Civil Engineering Assessment

Dargaville Racecourse Plan Change

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Action	Name	Signed	Date
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on behalf of	**Lands and Survey Engineering Ltd**		

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1.0. Overview

1.1. Executive Summary

The information included in the summary below provides the reporting outcomes of this report and should be read in conjunction with the relevant sections as referenced herein.

Feature	Summary
Natural Hazards	According to the NRC hazard layers, the site is not located in an area susceptible to flooding. Assessment of Geotechnical hazards is not included in this assessment.
Wastewater Disposal	Combined gravity / low pressure system required to collect and convey wastewater to single pump station and low pressure rising main to convey wastewater from proposed development to Council network/wastewater treatment plant. Discharge points a) Pump Station 14, b) Wastewater Treatment Plant.
Stormwater Management	Runoff generated from proposed development to be collected and conveyed through sealed pipe network and discharge to proposed attenuation and detention devices (ponds). Controlled release of treated stormwater to the receiving environment to ensure no adverse effect on downstream infrastructure, property, or environment. Design, construction, and maintenance of proposed stormwater system to comply with relevant engineering standards and be designed to satisfy the provisions relating to *Te Mana o te Wai* and the objectives and policies for freshwater management in accordance with the National Policy Statement for Freshwater 2020.
Potable Water Supply	Council water supply is present in the direct vicinity of the proposed development. Network capacity to meet the demand of the proposed development was assessed and confirmed. Although, water treatment plant capacity assessment is inconclusive, it has been indicated that the plant has capacity to supply the proposed development, albeit that seasonal shortage of raw water to the treatment plant may be one of the major constraints to meet the demand of the development and expected growth for Dargaville. Alternative water supply or supplementary supply by way of rainwater harvesting and ground water supply should be considered. Testing and analysis of groundwater quality can be further investigated.
Sediment and Erosion Control	Appropriate erosion and sediment control practices to be implement in accordance with Guideline Document 2016/005 Erosion and Sediment Control Guide for Land Disturbing Activities in the Auckland Region (GD05). Proposed stormwater attenuation structures are recommended to be incorporated into the temporary works and service as settlement ponds during the development stage.
Other Utilities (Electricity and Telecommunications)	Network capacity for electrical supply is available, however may require upgrades to the local network to provide adequate supply. No fibre is available; however, the site is currently within area of benefit for VDSL and Wireless connectivity.

1.2. Introduction

This engineering assessment report has been prepared by Lands and Survey Engineering LTD for Dargaville Racing Club Incorporated (the client) in accordance with instructions received via the client's consultant (Griffiths & Associates) responsible for investigating the feasibility and preparing an application for a private plan change. The purpose of this report is to provide commentary and recommendations on the civil engineering aspects of the proposed development envisaged for which the plan change is sought.

An assessment of the following engineering aspects is included in this report:

- Wastewater disposal,
- Stormwater management,
- Potable water supply,
- Sediment and erosion control, and
- Telecommunications and power.

1.3. Site Description

The site is located on the corner of State Highway 14 and Awakino Point North Road, Dargaville. The legal description of the site is Part Lot 37 DP 7811. The site has an area of 46.6729ha, located approximately 3.5km north-east from Dargaville Town Centre, on the eastern side of State Highway 14.

The site is a decommissioned horseracing facility that was owned and operated by the Dargaville Racing Club. The site has several access crossings from State Highway and Awakino Point North Road with the main access from the State Highway. Buildings on site consists of a range of stables, outbuildings and the main viewing pavilion.

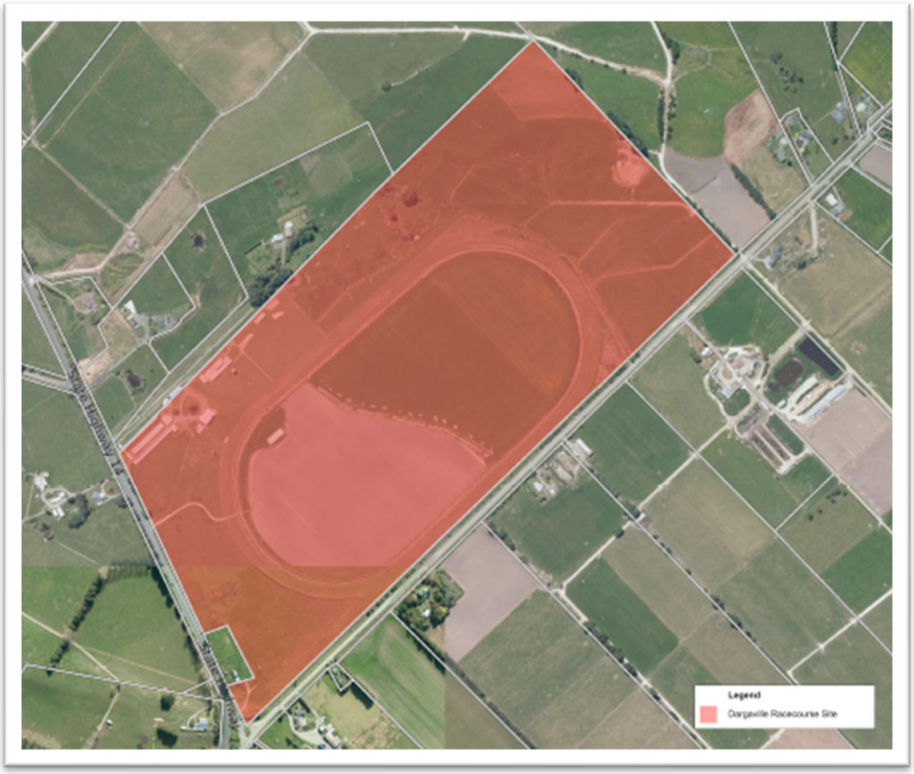


Figure 1: Site Locality (Source: https://map.grip.co.nz/map)

Several site visits were conducted between June and August 2021 to gain an understanding of the site and investigate aspects that may affect this assessment. The site is generally assessed as being relatively flat, with an overall flat slope in an eastern direction, with elevated sections towards the northern corner of the site.

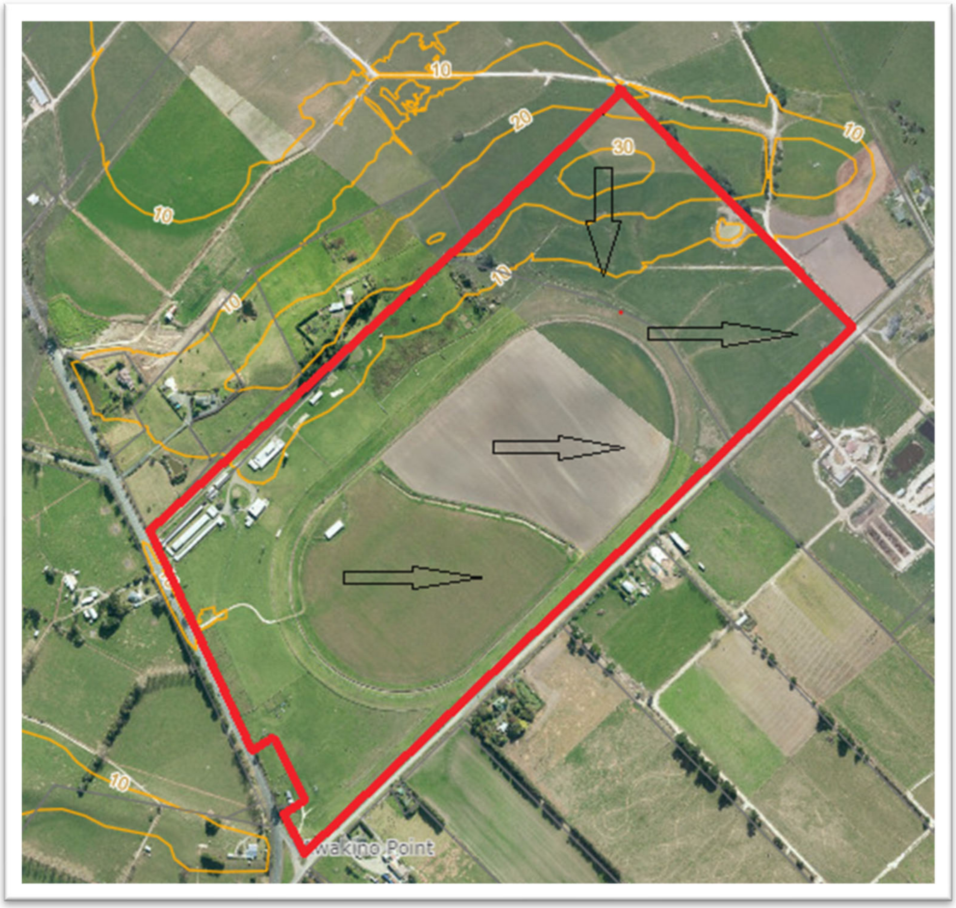


Figure 2: Depicting Contours and General Direction of Site Slope (Source: NRC Online Maps)

A desktop assessment through reviewing the Kaipara District Asset Maps indicates that the site has access to Council water infrastructure, however there is no stormwater or wastewater infrastructure in proximity of the site.

Roadside table drains were observed along Awakino Road intercepts all overland stormwater flow from the site. These drains were observed to have very flat grades, sloping in a north-eastern direction.

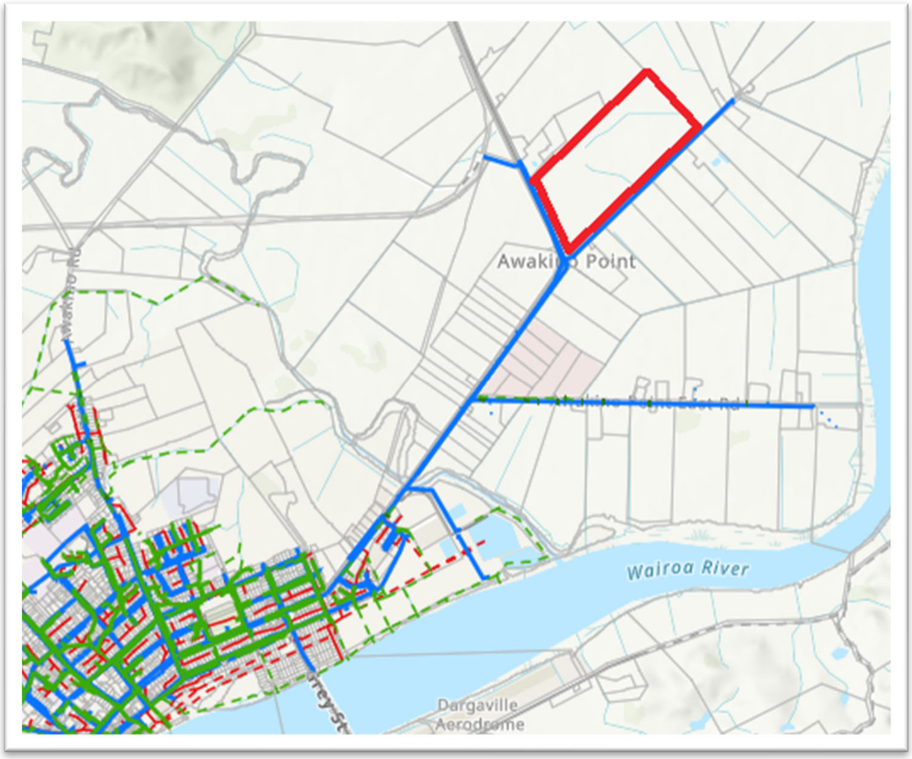


Figure 3: Overview of Council 3 Water Assets (Source: KDC Utility Services Maps)

1.4. Supplied Information

The following information has been supplied by the client's agent:

1. High level brief via email dated 7 April 2021 with subject title "*Civils & Three Waters Assessment for Dargaville Racecourse Plan Change*".

2. Draft Concept Development Plan, received via email dated 8 June 2021 with subject title "*Dargaville Racecourse Plan Change - approach going forward ...*"

3. Geotechnical Assessment prepared by Land Development & Engineering Ltd (LDE), dated 7 May 2021, received via email dated 19 July 2021 with subject title "*Geotech report for Dville Racecourse site*",

4. Draft Outline Development Plan, received via email dated 9 August 2021 with subject title "*Dargaville Racecourse - Outline Development Plan*".

5. Draft Outline Development Plan V.4.0-01 and CDP version 3.2_Yield Summary, received via email dated 22 November 2021 with subject title "Dville Racecourse Plan Change - update".

6. Notes of a consultation meeting held with neighbours on 30 November 2021.

7. Copy of the Cultural Impact Assessment prepared by Georgina Olsen, dated November 2021 with reference GO/LFC – 0112021.

8. Final Concept Development Plan, received via email dated 29 January 2022 with subject title "*Dville Racecourse Plan Change - final Development Area plan for inclusion your reports plse*".

1.5. Development Proposal

The development proposal, obtained from the client's agent, is to undertake a plan change of the site with a total area of 45.4347 ha into a mix of land uses.

The draft concept development plan V.4.0-01 and CDP version 3.2_Yield Summary outlines the following anticipated land use and activities:

- Light Industrial/Commercial – 9.53ha consisting of 24 allotments,
- Residential – 23.67ha consisting of 435 allotments,
- Neighbourhood Centre – 0.29ha consisting of 1 allotment,
- Public open space, buffer areas and reserve areas – 4.16ha,
- Infrastructure including stormwater detention ponds and pumpstation – 1.59ha,
- Road Reserve – 6.19ha.

The anticipated land use and activities is further detailed as per the table below:

Land Use		Area sum (m^2)	Number of lots
Light Industrial		9.53	24
	Small Industrial	0.67	14
	Large Industrial	8.86	10
Residential		23.67	435
	General Residential	20.23	428
	Lifestyle Lot Residential	3.44	6
Neighbourhood Centre		0.29	1
Open Space		4.16	n/a
Infrastructure		1.59	n/a
Road reserve		6.19	n/a
TOTAL		**45.43**	

The concept and outline development plans were a work in progress, informed by input from various consultants and stakeholders, undertaking assessments and consultations which include, but is not limited to Acoustics, Engineering (Traffic and Transport, Geotechnical, Civil), Local Iwi, Landscape and Visual, Market Research and Demand Analysis, and Urban Planning.

A copy of the outline development plan prepared by *The Urban Advisory* is included in ***Appendix A.***

2.0. Description of Environment

2.1. Geology and Geotechnical Considerations

The geological map of the area produced by the Institute of Geological and Nuclear Sciences shows the property as underlain by Late Quaternary alluvium and colluvium soils, which is described as unconsolidated, to poorly consolidated mud, sand, gravel, and peat. These geotechnical characteristics pose several development challenges, to the extent, and in extreme cases, that certain areas may not be suitable for development due to ongoing consolidation, differential settlement and liquefaction risk.

Figure 4: Site Location (Source: New Zealand Geology Web Map (GNS))

A geotechnical investigation report was sought from Land Development and Engineering (LDE) to provide the client with the following information:

- "Large scale cross-sections through the property showing the subsurface geology which will help identify areas which will require a higher level of investigation,
- Geomorphological map of the site
- Liquefaction analysis
- One dimensional static settlement analysis
- Consideration for lateral spread along the stream/drainage margins"

An investigation report, prepared by LDE dated 7 May 2021 with reference 19457 was reviewed. The key points summarised in the report includes:

1. "*Specific consideration will be required for the points summarised within this document when developing the scheme plan."* and

2. "*Consolidation and settlement analysis should be conducted in more detail and be site specific for the different stages of the scheme plan, with remediation methods considered to overcome potential consolidation settlement."*

The design and construction of 3 waters infrastructure, other infrastructure, and buildings would have to consider the restrictions and recommendations included in the LDE report. The selection of products, materials and construction methodologies should align with these recommendations to mitigate risks associated with differential settlement and should exhibit resilience characteristics in environments with ongoing consolidation.

It is strongly recommended that the findings, recommendations, restrictions, and limitations contained in the geotechnical report from LDE is considered, when the subdivision development staging is being contemplated.

2.2. Natural Hazards

The Northland Regional Council (NRC) natural hazard layers have been reviewed to identify potential hazards to be considered in this assessment. According to the NRC hazard layers, the site is not located in an area susceptible to:

Flood Risk

River flood hazards and coastal flood hazards, for up to and including the 100-year reoccurrence interval (including in a rapid sea level rise scenario). A small portion of the site is depicted on the updated regionwide flood hazard maps to be susceptible to river flooding during all event ranging from the 10year to 100yeaar reoccurrence interval.

Flood Susceptible Land

Gley Soils are indicateed along the south-eastern boundary of the subdivision, marginally within proposed footprint area for development, the site footprint areas for development fall outside this area.

[1]*Gley Soils are strongly affected by waterlogging and have been chemically reduced. They have light grey subsoils, usually with reddish brown or brown mottles. The grey colours usually extend to more than 100 cm depth. Waterlogging occurs in winter and spring, and some soils remain wet all year.*

Tsunami Risk

The NRC hazard layers indicate that the site is in an area marked as a safe zone in respect to Tsunami Evacuation Areas.

[1] *Gley Soils – Soils Portal: Characterising Soils » New Zealand Soil Classification (NZSC) » Soil orders » Gley Soils [G] - https://soils.landcareresearch.co.nz/describing-soils/nzsc/soil-order/gley-soils*

Figure 5: Flood Susceptible by Soil Type (Source: NRC Natural Hazard Map - Superseded)

Figure 6: Updated Regionwide Flood Hazard Maps (Source: NRC Natural Hazard Map)

3.0. Planning Assessment

This engineering assessment is undertaken to establish the total engineering infrastructure requirements to support such proposed development, however it is equally important to compare such requirements to the objective and outcomes included in district and regional policies. This will enable our client and local authorities to establish a baseline from where the additional impact of the development can be determined.

3.1. Kaipara District Plan

The Operative Kaipara District Plan depicts the site within the rural zone.

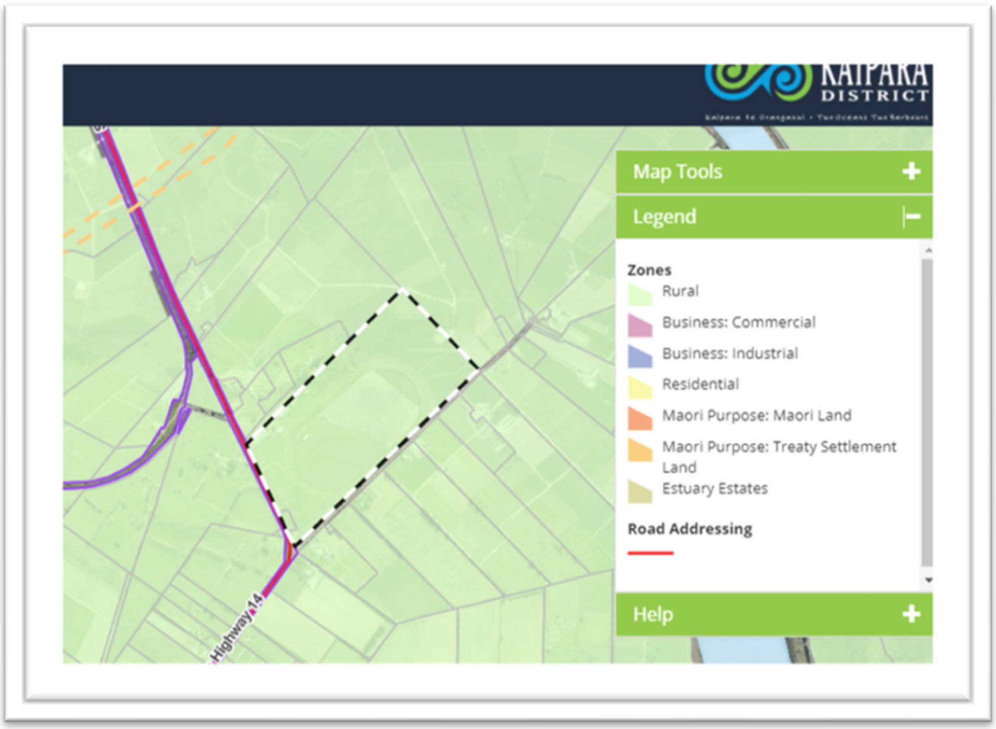


Figure 7: Current KDC Zoning (Source: KDC District Plan Map)

The Indicative Growth Area for Dargaville does not include the subject site as being a growth area in the Operative District Plan. Therefore, it is not envisaged that infrastructure development and planned capital works programmes would have included the subject site as an area of benefit.

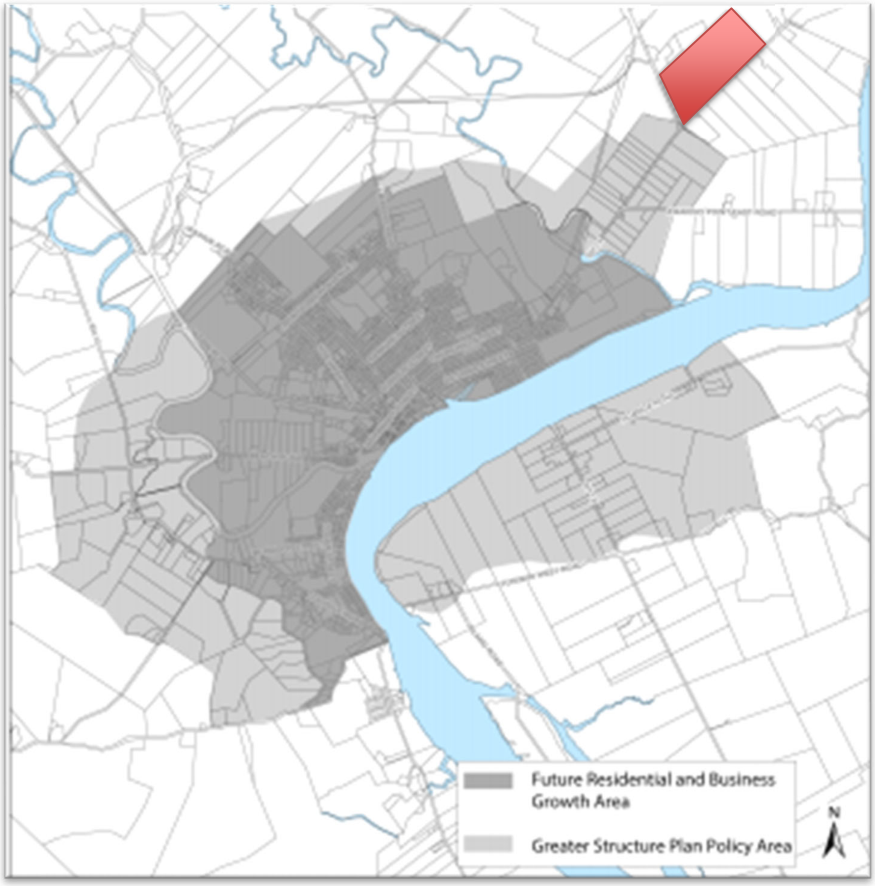


Figure 8: Mapped Growth Areas for Dargaville with site area indicated in red (Source: KDC District Plan Appendix A)

3.2. Kaipara Long Term Plan

The current Kaipara District Long Term Plan was adopted in June 2021, therefore considered to be current and relevant to matters associated with the proposed development of the subject site.

[2] The Long-Term Plan (LTP) sets Council's strategic direction and work programme for the 10 years ahead. It outlines the services Council will provide, the projects that will be undertaken, the cost of doing this work, how it will be paid for and how Council will measure performance.

Items identified in the long-term plan relevant to this assessment:

- Council have identified a project to upgrade the Dargaville Wastewater Treatment Plant to Increase Capacity (2024/2031).
- Council have listed the security of water supply for Dargaville as a challenging risk and issue and have identified several projects for the feasibility study, consents for water take and business cases for water storage.

[2] KDC Long Term Plan 2021 - 2031 – Web page cover statement on Long Term Plan: https://www.kaipara.govt.nz/ltp

3.3. Proposed Kaipara District Spatial Plan

The Kaipara District Spatial Plan for Dargaville have identified the subject site to be within new Industrial Zone, with 184ha area in Awakino Point to be developed, creating approximately 420 to 1472 new allotments, based on minimum lot sizes ranging between 1000-3500m^2.

The key observation in respect to infrastructure, is that Council envisaged that development is projected to start beyond the 10 years scope of the Long-Term Plan.

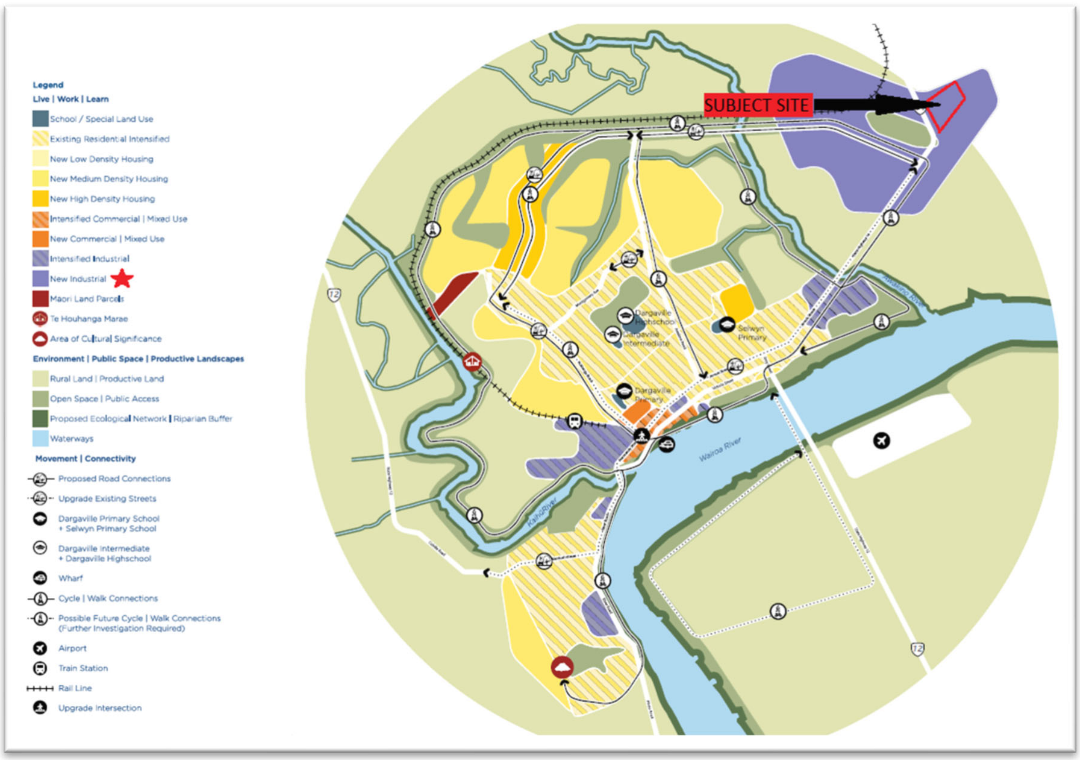


Figure 9: Spatial Plan for Dargaville depicting the subject site within new Industrial Zone (Source: KDC Spatial Plan)

4.0. Engineering Assessment

4.1. Wastewater Disposal

4.1.1. Existing Public Network

The KDC Asset Maps depict no wastewater network in proximity of the site. Therefore, wastewater collection and disposal will be through a combination of gravity and low-pressure systems, with storage and conveyance capacity to counter events that may impact the capacity of the public networks and treatment plant.

Although the site is relatively flat, it will be possible to construct an internal gravity system that will collect, convey, and store wastewater, from where it can be pumped towards the point of disposal, currently envisaged to be the Dargaville Wastewater Treatment Plant.

An internal low-pressure network can also be considered as a feasible option, however, does involve a rigid operation and maintenance regime. The benefit of low-pressure systems is that it provides better discharge and flow control and reduces the risk of inflow and infiltration.

The Dargaville wastewater treatment plant is approximately 3km from the site. The site has a mean elevation of 5MSL whereas the treatment plant is at 3MSL. Although the treatment plant is generally lower than the site, the distance to the treatment plant will render a gravity system unfeasible. Thus, all wastewater to be conveyed via a pressure pipeline. More information on the existing network and wastewater treatment plant was sought from Council and its Maintenance Contractor (Ventia).

Wastewater from the development will have to be conveyed via pump scheme to discharge to the existing Dargaville Wastewater Treatment Plant. A conservative static head from the invert of the storage tank on site to the discharge point at the treatment plant is taken as 2m.

A connection to Council's "Pump Station 14" may also be possible, subject to capacity, saving the construction of approximately 420m of the 3km rising main.

4.1.2. Wastewater Generation

With the anticipated land use, the following wastewater runoff is estimated at a point when the site is fully developed, determined in accordance with NZS440:2010.

	Population	Expected flow (l/d)
Light Industrial	556	137,233.44
General Residential	680	142,800.00
Lifestyle Lot Residential	15	3,150.00
Retirement* *(provisional)*	312	81,900.00
Neighbourhood Centre	50	10,500.00

** Calculations assume that there is a likelihood of provision for retirement housing, however in keeping with a 'worst case' scenario have based all calculations for retirement as being residential.*

Discharge from the site is recommended to be via 150mm dia PVC-U rising main, estimated to be approximately 3km long. Pump station with a wet well with storage provision to Council's requirements for wet weather flow conditions would be required.

It is noted that the 6 "Lifestyle Lot Residential" allotments are intended to rely on on-site wastewater treatment and disposal. Detailed assessments will be required at subdivision stage.

The table below provides a rough calculation to determine the parameters of a rising main to discharge wastewater from site.

Desired Minimum Velocity (m/s)	0.9
Total Average Daily Flow (m^3/d)	375.583
Peak Flow (6 Hour Pump Cycle) (l/s)	**8.694**
Mannings (f)	0.011
Peak Flow (m^3/d)	938.959
Peak Flow (m^3/s)	0.011
Peak Wet Weather Flow (m^3/s)	0.022
Diameter of Pipe for PWWF (m)	0.110
Flow Velocity (m/s)	0.914
Length of Pipe (m)	3000.00
Static Head (m)	2.00
Head loss through Pipe (m)	51.148
Total Head (m)	**53.148**

The duty points to convey estimated flow from the site to the pump station is estimated at 8.964 l/s @ 53.148 m dynamic head, for a 12-hour daily pump cycle. Any proposed low-pressure sewer systems and scheme conveyance pump station will be subject to specific design in accordance with section 7 of Council's Engineering Standards. Lower delivery heads could be achieved, however is recommended that a minimum velocity of at least 0.75m/s is maintained.

A PRELOS (or "Pressurised Liquid Only Sewer") system can also be considered for the development. Previously known as STEP systems, provides on-site retainment and digestion of solids, limiting off-site discharge to liquids only for further treatment.

Typical benefits of a PRELOS system compared to standards LPS systems to consider:

- Provides 50% or more treatment on the owner's property before discharging effluent to the network or treatment plant, a key consideration for addressing concerns raised in the CIA.
- Pumps are less susceptible to wear, and maintenance compared to grinder pumps.
- Tanks reduce solids by up to 80% and need desludging generally once a decade, reducing the solids loading on Council's treatment plant, which from understanding must be de-sledged on a regular basis.

