

SUMMARY MEMORANDUM

TO: Jake Woodward, Jake Woodward Resource Management Planner

FROM: James Hadley, Principal, Hadley Consultants Ltd

REVIEWED BY: Nigel Lloyd and Cameron Kerr, Hadley Consultants Ltd

DATE: 14 May 2026 – For Issue

SUBJECT: Shannon Farm Residential and Commercial Extension - 3 Waters Feasibility
Assessment for Fast-track Referral.

PROJECT: 254029

1.0 Introduction

Hadley Consultants Limited (HCL) have been engaged by Rippon Holdings Limited (RHL) and Rippon Investments Limited (RIL) to assess and report on the 3 Waters Servicing Feasibility for the proposed Shannon Farm Residential and Commercial Extension (SFRCE) proposal at Ripponvale Road, Cromwell.

This memorandum has been prepared to assist and supplement the Fast-track Referral Application prepared by Jake Woodward, Resource Management Planner, and Anderson Lloyd, on behalf of RHL and RIL. The memorandum outlines the nature of the proposed development and considers the demand parameters to apply for potable water supply, wastewater and stormwater, together with the available servicing capacity in the existing and proposed Central Otago District Council (CODC) 3 Waters infrastructure network.

The purpose of this memorandum is to demonstrate the considerable work that has already been undertaken to assess the 3 Waters demands arising from the SFRCE proposal, that CODC have been consulted over the future connection of the SFRCE proposal and that the CODC and SFRCE engineering teams are working together to confirm what, if any, capital upgrades are required to the existing CODC 3 Waters network to provide servicing to the SFRCE site.

We note that prior reporting which is referenced within and attached to this memorandum refers to the project as Cherry Orchard Development (COD), which was the prior working title for the development. For the avoidance of doubt, the prior reporting referencing COD is referring to this SFRCE, which is a change in name only.

2.0 Prior CODC Consultation and Previous Demand and Capacity Assessments

HCL have conducted previous work on demand and capacity assessments and have previously engaged with CODC over the capacity and potential constraints of CODC's existing water supply and wastewater networks. This prior consultation and engagement has included prior reporting by HCL comprising;

- 1) CODC Wastewater and Portable Water Network Review – 19 November 2025. (refer Attachment A)
- 2) Cherry Orchard Development – Water and Wastewater Demand Assessment – First issued for consultation 10 March 2026. (refer Attachment B, revision 1 dated 12 May 2026)

Meetings were held with CODC following issue of both the above technical reports. The meetings with CODC confirmed that;

- a) CODC are in general agreement that the CODC water supply network already has significant capacity to meet future potable water supply demands to 2030 and beyond. However, it was acknowledged that the capacity primarily exists in the headworks of the network (being existing water intakes and extraction consents). Piped reticulation constraints do exist in some locations, however it was agreed that these can be resolved by localised capital works upgrade projects or renewal works.
- b) CODC are also in general agreement that whilst there is less headroom in the capacity of their wastewater headworks (treatment + disposal + discharge consents), there is still residual capacity which can be further expanded in the future under their capital programme once they have confirmed what treatment standard they must meet.
- c) With regard to the SFRCE Fast-track proposal, and the associated Demand Assessment (Attachment B), CODC agreed that they would take forward the HCL Demand Assessment in to their network model assessments and stated that they acknowledged that any future capacity constraints for either potable water supply or wastewater could ultimately be addressed and resolved by way of appropriate capital works investment.
- d) It was agreed with CODC that as the SFRCE project proceeded through the Fast-track process, CODC and the SFRCE's engineering teams would work together to complete the necessary network modelling of development uptake (increased demand over time) to confirm the capital works upgrades necessary to achieve 3 waters servicing of the full development of the SFRCE project.

In addition to the work and discussion on Water and Wastewater, HCL have also noted to CODC that they have undertaken detailed catchment modelling of the wider Ripponvale Road catchment. As part of that work, CODC have acknowledged that they do not hold reliable as-built or GIS records of their own stormwater network east of the intersection of Ripponvale Road and State Highway 6. CODC are in the process of developing their own stormwater network models and Catchment Management Plans, but these are currently a work in progress and are not available for use at this time. For that reason, HCL have allowed for advancing entirely independent stormwater reticulation solutions to address runoff from the proposed SFRCE project.

3.0 Nature of Proposed Development and Land Use Demand

The SFRCE site is located on Lots 1 to 4, DP344529, Lot 1 to 4 DP420009 and part of Lot 1000, DP61599 and is approximately 69.8 hectares in area.

The development yield allowances have been developed collaboratively with input from HCL, Rough Milne Mitchell (RMM), and the wider SFRCE project team. Ultimately the RMM Development Masterplan as indicated in Figure 1 below has been adopted as representing the anticipated ultimate development density and yield across the various land use typologies that are proposed, including various residential, retail and industrial activities within the SFRCE site.



Figure 1 – SFRCE Masterplan, RMM, 06 May 2026

Based on our assessment of the RMM Development Masterplan the ultimate development yield is as shown in the following table.

Development Type	Area (Ha)	Lots
Medium Density Residential Lots	-	94
Low Density Residential Lots	-	581
Large Lot Residential Lots	-	80
Commercial (Light Industrial & Retail)	8.29	-
Total	8.29Ha	755 Lots
	(Commercial)	(Residential)

Table 1 – SFRCE Site Ultimate Yield

The yield as detailed above defines the development and forms the basis upon which demand generation has been assessed for potable water supply servicing and wastewater servicing. Similarly, the Masterplan and land use typologies have informed the assessment of stormwater runoff.

The residential lot creation figures above, and the area allowances for Industrial and Retail land use have been examined in detail within the Water and Wastewater Demand Assessment Memorandum included as Attachment B. Similarly, the land use allowances in the Masterplan form the basis of the analysis of the stormwater effects arising from the SFRCE proposal and these are documented in detail in the Stormwater Feasibility Memorandum included as Attachment C.

4.0 Potable Water Demand Assessment and Available Capacity

This section outlines the potable water demand adopted for the assessment of the SFRCE proposal. It is important to note the figures stated herein are ultimate development demands (eg full development of the SFRCE proposal over an 8 – 12 year adoption horizon) and as such are considered upper bound limits.

The key water demand allowances considered and adopted are summarised as follows;

- Per Capita Water Demand = 500 L/person/day
- Peak Day Peaking Factor = 3.0 (ADF to PDF)
- Peak Hour Peaking Factor = 5.0 (ADF to PHF)
- Minimum Fire Water Classification = FW3, 50L/s instantaneous for 60mins and 180m³ storage

The resultant ultimate peak water demand arising from the COD proposal is assessed as shown in the following table.

Development Type	Average Day Flow (ADF)	Peak Day Flow (PDF)	Peak Hour Flow (PHF)	Minimum Fire Flow (FF)	
Medium Density Residential Lots	141m ³ /day	423m ³ /day	8.2L/s		
Low Density Residential Lots	1,274m ³ /day	3,822m ³ /day	73.7L/s		
Large Lot Residential Lots	175m ³ /day	526m ³ /day	10.2L/s		
Retail and Industrial	58m ³ /day	173m ³ /day	3.3L/s		
Total	1,648m ³ /day	4,944m ³ /day	95.4L/s	50L/s	180m ³

Table 2 – SFRCE Ultimate Peak Water Demand

The SFRCE proposal will include servicing by a new reservoir(s) at the expanded Shannon Farm water reservoir site which is already vested in CODC ownership. Reticulation flows and the proposed fire fighting and peak hour flow water demands will be provided for internally within the SFRCE site and will not have a material impact on the external CODC water supply network. That is, the key ultimate demands on the CODC network relate to the supply side (rising main delivery) from existing CODC water supply headworks assets. Therefore, the key ultimate demand figures relevant to the CODC network supply capacity are the average day and peak day volumetric flow figures as presented above in Table 2 of 1,648m³/day and 4,944m³/day.

Whilst the ultimate average and peak daily demands are not insignificant, it is important to note that the ultimate demands will develop to these peak levels over a period of 8 to 12 years, thereby allowing progressive headworks upgrades to meet demand. This is a typical eventuality across any Council network with an increasing population.

The Wastewater and Potable Water Network Review conducted in November 2025 confirmed that for Water Supply CODC are currently estimated to be using approximately 50% of their consented maximum abstraction rate of 18,000 m³/day. This indicates that there are currently no headworks limitations to increasing daily water supply volumes to service new development areas other than localised constraints in the existing CODC reticulation network which can be addressed by relatively minor pipe upgrade works. Therefore, with the appropriate reticulation and reservoir upgrades, it is potentially feasible for CODC to service the early stages of the SFRCE development under their existing water extraction consent entitlements.

CODC and SFRCE engineering teams are presently working to model the existing CODC network in detail to determine the specific nature and timing of capital upgrades required to the CODC network, however it is agreed that water supply to the SFRCE to meet the ultimate demand level is feasible once the capital upgrades have been implemented.

6.0 Wastewater Demand and Available Capacity

This section summarises the wastewater demand adopted for the assessment of the SFRCE proposal. Again, it is important to note the figures stated herein are ultimate development demands (eg full development of the SFRCE proposal over an 8 – 12 year adoption horizon) and as such are considered upper bound limits.

The key wastewater generation allowances considered and adopted are summarised as follows;

- Per Capita Wastewater Generation = 250 L/person/day
- Diurnal Peaking Factor = 2.5 (ADWF to PDWF)
- Wet Weather Peaking Factor = 2.0 (PDWF to PWWF)

The resultant ultimate peak wastewater generation arising from the SFRCE is presented in the following table.

Development Type	Average Dry Weather Flow		Peak Dry Weather Flow (Diurnal)	Peak Day Wet Weather Flow	
Medium Density Residential Lots	70.5m ³ /day	0.8L/s	2.0L/s	141m ³ /day	4.1L/s
Low Density Residential Lots	637.1m ³ /day	7.4L/s	18.4L/s	1,280m ³ /day	37.0L/s
Large Lot Residential Lots	87.7m ³ /day	1.0L/s	2.5L/s	175m ³ /day	5.1L/s
Retail and Industrial	57.3m ³ /day	0.7L/s	1.7L/s	117m ³ /day	3.4L/s
Total	853m³/day	9.9L/s	24.7L/s	1,710m³/day	49.5L/s

Table 3 – SFRCE Ultimate Wastewater Generation

Note that the retail and industrial figures presented above have been back calculated from peak flow using the same peaking factors as applied to residential wastewater generation.

The SFRCE proposal will include servicing by way of connection to existing CODC wastewater reticulation east of State Highway 6. The peak wet weather flows of 1,710m³/day and 49.5 l/s are known to exceed the capacity of the existing downstream gravity reticulation in the CODC pipe network, and sections of the CODC network connecting the subject site to the Cromwell Wastewater Treatment Plant (WWTP) will be required to be upsized and upgraded as identified on Drawing 002 in Attachment A.

The Wastewater and Potable Water Network Review conducted in November 2025 confirmed that for Wastewater the capacity of the CODC system is currently limited by the total daily volume of nitrogen discharged under the CODC Discharge Consent from Otago Regional Council. CODC are aware of this issue and already have an \$11.3M WWTP upgrade provisioned in their Long Term Plan. Previously, this WWTP upgrade was proposed to occur in 2031, but we understand CODC are considered bringing this timing forward as a result of increasing development demand. CODC are also in a phase where they are waiting to confirm the discharge quality standards to be achieved under new nationwide policy guidelines.

Again, whilst the ultimate peak wet weather wastewater flows are not insignificant, it is important to note that the ultimate demands will develop to these peak levels over a period of 8 to 12 years, thereby by allowing progressive headworks upgrades to meet demand. This is a typical eventuality across any Council network with an increasing population.

Like the water supply capacity, the CODC and SFRCE engineering teams are presently working to model the existing CODC network in detail to determine the specific nature and timing of capital upgrades required to the CODC network, however it is agreed that connection of the SFRCE site and discharge of wastewater to the CODC wastewater network is feasible once capital upgrades have been implemented.

7.0 Stormwater Management

HCL have completed a comprehensive catchment modelling exercise of the Shannon Farm and Ripponvale Road catchment to examine pre and post development flows associated with the SFRCE proposal.

The stormwater modelling and analysis work utilises TUFLOW software to quantify the likely surface flooding effects of the proposed development. The TUFLOW model and analysis has also been calibrated against known and recent flood events to verify the model output. The analysis is fully documented in the HCL memorandum titled "Cromwell Cherry Orchard Development Stormwater Feasibility" dated 11 May 2026 and included as Attachment C to this memorandum.

The findings and key aspects of the stormwater analysis are summarised as follows;

- i. The existing undeveloped catchment already results in ponding and inundation beyond the site boundaries. This is evidenced by the recent inundation of areas either side of State Highway 6 to the southeast of the subject site.
- ii. Pre and post development analysis of the 1% AEP storm event shows that following incorporation of detention and attenuation devices within the subject site and within adjacent land (being the Shannon Farm Subdivision in common ownership) there is no increase in flooding impacts to surrounding properties. This is illustrated within Figure 18 of the Attachment C which, by colour coding, shows "Flooding Introduced" and "Flooding Removed".
- iii. The "white" colour in Figure 18 of Attachment C shows areas where there is no change in flooding impact. Examination of Figure 18 highlights that the only change in flooding impact is within the subject site and the neighbouring upslope land in common ownership where stormwater detention and attenuation devices are proposed. The "white" area tracking down the true left or north side of Ripponvale Road and the large "white" area encompassing State Highway 6 itself southeast of the subject site, confirms that there is an existing and unresolved flooding and inundation issue in this area under the 1% AEP storm event.
- iv. To provide flexibility for improvement of pre-existing flood risks, HCL have developed a stormwater solution that can be used to both ensure that development of the SFRCE site does not exacerbate any pre-existing flood hazard on neighbouring land and to provide the opportunity for CODC to resolve the existing flood hazard which exists to the southeast of the subject site.
- v. The stormwater solution proposed by HCL involves the construction of new inlet and sub-surface pipeline from Ripponvale Road, south along SH6 and then along McNulty Road and Barry Avenue to a discharge point at the Kawarau River. Preliminary design work involving hydraulic analysis and pipe long section development has been completed to confirm the feasibility of the proposed stormwater management solution.

- vi. Opportunities for optimisation of the proposed stormwater management solution exist and will be discussed and evolved with CODC through developed design. These discussions will inform whether CODC wish to take the opportunity to resolve pre-existing flood hazard issues by utilising the proposed McNulty Road pipe reticulation.

Again, like the water supply capacity and wastewater capacity issues, the CODC and SFRCE engineering teams are committed to working together to resolve the final stormwater solution to be adopted, based on the detailed analysis already undertaken for the SFRCE site. In summary, a feasible option for the safe control and disposal of stormwater from the SFRCE site exists, together with an opportunity for CODC to improve and enhance a pre-existing flood hazard elsewhere in their network.

7.0 Development Uptake Considerations

With infrastructure investment likely to be required to service the ultimate water demand and wastewater generation figures assessed for the full construction and development SFRCE proposal, it is important to appreciate the lead time and development uptake associated with such a development. The lead time provides time for any necessary large scale infrastructure headworks investment to be achieved. Therefore, to assist with infrastructure master planning and potential network upgrades we have also considered the likely development uptake and resultant demand over time.

At this stage the SFRCE is anticipated to occur over an estimated 8 to 12-year timeframe with the first development stages being completed in 2029. For this assessment we have considered two potential development scenarios being a fast-progression and a slow-progression pathway outlined as follows.

- The fast development scenario assumes that the full development is completed over a duration of 8 years with an initial surge of residential development occurring in 2029 (year 1) with the remainder spread evenly across years 2 to 8 and the retail and industrial development delivered rapidly in years 2 and 3.
- The slow development scenario assumes that the full development is completed over a duration of 12 years with an initial surge of residential development occurring in 2029 (year 1) with the remainder spread evenly across years 2 to 12 and the retail and industrial development delivered evenly across years 3 to 5.

The assumed residential and commercial development uptake across the two scenarios considered is indicated in the following chart.

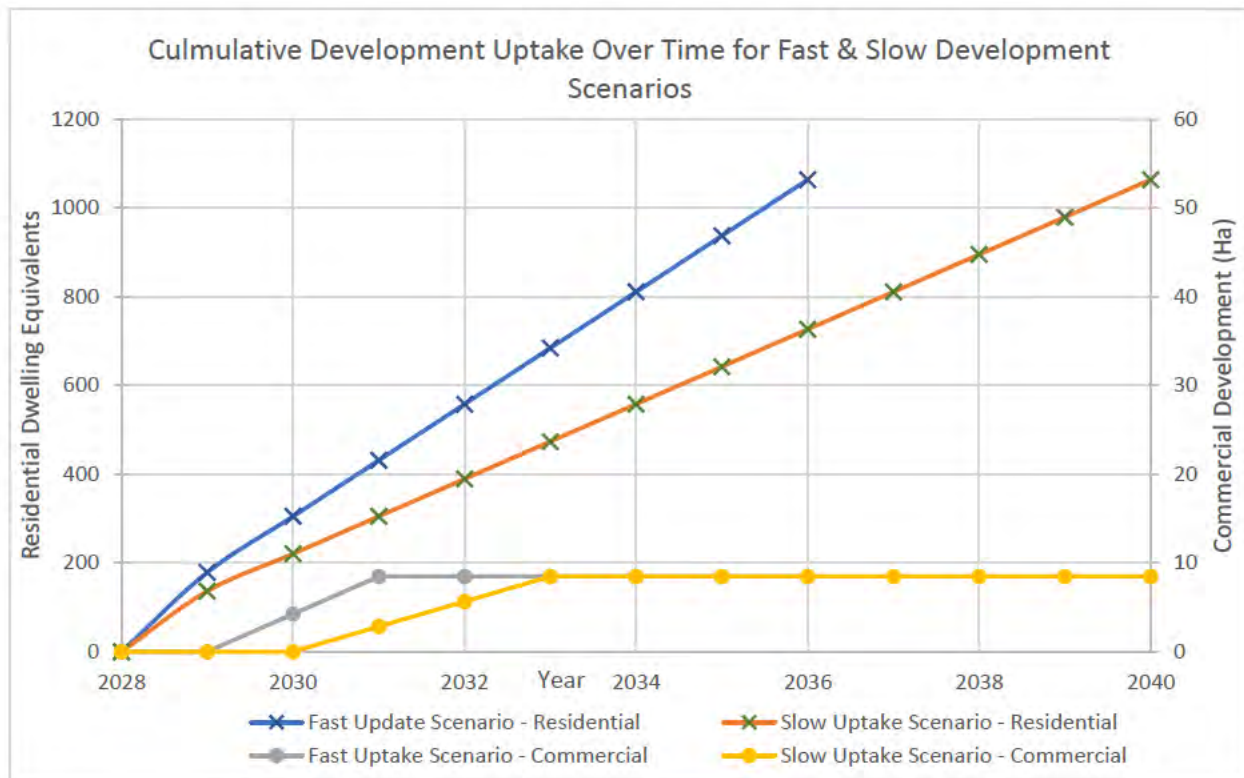


Figure 2 – Chart illustrating development uptake over time for various scenarios

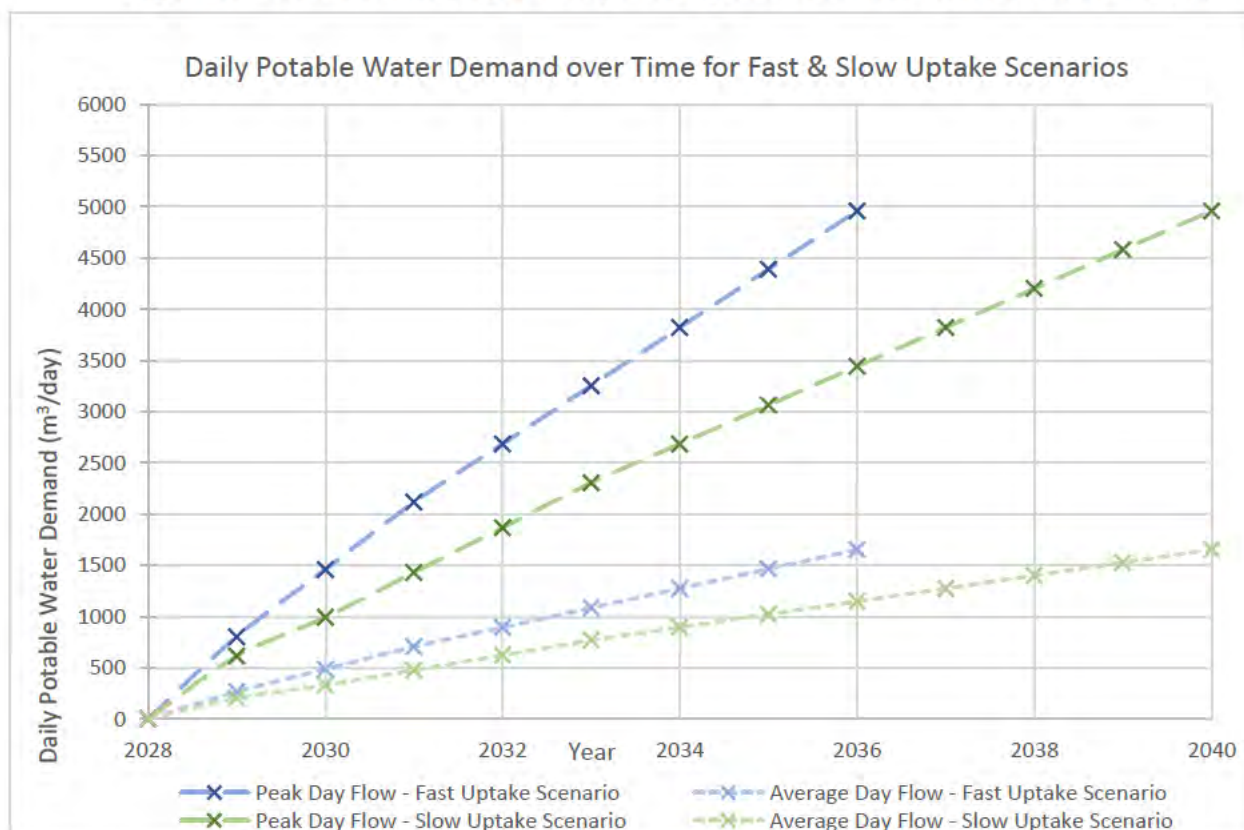


Figure 3 – Chart showing daily potable water demand over time for various scenarios

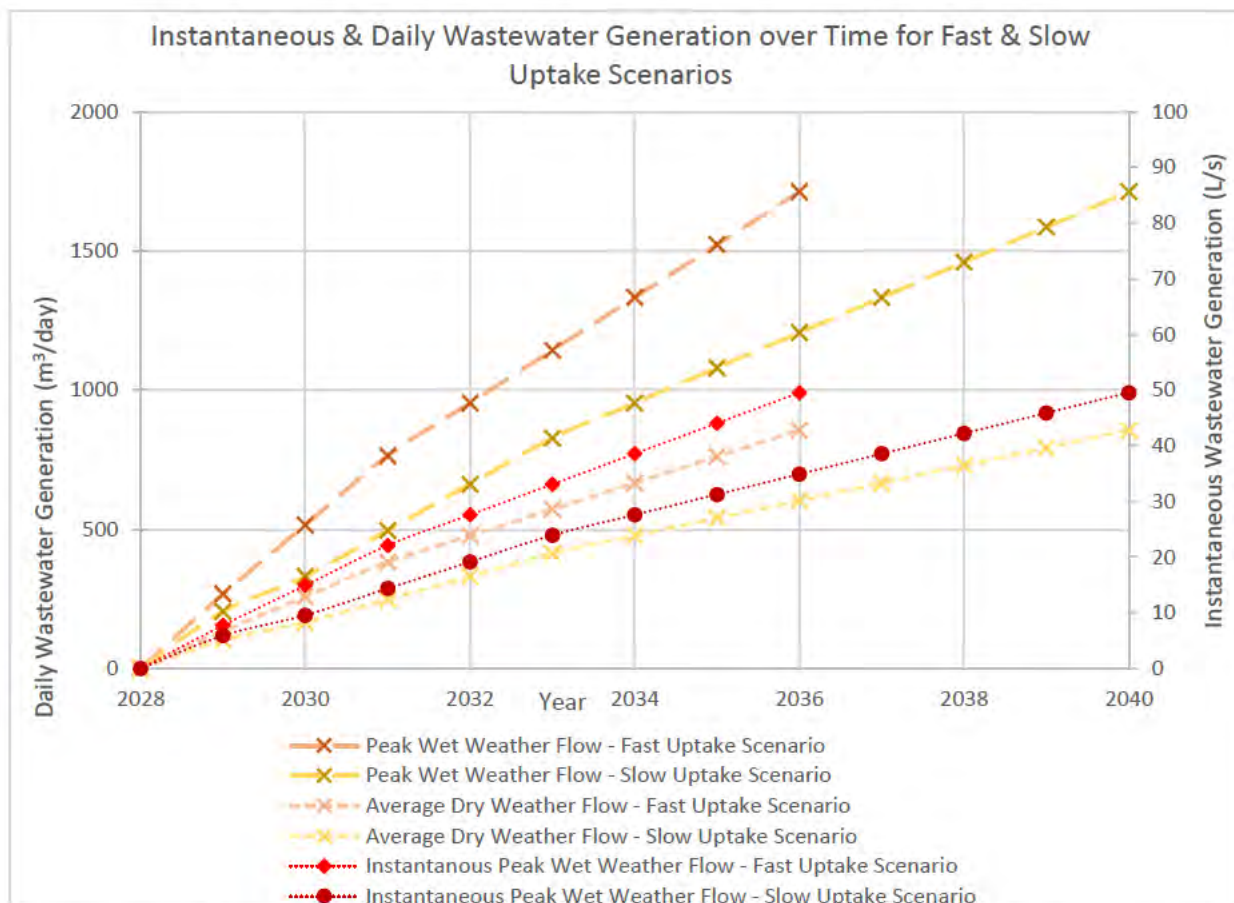


Figure 4 – Chart showing daily & instantaneous wastewater generation over time

The uptake analysis, combined with the preliminary capacity assessments already undertaken, illustrate that there is a significant time period available (4-5 years) for the consenting, design and construction of necessary network upgrades. We note that this assessment does not consider further mitigation measures such as off-peak pumping and flow buffering which could be implemented if necessary to provide additional time for new infrastructure to come on line.

8.0 Conclusions

Significant preliminary design, assessment and consultation work has been undertaken on the feasibility of 3 Waters servicing for the SFRCE Fast-track proposal located at Ripponvale Road, Cromwell.

The design and assessment work completed to date confirms that;

- The SFRCE can be serviced with potable water supply by way of connection to the existing CODC water supply network. It is acknowledged by both the developers of the SFRCE proposal and CODC that some network upgrades may be necessary, however these can be resolved by appropriate capital works investment.
- The SFRCE can also be serviced by connection of wastewater services to the existing CODC wastewater network. Again, it is acknowledged by both the developers of the SFRCE proposal and CODC that some network upgrades may be necessary, however these can be resolved by appropriate capital works investment.

- c) A stormwater solution for the site has been developed which confirms that the SFRCE proposal can be safely developed without adverse flood impacts to adjacent properties. Further, the stormwater solution proposed allows the opportunity for CODC to address a pre-existing flood hazard within their network.

9.0 Limitations

This report has been written for the particular brief to HCL from their client and no responsibility is accepted for the use of the report for any other purpose, or in any other context, prior to review and agreement.

In addition, this report contains information and recommendations based on information obtained from a variety of methods and sources including third parties, inspection, sampling or testing at specific times and locations with limited site coverage as outlined in this report. This report does not purport to completely describe all site characteristics and properties and it must be appreciated that the actual conditions encountered throughout the site may vary, particularly where ground conditions and continuity have been inferred between test locations. If conditions at the site are subsequently found to differ significantly from those described and/or anticipated in this report, HCL must be notified to advise and provide further interpretation.

List of Attachments

- Attachment A – Central Otago District Council (CODC) Wastewater and Potable Water Network Review – 19 November 2025
- Attachment B – Cromwell Cherry Orchard Development Water & Wastewater Demand Assessment – 11 May 2026 (Revision 1)
- Attachment C – Cromwell Cherry Orchard Development Stormwater Feasibility – 11 May 2026

Cherry Orchard Development – 3 Waters Assessment
for Fast Track Referral

ATTACHMENT A:

Central Otago District Council
(CODC) Wastewater and Potable
Water Network Review
19 November 2025

SUMMARY MEMORANDUM

TO: Central Otago District Council (CODC) and New Zealand Cherry Corp (Leyser)
Limited Partnership

FROM: Jake Allen, Design Manager, Hadley Consultants Ltd

DATE: 19 November 2025

SUBJECT: CODC Wastewater and Potable Water Network Review

PROJECT: 254006

1.0 Introduction

This memorandum has been prepared for New Zealand Cherry Corp (Leyser) Limited Partnership (NZCC) to summarise the findings of the preliminary CODC Wastewater and Potable Water Network Review undertaken by Hadley Consultants Limited (HCL).

The purpose of this network review is to assist NZCC in discussions with CODC staff about the additional 44 Lots applied for under RC250155.

2.0 Drawing Summary

A set of four drawing sheets have been prepared to summarise the existing infrastructure associated with both the wastewater and potable water networks in Cromwell. These are included with this memo as Attachment A. These drawing sheets describe the following;

- Sheet 001 – An overview of the CODC Wastewater Network in Cromwell, including the Wastewater Treatment Plant (WWTP) and the relevant connections from both Bannockburn and Pisa Moorings. This includes a summary of recent upgrades and proposed network upgrades from the Long Term Plan (LTP).
- Sheet 002 – A detailed view of the CODC Wastewater Network from the subject site to Cromwell WWTP. This illustrates the network hydraulic connections and reported surcharge issues due to downstream capacity.
- Sheet 003 – An overview of the CODC Potable Water Network in Cromwell, including the Bore field, Pump Station, Reservoir and Key Trunk Mains. Relevant connections to both Bannockburn and Pisa Moorings are also included. Again, recent upgrades and proposed LTP upgrades are listed.
- Sheet 004 - Detailed view of the CODC Potable Water Network from the Reservoir to the subject site. We have not identified any significant reticulation constraints in the water supply information we have reviewed.

3.0 Information Source

It is noted that the information contained within the drawing sheets has come from a variety of sources and to date only limited verification of the accuracy of the data gathered has been possible.

The majority of the capacity information has come from Mott McDonald (MM) reporting on water and wastewater networks that was undertaken to assess the Wooing Tree development up to 2022. The MM reporting contains the following disclaimer.

"As of 2022 all results produced using the Cromwell WW model (via MIKE URBAN 2020) should be considered with care as no recalibration of the model has been undertaken by Mott MacDonald and the most recent update was undertaken in 2015.

The present model was built by Rationale Ltd. and supplied to Mott MacDonald (by CODC) under their assumption that there was a reasonable level of confidence in the model's accuracy. Mott MacDonald was instructed to use it for assessment purposes and there have been no model simulation sensitivity tests or validation undertaken as part of this assessment."

The MM water and wastewater models appear to have had the Plan Change 19 demands integrated into them in 2023 and 2024. Therefore, it is hoped that modelling requested as part of the 44 Lot discussions under RC250155 will not contain the same disclaimer from MM. We note that we have not been able to obtain any MM reporting from 2024 or 2025. It would be constructive for all parties if CODC can assist in the provision of the MM capacity modelling in these years.

There has been no ability to confirm with MM what yield assumptions future demand they have used in the wastewater and water supply modelling at this stage.

4.0 CODC Demand Allowances

It is understood that the water and wastewater demand allowed for within the MM models are sourced from the "Cromwell Yield Assessment" prepared by Rationale and included as Attachment B. For context the Rationale Report refers to Cromwell as incorporating both Bannockburn and Pisa Moorings. The subject site is excluded from and outside both the MM and the Rationale reporting.

Based on the Cromwell Yield Assessment the existing Cromwell area currently contains 4,241 dwellings and a feasible yield of 12,104 dwellings is available under Cromwell master planning and PC19. The upper bound of the Yield assessment is 16,360 dwellings. It is unclear if MM have used 12,104 or 16,360 in their 2023 and 2024 base models.

The Cromwell Yield Assessment was reported on in September 2022 and assumed certain densities at various locations under PC19. Since 2022, we understand many of the zonings have been refined, appealed or formalised. We recommend that a planning expert researches the yield assumptions in the Cromwell Yield Assessment report and compares those against actual confirmed zonings and densities in 2025. If the actual 2025 yields (based on confirmed zonings and densities) are lower than the 2022 reporting by Rationale, this would significantly assist capacity discussions with CODC for the 44 Lot application under RC250155.

5.0 Initial Findings

Various constraints within the existing CODC water and wastewater networks have been identified as indicated on the attached plans. Matters relevant to the subject site are summarised as follows:

The relevant wastewater capacity constraints for the subject site that have been identified are;

- a. There are a number of network capacity constraints identified in the past MM Development Impact Assessments. The relevant pipes with known capacity constraints that are relevant to the subject site are highlighted on sheet 002 of Attachment A.
- b. The WWTP at present is understood to be discharging nitrogen concentrations close to the permitted levels in CODC's Otago Regional Council (ORC) discharge consent. The ORC discharge consent currently allows nitrogen concentrations of up to 60mg/L provided the total daily volume of nitrogen is under 148kg. Once the daily volume is exceeded, additional nitrogen removal processes are required at the WWTP. Previously this was anticipated to occur in 2031 but given faster than expected dwelling growth it appears that CODC now expects that nitrogen discharge limits could be reached or exceeded around 2026 / 2027. This issue should be addressed by the \$11.3M upgrade of the WWTP, as described in the LTP. The timing of the WWTP upgrade means there may be a lag between the current limits being reached and the upgrades being implemented this would create a temporary breach of the nitrogen limit under the current consent. It is our experience with ORC that provided a consent holder is committed to upgrades, temporary breaches are permitted, this avenue should be available to CODC. It is also noted that the nitrogen discharge limits could be altered by central government in the near future. The water regulator, Taumata Arowai, plans to introduce new 'national standards' for the amount of pollutants, such as E coli or nitrogen, that sewage plants can put into the environment. This may further alleviate any existing nitrogen limit concerns.
- c. Further expansion to the membrane treatment plant at the WWTP will be required to meet growth expectations.

The potential potable water capacity constraints or issues that have been identified are.

- d. The Cromwell water supply water take is consented by the Otago Regional Council for a maximum abstraction of 18,000 cubic metres of water per day until 2028. The expiry of the water take consent is a risk to the water supply if there are issues renewing the current volume, however, we consider this to be a very low risk. We have been unable to obtain the volume of water that is currently used, but based off first principles it is expected to be in the order of 9,000 cubic meters per day. This is well below current consented peak abstraction rates and there should be no constraint in this regard. CODC has indicated that demand is expected to be 20,000 cubic meters per day in 30 years' time.

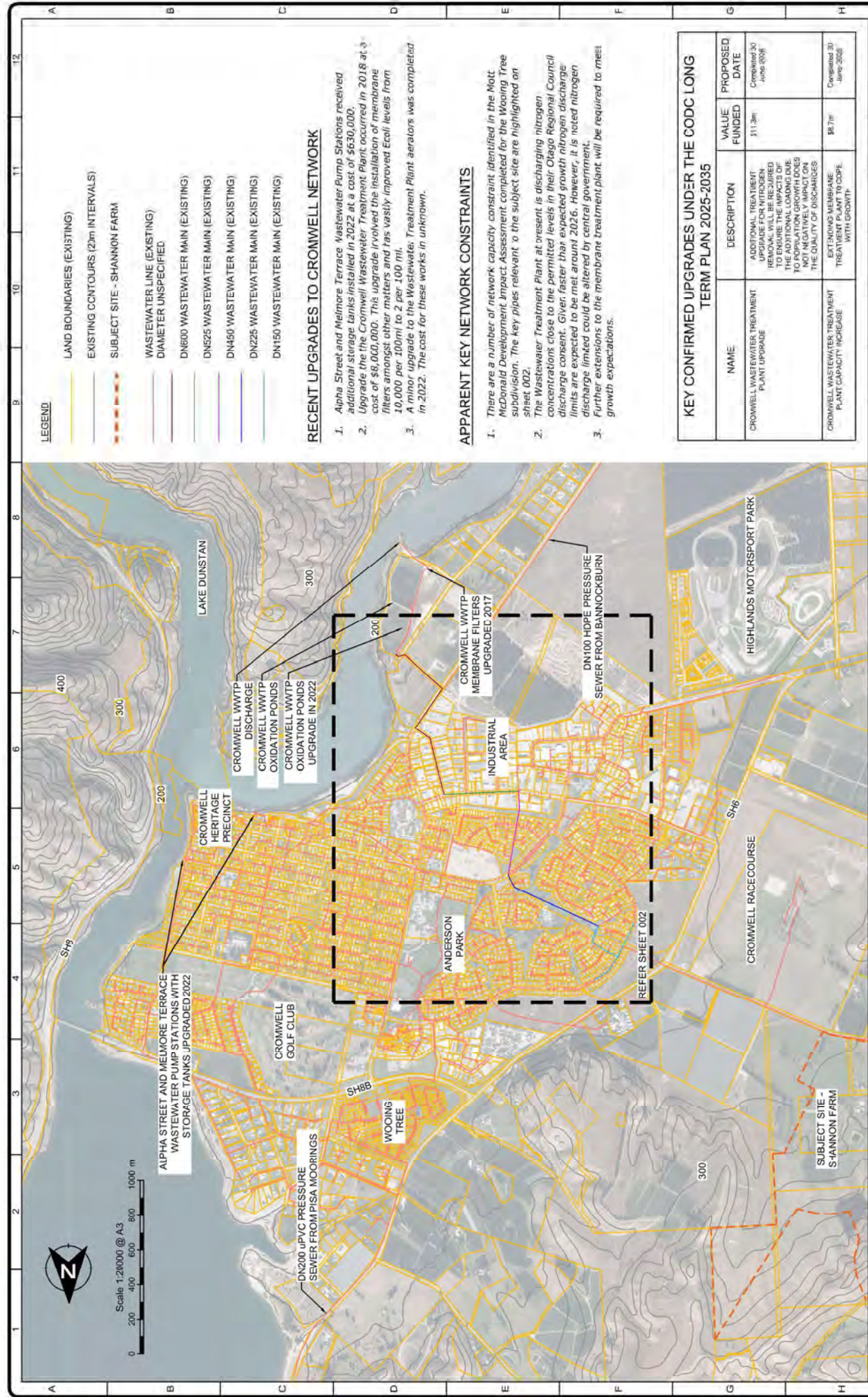
It appears that given the favourable location of the subject site relatively near the central portion of both the potable water and wastewater networks it is expected that the capacity constraints are primarily related to the WWTP infrastructure capacity. No significant constraints have been identified in the Water Treatment Plant at this stage. While some capacity constraints appear to be present within some areas of the existing reticulation networks these appear to be relatively localised and could be addressed by relatively minor pipe upgrades or providing off peak discharge for wastewater if approved by CODC.

Key to resolving any current capacity limitations for the proposed 44 Lots under RC250155 is confirming the following;

- Which Cromwell Yield Assessment figures have been used in the MM modelling (12,104 or 16,360); and
- The actual yields (based on confirmed zoning and densities under PC19) and whether they are less than those under the Cromwell Yield Assessment report, thereby providing some headroom for connection to the proposed 44 Lots at Shannon Farm.
- The current status of the nitrogen discharge from the WWTP and the anticipated timing of exceeding the current consented daily discharge limit of 148kg nitrogen.
- Expected dwelling capacity of the WWTP once the upgrade is completed.

Attachment A: HCL CODC Network Plans

Attachment B: Cromwell Yield Assessment, Rationale Ltd, 5 September 2022



KEY CONFIRMED UPGRADES UNDER THE CODC LONG TERM PLAN 2025-2035			
NAME	DESCRIPTION	VALUE FUNDED	PROPOSED DATE
CROWMELL WASTEWATER TREATMENT PLANT UPGRADE	ADDITIONAL TREATMENT CAPACITY FOR REMOVAL OF NITROGEN TO ENSURE THE IMPACTS OF THE ADDITIONAL LOADING DUE TO POPULATION GROWTH DOES NOT EXCEED THE QUALITY OF DISCHARGE.	\$11.3m	Completed 30 June 2026
CROWMELL WASTEWATER TREATMENT PLANT CAPACITY INCREASE	EXTENDING MEMBRANE TREATMENT TO DEEPEN WITH GROWTH.	\$6.7m	Completed 30 June 2026

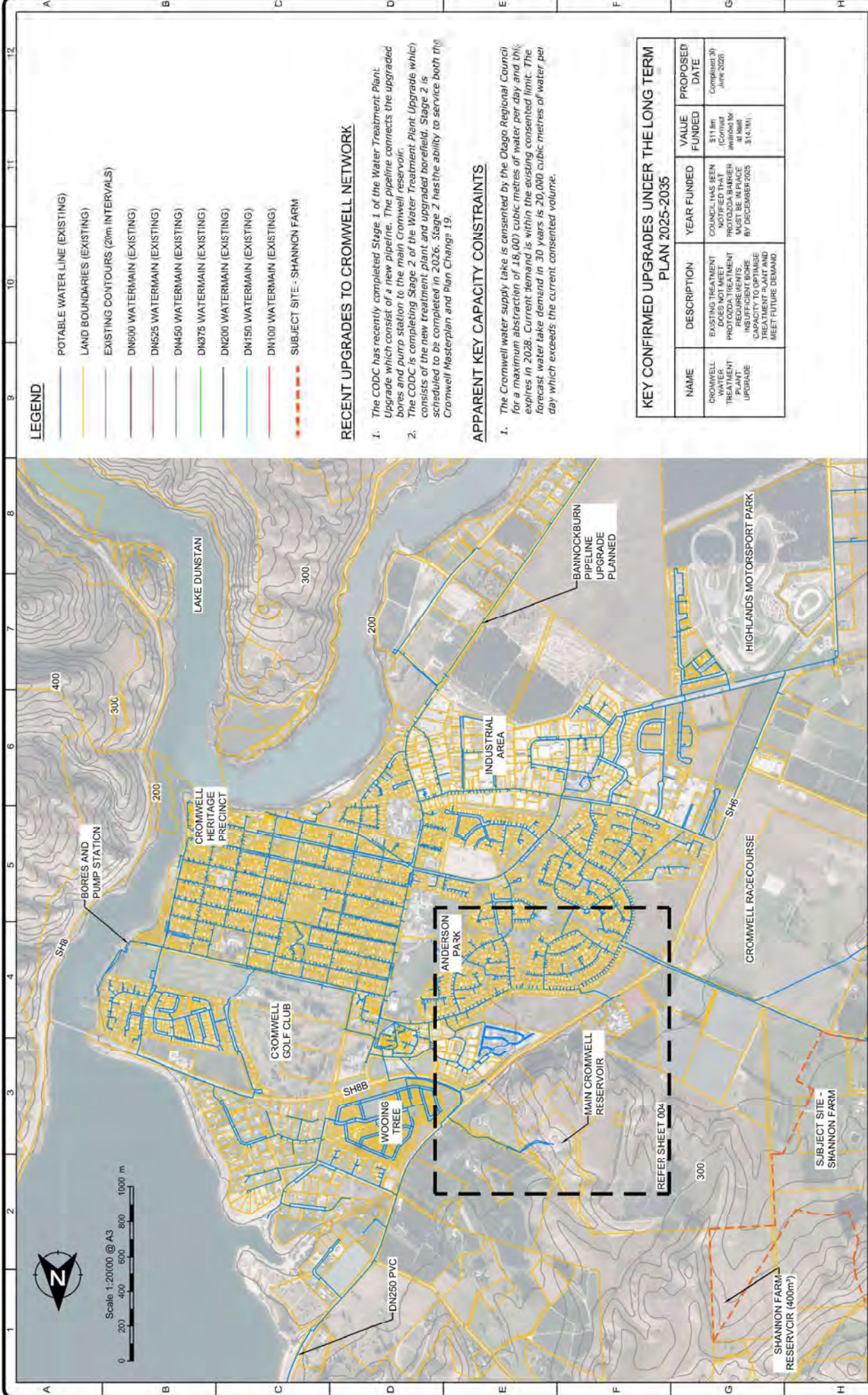


Notice:		1	2	3	4	5	6	7	8	9	10	11	12
A		Description		By	Date	Project		Client:					
FOR CLIENT INFORMATION				MB	07/11/25	WASTEWATER AND POTABLE WATER ASSISTANCE		NEW ZEALAND CHERRY CORP (LEYSER) LIMITED					
Title:		EXISTING CODC WASTEWATER NETWORK DETAIL											
Drawn		Checked		Scale		Drawing Number							
BB		JA		1:8000 @A3		Project: 254006		Sheet: 002					
Drawing Number		Project		Sheet		This drawing is included on the understanding that the client and other party without written consent from Hadley Consultants Ltd.							
Project: 254006		Sheet: A											



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At Ebbw Road PO Box 195, Christchurch, New Zealand 8140 (Tel: 03 378 1000) (Fax: 03 378 1001) (Email: info@hadley.co.nz)



LEGEND

- POTABLE WATER LINE (EXISTING)
- LAND BOUNDARIES (EXISTING)
- EXISTING CONTOURS (20m INTERVALS)
- DN600 WATERMAIN (EXISTING)
- DN525 WATERMAIN (EXISTING)
- DN450 WATERMAIN (EXISTING)
- DN375 WATERMAIN (EXISTING)
- DN200 WATERMAIN (EXISTING)
- DN150 WATERMAIN (EXISTING)
- DN100 WATERMAIN (EXISTING)
- SUBJECT SITE - SHANNON FARM

RECENT UPGRADES TO CROMWELL NETWORK

- The CODC has recently completed Stage 1 of the Water Treatment Plant Upgrade which consist of a new pipeline. The pipeline connects the upgraded bores and pump station to the main Cromwell reservoir.
- The CODC is completing Stage 2 of the Water Treatment Plant Upgrade which consists of the new treatment plant and upgraded borefield. Stage 2 is scheduled to be completed in 2026. Stage 2 has the ability to service both the Cromwell Masterplan and Plan Change 19.

APPARENT KEY CAPACITY CONSTRAINTS

- The Cromwell water supply lake is consented by the Otago Regional Council for a maximum abstraction of 18,000 cubic metres of water per day and this expires in 2028. Current demand is within the existing consented limit. The forecast water take demand in 30 years is 20,000 cubic metres of water per day which exceeds the current consented volume.

KEY CONFIRMED UPGRADES UNDER THE LONG TERM PLAN 2025-2035

NAME	DESCRIPTION	YEAR FUNDED	VALUE FUNDED	PROPOSED DATE
CROMWELL WATER TREATMENT PLANT UPGRADE	EXISTING TREATMENT DOES NOT MEET PHOTOGRAPHIC TREATMENT REQUIREMENTS. INSUFFICIENT BORE CAPACITY TO OPTIMISE TREATMENT PLANT AND MEET FUTURE DEMAND.	COUNCIL HAS BEEN NOTIFIED THAT PHOTOGRAPHIC TREATMENT REQUIREMENTS WILL BE BY DECEMBER 2025	\$11.8m (Consent available for \$14.1m)	Completed 30 June 2026

WASTEWATER AND POTABLE WATER ASSISTANCE

EXISTING CODC WATERSUPPLY NETWORK OVERVIEW

NEW ZEALAND CHERRY CORP (LEYSER) LIMITED

hadley consultants LTD

100000 @A3

Project: 254006

Sheet: 003

Issue: A

This drawing is prepared on the understanding that the client has provided all necessary information and that the client is responsible for obtaining all necessary consents and approvals from the relevant authorities.



LEGEND

- LAND BOUNDARIES (EXISTING)
- EXISTING CONTOURS (20m INTERVALS)
- WATER MAIN UNSPECIFIED DIAMETER (EXISTING)
- DN600 WATERMAIN (EXISTING)
- DN525 WATERMAIN (EXISTING)
- DN450 WATERMAIN (EXISTING)
- DN375 WATERMAIN (EXISTING)
- DN200 WATERMAIN (EXISTING)
- DN150 WATERMAIN (EXISTING)
- DN100 WATERMAIN (EXISTING)
- SUBJECT SITE - SHANNON FARM

NOTES

- 1. AC = Asbestos Cement
- 2. upVC = Polyvinyl Chloride

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Cromwell Yield Assessment

September 2022



Document Title:

Cromwell Yield Assessment

Prepared for:

Central Otago District Council

Quality Assurance Statement

Rationale Limited	Project Manager:	Ben Gonzalez
5 Arrow Lane	Prepared by:	Ben Gonzalez
PO Box 226	Reviewed by:	Tom Lucas and Neil Jorgensen
Arrowtown 9351	Approved for issue by:	Tom Lucas and Neil Jorgensen
Phone: +64 3 442 1156	Job number:	J001316

Document Control History

Rev No.	Date	Revision Details	Prepared by	Reviewed by	Approved by
0.1	29 Aug 2022	Draft	BG		
2.0	5 Sep 2022	Final	BG	TL	TL

Current Version

Rev No.	Date	Revision Details	Prepared by	Reviewed by	Approved by
2.0	5 Sep 2022	Final	BG	NJ	NJ

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Executive Summary

Rationale was engaged by CODC to produce yield calculations for Plan Change 19 (PC19) as well as some private plan changes in the Cromwell Ward. The purpose was to determine whether these plans provide sufficient capacity for the forecasted demand.

The yield calculations were completed using the same methodology as that used in the Vincent Spatial Plan. Parcels of land were divided into Small Lots, Large Lots, and Greenfields in relation to their size compared to the minimum lot size. An infrastructure allowance was provided for in the Greenfield parcels. Development feasibility parameters were calibrated by taking Cromwell's current Low and Medium density zones as realistic levels of development. Three additional plan changes were included in the yield calculation: the Golf Course in Cromwell, Cherry Orchard in Pisa Moorings and Parkburn in Pisa Moorings.

The resulting yield calculations suggest that PC19 in conjunction with the private plan changes provide sufficient housing supply for the forecasted demand at a Ward level. Assessment at a township level shows that in both Cromwell and Pisa Moorings the forecasted supply outpaces the demand, while the opposite is true in Bannockburn.

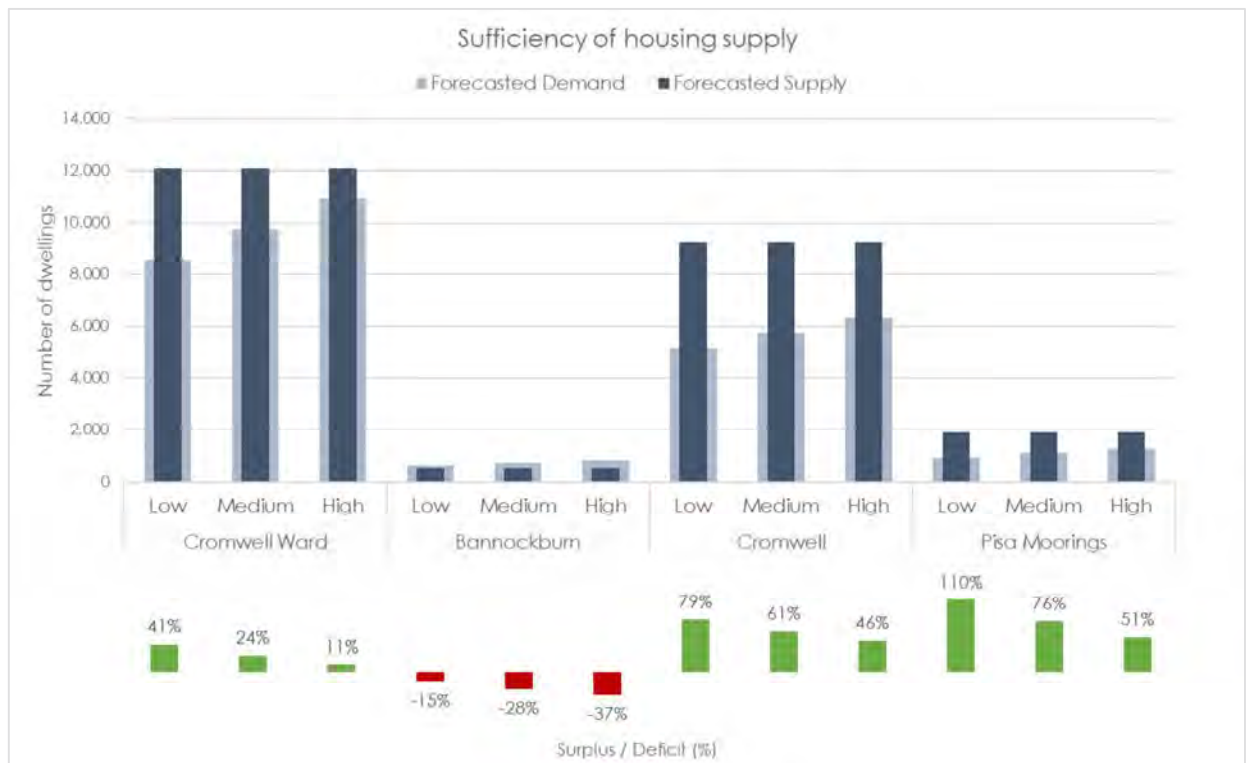


Figure 1: Sufficiency of housing supply in Cromwell Ward

1 Purpose

A Cromwell Housing and Business Development Capacity Assessment was prepared in 2018 to support the Cromwell Spatial Framework. The spatial framework recommended changes that would enable over 20,000 houses with a likely feasible capacity assessed at around 10,000 houses. The demand at the time was only forecast to be around 7,700 houses in 2048, meaning sufficient capacity was being proposed.

The latest growth projections suggest a demand for around 9,700 houses in 2054 for the Cromwell Ward, putting the sufficiency of housing at risk.

Further to this there have been a number of changes from the zoning recommendations in the spatial framework that will further affect the supply of housing, that need to be tested. A key one of these is the potential development of the golf course land which is now considered less feasible, resulting in a potential reduction of around 800 houses.

2 Methodology

2.1 Overview

The model used to calculate housing yield is the same as that used for the Vincent Spatial Plan and is summarised in the diagram below. It is based on classifying land parcels into the following categories:

- Brownfield - Small lots: sections smaller than the 2x min. lot size
- Brownfield - Large lots: sections up to 10x the min. lot size
- Greenfield: sections larger than 10x the min. lot size

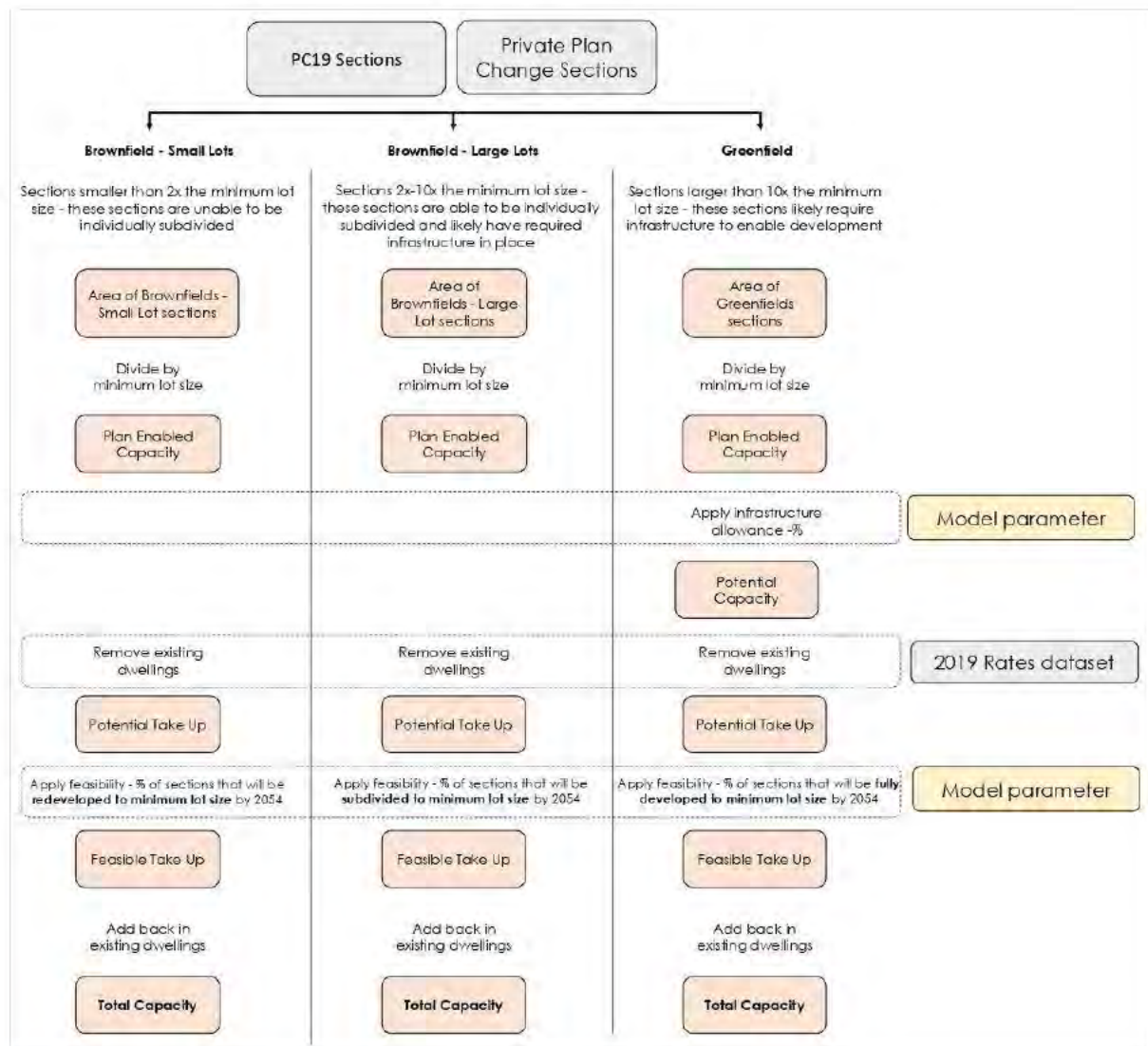


Figure 2: Overview of methodology for yield calculation

2.2 Plan Change 19

PC19 has six distinct zoning categories, with the following minimum lot sizes.

Table 1: PC19 Zones

Zone	Minimum lot size (m ²)
Medium Density	200
Low Density	500
Large Lot	2,000
Large Lot (P1)	1,000
Large Lot (P2)	3,000
Large Lot (P3)	6,000

CODC provided Rationale with the new zoning spatial data which was further enriched with location data using GIS software to enable summaries at a township level. Township boundaries are shown in Appendix A. There were 8 parcels where the zoning was ill-defined as the zoning boundary change cut across them. In all these instances, the higher density zone has been assumed.

A Rates dataset as of 2019 was also used in order to estimate the number of dwellings in each area of interest. Dwellings were estimated from “Residential” and “Lifestyle” rating units using the number of water and wastewater connections. The estimates obtained align reasonably well with the starting point of the Growth Projections, which is in turn derived from Stats NZ figures. The only area which showed significant discrepancy was Pisa Moorings, which is partly attributable to the definition and exclusion of commercial accommodation under Stats NZ figures, whereas in the Rates dataset some of these properties are still accounted for as Residential.

2.3 Additional Plan Change Areas

Three additional plan change areas were identified for inclusion into the yield calculation:

- Golf Course: around 330,000 m² in Cromwell assessed as Low Density.
- Cherry Orchard: around 240,000 m² in Pisa Moorings assessed as Low Density.
- Parkburn: around 990,000 m² in Pisa Moorings assessed as Low Density.

Outlines of the boundaries for each of these can be found in Appendix B.

2.4 Model Parameter Calibration

Infrastructure allowance percentages were set following the same methodology as used in the Vincent Spatial Plan, allowing 20% to 30% for infrastructure development in Greenfield lots.

Feasibility percentage calibration was carried out by using current percentage developed rates in Cromwell township as reference of realistic levels of development. The targeted percentage developed rates were 80% for Low Density and Large Lot, and about 60% for Medium Density, as highlighted in the table below. Large Lot (P1), (P2) and (P3) were set to very low feasibility percentages as very little further development is expected in them.

Table 2: Current percentage developed rates

Township	Brownfields - Small Lots	Brownfields - Large Lots	Greenfields
Bannockburn			
Large Lot	88%	31%	11%
Large Lot (P2)		0%	
Large Lot (P3)	0%	0%	
Cromwell			
Medium Density	60%	26%	1%
Low Density	79%	25%	2%
Large Lot	55%	34%	3%
Large Lot (P3)	77%	42%	
Pisa Moorings			
Low Density	65%	47%	0%
Large Lot (P1)	86%	19%	5%

Rural			
Large Lot			0%
Large Lot (P2)	58%	7%	3%

Table 3: Calibrated model parameters

Lot Classification and Zone	Infrastructure Allowance	Feasibility
Brownfields - Small Lots		
Medium Density	-	10%
Low Density	-	5%
Large Lot	-	2%
Large Lot (P1)	-	2%
Large Lot (P2)	-	2%
Large Lot (P3)	-	2%
Brownfields - Large Lots		
Medium Density	-	50%
Low Density	-	73%
Large Lot	-	70%
Large Lot (P1)	-	2%
Large Lot (P2)	-	2%
Large Lot (P3)	-	2%
Greenfields		
Medium Density	30%	60%
Low Density	30%	80%
Large Lot	20%	78%
Large Lot (P1)	20%	1%
Large Lot (P2)	20%	2%
Large Lot (P3)	20%	2%

3 Demand

The 2022 Growth Projection Review completed by Rationale for CODC was used as a forecast of housing demand needs in the Cromwell Ward. These projections estimate growth in Cromwell Ward to be more rapid than the rest of the district, with the medium forecast indicating 5,092 new dwellings required by 2054.

The table below provides a summary of the growth projections at the Ward level. More detailed results can be accessed in the Growth Projections – 2022 report.

Table 4: 2022 Growth Projections Review - Cromwell Ward Summary

	Historic Growth (2006 - 2021)			Short Term Forecast (2021 - 2034)			Long Term Forecast (2034 - 2054)		
	Total Growth	Av. Annual Growth	Av. Annual Growth Rate	Total Growth	Av. Annual Growth	Av. Annual Growth Rate	Total Growth	Av. Annual Growth	Av. Annual Growth Rate
Usually Resident Population	4,820	321.3	4.6%	4,362	335.6	2.9%	6,320	316.0	1.9%
Total Dwellings	1,694	112.9	3.1%	2,078	159.9	2.9%	3,013	150.7	1.9%
Total Rating Units				2,238	172.1	2.7%	3,258	162.9	1.8%
Residential and Lifestyle Rating Units				2,078	159.9	3.0%	3,013	150.7	1.9%

The figure below shows that majority of new dwellings are forecasted within Cromwell, as expected. However, there is also large growth forecasted to occur within rural areas. It is important to note that in these projections, the distribution of growth within the Ward is based on how growth has been occurring in recent years, but this apportionment may well change as urban towns grow in scale.

For the purposes of this assessment, 90% of the demand growth forecasted in rural areas has been shifted to occur within Cromwell, as advice from CODC suggests this reflects a more realistic scenario in line with the Council's long term growth strategies.

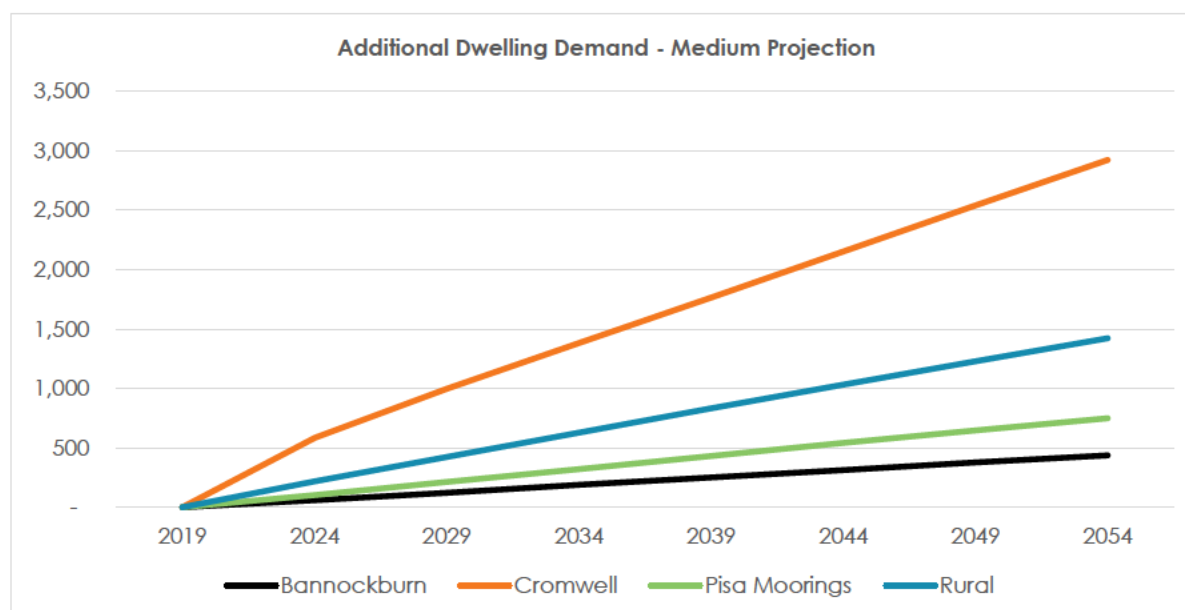


Figure 3: Additional Dwelling Demand by Township - Medium Projection

4 Supply

As discussed in Section 2, parcels of land were classified into Brownfields – Small Lots, Brownfields – Large Lots and Greenfields before the yield calculations were carried out.

Appendix C provides a breakdown of the almost 1,100 HA included in PC19 and the additional plan change areas. The following remarks are made:

- Cromwell contains about 60% of the plan's area, with the majority of lots being zoned as Low and Medium Density.
- Bannockburn is entirely zoned as Large Lot and makes up 13% of the plan's area.
- Pisa Moorings holds 22% of the area and is almost equally divided into Large Lot (P1) and Low Density zoning.
- PC19 does not provide zoning for much of the rural areas, making up only 5% of the plan's area, and most of it being zoned as Large Lot (P2).

The following table presents the resulting yields by area and zone. As expected, most of the additional capacity resides in Cromwell, followed by Pisa Moorings.

