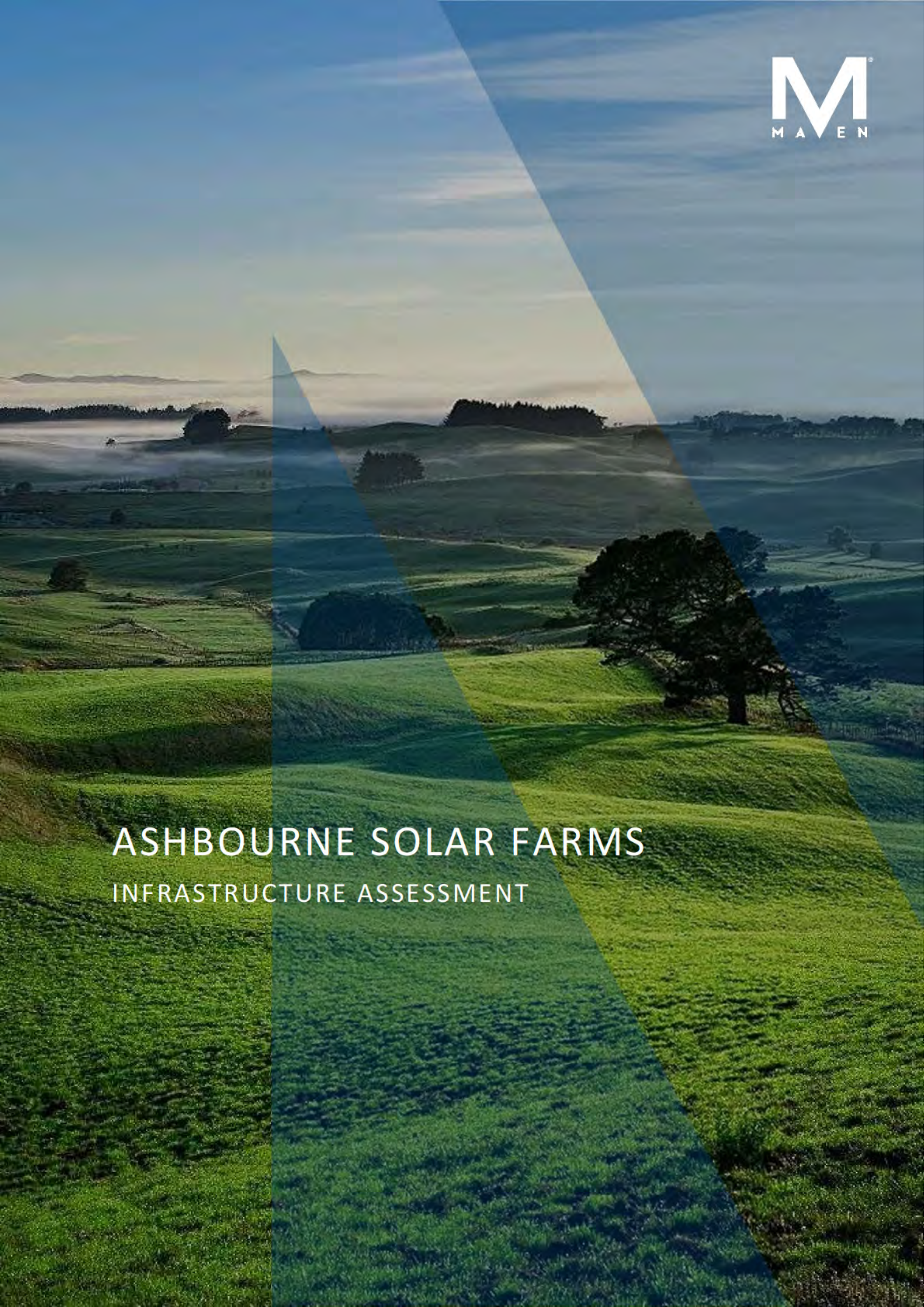

ASHBOURNE SOLAR FARMS

INFRASTRUCTURE ASSESSMENT

PROJECT INFORMATION

CLIENT	Matamata Developments Ltd
PROJECT	289001

DOCUMENT CONTROL

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Contents

1. Introduction

1.1. Background

Maven Waikato Ltd have been engaged by Matamata Developments Ltd to explore the feasibility and concept of land development for two new solar farms in Matamata. These solar farms sites will have dual use, with agrivoltaics farming proposed to be undertaken underneath the solar panels to promote sustainability and preserve the identified highly productive land. Typical landscaping, planting and security will complement the solar farms to ensure their integration with the wider Ashbourne development.

The two solar farm sites, which are known as the northern and southern solar farms will produce enough power energy for over 7,000 homes per year, and the ability to power not only the Ashbourne residential development but the wider community.

1.2. Purpose of this Report

The purpose of this report is to provide the developer an initial assessment of the earthworks, transportation, and three waters infrastructure to service these solar farm developments and new rural residential lots within the proposed northern solar farm site. This report is to be read in conjunction with the associated engineering drawings and calculations.

1.3. Northern Solar Farm Site Description

The Ashbourne Development area is a circa 13.5ha block of land within the Matamata Piako District. The subject site is adjacent to Station Road in Matamata, and the legal description of the site is Lot 2 DP 567678. Most of the site is low-lying flat farmland, that is interspersed with artificial farm drains.

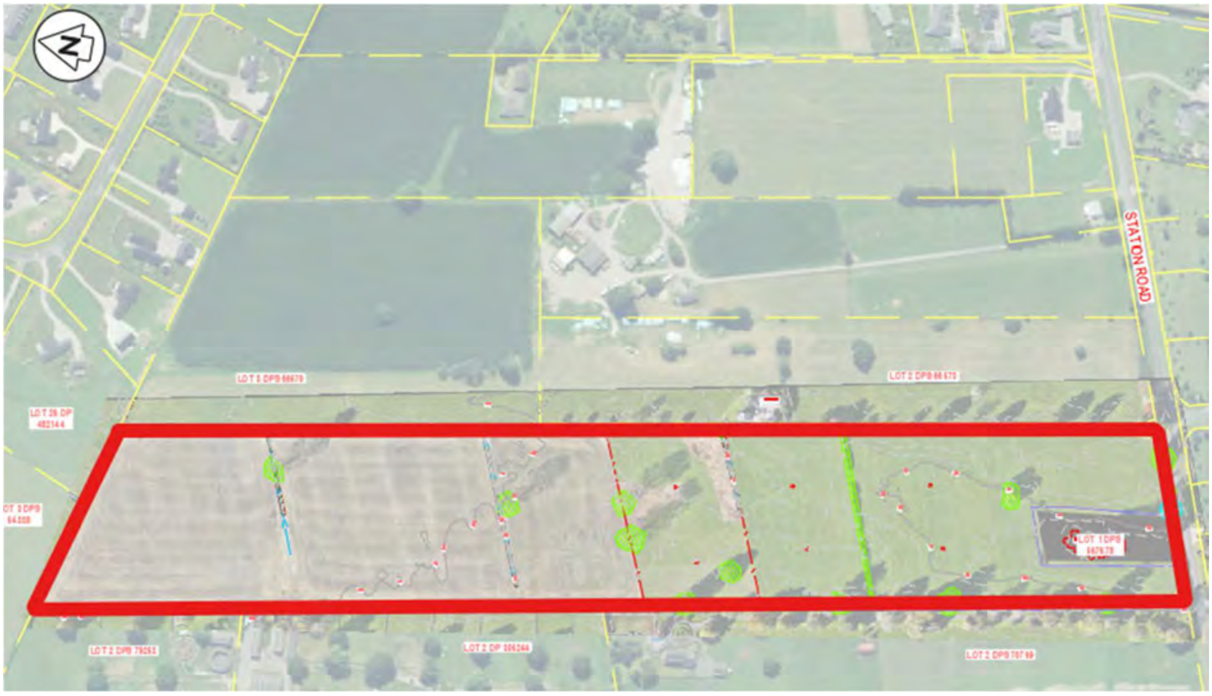

Figure 1: Northern Solar Farm Locality Plan

1.4. Southern Solar Farm Site Description

The Ashbourne Development area is a circa 27.2ha block of land within the Matamata Piako District. The subject site is south of Station Road in Matamata and adjacent to the Waitoa River. Most of the site is low-lying flat farmland, that is interspersed with artificial farm drains.

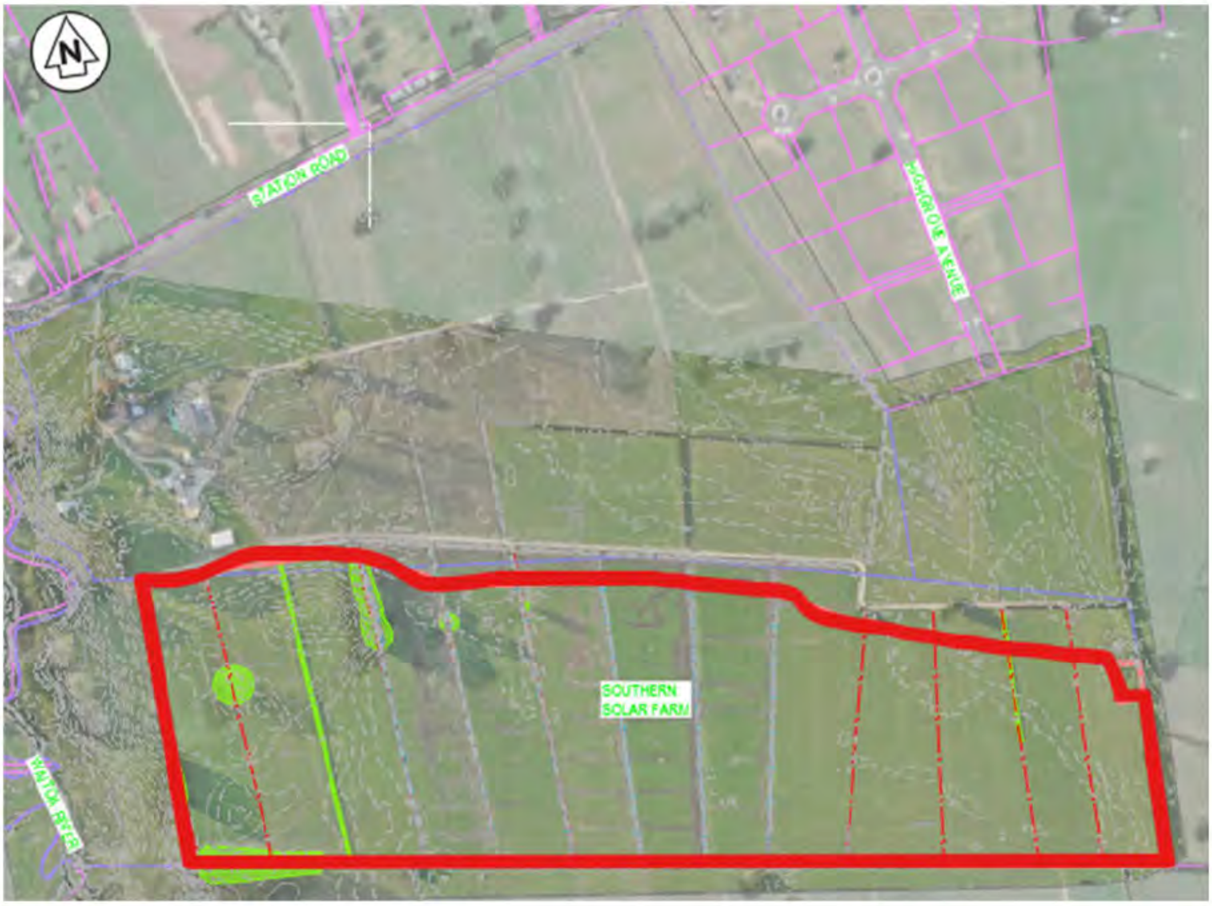

Figure 2: Southern Solar Farm Locality Plan

1.5. Southern Solar Site Legal Descriptions

The development site is currently split across three major parcels of land, and the legal descriptions for the parcels of land are listed in table 1 below:

Lot Description
Lot 3 DPS 14362
Lot 2 DP 21055
Lot 1 DP 21055

Table 1: Southern Solar Farm Existing Lot Description

2. Proposed Development

2.1. Northern Solar Farm Proposed Development

The northern solar farm site will have three solar string arrangement types: 26 solar panel strings, 44 solar panel strings and 72 solar panel strings. Each individual solar panel string will have a string inverter, that will convert the DC power from the solar panels to AC power. The number of solar panels and solar panel strings are summarised in the table 2 below.

Northern Solar Farm Strings		
Solar Panel String Type	Number of Strings	Total Solar Panels Per Type
26	15	390
44	31	1364
72	179	12888
Total Solar Panels		14642

Table 2: Northern Solar Farm Strings

Two power transformers will be installed onsite. These power transformers will manage the power within the solar farm, and they will control the power distribution from the solar farm site to the Powerco Browne Street power substation. Powerco will arrange the design and installation of the 11KV HV cable and ducts, within the existing public roads and through to the existing power substation.

A 10m metaled setback area will be provided around each power transformer. The metal area will provide vehicle access around the units, and it will also act as a firebreak in the event of one of the power transformers catches fire. Refer to Appendix A for the Maven Civil design drawings, Appendix C for the Lightyears Solar drawings and Appendix D for the Solar data sheets.

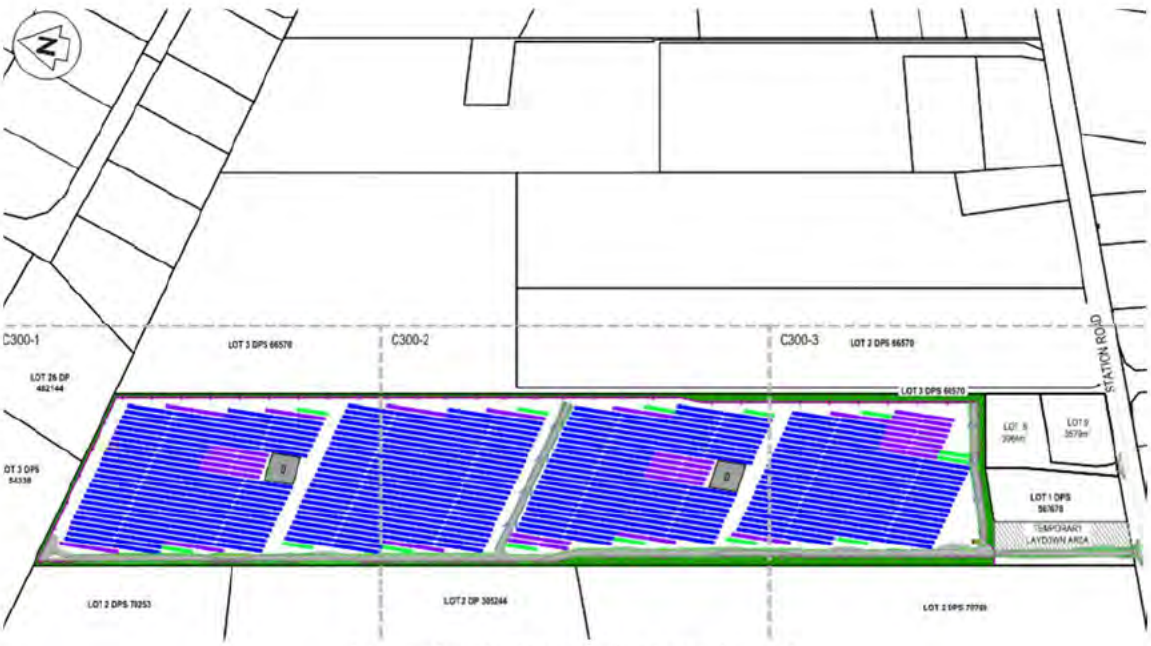

Figure 3 – Northern Solar Farm Layout

A container site office will be provided within the site entrance. The site office will be for maintenance staff coming to the site for routine maintenance visits or inspections. A landscaping buffer strip will be provided around the perimeter of both sites and there will be a 2.2m high security fence inside of the landscaping buffer strips and there will be a gated entry point.

Where the solar farm site will be closely onlooked by existing houses the proposed landscaping buffer strip will be 7m wide, instead of the 3m wide standard width. Refer to the Greenwood Associates landscaping design package for further details relating to the proposed solar farm landscaping.

The block of land on the eastern side of the solar farm access road will be used for storing materials and to provide some temporary parking during construction. A Matamata-Piako District Council (MPDC) intermediate vehicle entrance will be formed for access to the site off Station Road.

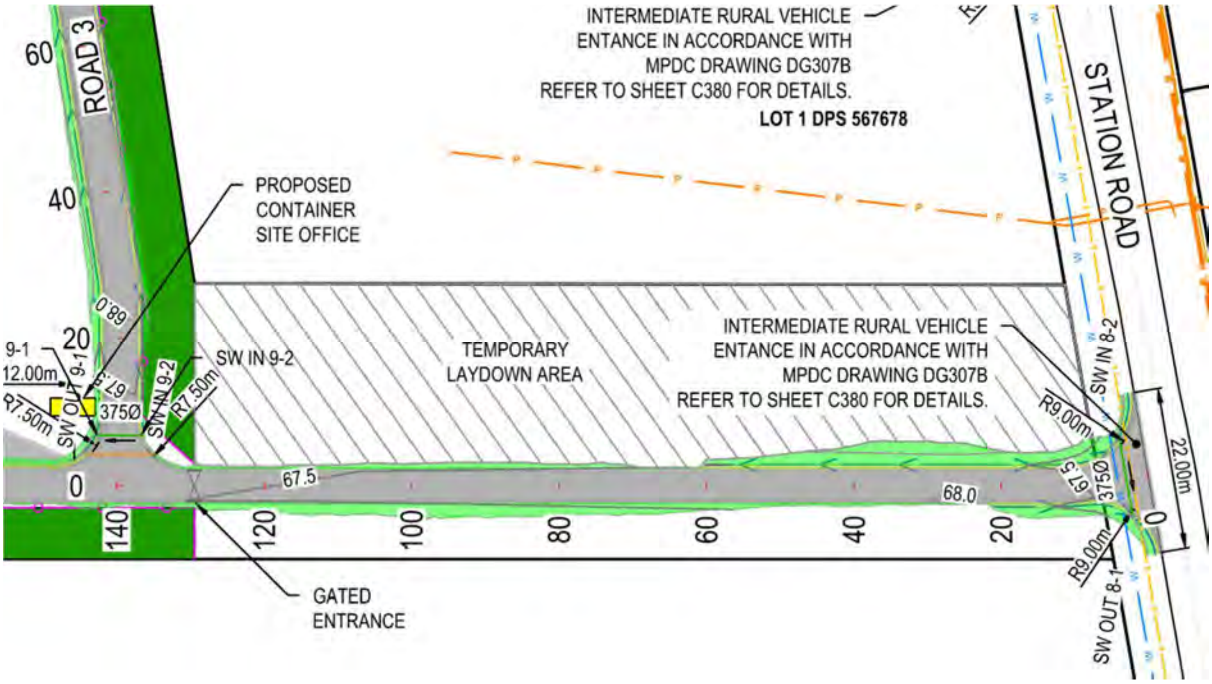

Figure 4 – Northern Solar Farm Temporary Laydown Area

The site will be subdivided to create two new rural residential lots in the southeastern corner of the site. A new intermediate rural vehicle crossing will be constructed to provide a single access point off Station Road for the two new rural residential lots. An easement will be created over the proposed access leg for Lot 8, for the purpose of right of access for the proposed Lot 9.

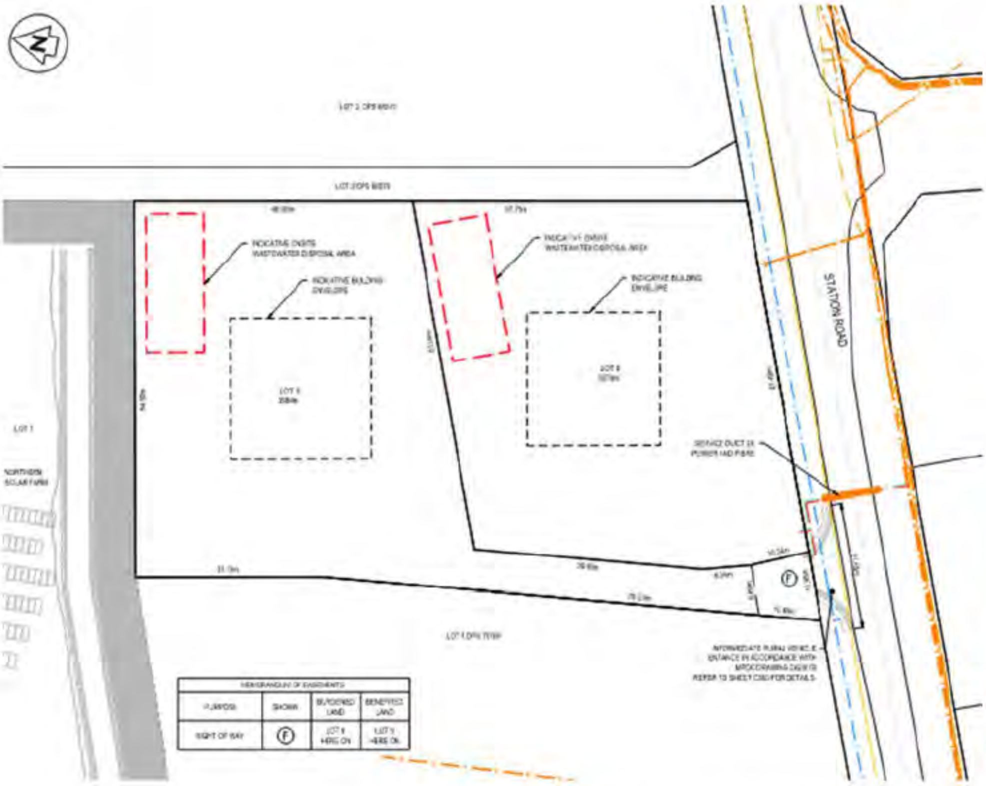

Figure 5 – Proposed Rural residential lots

2.2. Southern Solar Farm Proposed Development

The southern solar farm site will have two solar string arrangement types: 44 solar panel strings and 66 solar panel strings. Each individual solar panel string will have a string inverter, that will convert the DC power generated from the solar panels over to AC power. The number of solar panels and solar panel strings are summarized in the table 3 below.

Southern Solar Farm Strings		
Solar Panel String Type	Number of Strings	Total Solar Panels Per Type
44	365	16060
66	271	17886
Total Solar Panels		33946

Table 3: Southern Solar Farm Strings

Three power transformers will be installed onsite. The power transformers will manage the power within the solar farm area, and they will control the power distribution from the solar farm site to the Powerco Tower Road power substation. Powerco will arrange the design and installation of the 33KV HV cables and ducts, within the existing public roads and through to the existing power substation.

A 10m metaled setback area will be provided around each power transformer. The metal area will provide vehicle access around the units, and it will also act as a firebreak in the event of one of the power transformers catches fire. Refer to Appendix A for the Maven Civil design drawings, Appendix C for the Lightyears Solar drawings, and Appendix D for the Solar data sheets.

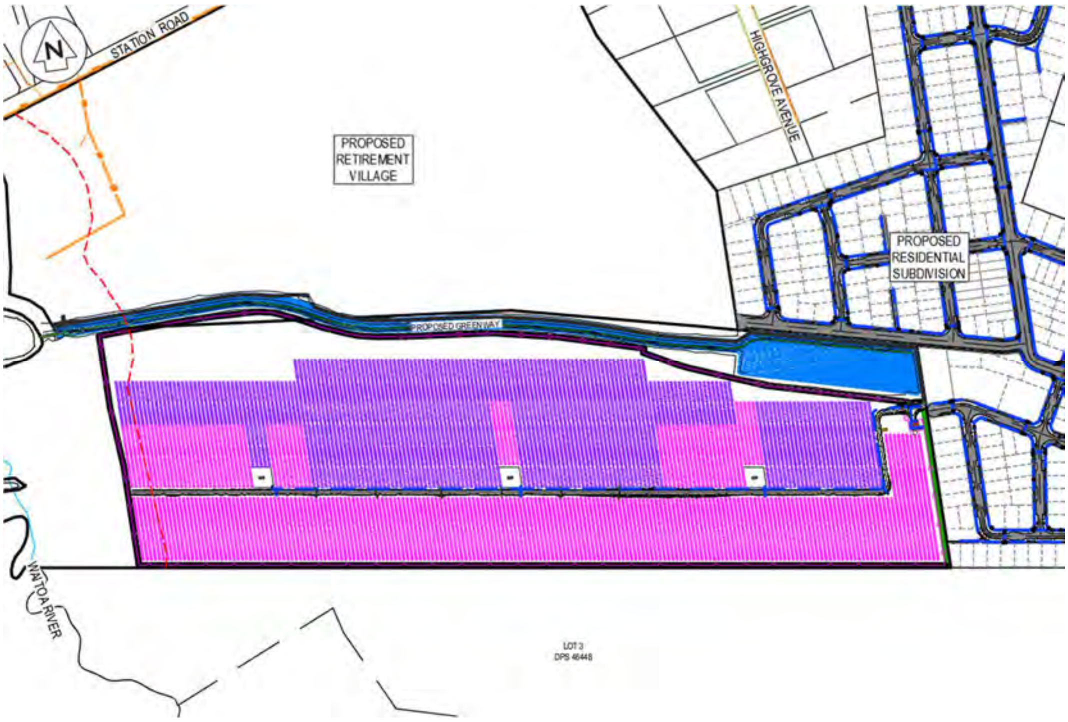

Figure 6 – Southern Solar Farm Layout

A container site office will be provided near the site entrance within the site. The site office will be for maintenance staff coming to the site, for their routine maintenance site visits. A landscaping buffer strip will be provided around the perimeter of both sites and there will be a 2.2m high security fence inside of the landscaping buffer strips and there will be a gated entry point.

Where the solar farm site is near residential housing the proposed landscaping buffer strip will be 7m wide along the perimeter and the remainder will have a 3m wide landscaping buffer strip around the site perimeter. Refer to the Greenwood Associates landscaping design package for further details relating to the proposed solar farm landscaping.

3. Earthworks

A geotechnical desktop review for the Ashbourne Development area was undertaken by CMW in May 2025. Refer to Appendix D for the CMW report.

Earthworks will be required to form the access ways through the solar farm sites and for constructing the drainage onsite. The existing farm drains will be retained onsite, and culverts will be installed to connect the existing drains where the proposed roads cross the existing drains.

3.1. Site Geology

The report identifies the approximate distribution of prevailing landforms and geologies for the local area. The published geological maps for the area are generally aligned with the geology encountered onsite as comprised of interbedded sand, silt, and gravel from the Hinuera Formation.

From the ground investigations undertaken by CMW, they have summarized the site geology results in the CMW Table 1 below.

Unit	Depth to base (m)		Thickness (m)**	
	Min	Max	Min	Max
Topsoil/Fill	0.1	0.5	0.1	0.5
Firm to Stiff Silt/Sandy Silt	0.8	1.2	0.5	0.9
Stiff to Very Stiff Silt (Hinuera Formation)	1.0	1.2	0.5	1.0
Loose to Medium Dense Sand/Silty Sand (Hinuera Formation)	1.4	2.5	0.6	1.7
Dense to Very Dense Sand with interbedded Silt (Hinuera Formation)	5.9	17.3	4.9	16.3
Very Stiff to Hard Silt/Clay (Walton Subgroup)	0.1	18.1	9*	18*
Very Dense Silty Sand (Walton Subgroup)	-	-	**	**
Notes: * Strata not encountered within all test locations. **Thickness only recorded were base of strata has been confirmed.				

3.2. Groundwater Table

Groundwater investigations were undertaken by CMW in May/June 2024 and groundwater was encountered within the CPT and boreholes. The groundwater depths varied across the sites with the northern solar farm site ranging from 1.8m-4.2m. For the southern solar farm site, the groundwater varied from 1.3m-3.3m.

3.3. Sediment and Erosion Control

Sediment and erosion control measures are to be established in accordance with Waikato Regional Council's (WRC) Erosion and sediment control guidelines for soil disturbing activities. Erosion and sediment controls should be in place before earthworks commence and checked onsite by the engineer. Sediment and erosion control drawings will be provided prior to construction.

3.4. Northern Solar Farm Earthworks

A preliminary earthworks assessment for the northern solar farm site was undertaken for this proposed development. The design terrain was developed based on the proposed solar farm roads and the new rural residential vehicle crossing. The average depth of topsoil observed onsite by CMW was 300mm in depth.

The earthworks volumes have not factored in any final shaping of the proposed rural residential development area. We anticipate minor earthworks will be required within the proposed rural residential lots and this will be managed by the new lot owners.

Earthworks Volumes	
Total Cut =	92m^3
Total Fill =	2,244m^3
Balance (Fill) =	2,152m^3

Table 4: Northern Solar Farm Earthworks Volumes

Topsoil Stripping	
Topsoil Stripping Area =	8,853m^2
Topsoil Stripping =	2,655.9m^3

Table 5: Northern Solar Farm Topsoil stripping

Volumes indicated are solid measure in place, no bulking or compaction factors have been applied. The anticipated earthworks bulking factor for this site 1.1. The fill material will be imported fill.

Based on the results of geotechnical investigations undertaken onsite, the Ashbourne northern solar farm development should be suitable for the intended development.

3.5. Southern Solar Farm Earthworks

A preliminary earthworks assessment for the southern solar farm site was undertaken for this proposed development. The design terrain was developed based on the proposed solar farm roads. The average depth of topsoil observed onsite by CMW was 300mm in depth.

Earthworks Volumes	
Total Cut =	$48m^3$
Total Fill =	$2731m^3$
Balance (Fill) =	$2683m^3$

Table 6: Northern Solar Farm Earthworks Volumes

Topsoil Stripping	
Topsoil Stripping Area =	$8196m^2$
Topsoil Stripping =	$2458.8m^3$

Table 7: Northern Solar Farm Topsoil stripping

Volumes indicated are solid measure in place, no bulking or compaction factors have been applied. The anticipated earthworks bulking factor for this site 1.1. The fill material will be imported fill.

Based on the results of geotechnical investigations undertaken onsite, the Ashbourne southern solar farm development should be suitable for the intended development.

4. Roading

The solar farm roads will be provided with 4m wide metaled access roads through both sites. To manage stormwater for these roads, they will have road table drains and stormwater culverts where required. The roads have been designed to cater for emergency fire trucks and 8m rigid truck turning movements.

The pavement design is based on a minimum CBR of 5, with 200mm GAP65 subbase layer and 100mm GAP40 basecourse layer. Further subgrade testing will be required to confirm the actual site CBRs for the proposed roads. The proposed access road is shown below. Refer to the C300 series drawings in Appendix A for further details. The primary access roads have been provided with passing bays, to cater for aerial appliance fire trucks, which are a 12.6m long rigid truck.

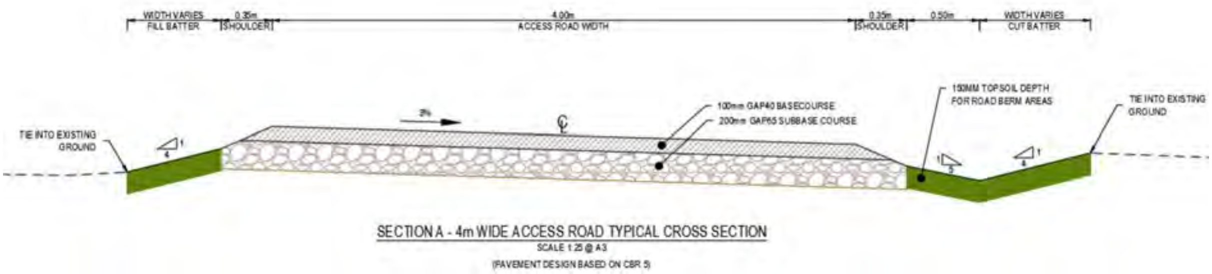

Figure 7 – 4m Wide Access Road Typical Section

4.1. Northern Solar Farm Roads

The northern Solar farm will include constructing 1.26km of metaled access roads. There will be one primary access road that will closely follow the western boundary and two connected side roads. A standard MPDC rural intermediate vehicle entrance will be provided off Station Road for the main access road.

A temporary laydown area will be established adjacent to Road 1 between the Station Road end and before the proposed gated entrance. The temporary laydown area will be used for temporary storage of materials and to provide some temporary parking during construction.

4.2. Southern Solar Farm Roads

The southern solar farm site will include constructing a 1.1km central metaled access road, commercial vehicle entrance and turning bay for the proposed central wastewater pump station. The turning bay has been designed for 8m rigid sucker truck turning movements for the wastewater pump station.

5. Stormwater

The Matamata-Piako District Council (MPDC) holds a discharge consent issued by the Waikato Regional Council (WRC). This consent outlines how stormwater runoff from the urbanised area of the Matamata town centre should be managed. The consent provides guidance on managing stormwater and flooding to support future urban development within the catchment area. Refer to Appendix A for the stormwater concept drawing.

5.1. Existing Stormwater

The existing stormwater infrastructure within the site is limited to farm/roadside drains and streams. There are farm drain networks through these sites, that all connect into a downstream primary farm drain.

5.2. Proposed Stormwater

The two solar farm sites are flat sites. These sites will utilise the existing farm drains and the roadside drains for conveyance and ground soakage to manage stormwater through these sites. Road culverts will be provided to convey the open channel flows through the road crossings.

For the post development scenario, the site will be managed in a similar way. New metalled access roads will be constructed, and the surface water from these roads will be managed by the road swales and to provide the required level of water quality treatment onsite.

Once the site is operational a local farmer will bring in livestock to graze the paddocks to manage the grass and weeds onsite periodically. Maintenance staff will come to site in either a work ute or van every 2-3 months to clean the solar panel units and or to do general maintenance checks, so traffic generation and potential vehicle contaminants will be low for these sites.

The solar panels themselves would not generate any additional contaminants, so rainwater spilling off these solar panels would be considered as clean water runoff. The stormwater would then soak into the ground like it already does, so there would be negligible changes to the stormwater runoff for the post development scenario.

The proposed rural residential will need to manage their stormwater onsite through either onsite soakage or roof collection tanks. Preliminary soakage testing was undertaken near these sites, and the results demonstrated that there was good soakage within area, however further investigations will be required for each site to confirm the actual soakage rates.

The stormwater infrastructure will need to comply with the conditions for resource consent and engineering approval. Where possible, the stormwater network will be designed and constructed within the roads.

5.3. Flood Modelling

We have undertaken flood modelling assessment using HEC-HMS and HEC-RAS for the proposed Ashbourne development for the 10-YR ARI and 100-YR ARI storm events for pre and post development. The Ashbourne development is split into four key precincts, which include the Southern Solar farm, residential area, retirement village and the northern solar farm. For the 100-YR ARI storm events the flood modelling has assumed that the existing and proposed culverts would be fully blocked. The base level of the power transformers will be constructed, with a minimum 150mm freeboard limit above the 100-YR ARI flood level.

5.3.1. Northern Solar Farm

The flooding results were generated through using the HEC-RAS 2D software. The 100-YR ARI RCP 8.5 post development flooding depth results, for the northern solar farm site are shown below. From comparing the pre-development vs post development results it showed there was a negligible increase in depth across the site.

With introducing the new main access road near the western boundary, the ponding depth has been slightly raised on the western side of the proposed road in some areas, due to partially raising the road levels above the existing ground level. For the detailed design stage, the roading design levels will be further refined to align with the proposed flood modelling results.

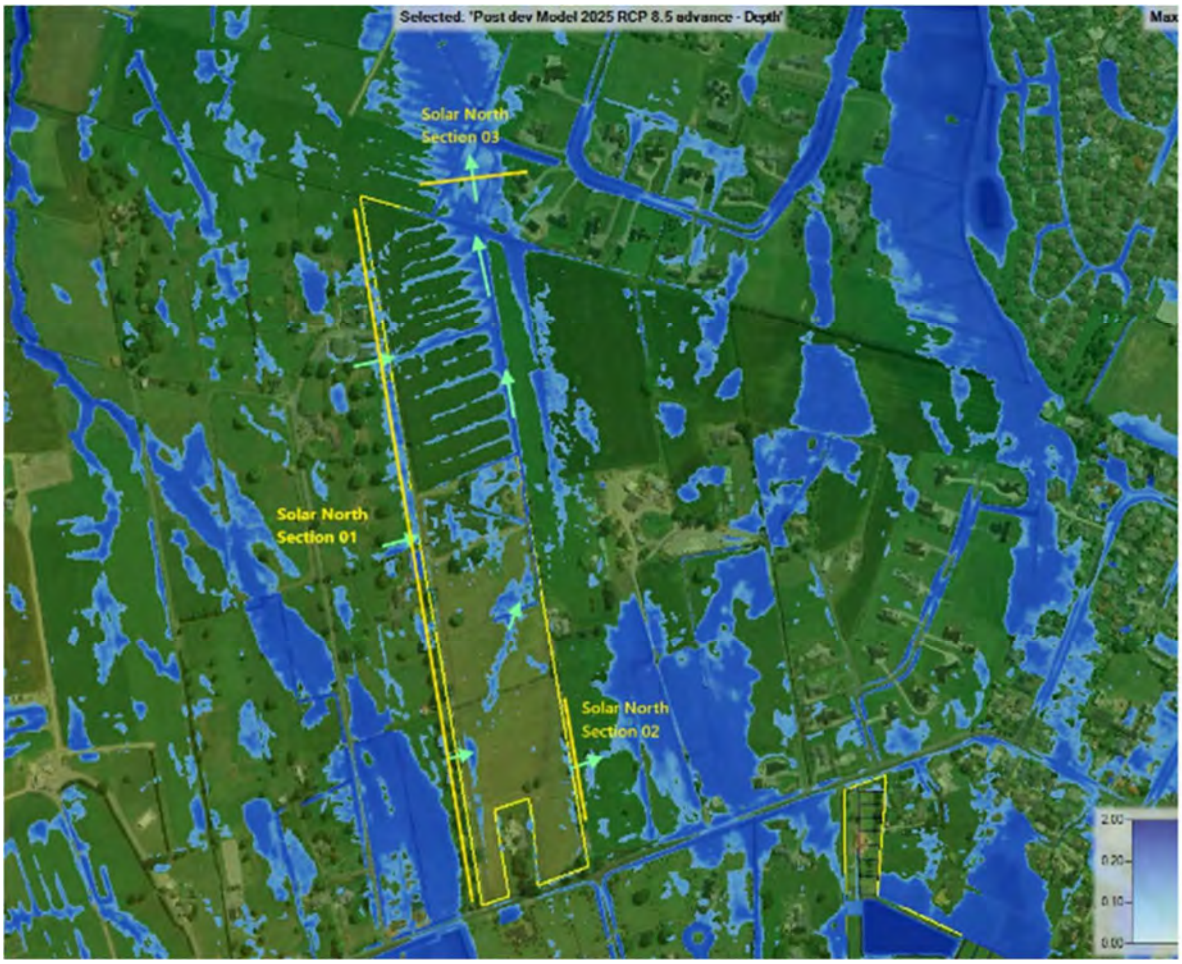

Figure 8: HEC-RAS flooding depth results for post development

5.3.2. Southern Solar Farm

The flooding results were generated through using the HEC-RAS 2D software. The 100-YR ARI RCP 8.5 post development flooding depth results for the southern solar farm site are shown below. Comparing the pre-development vs post development results some key differences were noted there is higher levels of ponding on the southern side of the proposed central access road, and they were lower on the northern side of the road. For the flood modelling we have assumed 100% blockage for the culverts in the 100YR ARI storm events, so the flood model is purely based on the overland flows. This is a conservative approach as the worst-case scenario, it is unlikely that all culverts would be fully blocked in a 100YR ARI storm event, if there is regular maintenance undertaken for the culverts.

Upstream flows enter the site from the south and then generally head in northern direction through the solar farm site. The overland flows reach the central road and then it ponds up behind the road, until the flows reach the height of the road spill points in the road sag points. The flows will then continue over the road and then they will eventually reach the greenway. With the raised ponding behind the new road, the results show the ponding is still being contained onsite, and no increased ponding in the neighboring properties.

For the detailed design stage, the roading design levels will be further refined to align with the proposed flood modelling results.

Figure 9: HEC-RAS flooding depth results for post development

6. Wastewater

Maven have completed a desktop study for the wastewater for this development. There is no existing private or public wastewater within either of the solar farm sites or near these sites. Wastewater services will not be required for the solar farm sites. A new central wastewater pump station will be constructed near the entrance of the southern solar farm site. The wastewater pump station will be vested with council, to service the proposed Ashbourne residential development.

There is no existing public wastewater reticulation network to connect into for the proposed rural residential lots and therefore wastewater will need to be managed through onsite wastewater treatment devices.

7. Water

Maven have completed a desktop study for the water supply for this development. MPDC GIS shows an existing 25mm public rider main running past the northern solar farm site within Station Road. There is no public water supply network shown through the proposed southern solar farm site.

The presence of water troughs through the proposed solar farm sites, suggests there could be existing private water supply lines running through these sites, that would fill these existing water troughs. The location of the existing water lines will need to be confirmed onsite and then removed and new private water network installed in accordance with the C700 service design drawings in Appendix A.

The solar farm sites will require firefighting supply for each site. Each site will have dual 25,000L water firefighting tanks inside the site near the entrance and they will have an attached 100mm diameter fire supply connection coupling, that will be compliant with the SNZ PAS 4509:2008 requirements.

The proposed rural residential lots will be provided with individual private 25mm water supply lot connections and they will connect to the existing rider main on Station Road.

8. Services

8.1. Power

Powerco are the power service providers for this area. Individual power connections will be provided for these rural residential lots and for the northern solar farm, by connecting into the existing power network located within Station Road. A new power connection will be extended into the southern solar farm site, from the proposed Ashbourne residential development. Low voltage power connections will be required for these solar farm sites.

8.2. Communications

Tuatahi First Fibre are the fibre service providers within this area. For the northern solar farm site and the rural residential lots will require individual fibre connections and they will connect into the existing network within Station Road. The southern solar farm site will require a new fibre connection from the proposed Ashbourne residential development.

8.3. Service Provisions

We have now received service provision confirmation letters from Powerco and Tuatahi First Fibre confirming that they can service the two solar farm sites and the two new rural residential lots. Powerco have confirmed some network upgrades will be required to support these developments. Refer to Appendix F for the service provision letters for the two solar farm sites.

9. Conclusions

Stormwater drainage can be provided for this development through stormwater culverts, existing farm/roadside drains and streams and onsite soakage devices.

No wastewater services are required for the solar farm sites and onsite wastewater treatment devices will be required for the proposed rural residential lots.

Potable water supply can be provided for the development by connecting into the existing and proposed water supply networks within the area.

This report will be further developed during the detailed design stage.

10. Limitations

The calculations and assessments included in this report are based on information available at time of issue. To the best of our knowledge, it represents a reasonable interpretation of available information.

This report is solely for our clients use for the purpose for which it is intended in accordance with the agreed scope of work. It may not be disclosed to any person other than the client and any use or reliance by any person contrary to the above, to which Maven has not given its prior written consent, is prohibited.

This report must be read in its entirety and no portion of it should be relied on without regard to the limitations and disclaimers set out.

Maven makes no assurances with respect to the accuracy of assumptions and exclusions listed within this report and some may vary significantly due to ongoing stakeholder engagement.

Appendix A – Civil Drawings

Appendix B – Stormwater Management Plan

STORMWATER MANAGEMENT PLAN

ASHBOURNE DEVELOPMENTS

PROJECT INFORMATION

CLIENT:	Matamata Development Ltd
PROJECT:	289001

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TABLE OF CONTENTS

1 OVERVIEW

This stormwater management plan ('SMP') outlines the proposed management of stormwater for the Ashbourne developments, located west of the Matamata town centre.

The overall development is illustrated in Figure 1 below. The project spans approximately 100 hectares of land, encompassing areas both north and south of Station Road, as shown below. The southern areas extend westward toward the Waitoa River.

The SMP outlines the overarching stormwater management principles that will form the basis of stormwater design to support future development of the proposed sites.

Discharge consent is being sought to develop the system to enable future stormwater discharge from the proposed sites. This SMP is prepared to support this discharge consent application.

The project is split into 4 sites.

1. Residential Development
2. Northern Solar Farm Development
3. Southern Solar Farm Development
4. Retirement Village

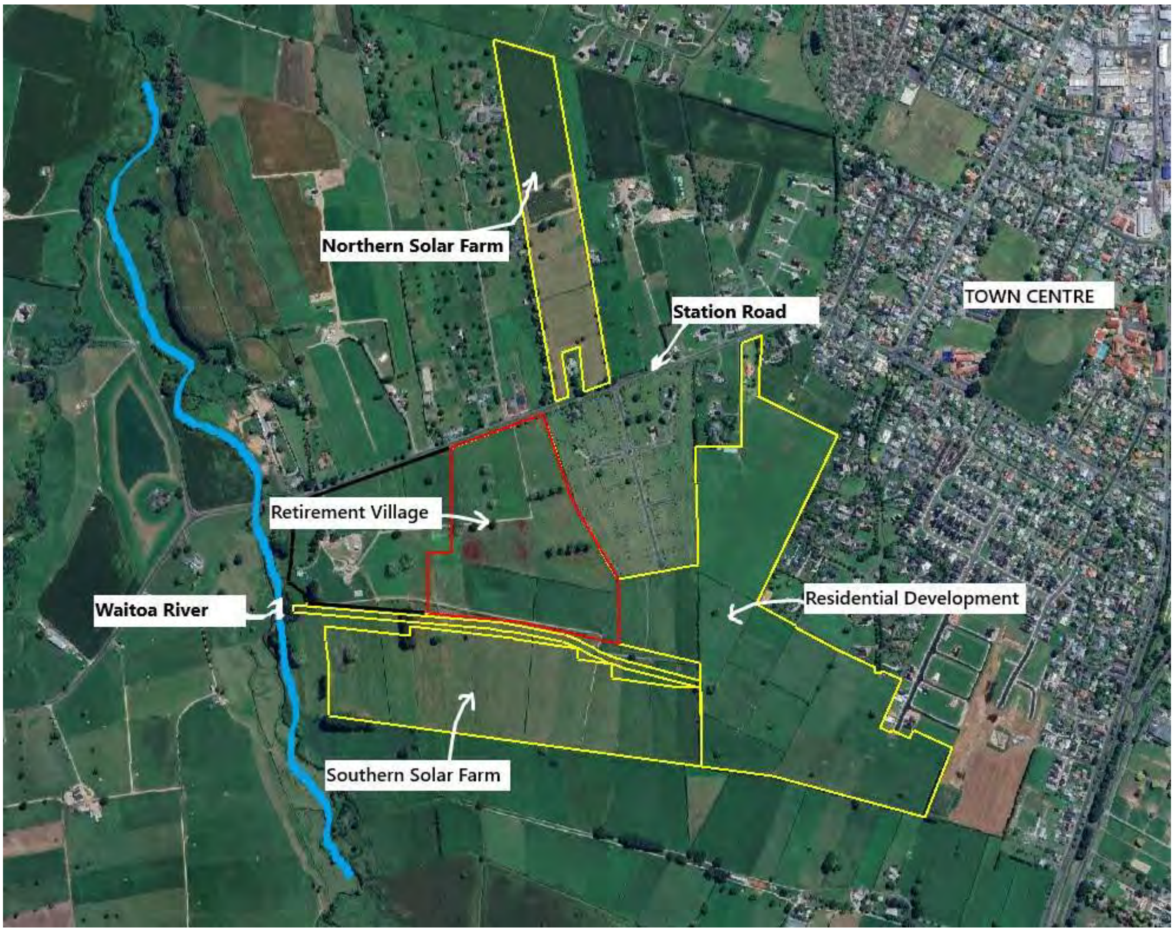


Figure 1: Site locality plan.

SITE	AREA (Ha)
Northern Solar Farm	13.00
Southern solar Farm	24.82
Retirement village	20.00
Residential	45.00

Table 1: Site Areas

There are total of 8 number of legal parcels distributed within the 4 project sites. List of these is provided is table 2 below.

PARCEL ID	AREA (ha)	OWNERS
Northern Solar Farm		
Lot 2 DP 567678	13.00	R.A Hemmings Limited
Southern Solar Farm and Retirement village		
Lot 1 DP 21055	33.23	R.A Hemmings Limited
Lot 2 DP 21055	27.38	R.A Hemmings Limited
Residential		
Lot 5 DP 365568	3.29	P & M Equipment Hire Limited
Lot 1 DPS 65481	4.20	CAT Limited, RM Craig, WJ Perry
Lot 5 DP 384886	8.10	Eldonwood Limited
Lot 3 DP 14362	13.71	R.A Hemmings Limited
Lot 204 DP 535395	24.14	Eldonwood Limited

Table 2: Legal Parcels, associated area, and Owners

The areas are currently zoned within rural, rural residential and future designated residential areas. The development has been master planned by the client in collaboration with Matamata-Piako District Council ('MPDC'), Waikato Regional Council ('WRC') and a design consultant specialists. As a result, this SMP is built upon previous discussions around stormwater management for the overall site with WRC and MPDC.

A discharge consent is required to enable the future stormwater discharge from these developments, which necessitate the importance of this SMP. The new discharge consent will be transferred to Council and ultimately form part of the Comprehensive Discharge Consent upon the vesting of the public network.

Stormwater is proposed to be discharged via the following methods:

- Soakage within the site using stormwater devices such as raingardens, soakage trenches and soakage basins.
- A proposed Greenway running east to west which discharges into Waitoa River at 80% of pre-development flows. The proposed greenway will also aid in diverting existing flows heading north from lands south of the project.
- Existing drains or overland flow paths leaving the project site at less than 80% of pre-development flows.

The overall Development area is divided into four distinct developments as listed and summarised below.

1.1 ASHBOURNE NORTHERN SOLAR FARM

This development spans approximately 13 Ha and will serve as the first solar farm constructed within the Ashbourne development. The site will feature 14,642 solar panels, generating required power to meet regional renewable energy requirements. The farm incorporates permeable ground coverage and minimal impervious surfaces to ensure effective stormwater infiltration and flow dispersion. A network of grass swales and drains will manage runoff from solar panels and ancillary infrastructure, maintaining water quality prior to discharging into the existing channel/farm drain along the northern boundary.

1.2 ASHBOURNE SOUTHERN SOLAR FARM

The Ashbourne southern solar farm spans approximately 24 ha and is the second solar farm planned for the Ashbourne development. The site will include 33,946 solar panels, generating required power to meet regional renewable energy requirements. The farm incorporates permeable ground coverage and minimal impervious surfaces to ensure effective stormwater infiltration and flow dispersion. A network of grass swales and drains will manage runoff from solar panels and ancillary infrastructure, maintaining water quality prior to discharging into the proposed greenway before discharging into Waitoa River.

1.3 RETIREMENT VILLAGE

The Retirement Village spans approximately 16 ha and is designed to accommodate 218 villas, one Aged Care Hospital, and other required facilities tailored to meet the needs of an aging population. The development aims to provide a high quality, connected environment for retirees, emphasizing on community living and accessibility.

Similar to the residential project, this development will require a stormwater basin to cater for overflows from upstream stormwater devices. Lot Areas and Road carriageways will be treated and discharged at source via the use of rain garden and soakage systems

1.4 RESIDENTIAL DEVELOPMENT

Spanning approximately 45 hectares, the residential development includes 518 Lots, designed with mix of housing typologies and densities to meet client and local authorities' requirements. Key feature of this development is the formation of the proposed greenway which serves as a stormwater management system for Catchment B of the residential areas. The Greenway conveys and attenuate overflows from the road and lot areas releasing at 80% of pre-development flows into the Waitoa River. Lot Areas and Road carriageways will be treated and discharged at source via the use of soakage systems.

1.5 STAGING, TIMING, RESPONSIBILITY AND FUNDING

The proposed development includes four distinct projects: the Residential Area, the Retirement Village, the northern solar farm, and the southern solar farm. Construction of these projects is not anticipated to occur simultaneously.

Key infrastructure elements, such as the greenway, provide shared downstream conveyance capacity; however, not all projects will depend on its availability. The Retirement Village will be proposing to install its own conveyance channel to discharge flows into the Waitoa River, and only Catchment B of the Residential area is set to discharge into the Greenway in addition to the southern catchment.

The southern solar farms will utilise existing natural channels or, where necessary, create new discharge pathways to the Waitoa River prior to the establishment of the Greenway, while the northern solar farm will operate independently of the greenway system.

The Residential and Retirement Village developments will be carried out in staged phases, with each stage designed to comply with the principles and requirements outlined in this Stormwater Management Plan (SMP). Interim measures, such as swales and temporary conveyance routes, will be implemented to ensure ongoing mitigation throughout the construction period.

1.6 COSTS, FUNDING AND VESTING ASSETS

The construction of the proposed stormwater management devices will be undertaken by the consent holder. The proposed stormwater infrastructure includes a greenway swale, public piped networks, soakage trenches, detention basins, raingardens, and private propriety devices.

The public assets will be vested to council at the appropriate time as the development progresses. Private assets will remain in private ownership where appropriate legal instruments will be set up to ensure ongoing operation and maintenance responsibilities are with the owners. Discussions will be undertaken with council(s) as to the design of the infrastructure, location, and purpose, with all public infrastructure subject to the relevant Engineering Approval process.

1.7 OPERATION, MAINTENANCE AND MONITORING PLAN

Operation and maintenance plans will be provided for all stormwater management devices that will be vested to Council(s), which will be required as a condition of any approved Resource Consent.

2 HYDRAULIC CONNECTIVITY

2.1 WATER SENSITIVE DESIGN

Stormwater management for the developments will include several devices which are considered as water sensitive design elements. These include the following:

- Dry basins and a greenway with landscaping.
- Soakage Devices
- On-lot stormwater Retention through Soakage
- Protection of existing bush and covenant features.
- Planting of riparian areas if applicable.

The Water Quality Volume (WQV) is treated at source for both Road and Lot areas. Overflows above the 10-year ARI cc event are directed to Dry basins for both RV and Residential Developments. With both solar farms, existing runoff route is to be maintained with minimal impervious runoff being added. The proposed stormwater solution ensures stormwater is being treated before discharging into the receiving environment.

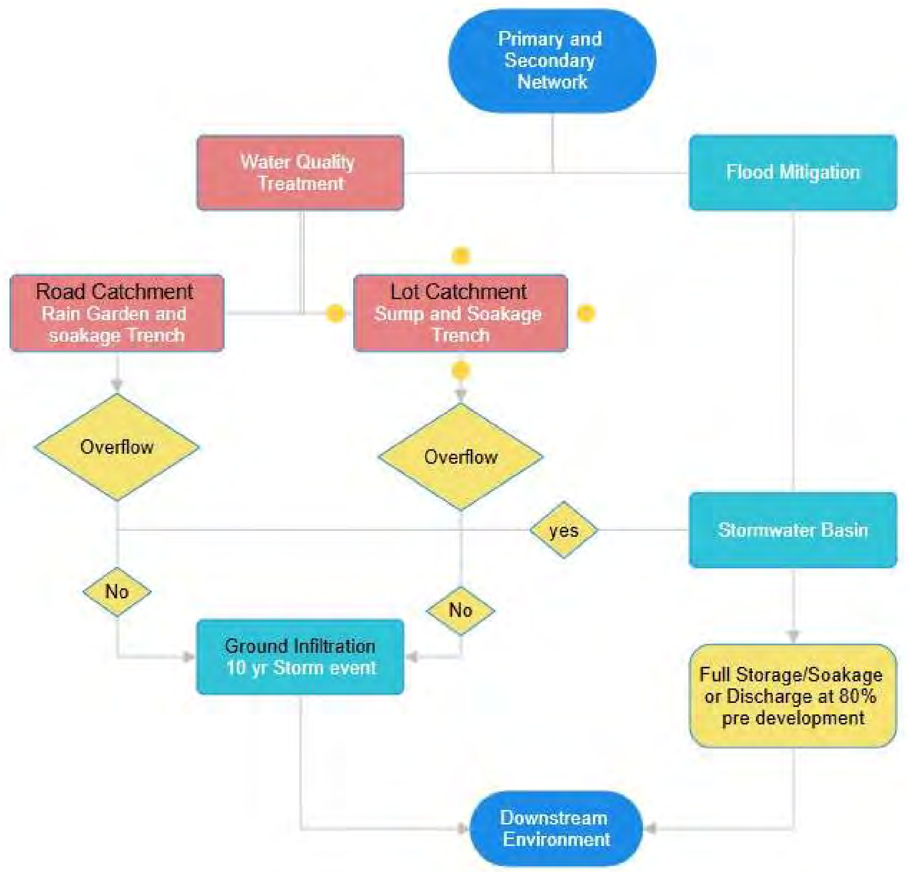


Figure 2: Hydraulics Connectivity Chart

3 EXISTING CATCHMENT CONTEXT

The Ashbourne development covers approximately 100 Ha, including areas, north and south of Station Road. The site features a flat to gently sloping topography shaped by its historical use as pastoral farmland. Gound elevations across the site range from RL 67m around the north area with RL 72m towards west and south, with the site generally draining in a northwest direction towards Waitoa River. The site is bounded to the west by Waitoa River, to the east by Existing Residential developments, and to the south by additional pastoral land, with Station Road dividing the northern and southern areas.

The existing land is predominantly grass covered, used for grazing livestock, characterised by grass covered paddocks, sporadic mature trees, and agricultural infrastructure such as farm buildings and swales. A network of farm drains across the site facilitates surface water conveyance, diverting water in line with the natural drainage pattern. Historical aerial photographs confirm the landform and drainage network have remained largely unchanged for the course of 50 years.

3.1 EXISTING CALCULATION PARAMETERS

The design and calculation assumptions used for pre-development are outlined in the table below. These values were established using the available GIS/aerial view, topography information, geotechnical report, and NIWA.

Pre-Development CN	
Pervious	**Impervious**
61	98
Pre-Development 24hr Rainfall Depth (mm)	
10yr cc	**100 yr cc**
128	200

Table 3: Design Parameters

The RCP8.5 climate change adjustment has been applied to both scenarios (pre- and post-development) to enable comparison between the two events when assessing compliance. Refer to Table 4 below for catchment areas. Approximately 2-5% of existing impervious coverage is likely present within the existing catchment; however, it is assumed in our calculation that there is none. This assumption is conservative.

3.2 EXISTING OVERALL CATCHMENT

A catchment analysis was undertaken to establish the extent of the Existing catchment within the proposed development areas. TR20/07 and HEC HMS were used for this analysis, with results shown below and further detailed in plans 410 series.

Figure 3 Existing catchments & Flow direction (Refer to Plans 410 Series)

The peak flow rates for both the 10- and 100-year events were calculated using HEC HMS, using the catchment analysis findings in addition to calculation parameters provided under section 3.1 and are provided in the table below. A few inflows from upstream areas were established from this catchment analysis. Either diversion away from development or accommodating it/them within catchment design for post development will be required (Proposal regarding these inflows are provided under Post development Section of this SMP).

			Pre-Development Flow	
SITE	**Catchment**	**Area (Ha)**	**10-year flow (m^3/s)**	**100-year flow (m^3/s)**
RESIDENTIAL	A1	2.39	0.18	0.37
	A2	8.78	0.47	0.96
	A3	0.52	0.04	0.09
	A4	3.33	0.21	0.43
	A5	0.47	0.04	0.08
	A6	9.33	0.52	1.06
	A7	28.51	1.64	2.97
	A8	0.54	0.05	0.1
SOUTH SOLAR	A9	121	4.22	8.63
RETIREMENT VILLAGE	A10	10.1	0.55	0.81
	A11	4.12	0.03	0.07
	A12	17	0.73	1.78
NORTH SOLAR	A13	0.26	0.05	0.11
	A14	2.33	0.15	0.28
	A15	17.89	0.83	1.71
	A16	0.66	0.05	0.11

Table 4: Overall Existing Catchment Flow table (refer to plan 410)

The above table shows pre development catchment areas and their respective flowrates, prior to development and serve as a baseline for checking overall flowrate pre vs post for the residential. This is linked to the 410 plans (Appendix A of infrastructure Report) and later referenced in tables 18 and 19.

3.3 WAITOA RIVER

The Waitoa River is one of the primary receiving environments for stormwater discharges from the Ashbourne development. It runs along the western boundary of the southern site and acts as the discharge point for both the residential and southern solar farm stormwater systems. The river plays a critical role in regional drainage and is known to experience periodic flood events, especially under prolonged rainfall conditions.

All stormwater outfalls from the development that discharge into the Waitoa River have been designed to include attenuation and treatment upstream, with flows directed either through dry basins, the greenway, or existing conveyance features. This discharge strategy ensures that post-development flows into the Waitoa River are maintained at or reduced from pre-development conditions, and energy dissipation structures are proposed to minimize erosion risk at outfall locations.

Future assessments of the river’s bank stability, riparian condition, and erosion susceptibility near discharge points will be conducted during the detailed design stage to ensure the integration of ecological and hydraulic objectives. Monitoring may also be proposed as a consent condition to assess any long-term impacts on the river system.

3.4 NORTHERN SOLAR FARM

The northern solar farm site, legally described as Lot 2 DP567678, encompasses an area of approximately 13hectares and is situated on the northern side of station road. The site comprises predominantly open pasture with sparse tree cover with no signs of existing dwellings. the topography shows a gradual slope across the site, generally falling from the southwest towards the northeast corner. The surrounding landscape features slightly elevated land to the west and south, contributing to the general eastward and northerly flow pattern across the site.

3.5 RETIREMENT VILLAGE

The southern Solar farm and the Retirement village comprise two adjoining rural land parcels located south of station road. Retirement Village site covers the northern portion of the two adjoining areas and Southern Solar farm to the South.

The area is predominantly open pastureland with generally flat typology with minimal existing infrastructures. A farm track currently crosses the site. The entrance is located on Station Road and provides access across the farmland, including the existing dwelling, which is situated to the west of the Retirement Village development boundary. Notably, the existing dwelling has its primary vehicle entrance located further west along Station Road.

The Retirement village is located adjacent to station road and occupies northern areas of the two adjoining parcel lands. General typology reveals majority of the site slopes towards east, with portion slopes south and westward towards the Waitoa river. This channel conveys flow to an existing culvert beneath station road, where it continues downstream.

Existing Inflows into the site are mainly from the southern solar farm areas and the residential area. Table 4 above summarises each sub catchment flows with plans 410 series (Appendix A) providing further details.

3.6 SOUTHERN SOLAR FARM

The southern Solar farm and the Retirement village comprise two adjoining rural land parcels located south of station road. Retirement Village site covers the northern portion of the two adjoining areas and Southern Solar farm to the South.

The area is predominantly open pastureland with generally flat typology with minimal existing infrastructures

The solar farm occupies the southern areas of the two parcel lands and receives additional inflows from the upstream catchment located to the south of the site. The general landform from the survey data reveals a general fall north until the flows splits, one drains down into Waitoa and rest discharges further north to the retirement village through the existing drain along the Eastern boundary, traverses over station Road, through further farmlands and finally will get discharge into the Waitoa River.

3.7 Existing Culvert – Station Road

Existing culverts and drains are located within and around the development sites, primarily to convey surface runoff, around the site, beneath station road and through ex accessways or farm tracks. The culverts form part of the existing drainage network conveying stormwater within the site and from upstream catchment. these culverts remain in service; however, their capacity and condition vary across the site.

Figure 4: Existing Culvert with blockage Issue

One critical culvert is located along the eastern boundary corner of the retirement village area. This is where an open drain collects runoff and dischargers under station road through this culvert. This culvert serves as a key outlet for surface water from the Retirement village site and further upstream catchment.

Visual inspection indicates that this culvert is in bad condition, with vegetation overgrowth and sediment accumulation presents at both the inlet and outlet locations. These conditions will restrict flow, capacity, particularly during higher rainfall events, and poses risk of upstream ponding.

3.8 RESIDENTIAL DEVELOPMENT

The Residential development site comprises of multiple existing land parcels, refer to overview section of the report. The land is currently used for rural and pastoral purposes with only one existing dwelling adjacent to station road.

The site lies to the south and west of established residential areas and is bounded by station road to the north, with rural properties including the solar farm site covering the western and southern areas. The site has a gentle and flat areas, sloping towards low lying areas in the central and northern parts of the site. Surface runoff generally conveyed through naturally formed low lying areas including existing farm drains etc.

The residential development existing catchment area has been subdivided into sub catchments, each draining towards the identified discharge points as outlined in the table below and stormwater plans 410 series.

3.9 ECOLOGICAL FEATURES

An Ecological assessment was conducted for the site covering the areas below. this assessment identified the western portion of the southern parcel, bordering the Waitoa River, as an area of heightened ecological sensitivity. This area contains a network of aquatic features, including oxbows, minor ponds, and a second order stream Channel, alongside existing riparian plantings of native species. While most of the site is highly modified and has low ecological value, this floodable corridor was recommended as a "no-development zone" due to its potential for ecological restoration and its contribution to habitat diversity and freshwater functions.

Figure 5: Existing Culvert with blockage Issue

While the proposed development areas do not fall within this "no development zone," the proposed greenway outlet does encroach into this area. As covered in section 6.2, the proposed greenway provides essential flood conveyance and will also enhance the ecological value of the site through planting and other measures; therefore, works within this area will further improve its ecological value.

Further value Engineering and design at detailed design stage will be undertaken to ensure full compliance with the ecological report recommendations and referenced standards.

3.10 GEOTECHNICAL REPORT

Two preliminary geotechnical investigations were undertaken to provide full coverage of the Ashbourne's development area. The investigations cover assessed ground water levels, soil soakage capacity, liquefaction potential, and lateral spreading risks, ensuring comprehensive data for stormwater management, earthworks, Roading and foundation design. The two referenced Geotechnical report in this SMP are listed below (Appendix Of the Infrastructure Report).

1. HAM2023-0124AB Rev 1 (dated 12th December 2023)
2. HAM2023-0124AE Rev 0 (dated 5 July 2024)

The site is characterised by a predominantly, flat to gently sloping topography, underlain by Hinuera formation soils, with ground water levels varying seasonally and across different areas.

The findings provide crucial insights into the site's Geotechnical behaviour, have informed the design completed to date and will inform future design considerations. The key findings are summarised below.

<u>Ground Water Levels</u>

- Depths recorded between 1.5m and 4.2m across the sites.
- Seasonal Fluctuations of up to 1m observed.

<u>Soil Soakage Capacity</u>

- 2 x 10^-6m/s to 5x10^-6m/s for Silty Soils
- 7 x 10^-6m/s to 6x10^-6m/s for Sandier Soils

Soakage test Area	Results	Soakage rate adopted for Design purposes (mm/hr)	Soakage Rate factored by 0.5 as per RITS for conservatism (mm/hr)
Basin D Area (SOA24-15&16)	31-78 mm/hr	54.5	27.25
Basin C Area (SOA24-13&14)	2-51 mm/hr	26.5	13.25
Basin A Area (SOA24-23&24)	178-345 mm/hr	261.5	130.75
Northern Residential Catchment (SOA23-01) *	171mm/hr	171	171
Southern Residential Catchment (SOA23-02) *	613mm/hr	613	613
Retirement Village and South-Western residential catchment (SOA24-20,21 7 22)	480-4829 mm/hr	206.5	100

Table 5: Soakage tests Results

* *Note unit conversion, mm/hr to L/min/m² which has been used in design of road and lot areas.*

Infiltration testing across the site revealed significant variability in soakage performance, with notably low rates, such as 2 mm/hr, recorded within the North-Western Residential Catchment. According to the Geotech report, these low rates are attributed to perched groundwater, where infiltrating water is impeded by underlying less permeable layers, resulting in temporary saturation near the surface. Observations from soakage testing and variable groundwater depths indicate that perched layers are likely present in parts of the site. As such, the adopted soakage rates for design have incorporated conservative assumptions, and further investigation at the detailed design stage is recommended to confirm underlying groundwater conditions beyond any perched layers.

The results in the Geotechnical report, shows soils at this site are generally suitable to provide soakage through use of SW Devices such as raingarden/ basins etc. This soakage rate has been derived from permeability test results, the results are averaged to adopt it for design purposes and factored by 0.5 to reflect long term reduction in soakage over time.

Topsoil and Sub-surface profile

- Topsoil depth: 0.1m to 0.5m
- Subsurface Soilis: Hinuera Formation sands, silts, and gravels. Walton Subgroup soils at greater depths.

3.11 CN - VALUES

Soil Class Group B has been adopted for pre-development modelling based on the soil types encountered during geotechnical investigations (HAM2023-0124AE), which identified dense sands and stiff silts consistent with sandy loam and loam textures. These are typically associated with Group B under TP108 and USDA-NRCS classification systems. Additionally, the observed infiltration performance during site testing supports this classification.

Pre Development CN		Post Development CN	
Pervious	**Impervious**	**Pervious**	**Impervious**
61	98	74	98

Table 6: Design CN Values

For post-development conditions, a higher Curve Number of 74 has been used to account for reduced permeability due to compaction from earthworks, which may result in soil behaviour more consistent with Group C. This approach ensures a conservative assessment of stormwater runoff. The classification may be refined at the detailed design stage if additional testing is undertaken.

3.12 WGA MOUNDING ASSESSEMENT – SOAKAGE RATE

WGA conducted a mounding assessment to determine the long-term infiltration capacity of Basins A, C, and D within the Residential development. This assessment builds on the geotechnical soakage test data by incorporating additional site-specific factors such as aquifer thickness, existing groundwater levels, and the potential for mounding beneath each basin. While the geotechnical tests provide essential input on the soil’s infiltration characteristics, the mounding assessment enhances this by evaluating how infiltration behaves over time, particularly under a sustained 24-hour storm event. As a result, the average 1-day hydraulic conductivity values derived from this assessment have been adopted for design based on table below:

	mm/hr Rate
Basin A	36.03
Basin C	20.69
Basin D	20.74

Table 7: Soakage tests Results

These values reflect a realistic and technically robust approach to stormwater management for the site.

3.13 CONTAMINATION

Contamination Investigation will be carried out. Once this is completed, we will incorporate the results and information into this SMP

3.14 ARCHAEOLOGICAL ASSESSMENT

Archaeological Assessment will be carried out. Once this is completed, we will incorporate the results and information into this SMP

3.15 IWI CONSULTATION

Iwi consultation is currently ongoing, and we will incorporate the applicable information into this SMP once completed.

4 EXISTING DEVELOPMENT FLOOD MODEL

4.1 EXISTING FLOODING, MODELLING & PARAMETERS

A catchment Analysis was undertaken to assess the runoff and flow conditions dynamically during the 100year event, within and around the four development sites. This undertaking establishes existing conditions within and around the sites, and support and informs the design proposal within these areas.

A predevelopment flood analysis was undertaken using a calibrated base model and geometry. The parameters used specific to the site are detailed below.

A 100-year (24-hour) rainfall event was analysed using various points along the catchment, as shown in the figure below. Rainfall depth values were obtained at multiple locations throughout the nominated 2D area and were then averaged to determine the representative depth to be used. HIRDS RCP8.5 rainfall data was applied for both Pre-Development and Post-Development scenarios

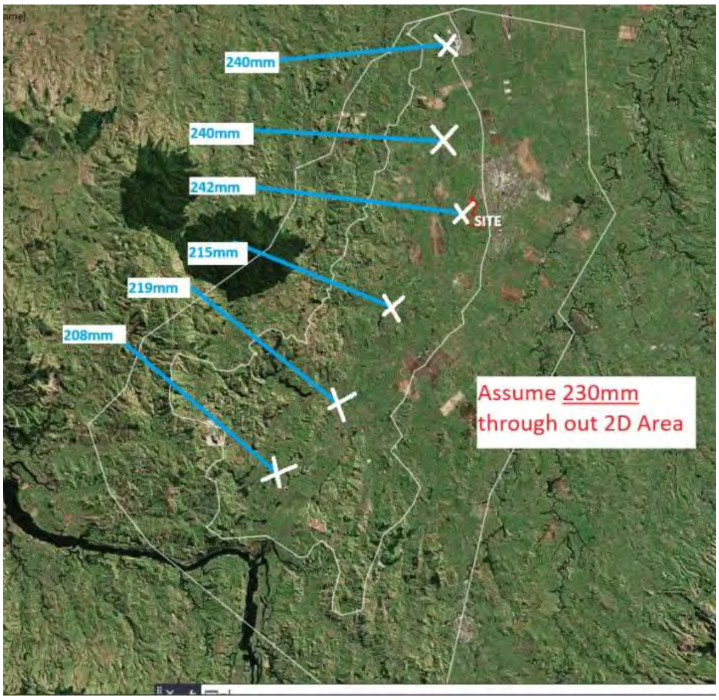


Figure 6 Rainfall depth across the 2D Area

Using HEC HMS software rainfall distribution within the shown 2D was created which has been inserted as a hyetograph onto the 2D area in HEC RAS.

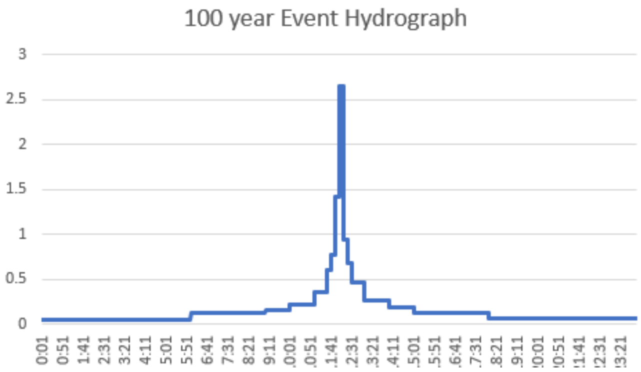


Figure 7 Hyetograph used in the Model (Data included in Appendix B)

CN Values used as shown below.

- CN numbers
    - Pre-development CN=61 for pervious Areas
    - Pre-development CN=98 for Impervious Areas
- Rainfall data from National Institute of Water and Atmospheric Research (NIWA) rainfall pattern and depth (Refer to Figure 6)

As part of the flood risk assessment for the site, an initial review was undertaken to identify any existing flood data from both the Waikato Regional Council (WRC) and the Matamata Piao District Council (MDPC). WRC confirmed that the wider area is within a flood hazard zone, however the available data was based on historical records dating back to 1996 and was not sufficiently detailed for direct use in assessing the site.

WRC did, however, provide access to gauge data from the Waitoa River at Sahara station and rainfall data from the Matamata Aerodrome Station. This formed the basis for the development and calibration of the hydraulic model, established as part of the overall flood Assessment.

The 2D flood model was built in HEC RAS version 6.6 and calibrated using observed flood events from the 2017 storm. The model simulates a 100year rainfall event (24hrs) duration and is simulated for over 3 days to ensure peak conditions are captured in the results. The calibration model/geometry was used therein to assess the existing and post development flows.

Additionally, this model will form base of the Sensitivity Analysis under section 7 of this report, the model assumes all primary system are 100% blocked.

4.2 FLOOD MODEL CALIBRATION

The Model was calibrated using observed rainfall data from the Tamhane (Matamata Aerodrome) rain gauge and river level data from the Waitoa River (Waharoa Control) Station, covering the period from April 4 to April 5, 2017. These stations are located within 6-9km downstream from the site. These data can be referred to in Appendix A of this memo.

The 2D area of the flood model was extended further downstream for calibration purposes to capture both data set available for this calibration exercise.

The datasets selected for calibration falls within the flood events that were caused by cyclone Debbie followed by Cyclone Cook in 2017. These two cyclones caused widespread of flooding across the Matamata region.

Figure 8 Calibration data Source Locations

Due to the 3km separation between the two stations, natural variability in flow response was anticipated in the model result.

The initial model result (Blue line as shown in graph below) indicated a quicker peak response than the observed data (Black Line as shown in graph below) with the modelled hydrograph rising more steeply and reaching its peak earlier than the observed data measurements.

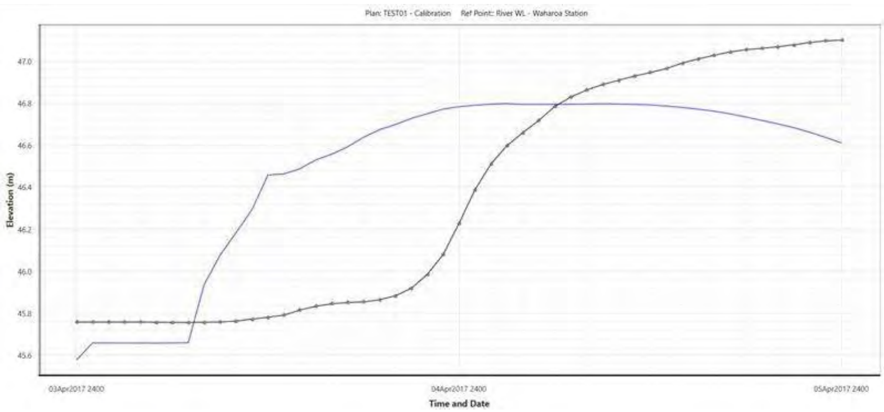


Figure 9 Elevation vs Time graph for both Model and Observed Data

Discrepancies in water levels at the beginning and end of simulation suggested differences in initial conditions, as model started with an empty channel while observed data included existing flow.

	Observed Data	Model Result	Difference
Initial Water RL	45.76	45.58	0.18
Max RL	47.1	46.8	0.3

Table 8 *First Run Results*

Area type	Road	Grass /farm	Stream Area	Rest of Area
Mannings n	0.02	0.045	0.05	0.04

Table 9 Mannings – Pre-Calibration of Model

The first calibration results presented above in first table, showing level differences between the observed data and the model. If we consider the model to have the same Water level as the observed data, the level difference is 120mm. Mannings numbers used in the model is shown in the above second table.

Multiple runs were further undertaken of the calibration model to achieve a better alignment of the two sets. Mannings were adjusted including cell sizes. Final calibration results shown below table, including Manning's finalised values.

The second graph shows results of one of the adjustments which has improved the difference between the two sets, demonstrating a better representation of flood behaviour within the Waitoa River.

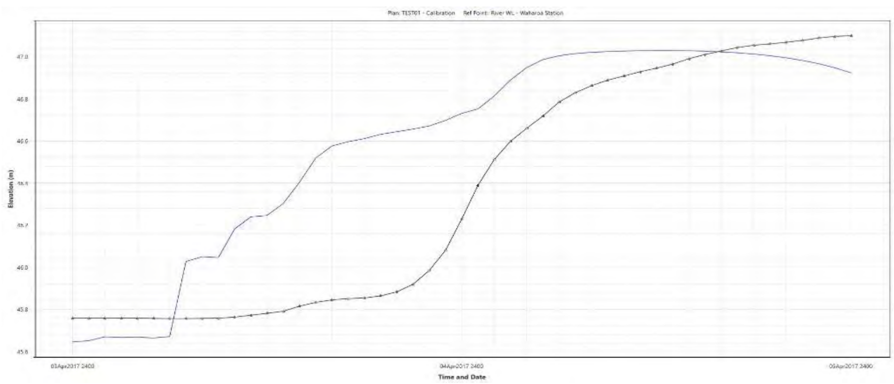


Figure 10 Elevation vs Time graph for both Model and Observed Data

	Observed Data	Model Result	Difference
Starting RL	45.76	45.58	0.18
Max RL	47.1	46.86	0.24

Table 10 Final Run Results

Area type	Road	Grass /farm	Stream Area	Rest of Area
Manning; s n	0.02	0.04	0.06	0.04

Table 11 Mannings – Final Run

If model is considered to have the same Water level as the observed data, the level difference is now 60mm. Mannings numbers used in the model is shown in the above table. Further refinement of the model is not possible as adjusted parameters will be unrealistic.

Due to the nature of this model and data available it is not practical nor necessary to do further refinement until both models are the same. Theres is variances through rainfall distribution, land use, un modelled hydraulic devices. Instead, the focus was on ensuring that water level in the model is as realistically close as practically possible to the observed data, as this will establish a robust model base to use for flood analysis on the proposed site. Based on the above, this calibration achieves a good overall balance between real world conditions and modelling.