4.1.3. Rising Main Construction

The construction of a rising main from the site to the wastewater treatment plant would require the proposed pipeline to be constructed in the public road reserve where available, and across private property were insufficient reserve space is available. A crossing over Awakino River on State Highway 14 (entering Dargaville) would also be required.

The bridge across the river currently hosts several services such as water, electrical cables, and comms as shown in figure 11.

Specific approval for the crossing would be required from NZTA. It is recommended that this requirement be addressed together with access connections between Dargaville and the development.

Figure 10: SH14 Bridge 545 Across Awakino River

4.2. Stormwater Management

4.2.1. Overview

The site has a mixture of grassed with isolated thicker vegetated areas. Stormwater drains have been formed diagonally across the site, appearing to be on the alignment of the natural overland flow paths depicted on Council maps. There is no clear connection and discharge points for these drains, however it may not be evident due to a lack of maintenance. It is assumed that the drains discharge to the table drains along Awakino Point North Road.

A review of the geology of the site and characteristics of the area, it is discovered that the permeability rate of the underlain soils is considered very low and negligible. The site appears to be generally waterlogged.

4.2.2. Current stormwater runoff discharge from the site

- Runoff flows overland towards the south-eastern boundary where runoff is intercepted by the overland flow paths,
- Channelled flow is directed and discharged towards the roadside table drain along the northern side of Awakino Point North Road.
- No culverts have been identified across Awakino road.
- Council's Utility Services Maps indicates that all overland flow generated by the site is conveyed towards the north-eastern end of Awakino Point North Road, from where it is directed south-eastwards towards the discharge point into the Wairoa River shown in figure 12 below.

Figure 101: Overland Flow Paths (Source: KDC Utility Services Maps)

4.2.3. Kaipara District Council Engineering Standards

Section 6.1.1 – Minimum Requirements in the KDC Engineering Standards provide that "on-site stormwater detention shall be provided to attenuate post development peak stormwater flows to no more than pre-development peak flows for storm events of up to 100 year ARI (1%AEP)."

Guidance Notes under section 6.1.1 includes the following:

- "2. Where stormwater attenuation is required, stormwater detention ponds or basins should be provided to serve the entire site catchment. A proliferation of small stormwater ponds or individual detention tanks will not generally be accepted because they are not as reliable or efficient as larger detention ponds or basins.

- Water quality treatment options should be considered where appropriate, particularly in conjunction with stormwater attenuation."

It is recommended to undertake a catchment wide management approach at resource consent stage, however for the purpose of concept development, a high-level analysis is undertaken to provide outcomes, sufficient to inform a more detailed concept development design, which includes stormwater detention locations, size and type, network.

4.2.4. Stormwater Management Approach

A numerical model was developed with the aid of a spreadsheet powered storage calculator, to estimate the runoff from the site pre- and post-development, determine the system volume, peak runoff and stormwater volume required.

The predevelopment state is taken as the site at current status quo where stormwater is conveyed through the site overland and through natural overland flow paths, discharged to the roadside table drain.

The post development state is allowing for the added impermeable surfaces (73.67%) associated with the anticipated development as indicated by the draft concept development plan.

The management approach is to maintain the pre-developed status quo post development, by attenuating runoff from the developed site, to ensure that peak discharge post development is equal or less to that estimated for the predeveloped state.

A second, more detailed deterministic model was built for the pre- and post-developed site to verify the numerical model and provide more detailed results. The modelling approach is to route the flow from the entire site through three detention devices, to throttle the post development flows to a value equal or less than the pre-developed state, whilst checking the flow depths of the downstream table drains.

4.2.5. Runoff Calculations

HIRDS V4 Depth-Duration-Frequency Results was utilised for the rainfall data. The rainfall was applied for respective storms from 10minutes to 24 hours for the 1% AEP event, utilising the Rational Calculation Method, analysing for the critical storm duration.

Rainfall depth selected (for 1%AEP event, Longitude: 173.901 Latitude: -35.912)

Pre-development - Historical Data

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h
100	0.01	21	29.2	35.3	48.5	65.8	104	135	171	212

Post Development – RCP8.5 for the period 2031-2050

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h
100	0.01	23.4	32.5	39.3	54.1	73.1	114	146	184	226

A copy of the HIRDS V4 Depth-Duration-Frequency Results is included in ***Appendix B***.

The catchment boundaries are limited to the boundaries of the site under this assessment. A more detailed analysis would be required to size and design infrastructure to cater for the conveyance of flow of runoff through the site from areas that may fall outside the boundaries of the site, but part of the contributing catchment.

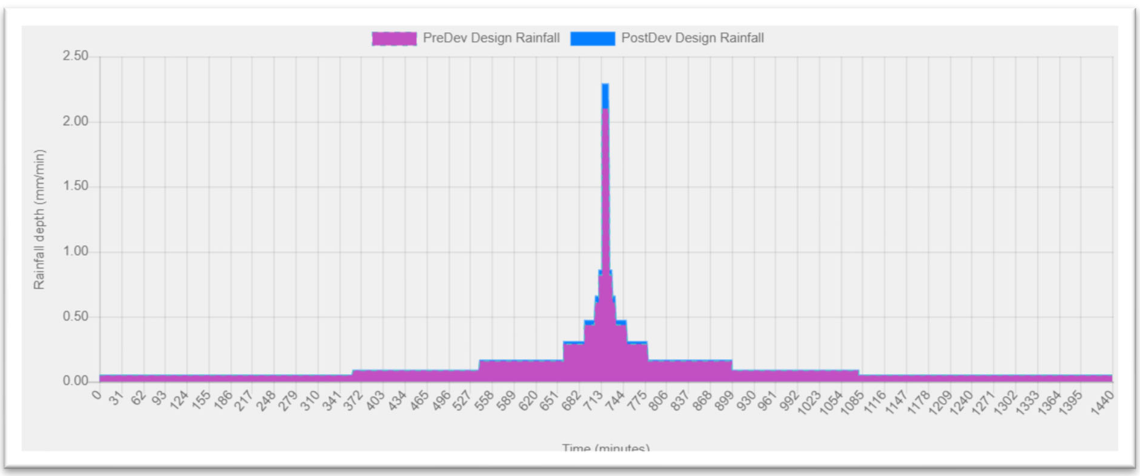


Figure 12: Pre and Post Development Rainfall Applied

<u>Deterministic Model</u>

The catchment for the model can be described as:

- Light Industrial
- Residential
- Open Space

- Infrastructure

Total Area (surveyed) 45.06ha

For the purposes of this report calculations are based on a high-density development to ensure a 'worst case scenario' for impermeable surfaces and are based on the conceptual model of the Outline Development Plan V4.0.

Thus, the model is as follows:

- Light Industrial –100% impervious surfaces
- Residential – 70% impervious surfaces
- Lifestyle Lot Residential – 40% impervious surfaces
- Neighbourhood Centre –100% impervious surface

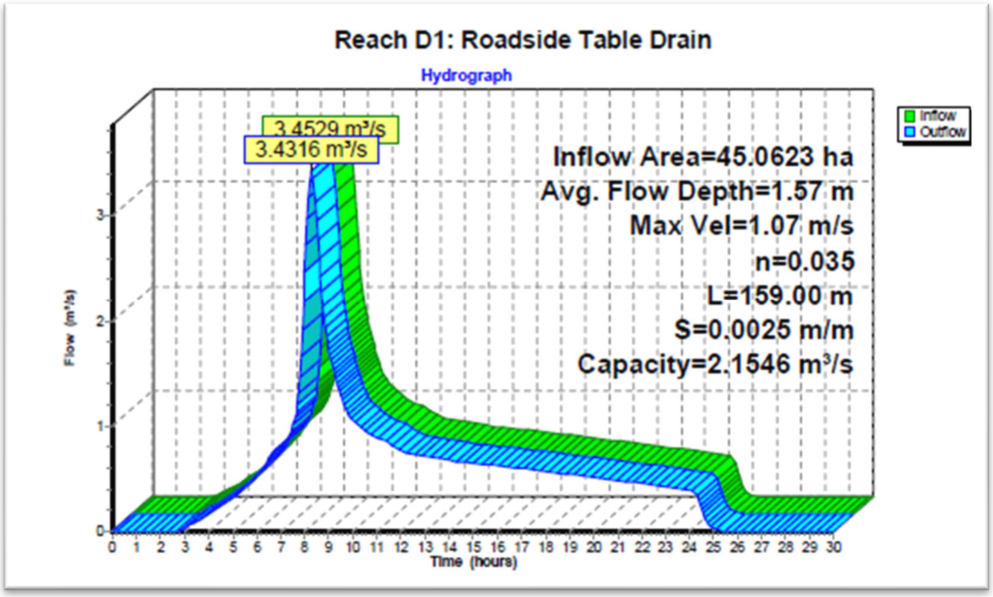
Note that the impervious surfaces are calculated at near 100% for infrastructure and do not make allowances for road reserve areas (including road verge, swales, buffers and other roadside stormwater mitigations and detention ponds)

Pre-Development calculations indicates that the stormwater flows across the site and enters the drain along Awakino Point North Road.

Calculations provided show that post-development will have a positive impact for both the 20% and 1% rainfall events.

Post-Development modelling results indicate that there is a reduction in peak runoff velocity and flow rate for the 1% rainfall design period from 3.4316 m3/s to 2.8475 m3/s and 1.07 m3/s to 1.04 m3/s respectively.

A copy of the detailed stormwater model report is included in ***Appendix C***.



Dargaville Pre-Devolpment *Type IA 24-hr 1% Pre Dev Rainfall=171 mm*
Prepared by Lands and Survey Printed 13/01/2022
HydroCAD® 10.10-7a s/n 12332 © 2021 HydroCAD Software Solutions LLC Page 6

Summary for Reach D1: Roadside Table Drain

[91] Warning: Storage range exceeded by 0.372 m
[55] Hint: Peak inflow is 160% of Manning's capacity

Inflow Area = 45.0623 ha, 0.00% Impervious, Inflow Depth = 124 mm for 1% Pre Dev event
Inflow = 3.4529 m³/s @ 8.21 hrs, Volume= 55.924 Ml
Outflow = 3.4316 m³/s @ 8.29 hrs, Volume= 55.924 Ml, Atten= 1%, Lag= 4.5 min
Routed to Reach D2 : Roadside Table Drain

Routing by Stor-Ind+Trans method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Max. Velocity= 1.07 m/s, Min. Travel Time= 2.5 min
Avg. Velocity = 0.60 m/s, Avg. Travel Time= 4.4 min

Peak Storage= 509.0 m³ @ 8.25 hrs
Average Depth at Peak Storage= 1.57 m , Surface Width= 3.20 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 2.1546 m³/s

1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 159.00 m Slope= 0.0025 m/m
Inlet Invert= 3.800 m, Outlet Invert= 3.400 m

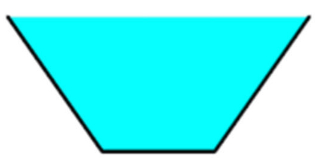
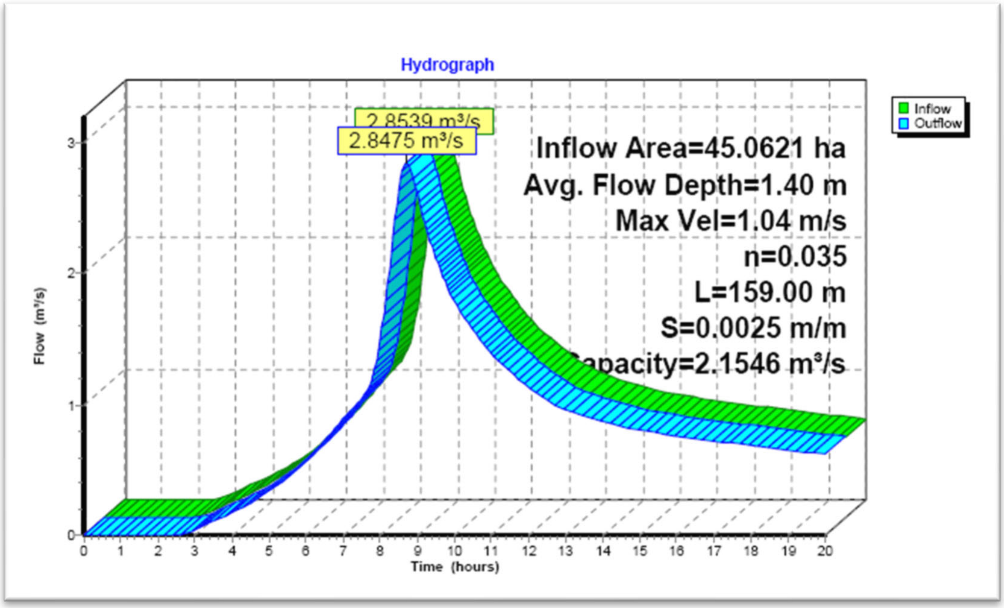


Figure 13: Hydrograph and Summary of Pre-development stormwater calculations



Dargaville Post Devel *Type IA 24-hr 1% Post Dev (RCP8.5 - 2031-2050) Rainfall=184 mm*
Prepared by Lands and Survey — Printed 13/01/2022
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Summary for Reach D1: Roadside Table Drain

[91] Warning: Storage range exceeded by 0.201 m
[55] Hint: Peak inflow is 132% of Manning's capacity
[62] Hint: Exceeded Reach 1R OUTLET depth by 0.269 m @ 8.90 hrs
[81] Warning: Exceeded Pond 2P by 0.323 m @ 8.70 hrs

Inflow Area = 45.0621 ha, 68.03% Impervious, Inflow Depth > 132 mm for 1% Post Dev (RCP8.5 - 20
Inflow = 2.8539 m³/s @ 8.63 hrs, Volume= 59.289 Ml
Outflow = 2.8475 m³/s @ 8.71 hrs, Volume= 59.056 Ml, Atten= 0%, Lag= 4.7 min
Routed to Reach D2 : Roadside Table Drain

Routing by Stor-Ind+Trans method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 1.04 m/s, Min. Travel Time= 2.6 min
Avg. Velocity = 0.73 m/s, Avg. Travel Time= 3.6 min

Peak Storage= 436.6 m³ @ 8.67 hrs
Average Depth at Peak Storage= 1.40 m , Surface Width= 2.96 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 2.1546 m³/s

1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 159.00 m Slope= 0.0025 m/m
Inlet Invert= 3.800 m, Outlet Invert= 3.400 m

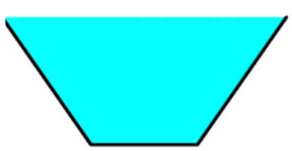


Figure 14: Hydrograph and Summary of Post-development calculations

The downstream roadside table drain (downstream receiving) was included in the model to check that conveyance capacity is maintained.

Figure 15: Picture depicting roadside table drain downstream of site

4.2.6. Stormwater Detention

The deterministic model results indicate that 3 detention ponds totalling 11,884m^3 storage is required for the respective sub catchments to ensure the post development discharge from the development is limited to the current pre-developed discharge. Details of these ponds are shown in figures 18-23.

The storage volume quoted above is based on storage device (pond) with a storage depth of approximately 1.2m deep. The depth has been determined based on the report be LDE (ref 19457, dated 7 May 2021) where ground water was encountered at depths ranging from 1.5m to 3.5m bgl with an assumed 0.5m bgl during winter. Following on from their analysis of the site geology, a ground water depth of 1.5 m bgl was adopted. Therefore, a conservative depth of 1.2m bgl was applied to the calculations we have provided.

Storage devices are detailed as trapezoidal structures with side slopes of 1:3, where a total reserve area to be set aside for stormwater detention with adequate buffer areas for riparian planting and landscaping.

For the purposes of this report the detention calculations have been presented to show that it is possible to improve stormwater management. A more economical solution is likely to be achieved at subdivision stage when a more detailed design would be undertaken that will include analysis to determine efficient use of storage, treatment and conveyance devices as pipe diversions, swales, vegetated filters, artificial wetlands and landscaped areas.

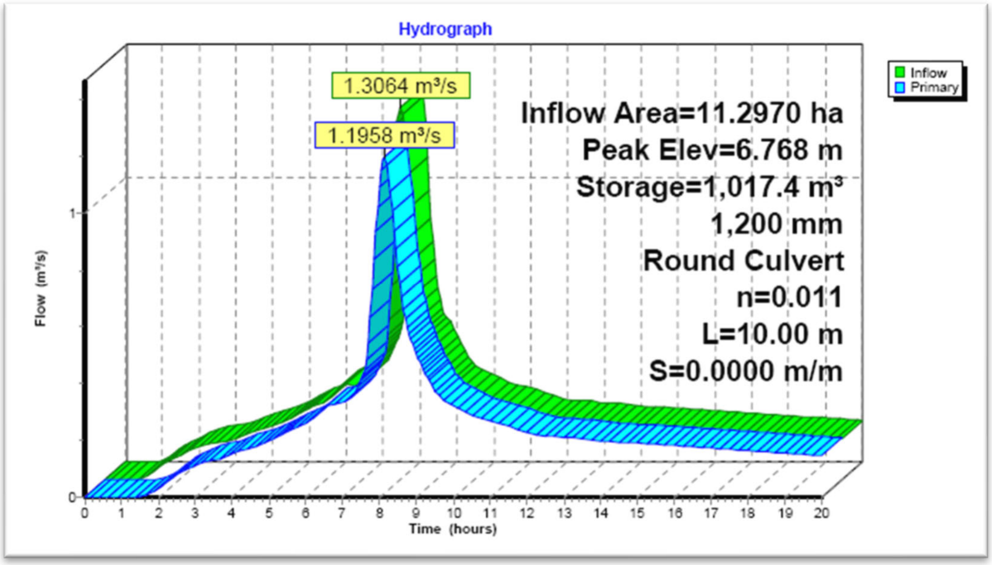


Figure 16: Detention Pond 1 Hydrograph for 1% Rainfall

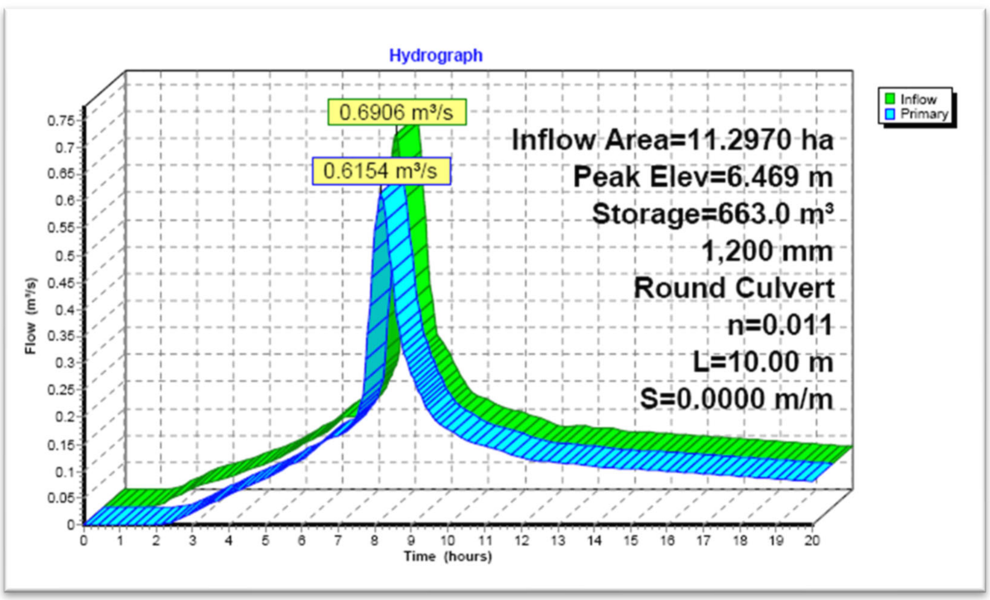


Figure 117: Detention Pond 1 Hydrograph for 20% Rainfall

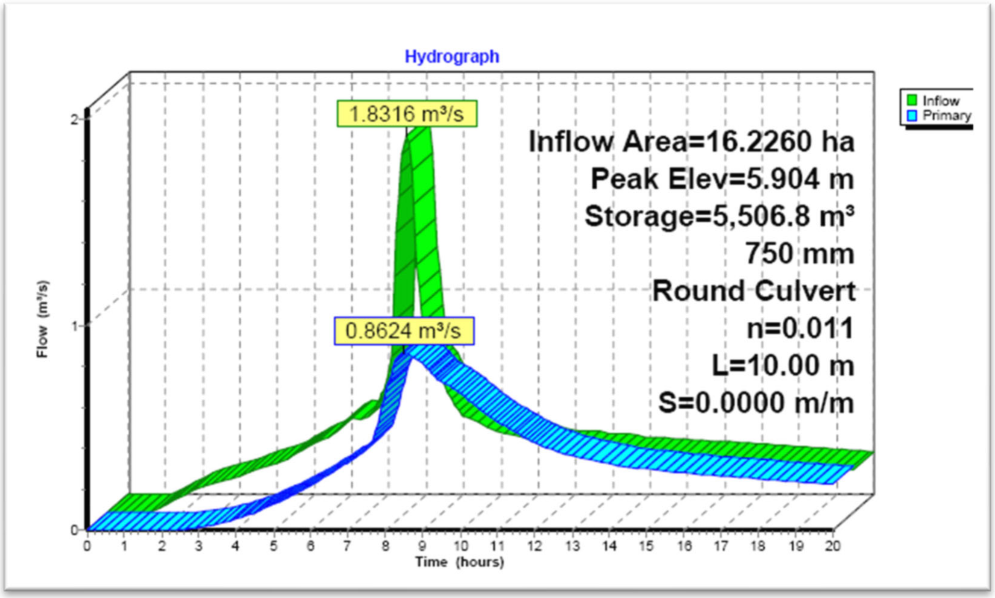


Figure 18: Detention Pond 2 Hydrograph for 1% Rainfall

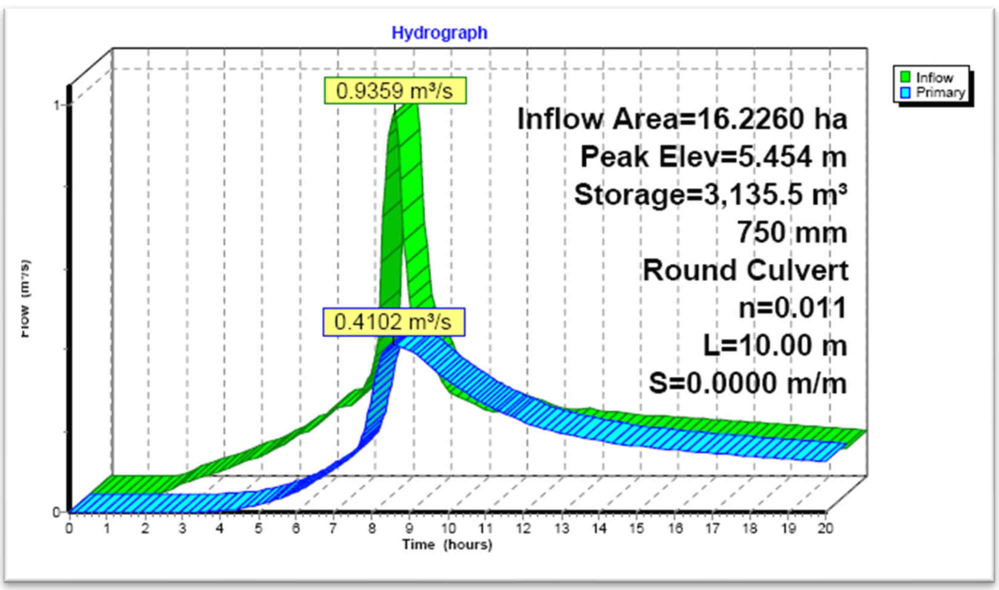


Figure 19: Detention Pond 2 Hydrograph for 20% Rainfall

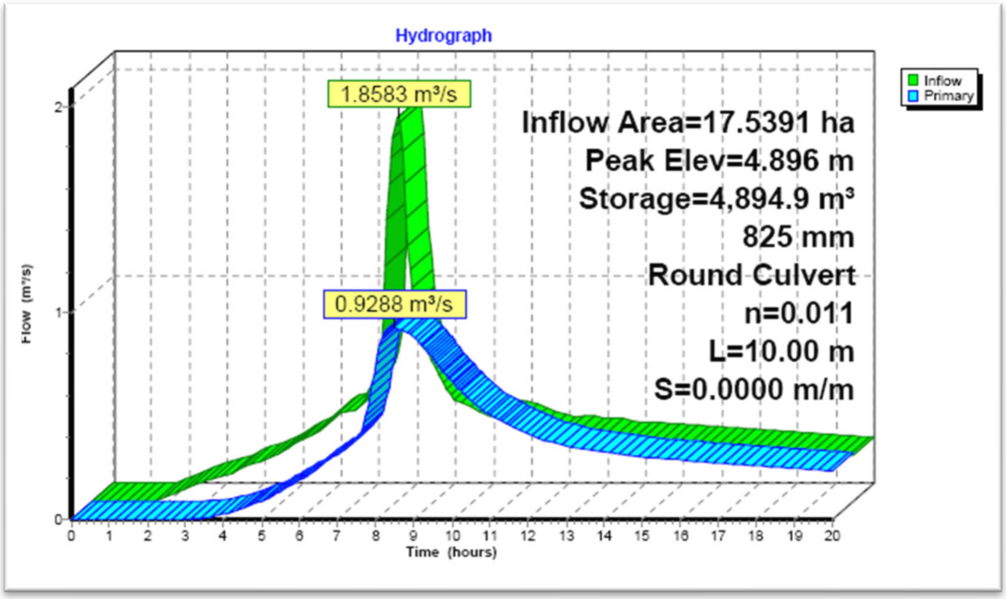


Figure 20: Detention Pond 3 Hydrograph for 1% Rainfall

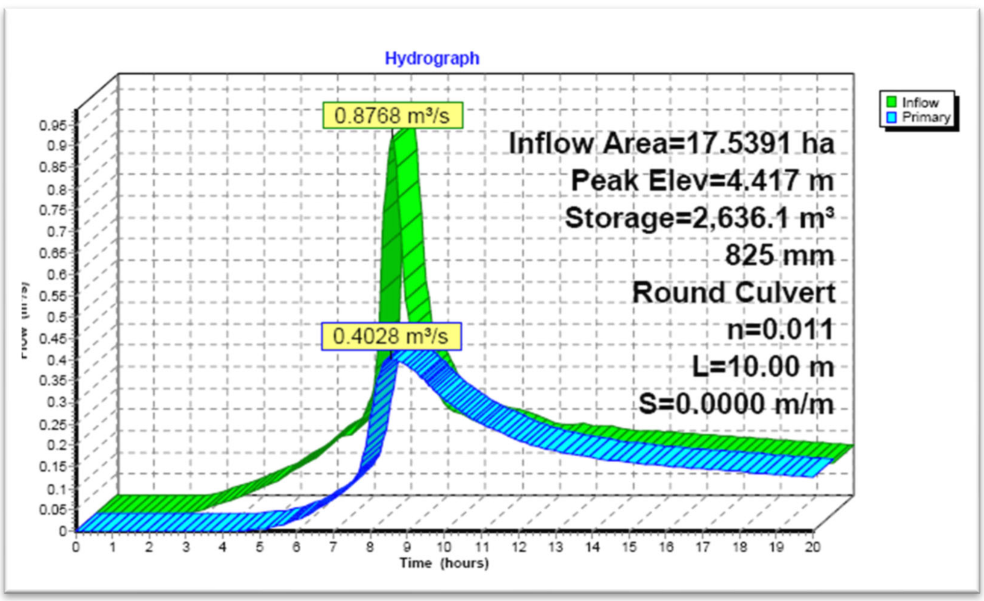


Figure 21: Detention Pond 3 Hydrograph for 20% Rainfall

4.3. Potable Water Supply

The Council water network is immediately available shown in figure 21 below. Feedback from Council confirmed that the network and treatment plant have current capacity to serve the proposed development, however several future challenges are anticipated, with a consistent and reliable supply of raw water to the treatment plant being one of the major challenges.

Figure 22: Map extract depicting 100mm and 180mm diameter water mains along the western and southern boundaries of the site (Source: KDC Utility Services Maps)

4.3.1. Network Capacity

To understand Council's network capacity to serve ongoing development and future growth, AWA Environmental have developed a network model of the Dargaville water network on behalf of Kaipara District Council. Lands and Survey Engineering engaged AWA to undertake a high-level analysis to identify the nodes and links likely to be point of constraints to meet the demand of the proposed development.

Although it was established that the existing network will have capacity to meet the additional demand created by the proposed development, it is recommended that a more detailed analysis is required, which includes a staged approach together with population growth for Dargaville, to inform the design of a staged development and integrate and align development with Council's capital works programme. It is also noted that the concept development plan has changed substantially since the analysis was undertaken, however the effect of such changes on the hydraulic performance of the network is expected to be less than minor.

A copy of the network model assessments by AWA Environmental is included in ***Appendix D.***

4.3.2. Water Treatment Plant Capacity

Investigation and feedback from Council's O&M contractor indicated that Dargaville water treatment plant have adequate capacity to meet demand for development and growth, subject to consistent and reliable raw water supply. This assessment finding is subject to further investigation and confirmation from Council at resource consent stage. Feedback from Council's maintenance contractor stated that Council's "Potable water plant (settling/rapid sand filtration) design flow rate at 210 m^3/hr, will require consent renewals and favourable seasonal conditions to achieve this. At present averaging around the 120-130 m^3/hr", whereas the network modelling undertaken by AWA Environmental undertook an assessment of the theoretical water drawn from the plant, with the following results:

	Pre-Development	Post Development
Maximum Flow (l/s)	111	128
Total Water Take (m^3)	3155	3533

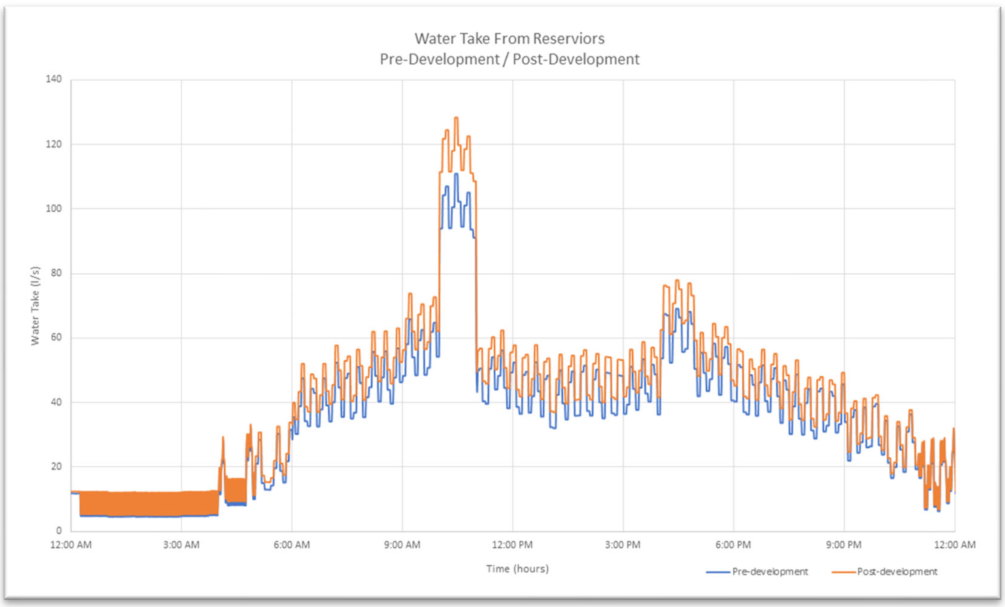


Figure 23: Water Take analysed over 24hour period (Source: AWA Environmental Hydraulic Model – Dargaville Water Reticulation Network)

The above result indicates that a total of 147.21 m^3/hr must be produced by the plant to serve the network, with the proposed development included. This theoretical demand is within the production design capacity communicated by Council's contractor, however processing enhancements may be required to increase the production from the current 120-130m3 per hour, as reported.

