BEFORE THE HEARING PANEL

IN THE MATTER of the Resource Management Act 1991

AND

IN THE MATTER of the Proposed Waikato District Plan

STATEMENT OF EVIDENCE OF CONSTANTINOS FOKIANOS (STORMWATER)

Dated 17 February 2021

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INTRODUCTION

1. My name is Constantinos Fokianos.

2. I hold a Master in Civil Engineering degree from the Democritus University of Thrace, Greece. I also undertook post-graduate studies on Hydraulic Engineering at the same university. I have been working in the water resource engineering field since 2005. I currently hold the position of Water Resource Engineer Manager at Bloxam Burnett & Olliver (**BBO**). I have been working for BBO since 2017. I have participated on a wide range of consulting, design, and modelling services for infrastructure and development projects. I have also provided peer reviewing services for Waikato Regional Council (**WRC**) and Waikato District Council (**WDC**).

3. I have been engaged by Shand Properties Limited (**Shand**) to provide a Stormwater Management Report to support its submission on the Waikato Proposed District Plan (**PDP**).

CODE OF CONDUCT

4. I have read the Environment Court Code of Conduct for expert witnesses contained in the Environment Court Practice Note 2014 and agree to comply with it. I confirm that the opinions expressed in this statement are within my area of expertise except where I state that I have relied on the evidence of other persons. I have not omitted to consider materials or facts known to me that might alter or detract from the opinions I have expressed.

SCOPE OF EVIDENCE

5. My evidence will address the following matters in relation to the Shand submission seeking to rezone land in Huntly North:

a) Hydrology;

b) Flood Regime;

c) Stormwater Drainage and Conveyance;

d) Stormwater Treatment, Attenuation, and Discharge; and

e) Residual Risk.

SUMMARY OF EVIDENCE

6. Through its submission on the PDP, Shand is seeking to rezone approximately 30.5 ha of land located in Huntly North from the current rural zoning to a mix of industrial (approximately 13 ha) and residential (approximately 17.5 ha) zoning.

7. I prepared a Stormwater Management Report to assess the stormwater aspect of the proposed zoning. The report is **Attachment 1** to my evidence. It addresses matters regarding hydrology, flood regime, drainage, conveyance, treatment, attenuation, and discharge. It also refers to the residual risk from flooding due to a potential stopbank breach.

8. The report's purpose is to support the rezoning submission by providing a high-level stormwater management plan/layout for the proposed zones. An overall catchment hydrology investigation was conducted to determine maximum flood levels and the corresponding minimum floor levels for the proposed zones. Appendix B to the Stormwater Management Report gives more information regarding the assumptions and methodology that was followed.

9. Further investigation has been conducted on the proposed industrial area as it is located within the Kimihia catchment floodplain, it is adjacent to the

railway and Great South Road, it is located close to the Kimihia rural stopbank and there are no discharge points located within the boundaries of the proposed industrial area.

10. A preliminary level layout has been developed to provide a solution that addresses these challenges and demonstrates the feasibility of the proposed area to be developed for industrial use while meeting all the criteria related to stormwater management. A residual risk assessment memorandum was prepared to address matters regarding potential flooding due to a breach on the Kimihia stopbank. This memorandum is Appendix D to the Stormwater Management Report.

11. The proposed residential area faces fewer challenges as it is set on higher ground and there is an existing watercourse that can be used as a discharge point for the post-development treated and attenuated runoff. The major challenge is its proximity to an existing wetland and how the development stormwater management layout can be implemented to improve the wetland, especially within the context of the recently updated Resource Management (National Standards for Freshwater) Regulations, 2020.

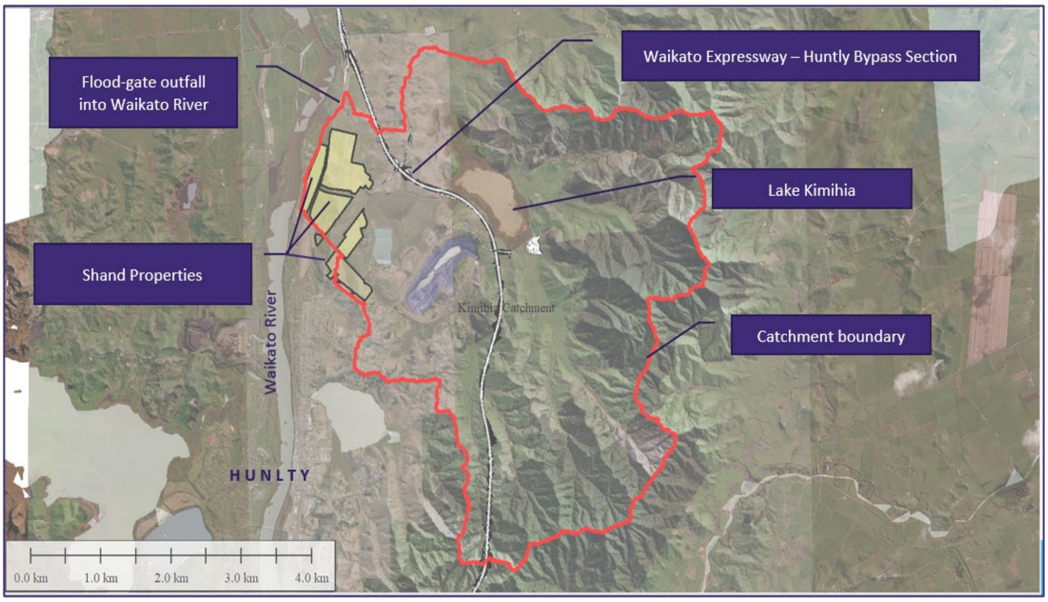
12. Overall, the areas proposed in the plan change are suitable to be zoned for residential and industrial activity. The Stormwater Management Report presents the principles by which the future developments should be configured in terms of stormwater and flood management.

    Detailed investigation of the identified stormwater issues can be conducted at the subdivision stage of the future development, with suitable conditions imposed as part of any subdivision resource consent.

OVERALL HYDROLOGY AND FLOOD REGIME

13. The areas proposed to be rezoned are located within the defended area of the Kimihia Catchment. The overall catchment covers approximately 2,473

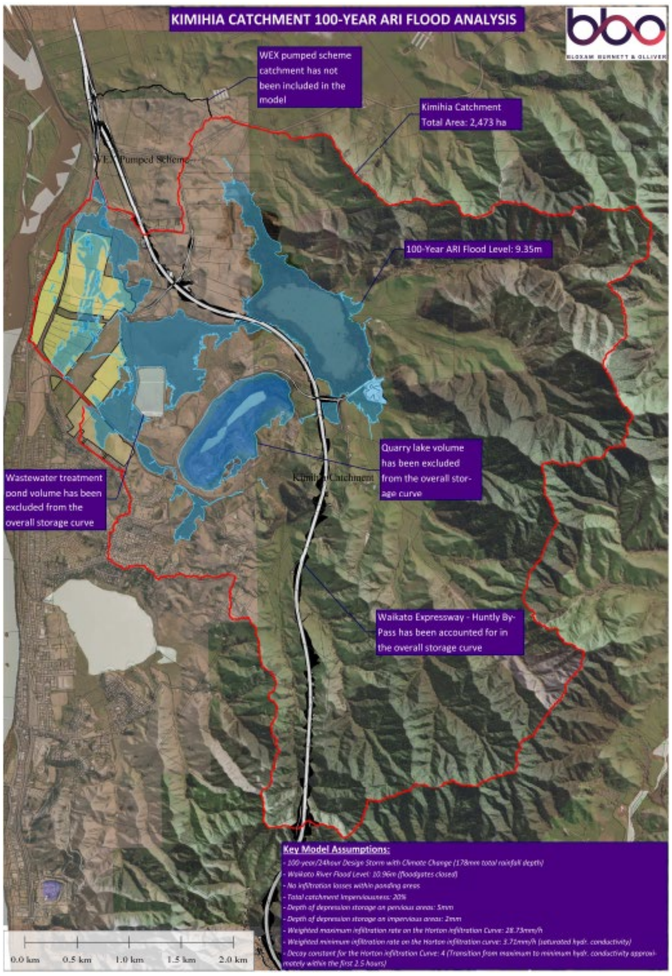
ha. The area falls within the WRC Kimihia Drainage scheme. It is considered defended as there is a stopbank along the Waikato River that keeps it protected during the river's high flows.



14. The overall catchment drains into the Waikato River through a flood gated culvert, allowing the runoff to drain freely during the river's low flows. During higher flow events the floodgates are closed and the runoff from the Kimihia catchment accumulates and ponds upstream of the gates.

15. That scenario could be considered as a conservative, yet safe, approach to determine maximum flood levels for the Kimihia catchment. These flood levels, varying as per design rainfall (2-year, 10-year and 100-year Annual Recurrence Interval (**ARI**)) can be used to set the minimum floor levels for any development within the proposed zones.

16. A hydrological study was carried out by BBO to determine these proposed minimum levels. The study is summarised in a memorandum that was sent to WRC and is Appendix B to the Stormwater Management Report. The hydrological analysis suggests that RL 9.35m could be used as the 100-year ARI flood level for the Kimihia catchment. According to the Regional Infrastructures Technical Specifications (**RITS**), the minimum freeboard between 100-year ARI flood level and the floor level of the industrial areas is 300mm. Hence, the proposed minimum floor level for the industrial zone is RL 9.65m.

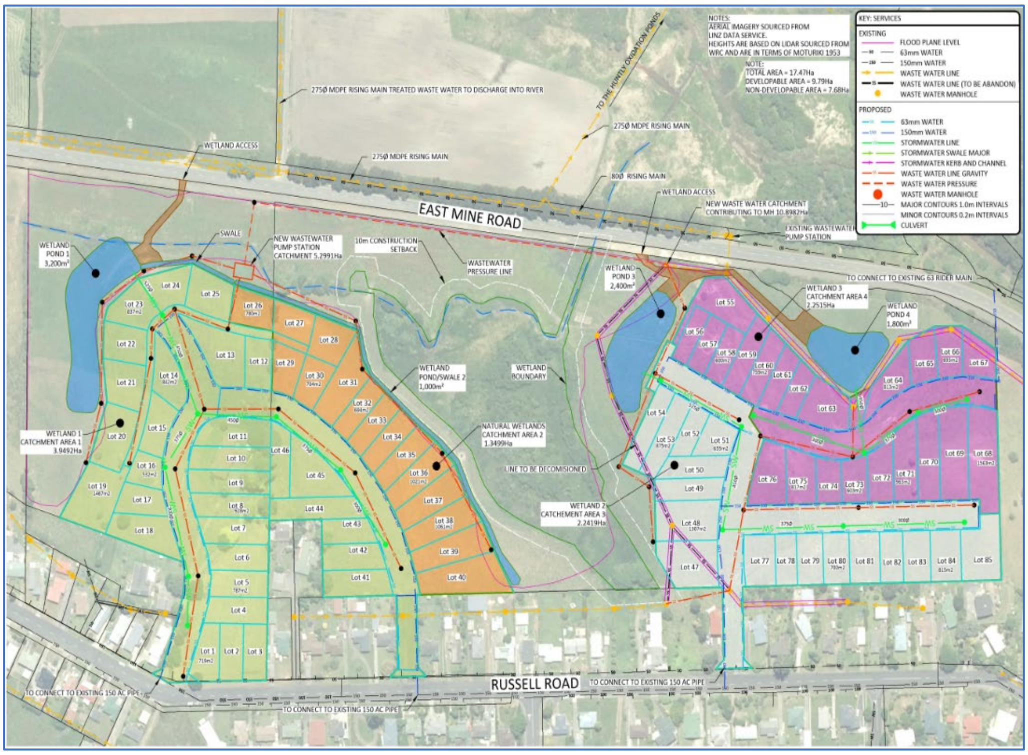


RESIDENTIAL AREA

17. The area proposed to be rezoned residential is currently pastureland. The land is adjacent to the residential area that has already been developed along Russell Road from the south and the East Mine road from the north. The terrain morphology is hilly, with two local high points forming four distinct sub-catchments. In the low-lying area between these two local high grounds, a natural wetland has been formed. According to the Ecological assessment conducted by Boffa Miskell Ltd, the wetland has an area of 1.84

ha and has medium ecological value. Although an analysis of existing wetland water quality has not been conducted, high levels of nutrients, BOD and ammonia is expected to be present due to the current grazing/pasture use of the surrounding area.

18. A high-level stormwater management layout was setup to investigate and demonstrate the feasibility of the proposed area to be rezoned as residential. The ground morphology dictates the delineation of four sub-catchments. A stormwater treatment device has been allocated to each sub-catchment.



19. The runoff from the residential road network is expected to be drained through kerb and channel and captured via catchpits. A stormwater reticulation network will convey the captured runoff to the treatment devices. Some of the lots sheet flow is also expected to be intercepted by the road drainage layout. The rest of the residential area's runoff will be drained in the form of sheet flow towards the lower areas. It is proposed that cut-off drains and/or swales are built on the downstream boundaries of the residential lots to intercept and convey the sheet flow to the

stormwater devices. This will protect the downstream areas against erosion due to the increased flows. The treatment devices will provide extended detention and attenuation of the flows down to pre-development levels to mitigate the effects of the developments and climate change and protect the receiving waters.

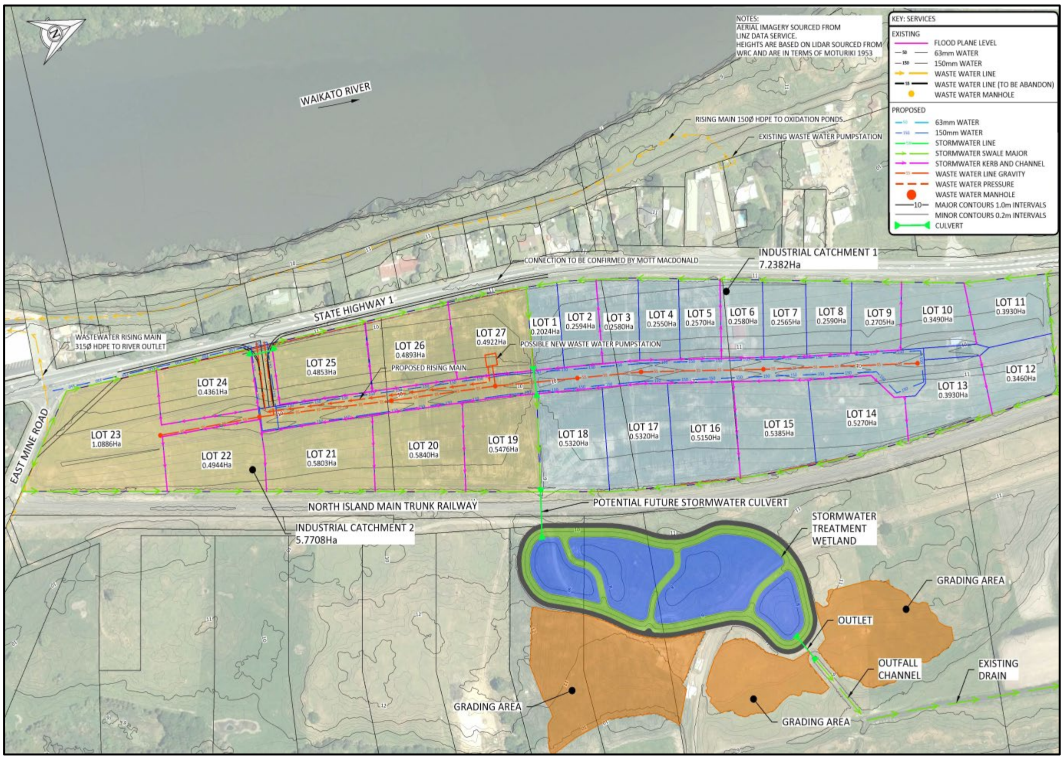
20. Special design will be needed to ensure that the existing wetland continues to receive the water volumes needed to maintain and improve its ecological value. A layout that will allow distribution of part of the treated stormwater back to the wetland in a form that replicates the current sheet flow will have to be considered. This approach could provide significant improvement to the existing wetland as it will protect the wetland against the increase of the runoff due to the development and the climate change.

21. The proposed land use change is also expected to reduce contaminant load into the wetland, especially regarding the levels of BOD, ammonia, and nutrients due to the removal of agricultural land use from the land. The residential nature of the development, along with the limited number of lots will not introduce any significant risk of heavy metal contamination since all of the future impervious runoff will be treated through stormwater treatment devices.

INDUSTRIAL AREA

22. The area proposed to be zoned industrial under its current status is also pasture/farmland. The area is delineated by old SH1 (now Great South Road) to the west, the railway to the east, East Mine Road to the south and another rural property to the north. The area is almost flat, with a small gradient towards the north. A local depression drains the surface runoff towards the north where, according to WRC input, there is a culvert that crosses the railway and discharges into the existing rural drainage network of Kimihia catchment.

23. A specimen design level layout was developed to demonstrate feasibility of the area to be developed/utilised for industrial purposes while meeting the district plan, the regional and the national rules and requirements for stormwater management.



24. The main challenge that the industrial zoning faces regarding stormwater is the availability of a discharge point. In its current condition, the area is flat, and a local depression forces it to drain towards the north. Currently there is no identified watercourse of concentrated flow such as a draining channel or stream. The runoff eventually drains across the railway through a culvert that is located further north, and away from the zoning boundaries.

25. An option of a new culvert across the railway has been considered. Both of the culvert ends (inlet/outlet) will be located within the proposed zoning. The proposed option also allows for the stormwater treatment and attenuation device to be located on the eastern side of the railway, allowing for the industrial area layout to be optimised. The treated and

attenuated flows can then be discharged into the existing Kimihia rural drainage network through a new channel. The discharge point and the connection channel are both located within property owned by Shand.

26. A preliminary terrain model was formed for the future development. The drainage layout is based on a network of peripheral swales surrounding the industrial development draining towards the proposed culvert under the railway. Each industrial lot can then be graded towards the swales forming a crown at the middle of the development. The internal roading could be adjusted to the crown to provide access to the future lots. The road runoff would be drained via kerb and channel and then through shallow channels along the boundaries of the lots towards the peripheral swales.

27. The stormwater swales could be planted to provide preliminary treatment and to enable a “treatment train” layout as promoted by WRC guidelines. Due to the low gradient, the swales would also function as buffer swales during high design events, adding further attenuation volume to the overall stormwater layout.

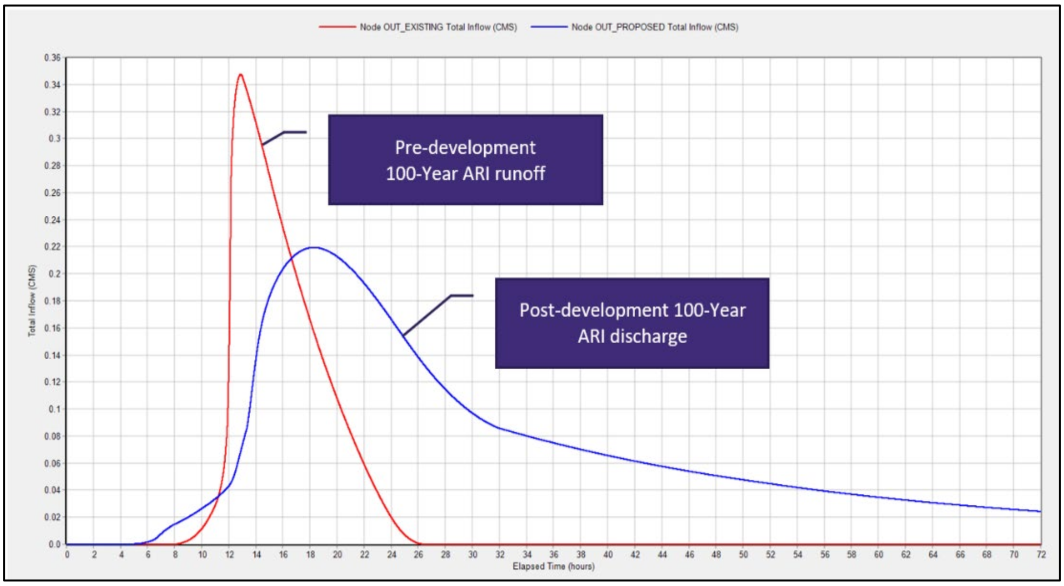
28. The proposed culvert under the railway (preliminary sized to be 1050mm diameter) could discharge into a stormwater treatment device. For the purposes of the report and taking into account the preliminary/specimen level of the design, a stormwater treatment and attenuation wetland is proposed. According to WRC stormwater guidelines, the RITS and other national technical documentation, stormwater wetlands are considered amongst the most efficient water quality treatment and attenuation devices.

29. The wetland shown on the drawings has been sized to attenuate future stormwater flows down to pre-development flows. Especially for the 100-year event, the proposed wetland attenuates discharge down to less than 80% of the pre-development flow. Extended detention was also accounted for in the design to reduce downstream erosion risk. The controlled flow

could then be discharged into the existing rural drainage network through a proposed outfall channel.

30. A stormwater model was built to evaluate the proposed layout. Design rainfall events for Water Quality (1/3rd of the two year/24hour rainfall), 2-year, 10-year and 100-year were run, all for 24-hour duration storms. The design rainfalls were also adjusted to climate change and a 2.1°C temperature rise was accounted for. The model also included a layout of the pre-development conditions in order to set the target flows for attenuation.

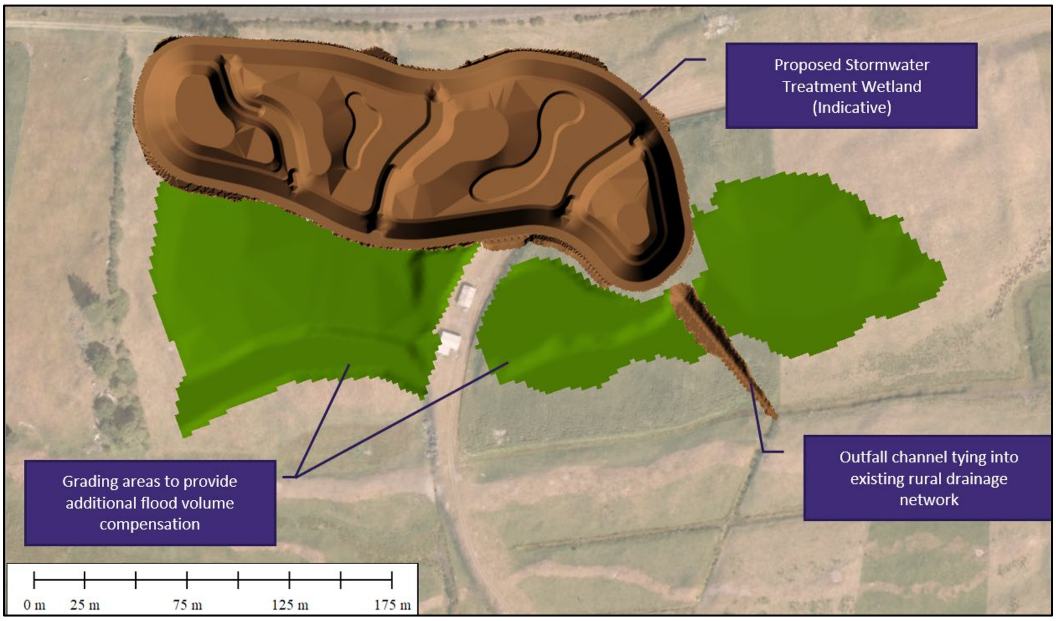
31. The modelling output suggests that the proposed stormwater layout can drain, treat, and discharge runoff from the future development in a controlled manner that meets WRC and national criteria regarding stormwater quality and quantity.



32. A direct ‘level for level’ scheme is proposed for the industrial rezoning whereby level for level and volume for volume compensation is provided to replicate ponding volumes lost by the development, such that at least the same volume is available at every flood level and ponded water can freely access (fill and drain) as currently occurs. In other words, in order to

mirror the existing situation for a flood, each stage or level is provided with at least the same storage volume.

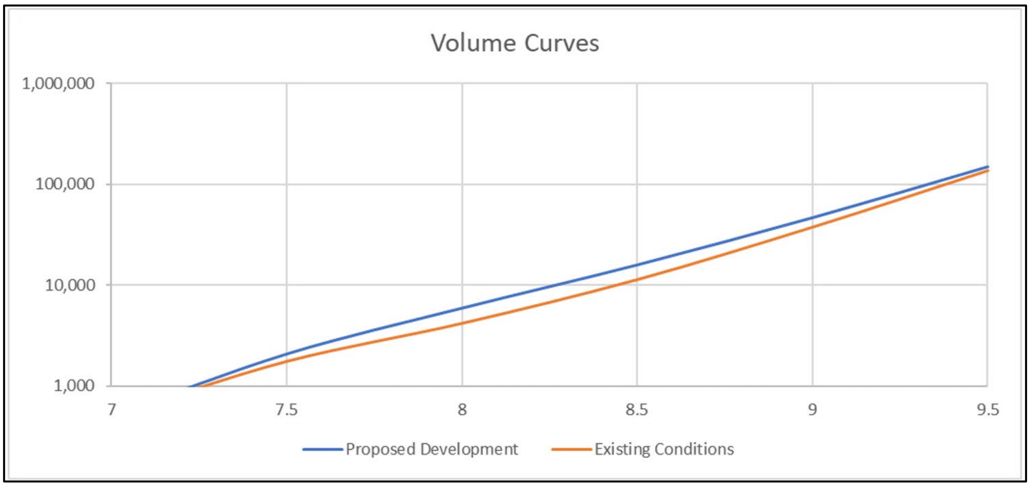
33. This will be achieved, by recontouring an area of the submitter's property east of the railway line to provide the stormwater treatment and attenuation wetland. Some additional recontouring outside the wetland could be needed, depending on the overall design. On the indicative scheme level design that has been applied for the needs of this report, some additional areas of grading were needed to satisfy the level for level flood compensation.