As mentioned in previous sections, The RCP8.5 climate change adjustment has been applied to both scenarios (pre- and post-development) to enable comparison between the two events when assessing compliance.

4.3 EXISTING MODEL RESULTS

The flood map results below display the extent of flooding with a tolerance of 50mm depth minimum. The results show the there are multiple interconnected overland flow paths traversing through sites. The general route for these flows is either a direct discharge into the Waitoa river or initially traversing ponding on land prior to discharging into the Waitoa River.

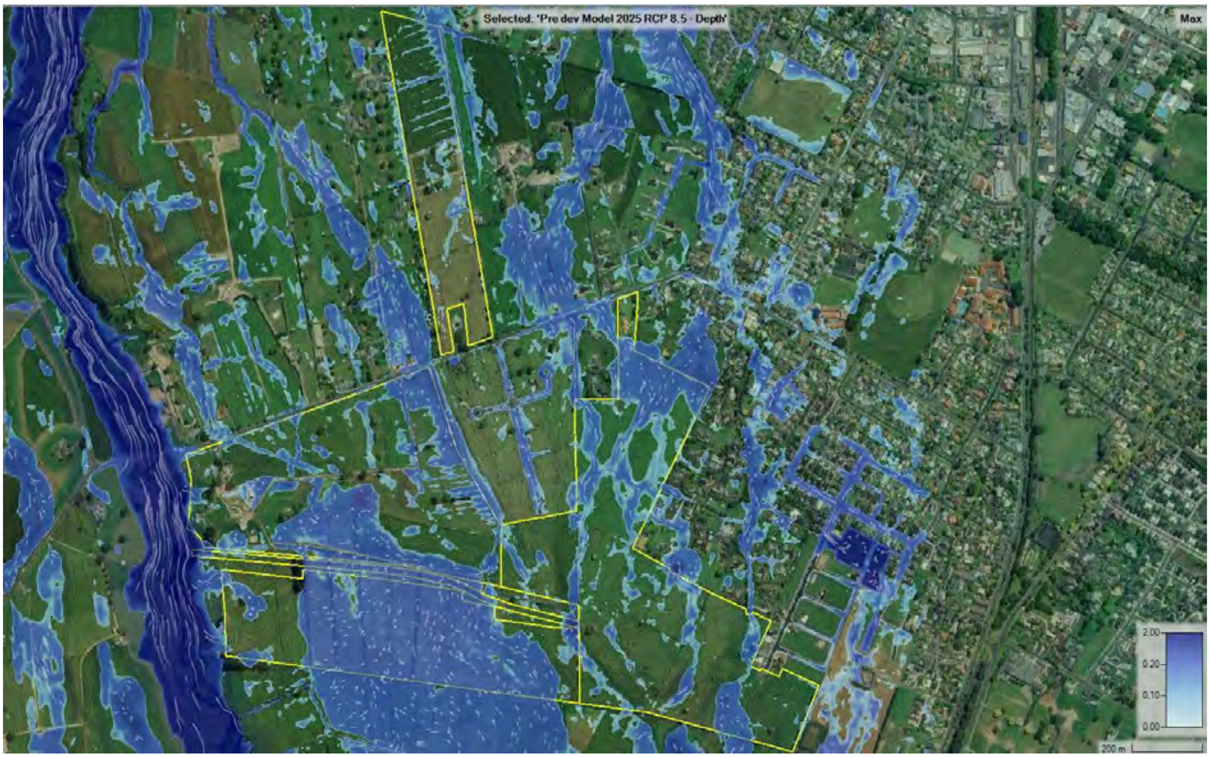


Figure 11: Overall Pre-Development Flood Map RCP 8.50

4.4 RESIDENTIAL AREA

As mentioned earlier in the report, there are several inflows into the residential area, originating from the upstream regions. These flows move northward into the site, following existing natural landforms and farm drains, before exiting along the lower northern boundary.

Notably, one of the discharge points from the residential catchment serves as an inflow into the retirement village catchment, as depicted in the screenshot, the flood model report, and the results below.

4.5 NORTHERN SOLAR FARM

inflow from the west enters the site and is conveyed through a network of existing farm drains. Most of the flows continues northward, with southern lower portion of the site discharges further east and west which is due to a crest line through the site within that portion of the site.

4.6 RETIREMENT VILLAGE

The retirement village catchment receives inflow from both southern solar farm and contributing portion (North-western Areas) of the residential catchments. There a few service tracks, farm drains and an existing conveyance channel running along the eastern boundary that leads to the existing culvert as mentioned in section 2 of this report

Due to blockage of the culvert beneath station road, as assumed in the flood model, floodwaters upstream of the culvert fills up the low land, flat areas of the RV site, flooded the area before it overtops station road.

4.7 SOUTHERN SOLAR FARM

This area receives upstream external catchment to the south. Runoff through the site is interfered by an existing farm track acting as a barrier, causing temporary ponding upstream of the farm track. The flow eventually overtops where the flow splits, north into the retirement village through the existing open channel while remainder continues westward, ponding within the localised depression before discharging into the Waitoa river. An isolated but much smaller area within the solar farm, northwest of the site, discharges directly into the Waitoa river

5 OPERATIVE CATCHMENT AND DEVELOPMENT PLANS

The development comprises both general residential and medium density residential zones as per the Operative Matamata-Piako District Plan (MPDP). This section provides a high-level summary of the planning and regulatory requirements.

5.1 REGULATORY AND DESIGN REQUIREMENTS

The relevant regulatory, technical and design requirements that the stormwater management for the development must meet are summarised in Table 12.

Table 12: Summary of regulatory and design requirements.

Requirements	Relevant regulatory/design to follow
Matamata-Piako District Plan (MPDP) Section 5.9.1 Infrastructure and servicing performance standards	• The subdivision and development of land shall be carried out so as to provide for effective stormwater management, in compliance with the Development Manual. • Be adequate to meet the maximum potential demand on site arising from the development and use of the land as permitted under the District Plan and taking into account the actual and potential land uses up-gradient of the site. • In the first instance be managed and disposed of on-site. Only where onsite disposal is unable to be achieved will Council consider new connections to public drainage, where available. • The design capacity of any piped stormwater facilities should be sufficient to accommodate the surface water flows resulting from a 10-year storm event in the case of Residential, Industrial and Business zones, and to avoid flood damage to the existing or potential principal buildings on the site, resulting from a 100-year storm event . • Overland flow paths must be able to cater for a minimum of a 1 in 100 year return period storm. Flood paths within the subject site must be protected by an easement registered against the titles affected throughout their length. Where flood paths are not feasible, the piped system must cater for a minimum of a 1 in 100 year return period storm. Where disposal is to ground soakage with no flood path, the soakage must cater for a 1 in 100 year return period storm. • Secondary flows exiting the subject site following development must not exceed pre-development overland flows. • Stormwater proposals must take into account the requirements of the Council's current stormwater discharge consents from the WRC. All proposals must be consistent with the conditions of this consent including requirements for low impact design principles, stormwater management devices and best practicable options as set out in the consent. • Stormwater works should be provided in a manner which avoids excessive modification of natural drainage systems and minimises any detriment to the environment particularly through potential contamination of natural water.

	• The integrity of the stormwater system should be maintained, and its safe and efficient operation facilitated, while ensuring an adequate level of safety to the public and those operating and maintaining the facilities. • Any detriment to the enjoyment and development of individual allotments arising from the provision and operation of the stormwater system needs to be minimised. • The known or predicted effects of climate change on a proposal, based on best available scientific knowledge, shall be taken into account. • In terms of operation and maintenance, the stormwater system shall be in line with community expectations regarding anticipated performance. • The lifecycle and maintenance costs meet community expectations. • Any disposal or treatment areas located off-site, other than to Council owned systems, shall be protected by easements as appropriate. • Council may require a detailed stormwater plan to ensure that there are no adverse stormwater effects off-site. • Any necessary consents shall be obtained from the WRC.
Comprehensive Network Discharge Consent 105063	Condition 28 of the comprehensive stormwater discharge consent (#105063) states: "The consent holder shall prepare a Stormwater Management Plan for the Matamata-Piako District Township and municipal stormwater diversions and discharge activities, which shall be submitted to the Waikato Regional Council within 12 months of the commencement of this consent". The Plan shall be developed in consultation with interested parties and shall detail the procedures, initiatives and stormwater management systems that will be implemented to operate in accordance with the conditions of this resource consent. As a minimum the Stormwater Management Plan shall describe the following: a) The relationship and integration of the Stormwater Management Plan with other Matamata-Piako District Council planning instruments and regulatory systems, including existing and proposed planning and regulatory controls that will be utilised to assist the control of routine and non-routine contaminant discharges to the stormwater system. b) Contributing catchments and the existing land uses, catchment receiving waters (including physical and biological characteristics, riparian vegetation, existing uses and values) and the municipal stormwater system characteristics (a diagram showing locations of the reticulation system, designated overland flow paths, treatment and disposal systems should be included). c) Potential risks to stormwater quality in the Matamata-Piako urban areas (i.e., resulting from routine and non-routine contaminant discharges to the municipal stormwater system;

	d) Stormwater management systems and implementation methods to avoid, remedy or mitigate routine contaminant discharges to the municipal stormwater system. e) Contingency measures and reporting systems to be implemented in the event of non-routine contaminant discharges from the municipal stormwater system. f) Methods which will be used to manage risks to stormwater quality, streambed scouring and erosion, and adverse flooding; g) Methods to identify and provide for stormwater overland flow paths in urban areas; h) Strategies for identifying municipal stormwater system structures that are impeding the upstream or downstream movement of fish, and system upgrades, implementation methods and timeframes to address these; i) Initiatives and implementation methods for improving the aesthetic appearance of drainage structures and stormwater detention areas; j) Street cleaning operations; k) All stormwater management devices and methods for ensuring that stormwater management devices are constructed and maintained in accordance with the Auckland Regional Council Technical Publication No 10 "Stormwater Management Devices; Design Guidelines Manual" (ARC, 2003) or any other technical publication approved in advance by the Waikato Regional Council acting in a technical certification capacity; l) Management and operational procedures to avoid contaminants discharging from the municipal stormwater system as a result of the various municipal operation and maintenance activities; m) Investigations and remediation programmes to discontinue informal wastewater system connections to the municipal stormwater system; n) Operation and maintenance programmes to minimise the discharge of any municipal wastewater system contaminants to the municipal stormwater system; o) Methods which will be encouraged by the Matamata-Piako District Council to minimise the effects of stormwater discharges from new subdivisions; p) Methods for ensuring consideration of Low Impact Design principles (as contained in the Auckland Regional Council Technical Publication No. 124 "Low Impact Design Manual for the Auckland Region" (ARC 2000)) for proposed greenfield development sites; q) Methods for identifying and implementing Best Practicable Options to manage the municipal stormwater system and prevent or mitigate adverse effects on aquatic ecosystems; r) Methods for implementing stormwater management education initiatives; s) A prioritised schedule for implementing the procedures, initiatives and stormwater management systems that are identified in the Stormwater management plan".

6 PROPOSED DEVELOPMENT & STORMWATER MANAGMENT

The overall development comprises of four distinct projects, as summarised below

1. Residential Development
2. Northern Solar Farm Development
3. Southern Solar Farm Development
4. Retirement Village

The residential and retirement village project share a common approach to stormwater management. Both developments incorporate roadside raingarden for treatment for high contaminant generating areas within the road carriageway area. Roadside soakage integrated with raingarden combines to fully store and soak incoming flows for event up to the 10year cc storm event. Flows exceeding this are conveyed via overland flow paths within road carriageway to downstream basins for attenuation and/or soakage.

Following discussions with WRC for on-lot stormwater management, they accept in most cases it is not practical for on-lot stormwater systems to have multiple devices providing treatment. Therefore, a driveway catchpit or strip drain is proposed which discharges up to the 10-year cc event into a soakage trench/soak pit. A lot connection into the roadside soakage trench will also be provided for larger events and provides redundancy should on-lot soakage fail. Flows exceeding this are conveyed via overland flow paths within the road carriageway to downstream basins for attenuation and/or soakage.

The two solar farm projects, located north and south of Station Road, introduces minimal land disturbance. The solar panels are elevated on stilts and placed on existing pastoral land, with only limited earthworks required for the two developments. Stormwater management approach for the two solar farm sites, is utilising the existing farm swales for conveyance, treatment, and attenuation.

As mentioned in previous sections, The RCP8.5 climate change adjustment has been applied to both scenarios (pre- and post-development)

6.1 POST DEVELOPMENT CATCHMENTS

The post development catchment layout for both the residential and retirement village areas has been defined based on the proposed stormwater basins, which also serve as the primary discharge points for each sub catchment. the placement of these basins has been carefully considered to align with existing discharge locations ensuring that no new discharge points are introduced as part of the development.

Each post development catchment incorporates not only the development catchment but also includes upstream inflows that have been identified and considered in the design. These inflow areas have been accounted for in the post-development analysis to ensure ongoing conveyance and to avoid any adverse effects both upstream and downstream resulting from the proposed development

The two solar farm projects are not expected to alter the existing catchment flow patterns, as existing flow routes are not altered within the sites. As the solar panels will be mounted on steel frames with minimal ground disturbance with only access tracks introduced offsetting existing ones likely to be removed due to the development, most of the land will remain as it was. Therefore, post development flow paths in these areas closely follow existing conditions, retaining predevelopment conditions.

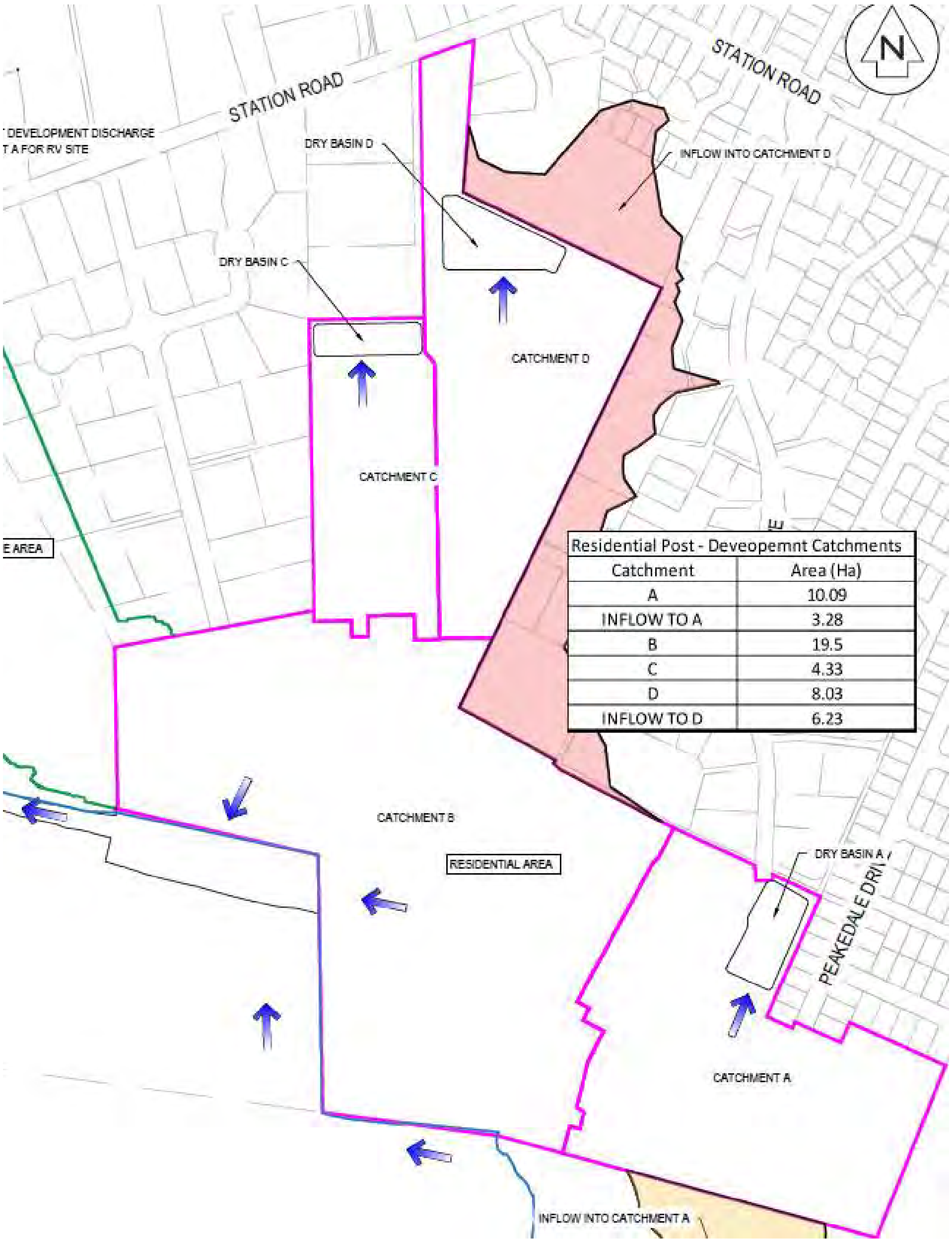
The post development model and calculations are built upon the following input and assumptions:

- CN numbers
    - Post-development CN=74 for Impervious Areas
    - Post-development CN=98 for Impervious Areas
- Rainfall data from National Institute of Water and Atmospheric Research (NIWA) rainfall pattern and depth:

<table>
<tr><td colspan="2">Post Development CN</td><td></td></tr>
<tr><td>Pervious</td><td>Impervious</td><td></td></tr>
<tr><td>74</td><td>98</td><td></td></tr>
<tr><td colspan="3">Pre Development 24hr Rainfall Depth (mm)</td></tr>
<tr><td>10yr RCP8.5</td><td>100 yr
RCP8.5</td><td>100yr-10yr</td></tr>
<tr><td>128</td><td>200</td><td>98</td></tr>
</table>

Table 13 Design Parameters table

6.2 RESIDENTIAL DEVELOPMENT – CATCHMENT A-D



Residential Post - Deveopemnt Catchments	
Catchment	Area (Ha)
A	10.09
INFLOW TO A	3.28
B	19.5
C	4.33
D	8.03
INFLOW TO D	6.23

Figure 13: Overall Residential Development Catchment Plan

Catchment A is the south-eastern portion of the site. Soakage will be provided within the lots and the road to manage the stormwater flows for up to 10-year cc storm events. Storm events more than the 10-year cc storm events, will be conveyed as overland flows to a new designated stormwater basin downstream of each catchment to provide further mitigation up to the 100year cc event. upstream inflow into catchment A has been accounted for in the HEC HMS model. Refer to Appendix B.

Catchment B is the south-central portion of the site that connects to the proposed greenway. due to high ground water relative to the invert of the basin, soakage was not incorporated into the design for mitigation of flow within catchment B. in line with the overall strategy, the flow up to the 10-year ARI cc will be accommodated in roadside raingarden/soakage and private soakage for lot areas. flow above this event will be drained to a newly formed Basin + greenway. The greenway will start from the western boundary, and it will continue west to connect into the Waitoa River. At the end of the greenway an orifice of 1.25m ins dimeter is to be set at base of the greenway to allow for attenuation of the post-development 100year flow to 80 % predevelopment level.

Catchment C is the north-western portion of the site. Soakage will be provided within the lots and the road to manage the stormwater flows within the road corridor for up to 10-year cc storm events. Storm events in excess of the 10-year ARI cc storm events, will be conveyed as overland flows to a new designated stormwater dry basin in the northern portion of the catchment area.

Catchment D is the northern portion of the site. Soakage will be provided within the lots and the road to manage the stormwater flows for up to 10-year ARI cc storm events. Storm events in excess of the 10-year ARI cc storm events, will be conveyed as overland flows within the road corridor to a new designated stormwater dry basin in the northern portion of the catchment area.

Residential Staging

The residential portion of the development is intended to proceed in stages. As each stage progresses, interim swales will be implemented to ensure that stormwater mitigation measures remain consistent with the intent of this Stormwater Management Plan (SMP). During early stages, undeveloped areas will retain their natural flow patterns, and any new works will be designed to avoid adverse impacts on these areas. The strategic use of swales will provide effective flow management and treatment during construction and staging, ensuring that runoff is appropriately directed and controlled until full development and permanent infrastructure are completed.

6.3 RETIREMENT VILLAGE

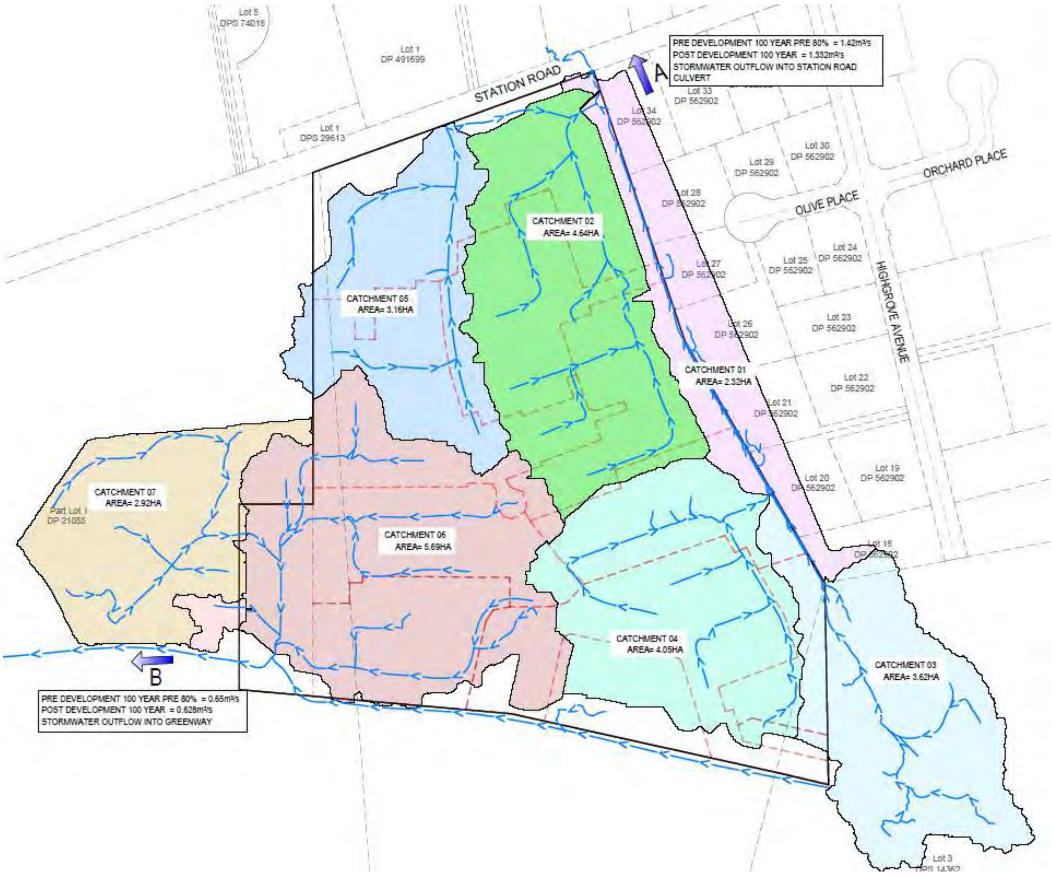


Figure 13: Overall Retirement Village Development Catchment Plan

The residential and retirement village project share a common approach to stormwater management. Both developments incorporate roadside raingarden for treatment for high contaminant generating areas within the road carriageway area. Roadside soakage integrated with raingarden combines to fully store and soak incoming flows for event up to the 10-year cc storm event. flows exceeding this are conveyed via overland flow paths within road carriageway to downstream two basins for attenuation and/or soakage, with slow release at, max 80% predevelopment.

6.4 SOLAR FARMS – NORTH AND SOUTH

As the solar panels themselves are built on steel frames, only approximately 5% of the solar farm will be changed to impervious surface. The remainder of the site remains in pasture suitable for sheep grazing. Therefore, assuming adherence to best practice stormwater management, the effects of increased stormwater runoff is considered to be 'Low'.

Southern solar will have combination of treatment methods where it will be initially treated using existing features such as drains etc, and eventually allow to flow to its natural flow path and existing streams. Farm site has 4m wide that runs from west to east centrally dividing the site. To manage stormwater flow from the southern side of the farm, open channels are proposed run in such a way that the flow is directed towards culverts. Culverts are placed at natural low points/existing streams allow discharge from southern side of the farm flow naturally to the other side discharging to proposed greenway and eventually flowing to Waitoa river. Culverts are designed to flow at 0.5% slope and average of 1m cover to comply with WRC standards. It will be sized to accommodate the discharge from receiving environment at detail design stages.

Northern solar farm site gently slopes from south to North. Existing farm drains will be incorporated around the site to facilitate pre-treatment of stormwater which will ultimately discharge into the Waitoa River.

6.5 CATCHMENT FLOW ANALYSIS

A hydrological model was created to assess the peak flows from the post development catchments. The model is based on the following input and assumptions:

- 10-year cc and 100-year cc rainfall from the post development catchments.
- Catchment extents as per C410 & 420 Series. refer to Appendix A
- Catchment characteristics as outlined in Appendix B (SW Calculations)
- Full soakage of up to the 10year cc event for Catchments for all lot and road catchments.
- Basin A, C and D full storage and Soakage for Excess flow from upstream up to 100year cc event. Attenuation to 80% predevelopment flow for Catchment B.
- For all Catchments: road corridors and accessways to convey secondary overland flow paths to the proposed Dry basins.
- For the Solar farm catchments: road corridors and accessways to convey secondary overland flow paths to the watercourses directly.

6.6 PRINCIPLES OF STORMWATER MANAGEMENT

The proposed stormwater management for the 4 developments incorporates a number of stormwater management principles, which are focused on:

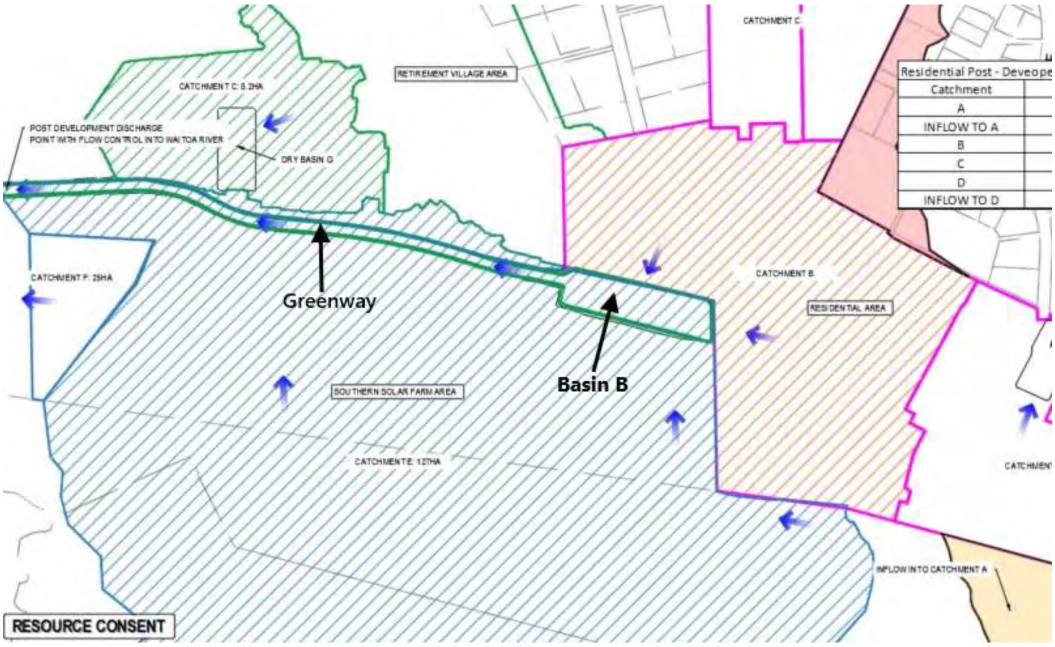
- Enhancing ecological value of the wider catchment.
- Preserving existing and waterways within the development sites.
- Mitigating flooding impacts.
- Treating stormwater runoff from the proposed impervious areas.

The principles set out in this SMP aligns with the previous projects SMP approved by WRC (Lockerbie Estate Subdivision AUTH141393.02.01) located in the same district of the proposed sites. The key components of the Ashbourne Developments SMP are as follows:

- Stormwater conveyance for up to 10year cc ARI rainfall event.
- Overland flow paths for 100-year cc ARI rainfall event to be accommodated within the site and conveyed by the road and green corridors.
- Downstream mitigation through attenuation of 100-year cc ARI rainfall event within the site:
    - Discharge limited to 80% pre-development levels (Maximum).
- Downstream mitigation through detention of 10-year cc ARI rainfall event within the site:
    - Discharge limited to pre-development levels (Maximum).
- Treatment of runoff prior to discharge into receiving environment in accordance with TP10 / GD01 / Waikato Stormwater Management Guidelines (WRC Technical Report 2020/07).
- Retention of initial abstraction runoff volume using raingardens and soakage devices for the development.

6.7 GREENWAY AND BASIN B-Residential

Basin B is connected and located upstream of the proposed Greenway. Both forms part of the overall greenway/Basin B stormwater Mitigation device designed to cater for attenuation of flow from catchment B of the Residential development area and the diversion of the existing flows south and north of the Greenway.



Catchment	Area (Ha)
RES B	19.5
Inflow RV	8.2
Inflow South	127

Table 14 Greenway & Basin B Catchments

The proposed Greenway corridor interconnects infrastructure, ecological wellbeing, connectivity, and amenity to support a place-based identity. Several uses are proposed along this corridor to encourage future residents to interact with the greenway, such as sheltered rest areas for relaxation and socialisation, active mode pathways, and play areas.

Figure 14: Greenway Cross Section (Plan 490-17)

The proposed greenway is sized to accommodate the 100-year ARI cc stormwater event flows less the 10-year ARI cc event from the Resident Area B. The 10 year cc ARI are proposed to be discharged via soakage in the road carriageway and the in the lot areas upstream of the proposed Greenway. Additionally, there is an inflow from the RV site through proposed South Easter RV Basin as shown in the above catchment diagram. This basin provides stormwater mitigation by capturing and discharging flows at 80% predevelopment flow into the greenway.

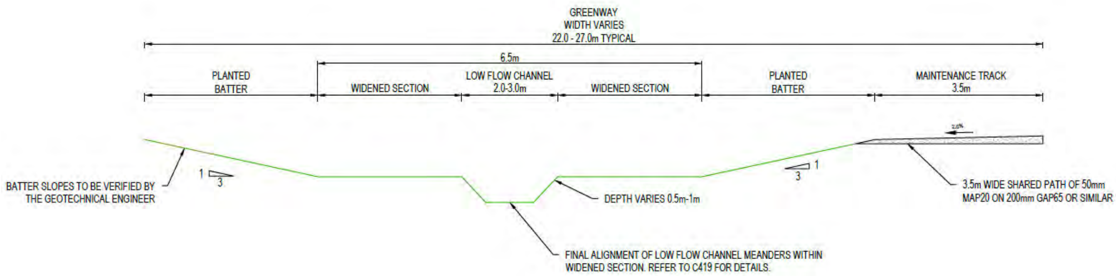


Figure 15: Greenway Cross Section (Plan 490-17)

A low-flow channel is incorporated at the base of the proposed greenway to replicate existing flow conditions and support continuous baseflows through the corridor. The channel is designed to have a width of approximately 2 to 3 meters and a depth of 0.5 to 1 meter, providing a defined conveyance path for low flows while maintaining ecological connectivity. This configuration ensures that hydraulic function is preserved during dry weather periods and provides controlled conveyance under baseflow conditions. The alignment and shape of the low-flow channel will follow the greenway's finished levels, and an impermeable liner will be considered where required to retain flow and minimize infiltration losses.

To provide for future maintenance of the greenway a 3.5m wide maintenance track will be constructed along the northern side of the greenway. The maintenance track will also provide a shared access track for pedestrians and cyclists. The greenway will have widened sections to provide some additional flood storage and to enhance the aesthetics of the greenway. The typical greenway section is shown above (figure 15).

6.8 DRY BASINS

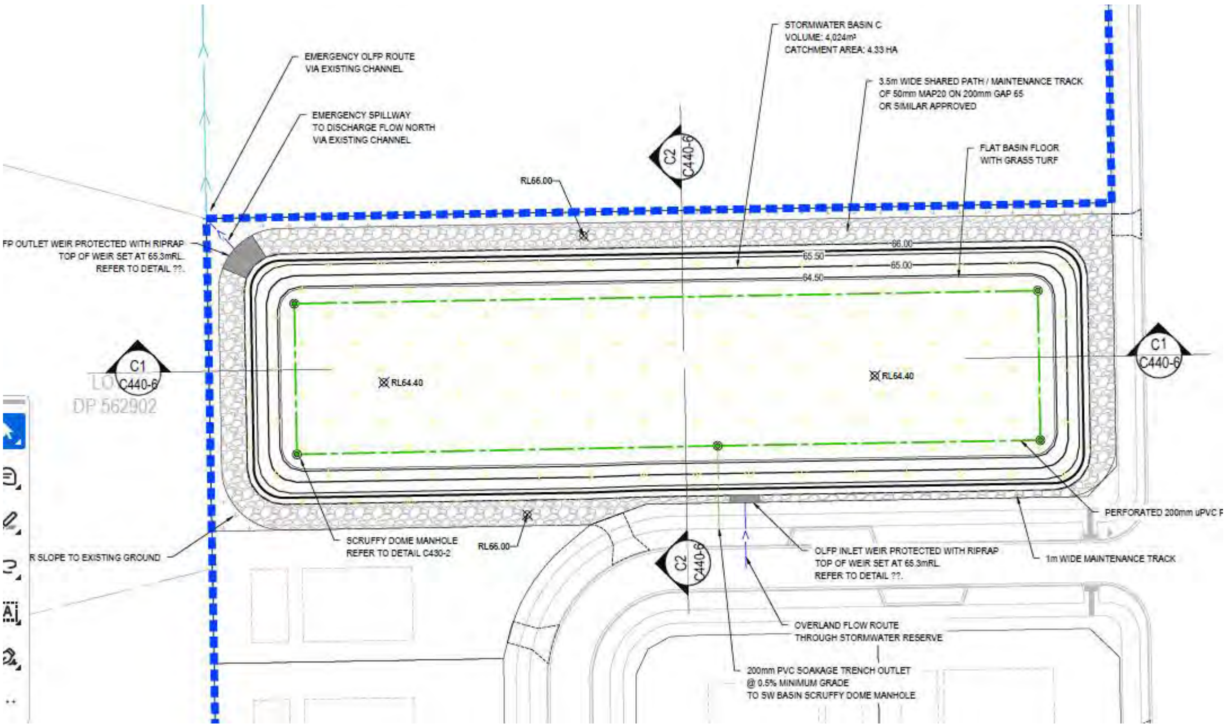


Figure 16: Dry Basin Typical

The proposed Dry basins captures excess runoff from upstream devices above the 10-year cc event. and up to the 100-year cc event. Attenuation, storage, and soakage is carried out within the dry basins with flow rate released no more than 80% Predevelopment rate.

Residential Dry basins for A, C and D, have been designed conservatively allowing full storage of the from upstream catchment. there is soakage element to them that has been applied in the HEC HMS model having a constant soakage throughout the 24hr period. Further calculations were carried out to determine time to drain from full condition (peak) with max allowable 48hrs.

The Retirement Village Dry Basins capture excess runoff from the upstream devices during event above the 10-year design event. Attenuation, storage, and soakage occur within the dry basins, and the flow rate is maintained at no more than 80% of the pre-development level rate.

Refer to calculation found in appendix B of this SMP.

For this resource consent application, stormwater dry basins have been designed to conservatively store the 100-year cc less 10-year cc event and to discharge via soakage without any release of flows into existing channels or via existing OLFPs. Therefore, there is potential for further refinement of dry basin sizing to utilise existing OLFP’s up to a maximum release rate of 80% pre-development

6.9 SOAKAGE DEVICES

6.9.1.1 ON LOT SOAKAGE

The on-lot drainage system for the Ashbourne Residential area is proposed to consist of the following.

- Soakage Device
- Slot drain (if needed) connected to a Catchpit in the driveway
- Pipe system from the roof and catchpit (driveway) to the soakage Device
- Lot Connection pipe that connects Catchpit to the roadside soakage device.

Driveway/Impervious area runoff will flow to the slot drain directing flow to proposed catchpit with sump where settlement of courser suspended solids can occur. The flow then gets discharged into the soakage trench where it will be treated further and then soak to the ground. Pre-treatment for sediments is not required for typical residential roof loadings therefore the roof runoff with be discharged directly to Soakage trench.

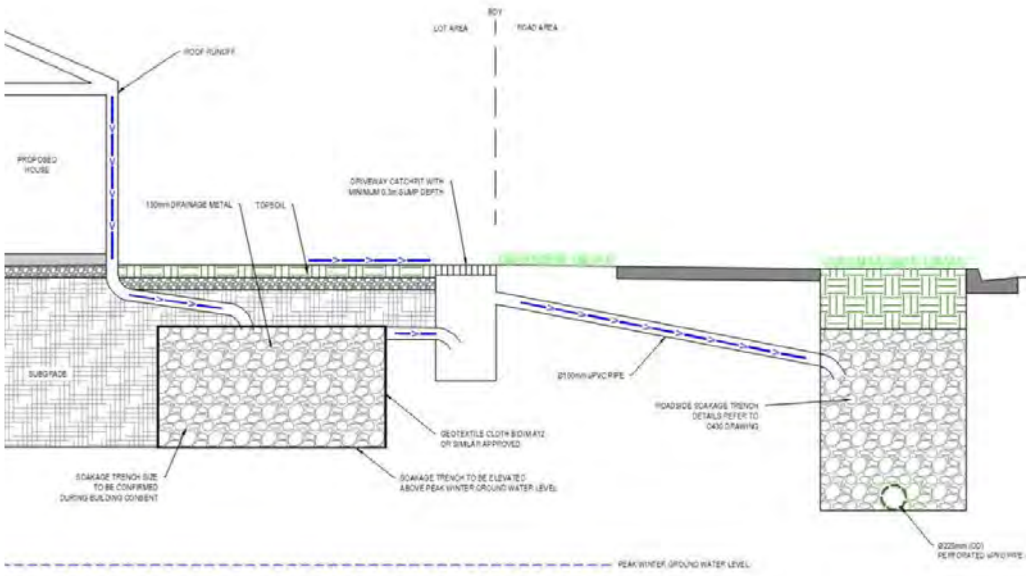


Figure 17: On Lot Drainage system/Soakage Typical Det

The proposed on lot systems have been designed to cater for up to the 10year cc rainfall event. Events above the 10year cc will be directed into the carriageway. This will be provided through a primary overflow mechanism, in the form of a connection from the proposed catchpit whin each lot to the stormwater soakage trench within the road reserve. This will form a lot connection for each of the lot areas within the Ashbourne Residential Area. Any flows above this, will be conveyed by overland flow into the road carriageway.

The sizing of soakage trenches for residential lots was based on the methodology outlined in E1/VM1. Stormwater catchment volumes were calculated for three typical impervious roof areas—150 m², 200 m², and 250 m². Corresponding soakage disposal volumes were determined using three representative soil infiltration rates (0.5, 1.5, and 3.0 L/min/m²).

The required trench storage volume for each scenario was derived by subtracting the volume of water disposed by soakage from the total catchment volume. In total, nine soakage trench volumes were calculated to represent different combinations of impervious area and soakage performance, allowing flexibility in design depending on lot-specific conditions. Refer to Appendix B for full calculation. Also refer to plans which states Soakage Device Sizing requirement table.

Additional site-specific geotechnical and/or infiltration testing will be undertaken prior to construction to confirm soakage values sizing of the proposed devices

6.10 On Lot RAIN SMART TANKS (RETIREMENT VILLAGE)

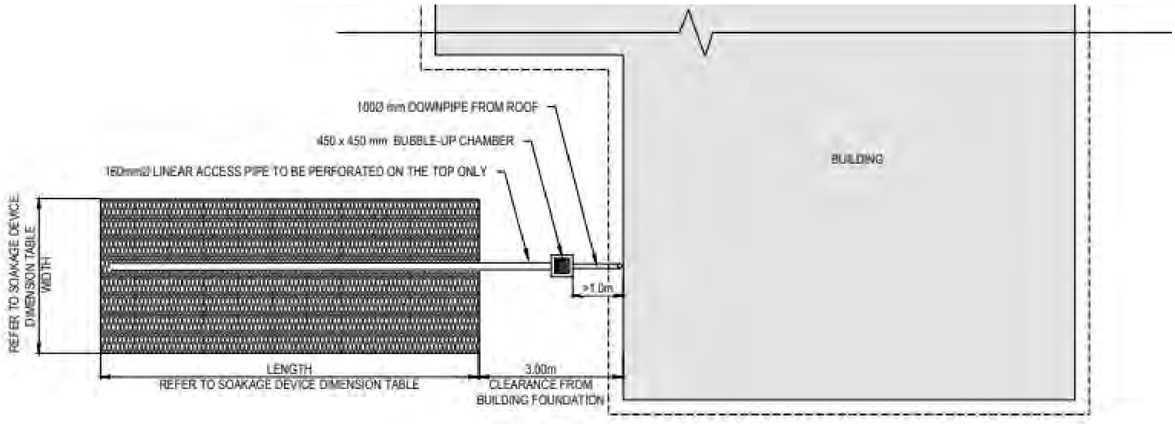


Figure 18: On Lot Soakage Typical Det

On-site stormwater management for the Retirement Village is achieved by using a modular Cirtex Rain smart Soakage system. This device is integrated within each unit's lot boundary to treat and dispose of roof runoff in accordance with regional requirements. Each device comprises modular tanks wrapped in AS410 geotextile, installed over a compacted sand base within a trench. The system includes a perforated 160mm diameter linear pipe to distribute flows evenly through the device, and a bubble-up chamber provides an overflow pathway. Refer to the RV infrastructure report containing the details of this product and design approach.

Additional site-specific geotechnical and/or infiltration testing will be undertaken prior to construction to confirm soakage values sizing of the proposed devices

6.11 Road Soakage & Raingarden – Residential

Combination of raingarden and soakage trench is used for managing impervious area flows of 10-year cc stormwater event. The raingardens will be connected via piped network to Stormwater basin.

Water from catchpits, roads and manhole will flow to raingarden where it will filter through the layers of raingarden thus removing contaminants and treating it initially. It will be placed at high point of soakage trench so it can flow naturally towards the soakage trench.

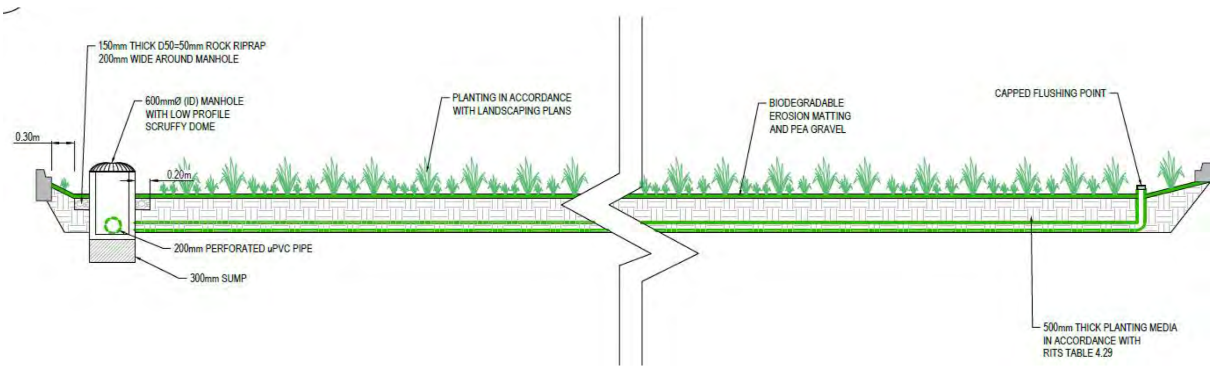


Figure 19: Typical RG (C430)

Raingardens were sized in accordance with RITS (Regional Infrastructure Technical Specification) requirements, using a volume equal to 2% of the contributing catchment area. To determine the required raingarden area, the total volume was divided by an assumed 1.0 m raingarden depth. This area was then divided by the road chainage within the relevant catchment to calculate the linear length of raingarden required per metre of road. This approach ensures adequate stormwater treatment and attenuation across road catchments, aligning with local council expectations for water quality management.

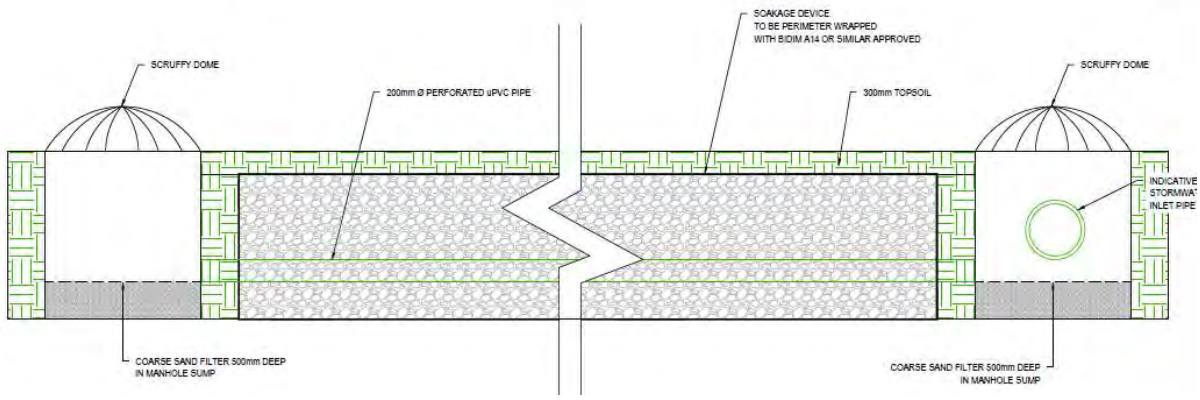


Figure 20: Typical Soakage Detail (C430-RES)

The soakage trench for the road carriageway area was designed in accordance with the Matamata-Piako District Council (MDPC) Soakage Manual, with the effective soakage area calculated using the methodology specified in the manual. The design objective is to ensure the trench can accommodate runoff up to a 10 year cc rainfall event.

Soakage rates were obtained from CMW Geotechnical testing,(allowing for 0.5 factor in accordance with the manual) using two representative rates, based on test pit results:

- 1.7 L/min/m²
- 5.7 L/min/m²

Due to the absence of a 10-year, 24-hour rainfall intensity graph in the MPDC Manual has been reasonably assumed/interpolated based on available information. Impervious area limits were estimated based on standard road widths.

For 18 m wide roads, with a trench dimension of 2.0 m wide x 1.5 m deep, the required trench length per meter of road was

- 0.55 m (0.28 m each side) for the 1.7 L/min/m² soakage rate.
- 0.27 m (0.135 m each side) for the 5.7 L/min/m² rate.

For 20-meter-wide roads, with a trench size of 1.2 m wide x 1.5 m deep, trench lengths of 1.1 m total (0.55 m per side) and 0.62 m total (0.31 m per side) were required for the lower and higher soakage rates, respectively.

- 1.1 m (0.55 m each side) for the 1.7 L/min/m² soakage rate.
- 0.62 m (0.31 m each side) for the 5.7 L/min/m² rate.

These trench sizing calculations above sets guidelines for sizing of soakage based on road typology and relevant soakage testing results onsite.

Additional site-specific geotechnical and/or infiltration testing will be undertaken prior to construction to confirm soakage values sizing of the proposed devices

6.12 Road Soakage & Raingarden – Retirement Village

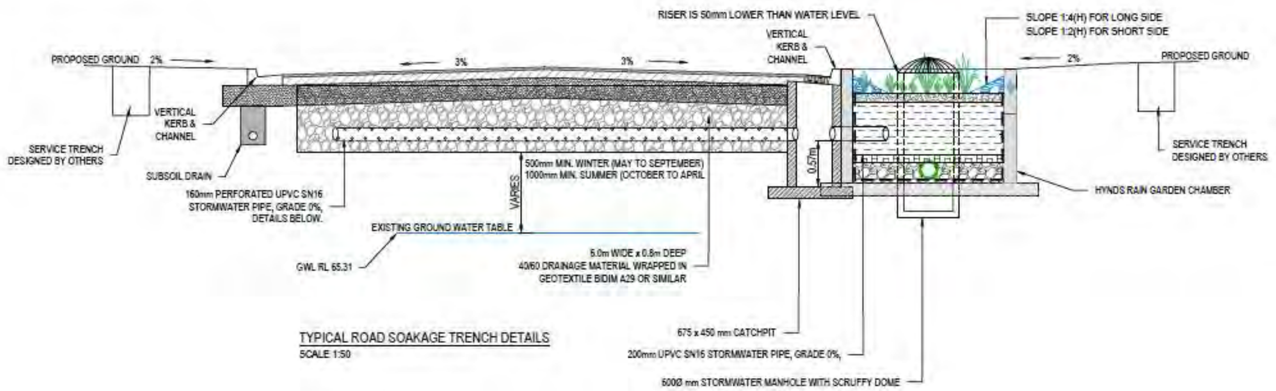


Figure 21: Typical Soakage Detail (C430-RES)

The soakage trench under road reserve is used for managing flows up to 10-year cc event for retirement village. Flows exceeding 10-year cc event will be redirected to road carriageway and discharged to the designated stormwater pond.

The retirement village reticulation system will allow stormwater to soak to the ground via roadside soakage trench for 10 yr cc event. Flows exceeding the 10-year cc soakage capacity are redirected back into the road carriageway and get discharged at the designated downstream Stormwater Pond.

Raingardens were sized in accordance with RITS (Regional Infrastructure Technical Specification) requirements, using a volume equal to 2% of the contributing catchment area. To determine the required raingarden area, the total volume was divided by an assumed 1.0 m raingarden depth. This area was then divided by the road chainage within the relevant catchment to calculate the linear length of raingarden required per metre of road. This approach ensures adequate stormwater treatment and attenuation across road catchments, aligning with local council expectations for water quality management.

6.13 INITIAL ABSTRACTION VOLUME

Initial abstraction volume of runoff from the proposed catchments will be retained by the proposed on-lot soakage Trenches and catchpit sumps. Initial abstraction volume of runoff from the proposed roads will be retained by the proposed raingardens/soakage devices. The preliminary design of these devices is summarised above and as provided in appendix B.

7 POST DEVELOPMENT FLOODING

7.1 OVERLAND FLOWPATHS (OLFPS) - CARRIAGEWAY

Additional branches of OLFPs will be created as roading corridors are formed. The following measures will be adopted to mitigate their effects of these overland flow paths on the proposed development.

- Identify and maintain natural overland flow/watercourse locations to convey concentrated stormwater from the site. Utilise existing culverts (where possible) to maintain the same discharge locations, post development.
- Identify and retain any upstream OLFPs and/or watercourses to avoid any upstream flooding.
- Ensure OLFPs are to be designed where possible within the roading network and discharge into watercourses and 100-year detention devices.

The preliminary OLFP design is shown in Maven Associates drawings C460, Appendix B Detailed design of the OLFPs will be provided at future detail design stage following the approval of the resource consent.

A Preliminary assessment of the post development overland flow paths (OLPs) has been carried out to evaluate the behaviour of surface runoff in the road carriageway under the proposed stormwater management system. The design scenario is based on the RCP8.5 climate change scenario, incorporating all proposed soakage and treatment devices and the assessment is done through Autodesk Hydraflow software. The OLFPs represents the conveyance of surface runoff as a result of the proposed system during the 100-year cc storm event.

Flow depths and velocities were assessed at key locations throughout the development covering all the various road/Accessway typologies ensuring and confirming conveyance o the OLFP is viable through proposed carriageway.

See below table showing results at the key locations and is linked to plans 430 For Residential Development

Section	Peak Flow (m^3/s)	Max Depth (m)	Max Velocity (m/s)	V x D (m^2/s)
A1	0.6	0.137	0.77	0.105
A2	1.1	0.158	0.97	0.153
A3	0.6	0.131	0.82	0.107
B1	0.9	0.167	0.81	0.135
B2	0.3	0.116	0.59	0.068
B3	0.2	0.091	0.61	0.056
B4	1.5	0.216	0.79	0.171
B5	0.6	0.121	0.912	0.11
C1	0.5	0.125	0.81	0.101
D1	0.3	0.116	0.59	0.068
D2	0.3	0.109	0.59	0.064

Table 15: OLFP Assessment Results (Residential Area)

Section	Peak Flow (m^3/s)	Max Depth (m)	Max Velocity (m/s)	V x D (m^2/s)
A	0.17	0.09	0.75	0.07
B	0.26	0.1	0.77	0.08
C	0.29	0.11	0.8	0.09
D	0.34	0.12	0.82	0.1
E	0.44	0.13	0.85	0.11
F	0.45	0.13	0.87	0.11
G	0.55	0.14	0.89	0.12
H	0.67	0.15	0.91	0.14
I	0.7	0.16	0.94	0.15
J	0.73	0.16	0.96	0.15
K	0.85	0.17	0.98	0.17
L	0.96	0.18	1.01	0.18
M	1.07	0.19	1.03	0.2
N	1.24	0.21	1.05	0.22
O	0.58	0.14	0.9	0.13

Table 16: OLFP Assessment Results (Retirement Village)

Most OLFP sections comply with standard design thresholds. However, three sections in the residential development and six sections in the retirement village recorded maximum water depths above the 150mm guideline.

Residential

Catchment A – Section 2: Max Depth = 0.158

- Catchment B – Section 1: Max Depth = 0.167

Catchment B – Section 4: Max Depth = 0.216

Retirement Village

Sections I – N: Highest Max depth is 0.21m and Lowest Max Depth: 0.16m

Despite minor exceedances in depth, depth x velocity (m2/s) values remain well below critical safety thresholds defined in Austroads 2012 Part 5, which specify.

- < 0.4m2/s pedestrian Safety
- <0.6m2/s for vehicle safety

The highest recorded value was $0.22m^2/s$ confirming safe flow conveyance for both pedestrians and vehicles under design conditions. Flow is primarily routed along proposed roads conveyed into roadside treatment and 10year cc event mitigation devices prior to spilling back (during event above the 10year cc) onto the road and get discharged into the proposed basins or Greenway.

It is noted that a separate flood sensitivity analysis has been completed using HEC-RAS 2D modelling assuming all stormwater devices are fully blocked. The assessment detailed in section 7 of this report, evaluates overland flow behaviour under worst case flooding conditions within and surrounding the site.

7.1 PROPOSED BASINS

The proposed basins located downstream of each catchment are primarily intended to capture rainfall runoff from events that exceed the 10 year cc event from the subject sites (RV and Residential). Please note that the table below presents the water levels for the 10-year cc event. This relates to upstream inflows, rather than those from the subject sites.

The table below displays the expected peak water levels for the basins during the critical duration 24-hour rainfall events for both the 10-year and 100-year periods.

SITE	BASIN	Water Level - HEC HMS		Min Platform Levels adjacent units
		10 years	100year	
RESIDENTIAL	A	64.10	66.20	66.70
	B	66.80	66.9	67.4
	C	-	65.45	65.95
	D	64.90	65.79	66.29
RETIREMENT VILLAGE	1	65.55	65.77	66.27
	2	66.41	66.82	67.32

Table 17: Stormwater Basin Water Levels

Note Min platforms levels for both the Residential and Retirement village complies with minimum freeboard requirements. This requirement is based on the 500mm freeboard referenced from both E1 Building Code and Matamata Piako district plan 11.4.

7.2 CONVEYANCE CHANNELS – EXTERNAL INFLOWS

The overall stormwater design accounts for all inflows from upstream catchments that flow through the site under pre-development conditions. As introduced in section 2.2 of this report, these inflows will require management to ensure conveyance while minimising impact to surrounding environment is maintained. To manage these flows, the proposal includes three conveyance channels positioned around the perimeter of the residential development. These channels aim to redirect upstream overland flows downstream, thereby minimizing flood risk and preventing adverse impacts on existing neighbouring properties.

The proposed channels vary in width from 3 to 4 meters. Preliminary sizing was conducted using HEC-RAS as part of the Flood RCP 8.5 sensitivity analysis. This process involved defining the channel alignments and iteratively adjusting their positions and widths to effectively accommodate flow during the design storm event while minimizing upstream impacts.

These inflows through the conveyance channel have been incorporated in the design of the greenway and the Stormwater Basins.

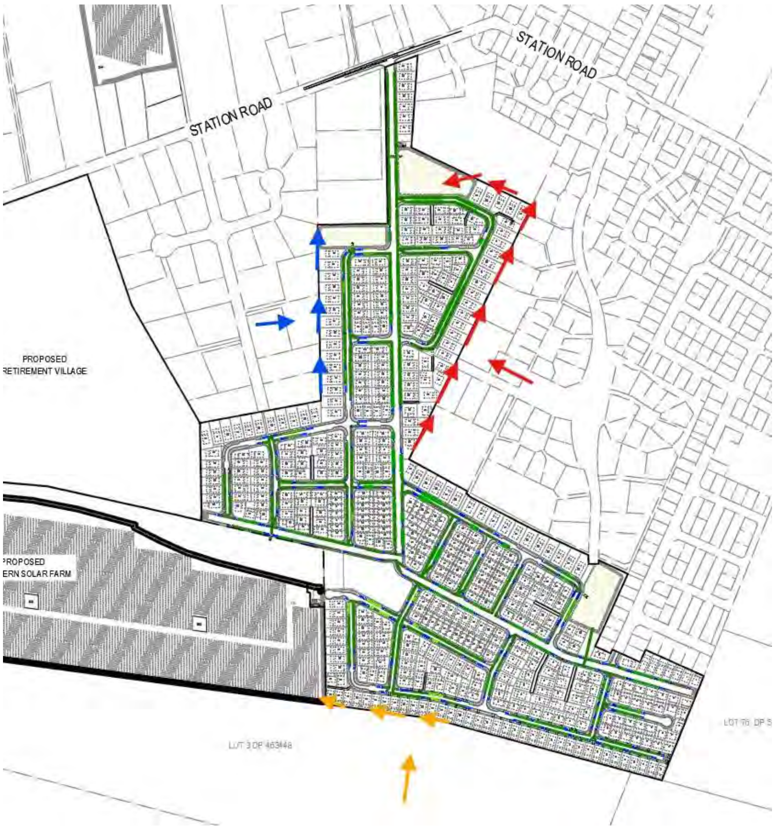


Figure 22: Greenway Cross Section (Plan 490-17)

Conveyance South of Residential Development - Orange

The proposed conveyance is required here for Runoff from the upstream portion of the post development Catchment B, located south of the residential development. This inflow will be redirected to the western edge of the residential area. From there, it will discharge into the southern solar farm zone. The redirected flow will ultimately be conveyed into the Waitoa River via the proposed greenway corridor.

Conveyance East of Residential Development – Red

Overland flow along the eastern boundary of Post Development Catchment D will be captured by this channel along the adjacent Eastern boundary. Under heavy rainfall, northern area of Basin D will pond within the neighbouring property, similar to existing pre-development conditions. Once ponding reaches elevation of RL 65.60, channel will overflow into Basin D. The basin also receives runoff from Catchment D (100-10yr cc event) and is designed with a soakage base, similar to Basins A and C, allowing for further storage for all inflows into Basin D.

Conveyance West of Residential Catchment C - Blue

A Conveyance channel is proposed along the western boundary of Post Development Catchment C to accommodate the natural overland flow from upstream areas. Under existing conditions, this flow would pass through the development area before entering the northern natural stream. The proposed channel will convey this existing flow downstream into the existing channel. This approach ensures that upstream flows are managed efficiently, preventing backwater effects due to infilling of the downstream areas.

7.3 DESIGN FLOW RESULTS – EXISTING DISCHARGE POINTS

SITE	Catchment (See Plan C-400 Series)	Pre-Development Flow: 10-year flow (m^3/s)	Pre-Development Flow: 100-year flow (m^3/s)	Post Development Flow: 10-year flow (m^3/s)	Post Development Flow: 100-year flow (m^3/s)
RESIDENTIAL	A1	0.18	0.37	0	0
	A2	0.47	0.96	0	0
	A3	0.04	0.09	0	0
	A4	0.21	0.43	0	0
	A5	0.04	0.08	0	0
	RES-CATCH B Discharge through Greenway			0.85	1.72
	A6	0.52	1.06	0	0
	A7	1.64	2.97	0	0
	A8	0.05	0.1	0	0
SOUTH SOLAR	A9	4.22	8.63	2.8	5.75
RETIREMENT VILLAGE	A10	0.55	0.81	0	0.63
	A11	0.03	0.07	0.03	0.07
	A12	0.73	1.78	0	1.33
NORTH SOLAR	A13	0.05	0.11	0.05	0.11
	A14	0.15	0.28	0.15	0.28
	A15	0.83	1.71	0.83	1.71
	A16	0.05	0.11	0.05	0.11

Table 18: HEC HMS discharge Table

The flow comparison table above presents the pre- and post-development peak flows across the various catchments within the Ashbourne development. Pre-development flows show a natural distribution of runoff across the site with majority discharging due north of the sites and into the Waitoa River.

In the post-development scenario, many of these flows are reduced to zero, particularly within the Residential and Retirement Village areas. This is due to the proposed use of basins with integrated soakage components, both within the basins themselves and in upstream areas, designed to retain stormwater on-site. Meanwhile, some catchments still show controlled discharge where infiltration is not feasible.

This pattern reflects a conservative design approach and highlights opportunities for further optimisation in the detailed design phase to improve land use efficiency while maintaining regulatory compliance.

8 POST DEVELOPMENT FLOODING – SENSITIVITY ANALYSIS

This section presents a sensitivity analysis carried out using HE RAS 2D for the 100year cc event which assumes all soakage and pond systems are fully blocked. This assessment builds on the Section 7 results but provides a more conservative view of surface flooding behaviour across the sites.

Objective of this analysis is to observe whether the overland flow paths can convey runoff when the mentioned key stormwater components are fully blocked, and how this will impact the development and the surrounding environment.

The map results shown below shows extent of the 100year cc flood event within and neighbouring site.

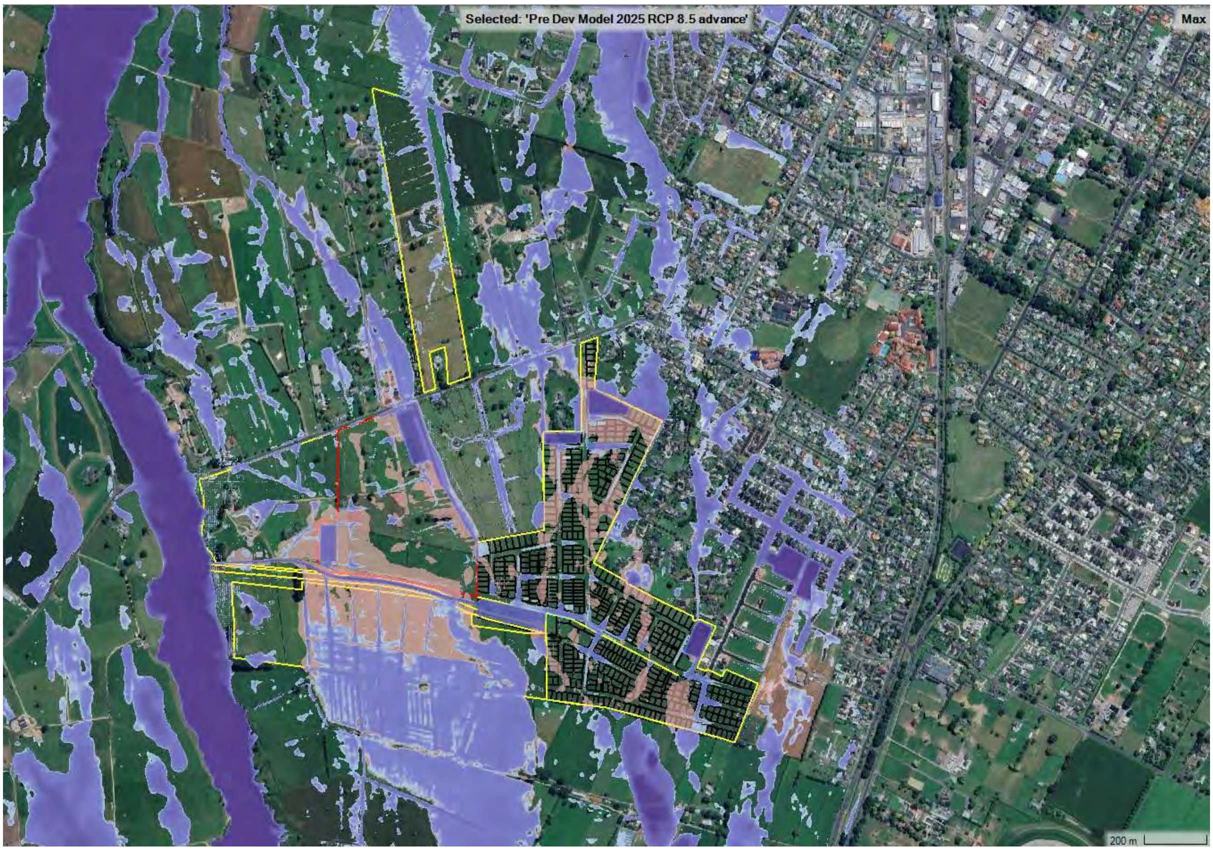


Figure 23: Overall Pre & Post Development Flood Map RCP 8.50 - Sensitivity

HEC RAS Catchment Flows (100 YEAR cc)m³/s				
SITE	**Catchment**	**Pre Development**	**HEC RAS Section**	**Post Development**
RESIDENTIAL	A1	0.85	RES SEC 11	0.2
	A2	0.96	RES SEC 10	0.96
	A3	2.05	RES SEC 09	0.05
	A4 & A5	0.85	RES SEC 03	0.08
	A6	2.3	RES SEC 06	0.3
	A7	2.6	RES SEC 04	1.2
	A8	0.13	RES SEC 12	0.05
SOUTH SOLAR	A9	1.7	GRNWAY SEC 01	9.8
RETIREMENT VILLAGE	A10		GRNWAY SEC 01	
	A11	0.84	RV SEC 01	0.9
	A12	2.99	RV SEC 02	1.25
NORTH SOLAR	A13	1.6	S.North SEC 01	1.5
	A14	0.9	S.North SEC 02	0.95
	A15		S.North SEC 01	
	A16	6.13	S.North SEC 03	6.1

Table 19: HEC RAS Sensitivity discharge Table

Referring to the table above, although the overall post-development scenario indicates reduced discharge at most locations, a few key areas require additional context.

For the Retirement Village, the increase in flow at the downstream section (RV Section 1) is attributed to preliminary surface levels, for small areas along the fringes of the RV development draining into the existing Road Swales on Station Road. These levels will be refined during the detailed design stage to ensure that all overland flows are directed appropriately into the proposed ponds, eliminating unintended bypass or overflow conditions.

The flood mapping will be refined during detailed design and fretboard levels set above the design 100year flood level scenario.

8.1 Greenway and Waitoa River Interface.

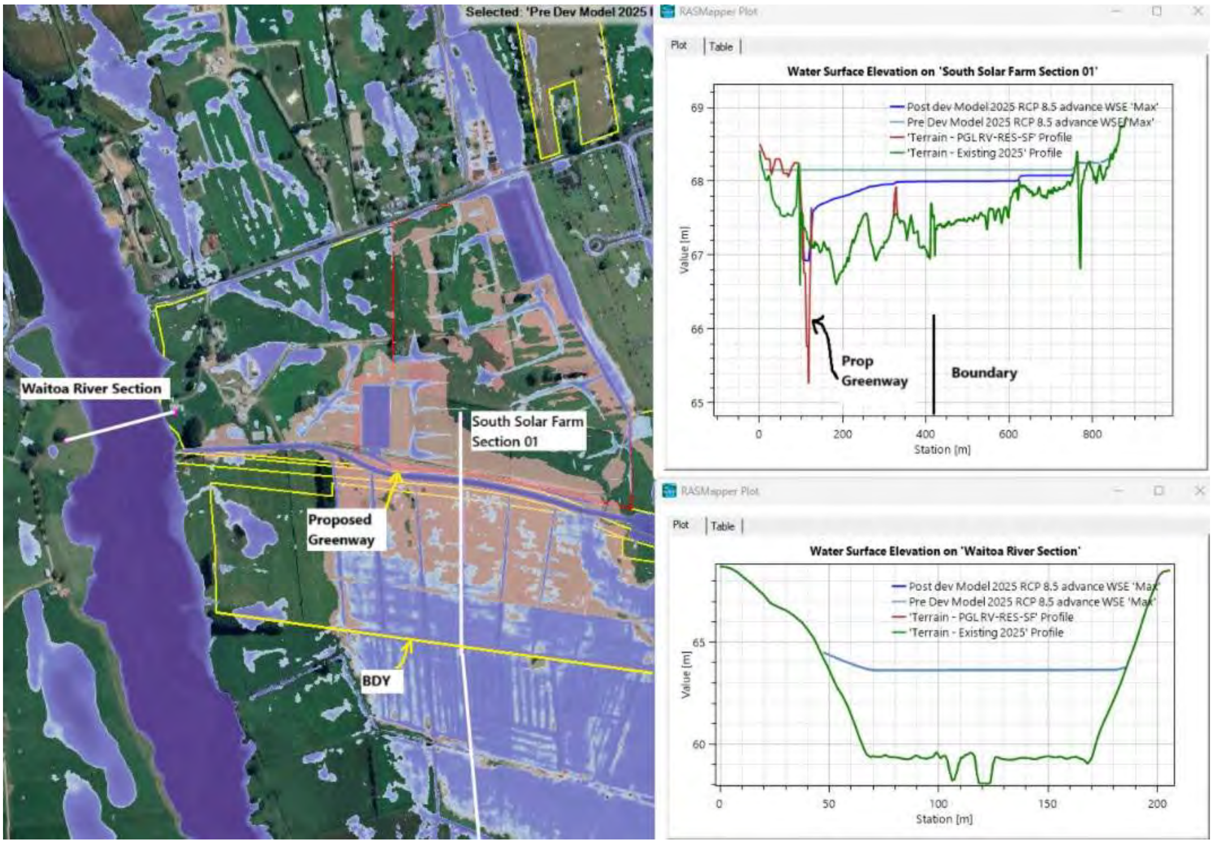


Figure 24: Overall Pre &Post Development Flood Map RCP 8.50 - Sensitivity

The proposed greenway plays a crucial role in the overall stormwater strategy for the development. One of its primary functions is to receive and attenuate flows from Catchment B of the residential area before discharging them into the Waitoa River. Another function is to convey flow from the south and north of the greenway. Refer to section 6.3, which provides more information the proposed greenway/Basin B and its function.

Under pre-development conditions, significant surface ponding was observed in the model within and upstream of the southern solar farm area, as seen in the results screenshot (blue—post dev & pink—pre dev). With the introduction of the greenway, this water is now conveyed more effectively downstream, resulting in an 80–90 mm reduction in flood depths in those upstream areas. This improvement supports the project's intent to alleviate localized flooding and enhance overland flow management.

However, the greenway's increased conveyance capability results in a higher runoff directed to the Waitoa River. This has led to a slight increase in the flood level, showing a 20 mm increase in water surface elevation observed in the flood sensitivity assessment. While this increase is acknowledged, it is considered minor and is outweighed by the substantial upstream benefits and overall reduction in localized flooding across the site.

8.2 North Of Basin D

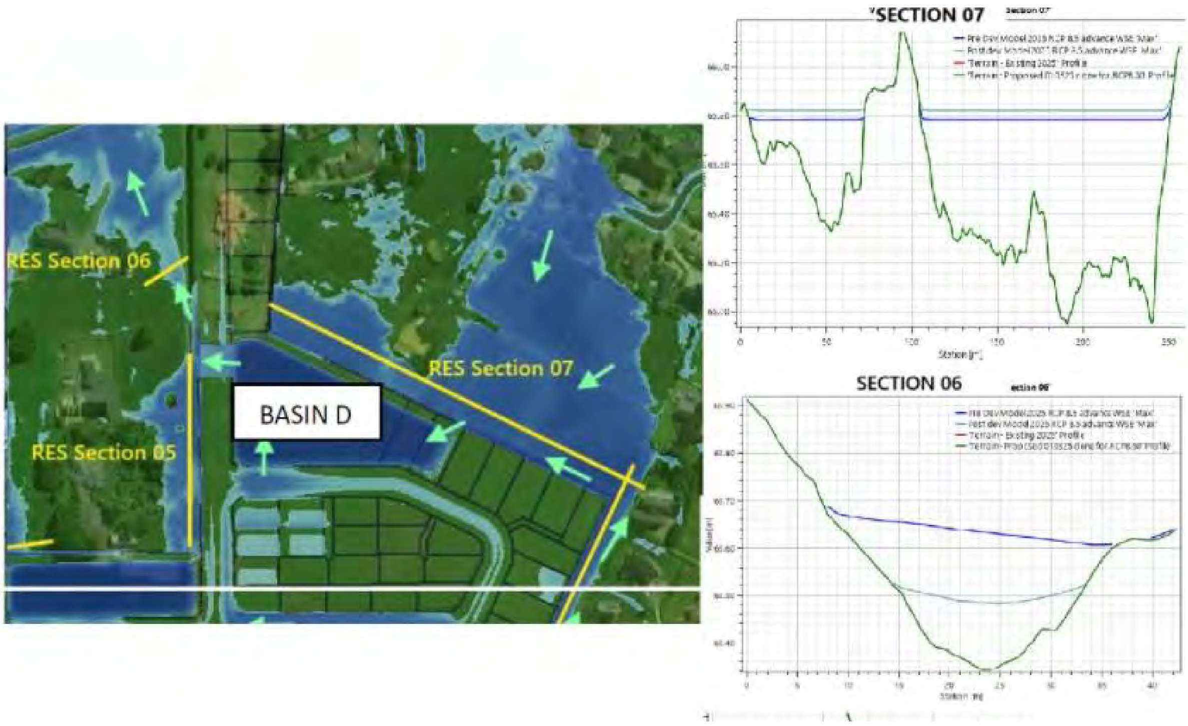


Figure 25: Basin D - RES Flood Map RCP 8.50 – Sensitivity

Basin D receives runoff from Catchment D and inflows from the northern and eastern areas upstream of Basin D, as referred to in the C410 catchment plan. As section 7.2 explains, this inflow is conveyed along the east boundary through the proposed convergence channel. Initially, the inflow will pond to the north of Basin D, similar to the pre-existing scenario. Over time, during the 24-hour event, it will eventually overtop and spill into Basin D. Once Basin D reaches its storage capacity, it will overflow westward onto the adjacent road corridor, before discharging into the neighbouring environment via the existing OLFP route.

The RES Section 07 has been placed to assess the development's impact on the neighbouring areas due to the rerouting of this inflow, as described above. The results indicate a slight increase in water depth ranging from 30 to 40 mm adjacent to the eastern boundary of Basin D, where it ponds before spilling into the Basin.

In contrast, RES Section 06 is located at the westward discharge point from the pond to assess the development's impact downstream. The results indicate a just over 100 mm reduction in post-development flood levels. This reduction underscores the effectiveness of the basin design in attenuating flows and, consequently, reducing flood risk in downstream areas.

9 SUMMARY OF PROPOSED STORMWATER MANAGEMENT

Stormwater management solutions for each post development sub-catchment is summarised in Table 20.

Table 20: Summary of stormwater management

Developments	Source	Water Quality	Flood management	Stream Protection	Water Sensitive Design
Residential Sub catchment-A, B, C &D	Lot Areas	Inert roof materials. Soakage Trench.	roadside raingardens and soakage trenches for 10yr ARI cc rainfall events. Catchment A, C & D; Overland flow paths for 100yr ARI cc events within road reserves, directed to detention basins. Catchment B: Overland flow paths for 100yr ARI cc events within road reserves, directed to detention basins and Greenway.	At source on lot soakage trench for: Full soakage/Mitigation of up to 10year cc event	**Stormwater detention basins with planting.** **75% TSS as per requirements of GD01/TR2018/01 for all impervious surfaces.** **Protection of streams and riparian planting.**
	Public Roads	At source Raingardens, soakage trenches and stormwater basins 75% TSS as per requirements of GD01/ TR2018/01 for all impervious surfaces.		Raingardens, soakage trench and dry basins	
Retirement Village	Lot Areas	Inert roof materials. Soakage Trench.	Catchment wide dry Basin the 10yr cc and 100yr ARI cc. roadside raingardens and soakage trenches for 10yr ARI cc rainfall events. Use of rain smart (Cirtex) or similar to capture the 10year event cc.	At source on lot soakage trench for: Full soakage/Mitigation of up to 10year cc event	
	Public Roads	**At source Raingardens, soakage trenches and stormwater basins** 75% TSS as per requirements of GD01/ TR2018/01 for all impervious surfaces.		**Raingardens, soakage trench and dry basins**	

Notes:

1. *On site refers to at source, e.g., adjacent to the road carriageway or within the lot boundary.*
2. *Catchment wide refers to outside of the road carriageway or lot boundary however within the Ashbourne Development Areas*

10 DESIGN OPTIMISATION

As part of the ongoing refinement of the stormwater management system, two key optimization strategies can be explored during the next stage of the project. These aim to maximize land availability further, improve hydraulic performance, and reduce the infrastructure footprint where appropriate:

- Reduction of Stormwater Pond Size via Controlled Discharge: The current post-development model assumes zero discharge from several catchments, which is conservative and beneficial from a flood mitigation perspective. This means the sizes of the proposed basins can be further optimized. To achieve this, we will investigate discharging post-development flows at up to 80% of the pre-development flow rate, which aligns with council standards and requirements. This approach balances flood control with land efficiency and has the potential to reduce pond size, thereby creating more developable land for additional units where possible.

- Incorporation of Base Flow Discharge to Existing Streams: To better mimic the natural hydrological conditions, base flow discharge will be reintroduced where feasible. This can be achieved by quantifying the existing base flow and assigning selected units to discharge directly into the stream at this defined rate and quantity. This strategy improves ecological outcomes and further reduces onsite storage demand, particularly for minor and frequent rainfall events.

These optimization works will be further assessed through refined modelling and consultation with the council and external consultants during the further design stage to ensure compliance with council requirements while maximizing development potential.

11 DEPARTURES FROM STANDARD

A departure from standard stormwater design has been identified in a few Overland Flow Path (OLFP) sections where the flow depth exceeds the recommended threshold of 150 mm according to RITS. These exceedances are minor and occur in isolated sections of the residential OLFP network.

Sections exceeding the 150mm limit.

Residential

- Catchment A – Section 2: Max Depth = 0.158
- Catchment B – Section 1: Max Depth = 0.167

Catchment B – Section 4: Max Depth = 0.216
Retirement Village

- Sections I – N: Highest Max depth is 0.21m and Lowest Max Depth: 0.16m

These areas remain safely contained within their respective flow corridors and do not pose a risk to vehicles and pedestrians as VD values remain well below the threshold. Further refinement of levels, swale profiles, or berms at the detailed design stage is expected to reduce these values and bring all OLFP sections fully into compliance.

12 CONCLUSIONS

This Stormwater Management Plan (SMP) has been prepared to support a discharge consent application for the proposed Ashbourne development, which comprises four key projects:

- the Residential Development
- Northern and Southern Solar Farms
- the Retirement Village.

Each site has been considered in detail through hydrological and hydraulic modelling, including sensitivity scenarios under future climate conditions.

The proposed stormwater management system has been designed to Provide

- full soakage and treatment for up to the 10- year ARI cc storm event through a combination of on- lot soakage devices, raingardens, and road soakage systems.
- Conveyance of overflows above the 10- year cc event into dry detention basins and a central greenway corridor sized for up to the 100- year cc storm event. Limit post- development peak discharges to no more than 80% of pre- development flows, where applicable.
- Preserve existing flow paths and discharge locations to avoid introducing new hydrological impacts.
- Integrate water- sensitive design elements, including green corridors, baseflow retention, and modular soakage systems such as Cirtex Rain Smart
- Account for external inflows and upstream catchments to ensure downstream impacts are not worsened under post- development conditions
- Support future maintenance, accessibility, and ecological enhancement through features such as maintenance tracks and riparian planting zones.