Table 5: Yields by Township and Zone

Township and Zone	Existing Capacity	Potential Capacity	Feasible Capacity	Additional Capacity	Current % Developed	Projected % Developed	Dwellings per HA
Bannockburn	267	612	507	240	44%	83%	3.5
Large Lot	240	579	480	240	41%	83%	3.7
Large Lot (P2)	-	18	0	0	0%	2%	0.1
Large Lot (P3)	-	15	0	0	0%	2%	0.0
Outside	27		27				
Cromwell	3,056	12,701	9,262	6,206	24%	73%	14.1
Large Lot	73	315	250	177	23%	79%	3.6
Large Lot (P3)	40	64	40	0	63%	63%	1.1
Low Density	2,042	6,818	5,470	3,428	30%	80%	13.1
Medium Density	799	5,504	3,399	2,600	15%	62%	25.5
Outside	102		102				
Pisa Moorings	498	2,884	1,907	1,409	17%	66%	7.8
Large Lot (P1)	416	1,012	423	7	41%	42%	3.7
Low Density	82	1,872	1,483	1,401	4%	79%	11.3
Outside	-		-				
Rural	420	163	428	8	258%	263%	8.4
Large Lot	-	8	6	6	0%	78%	3.1
Large Lot (P2)	56	155	58	2	36%	37%	1.2
Outside	364		364				
Grand Total	4,241	16,360	12,104	7,863	26%	74%	11.0

The figure below shows the resulting percentage developed rates by Zone. As discussed in Section 2, Low Density and Large Lot zones have been calibrated to yield 80% developed rates, while Medium Density zones have been calibrated to just over 60% and Large Lots (P1), (P2) and (P3) have been severely restricted.

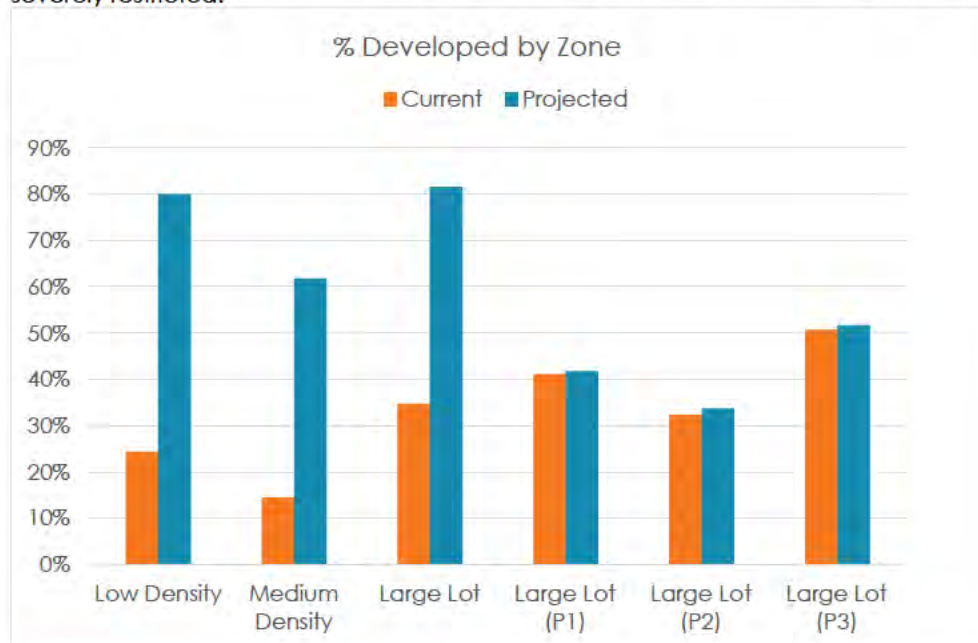


Figure 4: Percentage Developed Rates by Zone

5 Demand vs Supply

5.1 Cromwell Ward

At a Ward level, there is confidence that PC19 alongside the additional plan change areas will be sufficient for current forecasted housing demand. The forecasted supply of over 12,000 dwellings represents a surplus of 24% over the Medium forecasted demand, and 11% over the High forecasted demand.

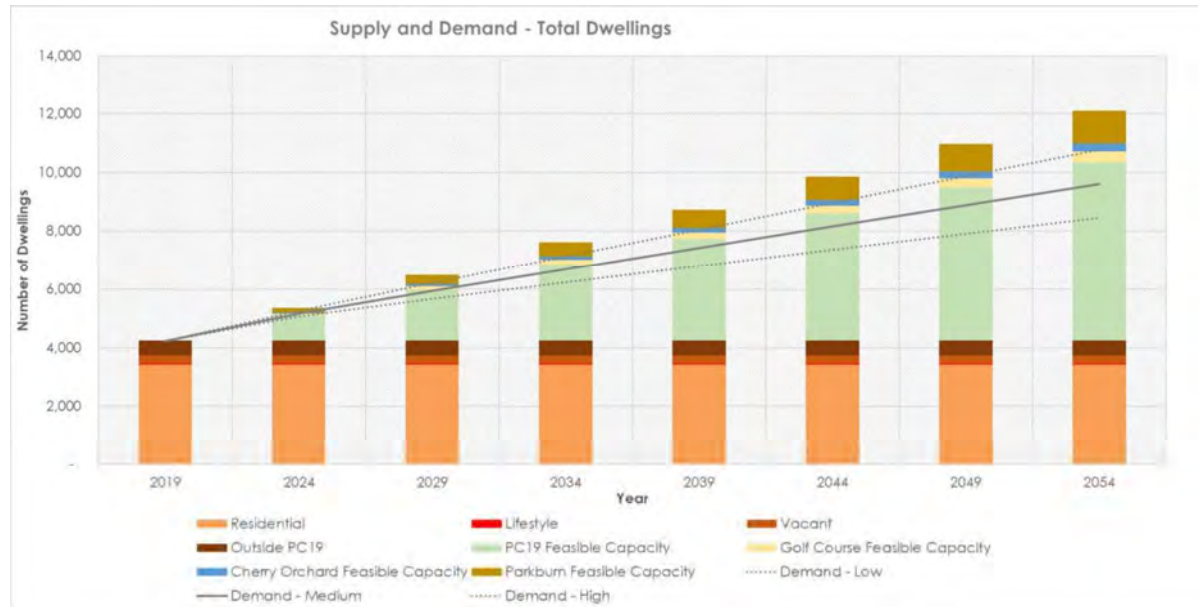


Figure 5: Cromwell Ward Demand vs Supply

5.2 Bannockburn

Current growth projections have moderate growth occurring within Bannockburn, while PC19 does not provide much scope for growth with all the zoning in this township being Large Lots. The forecasted supply of just over 500 dwellings represents a deficit of 28% under the Medium forecasted demand, and 37% under the High forecasted demand.

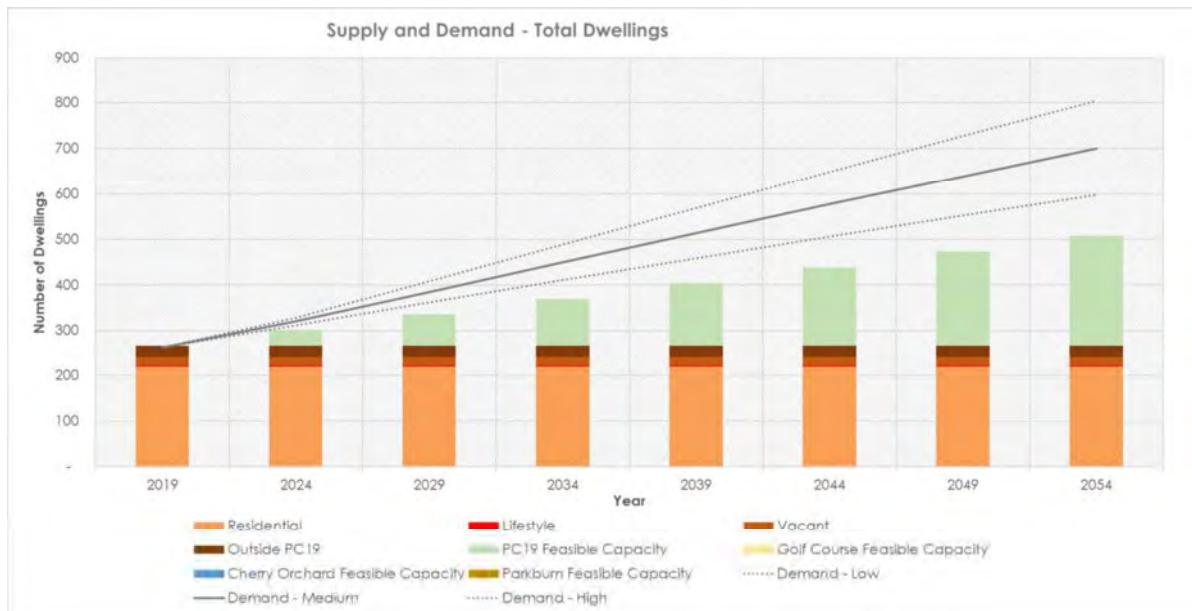


Figure 6: Bannockburn Demand vs Supply

5.3 Cromwell

PC19 has large areas of low and medium density zoning in Cromwell which allow for large growth in capacity. The forecasted supply of over 9,000 dwellings represents a surplus of 61% over the Medium forecasted demand, and 46% over the High forecasted demand. This massive surplus is expected to cover most of the deficit seen in Bannockburn, with large housing supply driving a shift to greater growth rates within Cromwell township.



Figure 7: Cromwell Demand vs Supply

5.4 Pisa Moorings

Pisa Moorings additional feasible capacity is almost entirely composed of development in the additional plan change areas: the Cherry Orchard and Parkburn lots. The forecasted supply of over 1,900 dwellings represents a surplus of 76% over the Medium forecasted demand, and 51% over the

High forecasted demand. This provides room for Pisa Moorings to adopt some of the remaining growth in surrounding rural areas.

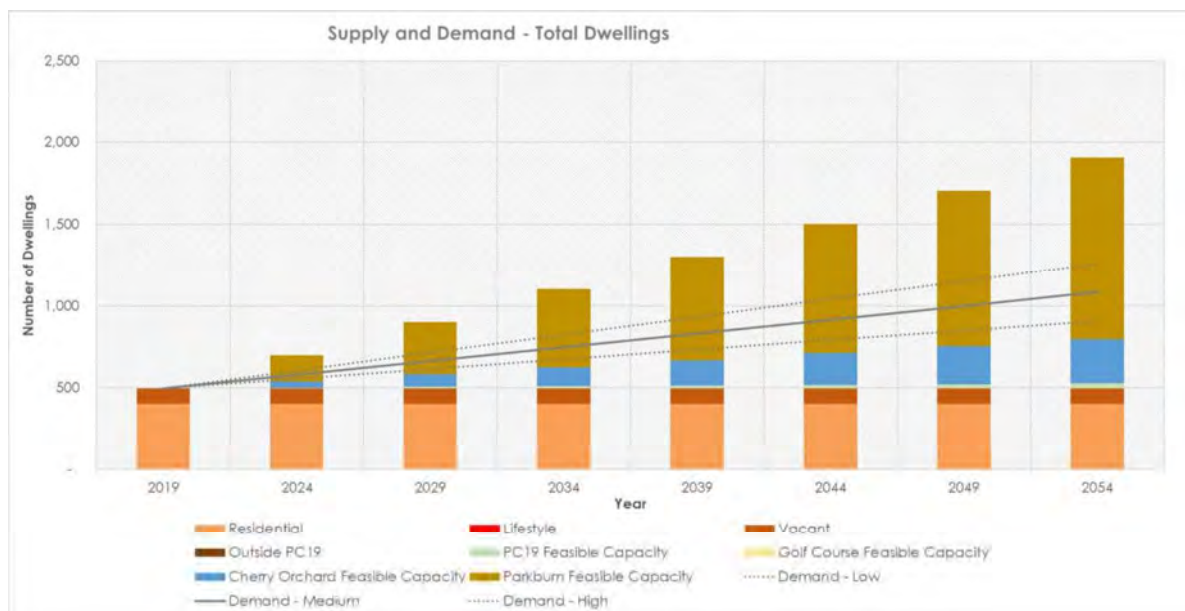


Figure 8: Pisa Moorings Demand vs Supply

5.5 Other (Rural)

As discussed in Section 2, PC19 does not provide zoning for the majority of rural lots, and as such a supply vs demand analysis is not meaningful for this area. Furthermore, most of the growth forecasted in rural areas has been shifted to the Cromwell township.

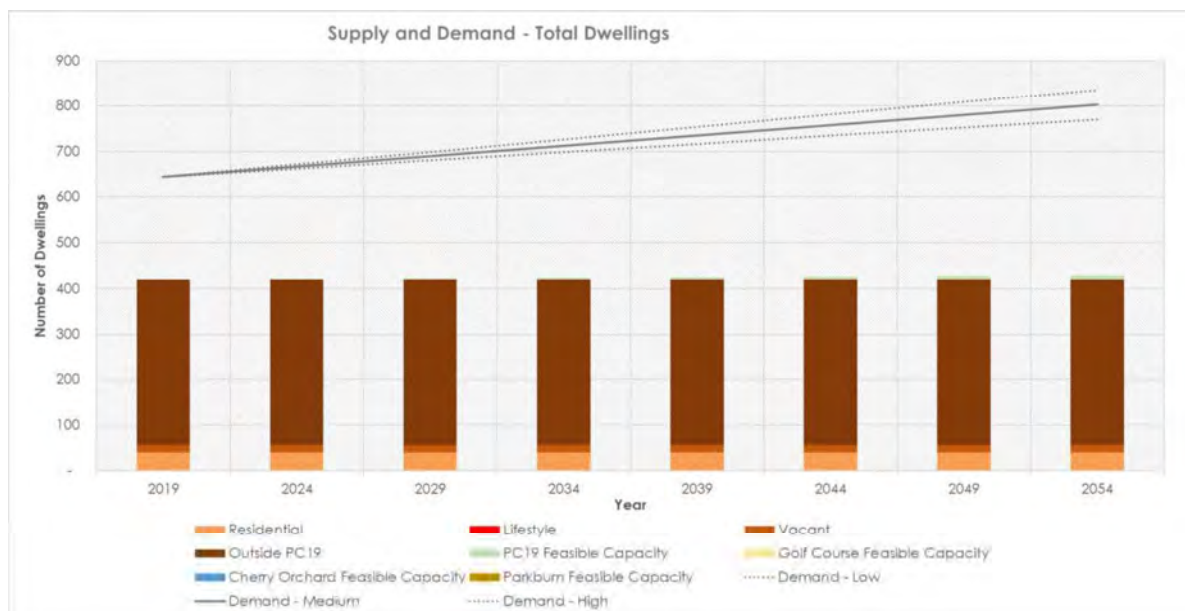
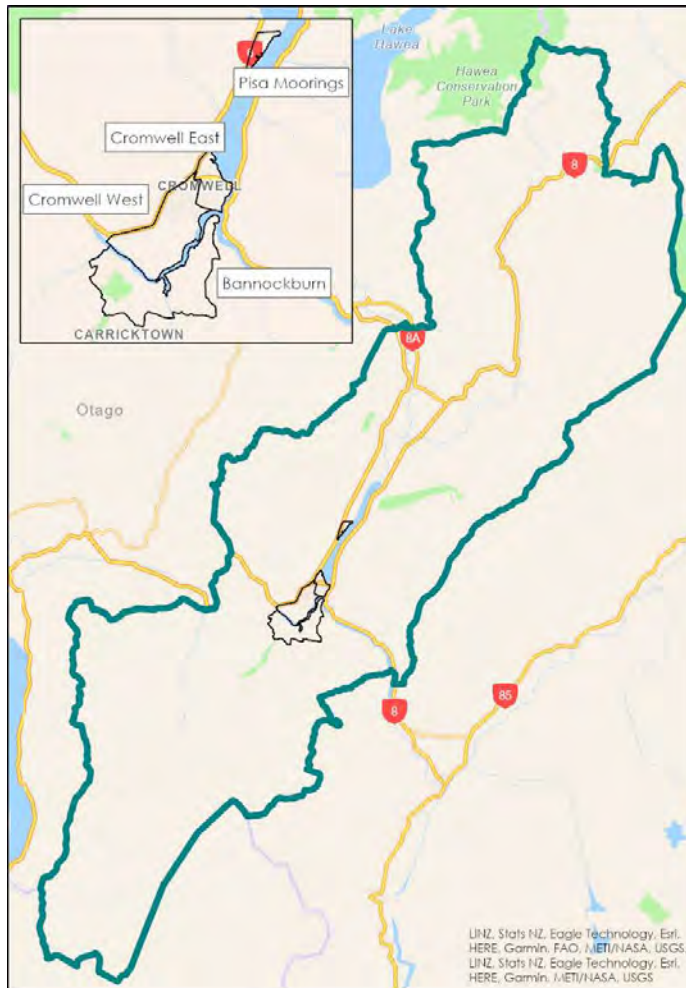


Figure 9: Rural Demand vs Supply

Appendices

Appendix A: Township Boundaries

Cromwell Ward



Bannockburn



Cromwell Township



Pisa Moorings



Appendix B: Additional Plan Change Zones

Golf Course - Cromwell



Cherry Orchard – Pisa Moorings



Parkburn – Pisa Moorings



Appendix C: Total Plan Area Breakdown

Table 6: Total Plan Area (m²) by Township and Lot Classification

Township and Lot Classification	Brownfields - Small Lots	Brownfields - Large Lots	Greenfields	Grand Total
Bannockburn	453,238	322,805	657,635	1,433,678
Large Lot	395,919	236,114	657,635	1,289,669
Large Lot (P2)		53,810		53,810
Large Lot (P3)	57,319	32,881		90,200
Cromwell	1,317,574	1,670,023	3,583,396	6,570,993
Medium Density	24,805	538,872	767,453	1,331,130
Low Density	1,058,550	570,816	2,542,317	4,171,683
Large Lot	7,264	404,069	273,625	684,958
Large Lot (P3)	226,956	156,266		383,222
Pisa Moorings	498,120	69,435	1,880,626	2,448,181
Low Density	55,713	10,622	1,242,246	1,308,582
Large Lot (P1)	442,407	58,812	638,380	1,139,599
Rural	276,404	81,775	153,304	511,483
Large Lot			20,199	20,199
Large Lot (P2)	276,404	81,775	133,105	491,284
Grand Total	2,545,336	2,144,038	6,274,961	10,964,335

Addendum – Sensitivity Analysis

Document Title:

Cromwell Yield Assessment. Addendum – Sensitivity Analysis

Date:

18/04/2023

Prepared for:

Central Otago District Council

Quality Assurance Statement

Rationale Limited	Project Manager:	Ben Gonzalez
5 Arrow Lane	Prepared by:	Ben Gonzalez
PO Box 226	Reviewed by:	Neil Jorgensen
Arrowtown 9351	Approved for issue by:	Neil Jorgensen
Phone: +64 3 442 1156	Job number:	J001316

Introduction

Further to the initial yield assessment, CODC engaged Rationale to do sensitivity analysis on the original model. The questions presented were:

- what is the yield from PC19 excluding plan change areas (Cherry Orchard and Parkburn),
- how would this yield change if the minimum lot size for the Low Density zone was reduced from 500sqm to 400sqm.

The same methodology and assumptions were used as per the original model. Note that the Golf Course which was considered an additional plan change area in the original report is now considered to be part of the PC19. As per the original model, 90% of growth in rural areas has been shifted to occur within Cromwell (see Section 3 for discussion on this).

Results

YIELDS

Table 7 presents the results, with the yellow section showing the yield from PC19, and the blue section showing the yield from plan change areas. The Grand Total at the bottom reflects the sum of the two.

Township and Zone	Existing Capacity	Total Feasible Capacity (@ 500sqm)	Total Feasible Capacity (@ 400sqm)	Additional Capacity @400sqm vs @500sqm
Bannockburn	267	507	507	0
Large Lot	240	480	480	0
Large Lot (P2)	0	0	0	0
Large Lot (P3)	0	0	0	0
Outside	27	27	27	0
Cromwell	3,056	9,261	10,313	1,052
Large Lot	73	250	250	0
Large Lot (P3)	40	40	40	0
Low Density	2,042	5,470	6,522	1,052
Medium Density	799	3,399	3,399	0
Outside	102	102	102	0
Pisa Moorings	498	528	563	35
Large Lot (P1)	416	423	423	0
Low Density	82	105	140	35
Outside	0	0	0	0
Rural	420	428	428	0
Large Lot	0	6	6	0
Large Lot (P2)	56	58	58	0
Outside	364	364	364	0
PC19 Subtotal	4,241	10,724	11,811	1,087
Cherry Orchard	0	271	338	67
Low Density	0	271	338	67
Parkburn	0	1,108	1,385	277
Low Density	0	1,108	1,385	277
Plan Change Areas Subtotal	0	1,379	1,723	344
Grand Total	4,241	12,103	13,534	1,431

Table 7: Yield assessment sensitivity

The results indicate that by changing the min. lot size rule the total feasible capacity of PC19 would increase by 1,087, from 10,724 to 11,811. Most of the additional capacity is generated in urban Cromwell where there is many Low Density lots.

The min. lot size rule change would also have material impact on the feasible capacity from the plan change areas, increasing the yield by 344 from 1,379 to 1,723.

SUFFICIENCY

Figure 10 compares feasible capacity against the demand forecast. Demand is shown as an orange line, with the confidence band around it representing the low and high scenarios of demand.

The @500sqm model (model that assumes a 500sqm min. lot size for Low Density zone) is shown with black lines, with the solid line showing the yield from PC19 only and the dashed line showing the yield from PC19 plus the plan change areas. Meanwhile, the @400sqm model (model that assumes a 400sqm min. lot size for Low Density Zone) is shown with green lines in a similar manner.

The results suggest that sufficiency under PC19 alone @500sqm would be tight in a scenario of high demand growth. Changing the rule to 400sqm would provide a more substantial buffer to allow for the potential of higher-than-anticipated growth.

Interestingly, the impact of changing the min. lot size rule generates a similar number of additional dwellings as that expected from the addition of the two plan change areas to PC19.

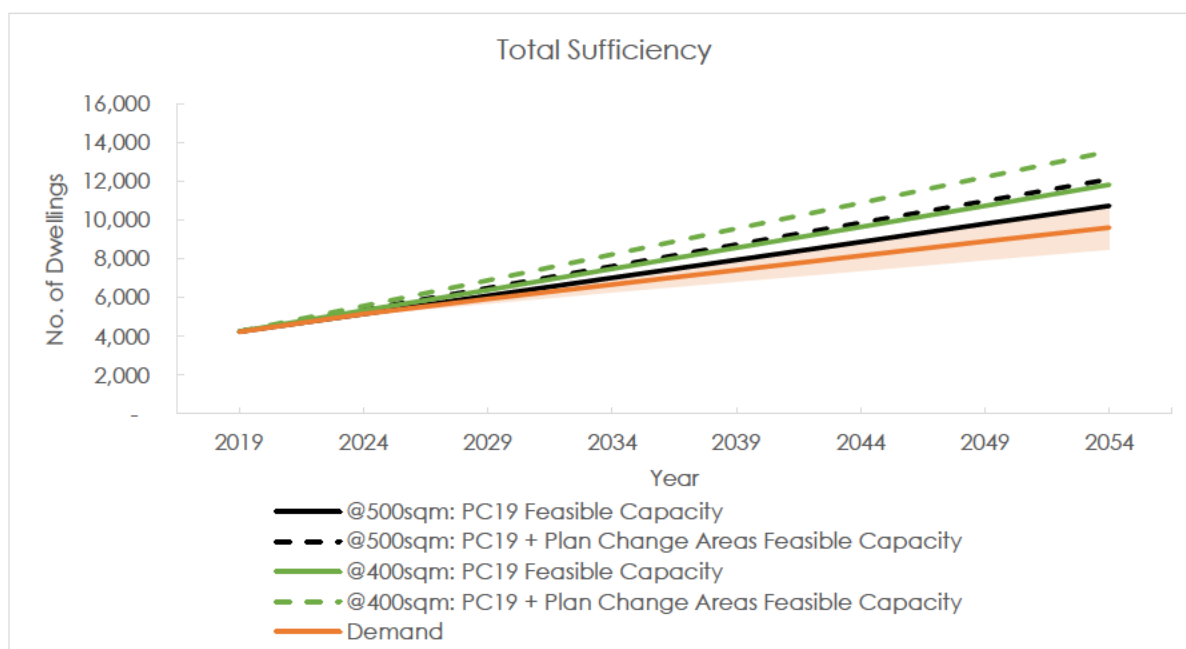


Figure 10: Total sufficiency under different sensitivity options

This same assessment is presented for each township in Figures 11, 12 and 13.

- Bannockburn's assessment remains unchanged as it does not hold Low Density lots or plan change areas.
- Cromwell's feasible capacity would increase by just over 1,000 dwellings with the min. lot size rule change. This would provide further capacity to accommodate spill-over from Bannockburn and neighbouring districts (Queenstown).
- Pisa Moorings' additional capacity is generated primarily by the plan change areas. Without these, there is little additional capacity and demand would not be met. The rule change generates an additional 380 dwellings in this area.

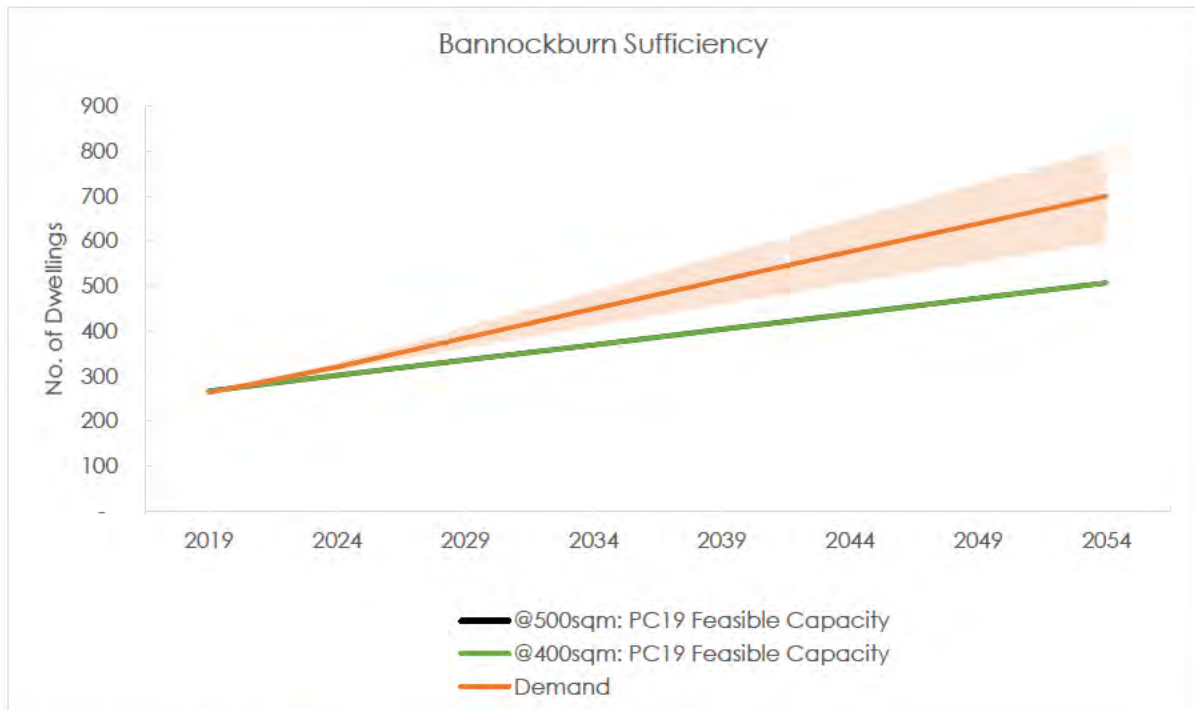


Figure 11: Bannockburn sufficiency under different sensitivity options

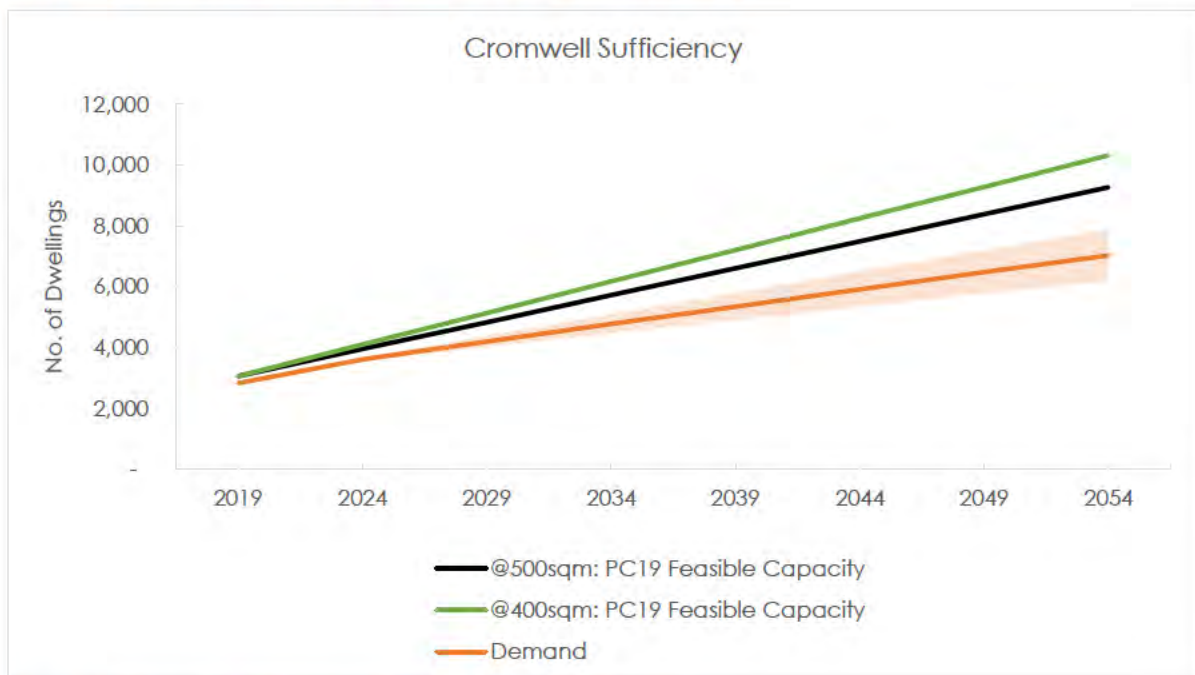


Figure 12: Cromwell sufficiency under different sensitivity options

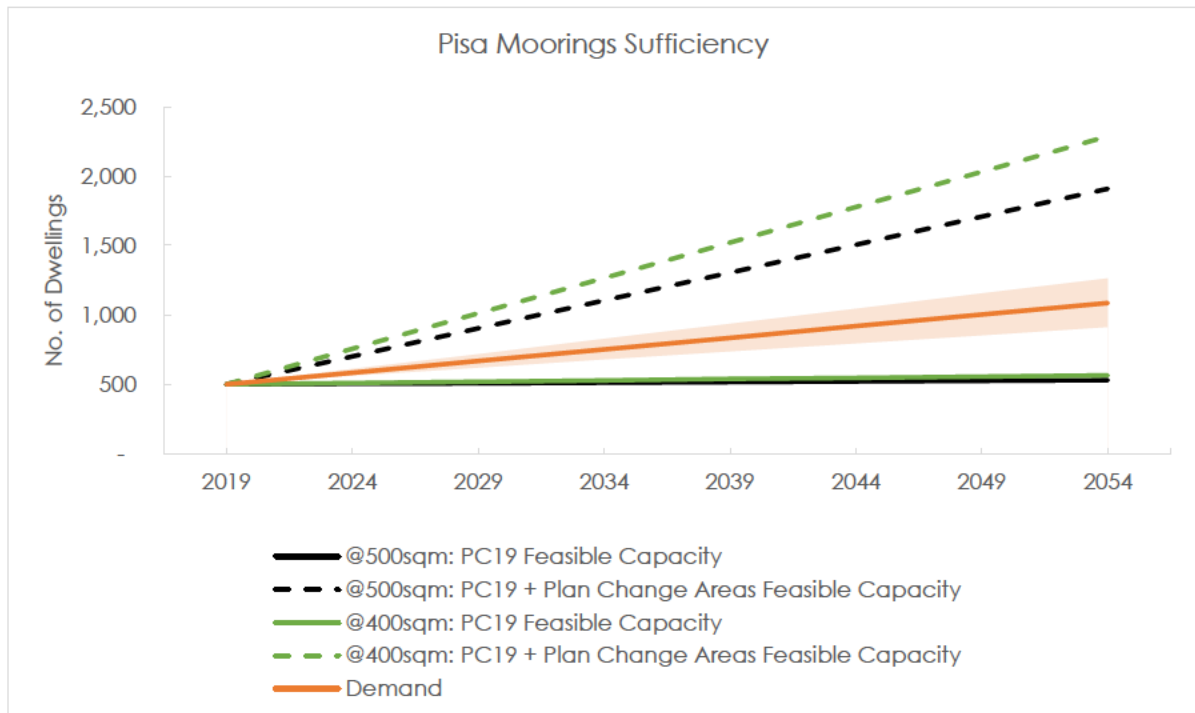


Figure 13: Pisa Moorings sufficiency under different sensitivity options

Conclusion

The sensitivity analysis suggests that changing the minimum lot size for the Low Density zone from 500sqm to 400sqm would have a material impact on the feasible capacity, allowing for an additional 1,087 dwellings from PC19 and 344 from plan change areas. Overall, this represents an additional 18% capacity.

Assessment of sufficiency requires considerations of growth scenarios. The original PC19 provides enough capacity for low-to-medium growth in the area. However, a high demand scenario would severely strain sufficiency under the original PC19, with a deficit of 50 dwellings expected by 2054. Either the inclusion of the plan change areas or the change to the minimum lot size would mitigate this risk and allow a buffer for high growth. The adoption of both these would allow for a total capacity of 13,534 dwellings, which is over 2,700 more dwellings than that demanded under the high growth scenario.

Cherry Orchard Development – 3 Waters Assessment
for Fast Track Referral

ATTACHMENT B:

Cromwell Cherry Orchard
Development Water and
Wastewater Demand Assessment
11 May 2026

SUMMARY MEMORANDUM

TO: Julie Muir (CODC)

FROM: Nigel Lloyd, General Manager Civil Engineering, Hadley Consultants Ltd

REVIEWED BY: James Hadley, Principal, Hadley Consultants Ltd

DATE: 11 May 2026 (initially issued 10 March 2026)

SUBJECT: Cromwell Cherry Orchard Development Water & Wastewater Demand Assessment
Ripponvale Road, Cromwell (Revision 1)

PROJECT: 254029

1.0 Introduction

Hadley Consultants Limited (HCL) have been engaged by Rippon Holdings Limited (RHL) and Rippon Investments Limited (RIL) to assess and report on the anticipated water demand and wastewater generation arising from the proposed Cherry Orchard Development (COD). This memorandum has been prepared to assist Central Otago District Council (CODC) with infrastructure master planning and consideration of the servicing feasibility. It outlines the development density and population assumptions adopted by HCL for the purposes of assessing the ultimate demand on potable water and wastewater infrastructure arising from the proposed COD.

It is expected that the demand and generation allowances as assessed will be applied to the network modelling for wastewater and potable water. The network modelling is to be undertaken by Mott McDonald and confirmed as a part of the detailed design of water supply and wastewater infrastructure that will be undertaken for the COD. The COD site is located on Lots 1 to 4, DP344529, Lot 1 to 4 DP420009 and part of Lot 1000, DP61599 and is approximately 69.8 hectares in area.

2.0 Overview of Assessment Methodology

This assessment has been developed in accordance with the CODC Land Development and Subdivision Engineering requirements that are based on NZS4404:2004 and the 2008 CODC Addendum to this document. We note that CODC are in the process of transition to NZS4404:2010.

Where possible we have also compared these allowances to other available standards and guidance including of NZS 4404:2010, Water NZ guidelines, Christchurch City Council (CCC) Infrastructure Design Standard (IDS) and the Queenstown Lakes District Council (QLDC) Land Development and Subdivision

Code of Practice (CoP) to ensure that a robust baseline for water demand and wastewater generation is adopted for infrastructure planning and design purposes.

3.0 Nature of Proposed Development and Ultimate Yield

The development yield allowances have been developed collaboratively with input from HCL, Rough Milne Mitchell (RMM), and the wider COD project team. Ultimately the RMM Development Masterplan as provided 06 May 2026 as indicated in Figure 1 below has been adopted as representing the anticipated ultimate development density and yield across the various land use typologies that are proposed, including various residential, retail and industrial activities within the COD site.



Figure 1 – COD Masterplan, RMM, 06 May 2026

Based on our assessment of the RMM Development Masterplan the ultimate development yield is as shown in the following table.

Development Type	Area (Ha)	Lots
Medium Density Residential Lots	-	94
Low Density Residential Lots	-	581
Large Lot Residential Lots	-	80
Commercial (Light Industrial & Retail)	8.29	-
Total	8.29Ha	755 Lots
	(Commercial)	(Residential)

Table 1 – COD Site Ultimate Yield

The yield as detailed above defines the development and forms the basis upon which the population and various demand and generation factors adopted for the water demand and wastewater generation assessment are applied in the following sections.

4.0 Occupancy / Population Allowances

Clause 6.11.5 of the CODC 2008 Addendum specifies that a population of 3.0 people per residential unit is to be used when assessing water demand. No guidance is provided for wastewater; therefore, we have referred back to NZS 4404:2004. The findings of our review of guidance contained in other similar standards are outlined in the table below.

Source	Recommended Population Allowance
CODC 2008 Addendum (water)	3.0 people / residential unit
NZS4404:2004 (wastewater)	2.5 to 3.5 people/dwelling
NZS4404:2010 (wastewater)	2.5 to 3.5 people/dwelling
QLDC CoP (water)	3.0 people/residence-
CCC IDS (wastewater)	2.7 persons/lot

Table 2 – Occupancy / Population Allowances

Following a review of the various guidance sources outlined above, we have adopted an occupancy / population allowance of 3.0 people per residential unit for the purposes of assessing both water demand and wastewater generation. This approach is aligned with the CODC allowance for water and the NZS 4404 allowance for wastewater.

No guidance is provided in regard to the number of dwellings or residential units to be allowed per lot. We note however that multi-unit house and income type developments are becoming relatively common and in light of the recent changes to the building consent exemptions for granny flats this is likely to increase further.

QLDC considers a residential flat or secondary dwelling, when contained on a lot with a main dwelling, to represent 66% of a dwelling equivalent.

Recent data from large developments within QLDC area, particularly Hanley's Farm, indicates that approximately 70% of lots are being developed to contain a residential flat.

In the absence of any other clear guidance, for conservatism we have adopted the above figures to assess the ultimate number of dwelling equivalents to be serviced within the COD site. It is noted that this approach has only been adopted for large lot residential and low density residential as medium density residential lots are considered too small for a residential flat.

Development Type	Lots	Minor Secondary Dwelling Uptake	Minor Secondary Dwellings	Dwelling Equivalents / Minor Secondary Dwelling	Dwelling Equivalents
Medium Density Residential Lots	94	0%	0	66%	94
Low Density Residential Lots	581	70%	408	66%	849
Large Lot Residential Lots	80	70%	56	66%	117
Retail and Industrial		-	-	-	-
Total	755 Lots (Residential)				1,060

Table 3 – COD Ultimate Dwelling Equivalents

The resulting ultimate residential population associated with the COD is presented in the table below.

Development Type	Lots	Dwelling Equivalents	People / Dwelling Equivalent	Population
Medium Density Residential Lots	94	94	3.0	282
Low Density Residential Lots	581	849	3.0	2,548
Large Lot Residential Lots	80	117	3.0	351
Retail and Industrial			Assessed separately-	
Total	757 Lots (Residential)	1,063		3,181

Table 4 – COD Ultimate Residential Population

5.0 Potable Water Demand Assessment

This section outlines the various potable water demand parameters adopted for the assessment and provides a comparison of the CODC demand parameters against relevant industry guidance, including the CCC IDS, NZS 4404, the QLDC Code of Practice and Water NZ data. The comparison has been undertaken to confirm that the demand assumptions applied are representative of anticipated ultimate development conditions and are consistent with accepted New Zealand practice for three waters infrastructure design. Where differences exist between the referenced documents, the adopted allowances have been selected to provide a robust and appropriately conservative basis for design.

5.1 Per Capita Water Demand

Clause 6.11.5 of the CODC 2008 Addendum specifies a per capita water demand of 500 litres/person/day for residential development. The table below provides a comparison of this per capita water demand against various other residential standards and guidelines.

Source	Recommended Daily Consumption Allowance
CODC 2008 Addendum	500 L/p/day
NZS4404:2004	250 L/p/day
NZS4404:2010	250 L/p/day
QLDC CoP	700 L/p/day
	250 L/p/day was supported by usage data
CCC IDS	Not individually specified
Water NZ	226 National Average (600 for Cromwell)

Table 5 – Per Capita Water Demand Allowances

Following review of the various guidance in the sources as outline above, for the purposes of this water demand assessment we have adopted an average per person water demand of 500 litres per day for residential land use. This aligns with the stated CODC daily consumption figures.

The resultant ultimate average residential water demand arising from the COD is presented in the following table

Development Type	Population	Per Capita Consumption	Average Daily Water Demand	
Medium Density Residential Lots	282	500L/p/day	141m ³ /day	1.6L/s
Low Density Residential Lots	2,548	500L/p/day	1,274m ³ /day	14.8L/s
Large Lot Residential Lots	351	500L/p/day	175m ³ /day	2.0L/s
Retail and Industrial	Assessed separately			
Total	3,181		1,600m ³ /d	18.4L/s

Table 6 – COD Ultimate Average Residential Water Demand

No specific guidance regarding water demand for commercial land use is provided within the CODC Addendum or NZS 4404. However, it is common practice to adopt the guidance used for commercial wastewater generation when assessing water demand. This approach is outlined in detail in Section 6.1 below. Based on a total retail and industrial area of 8.29 hectares (including roads and car parks) and applying a design flow allowance of 0.4 litres per second per hectare, the peak water demand from the retail and industrial area is assessed as 3.32 litres per second.

5.2 Fire Fighting Water Demand

Fire fighting water requirements are detailed in SNZ PAS4509:2008 and for an individual site typically depend on the hazard type, fire cell size and any mitigation provided on individual sites such as sprinkler systems. For the early stages of a greenfield development such as this when the nature of individual

development is unknown, it is considered appropriate to adopt a minimum level of service to be provided at the site and then design the relevant headworks elements of the water supply infrastructure accordingly.

It is anticipated that the COD will be serviced by new water reservoir(s) located adjacent to the existing Shannon Farm water supply reservoir. Additional land has recently been vested in CODC ownership in this location for this purpose.

Fire fighting water demand will be catered for by way of static reservoir storage and reticulation to the various fire hydrants. This reservoir and reticulation infrastructure will be new works and provided as a part of the COD. For this reason, we do not anticipate the fire water demand to add demand or impact the wider CODC water network, irrespective of the Fire Water classification adopted within the COD site.

For the purposes of this water demand assessment, at this stage we have adopted Fire Water classification FW3 which will typically accommodate small to moderate unsprinklered commercial developments or provide adequate water for fire sprinkler systems and the necessary residual FW2 fire fighting water supply. At this stage we have adopted this as the minimum level of service to be provided to the COD site. This equates to a fire demand of 50L/s for 60 minutes with a minimum fire fighting water storage volume of 180m³.

We recommend the actual Fire Water classification and level of service to be provided be assessed further and confirmed through the detailed design process when further information is likely to be available regarding the nature of the commercial development likely to occur. We note that a lesser FW2 classification could be applied to the residential portion of the development but the minimum FW3 classification will govern overall demand in terms of reticulation design and reservoir storage. In the case that the nature of the commercial development requires a higher Fire Water classification, this will simply require increased storage at the Shannon Farm reservoir site and larger diameter reticulation to be provided as a part of the COD. It would not impact or require any additional upgrades of the existing CODC water supply network.

There is no clear guidance provided in any of the standards reviewed in regard to how this type of infrequent emergency fire water demand should be applied in conjunction with mixed domestic and commercial demand.

Consistent with the approach adopted by QLDC, we have assumed that fire flows should be considered in conjunction with 60% of the Peak Day flow for the sizing of reticulation. For the purposes of this assessment, and in the absence of any other clear guidance we have adopted this approach for design of water supply reticulation downstream of the assumed water reservoir.

5.3 Peaking Factors

Clause 6.11.5 of the CODC 2008 Addendum specifies a peak day peaking factor of 3 and a peak hour peaking factor of 5 for residential development.

The table below provides a comparison of this per capita water demand against various other residential standards and guidelines.

Source	Peak Day Peaking Factor	Peak Hour Peaking Factor
CODC 2008 Addendum	3	5
NZS4404:2004	1.5 (population >10,000) 2 (population <2,000)	2 (population >10,000) 5 (population <2,000)
NZS4404:2010	1.5 (population >10,000) 2 (population <2,000)	2 (population >10,000) 5 (population <2,000)
QLDC CoP		4.0 Queenstown 6.6 Other areas
CCC IDS	Not individually specified	

Table 7 – Water Peaking Factors

We note that the CODC Peak Day peaking factor of 3 is high compared to other data sources that we have reviewed. We assume that this potentially reflects the seasonal nature of the area and also the high irrigation water demands that occur locally during prolonged dry periods. Following review of the various guidance in the sources as outline above, for the purposes of this water demand assessment we have adopted a Peak Day peaking factor of 3.0 and a Peak Hour peaking factor of 5.0 in accordance with the CODC standards. These will be applied to the average day water demand to calculate the peak day and peak hour water demand.

5.4 Overall Water Demand Assessment

The key water demand allowances considered and adopted above are summarised as follows.

- Per Capita Water Demand = 500 L/person/day
- Peak Day Peaking Factor = 3.0 (ADF to PDF)
- Peak Hour Peaking Factor = 5.0 (ADF to PHF)
- Minimum Fire Water Classification = FW3, 50L/s instantaneous for 60mins and 180m³ storage

The resultant ultimate peak residential water demand arising from the COD is presented in the following table.

Development Type	Average Day Flow (ADF)	Peak Day Flow (PDF)	Peak Hour Flow (PHF)	Minimum Fire Flow (FF)
Medium Density Residential Lots	141m ³ /day	423m ³ /day	8.2L/s	
Low Density Residential Lots	1,274m ³ /day	3,822m ³ /day	73.7L/s	
Large Lot Residential Lots	175m ³ /day	526m ³ /day	10.2L/s	
Retail and Industrial	57m ³ /day	172m ³ /day	3.3L/s	
Total	1,648m ³ /day	4,944m ³ /day	95.4L/s	50L/s 180m ³

Table 8 – COD Ultimate Peak Water Demand

As noted earlier it is anticipated that the COD will be serviced by a new reservoir(s) at the expanded Shannon Farm water reservoir and reticulation the proposed fire fighting and peak hour flow

water demands will be provided for internally and will not have a material impact on the external CODC water supply network. For the purposes of the wider CODC external water supply infrastructure the key ultimate demand figures are the average day and peak day flow figures as presented above in Table 8.

The ultimate demand will increase over time as the development proceeds and this is described in more detail within Section 7 to assist CODC with their modelling and Capital Works projections.

6.0 Wastewater Generation Assessment

This section outlines the various wastewater generation parameters adopted for the assessment and provides a comparison of the CODC generation parameters against relevant industry guidance, including the CCC IDS, NZS 4404, the QLDC Code of Practice and Water NZ data. The comparison has been undertaken to confirm that the generation assumptions applied are representative of anticipated ultimate development conditions and are consistent with accepted New Zealand practice for three waters infrastructure design. Where differences exist between the referenced documents, the adopted allowances have been selected to provide a robust and appropriately conservative basis for design.

6.1 Per Capita Wastewater Generation

The CODC 2008 Addendum does not specify wastewater generation figures so CODC would typically revert back to NZS4404:2004 which recommends an average dry weather flow of between 180 and 250 litres per person per day for residential development.

The table below provides a comparison of this per capita water demand against various other residential standards and guidelines.

Source	Recommended Daily Generation Allowance
CODC 2008 Addendum	Not specified
NZS4404:2004	180 to 250 L/p/day
NZS4404:2010	180 to 250 L/p/day
QLDC CoP	250 L/p/day
CCC IDS	220 L/p/day

Table 9 – Per Capita Wastewater Generation Allowances

Following review of the various guidance in the sources as outline above, for the purposes of this wastewater generation assessment we have adopted an average per person wastewater generation of 250 litres per day for residential land use. The resultant ultimate average dry weather wastewater generation arising from the residential component of the COD is presented in the following table.

Development Type	Population	Per Capita Generation	Average Dry Weather Flow	
Medium Density Residential Lots	282	250L/p/day	70.5m ³ /day	0.8L/s
Low Density Residential Lots	2,548	250L/p/day	637.1m ³ /day	7.4L/s
Large Lot Residential Lots	351	250L/p/day	87.7m ³ /day	1.0L/s
Retail and Industrial			Considered separately below	
Total	3,181		795.3m ³ /day	9.2L/s

Table 10 – COD Ultimate Residential Average Dry Weather Wastewater Flows

Wastewater generation from industrial / commercial landuse is covered by Clause 5.3.5.1(b) and Table 5.2 of NZS4404:2004. Where no specific flow information is available for the proposed activities, as is currently the case with the retail and industrial component of the COD, the real design flows presented in Table 5.2 should be used as shown in the figure below.

Table 5.2 – Industrial/commercial flows

Industry type (Water usage)	Design flow (Litre/second/hectare)
Light	0.4
Medium	0.7
Heavy	1.3

Figure 2 – Table 5.2, NZS4404:2004

It should be noted that the design flow allowances presented above represent peak design flows and are inclusive of peaking allowances. This methodology is consistent across both the 2004 and 2010 versions of NZS4404 and the QLDC Code of Practice.

For the purposes of this assessment, we have assumed that the retail and industrial area of the proposed development will represent light water usage as at this stage the area considered is total potential development area that will include internal roads, accesses, carparks and other non-productive areas when developed.

Based on a total retail/industrial area of 8.3 hectares and a design flow allowance of 0.4 litres per second per hectare the peak wastewater design flow from the retail and industrial area is assessed as 3.3 litres per second.

6.2 Peaking Factors

No wastewater peaking factors are specified in the CODC 2008 Addendum so the CODC standards would typically revert back to NZS4404:2004 that specifies a dry weather diurnal peaking factor of 2.5 and a wet weather infiltration / dilution peaking factor of 2. The table below provides a comparison of this per capita water demand against various other residential standards and guidelines.

Source	Dry Weather Diurnal Peaking Factor	Wet Weather Peaking Factor
CODC 2008 Addendum	Not specified	
NZS4404:2004	2.5	2
NZS4404:2010	2.5	2
QLDC CoP	2.5	2
CCC IDS	1.8	2.78

Table 11 – Wastewater Peaking Factors

Following review of the various guidance in the sources as outline above, for the purposes of this water demand assessment we have adopted a dry weather diurnal peaking factor of 2.5 and a wet weather peaking factor of 2.0 in accordance with NZS4404. These will be applied to the average dry weather wastewater generation to calculate the peak dry weather and peak wet weather flows.

6.3 Overall Wastewater Generation Assessment

The key wastewater generation allowances considered and adopted above are summarised as follows.

- Per Capita Wastewater Generation = 250 L/person/day
- Diurnal Peaking Factor = 2.5 (ADWF to PDWF)
- Wet Weather Peaking Factor = 2.0 (PDWF to PWWF)

The resultant ultimate peak wastewater generation arising from the COD is presented in the following table.

Development Type	Average Dry Weather Flow		Peak Dry Weather Flow (Diurnal)	Peak Day Wet Weather Flow	
Medium Density Residential Lots	70.5m ³ /day	0.8L/s	2.0L/s	141m ³ /day	4.1L/s
Low Density Residential Lots	637.1m ³ /day	7.4L/s	18.4L/s	1,280m ³ /day	36.9L/s
Large Lot Residential Lots	87.7m ³ /day	1.0L/s	2.5L/s	175m ³ /day	5.1L/s
Retail and Industrial	57.3m ³ /day	0.7L/s	1.7L/s	117m ³ /day	3.3L/s
Total	853m³/day	9.9L/s	24.7L/s	1,710m³/day	49.3L/s

Table 12 – COD Ultimate Wastewater Generation

Note that the retail and industrial figures presented above in italics have been back calculated from peak flow using the same peaking factors as applied to residential wastewater generation.

7.0 Development Uptake

The ultimate water demand and wastewater generation figures assessed above will not be realised until the COD is complete. Therefore, to assist with infrastructure master planning and potential upgrades we have also considered the likely development uptake and resultant demand over time.

At this stage the COD is anticipated to occur over an estimated 8 to 12-year timeframe with the first development stages being completed in 2029. For this assessment we have considered two potential development scenarios being a fast-progression and a slow-progression pathway outlined as follows.

- The **fast development** scenario assumes that the full development is completed over a duration of 8 years with an initial surge of residential development occurring in 2029 (year 1) with the remainder spread evenly across years 2 to 8 and the retail and industrial development delivered rapidly in years 2 and 3.
- The **slow development** scenario assumes that the full development is completed over a duration of 12 years with an initial surge of residential development occurring in 2029 (year 1) with the remainder spread evenly across years 2 to 12 and the retail and industrial development delivered evenly across years 3 to 5.

The assumed residential and commercial development uptake across the two scenarios considered is indicated in the following chart.

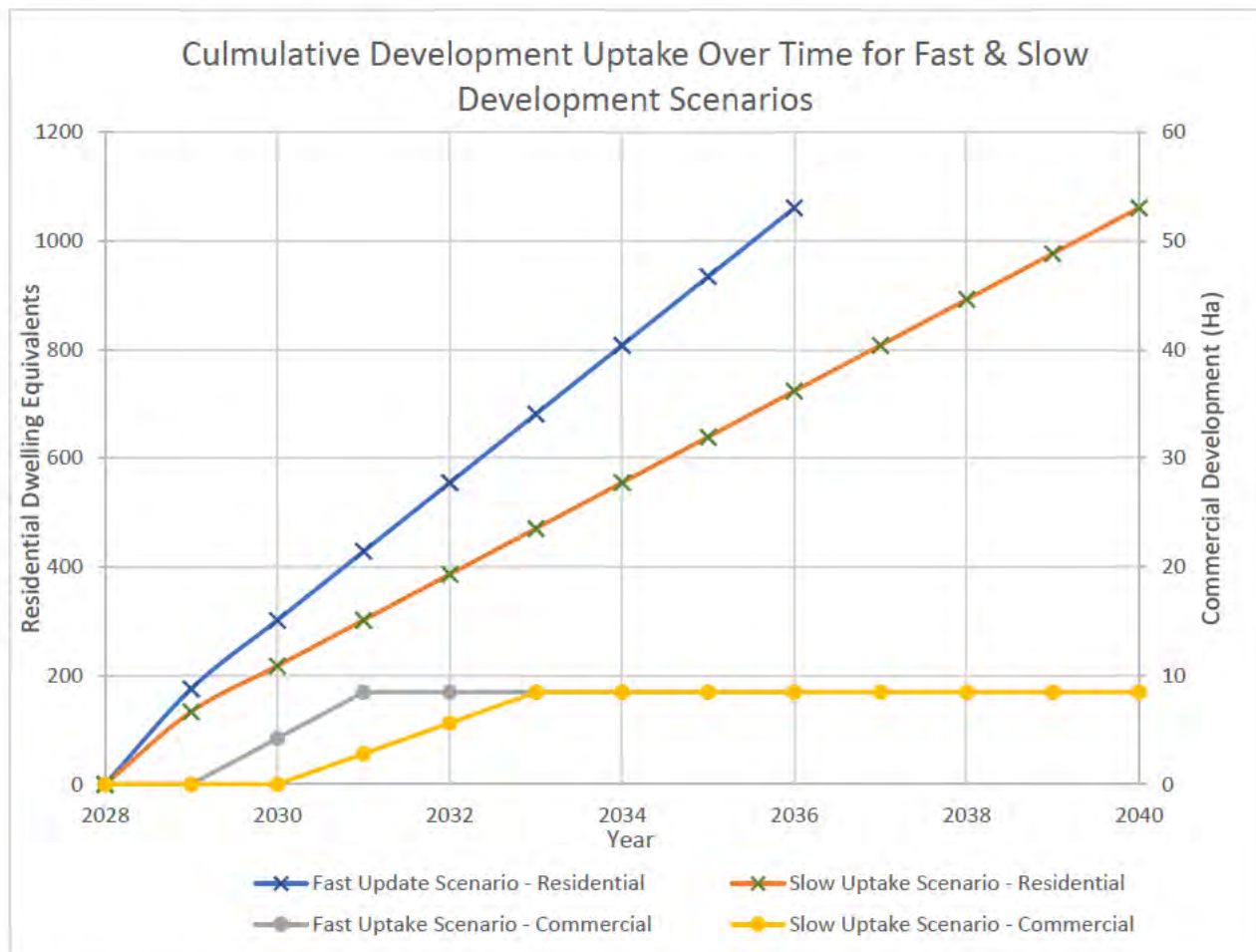


Figure 3 – Chart illustrating development uptake over time for various scenarios

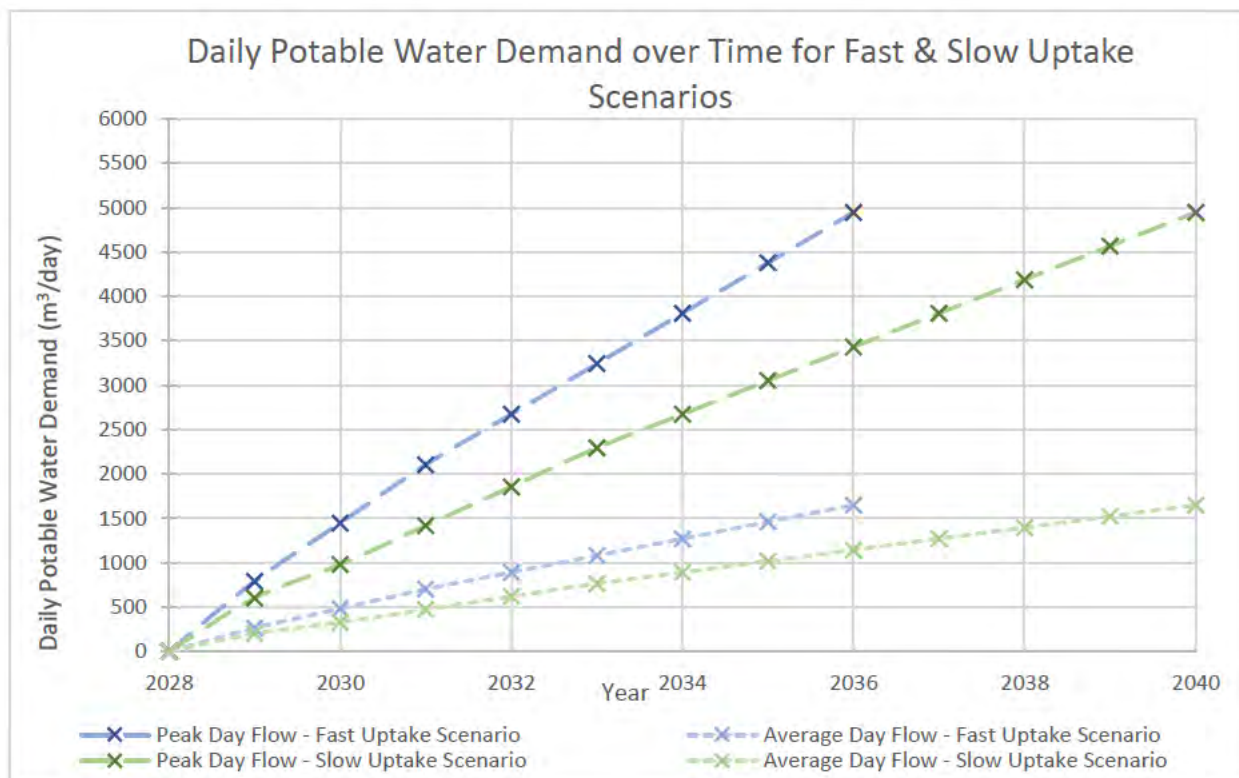


Figure 4 – Chart showing daily potable water demand over time for various scenarios

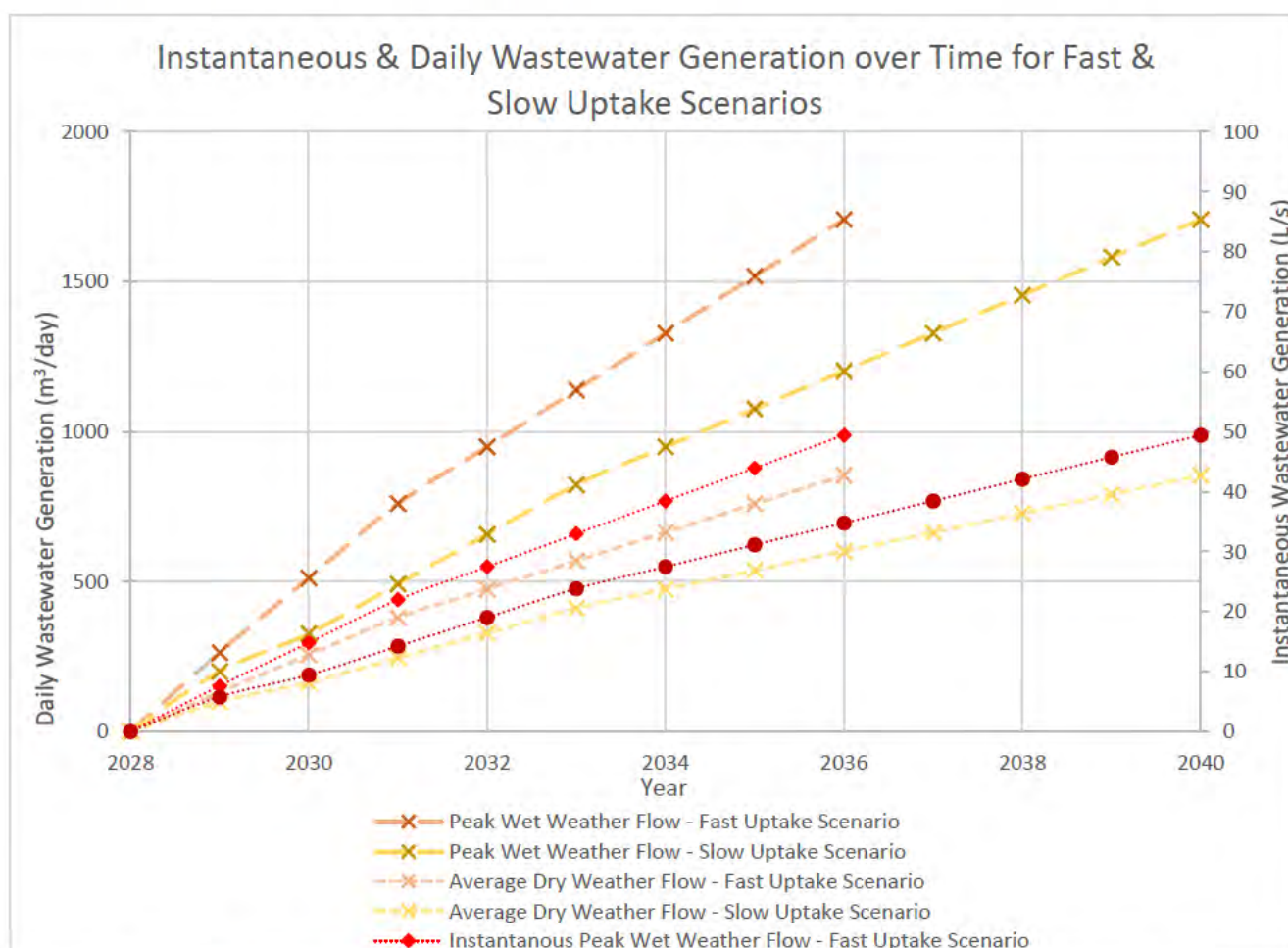


Figure 5 – Chart showing daily & instantaneous wastewater generation over time

8.0 Conclusions

The potable water demand and wastewater generation assessment for the COD outlined herein aligns with the CODC Engineering Standards within the 2008 Addendum and the guidance of NZS 4404:2004. Based on our review and comparison with other relevant standards, including NZS 4404:2010, the QLDC Code of Practice and the CCC Infrastructure Design Standards, we consider that a robust assessment has been undertaken for the ultimate demands of the COD.

The COD is anticipated to be delivered over an approximately 8–12-year period, and two development scenarios have been considered to indicate how demand will evolve over time. The proposed potable water demand and wastewater generation will occur incrementally, allowing the necessary infrastructure upgrades to be planned and implemented in a staged manner. The key metrics for the ultimate COD are summarised below.

Potable Water

- ADF = 1,648m³/day
- PDF= 4,944m³/day
- PHF= 95.4L/s
- Minimum Fire Flow= 50L/s and 180m³

Wastewater

- ADWF= 853m³/day
- PDWF = 24.7L/s
- PWWF = 1,705m³/day and 49.3L/s

To enable progression to detailed water and wastewater network modelling and internal design, HCL requests confirmation from CODC that the COD density assumptions presented in this memorandum are accepted. Specifically, we draw attention to and seek confirmation on the allowances made for secondary dwellings as these are outside of the guidance included within the CODC Engineering Standards within the 2008 Addendum and NZS 4404:2004.

On the basis that the allowances are acceptable we request that the demand information be provided to Mott McDonald to prepare both Wastewater and Potable Water Impact Assessments. We confirm that the author is available to discuss any queries or potential optionality regarding this demand assessment and request that for efficiency any Mott McDonald queries be sent direct to HCL. HCL will collaborate with and copy Mott McDonald on all responses to refine modelling and network impact assessments.

In parallel RHL/RIL will assess in more detail the level of service to be provided to the commercial portion of the development in regard to Fire Water classification, noting that this will not impact the wider CODC water supply network.

Cherry Orchard Development – 3 Waters Assessment
for Fast Track Referral

ATTACHMENT C:

Cromwell Cherry Orchard
Development Stormwater
Feasibility Memorandum
11 May 2026

SUMMARY MEMORANDUM

TO: Paul Croft and Jerome O'Sullivan (Rippon Holdings Limited and Rippon Investments Limited)

FROM: Nigel Lloyd - Hadley Consultants Ltd
Mark Pennington, Alex White - Tonkin and Taylor Ltd

REVIEWED BY: James Hadley, Managing Director, Hadley Consultants Ltd

DATE: 11 May 2026

SUBJECT: Cromwell Cherry Orchard Development Stormwater Feasibility Memorandum
Ripponvale Road, Cromwell

PROJECT: 254029

1.0 Introduction

Hadley Consultants Limited (HCL) has been engaged by Rippon Holdings Limited and Rippon Investments Limited (RIL) to assess and report on a feasible stormwater solution for the proposed Cherry Orchard Development (COD). The COD is located in Cromwell, adjacent to State Highway 6 and Ripponvale Road. The development encompasses a total area of approximately 69.79 hectares, distributed across seven land parcels listed below:

- Lot 2 and Lot 4, Deposited Plan 420009 (Record of Title 476974) – 2.68 hectares
- Lot 1 and Lot 3, Deposited Plan 420009 (Record of Title 476973) – 6.00 hectares
- Lot 1, Deposited Plan 344529 (Record of Title 182714) – 8.00 hectares
- Lot 2, Deposited Plan 344529 (Record of Title 182715) – 8.00 hectares
- Lot 3, Deposited Plan 344529 (Record of Title 182716) – 8.00 hectares
- Lot 4, Deposited Plan 344529 (Record of Title 182717) – 8.00 hectares
- Part of Lot 1000, DP 61599 (Record of Title 1233739) – 233.18 hectares

The proposed COD will convert the existing cherry orchard land into a mixed-use development comprising approximately 80 large residential lots, 581 low-density residential lots, 94 medium-density residential lots, a 3.10 hectare commercial precinct, and a 5.19 hectare industrial area. The proposed layout is illustrated in Figure 1.