34. The compensatory volume must be at the same level as the lost storage. In general, level for level compensation should only be applied in areas where flood water is stored; and flood flow routes should be protected as is the case for the proposed zoning as it is located within the Kimihia floodplain.

35. The figure below shows the existing cumulative volume curve of Areas 1 and 2 of Shand in their current condition, and the volume curve after the proposed indicative works for the development (earthworks within Area 1, treatment wetland, and additional grading). The graph shows that the proposed layout is introducing volumes are above existing up to the 100-

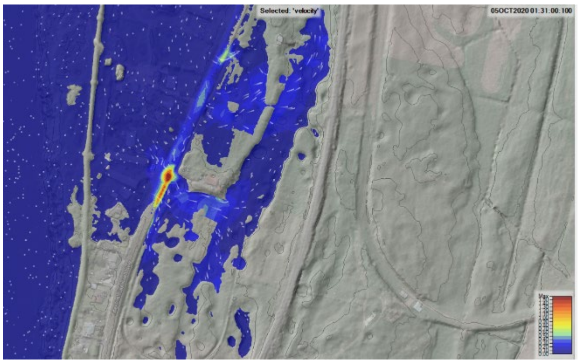
year level (9.35mRL). It therefore provides more flood storage then the existing conditions, which is an overall improvement for the Kimihia floodplain capacity, and it also provides contingency and flexibility for any changes to site layout during detailed design.

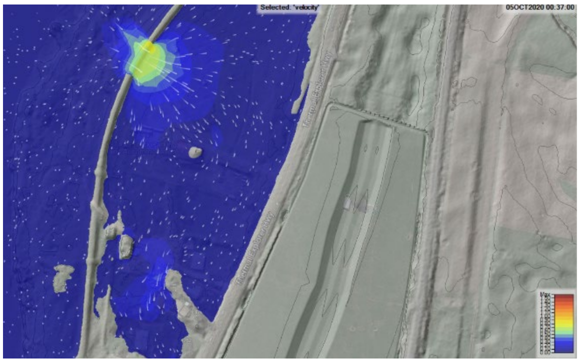


RESIDUAL RISK

36. The proposed industrial zone is within a defended area - defended from river flooding by stop banks and localised flooding by floodgates. Although this area is defended from river flooding up to the 100-year event, it still has the potential to flood. The site may flood due to a larger than design event, including the 100-year with climate change whereby the Waikato River may overtop the stop bank. The area may also flood due to failure of the stop bank defence. These scenarios are unlikely but can still happen. This risk that remains - once a defence is in place - is known as "residual risk".

37. A residual risk assessment was undertaken for this rezoning proposal. The assessment included modelling of Waikato river stop bank breach for both the existing and proposed conditions. The locations of the breach were defined in coordination with WRC. Two-dimensional hydraulic modelling was conducted to assess the velocity and the depth of the flood wave as it spreads for the breach into the floodplain area.





38. The hydraulic analysis and modelling of the stopbank breach scenarios show that there is enough time for an emergency plan to be implemented on staff evacuation if there is a monitoring and warning system in place for the stopbank. The proposed industrial development earthworks and stormwater infrastructure contribute to allowing more time for a reaction, adding more time for the flood wave to reach the internal access road. In case a breach occurs close to the existing Kimihia floodgate, the proposed stormwater layout within the industrial area could protect it from flooding by routing the flood wave towards the proposed wetland, providing full protection against flooding for at least 36 hours.

CONCLUSION

39. Shand proposes to rezone two separate areas, one for residential and one for industrial use. Both of these areas are located within the Kimihia

catchment, which is a defended area. The entire industrial area and part of the residential zone are also within the Kimihia floodplain.

40. A hydrological analysis was undertaken to determine the maximum flood levels of the Kimihia catchment during 100-year ARI climate-adjusted design storm with the floodgates considered closed. The analysis indicated that RL 9.35m is the maximum water surface level. That has been proposed to be used to set the minimum floor levels for the proposed industrial zoning (RL9.65m). The Operative District Plan shows RL 10.30m as the 100-year flood level at the area of the proposed residential zone. This level is proposed to determine the minimum floor level for the residential rezoning (RL 10.80m).

41. Initial Scheme level design shows that the residential zone can be serviced by several stormwater treatment devices and the layout includes reticulation, swales and cut-off drains. The existing wetland would be protected, and the quality of the stormwater runoff would be improved due to the proposed treatment devices. A discharge distribution layout should be introduced during detail design to ensure that the wetland will be receiving the base flows in a manner that replicates the existing situation.

42. The industrial zoning stormwater management solution includes a centralised treatment and attenuation device located on the eastern side of the railway. A new 1050mm diameter culvert under the railway has been proposed to allow for the runoff from the development to discharge into the proposed treatment device. The drainage layout of the industrial area would consist of stormwater swales that would add treatment and attenuation properties into the overall layout and qualify as a treatment train approach.

43. The proposed layout has been modelled and shows the capability to attenuate the development flows down or even lower than the pre-

development runoff. The treated and attenuated runoff will be discharged into the existing rural Kimihia drainage network through a proposed channel. Level for level flood volume compensation has been taken into account in the indicative design of the proposed layout, ensuring that the development will not reduce the flood storage capacity of the overall defended area.

44. Residual risk assessment was carried out including two-dimensional river stopbank breach modelling. The results show that a proper emergency evacuation plan can be established and implemented for the proposed industrial area.

45. The above conclusions indicate that the proposed rezoning can be serviced within the local, regional, and national requirements regarding stormwater management.

Constantinos Fokianos

17 February 2021

Attachment 1

Stormwater Management Report

Shand Properties Ltd

Huntly North Rezoning

Stormwater Management Report

2 November 2020

BLOXAM BURNETT & OLLIVER

Document control

Project identification		
Client	Shand Properties Ltd	
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Date / period ending	2/11/2020	
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Checked by	Chris Dawson		6 / 11/ 2020
Approved for issue	Jarred Stent		

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Appendix C – Industrial Zoning SWMM Model Output
Appendix D – Memorandum on Residual Risk assessment & River stopbank breach analysis

1. Introduction

BBO has been engaged by Shand Properties Limited (Shand) to support their submissions to the Proposed Waikato District Plan (PWDP). Shand are seeking to re-zone approximately 30.5 ha of land located in Huntly North from the current rural zoning to a mix of industrial (approximately 13 ha) and residential (approximately 17.5 ha) zoning.

As part of the services, BBO was engaged to produce a 3 Waters Report for the proposed plan change. This report refers to the stormwater aspect of the proposed zoning. It addresses matters regarding hydrology, flood regime, drainage, conveyance, treatment, attenuation and discharge.

The report's purpose is to support the rezoning submission by providing a high-level stormwater management plan/layout for the proposed zones. Further investigation has been conducted on the proposed industrial area as it is located within the Kimihia catchment floodplain, it neighbours the railway and Great South Road, it is located close to the Kimihia rural stopbank, and there are no discharge points located within the boundaries of the proposed industrial area. A preliminary level layout has been developed to provide a solution that addresses these challenges and demonstrates the feasibility of the proposed area to be developed for industrial use while meeting all the criteria related to stormwater management.

The proposed residential area faces fewer challenges as it is set on higher ground and there is an existing watercourse that can be used as a discharge point for the post-development treated and attenuated runoff. The major challenge is its proximity to an existing wetland and how the development stormwater management layout can be implemented to improve the wetland, especially within the context of the recently updated Resource Management (National Standards for Freshwater) Regulations, 2020.

Overall, the areas proposed in the plan change are suitable to be zoned for residential and industrial activity. This report presents the principles by which the future developments should be configured in terms of stormwater and flood management.

2. Hydrology & Flood Levels

The proposed rezoning areas are located within the defended area of the Kimihia Catchment. The overall catchment covers approximately 2,473 hectares (ha). The area falls within WRC Kimihia Drainage scheme. It is considered defended as there is a stopbank along the Waikato River that keeps it protected during the river's high flows.

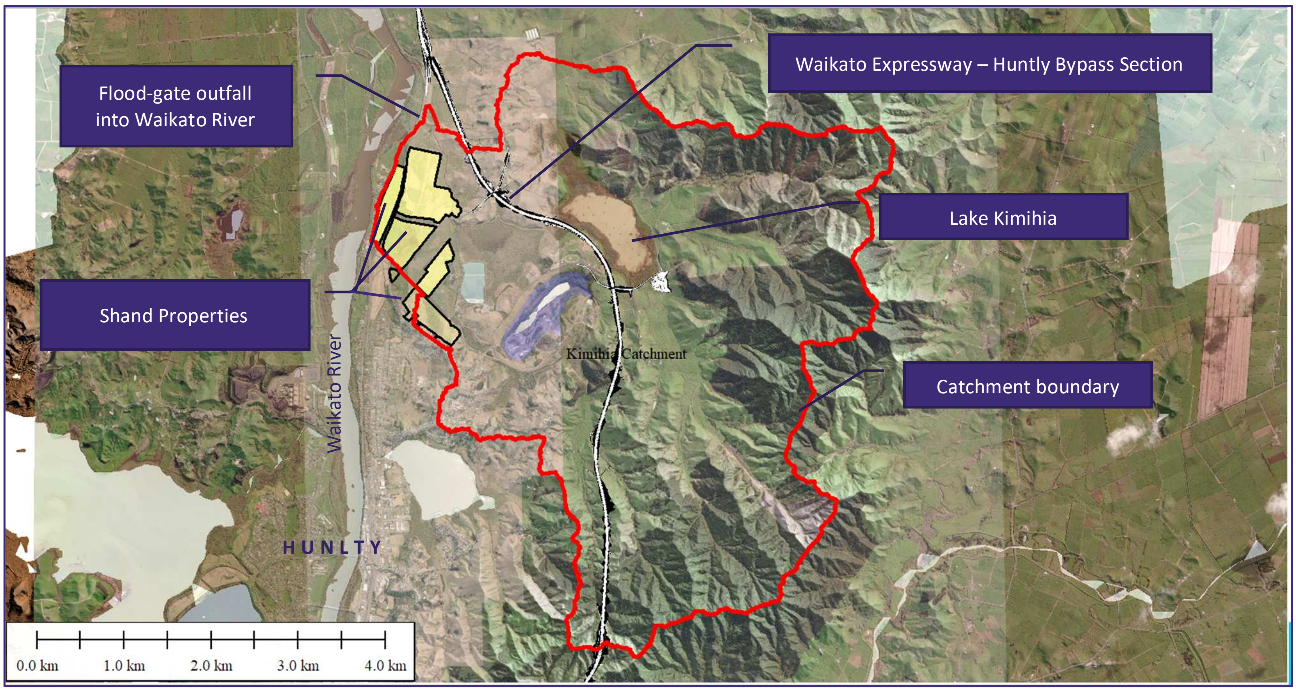


Figure 2.1 Overview of the Kimihia Catchment.

The overall catchment drains into the Waikato river through a flood gated culvert, allowing the runoff to drain freely during the river's low flows. During higher flow events the floodgates are closed and the runoff from the Kimihia catchment accumulates and ponds upstream of the gates.

Figure 2.2 *Flood-gated culvert outfall of Kimihia stream into Waikato River.*

That scenario could be considered as a conservative, yet safe approach to determine maximum flood levels for the Kimihia catchment. These flood levels, varying as per design rainfall (2-year, 10-year and 100-year Annual Recurrence Interval - ARI) can be used to set the minimum floor levels for any development within the proposed zones.

A hydrological study was carried out by BBO to determine these proposed minimum levels. The study is summarised in a memorandum that was sent to WRC and included as **Appendix B** of this report. The hydrological analysis suggests that RL 9.35m could be used as the 100-year ARI flood level for the Kimihia catchment. According to the Regional Infrastructures Technical Specifications (RITS), the minimum freeboard between 100-year ARI flood level and the floor level of the industrial areas is 300mm. Hence, the proposed minimum floor level for the industrial zone is RL 9.65m.

The residential zone is located in an area where design flood levels have already been established by the Operative District Plan, it is therefore proposed that RL 10.3m is used as the 100-year ARI flood level, as shown on Figure 2.2 below. For residential buildings, RITS mandate 500mm of freeboard, hence RL 10.8m should be used as the minimum floor level for the proposed residential zone.

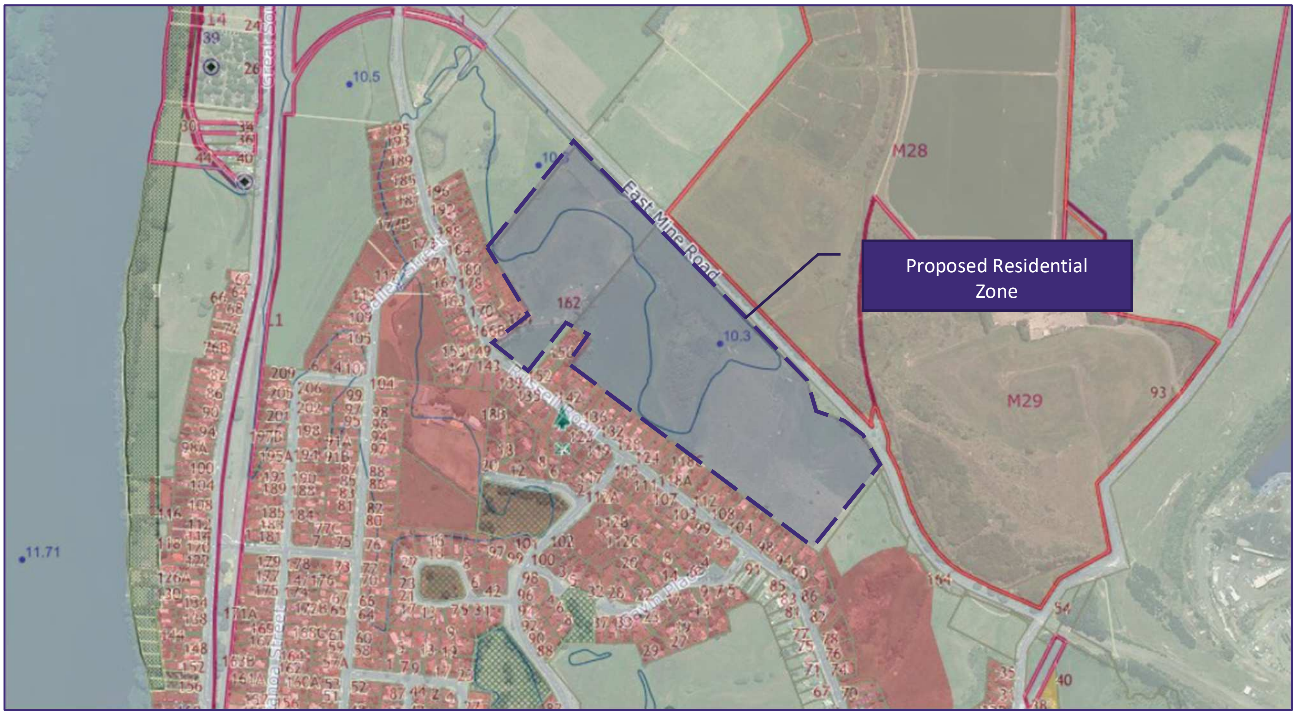


Figure 2.2 *Abstract from the Operative District Plan. Light blue lines represent flood boundaries. The spot elevations represent the 100-year ARI flood level. Dashed dark blue line represents the boundaries of the proposed Shand Residential Zone.*

3. Residential Area

3.1 Existing Conditions

The proposed residential zone is currently pastureland. There is only one residence currently in place (162 Russell Road). The land is adjacent to the residential area that has already been developed along Russell road from the south and the East Mine road from the north.

The terrain morphology is hilly, with two local high points forming four distinct sub-catchments. In the low-lying area between these two local high grounds, a natural wetland has been formed. According to the Ecological assessment conducted by Boffa Miskell Ltd, the wetland has an area of 1.84 ha and has medium ecological value.

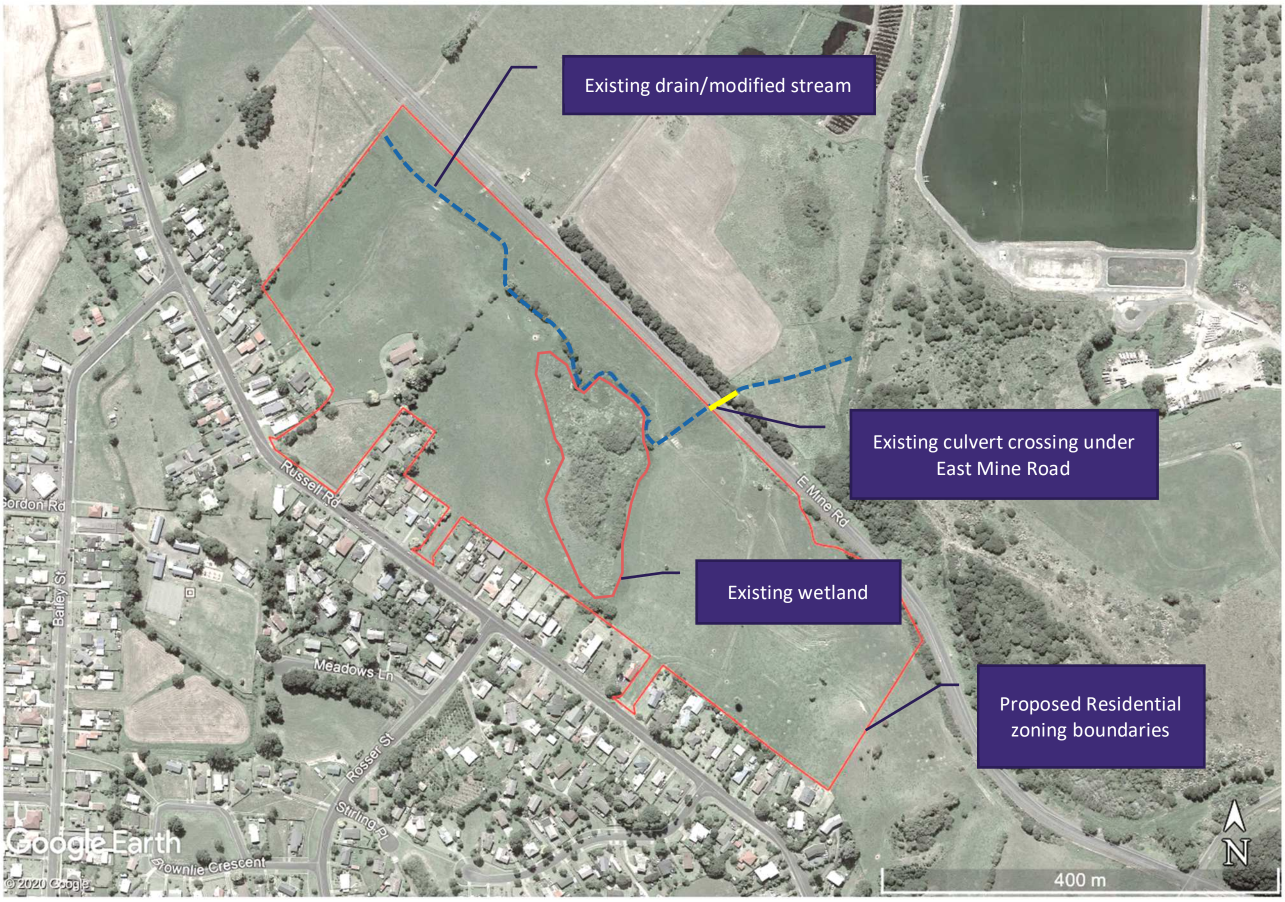


Figure 3.1 *Proposed Residential Zone Area in its current condition. Aerial image from Google Erath.*

An artificial drain (or modified stream) running along the northern boundary of the property delineates the existing wetland. The drain then crosses East Mine Road through a culvert and discharges into the Kimihia drainage network. The rain runoff is considered to form sheet flow and eventually drain through the wetland and/or the existing drain, as there are no other distinct streams, drains or any other accumulated flow patterns visible.

The predominant soil textures are clay and peat and this correlates well with the existence of the wetland in the low-lying area. Runoff coefficient is considered to be high as the soil textures indicate low infiltration rates. Although an analysis of existing wetland water quality has not been conducted, high levels of nutrients, BOD and ammonia is expected to be present due to the current grazing/pasture use of the surrounding area.

3.2 Proposed Indicative Stormwater Layout

A high-level stormwater management layout was setup to investigate and demonstrate the feasibility of the proposed area to be re-zoned as residential. **Drawing 0702** presents on overview of the potential stormwater layout.

As mentioned earlier in this report, the ground morphology dictates the delineation of four sub-catchments. A stormwater treatment device has been allocated to each sub-catchment. Three stormwater treatment wetlands and one stormwater treatment wetland swale are proposed to treat and attenuate the runoff from the future development. The devices have been sized to cater for up to 65% imperviousness of the future developed sub-catchments, according to RITS. The wetland swale has been proposed for Sub-catchment 3 as there are space restrictions that do not allow for a treatment wetland to be deployed.

Taking into account the design flood level shown on the Operative Plan and the requirement that no flood volume should be displaced from the floodplain, the residential development will only take place in that area above the 10.3m elevation level, with the dwelling levels to be at level 10.8m or above. These level restrictions place the development lots on the hilly area of the property. The runoff from the residential road network is expected to be drained through kerb and channel and captured via catchpits. A stormwater reticulation network will convey the captured runoff to the treatment devices. Some of the lots sheet flow is also expected to be intercepted by the road drainage layout. The rest of the residential area's runoff will be drained in the form of sheet flow towards the lower areas.

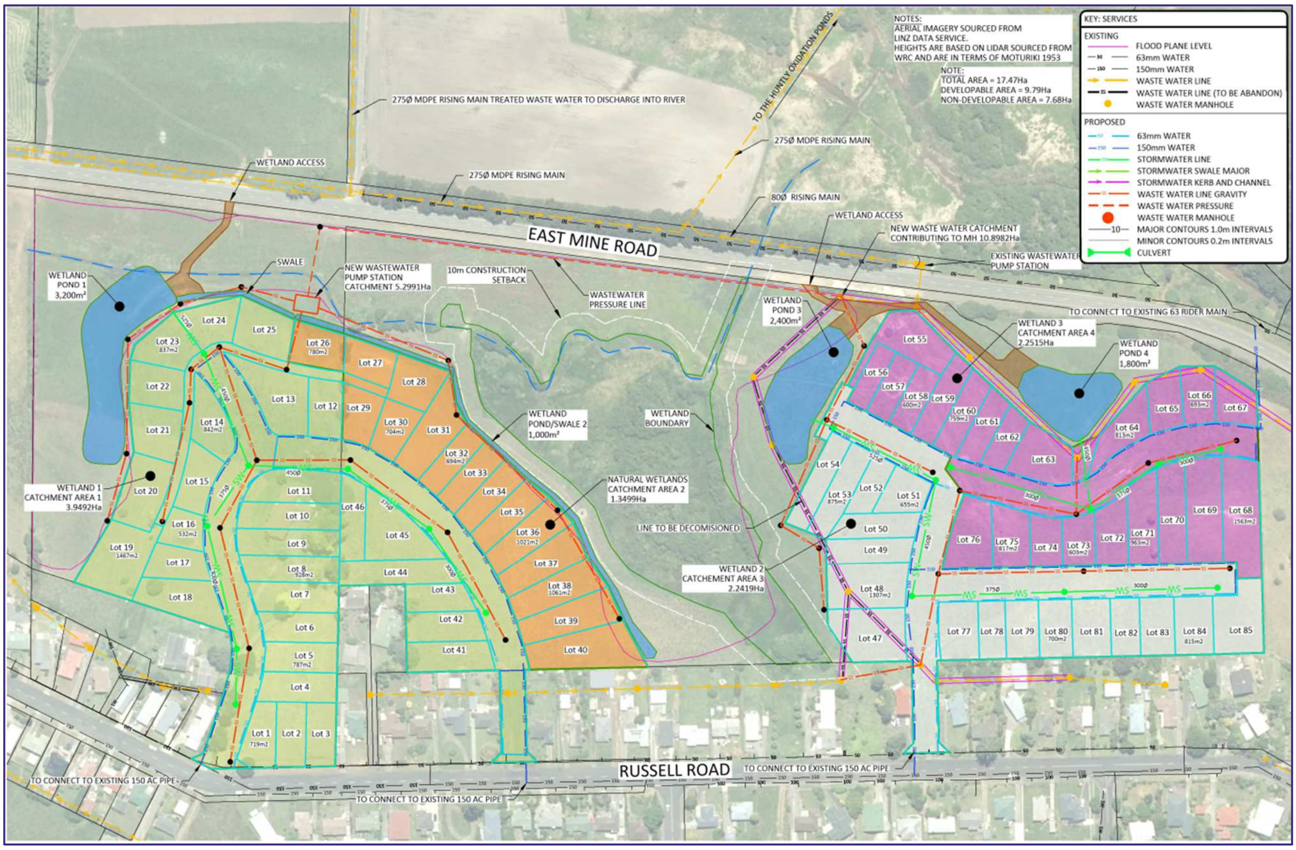


Figure 3.2 *Proposed Residential Zone layout. Abstract from drawing 144370-02-0702.*

The overall stormwater runoff is expected to rise due to the introduction of the impervious, residential areas of the development and the expected climate change. It is therefore proposed that cut-off drains and/or swales are built on the downstream boundaries of the residential lots to intercept and convey the sheet flow to the stormwater devices. This will protect the downstream areas against erosion due to the increased flows. The treatment devices will provide extended detention and attenuation of the flows down to pre-development levels to mitigate the effects of the developments and climate change and protect the receiving waters.

Special design will be needed to ensure that the existing wetland continues to receive the water volumes needed to maintain and improve its ecological value. In its existing condition, the wetland's water intake is realized mainly through sheet flow rather than seepage, as no springs were identified by the ecological assessment. Additional onsite information will have to be obtained during the detailed design of the development to identify the wetland's water balance. A layout that will allow distribution of part of the treated stormwater back to the wetland in a form that replicates the current sheet flow will have to considered.