A flood model has been developed and calibrated using the region's observed rainfall and river data. The model has been used to test a sensitivity scenario in which all primary stormwater devices are blocked. Even under this worst-case condition, the model demonstrates that the development maintains flood immunity, with only minor exceedances expected to be mitigated through detailed design refinements.

This SMP aligns with the requirements of both the Matamata- Piako District Council and Waikato Regional Council and has been developed in accordance with relevant planning instruments, including MPDC Section 5 Infrastructure Standards and the Waikato Comprehensive Network Discharge Consent (#105063).

Several design aspects of the proposed Stormwater will require further refinement during detailed design stage to ensure compliance and for further value engineering covered under design optimisation section of this SMP.

In conclusion, the proposed stormwater management approach is robust, resilient, and environmentally sensitive. It provides a strong foundation for the ongoing development of the Ashbourne site and offers a balanced integration of flood mitigation, water quality treatment, ecological protection, and long-term sustainability.

13 APPENDICES

Appendix A – Engineering Plans - Bounded with Infrastructure reports

APPENDIX B – STORMWATER CALCULATIONS AND RESULTS

Maven Associates Ltd.	Job Number 289001	Sheet 1	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B	Open Space (Sandy Loam or Silty Loam)	61	10.10	616.10
* from Appendix B		Totals =	10.10	616.10

WQV

CN (weighted) = total product / total area = 616.10 / 10.10 = 61.0

Ia (weighted) = 5 x pervious area / total area = 5 x 10.10 / 10.10 = 5.0 mm

2. Time of Concentration

Channelisation factor C = 1 (From Table 4.2) natural channels

Catchment length L = 0.39 km (along drainage path)

Catchment Slope Sc= 0.005 m/m (by equal area method)

Runoff factor, CN / (200 - CN) = 61.0 / (200- 61.0) = 0.44

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0 1 0.54 1.57 4.90 = 0.580 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.388 hrs

OK
use
0.39 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Cover description — Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group — A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd.	Job Number 289001	Sheet 2	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

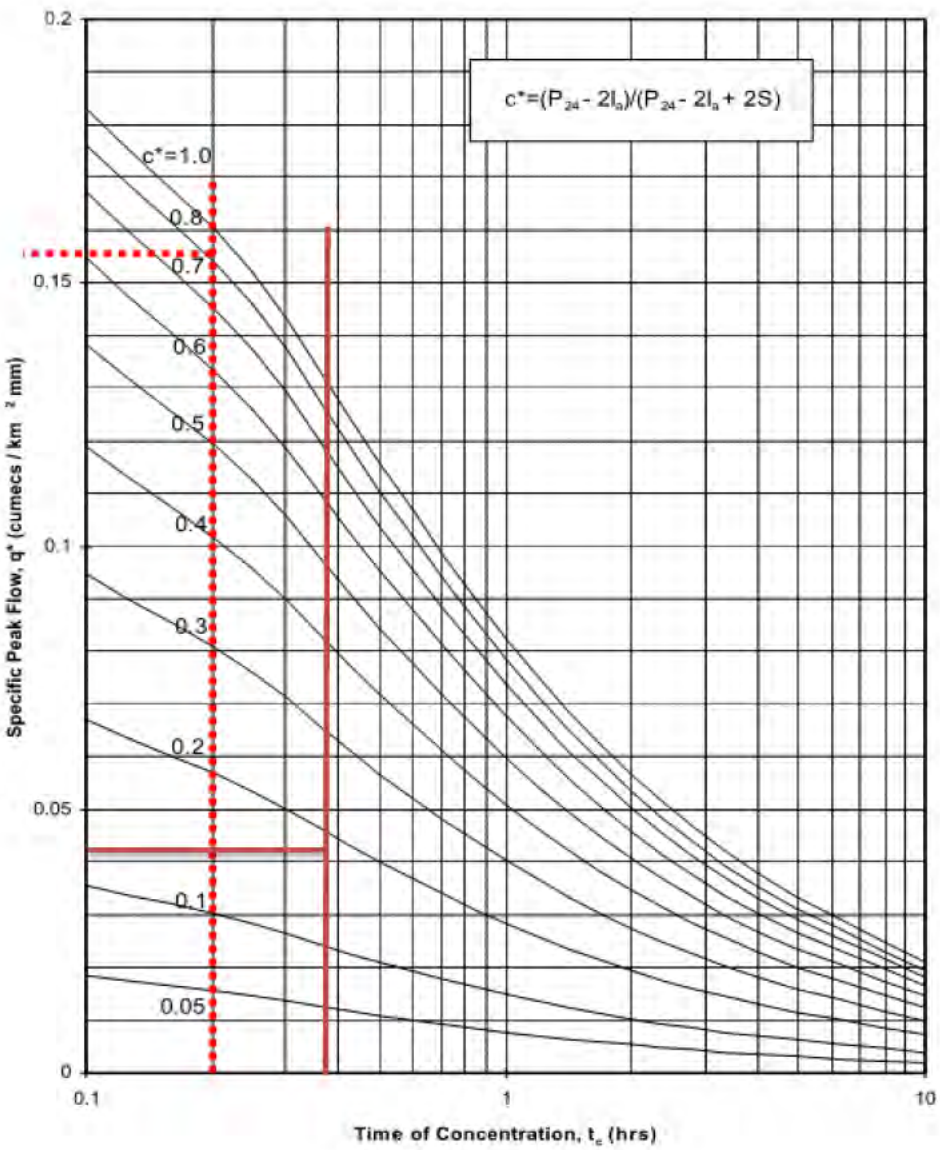
Catchment Area	A=	0.10100	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.39	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	2	(yr)
4. 24 hour rainfall depth, P24	83	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.184	
6. Specific peak flow rate q*	0.042	
7. Peak flow rate, q_p=q*A*P_{24}	**0.352**	(m3/s)
8. Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	25.3	
9. Runoff volume, V_{24} = 1000xQ_{24}A	2556.16	(m3)

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km ² mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 3	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

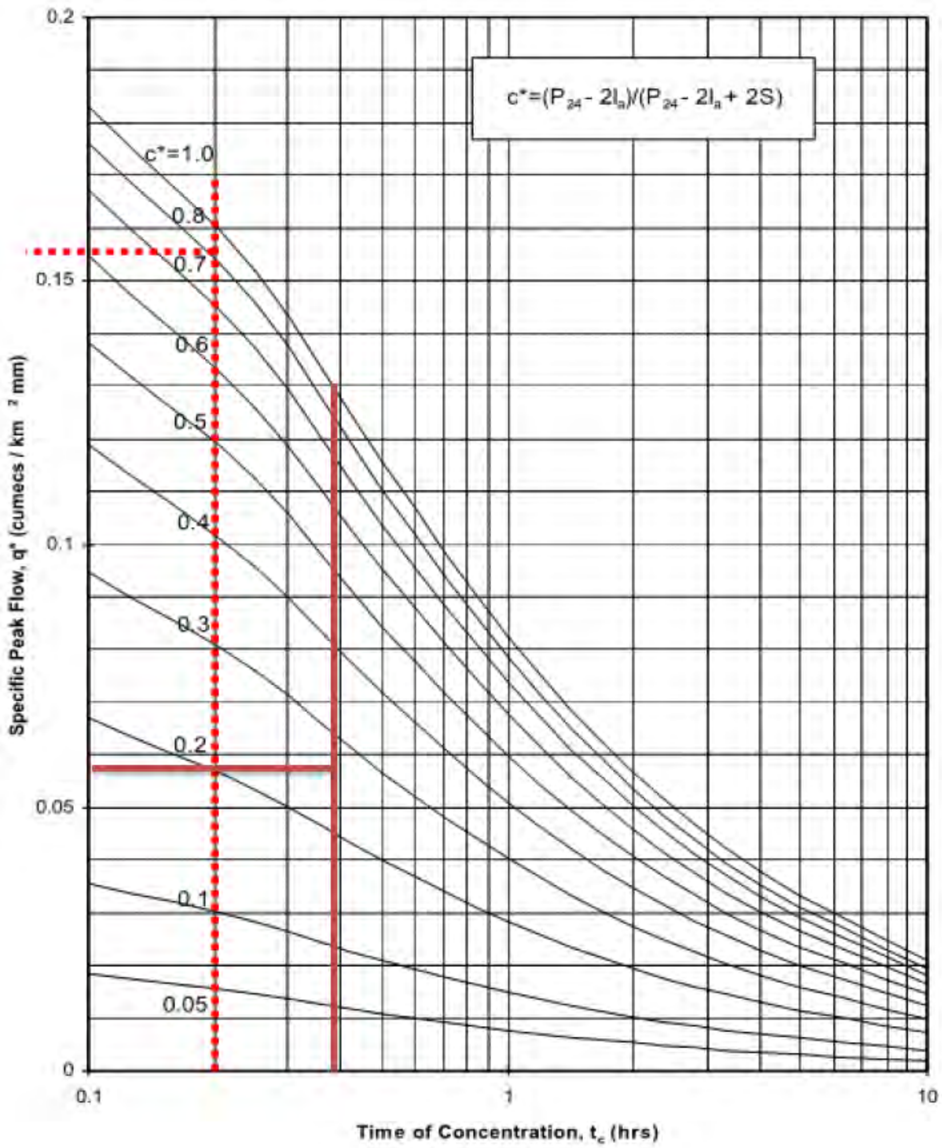
Catchment Area	A=	0.10100	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.39	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	10	(yr)
4. 24 hour rainfall depth, P24	128	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.266	
6. Specific peak flow rate q*	0.058	
7. Peak flow rate, q_p=q*A*P_{24}	**0.750**	(m3/s)
8. Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	53.0	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	5354.11	(m3)

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 4	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

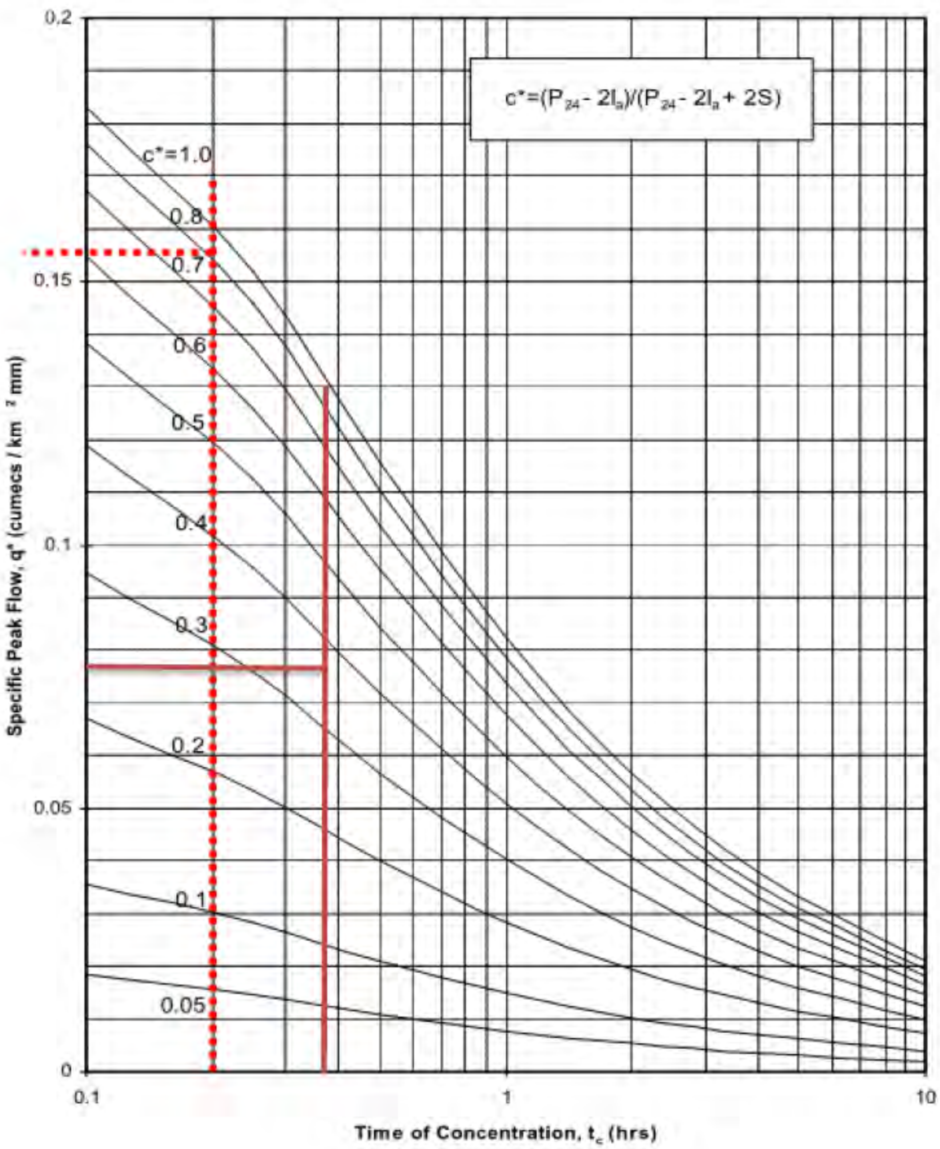
1. Data

Catchment Area	A=	0.10100	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.39	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

3. Average recurrence interval, ARI	100	(yr)	NO CLIMATE CHANGE
4. 24 hour rainfall depth, P24	200	(mm)	
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.369		
6. Specific peak flow rate q*	0.078		
7. Peak flow rate, q_p=q*A*P_{24}	**1.576**	(m3/s)	
8. Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	106.4		
9. Runoff volume, $V_{24} = 1000\text{x}Q_{24}A$	10745.93	(m3)	

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 5	Rev A
Job Title / Calc Title — Eldonwood, Matamata / Post-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road	98	2.76	270.5
C	Residential District (30% PERVIOUS)	74	2.2	163.5
C	Residential District (70% IMPERVIOUS)	90	5.13	461.7
* from Appendix B		Totals =	10.10	895.7

CN (weighted) = total product / total area = 895.72 / 10.100 = 88.7

Ia (weighted) = 5 x pervious area / total area = 5 x 2.21 / 10.10 = 1.1 mm

2. Time of Concentration

Channelisation factor C = 0.6 (From Table 4.2) piped

Catchment length L = 0.39 km (along drainage path)

Catchment Slope Sc= 0.01 m/m (by equal area method)

Runoff factor, CN / (200 - CN) = 88.7 / (200- 88.7) = 0.80

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0.1 0.6 0.54 1.13 3.98 = 0.204 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.136 hrs

NO GOOD
use
0.170 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

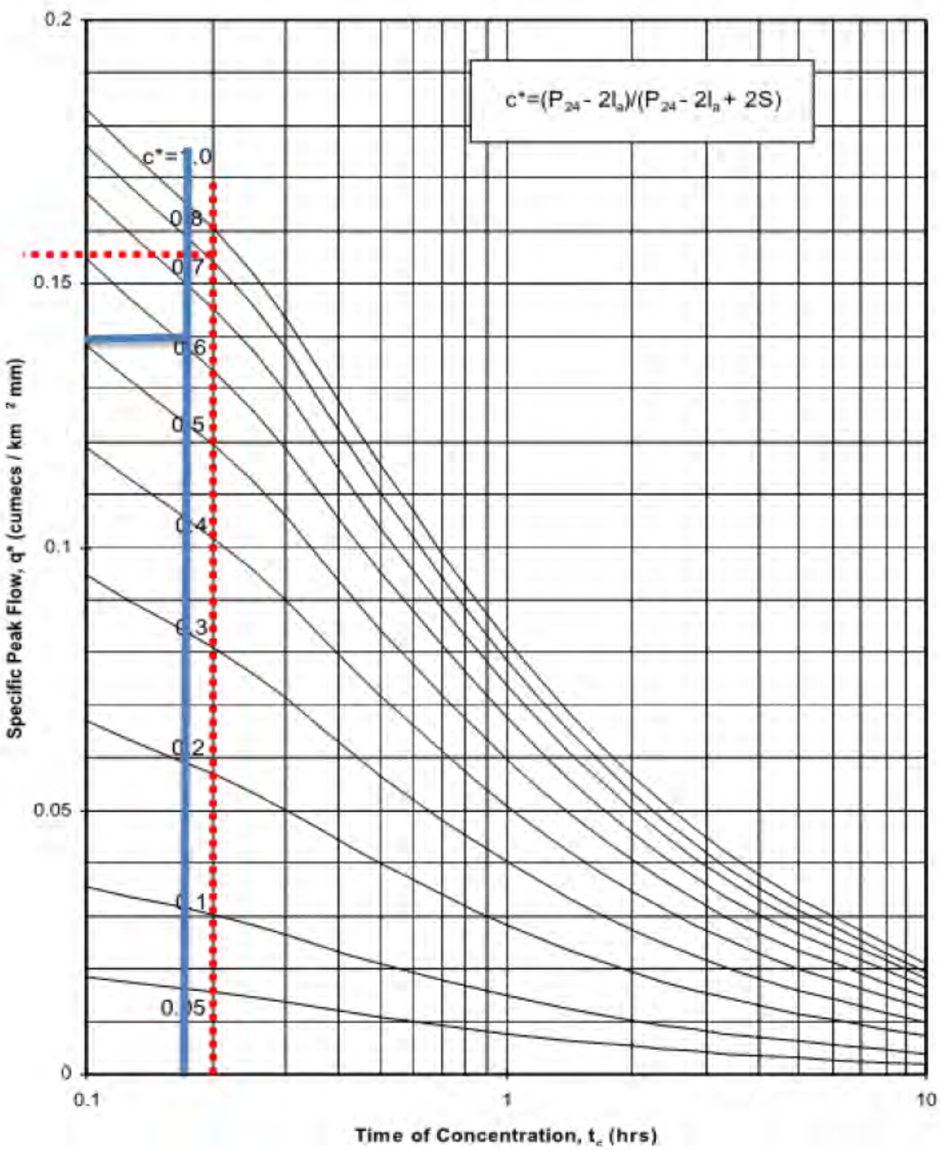
Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd	Job Number 289001	Sheet 6	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1.	Data				
	Catchment Area	A=	0.101	km2(100ha =1km2)	
	Runoff curve number	CN=	88.7	(from worksheet 1)	
	Initial abstraction	Ia=	1.1	mm (from worksheet 1)	
	Time of concentration	tc=	0.170	hrs (from worksheet 1)	
2.	Calculate storage, S =(1000/CN - 10)25.4	=	32	mm	

				HEC-HMS Check	
3.	Average recurrence interval, ARI	2	(yr)		
4.	24 hour rainfall depth, P24	106	(mm)		
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.616			
6.	Specific peak flow rate q*	0.140			
7.	Peak flow rate, q_p=q*A*P_{24}	**1.499**			Pre-Dev
8.	Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	80.1			
9.	Runoff volume, $V_{24} = 1000xQ_{24}A$	8094.91	(m3)		
	Pre development run off volume	**2556.16**	(m3)		
	Post development run off volume	**8094.91**	(m3)		
	Pre development flow rate	**0.35**	(m3/s)		
	Post development flow rate	**1.50**	(m3/s)		

Worksheet 2: Graphical Peak Flow Rate

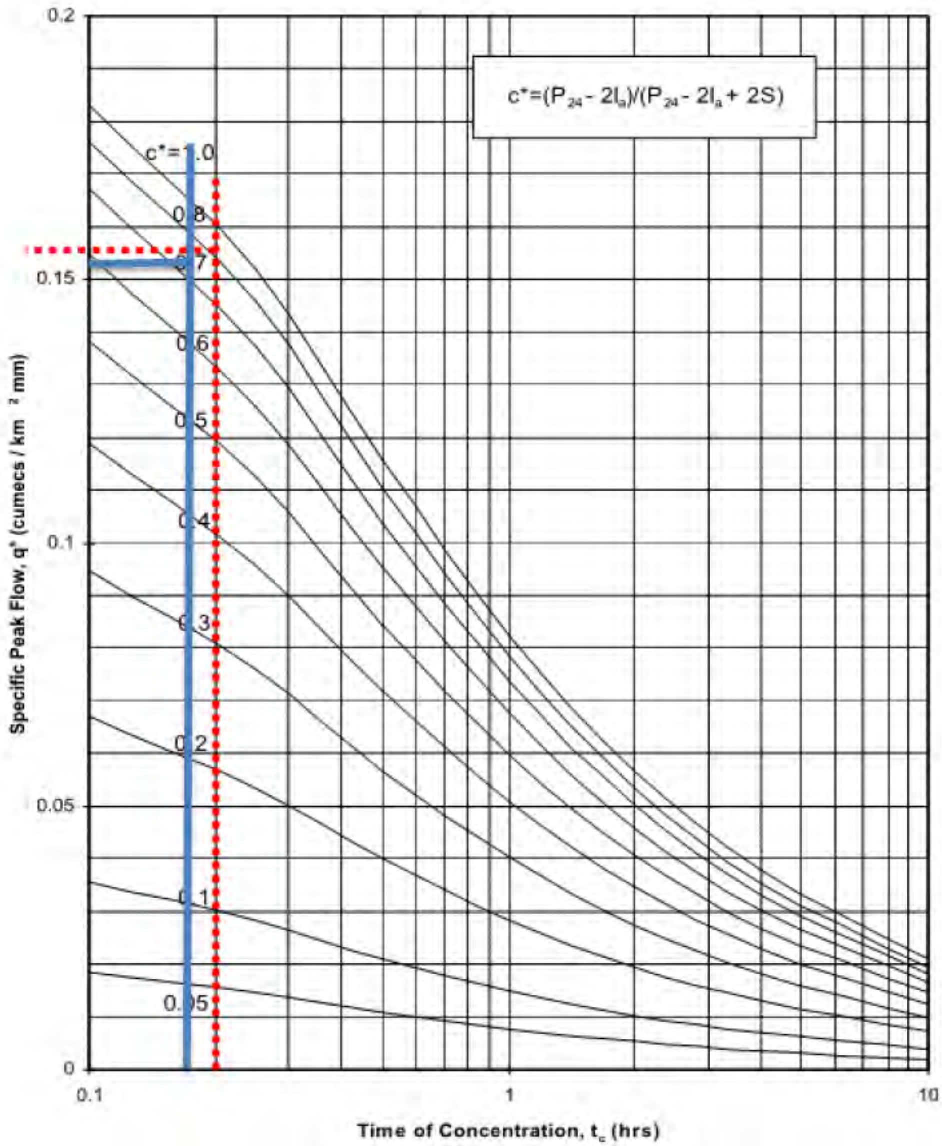


c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)

1.	Data			
	Catchment Area	A=	0.101	km2(100ha =1km2)
	Runoff curve number	CN=	88.7	(from worksheet 1)
	Initial abstraction	Ia=	1.1	mm (from worksheet 1)
	Time of concentration	tc=	0.170	hrs (from worksheet 1)
2.	Calculate storage, S =(1000/CN - 10)25.4	=	32	mm

3.	Average recurrence interval, ARI	10	(yr)	
4.	24 hour rainfall depth, P24	167	(mm)	
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.718		
6.	Specific peak flow rate q*	0.153		***HEC-HMS Check***
7.	Peak flow rate, q_p=q*A*P_{24}	**2.581**		Pre-Dev
8.	Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	138.8		
9.	Runoff volume, $V_{24} = 1000xQ_{24}A$	14018.30	(m3)	
	Pre development run off volume	**5354.11**	(m3)	
	Post development run off volume	**14018.30**	(m3)	
	Pre development flow rate	**0.75**	(m3/s)	
	Post development flow rate	**2.58**	(m3/s)	

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd	Job Number 289001	Sheet 8	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

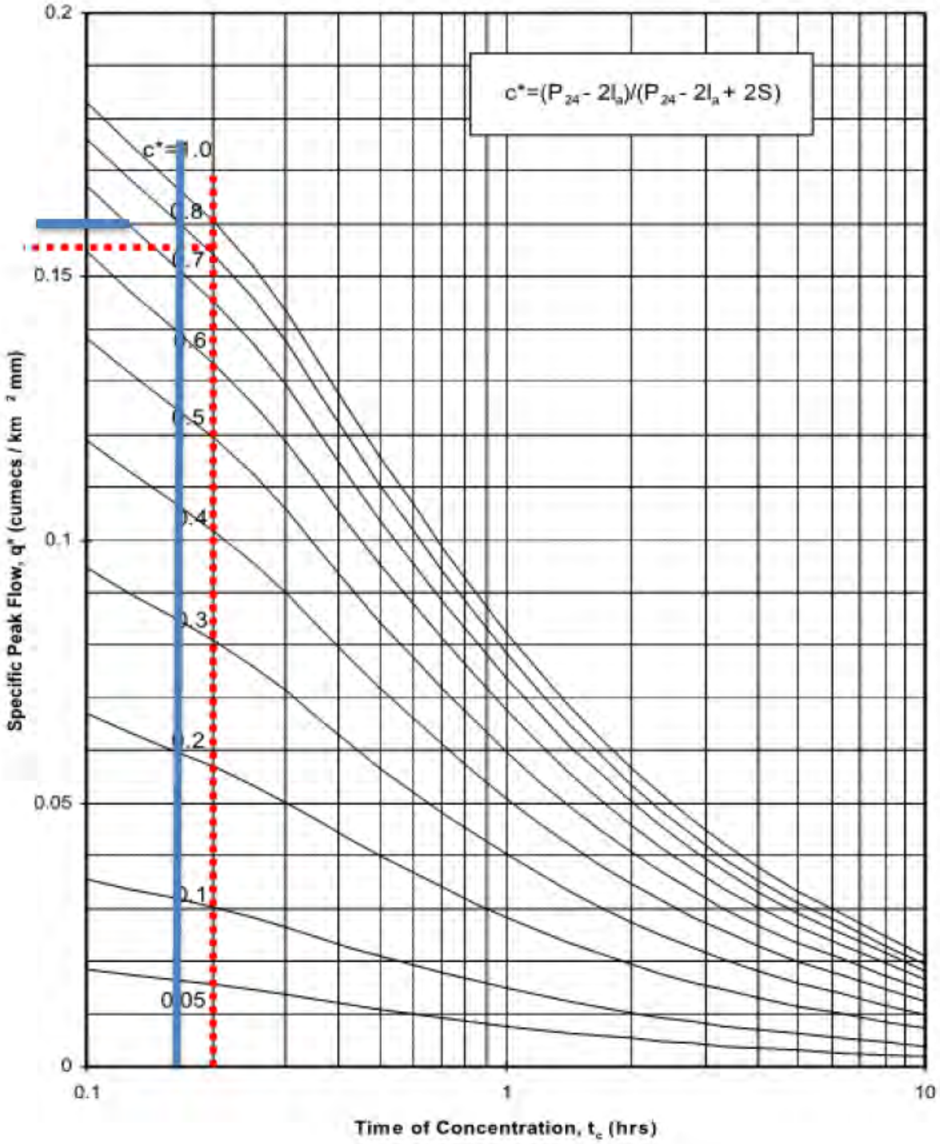
1. Data

Catchment Area	A=	0.101	km2(100ha =1km2)
Runoff curve number	CN=	88.7	(from worksheet 1)
Initial abstraction	Ia=	1.1	mm (from worksheet 1)
Time of concentration	tc=	0.170	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 32 mm

				HEC-HMS Check	
3.	Average recurrence interval, ARI	100	(yr)		
4.	24 hour rainfall depth, P24	265	(mm)		
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.802			
6.	Specific peak flow rate q*	0.160		*HEC-HMS Check*	
7.	Peak flow rate, q_p=q*A*P_{24}	**4.282**		**1.260**	80% Pre
8.	Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	235.0			
9.	Runoff volume, $V_{24} = 1000xQ_{24}A$	23739.41	(m3)		
	Pre development run off volume	**10745.93**	(m3)		
	Post development run off volume	**23739.41**	(m3)		
	Pre development flow rate	**1.58**	(m3/s)		
	Post development flow rate	**4.28**	(m3/s)		
	100yr - 10yr post development	**9721.11**	(m3)		

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 1	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B	Open Space (Sandy Loam or Silty Loam)	61	19.50	1189.50
* from Appendix B		Totals =	19.50	1189.50

WQV

CN (weighted) = total product / total area = 1189.50 / 19.50 = 61.0

Ia (weighted) = 5 x pervious area / total area = 5 x 19.50 / 19.50 = 5.0 mm

2. Time of Concentration

Channelisation factor C = 1 (From Table 4.2) natural channels

Catchment length L = 0.6 km (along drainage path)

Catchment Slope Sc= 0.005 m/m (by equal area method)

Runoff factor, CN / (200 - CN) = 61.0 / (200- 61.0) = 0.44

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0 1 0.71 1.57 4.90 = 0.770 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.516 hrs

OK
use
0.52 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd.	Job Number 289001	Sheet 2	Rev A
Job Title Station Road, Matamata Calc Title Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

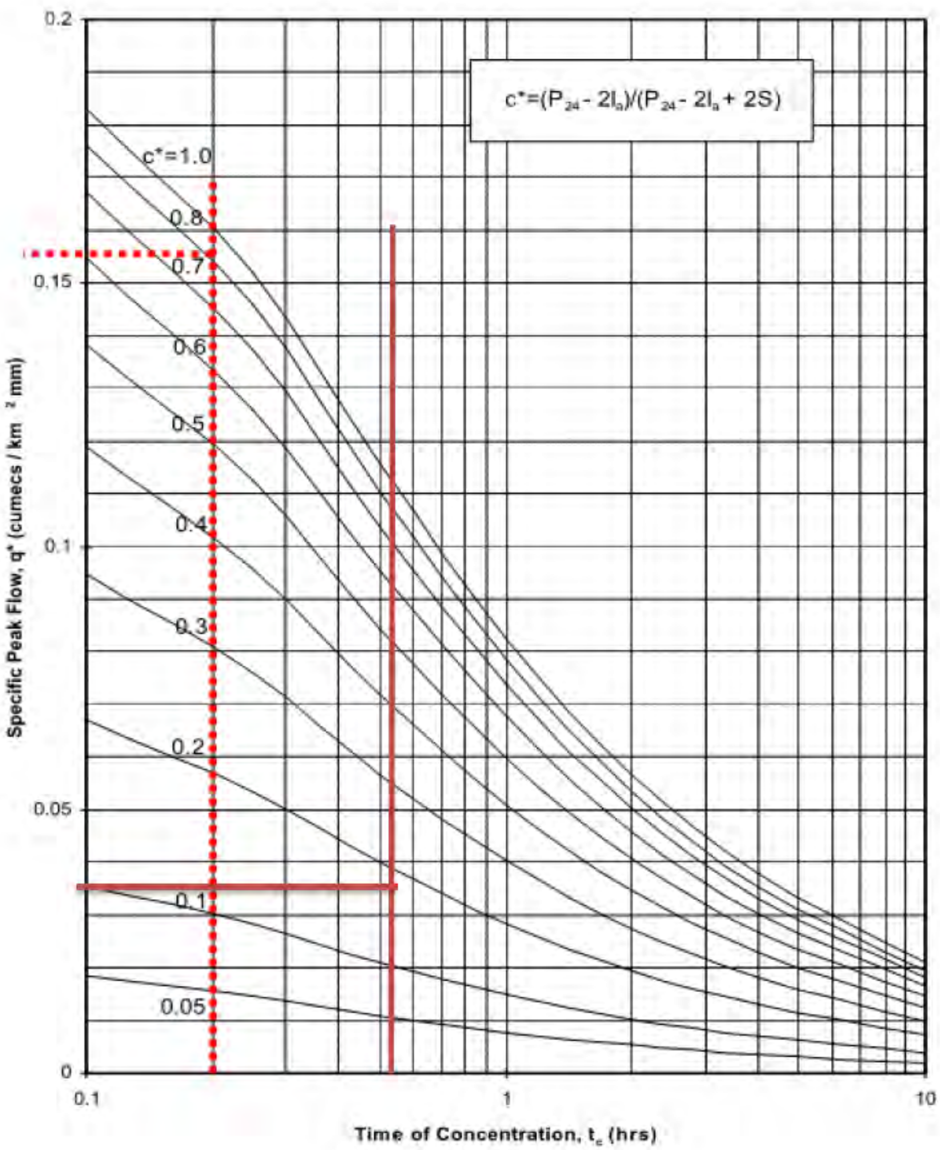
Catchment Area	A=	0.19500	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.52	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	2	(yr)
4. 24 hour rainfall depth, P24	83	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.184	
6. Specific peak flow rate q*	0.036	
7. Peak flow rate, q_p=q*A*P_{24}	**0.583**	(m3/s)
8. Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	25.3	
9. Runoff volume, V_{24} = 1000xQ_{24}A	4935.16	(m3)

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 3	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

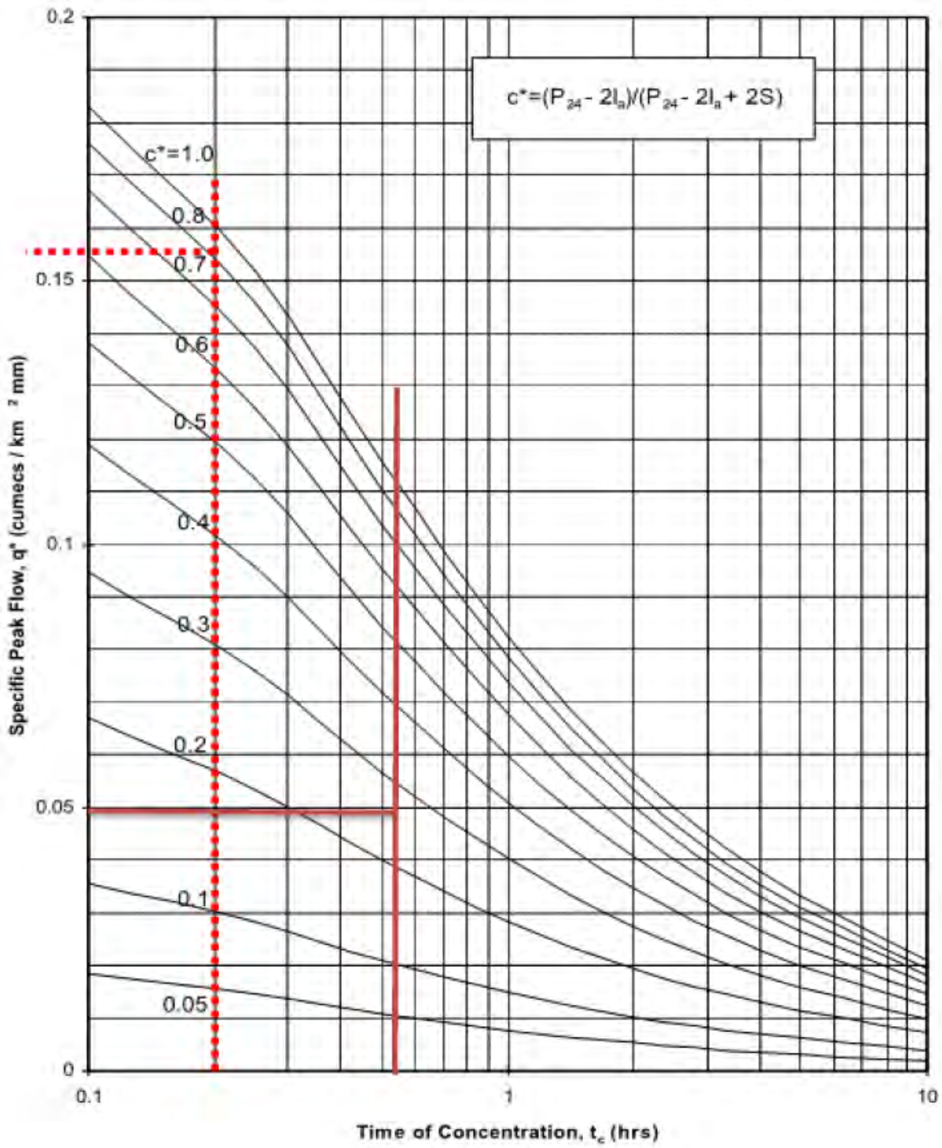
Catchment Area	A=	0.19500	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.52	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	10	(yr)
4. 24 hour rainfall depth, P24	128	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.266	
6. Specific peak flow rate q*	0.049	
7. Peak flow rate, q_p=q*A*P_{24}	**1.223**	(m3/s)
8. Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})$+S	53.0	
9. Runoff volume, V_{24} = 1000xQ_{24}A	10337.15	(m3)

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km ² mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 4	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

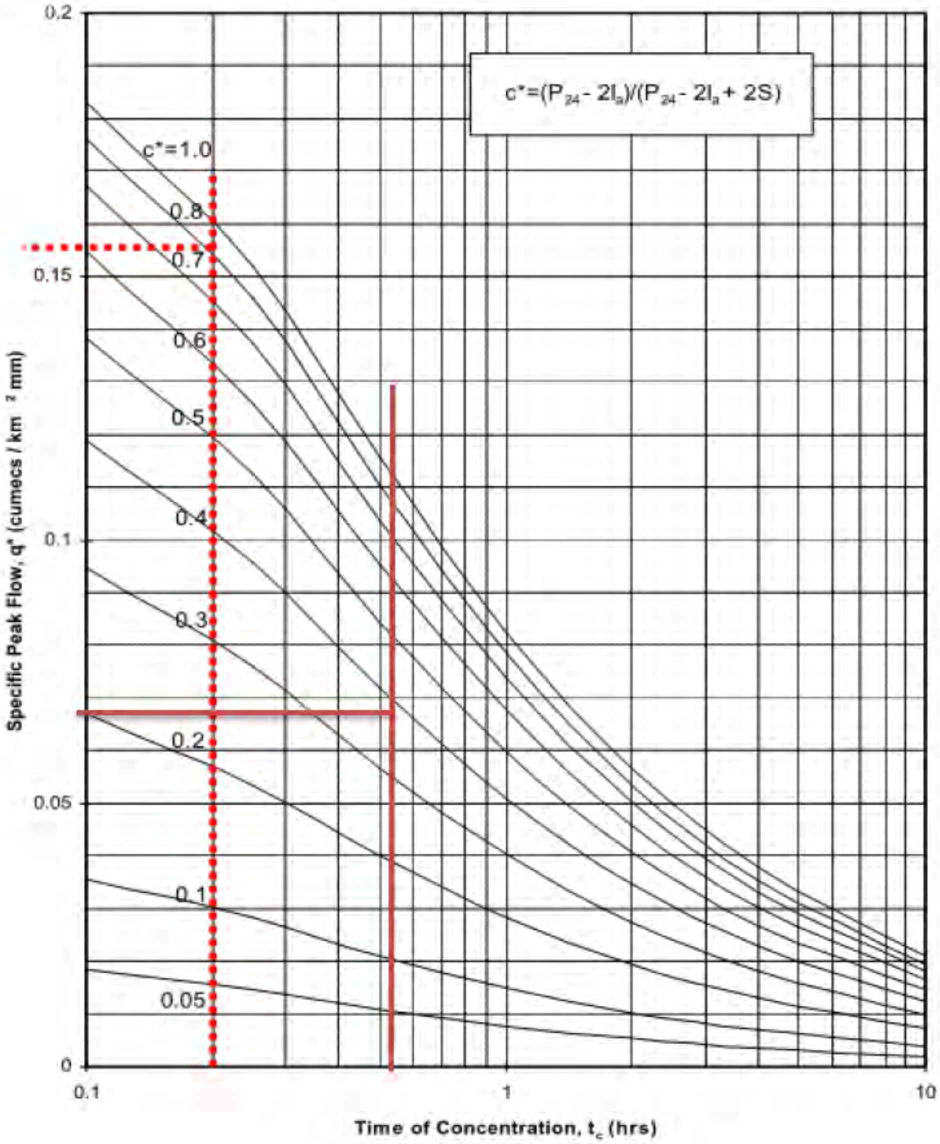
Catchment Area	A=	0.19500	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.52	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	100	(yr)
4. 24 hour rainfall depth, P24	200	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.369	
6. Specific peak flow rate q*	0.068	
7. Peak flow rate, q_p=q*A*P_{24}	**2.652**	(m3/s)
8. Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	106.4	
9. Runoff volume, V_{24} = 1000xQ_{24}A	20747.09	(m3)

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd.		Job Number 289001	Sheet 5	Rev A
Job Title Calc Title	Eldonwood, Matamata Post-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road	98	5.96	583.6
C	Residential District (30% PERVIOUS)	74	4.1	300.8
C	Residential District (70% IMPERVIOUS)	90	9.48	853.6
* from Appendix B		Totals =	19.50	1738.0

CN (weighted) = total product / total area = 1738.02 / 19.505 = 89.1

Ia (weighted) = 5 x pervious area / total area = 5 x 4.06 / 19.50 = 1.0 mm

2. Time of Concentration

Channelisation factor C = 0.6 (From Table 4.2) piped

Catchment length L = 0.6 km (along drainage path)

Catchment Slope Sc= 0.01 m/m (by equal area method)

Runoff factor, CN / (200 - CN) = 89.1 / (200- 89.1) = 0.80

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0.1 0.6 0.71 1.13 3.98 = 0.269 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.180 hrs

OK
use
0.180 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

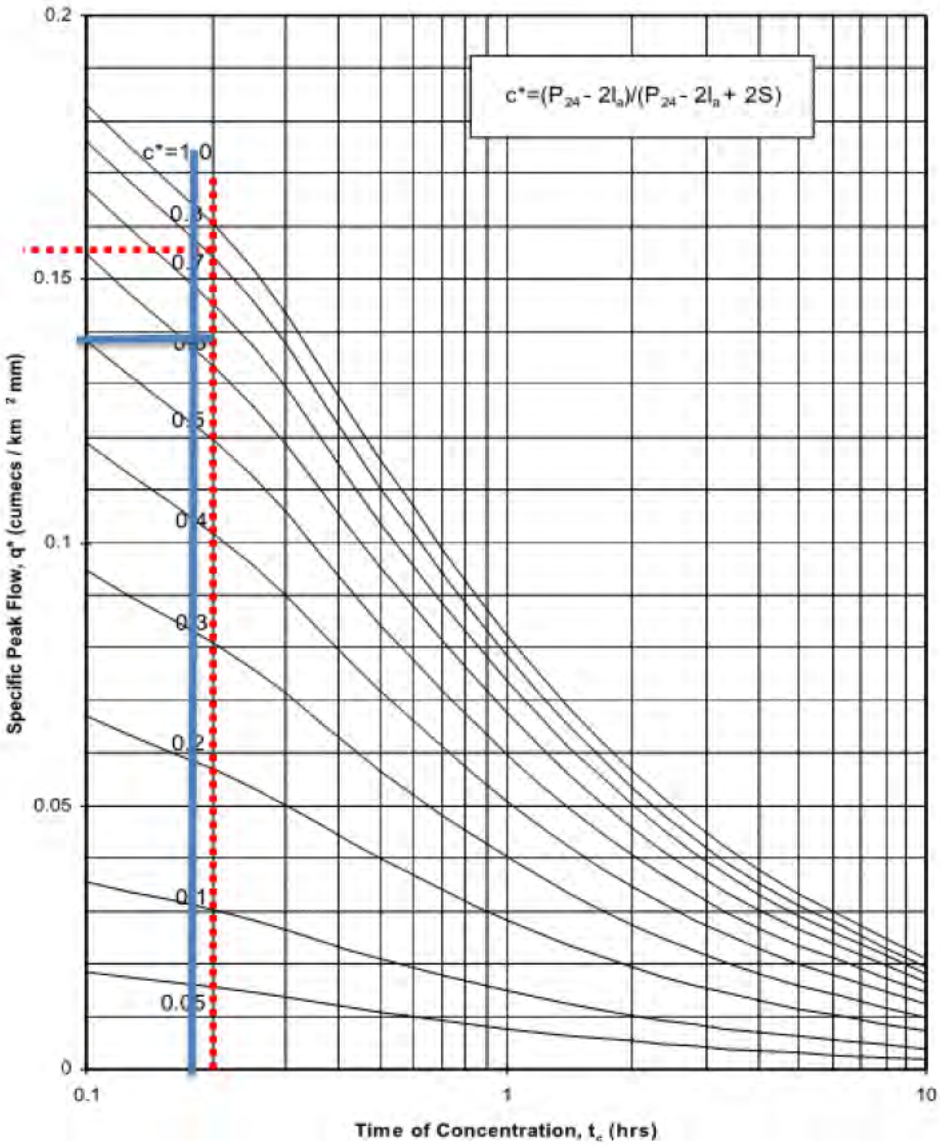
Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd	Job Number 289001	Sheet 6	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1.	Data				
	Catchment Area	A=	0.195	km2(100ha =1km2)	
	Runoff curve number	CN=	89.1	(from worksheet 1)	
	Initial abstraction	Ia=	1.0	mm (from worksheet 1)	
	Time of concentration	tc=	0.180	hrs (from worksheet 1)	
2.	Calculate storage, S =(1000/CN - 10)25.4	=	31	mm	

3.	Average recurrence interval, ARI	2	(yr)		
4.	24 hour rainfall depth, P24	106	(mm)		
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.626			
6.	Specific peak flow rate q*	0.139		***HEC-HMS Check***	
7.	Peak flow rate, q_p=q*A*P_{24}	**2.874**			Pre-Dev
8.	Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	81.0			
9.	Runoff volume, $V_{24} = 1000xQ_{24}A$	15798.35	(m3)		
	Pre development run off volume	**4935.16**	(m3)		
	Post development run off volume	**15798.35**	(m3)		
	Pre development flow rate	**0.58**	(m3/s)		
	Post development flow rate	**2.87**	(m3/s)		

Worksheet 2: Graphical Peak Flow Rate


c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)
0.2
0.15
0.1
0.05
0
0.1
1
10

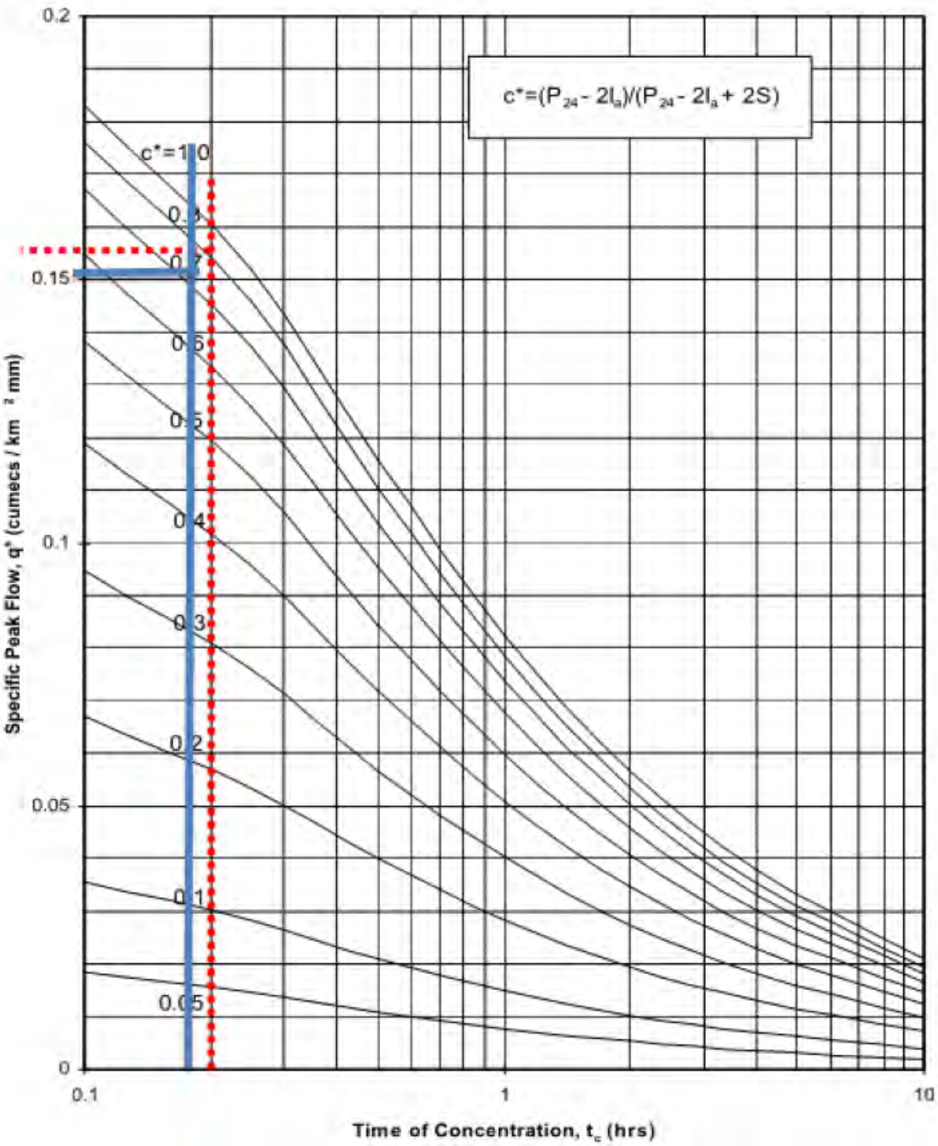
Maven Associates Ltd	Job Number 289001	Sheet 7	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1.	Data			
	Catchment Area	A=	0.195	km2(100ha =1km2)
	Runoff curve number	CN=	89.1	(from worksheet 1)
	Initial abstraction	Ia=	1.0	mm (from worksheet 1)
	Time of concentration	tc=	0.180	hrs (from worksheet 1)
2.	Calculate storage, S =(1000/CN - 10)25.4	=	31	mm
3.	Average recurrence interval, ARI		10	(yr)
4.	24 hour rainfall depth, P24		167	(mm)
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S		0.726	
6.	Specific peak flow rate q*		0.151	
7.	Peak flow rate, q_p=q*A*P_{24}		**4.918**	
8.	Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$		139.8	
9.	Runoff volume, V_{24} = 1000xQ_{24}A		27268.12	(m3)
	Pre development run off volume		**10337.15**	(m3)
	Post development run off volume		**27268.12**	(m3)
	Pre development flow rate		**1.22**	(m3/s)
	Post development flow rate		**4.92**	(m3/s)

HEC-HMS Check

	Pre-Dev

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, t_c (hrs)

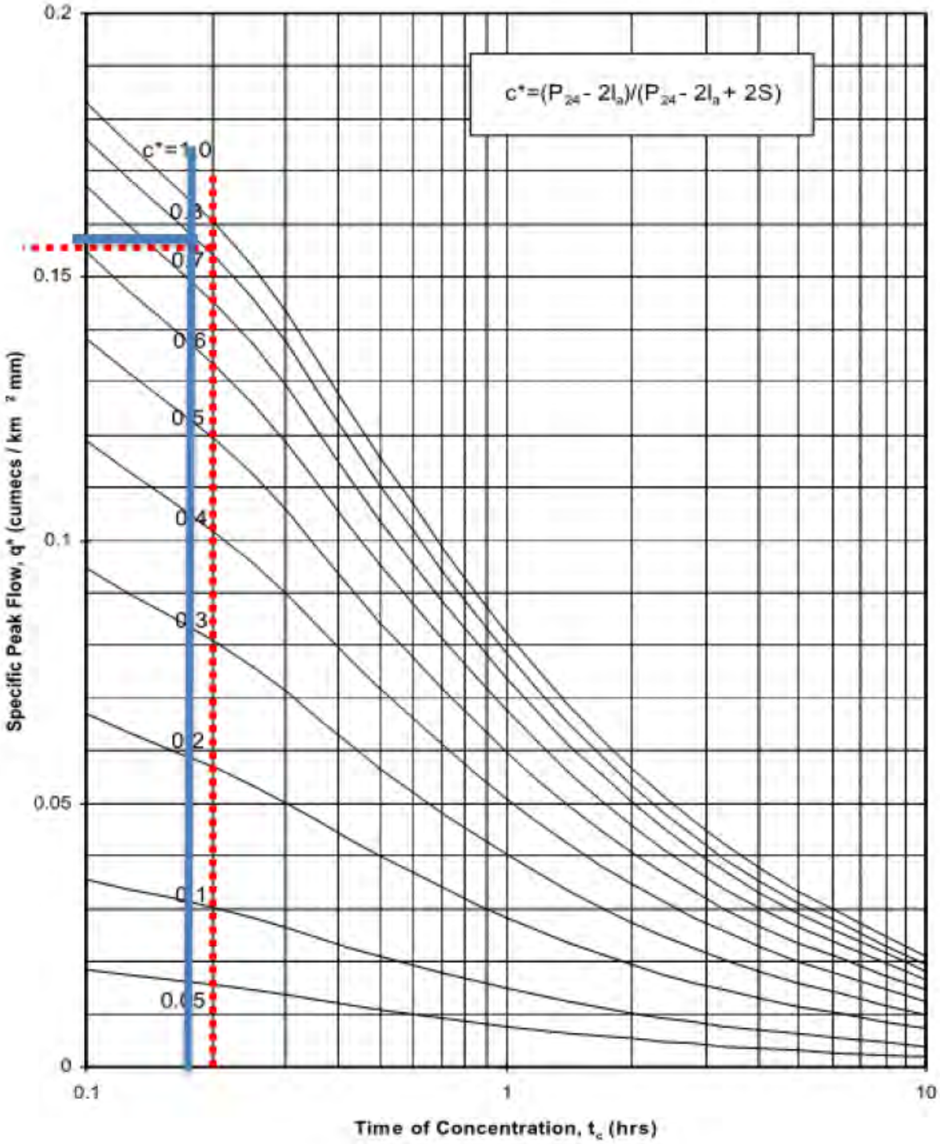
1. Data

Catchment Area	A=	0.195	km2(100ha =1km2)
Runoff curve number	CN=	89.1	(from worksheet 1)
Initial abstraction	Ia=	1.0	mm (from worksheet 1)
Time of concentration	tc=	0.180	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 31 mm

3.	Average recurrence interval, ARI	100	(yr)		
4.	24 hour rainfall depth, P24	265	(mm)		
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.809			
6.	Specific peak flow rate q*	0.158		***HEC-HMS Check***	
7.	Peak flow rate, q_p=q*A*P_{24}	**8.167**		**2.122**	80% Pre
8.	Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	236.2			
9.	Runoff volume, V_{24} = 1000xQ_{24}A	46065.59	(m3)		
	Pre development run off volume	**20747.09**	(m3)		
	Post development run off volume	**46065.59**	(m3)		
	Pre development flow rate	**2.65**	(m3/s)		
	Post development flow rate	**8.17**	(m3/s)		
	100yr - 10yr post development	**18797.47**	(m3)		

Worksheet 2: Graphical Peak Flow Rate


$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 1	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B	Open Space (Sandy Loam or Silty Loam)	61	4.33	264.41
* from Appendix B		Totals =	4.33	264.41

WQV

CN (weighted) = $\frac{\text{total product}}{\text{total area}}$ = $\frac{264.41}{4.33}$ = 61.0

Ia (weighted) = $\frac{\text{5 x pervious area}}{\text{total area}}$ = $\frac{5 \times 4.33}{4.33}$ 5.0 mm

2. Time of Concentration

Channelisation factor C = 1 (From Table 4.2) natural channels

Catchment length L = 0.34 km (along drainage path)

Catchment Slope Sc= 0.001 m/m (by equal area method)

Runoff factor, $\frac{CN}{200 - CN}$ = $\frac{61.0}{200- 61.0}$ = 0.44

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0 1 0.49 1.57 7.94 = 0.858 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.575 hrs

OK
use
0.58 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd.	Job Number 289001	Sheet 2	Rev A
Job Title Station Road, Matamata **Calc Title** Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

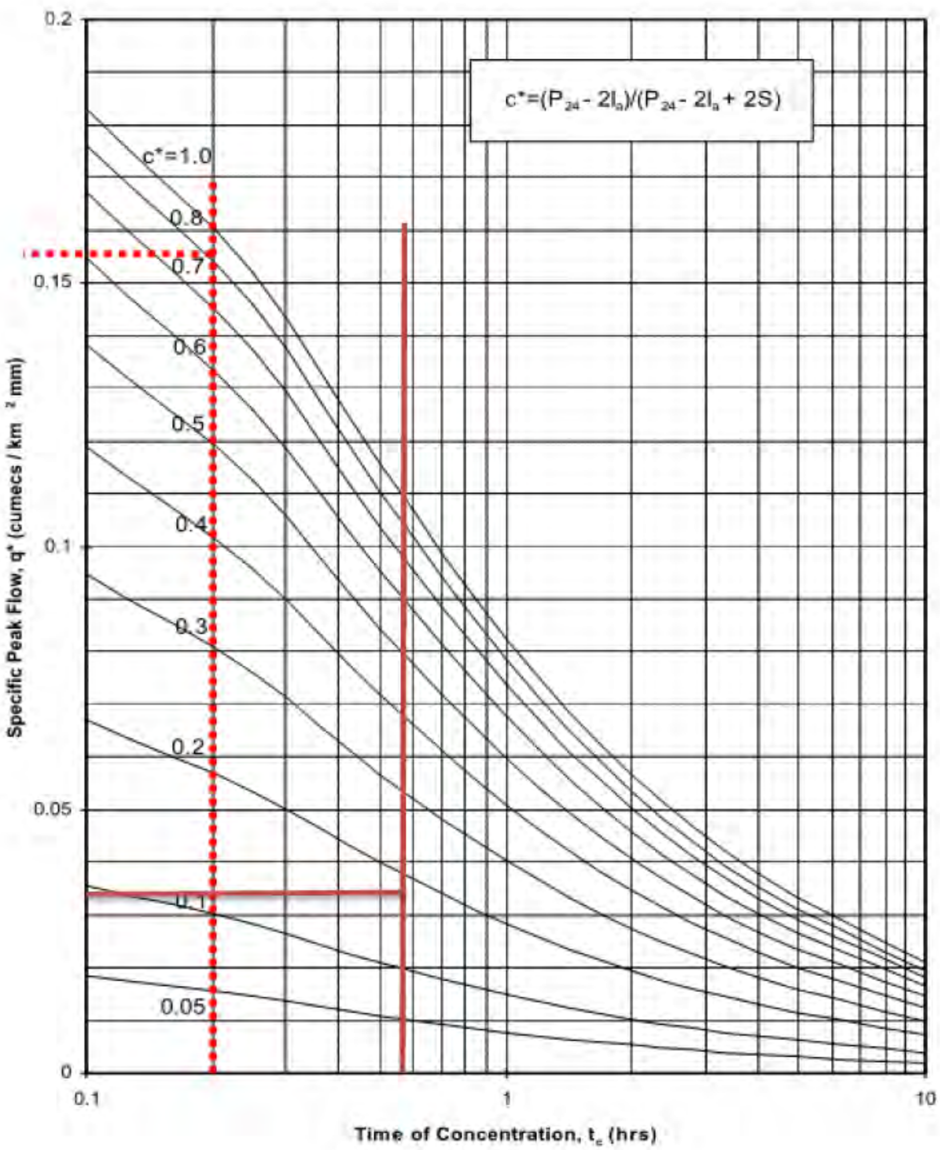
Catchment Area	A=	0.04335	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.58	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	2	(yr)
4. 24 hour rainfall depth, P24	83	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.184	
6. Specific peak flow rate q*	0.035	
7. Peak flow rate, q_p=q*A*P_{24}	**0.126**	(m3/s)
8. Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})\text{+S}$	25.3	
9. Runoff volume, V_{24} = 1000xQ_{24}A	1097.02	(m3)

Worksheet 2: Graphical Peak Flow Rate


$c^* = (P_{24} - 2I_a)/(P_{24} - 2I_a + 2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km² mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, t_c (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 3	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

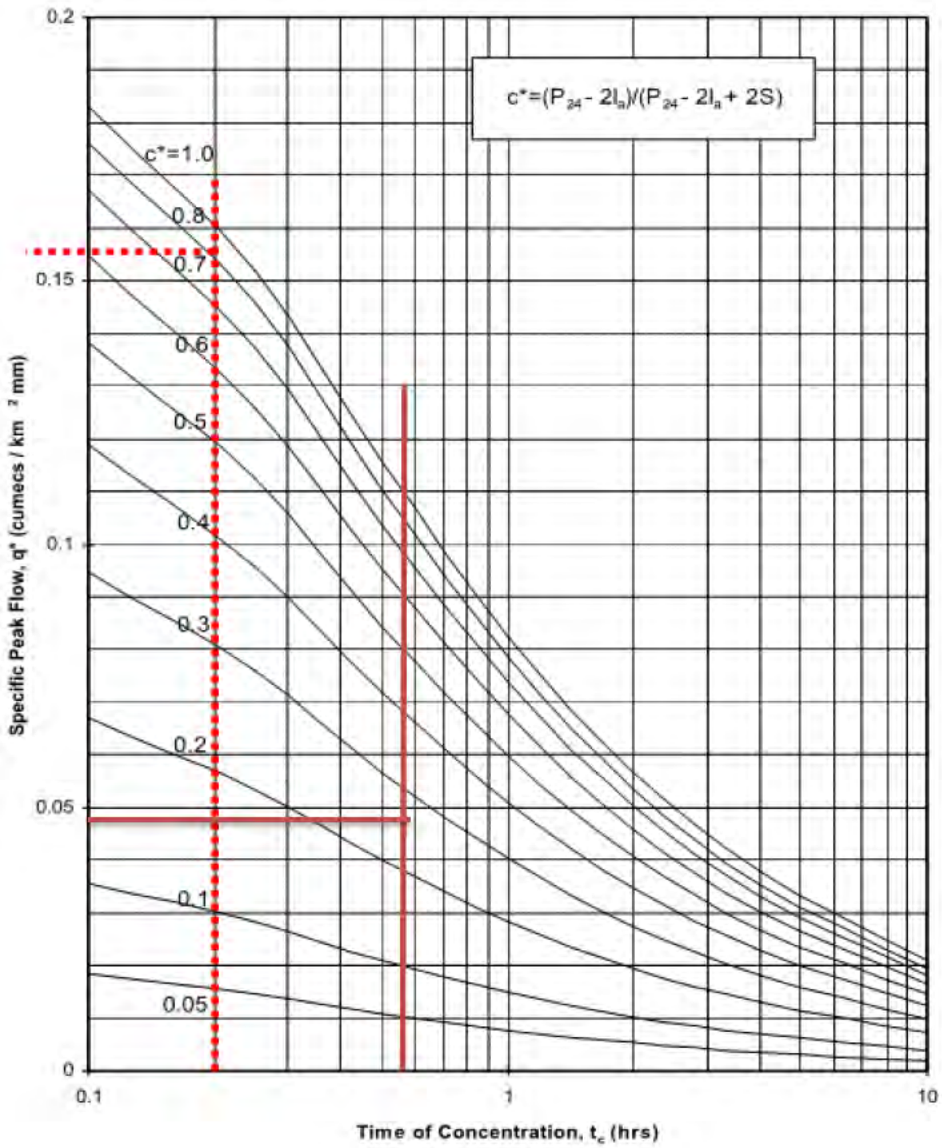
Catchment Area	A=	0.04335	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.58	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	10	(yr)
4. 24 hour rainfall depth, P24	128	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.266	
6. Specific peak flow rate q*	0.048	
7. Peak flow rate, q_p=q*A*P_{24}	**0.266**	(m3/s)
8. Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	53.0	
9. Runoff volume, $V_{24} = 1000\text{x}Q_{24}A$	2297.82	(m3)

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km 2 mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, tc (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 4	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

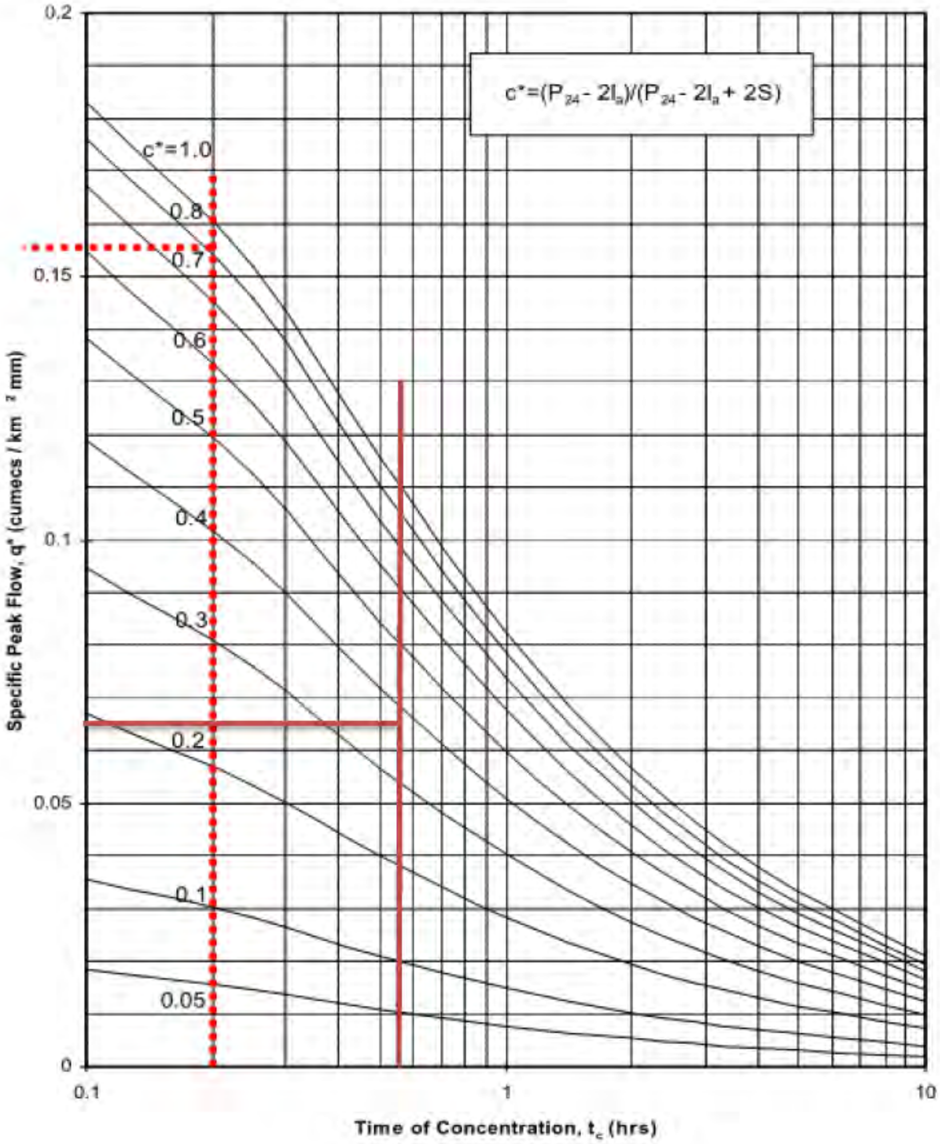
Catchment Area	A=	0.04335	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.58	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	100	(yr)
4. 24 hour rainfall depth, P24	200	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.369	
6. Specific peak flow rate q*	0.066	
7. Peak flow rate, q_p=q*A*P_{24}	**0.572**	(m3/s)
8. Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	106.4	
9. Runoff volume, $V_{24} = 1000\text{x}Q_{24}A$	4611.81	(m3)

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd.		Job Number 289001	Sheet 5	Rev A
Job Title Calc Title	Eldonwood, Matamata Post-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road	98	1.07	105.3
C	Residential District (30% PERVIOUS)	74	0.98	72.4
C	Residential District (70% IMPERVIOUS)	90	2.28	205.4
* from Appendix B		Totals =	4.33	383.0

CN (weighted) = total product = 383.05 = 88.4
total area 4.335

Ia (weighted) = 5 x pervious area = 5 x 0.98 1.1 mm
total area 4.33

2. Time of Concentration

Channelisation factor C = 0.6 (From Table 4.2) piped

Catchment length L = 0.34 km (along drainage path)

Catchment Slope Sc= 0.005 m/m (by equal area method)

Runoff factor, CN = 88.4 = 0.79
200 - CN 200- 88.4

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0.1 0.6 0.49 1.14 4.90 = 0.230 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.154 hrs

NO GOOD
use
0.170 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd	Job Number 289001	Sheet 6	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

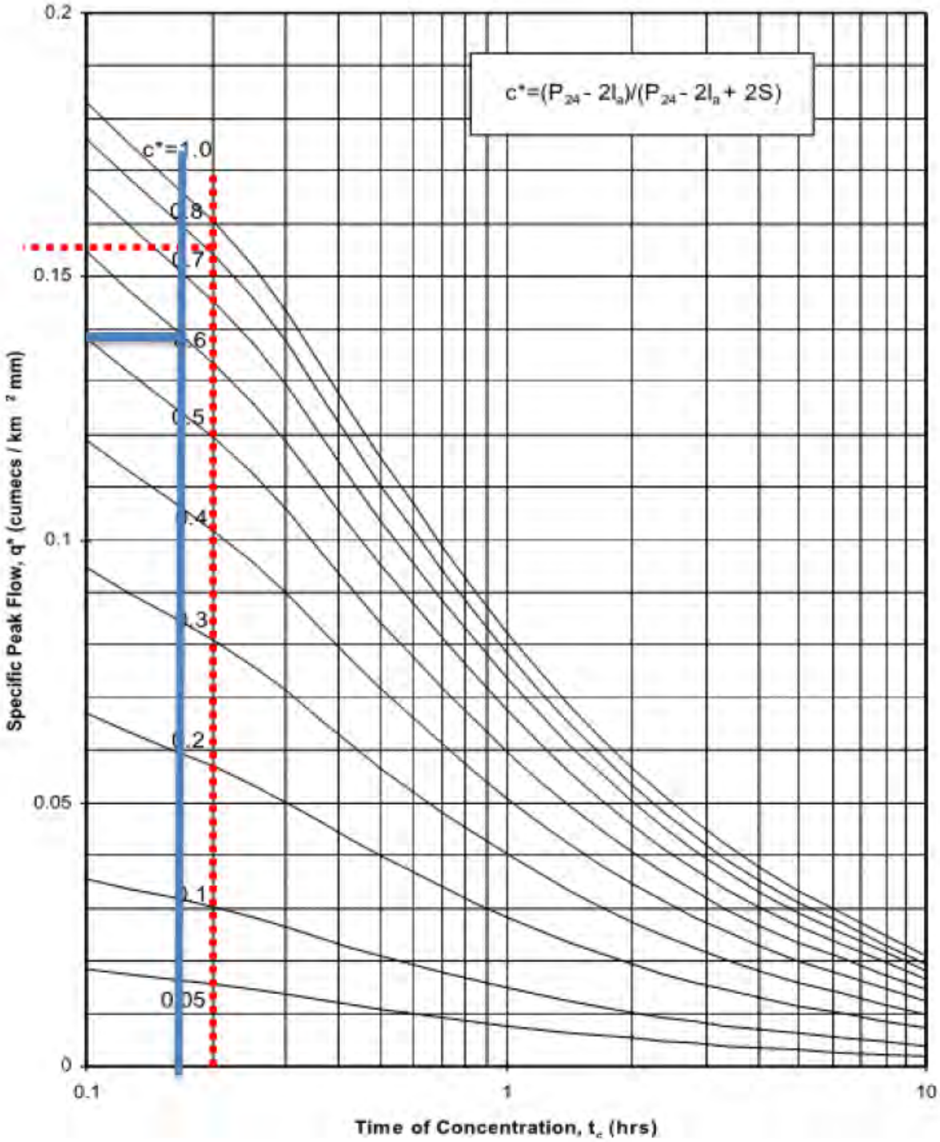
1. Data

Catchment Area	A=	0.043	km2(100ha =1km2)
Runoff curve number	CN=	88.4	(from worksheet 1)
Initial abstraction	Ia=	1.1	mm (from worksheet 1)
Time of concentration	tc=	0.170	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 33 mm

3.	Average recurrence interval, ARI	2	(yr)		
4.	24 hour rainfall depth, P24	106	(mm)		
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.608			
6.	Specific peak flow rate q*	0.139		***HEC-HMS Check***	
7.	Peak flow rate, q_p=q*A*P_{24}	**0.639**			Pre-Dev
8.	Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	79.5			
9.	Runoff volume, $V_{24} = 1000xQ_{24}A$	3447.06	(m3)		
	Pre development run off volume	**1097.02**	(m3)		
	Post development run off volume	**3447.06**	(m3)		
	Pre development flow rate	**0.13**	(m3/s)		
	Post development flow rate	**0.64**	(m3/s)		

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd	Job Number 289001	Sheet 7	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.043	km2(100ha =1km2)
Runoff curve number	CN=	88.4	(from worksheet 1)
Initial abstraction	Ia=	1.1	mm (from worksheet 1)
Time of concentration	tc=	0.170	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 33 mm

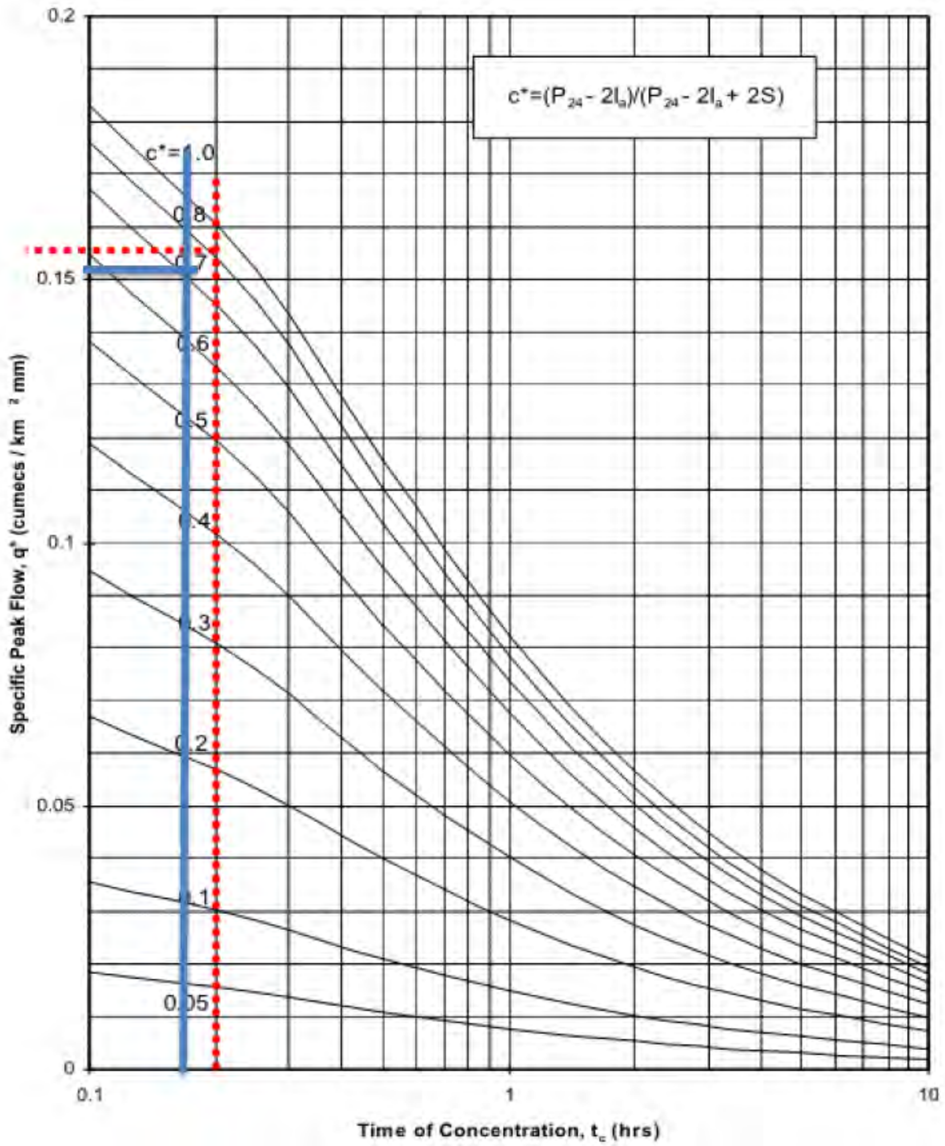
3.	Average recurrence interval, ARI	10	(yr)
4.	24 hour rainfall depth, P24	167	(mm)
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.711	
6.	Specific peak flow rate q*	0.152	
7.	Peak flow rate, q_p=q*A*P_{24}	**1.100**	
8.	Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	138.1	
9.	Runoff volume, $V_{24} = 1000xQ_{24}A$	5983.93	(m3)

HEC-HMS Check

Pre-Dev

Pre development run off volume	**2297.82**	(m3)
Post development run off volume	**5983.93**	(m3)
Pre development flow rate	**0.27**	(m3/s)
Post development flow rate	**1.10**	(m3/s)

Worksheet 2: Graphical Peak Flow Rate

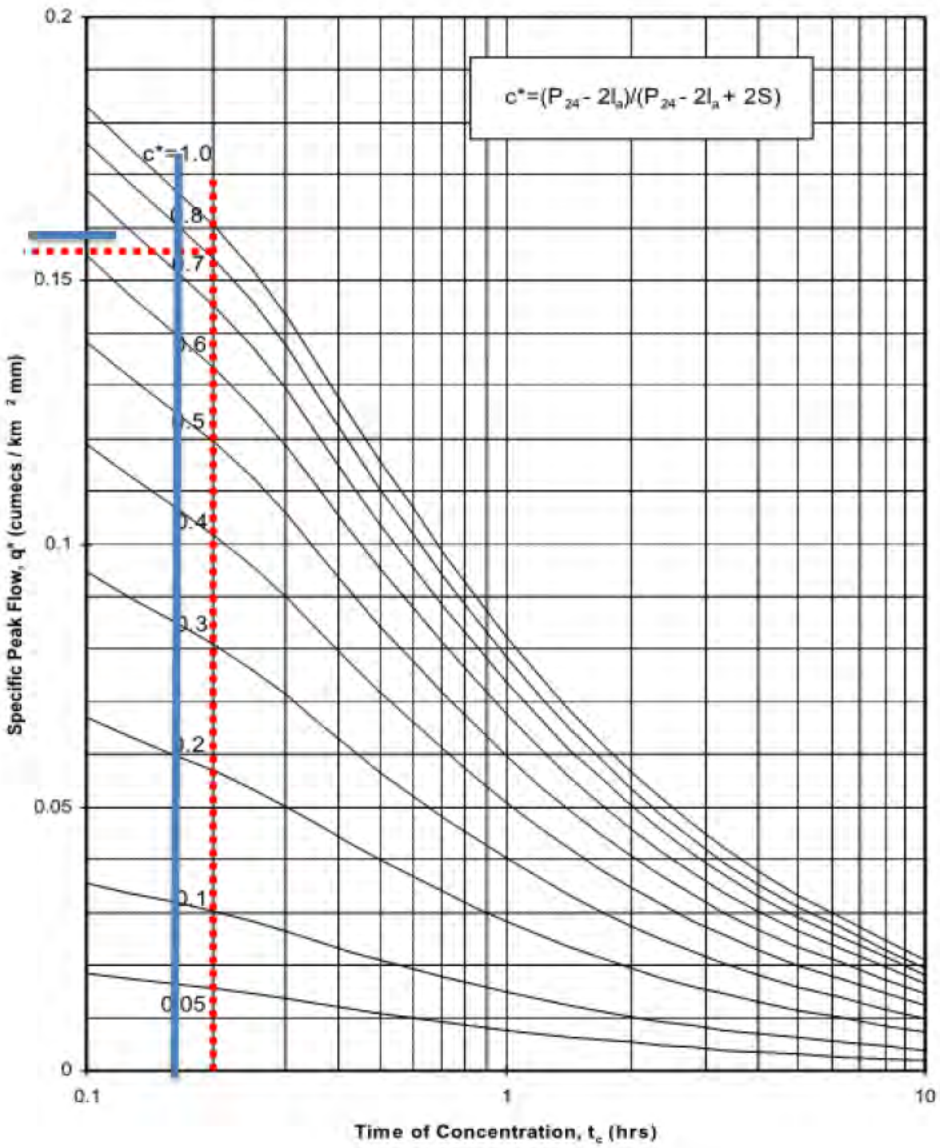


$c^* = (P_{24} - 2I_a)/(P_{24} - 2I_a + 2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd	Job Number 289001	Sheet 8	Rev A
Job Title Station Road, Matamata Calc Title Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1.	Data						
	Catchment Area	A=	0.043	km2(100ha =1km2)			
	Runoff curve number	CN=	88.4	(from worksheet 1)			
	Initial abstraction	Ia=	1.1	mm (from worksheet 1)			
	Time of concentration	tc=	0.170	hrs (from worksheet 1)			
2.	Calculate storage, S =(1000/CN - 10)25.4		=		33 mm		
3.	Average recurrence interval, ARI		100	(yr)			
4.	24 hour rainfall depth, P24		265	(mm)			
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S		0.797				
6.	Specific peak flow rate q*		0.159		***HEC-HMS Check***		
7.	Peak flow rate, q_p=q*A*P_{24}		**1.826**			**0.458**	80% Pre
8.	Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$		234.2				
9.	Runoff volume, V_{24} = 1000xQ_{24}A		10151.65	(m3)			
	Pre development run off volume		**4611.81**	(m3)			
	Post development run off volume		**10151.65**	(m3)			
	Pre development flow rate		**0.57**	(m3/s)			
	Post development flow rate		**1.83**	(m3/s)			
	100yr - 10yr post development		**4167.72**	(m3)			