Figure 1 – Extent of Proposed COD

This memorandum confirms the existence of a feasible stormwater drainage solution for the COD and outlines potential cost optimisation opportunities to be explored during later design stages. The following investigations have been undertaken:

- Detailed desktop review of existing reports.
- Site walkover.
- Soakage testing and physical investigations.
- Estimation of design rainfall using a TUFLOW model calibrated to the October 2025 event.
- Identification of flood-prone areas.
- Assessment of feasible servicing solutions.
- Consideration of potential design optimisations .

2.0 Overview of Rainfall in Cromwell

Cromwell experiences a relatively dry climate, with the average annual rainfall being approximately 398 mm, which is notably low compared with other regions in New Zealand.

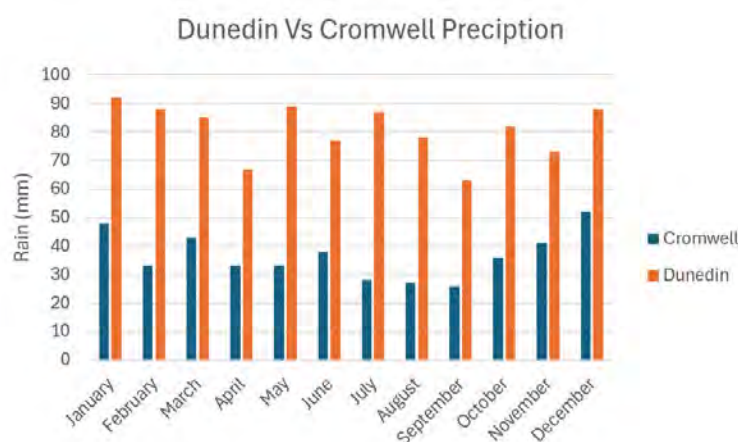


Figure 2 - Precipitation in Cromwell vs Dunedin

In many areas has sandy or gravelly soil conditions that generally allow for significant infiltration, however this is variable as pockets or layers of low-permeability material are found in various locations.

These conditions result in many catchments exhibiting non-linear behaviour, with infrequent runoff under typical rainfall conditions. However, during significant storm events, infiltration capacity, or near-surface storage capacity (where contained above layers of impermeable material) can be exceeded resulting in notable flooding and surface inundation.

3.0 Upslope Catchment

The COD site is influenced by both upslope and downslope catchments. Existing topography and stormwater infrastructure indicate five key overland flow paths (OFPs) affecting the site. Figures 3 and 4 demonstrate both the three-dimensional topography and the key OFP's in place at present. These key OFP's and other existing and proposed stormwater infrastructure are shown in Attachment A.



Figure 3 – Three Dimensional Terrain in the Area

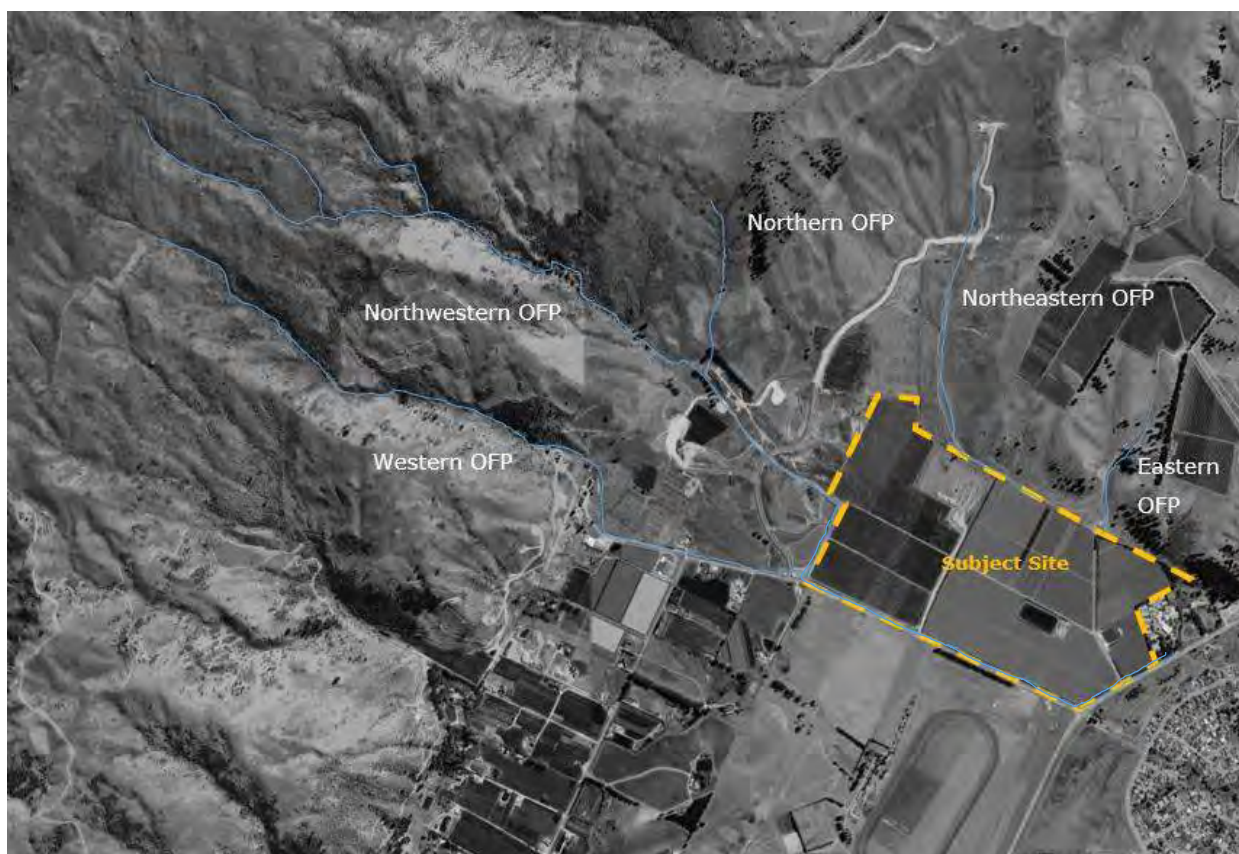


Figure 4 – Key Overland Flow Paths and Associated Shannon Farm Flood Channels

As part of the investigation completed by HCL two separate historic stormwater reports have been considered.

1. Attachment B: Flood Hazard Assessment for 114 Ripponvale Road prepared by Geosolve and dated 24 May 2019

2. Attachment C: Stormwater Modelling Report for Shannon Farm by Clark Fortune McDonald (CFMA) dated 23 June 2023 and a subsequent amendment dated 25 June 2025

These reports provide valuable background information, particularly regarding the operational Shannon Farm stormwater network.

Historical aerial imagery (sourced via Retrolens.co.nz) indicates clear paleochannel alignments west of State Highway 6, while no such features are evident to the east.

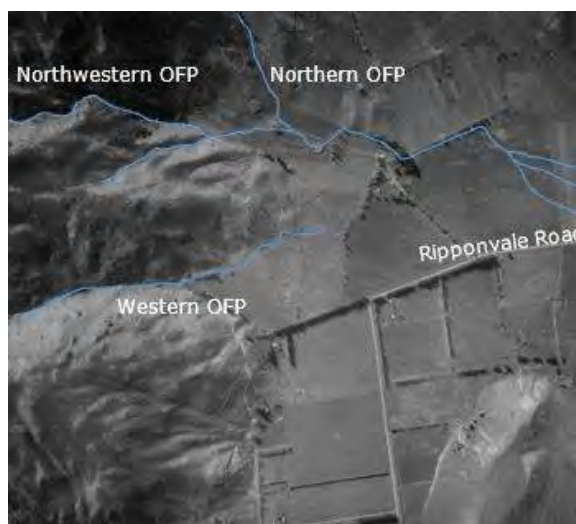


Figure 5A – Retrolens Aerial Image 1950 with visible paleochannels shown in blue



Figure 5B – Retrolens Aerial Image 1950 with visible paleochannels shown in blue



Figure 5C – Retrolens Aerial Image 1962 with visible paleochannels shown in blue



Figure 5D – Retrolens 1973 with visible paleochannels shown in blue

Five minor unnamed OFPs intersect the COD site from the northern boundary. Given their relatively small scale, these are expected to be managed through cut-off drains and flood bunding designed during detailed design. The minor unnamed OFP's have been identified in figure 6.

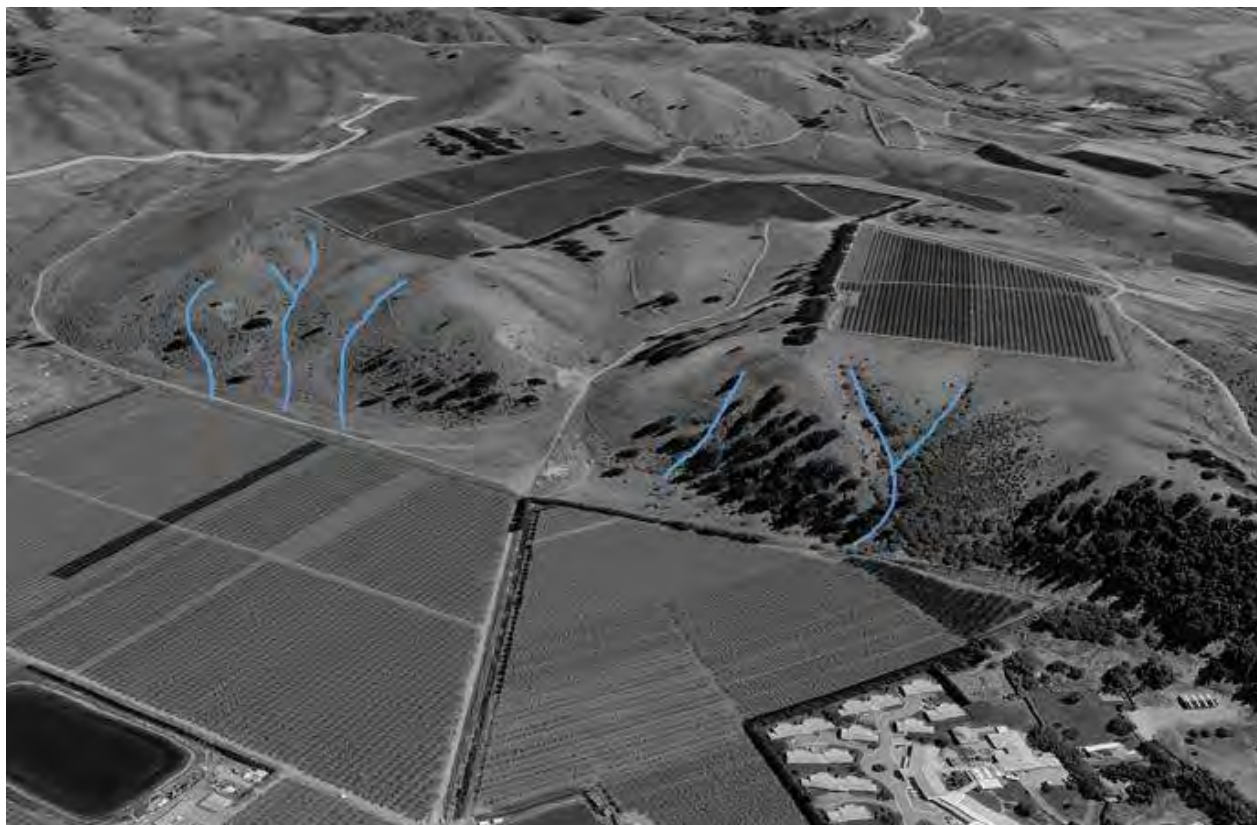


Figure 6 – Minor OFP Intersecting with the COD Site

4.0 Flood and Inundation Events

Design rainfall events occur periodically and are essential for understanding the site-specific factors influencing flooding behaviour at the COD. Two significant rainfall events relevant to the site have been identified to inform and calibrate the stormwater assessment:

- 17 November 1999
- 23 October 2025

The larger of these two events occurred in 1999. Our record of this event is contained in the attached Geosolve report dated 24 May 2019 (Attachment B). The 1999 flood is generally regarded as the flood of record in the Southern Lakes region and caused widespread damage. Significant flooding occurred in Queenstown Town Centre when Lake Whakatipu reached a maximum level of 312.77 m above sea level. In 2019, Geosolve consulted with the owner of the Shannon Farm property located upslope of the COD.

The property owner (who has occupied the site since the early 1970s) estimated the peak flow within the western gully to be approximately 0.4 m³/s at an unspecified location. Geosolve noted that this estimate is substantially lower than the peak flows for the western catchment, which they estimated at 4.74 m³/s. They further observed that:

- There is no reliable means of verifying the observed flow value
- Upstream processes (e.g. overflow or flow diversion) may have reduced flows at the observation point
- The estimate should therefore be treated with caution

Geosolve considered the reported flow to be anomalously low, noting that peak flood discharges in Central Otago catchments are typically in the order of: Approximately 1 m³/s per km² (subject to variability depending on catchment characteristics).

While the 1999 event is widely regarded as the flood of record and is generally estimated to have approximated a 100-year return period event in major catchments (largely due to snowmelt influences), Geosolve advised that this classification does not necessarily apply to smaller catchments such as that associated with this site. They further noted that:

- Short-duration, high-intensity rainfall drives peak flows in smaller catchments
- Such rainfall was not particularly extreme during the 1999 event

Accordingly, Geosolve concluded that peak flows associated with a design 100-year flood event may exceed those experienced in 1999.

The more recent possible design rainfall event on record occurred on 23 October 2025. For this event, more robust observations are available, including rainfall records from Anderson Park, approximately 1.5 km from the subject site. These rainfall records are included in the figures below.

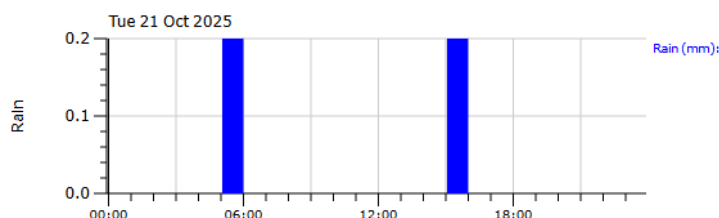


Figure 7A – Rainfall records 21 October 2025 Anderson Park, Cromwell – 0.4mm Total

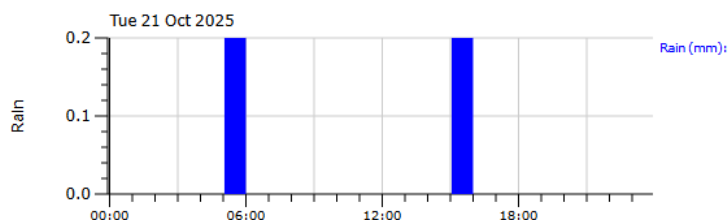


Figure 7B – Rainfall records 22 October 2025 Anderson Park, Cromwell – 0.4mm Total

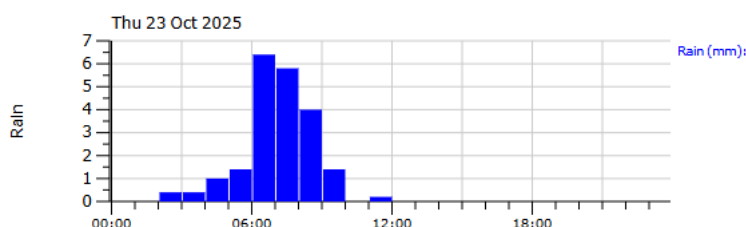


Figure 7C – Rainfall records 23 October 2025 Anderson Park, Cromwell – 21.0mm Total

The 23 October rainfall event predominantly occurred between 6:00 am and 9:00 am, with minor rainfall occurring approximately:

- Two hours prior to the main event
- One hour following the main event

Photographs of the rainfall event were captured and have been provided to HCL. When the rainfall record is compared to the NIWA HIRDS V4 tool, the event appears to align with design data as falling between a two- and five-year return period, based primarily on the below points:

- The critical duration for most of the catchment is approximately 2 hours.
- Recorded rainfall intensities align between NIWA 2-year and 5-year design values, as shown in the table below.

Table 1 – 23 October rainfall comparison to HIRDS design guidance.

Rainfall Type	1 hr (mm / hr)	2 hr (mm / hr)	3 hr (mm / hr)
Historic Rainfall NIWA HIRDS 2-year event	8.37	6.07	5.23*
Historic Rainfall NIWA HIRDS 5-year event	11.7	8.34	7.14*
Recorded Rainfall	6.25	6	5.33

*These values have been interpolated from NIWA HIRDS V4

The photographs from the 23 October event are displayed in the figure below with notations about the time of the photograph and it's location.



Figure 8A – 9.30am 23 October Flood Channel Three at Shannon Farm Detention Basin



Figure 8B – 9.30am 23 October Flood Channel Three at Shannon Farm Detention Basin looking down Rippon Vale Road towards SH6



Figure 8C – 12.40am 23 October Flood Channel Three at Shannon Farm Detention Basin

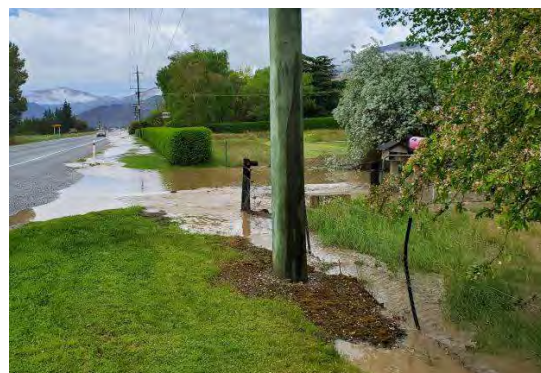


Figure 8D – 12.40am 23 October Flood Channel, Rippon burn Retirement Home looking west along SH6.

At the time of the rainfall event, the flood channels constructed as part of the Shannon Farm development were operational, resulting in Flood Channel Three along Rippon Vale Road becoming operational.

It should be noted that rainfall records at durations shorter than one hour are not available, meaning that 10-, 20- and 30-minute duration events cannot be directly assessed. However, the critical duration for most of this catchment is approximately two hours. Figure 9 illustrates the maximum inundation depth from a TUFLOW model run with rain-on-grid inputs designed to replicate the 23 October 2025 event and calibrate catchment conditions including infiltration.



Figure 9 – Tuflow Model Reflecting 23 October Storm Event – Maximum Inundation Depth

If a 100-year design rainfall event under an RCP 8.5 climate change scenario were to occur, an intensity of approximately 19.8 mm/hr over two hours would be expected. This represents approximately 3.3 times the rainfall intensity experienced during the 23 October 2025 event.

5.0 Downslope Catchment and Existing Information availability

Typically, territorial authorities in New Zealand will have both detailed GIS information associated with their stormwater assets and LiDAR terrain data obtained via Aeroplane. The LiDAR information is usually also held by Land Information New Zealand (LINZ).

In the case of the downstream catchment below COD there is no LiDAR and only rudimentary stormwater GIS information. While it is understood that Central Otago District Council (CODC) are in the process of gathering this information it doesn't exist currently. The figure below shows CODC stormwater assets that have sufficient information to prepare a three-dimensional stormwater model.

Typically, territorial authorities in New Zealand maintain detailed GIS datasets for stormwater assets, along with LiDAR-derived terrain data collected via aerial survey. LiDAR data are also commonly available through Land Information New Zealand (LINZ).

However, for the downstream catchment below the COD site, available information is limited. Specifically:

- No LiDAR data are currently available for the downstream catchment
- Stormwater GIS information is incomplete and considered rudimentary
- It is understood that the Central Otago District Council (CODC) is in the process of improving both the data sets with work expected to be completed this year

Figure 10 below illustrates the existing CODC stormwater assets for which sufficient information is currently available to support three-dimensional modelling.

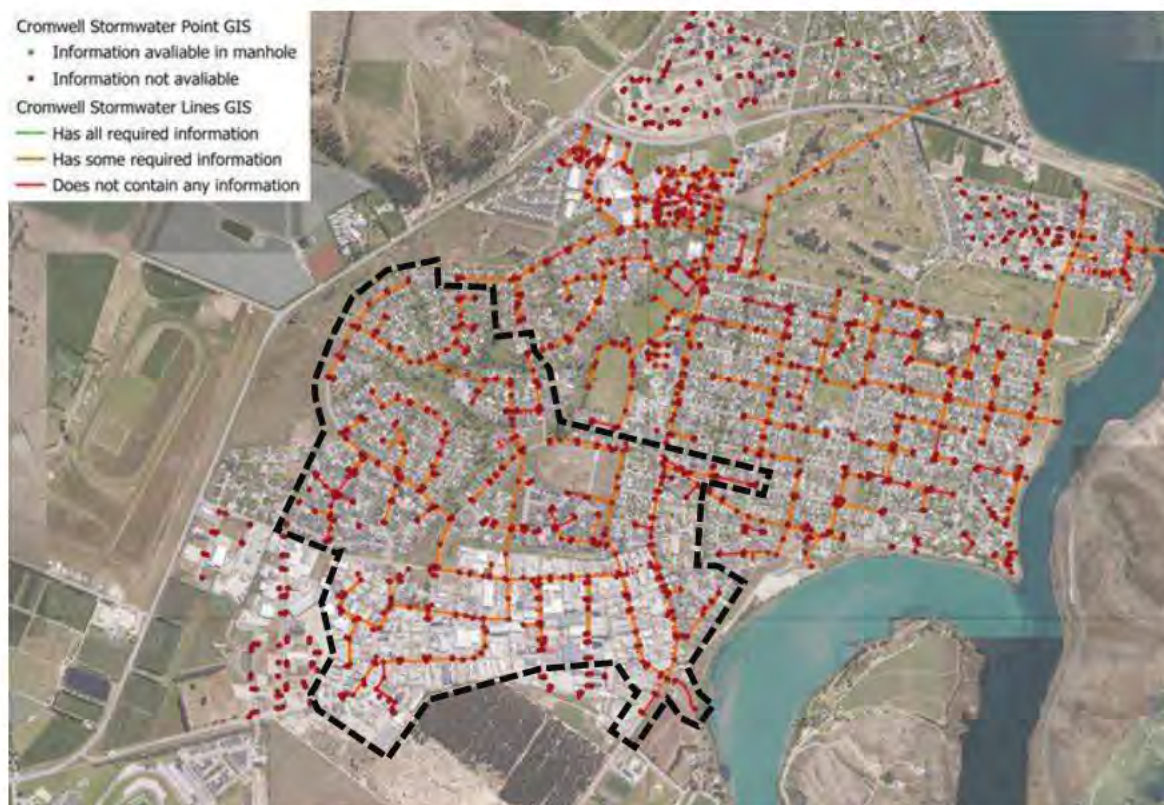


Figure 10 - CODC Stormwater GIS Cromwell

The limited availability of downstream data means that an optimised stormwater solution cannot be developed at this stage without undertaking extensive additional data collection. In particular, the absence of LiDAR surface data and detailed information on existing stormwater infrastructure constrains the accuracy of modelling and design.

The approximate reticulated catchment area downstream of the COD site is shown in Figure 10 as a black dashed line. Prior to completion of the detailed design phase, this area will need to be fully surveyed and clearly understood by the Civil Engineer to ensure that the final solution is effective. Indicatively a separate trunk line and outlet has been shown in the appended Stormwater drawing set, which has been used for the initial modelling purposes to confirm a solution is feasible. There is likely

scope for project optimisation by utilising some existing infrastructure, or providing improved level of service and futureproofing to the existing downstream catchment by utilising the new infrastructure.

6.0 Soakage & Soakage Testing

Cromwell typically has soil strata capable of soaking large volumes of stormwater into the ground. This is due to extensive areas being underlain by Middle Pleistocene river deposits. However, there are also areas with overlying or intermixed alluvial fan deposits, which can consist of cohesive soils with limited permeability. These alluvial fans are associated with smaller overland flow paths (OFPs), such as those that discharge into Shannon Farm and the COD site.

Recent developments in Cromwell, such as Wooing Tree, have successfully relied on soakage to ground to manage increased runoff from development. This approach has been effective due to the absence of significant OFPs and good access to permeable Middle Pleistocene deposits. Soakage systems can offer advantages over traditional reticulated stormwater systems, particularly in managing contaminants such as heavy metals and hydrocarbons from roads and roofs, thereby reducing impacts on downstream receiving environments. However, these systems are not without risk, as they may become clogged over time, reducing effectiveness and potentially leading to long-term flooding and inundation issues.

The COD and Shannon Farm catchments are more complex, as they are influenced by both on-site runoff and upslope OFPs. The upslope catchments include five key OFPs, as described in Section 3. Given the volume of water expected from these OFPs during design rainfall events, careful management of stormwater is critical when establishing a feasible solution for the proposed development.

Preliminary physical soakage testing was commissioned by HCL as part of this feasibility assessment and was undertaken on 9 March and 11 March.

The locations of the test sites are shown in Figure 11. Testing was carried out at three locations within the extents of Detention Basins 1, 2, and 3, where a total of eight tests were completed. Primarily designed as detention basins, these tests will inform the likelihood and quantity of soakage likely to occur in the basins.



Figure 11 – Soakage Test Locations

Two different testing types have been selected to obtain background information to inform the feasible solution put forward by this memorandum. The two-testing types selected are Double Ring Infiltration Tests and Constant Discharge Tests. In total eight test have been undertaken six Double Ring Infiltration Tests and two Constant Discharge Tests. We note the constant discharge test was not able to be completed at basin 3 due to the existing netting over the Cherry Orchard.

The Constant Discharge tests were augured into the ground approximately 3.5m deep in an attempt to get below the future Detention Basin Surface. The permeability testing of the aquifer was carried out using a 650mm diameter hole drilled by auger into the soil, and a 650mm diameter steel casing emplaced. During the drilling process some sand and silt material descended to the base of the hole; this was largely removed by pumping prior to the testing procedure. Following this a 50 mm diameter slotted standpipe was placed within the casing, near the casing side. Approximately 0.6 m³ of 100 mm diameter clean drainage metal was placed at the bottom of the cased hole, around the standpipe assembly. The casing was retracted by 0.5 m. The drainage metal effectively supports the exposed gravel strata at the bottom of the hole when the casing is retracted (in effect acting as a screen). The standpipe allows precise measurement of water levels within the cased bore. The idea was to understand the infiltration rates and hydraulic connectivity of the soils directly below the basins. However, the two tests obtained soakage rates of 0mm/hr as on both attempts the casing penetrated into a cohesive soil strata made up of fine grain material. The table below summarises the results with detailed test results summarised in Attachment D.

Table 2 – Test Result Constant Discharge Test

	Unfactored soakage (mm/hr)
Detention Basin 1	0
Detention Basin 2	0
Detention Basin 3	Not completed

It is likely that the impermeable alluvial fan could be penetrated to allow soakage if required. This was achieved at Shannon Farm Detention Basin three, where four Cauldwell pits were drilled through the alluvial fan to reach suitable Middle Pleistocene River deposits below. The Cauldwell pits extended six metres below the invert of Shannon Farm Detention Basin three, which itself is approximately 5.5 m below the existing ground level. A standard detail of a Cauldwell soak pit is included in figure 12.

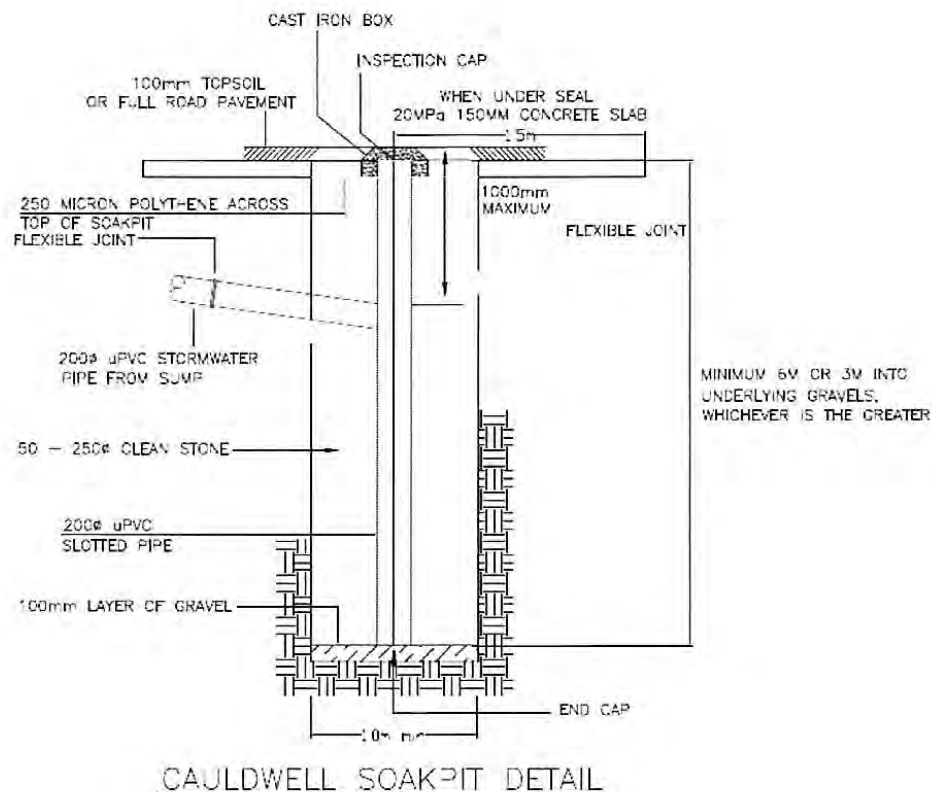


Figure 12 – Example Cauldwell Soak pit

It has been advised by the designers Clark Fortune McDonald and Associates that under test conditions the systems in place at Shannon Farm can discharge 80 L/s to the aquifer below. Further testing is required to confirm that COD Detention Basins 1, 2 and 3 could have similar soakage systems constructed.

The other tests completed at the subject site where six Double Ring Infiltration Tests which tested soakage of the existing surface strata. These tests consisted of large and small concentric rings driven into the ground to a depth of between 100mm and 150 mm. The inner ring and outer ring are filled with water to a depth of 100 to 200 mm above the surface after pre-soaking. The rate at which water

drops within the inner ring is measured at intervals ranging from 1 minute to 1 hour, depending on the soil type. Measurements are undertaken until there is an equal fall over three consecutive readings. The table below summarises the results with detailed test results summarised in Attachment D.

The results indicate that near-surface soakage is likely to occur, however the rate is quite variable.

Table 3 – Adopted Test Result Double Ring Infiltration Test

	Unfactored Soakage (mm/hr)	Measurement Frequency (min) after presoak
COD Detention Basin 1 - Test 1	660	1
COD Detention Basin 1 - Test 2	840	1
COD Detention Basin 2 - Test 1	300	1
COD Detention Basin 2 - Test 2	0*	1
COD Detention Basin 3 – Test 1	0*	1
COD Detention Basin 3 – Test 2	30	2

*(Prematurely terminated by Contractor on site, retesting required at detailed phase)

7.0 Master planning of the COD Site

The subject site receives flows from five key OFPs which, if unmanaged, would traverse the COD site as a combination of sheet flow and channel flow. Flows from the western, north-western, and northern OFPs would pass through the Shannon Farm flood channels and then generally follow Ripponvale Road. The north-eastern and eastern flood channels would flow as sheet flow across the development. The maximum expected flooding and inundation for the 1% Annual Exceedance Event (AEP), based on existing COD contours, is shown in figure 13 below.



Figure 13 – Current Flooding and Inundation Situation without Stormwater Master planning of the COD site 1% AEP critical durations RCP 8.5 design event

HCL worked with Rough Milne Mitchell Landscape Architects (RMM) and other consultants throughout late 2025 and early 2026 to coordinate the stormwater strategy for the site. As part of this process, HCL ensured that stormwater from the five OFPs can pass safely through the proposed COD site. The four largest OFP will pass through the site in flood channels and the remaining eastern OFP will pass through the site in a combination of flood channels and a piped network.

Within the development, a number of green spaces have been incorporated into the layout. These areas are connected by dry channels, forming an integrated internal drainage network. An example of similar dry basins and flood channels from subdivisions in the Lakes District are shown in the figures below.



*Figure 14A – Example Proposed Flood Channels
(Jacks Point, Queenstown)*



*Figure 14B – Example Proposed Dry basin
(Northlake, Wanaka)*

Based on the preliminary soakage testing undertaken to date, it is likely that in the optimised solution that these green spaces will be able to drain a portion of stormwater to ground via a series of Cauldwell pits, however, conservatively this hasn't been utilized at this stage. An example of the proposed masterplan from the RMM Urban design Report, including the open dry channels, is shown in figure 15.



Figure 15 – Proposed COD Masterplan with Flood Channels Terminating at Ripponvale Road

Three dry basins are proposed within the COD development, with a further two dry basins constructed as part of Shannon Farm incorporated into the feasible solution. As discussed above, these have been sized on the basis of no soakage occurring, and are shown to be feasible under that assumption. Further optimisation is likely possible.

The proposed flood channels are located within stormwater corridors ranging from 15m to 20m in width and have been modelled with depths between 0.9m and 1.1m. Flood channel diagrams and dimensions are included in the drawing set attached as Attachment 2.

It is also important to note that the Eastern and Northeastern OFP's will need some form of flood bunding or cut-off drains where they intersect with the subject site to prevent flooding of adjacent Lots and associated habitable floors. The flood bunding or cut-off drains for the Northeastern OFP will need to direct flows into the reserve and towards Detention Basin 2B, shown above as utilizing flood channels. The flood bunding or cut of drains for the Eastern OFP will need to direct flood waters towards COD Detention Basin 3, shown above as utilizing a piped connection.

8.0 Feasible Downstream Solution

This memorandum has focused on one feasible solution to transmit stormwater from the subject site through the downstream catchment below the subject site. Although several optimisations are discussed these are not relied upon and are considered only as a possibility for future investigation. It is acknowledged that the proposed feasible solution proposed requires significant capital investment and

may not represent the most cost-effective solution, however it is created at this phase to confirm that a viable solution exists.

The proposed feasible downstream solution is a 750mm pipe which runs 3.32 km from Ripponvale Road, along SH6, McNulty Road and out to the Kawarau River. The pipe has been modelled in three dimensions using basic contour information obtained from the Land Information New Zealand website.

The high level current conceptual design shows pipes with gradients of between 0.33% and 1.18% falling towards the outlet, confirming physical viability.

We note that the design has not yet been checked for clashes with other existing services, which may require resolution at later design stages. Manholes have been included in the design at changes in direction only, and additional MH's will be required to be compliant with NZS4404:2004 or other CODC amendments. The figure below shows the proposed alignment of the 750mm pipe from the COD to the Kawarau River.



Figure 16 – Overview of stormwater discharge alignment from COD to the Kavarau River

It is noted that if consenting of the proposed feasible solution is attempted through the Resource Management Act 1991 discharge consents from Otago Regional Council (ORC) may be required.

9.0 Conceptual Modelling and Calculation of the Feasible Downstream Solution

To progress the stormwater strategy and high-level design, a site-specific flood model has been developed using TUFLOW to quantify the likely surface flooding effects of the proposed development. The model is intended to provide a high-level understanding of the existing flood mechanisms affecting the site, and how these mechanisms are expected to change as a result of on-site earthworks and land use changes.

The model has been constructed using TUFLOW software and includes the following components:

- A model extent encompassing the entire hydrological catchment in which the site is located, largely covering the extent of the topographical survey provided by Clark Fortune McDonald and Associates
- Rain-on-grid hydrology, adopting the HIRDS temporal profile and HIRDS v4 rainfall depths for the 2-hour storm duration event (RCP 8.5 2081-2100), identified as critical during the model development stage. Total rainfall depths for the 5% Annual Exceedance Probability (AEP) and 1% AEP events are 24.5 mm and 35.7 mm, respectively
- Roughness zones derived from national datasets (created by Tonkin and Taylor), with roughness values assigned using standard lookup tables
- Soil infiltration parameters based on a preliminary calibration to the rainfall event that occurred in 23 October 2025.

The level of model refinement is intended to align with the current preliminary stage of design. As the design progresses and becomes more detailed, the flood model will be refined accordingly. Figure 17 illustrates the predicted maximum inundation depths during a 1% AEP RCP 8.5 storm event of critical duration, and generally indicates that inundation will be controlled to the proposed basins only.

Changes in peak flood level between pre- and post- development are shown in Figure 18 below. This figure shows that there are no increases in peak flood level outside of the development extent, and in some cases a reduction in peak flood levels in some existing ponding areas.

The site-specific flood model covers only the COD and the upslope catchments. As such, the feasibility of a downstream pipe solution has been assessed through a conceptual three-dimensional pipe layout and supporting calculations using Manning's equation. Given that gradients in several of these pipes reduce to approximately 0.33% to limit pipe depths, it is important to confirm that the pipes can convey the 1% AEP design event and, in accordance with NZS 4404:2004 Section 4.3.3.7, achieve adequate flushing velocities.

In flat areas, pipe gradients should be as steep as practicable to minimise silt deposition. The minimum velocity should be at least 0.6 m/s at a flow equal to half of the 50% AEP design flow. Indicatively both the minimum velocity requirement and the 1% AEP capacity can be achieved with the concept alignment.

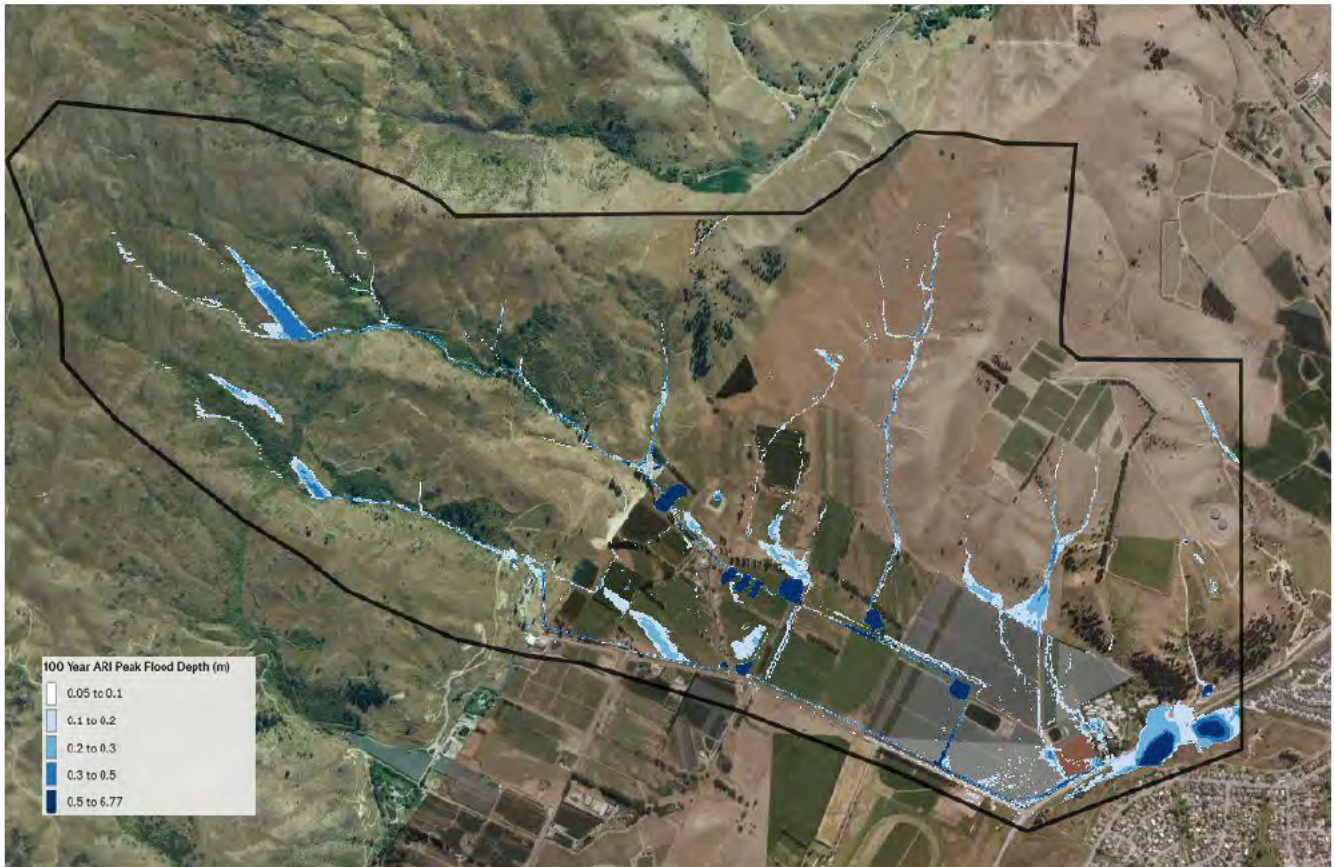


Figure 17 – Maximum inundation critical duration 1% AEP storm

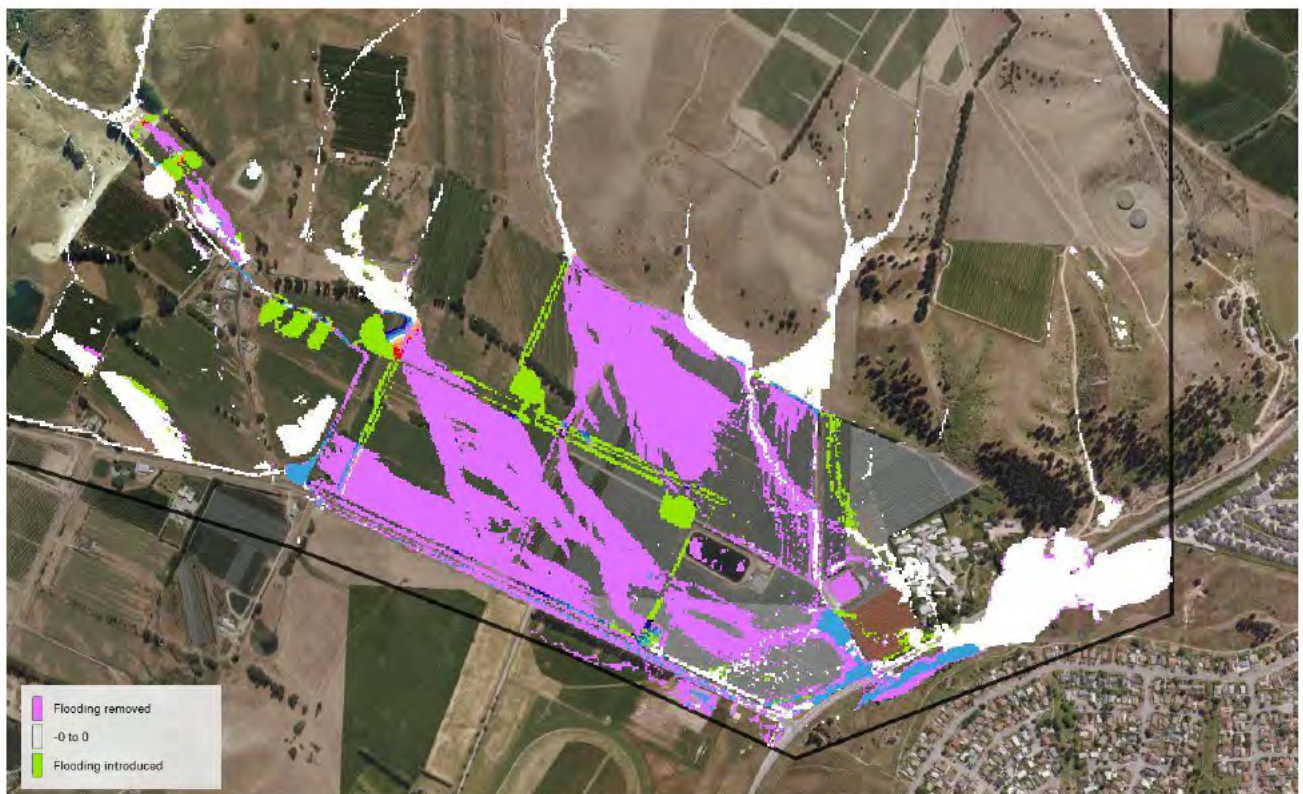


Figure 18 – Maximum inundation critical duration 1% AEP storm

10.0 Future Optimisation Options

It is expected that the optimal solutions developed during the later design phase will be a combination of both soakage and pipe discharge. A range of solutions were considered during the concept phase but have not been progressed at this stage. Possible future optimisation options are listed below:

1. Extend the flood channel along Ripponvale Road to reduce pipe length
2. Extend the flood channel along the Cromwell Racetrack to reduce pipe length
3. Utilise soakage via Cauldwell Pits in Detention Basins 1, 2A, 2B and 3
4. Utilise the existing Cromwell stormwater network for controlled low-level discharge:
 - a. Waenga Drive stormwater network
 - b. McNulty Road stormwater network
5. Add additional dry attenuation basins in Shannon Farm to further attenuate flows and reduce the downstream pipe.

11.0 Summary

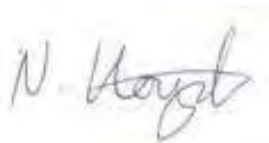
A stormwater model has been developed based on survey data and previous assessments of the subject site and its surrounds. This model has been calibrated based on recorded observations from known prior storms, most critically in October 2025.

Based on this stormwater model, a concept stormwater servicing option has been developed which is feasible to implement to service the site without adverse effects on surrounding properties.

The concept design requires significant further detailing and verification prior to implementation, and there exist multiple optimisations that could also be investigated (such as including onsite soakage), however it confirms that stormwater servicing of the site is feasible should the project be carried forward.

Yours faithfully,

Hadley Consultants Ltd



Nigel Lloyd

General Manager, Civil

ATTACHMENTS

Attachment A: HCL Drawings

Attachment B: Geosolve Flood Hazard Assessment for Shannon Farm

Attachment C: CFMA Shannon Farm Stormwater Report

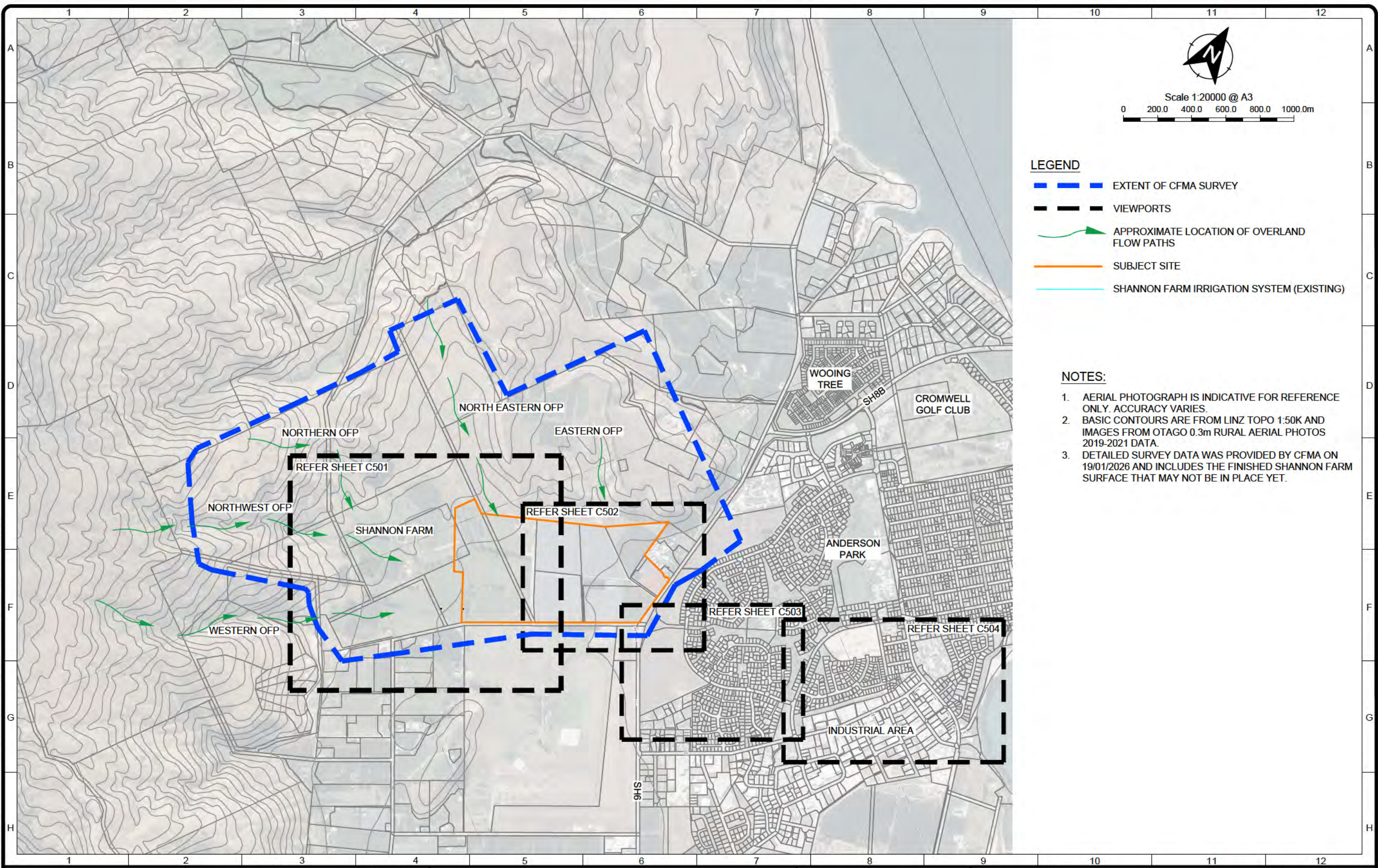
Attachment D: Soakage Testing Results

Cromwell Cherry Orchard Development Stormwater
Feasibility Memorandum

ATTACHMENT A:

HCL Stormwater Concept Design
Drawings

ORIGINAL SIZE A3 DO NOT SCALE, REFER ALL DISCREPANCIES TO THE ENGINEER



Scale 1:20000 @ A3
0 200.0 400.0 600.0 800.0 1000.0m

LEGEND

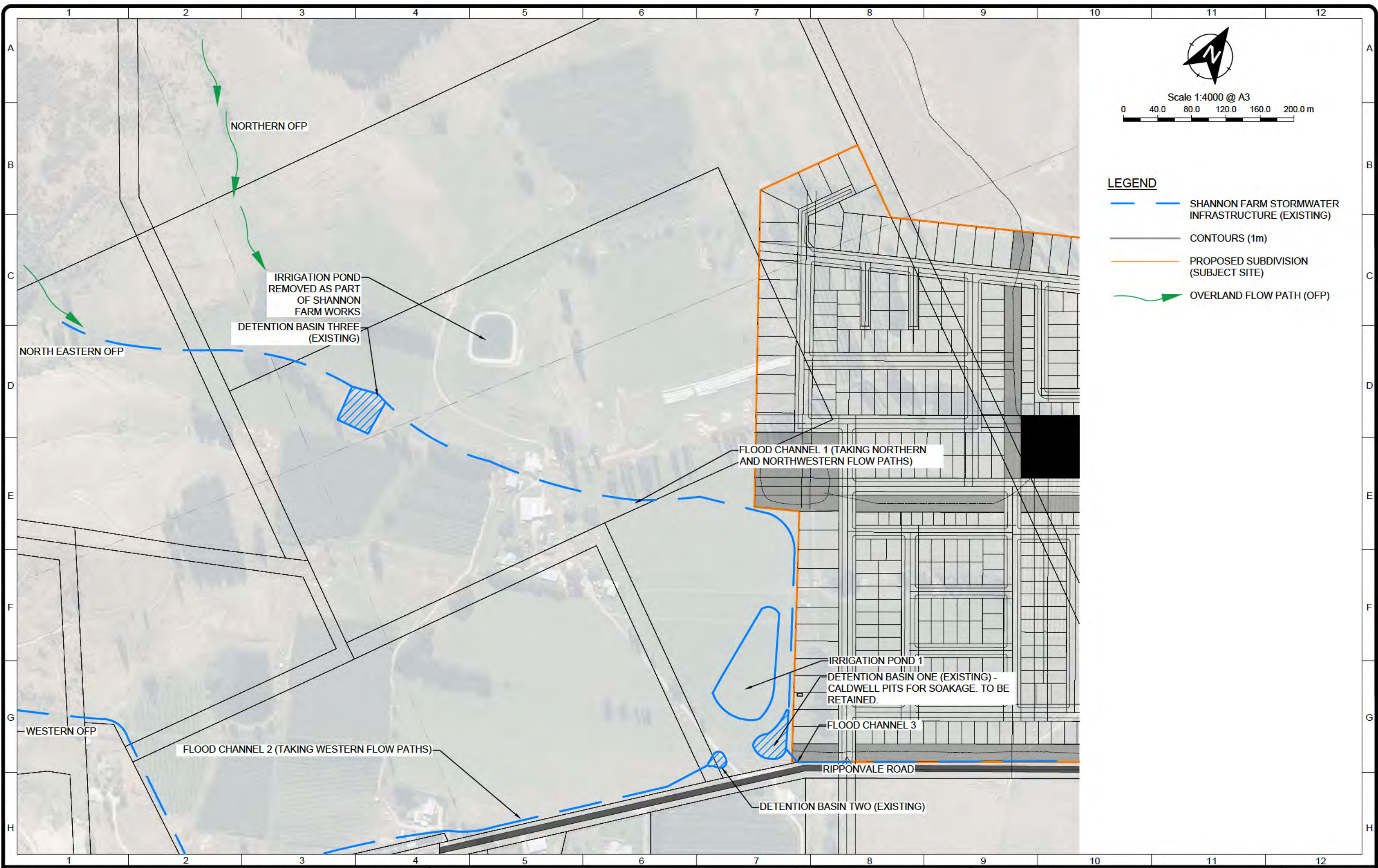
- EXTENT OF CFMA SURVEY
- VIEWPORTS
- APPROXIMATE LOCATION OF OVERLAND FLOW PATHS
- SUBJECT SITE
- SHANNON FARM IRRIGATION SYSTEM (EXISTING)

NOTES:

1. AERIAL PHOTOGRAPH IS INDICATIVE FOR REFERENCE ONLY. ACCURACY VARIES.
2. BASIC CONTOURS ARE FROM LINZ TOPO 1:50K AND IMAGES FROM OTAGO 0.3m RURAL AERIAL PHOTOS 2019-2021 DATA.
3. DETAILED SURVEY DATA WAS PROVIDED BY CFMA ON 19/01/2026 AND INCLUDES THE FINISHED SHANNON FARM SURFACE THAT MAY NOT BE IN PLACE YET.

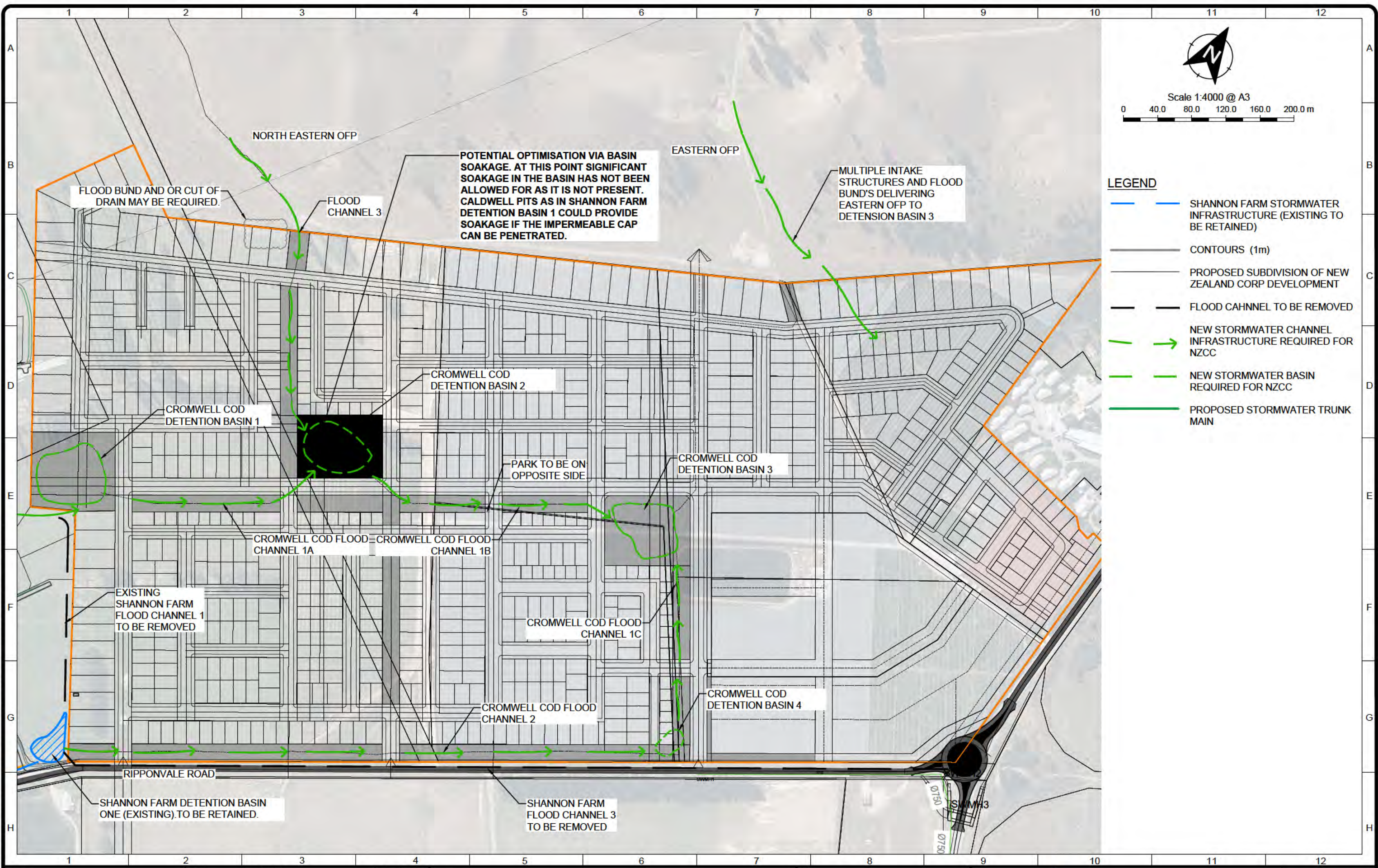
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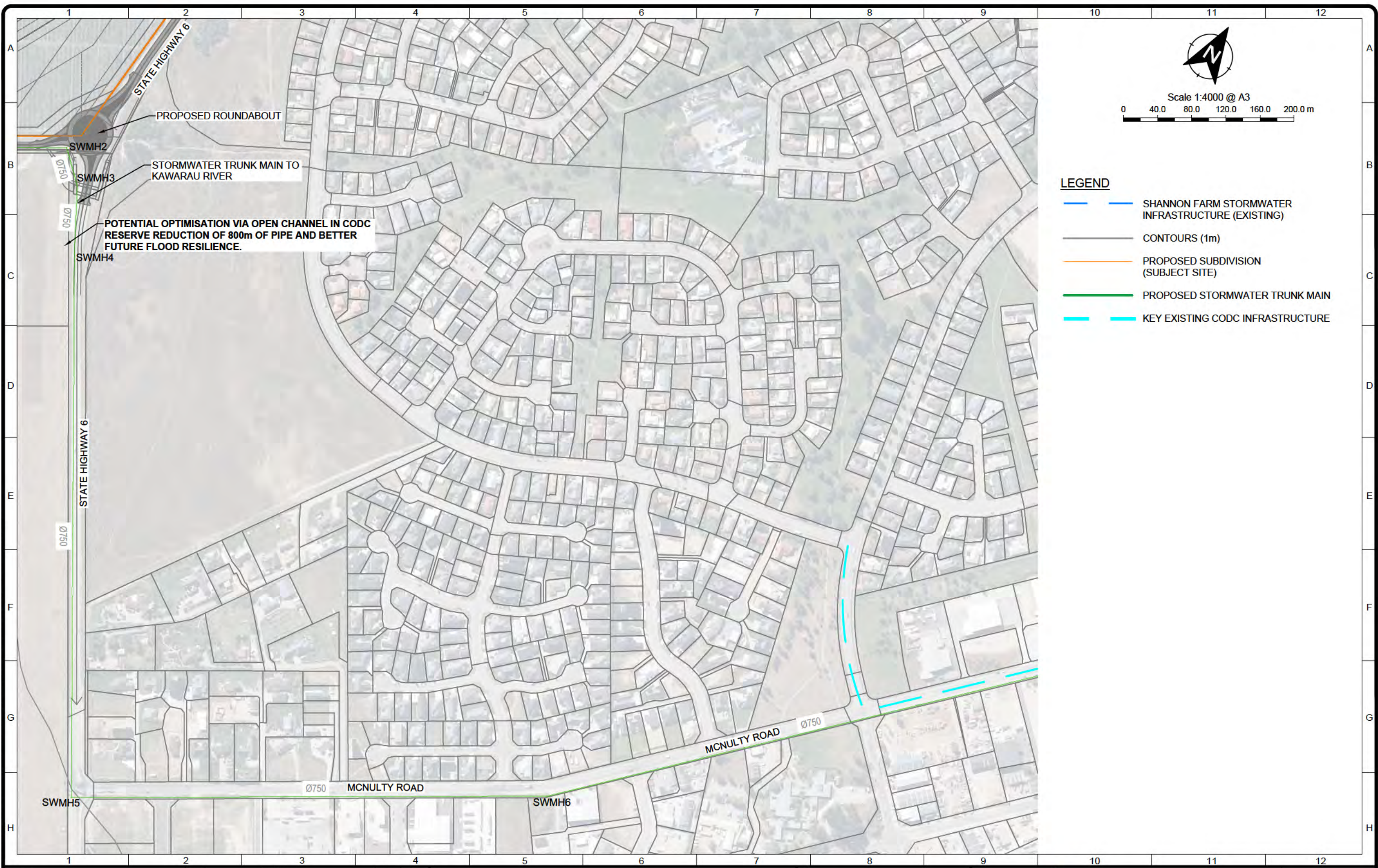
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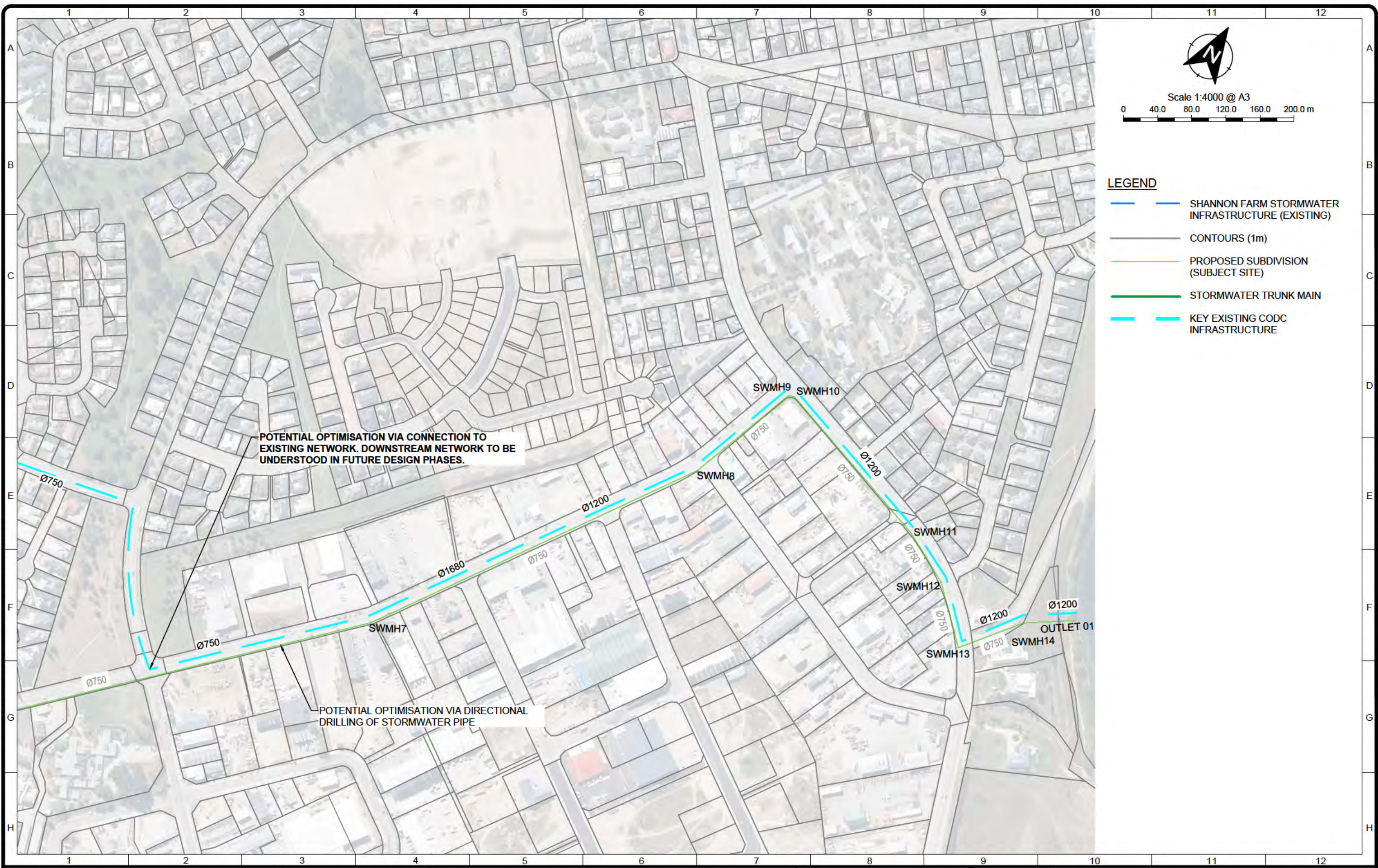


LEGEND

- SHANNON FARM STORMWATER INFRASTRUCTURE (EXISTING)
- CONTOURS (1m)
- PROPOSED SUBDIVISION (SUBJECT SITE)
- PROPOSED STORMWATER TRUNK MAIN
- KEY EXISTING CODC INFRASTRUCTURE

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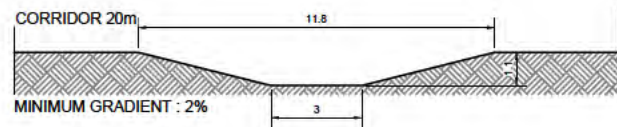
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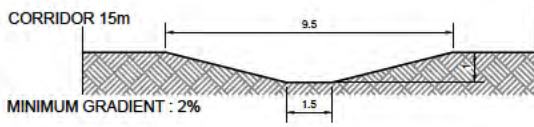
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- CONTOURS (1m)
- PROPOSED SUBDIVISION (SUBJECT SITE)
- STORMWATER TRUNK MAIN
- KEY EXISTING CODC INFRASTRUCTURE

