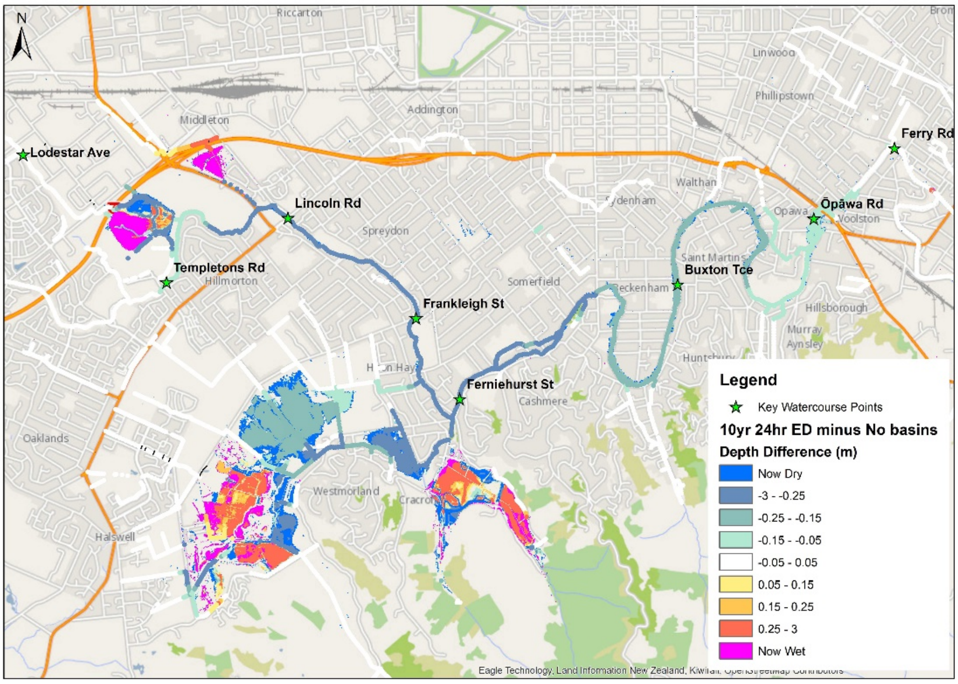

Figure 4-2: 10 year ARI, 24hr ED minus No Basins, Depth Difference

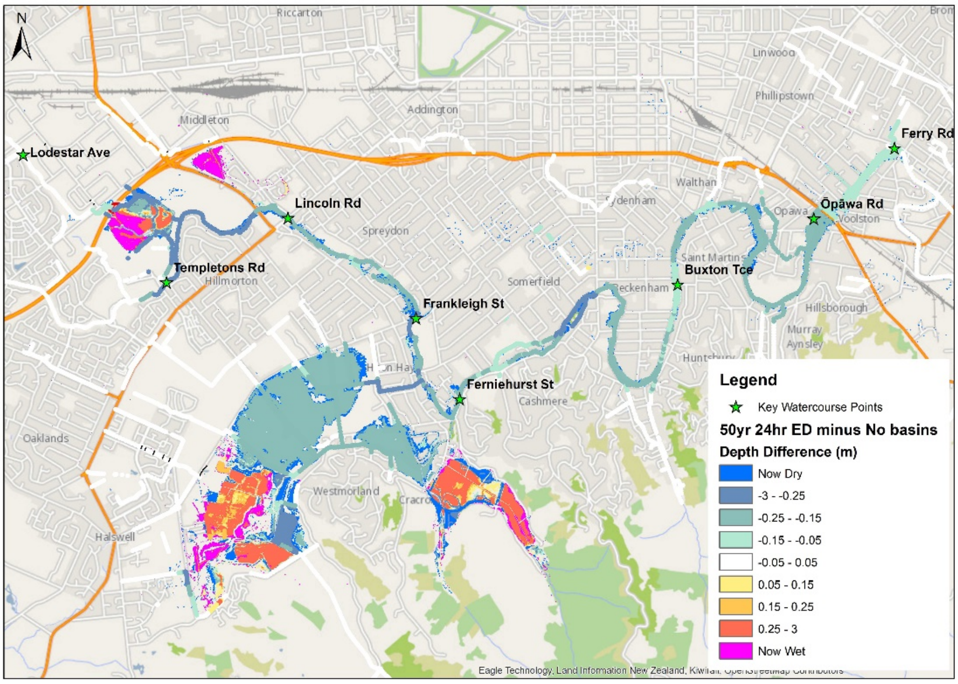

Figure 4-3: 50 year ARI, 24hr ED minus No Basins, Depth Difference

For each of the stormwater basins we have compared the outflow discharge with and without the basin upgrade, Figure 4-4 to Figure 4-7. These results are plotted below. The results show that volume discharged from all basins during the event has been reduced due to the larger storage volume and controlled release. The peak flows are also generally lower with some exceptions in the 50 year event, in these cases the control scheme may be able to be revised to improve the outcome if necessary.

Curletts Basin significantly reduces the outflow discharge in the 10 year event, however in the 50 year event the peak discharge is similar to the base case. This is due to the water levels in the basin reaching a critical level in the 50 year event, where release of the stored water is necessary.

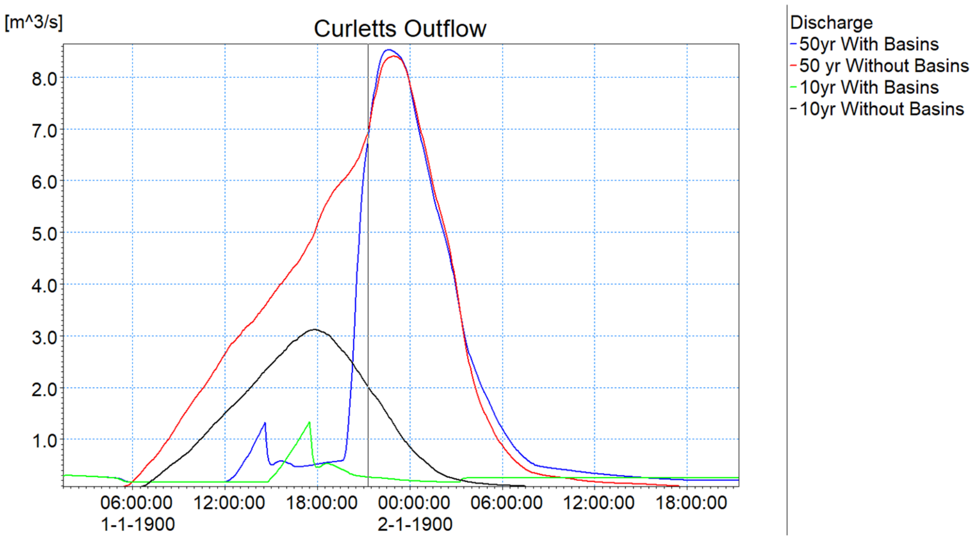

Figure 4-4: Curletts outflow with and without basin

The Wigram outflow discharge is measured on the Heathcote River downstream of the basin outflows, so includes some contribution from Awatea Stream. However, because this contribution is the same in both model simulations the comparison is still valid. The outflow discharge is reduced significantly in the 50 year event.

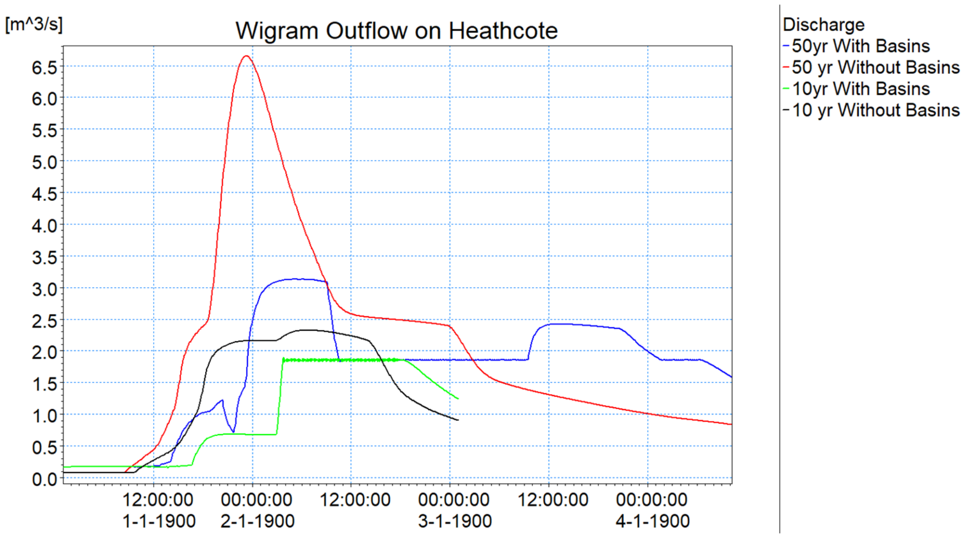

Figure 4-5: Wigram outflow with and without basin upgrade

The Cashmere Worsleys flow is only measuring the Cashmere Valley Stream and not Worsleys Stream, which adds approximately 2m^3/s in the 50 year and 1m^3/s in the 10 year no basins scenario. In the base scenario there is no flow through this branch. The outflow from the basin is reduced significantly in the 10 year event. In the 50 year event the peak discharge is equivalent in both scenarios but the volume discharged is less at the start of the event due to the gates closing. This has the effect of delaying the peak discharge by about an hour. Figure 4-6 shows the 1D flow at Buxton Terrace for each scenario. The peak flow at Buxton occurs earlier than the peak of the Cashmere Worsleys valley outflow (in part due to the contribution of the eastern hillside catchments). This means that the early closing of the gates results in a reduction in flow downstream.