4.3.3. Raw Water Supply

Investigation and feedback from Council, and its contractors indicate that the supply of raw water continues to be an issue, especially during summer. Council have experienced significant shortages in raw water supply, resulting in restrictions in capacity to produce potable water for Dargaville. Several projects have been identified towards providing relief to the water shortages during dry seasons, however status and progress on these projects are unknown.

4.3.4. Alternative Water Supply

It is recommended that development includes conservative approaches to the management of rainwater runoff and harvesting. Effective rainwater harvesting can reduce the system demand substantially.

Groundwater by way of community bore is another potable water source that can be explored. Bores and extraction of groundwater would be subject to resource consent from Northland Regional Council (NRC). An enquiry to NRC was submitted to query the current groundwater model. Initial feedback from NRC indicated that there is an unrestricted supply on site however, drainage through the site to be considered. Water levels and quality is unknown at this stage.

4.3.5. Water Demand

Water demand for the proposed development will gradually increase as the development progress through the stages. No information on staging and, priority areas or timelines are available at this stage, therefore water demand calculations and the network analysis are based on the development, connected to the network at its current status.

Water demand for the anticipated land uses is indicated below:

	Water Requirements (NZS4404:2010)									Firefighting Requirements	
	Area (ha)	Lot size (m^2)	Total Lots	Average daily flow (l/p/d)	Peak Flow Factor	Occupancy Rate	Population	Average Daily Flow (m^3/d)	Peak Daily Flow (m^3/d)	Supply Requirements (l/s)	Supply Requirements (m^3/h)
Small Industrial	0.6668	476	14	135	2	4	56	7.56	15.12	50	180
Large Industrial	8.8633	8863	10	135	2	50	500	67.5	135	50	180
General Residential	15.56	572	272	250	2	2.5	680	170	340	25	90
Lifestyle Lot Residential	2.7252	4542	76	250	2	2.5	15	3.75	7.5	25	90
Retirement* *(provisional)*	4.6714	299	156	250	2	2.5	312	97.5	195	25	90
Neighbourhood Centre	0.2862	2862	1	125	2	50	50	6.75	13.5	25	90
Totals							1613	353.1	668		

** Calculations assume that there is a likelihood of provision for retirement housing, however in keeping with a 'worst case' scenario have based all calculations as residential.*

The daily demand being estimated at 353.06 kl, it may serve the development to consider various forms of on-site storage or a communal elevated command reservoir, supplemented by on site rainwater harvesting and groundwater supply for specific use.

4.3.6. Connection Size and Location

It is envisaged that a connection will be available on the 180mm diameter pipeline along State Highway 14, however in terms of table 6.2 of NZS 4404:2020 (Empirical guide for principal main sizing) indicates that for single direction feeds the following connection would be required:

	Nominal Diameter of Main (mm)
Light Industrial	150
General Residential	150
Residential/Retirement	100
Neighbourhood Centre	n/a

This indicates that the Council supply may suffice for consumption supply, however, may be restricted under peak demands or to meet firefighting flows at adequate pressure. Detailed design and analysis will be required at development stage to ensure adequate provision of water for firefighting purposes.

4.4. Erosion and Sediment Control

4.4.1. Purpose

The purpose of sediment and erosion control plans are to describe and detail the methods and practices recommended to minimise the effects of sediment generation and yield on the receiving environments associated with earthworks and other activities during and after development.

4.4.2. General Principles

Erosion and sediment control measures will be undertaken and implemented with a hierarchy and priority order as follows:

- Avoidance of effects will be the priority. Any discharge locations will be carefully selected, and any stream works will only be undertaken where they are a necessary component of the Project construction.

- Erosion control will be a priority in all circumstances by preventing sediment generation through a range of structural (physical measures) and non-structural (methodologies and construction sequencing) means.

- Sediment Retention Ponds (SRPs) will be utilized where appropriate and if required. Priority of controls will then be decanting earth bunds, super silt fences and silt fences. Various innovative products may also be used and could include measures such as filter socks.

Erosion and sediment control measures will be implemented in accordance with this ESCP and are based upon appropriate solutions for the site to achieve an outcome, that ensures no adverse effect to the receiving environment and comply with anticipated resource consent conditions as a minimum.

No specific discharge water quality standards are proposed; however, the discharge from the Project should be designed to avoid conspicuous change in the colour or visual clarity of the discharge (after reasonable mixing) in the receiving environment.

All erosion and sediment control devices should be located outside the 5% AEP flood level unless no other viable alternative exists. During construction activity and where it is considered to be the only option and devices are required within this flood level, then the placement of such a device will be undertaken with consideration of minimizing catchment areas and ensuring more regular maintenance activities.

4.4.3. Erosion And Sediment Control Methodology

All sediment and erosion control measures recommended, to be implemented prior to commencement of any earthworks.

Exposed areas should be limited during construction where possible to reduce the potential for generating erosion and wash from site.

Clean water diversion channels or benching should be constructed above the works area to divert any runoff from entering the site where bare soils are exposed. Outlets must discharge water down slope in a controlled manner.

Silt fences to be utilised to prevent silt transportation during construction phases. Silt fences are recommended to be placed along the downstream edge of runoff diversion bunds and areas where overland flow is expected that have the potential to transport sediment during rainfall events.

Excavated material must be stockpiled away from sensitive areas and on vegetated areas that act as runoff buffers.

Runoff from the works area to be diverted by earth bunds to low points as indicated on indicative layouts attached.

Bunds can be constructed using any topsoil found onsite. The bund construction on steeper slopes (>2%) must be stabilised. Method of stabilisation can be proposed at construction stage; however, it is recommended that such proposals are to be assessed and endorsed by an engineering professional.

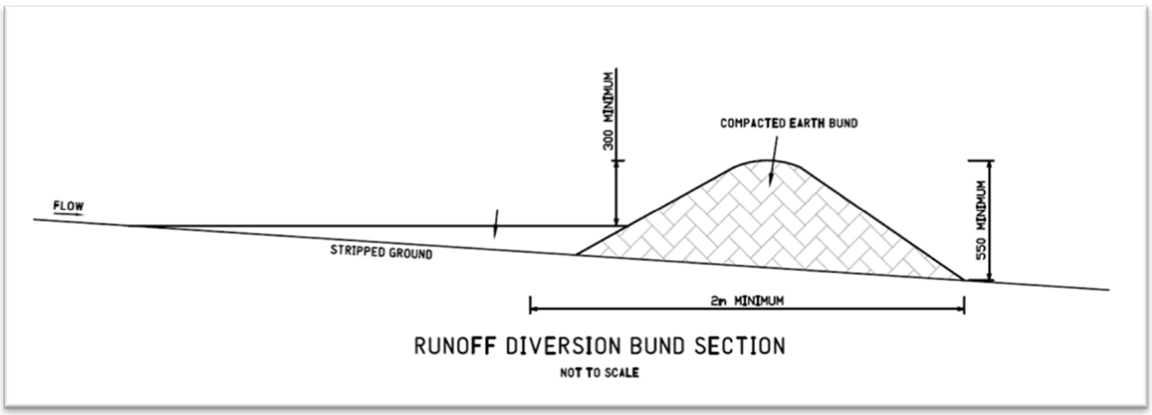


Figure 24: Typical Section of Recommended Earth Diversion Bund

Drainage should follow a uniform gradient so as not to cause sediment build up and impede flow.

A desktop assessment of the site identifying possible settlement and decanting points (at low points) can be located where water is likely to gather.

Log rolls, cascades or rip rap material must be placed in drain outlets where flow is concentrated in order to reduce flow velocity that will minimise the risk of erosion and mobilising silts and soils.

It is recommended that erosion controls and silt detention measures be inspected and maintained weekly, and that any sediment build up is removed. Check the inlet and outlet for signs of scouring and implement remedial fixes immediately.

Details and guidelines for the design and construction of decanting earth bunds and silt fences are included in ***Appendix G.***

Heavy Rainfall Response:

1. Works to be planned for periods with settled weather where possible.
2. Following any heavy rainfall warning, the erosion and sediment control measures must be inspected, repaired and cleaned (if necessary). Exposed surfaces to be prepared by removing loose material and ensuring silt fences are in place to contain sediment laden runoff.
3. If extreme weather is forecast, the site should be secured and protected as much as practicable.
4. Machinery and loose material must be removed from site or moved into a location where it cannot be mobilised by overland runoff.
5. Exposed slopes to be grassed, and silt fences will not be removed until the grass has established.
6. No machinery to be entering watercourses and areas of concentrated overland flow stream, nor refuelled nearby.
7. Remove temporary controls only after works have ceased and the area below bund has been secured. Topsoil can then be reused onsite.

4.4.4. Monitoring

As part of the erosion and sediment control methodology, ongoing site monitoring by the Project team must occur to ensure that the proposed erosion and sediment control measures have been installed correctly, methodologies are being followed and are functioning effectively throughout the duration of the works.

Any measures requiring attention must be identified, and if necessary, relevant team members consulted to ensure continual improvement is sought. This may include undertaking further assessment of risk, including sediment yields. In the circumstance of higher risk areas being identified more stringent controls must be considered, in particular more progressive stabilisation.

Visual assessments of the receiving environment should continue to be undertaken during the works period by the Project team with particular attention during and after periods of rainfall and activities likely to increase the risk of sedimentation. In the context of visual assessment, the receiving environment is defined as the immediate receiving environment adjacent to the area of works. Any noticeable change in water clarity from that prior to the rainfall event, or upstream of the site of works as a result of the construction activity, will require a review of the erosion and sediment control measures implemented and changes to be made as necessary.

Weather forecast monitoring will also ensure that critical works likely to increase the risk of sedimentation will only occur during a suitable weather window. Internet weather forecast sites such as www.metvuw.co.nz will provide one of the key tools in this regard with local weather forecasting utilised as necessary.

4.5. Other Utility Services

4.5.1. Electricity

Accessibility to power supply for the proposed development will be sought from Northpower. Development specific enquiries will have to be lodged with the services provider at resource consent stage. Detailed feedback from the Northpower engineering team has been sought. Interim feedback from Northpower advised that *"there is about 0.25MVA spare capacity currently but with some upgrades there could be 0.75MVA capacity. If any more than this is required, a new feeder would need to be installed."*

4.5.2. Telecommunications

Fibre is not available in proximity of the site, nor is it planned at this stage; however, the site is currently within area of benefit for VDSL and Wireless as depicted below.

Figure 12: Map depicting special reach of telecommunication infrastructure (Source: https://broadbandmap.nz/)

5.0. Conclusion and Recommendations

Engineering services included in this assessment, required to deem the proposed development feasible (conceptually), and can be provided to support the proposed development. Complexities in servicing the proposed development and surrounding areas, will require further investigation and investment to determine appropriate engineering solutions consistent with the aspirations identified in Council's long-term planning and investment strategies. However, from this assessment, such solutions are not outside the norm for a development of this scale and nature.

5.1. Wastewater Disposal

Combined gravity / low pressure system required to collect and convey wastewater to single pump station and low pressure rising main to convey wastewater from proposed development to Council network/wastewater treatment plant. Discharge points a) Pump Station 14, b) Wastewater Treatment Plant.

Alternative low-pressure systems (liquids only sewer) where solids loading on the receiving treatment plants is reduced, could also be considered where the wastewater treatment plant is found to lack treatment capacity due to sludge build up, compounded by lack of pre-treatment and screening. Information on an alternative (Pressurised Liquid Only Sewer) system is included in ***Appendix F***.

5.2. Stormwater Management

The effect of the proposal in respect to stormwater runoff quantity and quality, is less than minor, provided mitigation by way of detention, on-site treatment and controlled discharge is provided.

Runoff generated from proposed development to be collected and conveyed through sealed pipe network and discharge to proposed attenuation and detention devices (ponds). Controlled release of treated stormwater to the receiving environment to ensure no adverse effect on downstream infrastructure, property, or environment.

Design, construction, and maintenance of proposed stormwater system to comply with relevant engineering standards and be designed to satisfy the provisions relating to *Te Mana o te Wai* and the objectives and policies for freshwater management in accordance with the National Policy Statement for Freshwater 2020.

5.3. Potable Water Supply

Council water supply is present in the direct vicinity of the proposed development. Network capacity to meet the demand of the proposed development was assessed and confirmed.

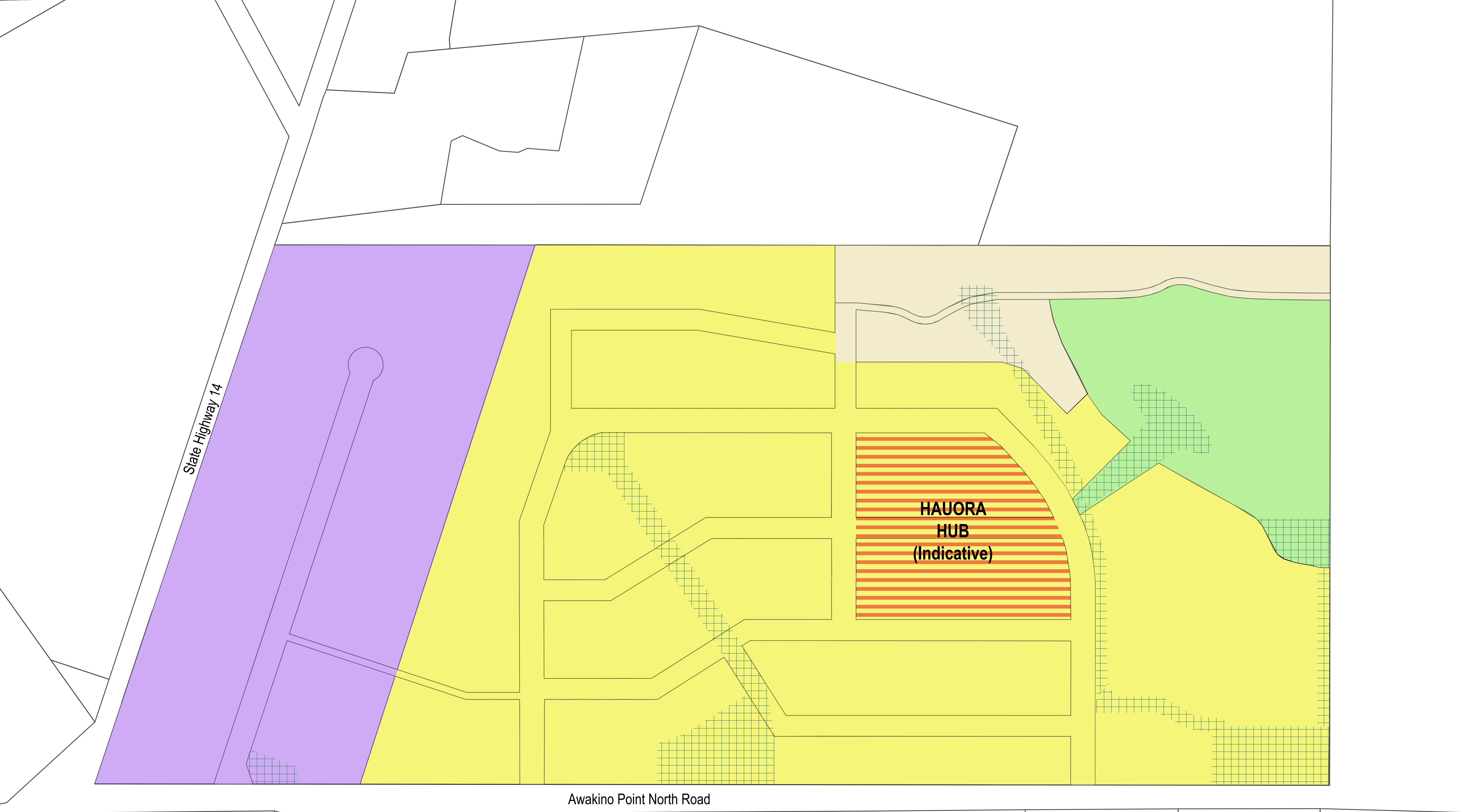
Although, water treatment plant capacity assessment is inconclusive, it has been indicated that the plant has capacity to supply the proposed development, albeit that seasonal shortage of raw water to the treatment plant may be one of the major constraints to meet the demand of the development and expected growth for Dargaville.

Alternative water supply or supplementary supply by way of rainwater harvesting and ground water supply should be considered. Testing and analysis of groundwater quality can be further investigated.

6.0. Limitations

This report has been prepared solely for the benefit of our client (Dargaville Racing Club Incorporated) and its authorised agents in relation to the private plan change application for which this document has been prepared. The comments herein are limited to the purpose identified within this report. No responsibility is accepted by Lands and Survey Engineering Limited for the accuracy of information provided by third parties and/or the use of any part of this report in any other context or for any other purposes. The reliance by other parties on the information or opinions contained herein shall, without our prior review and agreement in writing, do so at their own risk. This report is for the use of by our Client and should not be or relied upon by any other person or entity.

Appendix A - Concept Development Plan



PROPOSED AREAS

- Light Industrial (LIA)
- General Residential (GRA)
- Large Lot Residential (LLRA)
- Open Space (OSA)
- Hauora Hub - Mix of GRA, OSA & Neighbourhood Centre Area (NCA)

OTHER ELEMENTS (Indicative Layout)

- Blue-green Network
- Roading

THE URBAN ADVISORY
RETHINKING OUR CITIES

Matakohe Architecture + Urbanism

ISSUE	DATE	REVISION		
PROJECT	Dargaville Racing Club Redevelopment			PROJECT # 044
CLIENT	Tripartite Group	DATE #	28/01/22	DWG #
		SCALE @ A3	1:3000	V.6 .0
DWG	Trifecta Development Area Plan	DRAWN	MD/AL	
		CHKD	JK	REVISION

The Urban Advisory Ltd
74D France St S, Eden Terrace, Auckland 1010
www.theurbanadvisory.com

Matakohe Architecture and Urbanism Ltd
158b Bank St, Whangārei 0112
www.matakohe.co.nz

Appendix B - HIRDS V4 Depth-Duration-Frequency Results

HIRDS V4 Depth-Duration-Frequency Results
Sitename: Dargaville Racecourse
Coordinate system: WGS84
Longitude: 173.901
Latitude: -35.912

DDF Mode Parameter	c	d	e	f	g	h	i
Values:	0.001344	0.444183	-0.01219	-0.00146	0.25789	-0.01217	2.95172
Example:	Duration (l	ARI (yrs)	x	y	Rainfall Depth (mm)		
	24	100	3.178054	4.600149	171.0399		

Rainfall depths (mm) :: Historical Data

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
100	0.01	21	29.2	35.3	48.5	65.8	104	135	171	212	237	256	270
250	0.004	23.7	33	39.9	54.8	74.5	117	153	194	241	270	291	307

Depth standard error (mm) :: Historical Data

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	1.1	1.4	1.6	2.3	3.1	6	7.8	12	16	18	19	21
2	0.5	1.3	1.6	1.7	2.5	3.4	6.6	8.7	14	18	20	21	23
5	0.2	1.8	2.3	2.5	3.5	4.7	8.8	12	19	24	27	29	31
10	0.1	2.3	3	3.3	4.4	6	11	15	22	29	33	34	38
20	0.05	2.9	3.8	4.4	5.6	7.6	13	18	26	34	39	40	44
30	0.033	3.3	4.5	5.2	6.5	8.8	15	20	29	37	42	44	49
40	0.025	3.7	5	5.8	7.2	9.7	16	22	31	39	45	47	52
50	0.02	4	5.4	6.3	7.8	11	18	24	32	41	47	49	54
60	0.017	4.3	5.8	6.8	8.3	11	19	25	34	43	49	51	57
80	0.012	4.7	6.4	7.6	9.2	13	20	27	36	46	53	54	60
100	0.01	5.1	7	8.3	10	14	22	29	38	48	55	57	63
250	0.004	7.1	9.7	12	14	19	30	39	46	59	68	69	77

Rainfall depths (mm) :: RCP2.6 for the period 2031-2050

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	8.96	12.4	15	20.5	27.6	42.7	54.8	68.9	84.5	94	101	106
2	0.5	9.84	13.6	16.5	22.5	30.4	47	60.4	75.9	93.1	104	111	117
5	0.2	12.9	17.8	21.5	29.5	39.8	61.8	79.4	99.9	123	137	147	154
10	0.1	15.1	20.9	25.3	34.6	46.8	72.7	93.6	118	145	161	173	182
20	0.05	17.3	24.1	29.1	39.9	53.9	83.9	108	136	167	186	200	211
30	0.033	18.7	25.9	31.3	43	58.2	90.5	117	147	181	202	216	228
40	0.025	19.6	27.3	32.9	45.2	61.2	95.3	123	155	190	212	228	240
50	0.02	20.4	28.3	34.2	47	63.6	99	128	161	198	221	237	250
60	0.017	21	29.1	35.2	48.4	65.5	102	132	166	204	228	244	257
80	0.012	21.9	30.5	36.9	50.6	68.5	107	138	174	214	239	256	270
100	0.01	22.6	31.5	38.1	52.3	70.9	111	143	180	221	247	265	279
250	0.004	25.6	35.6	43.1	59.2	80.3	125	162	204	252	281	302	318

Rainfall depths (mm) :: RCP2.6 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
100	0.01	22.6	31.5	38.1	52.3	70.9	111	143	180	221	247	265	279
250	0.004	25.6	35.6	43.1	59.2	80.3	125	162	204	252	281	302	318

Rainfall depths (mm) :: RCP4.5 for the period 2031-2050

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	9.11	12.6	15.2	20.8	28	43.2	55.4	69.6	85.2	94.7	101	107
2	0.5	10	13.9	16.7	22.9	30.9	47.6	61.1	76.7	94	104	112	118
5	0.2	13.1	18.2	21.9	30	40.5	62.7	80.5	101	124	138	148	156
10	0.1	15.3	21.3	25.7	35.3	47.6	73.8	94.8	119	146	163	175	184
20	0.05	17.6	24.5	29.6	40.6	54.9	85.2	109	137	169	188	202	212
30	0.033	19	26.4	31.9	43.8	59.2	92	118	149	183	203	218	230
40	0.025	20	27.8	33.6	46.1	62.3	96.8	125	156	192	214	230	242
50	0.02	20.7	28.8	34.9	47.8	64.7	101	129	163	200	223	239	252
60	0.017	21.4	29.7	35.9	49.3	66.7	104	133	168	206	230	247	260
80	0.012	22.3	31.1	37.6	51.6	69.8	109	140	176	216	241	259	272
100	0.01	23.1	32.1	38.8	53.3	72.2	112	145	182	224	249	268	282
250	0.004	26.1	36.3	43.9	60.4	81.8	127	164	207	255	284	305	321

Rainfall depths (mm) :: RCP4.5 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	9.57	13.3	16	21.9	29.4	45	57.4	71.8	87.5	96.9	104	109
2	0.5	10.5	14.6	17.6	24.1	32.4	49.7	63.4	79.1	96.5	107	115	120
5	0.2	13.8	19.1	23.1	31.7	42.6	65.5	83.7	105	128	142	152	159

10	0.1	16.2	22.5	27.2	37.2	50.2	77.2	98.8	123	151	168	179	188
20	0.05	18.6	25.9	31.3	42.9	57.9	89.3	114	142	174	194	208	218
30	0.033	20.1	27.9	33.8	46.3	62.5	96.4	123	154	189	210	225	236
40	0.025	21.1	29.4	35.5	48.7	65.7	102	130	162	199	221	237	249
50	0.02	21.9	30.5	36.9	50.6	68.3	105	135	169	207	230	246	258
60	0.017	22.6	31.4	38	52.1	70.4	109	139	174	213	237	254	267
80	0.012	23.6	32.9	39.7	54.6	73.7	114	146	182	223	248	266	280
100	0.01	24.4	34	41.1	56.4	76.2	118	151	189	231	257	276	290
250	0.004	27.6	38.4	46.5	63.9	86.3	134	171	215	263	293	314	330

Rainfall depths (mm) :: RCP6.0 for the period 2031-2050

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	9.05	12.5	15.1	20.7	27.8	43	55.2	69.4	84.9	94.4	101	107
2	0.5	9.94	13.8	16.6	22.7	30.7	47.4	60.8	76.4	93.6	104	112	117
5	0.2	13	18	21.8	29.8	40.2	62.3	80.1	101	123	137	147	155
10	0.1	15.2	21.2	25.5	35	47.3	73.3	94.3	119	146	162	174	183
20	0.05	17.5	24.3	29.4	40.3	54.5	84.7	109	137	168	187	201	212
30	0.033	18.9	26.2	31.7	43.5	58.8	91.4	118	148	182	203	218	229
40	0.025	19.8	27.6	33.3	45.7	61.8	96.2	124	156	192	214	229	241
50	0.02	20.6	28.6	34.6	47.5	64.2	99.9	129	162	199	222	238	251
60	0.017	21.2	29.5	35.6	48.9	66.2	103	133	167	205	229	246	259
80	0.012	22.2	30.8	37.3	51.2	69.3	108	139	175	215	240	258	271
100	0.01	22.9	31.9	38.5	52.9	71.6	112	144	181	223	248	267	281
250	0.004	25.9	36	43.6	59.9	81.2	127	163	206	253	283	304	320

Rainfall depths (mm) :: RCP6.0 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	9.99	13.8	16.7	22.8	30.6	46.6	59.1	73.8	89.4	98.8	106	111
2	0.5	11	15.3	18.4	25.2	33.8	51.5	65.5	81.4	98.8	109	117	123
5	0.2	14.4	20	24.2	33.1	44.6	68.1	86.7	108	131	145	155	163
10	0.1	17	23.6	28.5	39	52.5	80.3	102	127	155	172	183	193
20	0.05	19.5	27.2	32.8	45	60.6	93	118	147	179	199	213	223
30	0.033	21.1	29.3	35.4	48.6	65.4	100	128	159	194	215	230	241
40	0.025	22.1	30.8	37.2	51.1	68.8	106	135	168	204	227	242	254
50	0.02	23	32	38.7	53.1	71.5	110	140	174	213	236	252	265
60	0.017	23.7	32.9	39.8	54.7	73.7	113	144	180	219	243	260	273
80	0.012	24.8	34.5	41.7	57.3	77.2	119	151	188	230	255	273	286
100	0.01	25.6	35.6	43.1	59.2	79.8	123	157	195	238	264	283	296
250	0.004	28.9	40.3	48.7	67	90.4	139	178	222	271	300	322	338

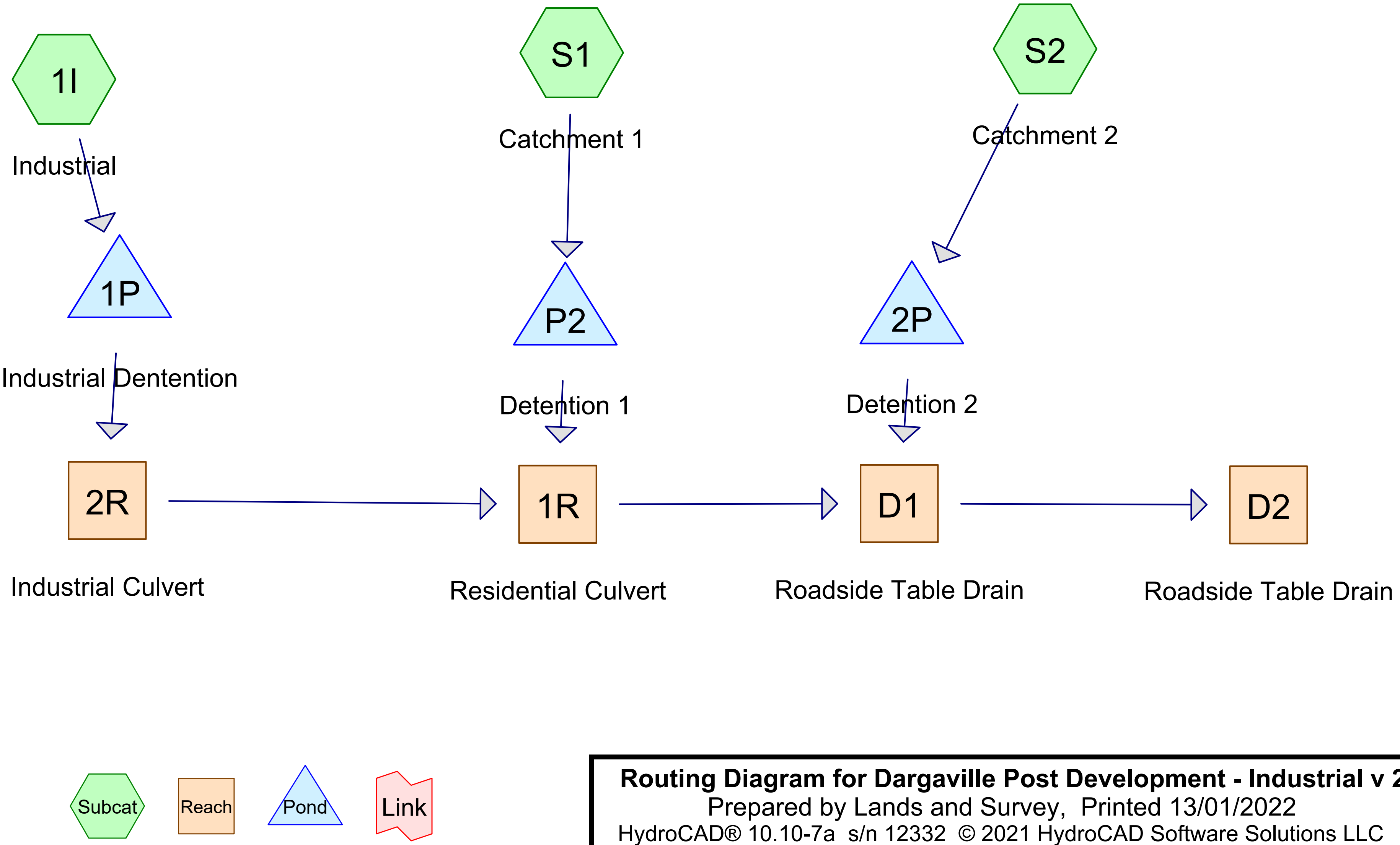
Rainfall depths (mm) :: RCP8.5 for the period 2031-2050