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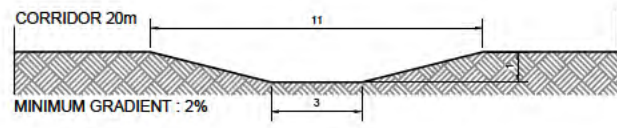
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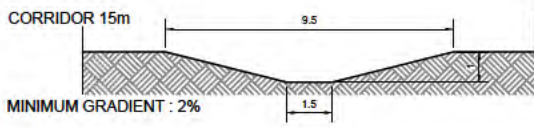
CROMWELL COD FLOOD CHANNEL 1A CROSS SECTION



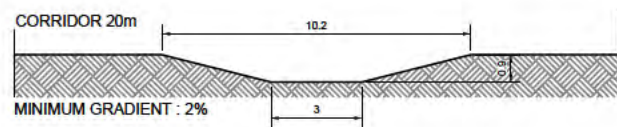
CROMWELL COD FLOOD CHANNEL 2 CROSS SECTION



CROMWELL COD FLOOD CHANNEL 1B CROSS SECTION



CROMWELL COD FLOOD CHANNEL 3 CROSS SECTION



CROMWELL COD FLOOD CHANNEL 1C CROSS SECTION



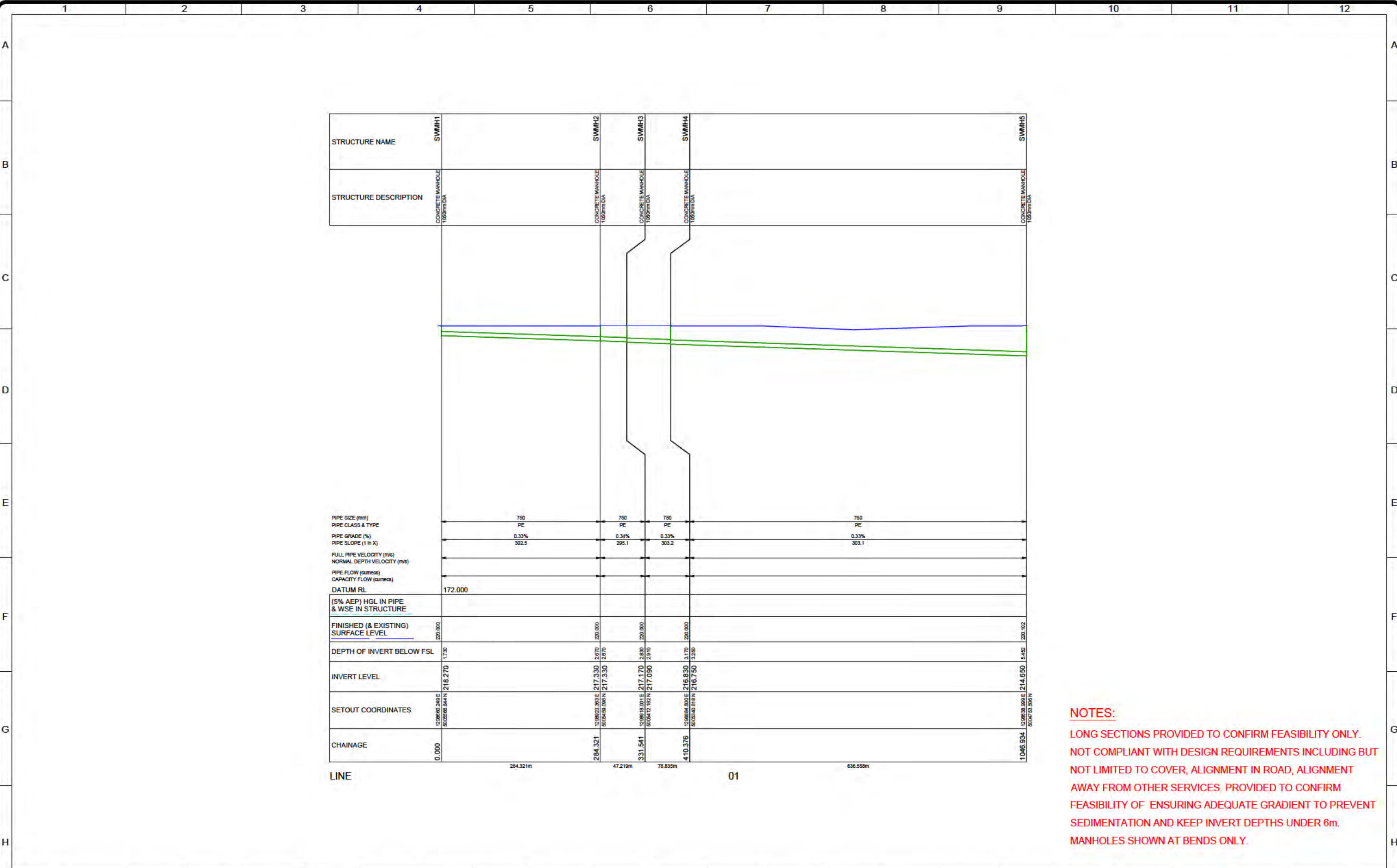
EXAMPLE OF PROPOSED DRY BASIN



EXAMPLE OF PROPOSED FLOOD CHANNELS

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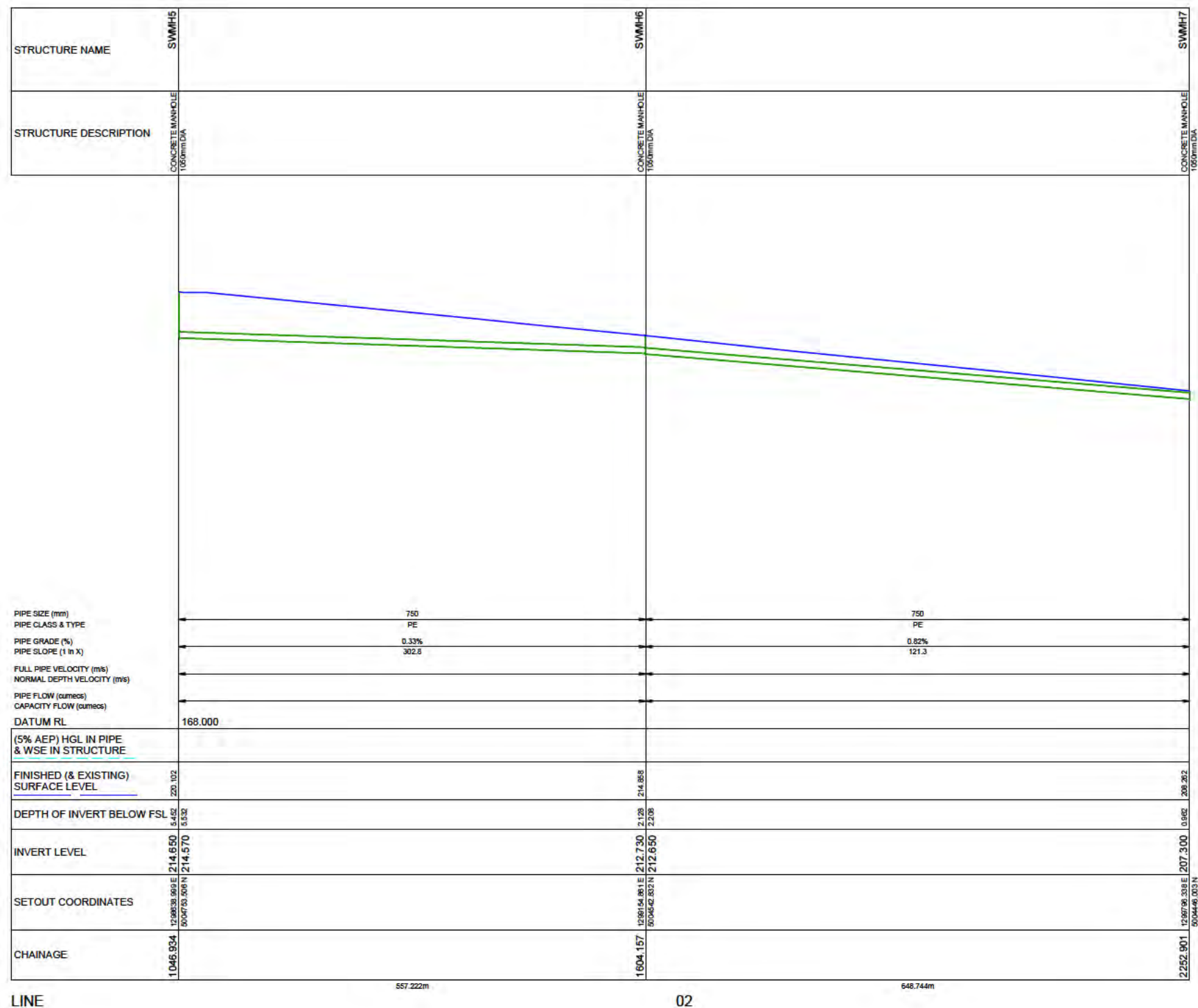
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Project:	CROMWELL PRECINCT
Title:	STORMWATER LONG SECTION PART 1

Client:	RIPPON HOLDINGS LIMITED AND RIPPON INVESTMENTS LIMITED
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Issue	Description	By	Date
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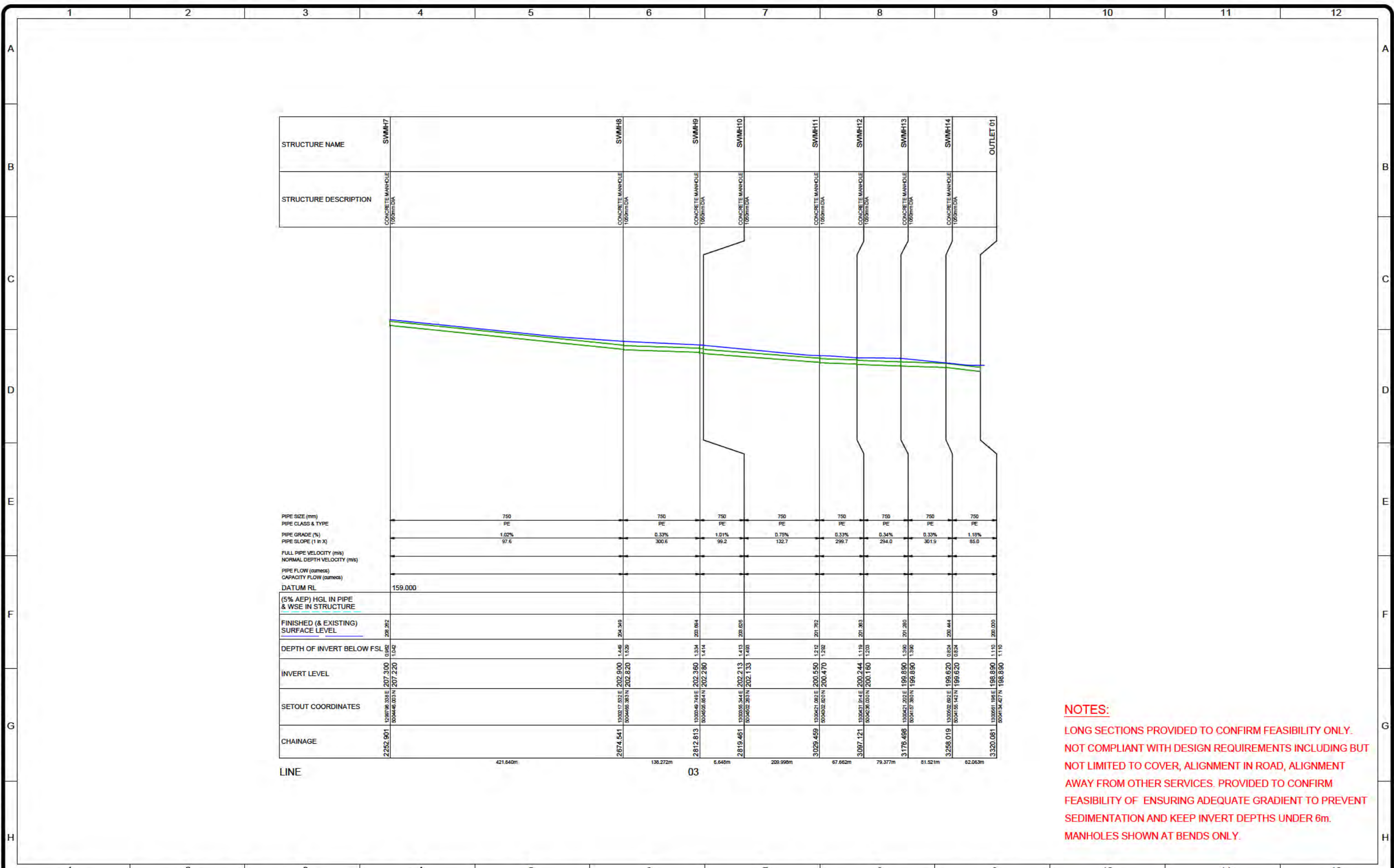
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Client:	RIPPON HOLDINGS LIMITED AND RIPPON INVESTMENTS LIMITED
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					STORMWATER LONG SECTION PART 3				
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Cromwell Cherry Orchard Development Stormwater
Feasibility Memorandum

ATTACHMENT B:

Geosolve Flood Hazard Assessment
for Shannon Farm



GEO**SOLVE**



GEOTECHNICAL



**WATER
RESOURCES**



PAVEMENTS

Flood Hazard Assessment

114 Ripponvale Road,
Cromwell

Report prepared for:

New Zealand Cherry Corp
(Leyser) LP Ltd

Report prepared by:

GeoSolve Limited

Distribution:

New Zealand Cherry Corp
(Leyser) LP Ltd

Town Planning Group (NZ) Ltd

Paterson Pitts Group Ltd

GeoSolve Limited (File)

May 2019

GeoSolve Ref: 180137.01

Revision	Issue Date	Purpose	Author	Reviewed
1	09/10/2018	DRAFT for Client review	DAH	EGM
2	25/10/2018	FINAL	EGM	
3	31/10/2018	Updated following site meeting with Paterson Pitts	DHA	EGM
4	25/03/2019	FINAL for ISSUE	EGM	
4	24/05/2019	Updated with new structure plan	EGM	



GEOTECHNICAL



**WATER
RESOURCES**



PAVEMENTS



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Appendix A. Existing flow paths and areas subject to spill under flood conditions

Appendix B. 20190415_Shannon Farm Structure Plan L1.1 Rev0 dated: 20/05/2019

1 Introduction

1.1 General

This report presents the results of a flood hazard assessment carried out by GeoSolve Ltd to support an application for a private plan change at 144 Ripponvale Road, Cromwell to be made to the Central Otago District Council.



Photo 1 – Property Boundary at 114 Ripponvale Road, Cromwell

The purpose of the report is to evaluate the flooding potential at the site to accompany an application for rural-residential zoning on the land. Our assessment involved:

- A walkover inspection of the site by a senior water resources engineer;
- Evaluation of catchment areas and hydrological parameters;
- Estimation of design rainfall intensities and flood flows;
- Identification of areas potentially affected by flooding; and
- Discussion of possible mitigation options as required.

This report should be read in conjunction with the other GeoSolve reports as necessary.

This work was carried out in accordance with GeoSolve Ltd.'s proposal dated 3 July 2018, ref:180137.01 which outlines the scope of work and conditions of engagement.



1.2 Site and Development

The subject property is located immediately north of Ripponvale Rd to the northwest of central Cromwell – as illustrated in Photo 1.

It is proposed to request an alternative zoning by way of a provide plan change request to the Central Otago District Council which would enable the future development of the site for up to 160 rural-residential sites (subject to further consenting).

As the application is for a zone change request and not a resource consent, no earthworks are proposed at this stage. We expect that cut and engineered fill will be required as part of the earthworks design at the time of subdivision and development should the zone request be successful and subsequent consent applied for.

Topography is gently sloping at the southern end of the site near the footfills and steepens towards Mt Michael to the north and west.

Present land use comprises of a working farm with the flatter land irrigated by border-dykes and spray irrigation. There are a number of orchard blocks that are generally spray irrigated. Most of the irrigation water is supplied by the Ripponvale Irrigation Company Limited race that utilises pumped supply from the Kawarau River. A number of water races are present throughout the property, and these can intercept runoff from land above and divert this to other parts of the property. Such runoff could be irrigation runoff water or overland flow after rainfall.

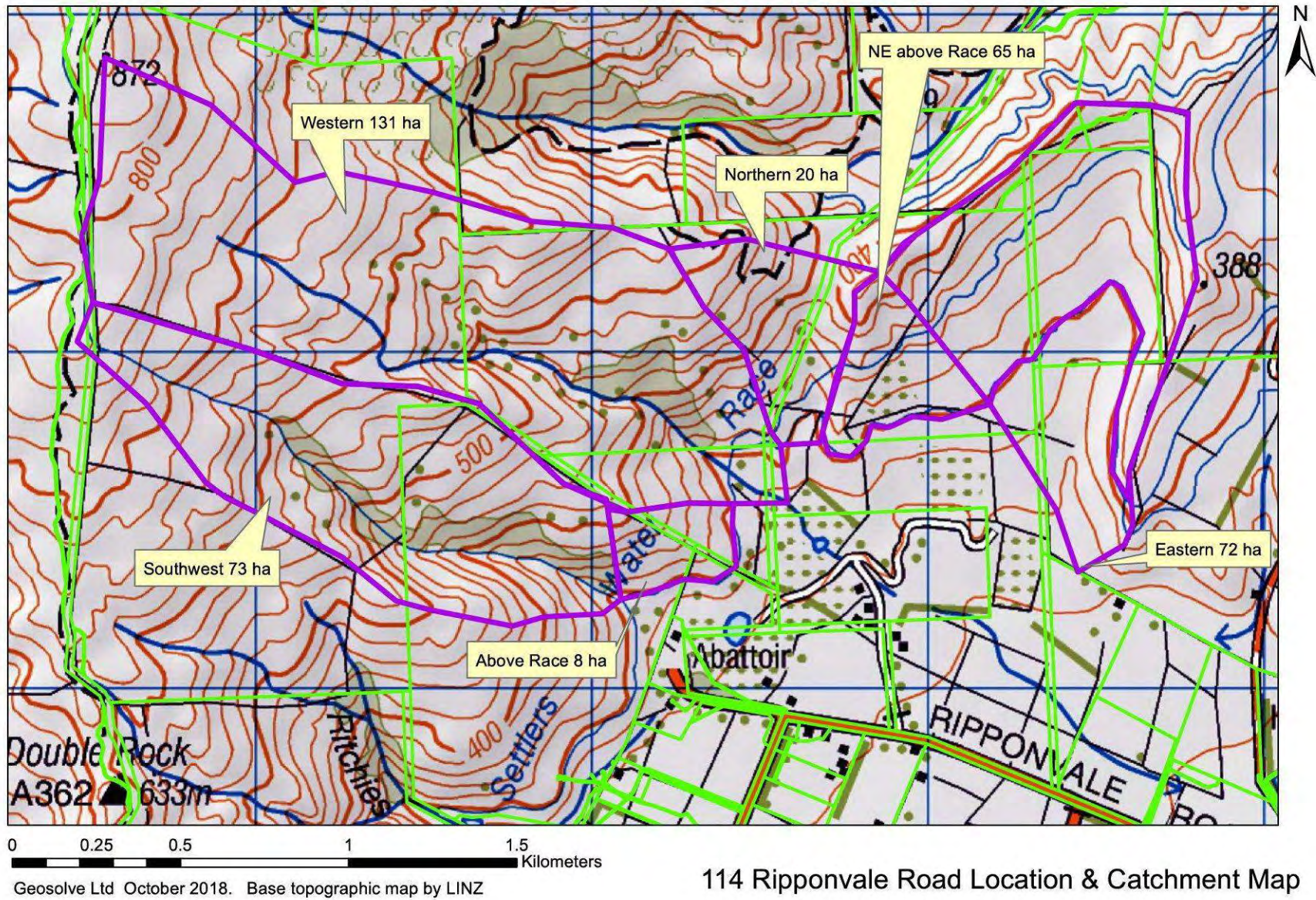


2 Catchments

There are three main catchments identified as West, North and East and one that touches the southwestern margins of the property identified as Southwest. Catchment boundaries, together with smaller internal catchment areas, are shown in Figure 1 attached.

Table 1. Catchment particulars

Catchment	Area (ha)	Length (m)	Max Elevation (mRL)	Outlet Elevation (mRL)	Elevation Difference (m)	Average Slope (%)	Soil Type
Southwest	73	1,850	790	300	490	26	Arrow
Western	131	2,675	872	278	594	22	Arrow + Annan
Northern	20	825	520	290	230	28	Shallow sandy
Eastern	72	1,435	410	250	160	11	Clare & Letts shallow sand-sandy loam
SW above race Abbattoir	8	400	420	295	125	31	Annan mod deep fine sandy loam
NE above race	65	500	420	300	120	24	Clare & Letts shallow sand-sandy loam
Border dyked paddocks per ha	1					5	Annan & Waenga sandy loam



114 Ripponvale Road Location & Catchment Map

Figure 1. Location and catchment map



3 Design Floods

3.1 Previous flood observations

In the 1999 flood, the property owner, who had been on the property since the early 1970s, estimated that he observed [at an unspecified location] a peak flow of approximately 15 heads ($0.4 \text{ m}^3/\text{s}$) down the western gully which is considerably less than estimated in Table 2. We have no way of verifying the accuracy of these observations or whether flows had already reduced by some mechanism upslope, i.e. via overspill or diversion.

It is considered that these observations are anomalously low, considering that peak floods in Central Otago catchments are typically in the order of $1 \text{ m}^3/\text{s}$ per km^2 but are variable. Although the 1999 flood, generally accepted to be the flood of record, was in the order of a 100-year flood in major catchments (owing to snowmelt etc.), this did not necessarily apply in smaller catchments such as on this site. For this event the short-duration, high-intensity floods that will generate large peak runoffs were not particularly high during this event either, and hence greater flows are likely to apply for a design 100-year flood.

Where there are defined hill catchments leading to a valley floor the pattern of drainage and therefore flood flow is fairly self-evident and could be observed on site. Once however, floodwater is captured by water races the exit point, if such races reach full capacity, can vary depending on what state the races have been maintained and managed prior.

Gate structures can be altered, and flows distributed in several directions. Branches, debris, dead animals, etc. can accumulate and block culverts resulting in overspill. If purposefully manipulated however, the flows could be spread out and minimise concentrated flows at any one point (as apparently occurred in the November 1999 floods) – refer photos 3509 and 3511 below.

Floods would have normally been actively managed in this manner under the previous land use, by sharing flood flows between the dam and upslope water races to reduce flooding effects downslope. This is unlikely to be available under the planned zoning and eventual future subdivision. Further, the pattern of flooding will be modified depending on the final road and earthworks layout.

Capacity of the sidling water race heading NE past the main buildings is estimated to be $1.5 \text{ m}^3/\text{s}$ on a much flatter gradient $S=0.0063$, bottom width 0.6m, top width 2.5m and 0.9m deep.

450 mm diameter concrete culverts within the water races will have a capacity of approximately 110-170 l/s (~4 to 6 heads in irrigation terms) depending on slope and may choke or attenuate incoming flows.



Photo 3509: Looking from water race up western gully, dam in middle distance. Flood path to left of road.



Photo 3511: Water race downstream of western gully outflow. 450mm culvert could block and overtop



3.2 Flood Parameters

The NZ Building Code Clause E1.3.2 Surface Water requires that *“Surface water, resulting from an event having a 2% probability of occurring annually, shall not to enter buildings.”* This applies to housing, communal residential and communal non-residential buildings.

NZS 4404:2010: Land Development and Subdivision Engineering clause 4.3.5.2 states:

“4.3.5.2 Freeboard The minimum freeboard height additional to the computed top water flood level of the 1% AEP design storm should be as follows or as specified in the district or regional plan:

	<i>Minimum height Freeboard</i>
<i>Habitable dwellings (including attached garages)</i>	<i>0.5 m</i>
<i>Commercial and industrial buildings</i>	<i>0.3 m</i>
<i>Non-habitable residential buildings and detached garages</i>	<i>0.2 m</i>

The minimum freeboard shall be measured from the top water level to the building platform level or the underside of the floor joists or underside of the floor slab, whichever is applicable.”

The Standard also states in Table 4.1 that the recommended annual exceedance probability (AEP) for primary systems where no secondary flow path is available should be the 1% AEP flood event.

For the purposes of this report the 1% AEP event has been used. An allowance for climate change has also been provided based on an 8% increase in high intensity rainfall per degree Celsius rise in predicted mean annual temperature.

3.3 High Intensity Rainfall

The National Institute of Water and Atmospheric Research (NIWA) provides information through their web application High Intensity Rainfall Design System (HIRDS) v3¹. The rainfall Depth-Duration-Frequency information has been obtained for the climate change scenario of a 2 degree rise in mean annual temperature.

The rainfall for durations of up to one hour are plotted for the 1% annual exceedance probability (AEP) event. This is shown as Figure 2 below.

¹ <https://hirds.niwa.co.nz/>

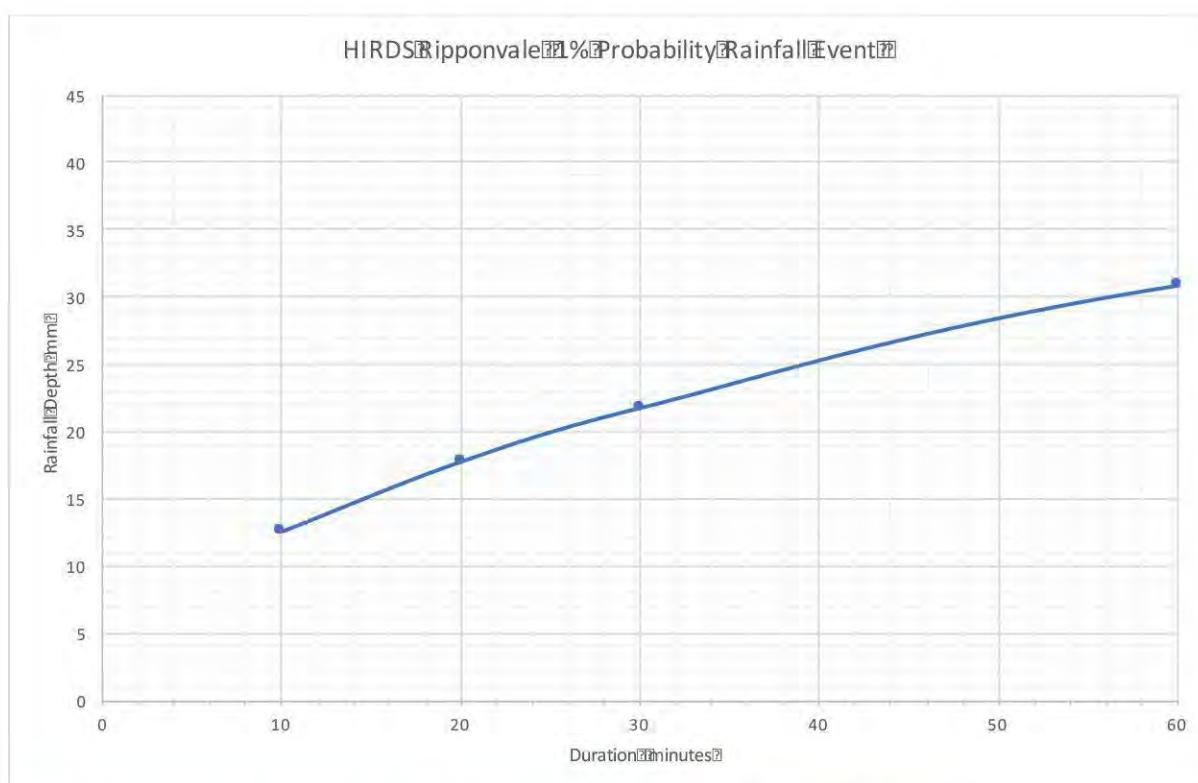


Figure 2. High Intensity Rainfall data for 1% AEP event. Source: <https://hirds.niwa.co.nz/>

3.4 Design Flood Flows

One method to estimate flood flows for a design storm is the Rational Method which is based on a runoff coefficient C , time of concentration and rainfall intensity. The time of concentration represents the time it takes for the entire catchment area to be full contributing to the flood flow and is estimated using catchment characteristics such as surface cover, soil types, length of catchment and slope.

A second method available is using Stream Explorer² software from NIWA and the Pearson Method (1991). This has derived specific discharge figures of 1.4 to 2.4 m³/s/km² that are generally lower than the Rational method even though they have also been adjusted for the 2 degree climate change scenario. The higher Pearson method figure has been used. For the purposes of this report the two methods estimates have been averaged as shown in the last column of Table 2.

Table 2 shows the derivation of the flood flows for the catchments in Table 1.

A check has been made using the most up-to-date Henderson & Collins (2018) method developed by NIWA, accessible through the web-based application on the NIWA website². Results indicate the adopted flood estimates are conservative, which is appropriate in this context.

²New Zealand River Flood Statistics, based on the Henderson & Collins 2018 Method:

<https://niwa.maps.arcgis.com/apps/webappviewer/index.html?id=933e8f24fe9140f99dfb57173087f27d>



Table 2. Flood estimates for 1% AEP flood including allowances for climate change

Catchment	Area (ha)	Rational Method including 2° climate change					Peason (1991) Method Q m³/s	Average Q m³/s	Check ¹
		Runoff Coefficient c	Time of concentration Tc minutes	Rainfall Depth (mm)	Rainfall intensity i mm/hr	Flood Flow Q m³/s			
Southwest	73	0.4	27	20.8	47	3.81	2.03	2.92	0.78
Western	131	0.4	35	23.5	40	5.85	3.65	4.74	1.12
Northern	20	0.4	19	17.2	56	1.23	0.56	0.90	
Eastern	72	0.35	27	20.8	46	3.22	2.00	2.61	0.73
SW above race Abbattoir	8	0.4	15	14.8	61	0.54	0.22	0.38	
NE above race	65	0.4	16	15.8	58	4.16	1.81	2.98	
Border dyked paddocks per ha	1	0.3	13	13	59	0.05	0.03	0.04	
¹ Check based on the web-based "Regional Flood Estimation Tool" developed by Henderson & Collins (2018) correlating relative catchment areas $A_1/A_2^{0.8}$ and including a 25% factor for climate change https://niwa.maps.arcgis.com/apps/webappviewer/index.html?id=933e8f24fe9140f99dfb57173087f27d									

4 Areas Potentially Affected by Flooding

4.1 General

We have reviewed the proposed Structure Plan (ref: 201902415_Shannon Farm Structure Plan L1.1 Rev0 dated: 20/05/2019 – reproduced in Appendix B). This shows an “Indicative Open Space and Stormwater Corridor” that provides for a 20 m width that will allow a 3 m bottom width channel, 0.6 m deep with 2:1 batters plus 7 m berm on either side. This is a no-build zone, as shown on the plan. Culverts on this main channel should be designed to accommodate 5.6 m³/s (Western plus Northern catchments) so should be a nominal 1600 mm to 1800 mm diameter pipe, depending on depth of cover. The northern catchment flood channel zone is also shown as 20 m, but the formed channel can be a nominal 1 m bottom width and 0.5 m deep grassed swale, also nominated a no-build zone.

There is an existing formed channel along the South-eastern boundary that directs flows along the boundary towards Ripponvale Road (refer Image 2130). The proposed flood channel zone will discharge into this channel so that outflow locations from the property will remain the same.

Appendix A presents the existing flows paths and areas potentially subject to inundation. As part of the development it is proposed to clear out water race structures including culverts.



Image 2130: Existing channel about 1.5 m bottom width 0.6 m deep along SE boundary looking upstream from Ripponvale Road (taken: 30 October 2018)

In general, it will be possible to locate future building sites well away from and above active watercourses. Accordingly, most future lots will only require a minimum level of flood protection to deal with localised shallow runoff, such as establishing minimum floor levels above surrounding ground as outlined in Section 3.2. The CODC minimum floor level is 300 mm above natural ground, but in some cases, this will need to be greater.

All building sites should be located away from flood flow paths.

Those building sites that are on slopes requiring excavation (cut and fill) should incorporate a stormwater catch drain placed at the base of the cut slope.

As previously noted, the old water races around the hill slopes can intercept runoff from the slopes above and redirect those flows. It is recommended that lowered sections of the downstream race bank be formed and grassed at larger watercourses so that if the old races become full they can overflow at known and manageable locations. If the building sites have been sited away from flood flow paths, then this will reduce the chances of them being affected by breakout at unexpected locations.

Shallow, grassed swales and roadside stormwater drains are typically also included in rural type subdivisions to deal with shallow runoff and retaining the system of existing border dykes could help in this regard. Roadside culverts and drainage infrastructure should be designed to accommodate any upstream catchment which is normal.

Confirmation of final earthworks plan and building locations could be assisted by the development of a hydraulic model to ensure that conveyance systems and minimum flood levels are appropriately set.

4.2 Specific Issue – Downstream of dams

Three small water storage dams are present on the site as illustrated on Figure 3:



Figure 3. Location of existing small dams circled in red



These dams could present a flood hazard to land immediately downstream if these were to fail, either through failure during normal operation say via piping or during an extreme weather event. This flood hazard would be additional to the hazard from stormwater runoff discussed previously, although the incremental increase in hazard may only be small.

If retained, these could also have a positive effect by storing and gradually releasing floodwater, reducing the impacts of flooding downstream, i.e. as stormwater retention basins. Emergency spillway capacities appear to be in excess of their respective upstream channel capacities, meaning that channels upslope are likely to overflow away from the dams in a large flood rather than enter the dam. Further engineering assessment can be undertaken at a later date to ensure that the dams comply with current standards if they were to perform as stormwater retention basins.

The largest of the dams is located on the Southwestern catchment and is fed by a Ripponvale Irrigation Company water race as well as the natural catchment – see bottom left on Figure 3. It has an estimated surface area of $\sim 6,000 \text{ m}^2$, is up to about 3.5 m deep and has a storage potential of about $8,000 \text{ m}^3$. On the basis of these estimates, it would note be classified as a 'Large Dam' in the Building Act.

Future building sites immediately downstream may require specific engineering works to manage flood risk of these dams. Conceptually, this this may involve a small flood diversion swale and bund upstream of building sites and/or a suitable minimum flood level established above the potential floodplain level to accommodate shallow flows arising from s potential dam failure. Final solutions should be subject to detailed design once development plans are confirmed.



5 Conclusions

Three main catchments totalling 370 ha were identified at the site. Flood flows from these catchments were estimated for a 1% AEP flood event using the Pearson (1991) method and Rational Method, with a check using the new Henderson & Collins (2018) method.

The proposed Structure Plan allows for a stormwater corridor capturing the western and northern catchments that will be directed into an existing formed channel along the South-eastern boundary to maintain existing outlet locations from the property. Culverts on the main channel should be designed to accommodate 5.6 m³/s. Roadside infrastructure should be designed to accommodate any upstream catchment as normal.

Generally, it will be possible to locate future building sites well away from and above active watercourses, meaning that minimum floor levels and shallow stormwater swales are likely to be all that is required in most areas.

The three existing small water storage dams present an additional flood hazard to land immediately downstream, although the incremental increase in hazard may only be small. If retained as stormwater retention ponds, these could be modified to have a positive effect by reducing the impacts of flooding downstream. Further engineering assessment is recommended at a later date to confirm their suitability for reuse in this way, and to inform any requirements for managing flood risk downstream.



6 Applicability

This report has been prepared for the benefit of New Zealand Cherry Corp (Leyser) LP Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

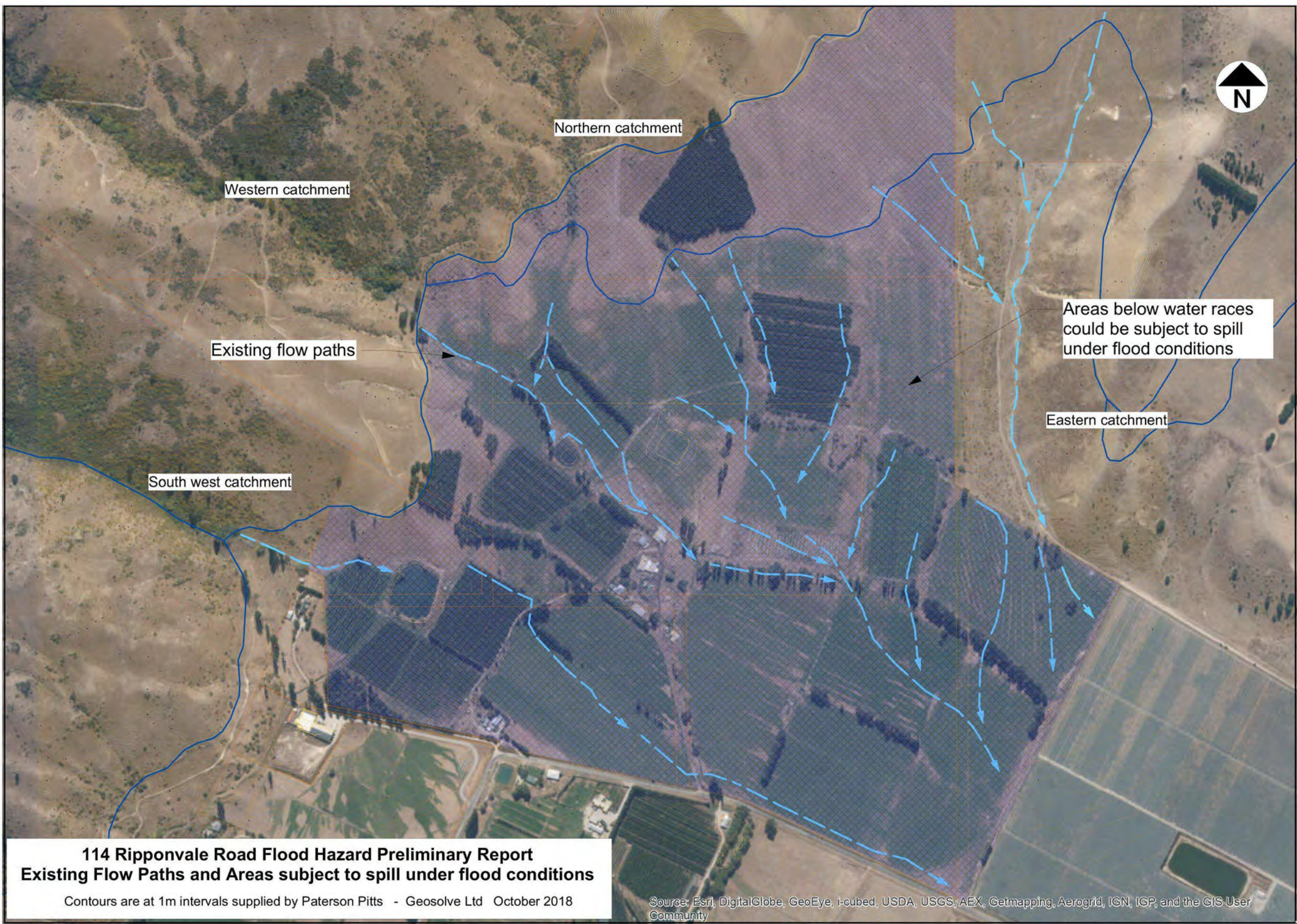
It is important that we be contacted if there is any variation in subsoil conditions from those described in this report.

Report prepared by:

.....
David Hamilton
Senior Water Resources Engineer

Reviewed for GeoSolve Ltd by:

.....
Eli Maynard
Geotechnical & Water Resources Engineer



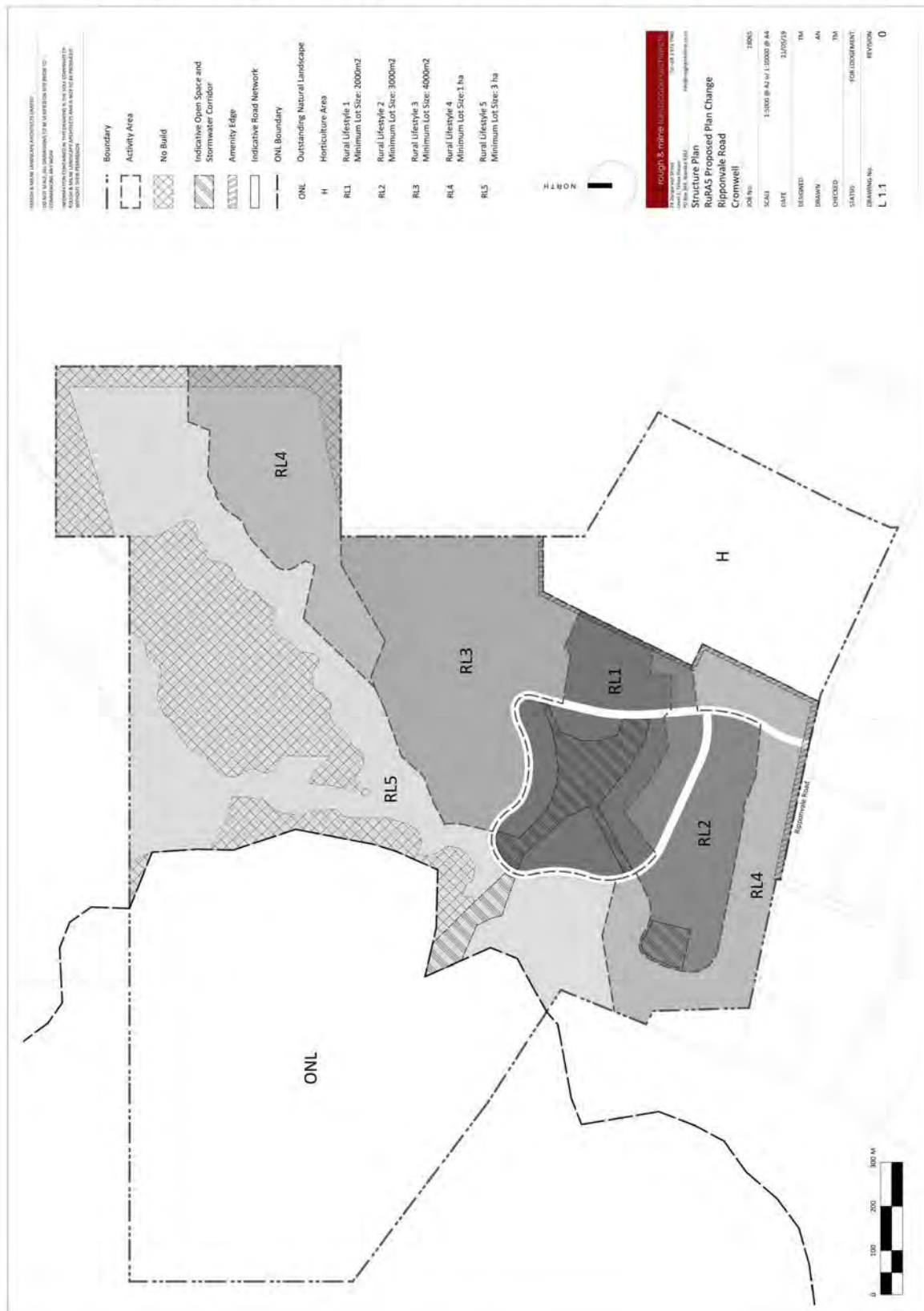
114 Ripponvale Road Flood Hazard Preliminary Report
Existing Flow Paths and Areas subject to spill under flood conditions

Contours are at 1m intervals supplied by Paterson Pitts - Geosolve Ltd October 2018

Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, and the GIS User Community

SCHEDULE 19.23 : STRUCTURE PLAN – RURAL RESOURCE AREA (5)

See Rules 4.7.2(ib), 4.7.2(ii)(a)(i), 4.7.2(ii)(a)(vi), 4.7.2(ii)(a)(vii), 4.7.5(viii), 4.7.5(ix), 4.7.6A(f), 4.7.6A(l), 4.7.6A(m), 4.7.6A(n), 4.7.6C(f)



Cromwell Cherry Orchard Development Stormwater
Feasibility Memorandum

ATTACHMENT C:

Clark Fortune McDonald Shannon
Farm Stormwater Report

STORMWATER MODELLING REPORT

SHANNON FARM DEVELOPMENT

FOR CONSENT
JUNE 2023



CLARK FORTUNE MCDONALD

Document register

DOCUMENT NAME	SHANNON FARM DEVELOPMENT - Stormwater modelling report		
PREPARED BY	Roy Buchanan		
REVIEWED BY	Neil Williman (Geosolve)		
APPROVED FOR ISSUE BY	Chris Hansen	DATE:	08.06.23

REVISION HISTORY		
-	24.04.23	DRAFT Issued for review
-	08.06.23	Consent issue

It is the responsibility of the designer and the project manager to ensure that they are referring to the latest revision of the Engineering Acceptance Report

This report has been prepared for the benefit of NZ CHERRY CORP (LEYSER) LP LTD. No liability is accepted by this company or any employee or sub-consultant of this company with respect to its use by any other person.

This disclaimer shall apply notwithstanding that the report may be made available to Central Otago District Council & Otago Regional Council and other persons for an application for permission or approval or to fulfil a legal requirement.



309 Lower Shotover Road - P.O Box 553 - Queenstown - 9348
T: (03) 441 6044 F: (03) 442 1066 E: admin@cfma.co.nz



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1 INTRODUCTION

1.1 EXECUTIVE SUMMARY

It is recognized that the proposed Shannon farm development will alter the existing storm water flow characteristics. Unmitigated, it is likely that peak runoff outflows will be increased.

These concerns have been considered along with managing potential flood risk and will be mitigated by the provision of stormwater channels & detention / soakage devices to ensure that the peak post development outflow is less than it would be in the predevelopment scenario.

Proposed channels will also convey flood flows through the development without risk to proposed allotments or adjoining property.

1.2 OVERVIEW

Clark Fortune McDonald has been engaged by NZ CHERRY CORP (LEYSER) LP LTD along with other specialist consultants to provide a design solution for managing stormwater for the Shannon Farm Development) located on Ripponvale Road, Cromwell.

The proposed stormwater system will mitigate the negative effects of development within the zone, resulting in a reduction in peak discharge and management of flood flows through the site.

The design is to provide an upfront solution to all stormwater management for the entire developed site rather than dealing with stormwater incrementally as the development progresses.

1.3 PURPOSE OF THIS REPORT

This report is to accompany the application submitted to the Otago Regional Council for resource Consent approval in relation to the Shannon Farm Development

2 BACKGROUND

2.1 APPROVED ZONE

The Shannon Farm development was approved by CODC plan change 14 which allows for the creation of up to 160 rural allotments ranging from 1500m² – 8000m²+ in size.

The zone also requires areas of productive soils to be protected & utilised.

The approved structure plan approved by plan change 14 is shown below:

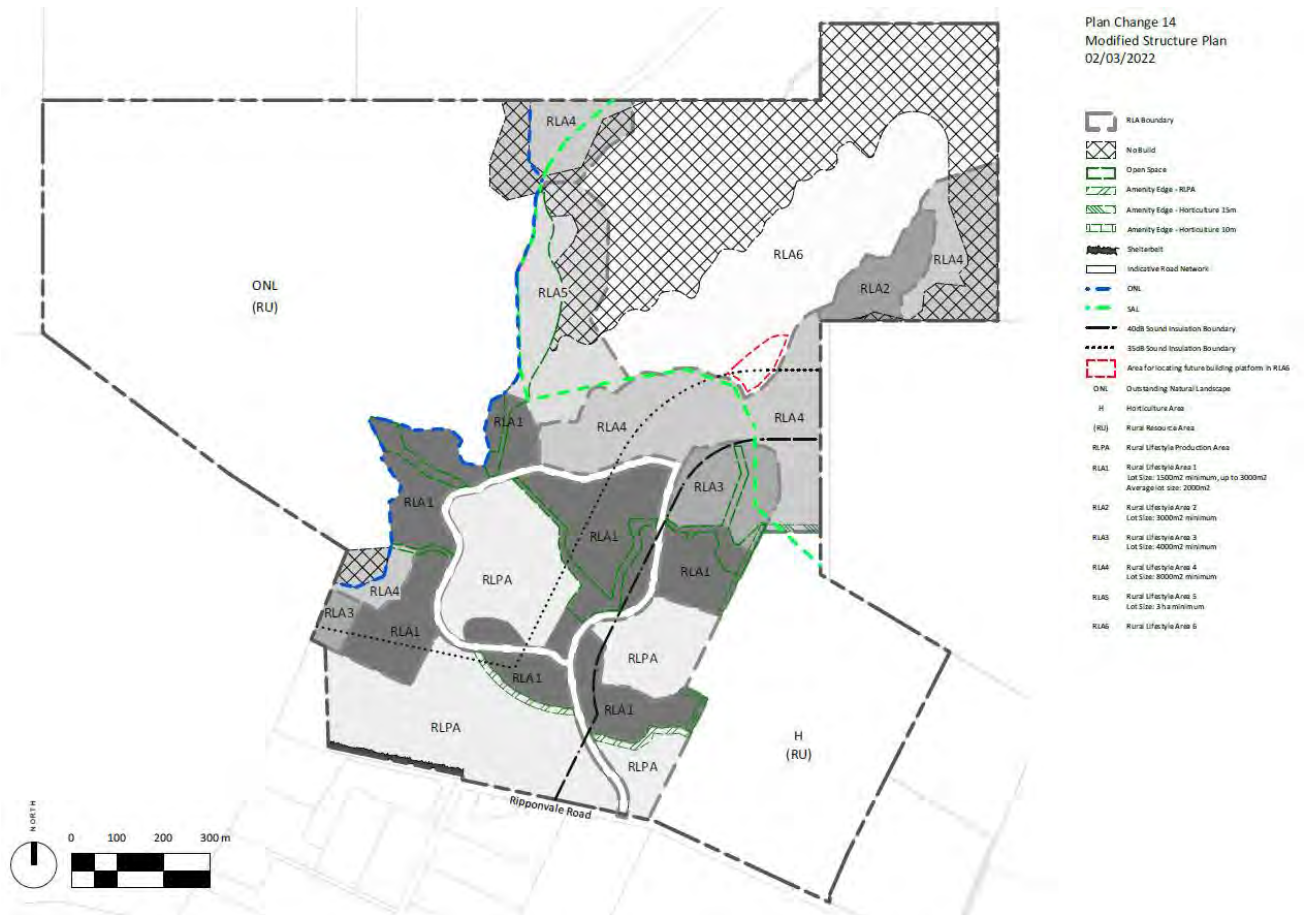


Fig 1 – PC14 structure plan

2.2 EXISTING CONSENTING & DEVELOPMENT

Following the approval of PC14, a resource consent application has been submitted to the Central Otago District Council for a 160-lot staged development.

Activity area H has been planted in Cherries as was a requirement of the plan change provisions. The remainder of the site remains an active farm.

3 EXISTING SITE

3.1 SITE LOCATION

Shannon Farm is located to the Northwest of Cromwell township with access off Ripponvale Road.

The site sits at the foot of the Pisa Range and is located on an Alluvial fan with defined gullies extending up into the upper reaches of the site. The site is bound by farmland & cherry orchards.

The NZ Cherry Corp orchard that sits downstream of the site is owned by the developer and is the downstream land directly affected by the development site.



Fig 2. Site Location

3.2 SITE TOPOGRAPHY & GEOLOGY

The proposed development is located at the foot of the Pisa Range on an inactive alluvial fan formation with allotments extending up the lower slopes of the Pisa Range and surrounding foothills.

The fan is inclined gently towards the southeast at around 3 – 5%.

Four large gullies discharge into the development area as identified in existing Geosolve reporting undertaken for the plan change approval. These have been identified as Western, North-western, Northern & Eastern as marked below.

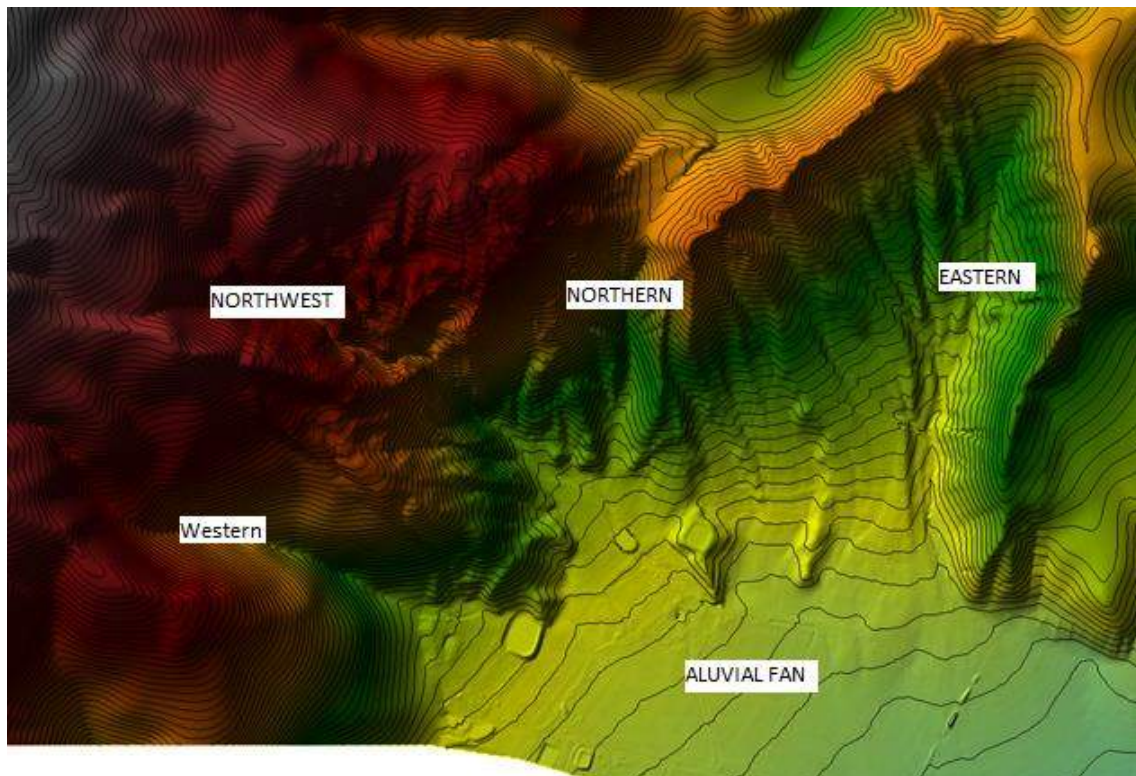


Fig 3. Existing site digital elevation model & identified gullies. 5m contours shown.

The site topography has been extensively modified by farming operations. The natural flow paths are modified by a network of irrigation races & border dyke paddocks, as well as historic mining races, many of which are now in a state of disrepair.

Three irrigation dams are also present onsite fed by irrigation water and diverted flow from the gullies.

The geology of the site is mostly alluvial outwash gravels & sediments. Soakage testing undertaken for the plan change identified good soakage conditions with soakage rates up to 2000mm/hr observed.

Please refer to Geosolve reporting for more information on site geology and soil structure.

3.3 ALUVIAL FAN HAZARD

The ORC hazards database identifies an alluvial fan hazard within the boundaries of the site. Geosolve have previously investigated the identified fan risk and an extract from their existing reporting is provided below:

“As indicated in Figure 4 below, the regional mapping indicates that there are ‘active debris-dominated’ and ‘active floodwater-dominated’ alluvial fans within the property bounds. These are confined to the lower section of the property, where slopes are largely sub-horizontal with areas towards the far eastern bounds gently inclined”.

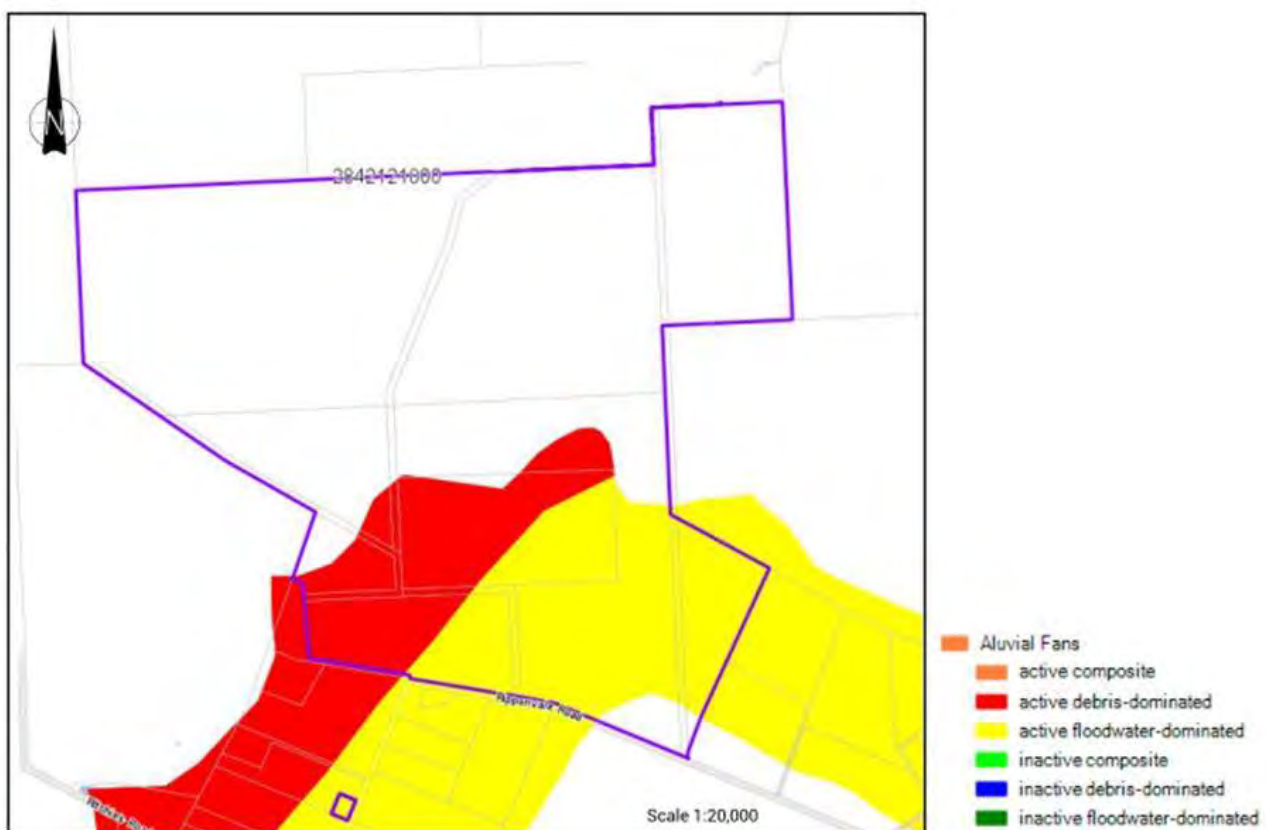


Figure 4 – Regional Alluvial Fan Mapping. Source: <http://hazards.orc.govt.nz>. Alluvial Fans – Fan Landform

“Higher resolution (1:10,000 scale) supplementary fan mapping from the ORC database as seen in Figure 5 below, shows a number of fan landforms within the boundaries of the property. The northern extent is mapped as ‘catchment gully erosion’ in two gullies, identified in field mapping; with the area surrounding as ‘catchment unspecified’, a default term for catchment areas. The eastern gully is mapped as a ‘fan less

recently active’ which is described as areas with stream activity >300 years. The southern extent is mapped as ‘fan undifferentiated’, this is areas that were not possible to differentiate as part of the study and therefore set to this as a default. A small section on the south-eastern boundary is mapped as ‘fan recently active’ in which there is relatively recent (<300 years) stream activity”.

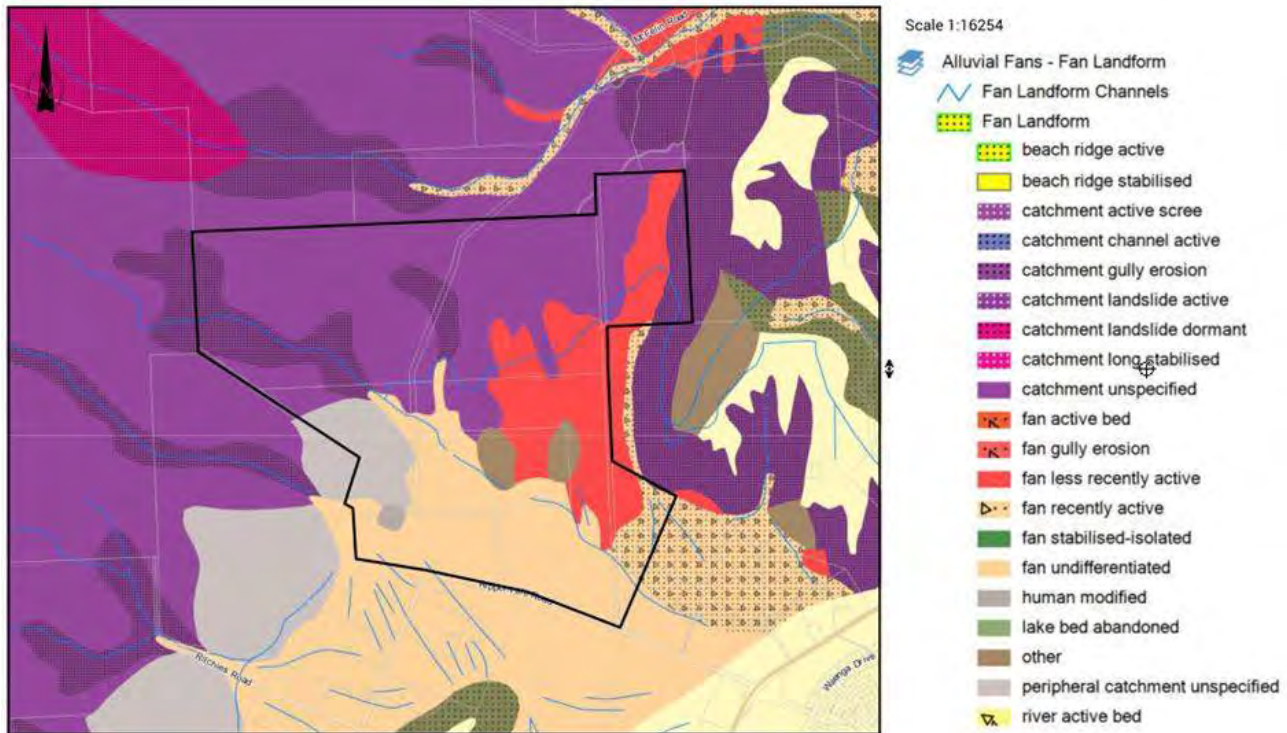


Figure 5 - Alluvial Fan- Fan Landform. Source: <http://hazards.orc.govt.nz>.

“No evidence of recent alluvial fan activity was identified during the desktop review or field inspections. Topsoil layers observed in test pits at the fan apexes range from 300-500 mm thick, and there is no evidence of surface gravel deposits indicating an extended period of quiescence. Fan surfaces are consistently smooth and there is no geomorphic evidence to suggest debris flow deposits are present, furthermore little evidence was observed in test pits. The numerous ephemeral gullies mapped within the subdivision are only expected to pose a shallow sheet flooding risk during heavy rainfall”.

The Geosolve investigations concluded that there was no recent alluvial fan activity and rebuts the active designation in the ORC GIS. The risk of future debris flows, or debris floods is considered to be generally low.

Additional freeboard is provided in the proposed stormwater channels to allow for debris originating from upstream catchments but debris flow originating from the post development fan surface is unlikely.

3.4 EXISTING CATCHMENTS

A full analysis of the existing site catchments has been undertaken to determine predevelopment flows.

As above, the site can be broken down into four primary gully catchments which all discharge out over the fan formation within the site. The flows are initially channelised, but flow is dispersed over the fan area after exiting the gullies. Catchment extent and flow paths from the HEC RAS modelling shown below:

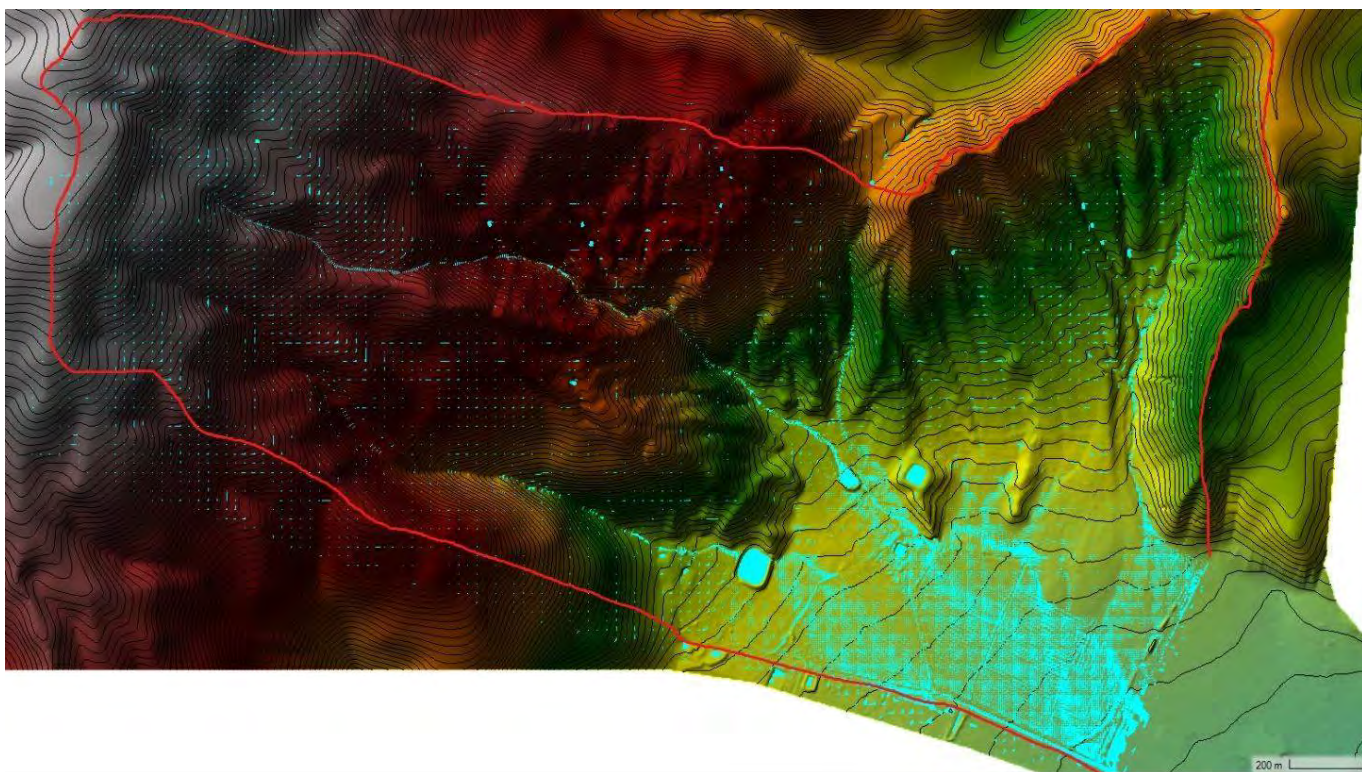


Fig 6a – Pre development catchment flow area with 100-year ARI predevelopment flow paths.