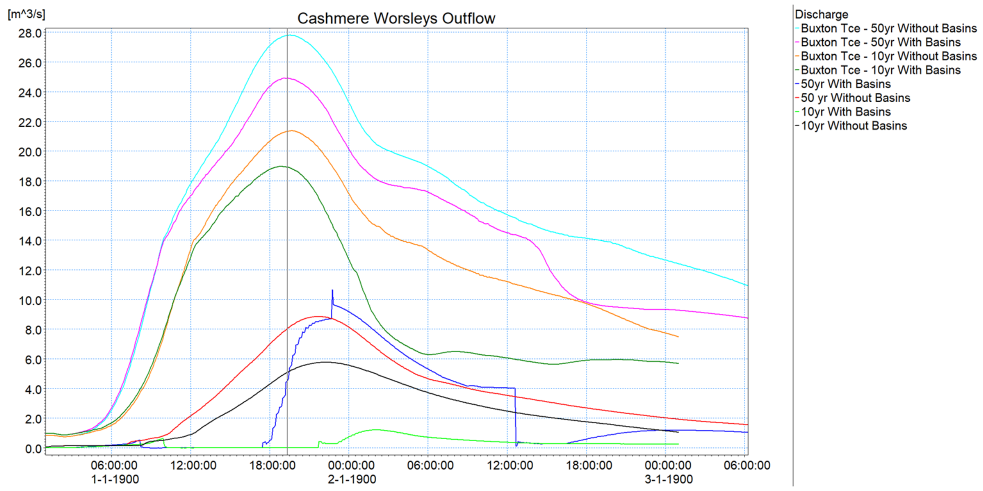

Figure 4-6: Cashmere Worsleys outflow with and without basin upgrade

The outflow from the Eastmans and Cashmere Stream control gates is measured by observing the discharge directly downstream of the Eastmans Basin, on Cashmere Stream (directly downstream of the confluence with Dunbar's Stream). The 10 year discharge is reduced significantly especially considering the time of the peak flow in the No Basins model. The water has been held back and is released later in the event once the peak levels downstream have receded. The 50 year reduction in discharge is less significant, and the gate hunting (oscillations) from the Eastmans control gate can be seen in the result. From the flow hydrograph it appears as if there is significantly more volume of water in the "With Basins" result. However, this is due to less water spilling into the North East section of Hendersons Basin, due to bunding. The flow hydrograph shown does not take this overflow into account however this section of Hendersons Basin shows a water level reduction of approximately 220mm indicating less water is entering this area.

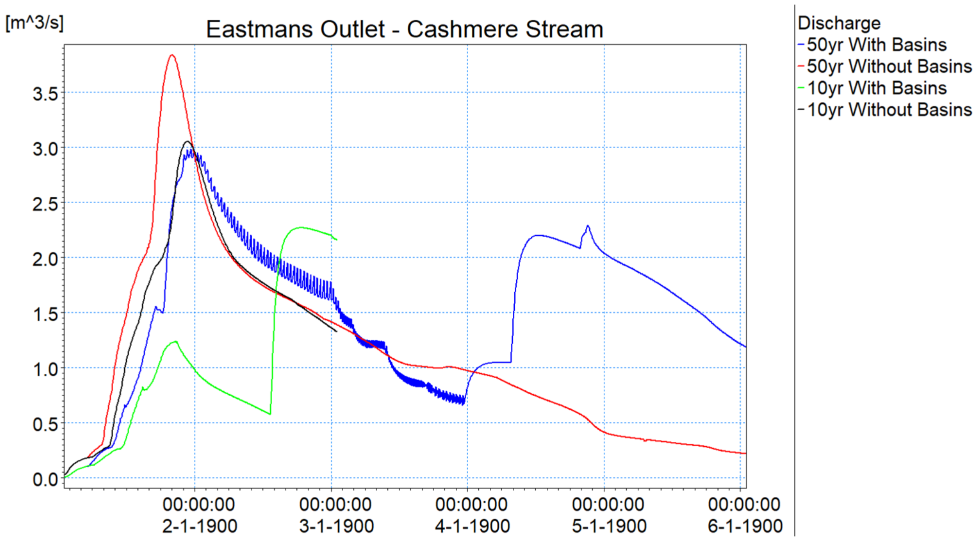

Figure 4-7: Eastmans outflow on Cashmere Stream with and without basins

The reduction in levels in the 10 year event can be mainly attributed to the significant reduction in outflow at Cashmere Worsleys, Eastmans/Sutherlands/Hoonhay and Curletts Basins, with a reduction in peak flow in the Heathcote River of $8m^3/s$ (measured downstream of Ferniehurst). The 50 year event shows a reduction in outflow of $4m^3/s$ at the same location on the Heathcote. In the 50 year event the Worsleys and Eastmans/ Sutherlands/Hoonhay Basins have the largest impact on a reduction in flow. In both the 10 year and 50 year events these reductions in flow in the Heathcote River correlate approximately with the sum of the reduction in peak outflow observed at each of the basins.

4.2 CSNDC modelling – 1991

A quasi-1991 model of the Heathcote catchment was developed, based on the calibration model, reported on by DHI in /4/ representing the catchment in 2014. The purpose of the CSNDC (Comprehensive Stormwater Network Discharge Consent, CRC190445) modelling is to allow for an estimation of the impacts of development since 1991 on the surrounding catchment, and in particular at key target points on the Heathcote River at Ferniehurst and Buxton Terrace. The results from this work will be used to provide an assessment of water quantity effects to inform the Heathcote River Stormwater Management Plan (SMP) and to establish consent compliance, for the CRC190445 discharge consent.

A total of 10 simulations have been run for the 1991 model, these are listed in Table 4-2.

Table 4-2: Pre-Development 1991, current climate simulations

ARI year Rain/Tide	Duration
10/2	Durations 6, 12, 18, 24, 30 hr
50/7	Durations 6, 12, 18, 24, 30 hr

4.2.1 Creating the quasi-1991 model

The 1991 model is based on the v26 model. This model has been reverted to a 1991 state focussing primarily on removing or reverting the stormwater ponds. In order to accurately represent the effects of removing the stormwater ponds surrounding terrain has also been reverted to a pre-development state. This was done because the landform changes for developments are designed so that water will flow into the receiving stormwater ponds, if we then remove the ponds but do not change the surface, the surface flow direction will still be towards where the ponds were removed and water may still pond in these areas. In discussion with CCC we agreed that it would be more appropriate if the surrounding terrain was also reverted to a pre-development state so that the overland flow was better represented.

In addition to the landform changes where streams in the 1D were representing storage ponds these were also adjusted, by reverting the watercourses to a pre-development state. For example the Ferrymead ponds near the Heathcote outlet. Some stream realignment was also included where these are impacted by the terrain changes required to properly represent the change in flow paths.

The model roughness and imperviousness layers were adjusted to represent conditions in 1991 as closely as possible. These layers represent the change in extent of the development areas but no account has been taken for representing the increase in intensification (i.e. housing and hard surface density) between 1991 and today.

The following updates were undertaken.