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	9.22	12.8	15.4	21.1	28.3	43.7	55.9	70.1	85.7	95.2	102	107
2	0.5	10.1	14	16.9	23.2	31.2	48.1	61.7	77.3	94.6	105	113	118
5	0.2	13.3	18.4	22.2	30.4	41	63.3	81.3	102	125	139	149	156
10	0.1	15.5	21.6	26.1	35.7	48.2	74.6	95.7	120	147	164	176	185
20	0.05	17.9	24.8	30	41.2	55.6	86.1	111	139	170	190	203	214
30	0.033	19.3	26.8	32.4	44.4	60	93	119	150	184	205	220	231
40	0.025	20.2	28.1	34	46.7	63.1	97.9	126	158	194	216	232	244
50	0.02	21	29.2	35.3	48.5	65.5	102	131	164	202	224	241	253
60	0.017	21.6	30.1	36.4	49.9	67.5	105	135	169	208	232	248	261
80	0.012	22.6	31.5	38.1	52.3	70.7	110	141	177	218	243	260	274
100	0.01	23.4	32.5	39.3	54.1	73.1	114	146	184	226	251	270	284
250	0.004	26.4	36.8	44.5	61.2	82.8	129	166	209	257	286	307	323

Rainfall depths (mm) :: RCP8.5 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h	48h	72h	96h	120h
1.58	0.633	10.9	15.2	18.3	25	33.3	50.2	63.1	78.2	93.9	103	110	115
2	0.5	12.1	16.7	20.2	27.6	37	55.7	70.1	86.3	104	115	122	128
5	0.2	15.9	22.1	26.6	36.5	48.9	73.9	93.2	115	138	153	163	170
10	0.1	18.7	26	31.4	43	57.7	87.4	110	136	164	181	193	202
20	0.05	21.6	30	36.2	49.7	66.7	101	128	157	190	210	224	234
30	0.033	23.3	32.3	39.1	53.6	72	109	138	170	206	227	243	254
40	0.025	24.5	34	41.1	56.4	75.8	115	146	179	217	240	255	268
50	0.02	25.4	35.3	42.7	58.7	78.8	120	151	186	226	249	266	278
60	0.017	26.2	36.4	44	60.4	81.2	124	156	192	233	258	274	287
80	0.012	27.4	38.1	46.1	63.3	85.1	130	164	202	244	270	288	301
100	0.01	28.3	39.4	47.6	65.5	88	134	170	209	253	280	298	312
250	0.004	32	44.5	53.9	74.1	99.7	152	193	237	288	318	340	355

Appendix C - Stormwater Calculations (HydroCAD Project Reports)



Routing Diagram for Dargaville Post Development - Industrial v 2
Prepared by Lands and Survey, Printed 13/01/2022
HydroCAD® 10.10-7a s/n 12332 © 2021 HydroCAD Software Solutions LLC

Rainfall Events Listing

Event#	Event Name	Storm Type	Curve	Mode	Duration (hours)	B/B	Depth (mm)	AMC
1	1% Post Dev (RCP8.5 - 2031-2050)	Type IA 24-hr		Default	24.00	1	184	2
2	20% Post Dev (RCP8.5 - 2031-2050)	Type IA 24-hr		Default	24.00	1	102	2

Area Listing (all nodes)

Area (hectares)	CN	Description (subcatchment-numbers)
2.7252	87	1/4 acre lots, 40% imp, HSG D (S2)
20.2310	92	1/8 acre lots, 70% imp, HSG D (S1, S2)
4.1623	80	>75% Grass cover, Good, HSG D (S1, S2)
6.5337	98	Paved roads w/curbs & sewers, HSG D (1I, S1, S2)
9.5301	95	Urban commercial, 100% imp, HSG D (1I)
0.2862	100	Urban commercial, 100% imp, HSG D (S2)
1.5936	98	Water Surface, HSG D (1I, S1, S2)
45.0621	**92**	**TOTAL AREA**

Time span=0.00-20.00 hrs, dt=0.05 hrs, 401 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1I: Industrial Runoff Area=11.2970 ha 100.00% Impervious Runoff Depth>152 mm
Flow Length=483.0 m Tc=15.0 min CN=95 Runoff=1.3064 m³/s 17.164 Ml

Subcatchment S1: Catchment 1 Runoff Area=16.2260 ha 77.36% Impervious Runoff Depth>146 mm
Flow Length=483.0 m Tc=15.0 min CN=93 Runoff=1.8316 m³/s 23.696 Ml

Subcatchment S2: Catchment 2 Runoff Area=17.5391 ha 53.28% Impervious Runoff Depth>134 mm
Flow Length=483.0 m Tc=15.0 min CN=89 Runoff=1.8583 m³/s 23.578 Ml

Reach 1R: Residential Culvert Avg. Flow Depth=1.20 m Max Vel=0.87 m/s Inflow=2.0067 m³/s 38.199 Ml
n=0.035 L=500.00 m S=0.0020 m/m Capacity=1.9211 m³/s Outflow=1.9315 m³/s 37.650 Ml

Reach 2R: Industrial Culvert Avg. Flow Depth=0.84 m Max Vel=0.88 m/s Inflow=1.1958 m³/s 16.868 Ml
n=0.035 L=350.00 m S=0.0029 m/m Capacity=2.2961 m³/s Outflow=1.1590 m³/s 16.704 Ml

Reach D1: Roadside Table Avg. Flow Depth=1.40 m Max Vel=1.04 m/s Inflow=2.8539 m³/s 59.289 Ml
n=0.035 L=159.00 m S=0.0025 m/m Capacity=2.1546 m³/s Outflow=2.8475 m³/s 59.056 Ml

Reach D2: Roadside Table Avg. Flow Depth=1.50 m Max Vel=0.92 m/s Inflow=2.8475 m³/s 59.056 Ml
n=0.035 L=222.00 m S=0.0018 m/m Capacity=2.8258 m³/s Outflow=2.8333 m³/s 58.686 Ml

Pond 1P: Industrial Dentention Peak Elev=6.768 m Storage=1,017.4 m³ Inflow=1.3064 m³/s 17.164 Ml
1,200 mm Round Culvert n=0.011 L=10.00 m S=0.0000 m/m Outflow=1.1958 m³/s 16.868 Ml

Pond 2P: Detention 2 Peak Elev=4.896 m Storage=4,894.9 m³ Inflow=1.8583 m³/s 23.578 Ml
825 mm Round Culvert n=0.011 L=10.00 m S=0.0000 m/m Outflow=0.9288 m³/s 21.639 Ml

Pond P2: Detention 1 Peak Elev=5.904 m Storage=5,506.8 m³ Inflow=1.8316 m³/s 23.696 Ml
750 mm Round Culvert n=0.011 L=10.00 m S=0.0000 m/m Outflow=0.8624 m³/s 21.495 Ml

Total Runoff Area = 45.0621 ha Runoff Volume = 64.437 Ml Average Runoff Depth = 143 mm
26.33% Pervious = 11.8667 ha 73.67% Impervious = 33.1954 ha

Pipe Listing (all nodes)

Line#	Node Number	In-Invert (meters)	Out-Invert (meters)	Length (meters)	Slope (m/m)	n	Width (mm)	Diam/Height (mm)	Inside-Fill (mm)
1	1P	5.800	5.800	10.00	0.0000	0.011	0	1,200	0
2	2P	3.800	3.800	10.00	0.0000	0.011	0	825	0
3	P2	4.800	4.800	10.00	0.0000	0.011	0	750	0

Summary for Subcatchment 1I: Industrial

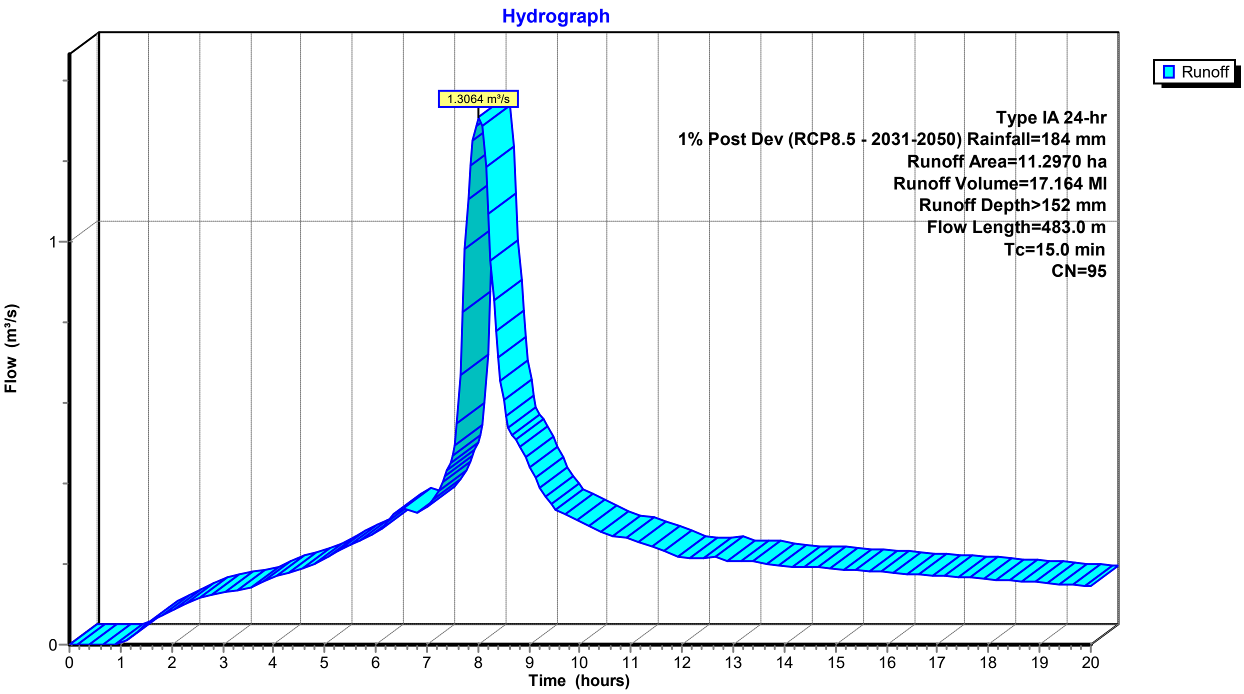
Runoff = 1.3064 m³/s @ 8.02 hrs, Volume= 17.164 Ml, Depth> 152 mm
Routed to Pond 1P : Industrial Dentention

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Type IA 24-hr 1% Post Dev (RCP8.5 - 2031-2050) Rainfall=184 mm

	Area (ha)	CN	Description
*	9.5301	95	Urban commercial, 100% imp, HSG D
	1.6334	98	Paved roads w/curbs & sewers, HSG D
	0.1335	98	Water Surface, HSG D
	11.2970	95	Weighted Average
	11.2970		100.00% Impervious Area

Tc (min)	Length (meters)	Slope (m/m)	Velocity (m/sec)	Capacity (m³/s)	Description
15.0	483.0		0.54		**Direct Entry, Storm**

Subcatchment 1I: Industrial



Summary for Subcatchment S1: Catchment 1

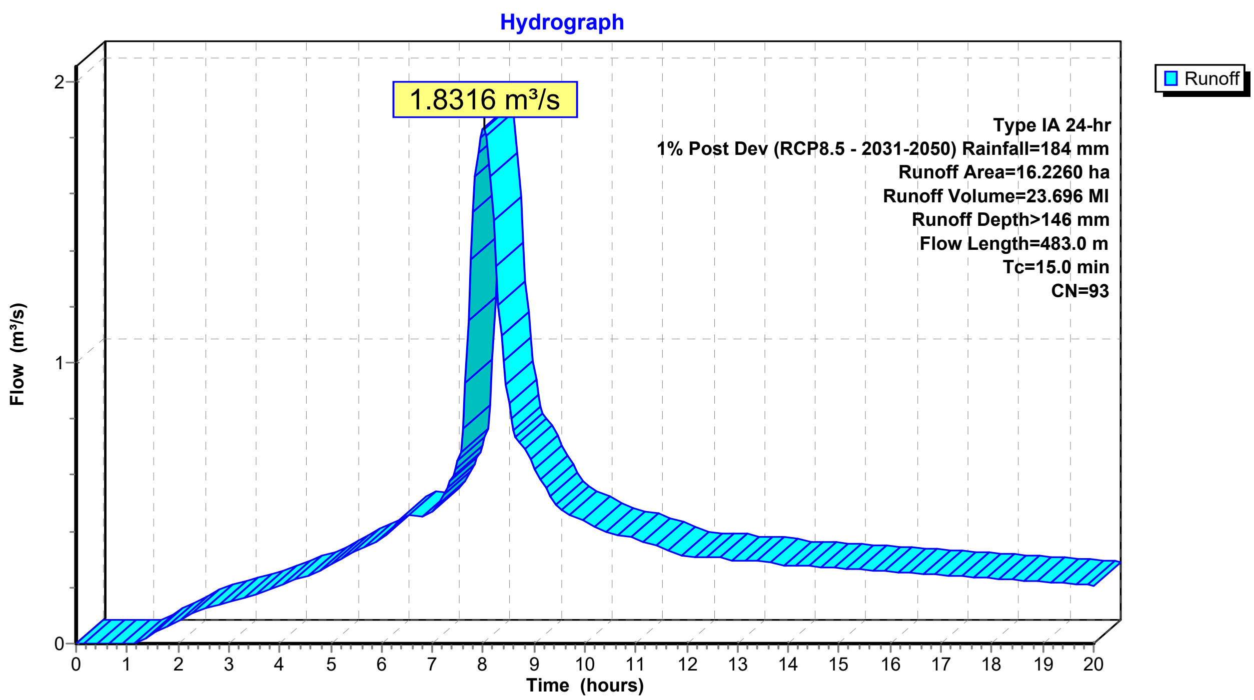
Runoff = 1.8316 m³/s @ 8.02 hrs, Volume= 23.696 Ml, Depth> 146 mm
Routed to Pond P2 : Detention 1

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Type IA 24-hr 1% Post Dev (RCP8.5 - 2031-2050) Rainfall=184 mm

	Area (ha)	CN	Description
*	11.9136	92	1/8 acre lots, 70% imp, HSG D
	3.2669	98	Paved roads w/curbs & sewers, HSG D
	0.9468	98	Water Surface, HSG D
	0.0987	80	>75% Grass cover, Good, HSG D
	16.2260	93	Weighted Average
	3.6728		22.64% Pervious Area
	12.5532		77.36% Impervious Area

Tc (min)	Length (meters)	Slope (m/m)	Velocity (m/sec)	Capacity (m³/s)	Description
15.0	483.0		0.54		**Direct Entry, Single S1**

Subcatchment S1: Catchment 1



Summary for Subcatchment S2: Catchment 2

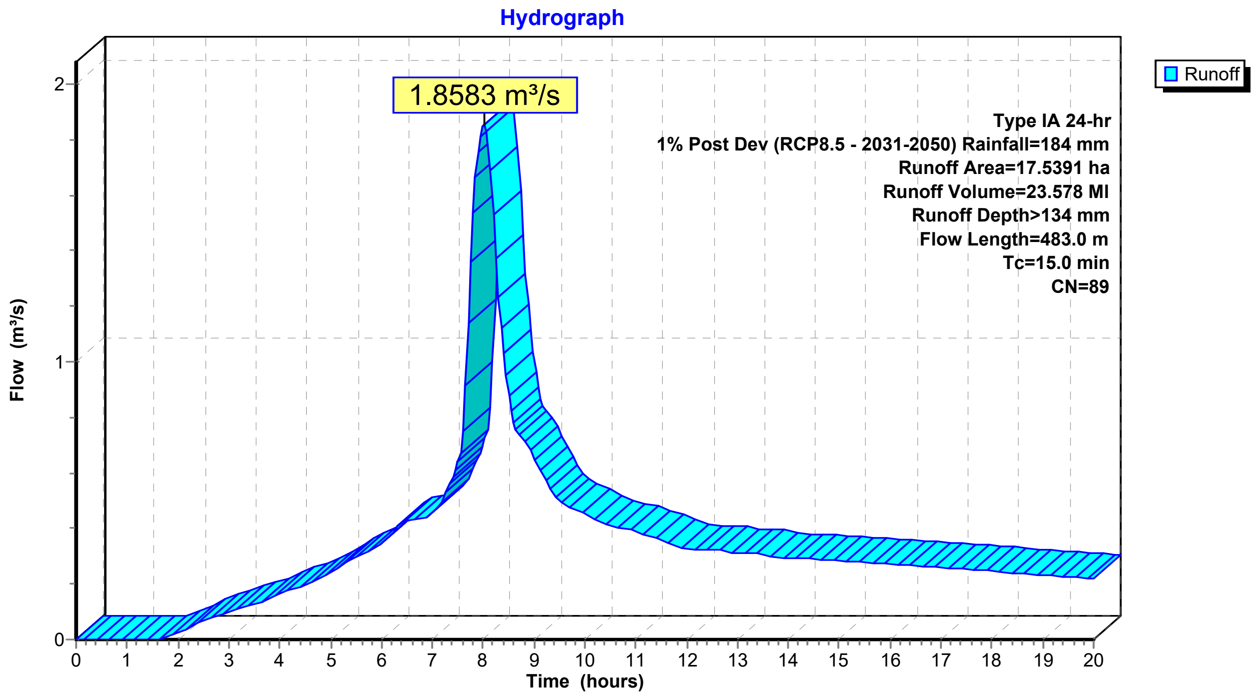
Runoff = 1.8583 m³/s @ 8.03 hrs, Volume= 23.578 Ml, Depth> 134 mm
Routed to Pond 2P : Detention 2

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Type IA 24-hr 1% Post Dev (RCP8.5 - 2031-2050) Rainfall=184 mm

	Area (ha)	CN	Description
*	3.6460	92	1/8 acre lots, 70% imp, HSG D
*	2.7252	87	1/4 acre lots, 40% imp, HSG D
	4.0636	80	>75% Grass cover, Good, HSG D
	1.6334	98	Paved roads w/curbs & sewers, HSG D
	0.5133	98	Water Surface, HSG D
*	4.6714	92	1/8 acre lots, 70% imp, HSG D
*	0.2862	100	Urban commercial, 100% imp, HSG D
	17.5391	89	Weighted Average
	8.1939		46.72% Pervious Area
	9.3452		53.28% Impervious Area

Tc (min)	Length (meters)	Slope (m/m)	Velocity (m/sec)	Capacity (m³/s)	Description
15.0	483.0		0.54		**Direct Entry, Single S2**

Subcatchment S2: Catchment 2



Summary for Pond 1P: Industrial Dentention

Inflow Area = 11.2970 ha, 87.35% Impervious, Inflow Depth > 152 mm for 1% Post Dev (RCP8.5 - 2031
Inflow = 1.3064 m³/s @ 8.02 hrs, Volume= 17.164 Ml
Outflow = 1.1958 m³/s @ 8.14 hrs, Volume= 16.868 Ml, Atten= 8%, Lag= 7.4 min
Primary = 1.1958 m³/s @ 8.14 hrs, Volume= 16.868 Ml
Routed to Reach 2R : Industrial Culvert

Routing by Stor-Ind method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Peak Elev= 6.768 m @ 8.14 hrs Surf.Area= 1,243.0 m² Storage= 1,017.4 m³

Plug-Flow detention time= 24.8 min calculated for 16.826 Ml (98% of inflow)
Center-of-Mass det. time= 14.7 min (620.4 - 605.8)

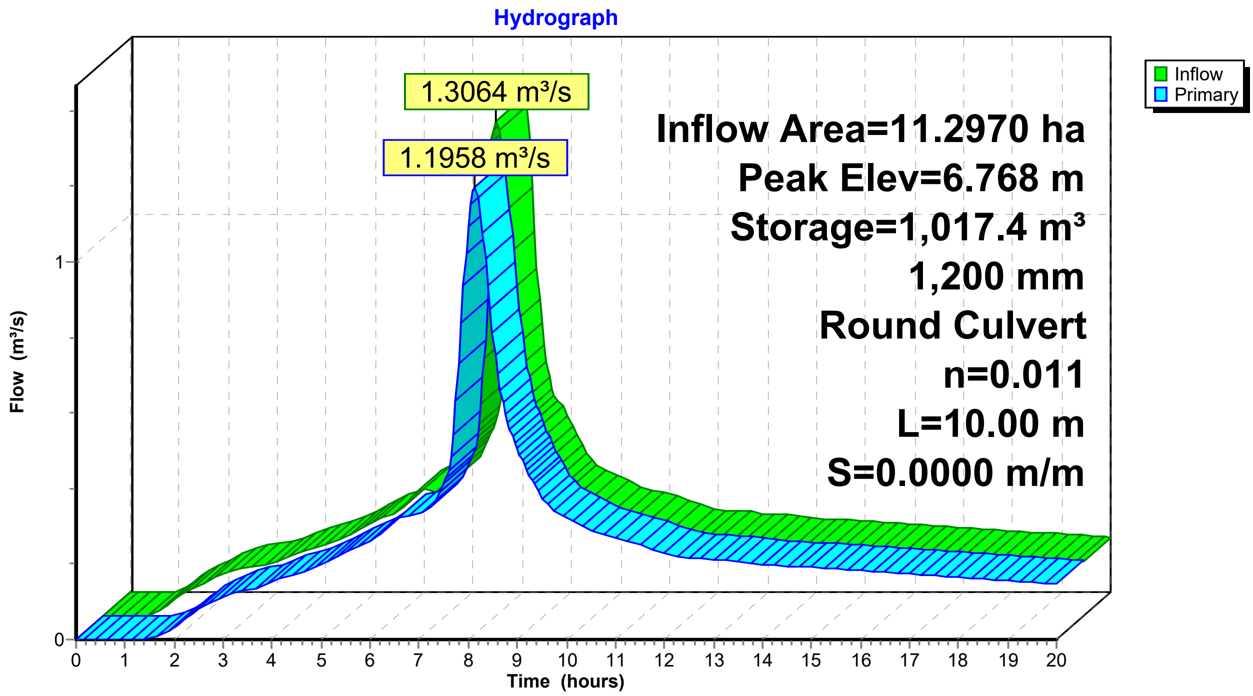
Volume	Invert	Avail.Storage	Storage Description
#1	5.800 m	1,316.0 m³	**Custom Stage Data (Prismatic)**Listed below (Recalc)

Elevation (meters)	Surf.Area (sq-meters)	Inc.Store (cubic-meters)	Cum.Store (cubic-meters)
5.800	858.3	0.0	0.0
7.000	1,335.0	1,316.0	1,316.0

Device	Routing	Invert	Outlet Devices
#1	Primary	5.800 m	**1,200 mm Round Culvert** L= 10.00 m Ke= 0.600 Inlet / Outlet Invert= 5.800 m / 5.800 m S= 0.0000 m/m Cc= 0.900 n= 0.011, Flow Area= 1.131 m²

Primary OutFlow Max=1.1940 m³/s @ 8.14 hrs HW=6.767 m (Free Discharge)
└—**1=Culvert** (Barrel Controls 1.1940 m³/s @ 1.67 m/s)

Pond 1P: Industrial Dentention



Summary for Pond P2: Detention 1

Inflow Area = 16.2260 ha, 73.69% Impervious, Inflow Depth > 146 mm for 1% Post Dev (RCP8.5 - 2031
Inflow = 1.8316 m³/s @ 8.02 hrs, Volume= 23.696 Ml
Outflow = 0.8624 m³/s @ 8.50 hrs, Volume= 21.495 Ml, Atten= 53%, Lag= 28.4 min
Primary = 0.8624 m³/s @ 8.50 hrs, Volume= 21.495 Ml
Routed to Reach 1R : Residential Culvert

Routing by Stor-Ind method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Peak Elev= 5.904 m @ 8.50 hrs Surf.Area= 5,464.3 m² Storage= 5,506.8 m³

Plug-Flow detention time= 125.7 min calculated for 21.442 Ml (90% of inflow)
Center-of-Mass det. time= 74.7 min (691.4 - 616.8)

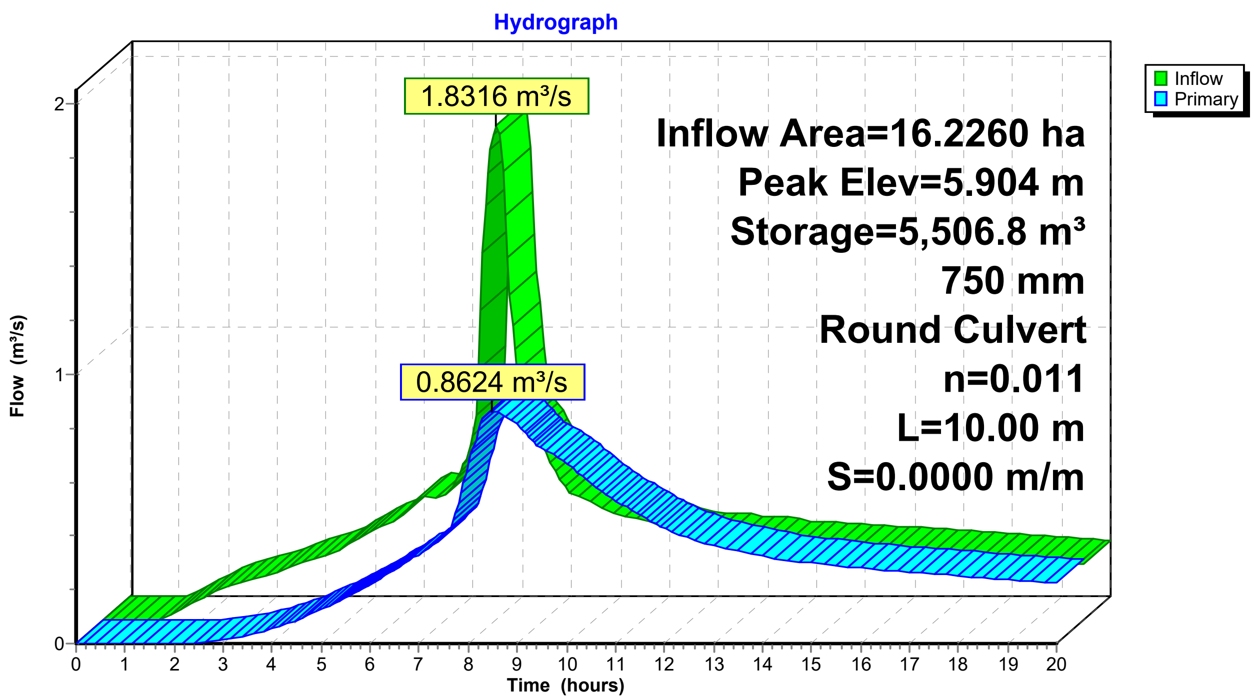
Volume	Invert	Avail.Storage	Storage Description
#1	4.800 m	6,037.0 m³	**Custom Stage Data (Pyramidal)**Listed below (Recalc)

Elevation (meters)	Surf.Area (sq-meters)	Inc.Store (cubic-meters)	Cum.Store (cubic-meters)	Wet.Area (sq-meters)
4.800	4,529.0	0.0	0.0	4,529.0
6.000	5,550.0	6,037.0	6,037.0	5,605.2

Device	Routing	Invert	Outlet Devices
#1	Primary	4.800 m	**750 mm Round Culvert** L= 10.00 m Ke= 0.600 Inlet / Outlet Invert= 4.800 m / 4.800 m S= 0.0000 m/m Cc= 0.900 n= 0.011 Concrete pipe, straight & clean, Flow Area= 0.442 m²

Primary OutFlow Max=0.8624 m³/s @ 8.50 hrs HW=5.904 m (Free Discharge)
└─**1=Culvert** (Barrel Controls 0.8624 m³/s @ 1.95 m/s)

Pond P2: Detention 1



Summary for Pond 2P: Detention 2

Inflow Area = 17.5391 ha, 50.36% Impervious, Inflow Depth > 134 mm for 1% Post Dev (RCP8.5 - 2031
Inflow = 1.8583 m³/s @ 8.03 hrs, Volume= 23.578 Ml
Outflow = 0.9288 m³/s @ 8.47 hrs, Volume= 21.639 Ml, Atten= 50%, Lag= 26.2 min
Primary = 0.9288 m³/s @ 8.47 hrs, Volume= 21.639 Ml
Routed to Reach D1 : Roadside Table Drain

Routing by Stor-Ind method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Peak Elev= 4.896 m @ 8.47 hrs Surf.Area= 4,911.4 m² Storage= 4,894.9 m³

Plug-Flow detention time= 105.6 min calculated for 21.639 Ml (92% of inflow)
Center-of-Mass det. time= 61.4 min (697.6 - 636.3)

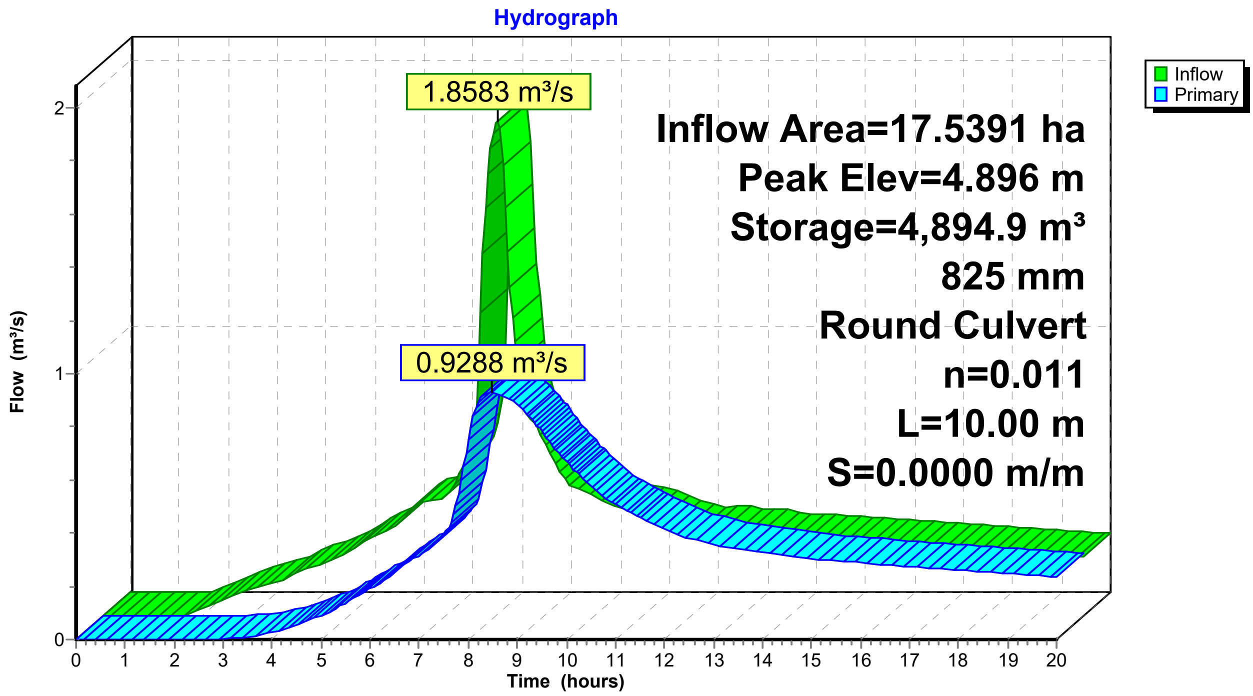
Volume	Invert	Avail.Storage	Storage Description
#1	3.800 m	5,408.3 m³	**Custom Stage Data (Pyramidal)**Listed below (Recalc)