An initial approach would be a network of shallow swales/drains that will discharge low flows into multiple locations around the wetland. During higher rainfall events, most of the runoff will be released into the existing drain downstream of the wetland. The proposed approach is considered to provide significant improvement to the existing wetland as it will provide higher water quality and protect the wetland against the increase of the runoff due to the development and the climate change. The proposed land use change is also expected to reduce contaminant load into the wetland, especially regarding the levels of BOD, ammonia, and nutrients due to the removal of agricultural land use from the land. The residential nature of the development, along with the limited number of lots will not introduce any significant risk of heavy metal contamination since all of the future impervious runoff will be treated through stormwater treatment devices.

4. Industrial Area

4.1 Existing Conditions

The proposed industrial zone under its current status is also pasture/farmland. There is only one residence currently in place. The area is delineated by old SH1 (now Great South Road) to the west, the railway to the east, East Mine Road to the south and another rural property to the north.

Figure 4.1 *Proposed Industrial Zone Area in its current condition. Aerial image from Google Erath.*

The area is almost flat, with a small gradient towards the north. A local depression, possible a sign of an ancient waterway, drains the surface runoff towards the north where, according to WRC input, there is a culvert that crosses the railway and discharges into the existing rural drainage network of Kimihia catchment.

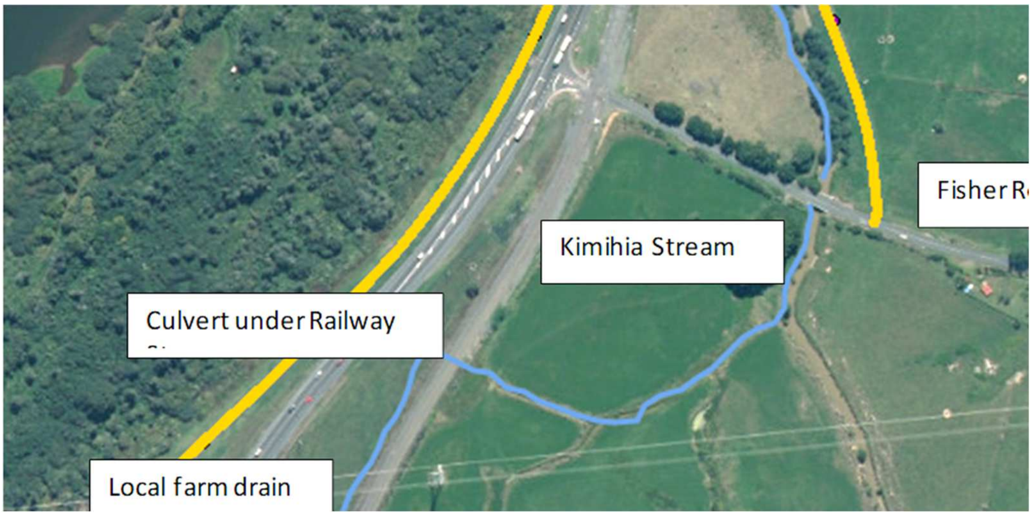


Figure 4.2 *Current draining pattern of Area1. Abstract from WRC correspondence.*

The predominant soil texture is loam over sand and belongs to the pumice soil order of the New Zealand soil classification. There are multiple local depressions located throughout the entire property indicating the existence of sites of archaeological interest within the proposed zone. No permanent water has been identified within these depressions verifying that, overall, the site is at least moderately drained and that the predominant soils allow for water to infiltrate.

4.2 Proposed Stormwater Layout

Part of the stormwater assessment is to demonstrate the feasibility of the area to be developed/utilised for industrial purposes while meeting the district plan, the regional and the national rules and requirements for stormwater management. A specimen design level layout was developed to provide this verification.

The main challenge that the industrial zoning faces regarding stormwater is the availability of receiving waters or point of discharge. In its current condition, the area is flat, and a local depression forces it to drain towards the north. Currently there is no identified watercourse of concentrated flow such as a draining channel or stream. The runoff eventually drains across the railway through a culvert that is located further north, and away from the zoning boundaries.

An option of a new culvert across the railway has been considered. Both of the culvert ends (inlet/outlet) will be located within the proposed zoning. The proposed option also allows for the stormwater treatment and attenuation device to be located on the eastern side of the railway, allowing for the industrial area layout to be optimised. The treated and attenuated flows can then be discharged into the existing Kimihia rural drainage network through a new channel. The discharge point and the connection channel are both located within property owned by Shand Properties Ltd.

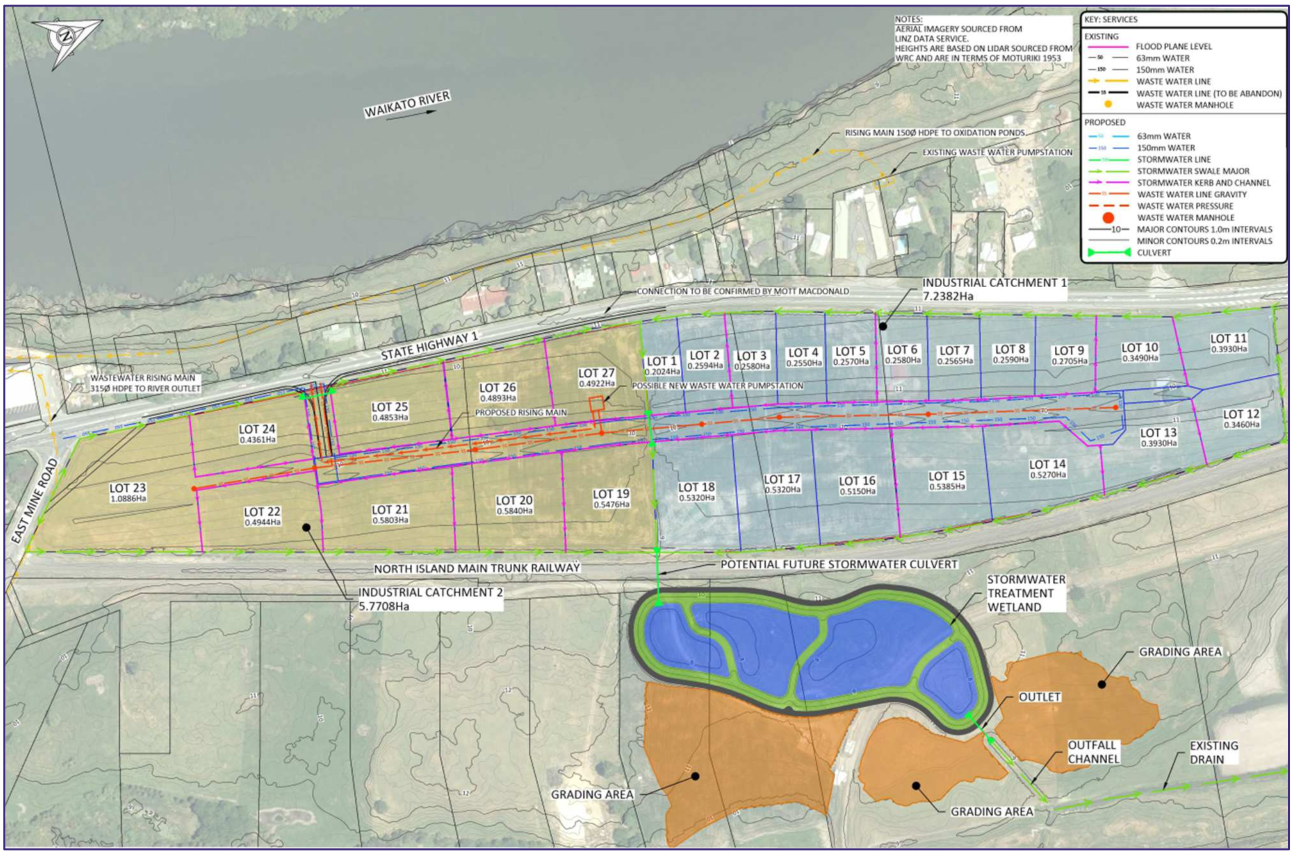


Figure 4.3 Proposed stormwater management layout for the development of Area 1. Abstract from drawing 144370-02-0704.

The layout was examined in relation to vertical constraints. The invert level of the rural drain at the discharge point, and the level of the railway track along with the minimum cover requirements below the railway line were considered to determine whether there is sufficient gradient to allow for stormwater flows to be drained from the proposed industrial area across the railway and into the existing rural drainage network. Based on the WRC LIDAR information, the proposed option is feasible.

Based on the vertical constraints mentioned above, and in combination with the minimum floor levels derived from the overall Kimihia catchment analysis, a preliminary terrain model was formed for the future development. The drainage layout is based on a network of peripheral swales surrounding the industrial development draining towards the proposed culvert under the railway. Each industrial lot can then be graded towards the swales forming a crown at the middle of the development. The internal roading could be adjusted to the crown to provide access to the future lots. The road runoff would be drained via kerb and channel and then through shallow channels along the boundaries of the lots towards the peripheral swales. The stormwater swales could be planted to provide preliminary treatment and to enable a “treatment train” layout as promoted by WRC guidelines. Planted swales are also easier to maintain when compared to grassed swales. On-lot gross-pollutant traps are recommended to further reduce maintenance requirements for the swales. Due to the low gradient, the swales would also function as buffer swales during high design events, adding further attenuation volume to the overall stormwater layout.

The proposed culvert under the railway (preliminary sized to be 1050mm diameter) could discharge into a stormwater treatment device. For the purposes of the report and taking into account the preliminary/specimen level of the design, a stormwater treatment and attenuation wetland is proposed. According to WRC stormwater guidelines, the RITS and other national technical documentation, stormwater wetlands are considered amongst the most efficient water quality treatment and attenuation devices. The wetland shown on the drawings has been sized to attenuate future flows down to pre-development flows. Especially for the 100-year event, the proposed wetland attenuates discharge down to less than the 80% of the pre-development flow. Extended detention was also accounted for in the design to reduce downstream erosion risk. The controlled flow could then be discharged into the existing rural drainage network through a proposed outfall channel.

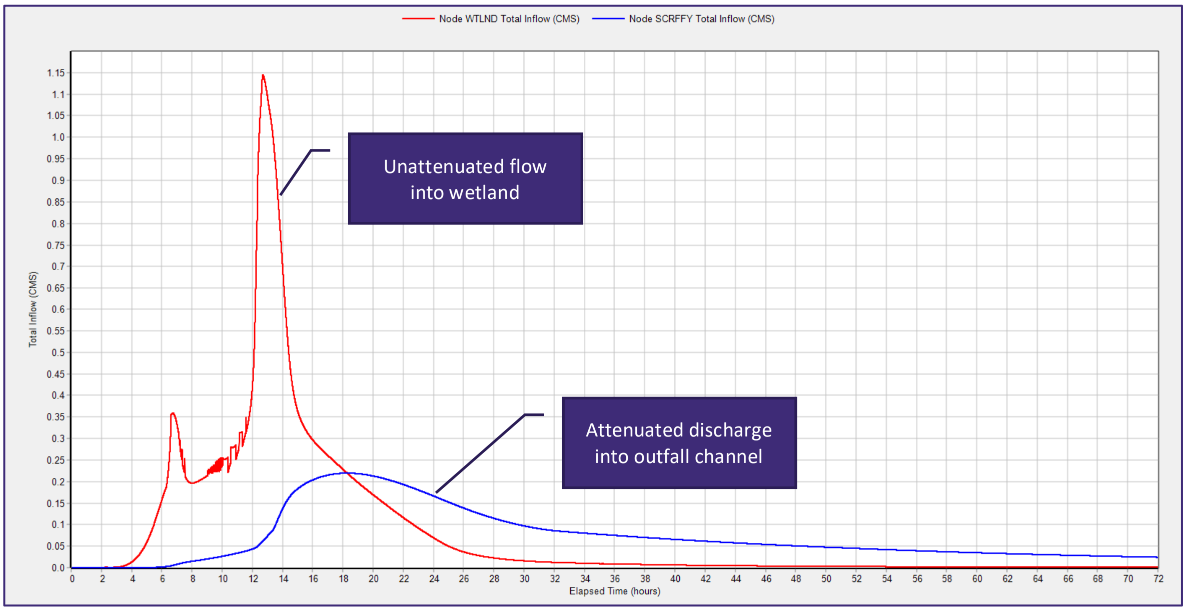


Figure 4.4 *Proposed Wetland inflow/outflow diagram, demonstrating the attenuation properties of the proposed wetland.*

A SWMM model was built to evaluate the proposed layout. Design rainfall events for Water Quality (1/3rd of the two year/24hour rainfall), 2-year, 10-year and 100-year were run, all for 24-hour duration storms. The design rainfalls were also adjusted to climate change and a 2.1°C temperature rise was accounted for. The model also included a layout of the pre-development conditions in order to set the target flows for attenuation. The design storms for the pre-development model did not include climate adjustment.

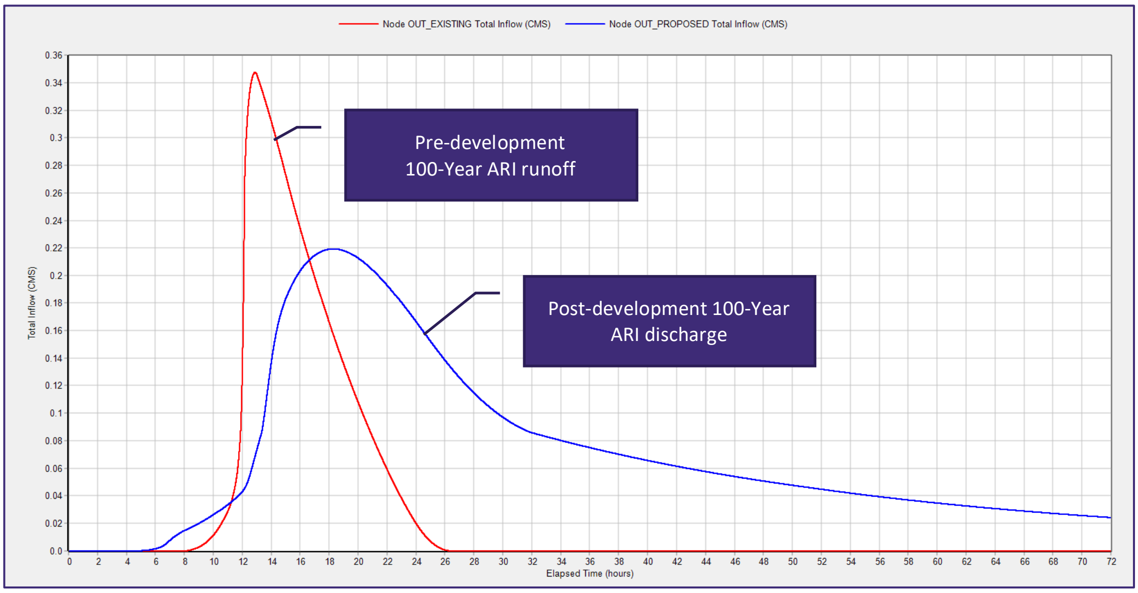


Figure 4.5 *Comparison graph of pre- and post-development flows.*

Two different downstream boundary conditions were considered: one of a normal flow discharge, and one where the downstream floodgates are closed and flood levels have reached the 100-year water surface level of RL 9.35m. The proposed stormwater layout was then sized and adjusted to cater for these conditions.

The modelling output suggests that the proposed stormwater layout can drain, treat, and discharge runoff from the future development in a controlled manner that meets WRC and national criteria regarding stormwater quality and quantity.

4.3 Flood management

In general, flood storage compensation works can be divided into direct and indirect. These terms come from UK CIRIA report C624 "Development and flood risk – guidance for the construction industry (2004)".

Direct or 'level for level' schemes re-grade the land at the same level as that taken up by the development. Direct schemes therefore provide a direct replacement for the lost storage volume. Indirect methods rely on water entering a new storage area via culvert or engineered structure and can be some distance from the infill area. Indirect schemes are less preferred because they are more vulnerable to failure.

A direct scheme is proposed for the industrial rezoning whereby level for level and volume for volume compensation is provided to replicate ponding volumes lost by the development, such that at least the same volume is available at every flood level and ponded water can freely access (fill and drain) as currently occurs. In other words, in order to mirror the existing situation for a flood, each stage or level is provided with at least the same storage volume.

This will be achieved by recontouring the site east of the railway line to provide the stormwater treatment and attenuation wetland. Some additional recontouring outside the wetland could be needed, depending on the overall design. On the indicative scheme level design that has been applied for the needs of this report, some additional areas of grading were needed to satisfy the level for level flood compensation.

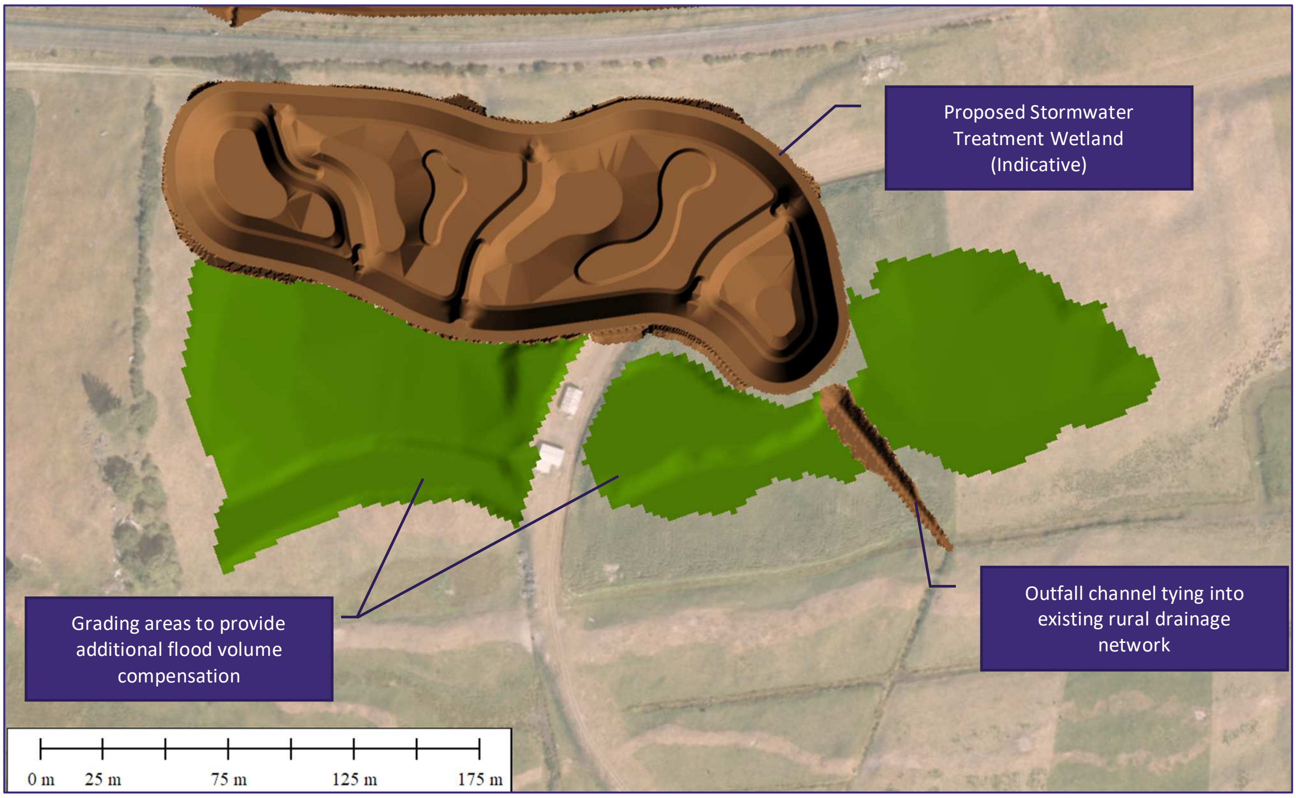


Figure 4.6 *Indicative wetland and additional grading to satisfy level for level flood volume compensation.*

The compensatory volume must be at the same level as the lost storage. In general, level for level compensation should only be applied in areas where flood water is stored; and flood flow routes should be protected as is the case for the proposed zoning as it is located within the Kimihia floodplain.

An earthworks map was produced using software Global Mapper, to provide more information on the cut and fill works that will be needed for the grading of the proposed rezoning area. The map is indicative and based on the scheme level design that was conducted for the needs of this report. Figure 4.7 shows an abstract of this map, while the full map is provided in **Appendix A**.

Figure 4.8 below shows the existing cumulative volume curve of Areas 1 and 2 of Shand Properties Ltd in their current condition, and the volume curve after the proposed indicative works for the development (earthworks within Area 1, treatment wetland, and additional grading). The graph shows that the proposed layout is introducing volumes are above existing up to the 100yr level (9.35mRL). It therefore provides more flood storage then the existing conditions, which is an overall improvement for the Kimihia floodplain capacity, and it also provides contingency and flexibility for any changes to site layout during detailed design.

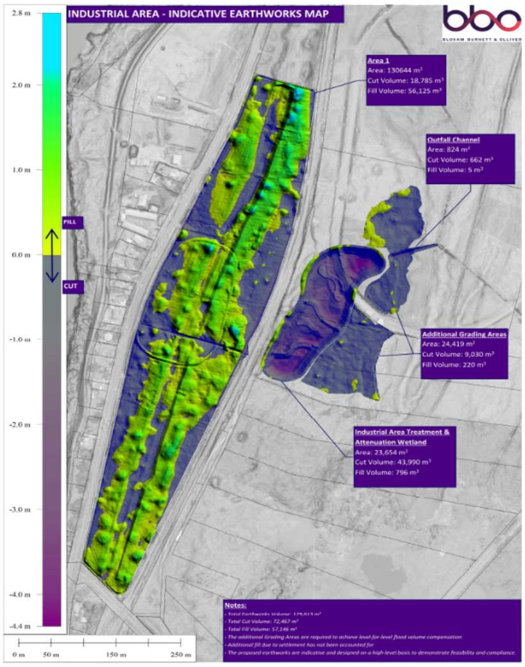


Figure 4.7 *Overview of the Earthworks map for the proposed rezoning layout.*

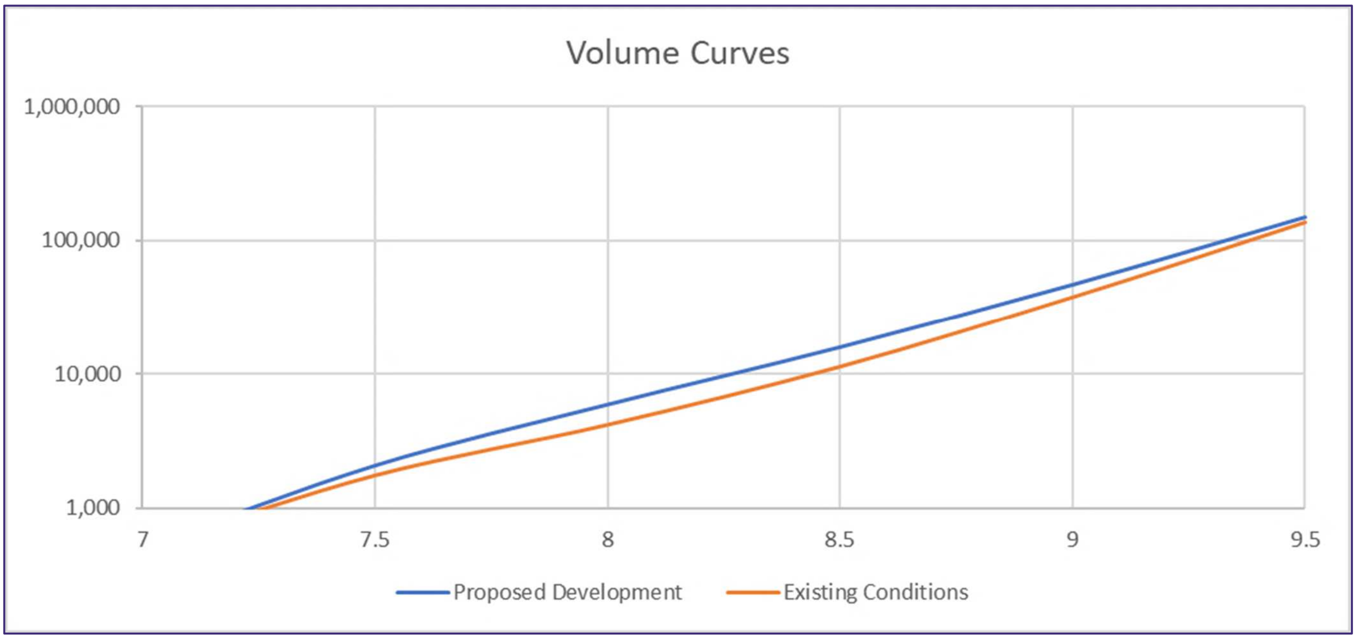


Figure 4.7 *Existing and proposed storage curves.*

4.4 Residual Risk

The existing site is within a defended area - defended from river flooding by stop banks and localised flooding by floodgates. Although this area is defended from river flooding up to the 100yr event, it still has the potential to flood. The site may flood due to a larger than design event, including the 100yr <u>with</u> climate change whereby the Waikato River may overtop the stop bank. The area may also flood due to failure of the stop bank defence. These scenarios are unlikely but can still happen.

This risk that remains - once a defence is in place - is known as "residual risk". The Waikato Regional Policy Statement includes policies and methods addressing residual risk, including that residual risk zones shall be identified in District Plans. Currently the Waikato District Plan does not show this area as being at residual risk of flooding due to a failure in the defence. The Waikato Regional Council though includes this area within the "Waipa and Waikato 1% AEP flood extent". That is the extent of the Waikato river flood in case of a stopbank failure.