Worksheet 2: Graphical Peak Flow Rate



$c^*=(P_{24}-2I_a)/(P_{24}-2I_a+2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, t_c (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 1	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B	Open Space (Sandy Loam or Silty Loam)	61	8.03	489.83
* from Appendix B		Totals =	8.03	489.83

WQV

CN (weighted) = total product / total area = 489.83 / 8.03 = 61.0

Ia (weighted) = 5 x pervious area / total area = 5 x 8.03 / 8.03 = 5.0 mm

2. Time of Concentration

Channelisation factor C = 1 (From Table 4.2) natural channels

Catchment length L = 0.62 km (along drainage path)

Catchment Slope Sc= 0.001 m/m (by equal area method)

Runoff factor, CN / (200 - CN) = 61.0 / (200- 61.0) = 0.44

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0 1 0.73 1.57 7.94 = 1.276 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.855 hrs

OK
use
0.85 hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd.	Job Number 289001	Sheet 2	Rev A
Job Title Station Road, Matamata Calc Title Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

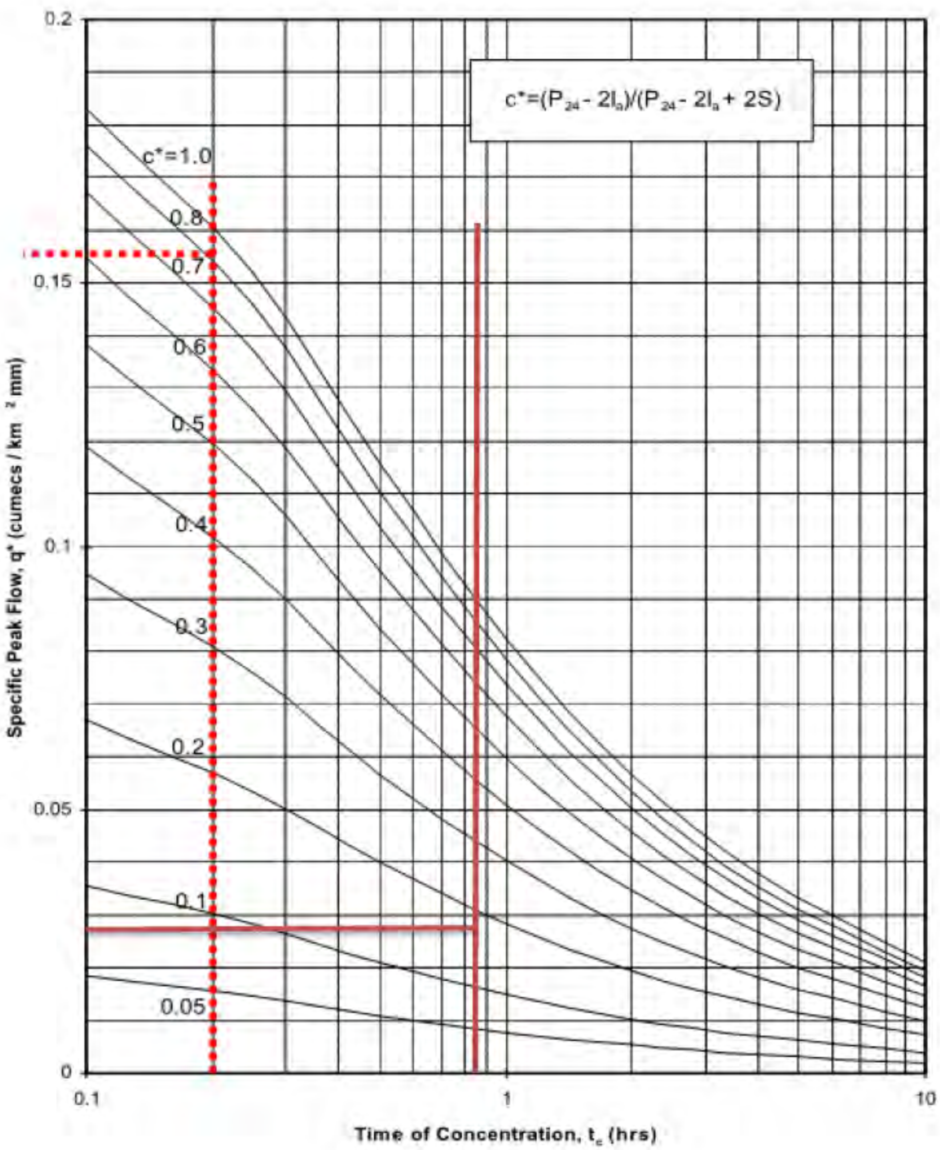
Catchment Area	A=	0.08030	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.85	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

3. Average recurrence interval, ARI	2	(yr)
4. 24 hour rainfall depth, P24	83	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.184	
6. Specific peak flow rate q*	0.028	
7. Peak flow rate, q_p=q*A*P_{24}	**0.187**	(m3/s)
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	25.3	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	2032.27	(m3)

NO CLIMATE CHANGE

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km 2 mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, tc (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 3	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

1. Data

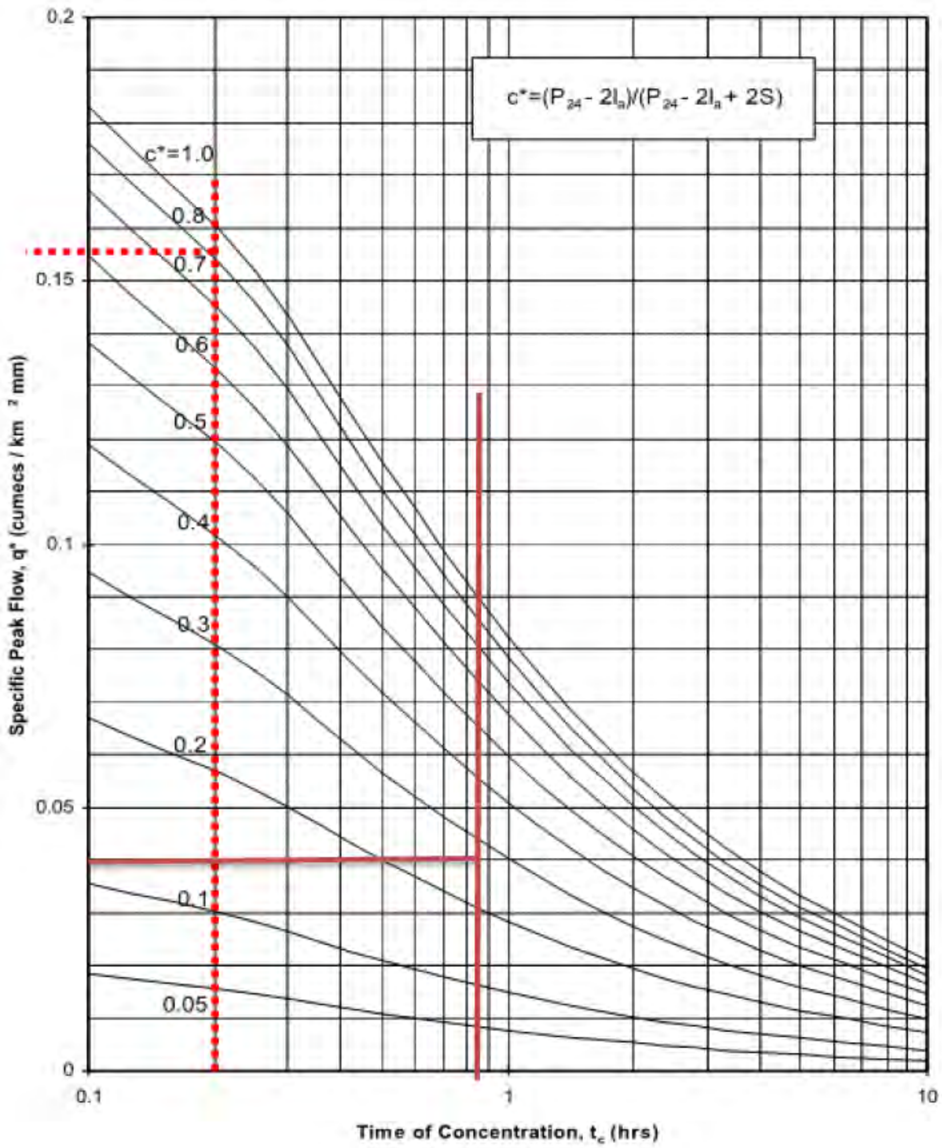
Catchment Area	A=	0.08030	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.85	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

NO CLIMATE CHANGE

3. Average recurrence interval, ARI	10	(yr)
4. 24 hour rainfall depth, P24	128	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.266	
6. Specific peak flow rate q*	0.040	
7. Peak flow rate, q_p=q*A*P_{24}	**0.411**	(m3/s)
8. Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	53.0	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	4256.79	(m3)

Worksheet 2: Graphical Peak Flow Rate



$c^* = (P_{24} - 2I_a)/(P_{24} - 2I_a + 2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km² mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, t_c (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 4	Rev A
Job Title: Station Road, Matamata Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

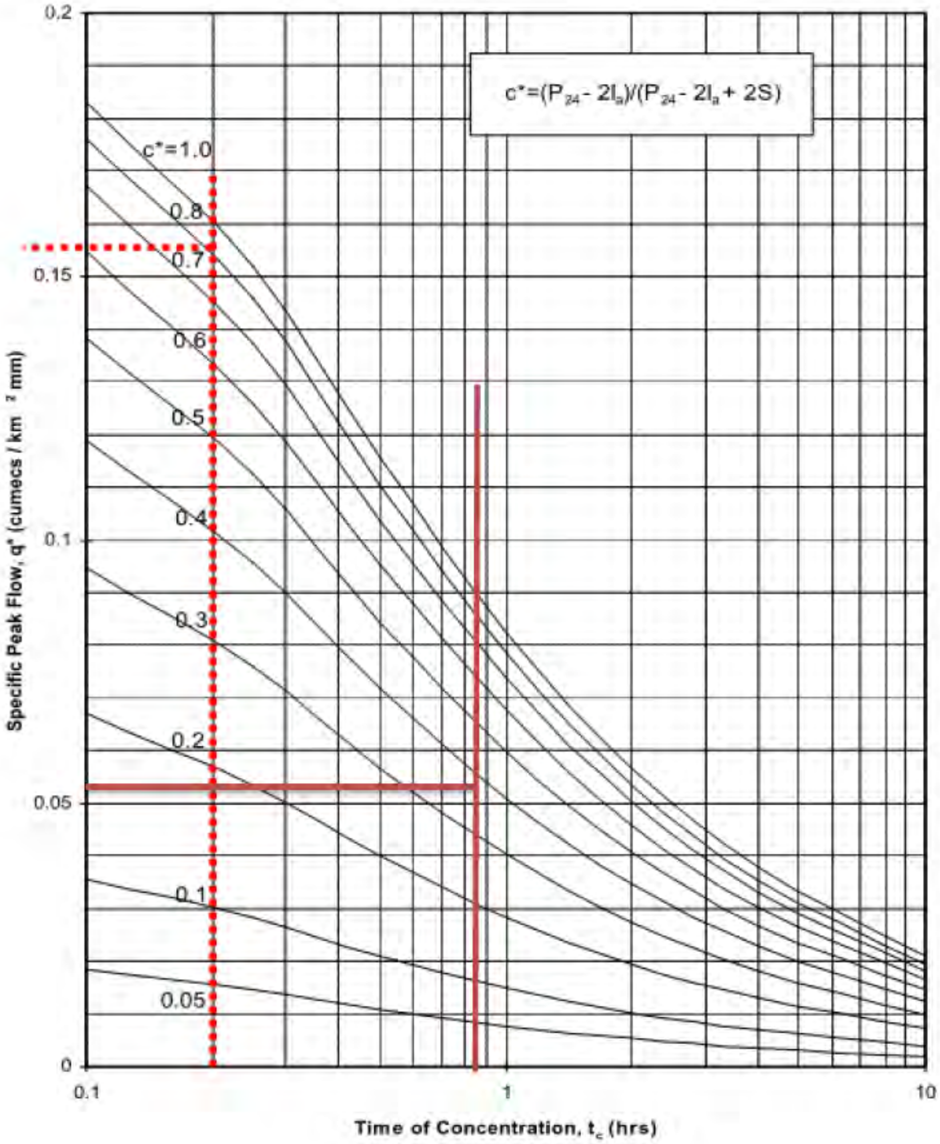
1. Data

Catchment Area	A=	0.08030	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	5.0	mm (from worksheet 1)
Time of concentration	tc=	0.85	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

3. Average recurrence interval, ARI	100	(yr)	NO CLIMATE CHANGE
4. 24 hour rainfall depth, P24	200	(mm)	
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.369		
6. Specific peak flow rate q*	0.053		
7. Peak flow rate, q_p=q*A*P_{24}	**0.851**	(m3/s)	
8. Runoff depth, Q_{24} = $(P_{24}$-Ia$)^2$/(P_{24}-Ia)+S	106.4		
9. Runoff volume, V_{24} = 1000xQ_{24}A	8543.55	(m3)	

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd.	Job Number 289001	Sheet 5	Rev A
Job Title: Eldonwood, Matamata Calc Title: Post-development	Author MKS	Date 13/03/2025	Checked DJM

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

Soil name and classification	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
C	Road	98	2.17	212.7
C	Residential District (30% PERVIOUS)	74	1.76	130.2
C	Residential District (70% IMPERVIOUS)	90	4.10	369.0
* from Appendix B		Totals =	8.03	711.9

CN (weighted) = total product / total area = 711.89 / 8.030 = 88.7

Ia (weighted) = 5 x pervious area / total area = 5 x 1.76 / 8.03 = 1.1 mm

2. Time of Concentration

Channelisation factor C = 0.6 (From Table 4.2) piped

Catchment length L = 0.62 km (along drainage path)

Catchment Slope Sc= 0.005 m/m (by equal area method)

Runoff factor, CN / (200 - CN) = 88.7 / (200- 88.7) = 0.80

$t_c = 0.14\ C\ L^{0.66}\ (CN/200\text{-}CN)^{-0.55}\ Sc^{-0.30}$

= 0.1 0.6 0.73 1.13 4.90 = 0.340 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.228 hrs

OK
use
0.228 hrs

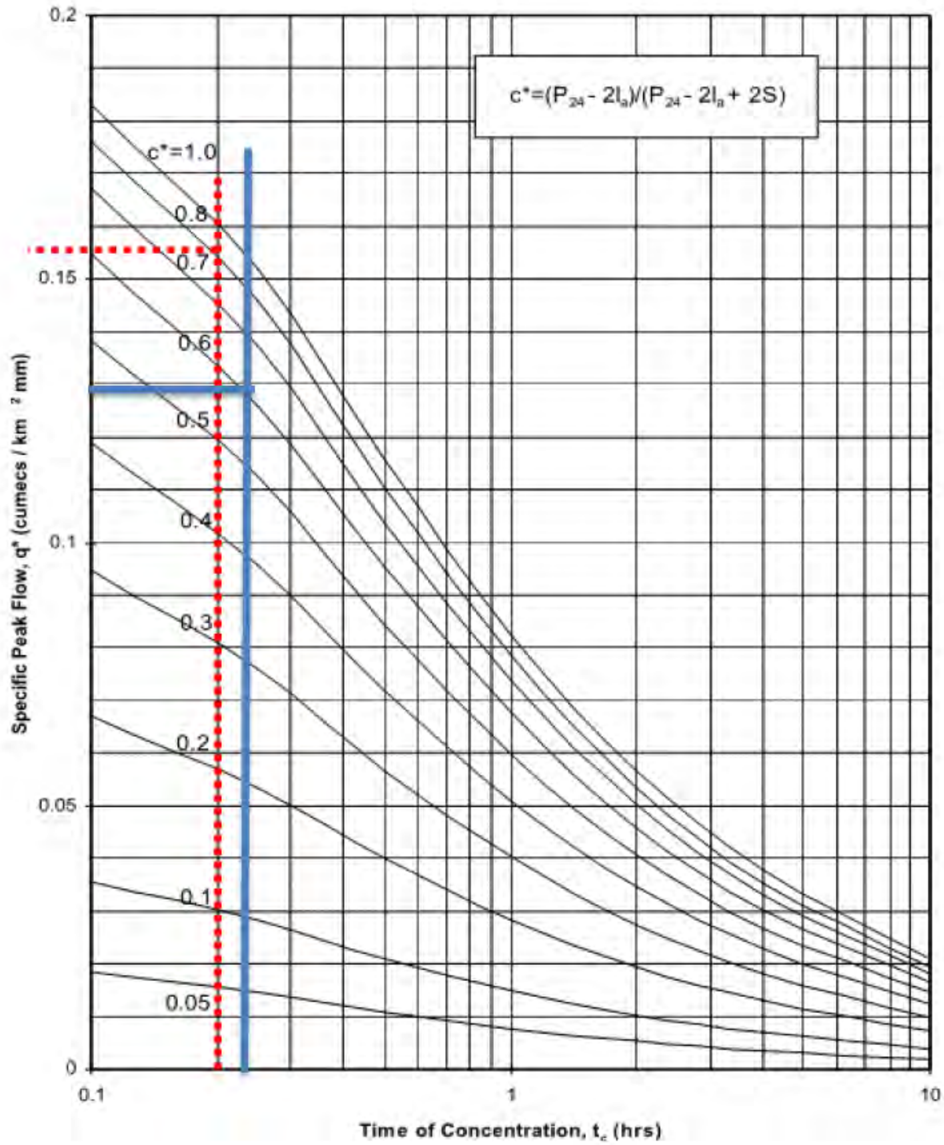
Worksheet 1: Runoff Parameters and Time of Concentration

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

Maven Associates Ltd	Job Number 289001	Sheet 6	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1.	Data				
	Catchment Area	A=	0.080	km2(100ha =1km2)	
	Runoff curve number	CN=	88.7	(from worksheet 1)	
	Initial abstraction	Ia=	1.1	mm (from worksheet 1)	
	Time of concentration	tc=	0.228	hrs (from worksheet 1)	
2.	Calculate storage, S =(1000/CN - 10)25.4	=	33	mm	
3.	Average recurrence interval, ARI		2	(yr)	
4.	24 hour rainfall depth, P24		106	(mm)	
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S		0.615		
6.	Specific peak flow rate q*		0.129		*HEC-HMS Check*
7.	Peak flow rate, q_p=q*A*P_{24}		**1.098**		Pre-Dev
8.	Runoff depth, Q_{24} = $(P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$		80.1		
9.	Runoff volume, V_{24} = 1000xQ_{24}A		6431.06	(m3)	
	Pre development run off volume		**2032.27**	(m3)	
	Post development run off volume		**6431.06**	(m3)	
	Pre development flow rate		**0.19**	(m3/s)	
	Post development flow rate		**1.10**	(m3/s)	

Worksheet 2: Graphical Peak Flow Rate



c*=(P24 - 2Ia)/(P24 - 2Ia + 2S)
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
0.2
0.15
0.1
0.05
0
0.1
1
10
Specific Peak Flow, q* (cumecs / km 2 mm)
Time of Concentration, tc (hrs)

Maven Associates Ltd	Job Number 289001	Sheet 7	Rev A
Job Title: Station Road, Matamata Calc Title: Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.080	km2(100ha =1km2)
Runoff curve number	CN=	88.7	(from worksheet 1)
Initial abstraction	Ia=	1.1	mm (from worksheet 1)
Time of concentration	tc=	0.228	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 33 mm

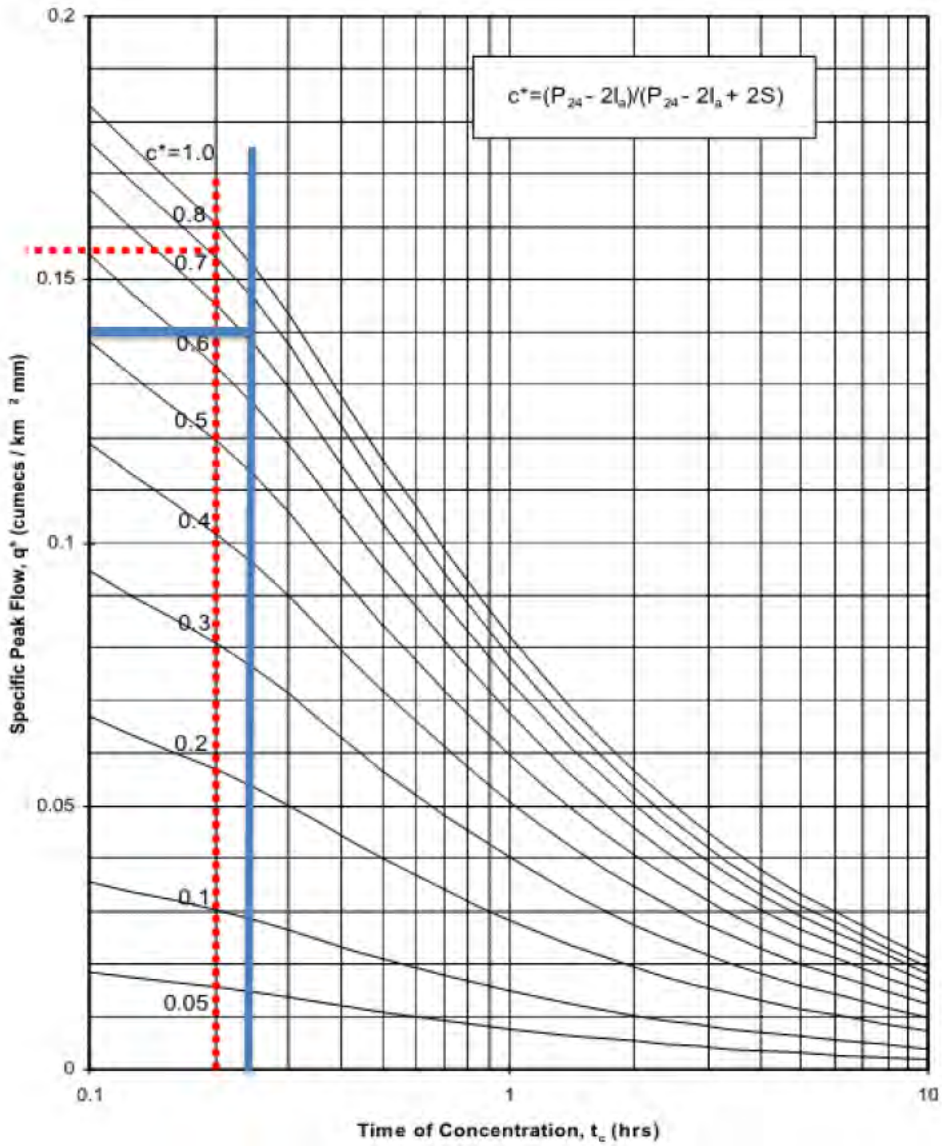
3.	Average recurrence interval, ARI	10	(yr)
4.	24 hour rainfall depth, P24	167	(mm)
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.717	
6.	Specific peak flow rate q*	0.140	
7.	Peak flow rate, q_p=q*A*P_{24}	**1.877**	
8.	Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	138.7	
9.	Runoff volume, $V_{24} = 1000xQ_{24}A$	11139.43	(m3)

HEC-HMS Check

Pre-Dev

Pre development run off volume	**4256.79**	(m3)
Post development run off volume	**11139.43**	(m3)
Pre development flow rate	**0.41**	(m3/s)
Post development flow rate	**1.88**	(m3/s)

Worksheet 2: Graphical Peak Flow Rate



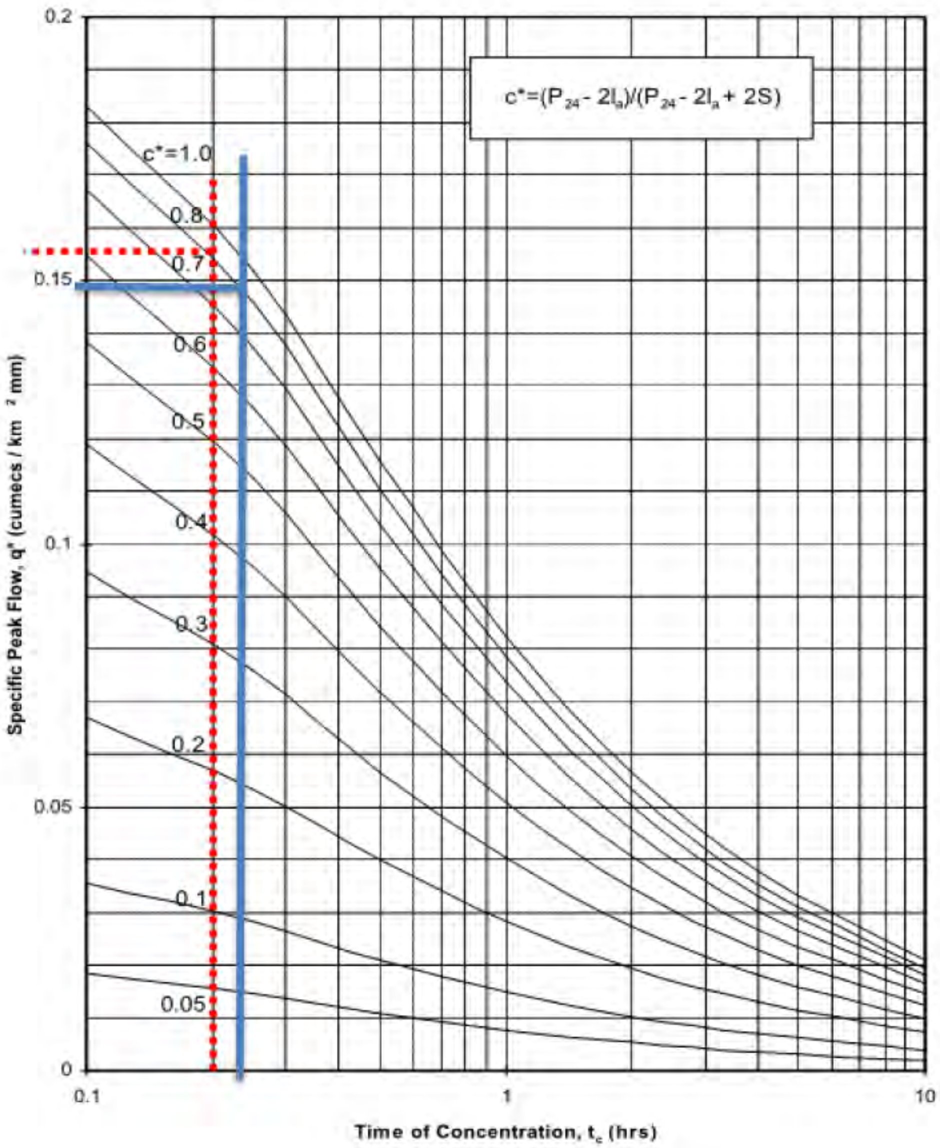
$c^* = (P_{24} - 2I_a)/(P_{24} - 2I_a + 2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km² mm)
0.2
0.15
0.1
0.05
0
0.1
1
10
Time of Concentration, t_c (hrs)

Maven Associates Ltd		Job Number 289001	Sheet 8	Rev A
Job Title Calc Title	Station Road, Matamata Post-development SW Demand	Author MKS	Date 13/03/2025	Checked DJM

1.	Data			
	Catchment Area	A=	0.080	km2(100ha =1km2)
	Runoff curve number	CN=	88.7	(from worksheet 1)
	Initial abstraction	Ia=	1.1	mm (from worksheet 1)
	Time of concentration	tc=	0.228	hrs (from worksheet 1)
2.	Calculate storage, S =(1000/CN - 10)25.4	=	33	mm

3.	Average recurrence interval, ARI	100	(yr)		
4.	24 hour rainfall depth, P24	265	(mm)		
5.	Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.802			
6.	Specific peak flow rate q*	0.149		***HEC-HMS Check***	
7.	Peak flow rate, q_p=q*A*P_{24}	**3.171**		**0.681**	80% Pre
8.	Runoff depth, $Q_{24} = (P_{24}\text{-Ia})^2/(P_{24}\text{-Ia})+S$	235.0			
9.	Runoff volume, V_{24} = 1000xQ_{24}A	18867.28	(m3)		
	Pre development run off volume	**8543.55**	(m3)		
	Post development run off volume	**18867.28**	(m3)		
	Pre development flow rate	**0.85**	(m3/s)		
	Post development flow rate	**3.17**	(m3/s)		
	100yr - 10yr post development	**7727.85**	(m3)		

Worksheet 2: Graphical Peak Flow Rate

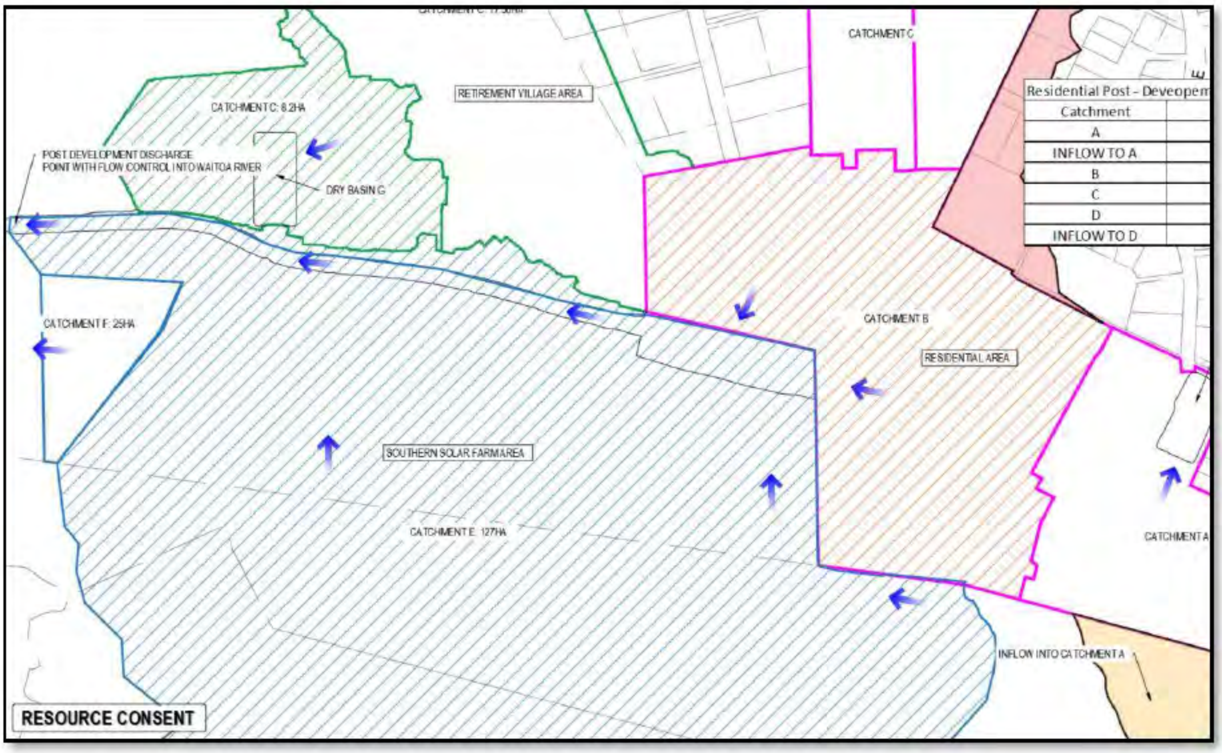


0.2
0.15
0.1
0.05
0
0.1
1
10
$c^* = (P_{24} - 2I_a)/(P_{24} - 2I_a + 2S)$
c*=1.0
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Specific Peak Flow, q* (cumecs / km² mm)
Time of Concentration, t_c (hrs)

GREENWAY DESIGN SUMMARY

Catchment Characteristics

The Greenway connects to Basin B and serves as an overall attenuation device for Basin B, as well as a diversion for the attenuated flow from RV and for inflow from the solar farm and upstream catchments. The catchment plan below, along with the tables further down, provides information regarding the catchment entering this stormwater device.Design



Catchment	Area (Ha)
RES B	19.5
Inflow RV	8.2
Inflow South	127

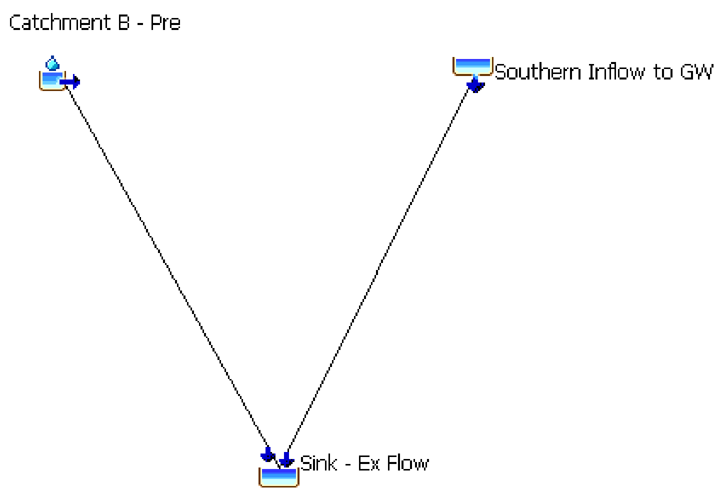
Pre-Development CN		Post Development CN	
Pervious	Impervious	Pervious	Impervious
61	98	74	98

	Pre Development		Post-Development (RCP 8.5)		
	10yr	100 yr	10yr	100yr	100yr-10yr
24 hour rainfall depth (mm)	128	200	167	265	98

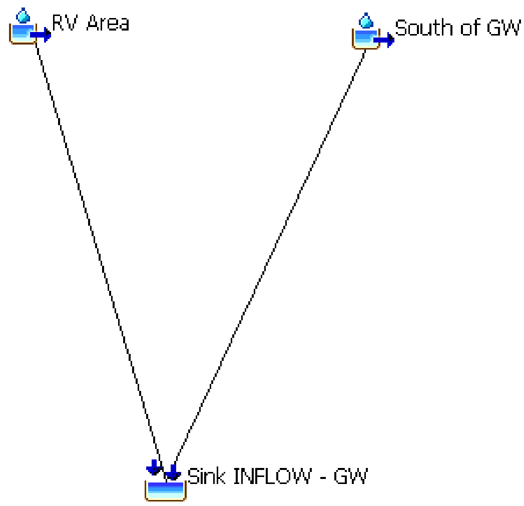
An HEC-HMS model has been prepared using the site design parameters mentioned above, along with the site hydrology data and TR20/07 data attached to this Design Summary.

Pre Development HEC HMS RESUTLS

Pre Development Basin Model



Southern Inflow to GW is separately modelled in the Basin Model shown below. Its hydrograph is inserted into the above main model representing the overall inflows outside of the Catchment B area. The same hydrograph is also incorporated into the post-development assessment of the Greenway + Basin model.



Pre Development Summary Results

Project: Basin A to D Simulation Run: Inflow 01 - GW
Sink: Sink INFLOW - GW

Start of Run: 04Apr2017, 00:00
End of Run: 05Apr2017, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

Basin Model: Inflow Basin Models
Meteorologic Model: 100 year pre
Control Specifications: Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge: 7.4724 (M3/S)
Date/Time of Peak Discharge 04Apr2017, 13:58
Volume: 136.6029 (1000 M3)

Inflow Results

Project: Basin A to D Simulation Run: Catchment B 100year Pre

Start of Run: 04Apr2017, 00:00
End of Run: 05Apr2017, 00:00
Compute Time: DATA CHANGED, RECOMPUTE

Basin Model: Pre Dev B
Meteorologic Model: 100 year pre
Control Specifications: Control 1

Show Elements: All Elements Volume Units: ○ MM ● 1000 M3 Sorting: Watershed Explorer

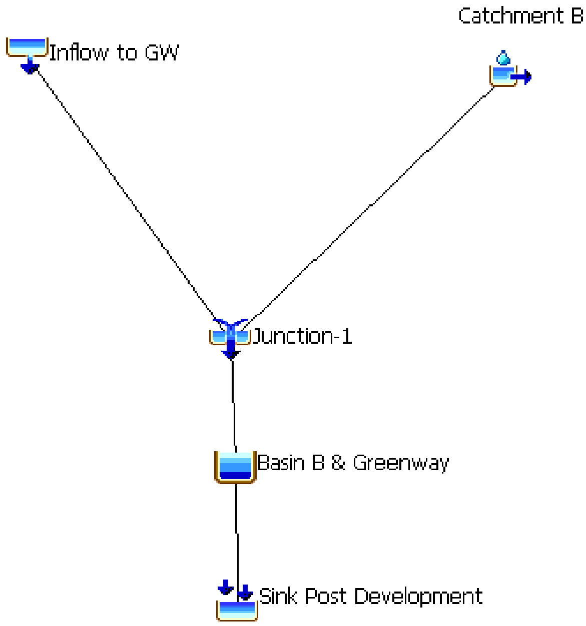
Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (1000 M3)
Catchment B - Pre	0.19500	2.1469	4 April 2017, 12:38	20.4084
Southern Inflow t...		7.4724	4 April 2017, 13:58	136.6028
Sink - Ex Flow		8.1179	4 April 2017, 13:55	157.0113

Combined Greenway + Catchment B Results

- Pre Development Peak Flow Rate Catchment B = 2.15m^3/s
- 80% Pre Development Peak Flow Rate Catchment B = **1.72m^3/s**
- Pre Development Peak Flow Rate Inflow Catchments = 7.47m^3/s
- Maximum Combined Flow Rate Required for Post Development = **9.19m^3/s**

<u>Post Development HEC HMS RESUTLS</u>

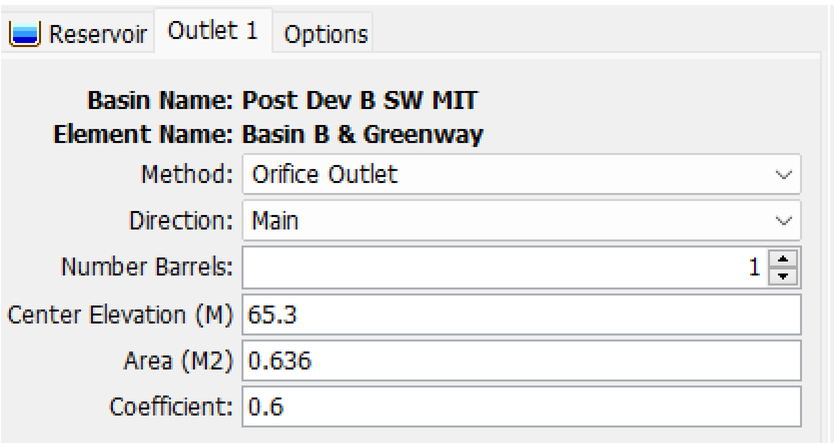
Post Development Basin Model



Basin B – Greenway – Set up

Outlet Assumptions;

- 1x Orifice 900mm @ Elevation 65.3m (note this is Center of Orifice)
- Spillway of 15m @ Elevation of 66.55m



Basin Name: Post Dev B SW MIT
Element Name: Basin B & Greenway
Method: Broad-Crested Spillway
Direction: Main
*Elevation (M) 66.55
*Length (M) 15
Coefficient (M^0.5/S) 1.6
Gates: 0

Greenway/Basin B Storage Function

Elevation	Volume (m^3)
67.00	19369.62
66.90	16927.66
66.80	14652.14
66.70	12520.01
66.60	10492.37
66.50	8536.09
66.40	6648.73
66.30	4832.60
66.20	3318.14
66.10	2359.96
66.00	1788.82
65.90	1338.32
65.80	997.34
65.70	757.29
65.60	580.37
65.50	432.76
65.40	310.75
65.30	212.29
65.20	135.32
65.10	77.79
65.00	37.66
64.90	12.88
64.80	1.41
64.75	0.03

Post Development Summary Results – Basin B

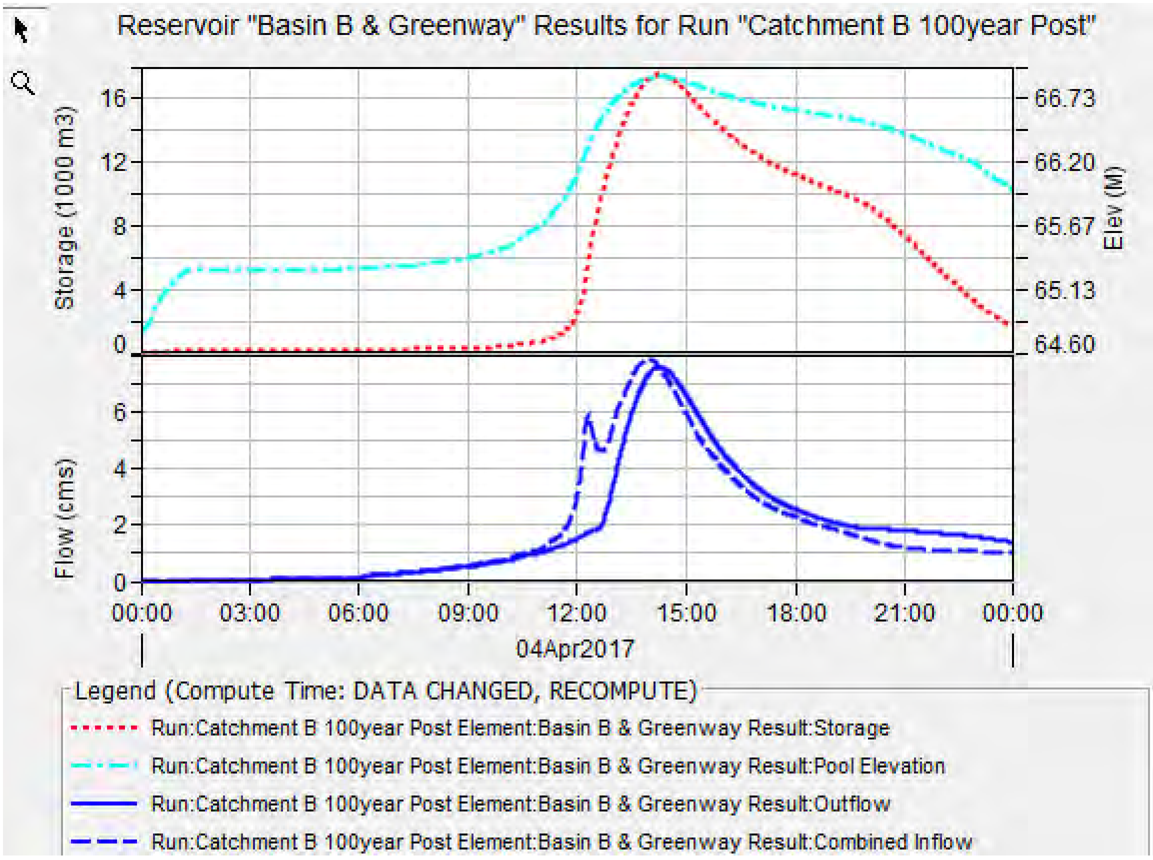
Project: Basin A to D   Simulation Run: Catchment B 100year Post
Reservoir: Basin B & Greenway

Start of Run:	04Apr2017, 00:00	Basin Model:	Post Dev B SW MIT
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100-10yr
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ◉ MM ○ 1000 M3

Computed Results

Peak Inflow:	7.8421 (M3/S)	Date/Time of Peak Inflow:	04Apr2017, 13:58
Peak Discharge:	7.5728 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 14:15
Inflow Volume:	n/a	Peak Storage:	17.5033 (1000 M3)
Discharge Volume:	n/a	Peak Elevation:	66.9209 (M)



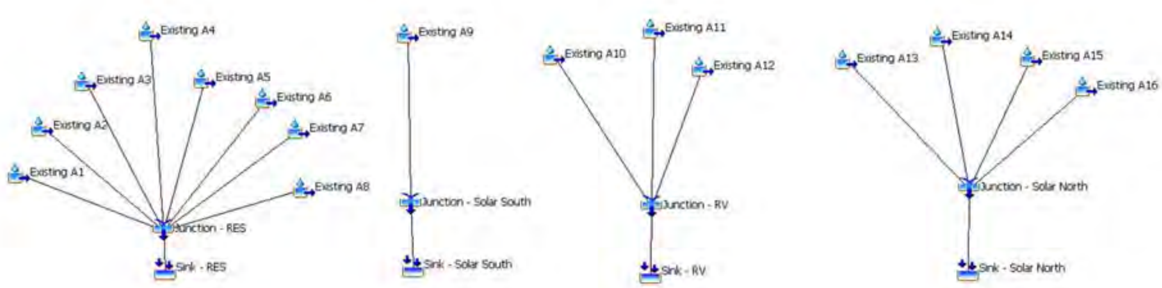
- Peak Water level @ RL: 66.92m
- Peak Discharge is 7.8m³/s
- Max Peak required is 9.19m³/s therefore OK

Matamata Developments Limited

Date: 30 May 2025

Ashbourne
HEC HMS Existing Inputs and Results

	AREA km2	CN	IMPERVIOUS %
A1	0.024	61	0
A2	0.0869	61	0
A3	0.0051	61	0
A4	0.033	61	0
A5	0.0047	61	0
A6	0.0927	61	0
A7	0.285	61	30
A8	0.0054	61	0
A9	1.21	61	0
A10	0.1	61	0
A11	0.00412	61	0
A12	0.17	61	0
A13	0.01	61	0
A14	0.0233	61	10
A15	0.179	61	0
A16	0.0066	61	0



Summary Results for Subbasin "Existing A1"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A1

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: MM 1000 M3

Computed Results

Peak Discharge:	0.083 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:22
Precipitation Volume:	2.0 (1000 M3)	Direct Runoff Volume:	0.6 (1000 M3)
Loss Volume:	1.4 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	0.6 (1000 M3)	Discharge Volume:	0.6 (1000 M3)

Summary Results for Subbasin "Existing A2"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A2

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: MM 1000 M3

Computed Results

Peak Discharge:	0.217 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:39
Precipitation Volume:	7.2 (1000 M3)	Direct Runoff Volume:	2.2 (1000 M3)
Loss Volume:	5.0 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	2.2 (1000 M3)	Discharge Volume:	2.2 (1000 M3)

Summary Results for Subbasin "Existing A3"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A3

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: MM 1000 M3

Computed Results

Peak Discharge:	0.021 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:16
Precipitation Volume:	0.4 (1000 M3)	Direct Runoff Volume:	0.1 (1000 M3)
Loss Volume:	0.3 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	0.1 (1000 M3)	Discharge Volume:	0.1 (1000 M3)

Summary Results for Subbasin "Existing A4"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A4

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.098 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:28
Precipitation Volume:	2.7 (1000 M3)	Direct Runoff Volume:	0.8 (1000 M3)
Loss Volume:	1.9 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	0.8 (1000 M3)	Discharge Volume:	0.8 (1000 M3)

Summary Results for Subbasin "Existing A5"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A5

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.019 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:16
Precipitation Volume:	0.4 (1000 M3)	Direct Runoff Volume:	0.1 (1000 M3)
Loss Volume:	0.3 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	0.1 (1000 M3)	Discharge Volume:	0.1 (1000 M3)

Summary Results for Subbasin "Existing A6"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A6

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.240 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:37
Precipitation Volume:	7.7 (1000 M3)	Direct Runoff Volume:	2.3 (1000 M3)
Loss Volume:	5.3 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	2.3 (1000 M3)	Discharge Volume:	2.3 (1000 M3)

Summary Results for Subbasin "Existing A7"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A7

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.910 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:58
Precipitation Volume:	23.7 (1000 M3)	Direct Runoff Volume:	11.9 (1000 M3)
Loss Volume:	11.5 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	12.1 (1000 M3)	Discharge Volume:	11.9 (1000 M3)

Summary Results for Subbasin "Existing A8"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A8

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.022 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:16
Precipitation Volume:	0.4 (1000 M3)	Direct Runoff Volume:	0.1 (1000 M3)
Loss Volume:	0.3 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	0.1 (1000 M3)	Discharge Volume:	0.1 (1000 M3)

Summary Results for Subbasin "Existing A9"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A9

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	1.968 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 13:24
Precipitation Volume:	100.4 (1000 M3)	Direct Runoff Volume:	29.3 (1000 M3)
Loss Volume:	69.8 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	30.6 (1000 M3)	Discharge Volume:	29.3 (1000 M3)

Summary Results for Subbasin "Existing A10"

Project: Basin A to D   Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A10

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.255 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:38
Precipitation Volume:	8.3 (1000 M3)	Direct Runoff Volume:	2.5 (1000 M3)
Loss Volume:	5.8 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	2.5 (1000 M3)	Discharge Volume:	2.5 (1000 M3)

Summary Results for Subbasin "Existing A11"

Project: Basin A to D   Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A11

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.015 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:20
Precipitation Volume:	0.3 (1000 M3)	Direct Runoff Volume:	0.1 (1000 M3)
Loss Volume:	0.2 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	0.1 (1000 M3)	Discharge Volume:	0.1 (1000 M3)

Summary Results for Subbasin "Existing A12"

Project: Basin A to D   Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A12

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.342 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:58
Precipitation Volume:	14.1 (1000 M3)	Direct Runoff Volume:	4.2 (1000 M3)
Loss Volume:	9.8 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	4.3 (1000 M3)	Discharge Volume:	4.2 (1000 M3)

Summary Results for Subbasin "Existing A13"

Project: Basin A to D   Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A13

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.025 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:38
Precipitation Volume:	0.8 (1000 M3)	Direct Runoff Volume:	0.2 (1000 M3)
Loss Volume:	0.6 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	0.3 (1000 M3)	Discharge Volume:	0.2 (1000 M3)

Summary Results for Subbasin "Existing A14"

Project: Basin A to D   Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A14

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.073 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:36
Precipitation Volume:	1.9 (1000 M3)	Direct Runoff Volume:	0.7 (1000 M3)
Loss Volume:	1.2 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	0.7 (1000 M3)	Discharge Volume:	0.7 (1000 M3)

Summary Results for Subbasin "Existing A15"

Project: Basin A to D   Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A15

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.388 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:50
Precipitation Volume:	14.9 (1000 M3)	Direct Runoff Volume:	4.4 (1000 M3)
Loss Volume:	10.3 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	4.5 (1000 M3)	Discharge Volume:	4.4 (1000 M3)

Summary Results for Subbasin "Existing A16"

Project: Basin A to D Simulation Run: HEC RAS 100 year pre -
Subbasin: Existing A16

Start of Run:	04Apr2017, 00:00	Basin Model:	New Existing Catchements
End of Run:	05Apr2017, 00:00	Meteorologic Model:	100 year pre
Compute Time:	DATA CHANGED, RECOMPUTE	Control Specifications:	Control 1

Volume Units: ○ MM ● 1000 M3

Computed Results

Peak Discharge:	0.024 (M3/S)	Date/Time of Peak Discharge:	04Apr2017, 12:20
Precipitation Volume:	0.5 (1000 M3)	Direct Runoff Volume:	0.2 (1000 M3)
Loss Volume:	0.4 (1000 M3)	Baseflow Volume:	0.0 (1000 M3)
Excess Volume:	0.2 (1000 M3)	Discharge Volume:	0.2 (1000 M3)

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title: Ashbourne RG - A Calc Title: Pre-development		Author MKS	Date 13/03/2025	Checked DJM

Eastern Rain Gardens
Total Catchment 27600

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B		Open Space (Sandy Loam or Silty Loam)	61	2.76	168.36
			Totals =	2.76	168.36

CN (weighted) = total product / total area = 168.36 / 2.760 = **61.0**

S= (1000 / CN -10) * 25.4 (1000 - 10) *2 66 / 61.0 = **162.4** mm

Ia= 0.05*S 0.05x 162.4 = **8.1** mm

2. Time of Concentration

Sheet and Shallow Flow (to be completed at detail design)
$T=100nL^{0.33}/S^{0.2}$
L= m
S= %
n=
T= 0.000 mins

Concentrated Network Flow (to be completed at detail design) 0

Open Channel Flow $V=R^{2/3}S^{1/2}/n$ w= 1 n= 0.04
R= A / Wp 0.33 d= 1 s= 1%
l= 50
(to be completed at detail design)
V= 1.201874642 m/s
T= 0.69 mins

$T_c = T_{sheet\ flow} + T_{concentrated\ flow} + T_{open\ channel\ flow}$

= 0 + 0.00 + 0.69 = 0.693 mins
0.011556021 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.008 hrs

NO GOOD use **0.17** hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 2	Rev: A
Job Title: Ashbourne RG - A Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

Ea Data

Catchment Area	A=	0.0276	km2(100ha =1km2)
Runoff curve number	CN=	61.0	(from worksheet 1)
Initial abstraction	Ia=	8.1	mm (from worksheet 1)
Catchment R1 (Impervious Area)			98
Time of conc Catchment R1 (pervious Area)			74

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24 *as per HIRDS RCP6.0 for the period 2081-2100	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.047		0.199	0.295	0.408	
6. Specific peak flow rate q*	0.015		0.059	0.083	0.108	
7. Peak flow rate, q_p=q*A*P_{24}	**0.014**	m^3/s				m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	3.1		31.4	67.6	136.4	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	87	m^3	**867.71**	**1865.53**	**3763.91**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 3	Rev: A
Job Title Calc Title	Ashbourne RG - A Post development (Pervious)	Author MKS	Date 13/03/2025	Checked DJM

Wetland **0**
Total Catchmer 27600

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (10%)	74	0.276	20.42
					0.00
			Totals =	0.276	20.42

CN (weighted) = total product = / total area = 20.42 / 0.276 = **74.0**

S= (1000 / CN -10) * 25.4 (1000 - 10) *25.4 / 74.0 = **89.2** mm

Ia= 0.05*S 0.05x 89.2 = **4.5** mm

2. Time of Concentration

Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 4	Rev: A
Job Title: Ashbourne RG - A Calc Title: Post development (Pervious)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.00276	km2(100ha =1km2)
Runoff curve number	CN=	74.0	(from worksheet 1)
Initial abstraction	Ia=	4.5	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 89 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations			
6. Specific peak flow rate q*			Unnecessary for volume calculations			
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations			m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	6.6		47.1	91.9	170.8	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	18	m^3	**130.02**	**253.73**	**471.45**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 5	Rev: A
Job Title Calc Title	Ashbourne RG - A Post development (Impervious)	Author MKS	Date 13/03/2025	Checked DJM

Wetland **0**
Total Catchmer 27600

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Impervious	98	2.48	243.43
					0.00
			Totals =	2.48	243.43

CN (weighted) = $\frac{\text{total product}}{\text{total area}}$ = $\frac{243.43}{2.484}$ = **98.0**

S= $(\frac{1000}{CN} - 10) * 25.4$ $(\frac{1000}{98.0} - 10) * 25.4$ = **5.2** mm

Ia= 0.05*S 0.05x 5.2 = **0.3** mm

2. Time of Concentration
Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

	Maven Associates	Job Number 289001.00	Sheet 6	Rev: A
Job Title Calc Title	Ashbourne RG - A Post development (Impervious)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.0248	km2(100ha =1km2)
Runoff curve number	CN=	98.0	(from worksheet 1)
Initial abstraction	Ia=	0.3	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 5 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations			
6. Specific peak flow rate q*			Unnecessary for volume calculations			
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations			m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	27.6		91.8	146.7	234.7	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	686	m^3	**2280.83**	**3644.73**	**5829.12**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates	Job Number 289001	Sheet 7	Rev: A
Job Title: Ashbourne RG - A Calc Title: Post development (whole site)	Author MKS	Date 13/03/2025	Checked DJM

Wetland **0**
Total Catchmer 27600

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Impervious (90%)	98	2.48	243.43
C		Pervious (10%)	74	0.28	20.42
			Totals =	2.76	263.86

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{263.86}{2.760}$ = **95.6**

S= $\left(\frac{1000}{CN} - 10\right) * 25.4$ $\left(\frac{1000}{95.6} - 10\right) * 25.4$ = **11.7** mm

Ia= 0.05*S 0.05x 11.7 = **0.6** mm

2. Time of Concentration

Sheet and Shallow Flow (to be completed at detail design)
$T=100nL^{0.33}/S^{0.2}$

L= 629 m
S= 5.00 %
n= 0.015
T= 9.117 mins

Concentrated Network Flow (to be completed at detail design) 0

Open Channel Flow $V=R^{2/3}S^{1/2}/n$
R= A / Wp 0.33

w= 1 n= 0.04
d= 1 s= 2%
l= 500

(to be completed at detail design)

V= 1.6997074 m/s
T= 4.90 mins

$T_c = T_{\text{sheet flow}} + T_{\text{concentrated flow}} + T_{\text{open channel flow}}$

= 9.117 + 0.00 + 4.90 = 14.020 mins
0.2336634 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\, t_c$ = 0.157 hrs

NO GOOD use **0.17** hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates		Job Number 289001	Sheet 8	Rev: A
Job Title Calc Title	Ashbourne RG - A Post development (whole site)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.0276	km2(100ha =1km2)
Runoff curve number	CN=	95.6	(from worksheet 1)
Initial abstraction	Ia=	0.6	mm (from worksheet 1)
Time of concentration	tc=	0.17	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 12 mm

		WQV				
3. Average recurrence interval, ARI	1/4 of 2yr	1/3 of 2yr	2	10	100	yr
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	24.25	32.3	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.497	0.571	0.804	0.866	0.911	
6. Specific peak flow rate q*	0.124	0.135	0.160	0.163	0.165	
7. Peak flow rate, q_p=q**A*P_{24}	**0.083**	**0.121**	**0.428**	**0.685**	**1.092**	m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$						
9. Runoff volume, $V_{24} = 1000xQ_{24}A$						m^3

	2	10	100	
Pre development run off volume	867.71	1865.53	3763.91	m^3
Post development run off volume - Pervious	130.02	253.73	471.45	m^3
Post development run off volume - Impervious	2280.83	3644.73	5829.12	m^3
Post development run off volume - total	2410.84	3898.46	6300.58	m^3
Pre development flow rate	0.16	0.35	0.71	m^3/s
Post development flow rate	0.43	0.68	1.09	m^3/s

Worksheet 2: Graphical Peak Flow Rate

Maven Associates	Job Number 289001	Sheet 9	Rev: A
Job Title Ashbourne RG - A Calc Title WQV and ED	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Runoff volume (pervious)	V_p=	18	m^3
Runoff volume (impervious)	V_{ip}=	686	m^3
Combined volume	V=	704	m^3
Pre-development initial abstration	I_{a1}=	8.1	mm
Post-development compacted pervious area CN		74	Class C
Post-development initial abstration of	I_{a2}=	4.5	mm
Post development Impervious area	A_{ip}=	2.48	ha.
Post development compacted perviou	A_{pp}=	0.28	ha.

2. Retention reduction

Impervious surface retention	Vrip=	201.7	m^3
Pervious surface retention	Vrp=	10.1	m^3

3. Water Quality Volume	WQV=	492	m^3
4. Extended Detention Volume	ED=	591	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title: Ashbourne RG - B Calc Title: Pre-development		Author MKS	Date 13/03/2025	Checked DJM

Eastern Rain Gardens
Total Catchment 59600

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B		Open Space (Sandy Loam or Silty Loam)	61	5.96	363.56
			Totals =	5.96	363.56

CN (weighted) = total product / total area = 363.56 / 5.960 = **61.0**

S= $\left(\frac{1000}{CN} - 10\right) * 25.4$ (1000 - 10) *2 66 / 61.0 = **162.4** mm

Ia= 0.05*S   0.05x 162.4 = **8.1** mm

2. Time of Concentration

Sheet and Shallow Flow (to be completed at detail design)

$T=100nL^{0.33}/S^{0.2}$

L= m
S= %
n=
T= 0.000 mins

Concentrated Network Flow (to be completed at detail design) 0

Open Channel Flow $V=R^{2/3}S^{1/2}/n$

R= A / Wp 0.33

w= 1   n= 0.04
d= 1   s= 1%
l= 50

(to be completed at detail design)

V= 1.201874642 m/s
T= 0.69 mins

$Tc = T_{sheet\ flow} + T_{concentrated\ flow} + T_{open\ channel\ flow}$

= 0 + 0.00 + 0.69 = 0.693 mins

0.011556021 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.008 hrs

NO GOOD use **0.17** hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 2	Rev: A
Job Title: Ashbourne RG - B Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

Ea Data

Catchment Area A= 0.0596 km2(100ha =1km2)

Runoff curve number CN= 61.0 (from worksheet 1)

Initial abstraction Ia= 8.1 mm (from worksheet 1)

Catchment R1 (Impervious Area) 98

Time of con(Catchment R1 (pervious Area) 74

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24 *as per HIRDS RCP6.0 for the period 2081-2100	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.047		0.199	0.295	0.408	
6. Specific peak flow rate q*	0.015		0.059	0.083	0.108	
7. Peak flow rate, q_p=q*A*P_{24}	**0.029**	m^3/s				m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	3.1		31.4	67.6	136.4	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	187	m^3	**1873.74**	4028.46	8127.86	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 3	Rev: A
Job Title / Calc Title	Ashbourne RG - B / Post development (Pervious)	Author MKS	Date 13/03/2025	Checked DJM

Wetland 0
Total Catchmer 59600

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (10%)	74	0.596	44.10
					0.00
			Totals =	0.596	44.10

CN (weighted) = total product = / total area = 44.10 / 0.596 = **74.0**

S= (1000 / CN -10) * 25.4   (1000 - 10) *25.4 / 74.0 = **89.2** mm

Ia= 0.05*S   0.05x 89.2 = **4.5** mm

2. Time of Concentration

Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 4	Rev: A
Job Title: Ashbourne RG - B Calc Title: Post development (Pervious)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.00596	km2(100ha =1km2)
Runoff curve number	CN=	74.0	(from worksheet 1)
Initial abstraction	Ia=	4.5	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 89 mm

	WQV		2	10	100	
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations			
6. Specific peak flow rate q*			Unnecessary for volume calculations			
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations			m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	6.6		47.1	91.9	170.8	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	40	m^3	**280.76**	**547.91**	**1018.07**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 5	Rev: A
Job Title Ashbourne RG - B Calc Title Post development (Impervious)		Author MKS	Date 13/03/2025	Checked DJM

Wetland **0**
Total Catchmer 59600

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Impervious	98	5.36	525.67
					0.00
			Totals =	5.36	525.67

CN (weighted) = total product = / total area = 525.67 / 5.364 = **98.0**

S= (1000 / CN -10) * 25.4 (1000 - 10) *25.4 / 98.0 = **5.2** mm

Ia= 0.05*S 0.05x 5.2 = **0.3** mm

2. Time of Concentration

Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 6	Rev: A
Job Title: Ashbourne RG - B Calc Title: Post development (Impervious)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.05364	km2(100ha =1km2)
Runoff curve number	CN=	98.0	(from worksheet 1)
Initial abstraction	Ia=	0.3	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 5 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations			
6. Specific peak flow rate q*			Unnecessary for volume calculations			
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations			m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	27.6		91.8	146.7	234.7	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	1481	m^3	**4925.27**	**7870.51**	**12587.53**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates	Job Number 289001	Sheet 7	Rev: A
Job Title: Ashbourne RG - B Calc Title: Post development (whole site)	Author MKS	Date 13/03/2025	Checked DJM

Wetland 0
Total Catchmer 59600

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Impervious (90%)	98	5.36	525.67
C		Pervious (10%)	74	0.60	44.10
			Totals =	5.96	569.78

CN (weighted) = total product = / total area = 569.78 / 5.960 = **95.6**

S= (1000 / CN -10) * 25.4 (1000 - 10) *25.4 / 95.6 = **11.7** mm

Ia= 0.05*S 0.05x 11.7 = **0.6** mm

2. Time of Concentration

Sheet and Shallow Flow (to be completed at detail design)
$T=100nL^{0.33}/S^{0.2}$
L= 629 m
S= 5.00 %
n= 0.015
T= 9.117 mins

Concentrated Network Flow (to be completed at detail design) 0

Open Channel Flow $V=R^{2/3}S^{1/2}/n$
R= A / Wp 0.33
w= 1 n= 0.04
d= 1 s= 2%
l= 500
(to be completed at detail design)
V= 1.6997074 m/s
T= 4.90 mins

$Tc = T_{sheet\ flow} + T_{concentrated\ flow} + T_{open\ channel\ flow}$

= 9.117 + 0.00 + 4.90 = 14.020 mins
0.2336634 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.157 hrs

NO GOOD use **0.17** hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 8	Rev: A
Job Title: Ashbourne RG - B Calc Title: Post development (whole site)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.0596	km2(100ha =1km2)
Runoff curve number	CN=	95.6	(from worksheet 1)
Initial abstraction	Ia=	0.6	mm (from worksheet 1)
Time of concentration	tc=	0.17	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 12 mm

		WQV				
3. Average recurrence interval, ARI	1/4 of 2yr	1/3 of 2yr	2	10	100	yr
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	24.25	32.3	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.497	0.571	0.804	0.866	0.911	
6. Specific peak flow rate q*	0.124	0.135	0.160	0.163	0.165	
7. Peak flow rate, q_p=q*A*P_{24}	**0.179**	**0.261**	**0.923**	**1.478**	**2.359**	m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$						
9. Runoff volume, $V_{24} = 1000xQ_{24}A$						m^3

	2	10	100	
Pre development run off volume	1873.74	4028.46	8127.86	m^3
Post development run off volume - Pervious	280.76	547.91	1018.07	m^3
Post development run off volume - Impervious	4925.27	7870.51	12587.53	m^3
Post development run off volume - total	5206.03	8418.42	13605.60	m^3

	2	10	100	
Pre development flow rate	0.34	0.75	1.54	m^3/s
Post development flow rate	0.92	1.48	2.36	m^3/s

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 9	Rev: A
Job Title Calc Title	Ashbourne RG - B WQV and ED	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Runoff volume (pervious)	V_p=	40	m^3	
Runoff volume (impervious)	V_{ip}=	1481	m^3	
Combined volume	V=	1521	m^3	
Pre-development initial abstration	I_{a1}=	8.1	mm	
Post-development compacted pervious area CN		74		Class C
Post-development initial abstration of	I_{a2}=	4.5	mm	
Post development Impervious area	A_{ip}=	5.36	ha.	
Post development compacted perviou	A_{pp}=	0.60	ha.	

2. Retention reduction

Impervious surface retention	Vrip=	435.5	m^3
Pervious surface retention	Vrp=	21.8	m^3

3. Water Quality Volume	WQV=	1063	m^3
4. Extended Detention Volume	ED=	1276	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title: Ashbourne RG - C Calc Title: Pre-development		Author MKS	Date 13/03/2025	Checked DJM

Eastern Rain Gardens
Total Catchment 10700

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B		Open Space (Sandy Loam or Silty Loam)	61	1.07	65.27
			Totals =	1.07	65.27

CN (weighted) = total product / total area = 65.27 / 1.070 = **61.0**

S= $\left(\frac{1000}{CN} - 10\right) * 25.4$ (1000 - 10) *2 66 / 61.0 = **162.4** mm

Ia= 0.05*S 0.05x 162.4 = **8.1** mm

2. Time of Concentration

Sheet and Shallow Flow (to be completed at detail design)
$T=100nL^{0.33}/S^{0.2}$
L= m
S= %
n=
T= 0.000 mins

Concentrated Network Flow (to be completed at detail design) 0

Open Channel Flow $V=R^{2/3}S^{1/2}/n$
R= A / Wp 0.33
w= 1 n= 0.04
d= 1 s= 1%
l= 50
(to be completed at detail design)
V= 1.201874642 m/s
T= 0.69 mins

$T_c = T_{sheet\ flow} + T_{concentrated\ flow} + T_{open\ channel\ flow}$

= 0 + 0.00 + 0.69 = 0.693 mins
0.011556021 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.008 hrs

NO GOOD use **0.17** hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 2	Rev: A
Job Title: Ashbourne RG - C Calc Title: Pre-development	Author MKS	Date 13/03/2025	Checked DJM

Ea Data

Catchment Area A= 0.0107 km2(100ha =1km2)

Runoff curve number CN= 61.0 (from worksheet 1)

Initial abstraction Ia= 8.1 mm (from worksheet 1)

Catchment R1 (Impervious Area) 98

Time of con‹ Catchment R1 (pervious Area) 74

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24	32.3	(mm)	97	152	240	(mm)
*as per HIRDS RCP6.0 for the period 2081-2100						
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.047		0.199	0.295	0.408	
6. Specific peak flow rate q*	0.015		0.059	0.083	0.108	
7. Peak flow rate, q_p=q*A*P_{24}	**0.005**	m^3/s				m^3/s
8. Runoff depth, Q_{24} = $(P_{24}-Ia)^2/(P_{24}-Ia)+S$	3.1		31.4	67.6	136.4	
9. Runoff volume, V_{24} = 1000xQ_{24}A	34	m^3	**336.39**	723.23	1459.20	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 3	Rev: A
Job Title / Calc Title	Ashbourne RG - C / Post development (Pervious)	Author MKS	Date 13/03/2025	Checked DJM

Wetland **0**
Total Catchmer 10700

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (10%)	74	0.11	7.92
					0.00
			Totals =	0.11	7.92

CN (weighted) = total product = / total area = 7.92 / 0.107 = **74.0**

S= (1000 / CN -10) * 25.4 (1000 - 10) *25.4 / 74.0 = **89.2** mm

Ia= 0.05*S 0.05x 89.2 = **4.5** mm

2. Time of Concentration
Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 4	Rev: A
Job Title Ashbourne RG - C Calc Title Post development (Pervious)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.00107	km2(100ha =1km2)
Runoff curve number	CN=	74.0	(from worksheet 1)
Initial abstraction	Ia=	4.5	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 89 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations			
6. Specific peak flow rate q*			Unnecessary for volume calculations			
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations			m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	6.6		47.1	91.9	170.8	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	7	m^3	**50.41**	**98.37**	**182.77**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 5	Rev: A
Job Title Calc Title	Ashbourne RG - C Post development (Impervious)	Author MKS	Date 13/03/2025	Checked DJM

Wetland **0**
Total Catchmer 10700

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Impervious	98	0.96	94.37
					0.00
			Totals =	0.96	94.37

CN (weighted) = total product = / total area = 94.37 / 0.963 = **98.0**

S= (1000 / CN -10) * 25.4 (1000 - 10) *25.4 / 98.0 = **5.2** mm

Ia= 0.05*S 0.05x 5.2 = **0.3** mm

2. Time of Concentration

Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 6	Rev: A
Job Title: Ashbourne RG - C Calc Title: Post development (Impervious)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.00963	km2(100ha =1km2)
Runoff curve number	CN=	98.0	(from worksheet 1)
Initial abstraction	Ia=	0.3	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 5 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations			
6. Specific peak flow rate q*			Unnecessary for volume calculations			
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations			m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	27.6		91.8	146.7	234.7	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	266	m^3	**884.23**	**1412.99**	**2259.84**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates	Job Number 289001	Sheet 7	Rev: A
Job Title: Ashbourne RG - C Calc Title: Post development (whole site)	Author MKS	Date 13/03/2025	Checked DJM

Wetland 0
Total Catchmer 10700

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Impervious (90%)	98	0.96	94.37
C		Pervious (10%)	74	0.11	7.92
			Totals =	1.07	102.29

CN (weighted) = total product / total area = 102.29 / 1.070 = **95.6**

S= (1000 / CN -10) * 25.4 (1000 / 95.6 - 10) *25.4 = **11.7** mm

Ia= 0.05*S 0.05x 11.7 = **0.6** mm

2. Time of Concentration

Sheet and Shallow Flow (to be completed at detail design)
$T=100nL^{0.33}/S^{0.2}$

L= 629 m
S= 5.00 %
n= 0.015
T= 9.117 mins

Concentrated Network Flow (to be completed at detail design) 0

Open Channel Flow $V=R^{2/3}S^{1/2}/n$
R= A / Wp 0.33
w= 1 n= 0.04
d= 1 s= 2%
l= 500

(to be completed at detail design)

V= 1.6997074 m/s
T= 4.90 mins

$Tc = T_{sheet\ flow} + T_{concentrated\ flow} + T_{open\ channel\ flow}$

= 9.117 + 0.00 + 4.90 = 14.020 mins
0.2336634 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.157 hrs

NO GOOD use **0.17** hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 8	Rev: A
Job Title: Ashbourne RG - C Calc Title: Post development (whole site)	Author MKS	Date 13/03/2025	Checked DJM

1. Data
   - Catchment Area A= 0.0107 km2(100ha =1km2)
   - Runoff curve number CN= 95.6 (from worksheet 1)
   - Initial abstraction Ia= 0.6 mm (from worksheet 1)
   - Time of concentration tc= 0.17 hrs (from worksheet 1)
2. Calculate storage, S =(1000/CN - 10)25.4 = 12 mm

		WQV				
3. Average recurrence interval, ARI	1/4 of 2yr	1/3 of 2yr	2	10	100	yr
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	24.25	32.3	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.497	0.571	0.804	0.866	0.911	
6. Specific peak flow rate q*	0.124	0.135	0.160	0.163	0.165	
7. Peak flow rate, q_p=q*A*P_{24}	**0.032**	**0.047**	**0.166**	**0.265**	**0.424**	m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$						
9. Runoff volume, $V_{24} = 1000xQ_{24}A$						m^3

Pre development run off volume	336.39	723.23	1459.20	m^3
Post development run off volume - Pervious	50.41	98.37	182.77	m^3
Post development run off volume - Impervious	884.23	1412.99	2259.84	m^3
Post development run off volume - total	934.64	1511.36	2442.62	m^3

Pre development flow rate	0.06	0.14	0.28	m^3/s
Post development flow rate	0.17	0.27	0.42	m^3/s

Worksheet 2: Graphical Peak Flow Rate

Maven Associates	Job Number 289001	Sheet 9	Rev: A
Job Title: Ashbourne RG - C Calc Title: WQV and ED	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Runoff volume (pervious)	V_p=	7	m^3
Runoff volume (impervious)	V_{ip}=	266	m^3
Combined volume	V=	273	m^3
Pre-development initial abstration	I_{a1}=	8.1	mm
Post-development compacted pervious area CN		74	Class C
Post-development initial abstration of	I_{a2}=	4.5	mm
Post development Impervious area	A_{ip}=	0.96	ha.
Post development compacted perviou	A_{pp}=	0.11	ha.

2. Retention reduction

Impervious surface retention	Vrip=	78.2	m^3
Pervious surface retention	Vrp=	3.9	m^3

3. Water Quality Volume	WQV=	191	m^3
4. Extended Detention Volume	ED=	229	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 1	Rev: A
Job Title Ashbourne RG - D Calc Title Pre-development		Author MKS	Date 13/03/2025	Checked DJM

Eastern Rain Gardens
Total Catchment 21700

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1ha	Product of CN x area
B		Open Space (Sandy Loam or Silty Loam)	61	2.17	132.37
			Totals =	2.17	132.37

CN (weighted) = total product / total area = 132.37 / 2.170 = **61.0**

S= (1000 / CN -10) * 25.4 (1000 - 10) *2 66 / 61.0 = **162.4** mm

Ia= 0.05*S 0.05x 162.4 = **8.1** mm

2. Time of Concentration

Sheet and Shallow Flow (to be completed at detail design)

$T=100nL^{0.33}/S^{0.2}$

L= m
S= %
n=
T= 0.000 mins

Concentrated Network Flow (to be completed at detail design) 0

Open Channel Flow $V=R^{2/3}S^{1/2}/n$ w= 1 n= 0.04
R= A / Wp 0.33 d= 1 s= 1%
l= 50

(to be completed at detail design)

V= 1.201874642 m/s
T= 0.69 mins

$Tc = T_{sheet\ flow} + T_{concentrated\ flow} + T_{open\ channel\ flow}$

= 0 + 0.00 + 0.69 = 0.693 mins
0.011556021 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.008 hrs

NO GOOD use **0.17** hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 2	Rev: A
Job Title Ashbourne RG - D Calc Title Pre-development	Author MKS	Date 13/03/2025	Checked DJM

Ea Data

Catchment Area A= 0.0217 km2(100ha =1km2)

Runoff curve number CN= 61.0 (from worksheet 1)

Initial abstraction Ia= 8.1 mm (from worksheet 1)

Catchment R1 (Impervious Area) 98

Time of con Catchment R1 (pervious Area) 74

2. Calculate storage, S =(1000/CN - 10)25.4 = 162 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24 *as per HIRDS RCP6.0 for the period 2081-2100	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.047		0.199	0.295	0.408	
6. Specific peak flow rate q*	0.015		0.059	0.083	0.108	
7. Peak flow rate, q_p=q*A*P_{24}	**0.011**	m^3/s				m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	3.1		31.4	67.6	136.4	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	68	m^3	**682.22**	**1466.74**	**2959.30**	m^3

Worksheet 2: Graphical Peak Flow Rate

	Maven Associates		Job Number 289001	Sheet 3	Rev: A
Job Title Calc Title	Ashbourne RG - D Post development (Pervious)		Author MKS	Date 13/03/2025	Checked DJM

Wetland **0**
Total Catchmer 21700

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Pervious (10%)	74	0.217	16.06
					0.00
			Totals =	0.217	16.06

CN (weighted) = total product / total area = 16.06 / 0.217 = **74.0**

S= (1000 / CN -10) * 25.4 (1000 - 10) *25.4 / 74.0 = **89.2** mm

Ia= 0.05*S 0.05x 89.2 = **4.5** mm

2. Time of Concentration
Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 4	Rev: A
Job Title Ashbourne RG - D Calc Title Post development (Pervious)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.00217	km2(100ha =1km2)
Runoff curve number	CN=	74.0	(from worksheet 1)
Initial abstraction	Ia=	4.5	mm (from worksheet 1)
Time of concentration	tc=	0.00	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 89 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations			
6. Specific peak flow rate q*			Unnecessary for volume calculations			
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations			m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	6.6		47.1	91.9	170.8	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	14	m^3	**102.22**	**199.49**	**370.67**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates		Job Number 289001	Sheet 5	Rev: A
Job Title / Calc Title	Ashbourne RG - D / Post development (Impervious)	Author MKS	Date 13/03/2025	Checked DJM

Wetland 0
Total Catchmer 21700

1. Runoff Curve Number (CN) and initial Abstraction (Ia)

CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Impervious	98	1.95	191.39
					0.00
			Totals =	1.95	191.39

CN (weighted) = total product = / total area — 191.39 / 1.953 = **98.0**

S= (1000 / CN -10) * 25.4 — (1000 - 10) *25.4 / 98.0 = **5.2** mm

Ia= 0.05*S — 0.05x 5.2 = **0.3** mm

2. Time of Concentration

Unnecessary for volume calculations

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 6	Rev: A
Job Title: Ashbourne RG - D Calc Title: Post development (Impervious)	Author MKS	Date 13/03/2025	Checked DJM

1. Data
   Catchment Area A= 0.01953 km2(100ha =1km2)

   Runoff curve number CN= 98.0 (from worksheet 1)

   Initial abstraction Ia= 0.3 mm (from worksheet 1)

   Time of concentration tc= 0.00 hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 5 mm

	WQV					
3. Average recurrence interval, ARI	1/3 of 2yr	(yr)	2	10	100	yr
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	32.3	(mm)	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S			Unnecessary for volume calculations			
6. Specific peak flow rate q*			Unnecessary for volume calculations			
7. Peak flow rate, q_p=q*A*P_{24}		m^3/s	Unnecessary for volume calculations			m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$	27.6		91.8	146.7	234.7	
9. Runoff volume, $V_{24} = 1000xQ_{24}A$	539	m^3	**1793.26**	**2865.61**	**4583.04**	m^3

Worksheet 2: Graphical Peak Flow Rate

Maven Associates	Job Number 289001	Sheet 7	Rev: A
Job Title: Ashbourne RG - D Calc Title: Post development (whole site)	Author MKS	Date 13/03/2025	Checked DJM

Wetland 0
Total Catchmer 21700

1. Runoff Curve Number (CN) and initial Abstraction (Ia) CALCS to WRC TR2020/06

Soil name and classification	ID	Cover description (cover type, treatment, and hydrologic condition)	Curve Number CN*	Area (ha) 10000m2=1 ha	Product of CN x area
C		Impervious (90%)	98	1.95	191.39
C		Pervious (10%)	74	0.22	16.06
			Totals =	2.17	207.45

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{207.45}{2.170}$ = **95.6**

S= $\left(\frac{1000}{CN} - 10\right) * 25.4$ $\left(\frac{1000}{95.6} - 10\right) * 25.4$ = **11.7** mm

Ia= 0.05*S   0.05x 11.7 = **0.6** mm

2. Time of Concentration

Sheet and Shallow Flow (to be completed at detail design)
$T = 100nL^{0.33}/S^{0.2}$

L= 629 m
S= 5.00 %
n= 0.015
T= 9.117 mins

Concentrated Network Flow (to be completed at detail design) 0

Open Channel Flow $V = R^{2/3}S^{1/2}/n$
R= A / Wp 0.33
w= 1 n= 0.04
d= 1 s= 2%
l= 500

(to be completed at detail design)

V= 1.6997074 m/s
T= 4.90 mins

$Tc = T_{sheet\ flow} + T_{concentrated\ flow} + T_{open\ channel\ flow}$

= 9.117 + 0.00 + 4.90 = 14.020 mins
0.2336634 hrs

SCS Lag for HEC-HMS.... $t_p = 2/3\ t_c$ = 0.157 hrs

NO GOOD use **0.17** hrs

Worksheet 1: Runoff Parameters and Time of Concentration

Maven Associates	Job Number 289001	Sheet 8	Rev: A
Job Title: Ashbourne RG - D Calc Title: Post development (whole site)	Author MKS	Date 13/03/2025	Checked DJM

1. Data

Catchment Area	A=	0.0217	km2(100ha =1km2)
Runoff curve number	CN=	95.6	(from worksheet 1)
Initial abstraction	Ia=	0.6	mm (from worksheet 1)
Time of concentration	tc=	0.17	hrs (from worksheet 1)

2. Calculate storage, S =(1000/CN - 10)25.4 = 12 mm

		WQV				
3. Average recurrence interval, ARI	1/4 of 2yr	1/3 of 2yr	2	10	100	yr
4. 24 hour rainfall depth, P24 as per HIRDS RCP 6.0 2081-2100 data	24.25	32.3	97	152	240	(mm)
5. Compute c* = P24 - 2Ia/P24 - 2Ia+2S	0.497	0.571	0.804	0.866	0.911	
6. Specific peak flow rate q*	0.124	0.135	0.160	0.163	0.165	
7. Peak flow rate, q_p=q*A*P_{24}	**0.065**	**0.095**	**0.336**	**0.538**	**0.859**	m^3/s
8. Runoff depth, $Q_{24} = (P_{24}-Ia)^2/(P_{24}-Ia)+S$						
9. Runoff volume, $V_{24} = 1000xQ_{24}A$						m^3

Pre development run off volume	682.22	1466.74	2959.30	m^3
Post development run off volume - Pervious	102.22	199.49	370.67	m^3
Post development run off volume - Impervious	1793.26	2865.61	4583.04	m^3
Post development run off volume - total	1895.48	3065.09	4953.72	m^3

Pre development flow rate	0.12	0.27	0.56	m^3/s
Post development flow rate	0.34	0.54	0.86	m^3/s

Worksheet 2: Graphical Peak Flow Rate

1. Data

Runoff volume (pervious)	V_p=	14	m^3	
Runoff volume (impervious)	V_{ip}=	539	m^3	
Combined volume	V=	554	m^3	
Pre-development initial abstration	I_{a1}=	8.1	mm	
Post-development compacted pervious area CN		74		Class C
Post-development initial abstration of	I_{a2}=	4.5	mm	
Post development Impervious area	A_{ip}=	1.95	ha.	
Post development compacted perviou	A_{pp}=	0.22	ha.	