Elevation (meters)	Surf.Area (sq-meters)	Inc.Store (cubic-meters)	Cum.Store (cubic-meters)	Wet.Area (sq-meters)
3.800	4,032.2	0.0	0.0	4,032.2
5.000	4,999.0	5,408.3	5,408.3	5,051.2

Device	Routing	Invert	Outlet Devices
#1	Primary	3.800 m	**825 mm Round Culvert** L= 10.00 m Ke= 0.600 Inlet / Outlet Invert= 3.800 m / 3.800 m S= 0.0000 m/m Cc= 0.900 n= 0.011 Concrete pipe, straight & clean, Flow Area= 0.535 m²

Primary OutFlow Max=0.9274 m³/s @ 8.47 hrs HW=4.896 m (Free Discharge)
└─**1=Culvert** (Barrel Controls 0.9274 m³/s @ 1.74 m/s)

Pond 2P: Detention 2



Summary for Reach 2R: Industrial Culvert

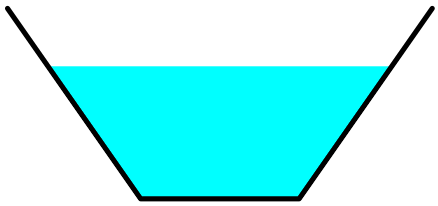
[79] Warning: Submerged Pond 1P Primary device # 1 by 0.834 m

Inflow Area = 11.2970 ha, 87.35% Impervious, Inflow Depth > 149 mm for 1% Post Dev (RCP8.5 - 203
Inflow = 1.1958 m³/s @ 8.14 hrs, Volume= 16.868 Ml
Outflow = 1.1590 m³/s @ 8.33 hrs, Volume= 16.704 Ml, Atten= 3%, Lag= 11.4 min
Routed to Reach 1R : Residential Culvert

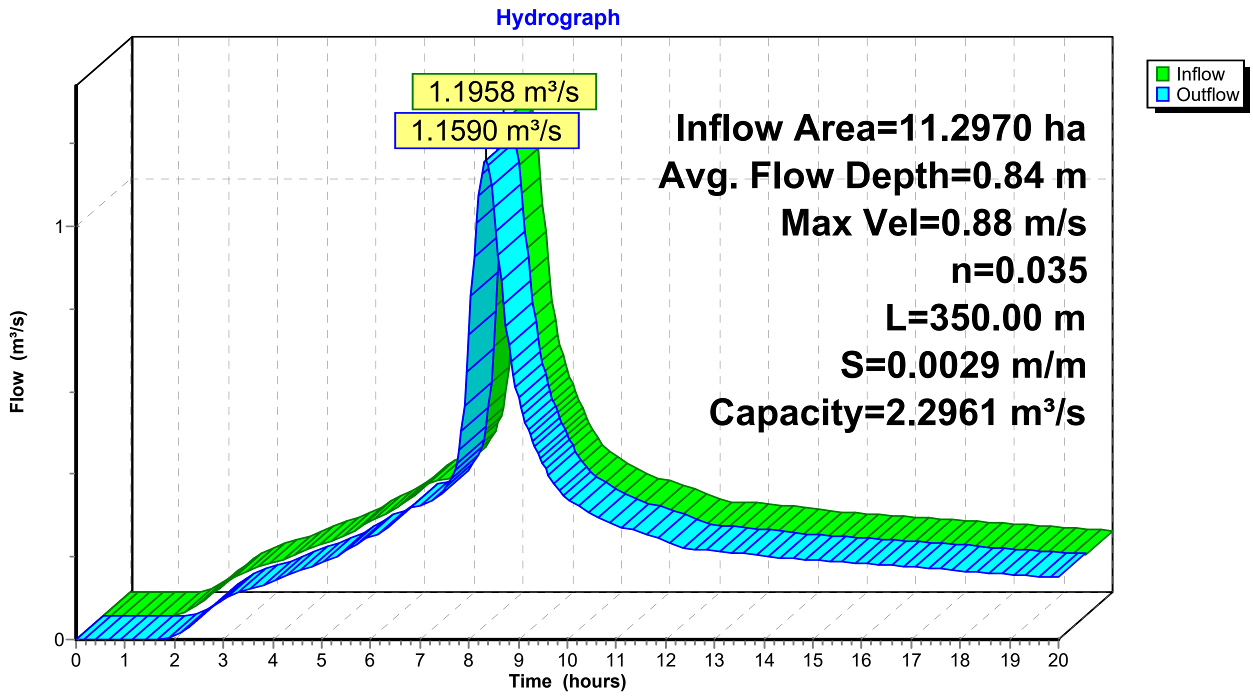
Routing by Stor-Ind+Trans method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.88 m/s, Min. Travel Time= 6.7 min
Avg. Velocity = 0.54 m/s, Avg. Travel Time= 10.7 min

Peak Storage= 463.2 m³ @ 8.22 hrs
Average Depth at Peak Storage= 0.84 m , Surface Width= 2.17 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 2.2961 m³/s

1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 350.00 m Slope= 0.0029 m/m
Inlet Invert= 5.800 m, Outlet Invert= 4.800 m



Reach 2R: Industrial Culvert



Summary for Reach 1R: Residential Culvert

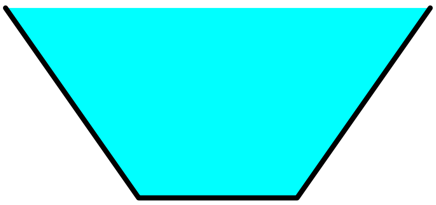
[91] Warning: Storage range exceeded by 0.004 m
[55] Hint: Peak inflow is 104% of Manning's capacity
[62] Hint: Exceeded Reach 2R OUTLET depth by 0.526 m @ 8.75 hrs
[81] Warning: Exceeded Pond P2 by 0.100 m @ 8.45 hrs

Inflow Area = 27.5230 ha, 79.30% Impervious, Inflow Depth > 139 mm for 1% Post Dev (RCP8.5 - 2031
Inflow = 2.0067 m³/s @ 8.35 hrs, Volume= 38.199 Ml
Outflow = 1.9315 m³/s @ 8.64 hrs, Volume= 37.650 Ml, Atten= 4%, Lag= 17.2 min
Routed to Reach D1 : Roadside Table Drain

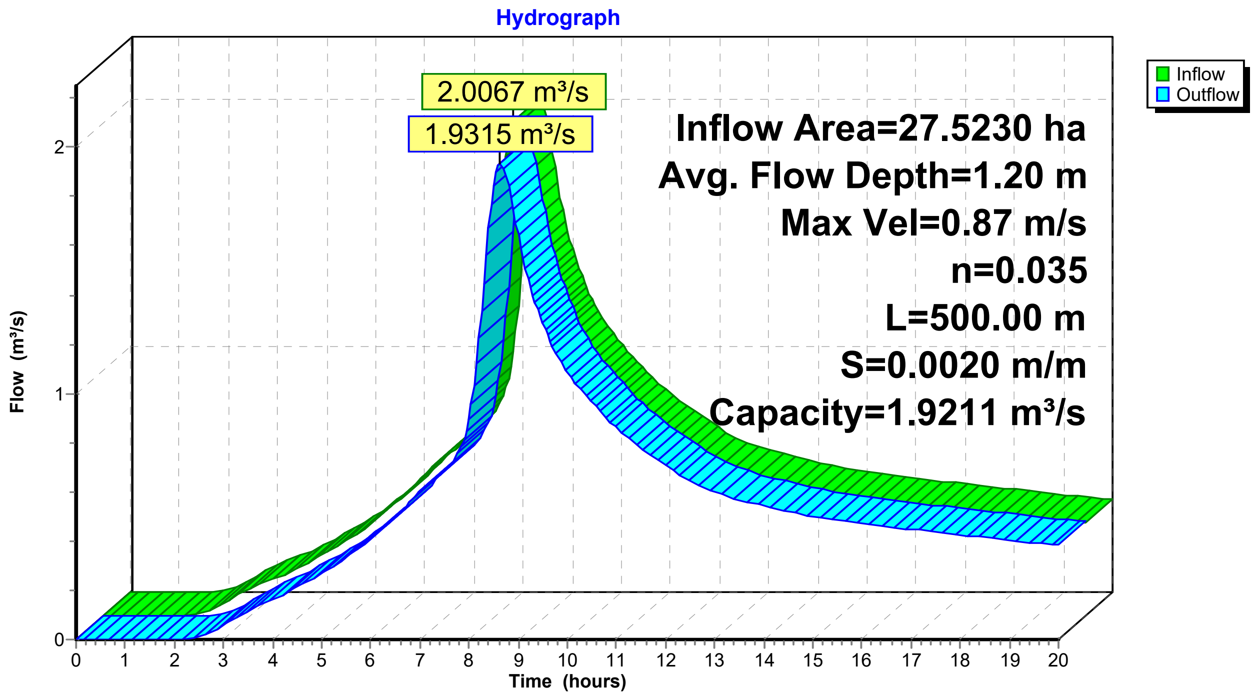
Routing by Stor-Ind+Trans method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.87 m/s, Min. Travel Time= 9.6 min
Avg. Velocity = 0.59 m/s, Avg. Travel Time= 14.0 min

Peak Storage= 1,109.2 m³ @ 8.48 hrs
Average Depth at Peak Storage= 1.20 m , Surface Width= 2.69 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 1.9211 m³/s

1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 500.00 m Slope= 0.0020 m/m
Inlet Invert= 4.800 m, Outlet Invert= 3.800 m



Reach 1R: Residential Culvert



Summary for Reach D1: Roadside Table Drain

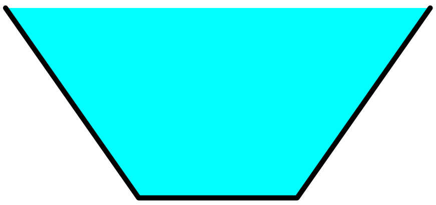
[91] Warning: Storage range exceeded by 0.201 m
[55] Hint: Peak inflow is 132% of Manning's capacity
[62] Hint: Exceeded Reach 1R OUTLET depth by 0.269 m @ 8.90 hrs
[81] Warning: Exceeded Pond 2P by 0.323 m @ 8.70 hrs

Inflow Area = 45.0621 ha, 68.03% Impervious, Inflow Depth > 132 mm for 1% Post Dev (RCP8.5 - 203
Inflow = 2.8539 m³/s @ 8.63 hrs, Volume= 59.289 Ml
Outflow = 2.8475 m³/s @ 8.71 hrs, Volume= 59.056 Ml, Atten= 0%, Lag= 4.7 min
Routed to Reach D2 : Roadside Table Drain

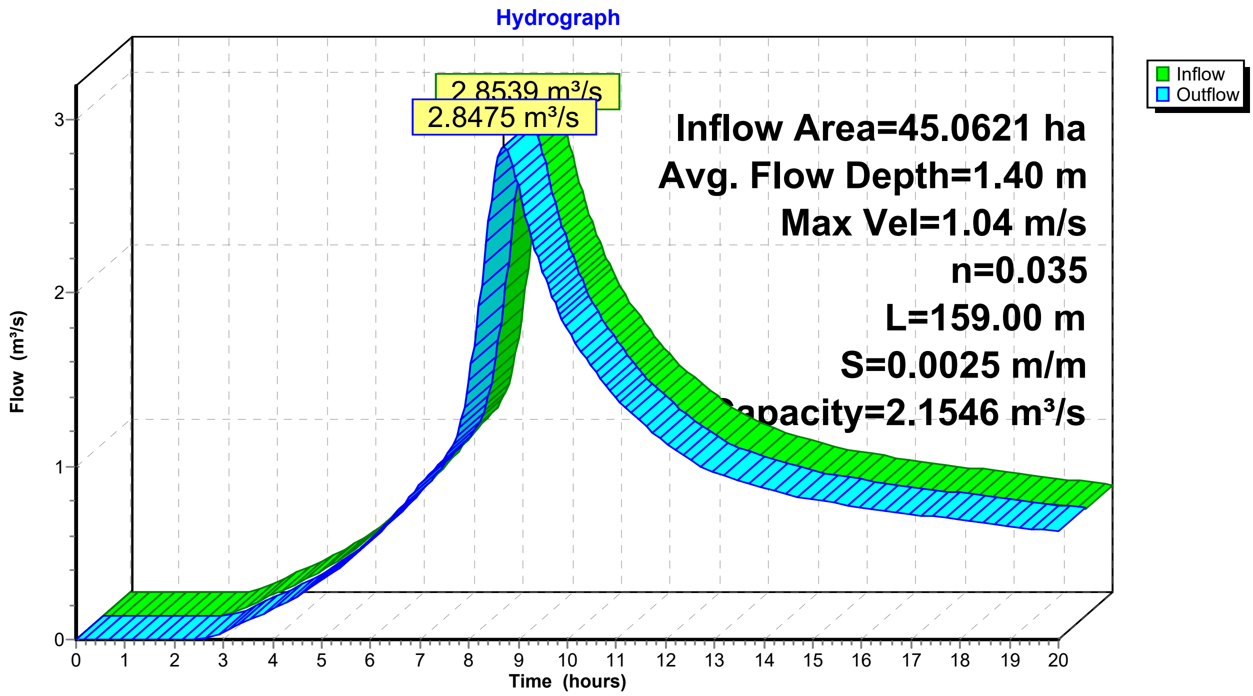
Routing by Stor-Ind+Trans method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 1.04 m/s, Min. Travel Time= 2.6 min
Avg. Velocity = 0.73 m/s, Avg. Travel Time= 3.6 min

Peak Storage= 436.6 m³ @ 8.67 hrs
Average Depth at Peak Storage= 1.40 m , Surface Width= 2.96 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 2.1546 m³/s

1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 159.00 m Slope= 0.0025 m/m
Inlet Invert= 3.800 m, Outlet Invert= 3.400 m



Reach D1: Roadside Table Drain



Summary for Subcatchment 1I: Industrial

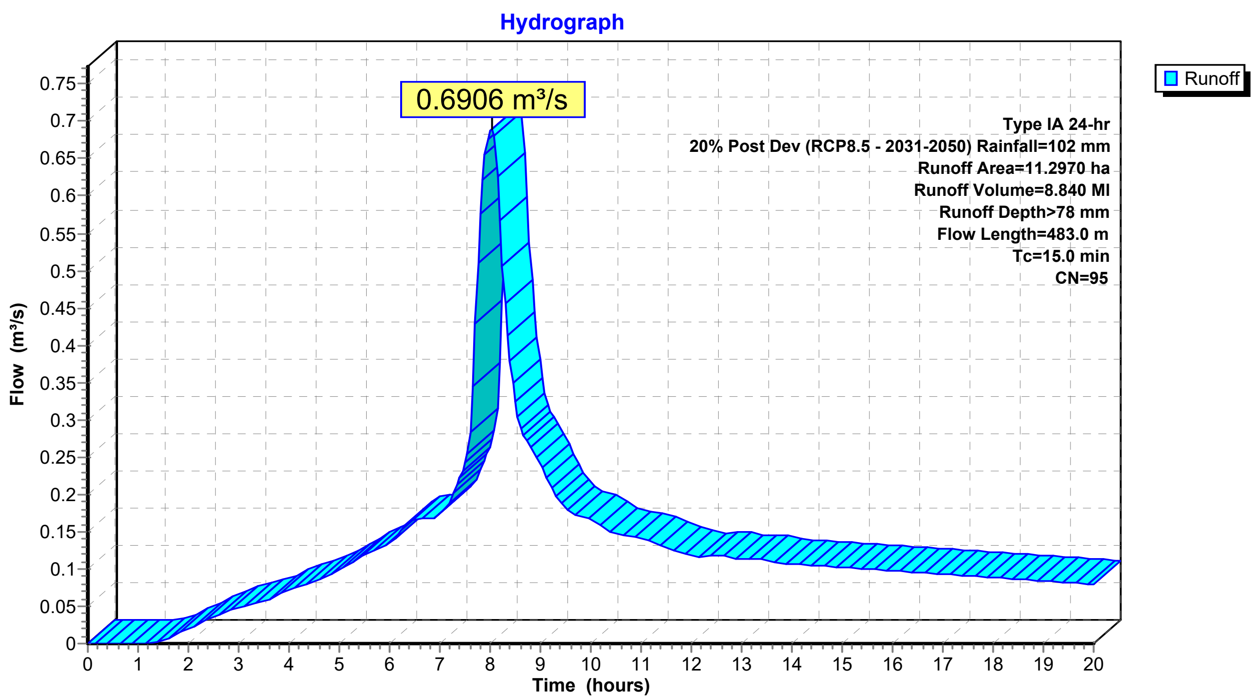
Runoff = 0.6906 m³/s @ 8.03 hrs, Volume= 8.840 Ml, Depth> 78 mm
Routed to Pond 1P : Industrial Dentention

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Type IA 24-hr 20% Post Dev (RCP8.5 - 2031-2050) Rainfall=102 mm

	Area (ha)	CN	Description
*	9.5301	95	Urban commercial, 100% imp, HSG D
	1.6334	98	Paved roads w/curbs & sewers, HSG D
	0.1335	98	Water Surface, HSG D
	11.2970	95	Weighted Average
	11.2970		100.00% Impervious Area

Tc (min)	Length (meters)	Slope (m/m)	Velocity (m/sec)	Capacity (m³/s)	Description
15.0	483.0		0.54		**Direct Entry, Storm**

Subcatchment 1I: Industrial



Summary for Subcatchment S1: Catchment 1

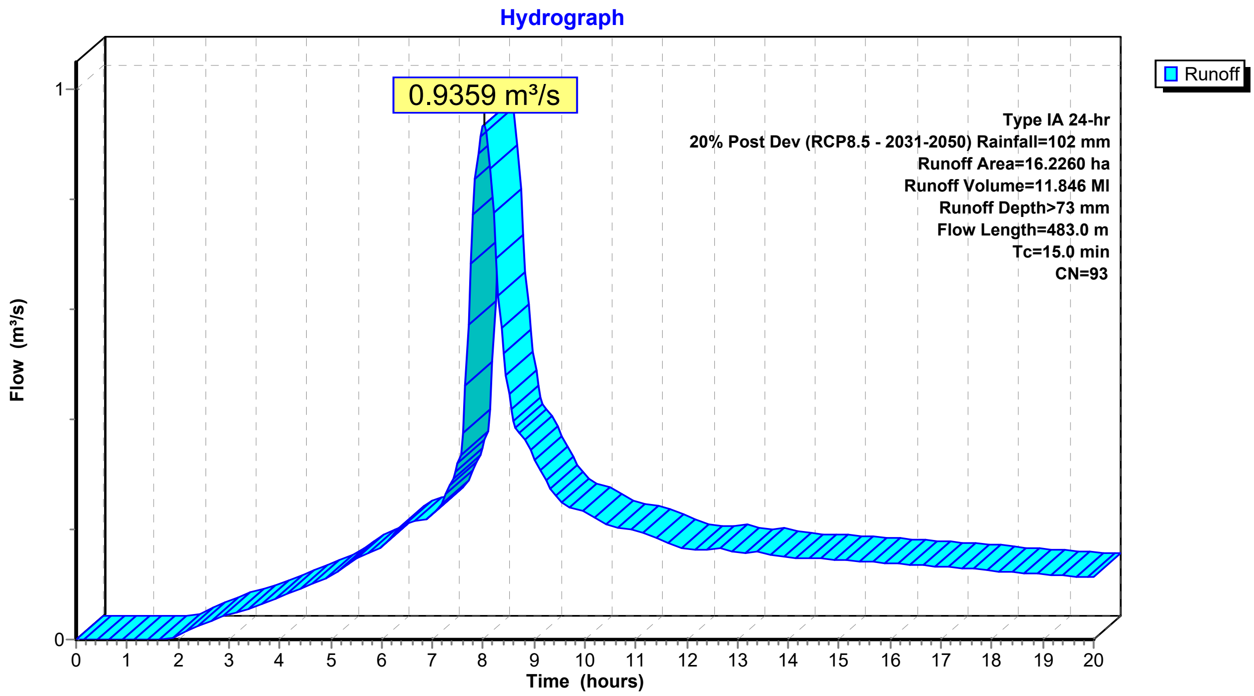
Runoff = 0.9359 m³/s @ 8.03 hrs, Volume= 11.846 Ml, Depth> 73 mm
Routed to Pond P2 : Detention 1

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Type IA 24-hr 20% Post Dev (RCP8.5 - 2031-2050) Rainfall=102 mm

	Area (ha)	CN	Description
*	11.9136	92	1/8 acre lots, 70% imp, HSG D
	3.2669	98	Paved roads w/curbs & sewers, HSG D
	0.9468	98	Water Surface, HSG D
	0.0987	80	>75% Grass cover, Good, HSG D
	16.2260	93	Weighted Average
	3.6728		22.64% Pervious Area
	12.5532		77.36% Impervious Area

Tc (min)	Length (meters)	Slope (m/m)	Velocity (m/sec)	Capacity (m³/s)	Description
15.0	483.0		0.54		**Direct Entry, Single S1**

Subcatchment S1: Catchment 1



Summary for Subcatchment S2: Catchment 2

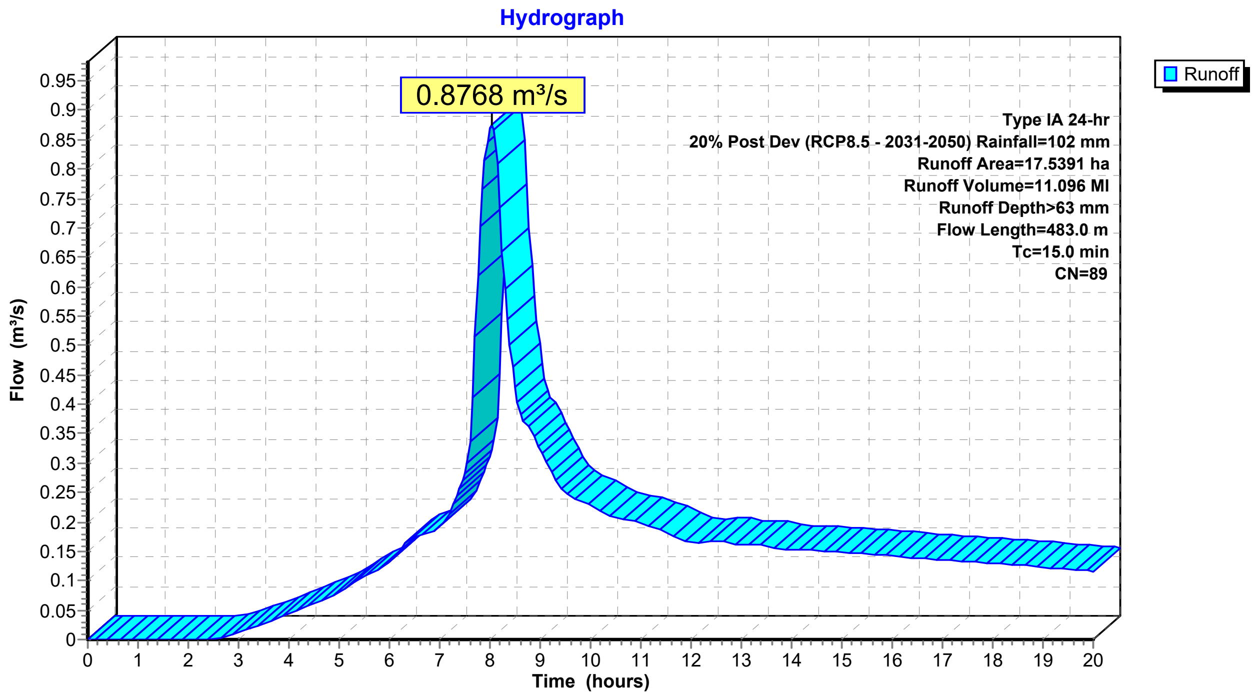
Runoff = 0.8768 m³/s @ 8.04 hrs, Volume= 11.096 Ml, Depth> 63 mm
Routed to Pond 2P : Detention 2

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Type IA 24-hr 20% Post Dev (RCP8.5 - 2031-2050) Rainfall=102 mm

	Area (ha)	CN	Description
*	3.6460	92	1/8 acre lots, 70% imp, HSG D
*	2.7252	87	1/4 acre lots, 40% imp, HSG D
	4.0636	80	>75% Grass cover, Good, HSG D
	1.6334	98	Paved roads w/curbs & sewers, HSG D
	0.5133	98	Water Surface, HSG D
*	4.6714	92	1/8 acre lots, 70% imp, HSG D
*	0.2862	100	Urban commercial, 100% imp, HSG D
	17.5391	89	Weighted Average
	8.1939		46.72% Pervious Area
	9.3452		53.28% Impervious Area

Tc (min)	Length (meters)	Slope (m/m)	Velocity (m/sec)	Capacity (m³/s)	Description
15.0	483.0		0.54		**Direct Entry, Single S2**

Subcatchment S2: Catchment 2



Summary for Pond 1P: Industrial Dentention

Inflow Area = 11.2970 ha, 87.35% Impervious, Inflow Depth > 78 mm for 20% Post Dev (RCP8.5 - 20
Inflow = 0.6906 m³/s @ 8.03 hrs, Volume= 8.840 Ml
Outflow = 0.6154 m³/s @ 8.16 hrs, Volume= 8.622 Ml, Atten= 11%, Lag= 8.1 min
Primary = 0.6154 m³/s @ 8.16 hrs, Volume= 8.622 Ml
Routed to Reach 2R : Industrial Culvert

Routing by Stor-Ind method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Peak Elev= 6.469 m @ 8.16 hrs Surf.Area= 1,124.0 m² Storage= 663.0 m³

Plug-Flow detention time= 32.9 min calculated for 8.600 Ml (97% of inflow)
Center-of-Mass det. time= 19.0 min (644.2 - 625.2)

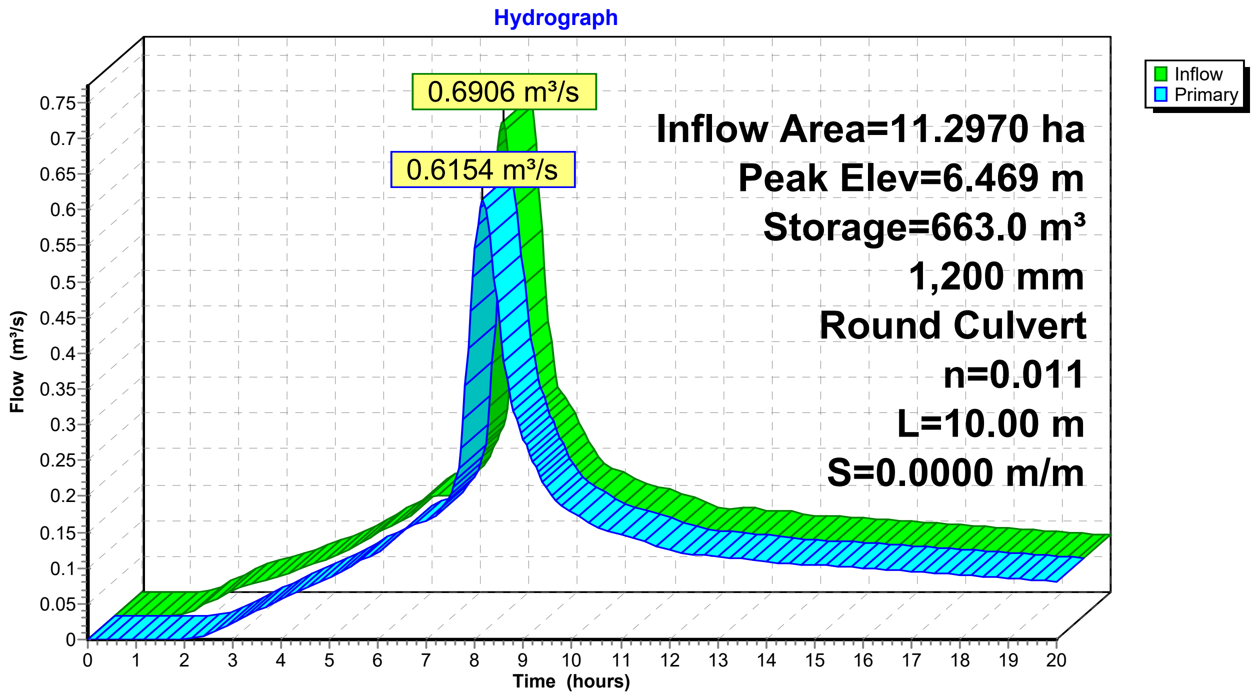
Volume	Invert	Avail.Storage	Storage Description
#1	5.800 m	1,316.0 m³	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (meters)	Surf.Area (sq-meters)	Inc.Store (cubic-meters)	Cum.Store (cubic-meters)
5.800	858.3	0.0	0.0
7.000	1,335.0	1,316.0	1,316.0

Device	Routing	Invert	Outlet Devices
#1	Primary	5.800 m	**1,200 mm Round Culvert** L= 10.00 m Ke= 0.600 Inlet / Outlet Invert= 5.800 m / 5.800 m S= 0.0000 m/m Cc= 0.900 n= 0.011, Flow Area= 1.131 m²

Primary OutFlow Max=0.6141 m³/s @ 8.16 hrs HW=6.468 m (Free Discharge)
└─**1=Culvert** (Barrel Controls 0.6141 m³/s @ 1.37 m/s)

Pond 1P: Industrial Dentention



Summary for Pond P2: Detention 1

Inflow Area = 16.2260 ha, 73.69% Impervious, Inflow Depth > 73 mm for 20% Post Dev (RCP8.5 - 20
Inflow = 0.9359 m³/s @ 8.03 hrs, Volume= 11.846 Ml
Outflow = 0.4102 m³/s @ 8.57 hrs, Volume= 10.252 Ml, Atten= 56%, Lag= 32.2 min
Primary = 0.4102 m³/s @ 8.57 hrs, Volume= 10.252 Ml
Routed to Reach 1R : Residential Culvert

Routing by Stor-Ind method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Peak Elev= 5.454 m @ 8.57 hrs Surf.Area= 5,072.2 m² Storage= 3,135.5 m³

Plug-Flow detention time= 157.7 min calculated for 10.252 Ml (87% of inflow)
Center-of-Mass det. time= 87.8 min (728.4 - 640.6)