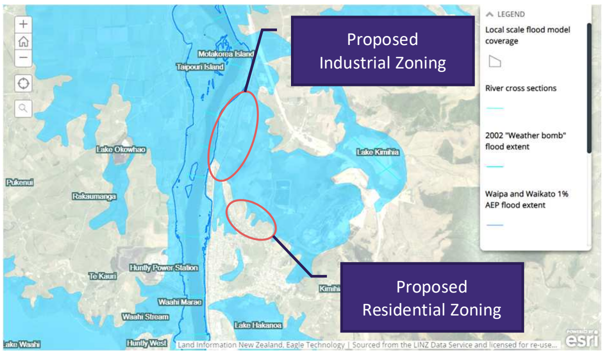


Figure 4.8 *Waikato River Flooding Map – WRC Regional Hazards Portal.*

A stop bank breach, failure, collapse, or overtopping event adjacent to the site could potentially result in rapid inundation of the site. The modelled 100yr maximum level is RL 10.96m as shown by main channel cross section results close to the site (WRC review feedback). This level is more than 1m above proposed minimum floor level (RL 9.65m). Residual risk is difficult to design and prepare for. WRC has no modelling information on the impact of breach or overtopping scenarios. In worst cases, while river levels are high, there may be little time to evacuate if the failure occurred without warning while people were at work.

A residual risk assessment was undertaken for this rezoning proposal. The assessment included modelling of Waikato river stopbank breach for both the existing and proposed conditions. The locations of the breach were defined in consultation with Waikato Regional Council. Two-dimensional hydraulic modelling was conducted to assess the velocity and the depth of the flood wave as it spreads for the breach into the floodplain area.

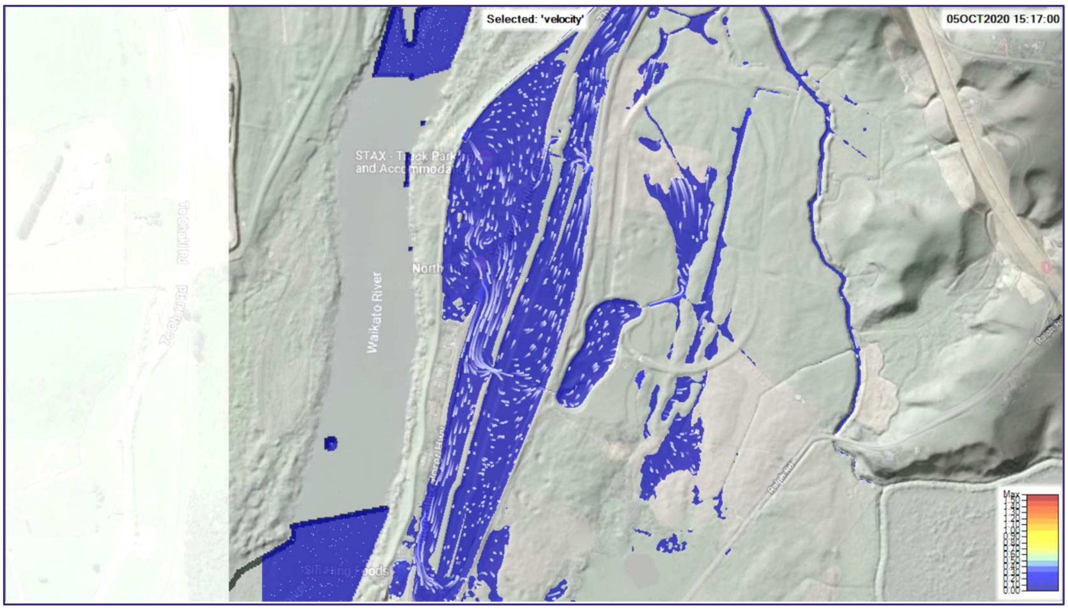


Figure 4.9 *Sample image from the HEC-RAS 2D stopbank breach model.*

A separate memo has been produced providing the details of the stopbank breach modelling. It is attached as **Appendix D** of this report. The results from the simulations indicated that the minimum time it takes for the flood wave to travel from a stopbank breach to the proposed Industrial Zoning is approximately 40min. However, if the breach happens near the southern floodgate this will take at least another 15min.

After reaching Area 1, propagation of the flood wave slows down. Minimum time to reach the road at approximately the middle of Area 1 is approximately 80 minutes. The modified terrain based on the proposed stormwater infrastructure adds significantly more time when the breach occurs close to Kimihia floodgate.

During the first 90 minutes after formation of the breach in the Waikato River stopbank, the maximum velocity along the proposed road on Area 1 is approximately 0.5m/s and the maximum inundation depth is about 0.14m. Velocities in the proposed swales are indicated to be up to 0.4m/s. Velocities at the majority of the indicative industrial lots were not greater than 0.1m/s throughout the 36hr simulations.

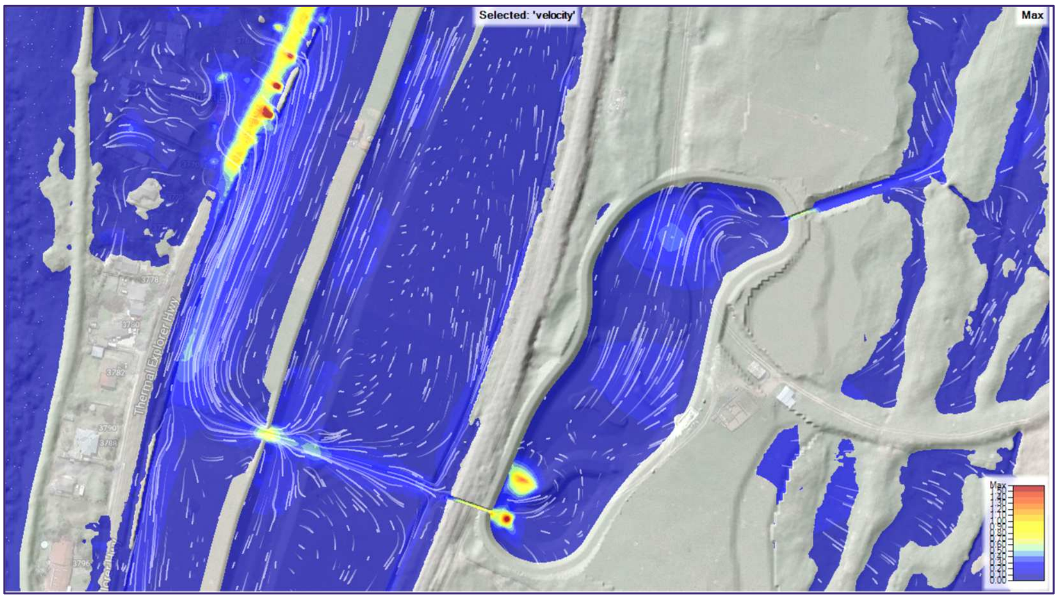


Figure 4.10 *Maximum velocities map on a 30m breach scenario. Sample image from the HEC-RAS 2D stopbank breach model.*

The hydraulic analysis and modelling of the stopbank breach scenarios show that there is enough time for an emergency plan to be implemented on staff evacuation if there is a monitoring and warning system in place for the stopbank. The proposed industrial development earthworks and stormwater infrastructure contribute to allowing more time for a reaction, adding approximately 10 minutes for the flood wave to reach the internal access road.

If a breach occurs close to the existing Kimihia floodgate, the proposed stormwater layout within the industrial area could protect it from flooding by routing the flood wave towards the proposed wetland, providing full protection against flooding for at least 36 hours.

5. Conclusions

Shand Properties Ltd propose two separate rezoning areas, one for residential and one for industrial use. Both of these areas are located within the Kimihia catchment, which is a defended area. The entire industrial area and part of the residential zone are also within the Kimihia floodplain.

A hydrological analysis was undertaken to determine the maximum flood levels of the Kimihia catchment during 100-year ARI climate-adjusted design storm with the floodgates considered closed. The analysis indicated that RL 9.35m is the maximum water surface level. That has been proposed to be used to set the minimum floor levels for the proposed industrial zoning (RL9.65m). The Operative District Plan shows RL 10.30m as the 100-year flood level at the area of the proposed residential zone. This level is proposed to determine the minimum floor level for the residential rezoning (RL 10.80m).

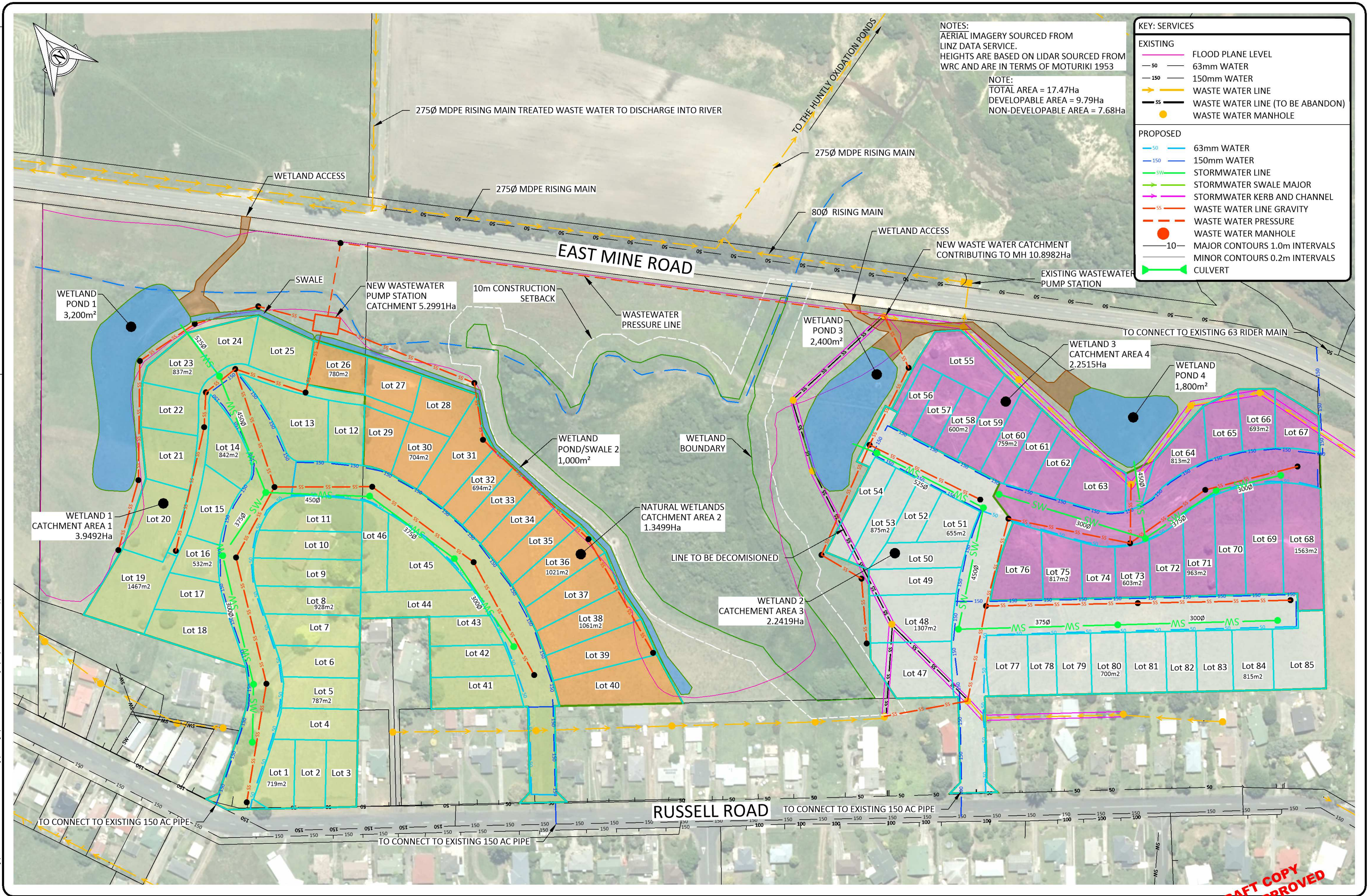
Initial Scheme level design shows that the residential zone can be serviced by several stormwater treatment devices and the layout includes reticulation, swales and cut-off drains. The existing wetland would be protected, and the quality of the stormwater runoff would be improved due to the proposed treatment devices. A discharge distribution layout should be introduced during detail design to ensure that the wetland will be receiving the base flows in a manner that replicates the existing situation.

The Industrial zoning stormwater management solution includes a centralised treatment and attenuation device located on the eastern side of the railway. A new 1050mm diameter culvert under the railway has been proposed to allow for the runoff from the development to discharge into the proposed treatment device. The drainage layout of the industrial area would consist of stormwater swales that would add treatment and attenuation properties into the overall layout and qualify as a treatment train approach. The proposed layout has been modelled and shows capability to attenuate the development flows down or even lower than the pre-development runoff. The treated and attenuated runoff will be discharged into the existing rural Kimihia drainage network through a proposed channel. Level for level flood volume compensation has been taken into account in the indicative design of the proposed layout, ensuring that the development will not reduce the flood storage capacity of the overall defended area.

Residual risk assessment was carried out including two-dimensional river stopbank breach modelling. The results show that a proper emergency evacuation plan can be established and implemented for the proposed industrial area.

The above conclusions indicate that the proposed rezoning can be serviced within the local, regional and national requirements regarding stormwater management.

Appendix A – Maps & Drawings



A	20/10/2020	INITIAL ISSUE	BSC	
	Date	Issue/revision detail	By	Chk / Appr

Designed: PP
Checked: BJM
Drawn: BSC
Approved:
mx model version:

BLOXAM BURNETT & OLLIVER

Client: SHAND PROPERTIES LIMITED

Project: HUNLTY NORTH REZONING

Drawing: RESIDENTIAL ZONE CONCEPTUAL THREE WATERS LAYOUT

Status: PRELIMINARY

Date: 20/10/2020

Scale (Original Size A3): 1:2000

Drawing Number: 144370/02/-/0702

Revision: -

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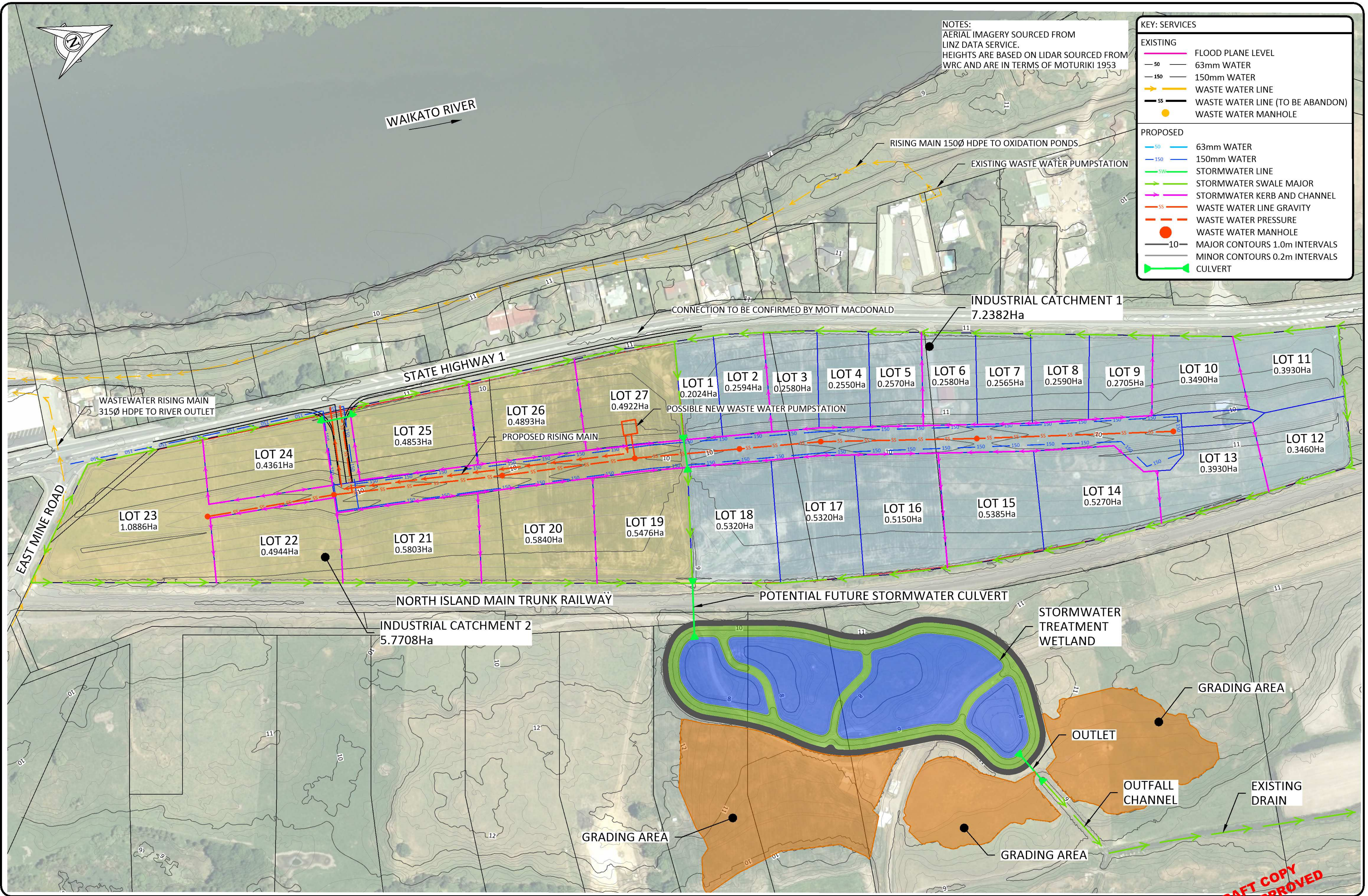
Version 2.04 – October 2013



SCALE FOR VALIDATING SIZE OF A3 PLOT ONLY 0 100mm

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Rev	Date	Issue/revision detail	By	Chk	Appr
A	10/09/2020	INITIAL ISSUE	BSC		

Designed: PP | Checked: BJM | Drawn: BSC | Approved: | mx model version:

PBLOXAM BURNETT & OLLIVER

Client: SHAND PROPERTIES LIMITED

Project: HUNTLY NORTH REZONING

Drawing: INDUSTRIAL ZONE CONCEPTUAL THREE WATERS LAYOUT

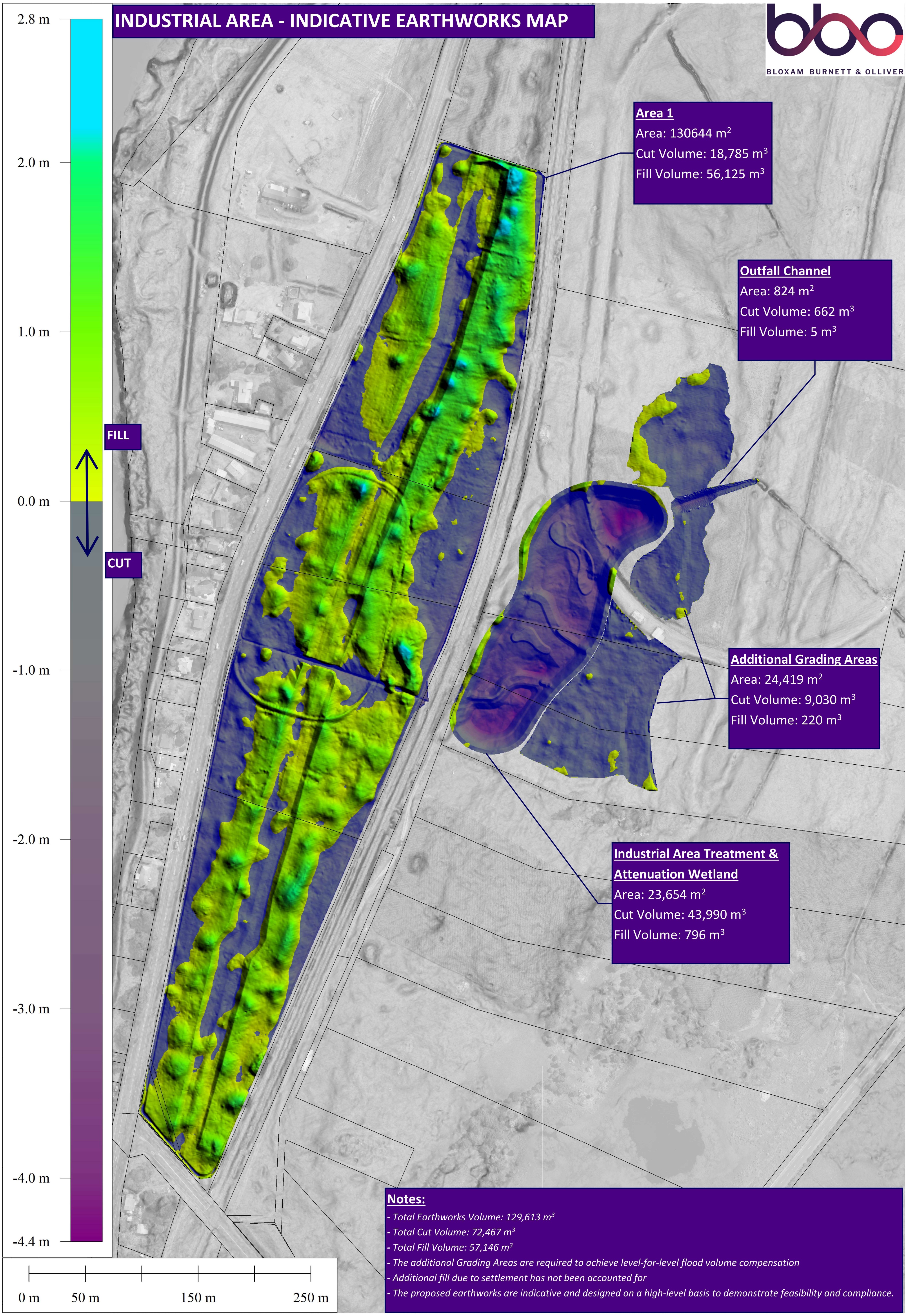
Status: PRELIMINARY

DRAFT COPY NOT APPROVED

Date: 10/09/2020 | Scale (Original Size A3): 1:2500

Drawing Number: 144370 / 02 / - / 0704 | Revision: -

Version 2.04 – October 2013



INDUSTRIAL AREA - INDICATIVE EARTHWORKS MAP
BLOXAM BURNETT & OLLIVER
2.8 m
2.0 m
1.0 m
0.0 m
-1.0 m
-2.0 m
-3.0 m
-4.0 m
-4.4 m
FILL
CUT
0 m
50 m
150 m
250 m
Area 1
Area: 130644 m^2
Cut Volume: 18,785 m^3
Fill Volume: 56,125 m^3
Outfall Channel
Area: 824 m^2
Cut Volume: 662 m^3
Fill Volume: 5 m^3
Additional Grading Areas
Area: 24,419 m^2
Cut Volume: 9,030 m^3
Fill Volume: 220 m^3
Industrial Area Treatment & Attenuation Wetland
Area: 23,654 m^2
Cut Volume: 43,990 m^3
Fill Volume: 796 m^3
Notes:
- Total Earthworks Volume: 129,613 m^3
- Total Cut Volume: 72,467 m^3
- Total Fill Volume: 57,146 m^3
- The additional Grading Areas are required to achieve level-for-level flood volume compensation
- Additional fill due to settlement has not been accounted for
- The proposed earthworks are indicative and designed on a high-level basis to demonstrate feasibility and compliance.

Appendix B – Memorandum on hydrological analysis of overall Kimihia catchment

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BLOXAM BURNETT & OLLIVER

Memo

To	Rick Liefting
CC	Ghassan Basheer
From	Constantinos Fokianos
Date	6 October 2020
Job No.	144370.02
Job name	Shand Properties Rezoning
Subject	**Kimihia Overall Catchment Stormwater Memo**

Introduction

BBO has been engaged by Shand Properties Limited (Shand) to support their submissions to the Proposed Waikato District Plan (PWDP). Shand are seeking to re-zone approximately 30.5 ha of land located in Huntly North from the current rural zoning to a mix of industrial (approximately 13 ha) and residential (approximately 17.5 ha) zoning.

As part of the services, BBO is engaged to produce a 3 Waters Report for the proposed plan change. This memo refers to the overall catchment hydrology and expected flood levels for various scenarios. It provides a high-level analysis of the overall Kimihia catchment, to determine the minimum platform level for the proposed industrial development.

Catchment delineation

The overall Kimihia catchment (2,473ha) delineation was based on WRC's LIDAR information, enhanced by the more-detailed LIDAR for lake Kimihia and as-built digital drawings from the Huntly By-Pass section of the Waikato Expressway (WEX) to establish an informed/updated terrain model. The updated digital terrain model enabled the delineation and exclusion of the WEX pumped drainage sub-catchment at the north of the overall catchment (refer to the overall catchment plan provided as an attachment of this memo) as this sub-catchment discharges to Waikato river through a separate floodgate.

Soil Characteristics

Infiltration was estimated based on typical hydraulic characteristics of typical soil texture classes, taken from the EPA SWMM-5 Manual and Horton's Infiltration Parameters from Soils Data (Rawls, W.J. et al., 1983, Journal of Hydraulic Engineering, 109:1(62). Soil textures from the site were determined from Manaaki Whenua, Landcare Research S-Map website. The predominant soil texture is clay. 54% of the area is considered poorly or imperfectly drained. The figure below shows the overall geology of the area as documented on S-Map website.