- A polygon shapefile was created which outlines the areas, since 1991, where development has occurred, using aerial photography, pipe network dates and differences between the 2003 and 2015/2020 LiDAR. The layer is an approximate estimate and does not include smaller isolated developments, such as single properties. Aerial photography covering Christchurch is available for 1988 and 1994, and the polygon is based for the most part on the 1994 photography.
- Where pipes were built after 1991 and are within the un-developed area, these have been removed from the MIKE Urban model, along with associated links. A common sense approach has been used to ensure that the model connectivity is maintained.
- MIKE 11 streams and cross sections were realigned or adjusted where the terrain had been reverted to pre-development levels. This occurred in locations such as the Ferrymead ponds which are modelled in 1D, within adjusted basin areas or where significant terrain adjustments were made where the MIKE 11 streams would not function correctly. Streams that were removed or adjusted are shown in Figure 4-8.
- Model terrain was reverted to either the 2011 or 2003 LiDAR surface to represent a pre-development model. Where possible the 2011 LiDAR was used as this is better quality than the 2003 dataset. A map of the areas using the new datasets is provided in Figure 4-8.
- In some locations where the basins were constructed before 2003 but after 1991 these needed to be flattened out of the terrain. Sixteen of these basins existed. The basins flattened for the 1991 model are shown in Figure 4-8.
- The infiltration and roughness files were adjusted to account for the change in land use from residential/industrial to assumed pervious areas. In areas where development has actually reduced since 1991, perviousness values have been assumed based on land use in 1991: 50% pervious for residential areas and 10% for industrial areas. In areas which were completely undeveloped in 1991, the latest Landcare soil drainage class layers were used to help with reverting infiltration rates to their fully pervious states.
- Standard links were adjusted or added in areas where the ground levels were changed or where a previous connection no longer existed. One such area this occurred was where the Awatea basins were removed, a standard link was added to allow water to naturally flow into the top of the Heathcote River.
- Where links had been deleted on hill catchments, some of these links had RORB hydrology inflows attached. In these cases, inflows were added instead to the MIKE 21 domain as source points, to ensure that the same runoff volume was used in all models. The locations of the source points were set to match those of the original MIKE Urban Model B inflow nodes.

Figure 4-9 shows where pipes were removed as well as the areas changed for the 1991 model. These areas were assigned categories based on the changes required, for example the “grass” category indicates that the area should be reverted to open space. For each land use change area the infiltration and roughness values were completely changed (i.e. not scaled from the postdevelopment values). Table 4-3 summarises the roughness and infiltration changes applied to each land use change identified. Note that “land use change” indicates the areas where the land use has changed since 1991, and the values indicate the land use at the time in 1991.

Table 4-3: Land use Changes

Land use Change	Roughness (M)	Infiltration Scaling
Grass	20	1
Industrial	50	0.1
Residential	8	0.5
Road/Paved	71	0
Vegetation	8	1

The infiltration rates used in the base model were extracted for each soil type at points where the land use was 100% pervious. This allowed for a timeseries to be generated that would match exactly with the base model infiltration rates. These timeseries were then scaled by the land use factors indicated in Table 4-3 and the rates were applied to the whole areas identified to be changed. Note that the scaling factor is just indicating the fraction imperviousness for the layer.

Scaling factors were estimated for the Port Hills runoff in the two areas where development had changed, however the estimation did not result in any significant change to the runoff hydrograph, so the scaling was not included in the final models. The estimation was done by comparing an estimate of the rainfall vs runoff coefficient between a residential catchment and a rural catchment and finding a scaling factor to apply to the areas which had changed from rural to residential.