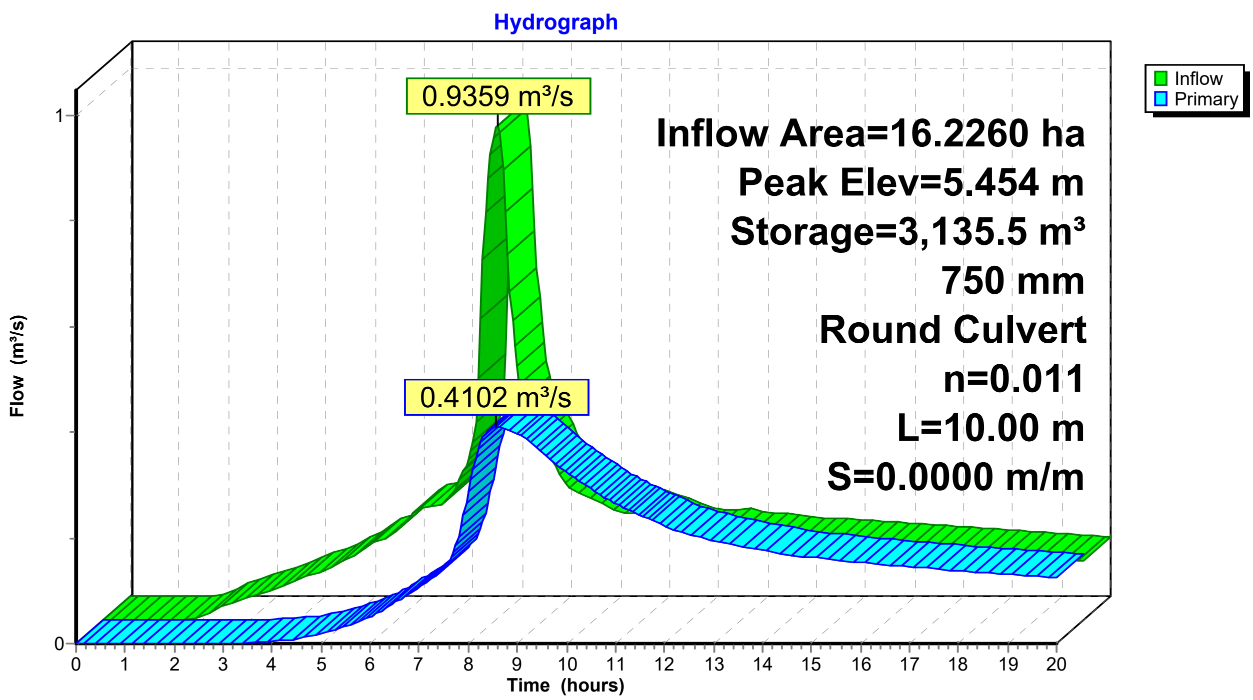
Volume	Invert	Avail.Storage	Storage Description
#1	4.800 m	6,037.0 m³	**Custom Stage Data (Pyramidal)** Listed below (Recalc)

Elevation (meters)	Surf.Area (sq-meters)	Inc.Store (cubic-meters)	Cum.Store (cubic-meters)	Wet.Area (sq-meters)
4.800	4,529.0	0.0	0.0	4,529.0
6.000	5,550.0	6,037.0	6,037.0	5,605.2

Device	Routing	Invert	Outlet Devices
#1	Primary	4.800 m	**750 mm Round Culvert** L= 10.00 m Ke= 0.600 Inlet / Outlet Invert= 4.800 m / 4.800 m S= 0.0000 m/m Cc= 0.900 n= 0.011 Concrete pipe, straight & clean, Flow Area= 0.442 m²

Primary OutFlow Max=0.4101 m³/s @ 8.57 hrs HW=5.453 m (Free Discharge)
└─**1=Culvert** (Barrel Controls 0.4101 m³/s @ 1.34 m/s)

Pond P2: Detention 1



Summary for Pond 2P: Detention 2

Inflow Area = 17.5391 ha, 50.36% Impervious, Inflow Depth > 63 mm for 20% Post Dev (RCP8.5 - 203
Inflow = 0.8768 m³/s @ 8.04 hrs, Volume= 11.096 Ml
Outflow = 0.4028 m³/s @ 8.55 hrs, Volume= 9.708 Ml, Atten= 54%, Lag= 30.9 min
Primary = 0.4028 m³/s @ 8.55 hrs, Volume= 9.708 Ml
Routed to Reach D1 : Roadside Table Drain

Routing by Stor-Ind method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Peak Elev= 4.417 m @ 8.55 hrs Surf.Area= 4,516.4 m² Storage= 2,636.1 m³

Plug-Flow detention time= 137.6 min calculated for 9.708 Ml (87% of inflow)
Center-of-Mass det. time= 74.7 min (741.6 - 666.9)

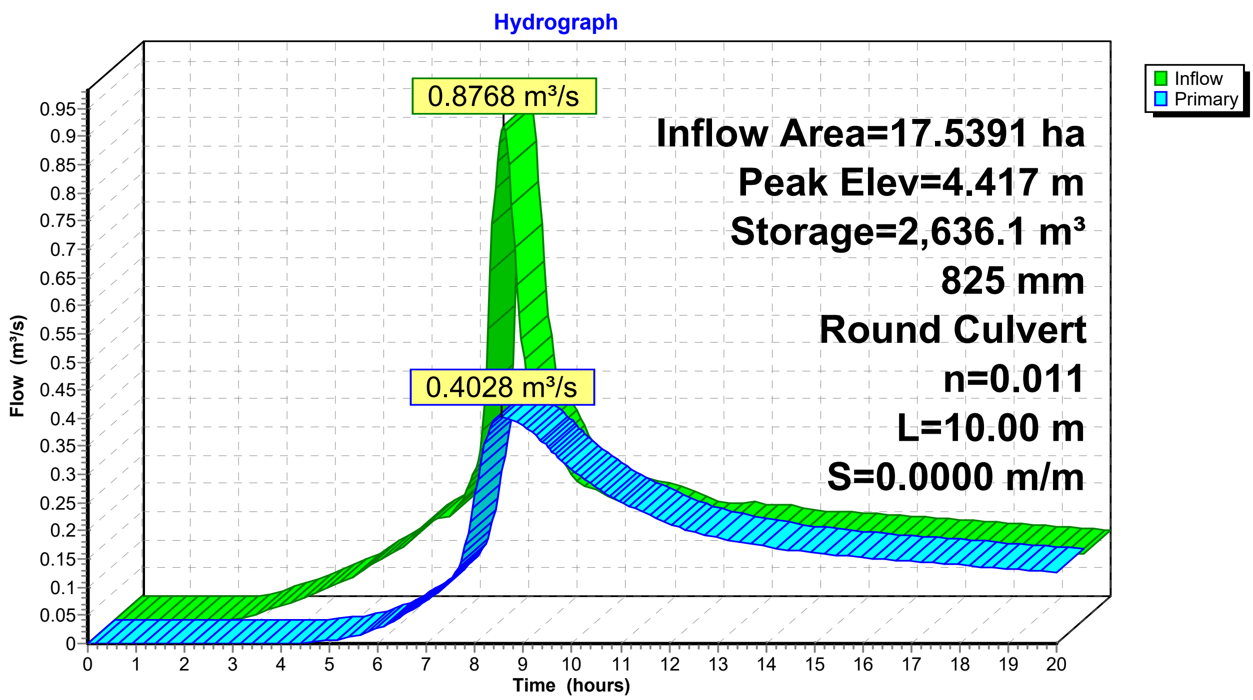
Volume	Invert	Avail.Storage	Storage Description
#1	3.800 m	5,408.3 m³	**Custom Stage Data (Pyramidal)** Listed below (Recalc)

Elevation (meters)	Surf.Area (sq-meters)	Inc.Store (cubic-meters)	Cum.Store (cubic-meters)	Wet.Area (sq-meters)
3.800	4,032.2	0.0	0.0	4,032.2
5.000	4,999.0	5,408.3	5,408.3	5,051.2

Device	Routing	Invert	Outlet Devices
#1	Primary	3.800 m	**825 mm Round Culvert** L= 10.00 m Ke= 0.600 Inlet / Outlet Invert= 3.800 m / 3.800 m S= 0.0000 m/m Cc= 0.900 n= 0.011 Concrete pipe, straight & clean, Flow Area= 0.535 m²

Primary OutFlow Max=0.4027 m³/s @ 8.55 hrs HW=4.417 m (Free Discharge)
└─**1=Culvert** (Barrel Controls 0.4027 m³/s @ 1.30 m/s)

Pond 2P: Detention 2



Summary for Reach 2R: Industrial Culvert

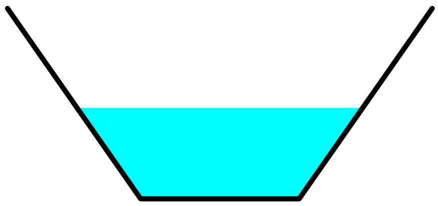
[79] Warning: Submerged Pond 1P Primary device # 1 by 0.573 m

Inflow Area = 11.2970 ha, 87.35% Impervious, Inflow Depth > 76 mm for 20% Post Dev (RCP8.5 - 203
Inflow = 0.6154 m³/s @ 8.16 hrs, Volume= 8.622 Ml
Outflow = 0.5902 m³/s @ 8.39 hrs, Volume= 8.513 Ml, Atten= 4%, Lag= 13.6 min
Routed to Reach 1R : Residential Culvert

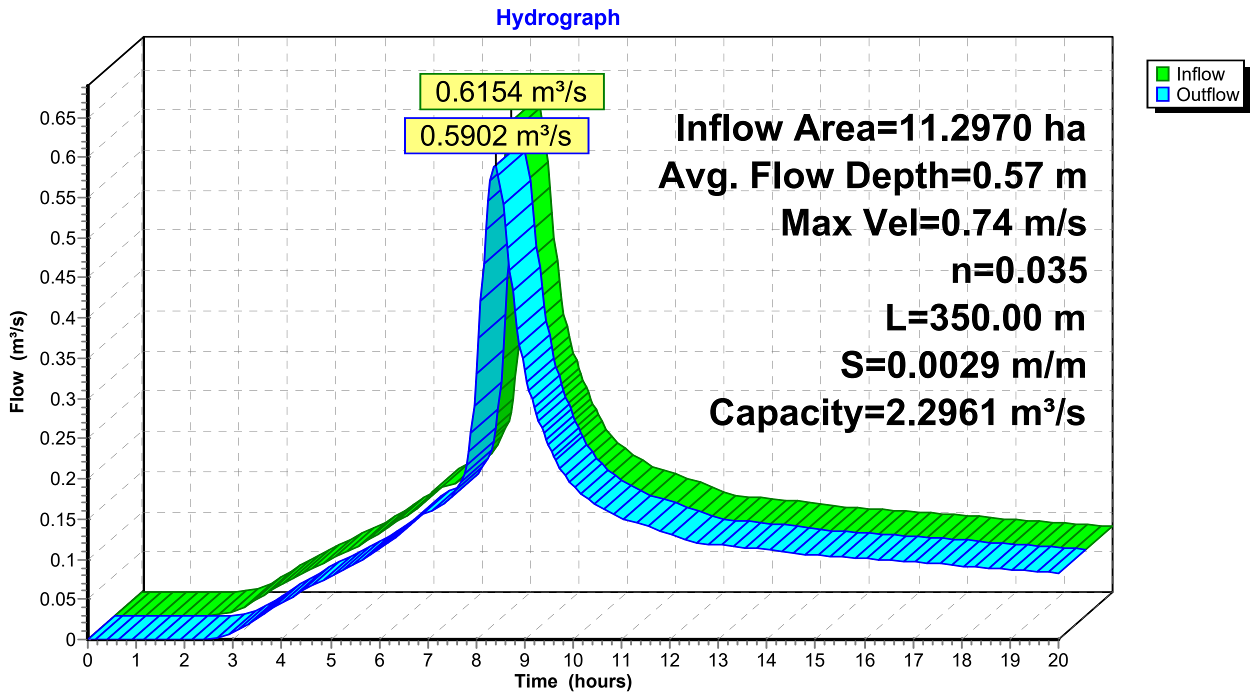
Routing by Stor-Ind+Trans method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.74 m/s, Min. Travel Time= 7.9 min
Avg. Velocity = 0.44 m/s, Avg. Travel Time= 13.2 min

Peak Storage= 281.2 m³ @ 8.25 hrs
Average Depth at Peak Storage= 0.57 m , Surface Width= 1.80 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 2.2961 m³/s

1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 350.00 m Slope= 0.0029 m/m
Inlet Invert= 5.800 m, Outlet Invert= 4.800 m



Reach 2R: Industrial Culvert



Summary for Reach 1R: Residential Culvert

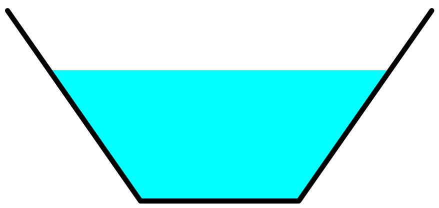
[62] Hint: Exceeded Reach 2R OUTLET depth by 0.364 m @ 8.80 hrs
[81] Warning: Exceeded Pond P2 by 0.170 m @ 8.55 hrs

Inflow Area = 27.5230 ha, 79.30% Impervious, Inflow Depth > 68 mm for 20% Post Dev (RCP8.5 - 20
Inflow = 0.9937 m³/s @ 8.41 hrs, Volume= 18.765 Ml
Outflow = 0.9468 m³/s @ 8.75 hrs, Volume= 18.406 Ml, Atten= 5%, Lag= 20.7 min
Routed to Reach D1 : Roadside Table Drain

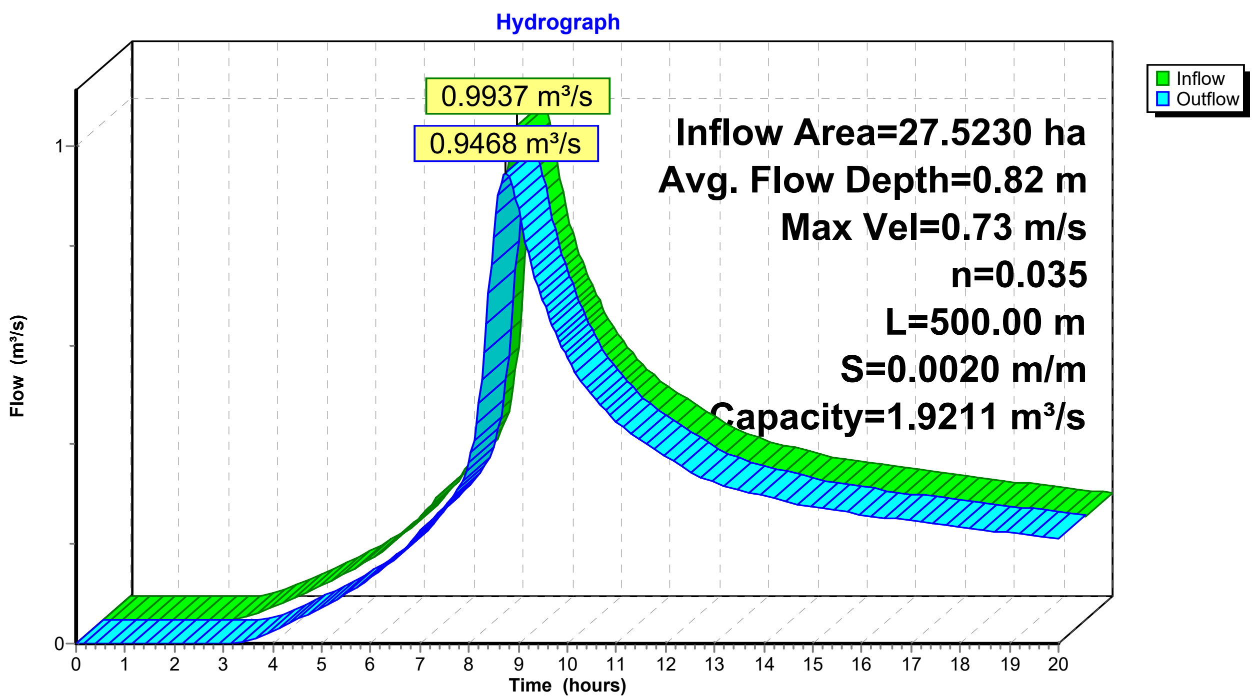
Routing by Stor-Ind+Trans method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.73 m/s, Min. Travel Time= 11.4 min
Avg. Velocity = 0.49 m/s, Avg. Travel Time= 17.1 min

Peak Storage= 649.5 m³ @ 8.56 hrs
Average Depth at Peak Storage= 0.82 m , Surface Width= 2.15 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 1.9211 m³/s

1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 500.00 m Slope= 0.0020 m/m
Inlet Invert= 4.800 m, Outlet Invert= 3.800 m



Reach 1R: Residential Culvert



Summary for Reach D1: Roadside Table Drain

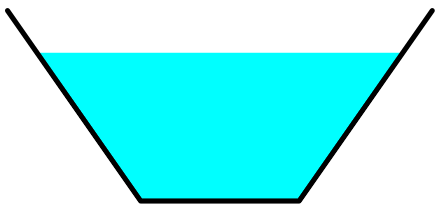
[62] Hint: Exceeded Reach 1R OUTLET depth by 0.166 m @ 9.05 hrs
[81] Warning: Exceeded Pond 2P by 0.323 m @ 8.80 hrs

Inflow Area = 45.0621 ha, 68.03% Impervious, Inflow Depth > 62 mm for 20% Post Dev (RCP8.5 - 203
Inflow = 1.3451 m³/s @ 8.74 hrs, Volume= 28.113 Ml
Outflow = 1.3422 m³/s @ 8.83 hrs, Volume= 27.962 Ml, Atten= 0%, Lag= 5.5 min
Routed to Reach D2 : Roadside Table Drain

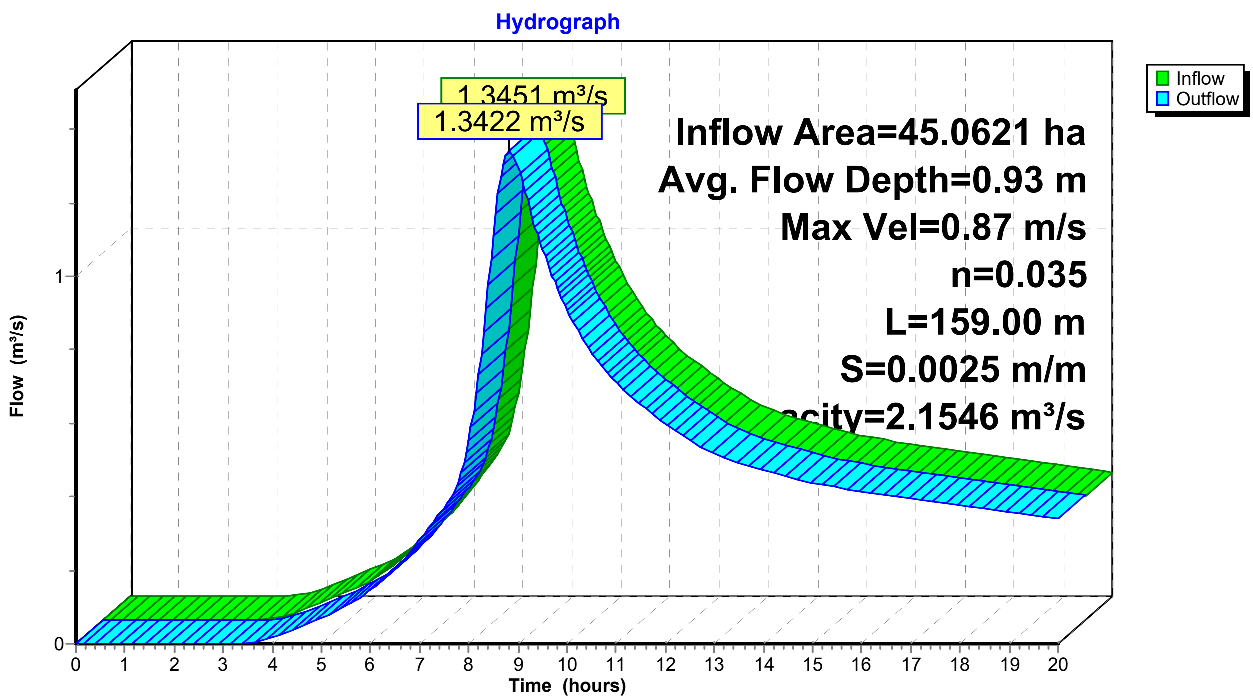
Routing by Stor-Ind+Trans method, Time Span= 0.00-20.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.87 m/s, Min. Travel Time= 3.1 min
Avg. Velocity = 0.59 m/s, Avg. Travel Time= 4.5 min

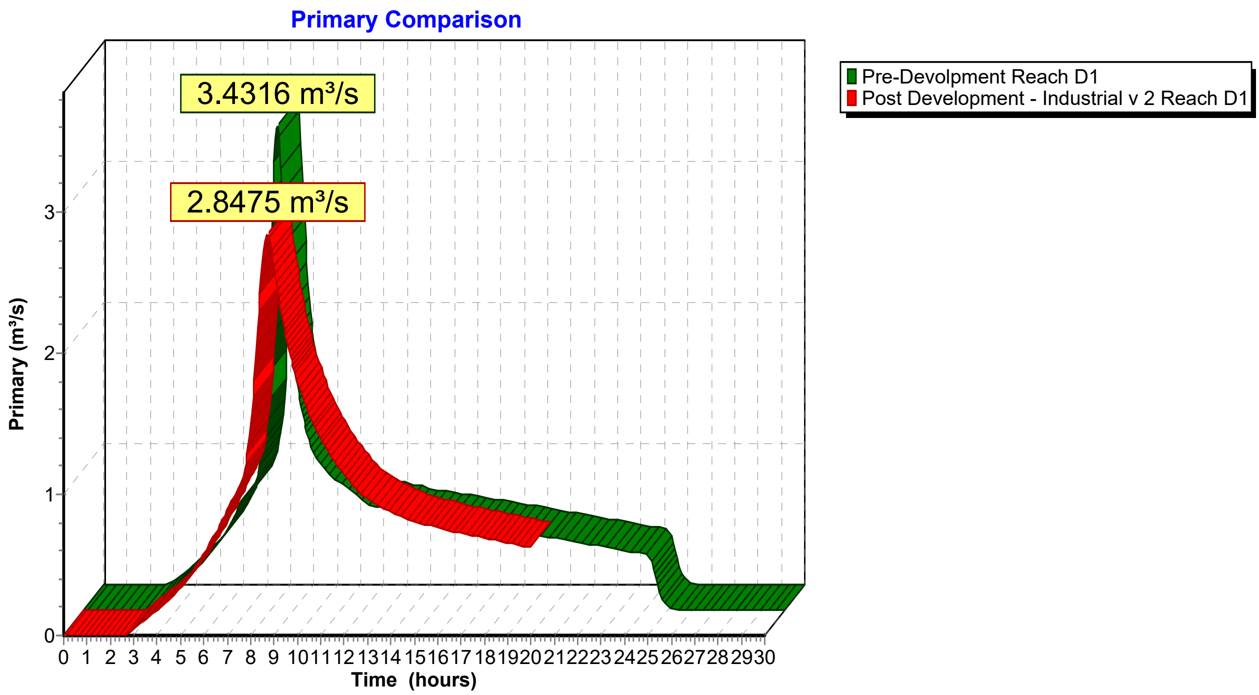
Peak Storage= 245.9 m³ @ 8.78 hrs
Average Depth at Peak Storage= 0.93 m , Surface Width= 2.31 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 2.1546 m³/s

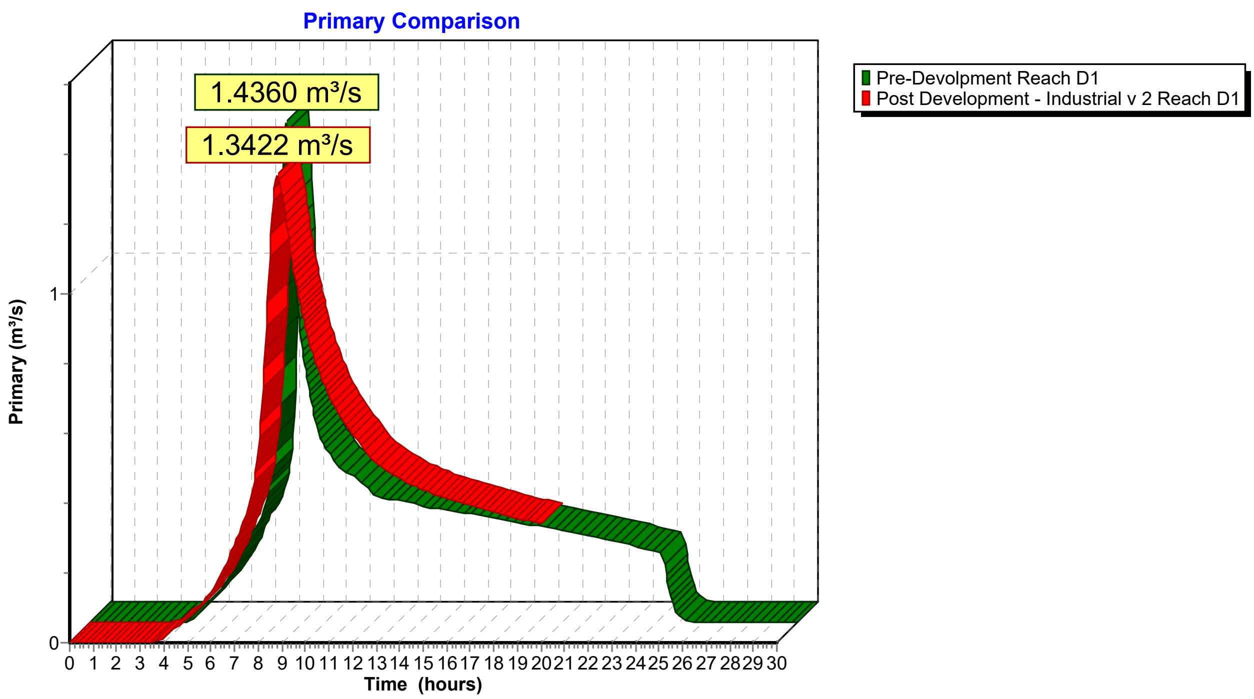
1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 159.00 m Slope= 0.0025 m/m
Inlet Invert= 3.800 m, Outlet Invert= 3.400 m

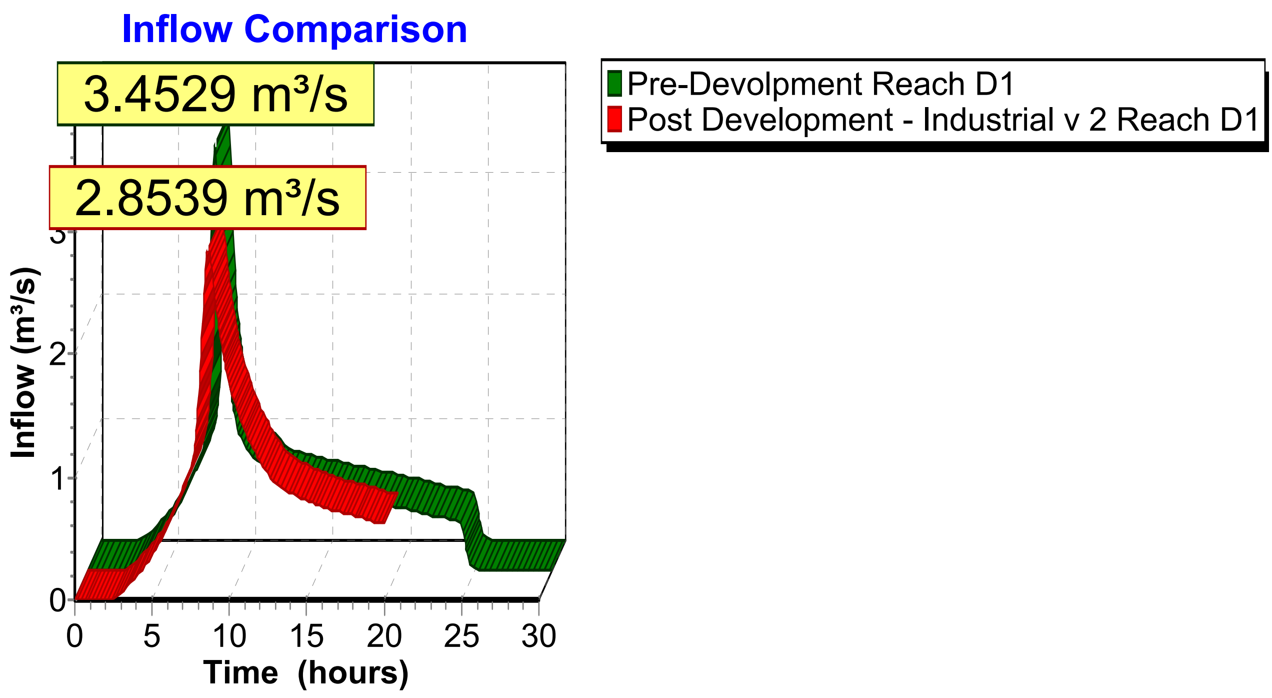


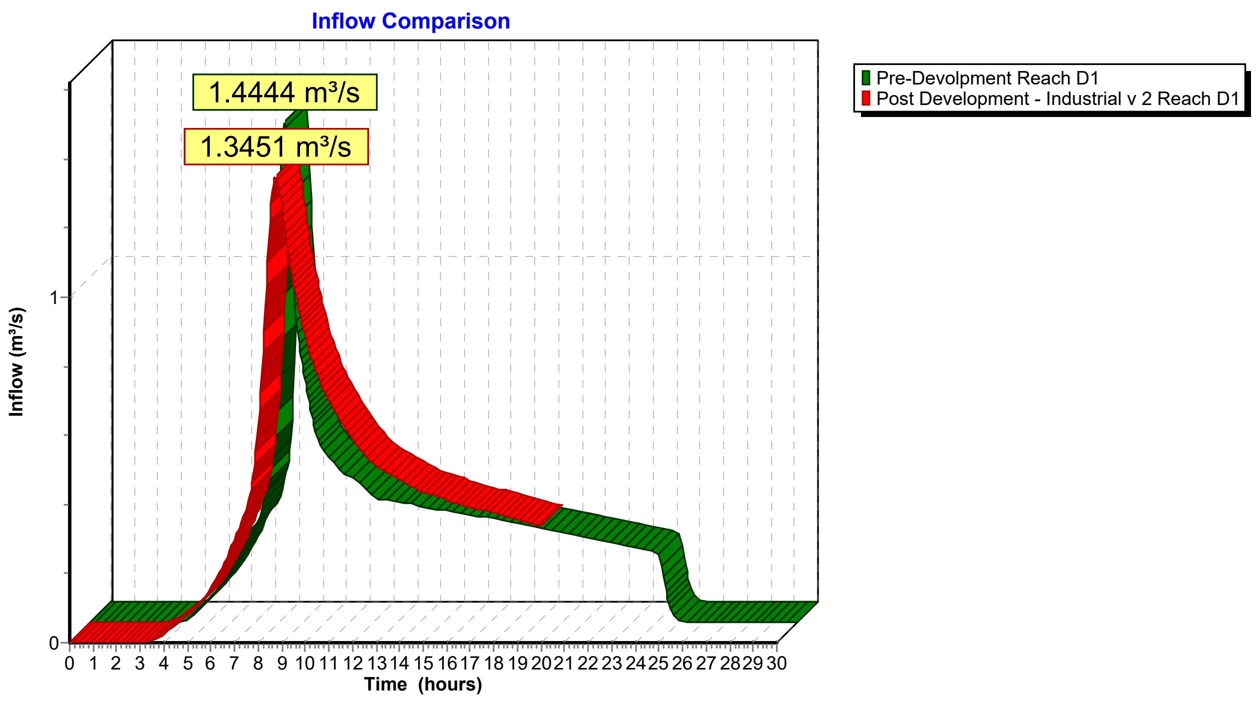
Reach D1: Roadside Table Drain

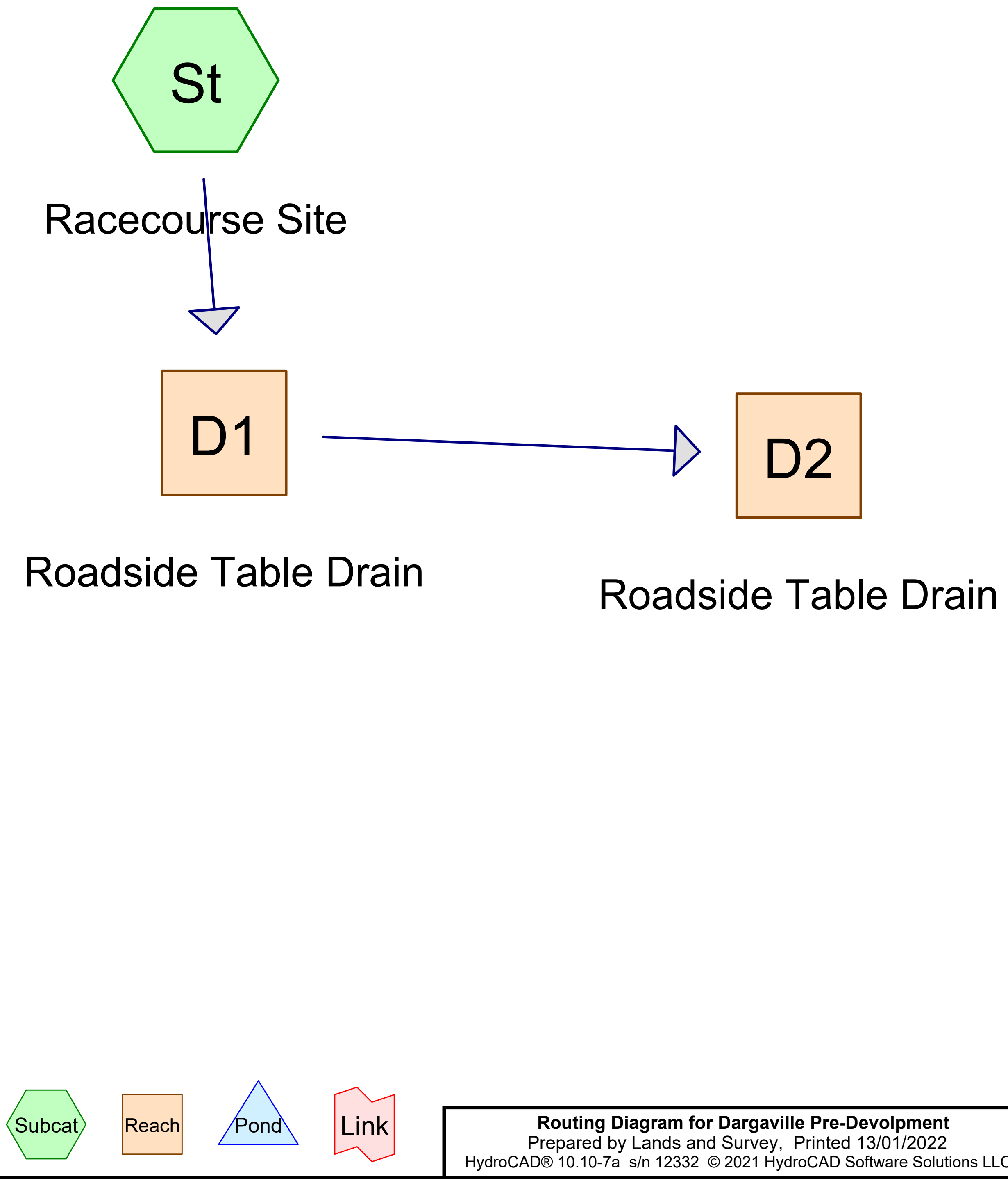



Primary Comparison
3.4316 m³/s
2.8475 m³/s
Primary (m³/s)
0
1
2
3
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
Time (hours)
Pre-Devolpment Reach D1
Post Development - Industrial v 2 Reach D1