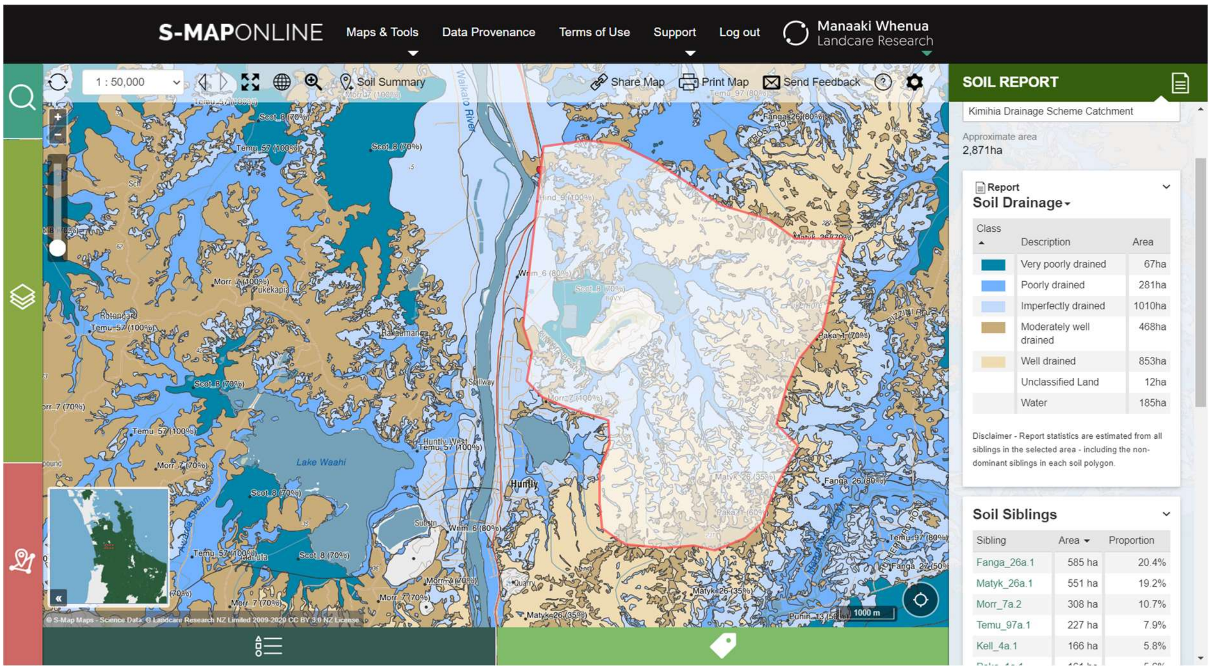

Figure 1. *Abstract on S-Map website information on soil properties regarding the project's catchment.*

Tables 1 and 2 below show the soil properties information and the calculated infiltration rates.

Table No. 1

Draining Properties Table		
Drainage status	**Area**	**% of total area**
Water/Unclassified/Very/Poorly drained	545	19%
Imperfectly Drained	1010	35%
Well/Moderately drained	1321	46%
Total	2876	100%

Table No. 2

Calculation of Infiltration Values								
Soil Family	**Area**		**Saturated infiltration Rate K**			**Max Infiltration Rate**		
	ha	**%**	**inches/hr**	**mm/hr**	**weighted mm/hr**	**inches/hr**	**mm/hr**	**weighted mm/hr**
Fanga_26a.1	585	22%	0.04	1.02	0.2244	0.5	12.7	2.794
Matyk_26a.1	551	21%	0.26	6.6	1.386	1.5	38.1	8.001
Morr_7a.2	308	12%	0.13	3.3	0.396	1.5	38.1	4.572
Temu_97a.1	227	9%	0.02	0.51	0.0459	0.5	12.7	1.143
Kell_4a.1	166	6%	0.06	1.52	0.0912	1	25.4	1.524
Paka_1a.1	161	6%	0.13	3.3	0.198	1.5	38.1	2.286
Matyk_27a.1	113	4%	0.13	3.3	0.132	1.5	38.1	1.524
Matyk_37a.1	113	4%	0.43	10.92	0.4368	2	50.8	2.032
Wnm_6a.1	83	3%	0.43	10.92	0.3276	2	50.8	1.524
Scot_8a.1	67	3%	0.01	0.25	0.0075	0.4	10.16	0.3048
Mai_4a.1	54	2%	0.13	3.3	0.066	1.5	38.1	0.762
Airf_7c.1	53	2%	0.06	1.52	0.0304	1	25.4	0.508
Fanga_27a.1	41	2%	0.04	1.02	0.0204	0.5	12.7	0.254
Airf_4b.1	29	1%	0.02	0.51	0.0051	0.5	12.7	0.127
Hind_9c.3	29	1%	0.13	3.3	0.033	1.5	38.1	0.381
Utuh_17a.2	29	1%	0.01	0.25	0.0025	0.4	10.16	0.1016
Turan_33a.1	21	1%	1.18	29.97	0.2997	2.5	63.5	0.635
Hast_67a.1	18	1%	0.04	1.02	0.0102	1	25.4	0.254
Weighted Total	2648	100%			**3.71**			**28.73**

The infiltration method applied was the Horton's Infiltration Equation. Horton's Equation uses infiltration rates for typical soil types in the sub-catchment. This method uses an initial infiltration rate, adjusted for an appropriate antecedent moisture condition. The initial infiltration rate decreases exponentially to a final infiltration rate for saturated soil conditions. The rate that the infiltration is decreased by is determined by a decay rate. A decay rate constant of 4 was applied. Using an initial weighted infiltration rate of 28.73mm/hr, a final infiltration rate of 3.71mm/hr, and a decay rate of 4 results in instantaneous infiltration rates of 20.48mm/hr at 6 minutes, 14.95mm/hr at 12 minutes, 7.1mm/hr at 30 minutes. The infiltration rate reaches 3.71mm/hr by 2 hours and 12 minutes, long before the rainfall peak of 24-hour design events. The following figure shows a plot of infiltration versus time, using Horton's Equation with the inputs that have been applied.

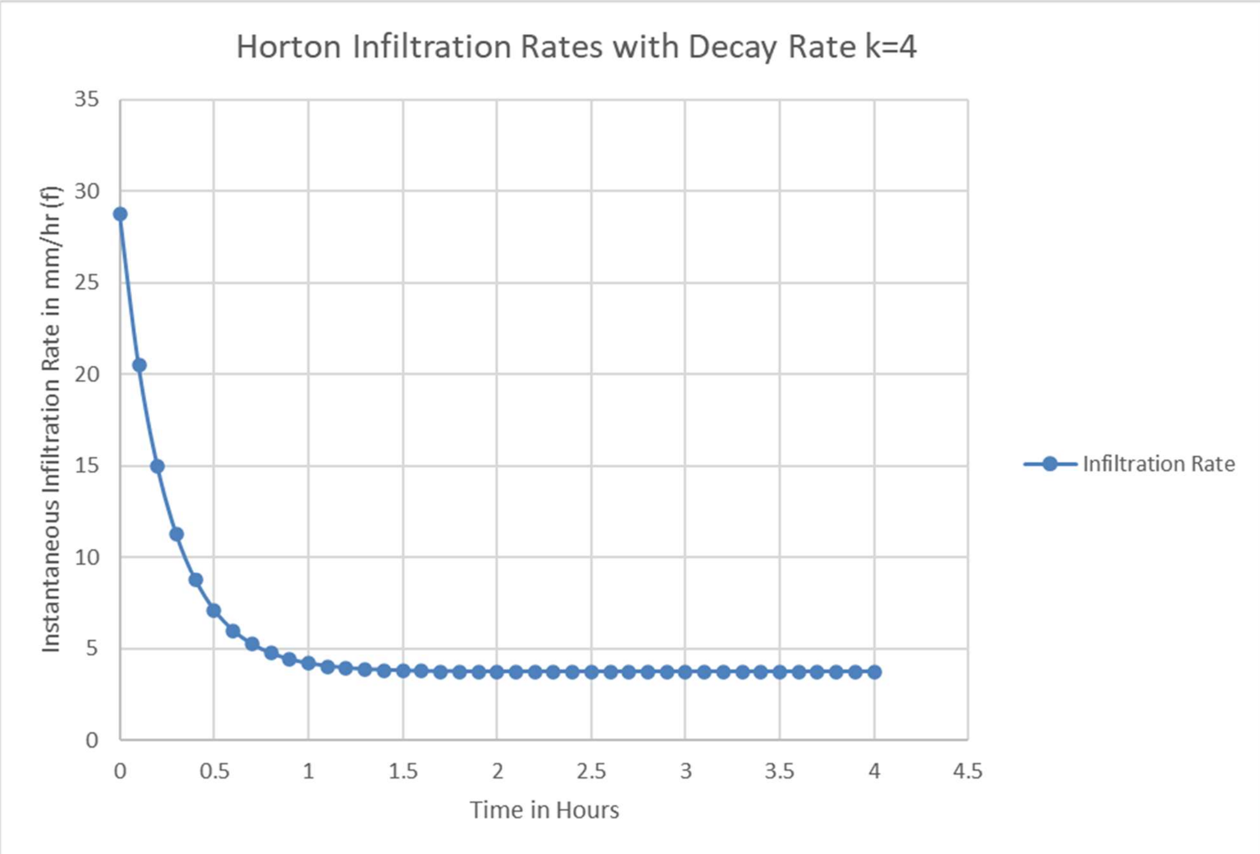

Figure 2. *Horton's equation plot.*

This infiltration function only applies to pervious areas. On the impervious areas, no infiltration is considered to take place. Depression storage was set to 5mm for pervious areas and 2mm for impervious areas. Regarding treatment and conveyance devices, a conservative approach was followed where no further losses due to infiltration ("soakage") were considered within ponding areas of Kimihia Lake, quarry lake and the wastewater treatment pond.

Hydrological & Hydraulic Modelling

Stormwater hydrology and hydraulics were modelled using EPA SWMM-5 (SWMM). SWMM develops sub-catchment runoff flows, based on imported rainfall patterns (synthetic design storms or continuous rainfall data), soil infiltration characteristics, and soil cover complexes. SWMM was used to route the stormwater flows, using the Dynamic Wave Method (application of the full Saint-Venant Equations). This allows hydraulic losses in manholes, bends or junctions to be accounted for and ponds with complex outlet structures to be modelled.

24-hour duration storms have been modelled, using rainfall intensities from High Intensity Rainfall System (HIRDS). The 24-hour design storms modelled were the 2-year, 10-year, and 100-year ARI storm events. All design storm events were adjusted to account for a 2.1°C temperature increase due to climate change. A set of design storms without climate change was also modelled to account for the existing conditions. The TP108 temporal pattern, also referred on WRC TR2108/02, table 4-1 was used for the rainfall distribution over 24-hour events.

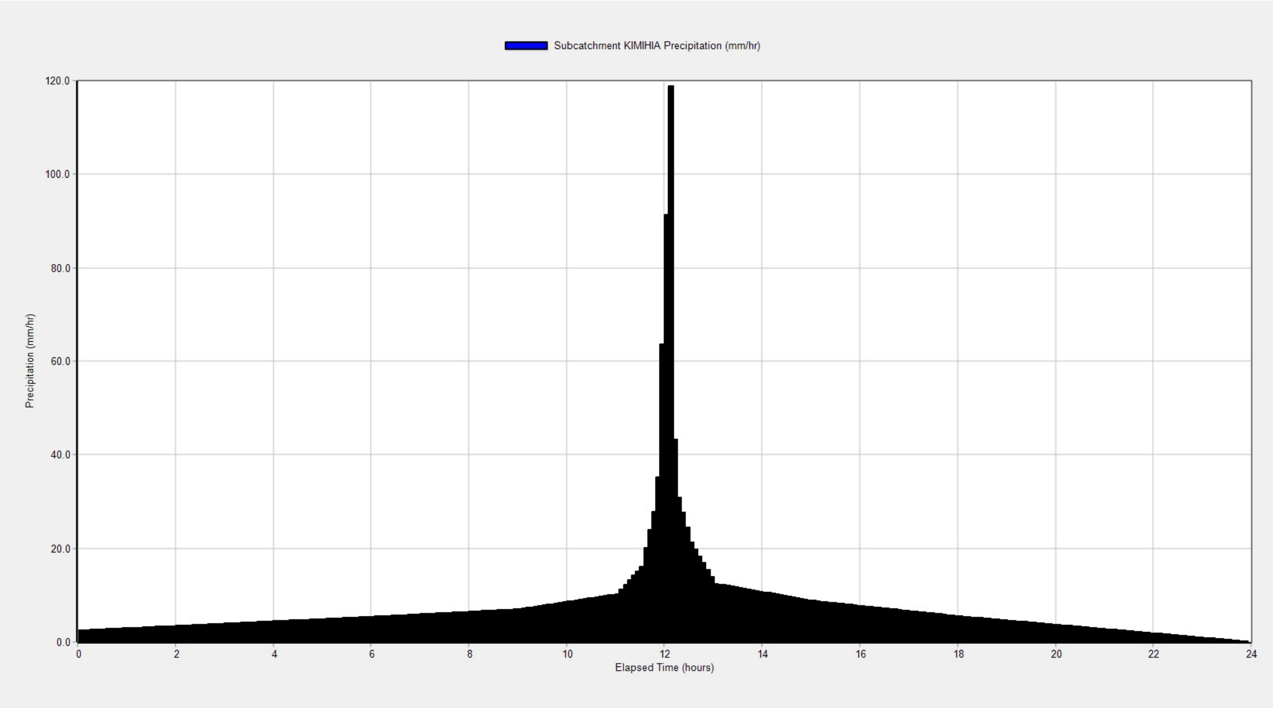

Figure 3. *Rainfall temporal patterns used for the 100-year/24hour design event, including climate change adjustment.*

The Waikato river floodgates that Kimihia stream discharges into were included in the model. For the basic scenario used to define the service level for the proposed development, the Waikato river 100-year water level at RL 10.96m was used as tailwater and the floodgates were modelled closed while the runoff was accumulating upstream of them. Other scenarios that included lower Waikato river flows were also run to provide a better understanding how the model performs and correlates with the empirical and statistical information provided by WRC.

Using the updated terrain model (WRC LIDAR updated with lake Kimihia detailed lass files and WEX digital terrain), a storage curve was defined using Global Mapper GIS software, for levels between RL 6.3m (invert of the floodgates) to RL 12.0m. The volumes of the quarry lake and the wastewater treatment pond were deducted from the curve as they are disconnected from the overall catchment inundation network. The elevation/storage curve is shown below (Figure 4 and Table 3).

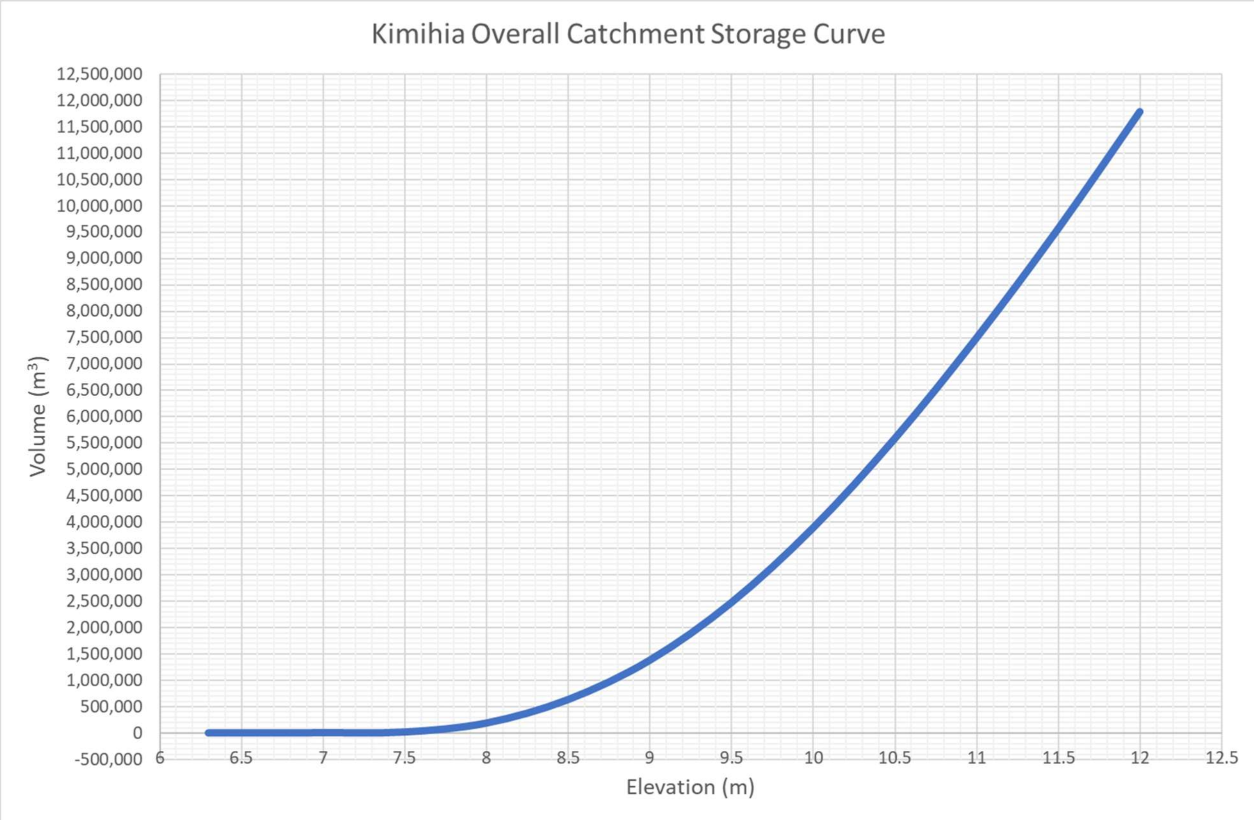

Figure 4. *Kimia catchment elevation/volume curve.*

Table No. 3

Overall Kimihia Catchment elevation/volume curve				
Elevation RL (m)	**Depth (measured at RL 6.3m)**	**Area (m^2)**	**Volume (m^3)**	**Cumulative Volume (m^3)**
6.3	0.0	1,020	0	0
6.5	0.2	1,420	244	244
7.0	0.7	11,771	3,298	3,542
7.5	1.2	43,341	13,778	17,320
8.0	1.7	635,301	169,661	186,980
8.5	2.2	1,142,646	444,487	631,467
9.0	2.7	1,843,946	746,648	1,378,115
9.5	3.2	2,535,895	1,094,960	2,473,075
10.0	3.7	3,137,429	1,418,331	3,891,406
10.5	4.2	3,640,465	1,694,474	5,585,880
11.0	4.7	3,991,226	1,907,923	7,493,803
11.5	5.2	4,305,071	2,074,074	9,567,877
12.0	5.7	4,576,155	2,220,307	11,788,183

The curve was used to model the accumulation of water upstream of the closed floodgates. A storage device with the storage curve defined above was imported into the model as the upstream node of the floodgate culvert.

Model Results

A combination of variable tailwater conditions was used for all design rainfalls. Table 4 below presents the resulted flood levels for each design rainfall and tailwater assumption. Table 5 presents the corresponding maximum inundation volumes within the overall catchment.

Table No. 4

Model Results – Flood levels						
Tail Water Level (Waikato River Flood Level)	Maximum Flood Level (RL m)					
	Existing Conditions Design Rainfall ARI			Climate Change Adjusted Design Rainfall ARI		
	2-Year	10-Year	100-Year	2-Year	10-Year	100-Year
RL 6.4m	7.82	8.14	8.75	7.90	8.32	9.02
RL 7.1m	7.82	8.14	8.75	7.90	8.32	9.02
RL 7.5m	7.87	8.17	8.76	7.94	8.34	9.03
RL 9.87m (10-year flood level)	8.18	8.52	9.10	8.27	8.70	9.35
RL 10.96m (100-year flood level)	8.18	8.52	9.10	8.27	8.70	**9.35**

Table No. 5

Model Results – Ponding Volumes						
Tail Water Level (Waikato River Flood Level)	Maximum Ponding Volume (m^3)					
	Existing Conditions Design Rainfall ARI			Climate Change Adjusted Design Rainfall ARI		
	2-Year	10-Year	100-Year	2-Year	10-Year	100-Year
RL 6.4m	92,824	284,278	962,856	128,046	445,322	1,412,713
RL 7.1m	93,299	284,582	963,287	128,422	445,648	1,413,223
RL 7.5m	113,639	306,346	982,784	149,850	466,413	1,432,158
RL 9.87m (10-year flood level)	320,280	656,122	393,493	393,493	889,777	2,116,759
RL 10.96m (100-year flood level)	320,280	656,122	393,493	393,493	889,777	**2,116,759**

The results show that level RL 9.35m could be used as the 100-year flood level for developments within the overall Kimihia Catchment. This flood level has been based on a conservative approach and represents a "worst case scenario" when the 100-year rainfall event coincides with Waikato River's 100-year flood levels.

It is therefore proposed that the flood level of reference for the Shand development area and the proposed plan change should be set at RL 9.35m. This level will be the upper boundary within which the level-for-level and volume-for-volume compensation approach should be applied regarding the future earthworks of the development.

Yours sincerely

Bloxam Burnett & Olliver

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Water Resource Engineering Manager
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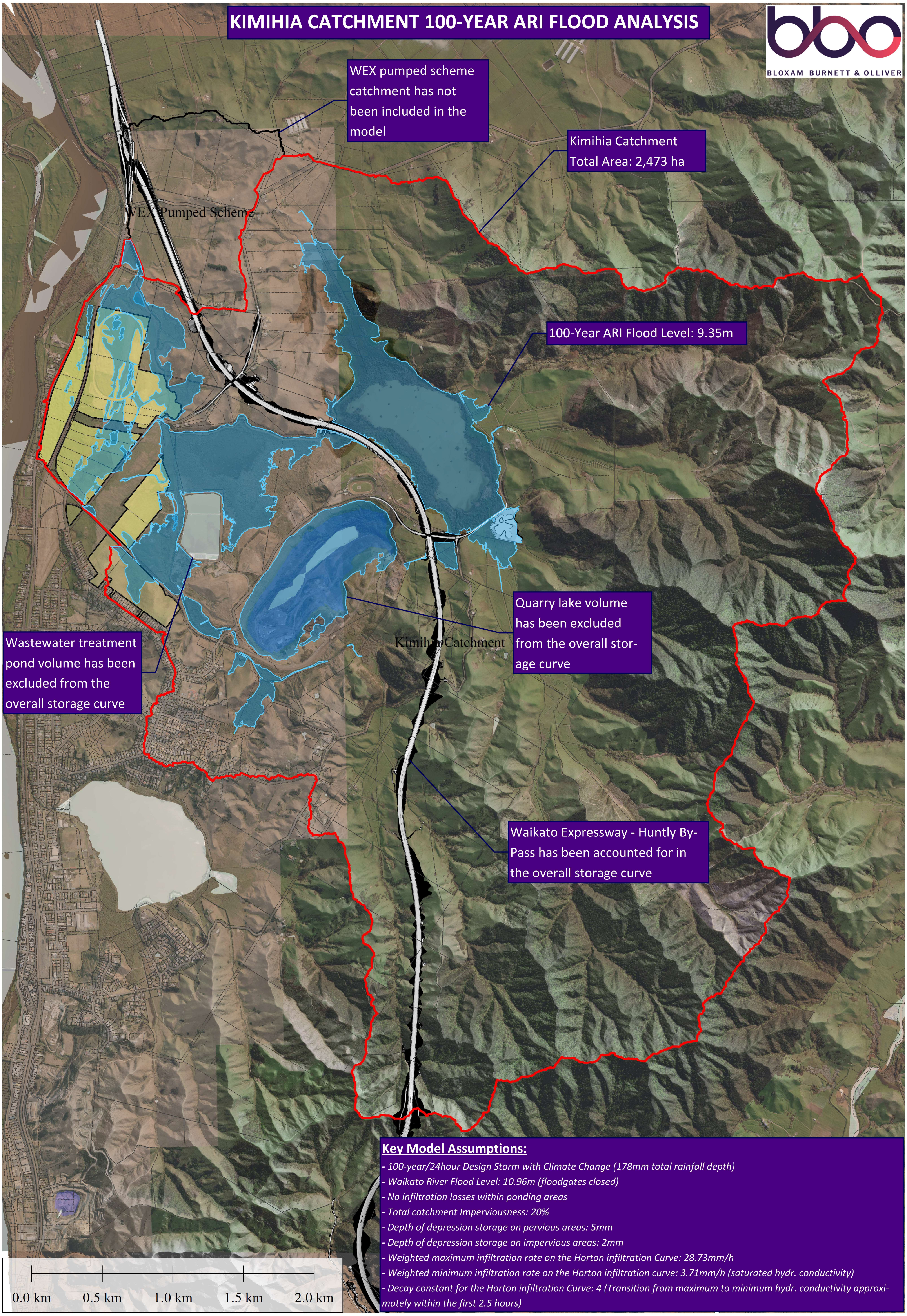
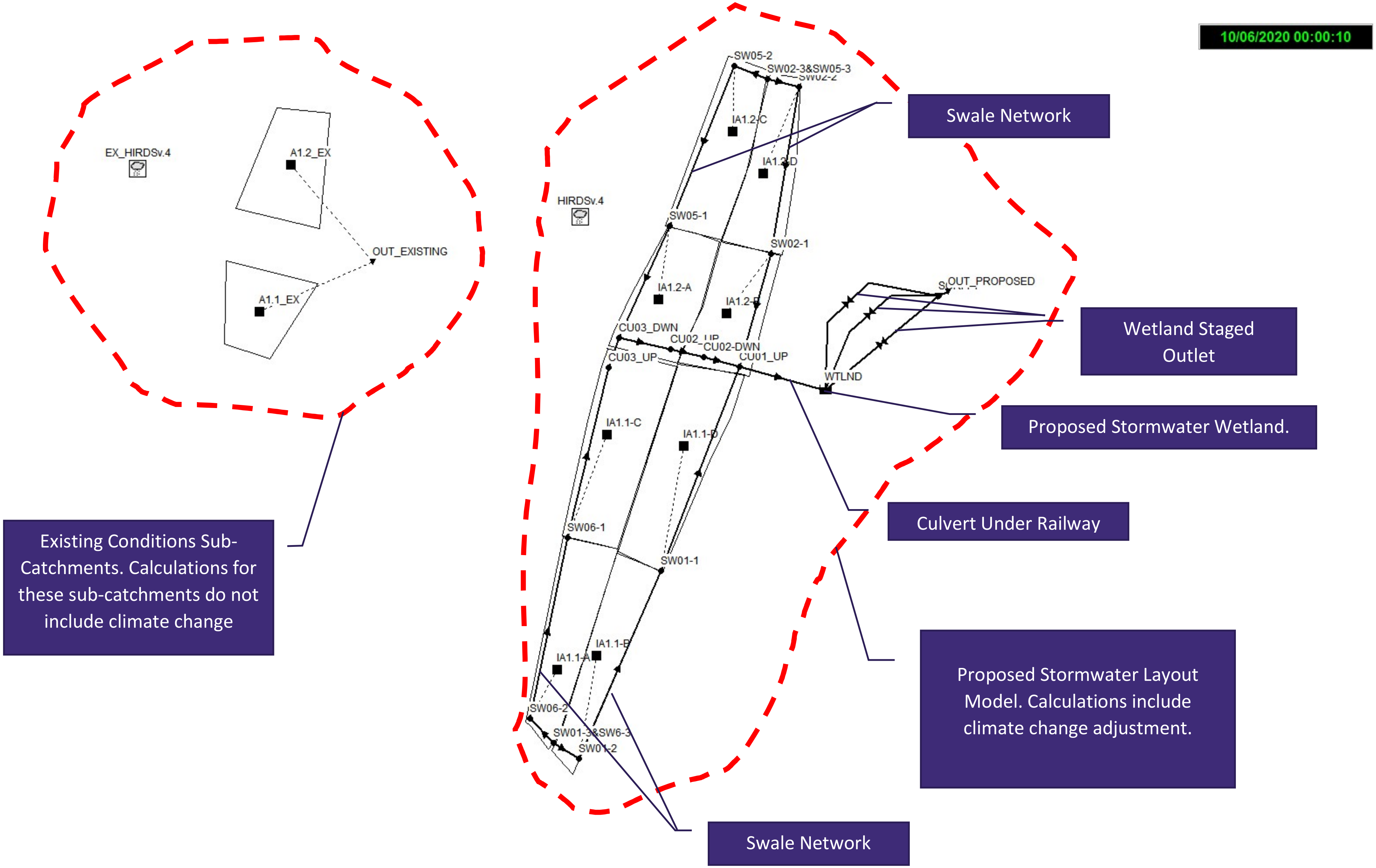
K:\144370 Shand Properties Rezoning\02 Industrial & Residential rezoning\Stormwater\Reporting\Stormwater Memo 06.10.2020.docx