2. Retention reduction

Impervious surface retention	Vrip=	158.6	m^3
Pervious surface retention	Vrp=	7.9	m^3

3. Water Quality Volume	WQV=	387	m^3
4. Extended Detention Volume	ED=	465	m^3

Worksheet 2: Graphical Peak Flow Rate

289001 – Ashbourne Soakage Calculation

1. The total effective area (per metre length of road)

= **Impervious Area + (0.3 * Pervious Area)**

Based on MPDC definition, we can calculate 2 effective areas for our design.

a. 20m wide road = 16.3 + (0.3*3.7) = **17.41m²**
b. 18m wide road = 14.6 + (0.3*3.4) = **15.62m²**

2. Design Soakage Rate

DSR of 4 Soakage holes from CMW Geotech report were calculated which are:
SO-01: 1.7 litres/min/m²
SO-02: 5.7 litres/min/m²
SO-03: 0.5 litres/min/m²
SO-04: 0.2 litres/min/m² -> But we go with 0.5 litres/min/m²

->Considering Test Pit Results, only 1.7 and 5.7 litres/min/m² will be used.

3. Maximum Impervious Area per Soakage Trench

There is no graph available for 10y 24h soakage trench with 1.2m width and 2.0m width, so we need a fair assumption for Maximum Impervious Area per soakage trench.

For 1.7 Design Soakage Rate (litres/min/m²)

0.4m wide	1m deep = 6m²
	2m deep = 12m²
0.8m wide	1m deep = 8m²
	2m deep = 16m²
1.2m wide	1m deep = 10.6m² (33.3% Increased per 0.4m)
	1.5m deep = 16m² (33.3% Increased per 0.4m)
	2m deep = 21.3m² (33.3% Increased per 0.4m)
2.0m wide	1m deep = 18.9m² (33.3% Increased per 0.4m)
	1.5m deep = 28.4m² (33.3% Increased per 0.4m)
	2m deep = 37.8m² (33.3% Increased per 0.4m)

For 5.7 Design Soakage Rate (litres/min/m²)

0.4m wide	1m deep = 9m²
	2m deep = 18m²
0.8m wide	1m deep = 13m²
	2m deep = 26m²
1.2m wide	1m deep = 18.7m² (44.4% Increased per 0.4m)
	1.5m deep = 28.2m² (44.4% Increased per 0.4m)
	2m deep = 37.5m² (44.4% Increased per 0.4m)
2.0m wide	1m deep = 39m² (44.4% Increased per 0.4m)
	1.5m deep = 58.7m² (44.4% Increased per 0.4m)
	2m deep = 78.2m² (44.4% Increased per 0.4m)

4. Comparison between Road and Soakage Trench

1.7 Design Soakage Rate (litres/min/m²) Zone:

2.0m wide and 1.5m deep soakage trench is recommended for 18m wide road, this will give us:

- 15.62 / 28.4 = 0.55 -> 1:0.55 ratio (18m wide road)

-> Every 1m long road construction, we need 0.55m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.28m long soakage trench in each side.

1.2m wide and 1.5m deep soakage trench is recommended for 20m wide road, this will give us:

- 17.41 / 16 = 1.1 -> 1:1.1 ratio (20m wide road)

-> Every 1m long road construction, we need 1.1m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.55m long soakage trench in each side.

5.7 Design Soakage Rate (litres/min/m²) Zone:

2.0m wide and 1.5m deep soakage trench is recommended for 18m wide road, this will give us:

- 15.62 / 58.7 = 0.27 -> 1:0.27 ratio (18m wide road)

-> Every 1m long road construction, we need 0.27m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.135m long soakage trench in each side.

1.2m wide and 1.5m deep soakage trench is recommended for 20m wide road, this will give us:

- 17.41 / 28.2 = 0.62 -> 1:0.62 ratio (20m wide road)

-> Every 1m long road construction, we need 0.62m long soakage trench to be constructed.

-> Every 1m long road construction, we need 0.31m long soakage trench in each side.

289001 – Ashbourne Soakage Calculation (E1/VM1)

1. Soakage Rate from the Percolation Test

- ➔ Soakage Rate 0.5l/min/m² -> 21mm/hr from CMW Geotech Report HAM2023-0124AB Rev1
- ➔ Soakage Rate 1.5l/min/m² -> 85mm/hr (half of 171mm/hr)
- ➔ Soakage Rate 3.0l/min/m² -> 171mm/hr from CMW Geotech Report HAM2023-0124AB Rev1

2. Stormwater Catchment Volume (Rc)

Formula: Rc = 10 * CIA

1. Assume: Impervious area: 150m²
2. Assume: Impervious area: 200m²
3. Assume: Impervious area: 250m

Run-off coefficient (C) = 0.9 from E1/VM1 Table 1 (Hard Surfaces)

Rain Intensity (I) = 48.8mm/hour (10-year, RCP 8.5)

1. Rc (10-year) = (10 * 0.9 * 48.8 * 0.015) = 6.588m³
2. Rc (10-year) = (10 * 0.9 * 48.8 * 0.02) = 8.784m³
3. Rc (10-year) = (10 * 0.9 * 48.8 * 0.025) = 10.98m³

3. Volume of Water Disposed by Soakage: V(soak)

Formula: V(soak) = Soakage Trench Area * Soakage Rate / 1000

Impervious area (150m²)

0.5 Soakage Rate (21mm/hr): Soakage Trench Size = 7m (L) * 2.44m (W) * 1m (H)

Soakage Trench Volume =

7 * 2.44 * 1.0 * 0.38 (void ratio) = 6.49m³

This gives V(soak): 7 * 2.44 * 21 /1000 = 0.36m³

1.5 Soakage Rate (85mm/hr): Soakage Trench Size = 6m (L) * 2.44m (W) * 1m (H)

Soakage Trench Volume =

6 * 2.44 * 1.0 * 0.38 (void ratio) = 5.56m³

This gives V(soak): 6 * 2.44 * 85 /1000 = 1.24m³

3.0 Soakage Rate (171mm/hr): Soakage Trench Size = 5m (L) * 2.44m (W) * 1m (H)

Soakage Trench Volume =

$5 * 2.44 * 1.0 * 0.38$ (void ratio) = $4.64m^3$

This gives V(soak): $5 * 2.44 * 171 /1000 = 2.086m^3$

Impervious area (200m²)

0.5 Soakage Rate (21mm/hr): Soakage Trench Size = 13.5m (L) * 1.65m (W) * 1m (H)

Soakage Trench Volume =

$13.5 * 1.65 * 1.0 * 0.38$ (void ratio) = $8.46m^3$

This gives V(soak): $13.5 * 1.65 * 21 /1000 = 0.47m^3$

1.5 Soakage Rate (85mm/hr): Soakage Trench Size = 11.5m (L) * 1.65m (W) * 1m (H)

Soakage Trench Volume =

$11.5 * 1.65 * 1.0 * 0.38$ (void ratio) = $7.21m^3$

This gives V(soak): $11.5 * 1.65 * 85 /1000 = 1.613m^3$

3.0 Soakage Rate (171mm/hr): Soakage Trench Size = 10m (L) * 1.65m (W) * 1m (H)

Soakage Trench Volume =

$10 * 1.65 * 1.0 * 0.38$ (void ratio) = $6.27m^3$

This gives V(soak): $10 * 1.65 * 171 /1000 = 2.82m^3$

Impervious area (250m²)

0.5 Soakage Rate (21mm/hr): Soakage Trench Size = 11.74m (L) * 1.6m (W) * 1.5m (H)

Soakage Trench Volume =

$11.74 * 1.6 * 1.5 * 0.38$ (void ratio) = $10.71m^3$

This gives V(soak): $11.74 * 1.6 * 21 /1000 = 0.39m^3$

1.5 Soakage Rate (85mm/hr): Soakage Trench Size = 11.74m (L) * 1.6m (W) * 1.4m (H)

Soakage Trench Volume =

11.74 * 1.6 * 1.4 * 0.38 (void ratio) = 9.99m^3

This gives V(soak): 11.74 * 1.6 * 85 /1000 = 1.60m^3

3.0 Soakage Rate (171mm/hr): Soakage Trench Size = 11.74m (L) * 1.6m (W) * 1.1m (H)

Soakage Trench Volume =

11.74 * 1.6 * 1.1 * 0.38 (void ratio) = 7.85m^3

This gives V(soak): 11.74 * 1.6 * 171 /1000 = 3.21m^3

4. Required Storage Volume: V(storage)

Formula: V(storage) = Rc – V(soak)

Impervious area (150m^2)

1. 21mm/hr

➔ Required 10-year V(storage) = 6.588 – 0.36 = 6.23m^3

➔ Proposed Soakage Trench Volume (6.49m^3) is larger than Required Volume (6.23m^3),

➔ Soakage trench sizing is OK

2. 85mm/hr

➔ Required 10-year V(storage) = 6.588 – 1.24 = 5.35m^3

➔ Proposed Soakage Trench Volume (5.56m^3) is larger than Required Volume (5.35m^3)

➔ Soakage trench sizing is OK

3. 171mm/hr

➔ Required 10-year V(storage) = 6.588 – 2.086 = 4.5m^3

➔ Proposed Soakage Trench Volume (4.64m^3) is larger than Required Volume (4.5m^3)

➔ Soakage trench sizing is OK

Impervious area (200m^2)

4. 21mm/hr

➔ Required 10-year V(storage) = 8.78 – 0.47 = 8.31m^3

➔ Proposed Soakage Trench Volume (8.46m^3) is larger than Required Volume (8.31m^3),

➔ Soakage trench sizing is OK

5. 85mm/hr

➔ Required 10-year V(storage) = 8.78 – 1.61 = 7.17m^3

- ➔ Proposed Soakage Trench Volume (7.21m^3) is larger than Required Volume (7.17m^3)
- ➔ Soakage trench sizing is OK

6. 171mm/hr

- ➔ Required 10-year V(storage) = 8.78 – 2.82 = 5.96m^3
- ➔ Proposed Soakage Trench Volume (6.27m^3) is larger than Required Volume (5.96m^3)
- ➔ Soakage trench sizing is OK

Impervious area (250m^2)

7. 21mm/hr

- ➔ Required 10-year V(storage) = 10.98 – 0.39 = 10.59m^3
- ➔ Proposed Soakage Trench Volume (10.71m^3) is larger than Required Volume (10.59m^3),
- ➔ Soakage trench sizing is OK

8. 85mm/hr

- ➔ Required 10-year V(storage) = 10.98 – 1.6 = 9.38m^3
- ➔ Proposed Soakage Trench Volume (9.99m^3) is larger than Required Volume (9.38m^3)
- ➔ Soakage trench sizing is OK

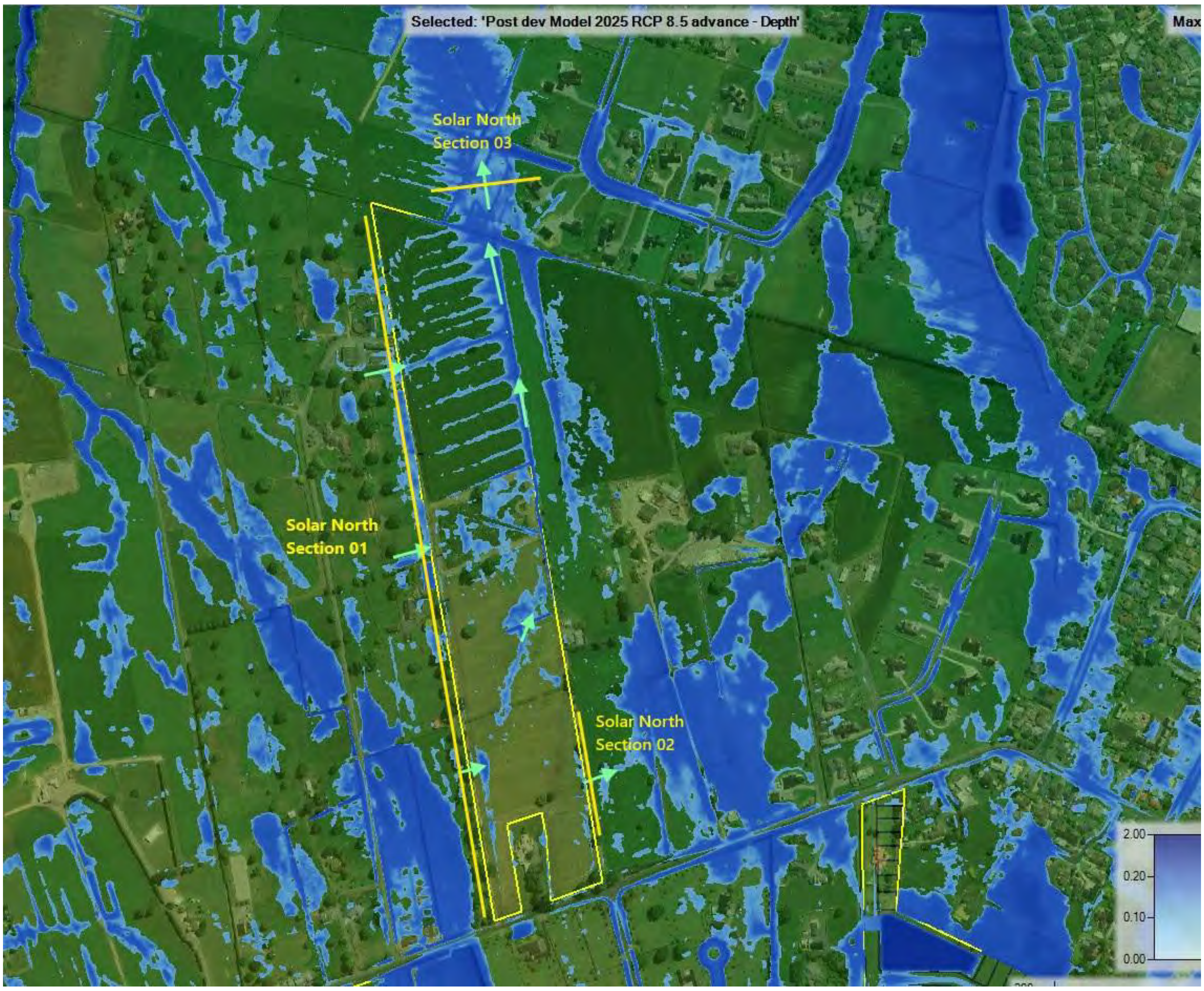
9. 171mm/hr

- ➔ Required 10-year V(storage) = 10.98 – 3.21 = 7.77m^3
- ➔ Proposed Soakage Trench Volume (7.85m^3) is larger than Required Volume (7.77m^3)
- ➔ Soakage trench sizing is OK

POST DEVELOMENT RESULTS RCP 8.5

Selected: 'Post dev Model 2025 RCP 8.5 advance - Depth'
Max
RES Section 03
Greenway &
Basin B Section
BASIN B
Waitoa River
Solar South
Section 01
RES Section 02


Selected: 'Post dev Model 2025 RCP 8.5 advance - Depth'
Max
RES Section 06
RES Section 07
BASIN D
RES Section 05
RES Section 04
BASIN C
RES Section 08
2.00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Max
Solar North
Section 03
Solar North
Section 01
Solar North
Section 02
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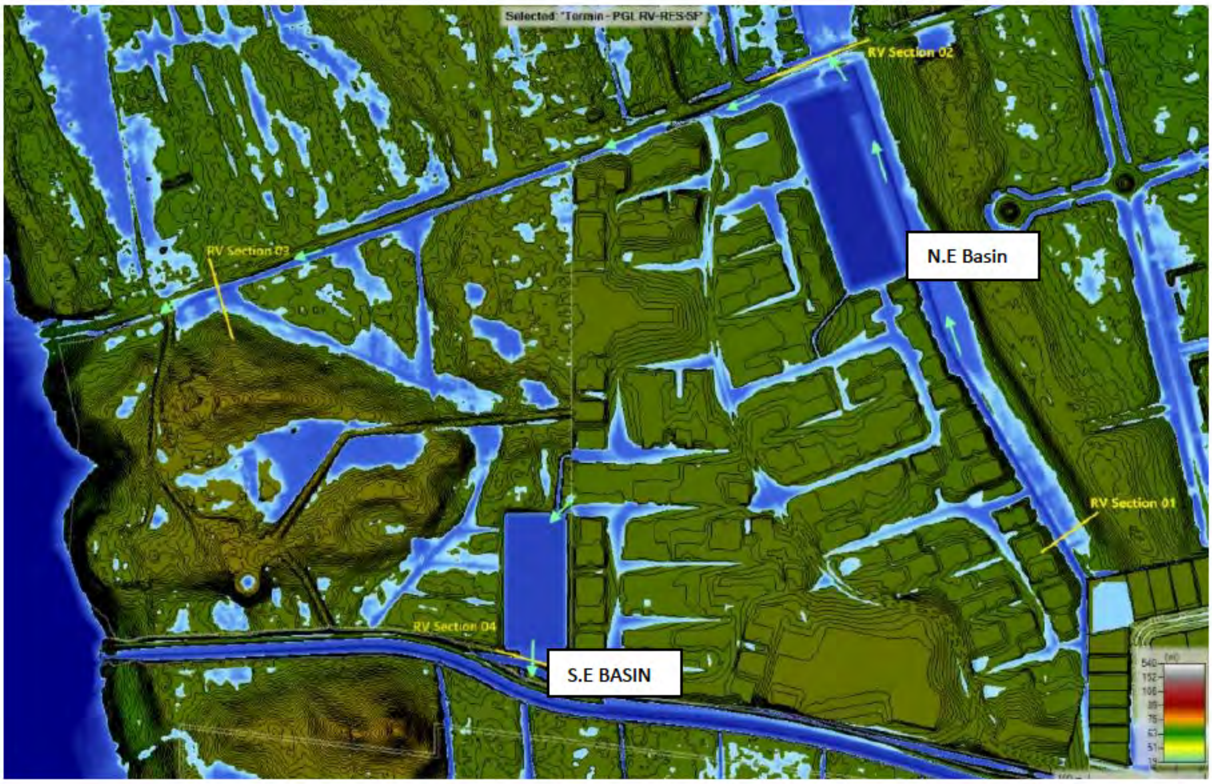
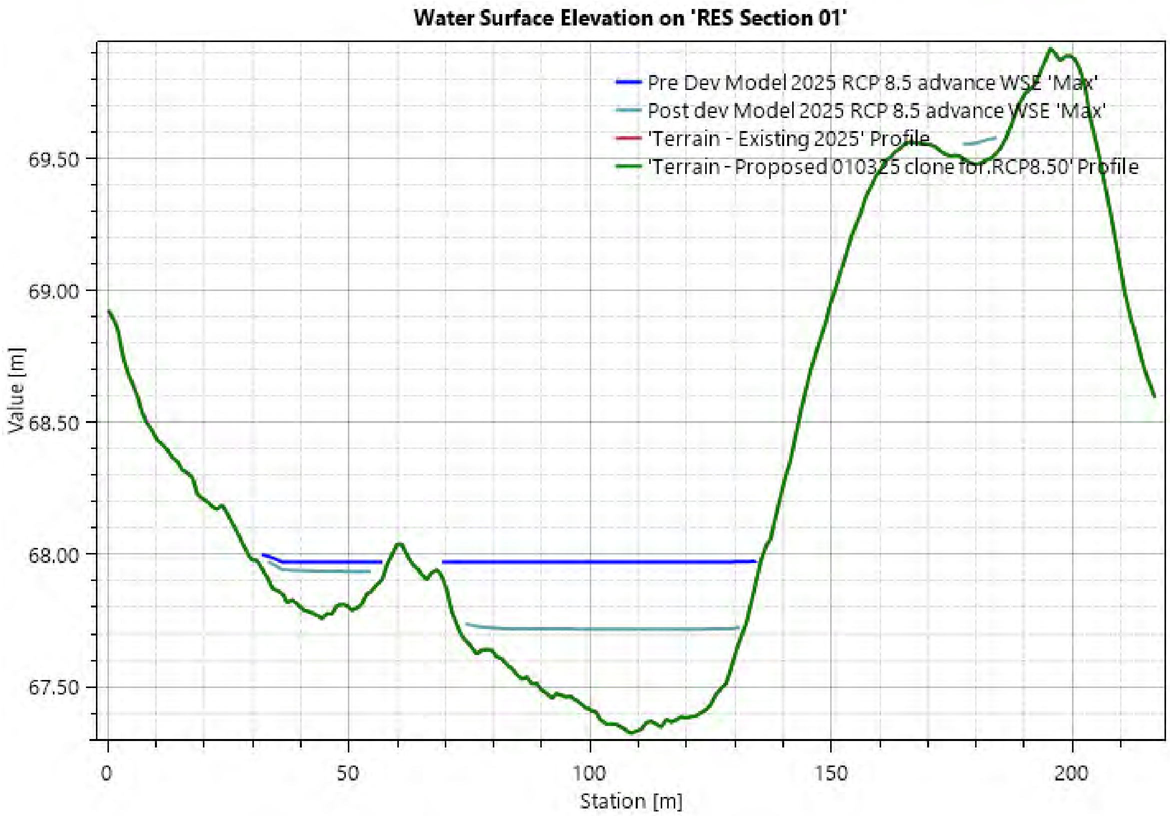
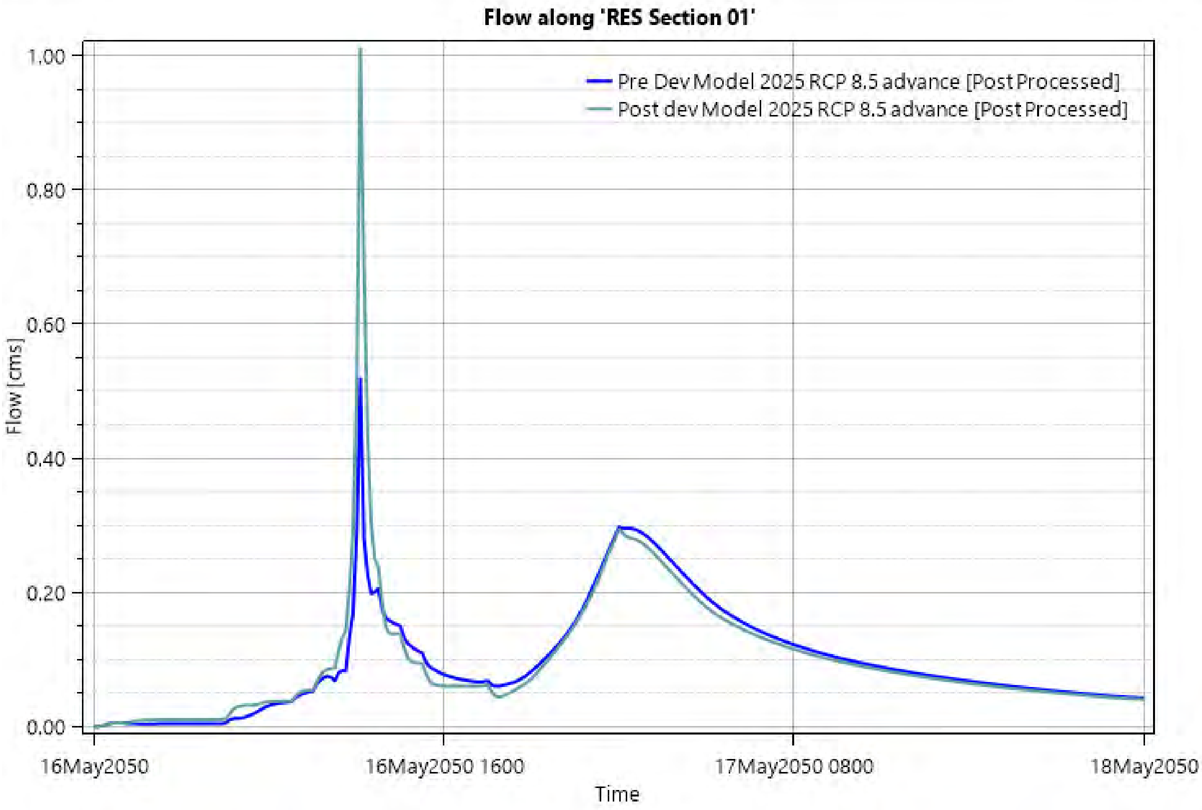
0.20
0.10
0.00



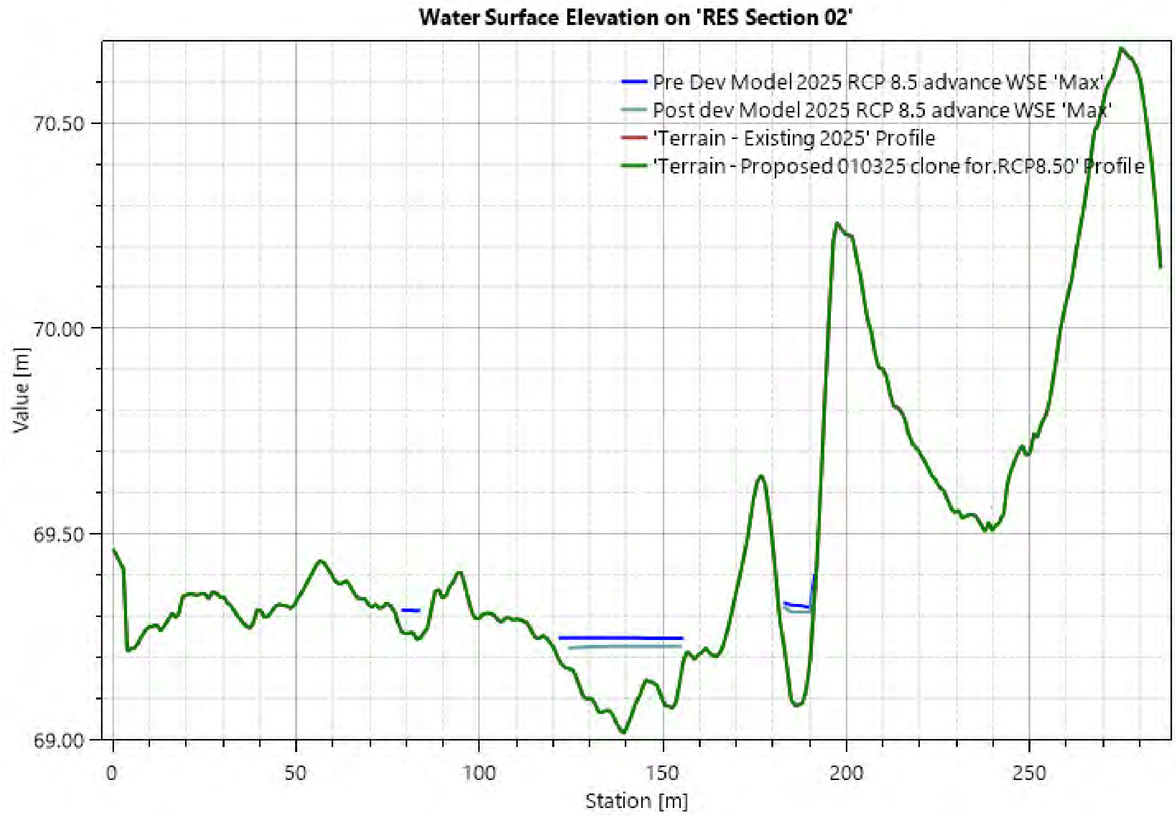
Selected: "Termin - PGL RV-RES SP"
RV Section 02
RV Section 03
N.E Basin
RV Section 01
RV Section 04
S.E BASIN

HEC RAS RESULTS – SECTIONS PRE & POST

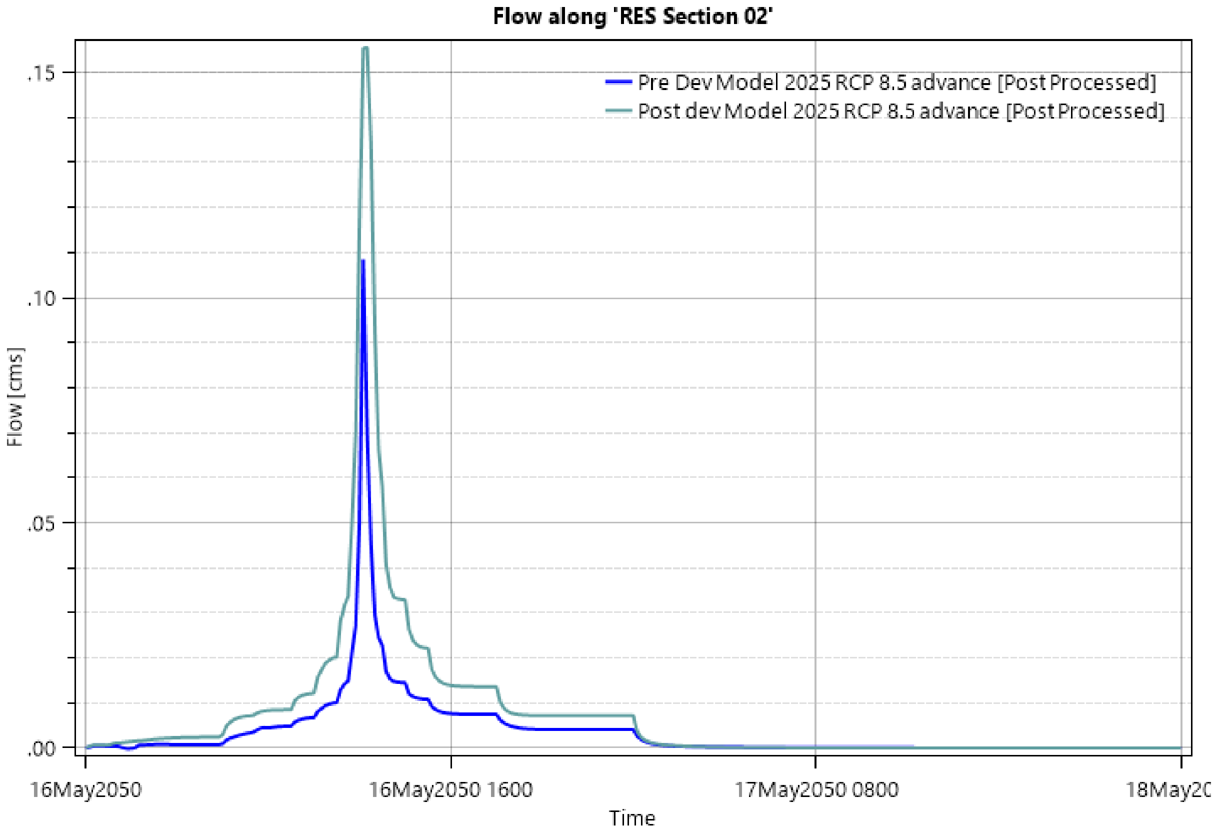


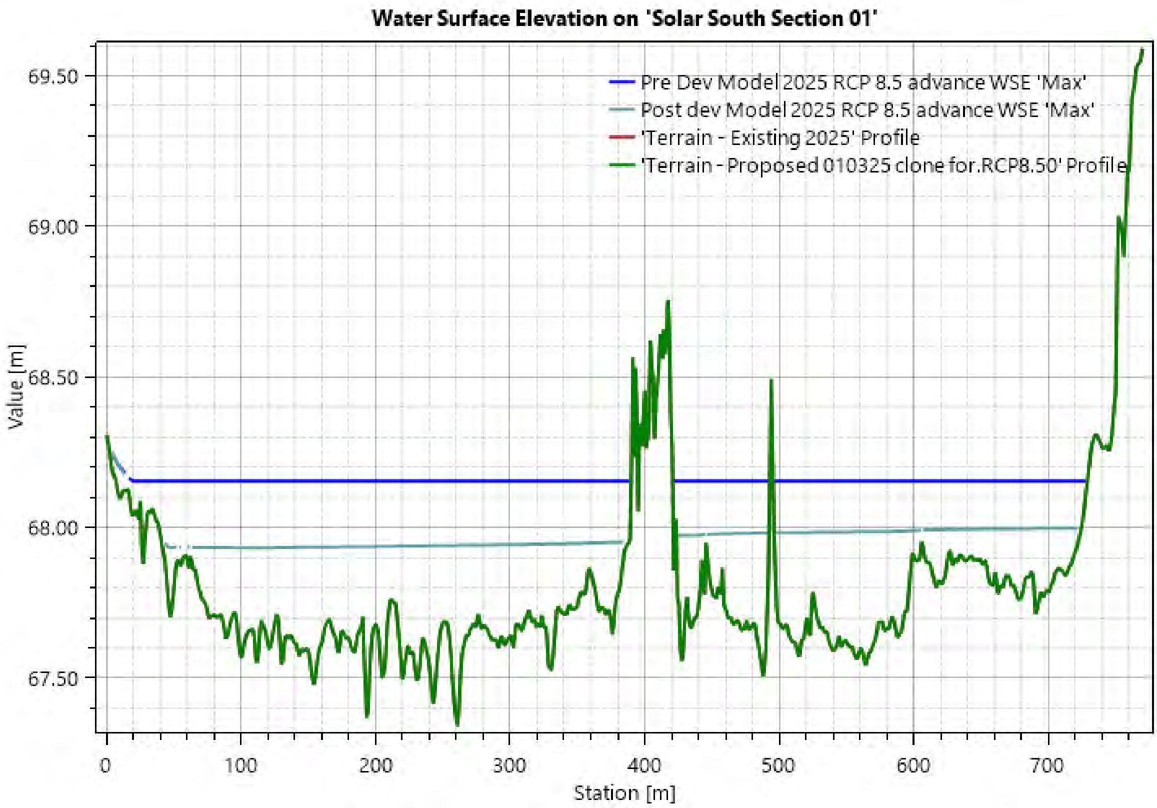


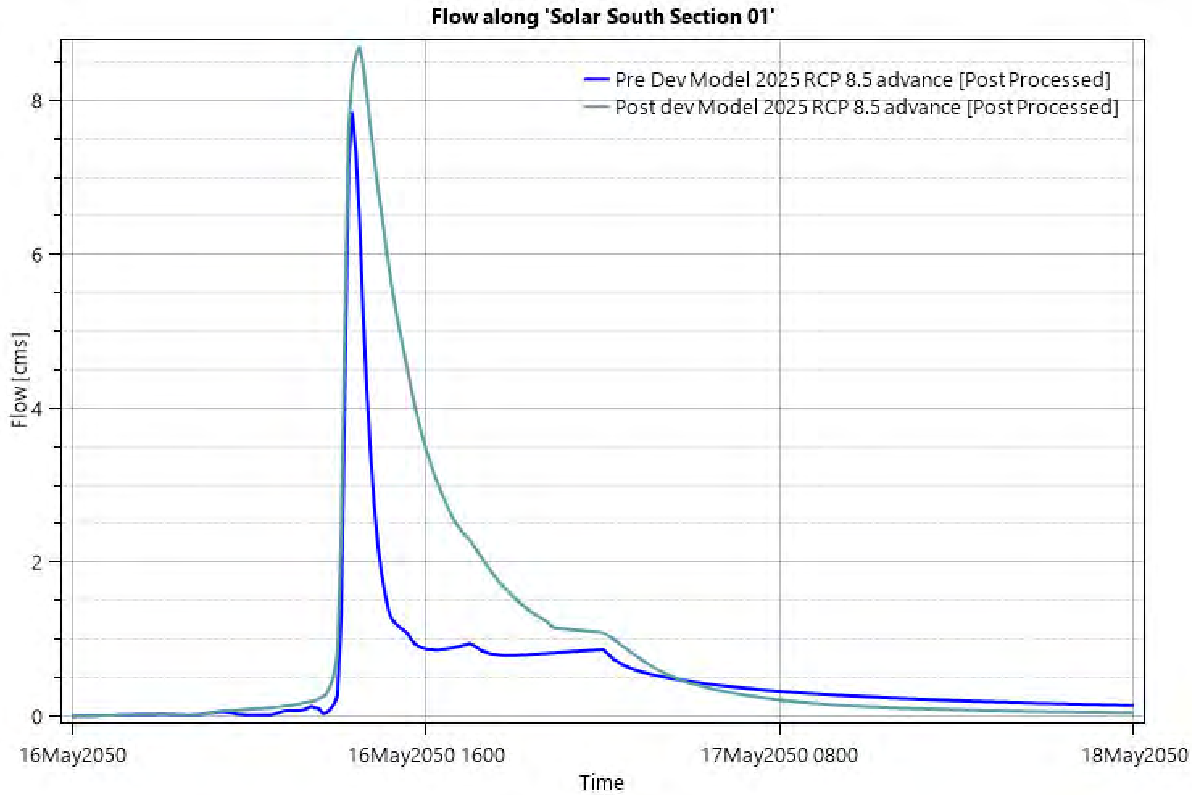
Flow along 'RES Section 01'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
1.00
0.80
0.60
0.40
0.20
0.00
Flow [cms]
16May2050
16May2050 1600
17May2050 0800
18May2050
Time



Water Surface Elevation on 'RES Section 02'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
70.50
70.00
69.50
69.00
Value [m]
0
50
100
150
200
250
Station [m]

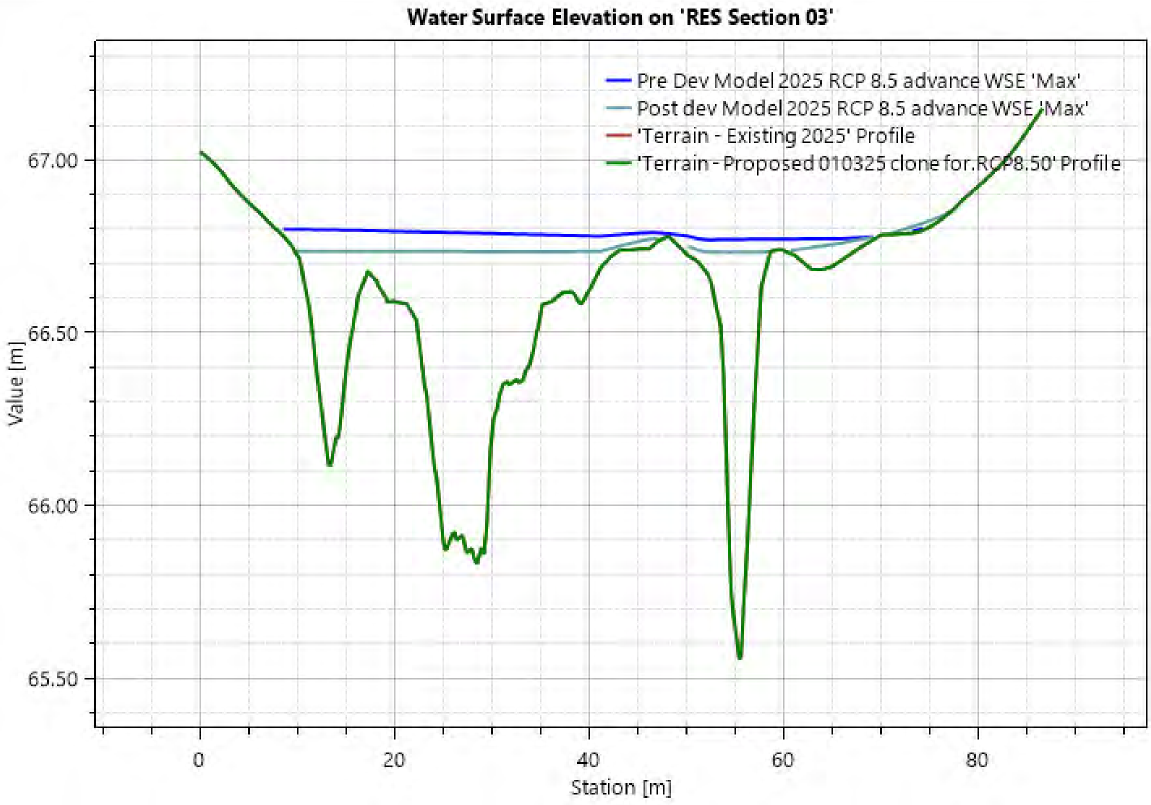

Flow along 'RES Section 02'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
.15
.10
.05
.00
16May2050
16May2050 1600
17May2050 0800
Time

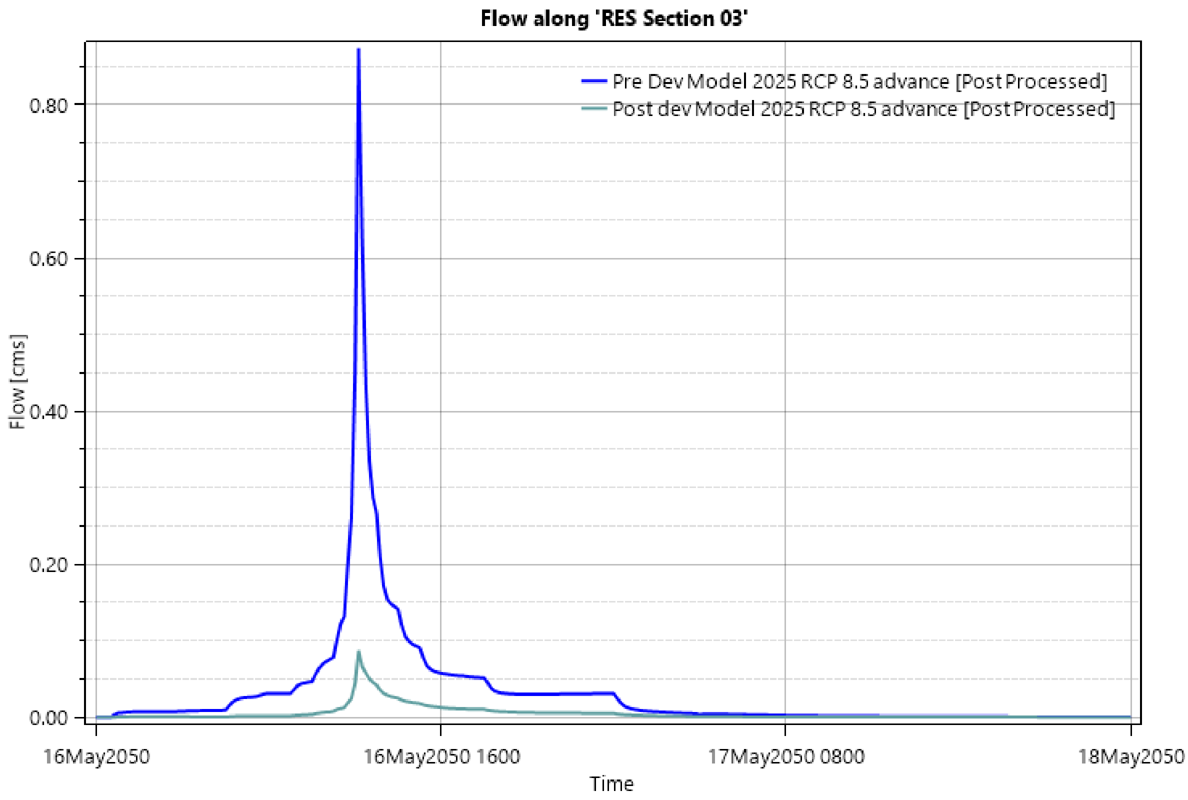

Water Surface Elevation on 'Solar South Section 01'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
69.50
69.00
68.50
68.00
67.50
0
100
200
300
400
500
600
700
Station [m]

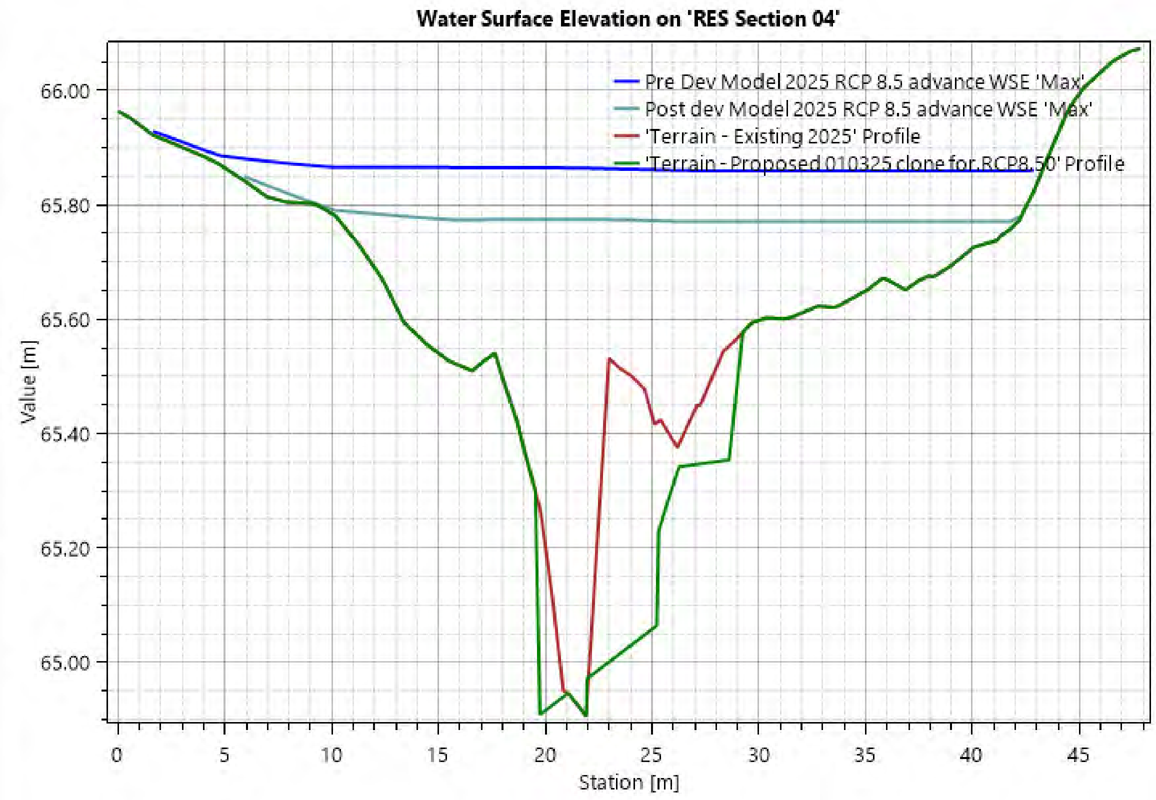

Flow along 'Solar South Section 01'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
0
2
4
6
8
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

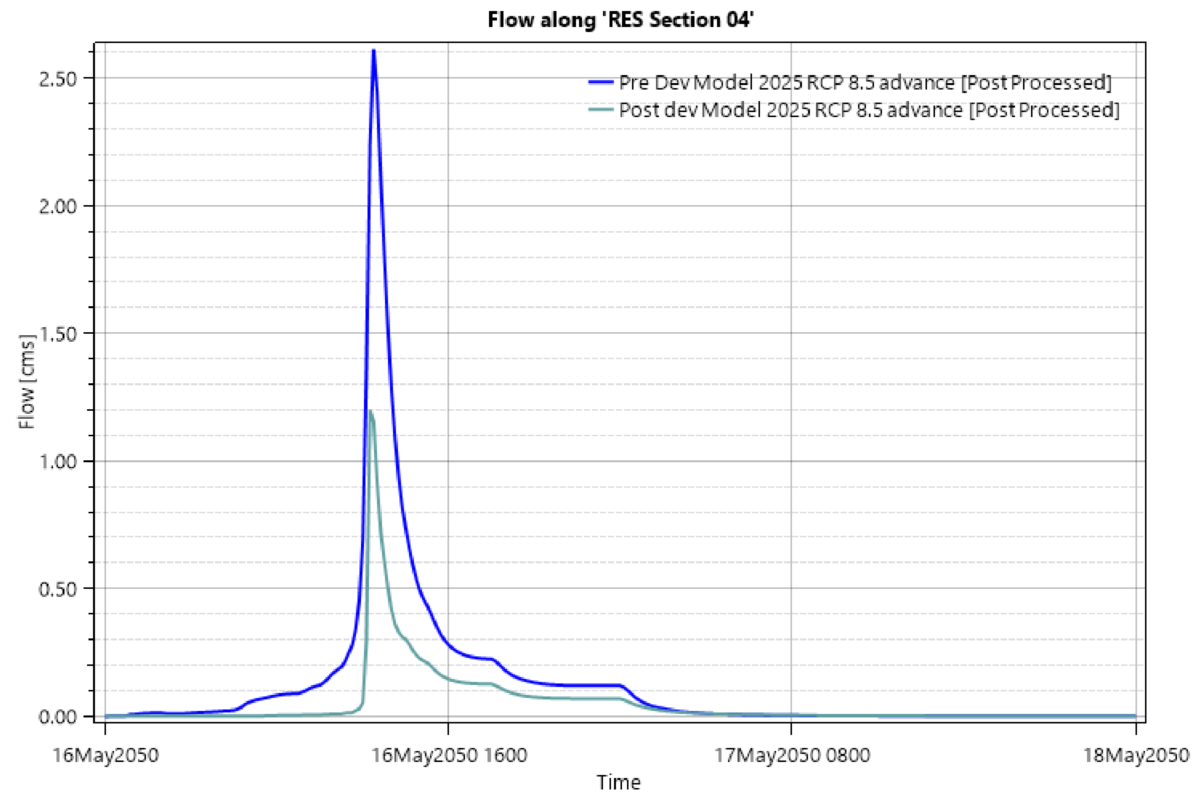
MAVEN

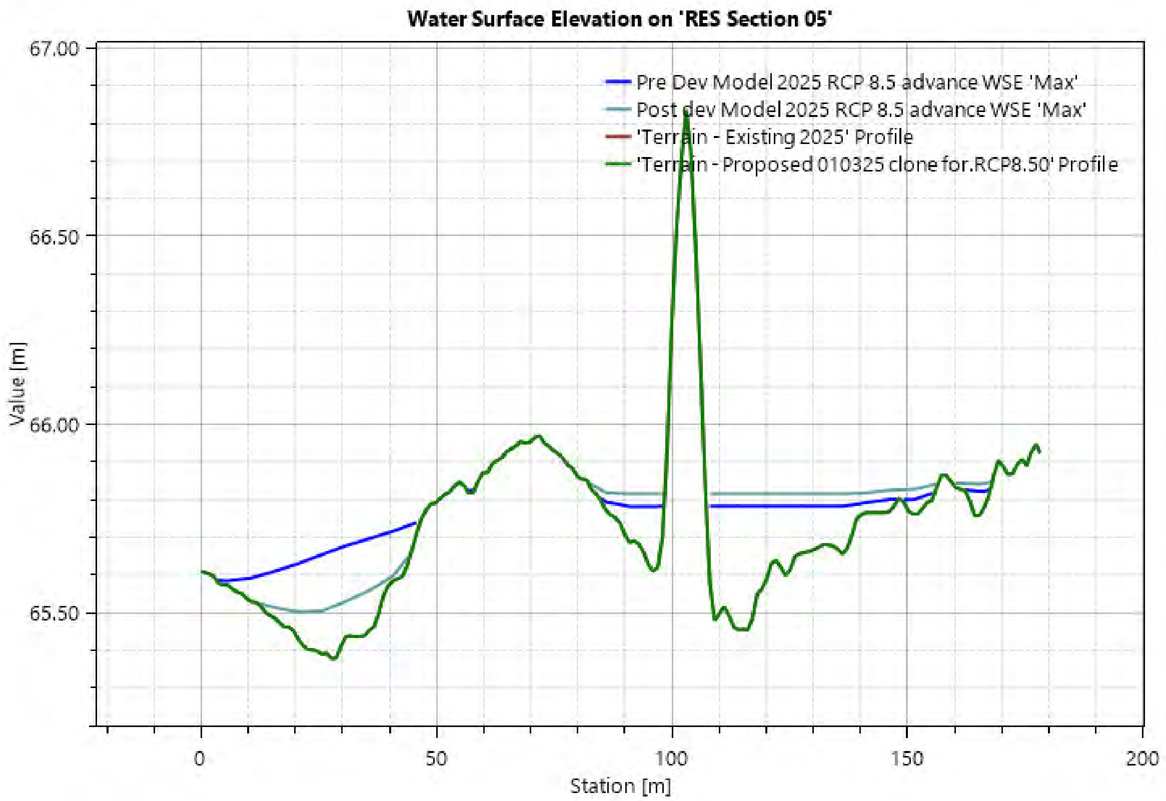
Water Surface Elevation on 'Greenway & Basin B RES Section'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
Station [m]
70
69
68
67
66
65
64
63
62
61
60
59
58
57
-100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400

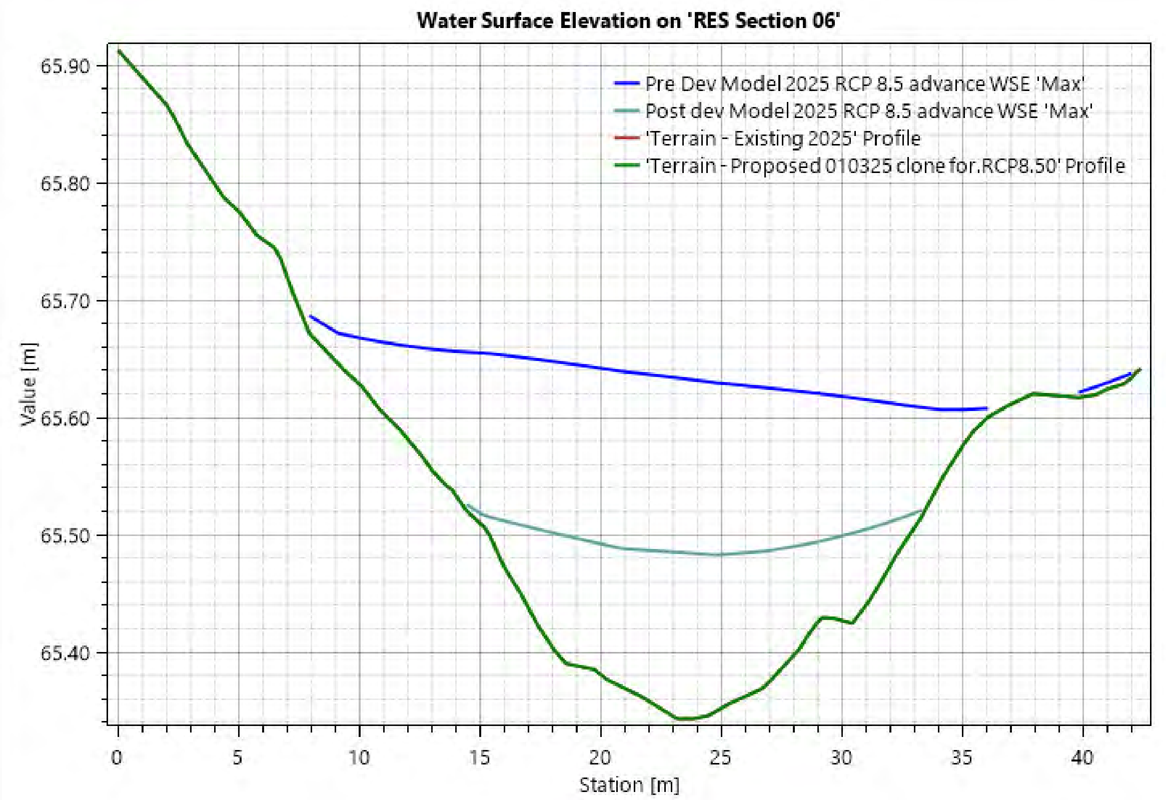

Water Surface Elevation on 'RES Section 03'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
67.00
66.50
66.00
65.50
0
20
40
60
80
Station [m]

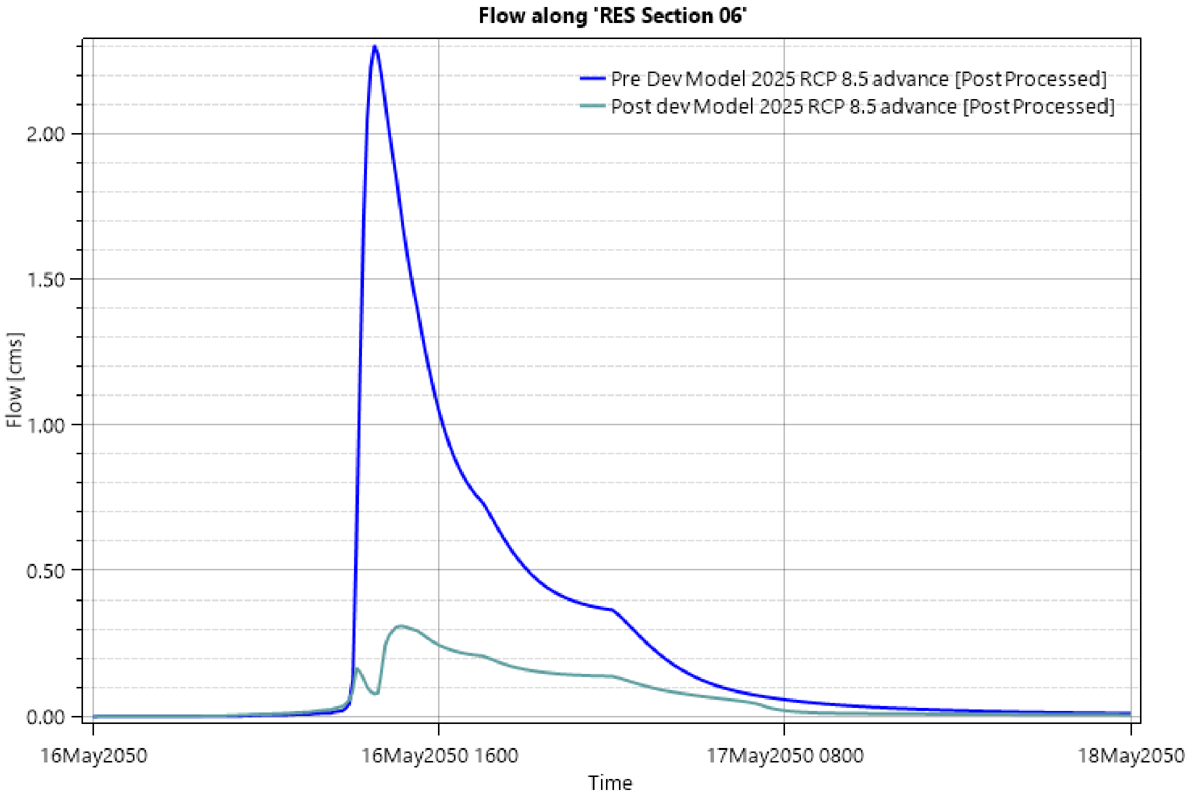

Flow along 'RES Section 03'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
0.80
0.60
0.40
0.20
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

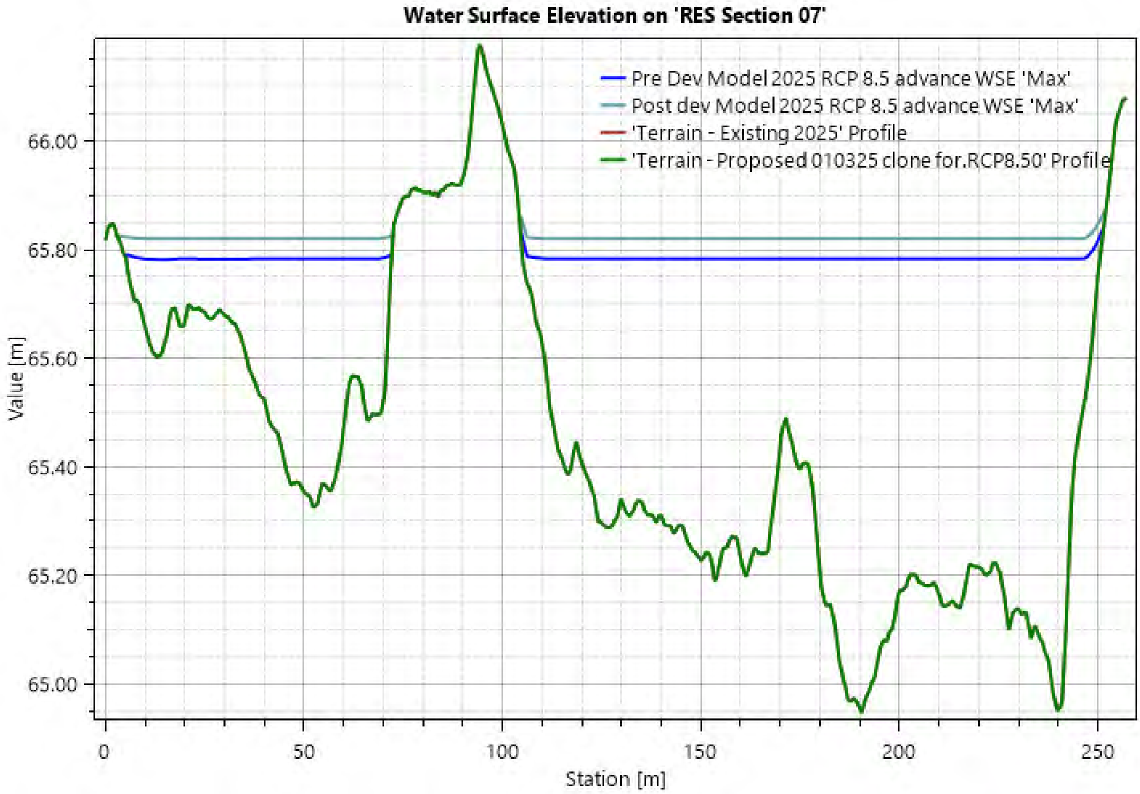

Water Surface Elevation on 'RES Section 04'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for RCP8.50' Profile
Value [m]
66.00
65.80
65.60
65.40
65.20
65.00
0
5
10
15
20
25
30
35
40
45
Station [m]

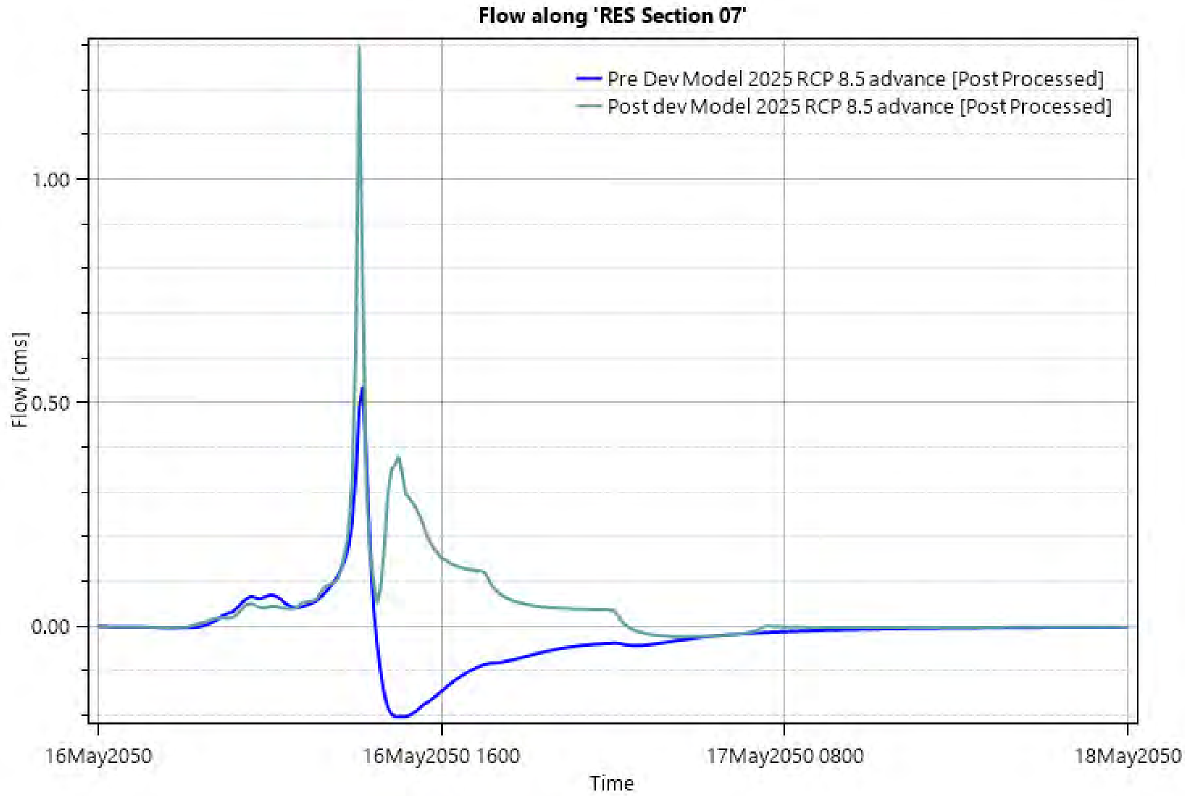

Flow along 'RES Section 04'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
2.50
2.00
1.50
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

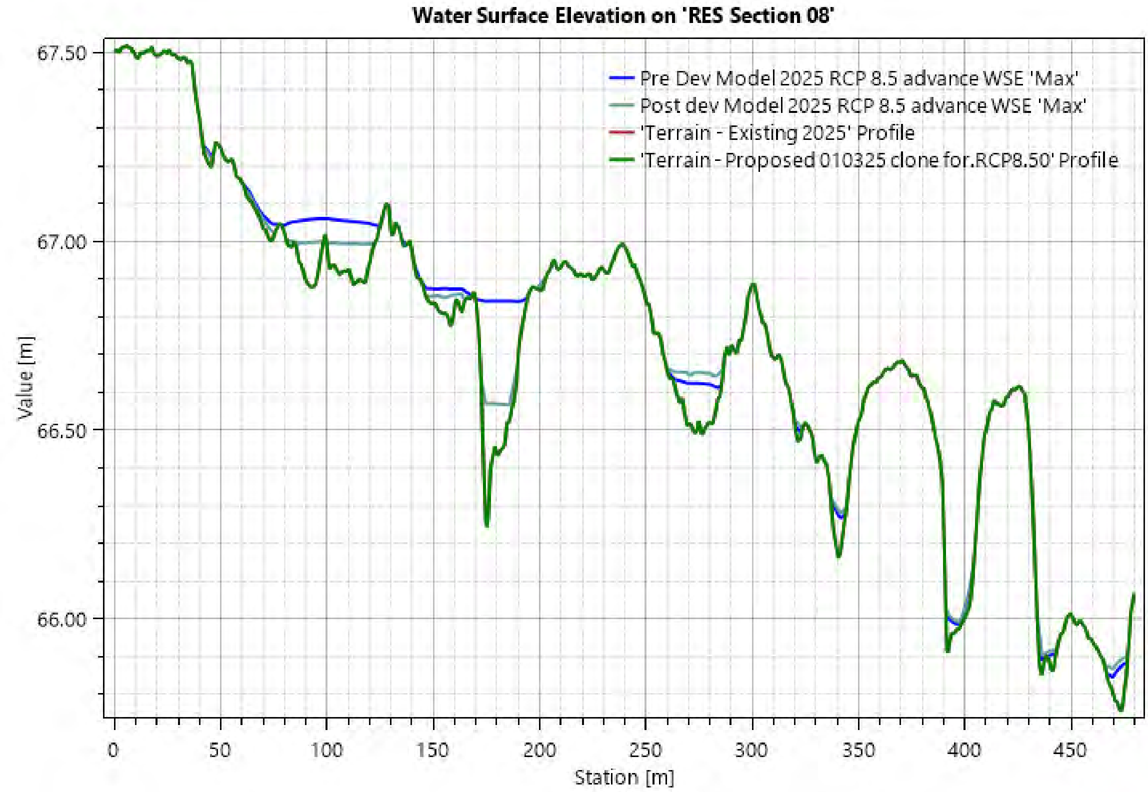

Water Surface Elevation on 'RES Section 05'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
67.00
66.50
66.00
65.50
0
50
100
150
200
Station [m]

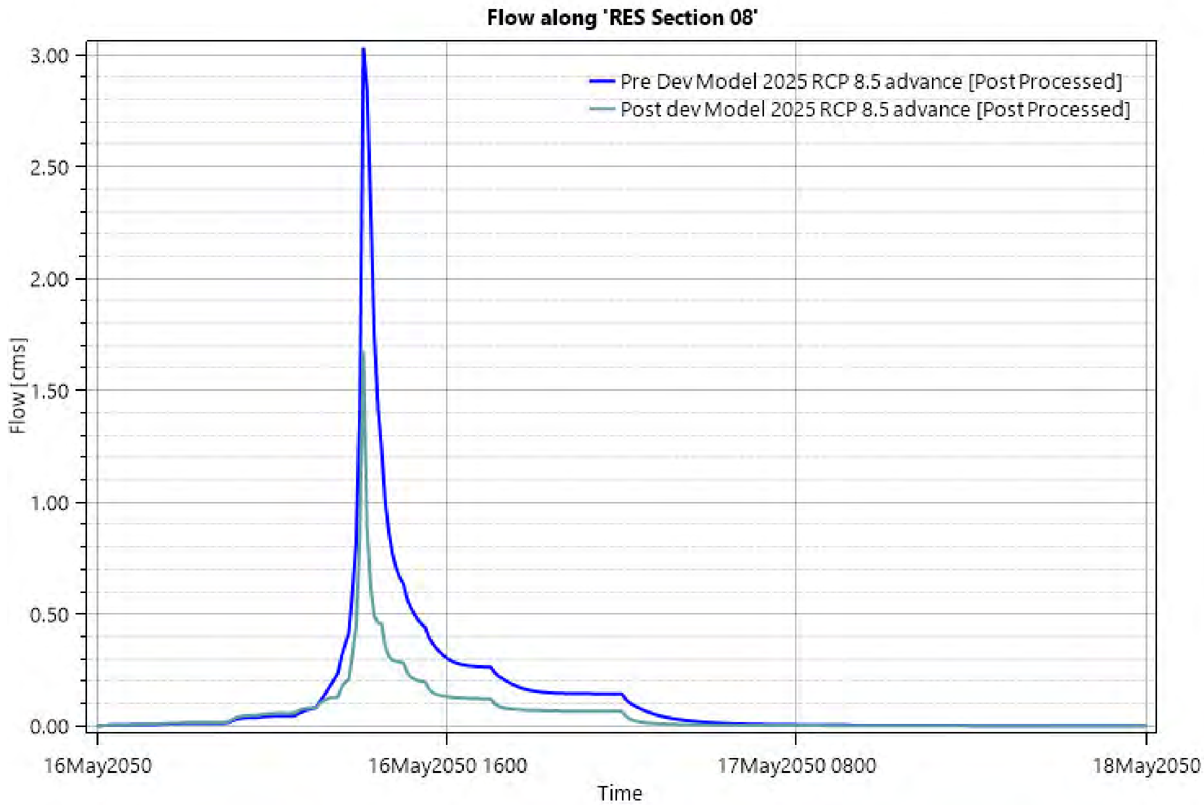

Water Surface Elevation on 'RES Section 06'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
65.90
65.80
65.70
65.60
65.50
65.40
0
5
10
15
20
25
30
35
40
Station [m]

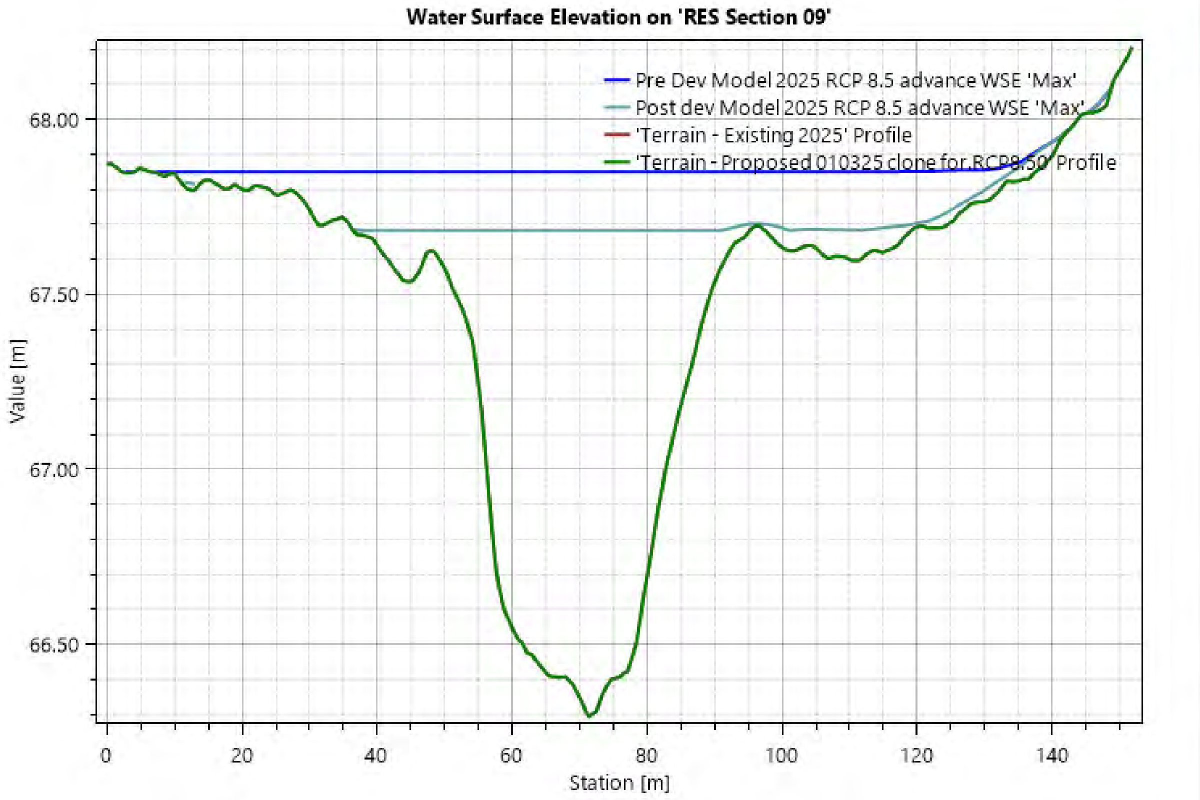

Flow along 'RES Section 06'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
2.00
1.50
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