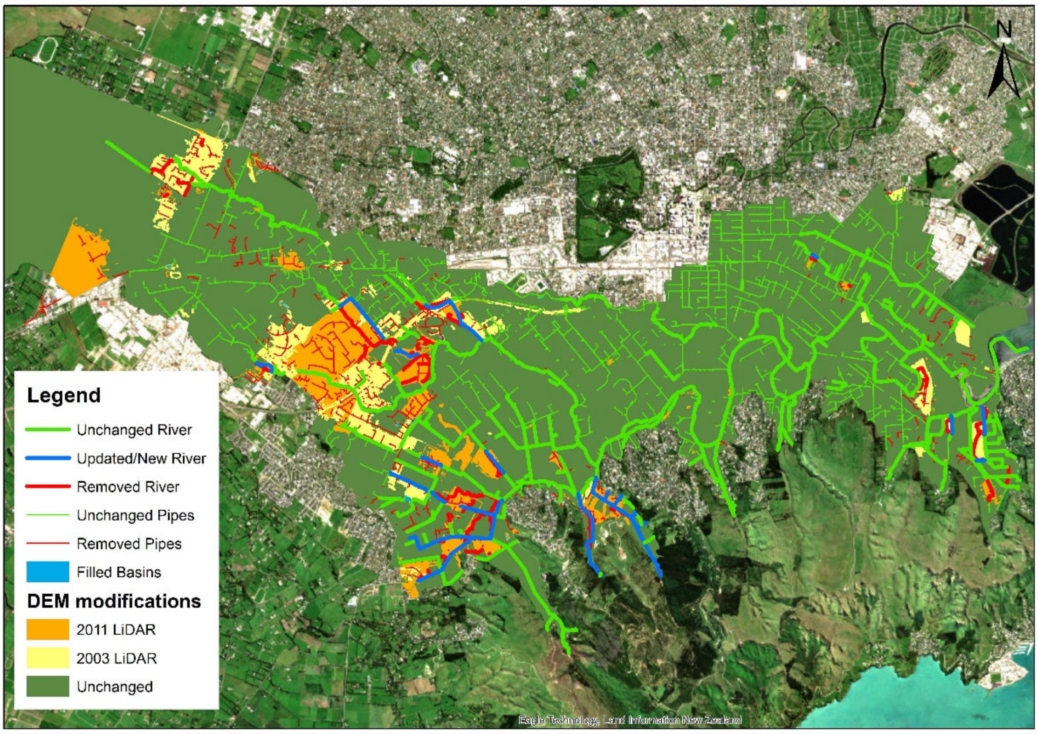

Figure 4-8: Model modifications for the 1991 model

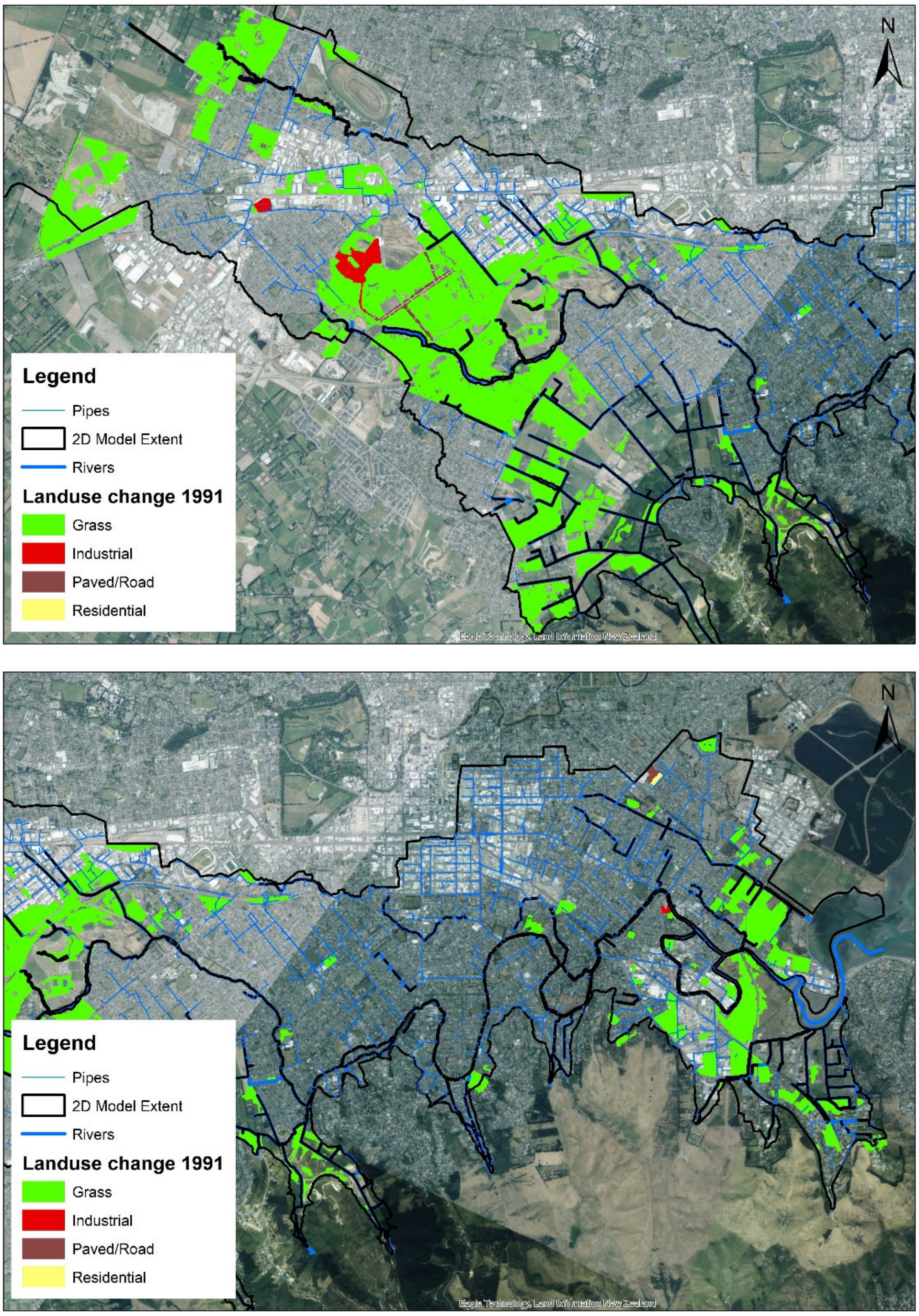

Figure 4-9: Land use Change areas

4.2.2 Results discussion

The model results, river and 2D, were combined into max of max rasters for each ARI modelled. From this the 1991 model results were compared back to the ED 2022 results. The ED 2022 max of max rasters were recalculated beforehand to remove the durations that were not modelled in the 1991 simulations, to ensure a like for like comparison was being made. The results at the key watercourse points are tabulated below in Table 4-4.

The results are consistently lower at all the key watercourse points. This is expected since the No Basins model results also show significant reduction at these locations and additional storage basins were removed as part of this simulation. This shows that the effect of the increase in impervious area from development, between 1991 & 2022, is not significant for the main stem of the Heathcote River. Looking at the wider catchment, some areas such as at Ferrymead and within the basin areas do show increases in levels between 1991 and 2022, however these areas are now being used as storage, so it is expected that water levels would increase.

It is noted that no account has been made for the increase in intensification in the existing built-up areas, between 1991 and 2022. It is expected that this would decrease levels in the 1991 model further, providing less margin on the differences between the 1991 and 2022 results. This is more likely to be significant at the downstream end of the river where the current margin is much smaller.

Table 4-4: Peak Water Levels ED2022 vs Pre Development (PD) 1991 simulations

ShortName	WL 10 ED	WL 10 PD	WL 50 ED	WL 50 PD	Diff 10yr	Diff 50yr
Lodestar Ave	27.82	28.55	28.35	28.78	-0.73	-0.43
Templetons Rd	22.57	24.67	22.88	24.81	-2.10	-1.93
Lincoln Rd	20.99	21.87	22.04	22.45	-0.88	-0.40
Frankleigh St	19.10	19.77	19.97	20.36	-0.67	-0.39
Ferniehurst St	16.90	17.52	17.69	18.08	-0.61	-0.39
Buxton Tce	13.17	13.25	13.61	13.67	-0.08	-0.06
Opawa Rd	11.24	11.30	11.93	12.00	-0.06	-0.07
Ferry Rd	10.99	11.01	11.31	11.34	-0.02	-0.04

Plots of the difference results for the two ARI's are shown in Figure 4-10 and Figure 4-11.