Primary Comparison
1.4360 m³/s
1.3422 m³/s
Pre-Devolpment Reach D1
Post Development - Industrial v 2 Reach D1
Primary (m³/s)
0
1
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
Time (hours)


Inflow Comparison
3.4529 m³/s
2.8539 m³/s
Inflow (m³/s)
0
1
2
3
0
5
10
15
20
25
30
Time (hours)
Pre-Devolpment Reach D1
Post Development - Industrial v 2 Reach D1


Inflow Comparison
1.4444 m³/s
1.3451 m³/s
Pre-Devolpment Reach D1
Post Development - Industrial v 2 Reach D1
Inflow (m³/s)
0
1
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
Time (hours)



Routing Diagram for Dargaville Pre-Devolpment
Prepared by Lands and Survey, Printed 13/01/2022
HydroCAD® 10.10-7a s/n 12332 © 2021 HydroCAD Software Solutions LLC

Rainfall Events Listing

Event#	Event Name	Storm Type	Curve	Mode	Duration (hours)	B/B	Depth (mm)	AMC
1	1% Pre Dev	Type IA 24-hr		Default	24.00	1	171	2
2	20% Pre Dev	Type IA 24-hr		Default	24.00	1	96	2

Area Listing (all nodes)

Area (hectares)	CN	Description (subcatchment-numbers)
45.0623	84	Pasture/grassland/range, Fair, HSG D (St)
45.0623	**84**	**TOTAL AREA**

Time span=0.00-30.00 hrs, dt=0.05 hrs, 601 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment St: Racecourse Site Runoff Area=45.0623 ha 0.00% Impervious Runoff Depth=124 mm
Tc=30.0 min CN=84 Runoff=3.4529 m³/s 55.924 Ml

Reach D1: Roadside Table Avg. Flow Depth=1.57 m Max Vel=1.07 m/s Inflow=3.4529 m³/s 55.924 Ml
n=0.035 L=159.00 m S=0.0025 m/m Capacity=2.1546 m³/s Outflow=3.4316 m³/s 55.924 Ml

Reach D2: Roadside Table Avg. Flow Depth=1.65 m Max Vel=0.96 m/s Inflow=3.4316 m³/s 55.924 Ml
n=0.035 L=222.00 m S=0.0018 m/m Capacity=2.8258 m³/s Outflow=3.3909 m³/s 55.924 Ml

Total Runoff Area = 45.0623 ha Runoff Volume = 55.924 Ml Average Runoff Depth = 124 mm
100.00% Pervious = 45.0623 ha 0.00% Impervious = 0.0000 ha

Summary for Subcatchment St: Racecourse Site

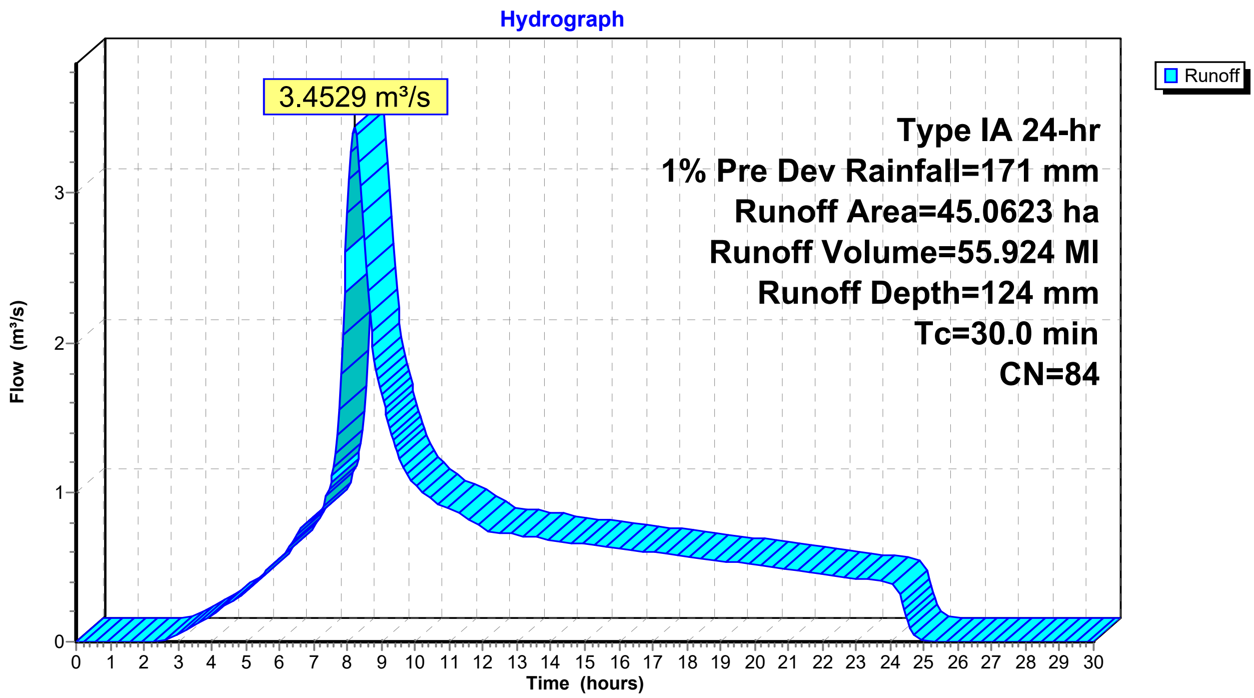
Runoff = 3.4529 m³/s @ 8.21 hrs, Volume= 55.924 Ml, Depth= 124 mm
Routed to Reach D1 : Roadside Table Drain

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 1% Pre Dev Rainfall=171 mm

Area (ha)	CN	Description
45.0623	84	Pasture/grassland/range, Fair, HSG D
45.0623		100.00% Pervious Area

Tc (min)	Length (meters)	Slope (m/m)	Velocity (m/sec)	Capacity (m³/s)	Description
30.0					**Direct Entry, Combined TC**

Subcatchment St: Racecourse Site



Summary for Reach D1: Roadside Table Drain

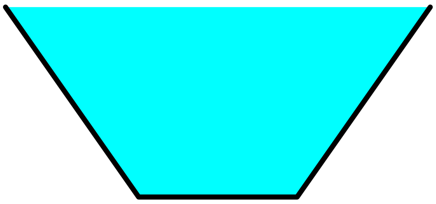
[91] Warning: Storage range exceeded by 0.372 m
[55] Hint: Peak inflow is 160% of Manning's capacity

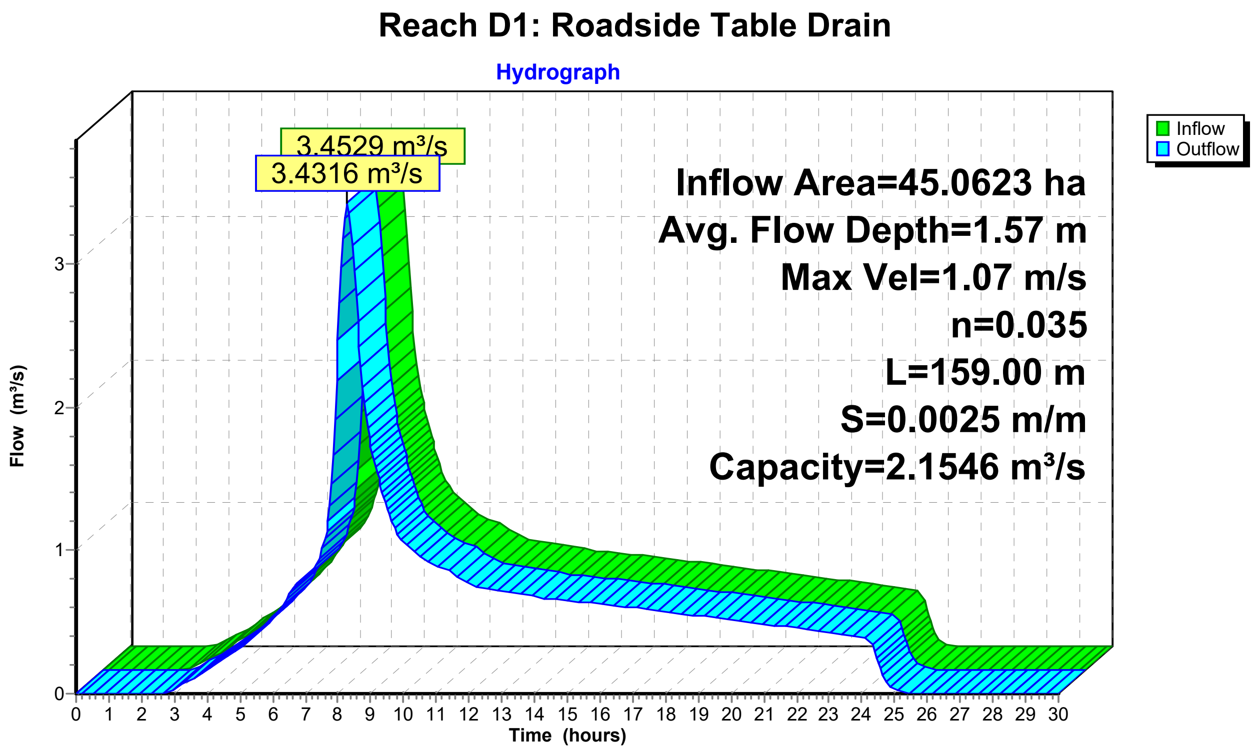
Inflow Area = 45.0623 ha, 0.00% Impervious, Inflow Depth = 124 mm for 1% Pre Dev event
Inflow = 3.4529 m³/s @ 8.21 hrs, Volume= 55.924 Ml
Outflow = 3.4316 m³/s @ 8.29 hrs, Volume= 55.924 Ml, Atten= 1%, Lag= 4.5 min
Routed to Reach D2 : Roadside Table Drain

Routing by Stor-Ind+Trans method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Max. Velocity= 1.07 m/s, Min. Travel Time= 2.5 min
Avg. Velocity = 0.60 m/s, Avg. Travel Time= 4.4 min

Peak Storage= 509.0 m³ @ 8.25 hrs
Average Depth at Peak Storage= 1.57 m , Surface Width= 3.20 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 2.1546 m³/s

1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 159.00 m Slope= 0.0025 m/m
Inlet Invert= 3.800 m, Outlet Invert= 3.400 m





Summary for Reach D2: Roadside Table Drain

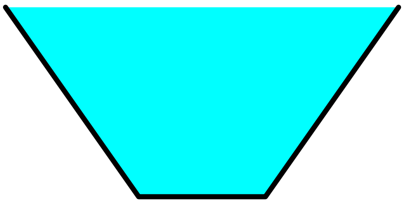
[91] Warning: Storage range exceeded by 0.153 m
[55] Hint: Peak inflow is 121% of Manning's capacity
[62] Hint: Exceeded Reach D1 OUTLET depth by 0.199 m @ 8.60 hrs

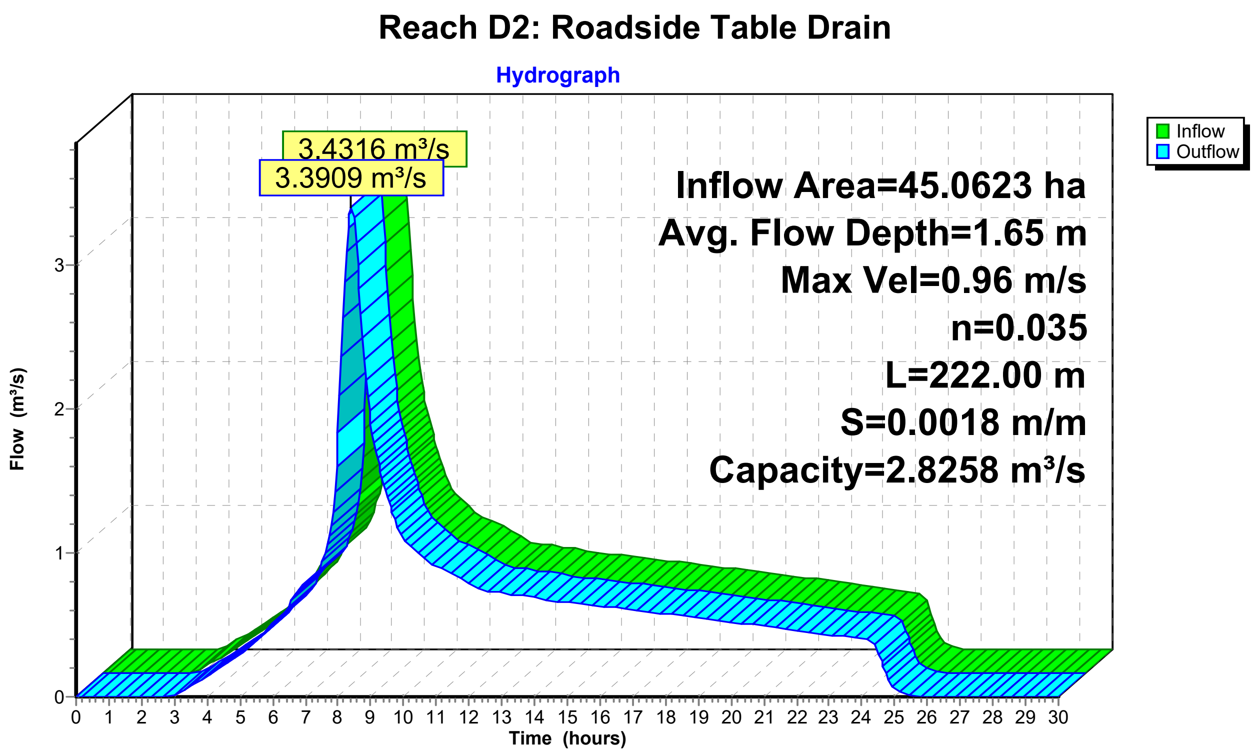
Inflow Area = 45.0623 ha, 0.00% Impervious, Inflow Depth = 124 mm for 1% Pre Dev event
Inflow = 3.4316 m³/s @ 8.29 hrs, Volume= 55.924 Ml
Outflow = 3.3909 m³/s @ 8.40 hrs, Volume= 55.924 Ml, Atten= 1%, Lag= 6.9 min
Routed to nonexistent node D3

Routing by Stor-Ind+Trans method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.96 m/s, Min. Travel Time= 3.9 min
Avg. Velocity = 0.52 m/s, Avg. Travel Time= 7.2 min

Peak Storage= 787.5 m³ @ 8.34 hrs
Average Depth at Peak Storage= 1.65 m , Surface Width= 3.31 m
Bank-Full Depth= 1.50 m Flow Area= 3.08 m², Capacity= 2.8258 m³/s

1.00 m x 1.50 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 3.10 m
Length= 222.00 m Slope= 0.0018 m/m
Inlet Invert= 3.400 m, Outlet Invert= 3.000 m





Time span=0.00-30.00 hrs, dt=0.05 hrs, 601 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment St: Racecourse Site Runoff Area=45.0623 ha 0.00% Impervious Runoff Depth=55 mm
Tc=30.0 min CN=84 Runoff=1.4444 m³/s 24.928 Ml

Reach D1: Roadside Table Avg. Flow Depth=0.97 m Max Vel=0.88 m/s Inflow=1.4444 m³/s 24.928 Ml
n=0.035 L=159.00 m S=0.0025 m/m Capacity=2.1546 m³/s Outflow=1.4360 m³/s 24.928 Ml

Reach D2: Roadside Table Avg. Flow Depth=1.05 m Max Vel=0.78 m/s Inflow=1.4360 m³/s 24.928 Ml
n=0.035 L=222.00 m S=0.0018 m/m Capacity=2.8258 m³/s Outflow=1.4121 m³/s 24.928 Ml

Total Runoff Area = 45.0623 ha Runoff Volume = 24.928 Ml Average Runoff Depth = 55 mm
100.00% Pervious = 45.0623 ha 0.00% Impervious = 0.0000 ha

Summary for Subcatchment St: Racecourse Site

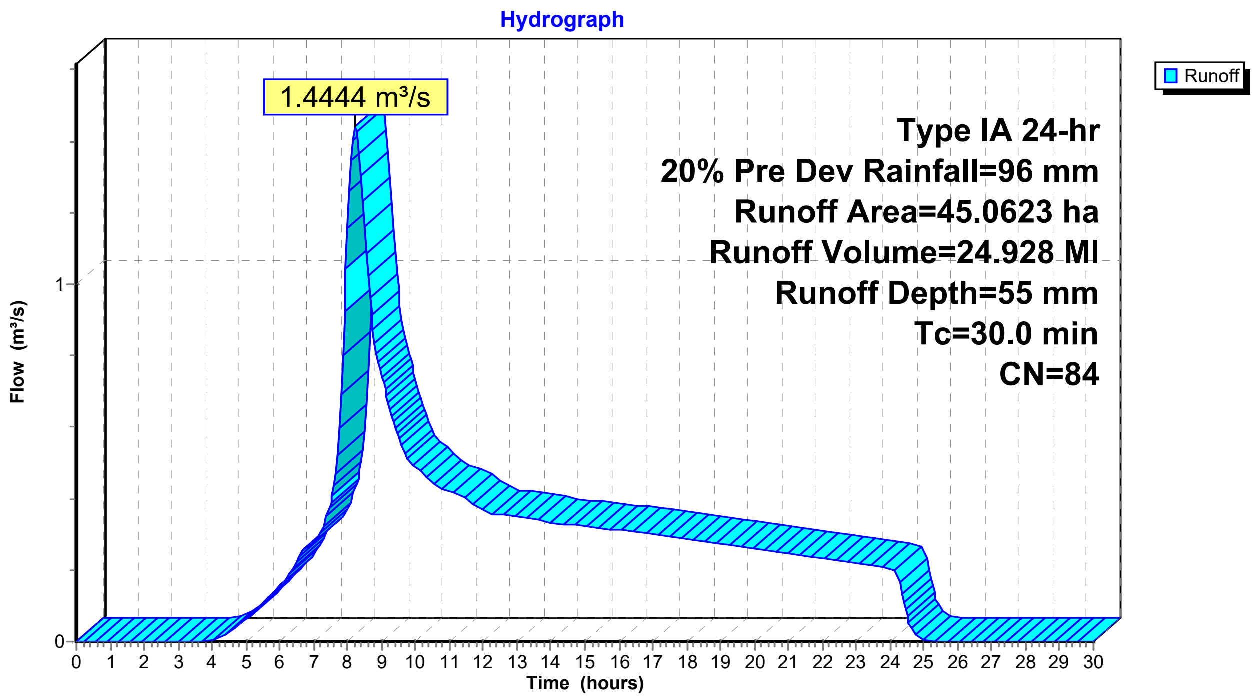
Runoff = 1.4444 m³/s @ 8.23 hrs, Volume= 24.928 Ml, Depth= 55 mm
Routed to Reach D1 : Roadside Table Drain

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 20% Pre Dev Rainfall=96 mm

Area (ha)	CN	Description
45.0623	84	Pasture/grassland/range, Fair, HSG D
45.0623		100.00% Pervious Area

Tc (min)	Length (meters)	Slope (m/m)	Velocity (m/sec)	Capacity (m³/s)	Description
30.0					**Direct Entry, Combined TC**

Subcatchment St: Racecourse Site



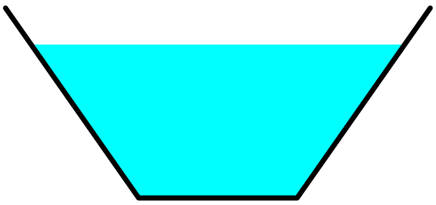
Summary for Reach D1: Roadside Table Drain

Inflow Area = 45.0623 ha, 0.00% Impervious, Inflow Depth = 55 mm for 20% Pre Dev event
Inflow = 1.4444 m³/s @ 8.23 hrs, Volume= 24.928 Ml
Outflow = 1.4360 m³/s @ 8.32 hrs, Volume= 24.928 Ml, Atten= 1%, Lag= 5.3 min
Routed to Reach D2 : Roadside Table Drain

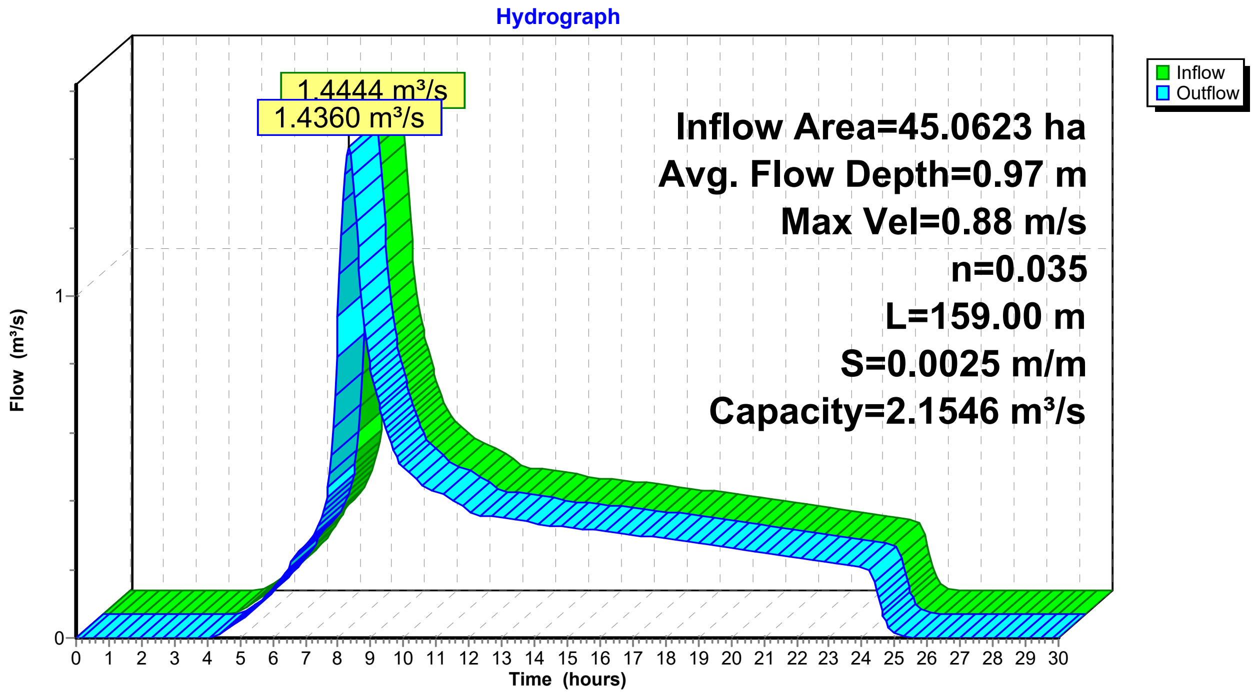
Routing by Stor-Ind+Trans method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.88 m/s, Min. Travel Time= 3.0 min
Avg. Velocity = 0.49 m/s, Avg. Travel Time= 5.4 min

Peak Storage= 258.7 m³ @ 8.27 hrs
Average Depth at Peak Storage= 0.97 m , Surface Width= 2.36 m
Bank-Full Depth= 1.20 m Flow Area= 2.21 m², Capacity= 2.1546 m³/s

1.00 m x 1.20 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 2.68 m
Length= 159.00 m Slope= 0.0025 m/m
Inlet Invert= 3.800 m, Outlet Invert= 3.400 m



Reach D1: Roadside Table Drain



Summary for Reach D2: Roadside Table Drain

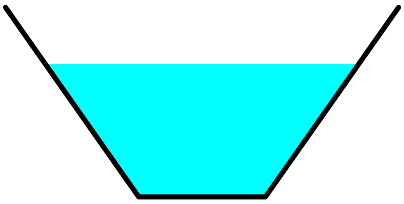
[62] Hint: Exceeded Reach D1 OUTLET depth by 0.145 m @ 8.60 hrs

Inflow Area = 45.0623 ha, 0.00% Impervious, Inflow Depth = 55 mm for 20% Pre Dev event
Inflow = 1.4360 m³/s @ 8.32 hrs, Volume= 24.928 Ml
Outflow = 1.4121 m³/s @ 8.46 hrs, Volume= 24.928 Ml, Atten= 2%, Lag= 8.5 min
Routed to nonexistent node D3

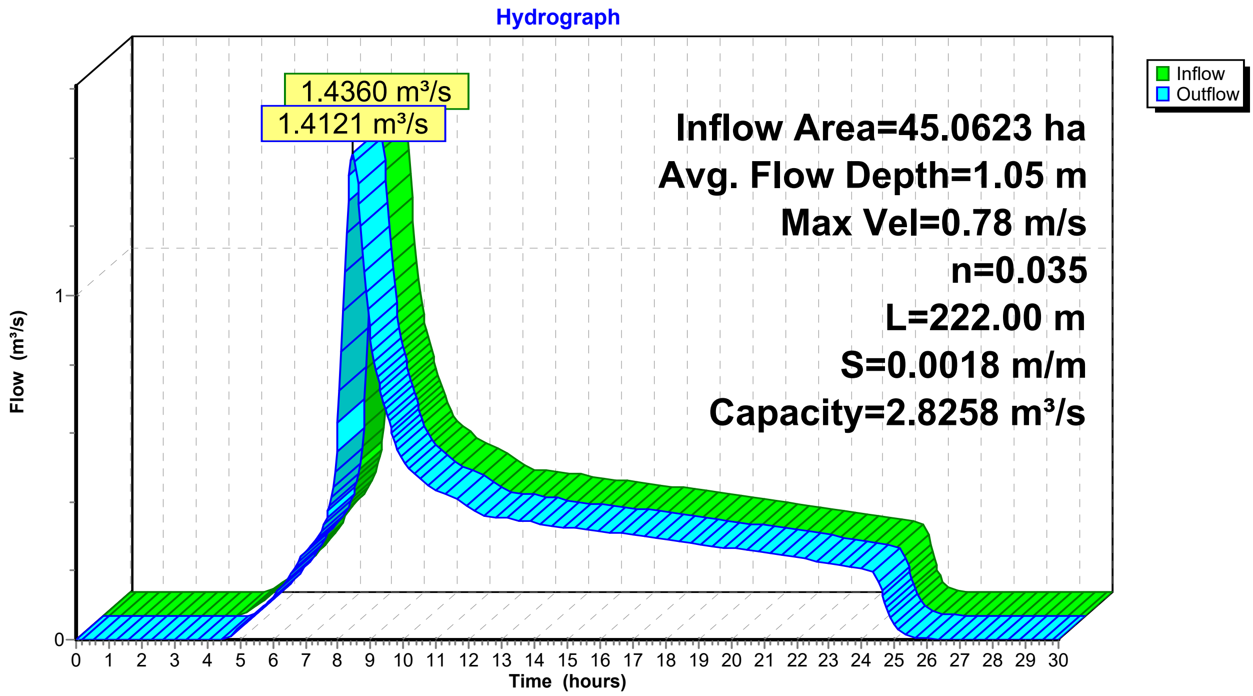
Routing by Stor-Ind+Trans method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Max. Velocity= 0.78 m/s, Min. Travel Time= 4.8 min
Avg. Velocity = 0.42 m/s, Avg. Travel Time= 8.8 min

Peak Storage= 405.0 m³ @ 8.38 hrs
Average Depth at Peak Storage= 1.05 m , Surface Width= 2.47 m
Bank-Full Depth= 1.50 m Flow Area= 3.08 m², Capacity= 2.8258 m³/s

1.00 m x 1.50 m deep channel, n= 0.035 Earth, dense weeds
Side Slope Z-value= 0.7 m/m Top Width= 3.10 m
Length= 222.00 m Slope= 0.0018 m/m
Inlet Invert= 3.400 m, Outlet Invert= 3.000 m



Reach D2: Roadside Table Drain



Appendix D - Water Network Capacity Analysis (Memo from AWA Environmental Limited)

MEMO

TO:	Henk de Wet	**DATE:**	13th August 2021
FROM:	Kirsten Henden	**PROJECT NO.:**	J000434
REVIEWED:	James Taylor		
SUBJECT:	Dargaville Racecourse Development		

INTRODUCTION

Dargaville Racing Club Incorporated intend to submit a Plan Change to support the development of the existing Dargaville Racing Club (DRC) site in Dargaville. The proposed Plan Change aims to allow a mixed-use development, consisting generally of the land use details supplied by Lands and Survey Limited as follows:

Table 1 Concept development plan with yields (supplied by Lands and Survey Limited)

ZONE	AREA OF ZONE (M^2)	DENSITY OF LOTS (M^2)	LOT YIELD (LOTS)	COMMENT
Light Industrial	72,000	500	115	Assumed occupancy is an average of 4 to 6 people from 8 am to 6 pm
Medium Density Residential	75,000	300	200	
General Residential	120,000	400-500	213	Used 450 m^2 for calculations
Low Density Residential	28,000	1,000	22	
Total			500	

Lands and Survey Limited have engaged Awa Environmental Ltd (Awa), on behalf of DRC, to undertake a high-level capacity assessment of the proposed new development on Kaipara District Council's water distribution network. The objective of the assessment will be to identify any adverse effects on the network as a result of the development, and to determine whether the network has capacity to meet the required level of service to support the intensified land use.