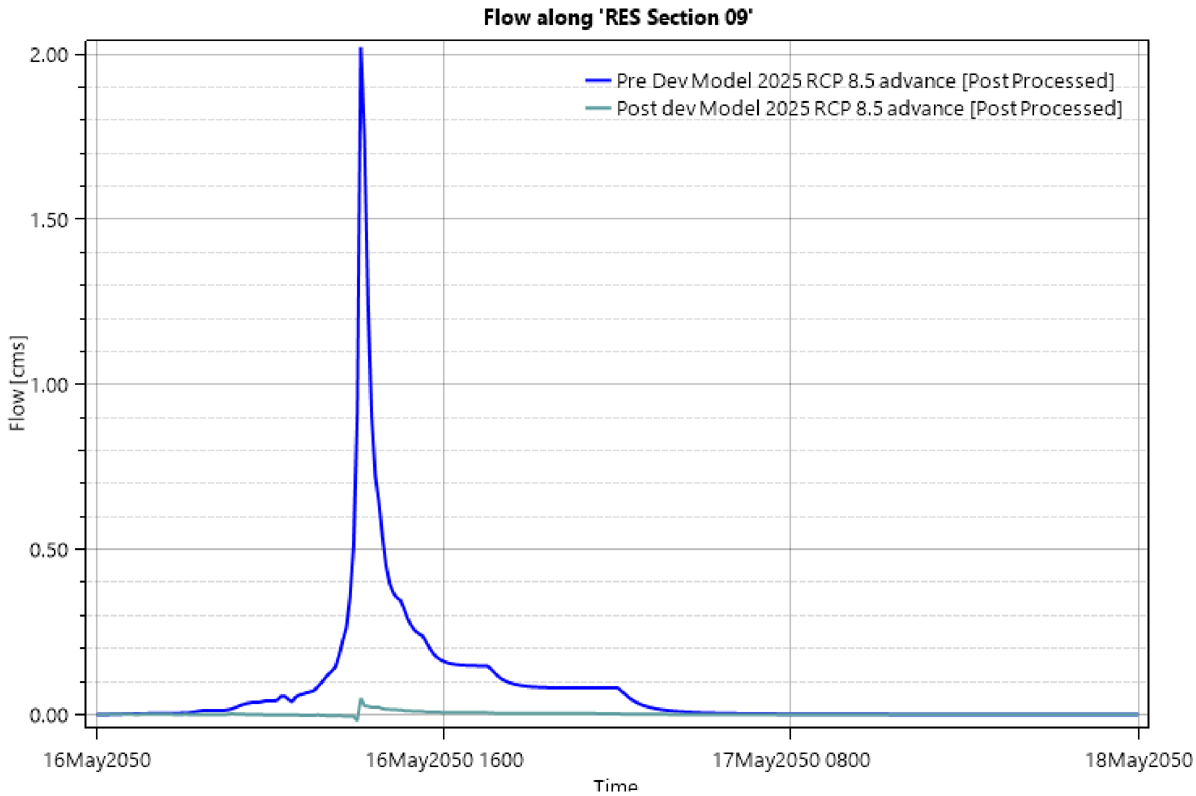

Water Surface Elevation on 'RES Section 07'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
66.00
65.80
65.60
65.40
65.20
65.00
0
50
100
150
200
250
Station [m]

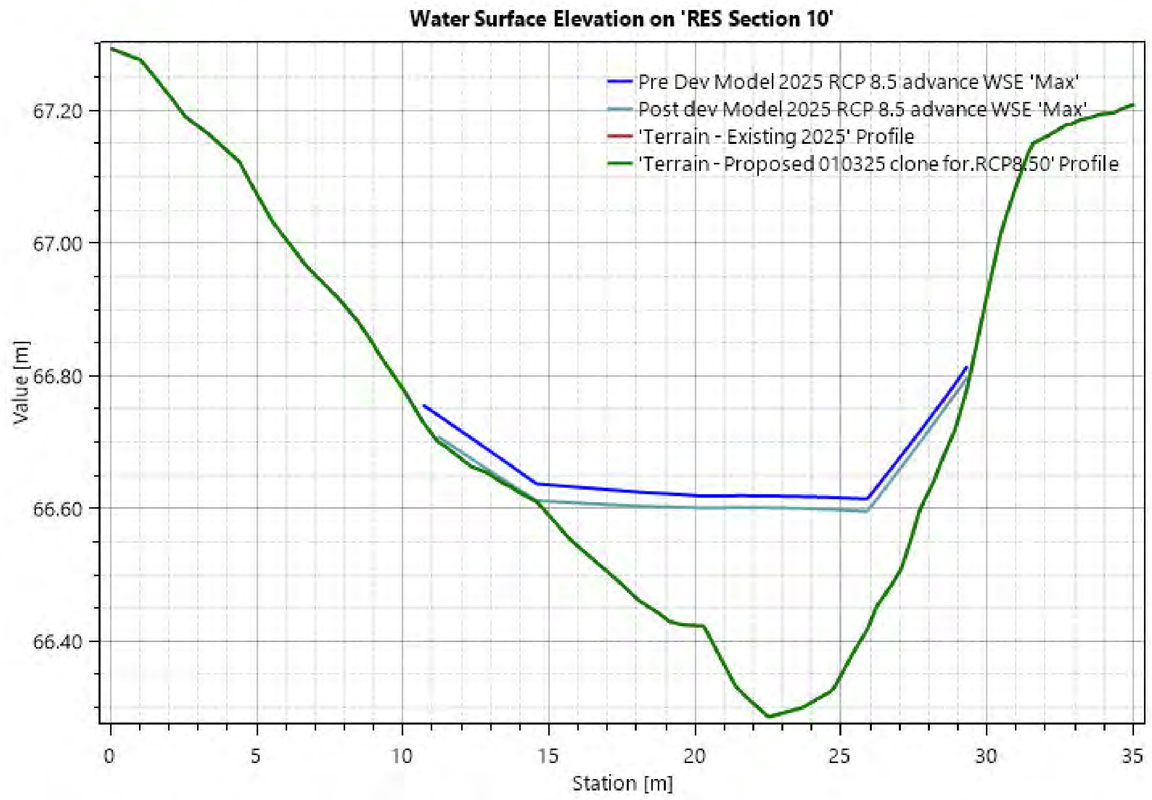

Flow along 'RES Section 07'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

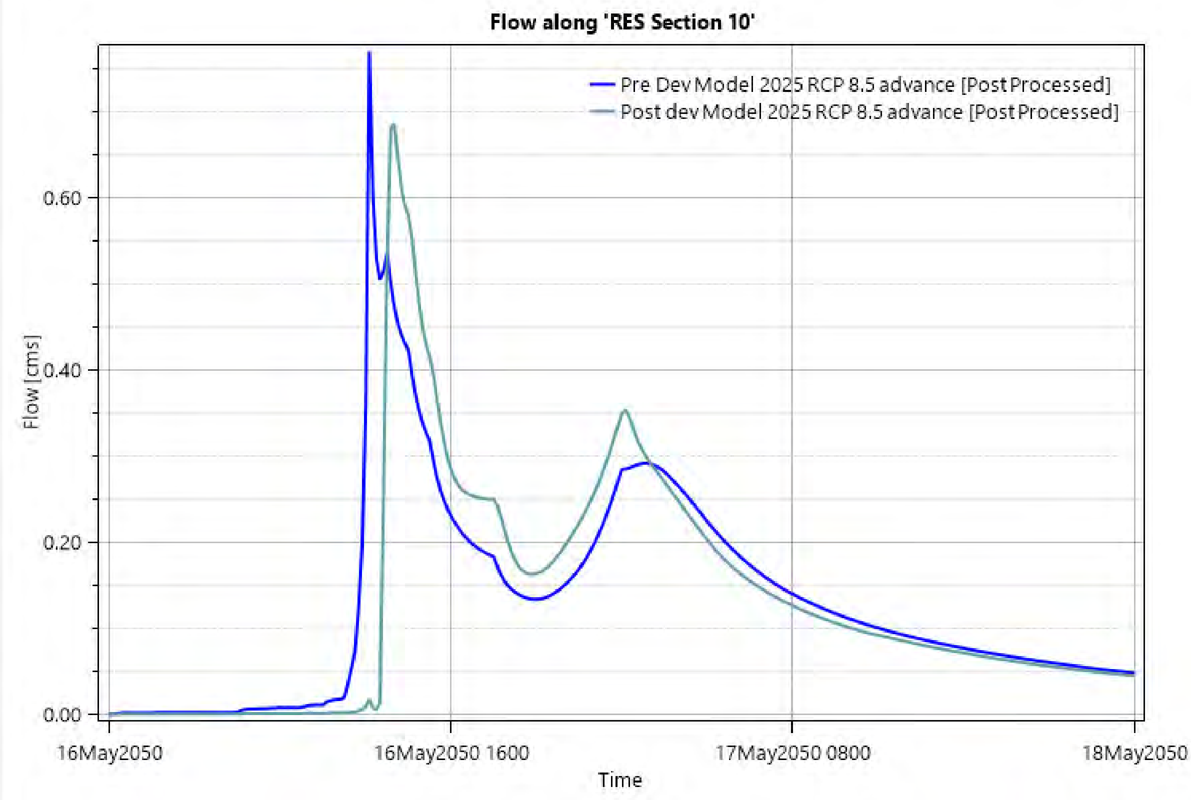

Water Surface Elevation on 'RES Section 08'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
Station [m]
67.50
67.00
66.50
66.00
0
50
100
150
200
250
300
350
400
450

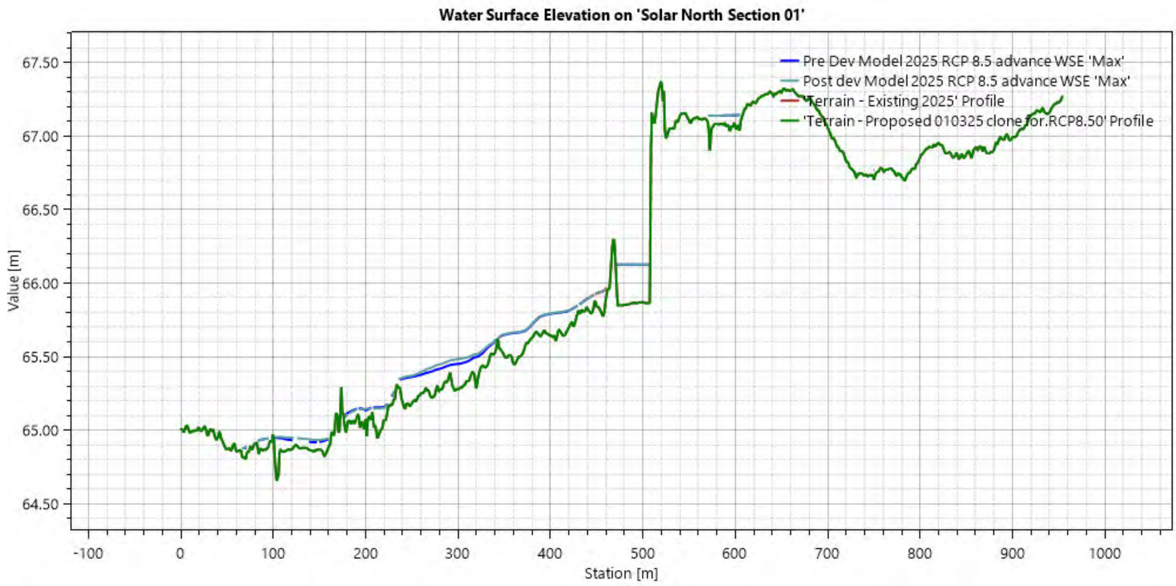

Flow along 'RES Section 08'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
Time
3.00
2.50
2.00
1.50
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050

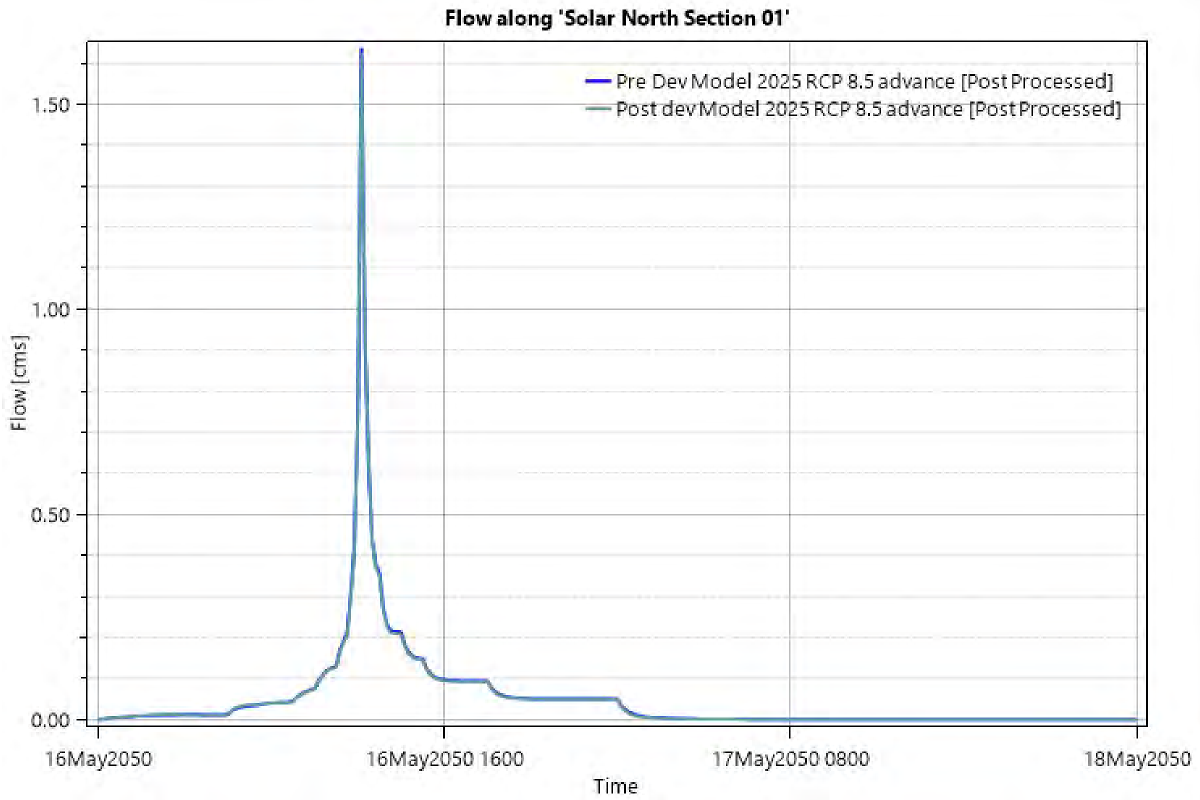

Water Surface Elevation on 'RES Section 09'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for RCP8.50' Profile
Value [m]
68.00
67.50
67.00
66.50
0
20
40
60
80
100
120
140
Station [m]

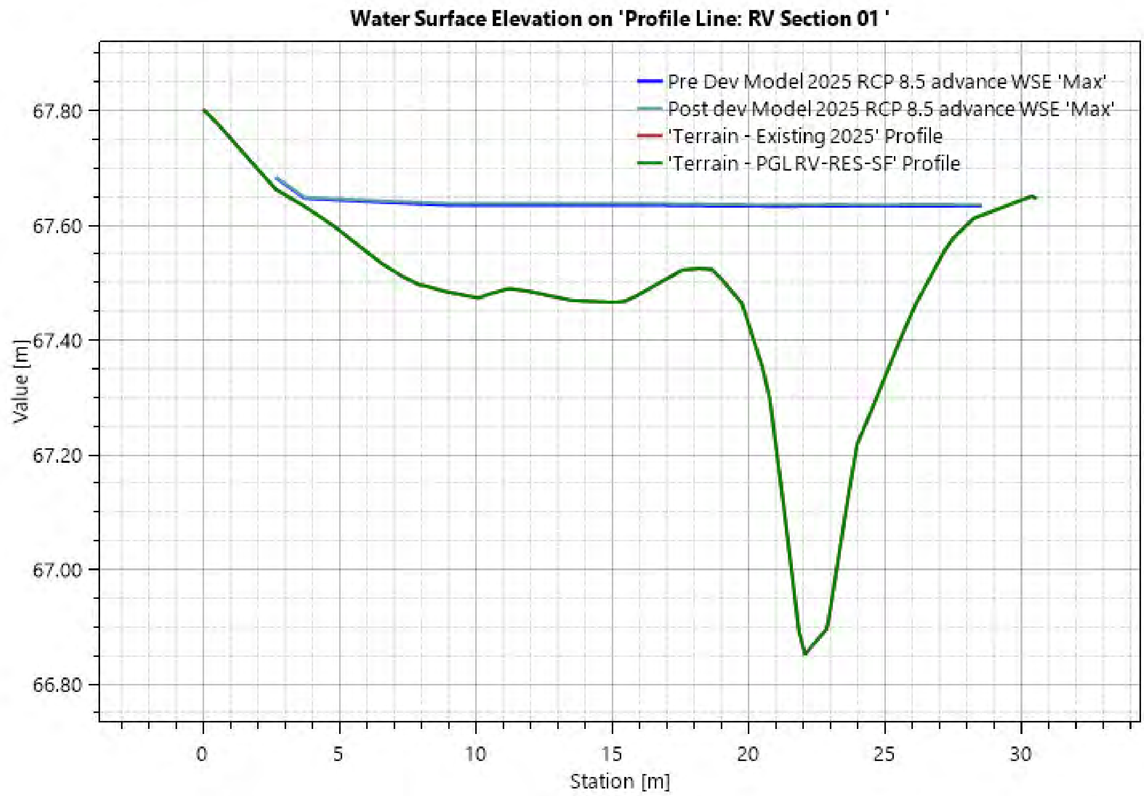

Flow along 'RES Section 09'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
2.00
1.50
1.00
0.50
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

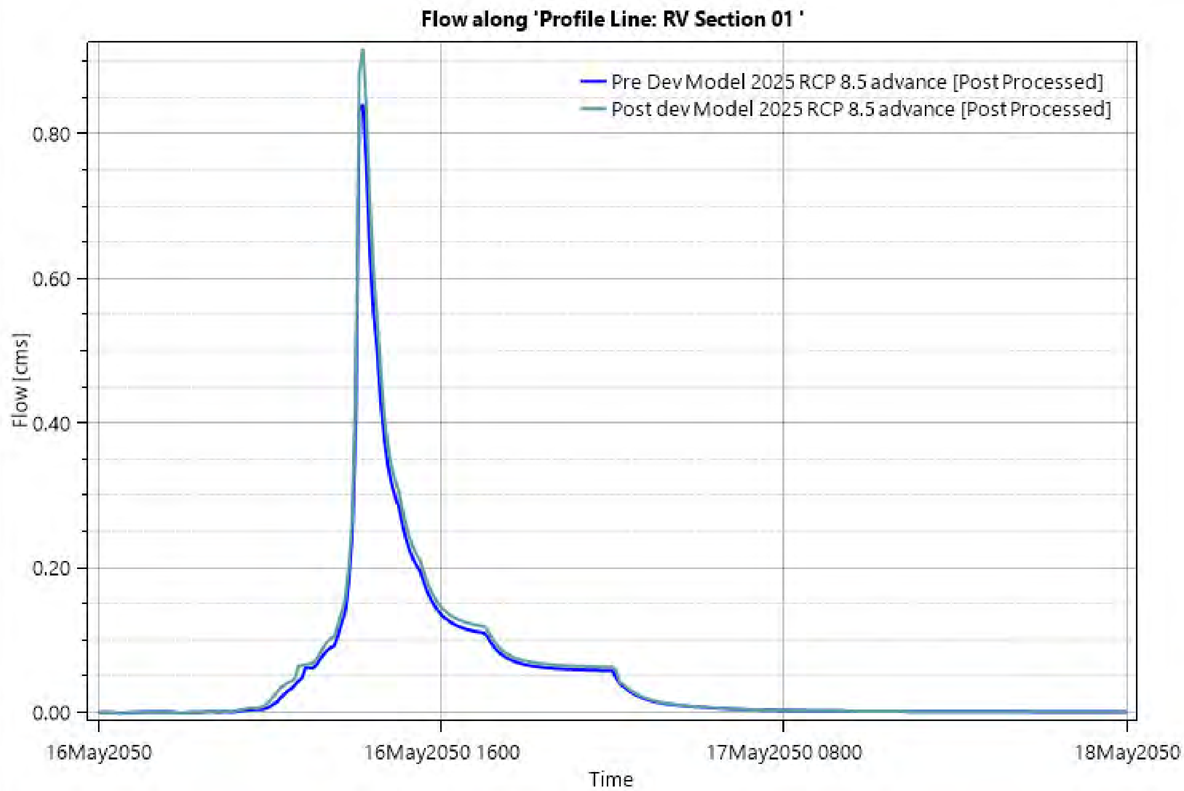

Water Surface Elevation on 'RES Section 10'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
Station [m]
67.20
67.00
66.80
66.60
66.40
0
5
10
15
20
25
30
35

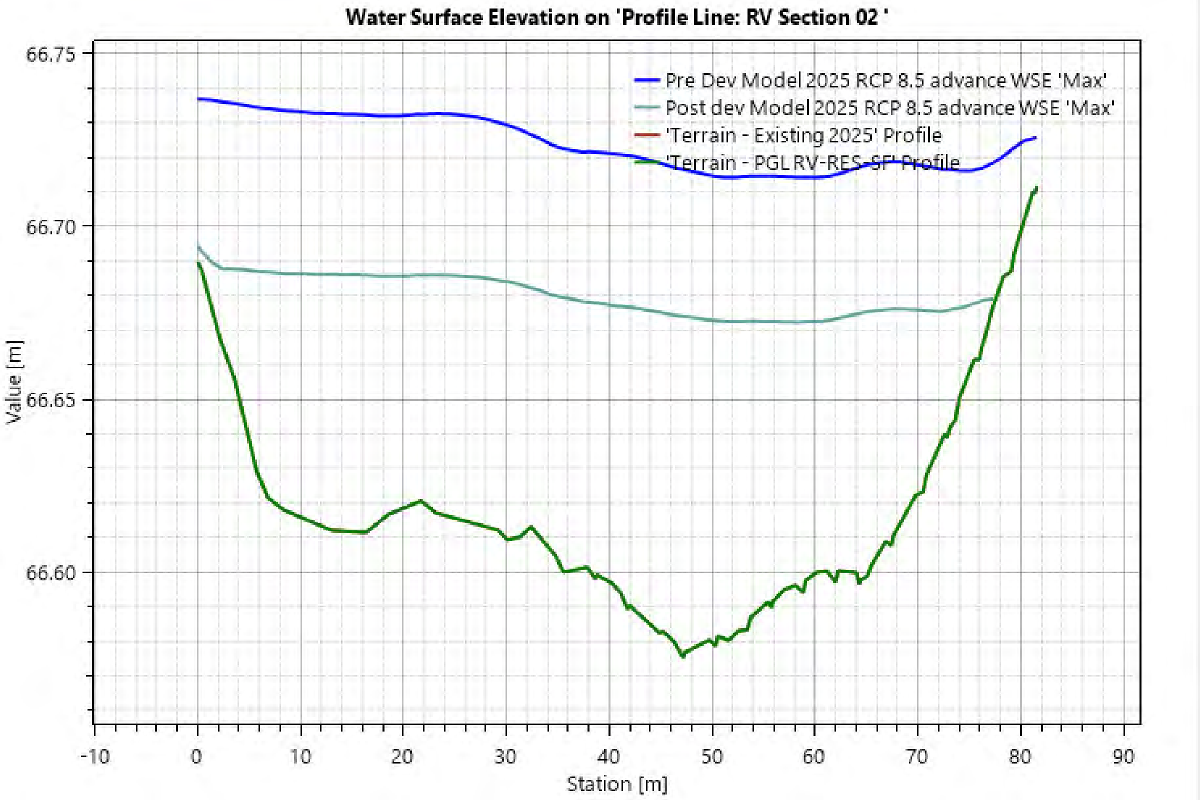

Flow along 'RES Section 10'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
Time
0.60
0.40
0.20
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050

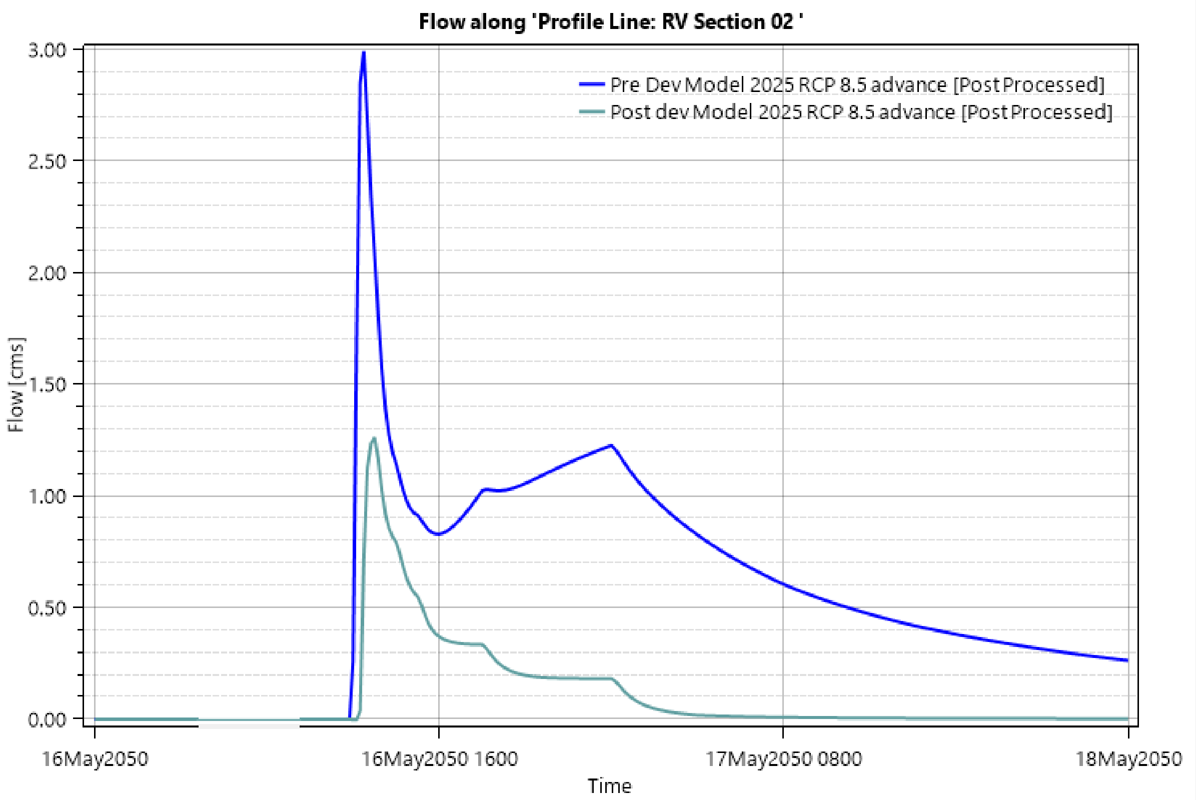

Water Surface Elevation on 'Solar North Section 01'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
Station [m]

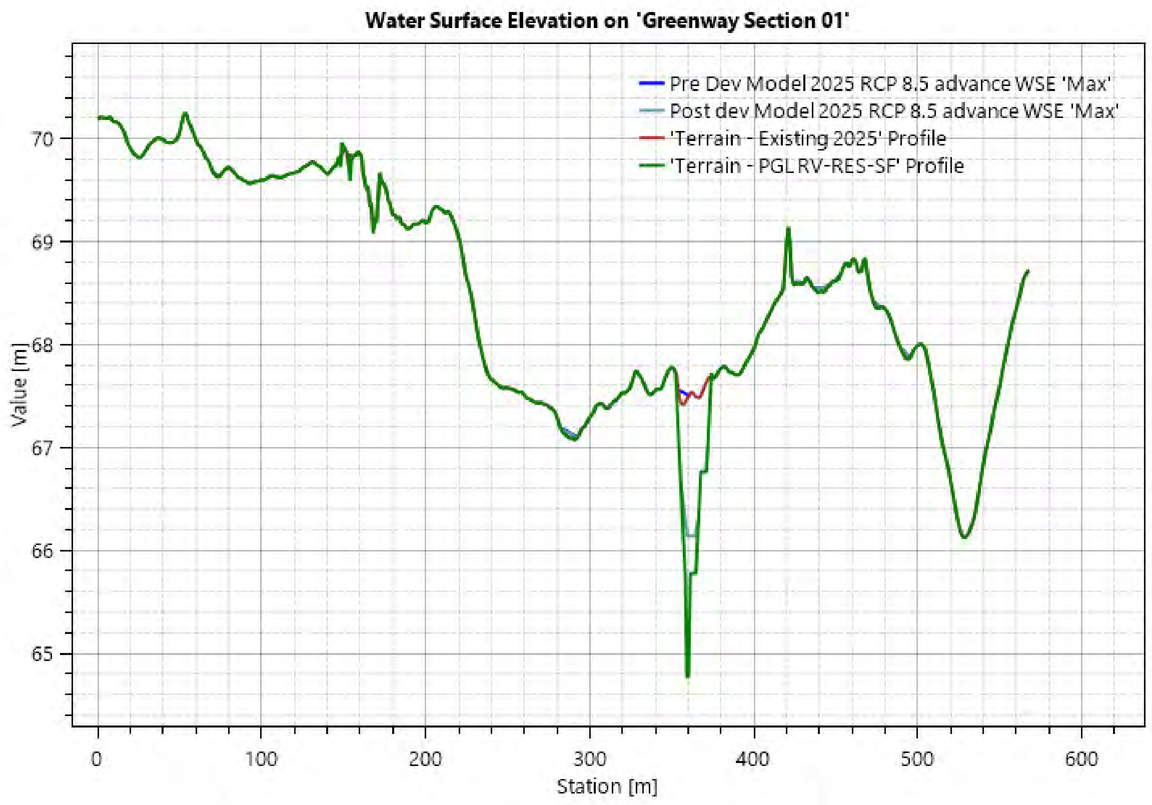

Flow along 'Solar North Section 01'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

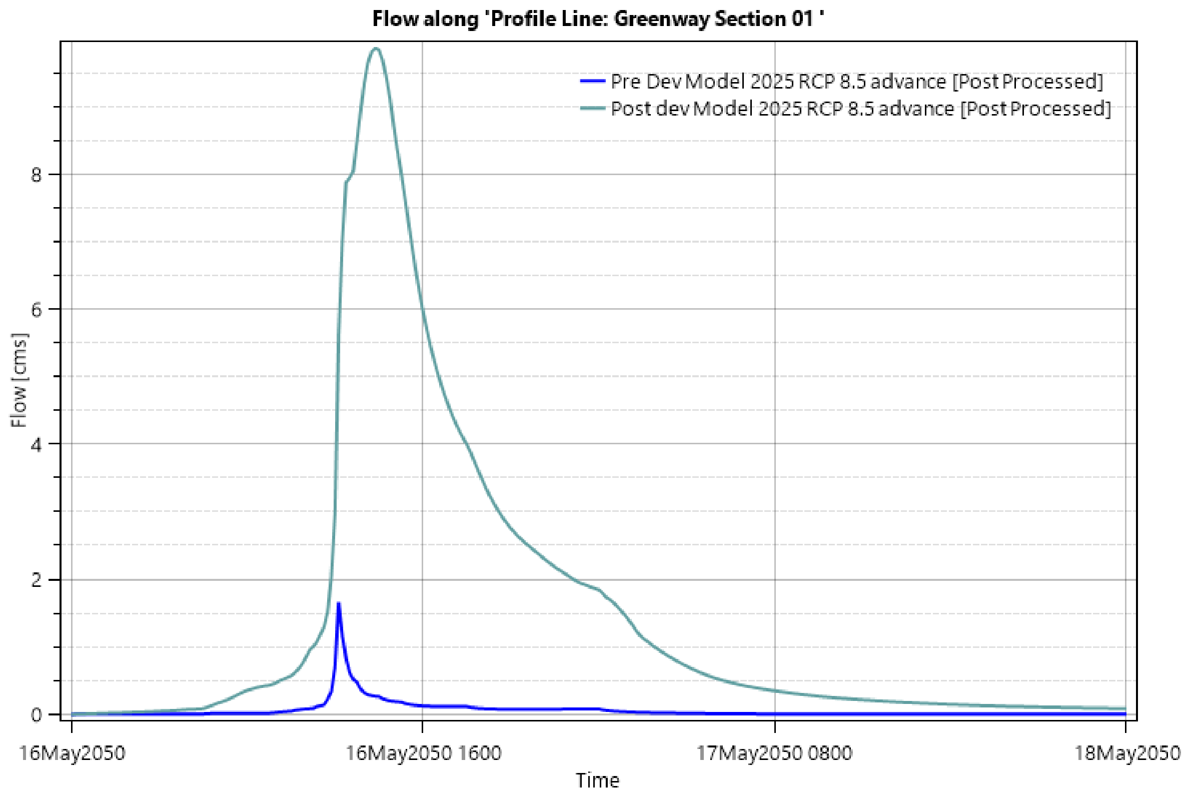

Water Surface Elevation on 'Profile Line: RV Section 01 '
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - PGL RV-RES-SF' Profile
Value [m]
67.80
67.60
67.40
67.20
67.00
66.80
0
5
10
15
20
25
30
Station [m]

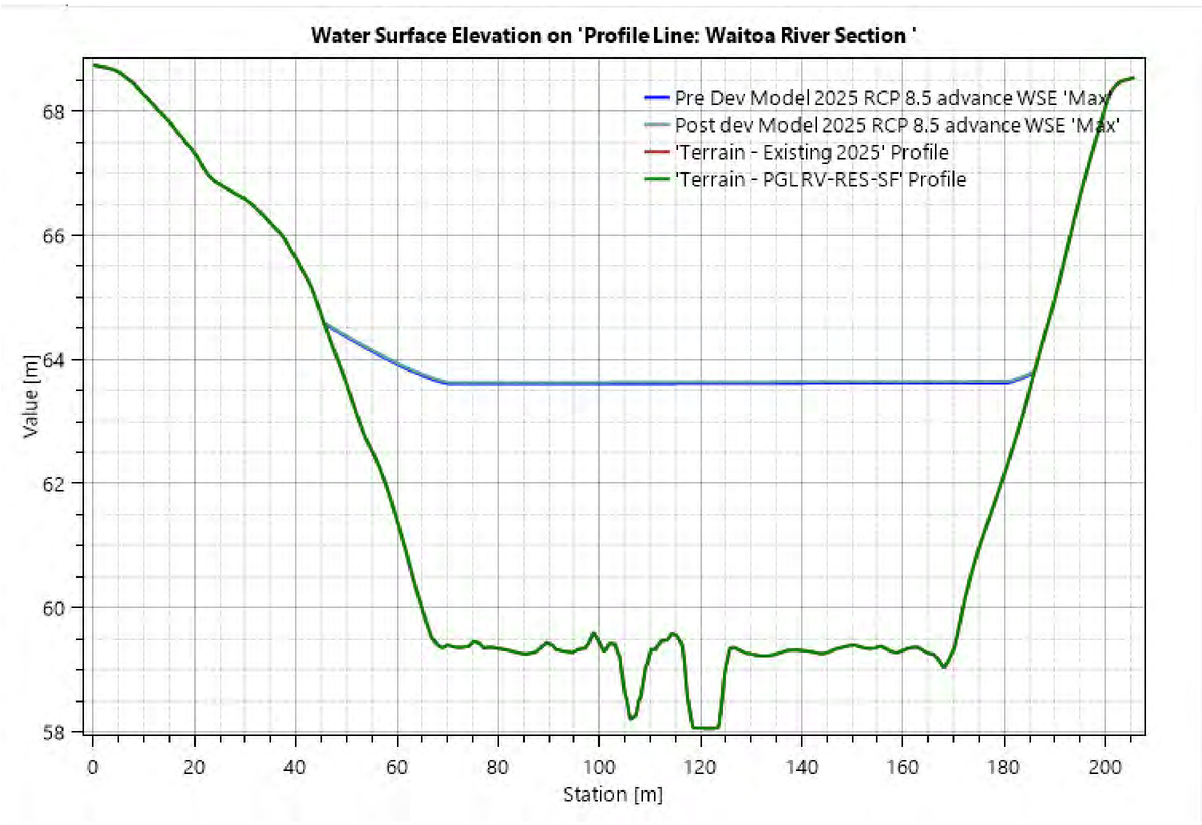

Flow along 'Profile Line: RV Section 01 '
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
0.80
0.60
0.40
0.20
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time


Water Surface Elevation on 'Profile Line: RV Section 02 '
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - PGL RV-RES-SF' Profile
66.75
66.70
66.65
66.60
Value [m]
-10
0
10
20
30
40
50
60
70
80
90
Station [m]

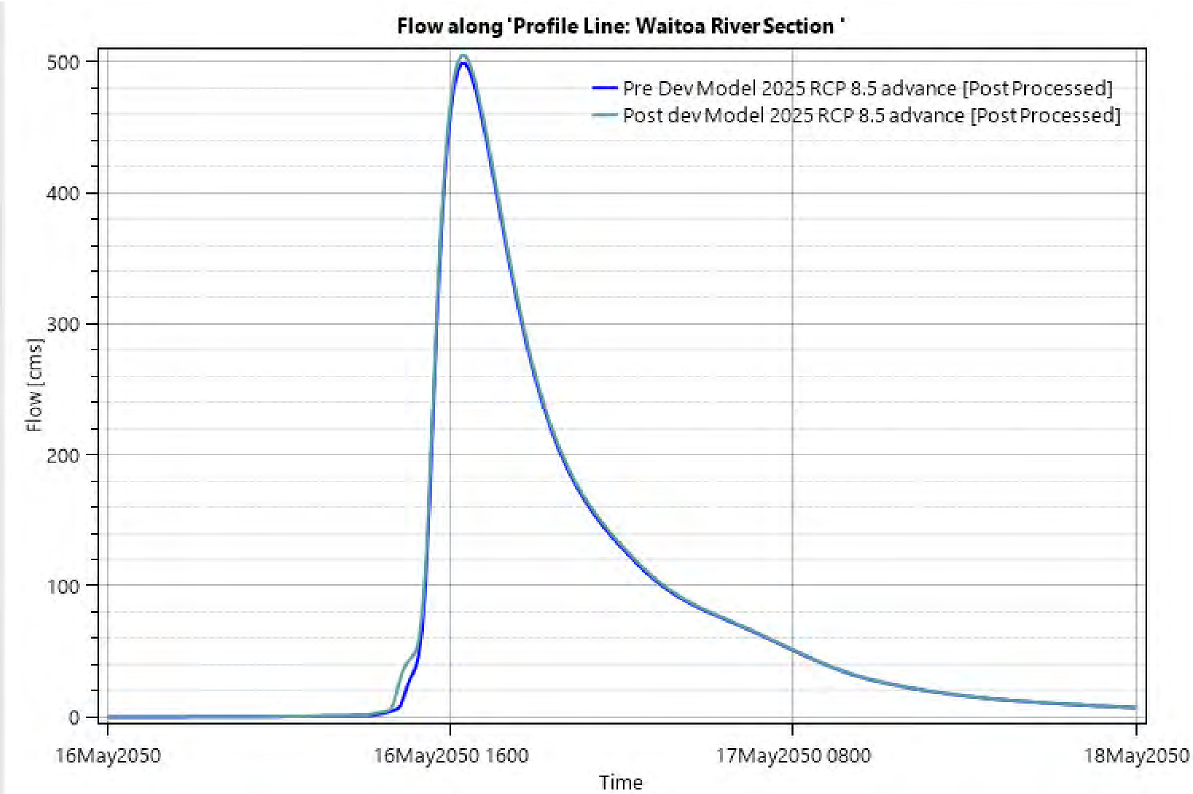

Flow along 'Profile Line: RV Section 02 '
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Flow [cms]
16May2050
16May2050 1600
17May2050 0800
18May2050
Time


Water Surface Elevation on 'Greenway Section 01'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - PGL RV-RES-SF' Profile
Value [m]
70
69
68
67
66
65
0
100
200
300
400
500
600
Station [m]

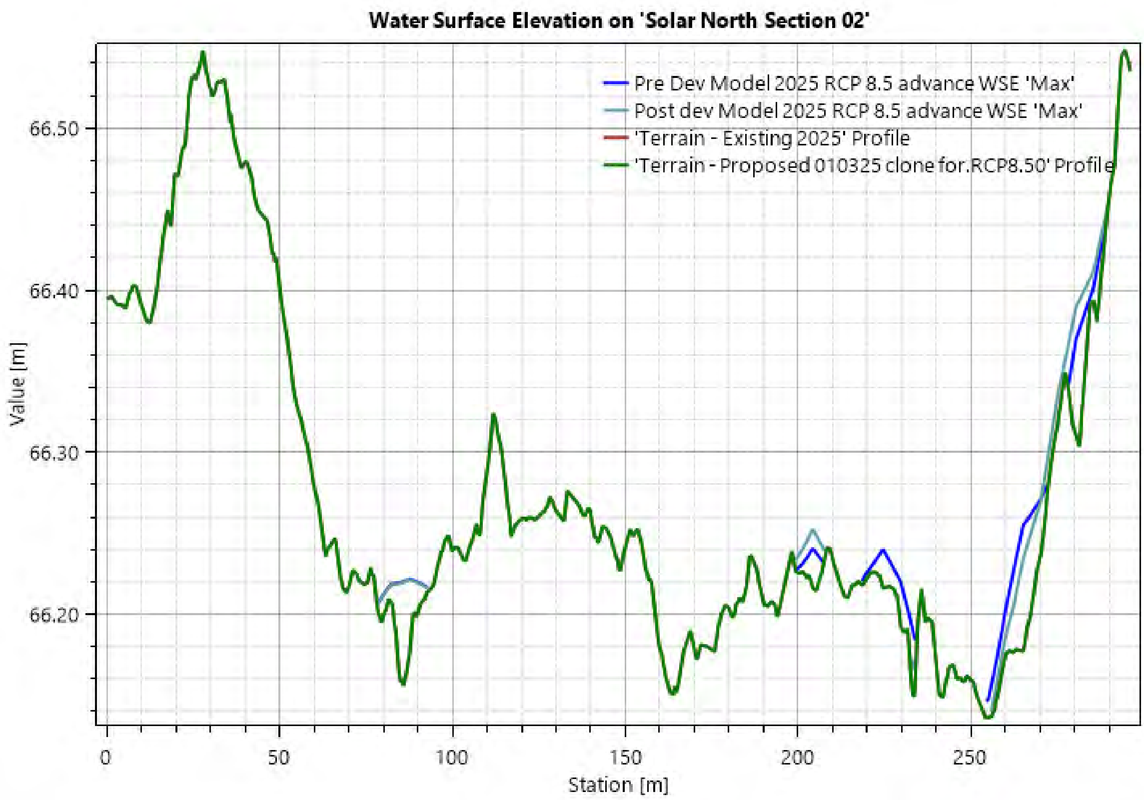

Flow along 'Profile Line: Greenway Section 01 '
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
8
6
4
2
0
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

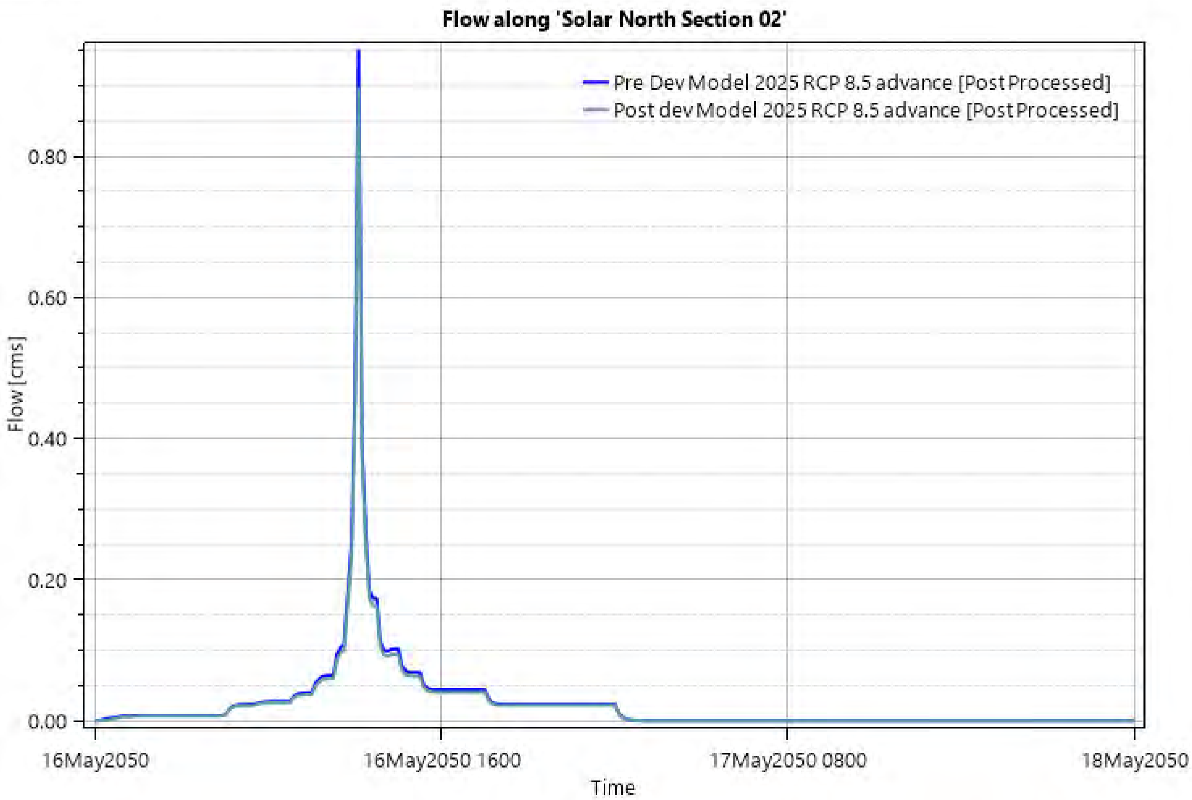


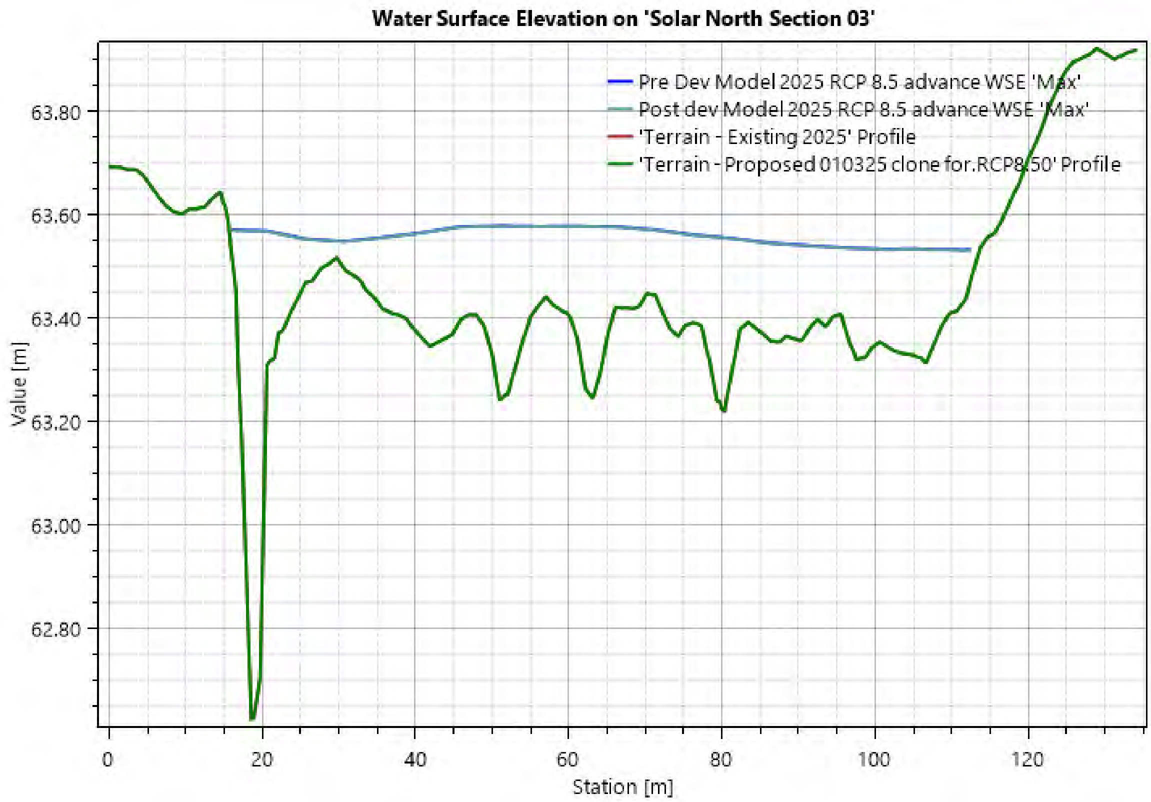
Water Surface Elevation on 'Profile Line: Waitoa River Section '
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - PGLRV-RES-SF' Profile
Value [m]
Station [m]
58
60
62
64
66
68
0
20
40
60
80
100
120
140
160
180
200

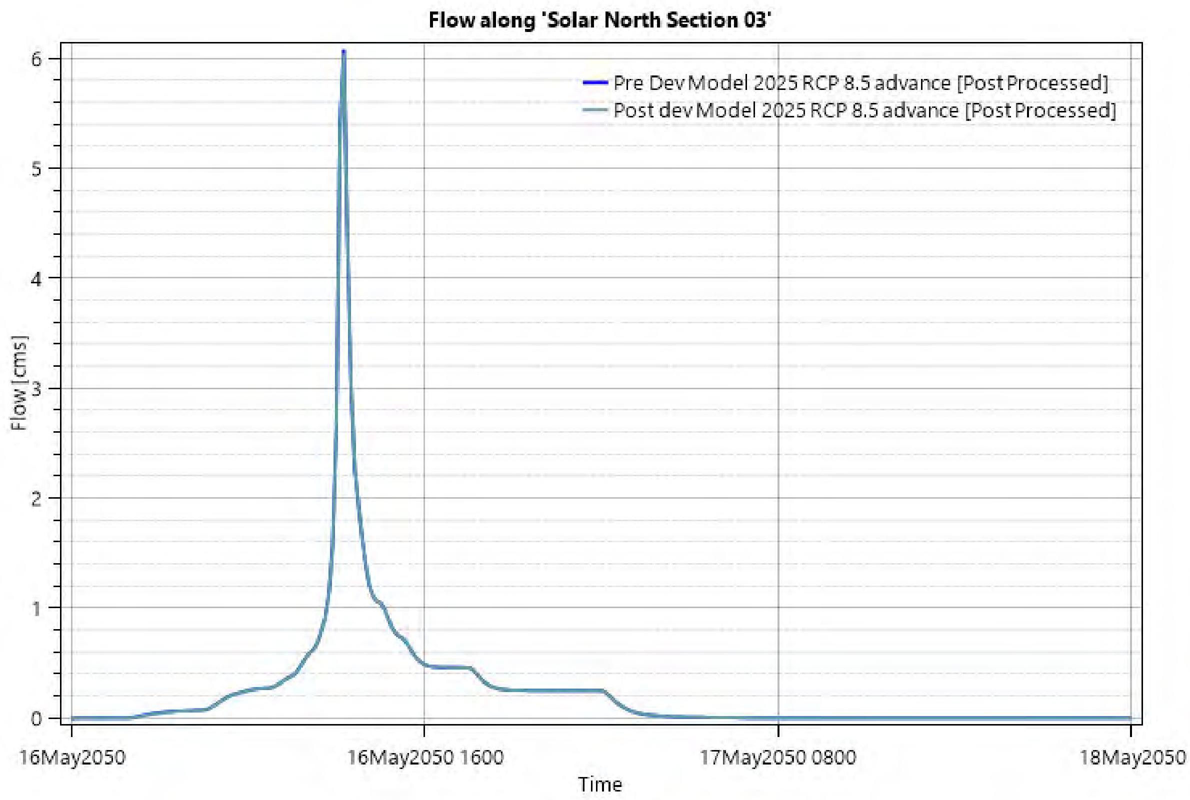


Flow along 'Profile Line: Waitoa River Section '
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
Time
0
100
200
300
400
500
16May2050
16May2050 1600
17May2050 0800
18May2050


Water Surface Elevation on 'Solar North Section 02'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
Value [m]
66.50
66.40
66.30
66.20
0
50
100
150
200
250
Station [m]


Flow along 'Solar North Section 02'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
Flow [cms]
0.80
0.60
0.40
0.20
0.00
16May2050
16May2050 1600
17May2050 0800
18May2050
Time


Water Surface Elevation on 'Solar North Section 03'
Pre Dev Model 2025 RCP 8.5 advance WSE 'Max'
Post dev Model 2025 RCP 8.5 advance WSE 'Max'
'Terrain - Existing 2025' Profile
'Terrain - Proposed 010325 clone for.RCP8.50' Profile
63.80
63.60
63.40
63.20
63.00
62.80
Value [m]
0
20
40
60
80
100
120
Station [m]


Flow along 'Solar North Section 03'
Pre Dev Model 2025 RCP 8.5 advance [Post Processed]
Post dev Model 2025 RCP 8.5 advance [Post Processed]
6
5
4
3
2
1
0
Flow [cms]
16May2050
16May2050 1600
17May2050 0800
18May2050
Time

Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt A Section 1

User-defined

Invert Elev (m)	=	66.6700
Slope (%)	=	0.7000
N-Value	=	0.015

Calculations

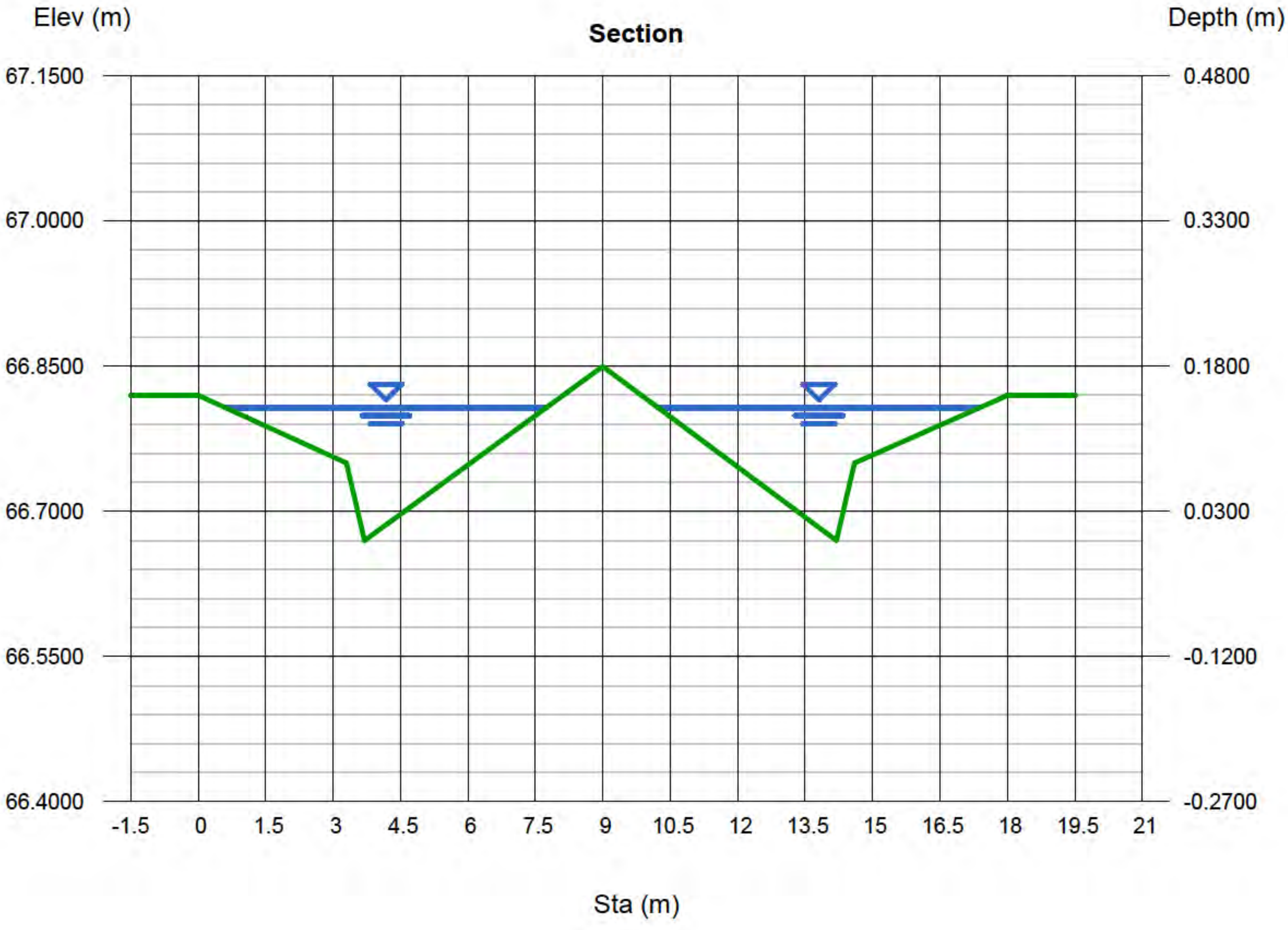
Compute by:		Known Q
Known Q (cms)	=	0.6000

Highlighted

Depth (m)	=	0.1372
Q (cms)	=	0.6000
Area (sqm)	=	0.7825
Velocity (m/s)	=	0.7668
Wetted Perim (m)	=	14.2849
Crit Depth, Yc (m)	=	0.1402
Top Width (m)	=	14.2631
EGL (m)	=	0.1671

(Sta, El, n)-(Sta, El, n)...

(0.0100, 66.8200)-(3.3000, 66.7500, 0.015)-(3.7000, 66.6700, 0.015)-(9.0000, 66.8500, 0.015)-(14.2000, 66.6700, 0.015)-(14.6000, 66.7500, 0.015)-(18.0000, 66.82



Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. | Monday, Mar 17 2025

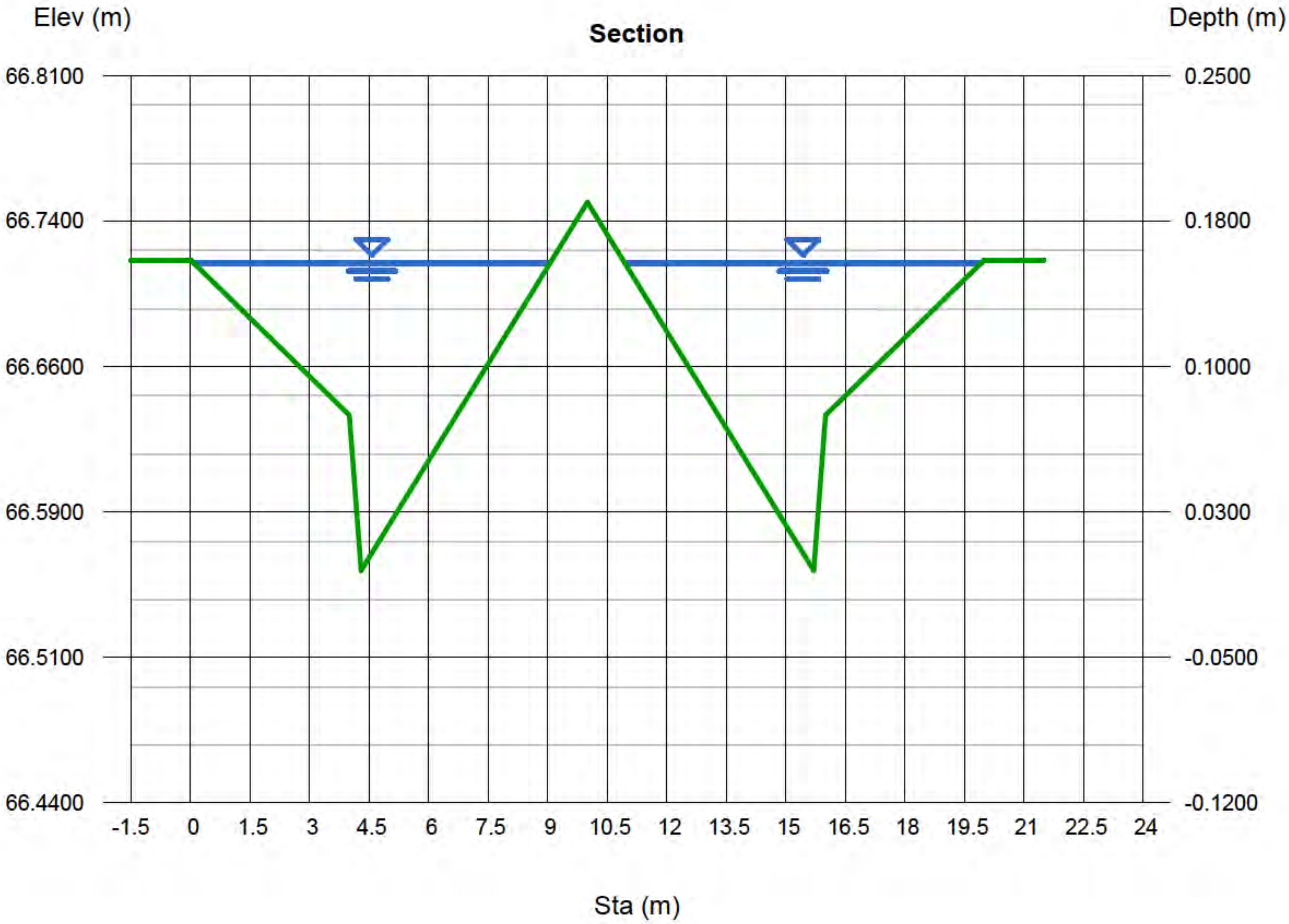
Catchemnt A Section 2

User-defined
Invert Elev (m) = 66.5600
Slope (%) = 0.9000
N-Value = 0.015

Calculations
Compute by: Known Q
Known Q (cms) = 1.1000

Highlighted
Depth (m) = 0.1585
Q (cms) = 1.1000
Area (sqm) = 1.1324
Velocity (m/s) = 0.9714
Wetted Perim (m) = 17.9771
Crit Depth, Yc (m) = 0.1707
Top Width (m) = 17.9492
EGL (m) = 0.2066

(Sta, El, n)-(Sta, El, n)...
(0.0100, 66.7200)-(4.0000, 66.6400, 0.015)-(4.3000, 66.5600, 0.015)-(10.0000, 66.7500, 0.015)-(15.7000, 66.5600, 0.015)-(16.0000, 66.6400, 0.015)-(20.0000, 66.7



Channel Report

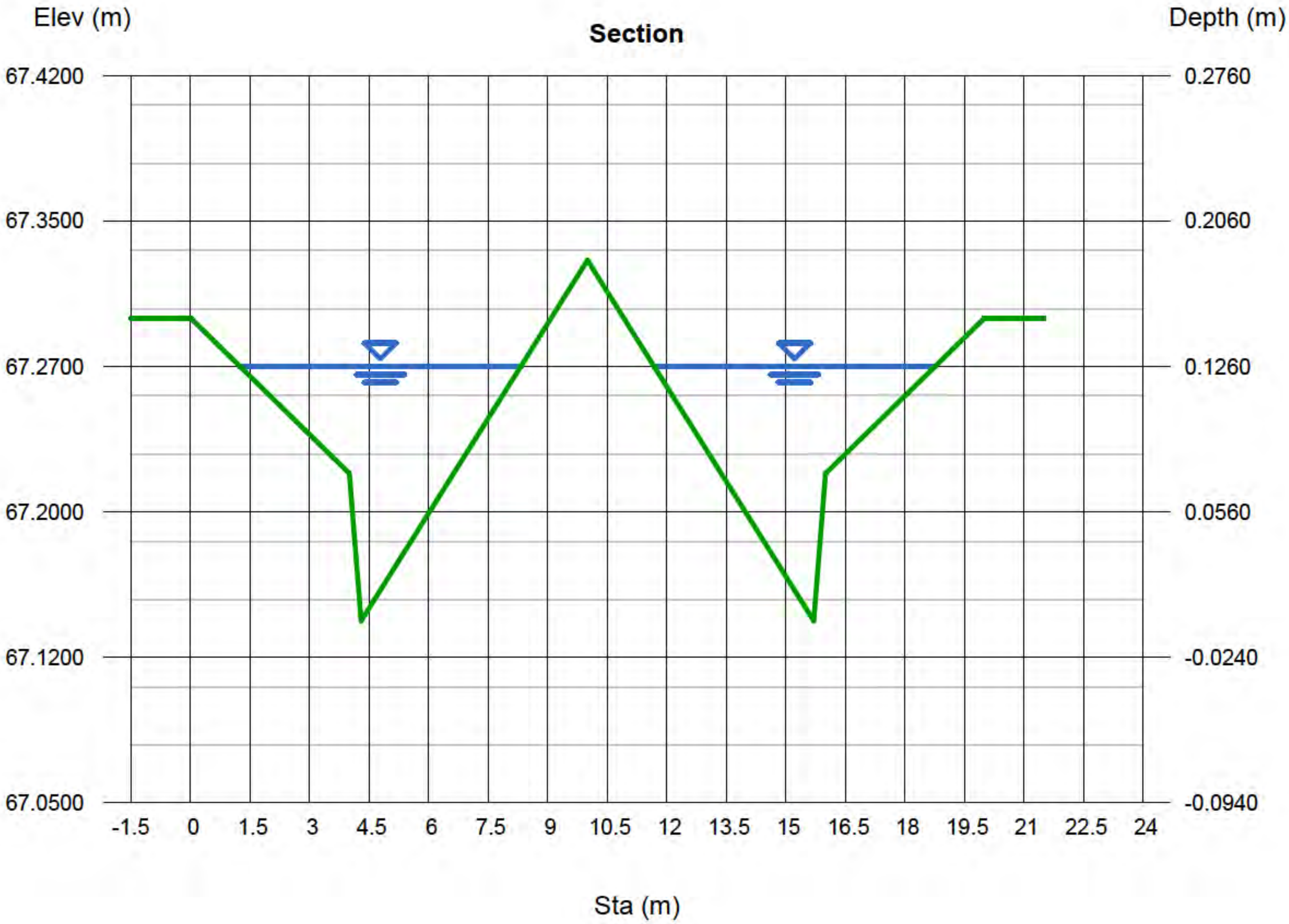
Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. | Monday, Mar 17 2025

Catchemnt A Section 3

User-defined		**Highlighted**	
Invert Elev (m)	= 67.1440	Depth (m)	= 0.1311
Slope (%)	= 0.9000	Q (cms)	= 0.6000
N-Value	= 0.015	Area (sqm)	= 0.7336
		Velocity (m/s)	= 0.8179
Calculations		Wetted Perim (m)	= 14.1554
Compute by:	Known Q	Crit Depth, Yc (m)	= 0.1372
Known Q (cms)	= 0.6000	Top Width (m)	= 14.1310
		EGL (m)	= 0.1652

(Sta, El, n)-(Sta, El, n)...

(0.0100, 67.3000)-(4.0000, 67.2200, 0.015)-(4.3000, 67.1440, 0.015)-(10.0000, 67.3300, 0.015)-(15.7000, 67.1440, 0.015)-(16.0000, 67.2200, 0.015)-(20.0000, 67.3



Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt B Section 1

User-defined
Invert Elev (m) = 68.0800
Slope (%) = 0.5000
N-Value = 0.015

Calculations
Compute by: Known Q
Known Q (cms) = 0.9000

Highlighted
Depth (m) = 0.1646
Q (cms) = 0.9000
Area (sqm) = 1.1164
Velocity (m/s) = 0.8062
Wetted Perim (m) = 14.7110
Crit Depth, Yc (m) = 0.1615
Top Width (m) = 14.6887
EGL (m) = 0.1977

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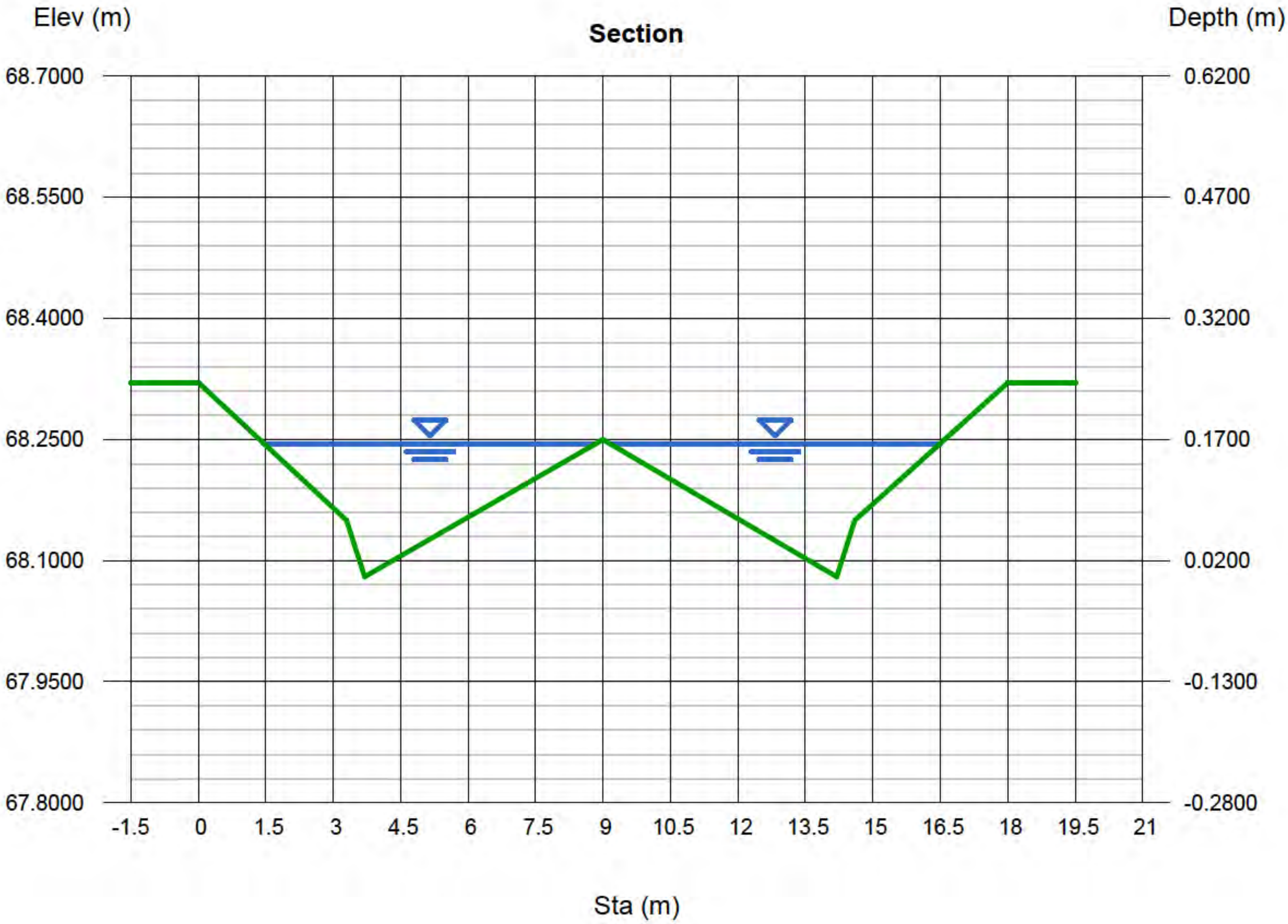

Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt B Section 2

User-defined		**Highlighted**	
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Slope (%)	= 0.5000	Q (cms)	= 0.3000
N-Value	= 0.015	Area (sqm)	= 0.5120
		Velocity (m/s)	= 0.5859
Calculations		Wetted Perim (m)	= 11.1529
Compute by:	Known Q	Crit Depth, Yc (m)	= 0.1128
Known Q (cms)	= 0.3000	Top Width (m)	= 11.1273
		EGL (m)	= 0.1333

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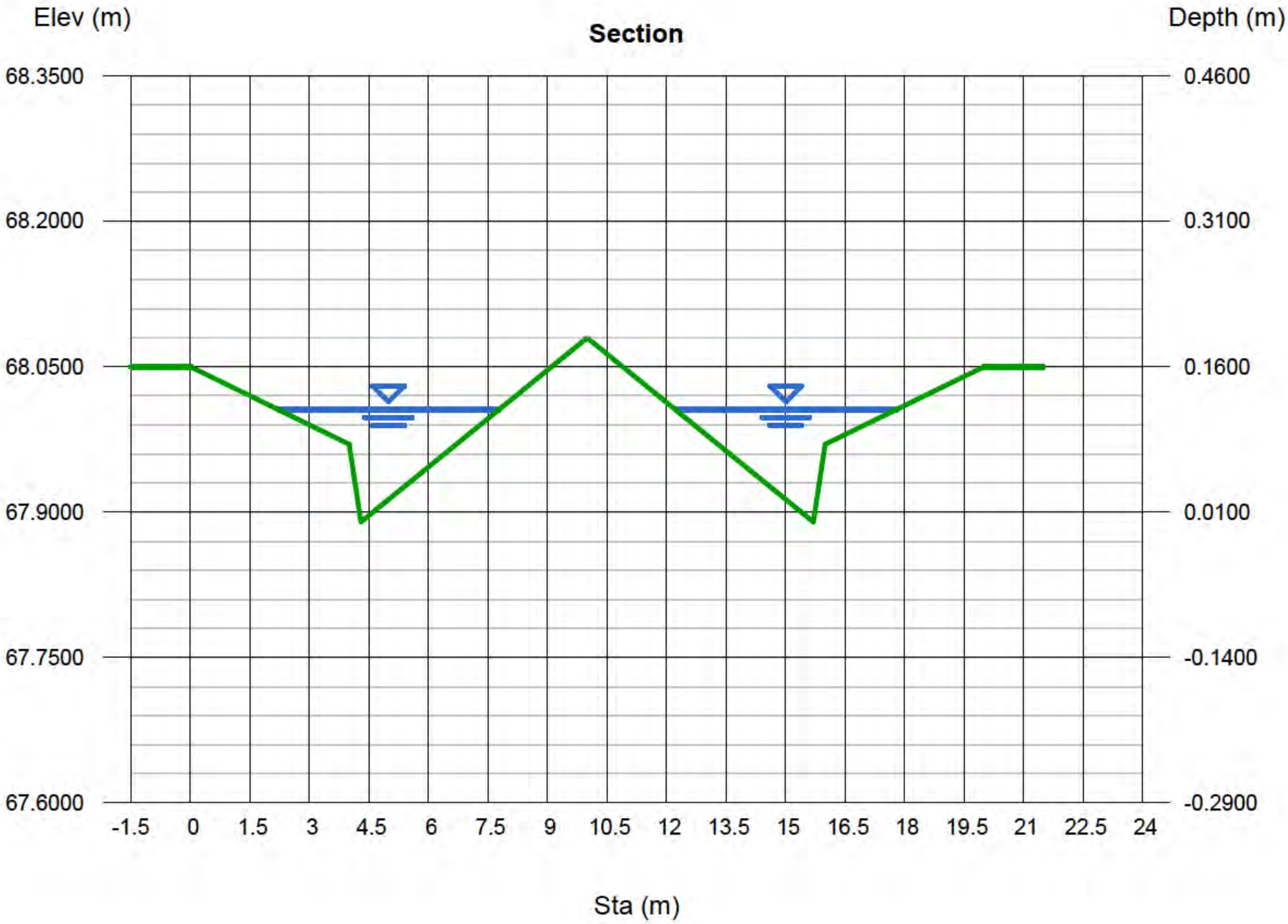

Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt B Section 3

User-defined

Invert Elev (m)	=	66.3200
Slope (%)	=	0.7000
N-Value	=	0.015

Calculations

Compute by:		Known Q
Known Q (cms)	=	0.2000

Highlighted

Depth (m)	=	0.0914
Q (cms)	=	0.200
Area (sqm)	=	0.3253
Velocity (m/s)	=	0.6147
Wetted Perim (m)	=	8.5125
Crit Depth, Yc (m)	=	0.0945
Top Width (m)	=	8.4970
EGL (m)	=	0.1107

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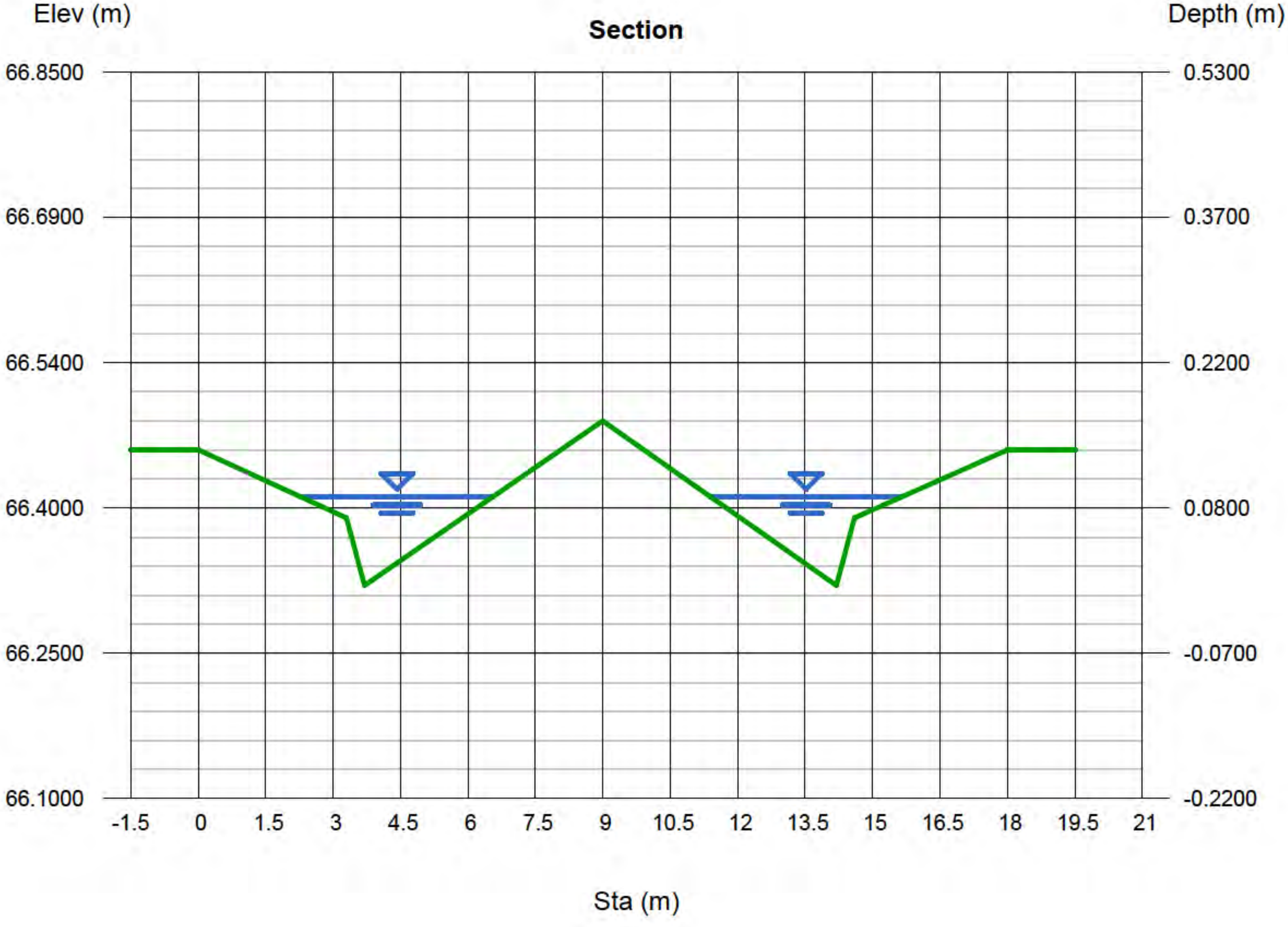

Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. | Monday, Mar 17 2025

Catchemnt D Section 4

User-defined
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Slope (%) = 0.3000
N-Value = 0.015

Calculations
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Known Q (cms) = 1.5000

Highlighted
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Q (cms) = 1.5000
Area (sqm) = 1.9030
Velocity (m/s) = 0.7882
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Top Width (m) = 18.0554
EGL (m) = 0.2481