4 PROPOSED STORMWATER SYSTEM

4.1 OVERVIEW

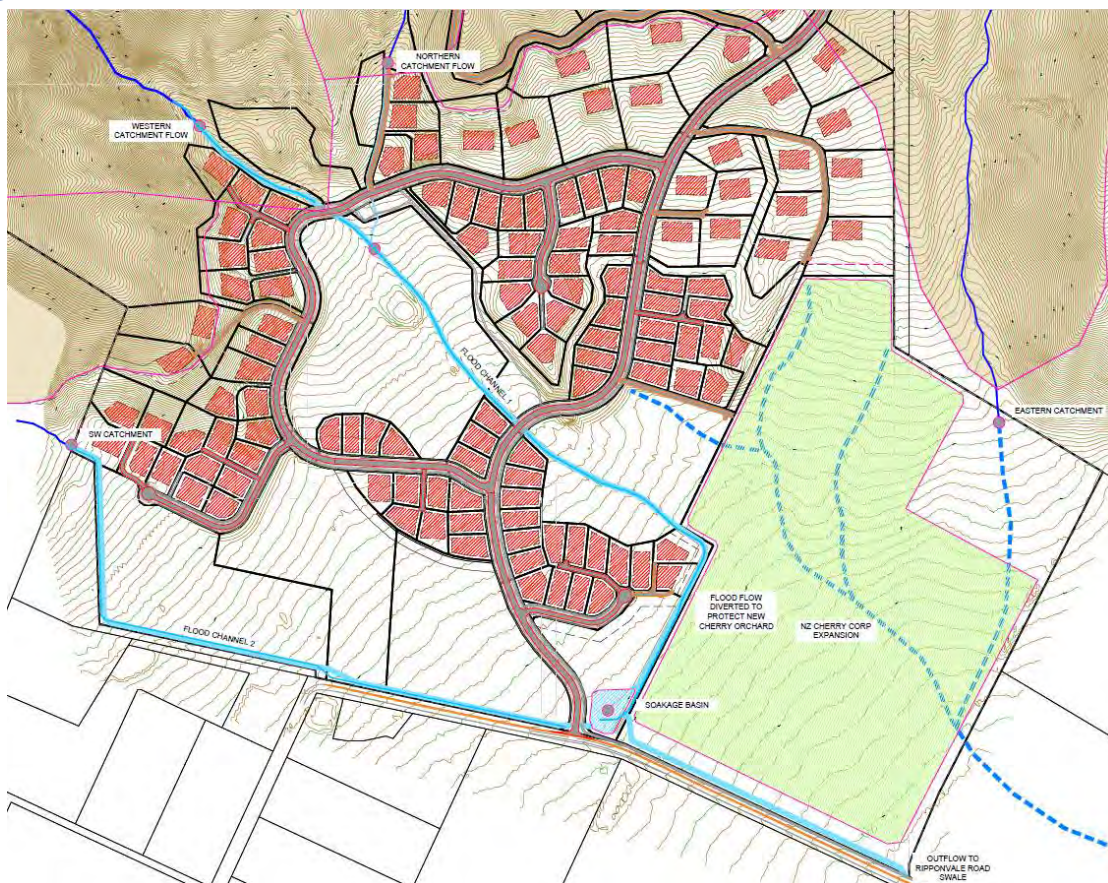


Fig 6b – Proposed system overview

The proposed stormwater system will capture and channelise flood flows through the development via three new channels as shown above.

The channel network is intended to convey predevelopment flood flows through the development. Runoff from impervious areas within the site will be captured in a separate system and discharged to ground. Only surplus flow from the proposed soakage system if any will discharge to the swale network.

The channels will discharge to a soakage / detention basin located in the South-eastern corner of the development site before flowing out into the Ripponvale Road swale in the same location as the predevelopment flow.

By concentrating storm flows in a channel, the peak downstream flow rate is increased as the storm flow is no longer spread out over a large surface area. Time of concentration and catchment storage are reduced as a result.

The proposed basin will mitigate this increase in flow rate. As a result, peak flows existing the site to the east will be reduced to less than pre development peak flow rates.



Fig 6c – Post development catchment flows with 100-year ARI post development flow paths.

4.2 SYSTEM COMPONENTS

4.2.1 STORM CHANNEL NETWORK

Storm flows will be conveyed via three new open channels. These channels have been designed to maintain a minimum of 500mm freeboard in a 100 year ARI storm event. Additional width & depth is provided through selected sections where mitigation of Aggradation risk is required.

4.2.1.1 CHANNEL 1 & 3

Channel 1 conveys existing storm flows from the northern & northwest catchments through the centre of the site to the proposed detention basin. The proposed channel diverts flood flows away from the down stream cherry orchard as well as protecting the proposed residential allotments.

Channel 3 conveys water from the soakage basin down to the south east corner of the site where the pre development flows exit the site at the lowest site elevation. This channel discharges out into the Ripponvale Road swale which ultimately discharges out over the paddocks adjoining the Cromwell Racecourse.

Channels 1 & 3 have a base width of 3m and has a minimum depth of 1000mm with side slopes of 1 in 3.

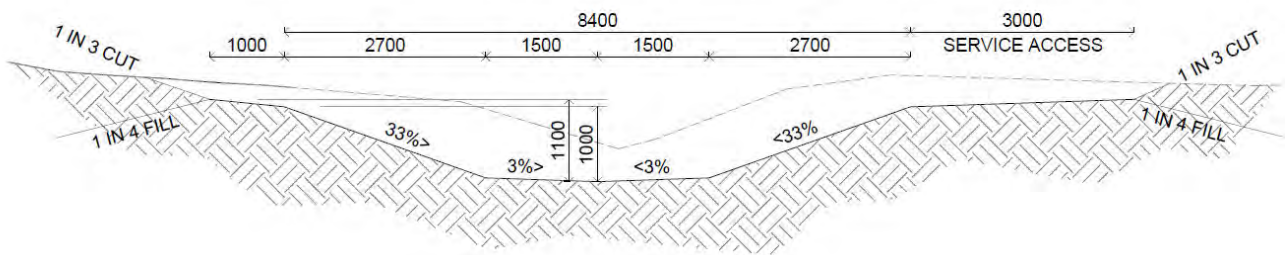


Fig 7 – Channel 1 & 3 profile.

4.2.1.2 CHANNEL 2

Channel 2 conveys flood flows from the Western Gully catchment along the southern boundary of the site adjoining Ripponvale Road to the proposed basin. The channel will also pick up flows from the proposed cherry blocks adjoining. Channel 2 has a base width of 2m and has a minimum depth of 1000mm with side slopes of 1 in 3.

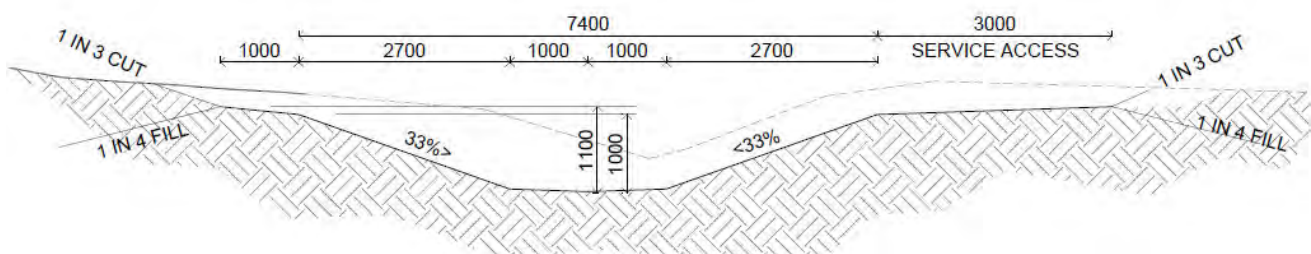


Fig 8 – Channel 2 profile.

4.2.2 CHANNEL DESIGN CONSIDERATIONS

4.2.2.1 VELOCITY MITIGATION

The proposed stormwater channels generally fall at between 1% – 5%. However, the upper extents of the alluvial fan formation gradually steepen to around 10% where flows exit the channelised gully.

Modelling demonstrates that unmitigated this gradient results in channel velocities that are above the desired maximum of 2m/s, with velocities up to 3.5m/s observed. The proposed channel will incorporate check dam structures to control flow velocities. The typical check dam arrangement is shown below:

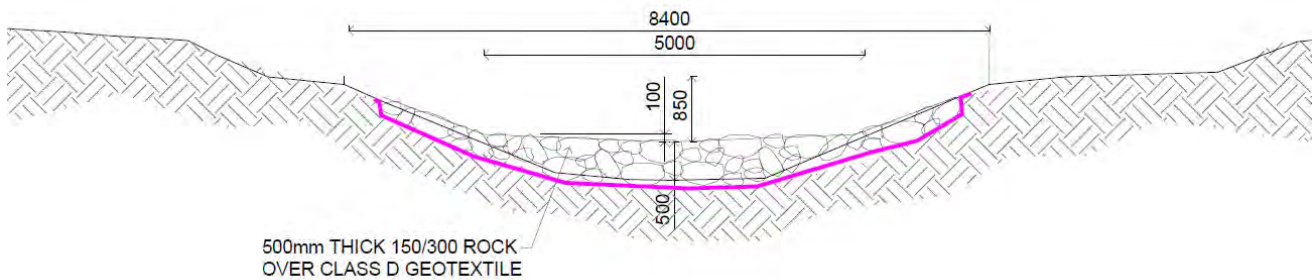


Fig 8a - Typical check dam structure – section view

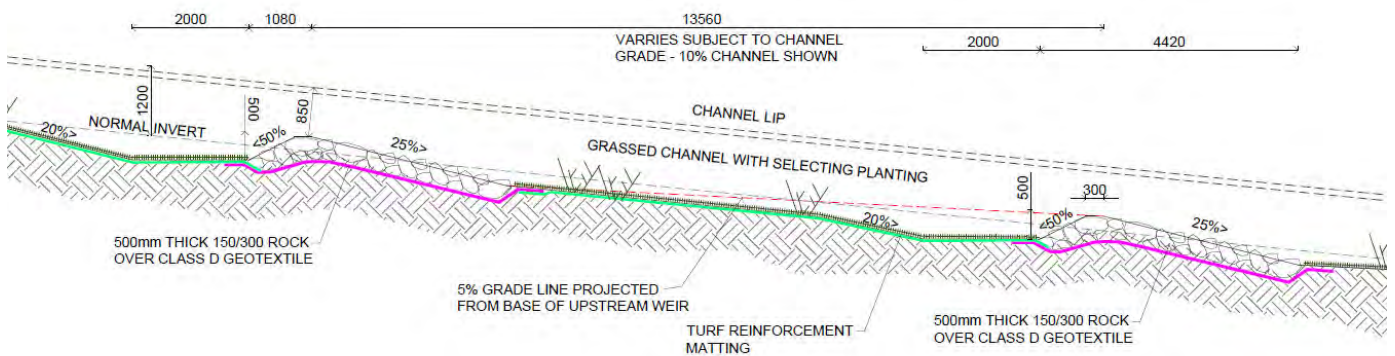


Fig 8b - Typical check dam structure – Profile view

Check dams will be constructed where channel grades exceed 5%. Spacing is based on a 5% gradient projected from the base of the upstream dam as shown above.

The proposed channel planting & grassing will also help control velocities.

4.2.3 EROSION & SCOUR PROTECTION

The risk of long-term erosion & scour in the channel network has been considered. Excessive scour can ultimately result in channel failure and must be mitigated to ensure the ongoing performance of the system.

Flow velocity is the primary factor in determining risk of erosion & scour. The velocity mitigation measures detailed in section 4.2.1 above are the primary mitigation feature, but additional measures are to be implemented as detailed below:

- Stabilisation & re vegetation of channel alignment. – The channel base and walls will be reinforced with turf reinforcement matting & grassed. A narrow gravel base will be maintained for normal catchment flows.
- Planting of channel walls. – Channel walls will be planted with suitable riparian species.
- Rip Rap armour – High energy locations such as culvert outlets, check dams and channel bends will be armoured with a suitably sized rip rap placed over a class D geotextile.

The proposed mitigation measures will significantly reduce the risk of scour, but it cannot be fully eliminated. Ongoing inspection of the channel network following significant rain events will be required to identify and remedy any failures that might occur. This will be included in the system O&M manual.

4.2.4 CHANNEL AGGRADATION

The proposed channel sits on an existing alluvial fan formation. Despite no evidence of recent fan activity or sedimentation of the existing flow paths, the risk of channel aggradation must be considered.

Mitigation measures have been implemented to allow for the risk of aggradation, these are detailed below:

- Mitigation of erosion & scour risk within channels – As detailed in section 4.2.3. This removes a primary sediment source which leads to downstream aggradation.
- Additional channel width & depth – Channels have been provided with additional freeboard beyond the minimum 500mm to allow for sedimentation of the channel base. Modelling indicates that channels generally have 700 – 800mm of freeboard available which provides a significant buffer to allow for aggradation in the channel. For the upper extents of the channel where sediment loading is expected to be the highest this is increased to over 1m freeboard.
- Check Dam structures – These incorporate a sediment basin behind the check dam to capture sedimentation. Refer section 4.2.1 above.
- Stormwater system O&M documentation – The O&M manual, to be provided the CODC who will be the long-term owner & operator of the system, will outline an inspection & maintenance schedule for the channels & basin. This will include inspection of sediment accumulation in the check dam sumps & channel base at regular intervals. If sediment levels exceed acceptable limits, it will be cleared.

One side of the channel formation includes a 3m wide bench to allow access for clearing of silt accumulation by an excavator & truck. The easement provisions will allow for property access for clearing over subject land.

4.2.5 FLOOD MITIGATION

The proposed flood channels pass in close proximity to the proposed platforms on lots 5, 6 75, 76, 77 & 301. Details of flood risk mitigation provided below:

- Lots 75 – 77 – These platforms are sited over 1m above the channel lip so have sufficient freeboard to mitigate any flood risk.
- Lots 5 & 6 – These lots are sited 400mm above the adjoining channel lip. The height of the lip on the downhill side of the channel adjoining the cherry orchard has been set lower to direct any overflow in that direction away from any building platforms.
- Lot 301 - The pre development flow path runs through Lot 301. This flow is to be diverted into the proposed channel 2. The existing flow path will be filled. The proposed channel incorporates additional depth and bunding to divert all flows away from the proposed platform. The channel arrangement is detailed below. This results in 1.4m of available freeboard above the 100 year ARI storm flow. Overflow analysis confirms a blockage at the bend does not cause any inundation of the Lot 301 platform.

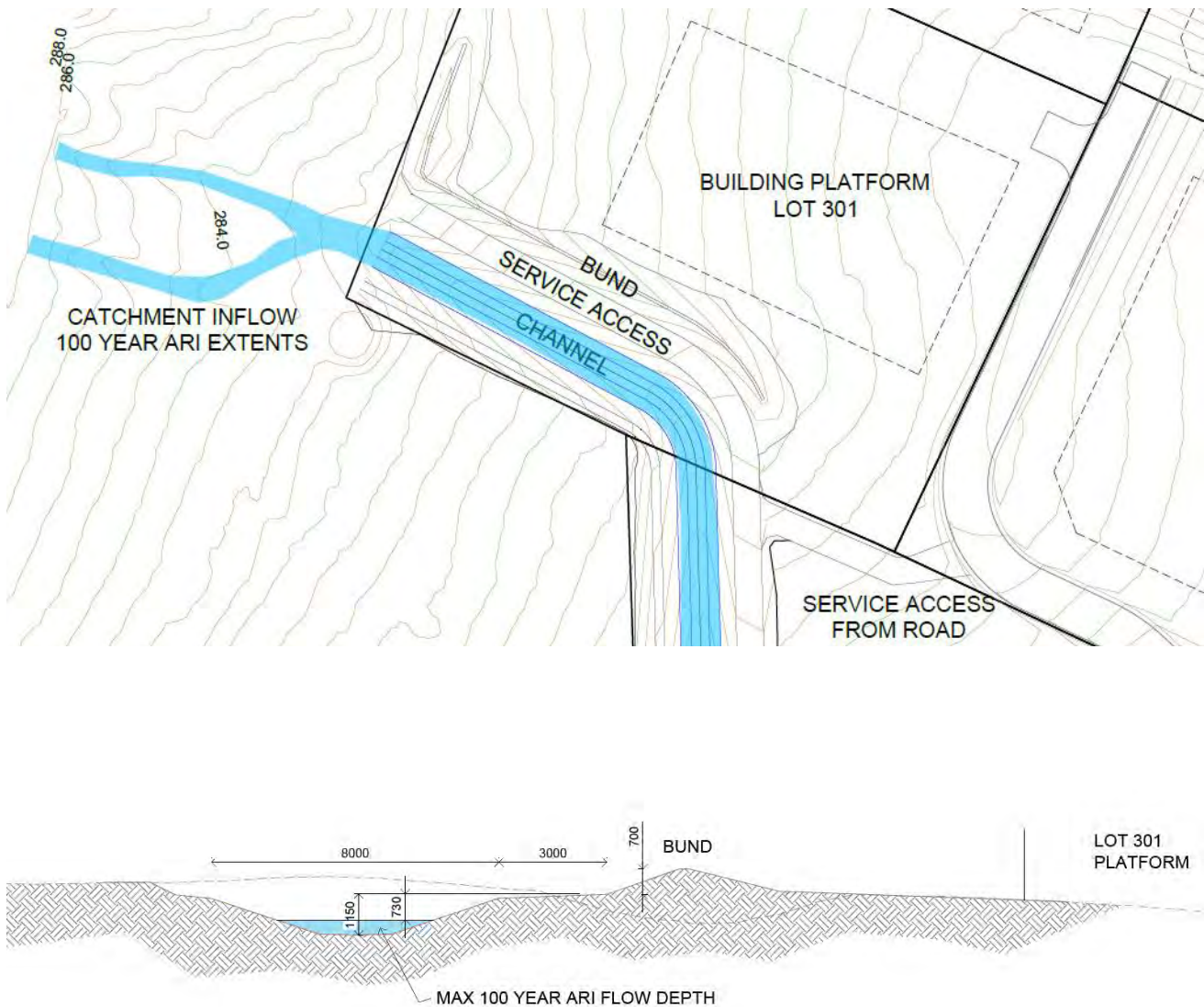


Fig 8c - Lot 301 channel layout & profile

4.2.6 SOAKAGE / DETENTION BASIN

4.2.6.1 DETENTION CONTROL LOGIC

The design calls for the soakage / Detention basin to capture flows from channel 1 & 2. During smaller events the flows will be captured and discharged to ground via the soakage area. In larger storms the soakage rate is not sufficient to discharge the storm flows so the basin starts to function as a detention device with a controlled release of water to the downstream catchment. The outlet weir is designed to ensure post development flows into the downstream catchment are less than the pre development scenario.

4.2.6.2 SOAKAGE / DETENTION BASIN

The soakage / detention pond has a base level of 239.00m, an embankment height of 241.20m and total storage capacity of 6,500m³ at max water level of 240.70m. The floor of the basin is lined with free draining gravels to allow smaller flows to soak away without any discharge from the basin.

The basin incorporates a forebay to capture silts & debris before discharging to the main soakage basin. Sedimentation & debris build up in the forebay would need to be cleared regularly to maintain system

efficiency. The maintenance obligations would be detailed in an O & M manual to be provided to the CODC who will ultimately own & operate the system.

A total area of 4040m² available for soakage within the basin. The average observed soakage rate for the site was 584mm/hr . Applying a 0.5 correction factor to allow for blockages and silting gives a design soakage rate of 292mm/h. This equates to a discharge of 0.327m³/h from the soakage basin which has been factored in the HEC RAS modelling.

A concrete weir is located at the SE corner which allows larger storms to discharge at a controlled rate to the downstream channel. This weir was modelled in HEC RAS. Weir details are shown below.

The max water level reached 240.50 during the simulation. This means there is additional capacity available in the basin to either reduce peak runoff further or absorb a unforeseen increase in runoff without exceeding the 500mm freeboard.

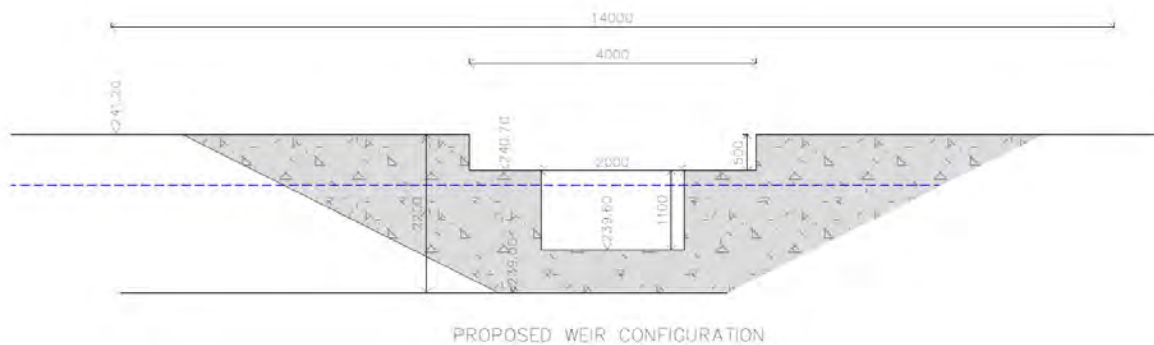


Fig 9 – Proposed basin Weir profile

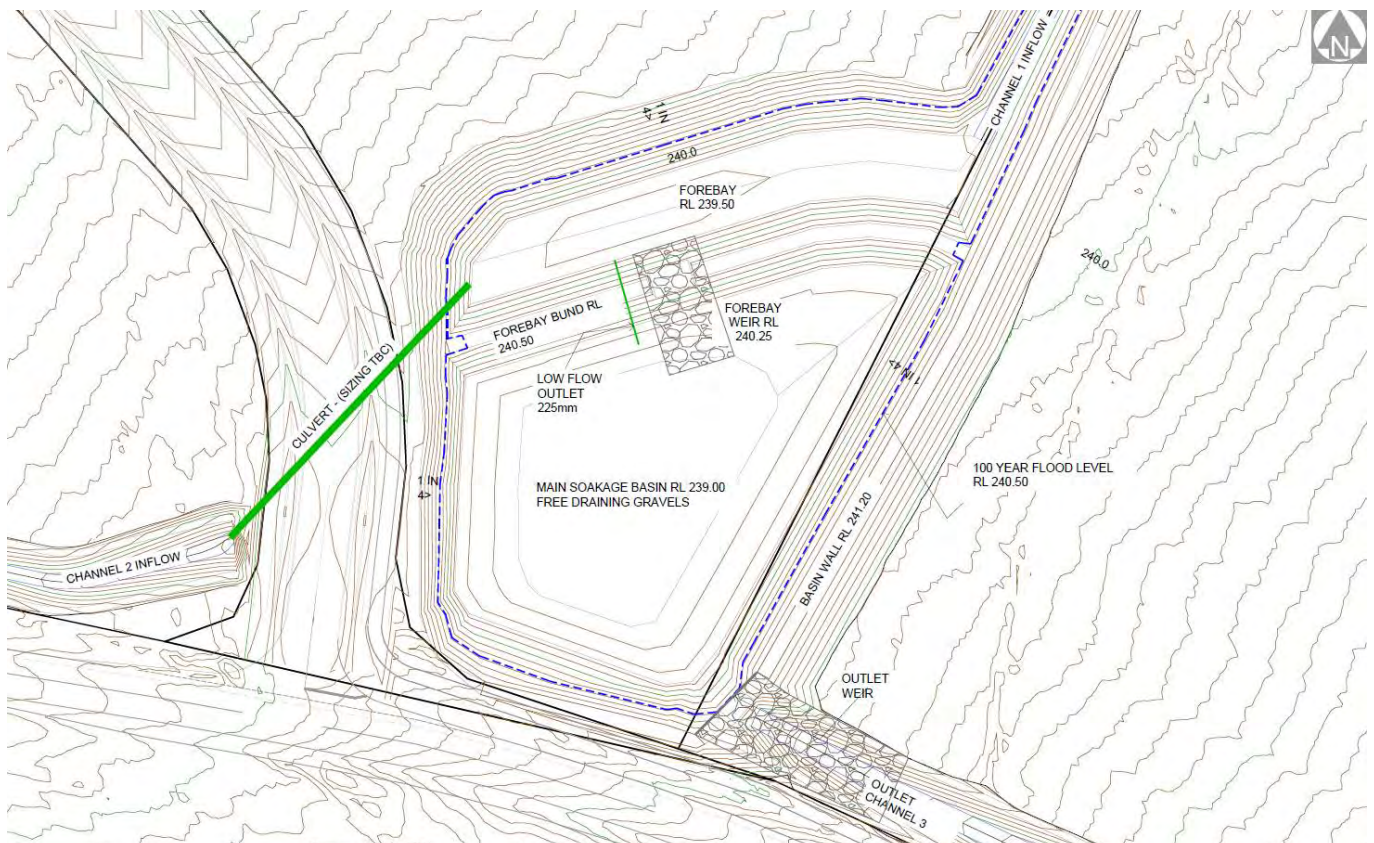


Fig 10 – Proposed soakage / detention basin layout.

5 HEC RAS MODELLING

5.1 INTRODUCTION

This section describes the modelling process applied to simulate rainfall and runoff scenarios and to evaluate flood mitigation measures for the Shannon Farm development.

5.2 OBJECTIVE

The objective of the modelling was to demonstrate that, once fully developed, the mitigation components introduced into the catchment would limit post-development peak flows exiting the catchment to predevelopment levels and mitigate the existing alluvial fan flood risk.

5.3 BACKGROUND

HEC RAS (Hydrologic Engineering Centre – River Analysis System) is an open channel flow analysis tool developed by the Hydrologic Engineering Centre (HEC) within the US Army Corps of Engineers. It is available as a free download and has become a widely adopted open channel flow analysis tool.

Until 2016 HEC RAS was limited to 1-dimensional (1-D) modelling. Nevertheless, it was recognised as a highly capable open channel analysis tool with capabilities that included steady and unsteady flow analysis, water quality modelling, sediment modelling, and the ability to analyse structures such as culverts, bridges, and weirs.

The latest HEC RAS version is a robust, capable, and widely used 2-dimensional (2-D) modelling package which allows flow over large complex terrain systems to be modelled effectively. It has proved to be an appropriate modelling tool for use in the Shannon Farm stormwater analysis by allowing 2-D rain on grid modelling over the entire project area.

The 2016 release of version 5.0 allowed 2-D unsteady flow modelling, the addition of mapping capabilities within the program (RAS Mapper), and the introduction of direct precipitation (rain on grid). The rain on grid capability made possible modelling of complex overland flow paths and wide flood floodplains. It has enabled an elevated level of modelling accuracy (significantly more so than if it were modelled in 1D only) with graphical outputs that are both high quality and descriptive. A further improvement came in June 2021 when version 6.0 was released making rainfall loss accounting possible.

It should be noted that HEC RAS does not have all the hydrologic capabilities of a specific hydrologic modelling package such as HEC-HMS. For example, it is not possible to account for conditions such as snow melt within the program directly. However, its hydrologic capabilities are adequate for land development and engineering applications provided that catchment size and run-off characteristics are well defined.

5.4 MODEL DEVELOPMENT

Direct precipitation on a HEC RAS 2-D model was used to assess predevelopment run-off from the Shannon farm catchment. Rain on grid modelling provided a graphical representation of how sheet flow developed from the hillsides above the development area, amalgamated as channel flow within the gullies, and dispersed out over the fan formation.

An analysis was then made of the flow as it exited the site to the east. The analysis outputs were both graphical, in the form of maximum depth maps and particle tracing arrows, and numerical in the form of flow hydrographs.

Post development run-off was then assessed to gauge the effect of the channelisation of storm flows and increase in impervious surfaces. A mitigation strategy (utilising primarily soakage & detention devices) was developed to buffer peak outflow. A 2-D post-development model including the mitigation devices was formed, rain on grid simulations carried out and the mitigation components incrementally refined until the modelling objectives had been achieved.

5.4.1 WORKFLOW SUMMARY

The modelling workflow started with sourcing terrain data and importing it into HEC RAS, thus creating a terrain model. Next the 2-D geometry was developed. This consisted of:

- the terrain model;
- the 2-D flow area;
- the SA/2D connections*;
- boundary condition lines;
- Manning's N regions; and
- SCS curve number regions.

2-D flow area consists of a perimeter, computation points, break lines and refinement regions. The perimeter was determined separately in HEC HMS using the watershed delineation function. Nominal grid cell size was specified. Break lines and refinement regions were added to delineate flow paths and ridgelines, and computation points were calculated.

This project only required the outlet weir as a SA/2D connection.

External boundary condition lines were digitised at key locations to allow flow to exit the model.

Mannings "N" regions were digitised to allow flow velocity to be simulated.

SCS curve number regions were digitised to allow infiltration losses to be accounted for.

Once the geometry had been thus described, unsteady (i.e., varying with time) flow data files were created to define the rainfall input (i.e., precipitation).

Unsteady flow analysis files were created to link the precipitation and the 2-D geometry so that simulation could be executed.

*SA/2D connections are structures within 2-D flow areas. They can include overflow weirs, culverts, linear routing devices and bridges.

5.4.2 2D FLOW AREA GEOMETRY FILE

5.4.2.1 TERRAIN MODEL

LiDAR data of the entire subject site (flown by landpro 2019) was utilised to create the terrain model in RAS Mapper. The data was imported in the form of a 1m resolution digital elevation model (DEM). This resolution was highly satisfactory as it allowed the complex terrain to be modelled precisely. Areas outside the lidar DEM were supplemented with 8m LINZ dem data for the upper reaches of the Pisa Range.

The terrain extents were selected so that a substantial buffer was provided around the 2-D flow area.

The post-development terrain model included the predevelopment surface overlain with terrain modifications such as roadways, building platforms, surface drainage channels and ponds. The terrain model was generated in 12D software, and a combined surface exported to HEC RAS.

The post-development terrain included the basin, roads, channels, and allotment levels. Two channels and a soakage / detention were incorporated into the design surface. The proposed soakage/ detention basin included the modelling of an outlet weir as a 2D connection in HEC RAS.

Culvert modelling is possible in HEC RAS. However, as the development is still subject to detailed design the culverts on the main channels have been modelled as channels until a final culvert design is confirmed.

For the channel overflow simulation, the terrain was modified to create a full blockage of the proposed channel at the 2 significant bends to assess the overflow route.

5.4.2.2 PERIMETER

The 2-D flow area perimeter was determined by using the automated watershed delineation function in HEC HMS. 2D perimeter was extended beyond the ridgelines surrounding the site to ensure all contributing flow was accounted for. Output shown below:

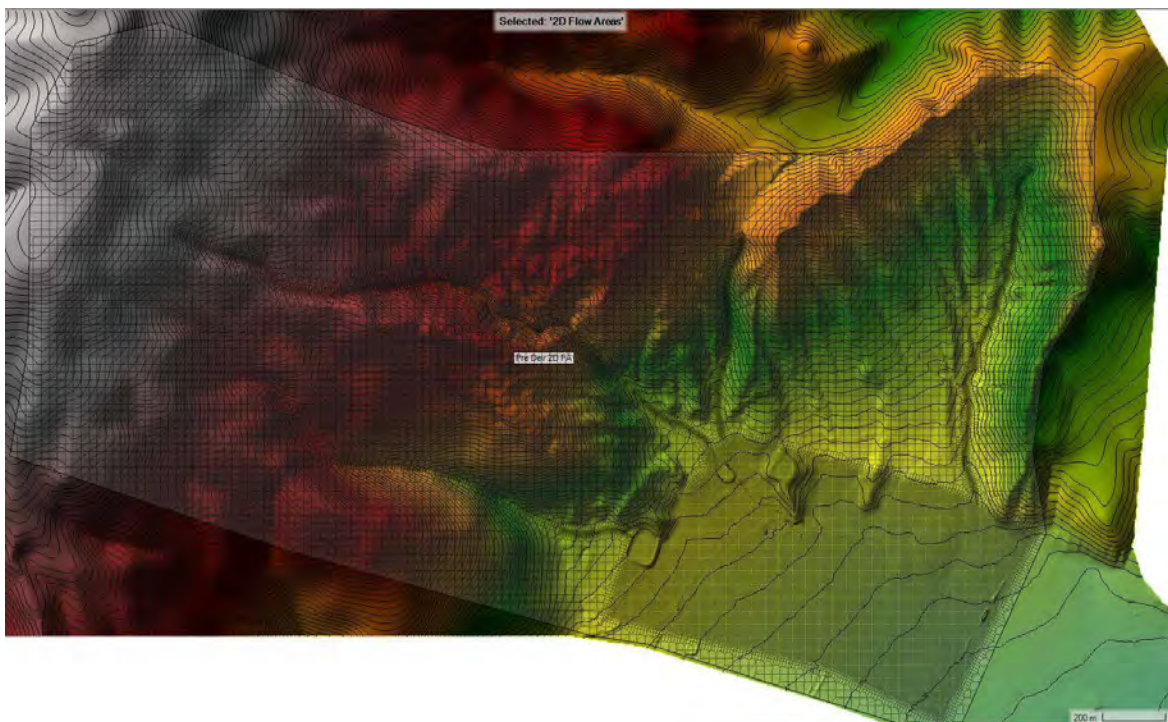


Fig 11 - 2D flow area

5.4.2.3 COMPUTATION POINTS, BREAK LINES AND REFINEMENT REGIONS

Computation points were created based on a nominal 25m grid oriented east – west.

Refinement regions and break lines were introduced to provide higher resolution and cell face orientation where necessary. The lower areas of the site where concentrated flow occurs was refined to a 5m base grid with 2m cells assigned along channel flow paths to ensure concentrated flow was accurately modelled. These all contributed to model stability during the simulations. The predevelopment 2-D flow area, once completed, contained 44,000 cells.

The post-development model Includes additional refinement regions and break lines to provide the necessary resolution around the design flow channels, ponds, development areas, and roadways (2m cells). Flows from the gully catchments were introduced as internal boundary conditions to reduce the processing required for

the post development model. As a result the post development 2-D flow area, once completed, contained 22,000 cells.

5.4.2.4 SA/2D CONNECTIONS (WEIRS & CULVERTS)

SA/2D connections are structures within 2-D flow areas. They can include overflow weirs, culverts and bridges.

No culverts or weirs were modelled in the pre development simulation as all flow was overland.

The post-development geometry incorporated an outlet weir on the soakage/ detention basin to control outlet flow the basin. Profile in HEC RAS shown below.

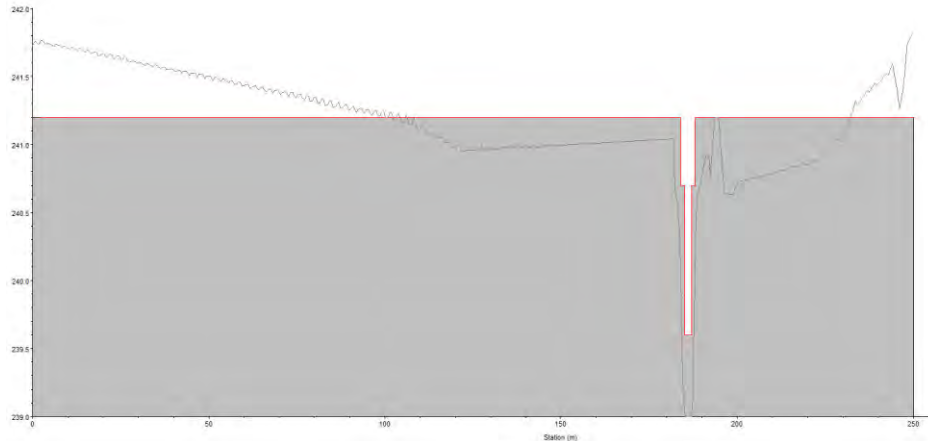


Fig 12 – Outlet Weir profile in HEC RAS.

5.4.2.5 BOUNDARY CONDITION LINES

The geometry was provided with an external boundary condition to allow flow to exit the model. This extends around the entire perimeter of the 2D flow area.

For the post development model, additional profile lines were placed upstream of Channel 1 & 2. HEC RAS allows internal boundary condition lines to be created so that steady or unsteady flow can be introduced into the model. In this case, an unsteady flow hydrograph was introduced as an internal boundary condition line and flows calculated during the pre-development analysis from the Western & Norwest catchments introduced. This cuts down the computational area allowing a more detailed analysis to be performed on the post development area without excessive processing times.

For the channel overflow analysis channel flow obtained from the post development model was introduced as an internal boundary condition above the channel obstruction.

5.4.2.6 MANNINGS N REGIONS

The Manning's n value is a unitless coefficient that represent the roughness or friction factor of a surface or conduit. Rougher conduits with higher friction have a higher value, and smoother conduits with lower friction have a lower value. The values for Manning n are derived from experiment and observation.

The following values were used in the HEC RAS model build:

Manning's N

Pasture & scrub cover	0.150
Flow channels	0.035
Stream	0.035
Seal	0.016
Gravel	0.018
Residential activity areas	0.016

5.4.2.7 SCS CURVE NUMBER REGIONS

A SCS curve numbers were assigned to the different land use types across the site allowing infiltration losses to be accounted for.

The following values were used in the model build:

SCS Curve Number

Pasture & grass cover	61 (group B soil, pasture, lightly grazed, good grass cover)
Bush, trees, Orchards	55
Roofs	98
Pavements - seal	98
Pavements – gravel	90

Curve numbers for activity areas have been obtained from ARC TP108 - Table 3.3. A Group B soil was selected as the alluvial sediments onsite fall within this category.

Land use	Group A Soil (volcanic granular loam)	Group B Soil (alluvial)	Group C Soil (mudstone/san dstone)
Bush, humid-climate, not-grazed	30	55	70
Pasture, lightly grazed, good grass cover	39	61	74
Urban lawns	39	61	74
Crops, straight rows, minimal vegetative cover	72	81	88
Sealed roads, roofs	98	98	98

ARC TP108 - Table 3.3

In accordance with the proposed resource consent conditions, all runoff from impervious areas within allotments shall be captured and stored in rainwater storage tanks to be used for irrigation or disposed to ground in a soak pit with sized to detain the 100-year storm event.

As such, additional flow from future impervious surfaces will be detained on each lot and will not factor in the overall site runoff in a storm event. As such, each allotment has been modelled as grass / gardens with a curve number of 61 as a baseline runoff.

Development roading has been modelled as impervious seal with a SCS curve number of 98.

Stormwater from Road surfaces will be captured by sumps and discharge to ground in CODC standard soak pits. Modelling this in a HEC RAS simulation is difficult. Open pits have been created to simulate the storage volume, but no infiltration is factored. This results in a conservative estimation of overall runoff from road surfaces.



Fig 13 - Pre development SCS curve areas

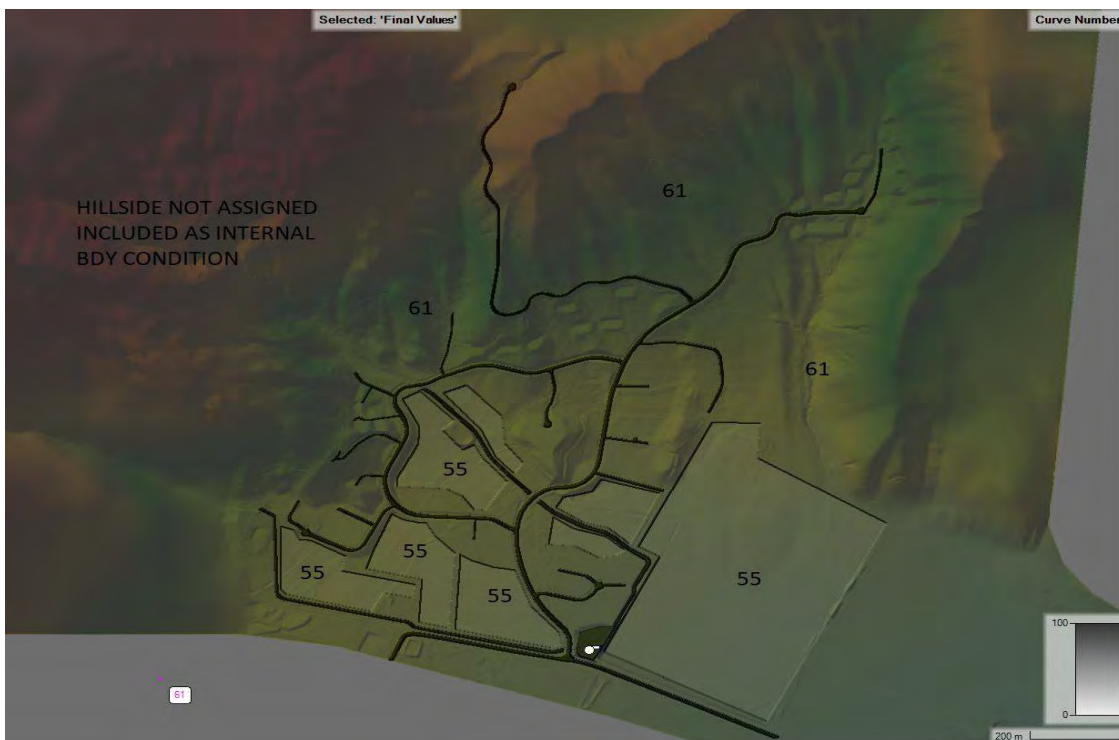


Fig 14 - Post development SCS curve areas

5.4.2.8 ABSTRACTION RATIO (INITIAL LOSSES)

The Shannon Farm HEC RAS model uses an abstraction ratio of 0.05 in accordance with model fitting undertaken by Hawkins et al (2002) and accepted modelling practice.

5.4.3 UNSTEADY FLOW DATA FILES

The rain on grid method was used for all hydraulic modelling to account for catchment storage and routing flows. The unsteady flow data file links inflow and outflow. The inflow was defined as the precipitation boundary condition, and the outflow was defined as the five external boundary conditions.

An unsteady flow data file was created for the 100-yearARI (1% AEP) storm.

24-hour nested storm hyetographs were prepared in Microsoft Excel from NIWA HIRDS V4 for use in the precipitation boundary condition.

Rainfall intensities (mm/hr) :: RCP8.5 for the period 2081-2100									
ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h
1.58	0.633	23	16.3	13.5	9.82	7.05	3.99	2.66	1.7
2	0.5	26.2	18.5	15.2	11.1	7.94	4.44	2.95	1.87
5	0.2	38.4	26.7	21.8	15.6	11	6.05	3.96	2.47
10	0.1	48.9	33.6	27.3	19.3	13.5	7.3	4.73	2.92
20	0.05	60.9	41.5	33.5	23.5	16.3	8.68	5.56	3.39
30	0.033	68.8	46.7	37.6	26.2	18.1	9.53	6.07	3.68
40	0.025	74.8	50.5	40.6	28.2	19.3	10.2	6.46	3.9
50	0.02	79.8	53.8	43.1	29.8	20.4	10.7	6.75	4.07
60	0.017	84	56.4	45.2	31.2	21.3	11.1	7.01	4.21
80	0.013	91.1	61	48.7	33.5	22.8	11.8	7.41	4.44
100	0.01	96.7	64.6	51.5	35.3	24	12.4	7.74	4.62
250	0.004	123	81	64.1	43.4	29.1	14.7	9.11	5.37

Fig 15 – HIRDS V4 rainfall depths

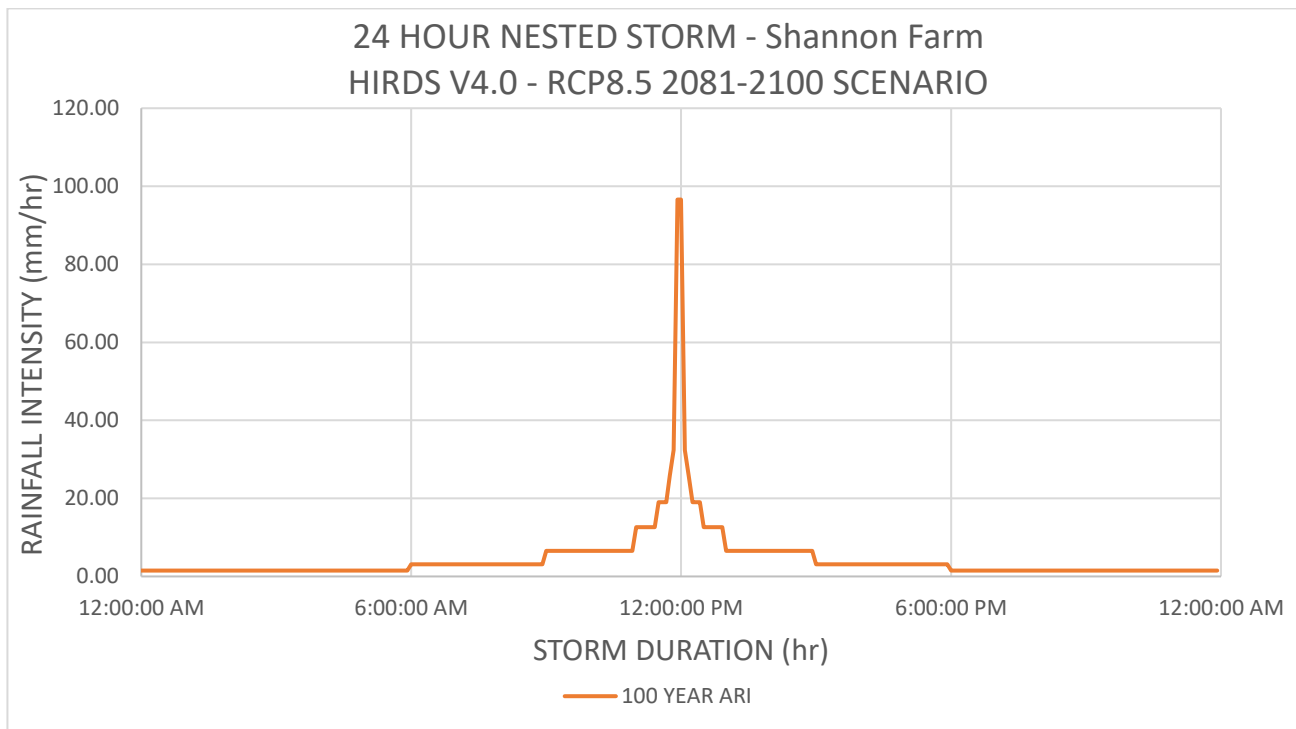


Fig 16 - rainfall hyetograph

The RCP 8.5 2081 – 2100 scenario precipitation data was selected: the most pessimistic climate change scenario.

Precipitation hyetographs for the 100-yearARI (1% AEP) storm were tabulated in five-minute data time intervals. The precipitation values were then transferred to the unsteady flow data file precipitation boundary condition fields.

5.4.4 UNSTEADY FLOW ANALYSIS FILE

The unsteady flow analysis file links the 2-D geometry and the unsteady flow file (in other words the terrain and the rainfall) and allows for simulation parameters to be specified as follows:

Unsteady flow analysis parameters:

- the geometry file;
- the unsteady flow file;
- the simulation start and end date (set nominally to 1 Jan 2023);
- the simulation start and end time (set nominally to 0:00:00hrs and 23:59:59hrs);
- the computation interval (set at 0.5 seconds);
- the mapping output interval, hydrograph output interval and detailed output interval (these were all set at five minutes); and
- the equation set to be specified in the unsteady flow operations and tolerances (SWE-ELM* was used)

The computation interval of 0.3 seconds was required to provide model stability during the simulation. Using longer computation intervals mean shorter simulation run times however this can cause instabilities to develop depending on the complexity of the model. It is often necessary to use a short computation interval and to accept longer simulation run times to achieve sufficient accuracy.

The mapping output interval, hydrograph output interval and detailed output interval were all set at one minute to keep file size to reasonable limits. SWE-ELM was specified as the equation set in the 2-D unsteady flow options and tolerances.

* HEC-RAS has the option of running the following equation sets: 2D Diffusion Wave equations; Shallow Water Equations (SWE-ELM) or a Shallow Water Equation solver (SWE-EM). SWE-EM is more momentum conservative; it requires smaller time steps longer run times. The default is the 2D Diffusion Wave equation set. In general, many flood applications will work satisfactorily with the 2D Diffusion Wave equations. The Diffusion Wave equation runs faster and is inherently more stable.

The SWE-EM solver is an explicit solution scheme that is based on a more conservative form of the momentum equation. This solver produces less numerical diffusion than the SWE-ELM solver. However, in general, it is only needed when users are interested in taking a close look at changes in water surfaces and velocities at and around hydraulic structures. The original SWE-ELM solver is more than adequate for most problems requiring the full momentum equation-based solution scheme.

5.5 HEC RAS OUTPUTS

Once complete, the results were viewed in RAS Mapper. RAS Mapper allows animated viewing of the simulated water depth, velocity, and elevation.

RAS Mapper functionality also allows profile lines to be digitised within the geometry for post simulation analysis. HEC RAS enables flow characteristics across the profile lines to be analysed such as depth, flow, velocity versus time, and accumulated volume. The results can be graphed directly in HEC RAS or tabulated for inclusion in external software such as Microsoft Excel.

Profile lines were placed at key points in the pre-development and post-development geometries allowing hydrographs to be generated, comparisons to be made between predevelopment and post-development flow, and design iterations to be carried out until the project objectives had been met.

5.5.1 KEY PROJECT OUTPUTS

For the Shannon Farm project the key outputs were:

1. Pre-development peak outflow hydrographs for flows from the western & north western catchments to confirm the flow that needs to be conveyed by the channels.

2. Confirmation that capacity exists in the proposed channels to convey the 100 year ARI storm event ;
3. Pre development peak flow hydrograph for 100 year ARI flows exiting the site; and
4. Post development peak flow hydrograph for 100 year ARI flows exiting the site
5. Channel overflow flow path at the bends on channel 1 & 2 to confirm no risk to proposed allotments.

5.6 MODELLING RESULTS

The outputs from the HEC RAS modelling as per section 5.5.1 above are shown below:

	Peak outflow (m ³ /s)	
	Pre-development 100 Year ARI	Post-development 100 Year ARI
Location		
1 Western Gully	2.65	2.65
2 North West Gully	3.69	3.69
3 Northern Gully	0.98	1.29
4 Eastern Gully	3.28	3.82
5 Total Catchment outflow	13.27	12.51

Table 1 – summary of HEC RAS modelling results

The location of the profile lines used to derive the flow data is shown below:



Fig 16 – profile line locations used in analysis. Blue sections indicate channel profile locations. Yellow is the location of channel overflow simulation.

The proposed stormwater system results in a 0.76m³.s reduction in overall peak outflow in a 100-year RCP8.5 storm event.

Inspection of the flow modelling in the channels (refer appendix A) confirms that they can convey these flows, while achieving the required 500mm freeboard throughout the channel network. Channels generally have 700 – 800mm of freeboard available. This allows a significant buffer to account for sediment & debris flow in the channel in a storm event. Proposed allotments are sited an additional 500mm above the lip of channel.

HEC RAS calculates flow velocities in the channels. This can be used to identify where velocity mitigation as detailed in section 4.2.1 will be required. These are identified as yellow & orange sections in the overall velocity map detailed in figure 78 of the HEC RAS outputs.

Modelling of the channel overflow at locations 1 & 2 confirms there is no risk to the proposed residential allotments in the event of an overflow due to a channel blockage or excessive aggradation at the bends.

Modelling of the proposed soakage basin shows a peak 100-year ARI water level of 240.50m with the proposed outlet weir configuration. This leaves 700mm of freeboard in the basin. The extra basin volume available provides an additional storage buffer to account for inefficiencies in the system such as sedimentation.

5.7 CONCLUSION

HEC RAS is a robust and capable 2-D modelling package which allows large complex systems to be modelled effectively. It proved to be an appropriate modelling tool for use in the Shannon Farm concept stormwater design project by allowing 2-D rain on grid modelling over the entire project area.

Based on the results of the modelling it can be concluded that the project objectives have been achieved. The modelling has demonstrated that, once fully developed, the effects of increased peak runoff resulting from the development will be mitigated through the provision of detention dams & stream diversions so that the peak runoff flows, coming off the new development will be no worse than they would be during the pre-development scenario.

The implementation and management of the detention areas creates a net overall reduction in peak outflow to the downstream land and mitigates the alluvial fan flood risk to the proposed allotments.

The findings and recommendations of this report are to be further refined at detailed design stage for engineering approval, for example specific scour protection in higher velocity areas and culvert design where channel crossings are required.