MODELLING APPROACH

The Dargaville Water Supply Projected Population Growth Model has been used to assess the impact of the proposed new development on the water distribution network. This model has been built using DHI's MIKE URBAN modelling software and includes the entirety of the Dargaville water reticulation network downstream of the Dargaville Water Treatment Plant.

This model includes an increase in demand as a result of future development and intensification to represent a future scenario representative of the Proposed KDC Spatial Plan, and projected population increases in the Kaipara District.

ASSUMPTIONS

The following assumptions were made in modelling:

- The Demand Pattern is based on flow monitoring completed in 2012, adjusted to include a 4x peaking factor, whilst retaining the overall daily volume.
- Existing demand has been included based on a 4-year period of water meter readings.
- Future domestic demand has been calculated based on a daily usage of 250L/person and occupancy of 2.2 persons per lot. For the Light Industrial zone, an occupancy of 5 persons has been used based on the information provided in Table 1. The Daily Demand Volume for each zone is shown in Table 2.
- The total Daily Demand Volume for the proposed development has been applied at a single node within the network.

Table 2 Daily Demand Volume for each zone

ZONE	LOT YIELD (LOTS)	OCCUPANCY (PERSONS)	DAILY DEMAND (L/S)
Light Industrial	115	5	1.66
Medium Density Residential	200	2.2	1.27
General Residential	213	2.2	1.36
Low Density Residential	22	2.2	0.14
Total			4.43

NETWORK ANALYSIS

The network has been upgraded since the Dargaville water supply model was built and there is now a 180mm PE main in place of a 100mm AC main along State Highway 14, adjacent to the Racecourse. The model has been updated to include this upgrade, which will be referred to as the pre-development scenario. An assessment of the pre-development, in comparison with the post-development scenario has been carried out.

The model suggests that there is sufficient capacity within the network to supply the proposed development. The model predicts a reduction in supply pressures from 35m head to 25m head, however this reduction is still within recommended limits (25-80m head). The minimum network pressure and maximum head loss pre- and post-development is shown in Figure 1.

Figure 1 Pre-Development (left) and Post-Development (right) network capacity

CONCLUSIONS

Modelling of the proposed development in conjunction with the new 180 mm PE main shows that there is sufficient capacity in the network to supply the development, and that there are no adverse on the ability of the network to meet KDC's Levels of Service.

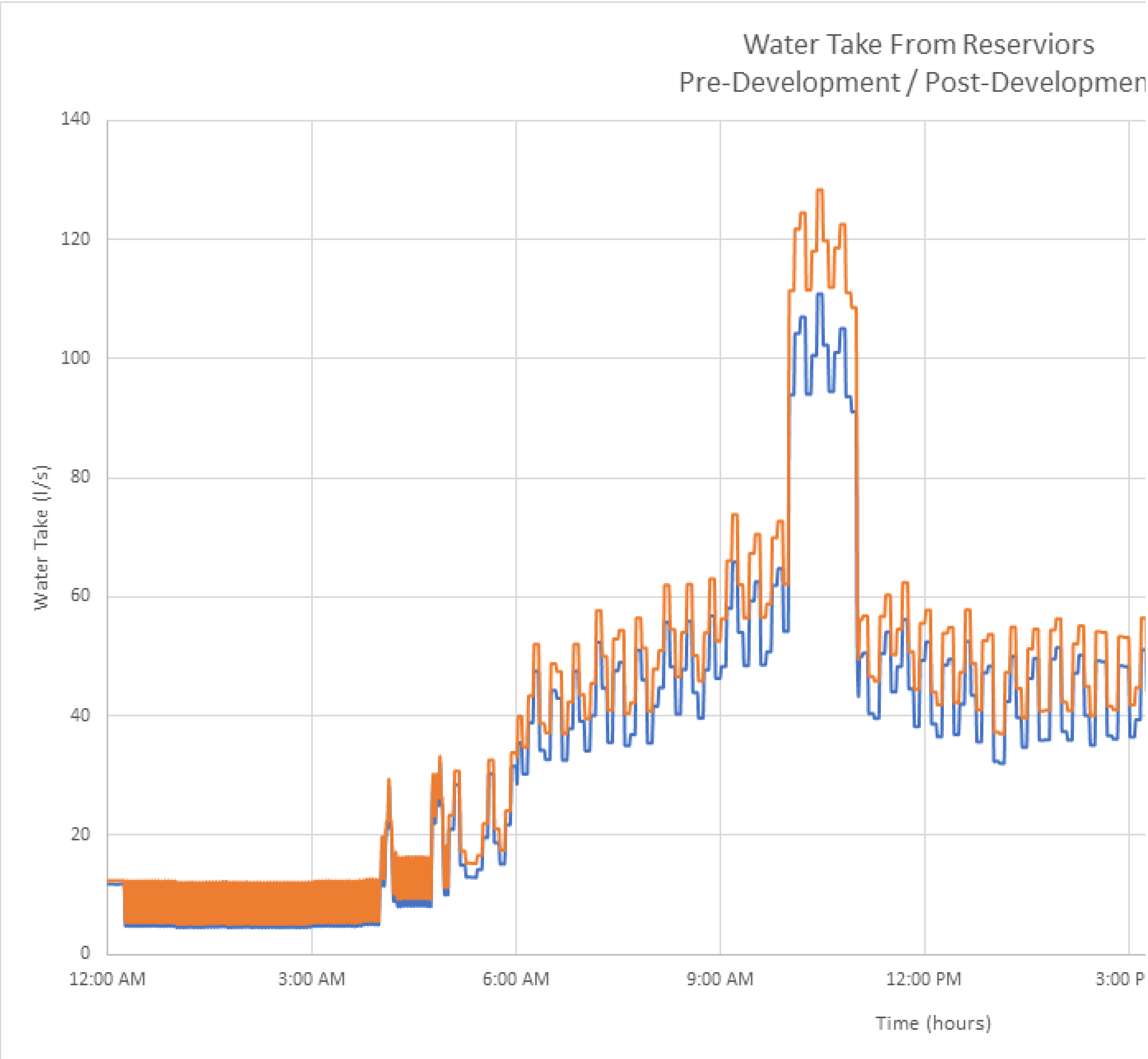
Henk de Wet

From: Kirsten Henden <Kirsten.Henden@awa.kiwi>
Sent: Friday, 24 September 2021 8:21 am
To: Henk de Wet
Cc: Rathika Jebamony
Subject: RE: Dargaville racecourse re-development - 180mm PE pipe

Hi Henk,

Thanks for your enquiry. The information you've requested is below:

	Pre-Development	Post-Development
Maximum flow (l/s)	111	128
Total water take (m3)	3155	3533



Let me know if we can help with anything else.

Regards,
Kirsten

KIRSTEN HENDEN
MODELLER
a: Level 9, 4 Williamson Ave, Grey Lynn, Auckland 1021
m: +64 27 366 8334 e: kirsten.henden@awa.kiwi w: www.awa.kiwi

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From: Henk de Wet <henk@landsandsurvey.co.nz>
Sent: Tuesday, 14 September 2021 9:32 AM
To: Rathika Jebamony <rathika.jebamony@awa.kiwi>
Cc: Kirsten Henden <Kirsten.Henden@awa.kiwi>
Subject: RE: Dargaville racecourse re-development - 180mm PE pipe

Hi Rathika

Thanks for taking my call earlier. As requested during our conversation, please can you enquire the water network model to establish the following:

- Water take from the command reservoir over a 24-hour period (for current and with the proposed development respectively i.e. pre and post development)
- Peak flow from the reservoir

It may be useful if you can produce water take curves on the outlet from the reservoir over a 24-hour period for the 2 scenarios (i.e. pre and post development)

Council's maintenance contractor (Ventia) has provided me with water production rates for the plant which I want to compare to the theoretical demand from the network, pre and post development.

Time and associated cost to extract this information can be charged as a Variation to the initial agreement.

Thanks,

Henk de Wet | Technical Director - Lands and Survey Engineering |
021 024 99917 | henk@landsandsurvey.co.nz |

From: Kirsten Henden <Kirsten.Henden@awa.kiwi>
Sent: Friday, 13 August 2021 3:44 pm
To: Henk de Wet <henk@landsandsurvey.co.nz>
Cc: Rathika Jebamony <rathika.jebamony@awa.kiwi>
Subject: RE: Dargaville racecourse re-development - 180mm PE pipe

Hi Henk,

As Rathika mentioned below, the model suggests that there is sufficient capacity within the network (with 180mm PE pipe) to supply the proposed development. Please see attached for a summary of our assessment.

Regards,
Kirsten

KIRSTEN HENDEN
MODELLER
a: Level 9, 4 Williamson Ave, Grey Lynn, Auckland 1021
m: +64 27 366 8334 e: kirsten.henden@awa.kiwi w: www.awa.kiwi

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From: Rathika Jebamony <rathika.jebamony@awa.kiwi>
Sent: Thursday, 12 August 2021 12:37 PM
To: Henk de Wet <henk@landsandsurvey.co.nz>
Cc: Kirsten Henden <Kirsten.Henden@awa.kiwi>
Subject: Dargaville racecourse re-development - 180mm PE pipe

Hi Henk,

It looks like there is indeed a 180mm PE pipe already heading towards Awakino point that replaces the 100mm AC pipe. (100mm AC pipe is now abandoned). It looks like this pipe was installed last year and updated in the KDC GIS system in June 2021.

KDC's base water distribution model for Dargaville did not include this upgrade (and we were not aware of it). Therefore our analysis assumed the old network was still in operation (100mm AC pipe).

Regardless, it looks like a 180mm PE pipe should be sufficient for the Dargaville Racecourse redevelopment.

Regards,

RATHIKA JEBAMONY
WATER TEAM LEADER
a: Level 9, 4 Williamson Ave, Grey Lynn, Auckland
m: +64 21 770 637 e: rathika.jebamony@awa.kiwi w: www.awa.kiwi

Appendix E - Emailed Discussions - WDC Maintenance Contractor (Ventia)

Henk de Wet

From: Guy, Johan <Johan.Guy@broadspectrum.com>
Sent: Tuesday, 27 July 2021 2:24 pm
To: Henk de Wet
Subject: FW: Dargaville Racecourse Plan Change - Three Waters discussion

Hi Henk

I haven't heard from KDC. I would have thought they send it to you. (Please see below answers in red.)

Kind regards

Johan Guy
Contract Manager – Kaipara 3 Waters

Ventia

M +64 (27) 265 3871

E johan.guy@broadspectrum.com
W www.ventia.com

A list of Ventia Group entities can be found here.

From: Guy, Johan
Sent: Friday, 16 July 2021 8:21 am
To: Donnick Mugutso (dmugutso@kaipara.govt.nz) <dmugutso@kaipara.govt.nz>
Cc: Brian Armstrong <barmstrong@kaipara.govt.nz>; Donna Powell <dpowell@kaipara.govt.nz>
Subject: Dargaville Racecourse Plan Change - Three Waters discussion

Hi Donnick

I've been contacted by Land Survey Engineering asking a view questions.
Question raised by "Land Survey Engineering"

Water and Wastewater

1. Known network issues or constraints, to receive addition flow or provide supply to new development, (note that this query refers to operations issues. Hydraulic capacity will be assessed by AWA by querying the network models),
2. Preferred / available point of connection / discharge including arrangement of such connections, (nodes / pump stations etc, subject tom capacity to receive such connections), New 125mm PE water main to stables. No wastewater connections. Mains and pumpstation required.
3. General operational status of the respective treatment plants, including their treatment / production capacity, current and near future capacity envisaged, based on information at hand and confirmed improvement projects (if any), Potable water plant (settling/rapid sand filtration) design flow rate at 210m3hr, will require consent renewals and favourable seasonal conditions to achieve this. At present averaging around the 120-130 m3hr. Wastewater process design rate 10ha conventional pond treatment (Primary/tertiary ponds) just domestic loading at 84kg BOD could process a population of 12000. We have industrial loading and the process will require an upgrade to achieve this, e.g.. screening, primary or activated sludge process.
4. Actual treatment and productions rates, with possible variation in time of day / seasonal, as per above

5. Restrictions and limitations on treatment / production capacity (i.e. wet weather events with wastewater treatment, raw water supply restrictions for water production) as per above.
6. Details of known/confirmed upgrades / enhancement projects and when they are envisaged to be implemented (if any), Principal to answer.
7. Description of the treatment plants (Type and process). as per above.

"A response to the above items for the respective water and wastewater systems will provide us with a good point of departure and steer to develop conceptual solutions for our client, in support to the application for the proposed private plan change."

We answered the questions to the best of our ability. KDC can access and probably answer remaining question before it is forwarded to Land Survey Engineering.

Kind regards

Johan Guy
Contract Manager – Kaipara 3 Waters

Ventia

M +64 (27) 265 3871

E johan.guy@broadspectrum.com
W www.ventia.com

A list of Ventia Group entities can be found here.

Appendix F - Emailed Discussion and Case Study for Wastewater Alternative

Henk de Wet

From: Brent Hawthorn <brenth@innoflow.co.nz>
Sent: Friday, 3 September 2021 4:19 pm
To: Henk de Wet
Subject: Waipu and Ngati Whatua Projects
Attachments: NCS-55_JacksPoint_v1.pdf; Prelos Brochure.pdf; SHO-EFS-1 Considering a Pressure Sewer.pdf

Follow Up Flag: Follow up
Flag Status: Flagged

Hi Henk,

Thank you for your time on the phone today.

To get the ball rolling please find attached some information on our Liquid Only Sewers (equivalent to a LPS but discharging liquid).

We call our tanks Prelos (PRessurised Liquid Only Sewer), but they have been known as STEP sewers for 25 years. The first STEP sewer was installed in Mangawhai in 1994, so we have a long history of their use in NZ.

Our largest liquid only sewer is Jacks Point Subdivision in Queenstown, which will be over 850 tanks once fully built. There are over 500 installed currently.

I am attaching a Brochure on Prelos and a Case Study on Jacks Point, and a comparison between (STEP/Prelos and Grinders).

Some other things to consider:

1. Prelos provide 50% or more treatment on the owners property before discharging effluent into a main and off to a Council/Private treatment plant. This has been a significant criterion for other iwi groups around NZ.
2. We offer a 10 year warranty on our Prelos pump. Grinders only have 2 years.
3. Prelos Tanks reduce solids by up to 80% and need desludging generally once a decade.
4. Prelos has a fraction of the OPEX cost of grinders (at least half).
5. There are plenty more benefits which we can cover if this goes further.

We also have plenty of reference sites and clients (Council/consultants and developers), which we can pass on as needed.

Please let me know if you would like anything more specific for these projects.

Regards

Brent Hawthorn
021-749-126
innoflow
wastewater specialists
311A Postman Road, RD4,
Albany 0794
New Zealand
PO Box 300 572, Albany,
North Shore City 0752,
New Zealand
www.innoflow.co.nz| 09-426-1027| 09-426-1047

Appendix G - Guidelines for The Construction of Decanting Earth Bunds and Silt Fences

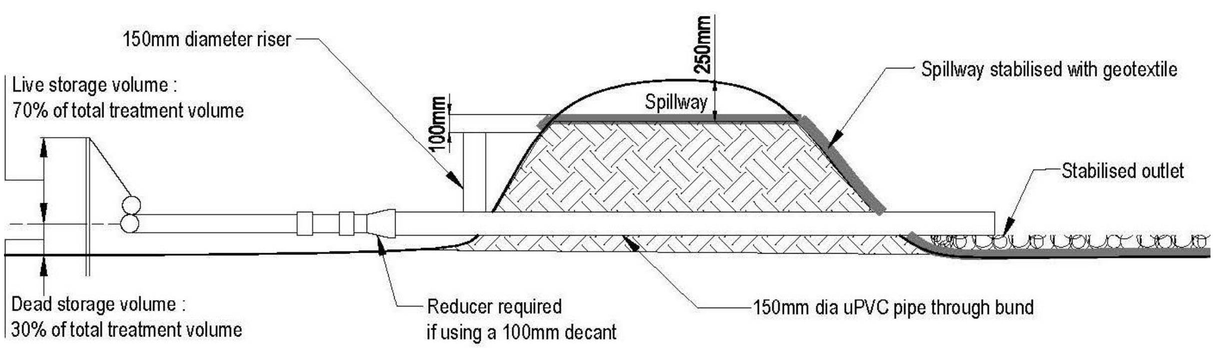
Appendix C1.13 Decanting earth bund (DEB)

Contractor:	Date: Time:	Consent #:	Site:

Construction checklist (refer Figures over page and Section F1.2 of GD05 for further details)	Yes (✓)	No (X) (Add comments to explain)
DEB has been built along the contour to obtain the required volumes		
All organic/ vegetation is removed before construction		
The DEB is keyed into the existing ground to a minimum depth of 0.3 m		
The DEB is built with a clay-silt mix of suitable moisture content to achieve a reasonable compaction standard (90%). This can be achieved, in most instances, by track rolling at 150 – 200 mm lifts. Particular care is required to achieve good compaction around the outlet pipe that passes through the bund to avoid seepage and potential failure		
A 150 mm diameter non-perforated outlet pipe has been installed through the bund and discharges to a stable erosion proofed area or stormwater system		
A T-Bar decant has been attached by way of a standard joint (glued and screwed). The decant is 100 or 150 mm dia. PVC pipe, 0.5 m long with equally spaced holes of 10 mm diameter and fixed firmly to a waratah standard to achieve 0.3 litres/ second/1,000 m^2 of contributing catchment		
A sealed PVC pipe (with endcaps) has been placed on top of the decant to provide buoyancy		
A flexible thick rubber coupling has been used to provide a connection between the decant arm and the discharge pipe. The flexible coupling has been fastened using strap clamps, glue and screws		
The decant is fastened to two waratahs by way of a nylon cord to the correct height		

Construction checklist (refer Figures over page and Section F1.2 of GD05 for further details)	Yes (✓)	No (X) (Add comments to explain)
An emergency spillway has been provided to a stabilised outfall 100 mm freeboard height above the primary spillway. This can be a trapezoidal spillway with a minimum invert length of 2 m that is smooth, has no voids and is lined with a soft needle punched geotextile to the stabilised outfall. The geotextile is pinned at 0.5 m centres		
The emergency spillway has a minimum freeboard of 250 mm, i.e. between the invert of the spillway to the lowest point of the top of the bund		
An as-built assessment has been completed at the completion of construction to check against the design. Any discrepancies have been rectified.		

Note: The purpose of this checklist is for contractors to complete on-site self-checks of construction quality for ESC practices. This is not a compliance or as-built checklist.



Cross - section

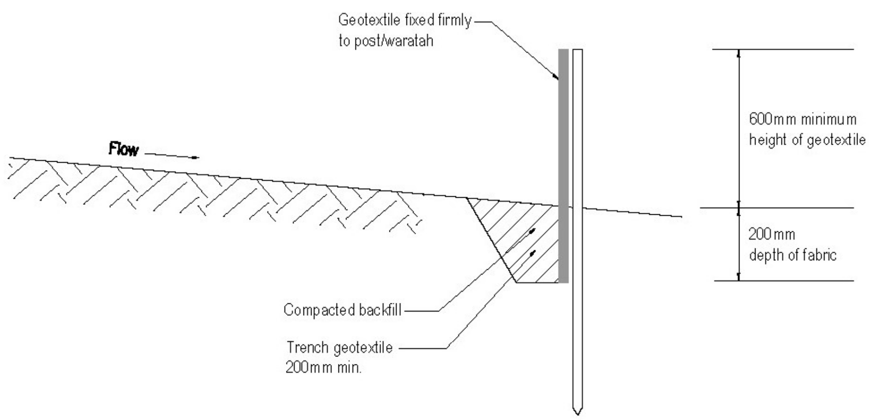
Appendix C1.14 Silt fence

Contractor:	Date: Time:	Consent #:	Site:

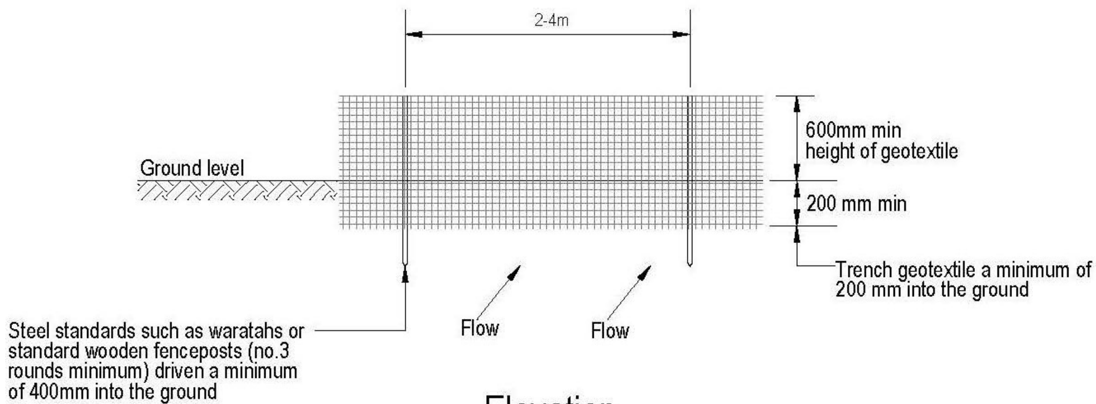
Construction checklist (refer Table and Figure over page and Section F1.3 of GD05 for further details)	Yes (✓)	No (X) (Add comments to explain)
The silt fence material used is appropriate to the site conditions and in accordance with the manufacturer's specifications		
Silt fences have been installed along the contour		
A trench of a minimum of 100 mm wide and 200 mm deep has been excavated along the proposed line of the silt fence		
Supporting posts /steel waratahs are installed at least 1.5 m length and 2-4 m apart		
Support posts/waratahs are installed on the down-slope edge of the trench, with silt fence fabric on the up-slope side of the support posts to the full depth of the trench. The trench is backfilled with compacted soil		
The top of the silt fence fabric is reinforced with a support made of high tensile 2.5 mm diameter galvanised wire. The wire is tensioned using permanent wire strainers attached to angled waratahs at the end of the silt fence		
The silt fence fabric is doubled over and fastened to the wire with silt fence clips at 500 mm spacings		
Where ends of the silt fence fabric come together, they are overlapped, folded and stapled/screwed to prevent sediment bypass		

Note: The purpose of this checklist is for contractors to complete on-site self-checks of construction quality for ESC practices. This is not a compliance or as-built checklist.

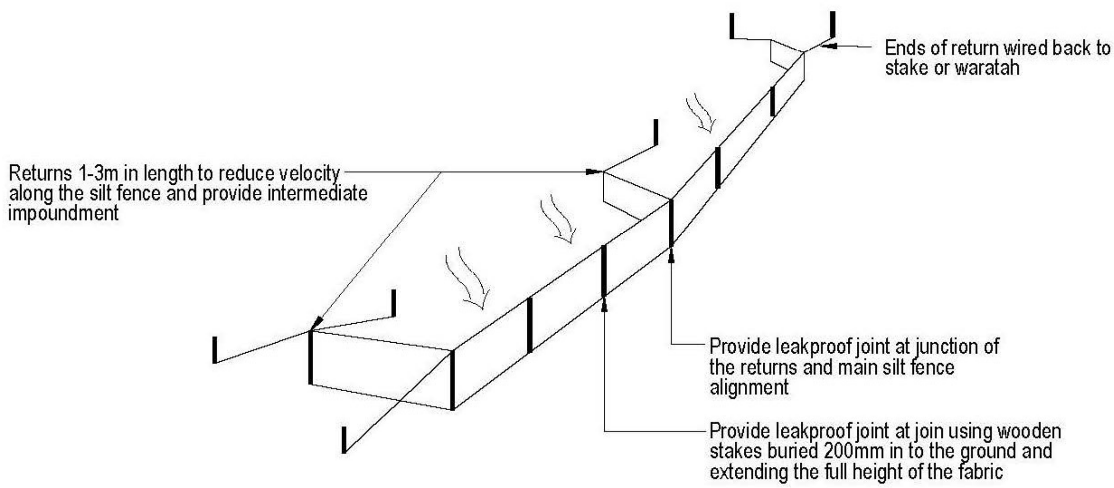
Slope steepness %	Slope length (m) (maximum)	Spacing of returns (m)	Silt fence length (m) (maximum)
Flatter than 2%	Unlimited	N/A	Unlimited
2 – 10%	40	60	300
10 – 20%	30	50	230
20 – 33%	20	40	150
33 – 50%	15	30	75
> 50%	6	20	40



Cross section



Elevation



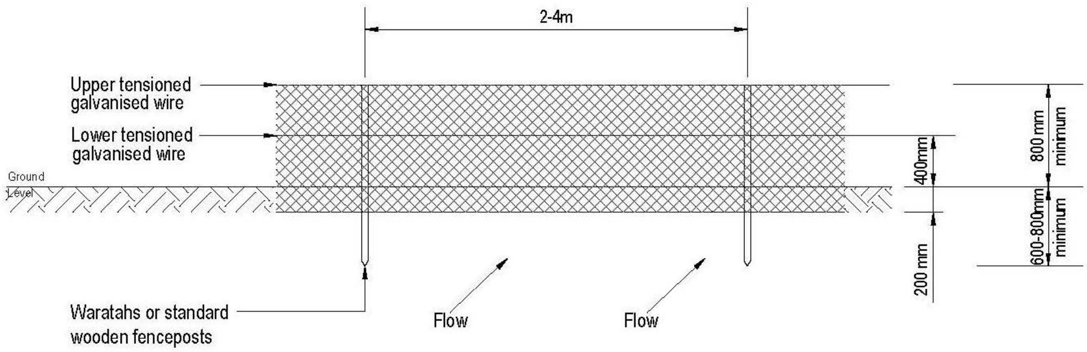
Silt fence with returns and support wire

Appendix C1.15 Super silt fence

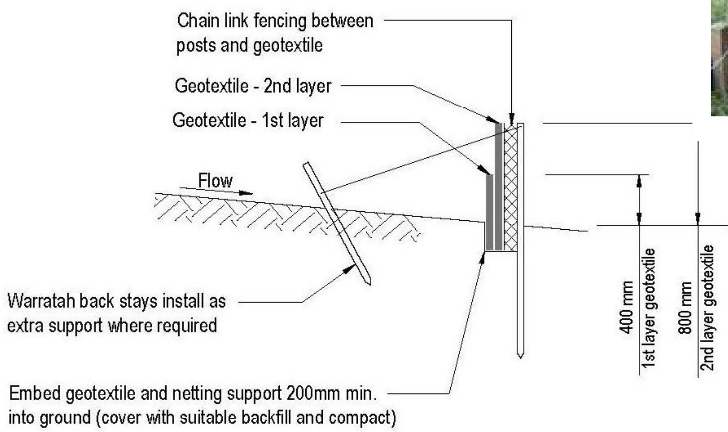
Contractor:	Date: Time:	Consent #:	Site:

Construction checklist (refer Figure and table over page and Section F1.4 of GD05 for further details)	Yes (✓)	No (X) (Add comments to explain)
Super silt fence material used is appropriate to the site conditions and in accordance with the manufacturer's specifications		
Super silt fences are installed along the contour		
A trench of a minimum of 100 mm wide and 200 mm deep has been excavated along the proposed line of the silt fence		
Supporting posts /steel waratahs are installed at least 1.8 m length and 2–4 m apart		
Support posts/waratahs are installed on the down-slope edge of the trench, with silt fence fabric on the up-slope side of the support posts to the full depth of the trench. The trench is backfilled with compacted soil		
Tensioned galvanised wire (2.5 mmHT) is installed at 400 mm and again at 800 mm above ground. The wire has been tensioned using permanent wire strainers attached to angled waratahs at the end of the super silt fence		
Chain link fence is secured to the fence posts with wire ties or staples, ensuring the chain link fence goes to the base of the trench		
Two layers of geotextile fabric are secured to the base of the trench (a minimum of 200 mm into the ground), with compacted backfill installed to the original ground level		
Where ends of the silt fence fabric come together, they are overlapped, folded and stapled/screwed to prevent sediment bypass		

Note: The purpose of this checklist is for contractors to complete on-site self-checks of construction quality for ESC practices. This is not a compliance or as-built checklist.



Elevation



Cross - section

Slope steepness %	Slope length (m) (maximum)	Spacing of returns (m)	Super silt fence length (m) (maximum)
0 – 10%	Unlimited	60	Unlimited
10 – 20%	60	50	450
20 – 33%	30	40	300
33 – 50%	30	30	150
> 50%	N/A	20	N/A