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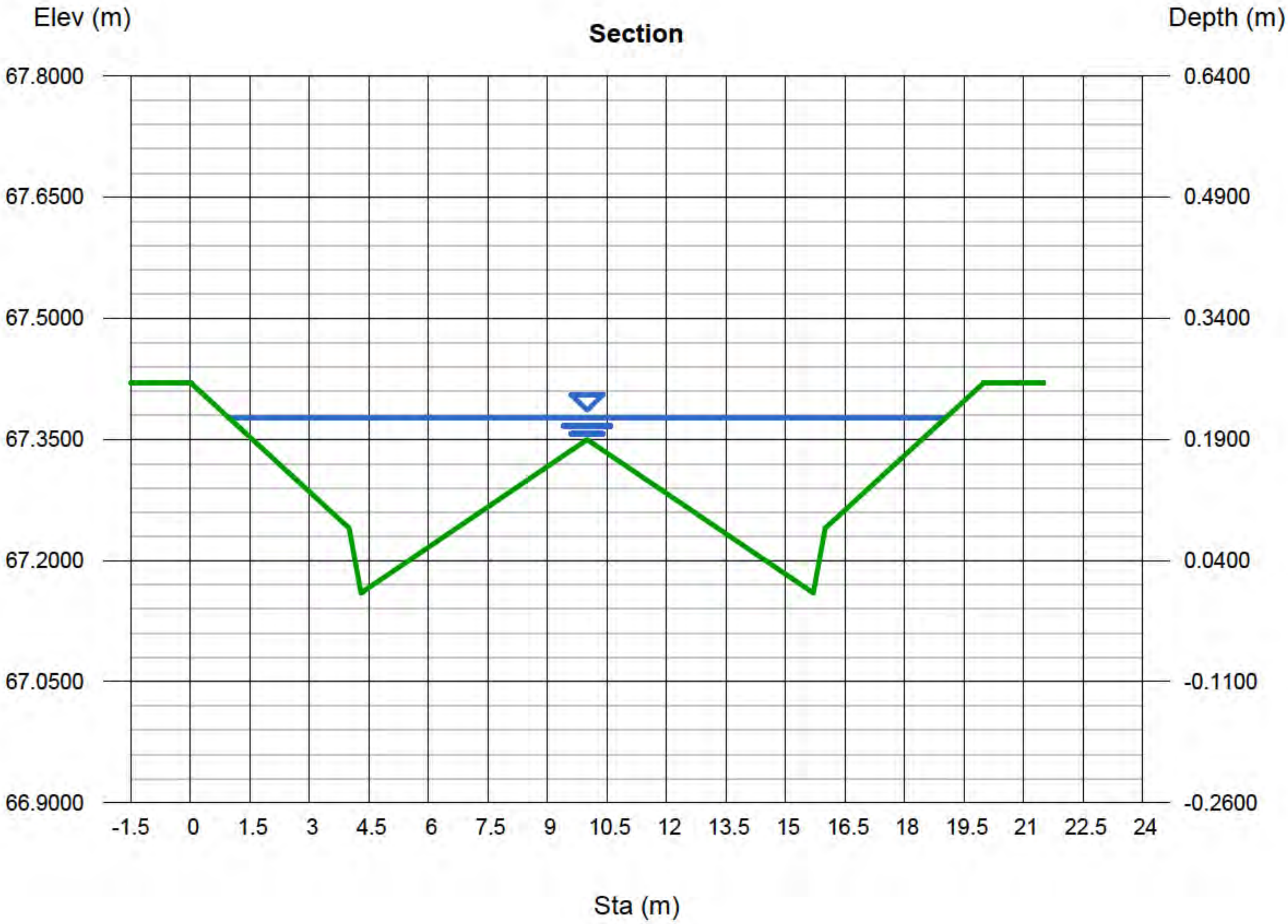

Channel Report

Catchemnt B Section 5

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N-Value = 0.015

Calculations
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Known Q (cms) = 0.6000

Highlighted
Depth (m) = 0.1219
Q (cms) = 0.6000
Area (sqm) = 0.6574
Velocity (m/s) = 0.9127
Wetted Perim (m) = 13.3098
Crit Depth, Yc (m) = 0.1341
Top Width (m) = 13.2927
EGL (m) = 0.1644

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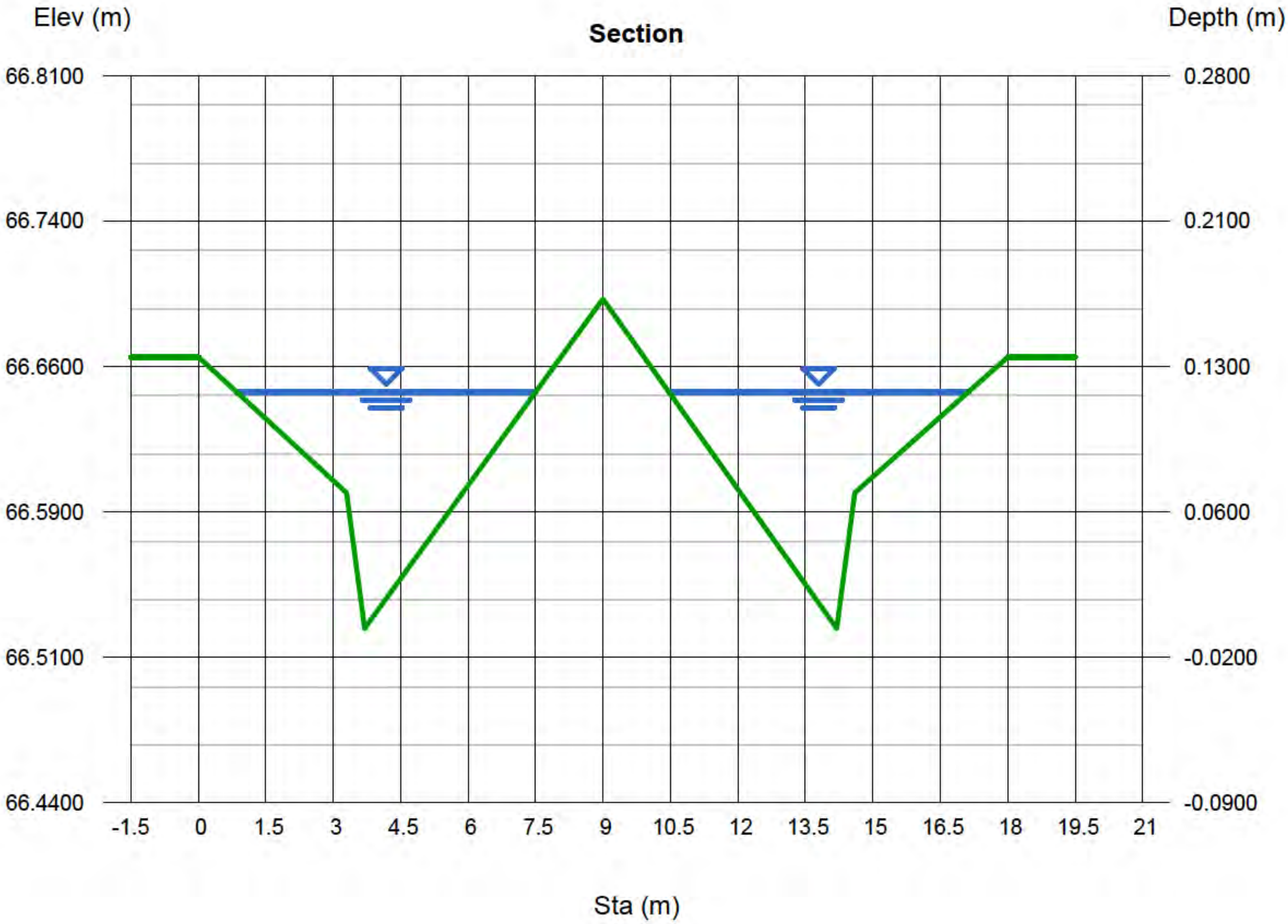

Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt C Section 1

User-defined		**Highlighted**	
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Slope (%)	= 0.9000	Q (cms)	= 0.5000
N-Value	= 0.015	Area (sqm)	= 0.6205
		Velocity (m/s)	= 0.8058
Calculations		Wetted Perim (m)	= 12.6157
Compute by:	Known Q	Crit Depth, Yc (m)	= 0.1341
Known Q (cms)	= 0.5000	Top Width (m)	= 12.5896
		EGL (m)	= 0.1581

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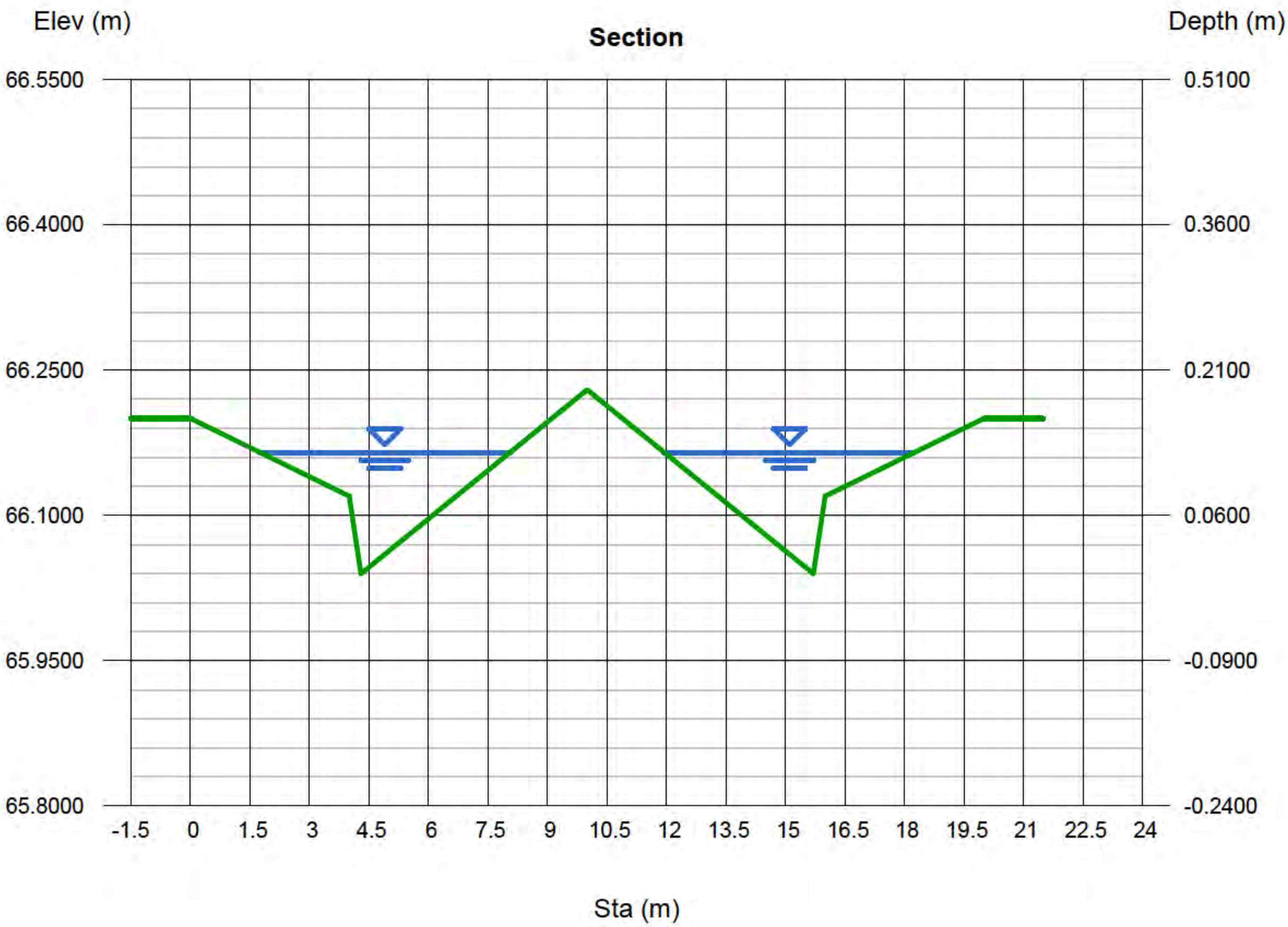

Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt D Section 1

User-defined		**Highlighted**	
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Slope (%)	= 0.5000	Q (cms)	= 0.3000
N-Value	= 0.015	Area (sqm)	= 0.5121
		Velocity (m/s)	= 0.5858
Calculations		Wetted Perim (m)	= 11.1542
Compute by:	Known Q	Crit Depth, Yc (m)	= 0.1128
Known Q (cms)	= 0.3000	Top Width (m)	= 11.1286
		EGL (m)	= 0.1333

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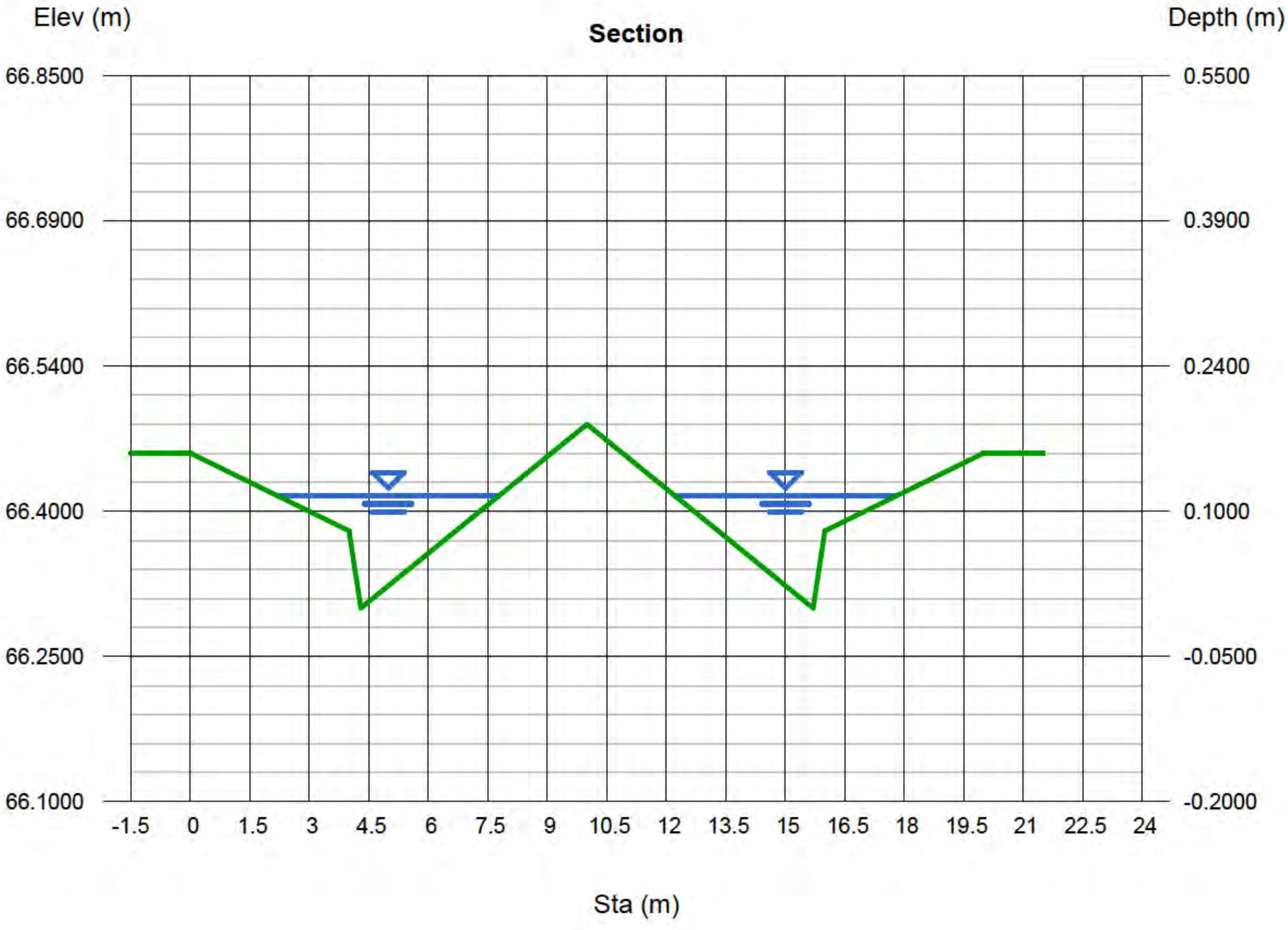

Channel Report

Hydraflow Express Extension for Autodesk® Civil 3D® by Autodesk, Inc. Monday, Mar 17 2025

Catchemnt D Section 2

User-defined		**Highlighted**	
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Slope (%)	= 0.5000	Q (cms)	= 0.3000
N-Value	= 0.015	Area (sqm)	= 0.5070
		Velocity (m/s)	= 0.5917
Calculations		Wetted Perim (m)	= 11.3907
Compute by:	Known Q	Crit Depth, Yc (m)	= 0.1067
Known Q (cms)	= 0.3000	Top Width (m)	= 11.3742
		EGL (m)	= 0.1276

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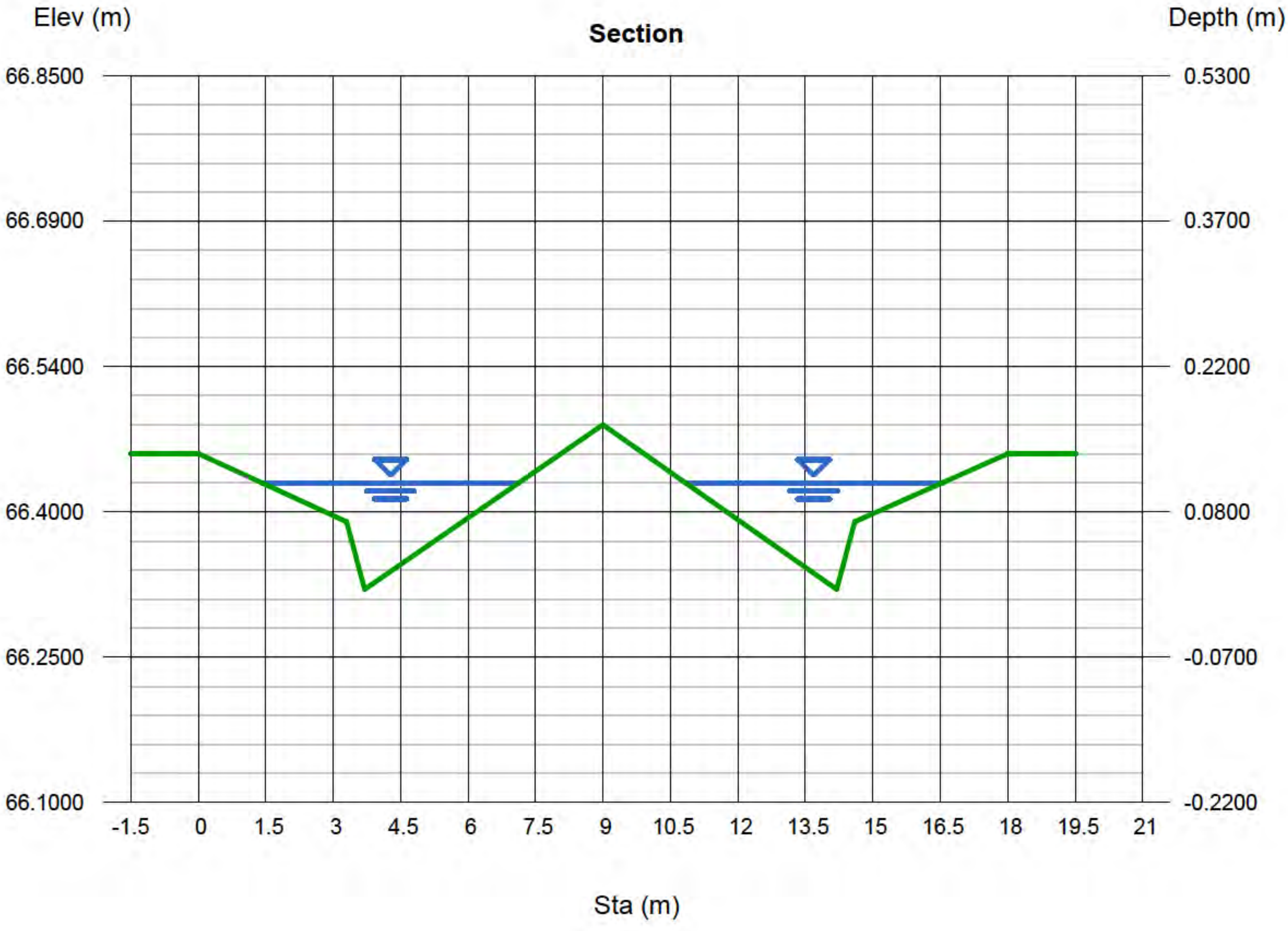
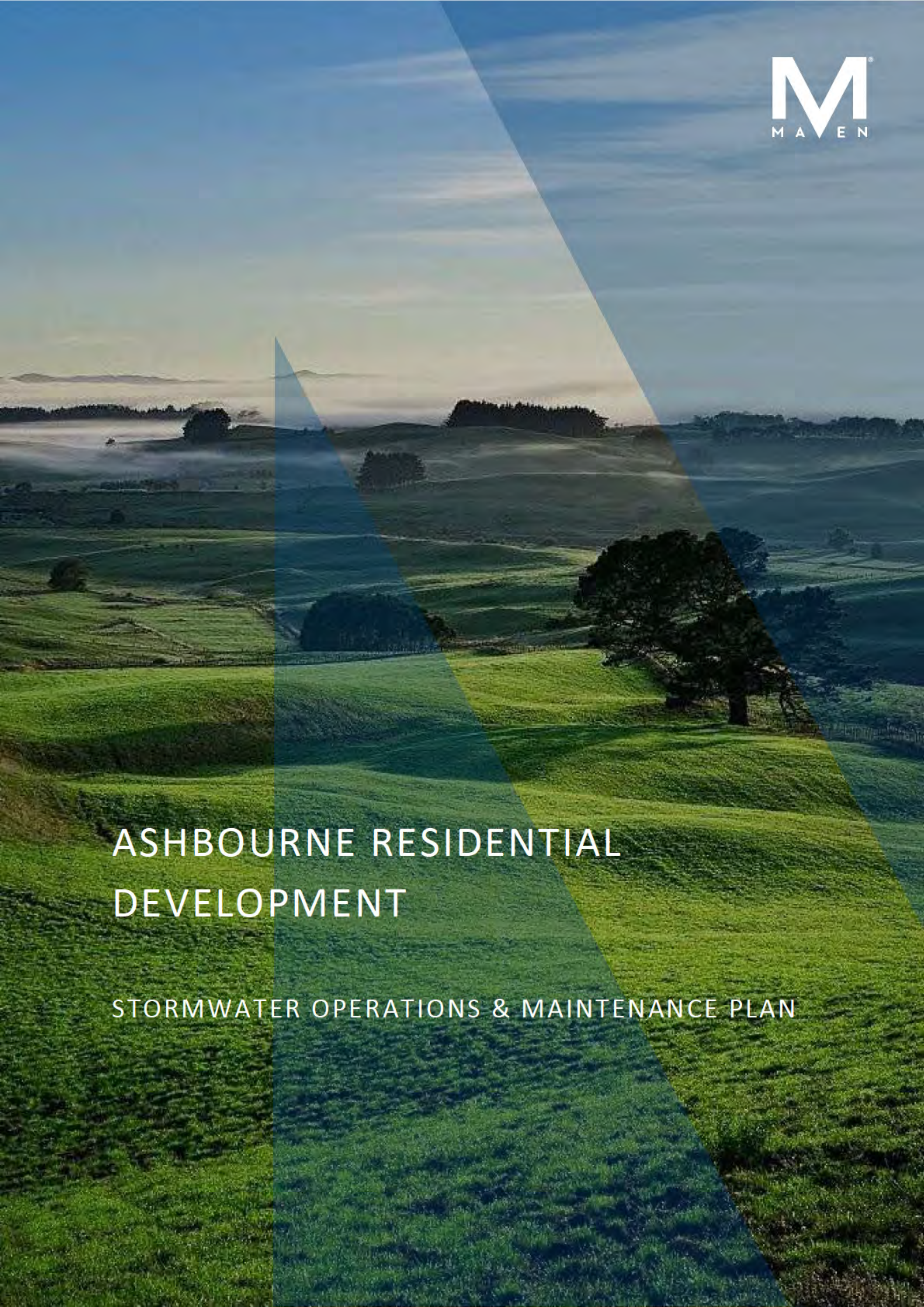


Appendix C - CMW Geotech Report- Bounded with Infrastructure reports

APPENDIX D – STORMWATER DEVICE OPERATIONS MAINTENANCE PLAN



ASHBOURNE RESIDENTIAL DEVELOPMENT

STORMWATER OPERATIONS & MAINTENANCE PLAN

PROJECT INFORMATION

CLIENT	Matamata Developments Ltd
PROJECT	289001

DOCUMENT CONTROL

DATE OF ISSUE	30/05/2025
REVISION	A
AUTHOR	Min Shon Engineer
REVIEWED BY	Raatite Kanimako Engineer
APPROVED BY	Dean Morris Regional Director

Level 1, 286 Victoria Street, Hamilton Central, New Zealand
Phone 07 242 0601
www.maven.co.nz

Contents

1. Introduction

1.1. Background

Maven Waikato Ltd have been engaged by Matamata Developments Ltd to undertake Infrastructure Design in support of Ashbourne Residential Development at 127 Station Road, Matamata.

1.2. Purpose of this report

The purpose of this operation and maintenance report is to ensure the correct ongoing operation of the stormwater quality management devices of Ashbourne Residential development. The information provided herein outlines the methodology associated with the stormwater infrastructure onsite. This report is to be read in conjunction with the engineering drawings around stormwater management within the site held between WRC and Matamata-Piako District Council (MPDC).

It is the responsibility of the nominated maintenance contractor for MPDC to carry out maintenance of the stormwater system devices. The maintenance will be in generally accordance with this document, WRC Stormwater Guidelines and MPDC's SW Guidelines.

1.3. Stormwater Assets

The Public Assets constructed which will require operation and maintenance are:

- Stormwater discharge from Ashbourne Residential will be conveyed via roadside soakage trench and piped network (perforated), and road for OLFP to each Soakage Basin.
- Stormwater Basins with soakage devices in the base.
- Raingardens constructed within the road reserves providing quality for whole catchment. These devices are at-source rain gardens providing water quality, infiltration, and detention.
- Stormwater Greenway which carries stormwater runoff from Catchment B.

1.4. Contact Information

A summary of the contact information relating to the ownership, maintenance manager, and designer for the stormwater system is included below.

Asset ID:		Resource Consent Number:	
Location:	127 Station Road, Matamata	Development Name / Legal Description:	
Asset Owner Details:			
Name:	Matamata-Piako	Postal Address: PO Box 266, Te Aroha 3342 Physical Address: Te Aroha Office - 35 Kenrick Street, Te Aroha Matamata-Piako Civic and Memorial Centre - 11 Tainui Street, Matamata Morrinsville Area Office - 56-62 Canada Street, Morrinsville	
Telephone Number:	0800 746 467		
Email:	Refer to Online Contact Form		
Maintenance Manager Emergency Contact Details			
Name:	MPDC call centre	Address: 56-62 Canada Street, Morrinsville	
Telephone Number: (Daytime)	0800 746 467		
Telephone Number: (Out-Of-Hours)	0800 746 467		
Email:	Relevant operational area inbox		

Designer Details:		
Name:	Dean Morris	Level 1, 286 Victoria Street, Hamilton Central, Hamilton 3204
Telephone Number:	09 242 2724	
Email:	deanm@maven.co.nz	
Applicant Details		
Name:	Matamata Developments Ltd	Address: 127 Station Road, Matamata
Telephone Number		
Email:		
Landowner Details:		
Name:	Matamata Developments Ltd	
Telephone Number		
Email:		
Notes / Restrictions / Access		

2. Stormwater System Description

2.1. Site Description

The Ashbourne Residential area is a circa 45.2ha block of land within the Matamata-Piako District. The current site access is through 127 Station Road in Matamata. The site adjoins with the new Highgrove Development to the north-west, and Peakedale and Pippins Development to the east, and the remainder of the site is surrounded by agricultural land.

There is an existing stormwater swale that follows the southern and western boundary. The Waitoa River which runs south to north is approximately 1km to the west of the subject site.

The site has an existing farmhouse located at 127 Station Road. Most of the site is low-lying flat farmland, that is interspersed with artificial farm drains.

2.2. Design Standards

The MPDC Development Manual sets out design and construction standards for stormwater and requires all land development projects to be provided with a mean of stormwater disposal.

Stormwater systems have been designed in accordance with RITS and other relevant standards including the MPDC Development Manual 2010 and caters for the primary pipe system up to the 10-year event as well as the secondary system and overland flow paths to manage excess runoff that cater for events exceeding the capacity of the primary piped system for events exceeding the 10-year event.

2.3. Stormwater Management Plan (SMP)

The planned development straddles the existing Network Discharge Consent ('NDC') boundary. As the development relates to undeveloped land stormwater discharge consents have been obtained from Waikato Regional Council.

The overarching stormwater strategy has been derived from the Maven Waikato SMP which sets out the high-level, best practice approach to stormwater management within the Ashbourne Residential development site. The SMP outlines the overarching stormwater management principles which will form the basis of stormwater design to support future development on the proposed sites.

Furthermore, the stormwater management strategy, as detailed within the SMP, establishes a robust long term stormwater solution, which integrates desired urban form outcomes, with the mitigation of flooding (flood plains and OLFPs) and consideration of best-practice design outcomes as detailed within relevant Waikato guidance documents.

The key components of the Ashbourne stormwater management strategy are as follows:

- Stormwater conveyance for 10yr cc ARI rainfall event
- Overland flow paths for (100yr – 10yr) cc ARI rainfall event to be accommodated within the site and conveyed to basins.
- Treatment of runoff prior to discharge into receiving environment in accordance with TP10 / GD01 / Waikato Stormwater Management Guidelines (WRC Technical Report 2018/01).
- Usage of soakage where possible

For further details please refer to the SMP prepared by Maven Waikato Ltd dated April 2025.

2.4. Capacity and Quality

Stormwater Strategy for Lot Areas

Roof runoff is managed using inert roofing materials, while driveway runoff is directed through a catch pit with a sump for pre-treatment before disposal into a private soakage device. Overflow is located in the catchpit system for flows surpassing the 10-year event within the lot areas. Excess flows will be diverted into the downstream basin via the road carriageway.

Stormwater Strategy for Road Carriageway

The initial runoff volume (WQV) is treated via proposed roadside raingardens. the proposed rain gardens are integrated with the roadside soakage trench combined to cater for the 10-year event. Flows exceeding the 10-year soakage capacity are redirected back into the road carriageway and get discharged at the downstream stormwater Basin.

Stormwater Strategy for SW Basin A, C, and D

These basins forms critical part of the overall stormwater Mitigation system. They have been sized to accommodate the 100-year event or excess flows from both the road carriageway and on lot flows exceeding the 10-year event. Additionally, the upstream inflows particularly in basins A and D has been accounted for as well in these basins. it is anticipated that through soakage and storage capacity of the proposed basins, no flows are expected to discharge into the downstream environment from these basins.

Stormwater Strategy for SW Basin B and Greenway

Stormwater Basin B is connected with the greenway and both serve a dual purpose; Attenuating flows from Catchment B flows (to at least 80% pre-development) and conveying flows from the southern solar farm and external inflow from the southern external catchment as depicted in plans.

2.4.1. Hydrogeological Assessment

Wallbridge Gilbert Aztec Ltd has been prepared the Hydrogeology – Assessment of Effects for the Ashbourne construction. Please consult the Hydrogeological Effects Assessment prepared by WGA Ltd which provides detailed guidance on hydrogeological measures.

2.5. Flooding

The WRC hazard portal has indicated there is potential flooding along the western side of Highgrove Development in the 100 years storm event, however there is no flooding indicated within the subject development.

A flood model has been developed and calibrated using the region's observed rainfall and river data. The model has been used to test a sensitivity scenario in which all primary stormwater devices are blocked. Even under this worst-case condition, the model demonstrates that the development maintains flood immunity, with only minor exceedances expected to be mitigated through detailed design refinements.

2.6. Overland Flow

Additional branches of OLFPs will be created as roading corridors are formed. The following measures will be adopted to mitigate their effects of these overland flowpaths on the proposed development.

- Identify and maintain natural overland flow/watercourse locations to convey concentrated stormwater from the site. Utilise existing culverts (where possible) to maintain the same discharge locations, post development.
- Identify and retain any upstream OLFPs and/or watercourses to avoid any upstream flooding.

- Ensure OLFPs are to be designed where possible within the roading network and discharge into watercourses and 100-year detention devices.

The preliminary OLFP design is shown in Maven Associates drawings C460 contained within Appendix A. summary of results provided below Detailed design of the OLFPs will be provided at future detail design stage following the approval of the resource consent.

An assessment of the post development overland flow paths (OLFPs) has been carried out to evaluate the behaviour of surface runoff in the road carriageway under the proposed stormwater management system. The design scenario is based on the RCP8.5 climate change scenario, incorporating all proposed soakage and treatment devices and the assessment is done through Autodesk Hydroflo software. The OLFPs represents the conveyance of surface runoff as a result of the proposed system during the 100-year storm event.

Flow depths and velocities were assessed at key locations throughout the development covering all the various road/Accessway typologies ensuring and confirming conveyance of the OLFP is viable through proposed carriageway. See below table showing results at the key locations.

	SECTION	CATCHMENT AREA HA	FLOWRATE m3/s	MAX DEPTH m	VELOCITY m/s	DEPTH x VELOCITY
CATCHMENT A	1	3.1	0.6	0.137	0.77	0.105
	2	6.2	1.1	0.158	0.97	0.153
	3	2.9	0.6	0.131	0.82	0.107
CATCHMENT B	1	5.9	0.9	0.167	0.81	0.135
	2	2.3	0.3	0.116	0.59	0.068
	3	1.1	0.2	0.091	0.61	0.056
	4	12.5	1.5	0.216	0.79	0.171
	5	4.2	0.6	0.121	0.912	0.110
CATCHMENT C	1	2.3	0.5	0.125	0.81	0.101
CATCHMENT D	1	2.8	0.3	0.116	0.59	0.068
	2	2.7	0.3	0.109	0.59	0.064

Table 1: OLFP Results

Most OLFP sections comply with standard design thresholds. However, three sections recorded maximum water depths above the 150mm guideline.

- Catchment A – Section 2: Max Depth = 0.158
- Catchment B – Section 4: Max Depth = 0.216
- Catchment C – Section 1: Max Depth = 0.137m (just below threshold)

Despite minor exceedances in depth, depth x velocity (m2/s) values remain well below critical safety thresholds defined in Austroads 2012 Part 5, which specify;

- < 0.4m^2/s pedestrian Safety
- <0.6m^2/s for vehicle safety

The highest recorded value was 0.171m^2/s confirming safe flow conveyance for both pedestrians and vehicles under design conditions. Flow is primarily routed along proposed roads conveyed into roadside treatment and 10year mitigation devices prior to spilling back (during event above the 10year) onto the road and get discharged into the proposed basins or Greenway.

It is noted that a separate flood sensitivity analysis has been completed using HEC-RAS 2D modelling assuming all stormwater devices are fully blocked. The assessment detailed in section 7 of SMP, evaluates overland flow behaviour under worst case flooding conditions within and surrounding the site.

3. Stormwater System Devices

3.1. Raingardens

Raingardens are the primary stormwater treatment mechanism for Ashbourne Residential development. They have been designed to treat stormwater run-off from hardstand areas such as roads, footpaths, car parks etc. by filtering it through vegetation and then soaking vertically through an organic loam soil mix before draining into the piped stormwater network.

Vegetation

Vegetation enhances raingarden performance for stormwater treatment and therefore requires close attention.

Maintenance includes fertilising plants, removing noxious plants or weeds, re-establishing plants that die and maintaining mulch cover.

Regular inspections by the responsible entity must be done to ensure that the desired vegetation remains and is not overtaken by invasive undesirable plants.

In some situations, the replacement of the planted vegetation by a volunteer species may be beneficial, but only if the invasive species provides equal or increased water quality benefits and is accepted by the owners of the site.

Plants

Use native plants as per the approved landscaping plans to replace plants if this is required.

Sediment

Sediments accumulate in raingardens. Removal should occur when surface ponding lasts significantly longer than the one day drain time, which indicates surface clogging. When sediments are to be removed, it is essential to restore the vegetation and soil conditions to the originally constructed condition.

Sediment removal will necessitate disturbance of the vegetation, so steps will have to be taken to re-establish the vegetation upon completion of sediment removal.

Erosion control in the contributing drainage area also will be necessary to prevent scour and excessive sedimentation in the rain garden until there is once again a dense stand of vegetation.

Sediment may also impede effective performance of a rain garden by clogging the soil surface and preventing design storms from being treated. If stormwater backs up into the upstream drainage area, overflow may occur and bypass the treatment area.

Debris

Similar to other types of practices, debris removal is an ongoing maintenance function at all rain garden systems.

Debris, if not removed, can block inlets or outlets, and can be unsightly if located in a visible location.

Soil

Only use approved raingarden soil (usually a sandy loam compost) which is readily available at some horticultural centres.

Drainage Testing

If water is not observed freely draining from the rain garden outlet it may be blocked. Back wash through the outlet and/or maintenance access port until the rain garden is freely draining. If this does not help then the soil may be blocked and need to be removed, pipes inspected/cleared.

Avoid

- The use of sprays to kill weeds or algae as this will contaminate the downstream waterways.
- Do not compact the rain garden soil mix.
- Do not add clay or silt in the rain garden soil mix as this will restrict infiltration.

Inspection requirements

- Debris cleanout
    - Removal of debris
    - No dumping of wastes into raingarden
    - Litter has been removed
- Vegetation
    - Plant height not less than design water depth
    - Fertilised per specifications
    - No evidence of erosion
    - Is planting composition still according to approved plans
    - No placement of inappropriate plants
- Dewatering and sedimentation
    - Raingarden dewaters between storms
    - No evidence of standing water
    - No evidence of surface clogging
    - Sediments should not be > than 20% raingarden design depth
- Outlets / Overflow Spillway
    - No evidence of erosion
    - No evidence of any blockages
- Integrity of Biofilter
    - Raingarden has not been blocked or filled inappropriately
    - Mulch layer still in place
    - Noxious plants or weeds removed

Maintenance procedures

Timing	Component	Action
Following storms	Grass filter strip, kerbing, and paved area	Remove rubbish, leaves, and other debris from the grass filter strip and surrounding drainage area
	Ponding area	Clear inflow points of sediments, rubbish, and leaves Check for erosion or gouging and repair Test drainage of ponding area
	Mulch	Mulch may need to be redistributed or added around inflow points.
3 monthly	Grass filter strip, kerbing, and paved area	Mow no shorter than 50mm. Re-sow grass as necessary. Remove rubbish, leaves, and other debris. Remove excess mulch/soil if required.

	Ponding area	Clear inflow points of built-up sediment, rubbish, and leaves. Check for erosion or gouging.
	Mulch layer	Remove rubbish, leaves, and other debris. After storm events, mulch may need to be redistributed or added around inflow points.
	Plants	Water establishing plants monthly during extended dry periods. Check plant health and replace dead plants. Use native species to suit garden conditions. Remove weeds – do not use herbicides, pesticides, and fertilisers.
Annually	Ponding area	Clear inflow points of sediment, rubbish, and leaves. Check for erosion or gouging and repair. Check all water has drained 24 hours after heavy rain.
	Raingarden soil mix	Check soil level is below surrounding hard surface level and overflow grate.
	Mulch layer	Check surface of mulch for build-up of sediment, remove and replace.
	Underdrain system	Use inspection well to check underdrain is working properly.

Troubleshooting

Symptom	Possible problems	Solutions
Stormwater runoff is bypassing the raingarden	Local earthworks increasing sediment load to raingarden, blocking raingarden outlets, or raising surface level of the raingarden	Check surface of the raingarden is below the surrounding areas. Remove any sediments and debris from inflow areas and from the surface of the raingarden. Protect raingarden from future construction sediments.
	Rubbish and other debris blocking the inflow points to the raingarden	Regularly remove rubbish leaves, and any other debris from inflow points.
Raingarden is ponding for longer than 24 hours	Incorrect blend of soil mix	Replace soil mix with the correct raingarden soil mix.
Stormwater and/or mulch flowing off the raingarden	The soil within the garden compacted during construction or other activities	Loosen the top 500mm of soil by tiling or forking.
	Raingarden filled with too much mulch or soil	Remove excess mulch or soil so that surface of ponding area is approximately 200-300mm below the surrounding hard surfaces and overflow
Sulphur smell coming from the raingarden	Plants and soils lacking oxygen.	Inspect raingarden after rain event to check garden drains within 12 to 24hours.

Erosion and gouging occurring within the raingarden	Kerbs and other hard structures channelling stormwater flow.	Create openings in the kerb to increase number and width of run off points or replace kerbing with a different design.

3.2. Soakage Basin

Soakage basins have been designed to restrict surface water flows from the site to predevelopment levels by retaining surface water on site within catchment areas.

The Ashbourne Residential development includes the construction of four (4) stormwater basins which will act primarily as dry ponds and will not have a permanent water level. These basins are primarily for stormwater attenuation. The design allows for infiltration within the base of stormwater basins. Each stormwater basin will have a maintenance access track around the perimeter, with sufficient widths for an excavator and cartage trucks.

Inspection requirements

- Embankment & Emergency Spillway
  - Level of spillway
  - Vegetation and ground cover
  - Freeboard
  - No evidence of embankment erosion
  - Removal of debris on emergency spillway
- Riser & Service Spillway
  - No low flow orifice obstructed
  - No excessive sediment accumulation inside the riser
  - Function of outfall channels
  - Slope protection
  - No rip-rap failures
- Dry Pond
  - Vegetation cover
  - No presence of undesirable vegetation
  - No standing water of wet spots
  - Sediment and/or trash not accumulated
  - Low flow channels not observed

- Sediment Forebays
  - Sediment is not accumulated more than 50%
  - Provision of access of maintenance

Maintenance procedures

Timing	Component	Action
Following storms / Monthly	Inlet	Inspect and remove rubbish and debris from inlets.
	Trash racks and debris screens	Inspect and clear all litter, including leaves, rubbish, branches, and any other materials.
	Sediment forebay	Check the forebay for accumulated sediment. Test sediments for contaminants prior to dredging and dispose of sediment to landfill or similar, suitable for contaminant levels.

	Risers, control structures, grates, outlet pipes, skimmers, weirs, and orifices	Inspect control structures, weirs, orifices, outfall pipes for leaks and blockages. Clear and remove all blockages to avoid local flooding. Inspect outflow pipes for leaky joints or soil piping erosion. Check if anti-seep collars need repair or replacement. Check outfall and water discharge areas for erosion and restore and stabilise erosion.
	Emergency overflow or spillway	Check emergency overflow path remains clear of debris and blockages and remove any blockages. Check flow paths for erosion and repair as necessary.
	Erosion and bank stability	Inspect banks for settlement, erosion, scouring, cracking, sloughing, seepage and rilling.
	Water body	Remove rubbish and other floating debris from wetland pond.
	Wildlife	Remove dead animals to prevent disease spread.
	Soil	Inspect for loss of soil on wetland banks from erosion.
Annually	Valves and pumps	Check pumps and valves. Check moving parts for corrosion and lubricate.
2+ years	Wetland liners	Inspect liner for leaks and fix as per manufacturers or design specifications.
	Sediment forebay	Check the forebay for accumulated sediment. Test sediment for contaminants prior to dredging and dispose of sediment to landfill or similar suitable for contaminant levels.

Troubleshooting

Symptom	Possible problems	Solutions
Wetland water levels remain high	The outlet riser openings may be too narrow to allow fast draining after a storm	Unless water levels remain high for more than two days or flooding is a threat, action may not be necessary,
	Outlets structures are clogged	Check outlet structures and openings for blockage by debris or sediment, and clean as necessary.
Wetland is dry	Invasive plants	Remove plants by hand. (no herbicide)
	A maintenance valve is open	Check drain valves and shut if open
	Water leaking from cracks in outlet structure	Inspect for cracks and repair as necessary. Inspect for leaky joints at outlet pipes and repair.
	Wetland in area of changing groundwater levels	Pond will remain dry as long as groundwater levels are low. Design for pond should have taken this into account, so this may be normal for this wetland.

Stormwater discharging from the wetland looks dirty, muddy, or dark	High concentration of sediments washing into wetland, especially silts and clays, due to erosion or construction in the catchment area	Check catchment for erosion areas, including construction works. Check erosion controls are in place.
	Forebay full of sediment	Forebay usually needs more frequent clearing of sediment than wetland pond.
	Local works disturbing soils, with rain washing these into wetland	Check erosion and sediment controls in place on local construction sites.
Pond banks are eroding	Water flowing down pond banks is eroding soils	Minor erosion can be repaired by replacing soil and stabilising with planting or other methods.
	Stormwater outlet pipes direct flow at banks	Cause of erosion from direct discharge may be required, for example, by extending pipes down into pond. Extensive erosion due to continuing discharge may require erosion protection.
Water is leaking form the wetland and through the banks along pipes	Leak collars around pipes have failed or have not been fitted correctly.	Qualified contractors should make immediate repairs. It usually requires pond to be drained, banks excavated, leak collars repaired, and pond banks.

3.3. Temporary Swales

Temporary swales for stage 3 and 4 of Ashbourne Development will be constructed to capture surface water from rainfall events exceeding 100 year and discharge to stormwater basin B.
The temporary swales shall be inspected in line with the Waikato Stormwater Management Guideline 2020. This will include manual/mechanical prevention of undesired overgrowth from taking over the area (mowing/weeding) and manual debris and sediment removal from the outlets discharging into the temporary swales.

Inspection requirements

- Debris cleanout
  - Removal of debris
  - No dumping of wastes into swales
  - Litter has been removed
- Vegetation
  - Plant height not less than design water depth
  - Fertilised per specifications
  - No evidence of erosion
  - Grass height not greater than 250mm
  - No placement of inappropriate plants
- Dewatering
  - Swales dewater between storms
  - No evidence of standing water

Maintenance procedures

Timing	Component	Action
Following storms	Inflow points	Check for scouring, channelling, and erosion and repair as necessary.
	Side slopes	Check for scouring, channelling, and erosion and repair by adding soil and replanting as necessary.
	Channel base	Check for scouring, channelling, and erosion and repair by adding soil and replanting as necessary.
	Plants and soil	Check stormwater is filtering through soil following stormwater runoff. Remove weeds.
Monthly	Outlet	Check for scouring or erosion, and repair to suit.
	Inflow points	Remove rubbish and debris.
	Channel base	If grassed, mow channel no shorter than 150mm Re-seed bare patches of grass.
	Plants and soil	Replant gaps and water ne plants in dry conditions until established.
Two yearly	Outlet	Remove rubbish and debris from outlet grate or catchpit.
	Channel base	Check for boggy patches and ponding water. Check soil is not compacted and aerate surface or tip up dips to repair.
	Grass, plants, and soil	Remove weeds, rubbish, and debris. Re-plant gaps and re-seed bare patches, and water if required to establish. Aerate soil to prevent natural compaction. Check Stormwater is filtering through soil.

Troubleshooting

Symptom	Possible problems	Solutions
Water not draining	Soil compacted	Aerate soil with rotating aerator or core.
	Soil clogged with fine sediments	Remove top layer of soil and replace, turning soil.
	Underdrain, if present, may be blocked	Re-build underdrain.
Water flowing straight to outlet	Soil not free draining	Aerate soil, replace top layer of soil, replace soil with free draining mix.
	Swale slope is too steep	If slope is over 5%, construct check dams to slow flows.
	Plants or grass is not dense enough	Leave grass longer, and re-seed to increase density.
Scouring / Channels appearing	Inflow is concentrated at inlets	Remove blockages including rubbish, debris, and sediment build up.

4. Reporting and Scheduling

Recording and Reporting of Operation and Maintenance activities to the WRC

Recording of information and device tracking are important components of the maintenance of stormwater system devices. It is important that site operator and/or owners track maintenance by use of database. This helps inspectors to understand what devices need to be inspected, when they need to be inspected, and when was the last maintenance. Contractors nominated by MPDC will record operation and maintenance activities and report to Waikato Regional Council by use of a database. Established checklists will be used during the inspection and maintenance activities, and the activities will never rely on the memory of any one individual.

Appendix A – Auckland Council's Wetlands Operation and Maintenance Guide

WETLANDS

Operation & Maintanence Guide

STORMWATER DEVICE INFORMATION SERIES

Auckland Council
Te Kaunihera o Tāmaki Makaurau

What are constructed wetlands?

Constructed wetlands are large shallow planted ponds that filter stormwater runoff, slow flows and help control flooding downstream. Similar to natural wetlands, they look attractive and provide home and shelter to wildlife. Constructed wetlands help remove sediments, nutrients and contaminants from incoming stormwater before discharging to downstream stormwater system or waterways.

This guide offers a general description of constructed wetlands. Each constructed wetland is specifically designed to suit a particular site, so construction details will be on design and site construction plans. Correct construction levels are crucial for supplying suitable drainage for wetland plants.

How and when should maintenance be carried out?

Constructed wetlands need to be maintained in two main ways. Firstly, so they continue to work as designed (filtering stormwater, slowing flows and controlling downstream flooding) and secondly, to look attractive. A full inspection of constructed wetlands should take place a year after construction is completed.

This may be carried out by the construction contractor to coincide with the end of the defects liability period. The tables below give only typical timelines and actions for maintaining constructed wetlands. This is a general guide - each wetland should have its own detailed maintenance plan to suit the particular catchment size, pollutant loads and inflows.

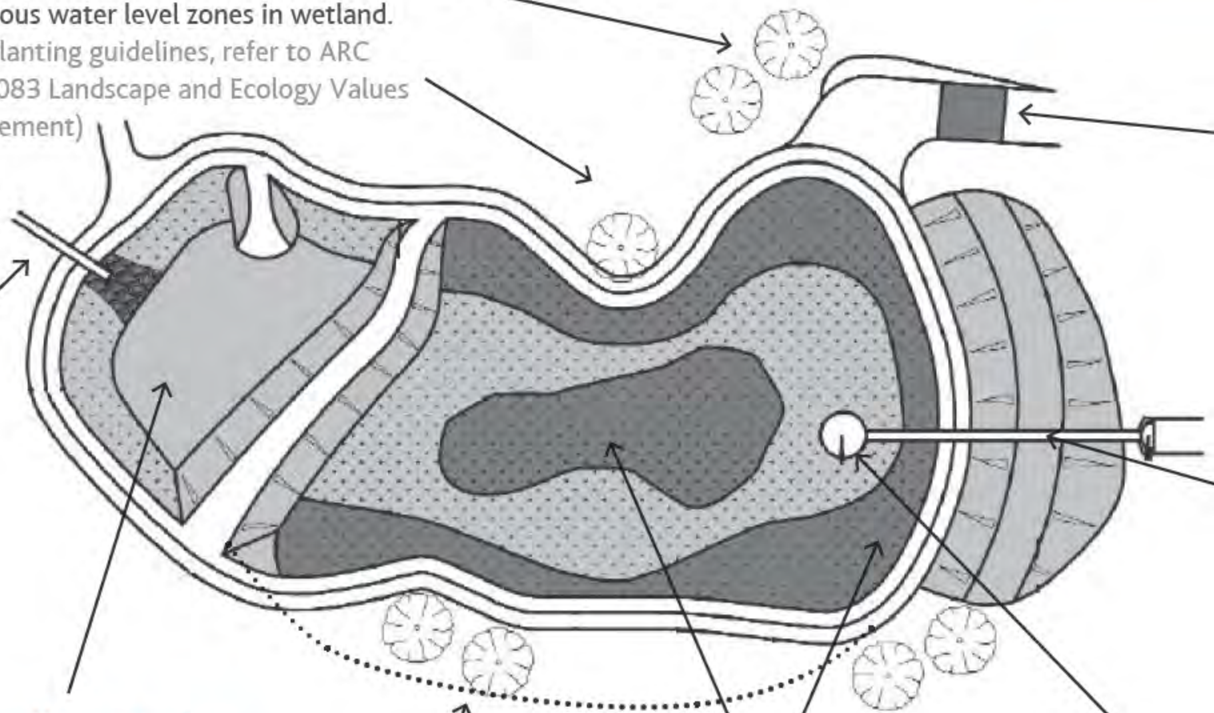
WARNING - CONTAMINATED SOIL

Constructed wetlands treat stormwater run-off, so will collect contaminants in the sediments of the pond and forebay. All material removed from these sites should be tested for contaminants before being disposed of at a suitable secure landfill.

Eight key components of a constructed wetland

5. Plants

Usually native plants, in the pond and on littoral shelf.
Species chosen to suit various water level zones in wetland.
(For suitable species and planting guidelines, refer to ARC Technical Report TR2009/083 Landscape and Ecology Values within Stormwater Management)



7. Emergency overflow

Structure to allow extreme heavy rain flows to bypass wetland and drain downstream, to prevent overtopping of wetland banks. May be in outlet riser or separate.

8. Anti-seep collars

Collars are fitted to all pipework to prevent pond leakage and potential bank collapse from erosion.

1. Inlet

Inlet pipe, receiving runoff. Erosion controls at inlet (rip rap, energy dissipaters) slow flows. Debris screens or trash racks capture rubbish.

2. Sedimentation forebay (if included)

Forebay helps slow runoff and sediment drops to the bottom. Separated from main pond by a bund or low dam.

3. Main wetland

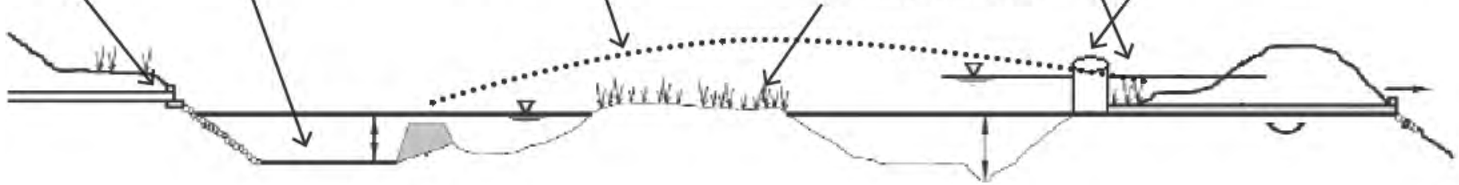
Shallow wide pond of variable depth to 1m, planted with aquatic species. Fine sediments settle to bottom and contaminants such as oil and grease break down.

4. Shallow wetland area

(Littoral shelf) Shoreline of the pond, planted with swamp species submerged at times. Plants take up nutrients (nitrogen and phosphorous) as well as slow flows and trap sediment.

6. Risers/outlets

Outlet riser pipe or weir for discharge of treated runoff. Risers may have scruffy dome to trap debris, or baffles/skimmer to prevent water life and debris from flowing downstream. Some risers have drain-down valve for maintenance.



MAINTENANCE SCHEDULE

Timing	Component	Action
Following storms	Inlet	• Inspect and remove rubbish and debris from inlets. • Check area around inlet, especially energy dissipation (rip rap) structures for erosion and cracking, and if present, repair.
	Trash racks and debris screens (if fitted)	• Inspect and clear all litter, including leaves, rubbish, branches and any other material that would block flows. Check racks for corrosion and replace if necessary.
	Sediment forebay	• Check the forebay for accumulated sediment. In general the forebay should be dredged if sediment fills over 50% of design volume. • Test sediments for contaminants (eg heavy metals, PAHs) prior to dredging and dispose of sediment to landfill or similar suitable for contaminant levels.
	Bund	• Check for erosion or instability and repair if required.
	Risers, control structures, grates, outlet pipes, skimmers, weirs and orifices	• Inspect control structures, weirs, orifices, outfall pipes for leaks and blockages. Blockage could be sediment build up, floating debris, rubbish. • Control structures could be overgrown with vegetation. • Clear and remove all blockages to avoid local flooding. Areas around control structure need to be clear of vegetation and rubbish to maintain stormwater flow. A boat may be required to access the outlet. • Inspect outflow pipes for leaky joints or soil piping erosion. • Check if anti-seep collars need repair or replacement. • Check outfall and water discharge areas for erosion and restore and stabilise erosion. • Check energy dissipaters are adequate.
	Emergency overflow or spillway	• Check emergency overflow path remains clear of debris and blockages, and remove any blockages. Check flow path for erosion and repair as necessary. Structural repairs must be repaired immediately to avoid catastrophic failure.

MAINTENANCE SCHEDULE cont...

TIMING	COMPONENT	ACTION
Following storms	Erosion and bank stability	• Inspect banks for settlement, erosion, scouring, cracking, sloughing, seepage and rilling. • Remove woody vegetation growth (unless species specifically included in pond planting plans) to avoid future root damage to banks. Removal will require bank material replacement and repair, compacted to design specification of maximum 90% dry soil density. • Inspect for pedestrian and cycle traffic or pathways on banks. • Either restrict traffic by closing paths off, or provide suitable resistant ground cover to avoid erosion from traffic.
	Water body	• Remove rubbish and other floating debris from wetland pond. • Inspect for algal blooms (usually dense water discolouration or surface scum) or fish kills – these could indicate water has extremely low levels of oxygen (eutrophication), or high nutrient loads or pollutants. • Test water quality if these problems suspected.
	Wildlife	• Control pest species so they do not threaten birds and aquatic life of the wetland. • Remove dead animals, especially water birds, to prevent disease spread. Wet areas where mosquito (mosquito larvae) could breed need careful maintenance.
	Soil	• Inspect for loss of soil on wetland banks from erosion. If plants are struggling to grow soil fertilizer may be required, but extra care must be taken to prevent fertilizer from entering wetland and local waterways.
Monthly	Inlet	• Inspect and remove rubbish and debris from inlets.
	Trash racks and debris screens (if fitted)	• Inspect and clear all litter, including leaves, rubbish, branches and any other material that would block flows. • Check racks for corrosion and replace if necessary.

MAINTENANCE SCHEDULE cont...

TIMING	COMPONENT	ACTION
Monthly	Risers, control structures, grates, outlet pipes, skimmers, weirs and orifices	• Inspect control structures, weirs, orifices, outfall pipes for leaks and blockages. Blockage could be sediment build up, floating debris, rubbish. Control structures could be overgrown with vegetation. • Clear and remove all blockages to avoid local flooding. Areas around control structure need to be clear of vegetation and rubbish to maintain stormwater flow. • Boat may be required to access outlet.
	Emergency overflow or spillway	• Check emergency overflow path remains clear of debris and blockages, and remove any blockages. • Check flow path for erosion and repair as necessary. Structural repairs must be repaired immediately to avoid catastrophic failure.
	Erosion and bank stability	• Inspect banks for settlement, erosion, scouring, cracking, sloughing, seepage and rilling. • Remove woody vegetation growth (unless species specifically included in pond planting plans) to avoid future root damage to banks. Removal will require bank material replacement and repair, compacted to design specification (of maximum 90% dry soil density). • Inspect for pedestrian and cycle traffic or pathways on banks. • Either restrict traffic by closing paths off, or provide suitable resistant ground cover to avoid erosion from traffic.
	Landscaping	• Clear wetland plants of weeds and prune and replace three-monthly. Mow split grass around pond monthly. Schedules may vary depending on seasonal growth.
	Water body	• Remove rubbish and other floating debris from wetland pond. Inspect for algal blooms (usually dense water discolouration or surface scum) or fish kills – these could indicate water has extremely low levels of oxygen (eutrophication), or high nutrient loads or pollutants. Test water quality if these problems suspected.

MAINTENANCE SCHEDULE cont...

TIMING	COMPONENT	ACTION
	Wildlife	• Control pest species so they do not threaten birds and aquatic life of the wetland. Remove dead animals, especially water birds, to prevent disease spread. • Wet areas where mosquito (mosquito larvae) could breed need careful maintenance.
	Soil	• Inspect for loss of soil on wetland banks from erosion. If plants are struggling to grow soil fertilizer may be required, but extra care must be taken to prevent fertilizer from entering wetland and local waterways.
6 Monthly	Inlet	• Check area around inlet, especially energy dissipation (rip rap) structures for erosion and cracking, and if present, repair.
	Bund	• Check for erosion or instability and repair if required.
	Risers, control structures, grates, outlet pipes, skimmers, weirs and orifices	• Inspect outflow pipes for leaky joints or soil piping erosion. • Check if anti-seep collars need repair or replacement. • Check outfall and water discharge areas for erosion and restore and stabilise erosion. Check energy dissipaters are adequate.
	Littoral zones	• Inspect wetland plants for exotic or invasive/nuisance water species and remove. • Control may be done manually, or with appropriate herbicide by properly licensed and registered professional. Follow up inspections may be needed during growing season.
Anually	Valves and pumps	• Check pumps and valves, if present, are functioning properly. • Check moving parts for corrosion and lubricate if required.
2+ Years	Wetland liner	• Inspect liner for leaks and fix as per manufacturer's or design specifications.
	Sediment forebay	• Check the forebay for accumulated sediment. In general the forebay should be dredged if sediment fills over 50% of design volume. • Test sediments for contaminants (eg heavy metals, PAHs) prior to dredging and dispose of sediment to landfill or similar suitable for contaminant levels.

TROUBLESHOOTING

SYMPTOM	POSSIBLE PROBLEMS	SOLUTION
Wetland water levels remain high	The outlet riser openings may be too narrow to allow fast draining after a storm	• Unless water levels remain high for more than two days or flooding is a threat, action may not be necessary. Refer decision to supervisor if necessary.
	Outlet structures are clogged	• Check outlet structures and openings for blockage by debris or sediment, and clean as necessary.
Wetland is dry	Invasive plants (such as raupo) clogging pond area	• Remove plants by hand – do not use herbicides.
	A maintenance valve is open.	• Check drain valves and shut if open.
	Water leaking from cracks in outlet structure.	• Inspect for cracks and repair as necessary. • Inspect for leaky joints at outlet pipes and repair.
	Wetland in area of changing groundwater levels.	• Pond will remain dry as long as groundwater levels are low. • Design for pond should have taken this into account, so this may be normal for this wetland.
	Ground water levels have dropped due to drought conditions	• Drought conditions cannot be solved, until wet season restores wetland pond levels. Use drought opportunity to clean sediments from forebay and repair stormwater infrastructure.
Stormwater discharging from the wetland looks dirty, muddy or dark	High concentration of sediments washing into wetland, especially silts and clays, due to erosion or construction in the catchment area.	• Check catchment for erosion areas, including construction works. • Check erosion controls are in place. Add or repair erosion control as required.
	Forebay full of sediment.	• Forebay usually needs more frequent clearing of sediment than wetland pond. Dredging required when forebay water storage is around 50% of total volume.
	Local works disturbing soils, with rain washing these into wetland.	• Check erosion and sediment controls in place on local construction sites. • Repair if necessary and stabilise areas of exposed soil where erosion occurring.
	Wetland outlet constructed too close to inlet, preventing treatment of water before discharge.	• Should have been designed to suit. Well placed baffles or islands in wetland may redirect and slow flows to increase treatment between inlet and outlet points.

TROUBLESHOOTING cont...

SYMPTOM	POSSIBLE PROBLEMS	SOLUTION
Wetland plants are growing over the edges and across surface of the pond	Wetland plants are growing in shallow edges of pond.	• Constructed wetlands are designed to have plants growing large fringes across pond. No action required unless plants are affecting pond function, for instance, clogging outlet structure.
Pond banks are eroding	Water flowing down pond banks is eroding soils.	• Minor erosion can be repaired by replacing soil and stabilising with planting or other methods.
	Stormwater outlet pipes direct flow at banks.	• Cause of erosion from direct discharge may be repaired, for instance, by extending pipes down into pond. • Extensive erosion due to continuing discharge may require erosion protection such as rip-rap, geotextile.
Water is leaking from the wetland and through the banks along pipes	Leak collars around pipes have failed or have not been fitted correctly (or at all). This can lead to failure of banks.	• Failure of pond banks can cause major damage at pond and downstream, so qualified construction contractors should make immediate repairs. This usually requires pond to be drained, banks excavated, leak collars repaired, and pond banks reconstructed to original design specifications.
Dead or dying birds	Botulism is a common killer of pond birds. Birds ingest toxins produced by the bacteria Clostridium botulinum, either from the water or by eating maggots or other infected food sources. Botulism can occur when water levels are low, often mid to late summer when pond water stagnates. It can also appear after algal blooms, when water oxygen levels are low.	• Remove all dead birds and animals from the area to reduce the spread of Botulism. Avoid algal blooms (see below). • Maintain flows through the ponds to avoid stagnant water. Improve shading over the water.

TROUBLESHOOTING cont...

SYMPTOM	POSSIBLE PROBLEMS	SOLUTION
Algal blooms (Yellow, green, red or blue-green coloured scum on the surface of the water.)	Algae is naturally present in waterways. Algal blooms occur in good growing conditions, including stagnant or slow moving water, high levels of nutrients, and warm and sunny weather.	• Avoid blooms by reducing nutrients entering the wetland, (for instance, controlling fertilizers from the surrounding area) and by maintaining water flows. • Although there are a number of suggested ways to deal with blooms, few are proven to work. The use of barley straw bales in the pond may work in some cases.
Animal pests present	Dense plant cover and abundant food supply in wetlands supports many animals, including pest species.	• Thin out vegetation where possible. • Set traps and poison in the area, using recommended procedures such as careful poison placement and providing warning signs.
Plants on edge of pond dying	Plants are suffering extreme wet and dry conditions.	• Choose plant varieties suitable to local conditions. • New plants need watering until established. • Replace unsuitable varieties.

Quick maintenance checks

- ✓ Check for leaks and erosion on and around banks, especially at leak collars.
- ✓ Regularly clear rubbish and dead vegetation around outlet structures, trash racks and forebay.
- ✓ Remove dead birds in case of botulism, especially in hot, humid conditions
- ✓ Keep new plants watered and control weed species.

Avoid

- ✗ Do not let erosion go unchecked. Repair, and replace erosion controls if necessary.
- ✗ Do not let forebay volume reach over half-full of sediment. Dredge and dispose of to suitable landfill.
- ✗ Prevent fertilizers, pesticides and herbicides entering the pond to avoid algal blooms and polluting downstream waterways.
- ✗ Do not ignore algal blooms and unusually dirty or dark pond water. These can affect the health of the wetland and downstream waterways.

Appendix B – Recommended Maintenance Event and Frequency Checklists

Waikato Regional Council	STORMWATER MAINTENANCE INSPECTION FORM	
		Inspector:
		Date:
		Time:
		Weather: Rainfall over previous 2-3 days?
		Page 1 of 2
		File No:

Site Name:		ID No:	
Location		Catchment:	
		Needs immediate attention	
		Not Applicable	

SWALE AND FILTER STRIP PRACTICE MAINTENANCE INSPECTION CHECKLIST	×	Required Y / N	✓	Okay	?	Clarification Required

"As builts"	Required Y / N	Available Y / N	Adequate Y / N	Approx. check to verify vol(s). Y / N
"Operation & Maintenance Plan"	Required Y / N	Available Y / N	Adequate Y / N	
"Planting Plan"	Required Y / N	Available Y / N	Adequate Y / N	

Swale And Filter Strip Components:

Items Inspected	Checked		Maintenance Needed		Inspection Frequency		Checked		Maintenance Needed		Inspection Frequency
DEBRIS CLEANOUT	Y		Y	N	**M**	**CHECK DAMS / ENERGY DISSIPATORS / SUMPS**	Y	N	Y	N	**A**
1. Swales and filter strips and contributing areas clean of debris											
2. No dumping of wastes into swales or filter strips											
3. Litter (branches, etc) have been removed											
VEGETATION					**M**						
4. Plant height not less than design water depth											
5. Fertilised per specifications											
6. No evidence of erosion											
7. Grass height not greater than 250mm											
8. Is plant composition according to design plans											
9. No placement of inappropriate plants											
DEWATERING					**M**						
10. Swales and filter strips dewater between storms											
11. No evidence of standing water											

Inspection Frequency Key **A = Annual, M = Monthly**

INSPECTOR REMARKS:

OVERALL CONDITION OF DEVICE:

In accordance with approved design plans? Y / N In accordance with As Built plans? Y / N

Maintenance required as detailed above? Y / N Compliance with other consent conditions? Y / N

Comments: ______________________________

Dates by which maintenance must be completed: / /

Dates by which outstanding information as per consent conditions is required by: / /

Inspector's signature: ______________________________

Waikato REGIONAL COUNCIL Te Kaunihera ā Rohe o Waikato	**STORMWATER MAINTENANCE INSPECTION FORM**	Inspector:
		Date:
		Time:
		Weather: Rainfall over previous 2-3 days?
		Page 1 of 2

Site Name:		File No:	
Location:		Consent No:	
		Catchment:	

RAIN GARDEN MAINTENANCE INSPECTION CHECKLIST	×	Needs immediate attention	✓	Okay	?	Clarification Required
	-	Not Applicable				

"As builts"	Required Y / N	Available Y / N	Adequate Y / N	Approx. check to verify vol(s). Y / N
"Operation & Maintenance Plan"	Required Y / N	Available Y / N	Adequate Y / N	
"Planting Plan"	Required Y / N	Available Y / N	Adequate Y / N	

Rain Garden Components:

Items Inspected	Checked		Maintenance Needed		Inspection Frequency		Checked		Maintenance Needed		Inspection Frequency
DEBRIS CLEANOUT	Y	N	Y	N	**M**	**OUTLETS/OVERFLOW SPILLWAY**	Y	N	Y	N	**A, AMS**
1. Rain gardens and contributing areas clean of debris						13. Good condition, no need for repair					
2. No dumping of yard wastes into rain garden						14. No evidence of erosion					
3. Litter (branches, etc) have been removed						15. No evidence of any blockages					
VEGETATION					**3M**	**INTEGRITY OF BIOFILTER**					**A**
4. Planting height not less than design water depth						16. Rain garden has not been blocked or filled inappropriately					
5. Fertilised per specifications						17. Mulch layer still in place					
6. No evidence of erosion						18. Noxious plants or weeds removed					
7. Is plant composition still according to approved plans											
8. No placement of inappropriate plants											
DEWATERING AND SEDIMENTATION											
9. Rain garden dewaters between storms					**3M**						
10. No evidence of standing water											
11. No evidence of surface clogging											
12. Sediments should not be > than 20% of rain garden design depth											

Inspection Frequency Key **A = Annual, M = Monthly, AMS = After Major Storm**

INSPECTOR REMARKS:

OVERALL CONDITION OF DEVICE:

In accordance with approved design plans? Y / N In accordance with As Built plans? Y / N

Maintenance required as detailed above? Y / N Compliance with other consent conditions? Y / N

Comments: ____

Dates by which maintenance must be completed: / /

Dates by which outstanding information as per consent conditions is required by: / /

Inspector's signature: ____________________

Waikato REGIONAL COUNCIL Te Kaunihera ā Rohe o Waikato	**STORMWATER MAINTENANCE INSPECTION FORM**	Inspector: Date: Time: Weather: Rainfall over previous 2-3 days? Page 1 of 2

Site Name:		File No:	
Location		Consent No:	
		Catchment:	

INFILTRATION TRENCH MAINTENANCE INSPECTION CHECKLIST — × Needs immediate attention; – Not Applicable; ✓ Okay; ? Clarification Required

"As builts"	Required Y / N	Available Y / N	Adequate Y / N	Approx. check to verify vol(s). Y / N
"Operation & Maintenance Plan"	Required Y / N	Available Y / N	Adequate Y / N	
"Planting Plan"	Required Y / N	Available Y / N	Adequate Y / N	

Infiltration Trench Components:

Items Inspected	Checked		Maintenance Needed		Inspection Frequency		Checked		Maintenance Needed		Inspection Frequency
DEBRIS CLEANOUT	Y	N	Y	N	**M**	**INLETS**	Y	N	Y	N	**A**
1. Trench surface clear of debris						13. Good condition					
2. Inlet areas clear of debris						14. No evidence of erosion					
3. Inflow pipes clear of debris						**OUTLETS/OVERFLOW SPILLWAY**					**A**
4. Overflow spillway clear of debris						15. Good condition, no need for repair					
SEDIMENT TRAPS, FOREBAYS, OR PRETREATMENT SWALES					**A**	16. No evidence of erosion					
5. Obviously trapping sediment						**AGGREGATE REPAIRS**					**A**
6. Greater than 50% of storage volume remaining						17. Surface of aggregate clean					
VEGETATION					**M**	18. Top layer of stone does not need replacement					
7. Mowing done when needed						19. Trench does not need rehabilitation					
8. Fertilized per specifications						**VEGETATED SURFACE**					**M**
9. No evidence of erosion						20. No evidence of erosion					
DEWATERING					**3M**	21. Perforated inlet functioning adequately					
10. Trench dewaters between storms						22. Water does not stand on vegetative surface					
SEDIMENT CLEANOUT OF TRENCH					**A**	23. Good vegetative cover exists					
11. No evidence of sedimentation in trench											
12. Sediment accumulation does not yet require cleanout											

Inspection Frequency Key A = Annual, M = Monthly

INSPECTORS REMARKS:

OVERALL CONDITION OF DEVICE:

In accordance with approved design plans? Y / N In accordance with As Built plans? Y / N

Maintenance required as detailed above? Y / N Compliance with other consent conditions? Y / N

Comments: ____

Dates by which maintenance must be completed: / /

Dates by which outstanding information as per consent conditions is required by: / /

Inspector's signature: ____________________

Waikato REGIONAL COUNCIL Te Kaunihera ā Rohe o Waikato	**STORMWATER MAINTENANCE INSPECTION FORM**	Inspector:
		Date:
		Time:
		Weather: Rainfall over previous 2-3 days?
		Page 1 of 2

Site Name:		File No:	
Location:		Consent No:	
		Catchment:	

STORMWATER POND/WETLAND MAINTENANCE INSPECTION CHECKLIST	×	Needs immediate attention	✓	Okay	?	Clarification Required
	–	Not Applicable				

"As builts"	Required Y / N	Available Y / N	Adequate Y / N	Approx. check to verify vol(s). Y / N
"Operation & Maintenance Plan"	Required Y / N	Available Y / N	Adequate Y / N	
"Planting Plan"	Required Y / N	Available Y / N	Adequate Y / N	

Pond/Wetland Components:

Items Inspected	Checked		Maintenance Needed		Inspection Frequency		Checked		Maintenance Needed		Inspection Frequency
EMBANKMENT & EMERGENCY SPILLWAY	Y	N	Y	N	**A,S**		Y	N	Y	N	
1. Is the spillway level?						20. Concrete/Masonry condition					
2. Adequate vegetation & ground cover?						Riser and barrels					
3. Appropriate plants / weeds?						a) Cracks or displacement?					
4. Adequate freeboard?						b) Minor spalling (0.25mm)?					
5. Embankment erosion evident?						c) Major spalling (rebars exposed)?					
6. Cracking, bulging or sliding of dam						d) Joint failures?					
a) Upstream embankment						e) Water tightness adequate?					
b) Downstream embankment						21. Pond drain valve:					
c) At or beyond toe upstream						a) Operational / exercised?					
d) At or beyond toe downstream						b) Chained and locked?					
e) Emergency spillway						22. Slope protection or rip-rap failures?					
7. Pond & toe drains clear & functioning?						23. Other?					
8. Evidence of animal burrows?						**PERMANENT POOL (WET POND)**					**3M**
9. Seeps/leaks on downstream face?						24. Undesirable vegetative growth?					
10. Vertical & horizontal alignment of top of dam as per As-Built plans?						25. Removal of floating debris required?					
						26. Visible pollution?					
11. Emergency spillway clear of obstructions & debris						27. Evidence of 'edge' erosion?					
						28. Other?					
12. Provision of access for maintenance?						**DRY POND**					**3M**
a) By hand?						29. Adequate vegetation cover?					
b) For machinery?						30. Presence of undesirable vegetation / woody growth?					
13. Other?											
RISER & SERVICE SPILLWAY					**A**	31. Standing water or wet spots?					
Type: Reinforced concrete						32. Sediment and/or trash accumulation?					
Metal pipe						33. Low flow channels unobstructed?					
Masonry						34. Other?					
14. Low flow orifice obstructed?						**SEDIMENT FOREBAYS**					
15. Low flow trash rack						35. Is sediment accumulation > 50% (maintenance req'd immed. If Yes)					
a) Is debris removal necessary?											
b) Is corrosion evident?						36. Provision of access for maintenance:					
16. Weir trash rack maintenance						a) By hand?					
a) Is debris removal required?						b) For machinery?					
b) Is corrosion evident?						**OUTFALLS INTO PONDS**					**A,S**
17. Is there excessive sediment accumulation inside the riser?						37. Riprap failures?					
						38. Condition of endwalls / headwalls	Good		Fair		Poor
18. Metal pipe condition	Good		Fair		Poor	39. Evidence of slope erosion?					
19. Outfall channels functioning?						40. Condition of any inflow pipes	Good		Fair		Poor
						41. Other?					

Items Inspected	Checked		Maintenance Needed		Inspection Frequency		Checked		Maintenance Needed		Inspection Frequency
OTHER	Y	N	Y	N	**6M**	**CONSTRUCTED WETLAND AREAS**	Y	N	Y	N	**A**

1

41. Encroachments on pond or easement area?						45. Vegetation healthy and growing?					
42. Complaints from residents?						46. Evidence of invasive species?					
43. Aesthetics						47. Excessive sedimentation in wetland area?					
a) grass mowing required?											
b) graffiti removal needed?											
c) other (specify)?											
44. Any public hazards (specify)?											

Inspection Frequency Key **A = Annual, M = Monthly, S = after monthly storm**

INSPECTOR REMARKS:

OVERALL CONDITION OF DEVICE:

In accordance with approved design plans? Y / N In accordance with As Built plans? Y / N

Maintenance required as detailed above? Y / N Compliance with other consent conditions? Y / N

Comments: ____________________

Dates by which maintenance must be completed: / /

Dates by which outstanding information as per consent conditions is required by: / /

Inspector's signature: ____________________

Appendix C – Trouble Shooting Guide

Timing	Component	Action
		• Remove weeds – do not use herbicides, pesticides and fertilisers as these chemicals will pollute the stormwater runoff.
Annually	Ponding area	• Clear inflow points of sediment, rubbish and leaves. • Check for erosion or gouging and repair. • Check all water has drained 24 hours after heavy rain. • Alternatively test drainage of ponding area. Dig a hole 200mm wide x 200mm deep. Pour in 10 litres of water in hole. Check drainage rate over 1 hour period – minimum 25mm/hour. • If crust of fine sediment present on surface of soil mix, remove with spade and rework using rake. Top up soil and mulch as necessary (ensuring level is below surrounding hard surface and overflow). Dispose of contaminated crusted topsoil in a secure landfill (unless soil testing shows no contamination).
	Rain garden soil mix	• Check soil level is below surrounding hard surface level and overflow grate. Use drainage test described above to check soil is free draining. •
	Mulch layer (bark, pebbles, etc.)	• Check surface of mulch for build-up of sediment, remove and replace as required.
	Underdrain system	• Use inspection well (if present) to check underdrain is working properly. • Check rain garden draining freely using the drainage test described above. If rain garden is not free-draining, the underdrain may be blocked. Try back-washing under drain from the outlet. If still blocked, the rain garden may need plants and rain garden soil mix removed and replaced.

Table 18-5: Troubleshooting for bioretention devices[175]

Symptom	Possible problems	Solution
Stormwater runoff is bypassing the rain garden	Local earthworks increasing sediment load to rain garden, blocking rain garden outlets or raising surface level of the rain garden	• Check surface of the rain garden is below the surrounding areas. • Remove any sediments and debris from inflow areas and from the surface of the rain garden. • Protect rain garden from future construction sediments.
	Rubbish and other debris blocking the inflow points to the rain garden	• Regularly remove rubbish, leaves and any other debris from inflow points.
Rain garden is ponding for longer than 24 hours	Incorrect blend of soil mix	• Replace soil mix with the correct rain garden soil mix. Do Ribbon test or Percolation test to test soil mix is free-draining.