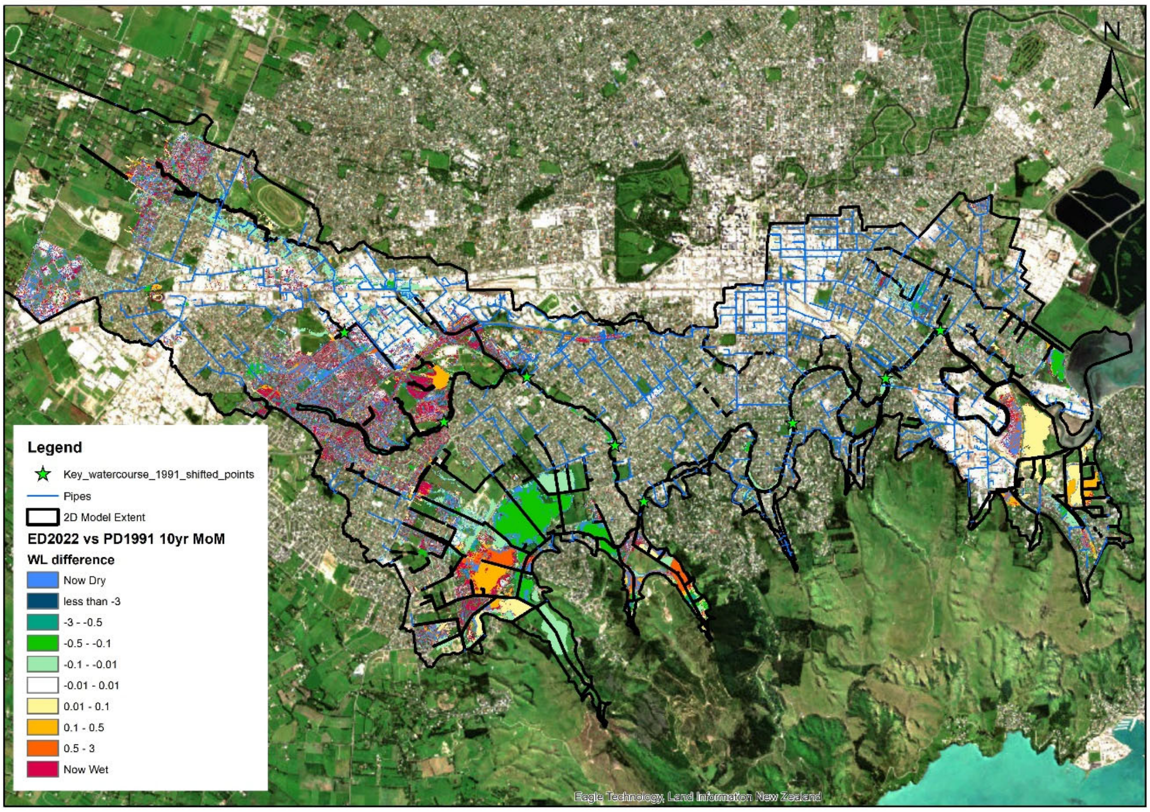

Figure 4-10: Water Level Difference ED2022 minus PD1991 10 year ARI

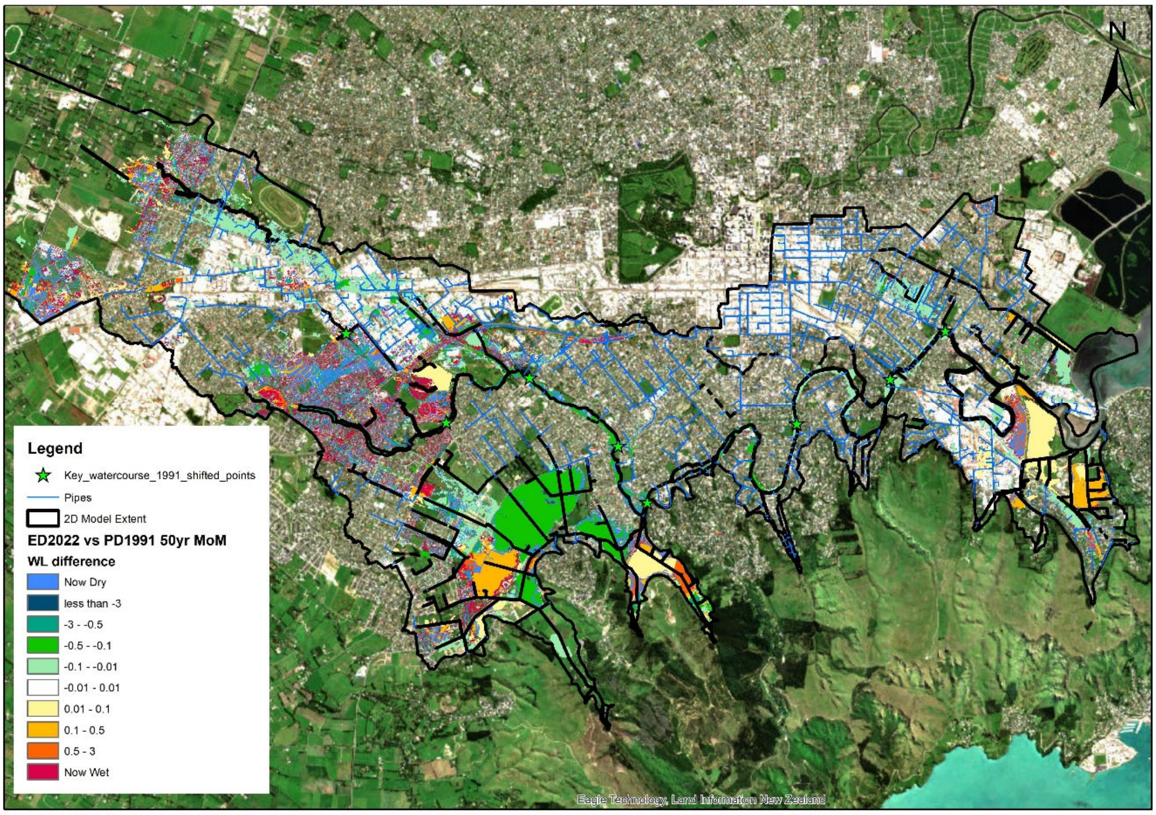

Figure 4-11: Water Level Difference ED2022 minus PD1991 50 year ARI

5 Model Stability

Once model build was complete, preliminary stability work was undertaken to ensure that the model was producing sensible velocities in the 2D and no high oscillations in the 1D. Some fixes were implemented during this stage, for example removing overlap of standard links and dikes, which were creating high velocities in the 2D results. Once the model was running well the model was run for the 7 stability standards simulations, these were:

- ED2020 10yr 1hr
- ED 2020 10yr 36hr
- ED 2020 50yr 6hr
- ED 2020 200yr 2hr
- MPD2060 50yr 18hr
- MPDCC16 200yr 18hr
- MPD2100 500yr 12hr

The simulations were chosen to represent a wide range of rainfall durations, tide levels and rainfall depths.

Water level oscillations were calculated from the MIKE Urban, MIKE 11 and MIKE 21 time-varying results. These have been grouped into bins, i.e. below 0.15, 0.15 to 0.3, above 0.3 etc.

For applying model fixes the stability guidelines state that:

- 1D oscillations between 0.15-0.3m can be ignored, unless the MIKE 21 oscillations are over 0.1m
- 1D oscillations above 0.3m should be attempted to be fixed, and if this is not successful then these should be reported on.

5.1 Process

The assigned models were run and the stability calculated in 1D, and 2D. The results were grouped into 32 areas where oscillations, generally above 0.3m, were occurring.

Types of issues encountered are listed below with the general fixes applied:

- M11 structure issues – checking cross sections fit well and have positive slope downstream
- MU-M21 link issues – applying smoothing factors, qDH to dampen oscillations
- MU-M11 link issues – correcting invert levels at intersection of the two models
- MU pipe/open channel issues – adjusting invert levels on steep pipes
- Overloaded nodes from hydrology – widening manhole diameter and increasing headlosses

After rerunning the models, it was found that the stability fixes improved 1/3 of issues, 1/3 had little or no change, 1/3 were made worse. For those made worse the fixes were reverted and different fixes were applied. The majority of the areas where the issue was made worse were in the downstream tidal area, where the MIKE Urban links were submerged. In these areas exponential smoothing was attempted instead but did not improve the issue.

Some additional iterations were undertaken on the 500yr CC event, which was showing the worst instabilities, to see if additional improvements could be made.

Some stability issues still remained in the final set of results. These most significant of these are described below. Note all plots show the ED 2020 200 year 18hr water level, with pipes, nodes and lateral links plotted.

Full oscillation shapefiles have been produced for all model results as part of the project deliverable.

5.1.1 CashBr 695

This area has a closed cross section representing a culvert, with a pipe entering from the north just upstream. The stability issue is very localised and the oscillations dissipate after a couple of h-points. The 2D area is not impacted, however the 1D peak level will be. Resolving the discrepancy between the MIKE Urban outlet and the channel, and/or potentially modelling the closed section as a culvert structure may be other ways this area could be improved.

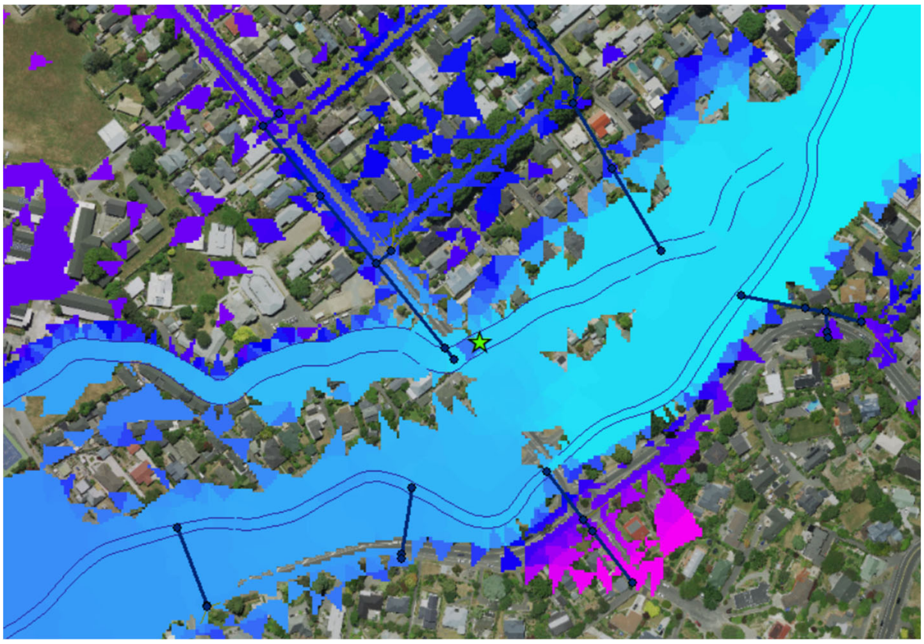

Figure 5-1: CashBr instability off the Heathcote

5.1.2 Pipe network in the tidal region

The downstream tidal area has MIKE Urban instabilities throughout, especially in the higher tide level events, this is likely due to the network being submerged and small changes in the 2D levels creating too high discharges through the links, forcing water to jump in and out of the links. Unfortunately using smoothing factors, Qdh and the exponential smoothing, did not improve the model stability here, and in fact made it worse in some cases. Where these smoothing factors made the stability worse they were reverted. Because these instabilities occur once the network is submerged, levels can be used from the 2D surface results, so this becomes a reasonably minor issue.

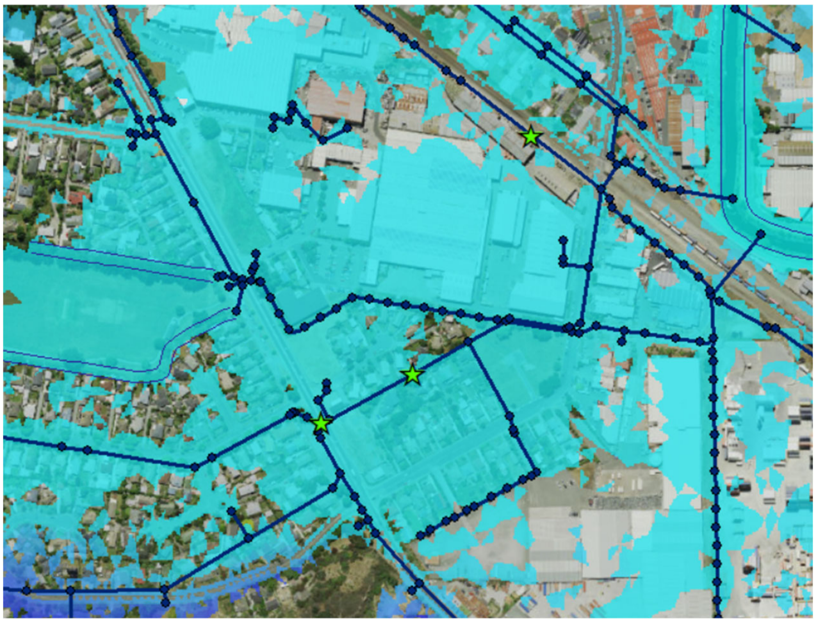

Figure 5-2: Flood area south of Woolston Loop

5.1.3 Richardson Pump Station

The Richardson pump station is modelled in the MIKE 11 model however it is connected at the upstream end to a pipe network. The MIKE 11 model was used in order to utilise the control structure module to apply the required control logic (available options were easier to implement in MIKE 11 rather than MIKE Urban). Unfortunately, the MIKE FLOOD link between MIKE 11 and MIKE Urban at the upstream end becomes unstable in some situations. The pump levels also fluctuate as part of the pump operation which can be flagged as an instability as well. This area could be improved by moving the pump to the MIKE Urban model, or modelling in MIKE+ (in the future when the model is moved to MIKE+) where more options are available.