KIMIHIA CATCHMENT 100-YEAR ARI FLOOD ANALYSIS
BLOXAM BURNETT & OLLIVER
WEX pumped scheme catchment has not been included in the model
Kimihia Catchment Total Area: 2,473 ha
WEX Pumped Scheme
100-Year ARI Flood Level: 9.35m
Quarry lake volume has been excluded from the overall storage curve
Kimihia Catchment
Wastewater treatment pond volume has been excluded from the overall storage curve
Waikato Expressway - Huntly By-Pass has been accounted for in the overall storage curve
0.0 km
0.5 km
1.0 km
1.5 km
2.0 km
Key Model Assumptions:
- 100-year/24hour Design Storm with Climate Change (178mm total rainfall depth)
- Waikato River Flood Level: 10.96m (floodgates closed)
- No infiltration losses within ponding areas
- Total catchment Imperviousness: 20%
- Depth of depression storage on pervious areas: 5mm
- Depth of depression storage on impervious areas: 2mm
- Weighted maximum infiltration rate on the Horton infiltration Curve: 28.73mm/h
- Weighted minimum infiltration rate on the Horton infiltration curve: 3.71mm/h (saturated hydr. conductivity)
- Decay constant for the Horton infiltration Curve: 4 (Transition from maximum to minimum hydr. conductivity approximately within the first 2.5 hours)

Appendix C – Industrial Zoning SWMM Model Output

SWMM Model Layout



2year/24hour Design Storm

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.015)
--------------------------------------------------------------

*********************************************************
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
*********************************************************

****************
Analysis Options
****************
Flow Units ............... CMS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... NO
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Surcharge Method ......... EXTRAN
Starting Date ............ 10/06/2020 00:00:00
Ending Date .............. 10/09/2020 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:00:10
Wet Time Step ............ 00:00:01
Dry Time Step ............ 01:00:00
Routing Time Step ........ 0.50 sec
Variable Time Step ....... YES
Maximum Trials ........... 20
Number of Threads ........ 1
Head Tolerance ........... 0.001500 m


**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......         1.748        66.967
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         0.847        32.435
Surface Runoff ...........         0.884        33.854
Final Storage ............         0.018         0.678
Continuity Error (%) .....         0.000


**************************        Volume        Volume
Flow Routing Continuity        hectare-m     10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.884         8.836
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         0.684         6.842
Flooding Loss ............         0.000         0.000
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.199         1.993
Continuity Error (%) .....         0.005


***************************
Time-Step Critical Elements
***************************
None


********************************
Highest Flow Instability Indexes
********************************
Link CU02 (1)


*************************
Routing Time Step Summary
*************************
Minimum Time Step           :     0.40 sec
Average Time Step           :     0.50 sec
Maximum Time Step           :     0.50 sec
Percent in Steady State     :     0.00
Average Iterations per Step :     2.00
Percent Not Converging      :     0.00
Time Step Frequencies       :
    0.500 -  0.362 sec      :   100.00 %
    0.362 -  0.263 sec      :     0.00 %
    0.263 -  0.190 sec      :     0.00 %
    0.190 -  0.138 sec      :     0.00 %
    0.138 -  0.100 sec      :     0.00 %
```

****
Subcatchment Runoff Summary
****

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
IA1.1-A	71.58	0.00	0.00	5.74	63.07	1.42	64.49	0.71	0.09	0.901
IA1.1-B	71.58	0.00	0.00	5.75	63.07	1.41	64.48	0.76	0.10	0.901
IA1.1-C	71.58	0.00	0.00	5.74	63.07	1.42	64.49	1.07	0.14	0.901
IA1.1-D	71.58	0.00	0.00	5.75	63.07	1.41	64.48	1.05	0.13	0.901
IA1.2-A	71.58	0.00	0.00	5.75	63.07	1.41	64.48	1.48	0.18	0.901
IA1.2-B	71.58	0.00	0.00	5.75	63.07	1.41	64.48	1.26	0.16	0.901
IA1.2-C	71.58	0.00	0.00	5.75	63.07	1.41	64.48	1.10	0.14	0.901
IA1.2-D	71.58	0.00	0.00	5.75	63.07	1.41	64.48	0.97	0.12	0.901
A1.2_EX	62.36	0.00	0.00	58.89	0.00	3.47	3.47	0.20	0.01	0.056
A1.1_EX	62.36	0.00	0.00	59.25	0.00	3.11	3.11	0.22	0.01	0.050

****
Node Depth Summary
****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	Time of Max Occurrence hr:min	Reported Max Depth Meters
CU01_UP	JUNCTION	0.15	0.56	9.51	0	13:01	0.56
CU02_UP	JUNCTION	0.07	0.47	9.67	0	12:47	0.47
CU02-DWN	JUNCTION	0.09	0.48	9.58	0	12:55	0.48
CU03_DWN	JUNCTION	0.09	0.47	9.77	0	12:43	0.47
CU03_UP	JUNCTION	0.05	0.40	9.80	0	12:42	0.40
SCRFFY	JUNCTION	0.08	0.13	8.88	0	22:10	0.13
SW01-1	JUNCTION	0.06	0.35	9.90	0	12:31	0.35
SW01-2	JUNCTION	0.04	0.30	10.45	0	12:23	0.30
SW01-3&SW6-3	JUNCTION	0.01	0.13	10.63	0	12:27	0.13
SW02-1	JUNCTION	0.06	0.39	9.96	0	12:28	0.39
SW02-2	JUNCTION	0.05	0.33	10.53	0	12:23	0.33
SW02-3&SW05-3	JUNCTION	0.01	0.16	10.81	0	12:26	0.16
SW05-1	JUNCTION	0.07	0.39	10.32	0	12:27	0.39
SW05-2	JUNCTION	0.05	0.32	10.87	0	12:19	0.32
SW06-1	JUNCTION	0.06	0.37	10.27	0	12:23	0.37
SW06-2	JUNCTION	0.04	0.28	10.68	0	12:18	0.28
OUT_PROPOSED	OUTFALL	0.08	0.12	8.72	0	22:10	0.12
OUT_EXISTING	OUTFALL	0.00	0.00	0.00	0	00:00	0.00
WTLND	STORAGE	0.23	0.42	9.22	0	22:10	0.42

****
Node Inflow Summary
****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days	Time of Max Occurrence hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
CU01_UP	JUNCTION	0.000	0.536	0	12:39	0	8.43	0.369
CU02_UP	JUNCTION	0.000	0.264	0	12:42	0	4.19	0.018
CU02-DWN	JUNCTION	0.000	0.261	0	12:44	0	4.19	-0.046
CU03_DWN	JUNCTION	0.000	0.282	0	12:32	0	4.2	0.190
CU03_UP	JUNCTION	0.000	0.142	0	12:27	0	1.73	0.290
SCRFFY	JUNCTION	0.000	0.048	0	22:10	0	6.42	0.006
SW01-1	JUNCTION	0.132	0.176	0	12:10	1.05	1.86	-0.555
SW01-2	JUNCTION	0.095	0.098	0	12:09	0.758	0.808	-0.198
SW01-3&SW6-3	JUNCTION	0.000	0.011	0	12:18	0	0.0502	0.427
SW02-1	JUNCTION	0.156	0.217	0	12:10	1.26	2.35	-0.533
SW02-2	JUNCTION	0.120	0.127	0	12:09	0.974	1.08	-0.180
SW02-3&SW05-3	JUNCTION	0.000	0.019	0	12:18	0	0.11	0.258
SW05-1	JUNCTION	0.183	0.242	0	12:10	1.48	2.47	-0.164
SW05-2	JUNCTION	0.136	0.136	0	12:09	1.1	1.1	-0.198
SW06-1	JUNCTION	0.143	0.183	0	12:10	1.07	1.73	-0.200
SW06-2	JUNCTION	0.095	0.095	0	12:09	0.708	0.708	-0.145
OUT_PROPOSED	OUTFALL	0.000	0.048	0	22:10	0	6.41	0.000
OUT_EXISTING	OUTFALL	0.025	0.025	0	13:39	0.428	0.428	0.000
WTLND	STORAGE	0.000	0.461	0	13:01	0	8.4	0.032

****
Node Surcharge Summary
****

No nodes were surcharged.

****
Node Flooding Summary
****

No nodes were flooded.

****
Storage Volume Summary
****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
WTLND	3.379	10	0	0	6.104	18	0 22:10	0.048

****
Outfall Loading Summary
****

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
OUT_PROPOSED	91.30	0.027	0.048	6.415
OUT_EXISTING	11.70	0.014	0.025	0.428
System	51.50	0.041	0.059	6.842

****
Link Flow Summary
****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
CU01	CONDUIT	0.461	0 13:01	1.85	0.46	0.43
CU02	CONDUIT	0.261	0 12:44	0.90	0.44	0.63
CU03	CONDUIT	0.113	0 12:37	0.53	0.34	0.72
OUTLET_PIPE	CONDUIT	0.048	0 22:10	0.88	0.03	0.12
SW01.1	CHANNEL	0.129	0 12:31	0.12	0.05	0.44
SW01.2	CHANNEL	0.067	0 12:23	0.09	0.03	0.32
SW01.3	CHANNEL	0.009	0 12:27	0.03	0.00	0.21
SW02.1	CHANNEL	0.173	0 12:28	0.14	0.06	0.46
SW02.2	CHANNEL	0.092	0 12:23	0.11	0.03	0.36
SW02.3	CHANNEL	0.017	0 12:26	0.05	0.00	0.25
SW03	CHANNEL	0.256	0 12:46	0.15	0.10	0.52
SW04	CHANNEL	0.264	0 12:42	0.17	0.11	0.47
SW05.1	CHANNEL	0.174	0 12:27	0.14	0.06	0.42
SW05.2	CHANNEL	0.080	0 12:19	0.10	0.03	0.35
SW05.3	CHANNEL	0.019	0 12:18	0.06	0.01	0.24
SW06.1	CHANNEL	0.142	0 12:27	0.13	0.05	0.37
SW06.2	CHANNEL	0.055	0 12:18	0.08	0.02	0.32
SW06.3	CHANNEL	0.011	0 12:18	0.05	0.00	0.20
ORFC	ORIFICE	0.030	0 22:10			1.00
2&10YR_WEIR	WEIR	0.019	0 22:10			0.14
100YR_WEIR	WEIR	0.000	0 00:00			0.00

****
Flow Classification Summary
****

Conduit	Adjusted /Actual Length	---------- Fraction of Time in Flow Class ---------- Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl
CU01	1.00	0.02	0.00	0.00	0.92	0.07	0.00	0.00	0.00	0.15
CU02	1.00	0.03	0.00	0.00	0.97	0.00	0.00	0.00	0.00	0.46
CU03	1.00	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.68
OUTLET_PIPE	1.00	0.05	0.00	0.00	0.95	0.00	0.00	0.00	0.06	0.00
SW01.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.93	0.00
SW01.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.96	0.00
SW01.3	1.00	0.00	0.25	0.00	0.75	0.00	0.00	0.00	0.92	0.00
SW02.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.92	0.00
SW02.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.96	0.00
SW02.3	1.00	0.00	0.19	0.00	0.81	0.00	0.00	0.00	0.94	0.00
SW03	1.00	0.02	0.03	0.00	0.96	0.00	0.00	0.00	0.70	0.00
SW04	1.00	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.00
SW05.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.93	0.00
SW05.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.96	0.00
SW05.3	1.00	0.00	0.19	0.00	0.81	0.00	0.00	0.00	0.70	0.00
SW06.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.45	0.00
SW06.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.96	0.00
SW06.3	1.00	0.00	0.25	0.00	0.75	0.00	0.00	0.00	0.73	0.00

****
Conduit Surcharge Summary
****

No conduits were surcharged.

Analysis begun on: Fri Nov 6 10:39:29 2020
Analysis ended on: Fri Nov 6 10:39:41 2020
Total elapsed time: 00:00:12

10year/24hour Design Storm

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.015)
 --------------------------------------------------------------

  *********************************************************
 NOTE: The summary statistics displayed in this report are
 based on results found at every computational time step,
 not just on results from each reporting time step.
 *********************************************************

 ****************
 Analysis Options
 ****************
 Flow Units ............... CMS
 Process Models:
   Rainfall/Runoff ........ YES
   RDII ................... NO
   Snowmelt ............... NO
   Groundwater ............ NO
   Flow Routing ........... YES
   Ponding Allowed ........ NO
   Water Quality .......... NO
 Infiltration Method ...... HORTON
 Flow Routing Method ...... DYNWAVE
 Surcharge Method ......... EXTRAN
 Starting Date ............ 10/06/2020 00:00:00
 Ending Date .............. 10/09/2020 00:00:00
 Antecedent Dry Days ...... 0.0
 Report Time Step ......... 00:00:10
 Wet Time Step ............ 00:00:01
 Dry Time Step ............ 01:00:00
 Routing Time Step ........ 0.50 sec
 Variable Time Step ....... YES
 Maximum Trials ........... 20
 Number of Threads ........ 1
 Head Tolerance ........... 0.001500 m


 **************************        Volume         Depth
 Runoff Quantity Continuity     hectare-m            mm
 **************************     ---------       -------
 Total Precipitation ......         2.717       104.088
 Evaporation Loss .........         0.000         0.000
 Infiltration Loss ........         1.098        42.062
 Surface Runoff ...........         1.601        61.349
 Final Storage ............         0.018         0.678
 Continuity Error (%) .....         0.000


 **************************        Volume        Volume
 Flow Routing Continuity        hectare-m      10^6 ltr
 **************************     ---------     ---------
 Dry Weather Inflow .......         0.000         0.000
 Wet Weather Inflow .......         1.601        16.012
 Groundwater Inflow .......         0.000         0.000
 RDII Inflow ..............         0.000         0.000
 External Inflow ..........         0.000         0.000
 External Outflow .........         1.316        13.160
 Flooding Loss ............         0.000         0.000
 Evaporation Loss .........         0.000         0.000
 Exfiltration Loss ........         0.000         0.000
 Initial Stored Volume ....         0.000         0.000
 Final Stored Volume ......         0.285         2.852
 Continuity Error (%) .....         0.001


 ***************************
 Time-Step Critical Elements
 ***************************
 None


 ********************************
 Highest Flow Instability Indexes
 ********************************
 All links are stable.


 *************************
 Routing Time Step Summary
 *************************
 Minimum Time Step           :     0.40 sec
 Average Time Step           :     0.50 sec
 Maximum Time Step           :     0.50 sec
 Percent in Steady State     :     0.00
 Average Iterations per Step :     2.00
 Percent Not Converging      :     0.00
 Time Step Frequencies       :
     0.500 -  0.362 sec      :   100.00 %
     0.362 -  0.263 sec      :     0.00 %
     0.263 -  0.190 sec      :     0.00 %
     0.190 -  0.138 sec      :     0.00 %
     0.138 -  0.100 sec      :     0.00 %
```

****
Subcatchment Runoff Summary
****

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
IA1.1-A	112.34	0.00	0.00	7.05	99.75	4.18	103.94	1.14	0.17	0.925
IA1.1-B	112.34	0.00	0.00	7.06	99.75	4.18	103.93	1.22	0.17	0.925
IA1.1-C	112.34	0.00	0.00	7.05	99.75	4.18	103.94	1.72	0.25	0.925
IA1.1-D	112.34	0.00	0.00	7.06	99.75	4.18	103.93	1.70	0.24	0.925
IA1.2-A	112.34	0.00	0.00	7.06	99.75	4.17	103.92	2.39	0.33	0.925
IA1.2-B	112.34	0.00	0.00	7.06	99.75	4.17	103.92	2.04	0.28	0.925
IA1.2-C	112.34	0.00	0.00	7.06	99.75	4.17	103.92	1.77	0.24	0.925
IA1.2-D	112.34	0.00	0.00	7.06	99.75	4.17	103.92	1.57	0.22	0.925
A1.2_EX	95.85	0.00	0.00	76.38	0.00	19.47	19.47	1.14	0.05	0.203
A1.1_EX	95.85	0.00	0.00	77.52	0.00	18.32	18.32	1.32	0.05	0.191

****
Node Depth Summary
****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters
CU01_UP	JUNCTION	0.27	0.78	9.73	0 12:53	0.78
CU02_UP	JUNCTION	0.11	0.67	9.87	0 12:45	0.67
CU02-DWN	JUNCTION	0.16	0.65	9.75	0 12:52	0.65
CU03_DWN	JUNCTION	0.11	0.60	9.90	0 12:42	0.60
CU03_UP	JUNCTION	0.07	0.56	9.96	0 12:39	0.56
SCRFFY	JUNCTION	0.11	0.17	8.92	0 21:17	0.17
SW01-1	JUNCTION	0.07	0.44	9.99	0 12:24	0.44
SW01-2	JUNCTION	0.05	0.37	10.52	0 12:19	0.37
SW01-3&SW6-3	JUNCTION	0.01	0.19	10.69	0 12:20	0.19
SW02-1	JUNCTION	0.08	0.48	10.05	0 12:24	0.48
SW02-2	JUNCTION	0.06	0.41	10.61	0 12:19	0.41
SW02-3&SW05-3	JUNCTION	0.02	0.22	10.87	0 12:19	0.22
SW05-1	JUNCTION	0.08	0.48	10.41	0 12:21	0.48
SW05-2	JUNCTION	0.06	0.38	10.93	0 12:15	0.38
SW06-1	JUNCTION	0.07	0.44	10.34	0 12:18	0.44
SW06-2	JUNCTION	0.05	0.34	10.74	0 12:15	0.34
OUT_PROPOSED	OUTFALL	0.10	0.15	8.75	0 21:17	0.15
OUT_EXISTING	OUTFALL	0.00	0.00	0.00	0 00:00	0.00
WTLND	STORAGE	0.35	0.64	9.44	0 21:17	0.64

****
Node Inflow Summary
****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
CU01_UP	JUNCTION	0.000	0.935	0 12:29	0	13.6	0.667
CU02_UP	JUNCTION	0.000	0.430	0 12:32	0	6.67	0.030
CU02-DWN	JUNCTION	0.000	0.420	0 12:35	0	6.67	-0.049
CU03_DWN	JUNCTION	0.000	0.500	0 12:24	0	6.69	0.166
CU03_UP	JUNCTION	0.000	0.257	0 12:18	0	2.75	0.314
SCRFFY	JUNCTION	0.000	0.083	0 21:17	0	10.7	0.005
SW01-1	JUNCTION	0.237	0.336	0 12:10	1.7	3.04	-0.884
SW01-2	JUNCTION	0.170	0.181	0 12:09	1.22	1.34	-0.182
SW01-3&SW6-3	JUNCTION	0.000	0.027	0 12:14	0	0.117	0.264
SW02-1	JUNCTION	0.280	0.414	0 12:10	2.04	3.84	-0.807
SW02-2	JUNCTION	0.216	0.236	0 12:10	1.57	1.8	-0.167
SW02-3&SW05-3	JUNCTION	0.000	0.044	0 12:15	0	0.231	0.153
SW05-1	JUNCTION	0.329	0.450	0 12:10	2.39	3.93	-0.200
SW05-2	JUNCTION	0.243	0.243	0 12:09	1.77	1.77	-0.178
SW06-1	JUNCTION	0.253	0.338	0 12:10	1.72	2.75	-0.236
SW06-2	JUNCTION	0.168	0.168	0 12:09	1.14	1.14	-0.145
OUT_PROPOSED	OUTFALL	0.000	0.083	0 21:17	0	10.7	0.000
OUT_EXISTING	OUTFALL	0.105	0.105	0 13:34	2.46	2.46	0.000
WTLND	STORAGE	0.000	0.767	0 12:53	0	13.5	0.028

****
Node Surcharge Summary
****

No nodes were surcharged.

****
Node Flooding Summary
****

No nodes were flooded.

```
*********************
Storage Volume Summary
*********************
```

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
WTLND	5.167	15	0	0	9.549	28	0 21:17	0.083

```
**********************
Outfall Loading Summary
**********************
```

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
OUT_PROPOSED	92.83	0.044	0.083	10.700
OUT_EXISTING	17.46	0.054	0.105	2.460
System	55.14	0.099	0.166	13.160

```
*******************
Link Flow Summary
*******************
```

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
CU01	CONDUIT	0.767	0 12:53	1.94	0.77	0.63
CU02	CONDUIT	0.420	0 12:35	1.08	0.71	0.88
CU03	CONDUIT	0.183	0 12:30	0.67	0.56	0.96
OUTLET_PIPE	CONDUIT	0.083	0 21:17	1.01	0.05	0.16
SW01.1	CHANNEL	0.248	0 12:25	0.14	0.09	0.59
SW01.2	CHANNEL	0.133	0 12:19	0.13	0.05	0.40
SW01.3	CHANNEL	0.023	0 12:20	0.05	0.00	0.28
SW02.1	CHANNEL	0.320	0 12:24	0.17	0.11	0.60
SW02.2	CHANNEL	0.182	0 12:19	0.15	0.07	0.44
SW02.3	CHANNEL	0.041	0 12:19	0.07	0.01	0.32
SW03	CHANNEL	0.401	0 12:39	0.16	0.16	0.72
SW04	CHANNEL	0.430	0 12:32	0.19	0.17	0.64
SW05.1	CHANNEL	0.327	0 12:21	0.19	0.12	0.53
SW05.2	CHANNEL	0.144	0 12:15	0.13	0.05	0.43
SW05.3	CHANNEL	0.044	0 12:15	0.09	0.02	0.30
SW06.1	CHANNEL	0.257	0 12:18	0.18	0.09	0.48
SW06.2	CHANNEL	0.100	0 12:15	0.10	0.04	0.39
SW06.3	CHANNEL	0.027	0 12:14	0.07	0.01	0.26
ORFC	ORIFICE	0.036	0 21:17			1.00
2&10YR_WEIR	WEIR	0.047	0 21:17			0.26
100YR_WEIR	WEIR	0.000	0 00:00			0.00

```
**************************
Flow Classification Summary
**************************
```

Conduit	Adjusted /Actual Length	---------- Fraction of Time in Flow Class ---------- Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl
CU01	1.00	0.01	0.00	0.00	0.93	0.06	0.00	0.00	0.00	0.05
CU02	1.00	0.02	0.00	0.00	0.97	0.00	0.00	0.00	0.00	0.26
CU03	1.00	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.66
OUTLET_PIPE	1.00	0.04	0.00	0.00	0.96	0.00	0.00	0.00	0.05	0.00
SW01.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.94	0.00
SW01.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.97	0.00
SW01.3	1.00	0.00	0.19	0.00	0.81	0.00	0.00	0.00	0.96	0.00
SW02.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.94	0.00
SW02.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.96	0.00
SW02.3	1.00	0.00	0.16	0.00	0.84	0.00	0.00	0.00	0.96	0.00
SW03	1.00	0.01	0.02	0.00	0.97	0.00	0.00	0.00	0.32	0.00
SW04	1.00	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.17	0.00
SW05.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.94	0.00
SW05.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.97	0.00
SW05.3	1.00	0.00	0.16	0.00	0.84	0.00	0.00	0.00	0.68	0.00
SW06.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.45	0.00
SW06.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.97	0.00
SW06.3	1.00	0.00	0.19	0.00	0.81	0.00	0.00	0.00	0.70	0.00

```
************************
Conduit Surcharge Summary
************************
```

Conduit	--------- Hours Full -------- Both Ends	Upstream	Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
CU03	0.01	0.01	0.21	0.01	0.01

```
Analysis begun on:  Fri Nov  6 10:46:24 2020
Analysis ended on:  Fri Nov  6 10:46:36 2020
Total elapsed time: 00:00:12
```

100year/24hour Design Storm

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.015)
--------------------------------------------------------------

*********************************************************
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
*********************************************************