5.8 APPENDIX A - FLOW ANALYSIS

5.8.1 HEC RAS OUTPUTS – FLOOD FLOWS



Fig17 – Western Catchment – Pre Dev-Flooded extents

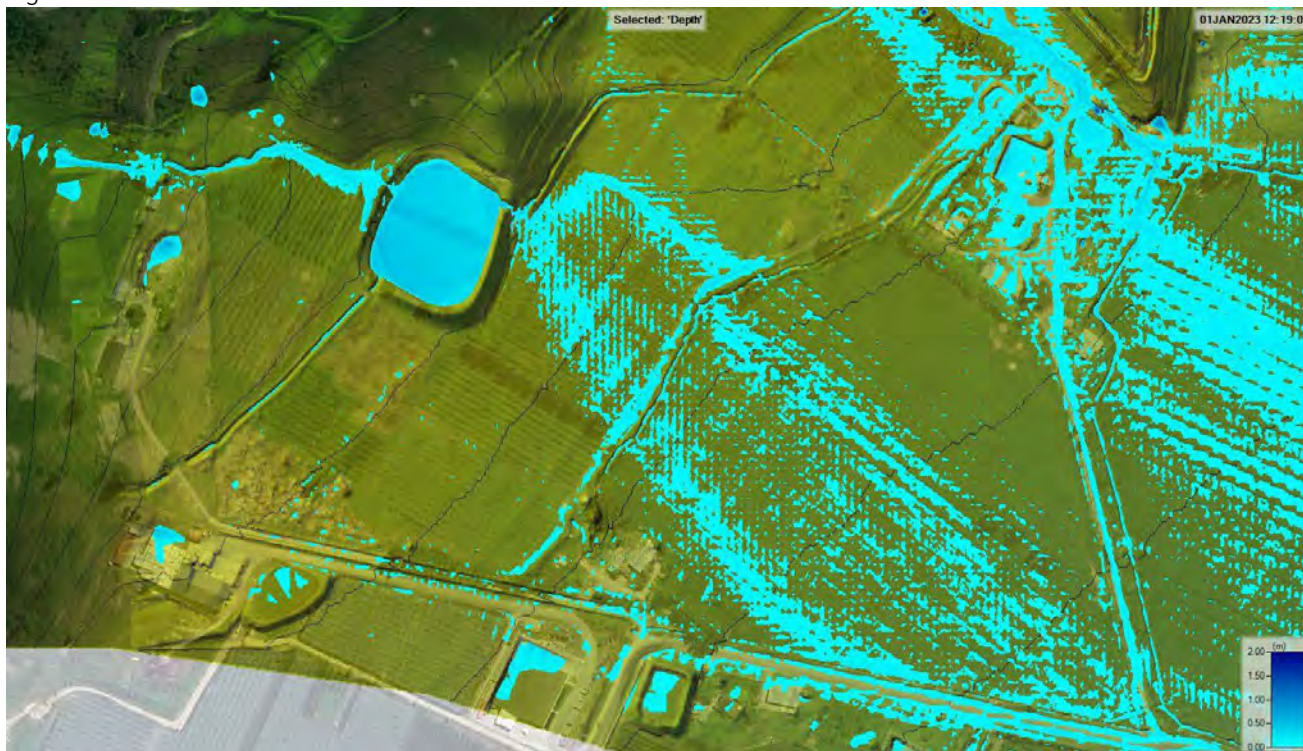


Fig17 & 18 – Western Catchment – Pre dev Flooded extents

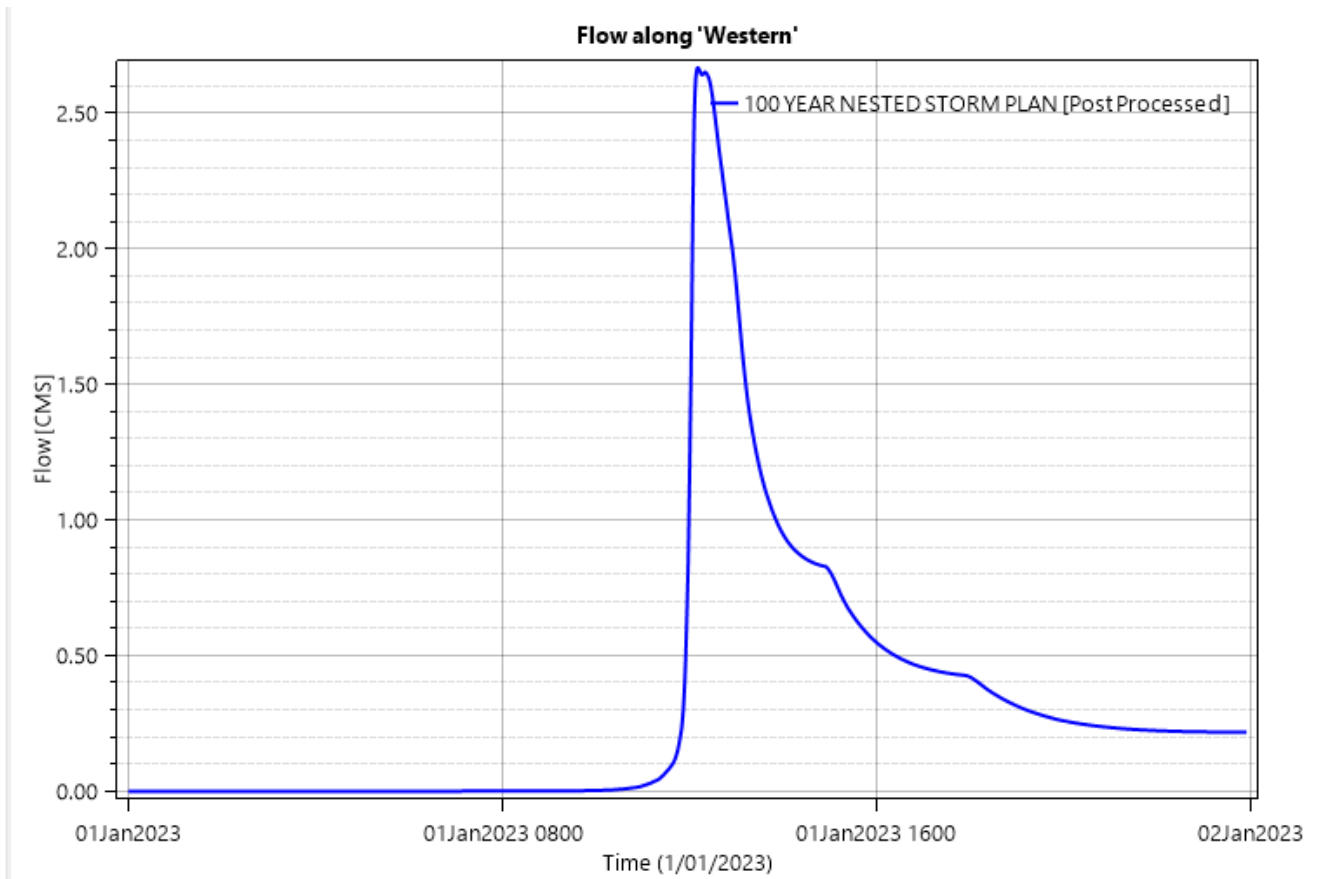


Fig 19 – Western Catchment Pre dev flow hydrograph

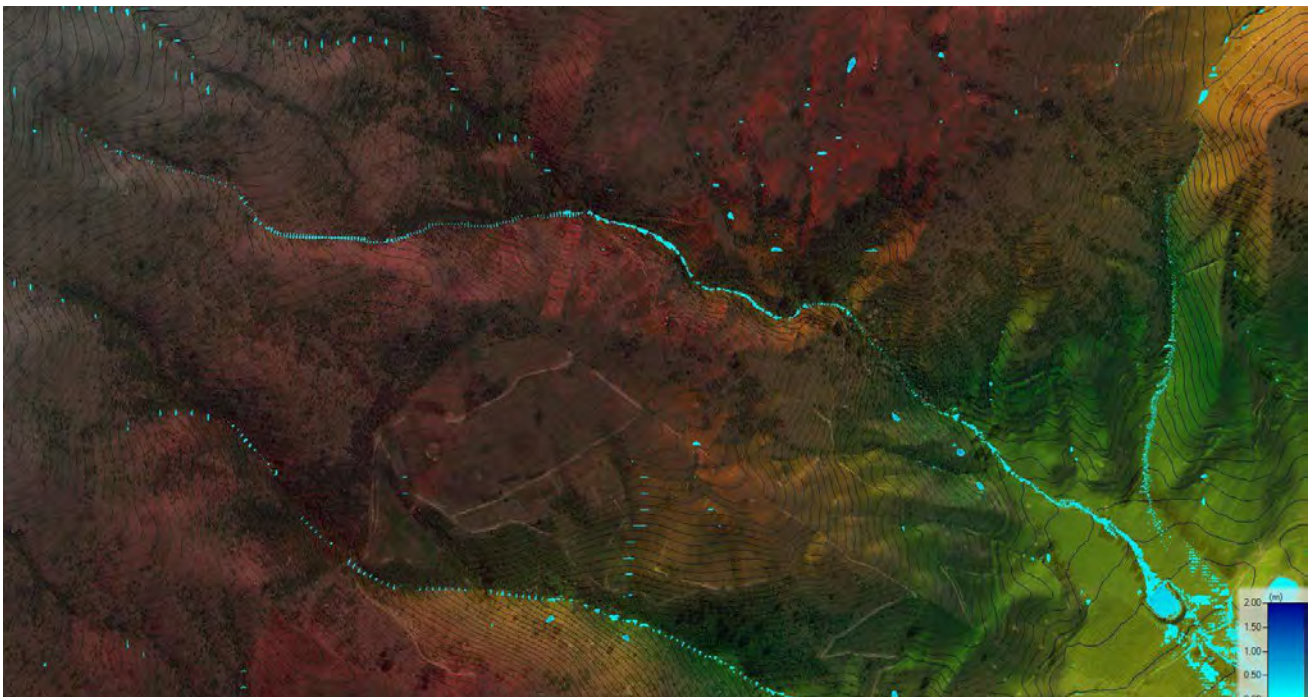


Fig 20 – North Western Catchment – Flooded extents

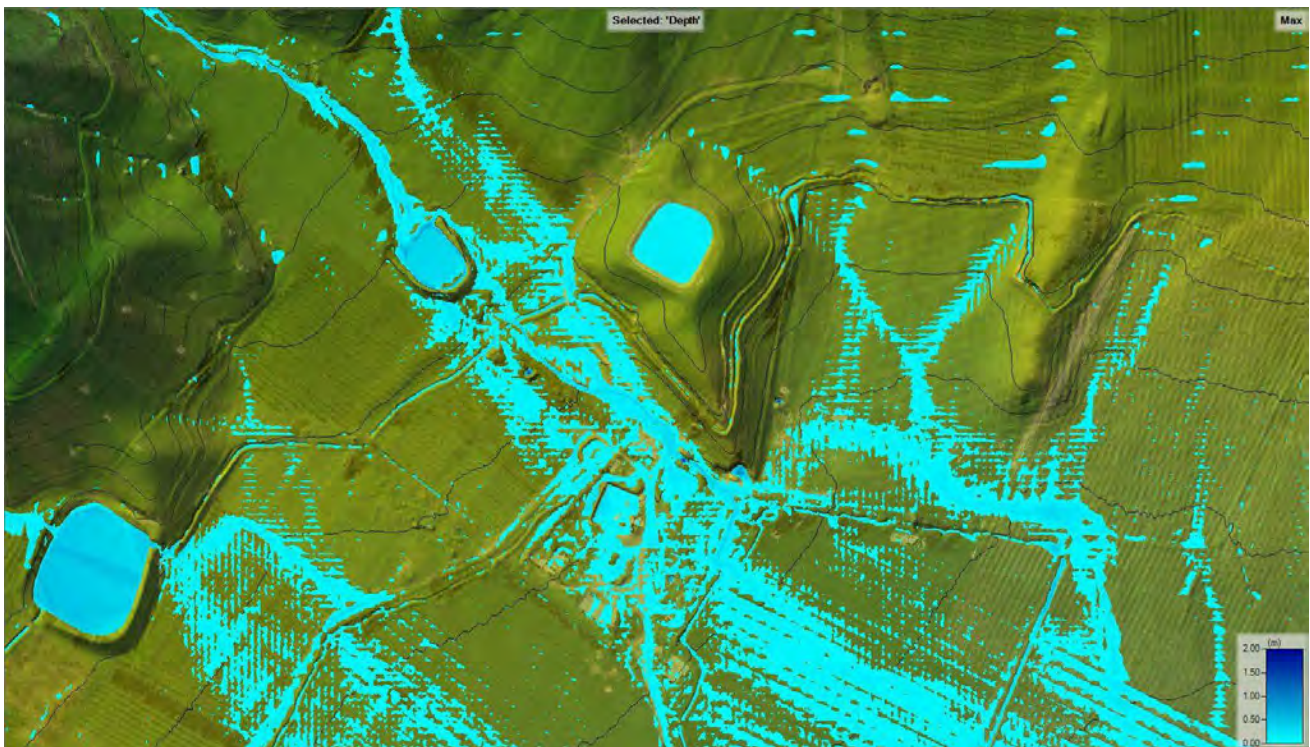


Fig 21 – North Western Catchment – Pre dev Flood flows exiting gully

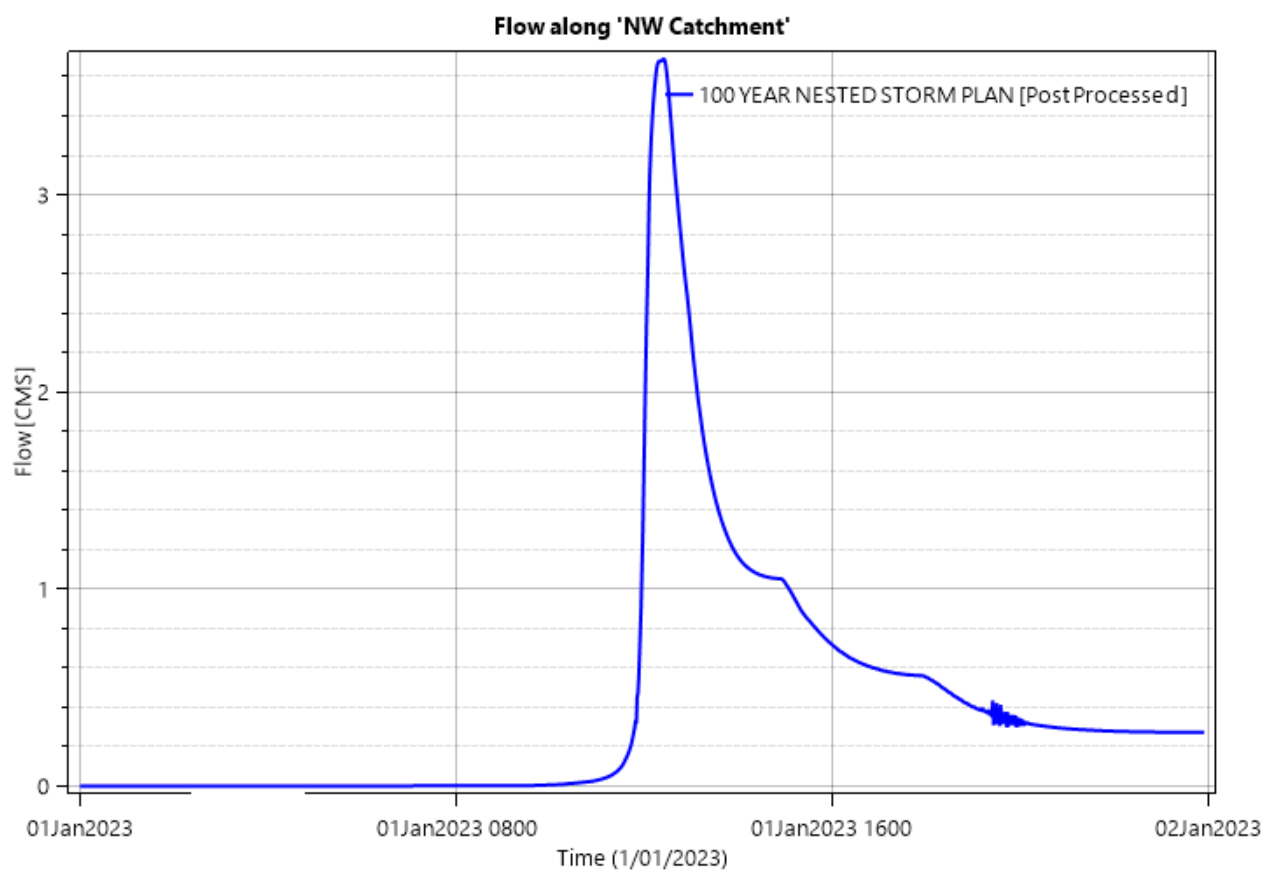


Fig 22 – Western Catchment flow hydrograph

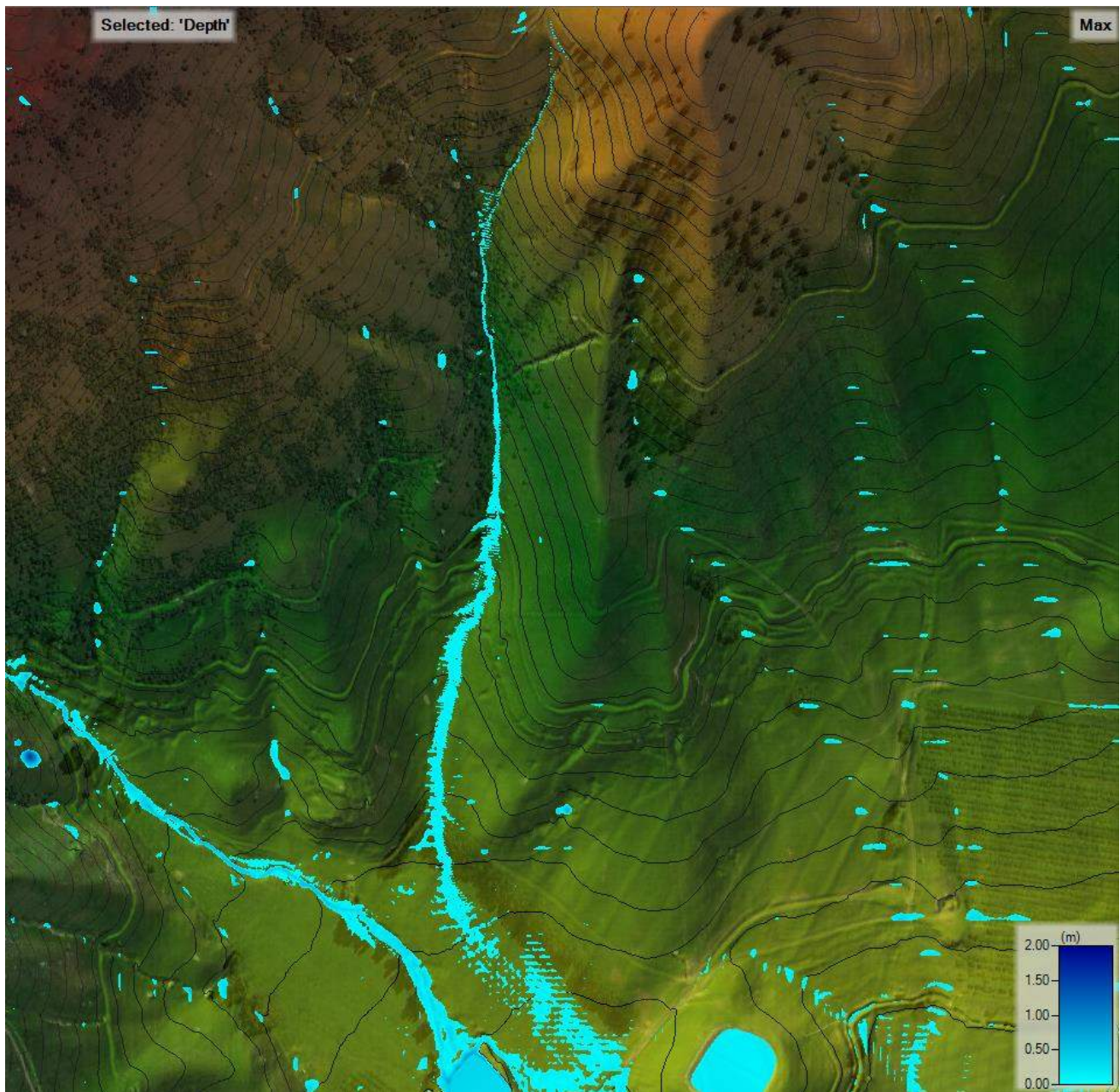


Fig23 – Northern Catchment – Flooded extents – Pre dev



Fig24 – Northern Catchment – Flooded extents – post dev

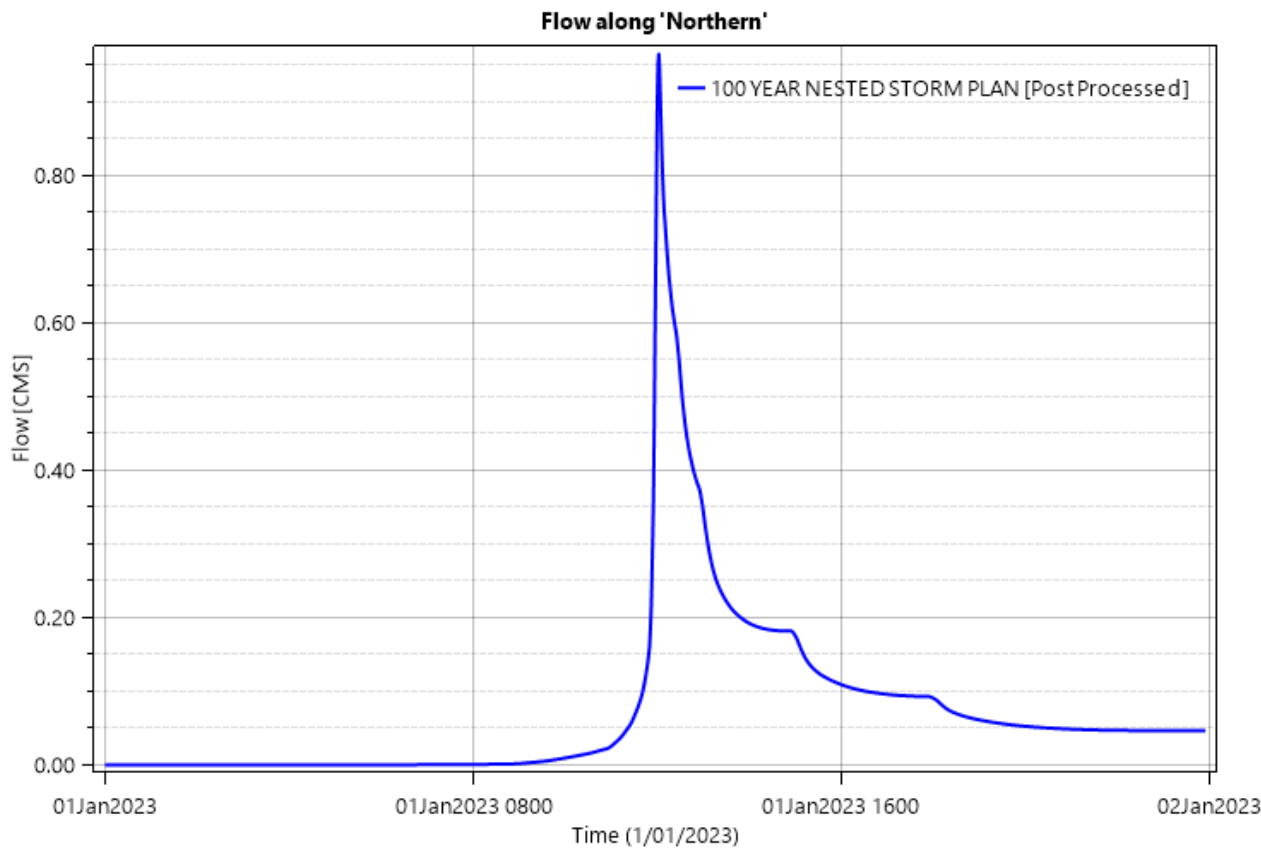


Fig 25 – Northern Catchment pre dev flow hydrograph

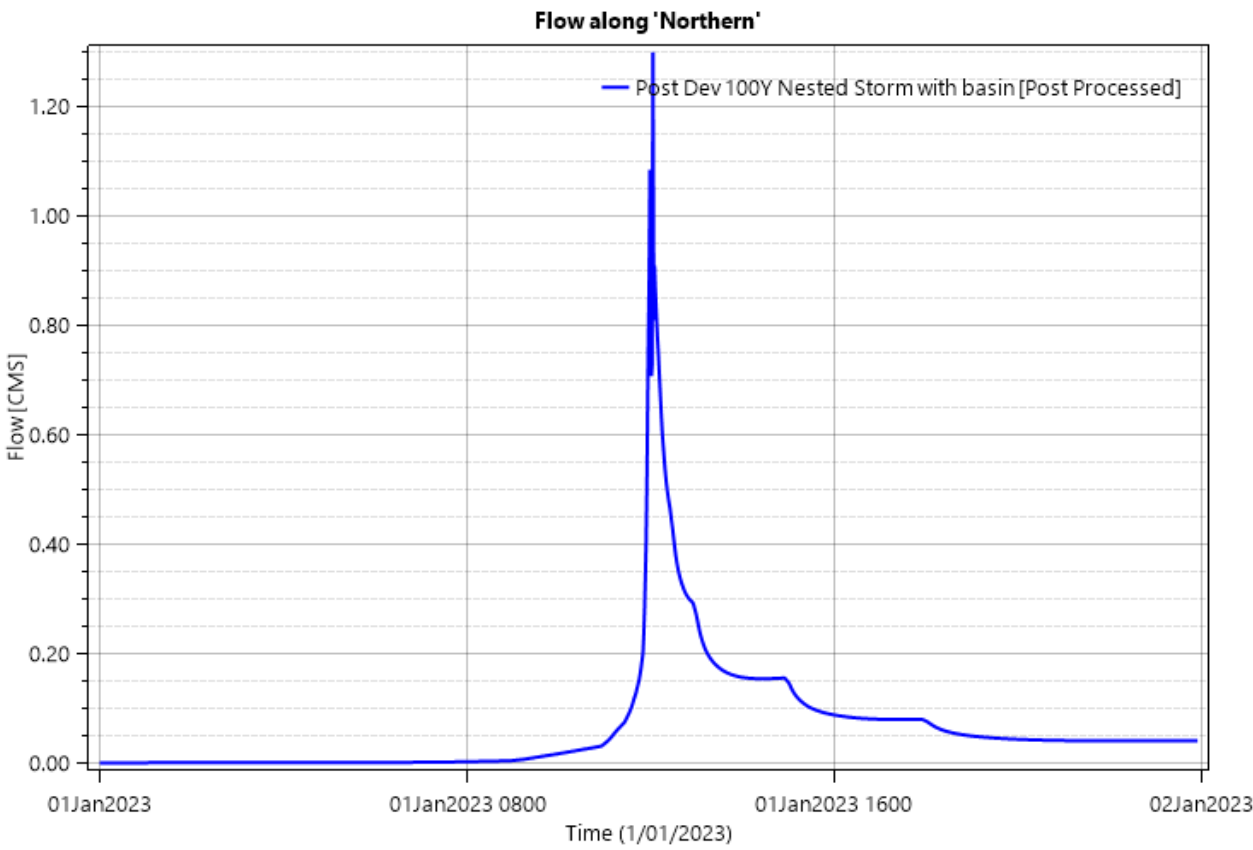


Fig 26 – Northern Catchment post dev flow hydrograph

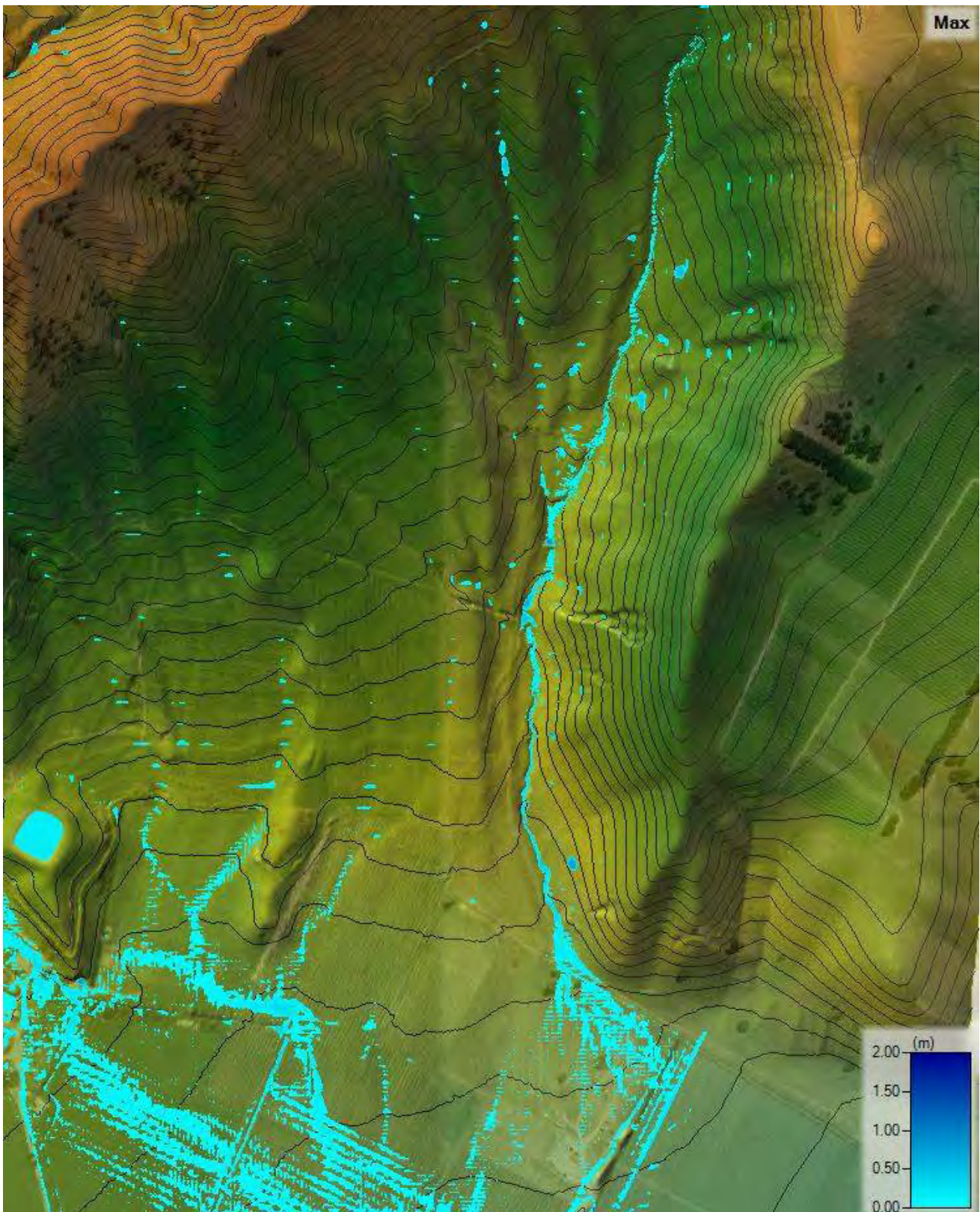


Fig27 – Eastern Catchment – Flooded extents – Pre dev



Fig28 – Eastern Catchment – Flooded extents – post dev

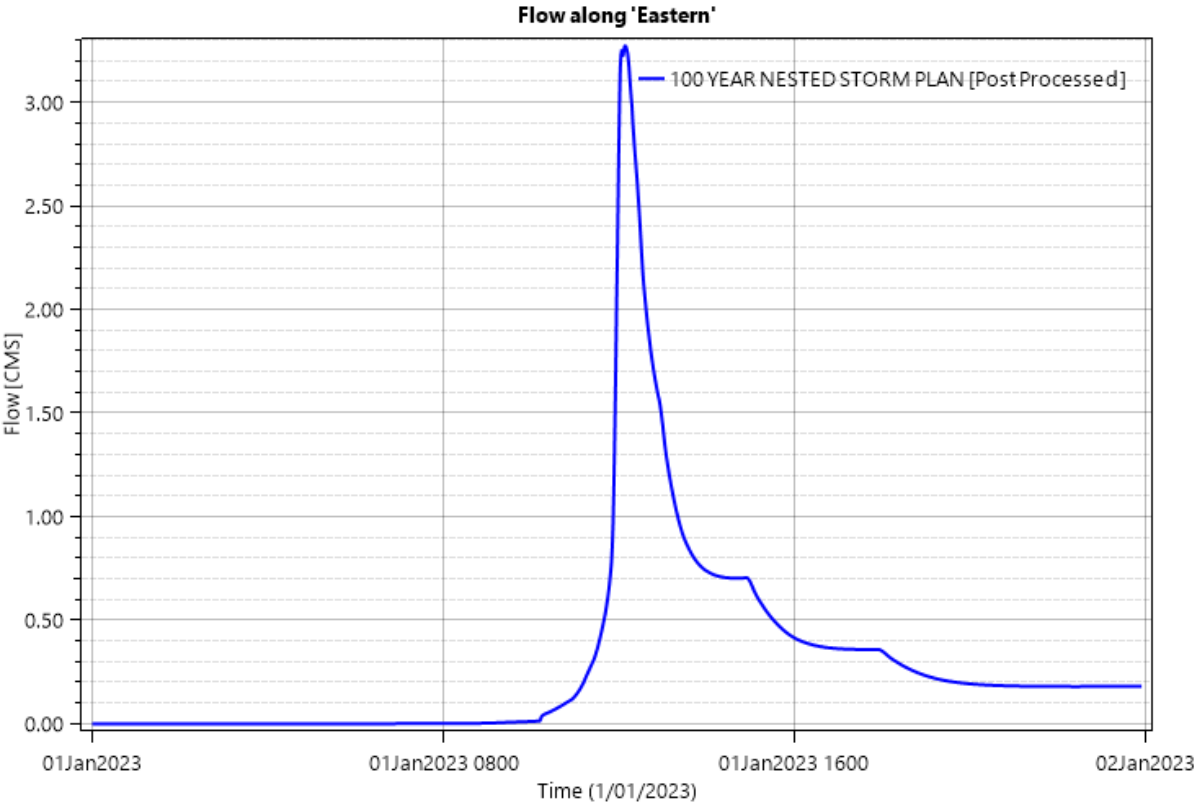


Fig 29 – Eastern Catchment pre dev flow hydrograph

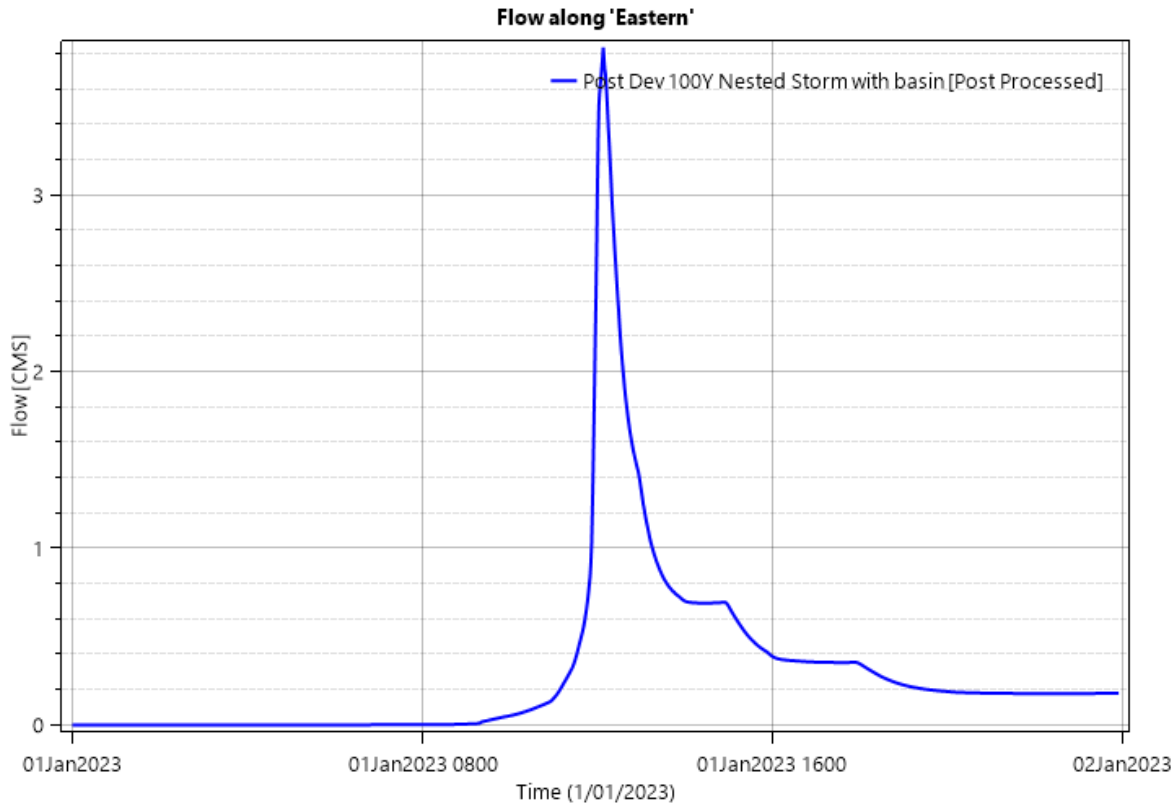


Fig 30 – Eastern Catchment post dev flow hydrograph

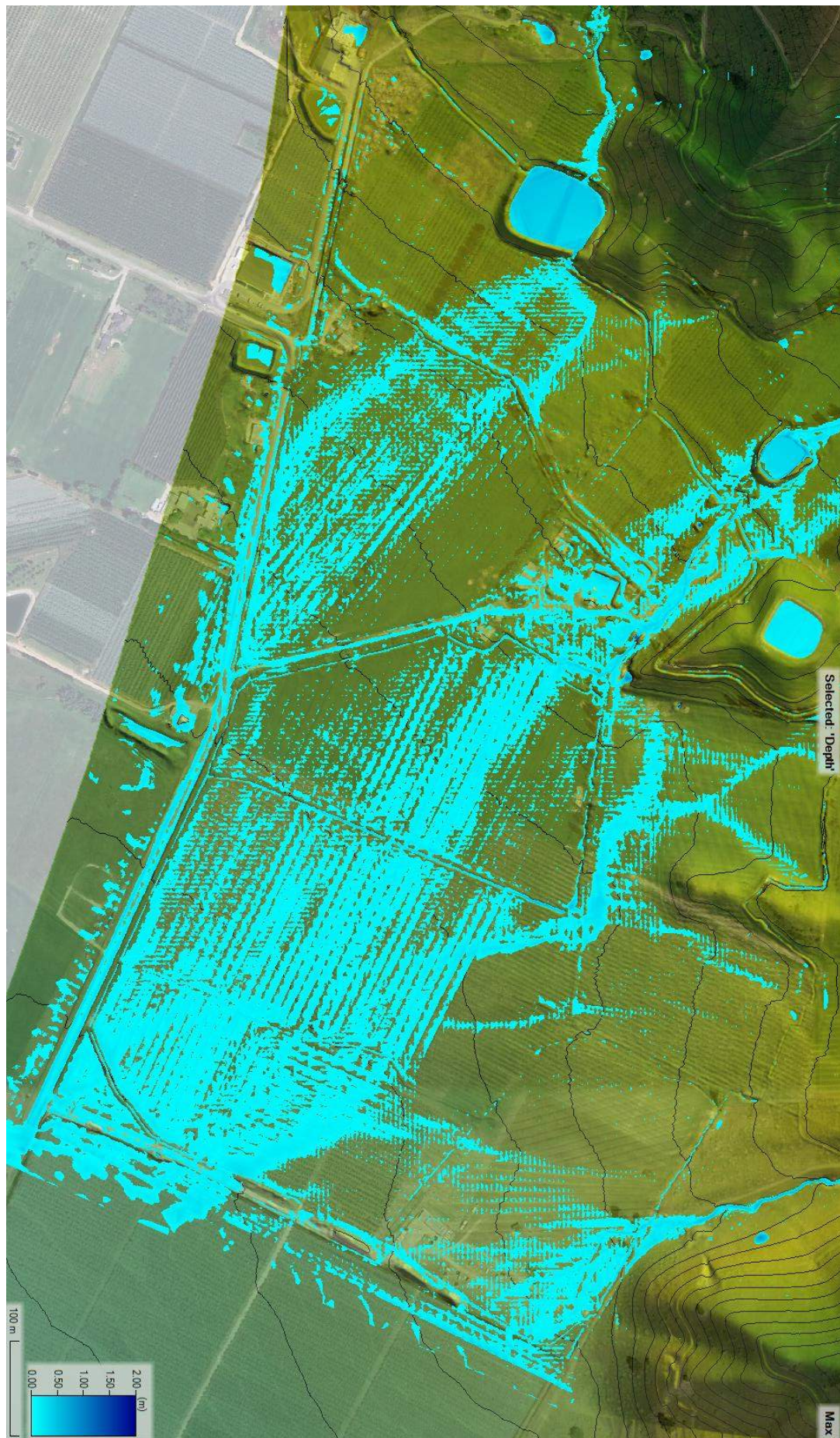


Fig 31 – Total outflow – Pre development (flow depths over 30mm shown)



Fig 32 – Total outflow – Post development (flow depths over 30mm shown)