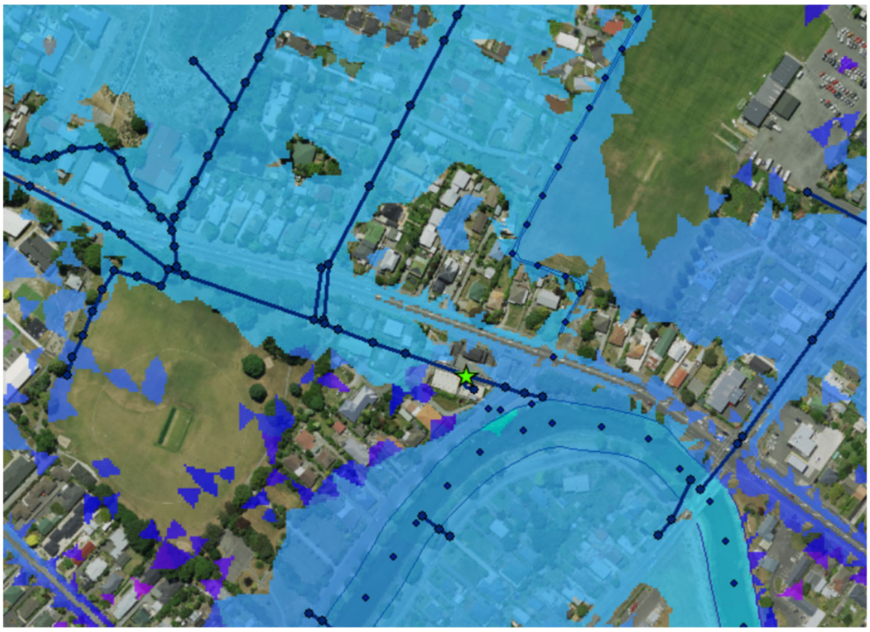

Figure 5-3: Richardson Pump Station

5.1.4 Hayton Stream

Instability occurs near the closed section culvert on Heth.Hayton around chainage 1930. Attempts were made to adjust the cross section to allow a smoother transition to the closed cross section, however some instability still remains in one event. A possible improvement could be to switch the open cross section into a culvert structure.

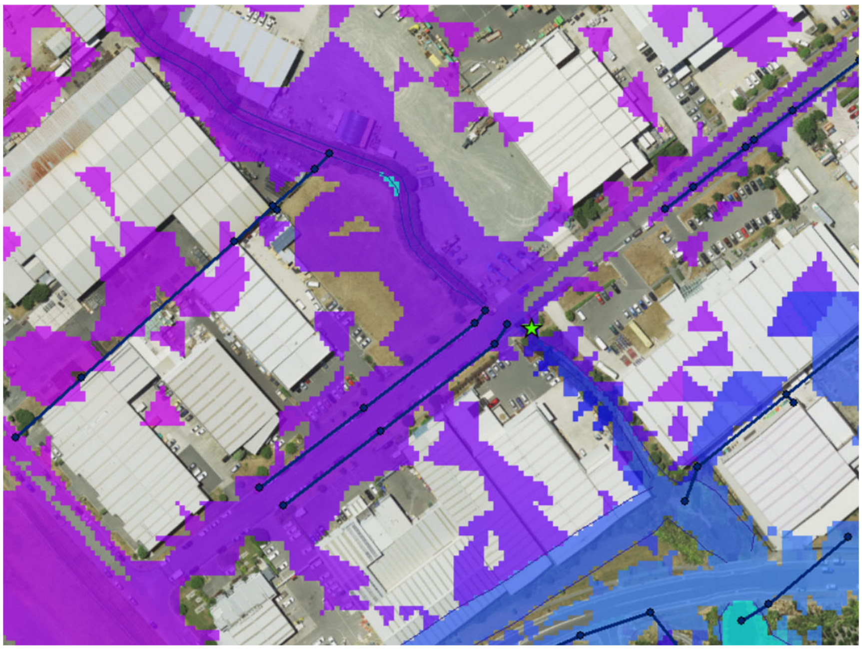

Figure 5-4: Hayton Stream

6 Design Model

6.1 Simulations

The model was run for a total of 74 design simulations. These varied in terms of annual return period (ARI), storm duration, tidal return period, land use and climate change. A table of all runs completed is shown Table 6-1.

Table 6-1: Model Simulations

ARI year Rain/Tide	Storm Duration (hours)	SLR (m)
Existing Development 2022, current climate		+0
10/2	1, 3, 6, 9, 12, 18, 24, 30, 36, 18T	
50/5	1,3	
50/7	6, 9,12,18, 24, 30, 36, 18T	
200/7	0.5, 2	
200/13	6, 12, 18, 24, 30, 24T	
500/50	2, 6, 12, 24, 30, 24T	
Pre-Development 1991, current climate		+0
10/2	6, 12, 18, 24, 30	
50/7	6, 12, 18, 24, 30	
Maximum Probable Development 2068, 2068 climate		
50/5	1,3	+0.45
50/7	6 ,9, 12, 18, 24, 30, 36, 18T	+0.45
500/50	2, 6, 12, 24, 30	+0.5
50/500	24	+0.5
Maximum Probable Development 2068, +16% climate		
200/20	0.5, 2, 6, 12, 18, 24, 30, 24 T	+1.0
Maximum Probable Development 2068, 2100 climate		
500/50	2, 6, 12, 24, 30, 24T	+1.0

*Orange durations run for an extended 6-day period to track active management

T indicates a tidal simulation where the rainfall and tide ARI are switched

6.1.1 Tide

The tidal timeseries to be used in the model have been updated based on the joint probability of pluvial and tidal flooding, work completed as part of the Multihazards work completed by HKV and GHD /5/. The work includes updated recommendations to the tide and rainfall combinations to be used in the design modelling as well as updates to the tidal timeseries. The tidal range derived for the Ferrymead Bridge was used. Tide and rainfall combinations are shown in Table 6-1. The timing of the tide was not adjusted from previous modelling.

6.1.2 Initial Conditions

The MIKE 11 hotstart definition was updated to allow for only one hotstart simulation to be run and used for all simulations with a similar sea level rise. i.e. the full range of return periods and durations. This hotstart works by using a variable tide level at the downstream boundary. For this variable tide, the level starts constant, to allow for the model to reach steady state, and then runs through a modified tide cycle, to include the full range of starting tide levels. This method allows for the dynamics in the tidal reach to be initialised taking into account the lag in the tidal propagation and the direction of flow. The secondary benefit to this method is that only one hotstart file needs to be used for a large range of simulations, saving on setup time.

When starting any simulation, a lookup table can be used to reference a starting level with the time at which to reference the hotstart res11 file.

The MIKE 21 initial conditions reflect the starting tide level and the initial water levels at some of the stormwater basins, as per Table 6-2.

Table 6-2: Initial Basin Levels

Basin	Initial Level RL m
Wigram, pond and wetland	23.56
Curletts wetland	21.65
Hoon Hay Basin	17.9
Linwood Lower Fields	10.0

When simulating higher sea level rise conditions, the starting tide levels were very high in some simulations. To avoid issues where the simulation would crash due to the starting initial conditions it was necessary to make some changes to the initial starting levels.

- The extent of the initial flooding for the tide conditions above RL 11.3m, the initial extent was reduced back to the extent just lower than RL 11.3m.
- For starting levels greater than RL 11.5m, a special MIKE 11 hotstart was used where the tidal flap gates were all set to allow flow in both directions. This allowed the channels to fill upstream and equalise better with the MIKE 21 initial condition.
- Where the starting levels are greater than RL 11.5m, the MIKE 11 hotstart start time was shifted to align the starting level at the Steamwharf branch with the MIKE 21 initial level. This was necessary due to lag in the MIKE 11 hotstart creating too much difference between the MIKE 21 starting level and the MIKE 11 level.

6.1.3 Infiltration and Groundwater

A number of new infiltration and groundwater files were generated for future climate scenarios, these were:

- Future Development 2068 with Mitigation, 0.45m Sea Level Rise (SLR)
- Future Development 2068 with Mitigation, 0.5m SLR
- Future Development 2068 with no Mitigation, 0.5m SLR
- Future Development 2068 with Mitigation, 1m SLR
- Future Development 2068 with no Mitigation, 1m SLR

The sea level rise is reflected in the groundwater (85th percentile) surface levels. While the future development 2068, with and without mitigation are used from the files produced by GHD in the geodabase dated 20/01/2022. These files were modified to account for the new developments added to the Eastmans Basin area. Otherwise, the imperviousness maps matched well to the existing development in the catchment and to the developments included within the model.