****************
Analysis Options
****************
Flow Units ............... CMS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... NO
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Surcharge Method ......... EXTRAN
Starting Date ............ 10/06/2020 00:00:00
Ending Date .............. 10/09/2020 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:00:10
Wet Time Step ............ 00:00:01
Dry Time Step ............ 01:00:00
Routing Time Step ........ 0.50 sec
Variable Time Step ....... YES
Maximum Trials ........... 20
Number of Threads ........ 1
Head Tolerance ........... 0.001500 m


**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......         4.287       164.257
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         1.282        49.103
Surface Runoff ...........         2.988       114.476
Final Storage ............         0.018         0.678
Continuity Error (%) .....         0.000


**************************        Volume        Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         2.988        29.879
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         2.638        26.384
Flooding Loss ............         0.000         0.000
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.350         3.496
Continuity Error (%) .....        -0.003


***************************
Time-Step Critical Elements
***************************
None


********************************
Highest Flow Instability Indexes
********************************
All links are stable.


*************************
Routing Time Step Summary
*************************
Minimum Time Step           :     0.28 sec
Average Time Step           :     0.50 sec
Maximum Time Step           :     0.50 sec
Percent in Steady State     :     0.00
Average Iterations per Step :     2.00
Percent Not Converging      :     0.00
Time Step Frequencies       :
    0.500 -  0.362 sec      :   100.00 %
    0.362 -  0.263 sec      :     0.00 %
    0.263 -  0.190 sec      :     0.00 %
    0.190 -  0.138 sec      :     0.00 %
    0.138 -  0.100 sec      :     0.00 %
```

****
Subcatchment Runoff Summary
****

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
IA1.1-A	177.86	0.00	0.00	7.92	158.72	9.86	168.58	1.85	0.29	0.948
IA1.1-B	177.86	0.00	0.00	7.93	158.71	9.85	168.57	1.98	0.29	0.948
IA1.1-C	177.86	0.00	0.00	7.92	158.72	9.86	168.58	2.79	0.43	0.948
IA1.1-D	177.86	0.00	0.00	7.93	158.71	9.85	168.57	2.76	0.41	0.948
IA1.2-A	177.86	0.00	0.00	7.93	158.71	9.85	168.57	3.88	0.57	0.948
IA1.2-B	177.86	0.00	0.00	7.93	158.71	9.85	168.57	3.30	0.49	0.948
IA1.2-C	177.86	0.00	0.00	7.93	158.71	9.85	168.57	2.87	0.42	0.948
IA1.2-D	177.86	0.00	0.00	7.93	158.71	9.85	168.57	2.55	0.37	0.948
A1.2_EX	150.68	0.00	0.00	89.17	0.00	61.51	61.51	3.60	0.17	0.408
A1.1_EX	150.68	0.00	0.00	91.06	0.00	59.62	59.62	4.30	0.18	0.396

****
Node Depth Summary
****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters
CU01_UP	JUNCTION	0.39	1.06	10.01	0 12:42	1.06
CU02_UP	JUNCTION	0.20	0.99	10.19	0 12:48	0.99
CU02-DWN	JUNCTION	0.26	0.91	10.01	0 12:43	0.91
CU03_DWN	JUNCTION	0.16	0.90	10.20	0 12:48	0.90
CU03_UP	JUNCTION	0.11	0.88	10.28	0 12:47	0.88
SCRFFY	JUNCTION	0.14	0.30	9.05	0 18:17	0.30
SW01-1	JUNCTION	0.09	0.54	10.09	0 12:20	0.54
SW01-2	JUNCTION	0.07	0.46	10.61	0 12:16	0.46
SW01-3&SW6-3	JUNCTION	0.02	0.24	10.74	0 12:15	0.24
SW02-1	JUNCTION	0.10	0.59	10.17	0 12:20	0.59
SW02-2	JUNCTION	0.07	0.51	10.71	0 12:16	0.51
SW02-3&SW05-3	JUNCTION	0.02	0.28	10.93	0 12:15	0.28
SW05-1	JUNCTION	0.10	0.59	10.51	0 12:17	0.59
SW05-2	JUNCTION	0.07	0.45	11.00	0 12:13	0.45
SW06-1	JUNCTION	0.08	0.53	10.43	0 12:16	0.53
SW06-2	JUNCTION	0.06	0.41	10.81	0 12:12	0.41
OUT_PROPOSED	OUTFALL	0.13	0.24	8.84	0 18:17	0.24
OUT_EXISTING	OUTFALL	0.00	0.00	0.00	0 00:00	0.00
WTLND	STORAGE	0.46	0.88	9.68	0 18:17	0.88

****
Node Inflow Summary
****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
CU01_UP	JUNCTION	0.000	1.508	0 12:22	0	22.1	0.762
CU02_UP	JUNCTION	0.000	0.576	0 12:21	0	10.7	0.035
CU02-DWN	JUNCTION	0.000	0.528	0 12:58	0	10.7	-0.032
CU03_DWN	JUNCTION	0.000	0.817	0 12:19	0	10.7	0.299
CU03_UP	JUNCTION	0.000	0.439	0 12:16	0	4.42	0.567
SCRFFY	JUNCTION	0.000	0.219	0 18:17	0	18.5	0.003
SW01-1	JUNCTION	0.410	0.613	0 12:10	2.76	4.99	-0.988
SW01-2	JUNCTION	0.294	0.328	0 12:10	1.98	2.23	-0.177
SW01-3&SW6-3	JUNCTION	0.000	0.057	0 12:12	0	0.244	0.127
SW02-1	JUNCTION	0.486	0.764	0 12:10	3.3	6.31	-0.908
SW02-2	JUNCTION	0.375	0.436	0 12:10	2.55	3	-0.146
SW02-3&SW05-3	JUNCTION	0.000	0.094	0 12:13	0	0.453	0.069
SW05-1	JUNCTION	0.570	0.794	0 12:10	3.88	6.3	-0.461
SW05-2	JUNCTION	0.422	0.422	0 12:09	2.87	2.87	-0.152
SW06-1	JUNCTION	0.433	0.592	0 12:10	2.79	4.4	-0.501
SW06-2	JUNCTION	0.287	0.287	0 12:09	1.85	1.85	-0.129
OUT_PROPOSED	OUTFALL	0.000	0.219	0 18:17	0	18.5	0.000
OUT_EXISTING	OUTFALL	0.347	0.347	0 12:54	7.9	7.9	0.000
WTLND	STORAGE	0.000	1.145	0 12:42	0	21.9	0.016

****
Node Surcharge Summary
****

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
CU01_UP	JUNCTION	0.53	0.057	1.593

****
Node Flooding Summary
****

No nodes were flooded.

****
Storage Volume Summary
****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
WTLND	6.874	20	0	0	13.598	40	0 18:17	0.219

****
Outfall Loading Summary
****

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
OUT_PROPOSED	94.11	0.076	0.219	18.486
OUT_EXISTING	25.49	0.120	0.347	7.897
System	59.80	0.195	0.459	26.383

****
Link Flow Summary
****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
CU01	CONDUIT	1.145	0 12:42	2.16	1.15	0.91
CU02	CONDUIT	0.528	0 12:58	1.20	0.89	1.00
CU03	CONDUIT	0.242	0 12:25	0.86	0.74	1.00
OUTLET_PIPE	CONDUIT	0.219	0 18:17	1.28	0.13	0.27
SW01.1	CHANNEL	0.462	0 12:20	0.17	0.17	0.75
SW01.2	CHANNEL	0.253	0 12:16	0.17	0.09	0.50
SW01.3	CHANNEL	0.052	0 12:15	0.08	0.01	0.35
SW02.1	CHANNEL	0.597	0 12:20	0.21	0.21	0.77
SW02.2	CHANNEL	0.347	0 12:16	0.20	0.13	0.55
SW02.3	CHANNEL	0.089	0 12:15	0.11	0.02	0.39
SW03	CHANNEL	0.545	0 13:00	0.14	0.22	0.96
SW04	CHANNEL	0.576	0 12:21	0.20	0.23	0.94
SW05.1	CHANNEL	0.588	0 12:17	0.24	0.21	0.69
SW05.2	CHANNEL	0.247	0 12:13	0.16	0.09	0.52
SW05.3	CHANNEL	0.094	0 12:13	0.13	0.04	0.37
SW06.1	CHANNEL	0.439	0 12:16	0.21	0.16	0.66
SW06.2	CHANNEL	0.173	0 12:12	0.13	0.06	0.47
SW06.3	CHANNEL	0.057	0 12:12	0.10	0.02	0.32
ORFC	ORIFICE	0.041	0 18:17			1.00
2&10YR_WEIR	WEIR	0.085	0 18:17			0.39
100YR_WEIR	WEIR	0.094	0 18:17			0.17

****
Flow Classification Summary
****

Conduit	Adjusted /Actual Length	---------- Fraction of Time in Flow Class ---------- Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl
CU01	1.00	0.01	0.00	0.00	0.96	0.03	0.00	0.00	0.00	0.05
CU02	1.00	0.02	0.00	0.00	0.98	0.00	0.00	0.00	0.00	0.22
CU03	1.00	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.53
OUTLET_PIPE	1.00	0.03	0.00	0.00	0.97	0.00	0.00	0.00	0.04	0.00
SW01.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.94	0.00
SW01.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.97	0.00
SW01.3	1.00	0.00	0.16	0.00	0.84	0.00	0.00	0.00	0.97	0.00
SW02.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.95	0.00
SW02.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.97	0.00
SW02.3	1.00	0.00	0.13	0.00	0.87	0.00	0.00	0.00	0.98	0.00
SW03	1.00	0.01	0.02	0.00	0.97	0.00	0.00	0.00	0.23	0.00
SW04	1.00	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.14	0.00
SW05.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.95	0.00
SW05.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.97	0.00
SW05.3	1.00	0.00	0.13	0.00	0.87	0.00	0.00	0.00	0.67	0.00
SW06.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.63	0.00
SW06.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.98	0.00
SW06.3	1.00	0.00	0.16	0.00	0.84	0.00	0.00	0.00	0.69	0.00

****
Conduit Surcharge Summary
****

Conduit	Hours Full Both Ends	Hours Full Upstream	Hours Full Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
CU01	0.01	1.16	0.01	0.95	0.01
CU02	1.24	1.38	1.24	0.01	1.24
CU03	1.49	1.49	1.68	0.01	0.30
SW01.1	0.01	0.01	0.53	0.01	0.01
SW02.1	0.01	0.01	0.53	0.01	0.01
SW03	0.01	0.01	0.53	0.01	0.01

Analysis begun on: Fri Nov 6 11:32:33 2020
Analysis ended on: Fri Nov 6 11:32:45 2020
Total elapsed time: 00:00:12

Water Quality (1/3rd for the 2year/24hour Design Storm)

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.015)
--------------------------------------------------------------

*********************************************************
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
*********************************************************

****************
Analysis Options
****************
Flow Units ............... CMS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... NO
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Surcharge Method ......... EXTRAN
Starting Date ............ 10/06/2020 00:00:00
Ending Date .............. 10/09/2020 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:00:10
Wet Time Step ............ 00:00:01
Dry Time Step ............ 01:00:00
Routing Time Step ........ 0.50 sec
Variable Time Step ....... YES
Maximum Trials ........... 20
Number of Threads ........ 1
Head Tolerance ........... 0.001500 m


**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......         0.311        23.859
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         0.031         2.386
Surface Runoff ...........         0.262        20.117
Final Storage ............         0.018         1.356
Continuity Error (%) .....         0.000


**************************        Volume        Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.262         2.623
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         0.169         1.693
Flooding Loss ............         0.000         0.000
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.093         0.930
Continuity Error (%) .....         0.020


***************************
Time-Step Critical Elements
***************************
None


********************************
Highest Flow Instability Indexes
********************************
All links are stable.


*************************
Routing Time Step Summary
*************************
Minimum Time Step           :     0.40 sec
Average Time Step           :     0.50 sec
Maximum Time Step           :     0.50 sec
Percent in Steady State     :     0.00
Average Iterations per Step :     2.00
Percent Not Converging      :     0.00
Time Step Frequencies       :
    0.500 -  0.362 sec      :   100.00 %
    0.362 -  0.263 sec      :     0.00 %
    0.263 -  0.190 sec      :     0.00 %
    0.190 -  0.138 sec      :     0.00 %
    0.138 -  0.100 sec      :     0.00 %
```

****
Subcatchment Runoff Summary
****

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
IA1.1-A	23.86	0.00	0.00	2.39	20.12	0.00	20.12	0.22	0.02	0.843
IA1.1-B	23.86	0.00	0.00	2.39	20.12	0.00	20.12	0.24	0.02	0.843
IA1.1-C	23.86	0.00	0.00	2.39	20.12	0.00	20.12	0.33	0.03	0.843
IA1.1-D	23.86	0.00	0.00	2.39	20.12	0.00	20.12	0.33	0.03	0.843
IA1.2-A	23.86	0.00	0.00	2.39	20.12	0.00	20.12	0.46	0.04	0.843
IA1.2-B	23.86	0.00	0.00	2.39	20.12	0.00	20.12	0.39	0.04	0.843
IA1.2-C	23.86	0.00	0.00	2.39	20.12	0.00	20.12	0.34	0.03	0.843
IA1.2-D	23.86	0.00	0.00	2.39	20.12	0.00	20.12	0.30	0.03	0.843

****
Node Depth Summary
****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters
CU01_UP	JUNCTION	0.06	0.45	9.40	0 13:38	0.45
CU02_UP	JUNCTION	0.04	0.24	9.44	0 13:39	0.24
CU02-DWN	JUNCTION	0.05	0.32	9.42	0 13:39	0.32
CU03_DWN	JUNCTION	0.06	0.29	9.59	0 13:22	0.29
CU03_UP	JUNCTION	0.03	0.19	9.59	0 13:22	0.19
SCRFFY	JUNCTION	0.04	0.06	8.81	1 00:54	0.06
SW01-1	JUNCTION	0.03	0.19	9.74	0 13:00	0.19
SW01-2	JUNCTION	0.02	0.17	10.32	0 12:42	0.17
SW01-3&SW6-3	JUNCTION	0.00	0.04	10.54	0 12:47	0.04
SW02-1	JUNCTION	0.04	0.21	9.79	0 12:59	0.21
SW02-2	JUNCTION	0.03	0.19	10.39	0 12:41	0.19
SW02-3&SW05-3	JUNCTION	0.00	0.06	10.71	0 12:47	0.06
SW05-1	JUNCTION	0.04	0.23	10.15	0 12:51	0.23
SW05-2	JUNCTION	0.03	0.19	10.74	0 12:34	0.19
SW06-1	JUNCTION	0.03	0.23	10.13	0 12:46	0.23
SW06-2	JUNCTION	0.02	0.16	10.56	0 12:32	0.16
OUT_PROPOSED	OUTFALL	0.04	0.06	8.66	1 00:55	0.06
WTLND	STORAGE	0.08	0.14	8.94	1 00:54	0.14

****
Node Inflow Summary
****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
CU01_UP	JUNCTION	0.000	0.110	0 13:25	0	2.62	0.008
CU02_UP	JUNCTION	0.000	0.060	0 13:28	0	1.34	0.023
CU02-DWN	JUNCTION	0.000	0.059	0 13:34	0	1.34	0.128
CU03_DWN	JUNCTION	0.000	0.062	0 13:11	0	1.34	0.307
CU03_UP	JUNCTION	0.000	0.030	0 13:02	0	0.551	0.396
SCRFFY	JUNCTION	0.000	0.012	1 00:54	0	1.69	0.008
SW01-1	JUNCTION	0.032	0.039	0 12:10	0.329	0.57	0.221
SW01-2	JUNCTION	0.023	0.023	0 12:09	0.237	0.241	-0.252
SW01-3&SW6-3	JUNCTION	0.000	0.001	0 12:28	0	0.00435	1.584
SW02-1	JUNCTION	0.037	0.047	0 12:15	0.394	0.711	0.187
SW02-2	JUNCTION	0.029	0.029	0 12:09	0.304	0.316	-0.240
SW02-3&SW05-3	JUNCTION	0.000	0.003	0 12:31	0	0.0121	0.822
SW05-1	JUNCTION	0.044	0.055	0 12:15	0.463	0.794	-0.165
SW05-2	JUNCTION	0.032	0.032	0 12:09	0.342	0.342	-0.228
SW06-1	JUNCTION	0.034	0.042	0 12:10	0.333	0.55	-0.225
SW06-2	JUNCTION	0.023	0.023	0 12:09	0.221	0.221	-0.140
OUT_PROPOSED	OUTFALL	0.000	0.012	1 00:55	0	1.69	0.000
WTLND	STORAGE	0.000	0.177	0 13:56	0	2.62	0.036

****
Node Surcharge Summary
****

No nodes were surcharged.

****
Node Flooding Summary
****

No nodes were flooded.

****
Storage Volume Summary
****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
WTLND	1.194	4	0	0	2.006	6	1 00:54	0.012

****
Outfall Loading Summary
****

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
OUT_PROPOSED	85.92	0.008	0.012	1.693
System	85.92	0.008	0.012	1.693

****
Link Flow Summary
****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
CU01	CONDUIT	0.177	0 13:56	1.54	0.18	0.27
CU02	CONDUIT	0.059	0 13:34	0.44	0.10	0.37
CU03	CONDUIT	0.027	0 13:23	0.26	0.08	0.40
OUTLET_PIPE	CONDUIT	0.012	1 00:55	0.61	0.01	0.06
SW01.1	CHANNEL	0.025	0 13:00	0.04	0.01	0.31
SW01.2	CHANNEL	0.013	0 12:42	0.05	0.01	0.18
SW01.3	CHANNEL	0.001	0 12:47	0.01	0.00	0.10
SW02.1	CHANNEL	0.033	0 12:59	0.04	0.01	0.32
SW02.2	CHANNEL	0.018	0 12:41	0.05	0.01	0.20
SW02.3	CHANNEL	0.002	0 12:47	0.02	0.00	0.12
SW03	CHANNEL	0.063	0 13:52	0.07	0.03	0.38
SW04	CHANNEL	0.060	0 13:28	0.09	0.02	0.26
SW05.1	CHANNEL	0.038	0 12:51	0.06	0.01	0.25
SW05.2	CHANNEL	0.018	0 12:34	0.05	0.01	0.20
SW05.3	CHANNEL	0.003	0 12:31	0.02	0.00	0.12
SW06.1	CHANNEL	0.030	0 13:02	0.06	0.01	0.20
SW06.2	CHANNEL	0.012	0 12:33	0.04	0.00	0.19
SW06.3	CHANNEL	0.001	0 12:28	0.02	0.00	0.10
ORFC	ORIFICE	0.012	1 00:54			0.94
2&10YR_WEIR	WEIR	0.000	0 00:00			0.00
100YR_WEIR	WEIR	0.000	0 00:00			0.00

****
Flow Classification Summary
****

Conduit	Adjusted /Actual Length	---------- Fraction of Time in Flow Class ---------- Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl
CU01	1.00	0.03	0.00	0.00	0.95	0.03	0.00	0.00	0.00	0.79
CU02	1.00	0.05	0.00	0.00	0.95	0.00	0.00	0.00	0.00	0.76
CU03	1.00	0.02	0.00	0.00	0.98	0.00	0.00	0.00	0.00	0.79
OUTLET_PIPE	1.00	0.09	0.00	0.00	0.91	0.00	0.00	0.00	0.08	0.00
SW01.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.89	0.00
SW01.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.93	0.00
SW01.3	1.00	0.00	0.41	0.00	0.59	0.00	0.00	0.00	0.83	0.00
SW02.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.89	0.00
SW02.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.93	0.00
SW02.3	1.00	0.00	0.37	0.00	0.63	0.00	0.00	0.00	0.84	0.00
SW03	1.00	0.03	0.04	0.00	0.93	0.00	0.00	0.00	0.30	0.00
SW04	1.00	0.02	0.00	0.00	0.98	0.00	0.00	0.00	0.00	0.00
SW05.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.89	0.00
SW05.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.94	0.00
SW05.3	1.00	0.00	0.37	0.00	0.63	0.00	0.00	0.00	0.78	0.00
SW06.1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.33	0.00
SW06.2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.95	0.00
SW06.3	1.00	0.00	0.41	0.00	0.59	0.00	0.00	0.00	0.81	0.00

****
Conduit Surcharge Summary
****

No conduits were surcharged.

Analysis begun on: Fri Nov 6 11:38:01 2020
Analysis ended on: Fri Nov 6 11:38:13 2020
Total elapsed time: 00:00:12

Appendix D – Memorandum on Residual Risk assessment & River stopbank breach analysis

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Memo

To	Rick Liefting
CC	Ghassan Basheer
From	Gustaaf Kikkert
Reviewed by	Constantinos Fokianos
Date	6 November 2020
Job No.	144320
Job name	Shand Properties Rezoning
Subject	Residual Flood Risk

Introduction

BBO has been engaged by Shand Properties Limited (Shand) to support their submissions to the Proposed Waikato District Plan (PWDP). Shand are seeking to re-zone approximately 30.5 ha of land located in Huntly North from the current rural zoning to a mix of industrial (approximately 13 ha) and residential (approximately 17.5 ha) zoning.

As part of the application for the proposed rezoning, an investigation has been carried out to quantify the risks related to floodwater inundation during a scenario where the floodgates of the Kimihia stream at the point of discharge into Waikato River are closed. Based on the outcomes of this investigation, the proposed minimum floor levels were set to mitigate these flood risks up to the 100yr ARI storm event.

The residual flood risks to the property are therefore related to the Waikato River which runs parallel and in a close proximity to the proposed rezoning. A stopbank has been built between Thermal Explorer Highway and the Waikato River (Figure 1). The stopbank is designed to rural standards and has a 300mm freeboard above the 100yr Design Flood Level (DFL). The stopbank therefore mitigates the flood risk due to overtopping during the 100yr ARI water surface elevation event in the Waikato River.

However, flood waters may still reach the property during the 100yr ARI water surface elevation event if breaches occur in the stopbank. To determine a suitable procedure to assess this residual flood risk, a hydraulic model was developed to enable breaches in the stopbank to be simulated and the impact of these breaches to be analysed. The hydraulic model was set up in HEC-RAS.

This memo gives a brief description of the model, followed by the results from the model to highlight the impact of breaches in the stopbank on Area 1 that is proposed to be rezoned for industrial use.

Model Description

A two-dimensional hydraulic model was developed in HEC-RAS. The terrain used for the HEC-RAS model was based on WRC LiDAR data. The terrain data was modified, and breaches were added into the stopbank on two locations. These locations were decided in coordination with WRC, which is responsible for the Kimihia stopbank. Simulations were for the existing situation and for the future development based on a high-level concept design of the important stormwater infrastructure. The development of the model, including the input parameters and the assumptions included, are presented below.

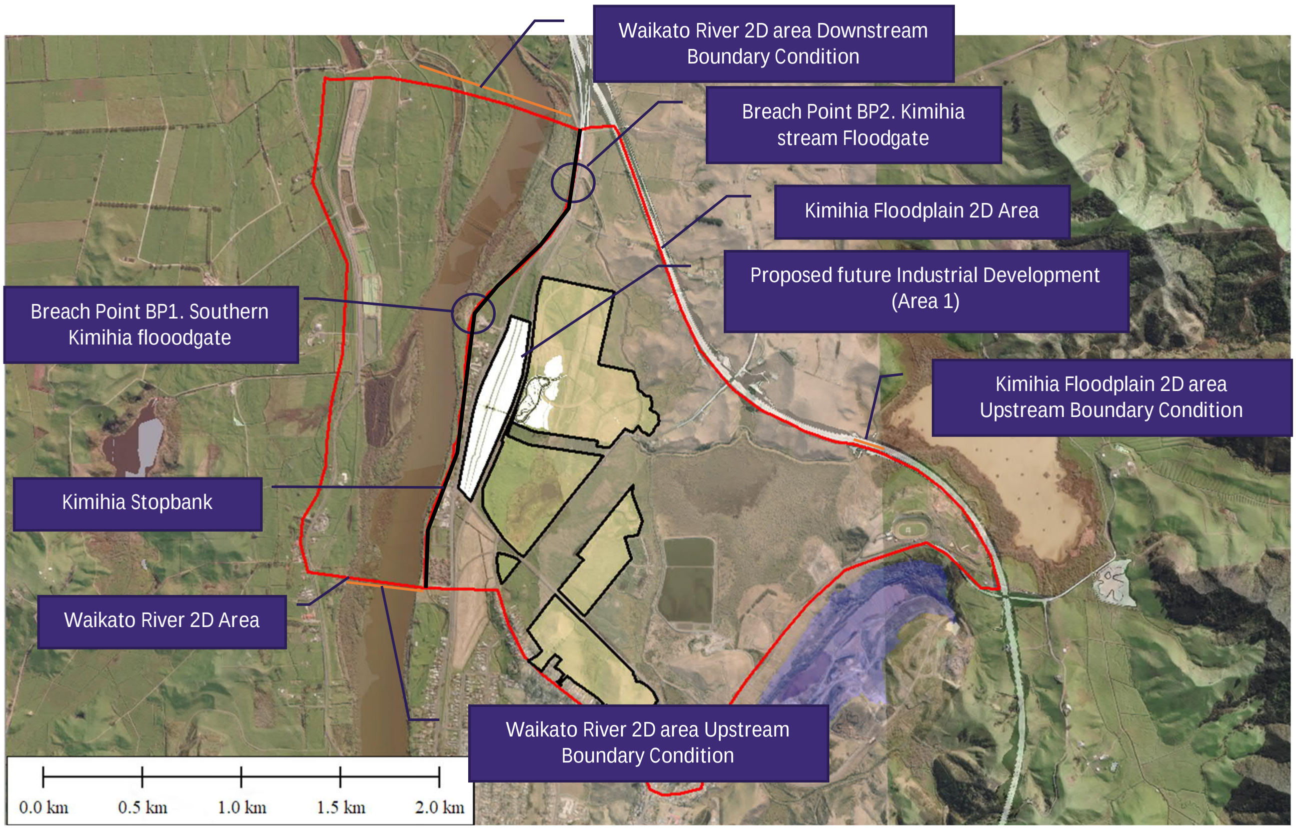

Figure 1 - Area of Interest for Simulation of stopbank breaches

Initial Water Surface Levels

WRC information yielded a 100yr ARI water surface level for the Waikato River at the Kimihia floodgate outlet of approximately R.L. 10.96m. Upstream of the Kimihia floodgate outlet, the 100yr ARI water surface level may be slightly higher, while downstream it may be slightly lower. To simplify the model, flow in the Waikato River is modelled as a two-dimensional area with boundary conditions upstream and downstream with a constant water surface level of R.L. 10.96m (Figure 1).