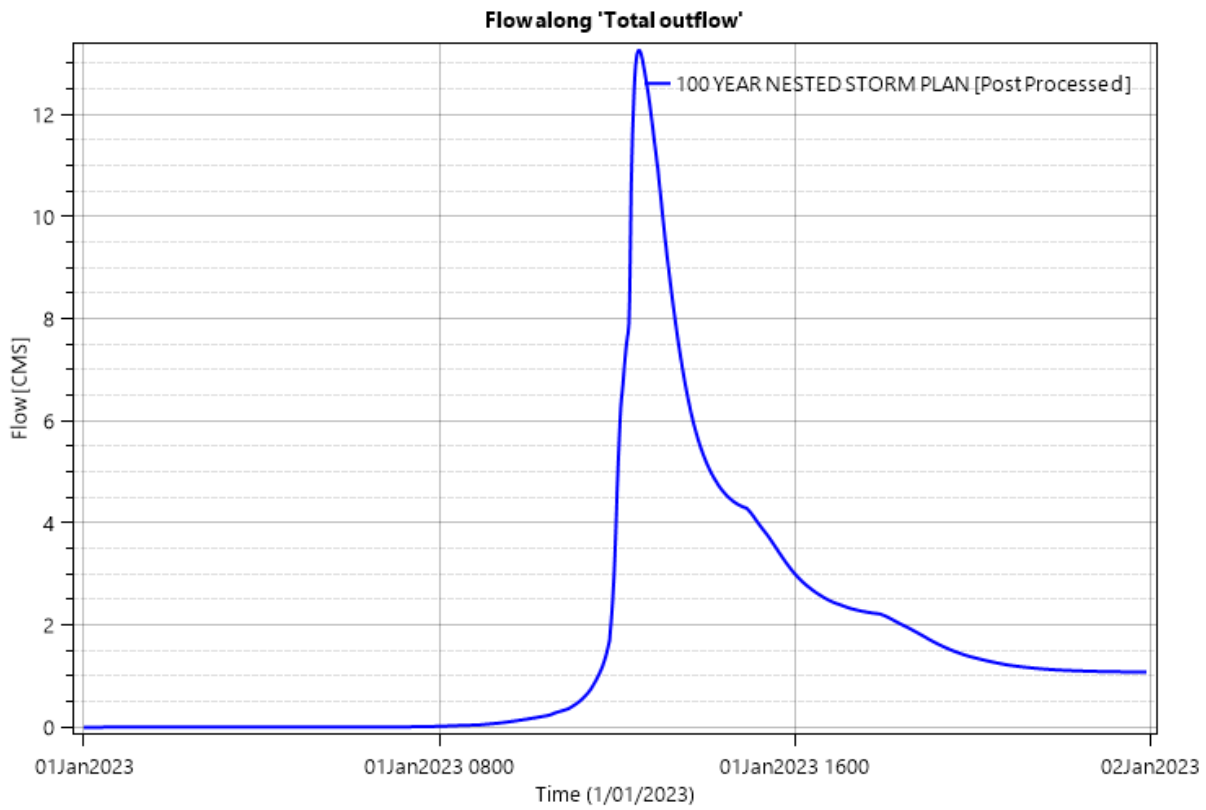


Fig 33 – Total pre dev outflow hydrograph

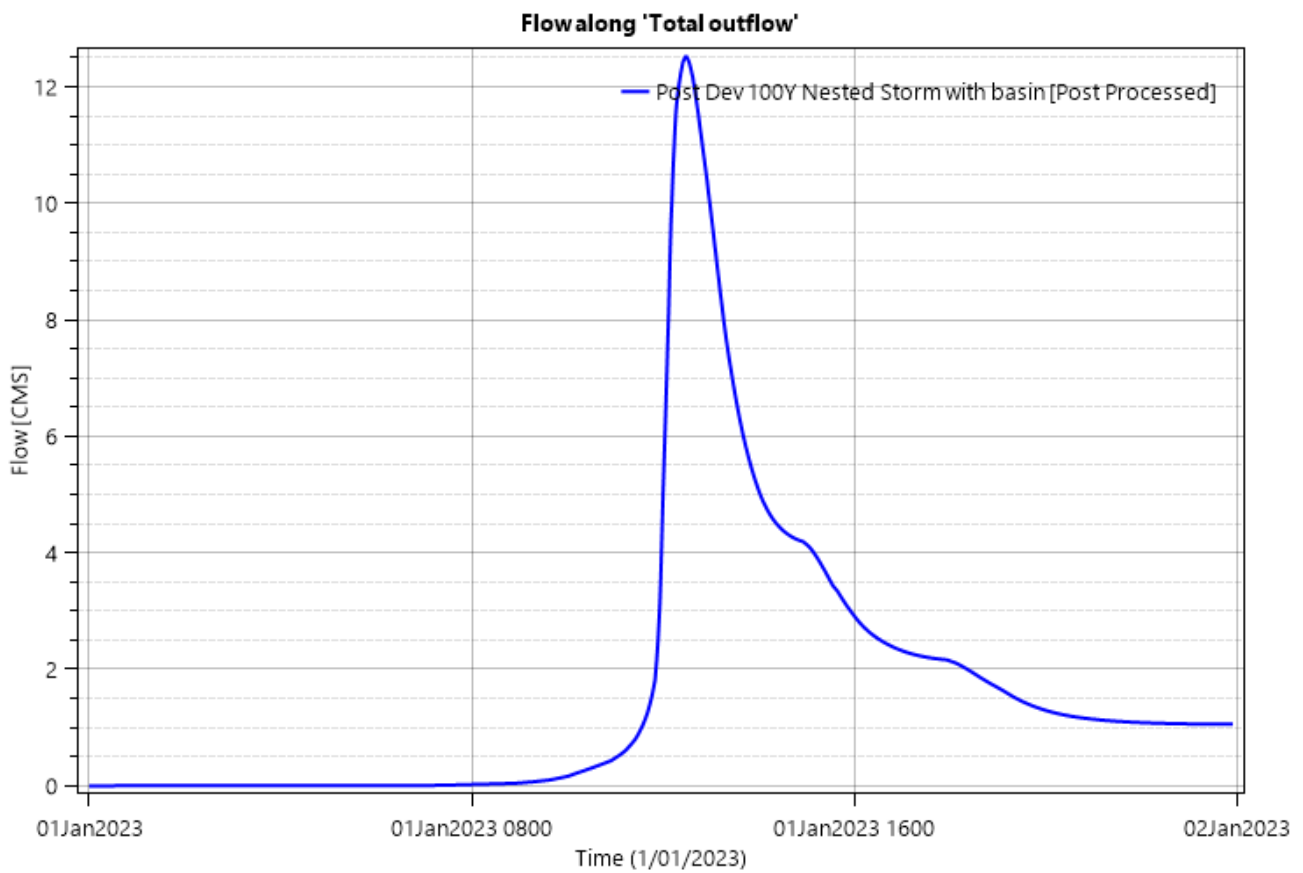


Fig 34 – Total post dev outflow hydrograph

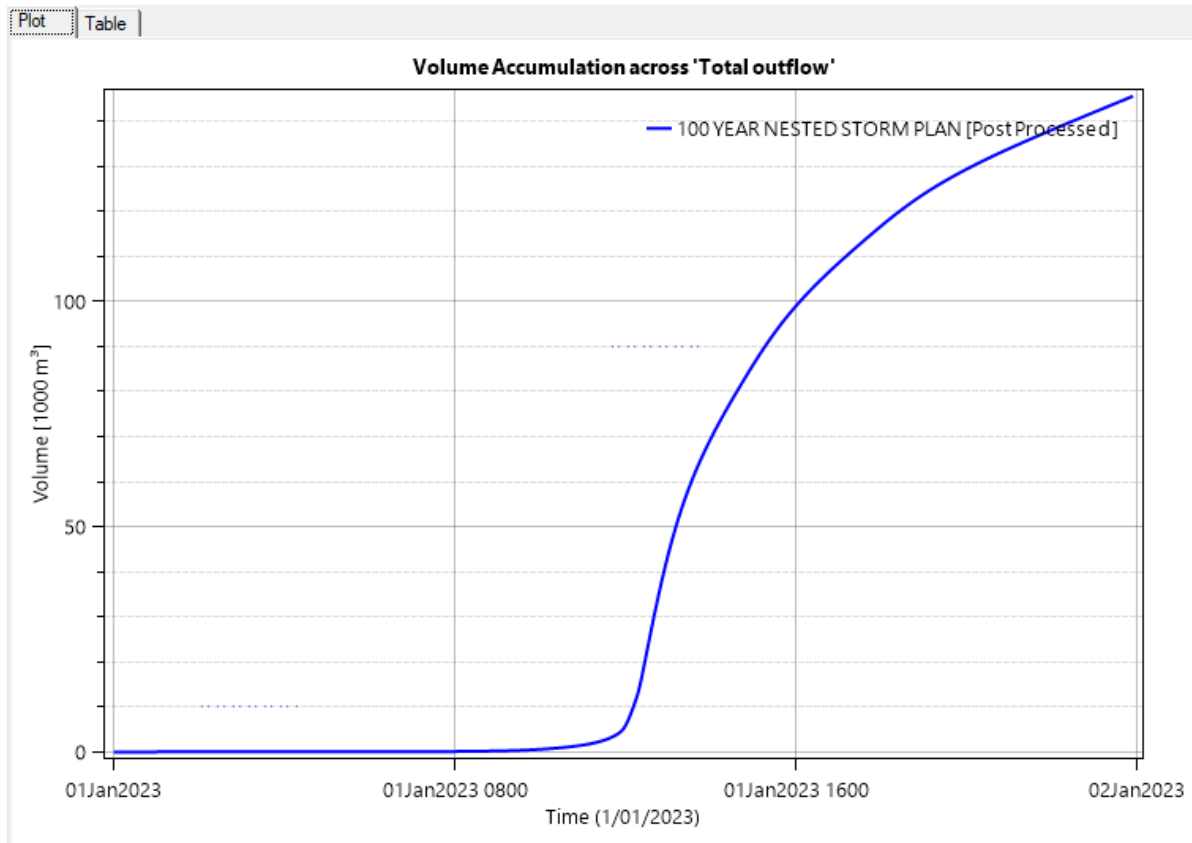


Fig 35 – Total predev outflow volume - 145.6m³ total flow

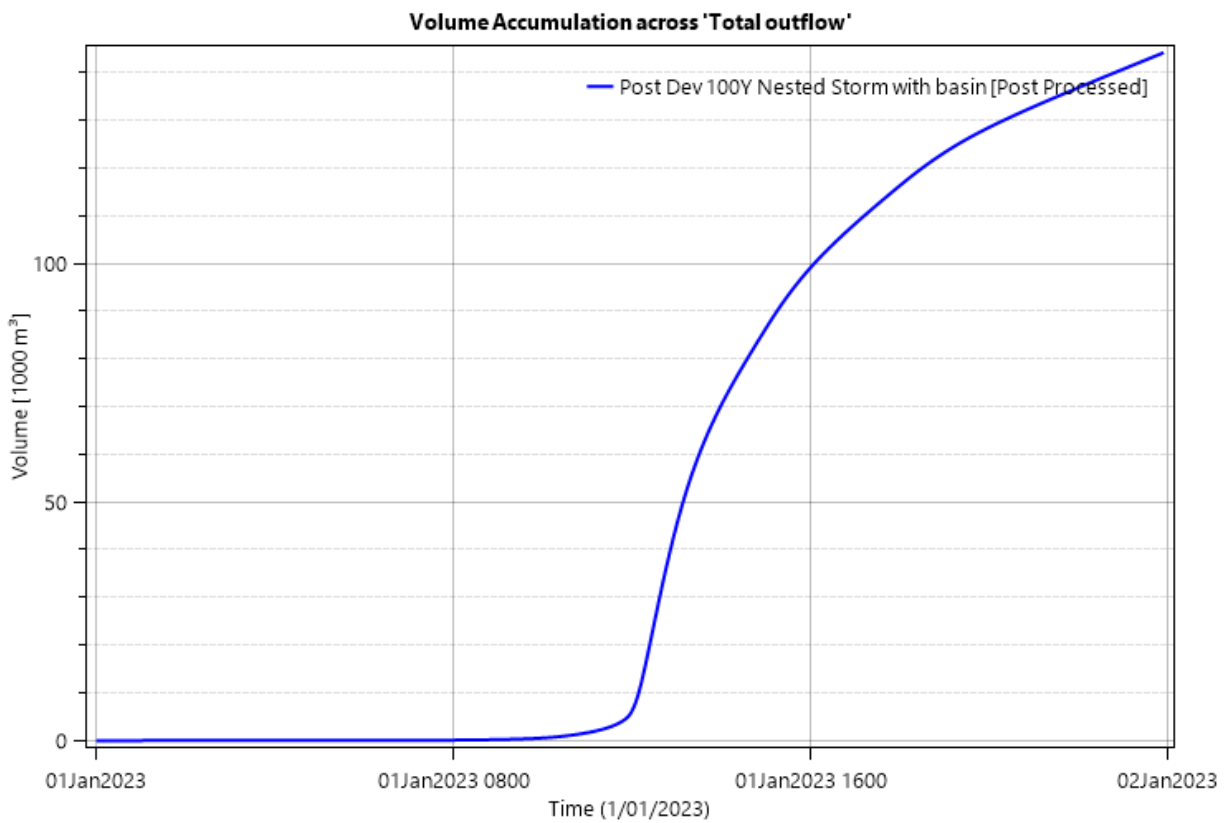


Fig 36 – Total post dev outflow volume – 144.2m³ total flow

5.8.2 HEC RAS OUTPUTS – CHANNEL FLOWS



Fig 37 – Channel 1 / section 1 location

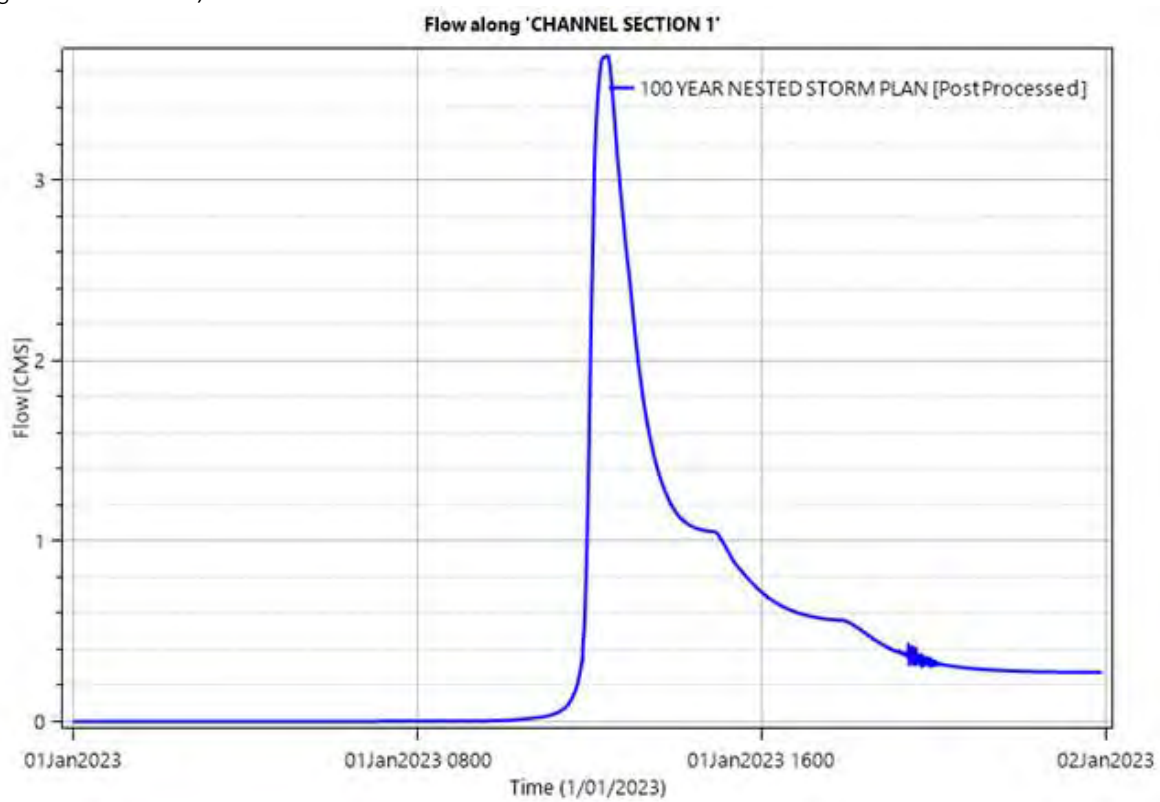


Fig 38 – Channel 1 / section 1 flow hydrograph

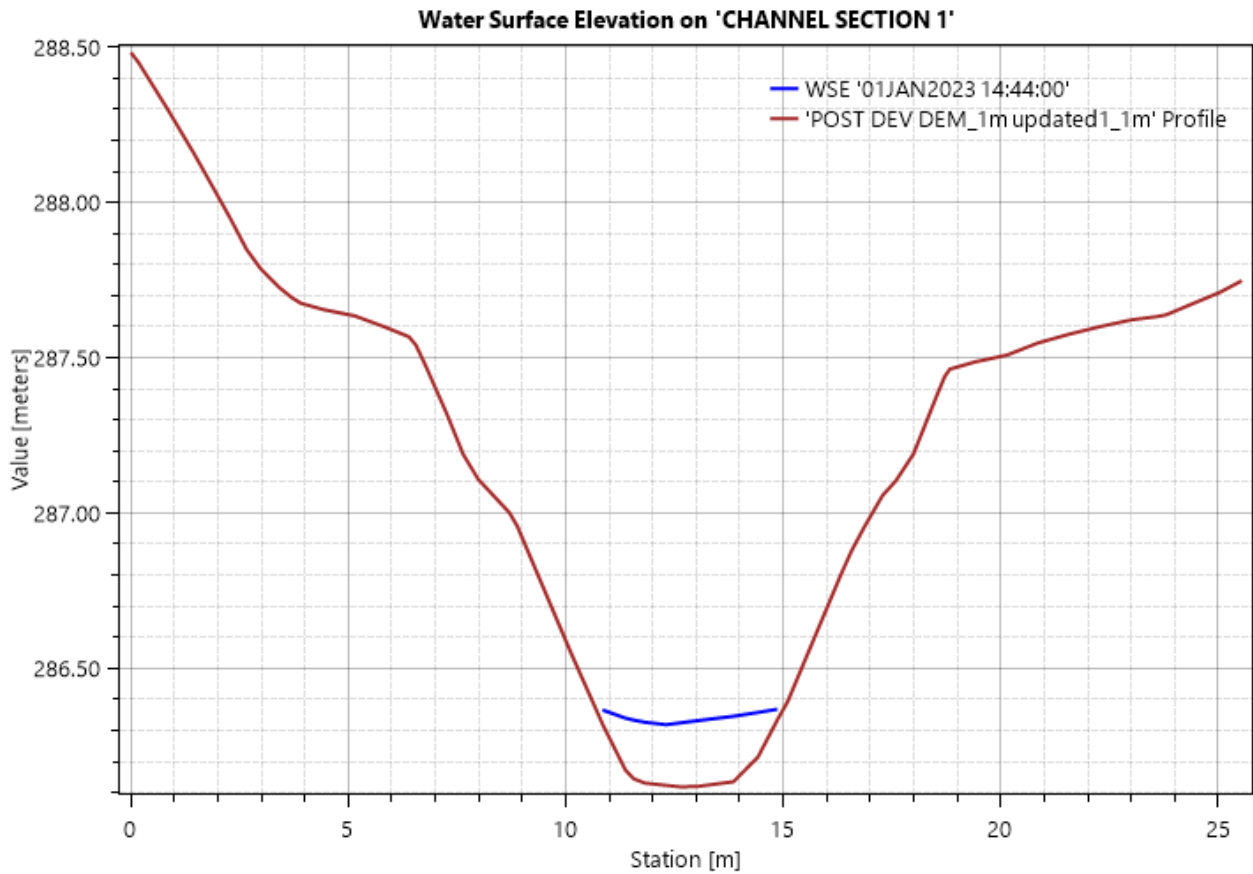


Fig 39 – Channel 1 / section 1 flow depth

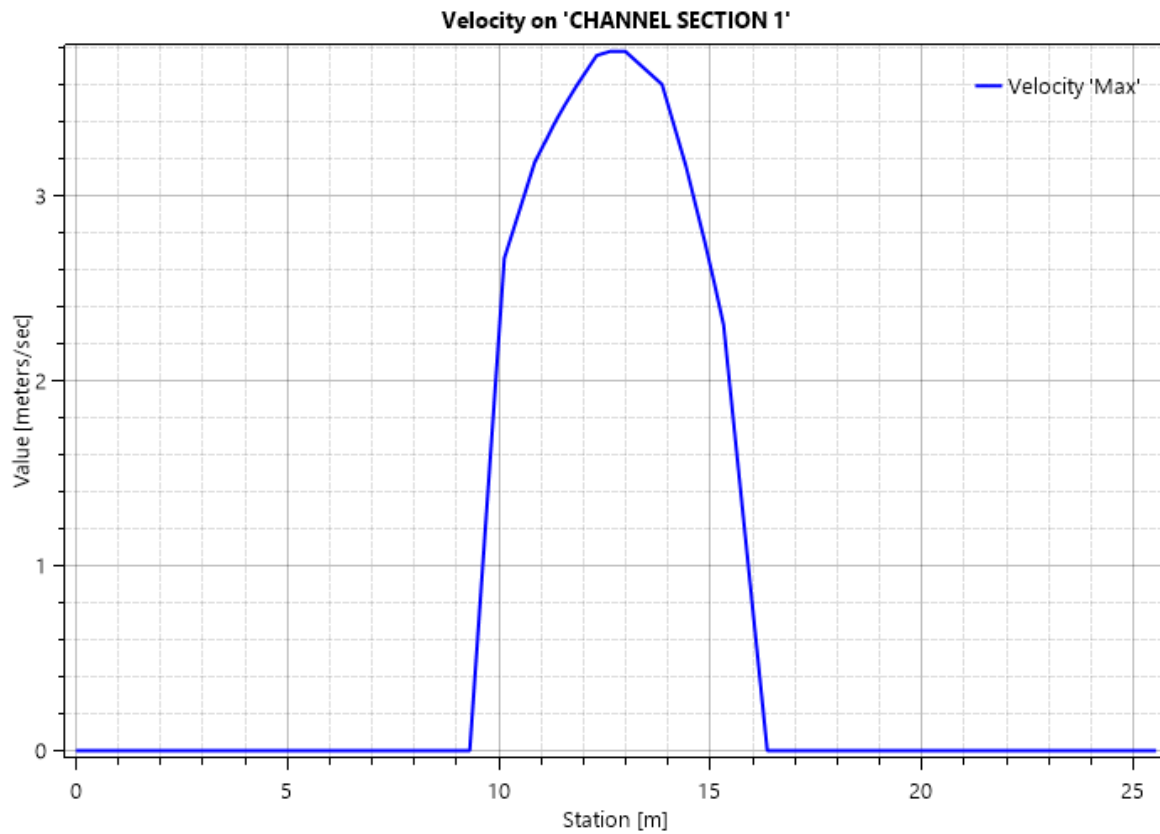


Fig 40 – Channel 1 / section 1 velocity

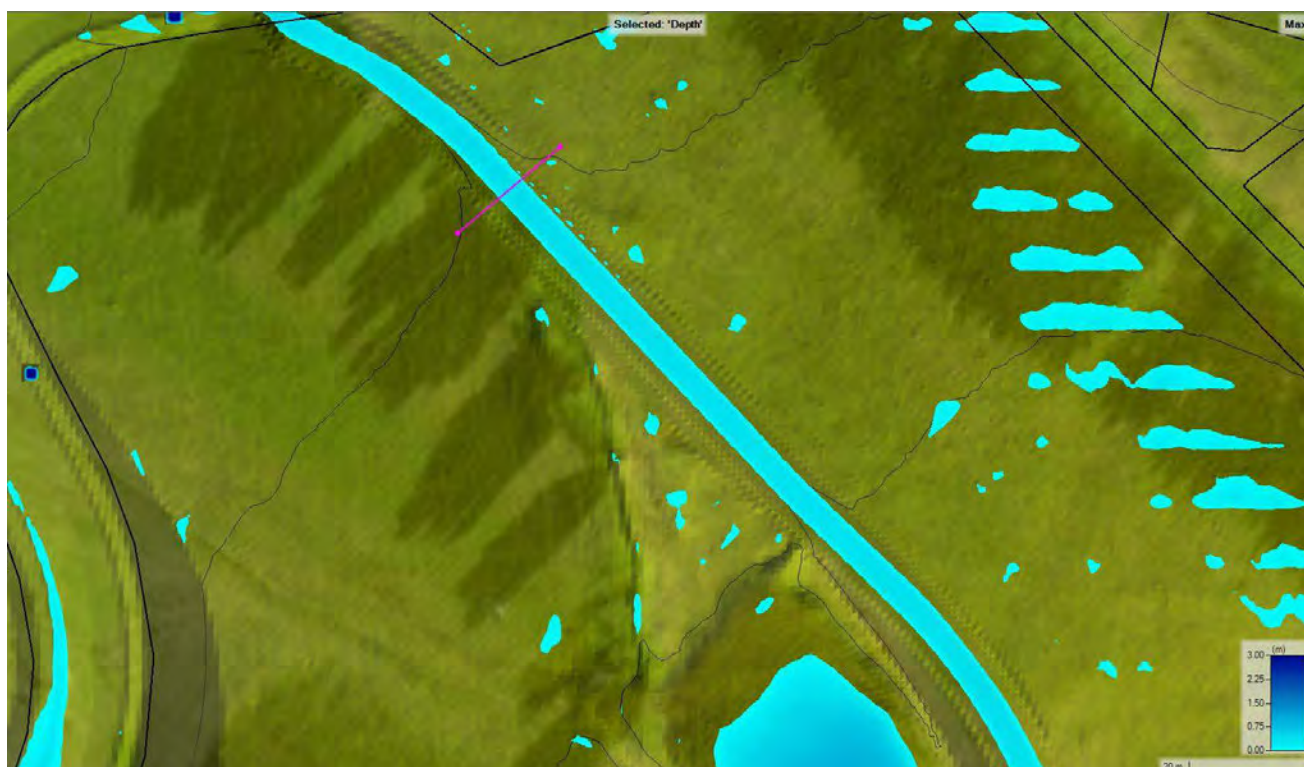


Fig 42 – Channel 1 / section 2 location

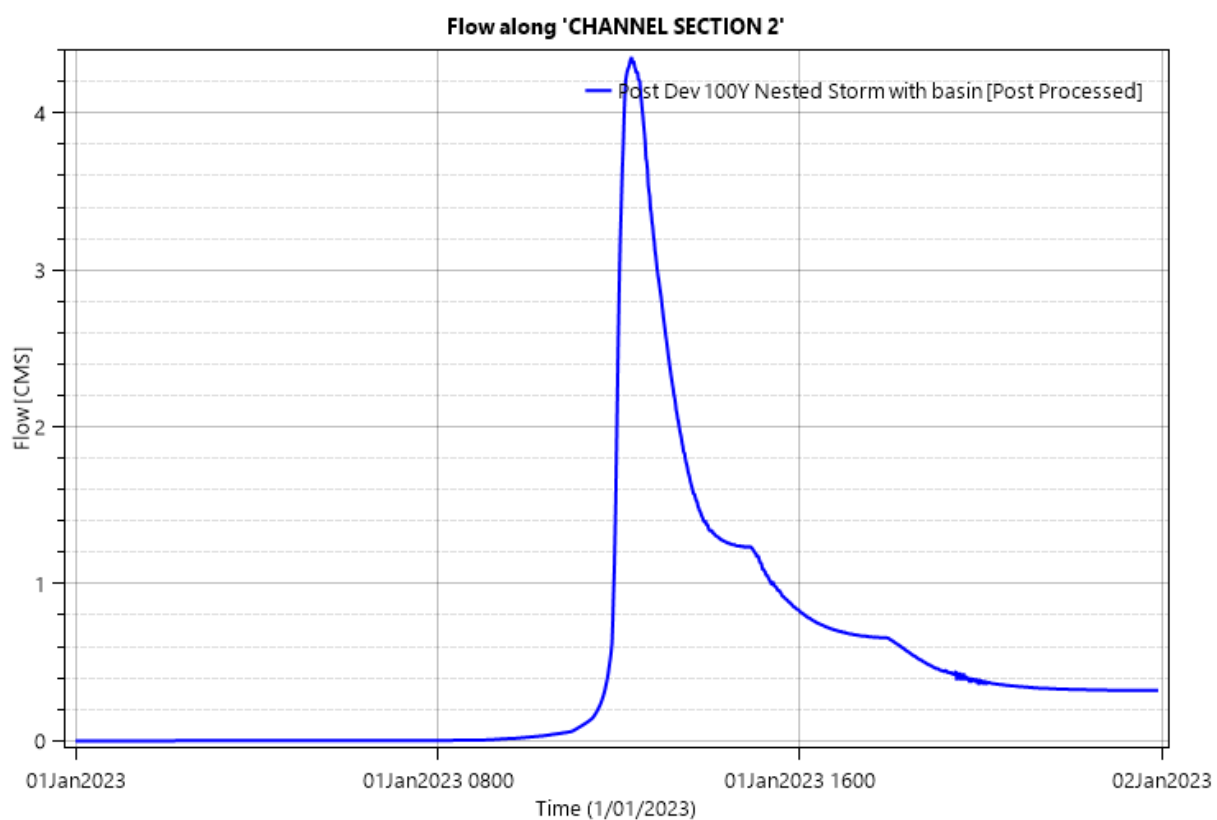


Fig 43 – Channel 1 / section 2 flow hydrograph

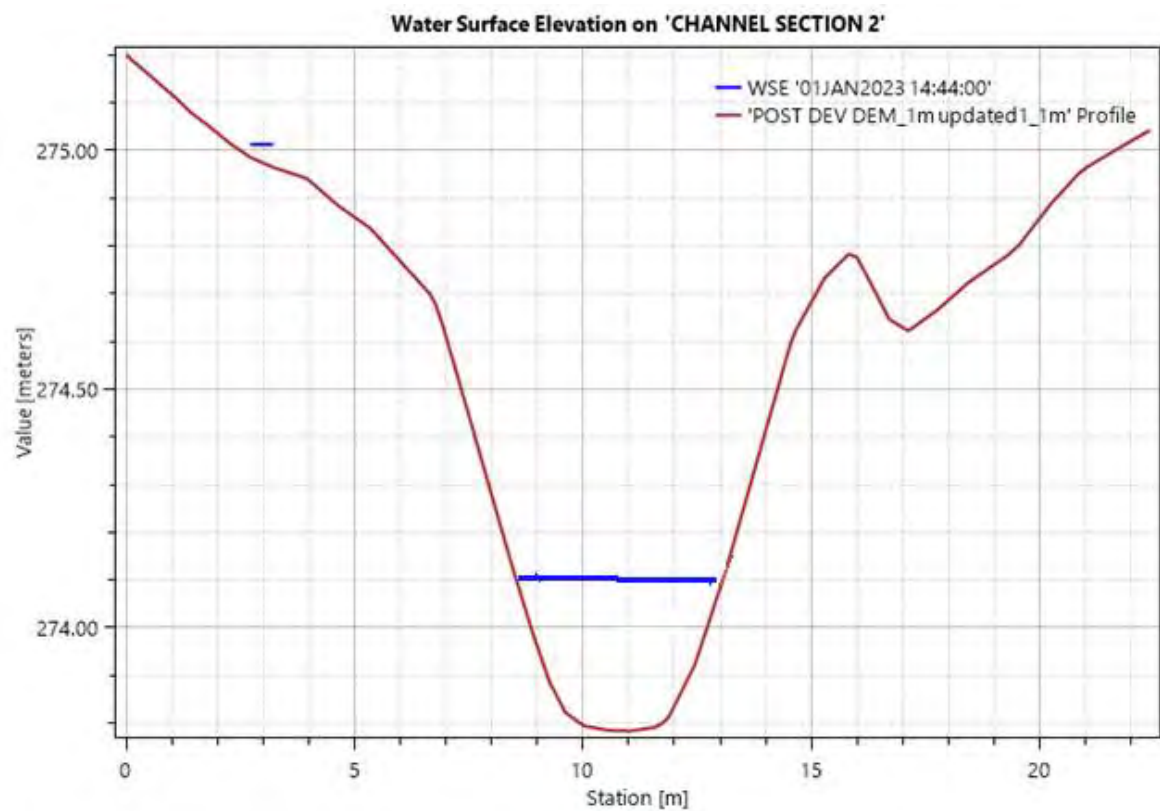


Fig 44 – Channel 1 / section 2 flow depth

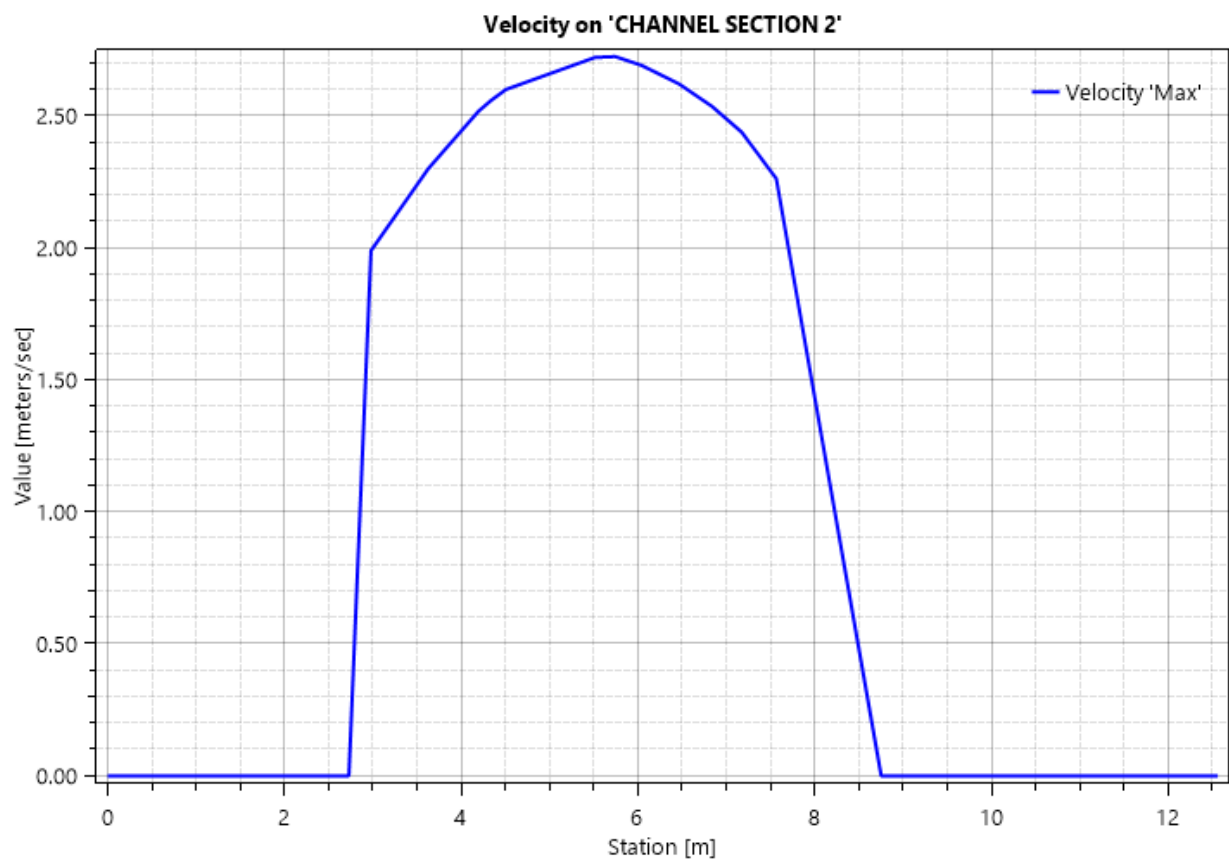


Fig 45 – Channel 1 / section 2 velocity

Fig

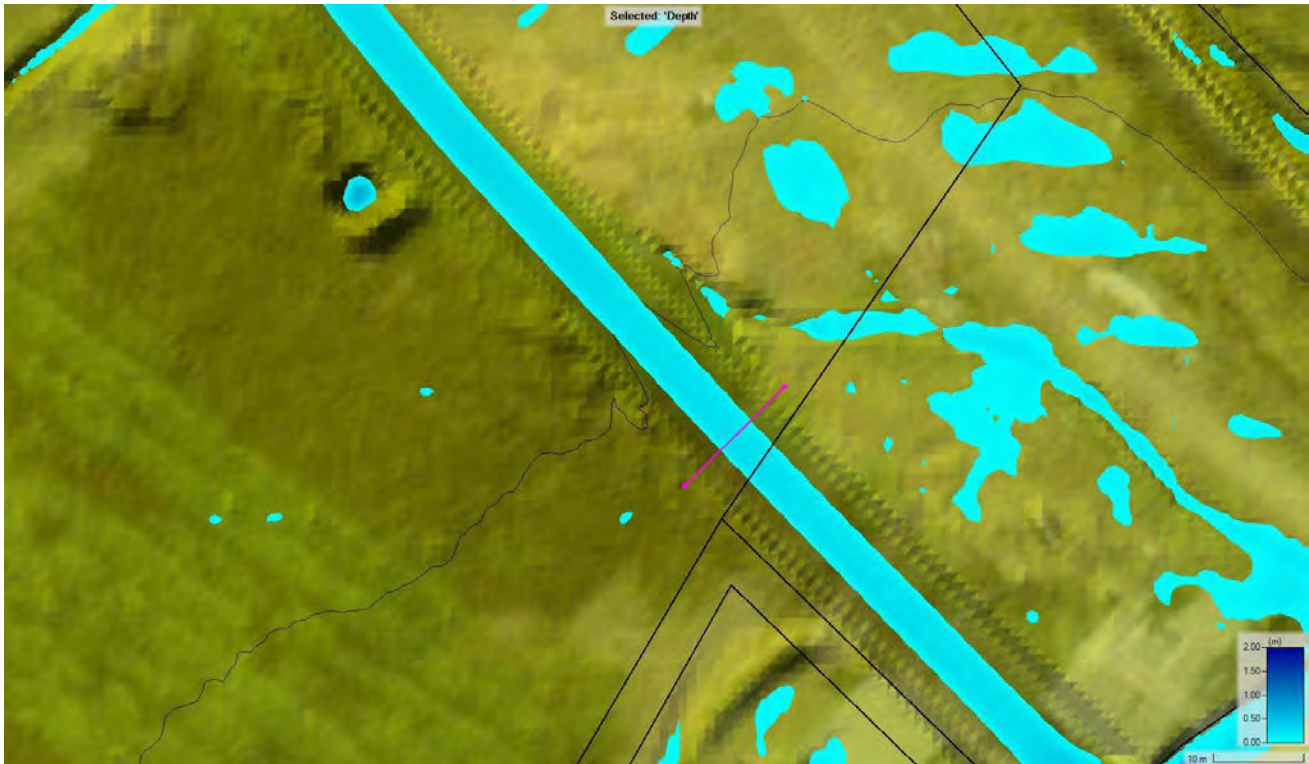


Fig 46 – Channel 1 / section 3 location

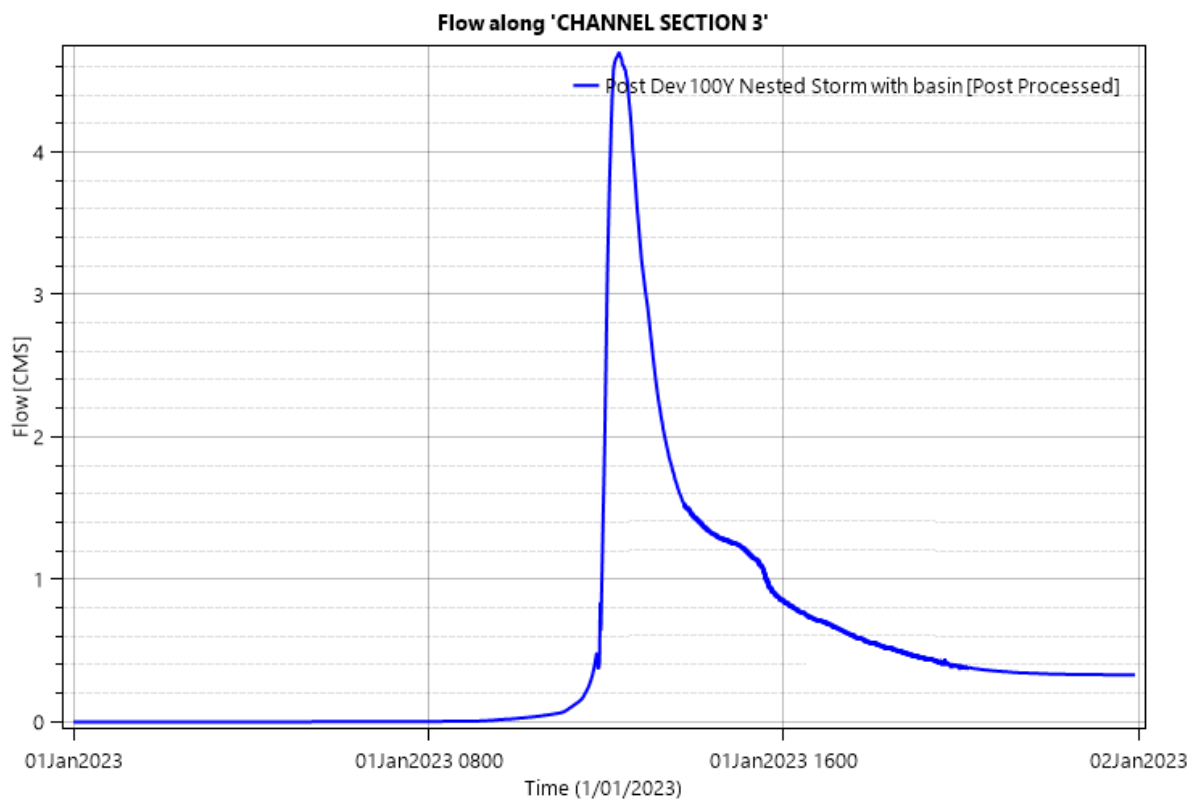


Fig 47 – Channel 1 / section 3 flow

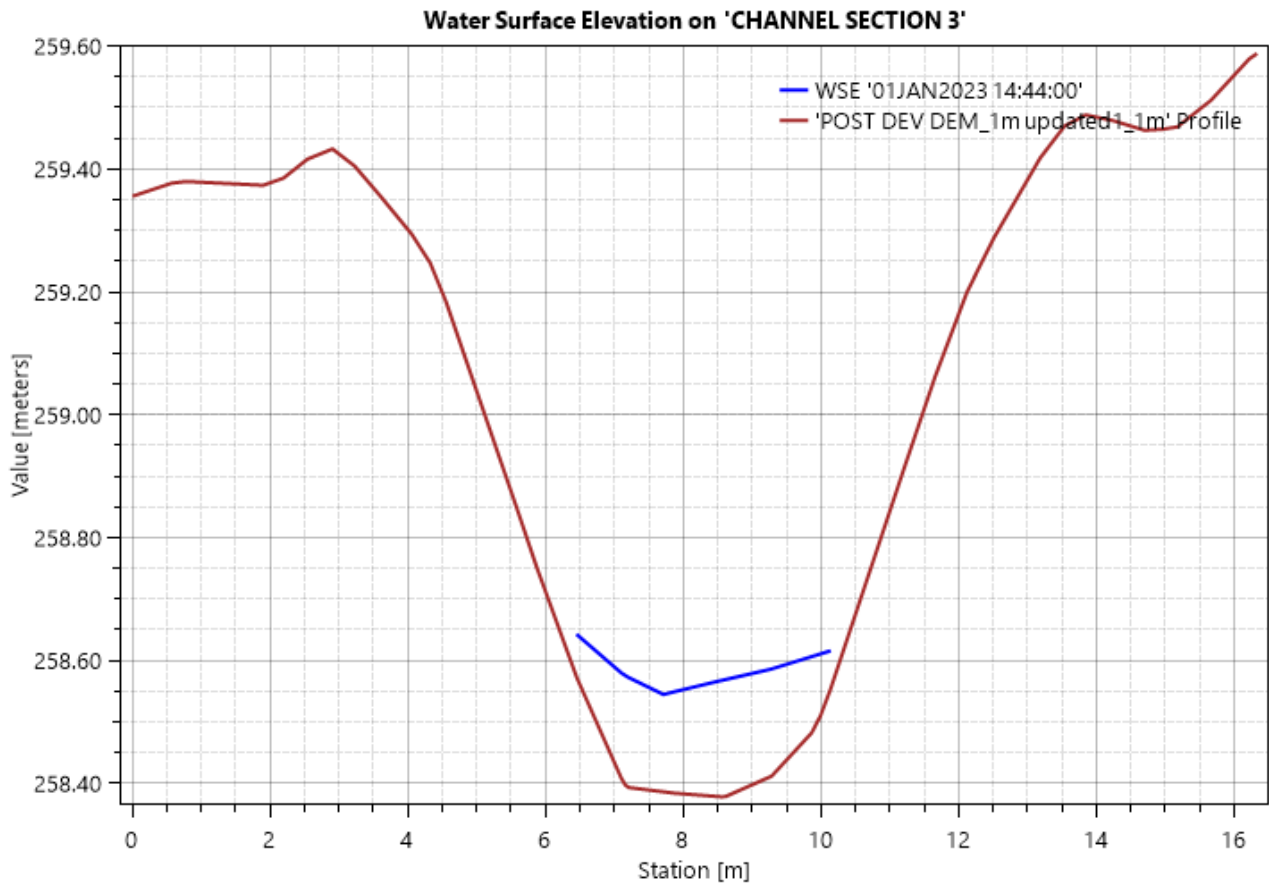


Fig 48 – Channel 1 / section 3 flow depth

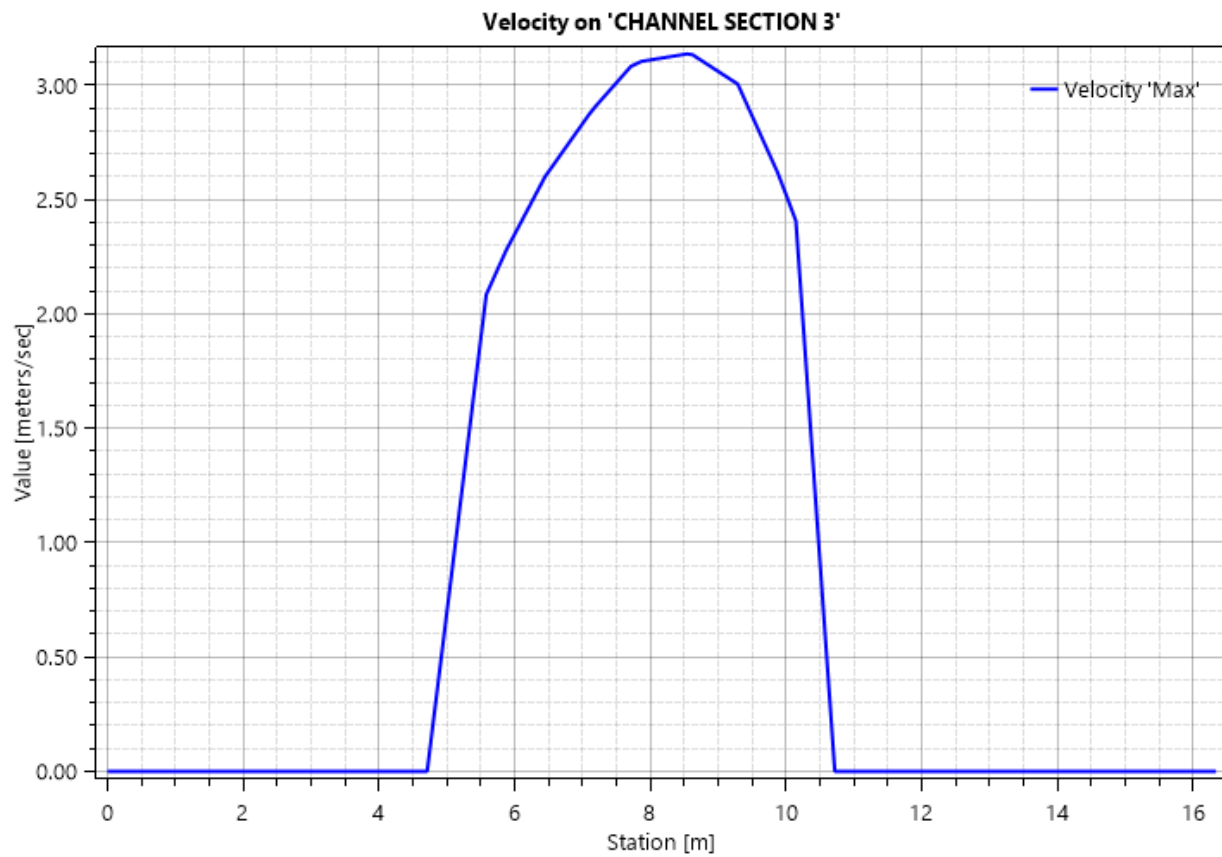


Fig 49 – Channel 1 / section 3 flow velocity



Fig 50 – Channel 1 / section 4 location

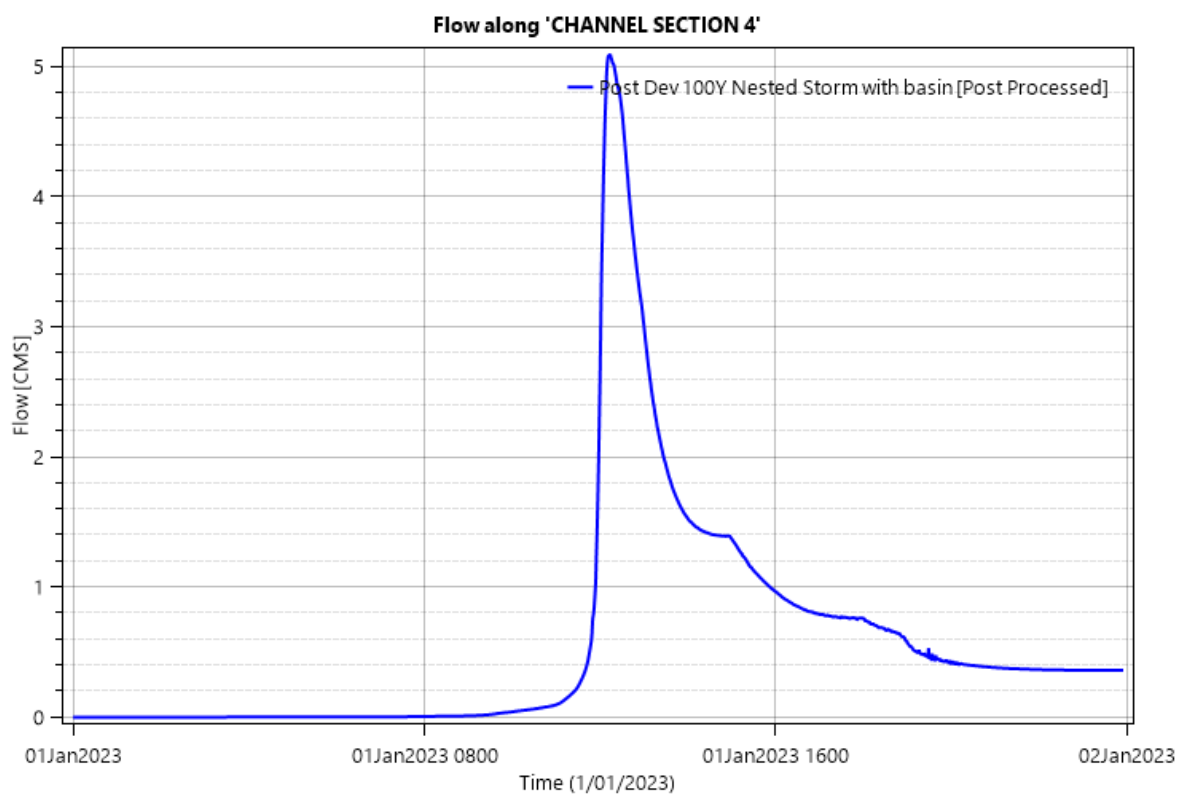


Fig 52 – Channel 1 / section 3 flow

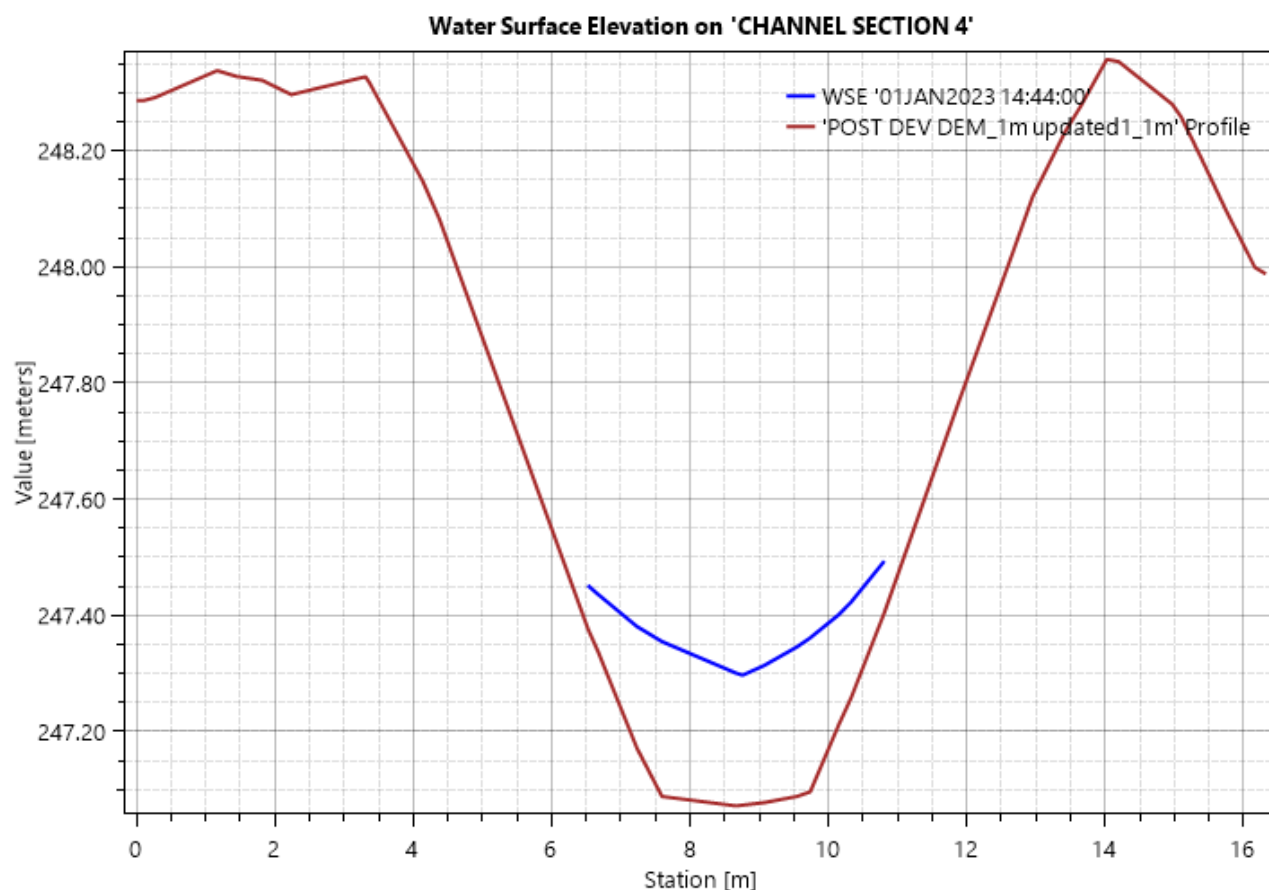


Fig 54 – Channel 1 / section 3 flow DEPTH

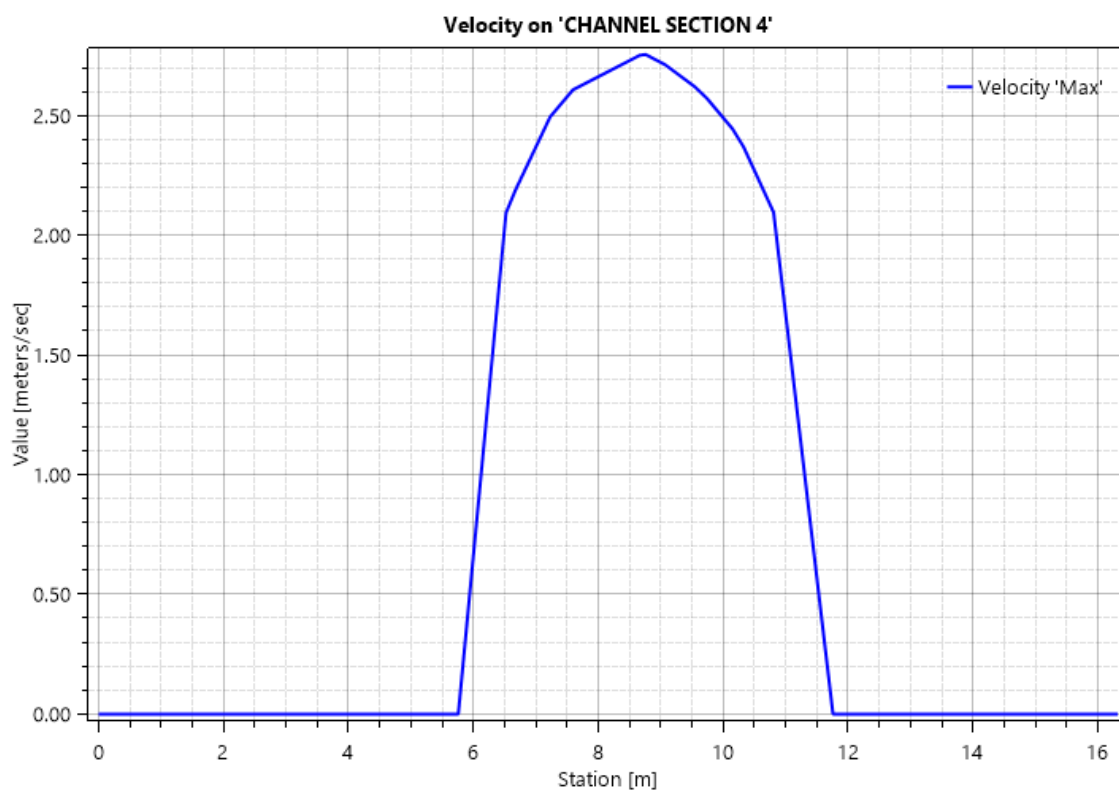


Fig 55 – Channel 1 / section 3 flow velocity

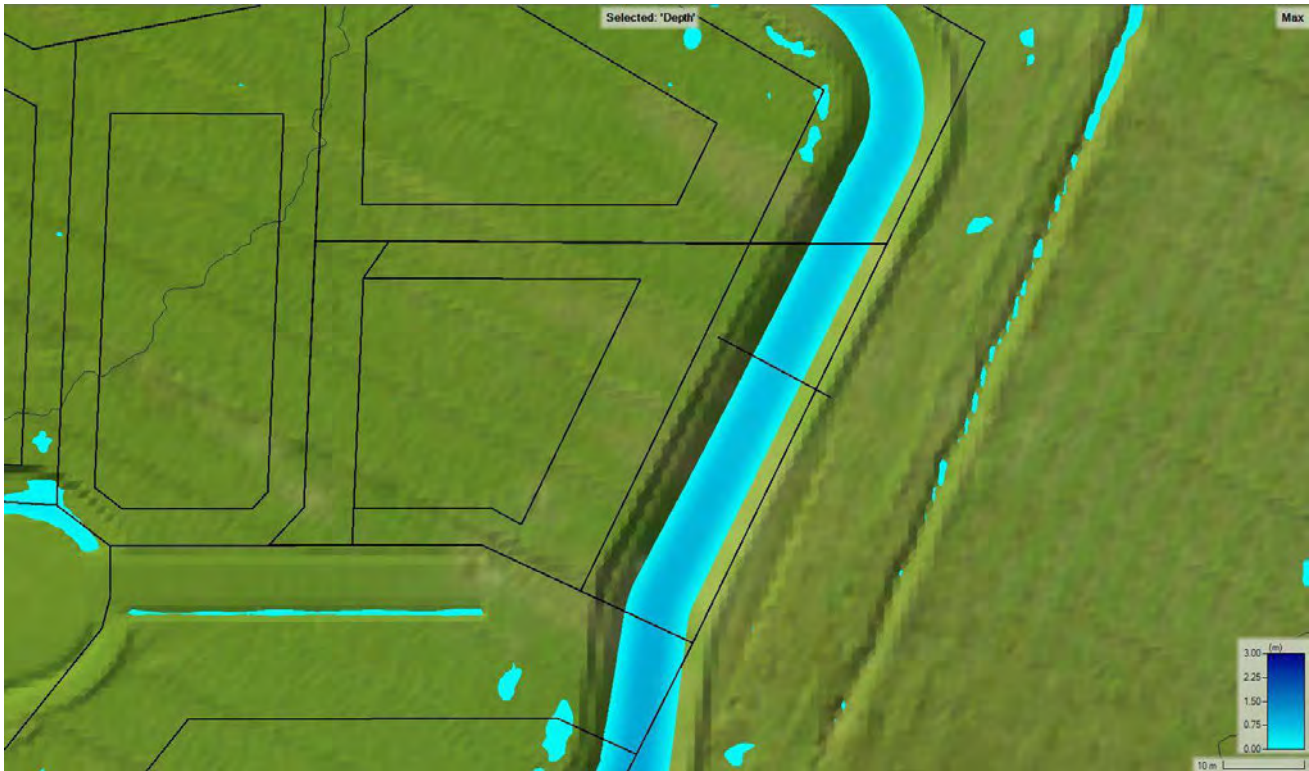


Fig 56 – Channel 1 / section 5 location

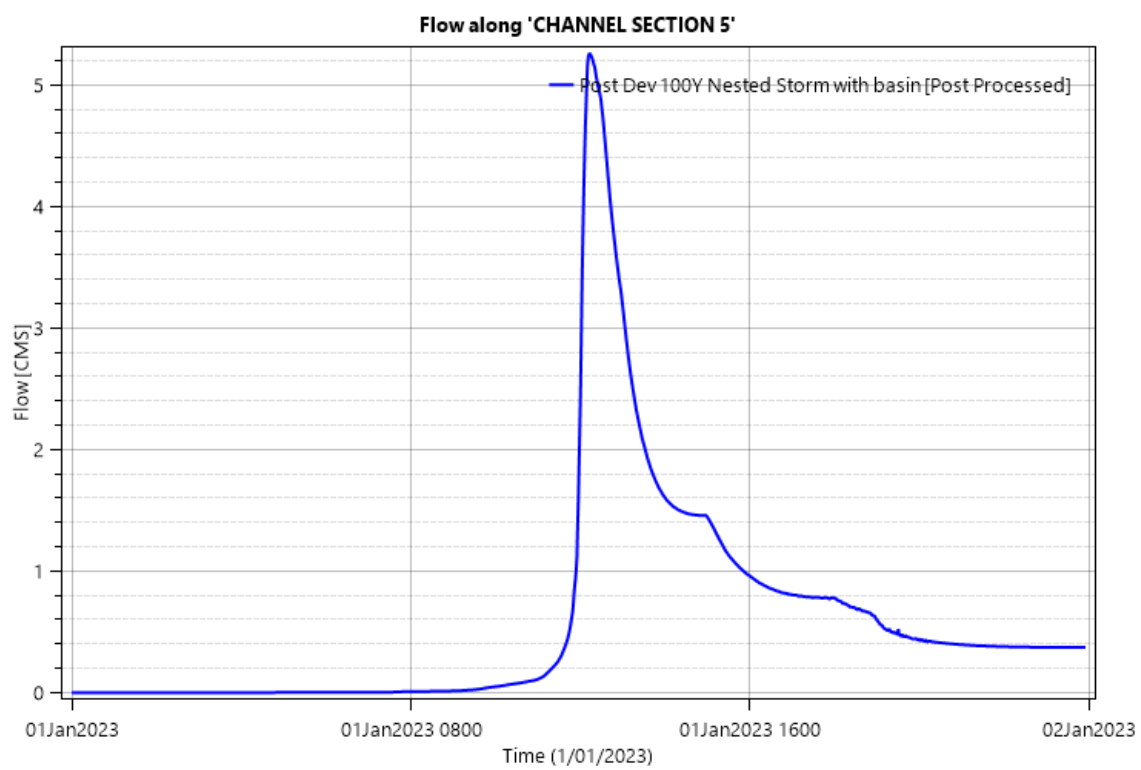


Fig 57 – Channel 1 / section 5 flow hydrograph

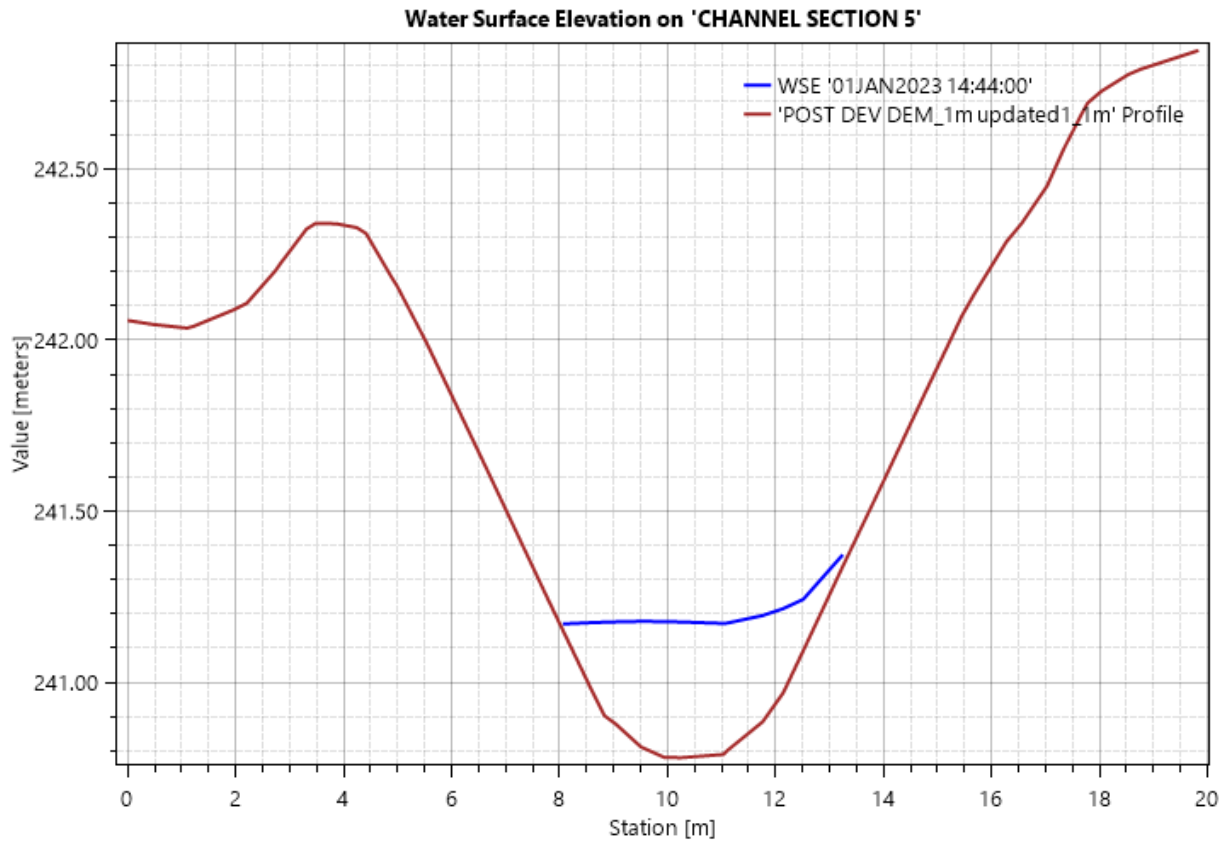


Fig 58 – Channel 1 / section 5 flow depth

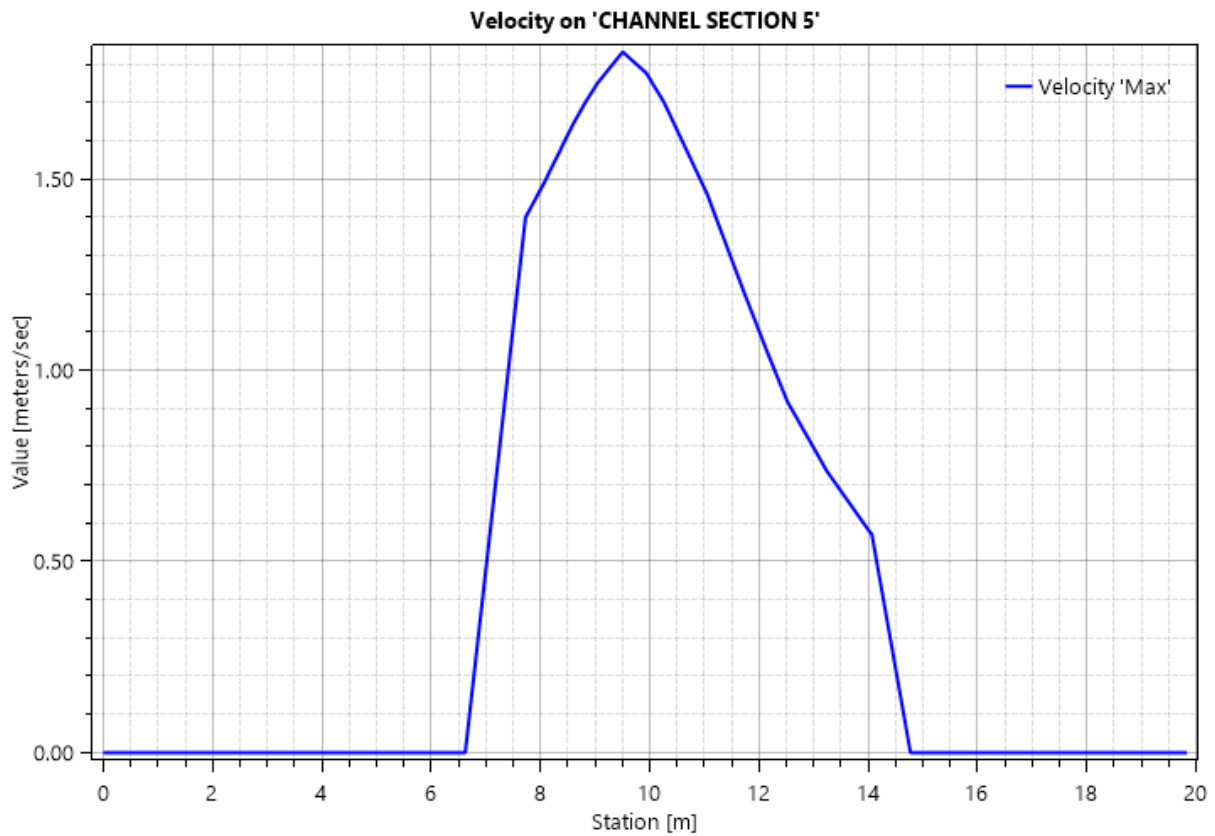


Fig 59 – Channel 1 / section 5 flow velocity

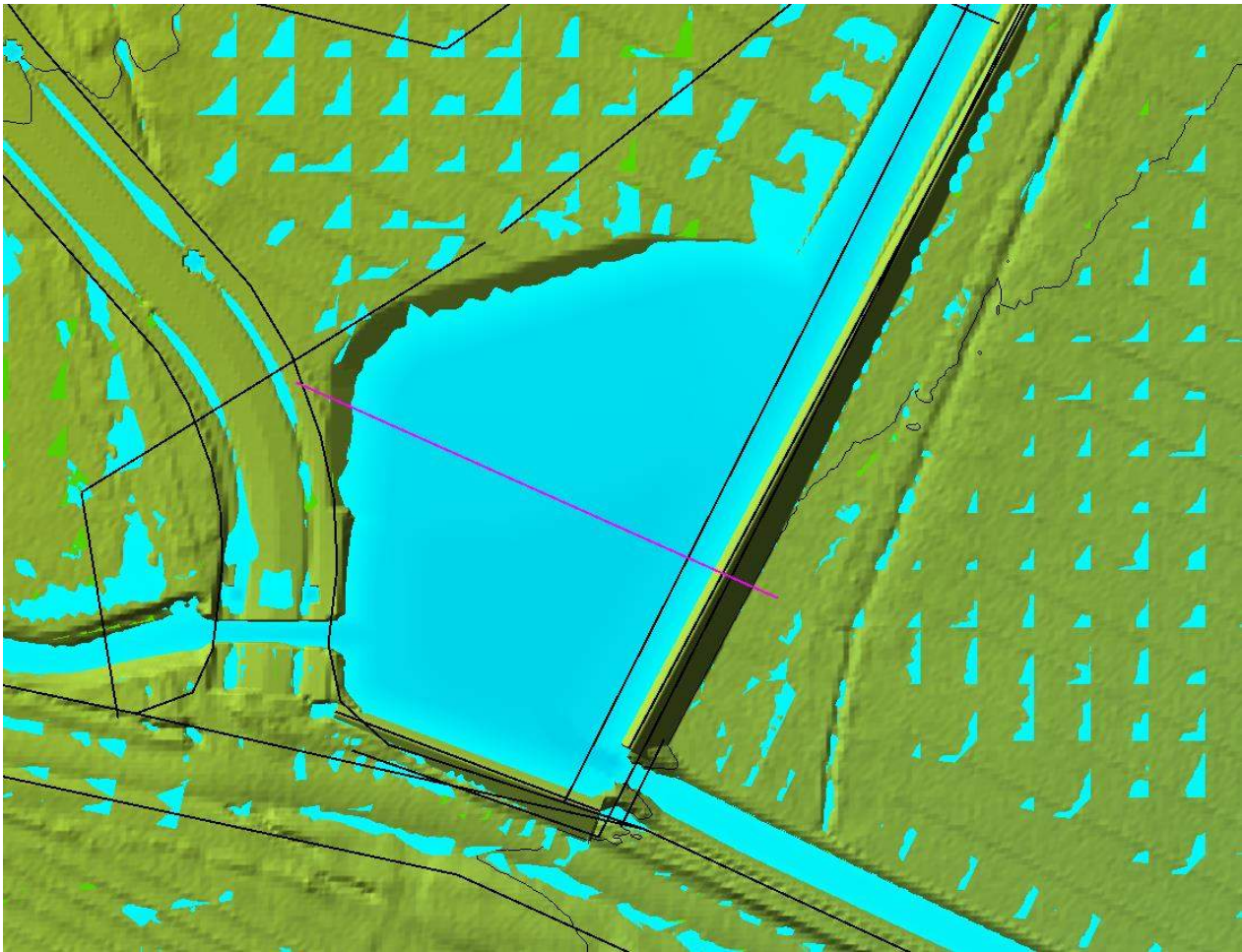


Fig 60 – basin / section 6 location

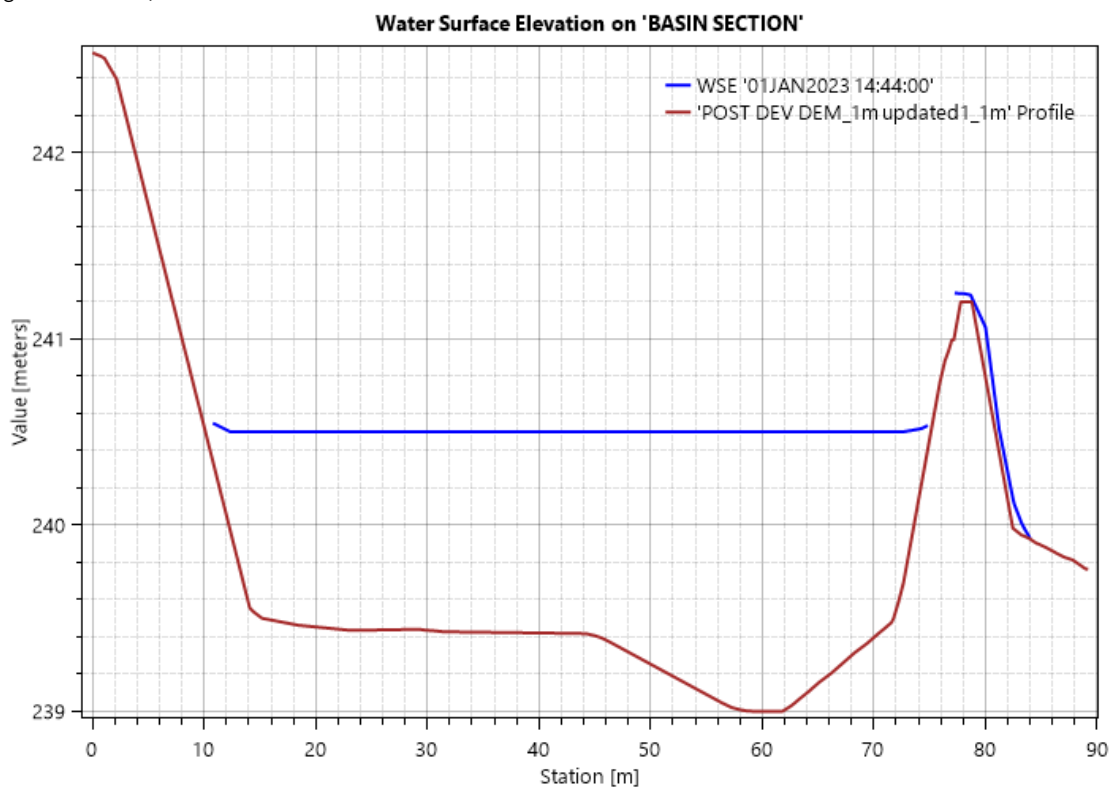


Fig 61 – basin / section 6 max water depth



Fig 62 – Channel 2 / section 7 location

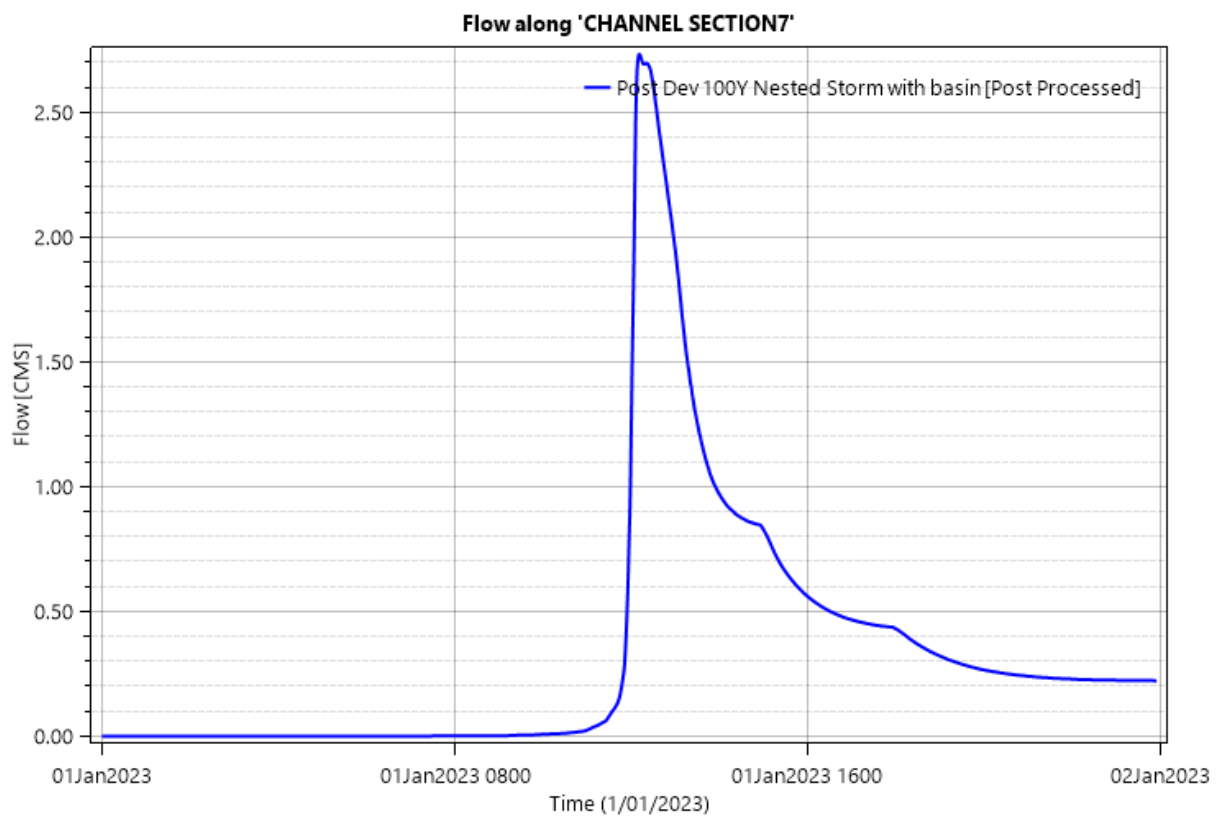


Fig 63 – Channel 2 / section 7 flow hydrograph

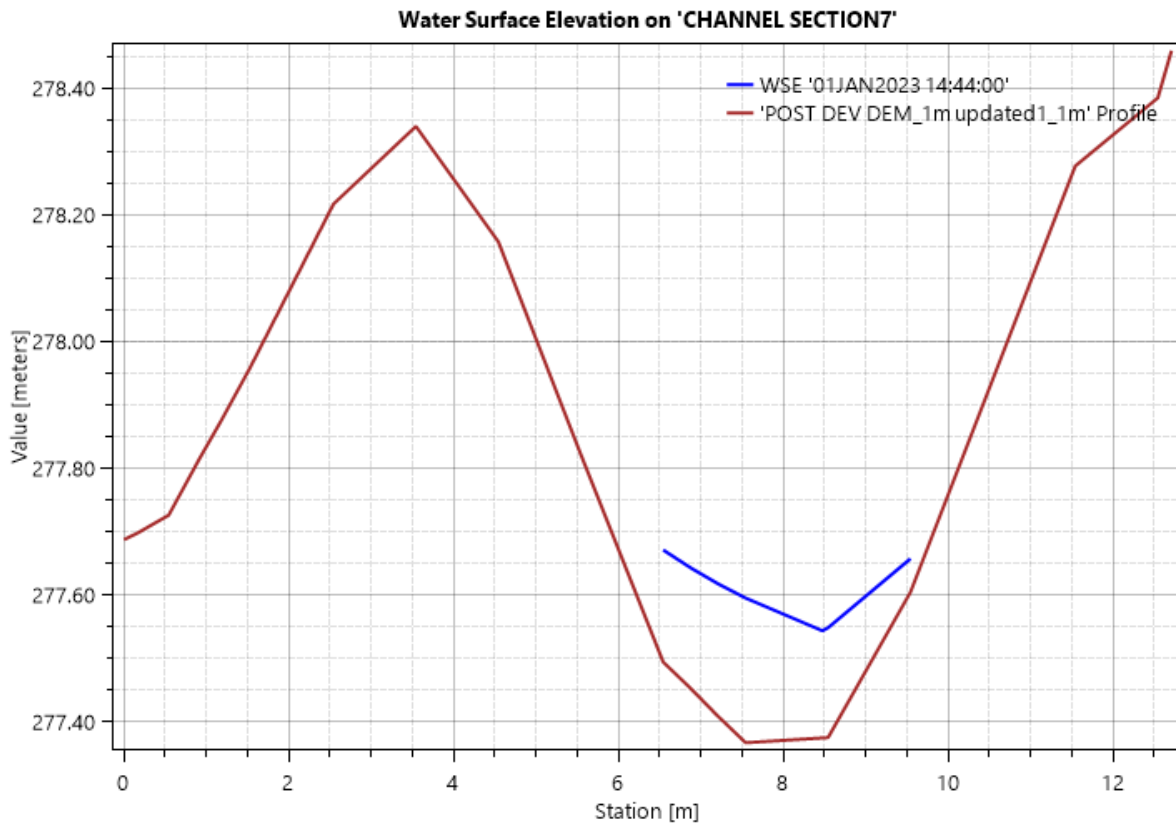


Fig 64 – Channel 2 / section 7 flow depth

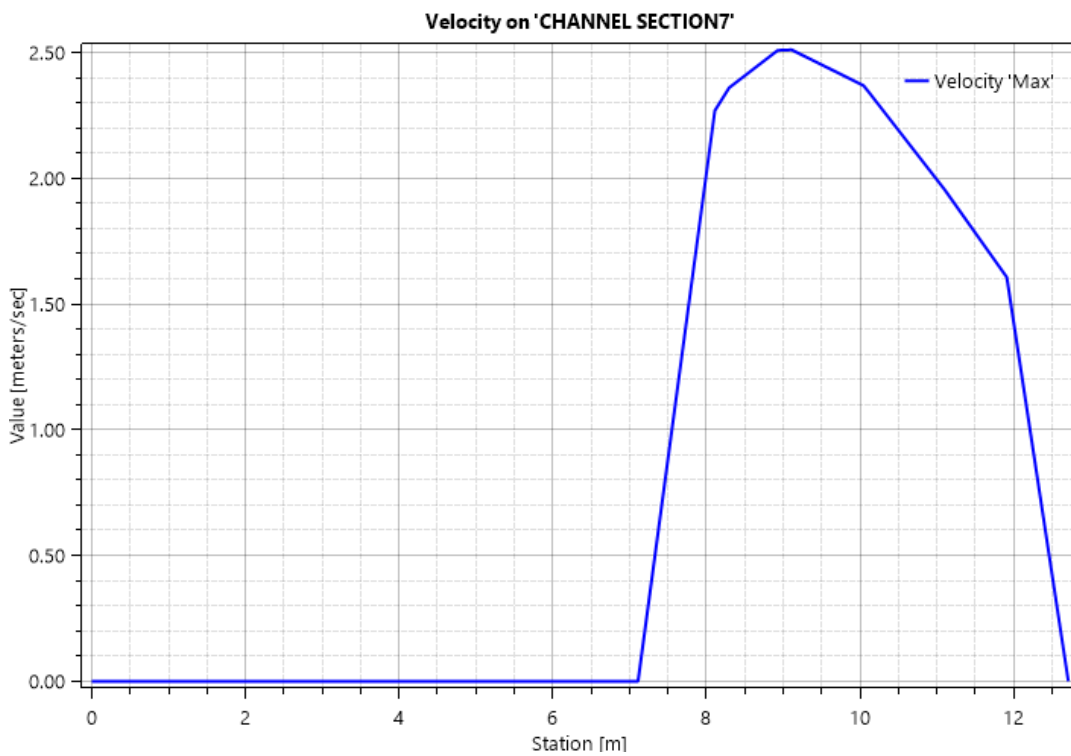


Fig 65 – Channel 2 / section 7 flow velocity



Fig 66 – Channel 2 / section 8 location

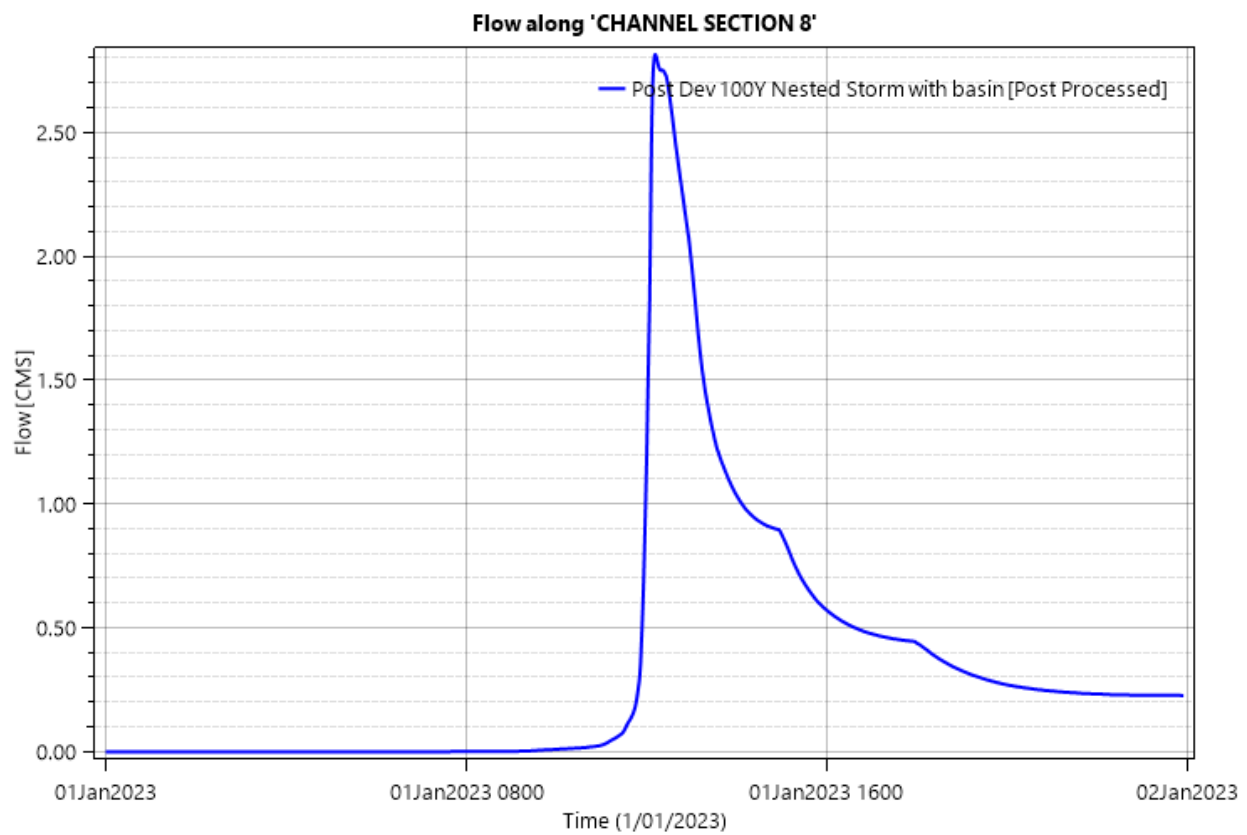


Fig 67 – Channel 2 / section 8 flow hydrograph

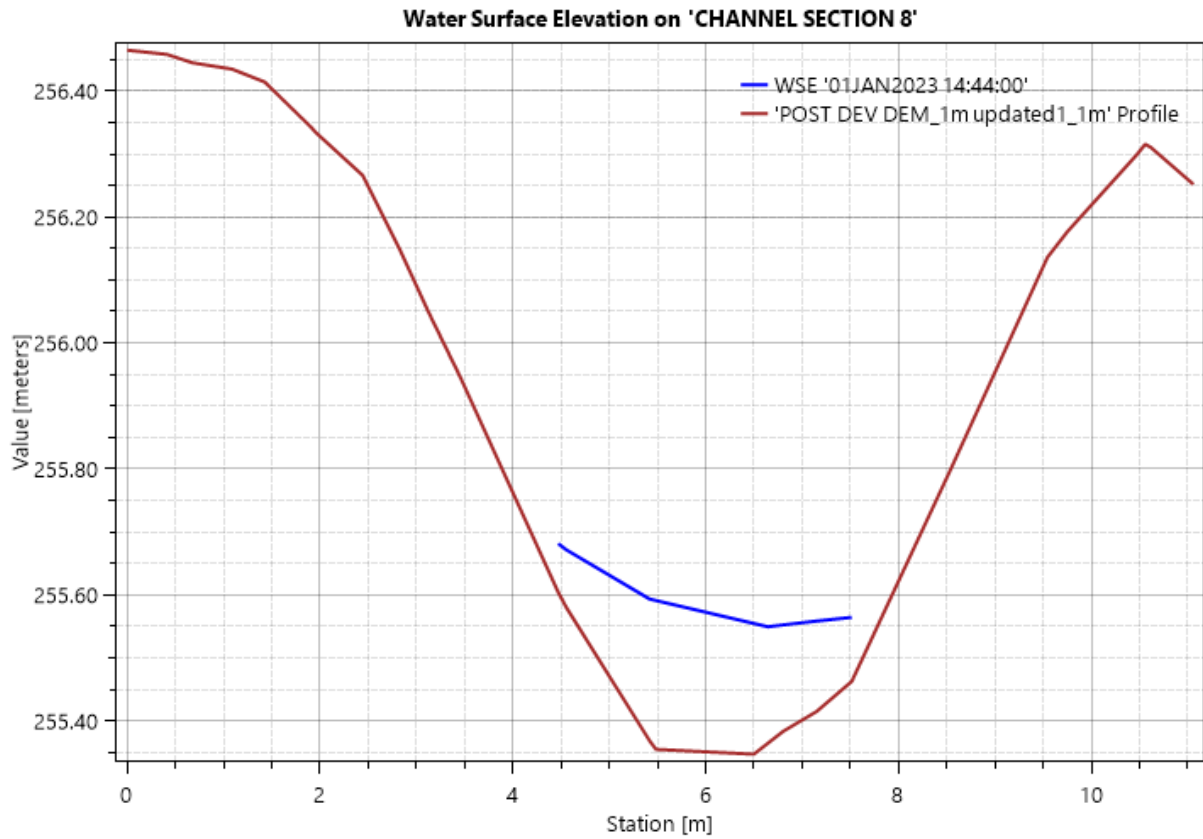


Fig 68 – Channel 2 / section 8 flow depth

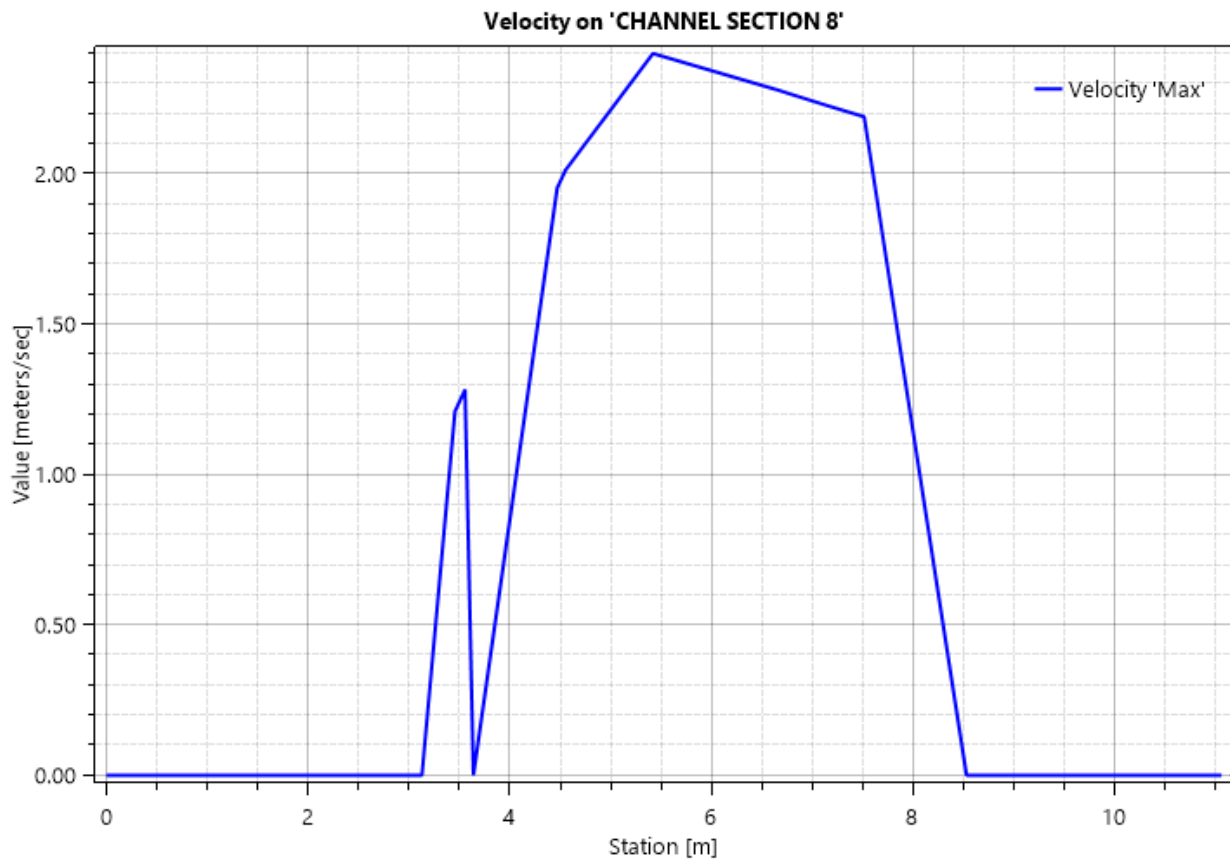


Fig 69 – Channel 2 / section 8 flow velocity

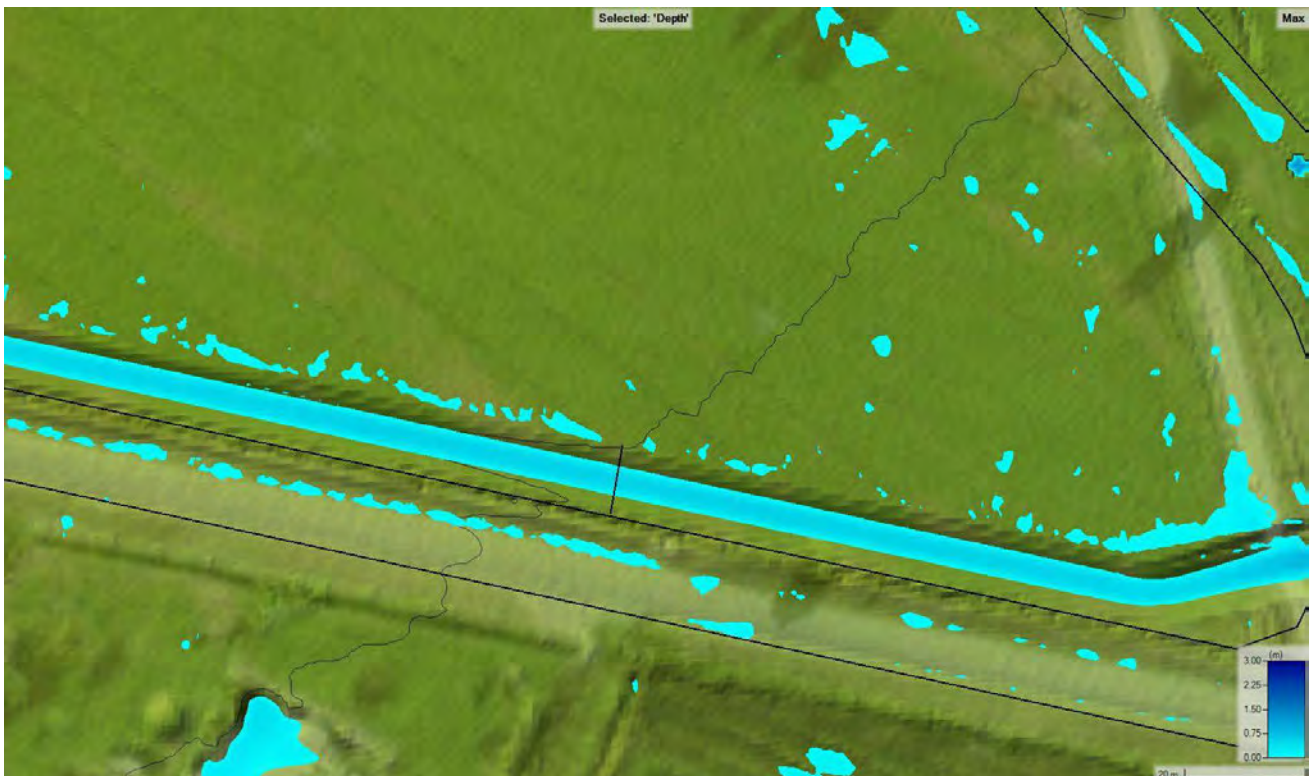


Fig 70 – Channel 2 / section 9 location

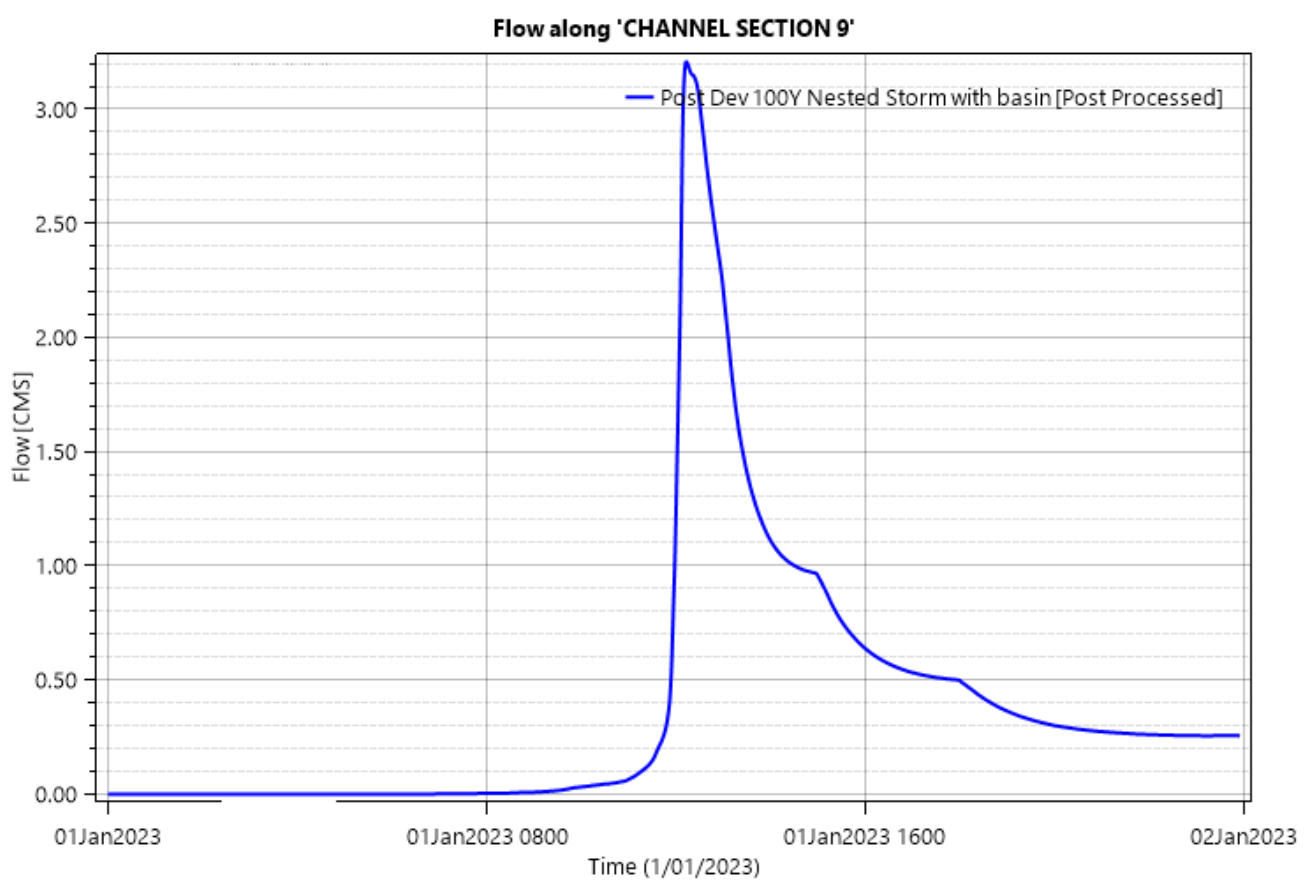


Fig 71 – Channel 2 / section 9 flow hydrograph

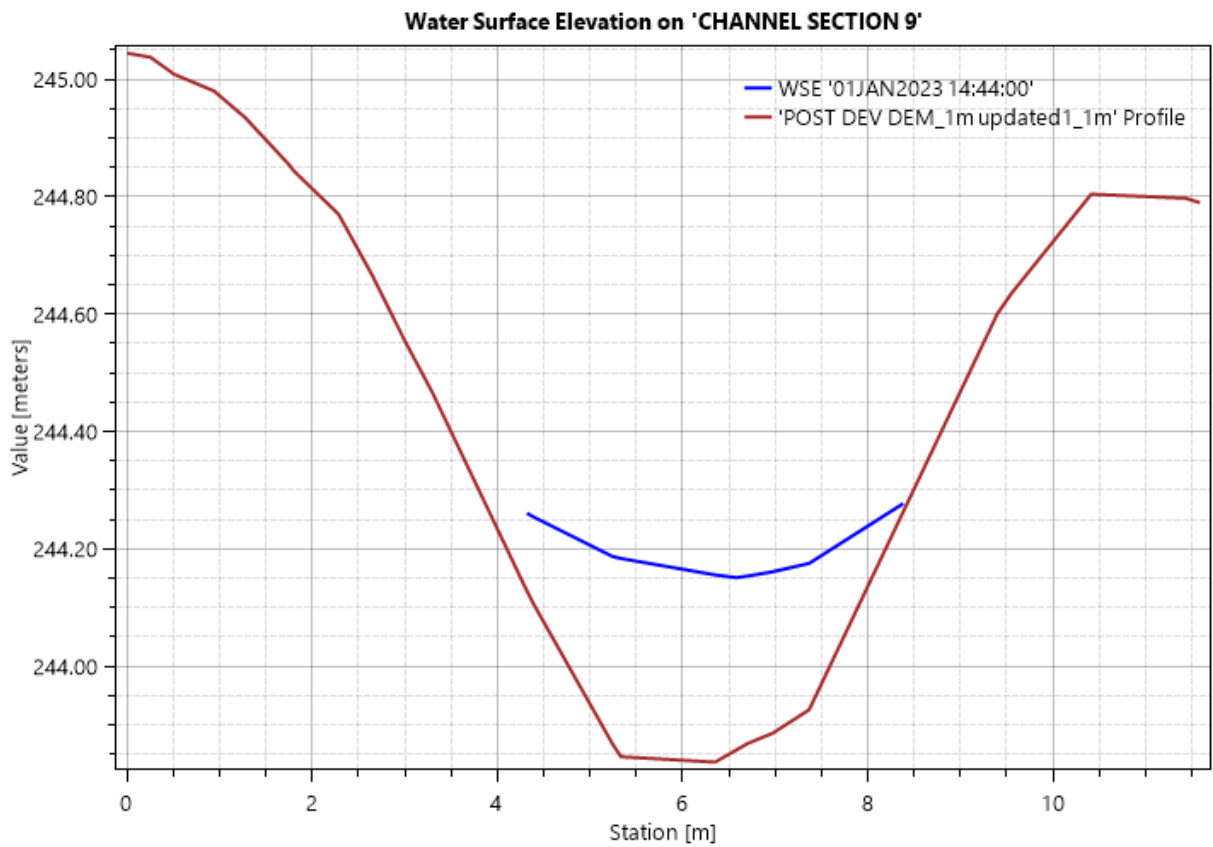


Fig 72 – Channel 2 / section 9 flow depth

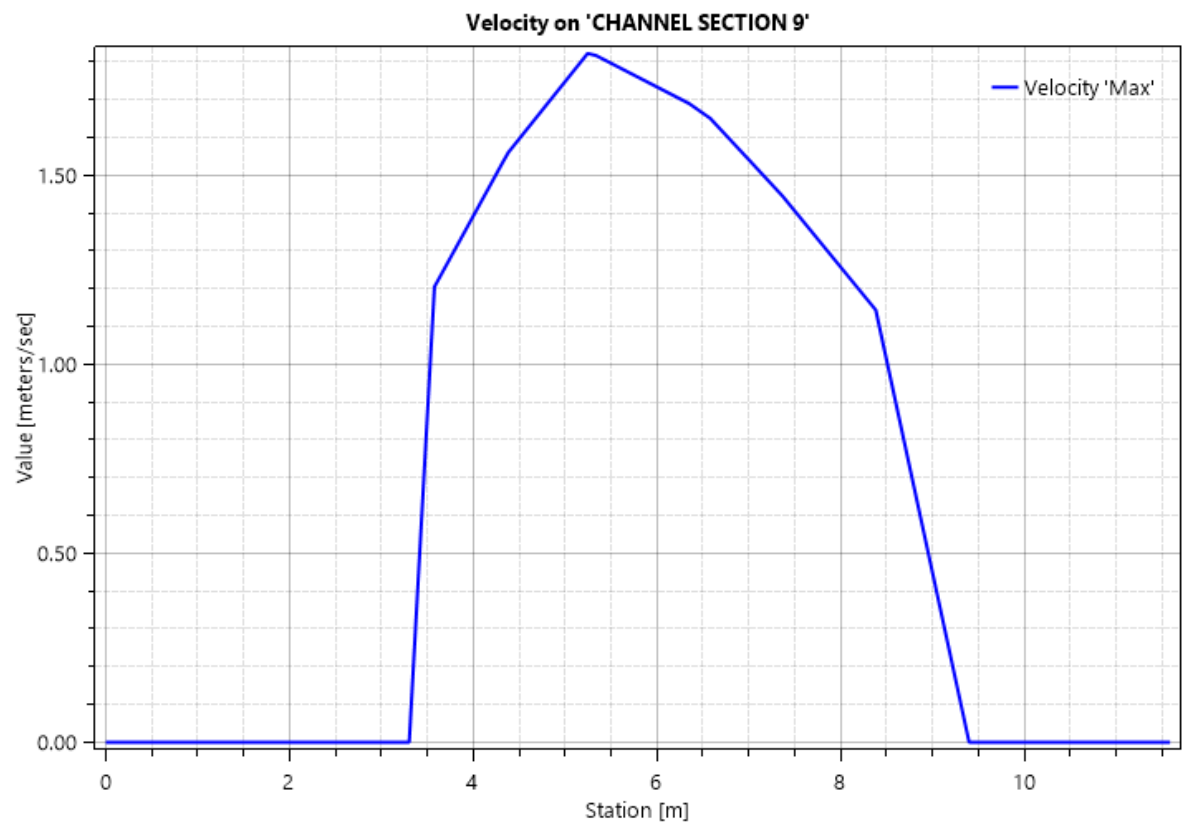


Fig 73 - Channel 2 / section 9 flow velocity

Fig 24 –

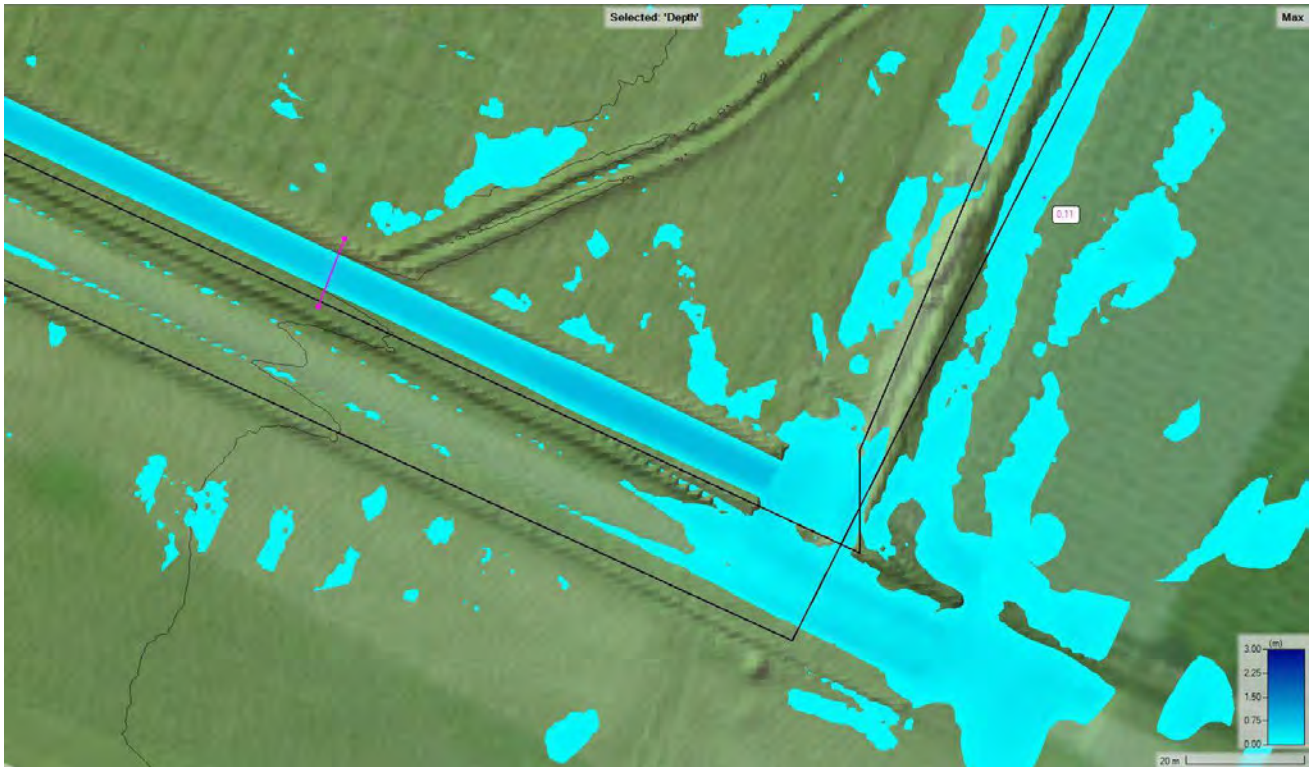


Fig 74 – Channel 3 / section 10 location

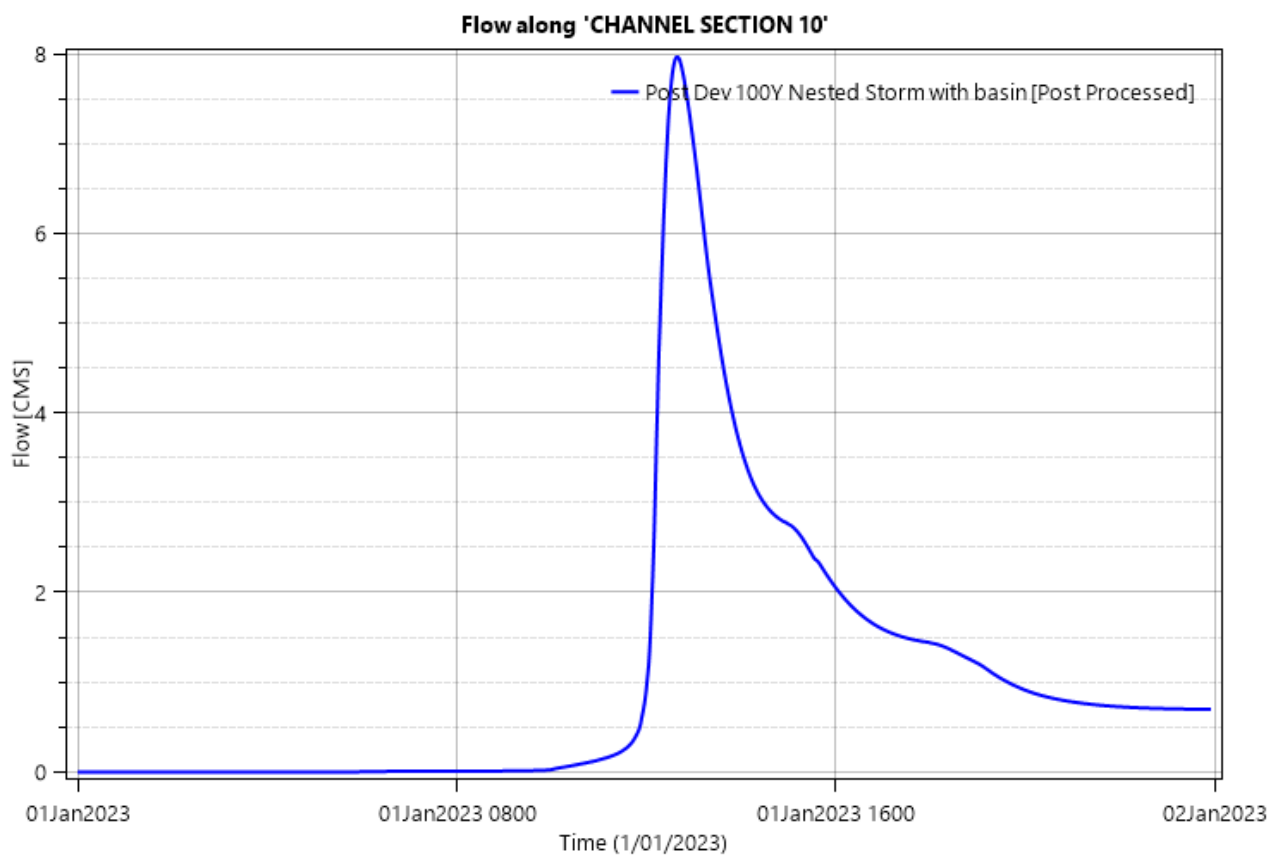


Fig 75 – Channel 3 / section 10 flow hydrograph

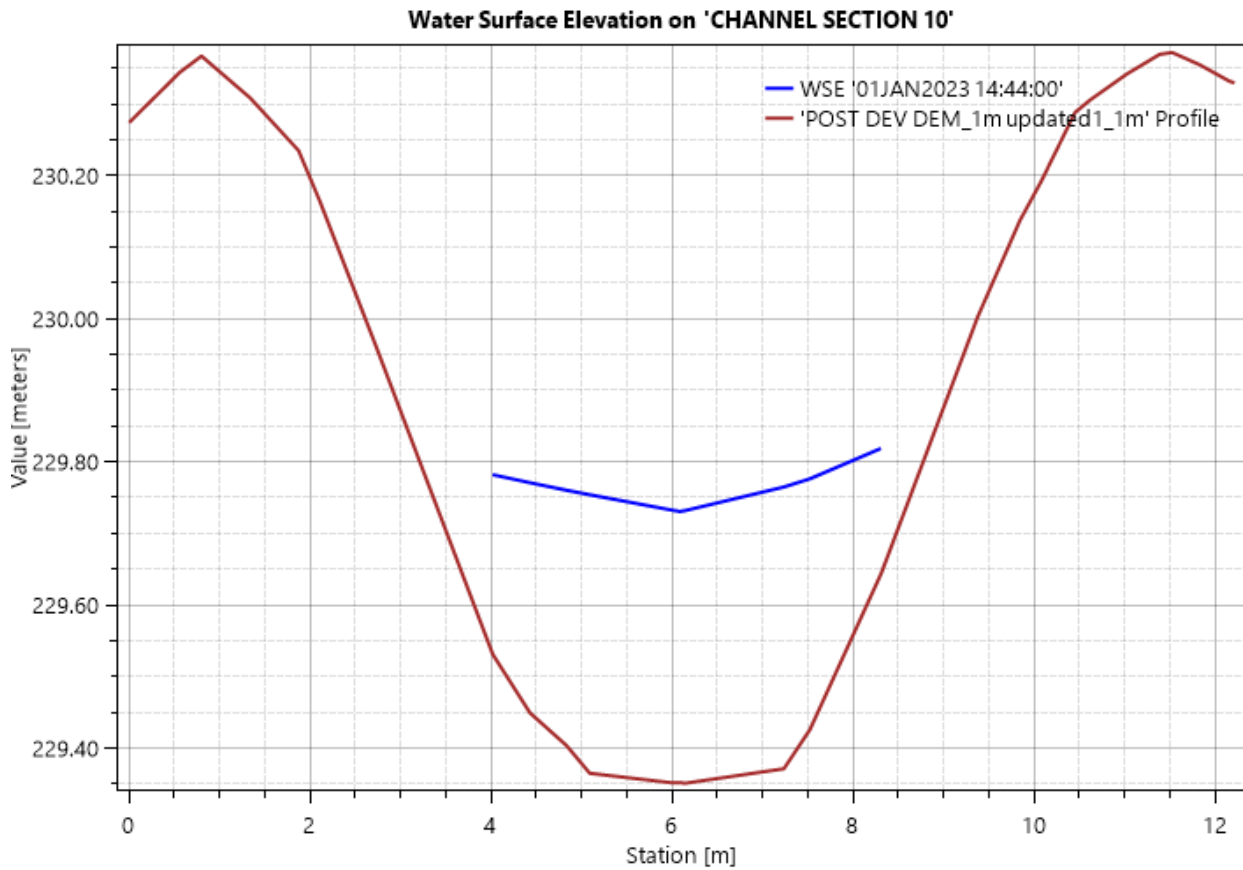


Fig 76 – Channel 3 / section 10 flow depth

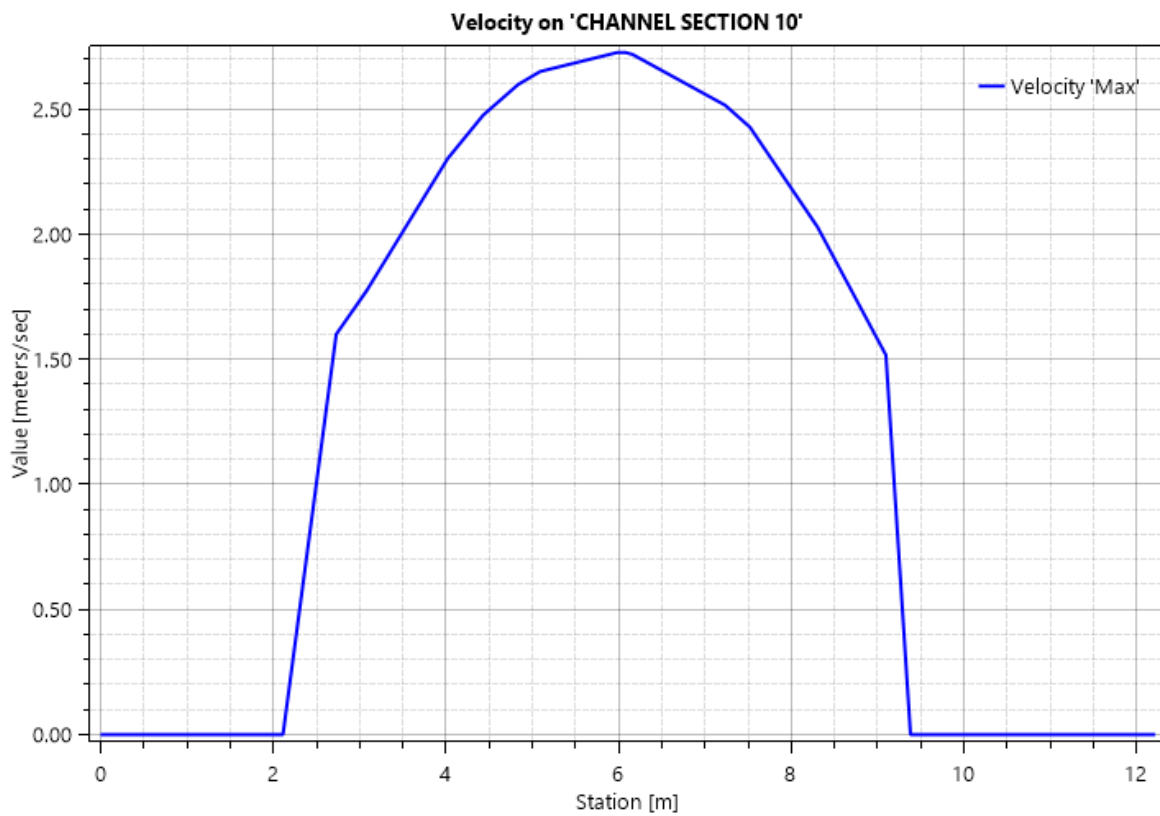
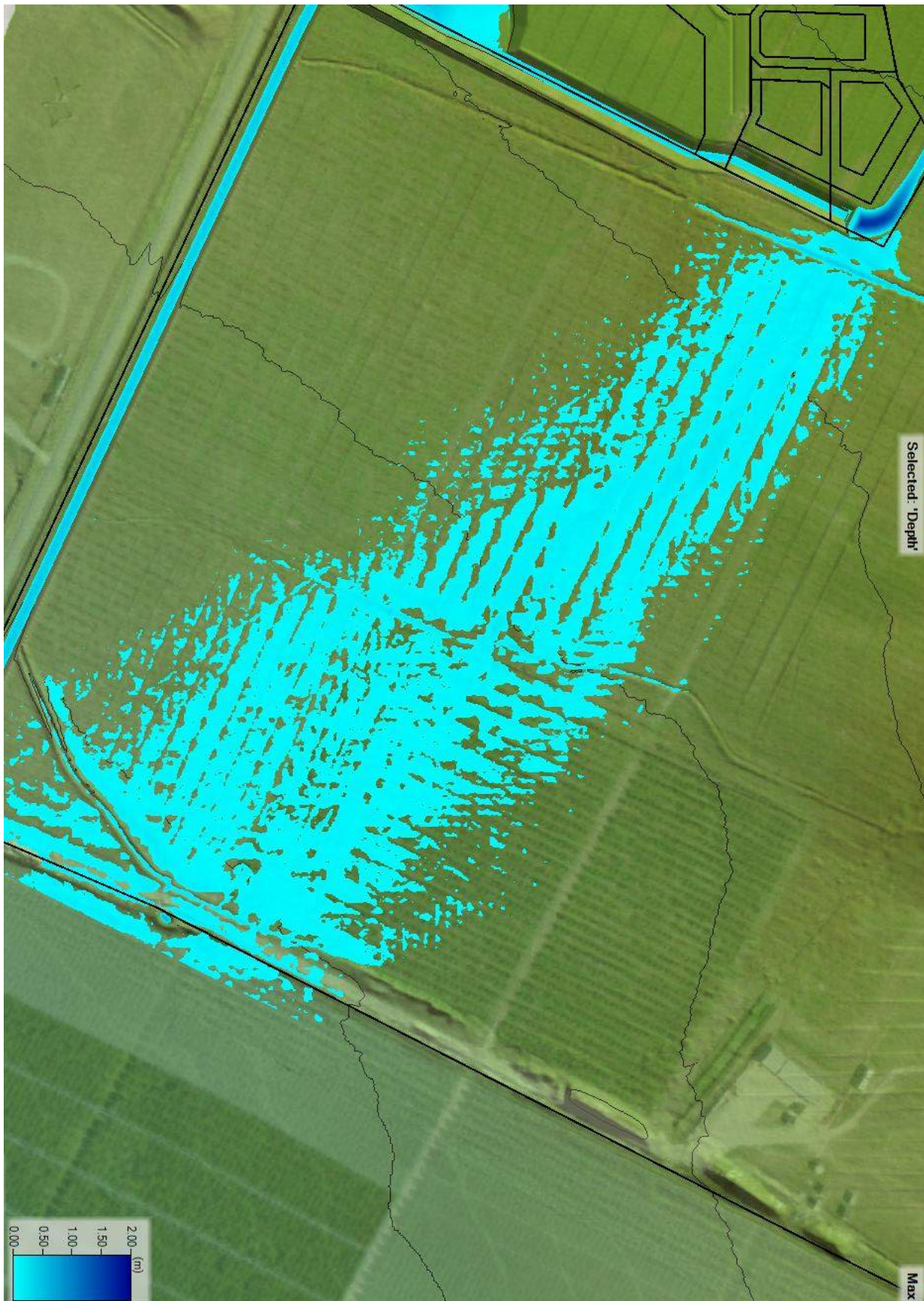


Fig 77 – Channel 3 / section 10 flow velocity

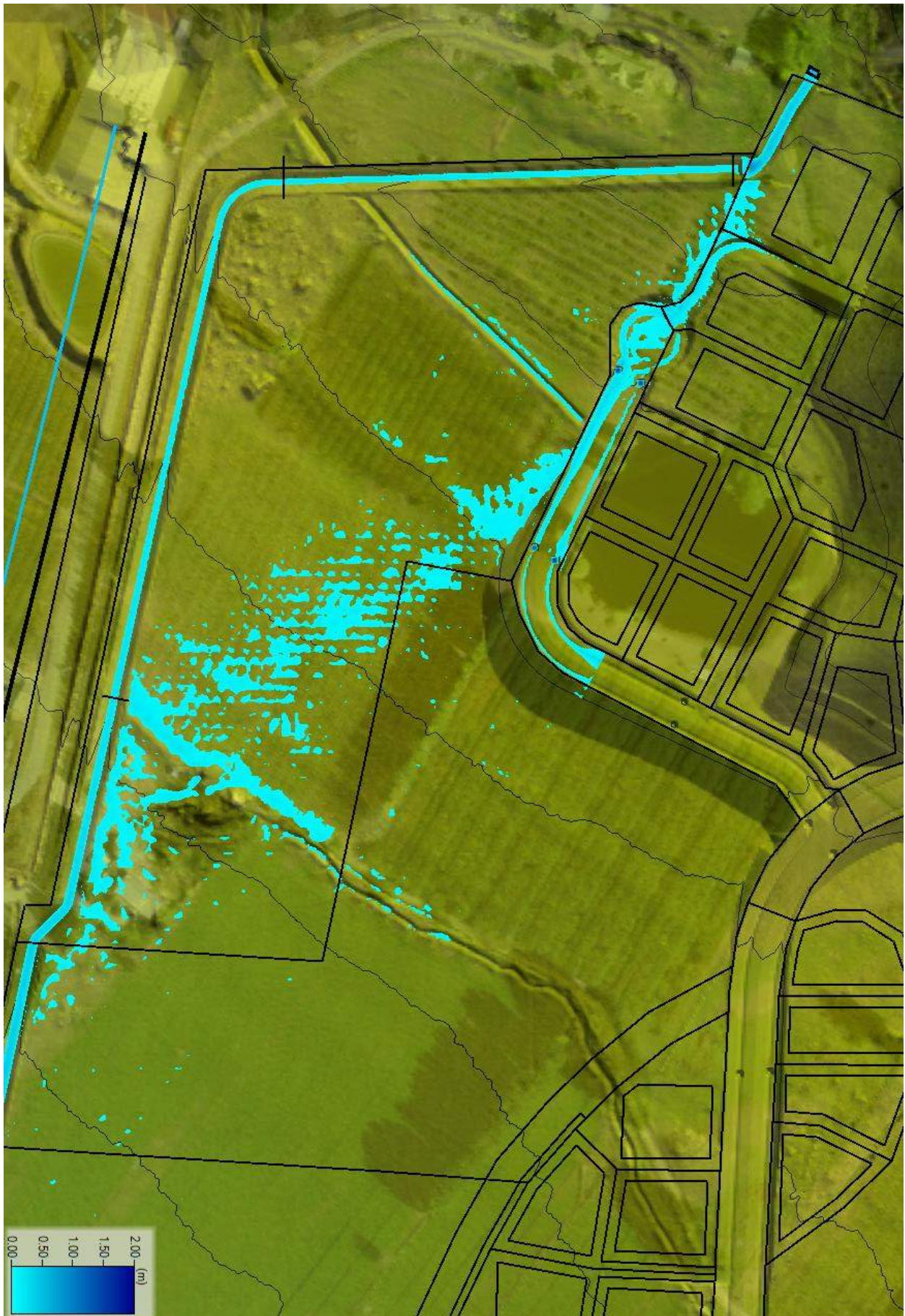


Fig 78 – overall channel flow velocities.

5.8.3 HEC RAS OUTPUTS – CHANNEL OVERFLOWS



Channel 1 bend – overflow route across cherry orchard. No inundation of building platforms



Channel 2 bend – overflow route down roadway & across cherry block. No inundation of building platforms



**UPDATED STORMWATER MODELLING.
SHANNON FARM DEVELOPMENT**

TO	Otago regional Council
From	Roy Buchanan
Date	25.06.2025

This memo has been prepared to accompany the consent variation application for works associated with the Shannon Farm Development

As apart of the engineering approval process with CODC some changes were made to the proposed stormwater system as consented by ORC. An existing irrigation dam was utilised to provide attenuation (Basin 2) allowing the original basin to be reduced in size (Basin 1).

This provided for attenuation of flood flows before the passed through the development rather than downstream from the development reducing flood risk to the new allotments. The overall downstream effects from the development are still reduced as with the original system.

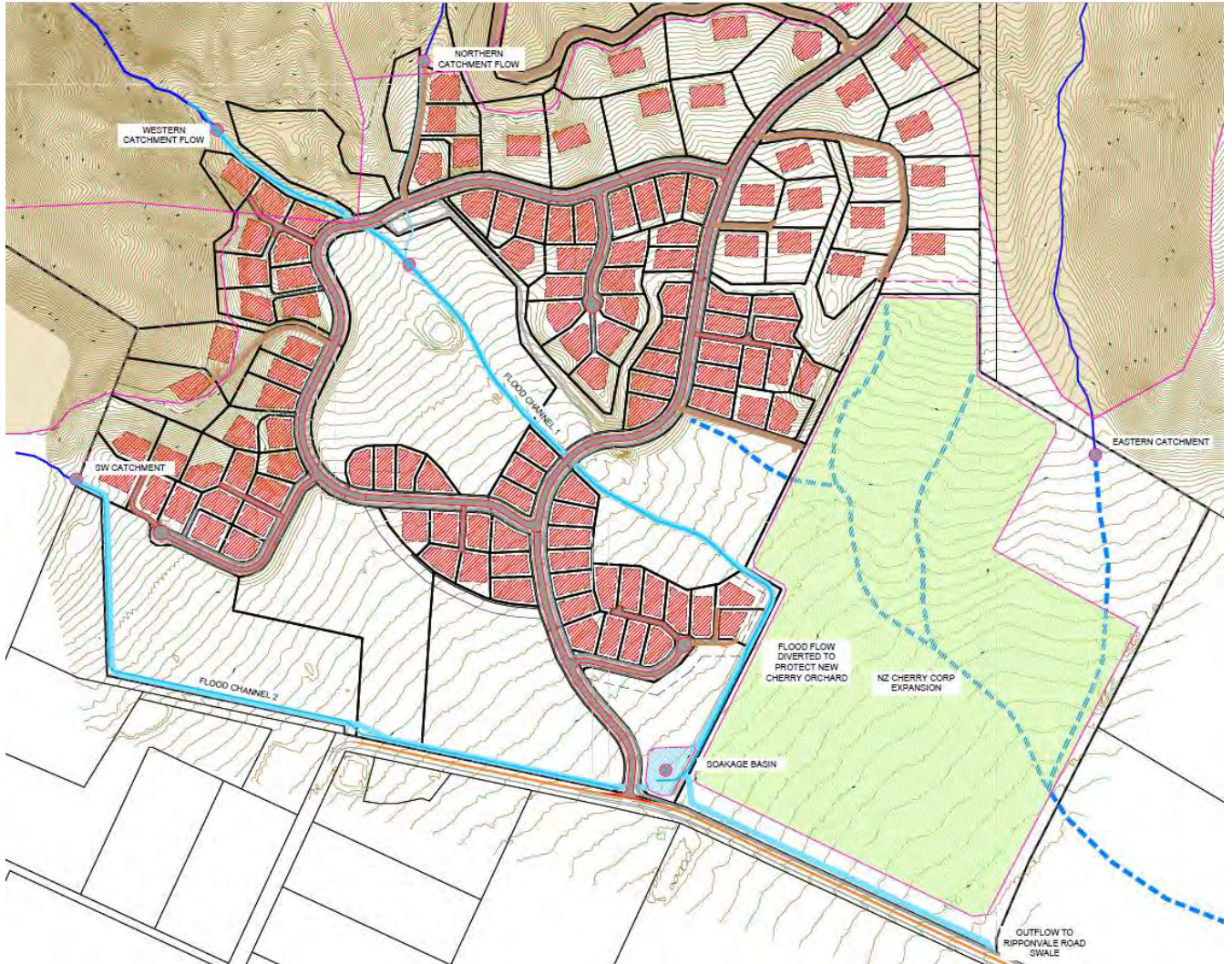
The reporting submitted and approved by CODC is included in this memo. The same model parameters we used as the original ORC reporting with the only change being the adjustments to the attenuation configuration.

This memo contains updated stormwater modelling reflecting this change & should be read in conjunction with the **15729_STORMWATER MODELLING REPORT_MAY 2023**, issued to ORC for the original consent approval.

1 STORMWATER NETWORK

1.1 CONSENTED FLOOD MITIGATION NETWORK

The existing flood mitigation as approved by ORC consents RM23.549.01 – 07 is shown below:



The existing network consisted of 2 flood channels draining to a central attenuation basin (Basin 1) at the bottom corner of the site. The channels convey the full flood flow through the development without risk to property.

The attenuation basin (Basin 1) was provided to mitigate the effects of concentrating storm flows in a channel and modelling confirmed that the resultant peak flows to the downstream land was required in the post development scenario.

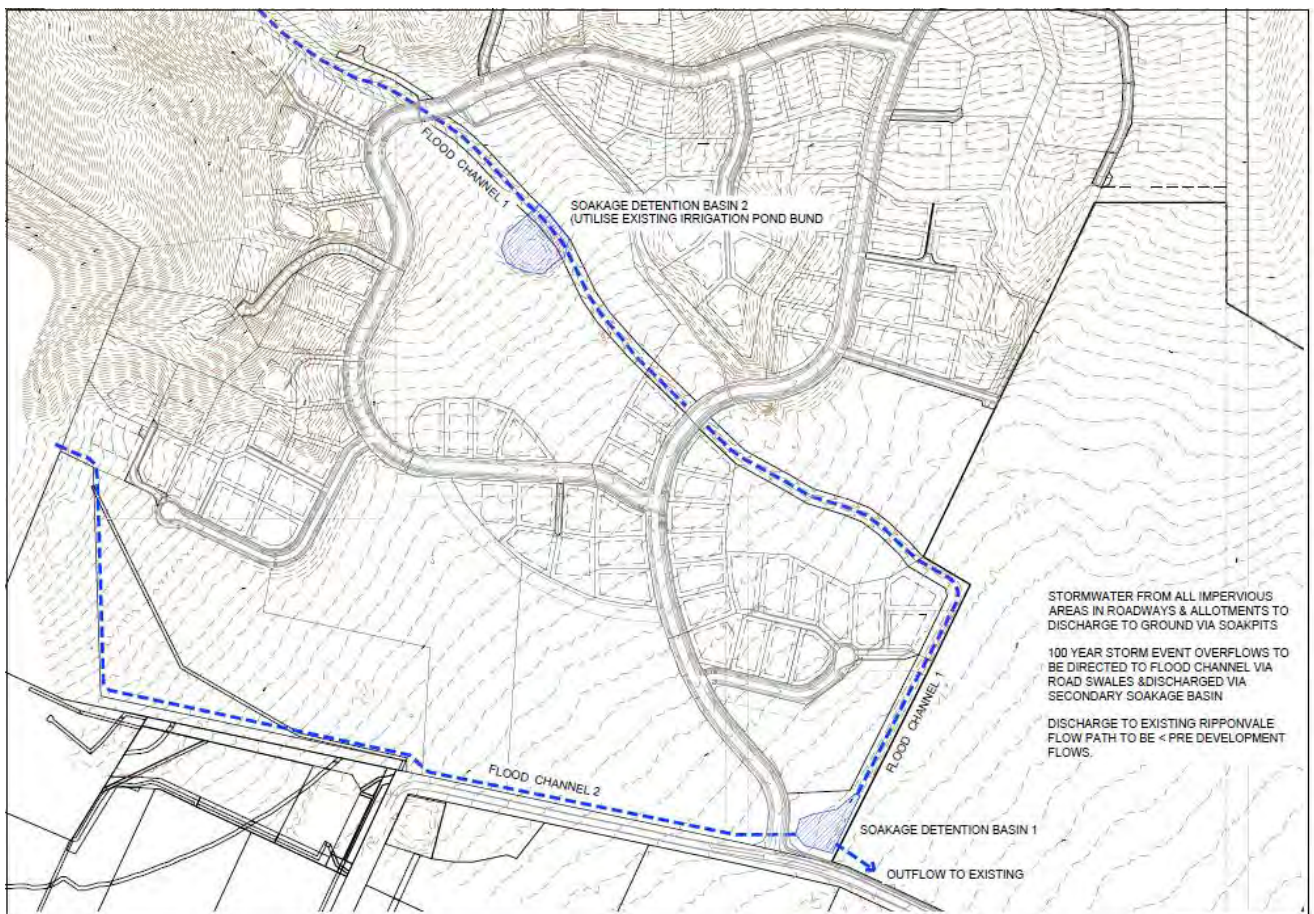
1.2 UPDATED FLOOD MITIGATION NETWORK

Through the CODC engineering acceptance process the original flood mitigation network was adjusted to provide attenuation of flows before they enter the stage 1 allotments.

This involved adding a second attenuation basin (Basin 2) along channel 1 which utilised the existing irrigation dam formation which the original channel alignment passes through.

The additional attenuation basin then allowed the original basin (Basin 1) to be reduced in size.

The alignment of the channels remains otherwise unchanged. This system results in a reduction in flows along the lower section of Channel 1 where it runs past the stage 1 allotments.



1.3 UPDATED SOAKAGE / DETENTION BASINS

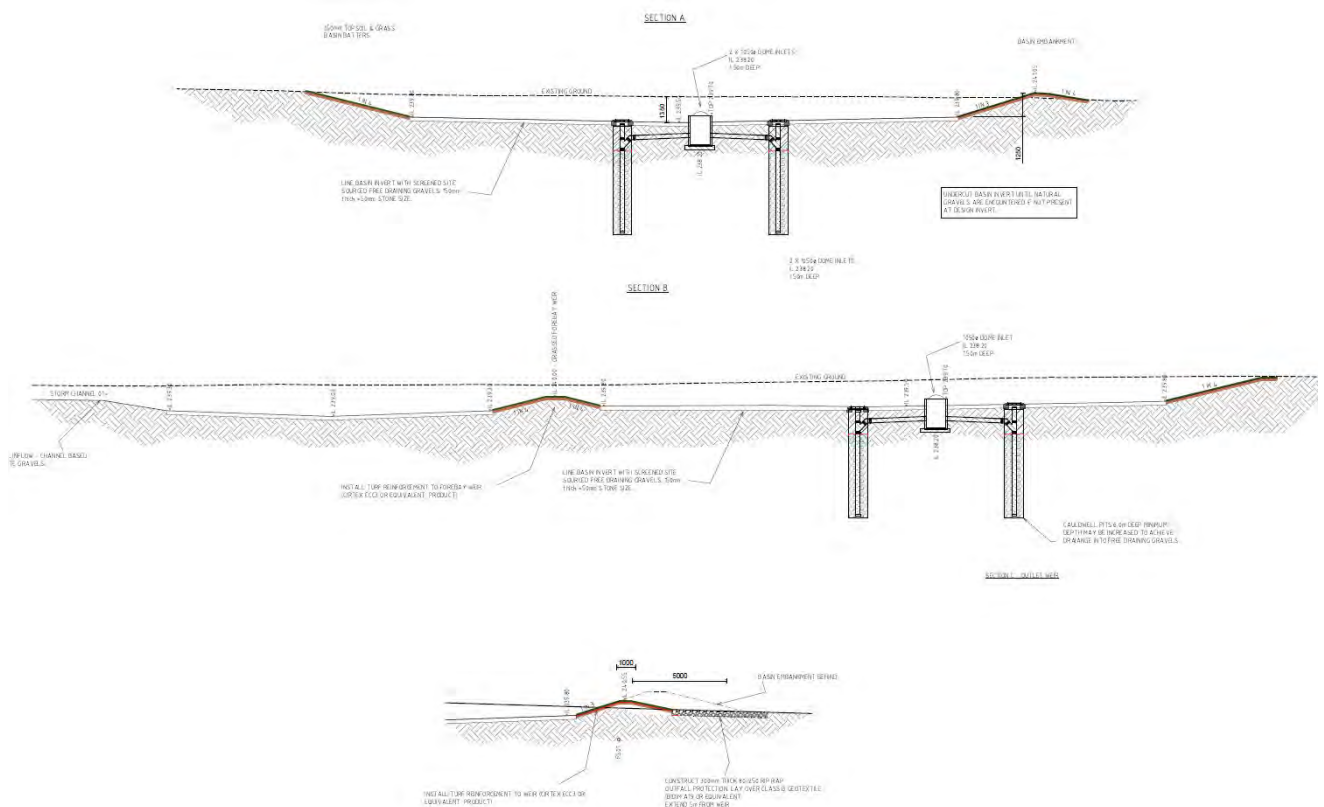
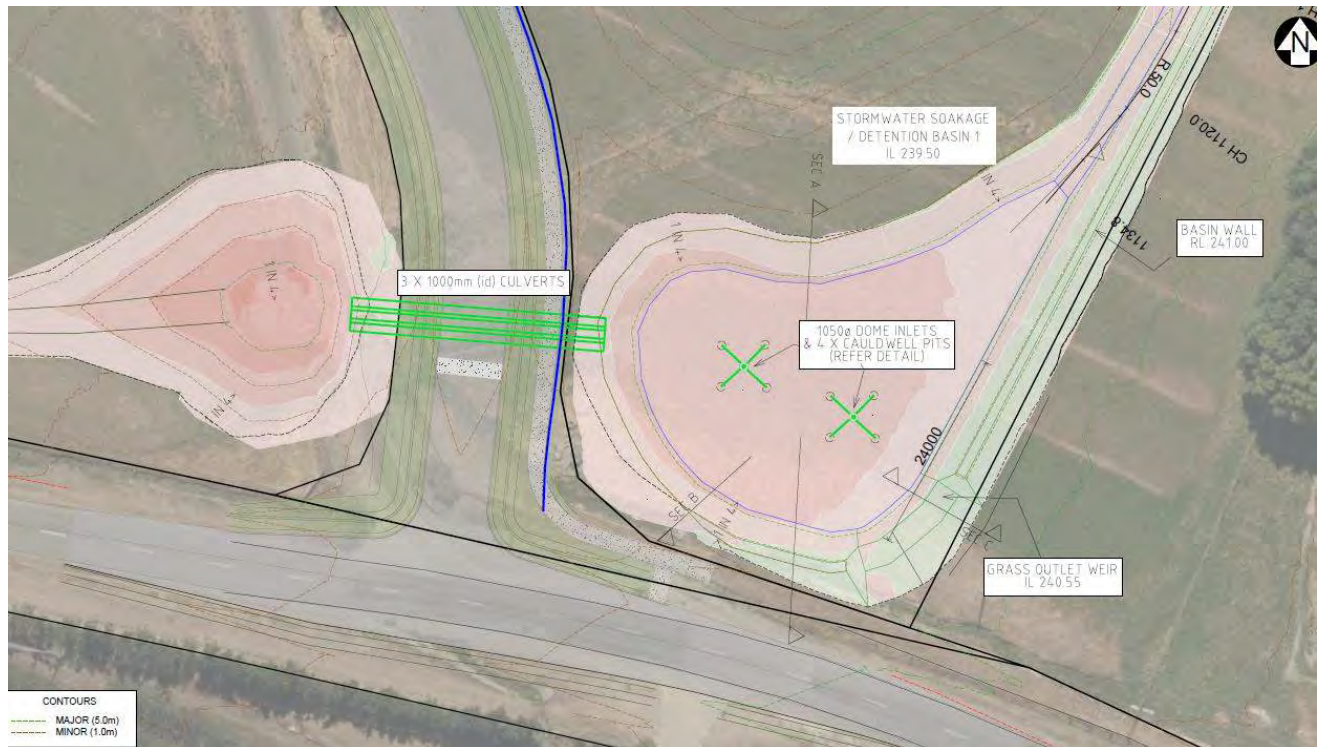
1.3.1 BASIN 1

Basin 1 has been reduced in size relative to the original design and now has the primary function to capture stormwater flows during smaller storm events and discharge flows to ground so they do not cause nuisance for the adjoining cherry orchard (owned by the developer).

As a secondary function this basin still provides additional flow detention during large storms to reduce downstream effects from the development. Basin 2 now provides the primary attenuation function in the system.

A dome inlet will be installed in the base of the basin which will connect to a series of Caldwell soak holes which are expected to provide better long term performance than soaking water through the gravels in the base of the basin. The sump and syphons in the inlet will allow for interception of silt and debris before entering the soak holes.

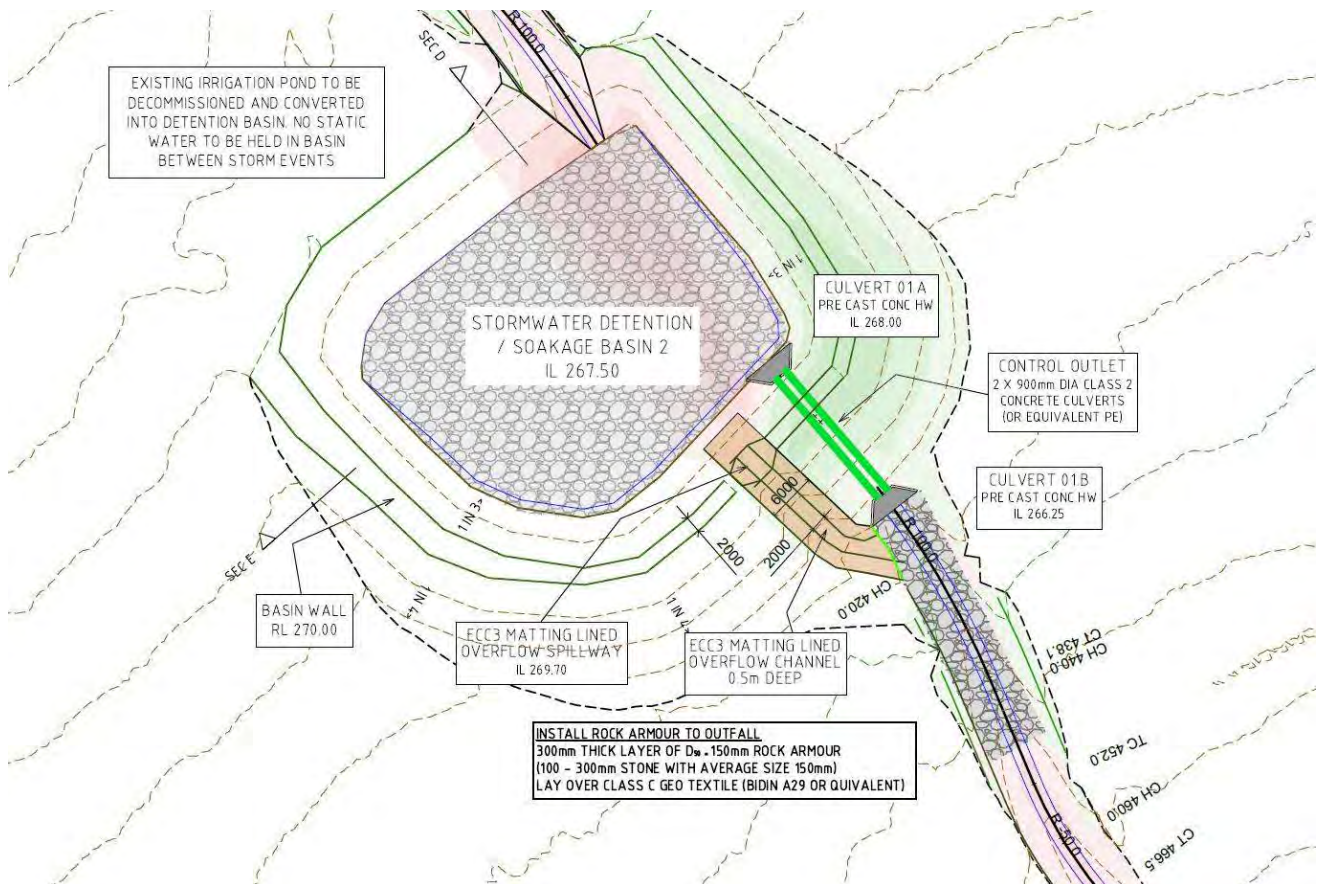
A low grassed weir is provided to control the overflow from the basin to the existing Ripponvale Road flow path.



1.3.2 BASIN 2

Basin 2 is to function primarily as a detention basin to buffer the peak flood flow in a 100-year ARI event. This basin significantly reduces the peak downstream channel flow (approximately 1000L/S reduction) which helps mitigate the flood risk to the stage 1 – 6 allotments and offsets the increases in peak flows to the downstream catchments due to the concentration of flow & increase in impervious within the development.

The basin utilises the existing irrigation pond located within Lot 703. 2x 900mm outlet pipes are provided to drain storm flows from the basin at a controlled rate. In the 100-year storm event a minimum of 500mm freeboard is maintained within the basin. In the event of an overflow a secondary spillway is provided which discharges back into the downstream channel.



Basin 2 layout

1.4 STORMWATER MODELLING.

1.4.1 INTRODUCTION

This section describes the modelling process applied to simulate rainfall and runoff scenarios and to evaluate flood mitigation measures for the Shannon Farm development.

1.4.2 OBJECTIVE

The modelling objectives are:

- Demonstrate that the proposed adjustments to the attenuation layout would not have a negative effect in comparison to the previously consented configuration.

1.4.3 MODEL DEVELOPMENT

The parameters used in the model are summarised below:

Summary of model parameters		
Software	12D model v15 & HEC RAS	
Loss method	SCS curve numbers	
Rainfall Data	HIRDS V4. RCP 8.5 2081-2100	
Rainfall hyetograph	24-hour nested storm based on intensity / frequency distribution rainfall data.	
Storm frequency	1% AEP storm event	
Terrain Data	Lidar Data (flow 2019)	
Soil Group	Group B soil.	
SCS curve numbers	Landuse	Curve Number
	Bush/Forest	55
	Pasture/Grass/Lawn	61
	Impervious	98
Initial abstraction	Impervious 0mm, Pervious 5mm (TP108)	
Manning's N	Description	N
	Unchanneled hillside	0.06