The groundwater depths were recalculated based on the latest model DEM. The groundwater and impervious area rasters provided by GHD needed to be extrapolated to cover the full catchment area.

6.2 Results

6.2.1 Results processing

Results were processed and provided as per the User Requirements spreadsheet /6/, which outlines all outputs and formats required for the model delivery. MIKE IO, Python and GIS tools were used to convert the results as per Table 6-3. For each of these results a "max of max" for each simulation group (same ARI and climate/land use), was calculated as well as the critical duration.

Table 6-3: Results conversions

Result Type	Converted into format
MIKE 21 dfsu	Arc GIS raster, Tiff of water level and depth
MIKE 11 .res11	Point shapefile of max level and max/min discharge
MIKE Urban .prf	Point shapefile of max water level, line shapefile of max/min discharge

6.2.2 Results QA

Additional processing of results was undertaken to check that the results were sensible and that all boundary inputs had been applied correctly.

A set of QA points were derived, 13 in MIKE 11, 14 in MIKE Urban and 27 in the MIKE 21 domain, where the peak water level was extracted for each simulation. The points were chosen throughout the model domain. The extracted results were then compared using pivot tables in excel to ensure levels compared in a sensible way against similar scenarios. i.e. you would expect levels to increase as the ARI increases, etc.

Mass error was calculated for each scenario, specifically looking at the MIKE 11 error and the MIKE 21 error (as the MIKE Urban log is not accurate). Note that a recalculation was done for the MIKE 11 error, as the active management logic was generating water within one of the dummy branches. This was necessary as part of how the logic variables work, however it does not affect any of the actual flood results. Once the additional water is removed from the mass balance calculation the values become reasonable. For this reason, we have manually recalculated the values and provided these as an output to replace the html version in the log files. The overall MIKE 11 mass balance error was between +2.7% and -0.3% all simulations, with an average of the absolute value of 0.3%. The MIKE 21 error has a maximum volume generation of 1.8m^3. The larger climate change scenarios resulted in the higher mass error.

The critical durations rasters were checked for each group. The Henderson's Basin area was consistently highest in the longest duration tested, either the 30 hour or 36 hour. For the larger events the longest duration was critical over a large portion of the catchment. This was especially prevalent in the FD2100 500 year scenarios. Where a large portion of the downstream Heathcote is highest in the 30 hour flood event. This does indicate that adding longer durations to the simulation list may be necessary to get an accurate peak level and critical duration. It is expected however that the difference between the peak levels in the different durations is fairly similar. For example at Buxton Terrace, the FD2100 500 year max water level shows only a 15mm increase from the 24 hour to the 30 hour event.

General spot checks were undertaken, such as plotting long sections in the 1D models.

The 2D results were checked to see if the model had past the peak in each simulation. This was done by subtracting the peak water level at the last timestep from the peak water level. This result is provided as part of the deliverables. It was found that each simulation had past the peak before it completed.

The 2D water level results were compared to adjacent scenarios to check that the differences are increasing in the expected direction for all grid points. For example it is expected that the 10 year ARI water level results would always be lower than the 50 year ARI results for the same storm duration, and that climate change will always increase water levels when looking at the same ARI and duration. A total of 63 of these difference maps were checked visually using a scale that specifically showed if the differences were either positive or negative. Where the differences were unexpected the results were checked in more detail. Only one area showed a significant unexpected difference, in the 2hr event for the 500 year ARI 2060 (comparing to the MPD2100). This occurred along Bells Creek. This discrepancy occurred due to a model instability however it does not impact on the critical duration in the area so it is not of concern.

Difference maps were also generated for the 2D to compare back to the original AECOM modelling. More detail of which is covered in Section 6.3.3.

To assess the flooding at the model boundary the results were processed to generate a shapefile for each simulation group that indicates where water is ponding significantly at the boundary. The process to generate this shapefile is as follows:

1. Create a point shapefile of the mesh boundary, offset by 3m internally from the edge
2. Transfer the values from the max of max depth results for each group to the point shapefile. Where the underlying depth value will be assigned.
3. Apply a filter to the point shapefile to show only points with a depth value greater than 0.3m
4. Convert the max of max depth result to a polygon – where depth is greater than 0.2m
5. Select the polygons that intersect with the filtered points
6. Clip these polygons where the extent is up to 500m from the boundary, or where there is an obvious high point which would restrict flow. This is to ensure the polygon does not just cover the entire flood extent in places.

The resulting boundary interaction shapefiles have been provided with the model deliverables. Figure 6-1 shows the downstream interaction between the Heathcote and Avon models. Key areas here include: along Madras St, where the Avon model is significantly higher than the Heathcote, along Buckleys Avenue and Humphries Drive where the two models are within 150mm. Figure 6-2 shows three key areas where water is pooling at the edge of the Heathcote/Halswell boundary. The most significant of these is where the

Awatea ponds overflow in the larger events and would enter into the Halswell model.

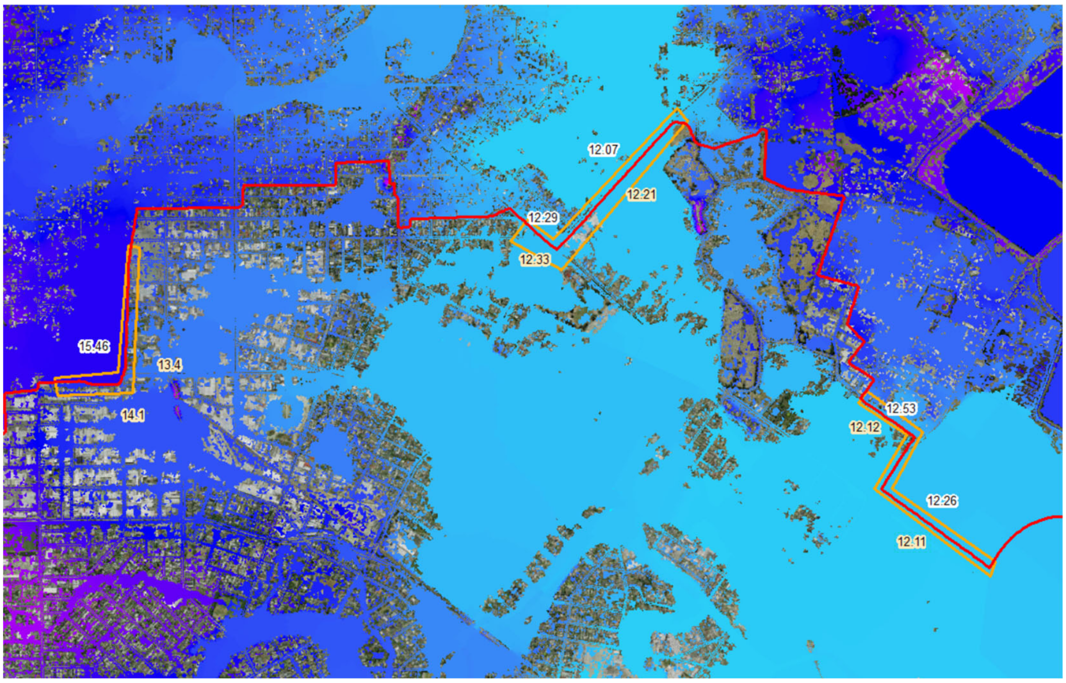

Figure 6-1: Downstream Heathcote and Avon boundary interaction - 500 year 2100 climate