During the 100yr ARI water surface level event, the water surface level will only be at the maximum for a certain amount of time, but a temporal pattern of the water surface level is not available. Running the simulation with a constant maximum water surface level will overestimate the volume of flood water through the breaches and therefore yield a conservative estimate for the residual flood risk due to those breaches.

LiDAR data obtained from WRC have been used as the terrain data for the two-dimensional HEC-RAS model. The LiDAR data included the streams that are part of the Kimihia drainage scheme that also drains Area 1. The LiDAR has not been modified to obtain the actual stream invert elevations.

It is likely that the 100yr ARI water surface level event for the Waikato River coincides with elevated water levels in the Kimihia drainage streams because the flap gates at the Kimihia floodgate outlet will be closed for some time. However, increasing the water surface elevations reduces the difference in the water levels on either side of the stopbank and hence reduces the impact of the breaches. For an assessment of the residual risk due to breaching of the stopbank, the maximum impact has been simulated. The water surface levels in the Kimihia drainage scheme have therefore not been increased.

The surface water levels in the Kimihia drainage scheme were set to non-storm levels. It is assumed that the surface elevations as given by the LiDAR data are representative water surface levels during these conditions.

Stopbank Breaches

WRC advised that breaches should be modelled at the southern end of the Kimihia stopbank and near the first southern floodgate within Kimihia stopbank. The first of these is opposite the intersection with East Mine Road and the second behind the truck stop.

The LiDAR information in the area opposite the intersection with East Mine Road indicates that the elevation of the land behind the stopbank is at or is higher than the R.L. 10.96m and hence no or very little water would flow from the Waikato River during the 100yr ARI water surface level event even with a breach in the stopbank at this location. Land behind the stopbank remains relatively high until the North End Motel.

The locations chosen for breaches in the stopbank were therefore north of the North End Motel. The first simulated breach point is at the first southern floodgate within Kimihia stopbank (Figure 1). Floodwater from this breach will fill up the paddock between the Thermal Explorer Highway and the stopbank and then flow across the highway into Area 1.

The second simulated breach point is located further north, near the Kimihia Floodgate outlet (Figure 1). Floodwaters from this breach will first fill up the swale between the Thermal Explorer Highway (old SH 1) and the stopbank. Floodwaters will enter Area 1 when the highway overflows south of the Fisher Road intersection.

The breaches in the stopbank were modelled by modifying the terrain at the breach locations. Two breach lengths were chosen to represent a narrow breach (10m) and a wide breach (30m). The level of terrain in the breach was set equal to the level of the land immediately downstream of the stopbank (10.25m for the first and 10.18m for the second breach point).

Model Set Up

With the initial water surface level of the Waikato River set to R.L. 10.96m, the potential flooding area due to a breach in the stopbank was identified. The available LiDAR and additional as built information from the Huntly section of the Waikato Expressway was transformed into terrain data for HEC-RAS. The area below the R.L. 11.0m contour was set up as a two-dimensional flow area (Figure 1).

To investigate the impact of flooding due to a breach in the stopbank on the proposed future scenario, simulations were carried with a modified terrain. These modifications were based on a high-level concept design of the stormwater infrastructure. The levels of the relevant stormwater infrastructure, including swales, the wetland, the road and elevated sections, were added to the terrain data for these simulations. The changes to the terrain are presented in Figure 3.

The initial cell size for the 2D flow area was 50m by 50m. To obtain refinement of cells, break lines were included along the stopbank, the Thermal Explorer Highway and the North Island Main Trunk railway line. The near spacing of cells along the stopbank was 10m and the near spacing of cells along the highway and railway was 5m. Additional refinement was added to the cell structure near the breach locations. The refinement region had cell sizes of 2m by 2m. An example of the cell structure of the model, focusing on the location of Area 1, is shown in Figure 2.

Figure 2 - Kimihia Stopbank Breach Simulation Model Set Up for Existing Conditions

For the simulations of the proposed future scenario, additional break lines were inserted to obtain cell refinement along the proposed stormwater infrastructure (Figure 3). The breaklines along the swales and proposed road edges had a near spacing of 5m, or even 2m meter. This increased the number of cells covering the 2D flow area.

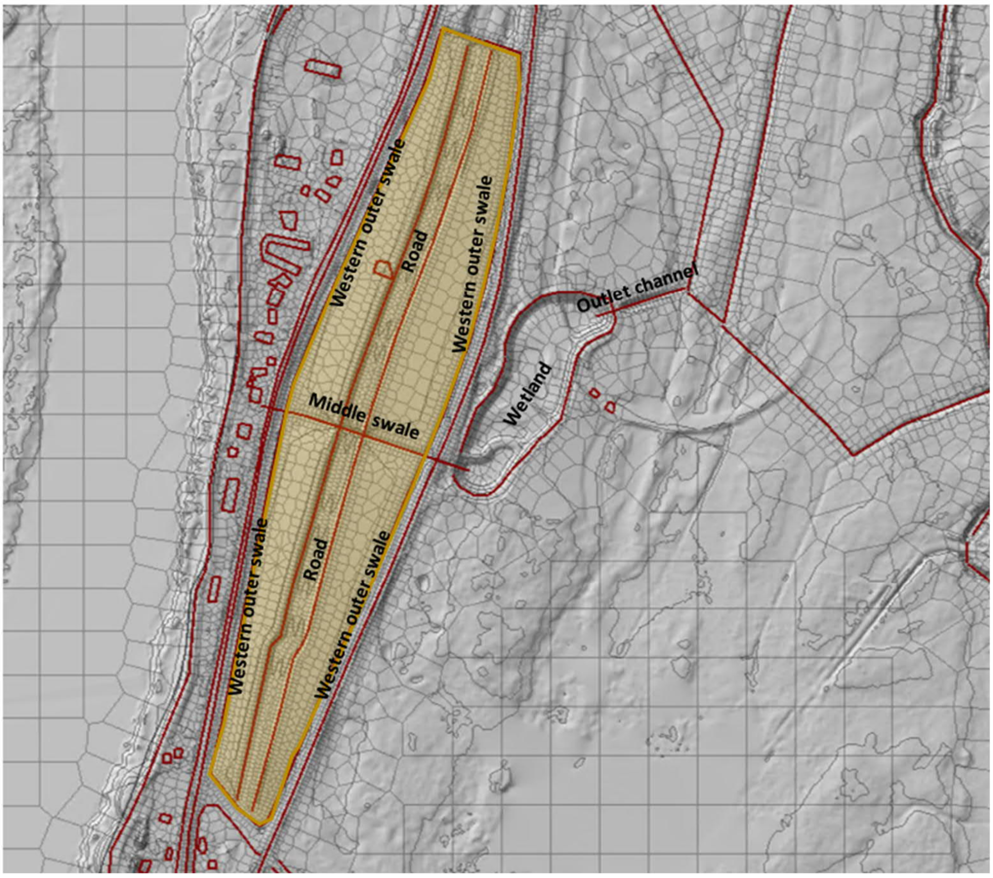

Figure 3 – Development Area Stormwater Infrastructure Model Set Up

For the majority of the 2D flow area, the flow is overland sheet flow. For the grassed areas, the roughness coefficient, Manning's n, has been set to 0.15 (brown in Figure 2). To include the impact of buildings on the flow, the roughness coefficient for buildings has been set to 1 (pink in Figure 2). Finally, the asphalt roads were identified in the model and given a roughness coefficient of 0.013 (blue in Figure 2).

Three boundary conditions were added to the model. The first two were the simulation of the Waikato River 2D area. The water surface level was set to 10.96m throughout the simulation. The second was at the culvert underneath SH 1, immediately downstream of Lake Kimihia. This uniform flow boundary condition based on normal flow depth enabled water to leave the 2D flow area if the flood waters reached this location.

With the two different breach points, two different breach lengths and two different terrains, a total of eight breach simulations were carried out. The results are briefly discussed below.

Model Results

For each of the breach simulations carried out, flood waters started to travel through the breach at t = 0 and the simulations ended after 36hrs. Example results of the propagation of the flood waters due the breaches of the stopbank are presented in Figures 4 and 5. The figures show the inundation extents and the flow velocities at five different times and eight different scenarios. These are the scenarios with the existing and proposed terrain with a 10m and a 30m breach occurring near the southern floodgate of Kimihia stopbank (BP 1), and in the stopbank near the Kimihia Floodgate outlet (BP 2).

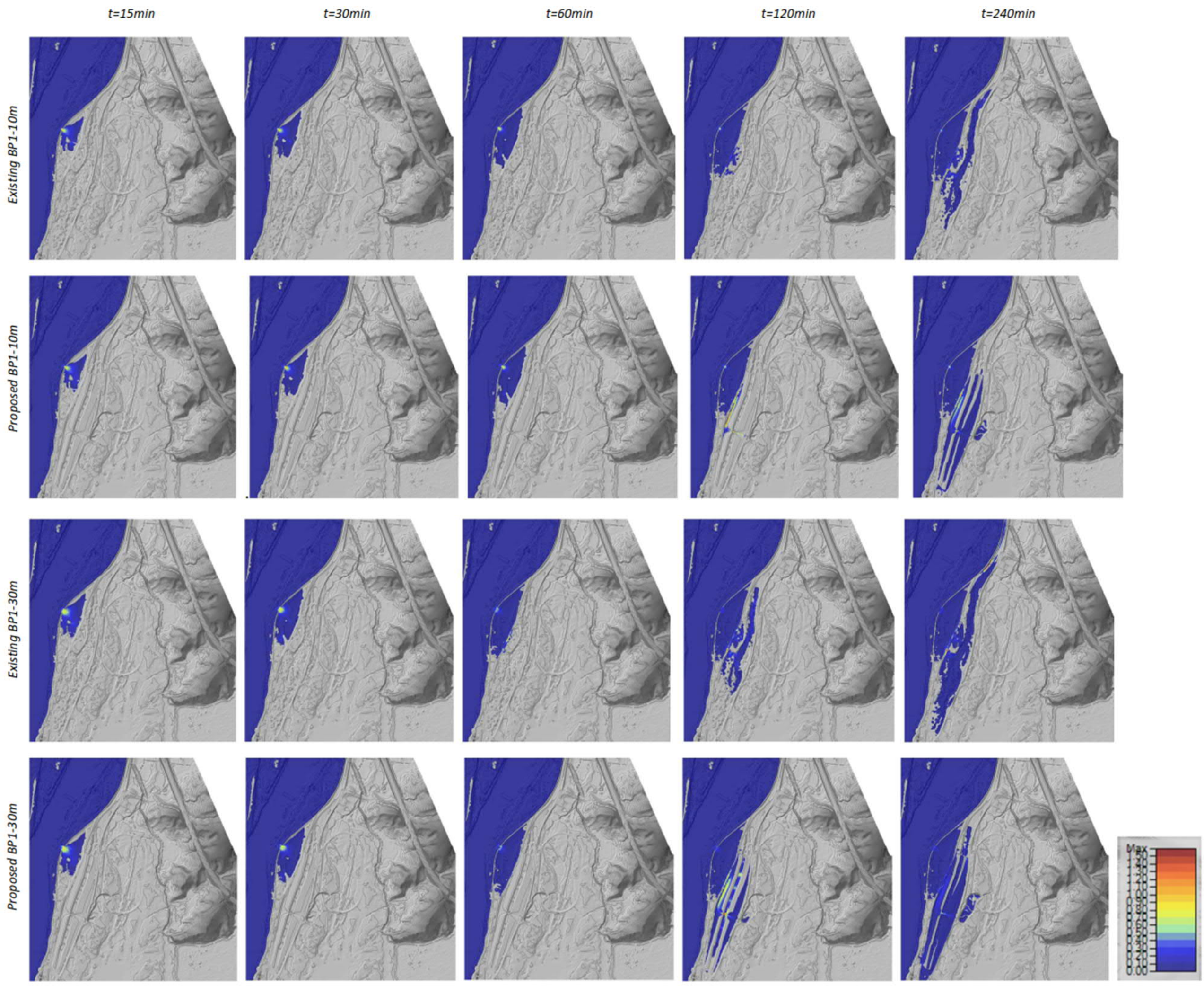

Figure 4 – Example HEC-RAS inundation extents and flow velocity results – BP1

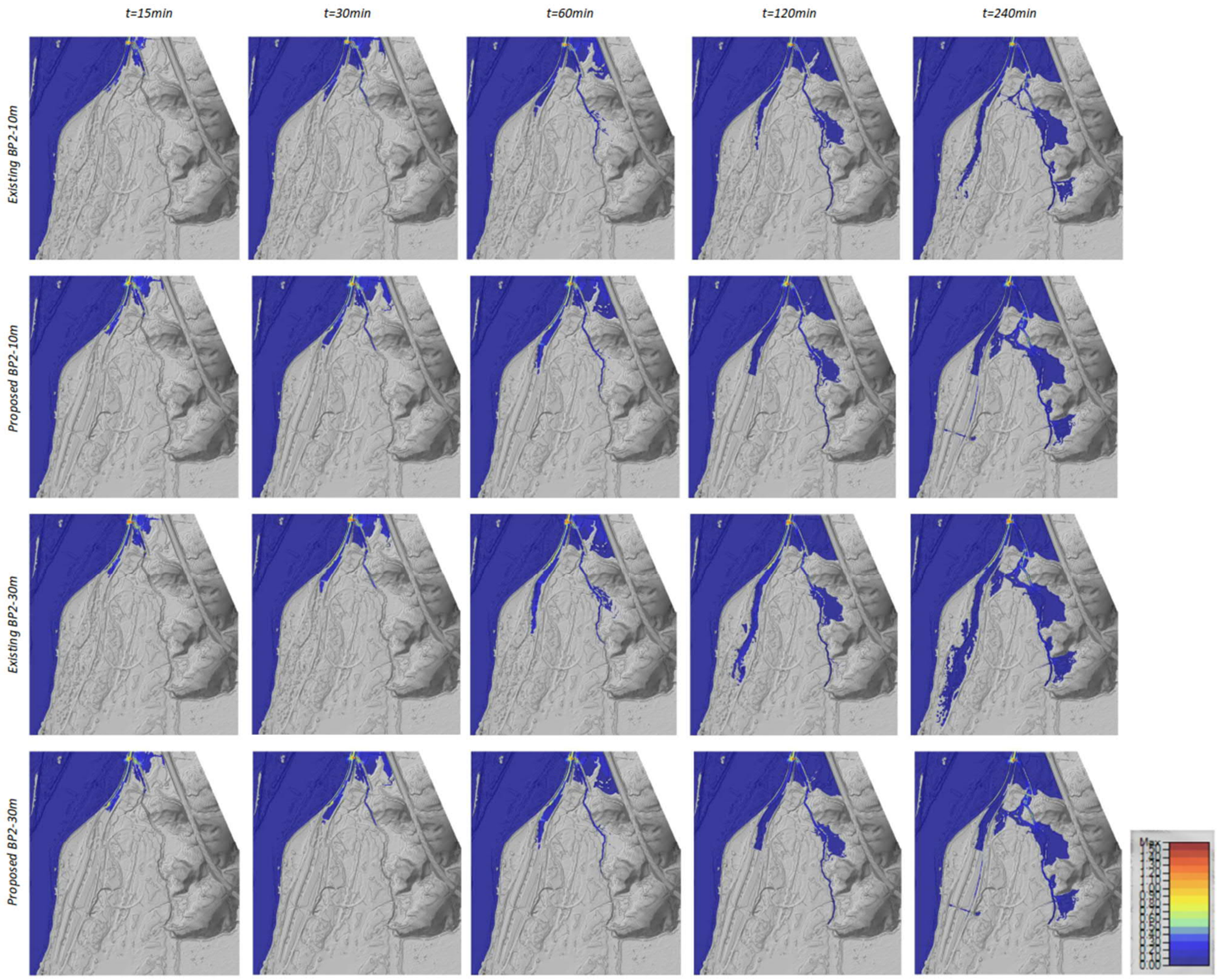

Figure 5 – Example HEC-RAS inundation extents and flow velocity results – BP2

The example results show that flood water is conveyed through different paths for the two breach points. While for the breach point 1 (southern floodgate), the majority of the floodwater flows eventually through the proposed rezoning, a breach at the existing Kimihia flood gates will only send part of the flood wave through the property, while the rest of the flow will overtop Thermal Explorer Highway and the railway and discharge into the Kimihia stream and propagate upstream towards the floodplain.

Table 1 – Propagation of flood waters from breach to Area 1.

Simulation	Flood waters enter development area	Flood waters reach road near middle swale	Flood waters reach southern boundary
Existing - BP1 – 10m	1hr 40min	02hr 20min*	6hr 05min
Existing - BP1 – 30m	0hr 53min	01hr 22min*	3hr 54min
Existing – BP2 – 10m	1hr 40min	03hr 39min*	10hr 36min
Existing – BP2 – 30m	0hr 43min	01hr 36min*	4hr 13min
Proposed - BP1 – 10m	1 hr 40min	02hr 10min	2hr 58min
Proposed - BP1 – 30m	0hr 58min	01hr 22min	1hr 40min
Proposed – BP2 – 10m	1hr 50min	25hr 47min	more than 36hr
Proposed – BP2 – 30m	1hr 07min	6hr 05min	18hr 09min

* based on location of road near middle swale in concept design

The propagation of the flood waters for all scenarios is summarized in Table 1 which gives the time when the flood waters first reach Area 1, the road level near the middle swale of the development (or the location of the road for the existing situation) and the southern boundary of Area 1.

The results indicate that the time between the stopbank breaching and flood waters reaching Area 1 is almost similar, and it is smaller when the 30m breach occurs in the stopbank near the Kimihia Floodgate outlet (Breach Point 2 – BP 2). The flood waters quickly fill the swale along the highway and almost immediately spill across the highway at multiple locations. One of these spills into the paddock south of the Fisher road intersection. The southern boundary of this paddock is the northern boundary of Area 1. The minimum time is just over 40 minutes.

If the breach occurs near the southern floodgate of Kimihia stopbank (Breach Point 1 – BP 1), the flood waters first fill the area between the stopbank and the highway before spilling over the highway into Area 1. The wider breach results in a smaller time for both BP 1 and BP 2. Results from the existing and proposed scenarios are the same as the modified terrain only influences the flow path of the flood waters after reaching Area 1.

The point along the Thermal Explorer highway where flood water from BP 1 spills onto Area 1 is further south than the northern boundary of the property where flood water from BP 2 enters the property (see Figure 5). The additional time it takes for the flood water to propagate across the property to the road near the middle swale (which is approximately in the middle of the property) is therefore generally smaller for floods from BP 1. As a result, the difference between the total times from BP 1 and BP 2 scenarios is smaller.

The modified terrain of the proposed scenarios does alter the flow paths for both the flood waters from BP 1 and BP 2. This affects the routing time differently for the two breach points. Taking the terrain modifications into account, the results indicate that a minimum time for flood waters to reach the road in the middle of the property is about one hour and twenty minutes for BP1, while for BP2 the times have been increased substantially in relation to the existing conditions scenario, indicating that the proposed swales could route the majority of the floodwave around the property and into the proposed wetland protecting the proposed rezoning from inundating.

For the proposed scenarios, the flood wave from BP 1 initially travels along the western swale (Figure 5). After it reaches the middle swale, the flow starts to spread out into the eastern swale, the wetland and onto the proposed road. When the flood wave comes from BP 2, the main flood wave on Area 1 is initially along the proposed road and the eastern swale. After reaching the middle swale, the flow also spreads into the western swale and the wetland.

The swale along the southern boundary of the property is relatively high to enable stormwater to drain to the middle swale and from there to the wetland. This increase in elevation means that it takes time for the water depth to become sufficiently high to enable the flood waters to propagate from the middle swale all the way to the southern boundary. For the proposed scenarios, flood waters are able to leave Area 1 into the wetland, reducing the volume of water available for propagation of the flood wave to the southern boundary of Area 1.

Example velocity results are presented in Figure 6. The velocity results indicate that the locations with the highest velocities coincided with the locations with greatest elevation changes, i.e. the breach points and locations where the flow spilled across the highways. Downstream of BP 1, velocities are around 0.8m/s (Figure 5) with depths up to 0.7m while downstream of BP2 velocities are around 2m/s with depths around 0.6m.

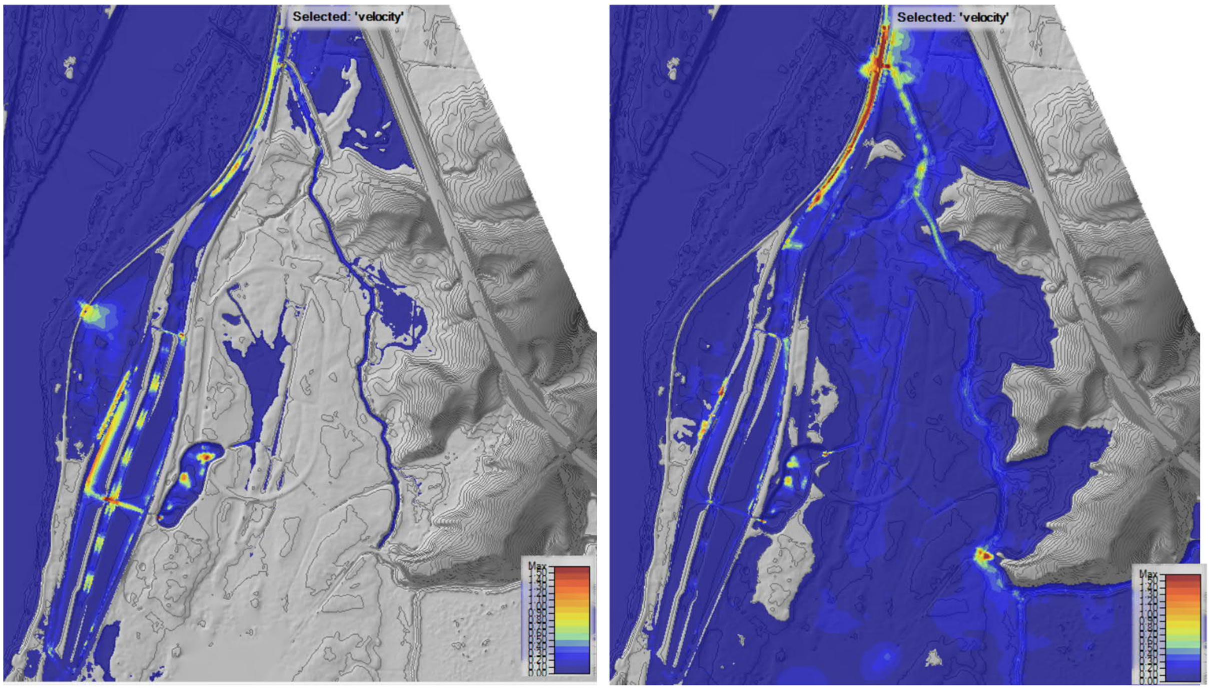

Figure 6 – Example HEC-RAS maximum velocity results – proposed scenarios with BP 1 (left) and BP 2 (right)

On Area 1, the maximum velocities are lower. For the existing scenarios, the maximum velocity is about 0.25m/s (not shown). For the proposed scenario, the maximum velocities occur in the swales at up to 0.4m/s (Figure 6). Along the road, the maximum velocity is about 0.8m/s while on the sections away from the overtopping area on Thermal Explorer Highway the maximum velocity is less than 0.1m/s.

Conclusions

To investigate the residual flood risk due to breaches in the Waikato River stopbank on Area 1, a 2D HEC-RAS model was developed. Simulations were run with the existing terrain and a modified terrain based on the proposed stormwater infrastructure from the high-level concept design. Breaches were inserted into the Waikato River stopbank at two different locations. The first was near the southern floodgate of the Kimihia Stopbank and the second near the Kimihia Floodgate outlet. For each of the breaches, simulations were run with a narrow breach length (10m) and wide breach length (30m).

Several important assumptions had to be made during the development of the HEC-RAS simulations and these assumptions will have a significant impact on the propagation and inundation results of the flood waves. The most important assumptions include:

- A constant flow depth in the Waikato River at the estimated 100 yr ARI water surface level.
- Almost Instantaneous breach (0.1hrs) of the stopbank over its full length and down to the surface level immediately downstream of the breach point.
- No attempt is made to fill the breach in the stopbank throughout the simulation.
- Land downstream of breach point is not already inundated due to flood waters from the upstream catchments.

The results from the simulations indicated that the minimum time it takes for the flood wave to propagate from a breach to Area 1 is approximately 40min. However, if the breach happens near the southern floodgate this will take at least another 15 minutes.

After reaching Area 1, propagation of the flood wave slows down. Minimum time to reach the road at approximately the middle of Area 1 is approximately 80 minutes. The modified terrain based on the proposed stormwater infrastructure adds significantly more time when the breach occurs close to Kimihia floodgate.

During the first 90 minutes after formation of the breach in the Waikato River stopbank, the maximum velocity along the proposed road on Area 1 is approximately 0.5m/s and the maximum inundation depth is about 0.14m. Velocities in the proposed swales are indicated to be up to 0.4m/s. Velocities at the majority of the indicative industrial lots were not greater than 0.1m/s throughout the 36hr simulations.

On the BP2 scenario, the proposed terrain provides significant protection of the industrial area, with the swales routing the majority of the floodwave around the zone and into the proposed wetland.

Yours sincerely
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