	Flow channels	0.035
	Concrete Pipe	0.013
	PVC/PE Pipe	0.011
1-D stormwater modelling software	12-D, Version 15, water network editor, and dynamic drainage module.	
2-D rain-on-grid SW modelling software	USACE HEC-RAS, Version 6.31	
Time of concentration method (Tc)	TP108 equation 4.3 $t_c = 0.14 C L^{0.66} \left(\frac{CN}{200 - CN} \right)^{-0.35} S_c^{-0.30}$	
Time to peak	0.75(tc) – TP108 section 4.1	

Table 1 – Model parameters

A description of the rationale behind the selected parameters is provided in the following sections.

1.4.4 WORKFLOW SUMMARY

The modelling workflow involved the following steps:

- Develop 12D 1D network to model flow channels & Basins

- Extract inflow hydrographs from existing 2D HEC RAS modelling (refer existing reporting) and introduce flows to 1D network to analyse channel & basin performance
- Confirm channel capacity & analyse results of resultant network outflows for comparison against predevelopment & previous outflows
- Prepare a table of results.
- Make recommendations.

1.4.5 1D FLOOD MITIGATION NETWORK

The channels and basins were modelled in 12D as a 1D network. Channels were modelled with a series of nodes following the horizontal and vertical alignment of the storm channels. The channel was assigned the design width and depth.

Basins were modelled as basin nodes with the outlet pipes and weirs full modelled.

Specific design parameters applied in the water network editor are detailed below:

- Culvert inlets are modelled as square-edged inlets flush to the dome inlet wall. $K_e=0.5$
- Channels modelled with a Mannings $N=0.035$ for a uniform weedy channel.
- Pipes have been modelled with roughness coefficients in accordance with NZS4404:2004.
- Flow from HEC RAS 2D modelling introduced to nodes at the inflow locations along the channel.

1.4.6 INFLOW HYDROGRAPHS

To account for flows arriving at the channels from upstream & internal catchments flow hydrographs for the channel modelled in the 2D rain on grid model were extracted from the HEC RAS 2D rain on grid simulation developed for the site at various locations along the channel.

Details of these flows are provided in the original reporting **15729_STORMWATER MODELLING REPORT_MAY 2023**, issued to ORC for the original consent approval.

These flows were assigned to the corresponding nodes along the channel network to account for the additional contributing catchments as the channel extends towards the bottom of the site. The total channel inflow aligned with the flows modelled in the 2D simulation.

1.4.7 PRIMARY NETWORK –1D DYNAMIC DRAINAGE SIMULATION RESULTS

Simulation runs were carried out for the 1% AEP storm event. Hydraulic elements were refined incrementally until the following modelling objectives had been satisfied:

- Demonstrate that the proposed adjustments to the attenuation layout would not have a negative effect in comparison to the previously consented configuration.

The Modelling confirms that the revised attenuation layout results in an additional reduction in peak flows leaving the site in relation to the predevelopment situation.

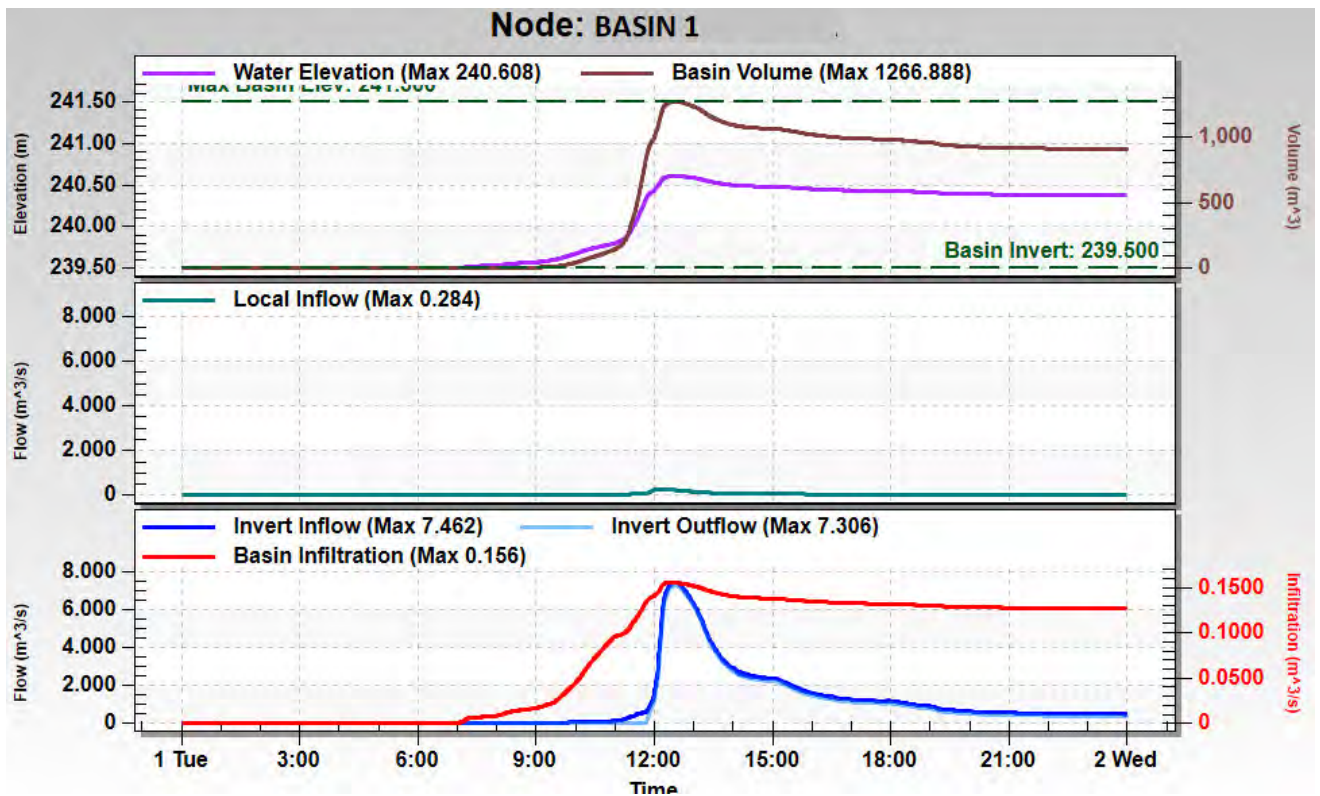
FLOW LOCATION	ORIGINAL NETWORK (m3/s)	UPDATED NETWORK (m3/s)	Diff
Channel 1 inflow upstream of basin 2	4.507	4.507	0
Channel 1 flow downstream of basin 2	4.507	3.591	-0.916
Basin 1 inflow from Channel 1	5.250	4.207	-1.043
Basin 1 inflow from Channel 2	3.394	3.394	0
Basin 1 outflow to downstream	7.980	7.306	-0.674

1.5 CONCLUSION

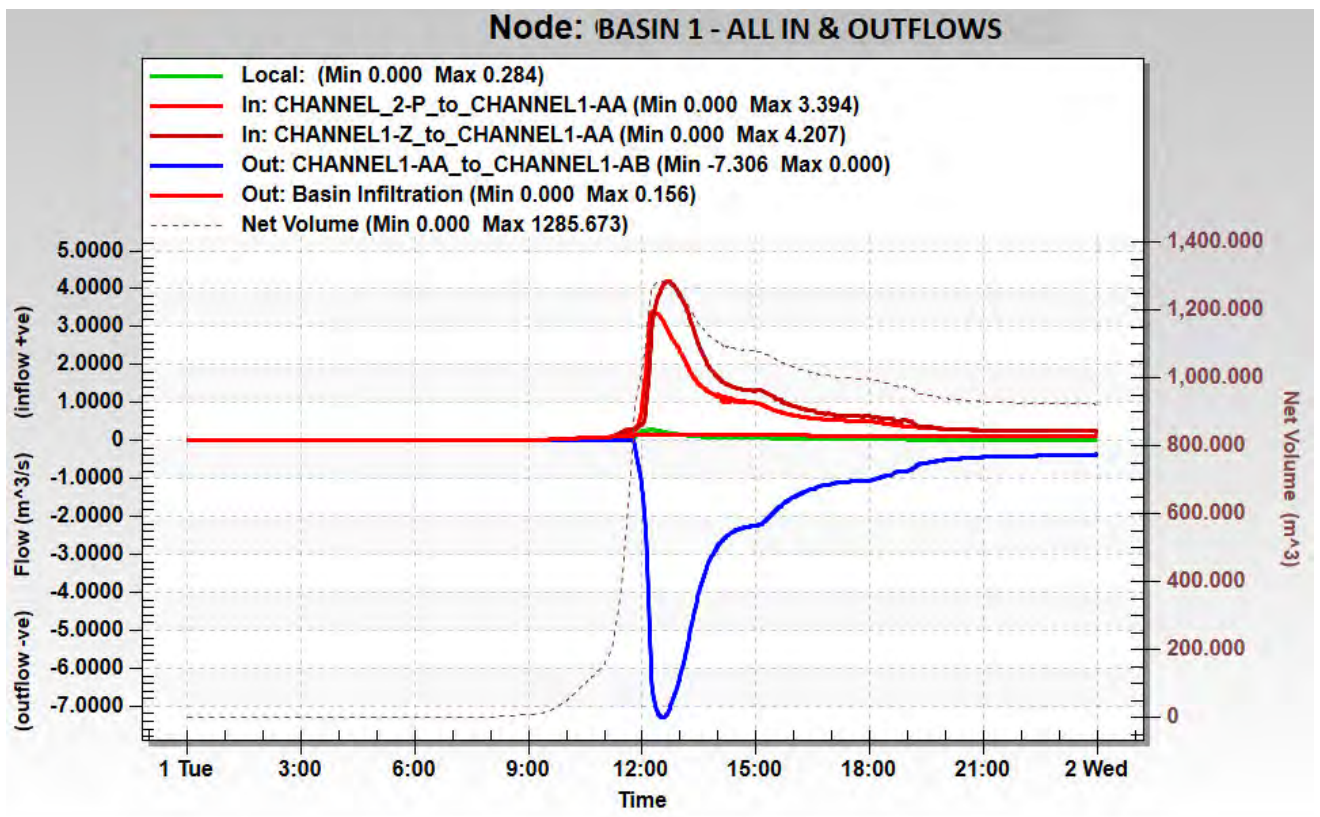
The modelling confirms that the project objectives outlined in section 1.4.2 have been achieved.

The proposed changes to the stormwater system will result in additional reduction of flows leaving the development.

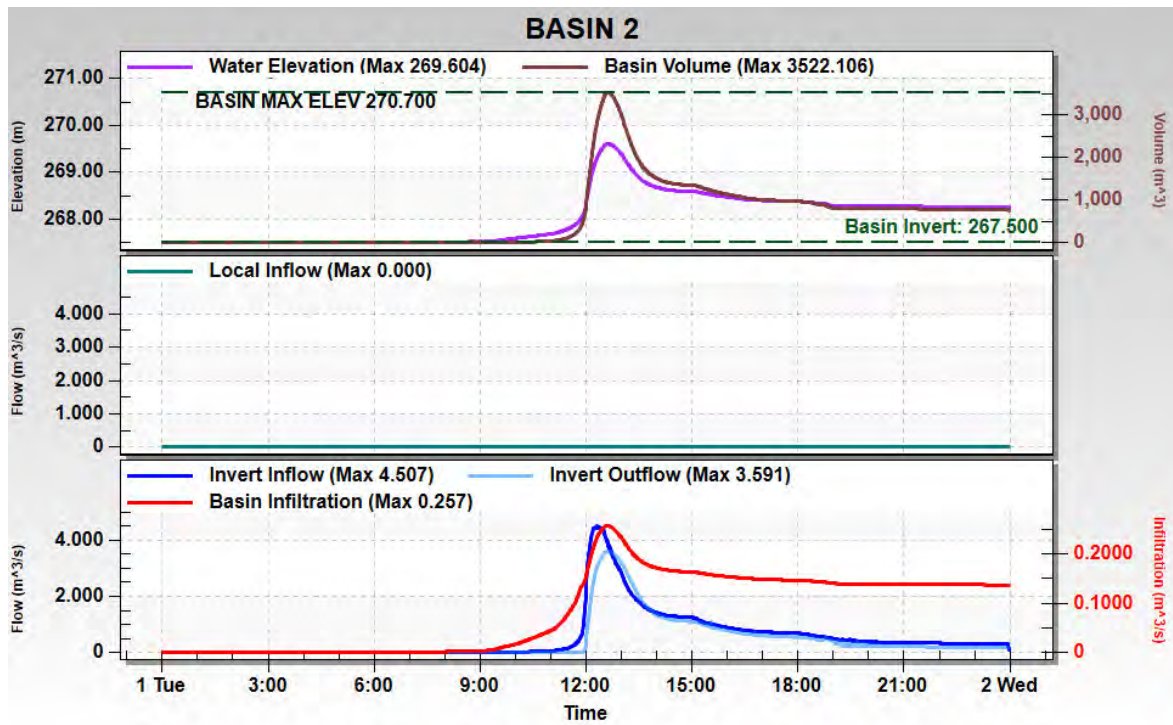
2 12D MODEL OUTPUTS



Basin 1 – total inflows & outflows



Basin 1 – All individual inflows & outflows



Basin 2 – total inflows & outflows

Cromwell Cherry Orchard Development Stormwater
Feasibility Memorandum

ATTACHMENT D:
Soakage Testing Results

TEST REPORT

Test completed to ASTM D3385-09

Test Locations:

The eight tests completed were focused on three locations: the proposed Basins 1, 2, and 3.



Testing Completed:

Testing completed on 9 and 11 March, testing on 10 March had to be scrubbed due to a broken auger.

Double Ring Infiltration Tests:

6

Constant Discharge Tests:

2

Note: The constant discharge test planned for Basin 3 was unable to be conducted due to the Cherry Orchard netting.

TEST REPORT

Test completed to ASTM D3385-09

Test Location:	Tests completed in proposed Basin 2 of the Cherry Orchard Development
Soil Description:	Soil generally a silty sand with some inclusions of gravels and larger material
Inner Ring Diameter:	300mm diameter
Outer Ring Diameter:	600mm diameter
Ring Height:	500mm
Ring thickness:	2.4mm

Test Results

	DRI-1		DRI-2	
Date:	9-March-2026		9-Mar-26	
Time:	11:35am		12:36pm	
Temperature (Air):	9 Degrees		12 Degrees	
Weather Conditions:	Sunny		Sunny	
Depth of Test:	600mm below existing ground		200mm below existing ground	
Water depth during presoak:	100mm - 200mm		100mm - 200mm	
Time taken to presoak:	15 minutes		15 minutes	
Rings driven to:	150mm		150mm	
	Test (mm/min)		Test (mm/min)	
Minutes	Water depth (mm)	Change in level (mm/min)	Water depth (mm)	Change in level (mm/min)
1	200	12	200	10
2	188	8	190	17
3	180	14	173	14
4	166	11	159	14
5	155	11	145	14
6	144	11	131	
7	133			
Adopted Soakage (unfactored):	11	mm/min	14	mm/min

Location	Photograph	Photograph
		

Test to be terminated once three equal declines in water depth are recorded

Location	Photograph	Photograph
 An aerial photograph of a vineyard. A dirt road runs diagonally from the bottom left towards the top right. To the left of the road is a grassy area. To the right is a dense vineyard. Three points are marked: CDT-2 (blue dot), DRI-3 (green dot), and DRI-4 (purple dot). CDT-2 and DRI-4 are close together, while DRI-3 is further down the road.	 A photograph of a well opening in a field. The well is a circular hole in the ground, surrounded by a concrete curb. The ground is dry and brown. There are trees and a blue sky in the background.	 A photograph of a well opening. A metal pipe is visible inside the well. The well is surrounded by a concrete curb. The ground is dry and brown. There are trees and a blue sky in the background.
Test to be terminated once three equal declines in water depth are recorded		

TEST REPORT

Test completed to ASTM D3385-09

Test Location:	Tests completed in proposed Basin 3 of the Cherry Orchard Development
Soil Description:	Soil generally a silty sand with some inclusions of gravels and larger material.
Inner Ring Diameter:	300mm diameter
Outer Ring Diameter:	600mm diameter
Ring Height:	500mm
Ring thickness:	2.4mm

Test Results

	DRI-5		DRI-6	
Date:	11-Mar-26		11-Mar-26	
Time:	2:00pm		2:28pm	
Temperature (Air):	22 Degrees		23 Degrees	
Weather Conditions:	Sunny		Sunny	
Depth of Test:	1400mm below existing ground		300mm below existing ground	
Water depth during presoak:	100mm - 200mm		100mm - 200mm	
Time taken to presoak:	15 minutes		15 minutes	
Rings driven to:	100mm		150mm	
	Test (mm/min)		Test (mm/min)	
Minutes	Water depth (mm)	Change in level (mm/min)	Water depth (mm)	Change in level (mm/min)
1	215	0	187	1
2	215	0		
3	215	0	186	1
4	215	0		
5	215	0	185	1
6	215			
7			184	1
8				
9			183	
10				
Adopted Soakage (unfactored):	0	mm/min	0.25	mm/min

[illegible]

 CONSULTING ENGINEERING STRUCTURAL / CIVIL / PROJECT MANAGEMENT / GEOTECHNICAL	JOB NAME: Cherry Orchard Development		PAGE No.	
	SECTION: Constant Discharge Test 1		5	
	JOB No. 254029	COMPLETED BY: Strata Civil		
	DATE: 9 March 2026	WITNESSED: Jake Allen (HCL)		
TEST REPORT				
Test Location:		Tests completed in proposed Basin 1 & 2 of the Cherry Orchard Development		
Soil Description:		Soil generally a silty sand with some inclusions of gravels and larger material. Soil was very dense and resulted in damage to the auger head during drilling.		
Casing Diameter:		650mm diameter.		
Measuring device:		Measurement undertaken with a dipmeter.		
Test Results				
	CDT-1		CDT-2	
Date:	9-Mar-26		11-Mar-26	
Time:	2:12pm		10.45am	
Temperature (Air):	11 Degrees		18 Degrees	
Weather Conditions:	Sunny		Sunny	
Bottom of Casing below EGL:	3000mm below existing ground		3000mm below existing ground	
PVC pipe above EGL:	960mm above existing ground		950mm above existing ground	
Depth of Gravel at bottom:	630mm		500mm	
Ground water level:	not encountered		not encountered	
Water delivery during presoak:	0.3 L/s		0.3 L/s	
Time taken to presoak:	30 minutes		30 minutes	
	100 min Vertical Test (mm/min)		100 min Vertical Test (mm/min)	
Minutes (Reading taken every minute but not recorded here for clarity)	Water below EGL (m)	Change in level (mm/min)	Water depth (mm)	Change in level (mm/min)
1	2.16	0	2.49	0
10	2.16	0	2.49	0
20	2.16	0	2.49	0
30 (Terminated CDT-2)	2.16	0	2.49	0
40	2.16	0		
50	2.16	0		
60	2.16	0		
70	2.16	0		
80	2.16	0		
88 (Terminated CDT-1)	2.16	0		
Adopted Vertical Soakage (unfactored):	0	mm/min	0	mm/min

	20 min Horizontal Test (mm/min)		20 min Horizontal Test (mm/min)	
	Water below EGL (m)	Change in level (mm/min)	Water depth (mm)	Change in level (mm/min)
Minutes (Reading taken every minute but not recorded here for clarity)				
1	2.03	0	2.24	0
5	2.03	0	2.24	0
10	2.03	0	2.24	0
20	2.03	0	2.24	0
Adopted Horizontal Soakage (unfactored):	0	mm/min	0	mm/min
Location	Photograph		Photograph	
				

The vertical soakage test was run for 100 minutes after pre-soak, followed by the horizontal test.