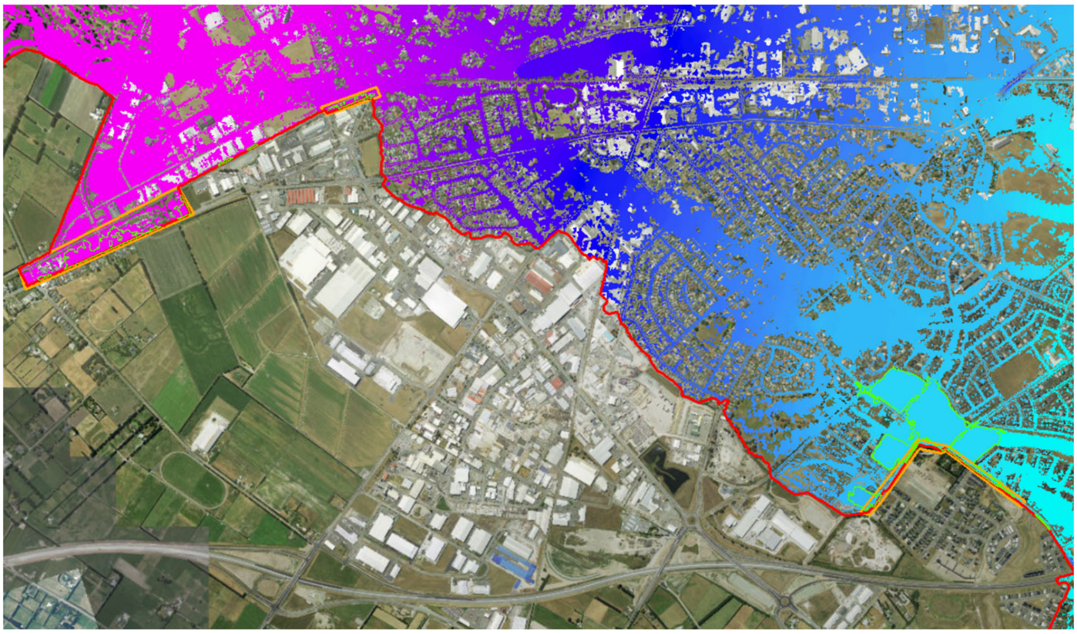

Figure 6-2: Interaction between Heathcote and Halswell boundary

6.3 Model Stability

6.3.1 Model Crashes

When running the final batch of simulations some simulations did not run to completion. Instead of leaving these, some stability fixes were applied which allowed the simulations to complete. The issues occurred in 3 main locations. Two additional versions were updated to allow the remainder of simulations to complete, these were:

V26b

Instability at Wilderness drain, HETH.Wc.heth.wildernd.116, in the MIKE Urban model– invert of open channel as it enters the pipe network, lifted by 0.5m to reduce slope.

The following models were run with version 26b

- HETH_ED2020_R0500ARI_30hr_T050ARI_SLR0p0
- HETH_MPD2060_R0500ARI_12hr_T050ARI_SLR0p5
- HETH_MPD2100_R0050ARI_24hr_T500ARI_SLR1p0
- HETH_MPD2060_R0050ARI_09hr_T007ARI_SLR0p45

V26c

Includes the same fix as in version b but also changed invert of node, Heth.wc.heth.hayton.543 down by 200mm, and increased the height of the CRS channel sides by 5m, to prevent errors regarding the cross section being too deep. Updated ground level of inlet, Heth.inlet.cccgis.12164 to match the LiDAR level (increase by around 2m). Made some adjustments to Popes drain around 565, filled in MIKE 21 level to 11.5 at bank as it was picking up bottom of channel, adjusted lateral link parameters to include smoothing of 0.01 and adjusted cross section slightly to add a small notch.

Update 26c was applied to the following scenarios:

- HETH_MPD2100_R0500ARI_02hr_T050ARI_SLR1p0
- HETH_MPD2100_R0500ARI_06hr_T050ARI_SLR1p0
- HETH_MPD2100_R0500ARI_24hr_T050ARI_SLR1p0

6.3.2 Stability Summary

The model stability was tested for each model simulation. The results from the simulation groups are shown in Table 6-4. The numbers represent the number of individual locations (elements/chainage points/nodes) where an instability occurs in the group of simulations. For example, if an instability occurs at chainage 10 and 20 of a river, but these same locations are unstable in multiple simulations within the group the instability "count" would still be two, because we are focussing on how many locations are unstable within the model. The total represents the total number of locations that are unstable covering all simulations.

The MIKE 21 stability is very good with all oscillations below 100mm. Note that in the 10 year ED2020 simulation the majority of the 2D instability occurs very late in one of the long simulations, so has no impact on the peak water levels.

The MIKE 11 stability is generally good except for a couple of isolated points, the two that occur in the 10 year ED 2020 event are those related to the MIKE 21 instabilities and thus occur well beyond the peak. The remaining instabilities with high level oscillations (>0.5) occur at:

- The structure on Cashbr 695, which is localised, however the instability is impacting on the peak level at this location.
- Bells Creek in the 2hr event, well below the critical water level, and
- The Richardson pump station, where the levels also oscillate due to the pump operation. In only two groups is this impacting the max of max peak level.

The MIKE Urban instabilities are reasonably widespread, with a larger number occurring in the downstream tidal region in the high sea level rise scenarios. Because the MIKE 21 oscillations are all below 100mm it can be considered that where there are instabilities in the MIKE Urban, it should be sufficient to use the MIKE 21 levels instead.

Table 6-4: Model Stability summary for all simulations

Group	1	2	3	4	5	6	7	8	9	10	Total
	10yr ED2020	10yr PD1991	50yr ED2020	50yr MPD2060	50yr PD1991	200yr ED2020	200yr MPDCC16	500yr ED2020	500yr MPD2060	500yr MPD2100	
M21											
0.05-0.1	358[1]	41	95	130	59	121	141	128	191	200	969
M11											
0.15-0.3	31	15	16	15	17	20	22	15	28	25	88
0.3-0.5	9	4	4	4	2	3	3	3	7	4	14
0.5-1.0	4	2	2	2	2	2	1	2	3	1	6
Above 1.0	2	0	0	1	1	1	1	1	1	1	4

[1] High values attributed to instability occurring well after the peak water level has passed, and does not exceed the peak water level, so impact on results is minimal.

MU											
0.015-0.3	186	49	109	62	79	28	414	41	112	270	681
0.3-0.5	24	4	15	7	13	3	64	9	21	47	119
0.5-1.0	2	1	3	0	1	1	2	0	1	22	28

6.3.3 Results comparison to previous modelling

As part of the QA process the model results were compared back to the v16 results, i.e. those produced by AECOM in 2019. Due to significant updates to the model it is not expected that the results will match well, however it is useful to understand where results have changed and to check that these changes are in the expected directions, i.e. water level increases where we would expect it to. The max of max results were compared for the existing development scenarios, for the 10 year, 50 year and 200 year results.

In general the levels have decreased throughout the catchment upstream of Ferniehurst and within Henderson's basin. The exception is the storage management basins which are storing more water, and thus have higher water levels. The lower catchment is generally showing an increase in water levels from the previous modelling. Reasons for these changes come down to a number of factors, the most significant of which are listed below.

- Mass balance errors in the v16 model were removing water from the model. This occurred in areas generally along the Heathcote. Fixing these errors have resulted in increased water levels mainly along the Heathcote River.
- Changing tide level timeseries to use the newest design levels. These new levels have a slightly higher peak, which will result in increased levels in the tidal area.
- Updated 2D infiltration rates from the calibration are slightly higher than the previous rates which will result in a lowering of water levels, generally in the upper catchment.
- New infiltration and groundwater methodology may result in higher water levels (less drainage) in areas with higher groundwater, this includes the tidal area and the Hendersons basin area.
- Active management and upgrade of the storage basins will result in an increased level in the basins with a decreased water level downstream, especially along the Heathcote River.
- Changes to the railway permeability has resulted in a reduction in level upstream of the railway upstream of Curletts Basin.
- In new development areas water levels will change up or down depending on how the surface has changed and what mitigation has been included.

Because many aspects of the model have been changed at once it is not suitable to draw specific conclusions from this analysis. For example it should not be concluded that because levels in the Heathcote have increased downstream that the Upper Basin Active Management scheme is not effective.

7 References

/1/ A.J. Tan, “Heathcote CWM Phase 1 and 2 Updates 2022”, DHI, New Zealand, 08/11/2022

/2/ M. Groves, “Cashmere stream enhancement hydraulic assessment”, WSP, New Zealand, Revision 4, 31/10/2022

/3/ S. Khasanyanova, “Upper Heathcote Storage Optimisation – UHSO Functional Description”, Jacobs, New Zealand, Revision 2, 13/05/2022

/4/ A.J. Tan, “Heathcote CWM Calibration 2022”, DHI, New Zealand, 08/11/2022

/5/ G. van, Rongen,G. Pleijter,T. Preston, “LDRP097 Multi-Hazard Baseline Modelling – Joint Risks of Pluvial and Tidal Flooding”, GHD and HKV, New Zealand, Rev 0, 04/02/2021

/6/ *User Requirements.xlsx* - User requirements deliverables Final Summary 090523.xlsx – Stored on CCC working group WSP SharePoint site