[175] Auckland Council Rain Garden Operation and Maintenance Guide

Symptom	Possible problems	Solution
Stormwater and/or mulch flowing off the rain garden	The soil within the garden compacted during construction or other activities.	• Loosen the top 500mm of soil by tilling or forking. Discourage vehicle, pedestrian and bicycle access to the rain garden.
	Layer of fine sediment settled on the garden surface	• Remove fine sediment layer and turn over the top layer of rain garden soil mix. Protect rain garden from surrounding sediment run off.
	Rain garden filled with too much mulch or soil	• Remove excess mulch or soil so that surface of ponding area is approximately 200-300mm below the surrounding hard surfaces and overflow.
	Overflows or discharge pipes clogged with sediments or debris	• Clear overflow and discharge pipes.
	Planting or rain garden soil mix clogged	• It may be necessary to remove some of the rain garden soil mix and replace with fresh rain garden soil mix.
Sulphur smell coming from the rain garden	Plants and soils lacking oxygen (anaerobic conditions). Organic material rotting within the garden The underdrain clogged and water is not properly draining out of the garden	• Inspect rain garden after rain event to check garden drains within 12 – 24 hours (see solutions above for rain garden ponding)
Erosion and gouging occurring within the rain garden	Kerbs and other hard structures channelling stormwater flow (rain gardens require an event sheet of flow of water to operate effectively)	• Create openings in the kerb to increase number and width of run off points, or replace kerbing with a different design (eg. kerbing slightly raised off the ground)
	Inflow points are too concentrated	• Increase kerb opening size by cutting kerbs or replacing with different design. If this is not possible install rip-rap (i.e. stones set into concrete) at the inflow point to spread flow and reduce erosion.
Plants are stressed or dying. Symptoms may include yellowing of leaves, unseasonal leaf fall, wilting.	Plant varieties selected for rain garden are unsuitable for the location and/or extreme wet/dry conditions.	• Select plants appropriate for the location (eg. full shade, partial shade, full sun, etc.) • Due to their hardy nature, native plants are recommended.
	Ponding or excessively long periods of flooding cause plants to become stressed or die.	• Inspect rain garden after rain event to check garden drains within 12 – 24 hour. If not, see above solutions for rain garden ponding.
	The plants poisoned by runoff from a hazards spill (fuel, paint, oil, etc).	• Check soil and mulch for evidence of heavily polluted runoff (eg. rainbow slick, coloured mulch, etc.)
	Pollutants accumulated in the rain garden reached a toxic level for plants.	• If contamination is extensive, clean out raingarden soil mix and replace fresh soil and new plants.
	The plants dehydrated from extended dry conditions	• Newly established plants need watering. • Check soil moisture content and water plants if dry.

Symptom	Possible problems	Solution
		• Establishing plants need watering in dry weather.
	Plants stressed due to attack by plant pests or diseases. Pests may include insects of animals.	• Check for leaf damage or pests and consult gardening manuals or a garden centre for the best treatment. • Stressed plants need replacing with healthy variety or pest-resistance species.
	Rain garden soil mix compacted	• Loosen the top 500mm of soil by tilling or forking. Do not allow vehicle, pedestrian and bicycle access to the rain garden.

18.2.1.4 Infiltration devices

Infiltration devices are very sensitive to impaired performance if excessive amounts of sediments or oils and greases are introduced into them. The greatest problem is clogging of soils in the sides and bottom or in the case of permeable paving surface clogging. This can occur fairly rapidly if inflow sediment loads are not reduced by pre-treatment devices.

Other contaminants, which are attached to sediments, are not considered a clogging concern.

Another problem is poor drainage as a result of high water table, groundwater mounding or a confining soil layer. Prolonged wetness encourages micro-organism growths that tend to clog soils.

18.2.1.5 Ponds and wetlands

[redacted]

One of the greatest benefits of stormwater management ponds and wetlands is their resilient performance even when excessive contaminant loads enter them. However, performance will suffer if sediment is introduced in large amounts over a lengthy time frame. Sediments reduce the volume of storage and reduce extended detention times, which ultimately reduce the pond or wetland's contaminant reduction potential.

This impaired function is not something that tends to occur dramatically in a short time period but rather occurs cumulatively over a longer time period if the incoming sediment load is consistently elevated.

Another problem that ponds and wetlands have that other devices do not have to such an extent is maintenance problems associated with debris clogging inlets and outlet areas. While other devices can have visual issues related to debris, pond outlets can become blocked, especially the extended detention orifices. Clogging of these outfall orifices can cause significant adverse effects by elevating water in the pond or wetland and potentially killing the vegetation, increasing safety concerns and increasing the zone of saturation in the pond or wetland embankment.

A recommended maintenance schedule for wetlands is provided below:

Timing	Component	Action
	Bund	• Check for erosion or instability and repair if required.
	Risers, control structures, grates, outlet pipes, skimmers, weirs and orifices	• Inspect outflow pipes for leaky joints or soil piping erosion. • Check if anti-seep collars need repair or replacement. • Check outfall and water discharge areas for erosion and restore and stabilise erosion. Check energy dissipaters are adequate.
	Littoral zones	• Inspect wetland plants for exotic or invasive/nuisance water species and remove. • Control may be done manually, or with appropriate herbicide by properly licensed and registered professionals. Follow up inspections may be needed during growing season.
Annually	Valves and pumps	• Check pumps and valves, if present, are functioning properly. • Check moving parts for corrosion and lubricate if required.
2+ years	Wetland liners	• Inspect liner for leaks and fix as per manufacturer's or design specifications.
	Sediment forebay	• Check the forebay for accumulated sediment. In general the forebay should be dredged if sediment fills over 50% of design volume. • Test sediment for contaminants (eg. heavy metals, PAHs) prior to dredging and dispose of sediment to landfill or similar suitable for contaminant levels.

Table 18-7: Trouble shooting for wetland[177]

Symptom	Possible problems	Solution
Wetland water levels remain high	The outlet riser openings may be too narrow to allow fast draining after a storm	• Unless water levels remain high for more than two days or flooding is a threat, action may not be necessary. Refer decision to supervisor if necessary.
	Outlet structures are clogged	• Check outlet structures and openings for blockage by debris or sediment, and clean as necessary.
Wetland is dry	Invasive plants (such as raupo) clogging pond area	• Remove plants by hand, do not use herbicide.
	A maintenance valve is open	• Check drain valves and shut if open
	Water leaking from cracks in outlet structure	• Inspect for cracks and repair as necessary • Inspect for leaky joints at outlet pipes and repair

[177] Auckland Council Wetlands Operation and Maintenance Guide

Symptom	Possible problems	Solution
	Wetland in area of changing groundwater levels	• Pond will remain dry as long as groundwater levels are low. • Design for pond should have taken this into account, so this may be normal for this wetland.
	Groundwater levels have dropped due to drought conditions	• Drought conditions cannot be solved, until wet season restores wetland pond levels. Use drought opportunity to clean sediments from forebay and repair stormwater infrastructure.
Stormwater discharging from the wetland looks dirty, muddy or dark	High concentration of sediments washing into wetland, especially silts and clays, due to erosion or construction in the catchment area	• Check catchment for erosion areas, including construction works. • Check erosion controls are in place. Add or repair erosion control as required
	Forebay full of sediment	• Forebay usually needs more frequent clearing of sediment than wetland pond. Dredging required when forebay water storage is around 50% of total volume.
	Local works disturbing soils, with rain washing these into wetland	• Check erosion and sediment controls in place on local construction sites • Repair if necessary and stabilise areas of exposed soil where erosion occurring
	Wetland outlet constructed too close to inlet, preventing treatment of water before discharge	• Should have been designed to suit. Well placed baffles or islands in wetland may redirect and slow flows to increase treatment between inlet and outlet points.
Wetland plants are growing over the edges and across surface of the pond	Wetland plants are growing in shallow edges of pond	• Constructed wetlands are designed to have plants growing large fringes across pond. No action required unless plants are affecting pond function, for instance, clogging outlet structure.
Pond banks are eroding	Water flowing down pond banks is eroding soils	• Minor erosion can be repaired by replacing soil and stabilising with planting or other methods
	Stormwater outlet pipes direct flow at banks	• Cause of erosion from direct discharge may be repaired, for instance, by extending pipes down into pond. • Extensive erosion due to continuing discharge may require erosion protection such as rip-rap, geotextile.
Water is leaking from the wetland and through the banks along pipes	Leak collars around pipes have failed or have not been fitted correctly (or at all). This can lead to failure of banks.	• Failure of pond banks can cause major damage at pond and downstream, so qualified construction contractors should make immediate repairs. This usually requires pond to be drained, banks excavated, leak collars repaired, and pond banks

Symptom	Possible problems	Solution
		reconstructed to original design specifications.
Dead or dying birds	Botulism is a common killer of pond birds. Birds ingest toxins produced by the bacteria Clostridium botulinum, either from the water or by eating maggots or other infected food sources. Botulism can occur when water levels are low, often mid to late summer when pond water stagnates. It can also appear after algal blooms, when water oxygen levels are low.	• Remove all dead birds and animals from the area to reduce the spread of Botulism. Avoid algal blooms (see below). • Maintain flows through the ponds to avoid stagnant water. Improve shading over the water.
Algal blooms (yellow, green, red or blue-green coloured scum on the surface of the water)	Algae is naturally present in waterways. Algal blooms occur in good growing conditions, including stagnant or slow moving water, high levels of nutrients, and warm and sunny weather	• Avoid blooms by reducing nutrients entering the wetland, (for instance, controlling fertilizers from the surrounding area) and by maintaining water flows. • Although there are a number of suggested ways to deal with blooms, few are proven to work. The use of barley straw bales in the pond may work in some cases.
Animal pests present	Dense plant cover and abundant food supply in wetlands supports many animals, including pest species	• Thin out vegetation where possible. • Set traps and poison in the area, using recommended procedures such as careful poison placement and providing warning signs.
Plants on edge of pond dying	Plants are suffering extreme wet and dry conditions.	• Choose plant varieties suitable to local conditions. • New plants need watering until established. • Replace unsuitable varieties.

18.2.1.6 Green roofs

Principal reasons why this device performance can deteriorate are the following:

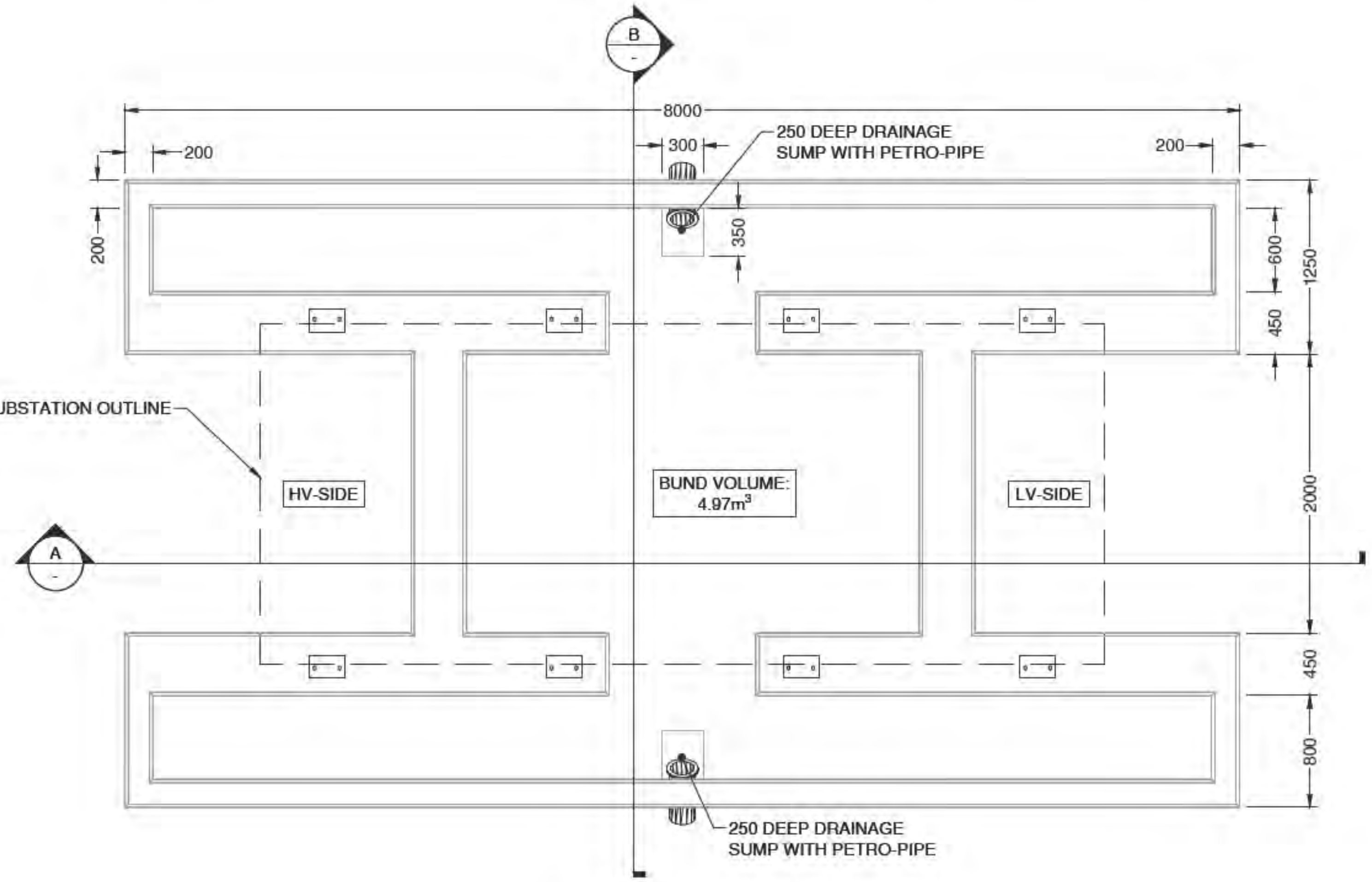
- Impermeable membrane failure due to leakage, puncture or UV deterioration
- Excessive weed growth outcompeting planted growth
- Ponding of water on flat roofs
- Concentration of flows across the green roof causing scour and discharge at locations not designed for
- Clogging of substrate, and
- Plugged outlets.

18.2.1.7 Water tanks

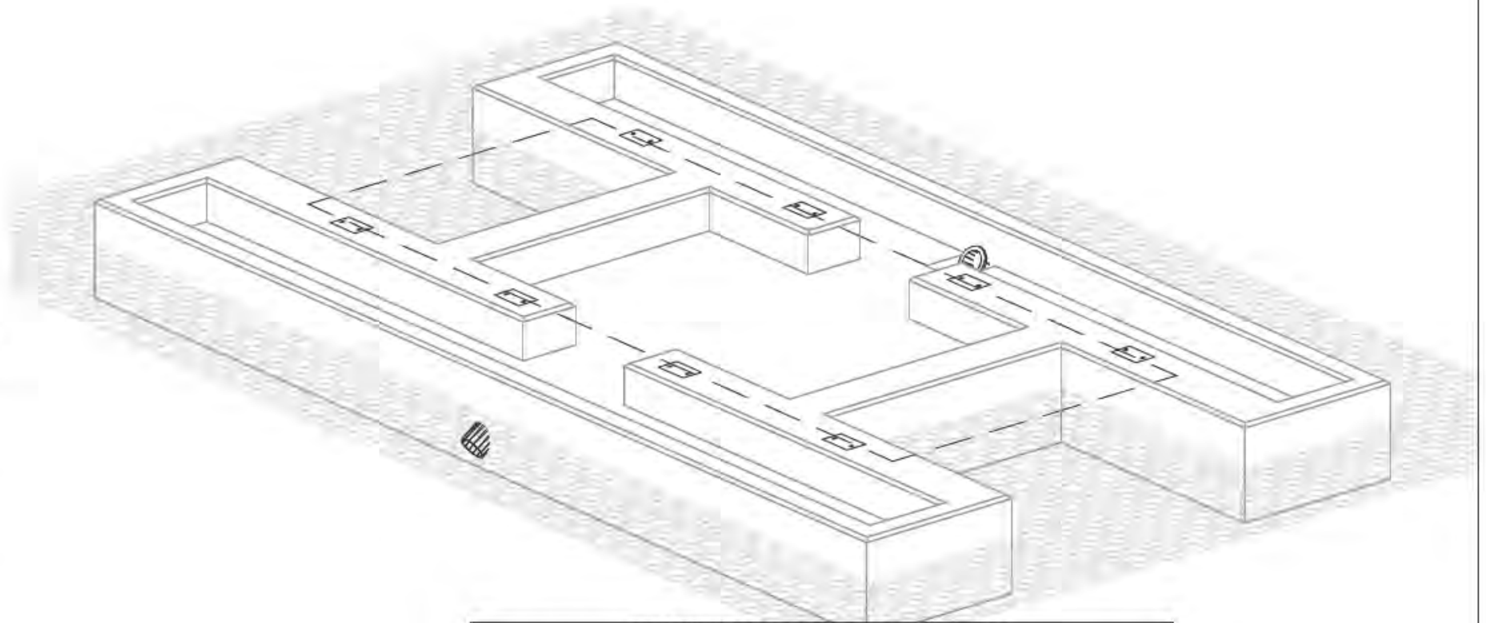
Water tank function can be compromised mainly due to two reasons:

1. Inadequate water supply where demand exceeds supply, and
2. The tank outlets or downspouts become clogged due to excessive vegetative entry into the tank from roof spouting.

Appendix C – Lightyears Solar Drawings

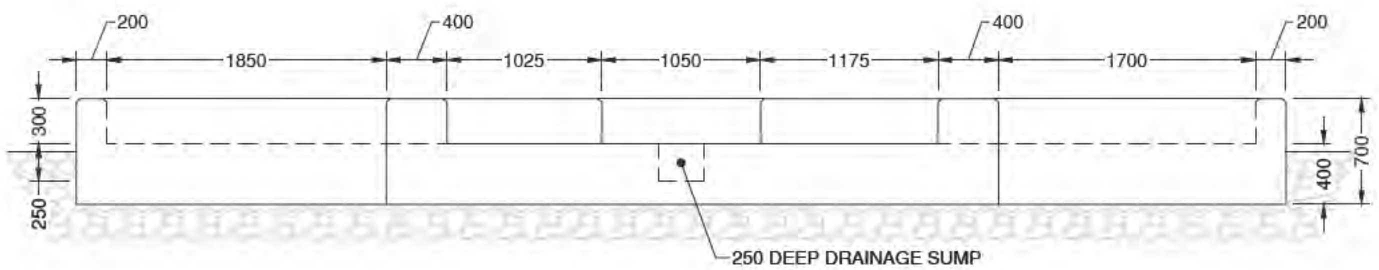


MV SUBSTATION FOUNDATION - PLAN VIEW
Scale 1:50

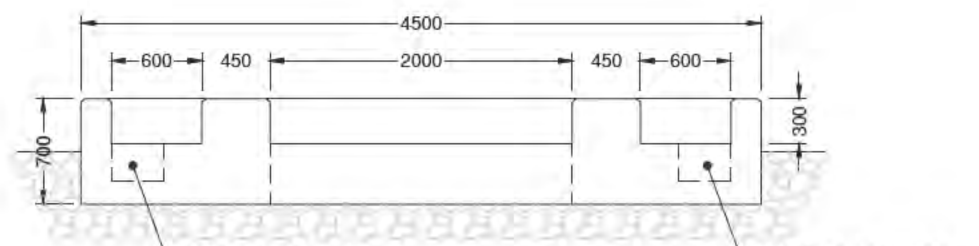


TRANSFORMER OIL WEIGHT <2.9t (3600L).
BUND TO BE FREE-DRAINING VIA SPI PETROPIPE - NO SITTING WATER.
BUND VOLUME 4.97m^3 / >138% OF TRANSFORMER OIL VOLUME.

MV SUBSTATION FOUNDATION - PERSPECTIVE
Scale NTS



A FRONT ELEVATION - SECTION A
Scale 1:50



B SIDE ELEVATION - SECTION B
Scale 1:50

PLOTTED: 8/10/2024 8:38 am

MASF-121.dwg

REV.	DESCRIPTION	DRAWN	CHECKED	APPROVED	DATE
A	FOR CONSENT	DM	GJ	MS	07.10.2024

CLIENT:

UNITS: | SHEET SIZE: A3 | SCALE: 1:50 (U.N.O.) | PROJECTION:

ADDITIONAL INFORMATION:

FOR CONSENT

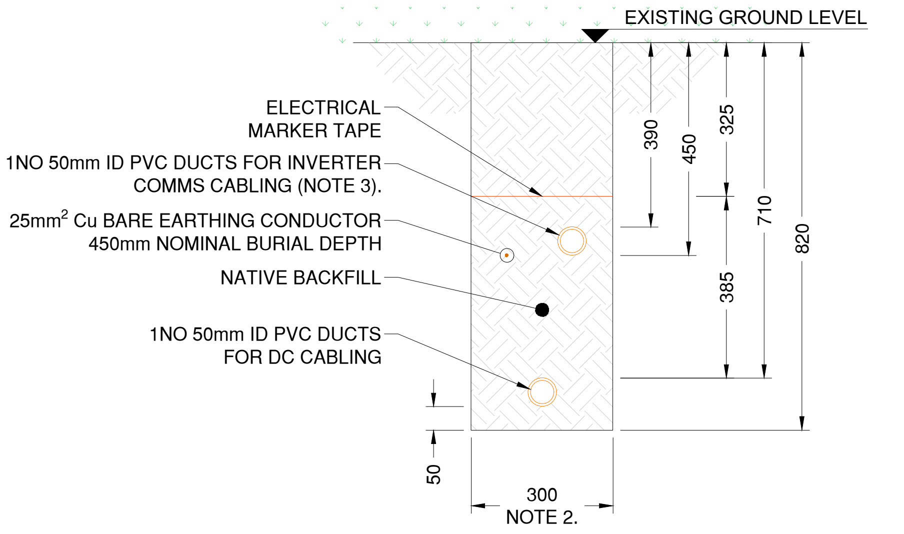
LIGHTYEARS SOLAR
AUCKLAND, NEW ZEALAND
www.lightyearssolar.co.nz

DESIGNED:	DM
DRAWN:	DM
DATE:	07/10/2024
CHECKED:	GJ
APPROVED:	MS

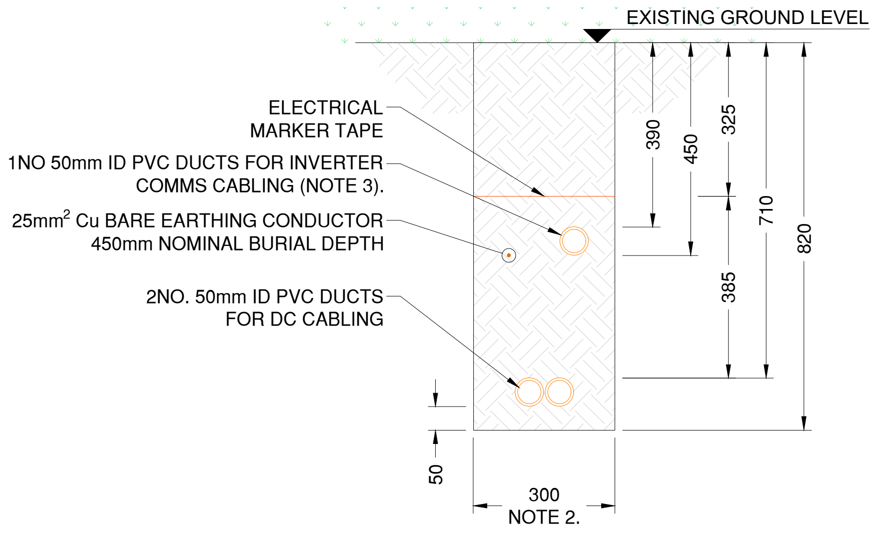
TITLE: MATAMATA SOLAR FARM
MV SUBSTATION FOUNDATION
GENERAL ARRANGEMENT

DRG. NO.: MASF-121

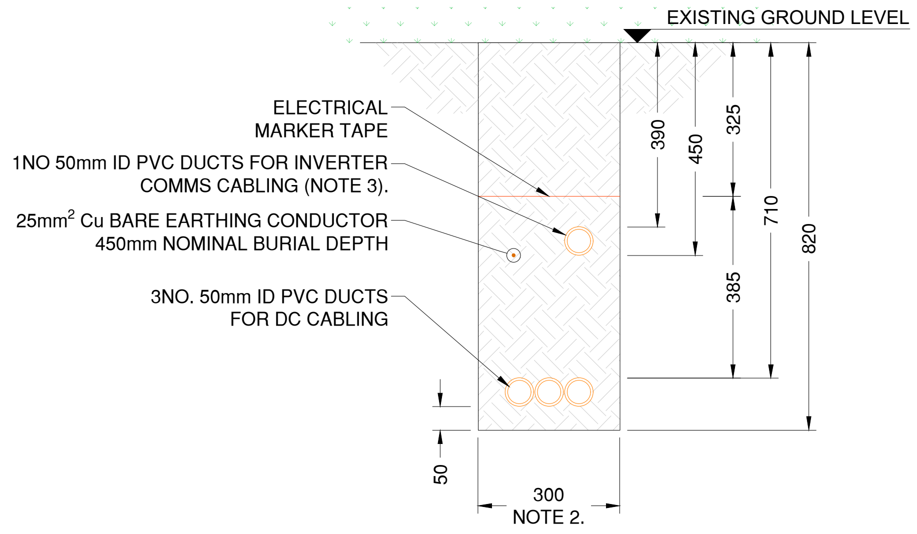
REVISION A



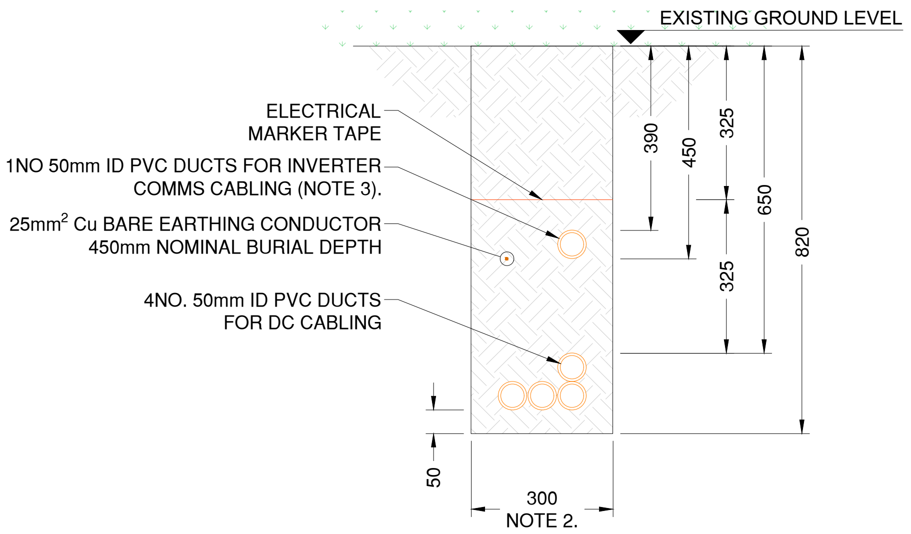
A | 1 DUCT DC TRENCH
- | Scale 1:16



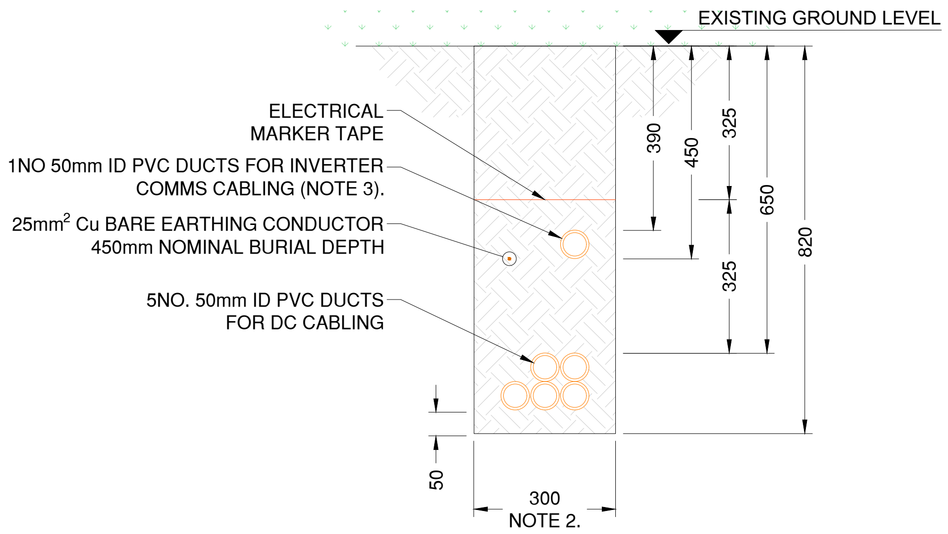
B | 2 DUCT DC TRENCH
- | Scale 1:16



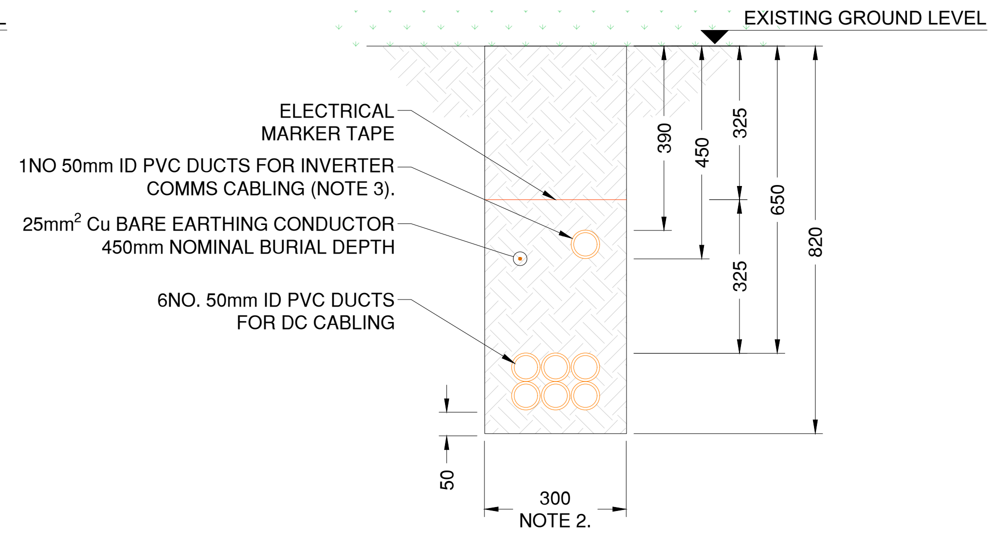
C | 3 DUCT DC TRENCH
- | Scale 1:16



D | 4 DUCT DC TRENCH
- | Scale 1:16



E | 5 DUCT DC TRENCH
- | Scale 1:16



F | 6 DUCT DC TRENCH
- | Scale 1:16

NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETRES UNO.
2. NO MINIMUM TRENCH WIDTH REQUIREMENT. THE CONTRACTOR MAY EXCAVATE TO NEAREST BUCKET WIDTH AVAILABLE, THAT ACCOMODATES THE WIDTH OF DUCTS.
3. DUCTS FOR INVERTER COMMS TO GO VIA ADJACENT INVERTERS (LOOP-IN / LOOP-OUT) TO THE RESPECTIVE FIELD CABINET.
4. WHERE LARGE BOULDERS ARE PRESENT IN THE NATIVE SOIL, COVER DUCTS & CABLES WITH MIN.100mm SAND OR FINES TO PREVENT DAMAGE TO DUCTS DURING BACKFILLING AND COMPACTION.

PLOTTED: 8/10/2024 8:38 am
MASF-212.dwg

REV.	DESCRIPTION	DRAWN	CHECKED	APPROVED	DATE
A	FOR CONSENT	DM	GJ	MS	07.10.2024

CLIENT:

UNITS:mm | SHEET SIZE: A3 | SCALE: NTS (U.N.O.) | PROJECTION:

ADDITIONAL INFORMATION:

FOR CONSENT

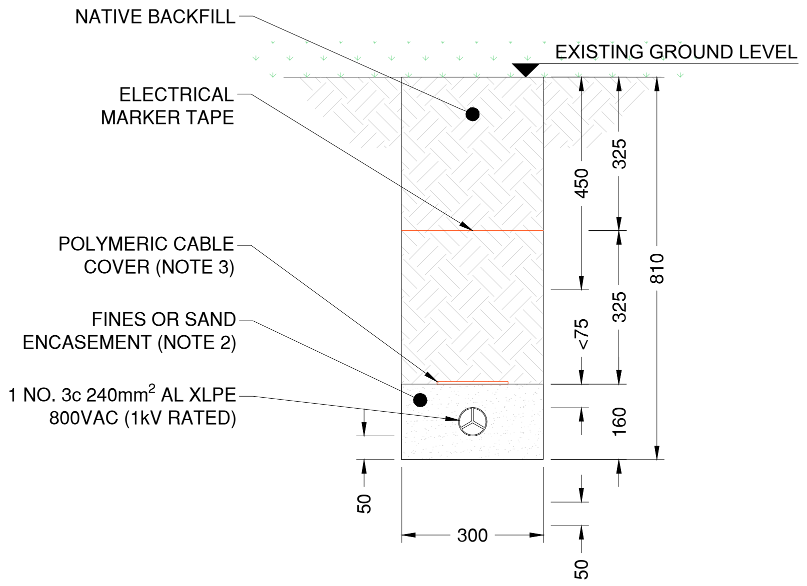
LIGHTYEARS SOLAR
AUCKLAND, NEW ZEALAND
www.lightyearssolar.co.nz

DESIGNED:	DM
DRAWN:	DM
DATE:	07.10.2024
CHECKED:	GJ
APPROVED:	MS

TITLE: MATAMATA SOLAR FARM
1500V DC
TRENCH PROFILES

DRG. NO.: MASF-212

REVISION A



A | 1 CABLE AC TRENCH
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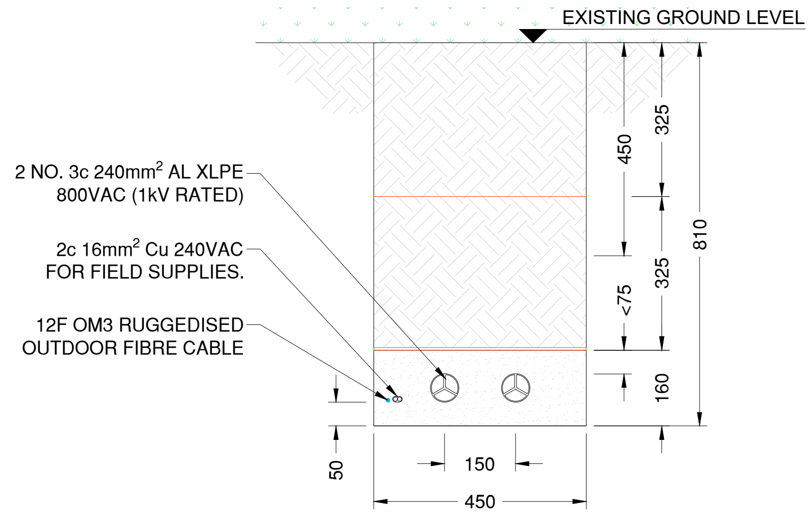
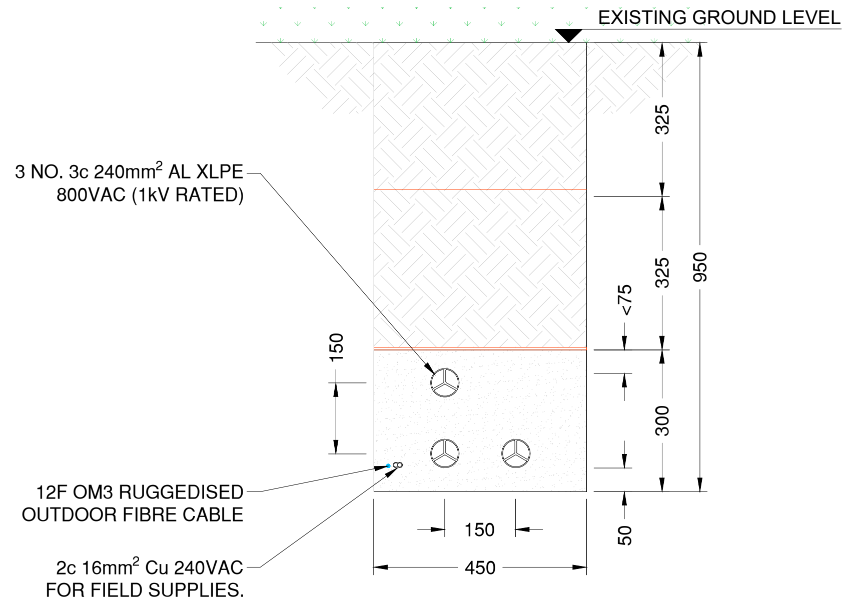
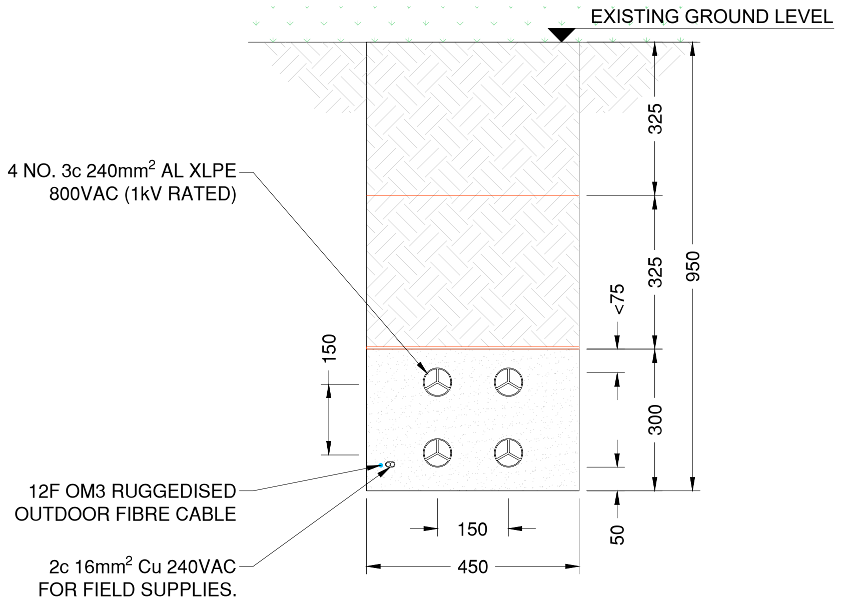


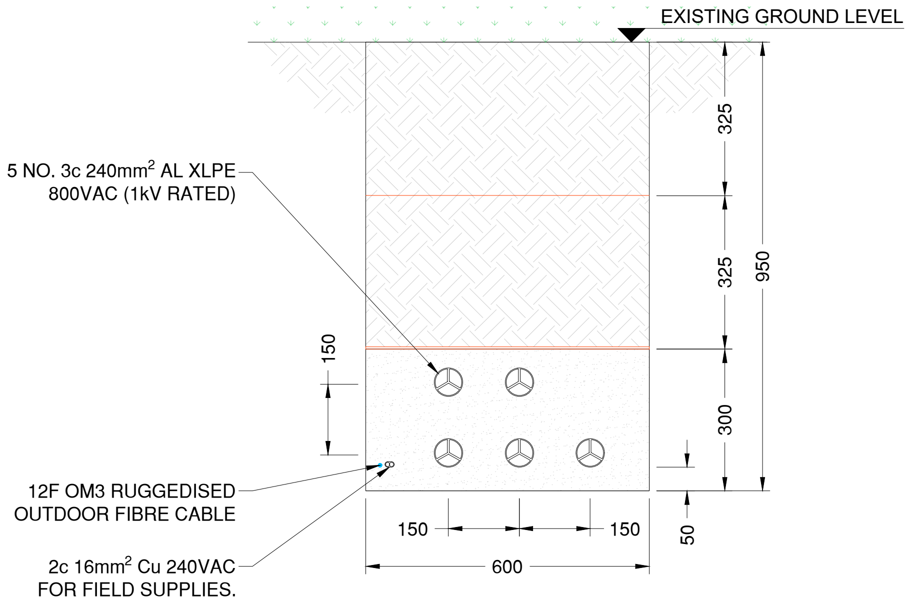
B | 2 CABLE AC TRENCH
- | Scale 1:16



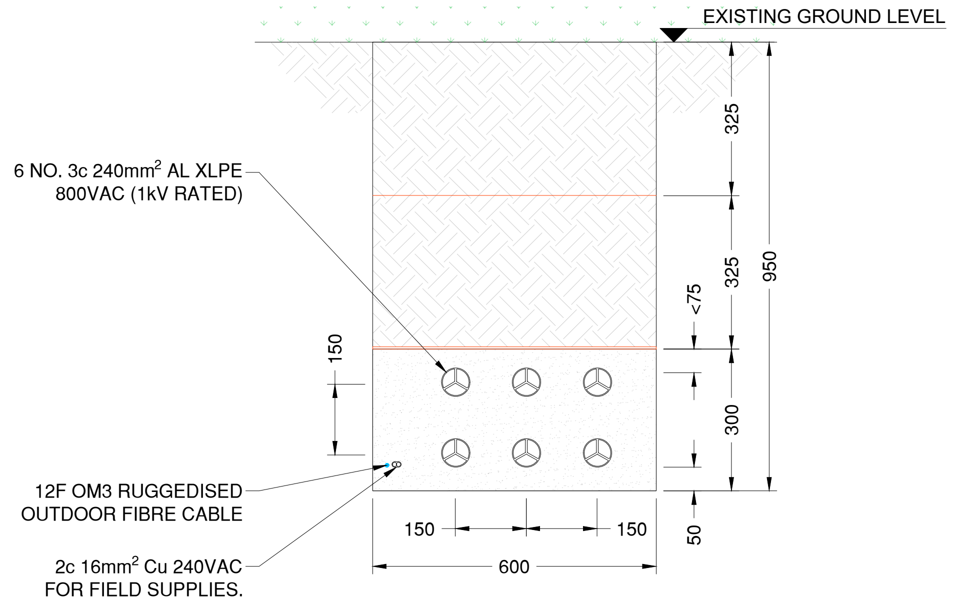
C | 3 CABLE AC TRENCH
- | Scale 1:16



D | 4 CABLE AC TRENCH
- | Scale 1:16



E | 5 CABLE AC TRENCH
- | Scale 1:16



F | 6 CABLE AC TRENCH
- | Scale 1:16

NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETRES UNO.
2. 800V AC CABLING TO BE ENCASED IN SAND OR FINES, MIN. 50mm TOP AND BOTTOM PRIOR TO BACKFILL WITH NATIVE SOIL.
3. AS/NZS 4702 POLYMERIC CABLE COVER OR MAGSLAB. TO EXTEND MINIMUM 40mm HORIZONTALLY BEYOND THE CABLE OR DUCT INSTALLED BELOW.
4. WHERE LARGE BOULDERS ARE PRESENT IN THE NATIVE SOIL, COVER DUCTS & CABLES WITH MIN.100mm SAND OR FINES TO PREVENT DAMAGE TO DUCTS DURING BACKFILLING AND COMPACTION.

PLOTTED: 8/10/2024 8:33 am

MASF-213.dwg

REV.	DESCRIPTION	DRAWN	CHECKED	APPROVED	DATE
A	FOR CONSENT	DM	GJ	MS	07.10.2024

CLIENT:

UNITS: mm | SHEET SIZE: A3 | SCALE: 1:16 (U.N.O.) | PROJECTION:

ADDITIONAL INFORMATION:

FOR CONSENT

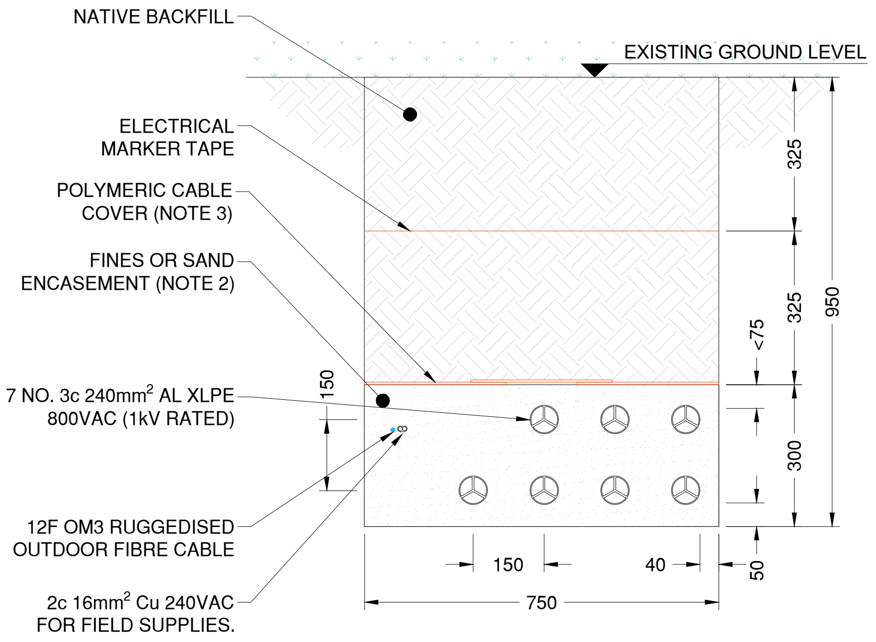
LIGHTYEARS SOLAR
AUCKLAND, NEW ZEALAND
www.lightyearssolar.co.nz

DESIGNED:	DM
DRAWN:	DM
DATE:	07.10.2024
CHECKED:	GJ
APPROVED:	MS

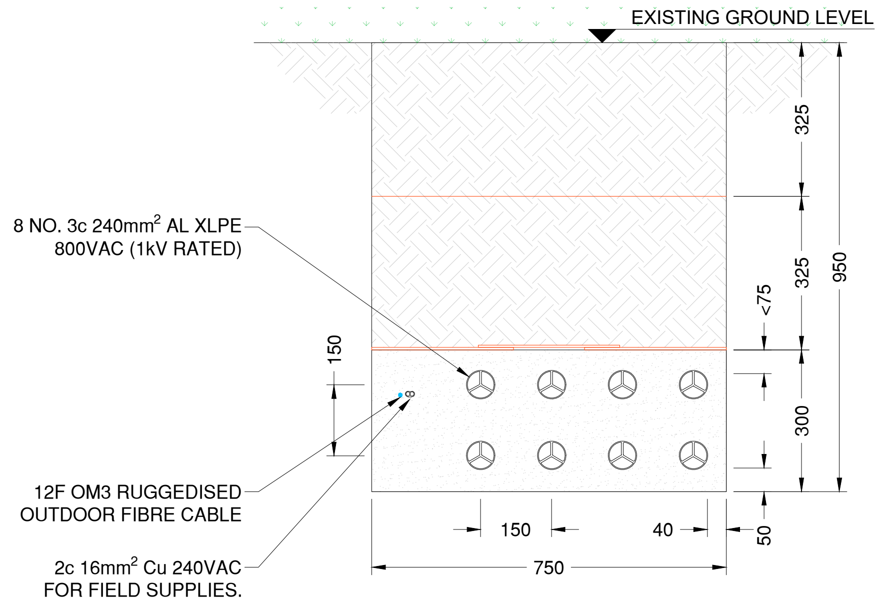
TITLE:
MATAMATA SOLAR FARM
800V AC
TRENCH PROFILES - SHEET 1 of 3

DRG. NO.: MASF-213

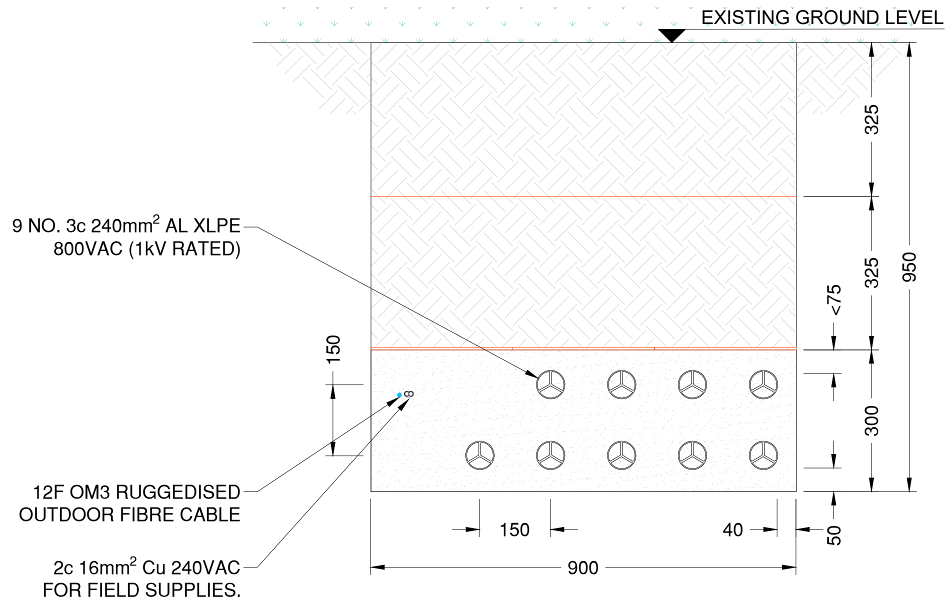
REVISION: A



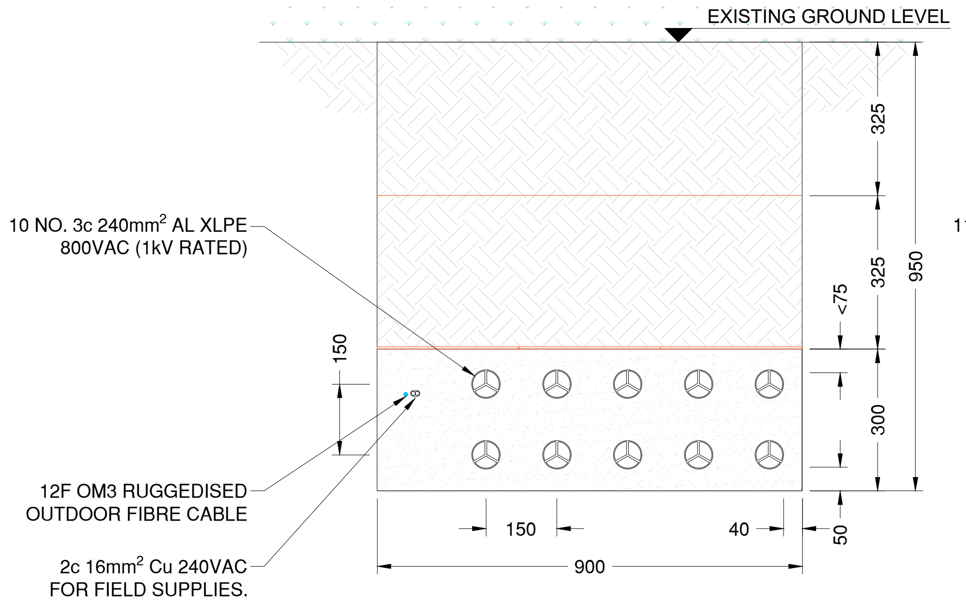
G | 7 CABLE AC TRENCH
- | Scale 1:16



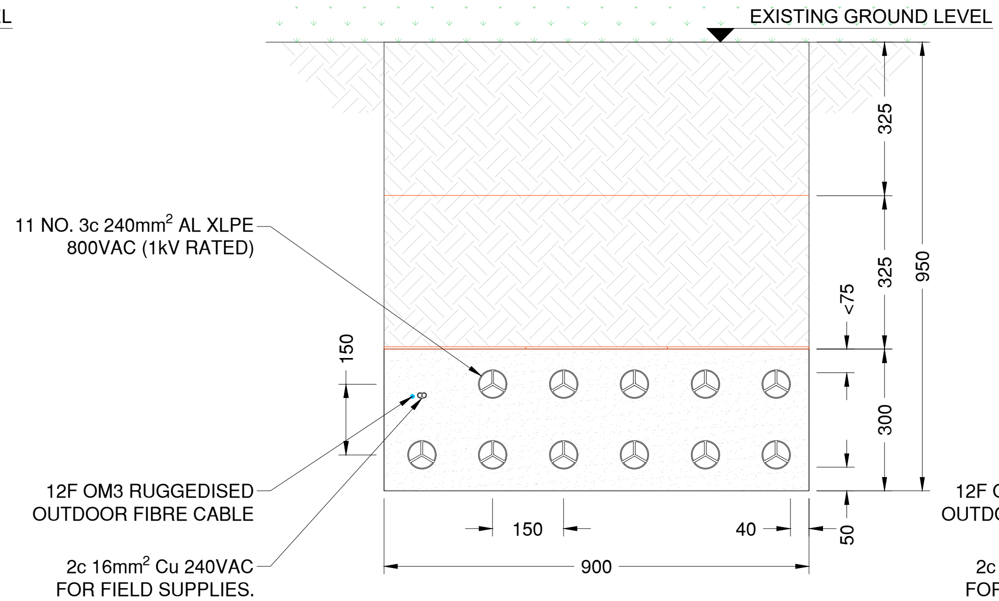
H | 8 CABLE AC TRENCH
- | Scale 1:16



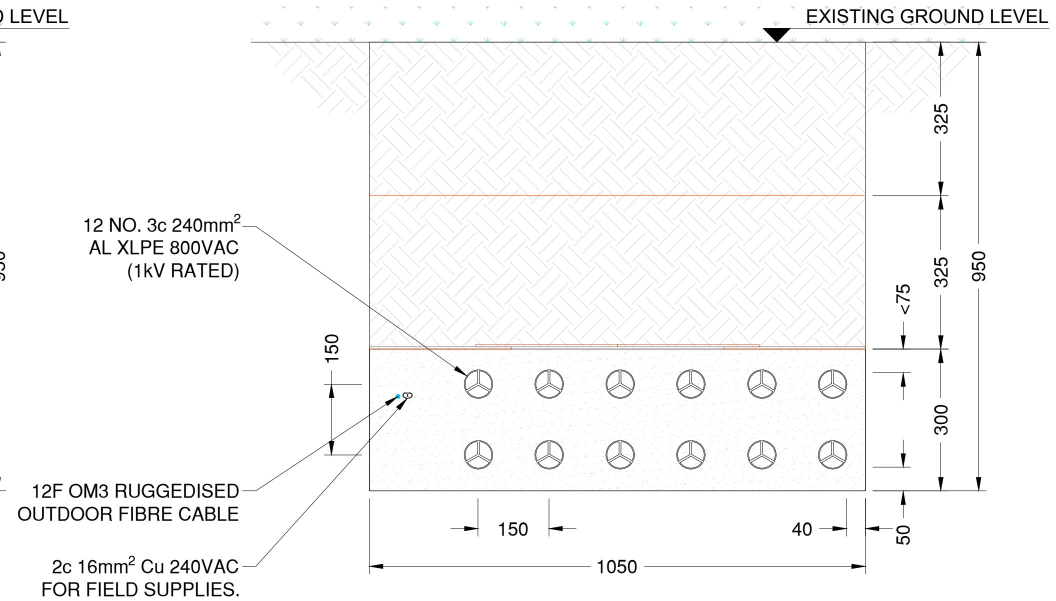
I | 9 CABLE AC TRENCH
- | Scale 1:16



J | 10 CABLE AC TRENCH
- | Scale 1:16



K | 11 CABLE AC TRENCH
- | Scale 1:16



L | 12 CABLE AC TRENCH
- | Scale 1:16

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETRES UNO.
2. 800V AC CABLING TO BE ENCASED IN SAND OR FINES, MIN. 50mm TOP AND BOTTOM PRIOR TO BACKFILL WITH NATIVE SOIL.
3. AS/NZS 4702 POLYMERIC CABLE COVER OR MAGSLAB. TO EXTEND MINIMUM 40mm HORIZONTALLY BEYOND THE CABLE OR DUCT INSTALLED BELOW.
4. WHERE LARGE BOULDERS ARE PRESENT IN THE NATIVE SOIL, COVER DUCTS & CABLES WITH MIN.100mm SAND OR FINES TO PREVENT DAMAGE TO DUCTS DURING BACKFILLING AND COMPACTION.

PLOTTED: 8/10/2024 8:34 am

MASF-214.dwg

REV.	DESCRIPTION	DRAWN	CHECKED	APPROVED	DATE
A	FOR CONSENT	DM	GJ	MS	07.10.2024

CLIENT:

UNITS: mm | SHEET SIZE: A3 | SCALE: 1:16 (U.N.O.) | PROJECTION:

ADDITIONAL INFORMATION:

FOR CONSENT

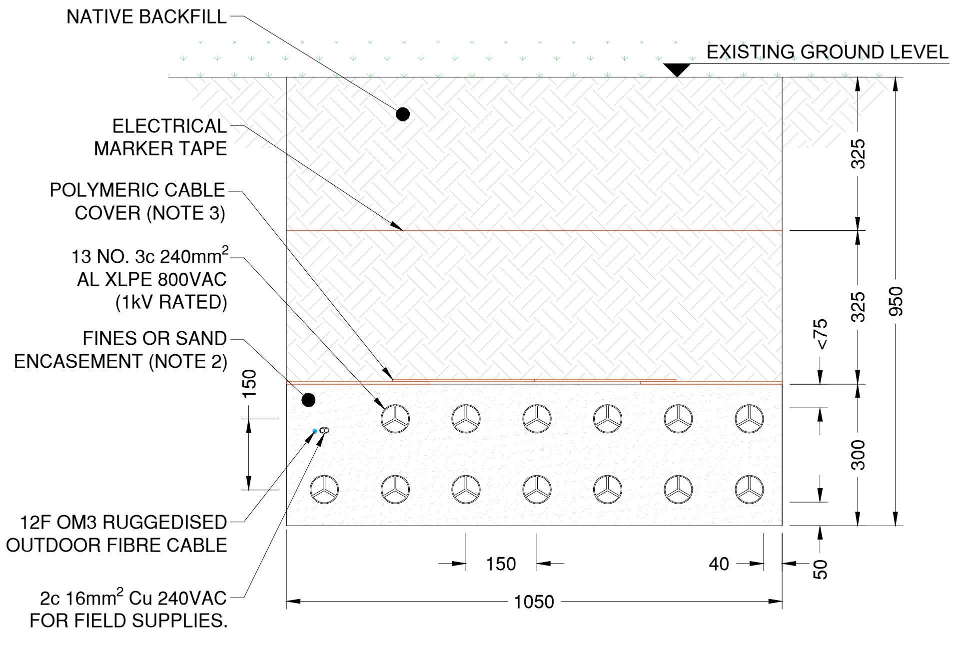
LIGHTYEARS SOLAR
AUCKLAND, NEW ZEALAND
www.lightyearssolar.co.nz

DESIGNED:	DM
DRAWN:	DM
DATE:	07.10.2024
CHECKED:	GJ
APPROVED:	MS

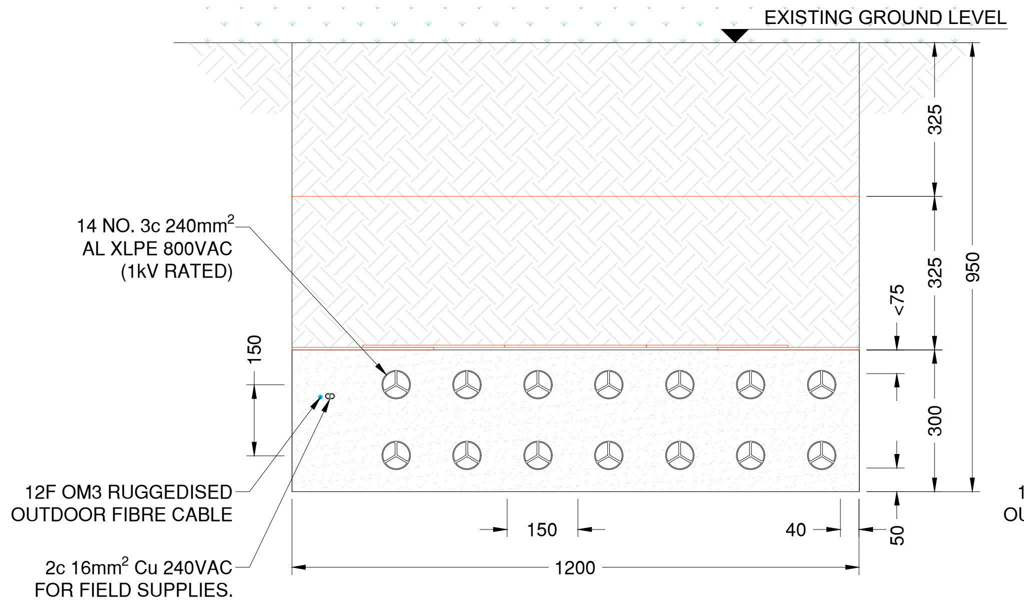
TITLE: MATAMATA SOLAR FARM
800V AC
TRENCH PROFILES - SHEET 2 of 3

DRG. NO.: MASF-214

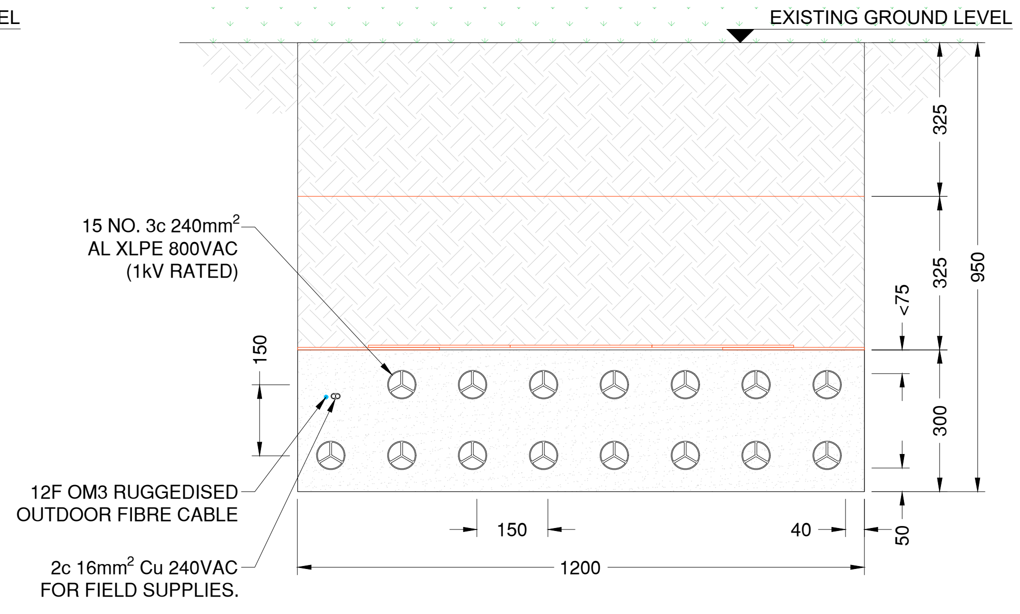
REVISION: A



M - 13 CABLE AC TRENCH
Scale 1:16



N - 14 CABLE AC TRENCH
Scale 1:16



O - 15 CABLE AC TRENCH
Scale 1:16

NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETRES UNO.
2. 800V AC CABLING TO BE ENCASED IN SAND OR FINES, MIN. 50mm TOP AND BOTTOM PRIOR TO BACKFILL WITH NATIVE SOIL.
3. AS/NZS 4702 POLYMERIC CABLE COVER OR MAGSLAB. TO EXTEND MINIMUM 40mm HORIZONTALLY BEYOND THE CABLE OR DUCT INSTALLED BELOW.
4. WHERE LARGE BOULDERS ARE PRESENT IN THE NATIVE SOIL, COVER DUCTS & CABLES WITH MIN.100mm SAND OR FINES TO PREVENT DAMAGE TO DUCTS DURING BACKFILLING AND COMPACTION.

PLOTTED: 8/10/2024 8:34 am
MASF-215.dwg

REV.	DESCRIPTION	DRAWN	CHECKED	APPROVED	DATE
A	FOR CONSENT	DM	GJ	MS	07.10.2024

CLIENT:

UNITS: mm | SHEET SIZE: A3 | SCALE: 1:16 (U.N.O.) | PROJECTION:

ADDITIONAL INFORMATION:

FOR CONSENT

LIGHTYEARS SOLAR
AUCKLAND, NEW ZEALAND
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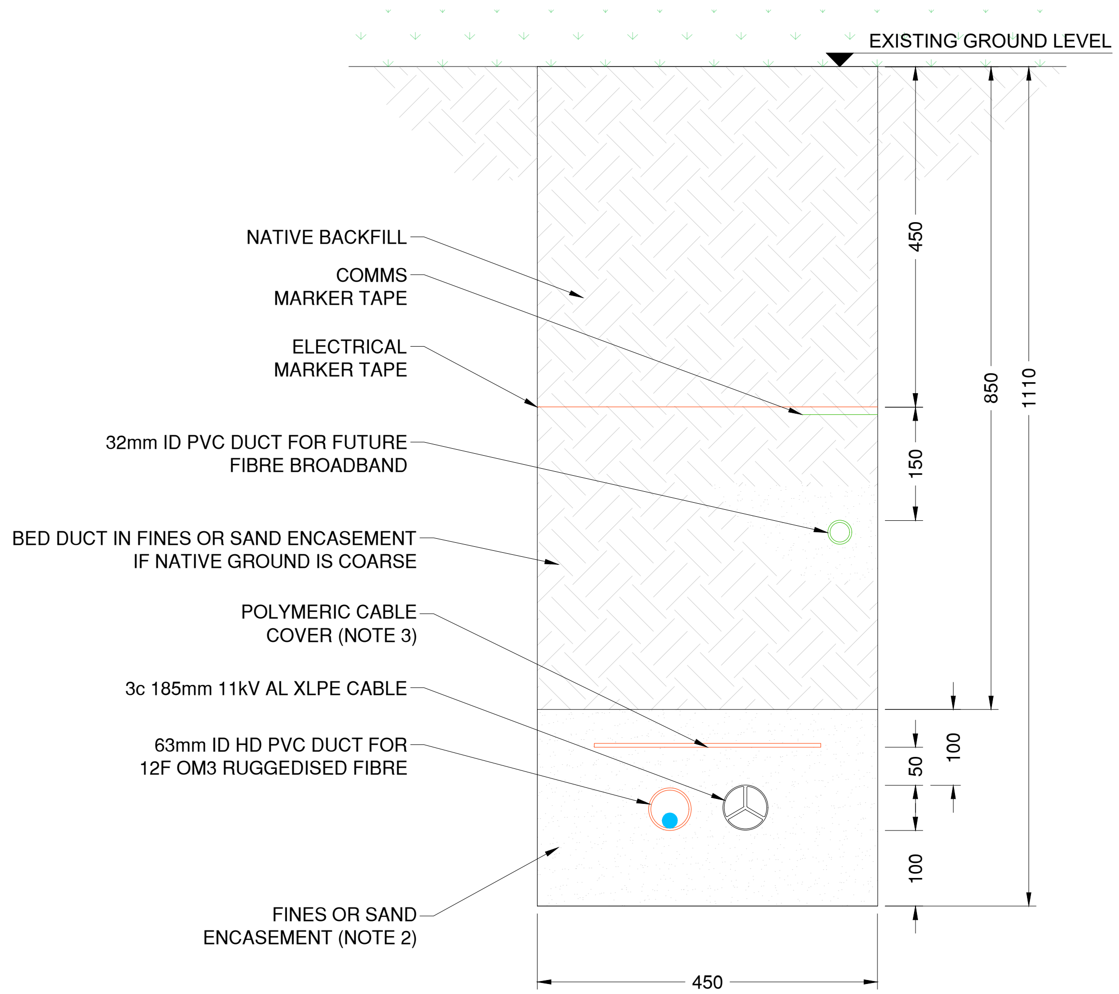
DESIGNED:	DM
DRAWN:	DM
DATE:	07.10.2024
CHECKED:	GJ
APPROVED:	MS

TITLE: MATAMATA SOLAR FARM
800V AC
TRENCH PROFILES - SHEET 3 of 3

DRG. NO.: MASF-215

REVISION: A



A | 11kV TRENCH
203 | Scale 1:10

NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETRES UNO.
2. DIRECT BURIED 11kV AND 800V AC CABLING TO BE ENCASED IN SAND OR FINES, MIN. 50mm TOP AND BOTTOM PRIOR TO BACKFILL WITH NATIVE SOIL.
3. AS/NZS 4702 POLYMERIC CABLE COVER OR MAGSLAB. TO EXTEND MINIMUM 40mm HORIZONTALLY BEYOND THE CABLE OR DUCT INSTALLED BELOW.
4. WHERE LARGE BOULDERS ARE PRESENT IN THE NATIVE SOIL, COVER DUCTS & CABLES WITH MIN.100mm SAND OR FINES TO PREVENT DAMAGE TO DUCTS DURING BACKFILLING AND COMPACTION.

REV.	DESCRIPTION	DRAWN	CHECKED	APPROVED	DATE
A	FOR CONSENT	DM	GJ	MS	07.10.2024

CLIENT:

UNITS:mm | SHEET SIZE: A3 | SCALE: 1:10 (U.N.O.) | PROJECTION:

ADDITIONAL INFORMATION:

FOR CONSENT

THE DESIGN AND DRAWINGS SHOWN ARE THE PROPERTY OF LIGHTYEARS SOLAR AND ARE NOT TO BE REPRODUCED WITHOUT WRITTEN AUTHORITY

LIGHTYEARS SOLAR
AUCKLAND, NEW ZEALAND
www.lightyearssolar.co.nz

DESIGNED:	DM
DRAWN:	DM
DATE:	07.10.2024
CHECKED:	GJ
APPROVED:	MS

TITLE: MATAMATA SOLAR FARM
MV CONNECTION
TRENCH PROFILE

DRG. NO.: MASF-225

REVISION A

PLOTTED: 8/10/2024 8:34 am
MASF-225.dwg

Appendix D – Solar Data Sheets

SG350HX

Multi-MPPT String Inverter for 1500 Vdc System

HIGH YIELD

- Up to 16 MPPTs with max. efficiency 99%
- 20A per string, compatible with 500Wp+ module
- Data exchange with tracker system, improving yield

LOW COST

- Q at night function, save investment
- Power line communication (PLC)
- Smart IV Curve diagnosis*,active O&M

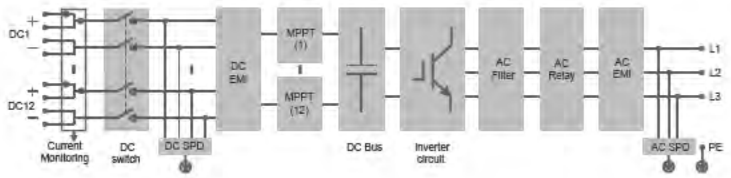
GRID SUPPORT

- SCR≥1.15 stable operation in extremely weak grid
- Reactive power response time <30ms
- Compliant with global grid code

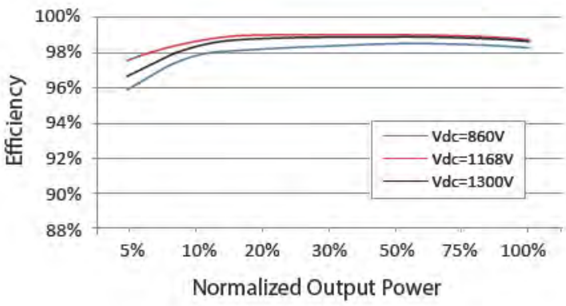
PROVEN SAFETY

- 2 strings per MPPT, no fear of string reverse connection
- 24h real-time AC and DC insulation monitoring

CIRCUIT DIAGRAM



EFFICIENCY CURVE



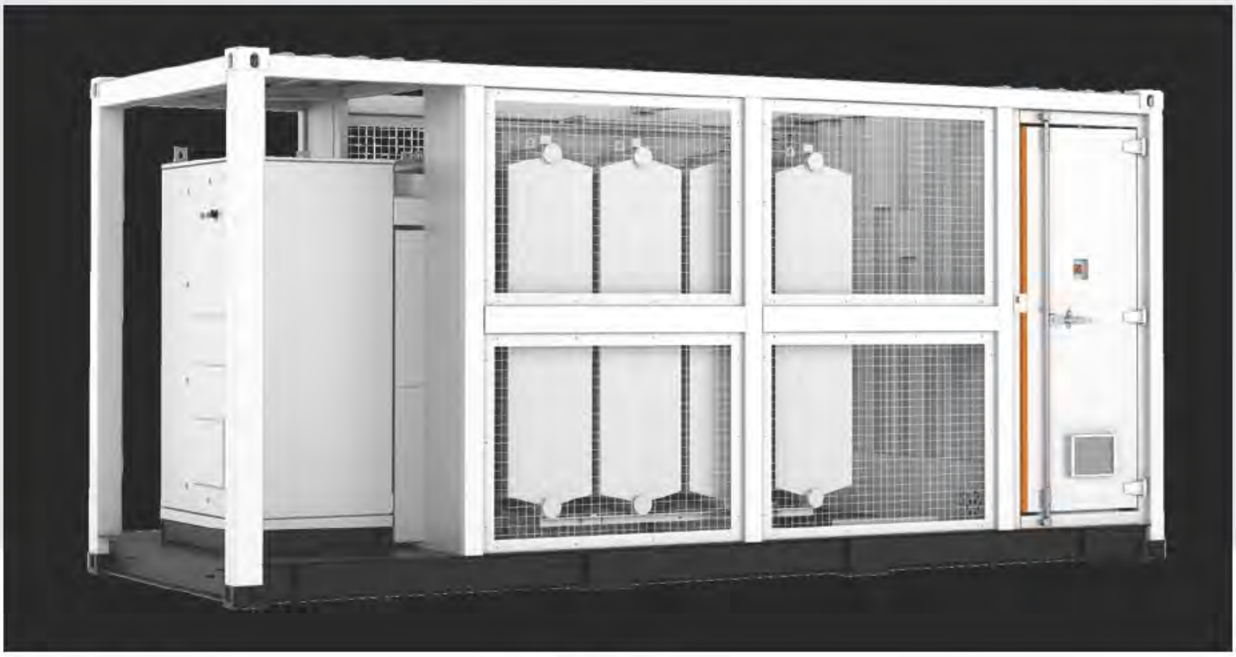
SUNGROW
Clean power for all

Type designation	SG350HX
Input (DC)	
Max. PV input voltage	1500 V
Min. PV input voltage / Startup input voltage	500 V / 550 V
Nominal PV input voltage	1080 V
MPP voltage range	500 V – 1500 V
No. of independent MPP inputs	12 (Optional: 16)
Max. number of input connector per MPPT	2
Max. PV input current	12 * 40 A (Optional: 16 * 30 A)
Max. DC short-circuit current per MPPT	60 A
Output (AC)	
AC output power	352 kVA @ 30℃ / 320 kVA @ 40 ℃ / 295 kVA @ 50℃
Max. AC output current	254 A
Nominal output current	231 A
Nominal AC voltage	3 / PE, 800 V
AC voltage range	640 V – 920 V
Nominal grid frequency / Grid frequency range	50 Hz / 45 Hz – 55 Hz, 60 Hz / 55 Hz – 65 Hz
THD	< 3 % (at nominal power)
DC current injection	< 0.5 % In
Power factor at nominal power / Adjustable power factor	> 0.99 / 0.8 leading – 0.8 lagging
Feed-in phases / Connection phases	3 / 3
Efficiency	
Max. efficiency / European efficiency	99.02 % / 98.8 %
Protection	
DC reverse connection protection	Yes
AC short circuit protection	Yes
Leakage current protection	Yes
Grid monitoring	Yes
Ground fault monitoring	Yes
DC switch / AC switch	Yes / No
PV string current monitoring	Yes
Q at night function	Yes
Anti-PID and PID recovery function	Optional
Surge protection	DC Type II / AC Type II
General data	
Dimensions (W*H*D)	1136 mm * 870 mm * 361 mm
Weight *	≤ 116 kg
Isolation method	Transformerless
Degree of protection	IP66
Power consumption at night	< 6 W
Operating ambient temperature range	-30 ℃ – 60 ℃
Allowable relative humidity range	0 % – 100 %
Cooling method	Smart forced air cooling
Max. operating altitude	4000 m (> 3000 m derating)
Display	LED, Bluetooth+APP
Communication	RS485 / PLC
DC connection type	MC4-Evo2 (Max. 6 mm², optional 10mm²)
AC connection type	Support OT/DT terminal (Max. 400 mm²)
Compliance	IEC 62109, IEC 61727, IEC 62116, IEC 60068, IEC 61683, VDE-AR-N 4110:2018, VDE-AR-N 4120:2018, EN 50549-1/2, UNE 206007-1:2013, P.O.12.3, UTE C15-712-1:2013
Grid Support	LVRT, HVRT, active & reactive power control and power ramp rate control, Q-U control, P-f control

* Due to the multi-supplier for some key components, the actual weight may have a ±10% deviation, please refer to the actually delivered product.

MVS3200/4480-LV

MV Turnkey Solution for 1500 Vdc String Inverter SG350HX / SG350HX-20



SAVED INVESTMENT

- Up to 4.48 MW block design
- Easy transportation due to standard container design
- All pre-assembled for easy set-up and commissioning

SAFETY

- MV and LV isolated, independent control room
- All key components front accessible, no need walk-in operation

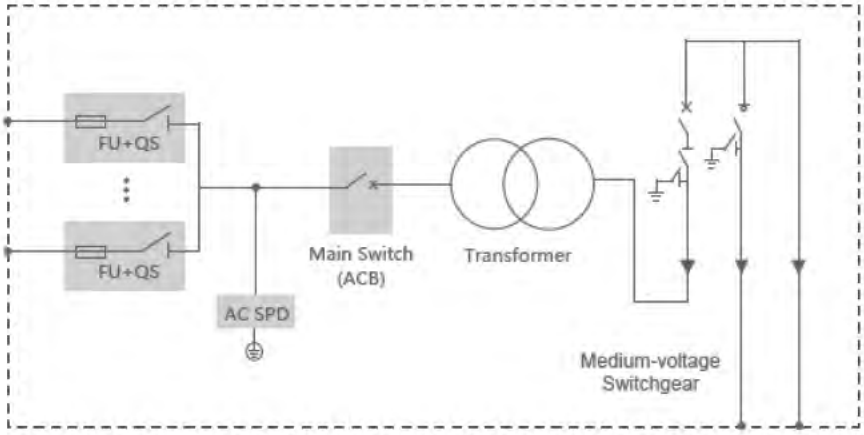
EASY O&M

- Online analysis for fast trouble shooting
- Modular design, main device easy replacement

RELIABLE

- All components type-tested
- Compliance with standards: IEC 60076, IEC 62271, IEC 61439

CIRCUIT DIAGRAM



SUNGROW
Clean power for all

Type designation	MVS3200-LV	MVS4480-LV
Transformer		
Transformer type	Oil immersed	
Rated power	3200 kVA @ 40 ℃	4480 kVA @ 40 ℃
Max. power	3520 kVA @ 30 ℃	4928 kVA @ 30 ℃
Vector group	Dy11	
LV / MV voltage	0.8 kV / (10 - 35) kV	
Maximum input current at nominal voltage	2540 A	3557 A
Frequency	50 Hz / 60 Hz	
Tapping on HV	0 , ± 2 * 2.5 %	
Efficiency	≥ 99 %	
Cooling method	ONAN (Oil Natural Air Natural)	
Impedance	7 % (± 10 %)	8 % (± 10 %)
Oil type	Mineral oil (PCB free)	
Winding material	Al / Al	
Insulation class	A	
MV switchgear		
Insulation type	SF6	
Rated voltage range	24 kV – 40.5 kV	
Rated current	630 A	
Internal arcing fault	IAC AFL 20 kA / 1 s	
LV panel		
Main switch specification	4000 A / 800 Vac / 3P, 1 pcs	
Disconnector specification	260 A / 800 Vac / 3P, 10 pcs	260 A / 800 Vac / 3P, 14 pcs
Fuse specification	400A / 800 Vac / 1P, 30 pcs	400 A / 800 Vac / 1P, 42 pcs
Protection		
AC input protection	Fuse+Disconnector	
Transformer protection	Oil-temperature, Oil-level, Oil-pressure, Buchholz	
Relay protection	50 / 51, 50N / 51N	
Surge protection	AC Type I + II	
General data		
Dimensions(W*H*D)	6058 mm * 2896 mm * 2438 mm	
Approximate weight	15 T	17 T
Operating ambient temperature range *	-20 ℃ – 60 ℃ (optional: -30 ℃ – 60 ℃)	
Auxiliary transformer supply	15 kVA / 400 V (optional: max. 40 kVA)	
Degree of protection	IP54	
Allowable relative humidity range (non-condensing)	0 % – 95 %	
Operating altitude	1000 m (standard) / > 1000 m (optional)	
Communication	Standard: RS485, Ethernet, Optical fiber	
Compliance	IEC 60076, IEC 62271-200, IEC 62271-202, IEC 61439-1, EN 50588-1	

* The ambient temperature is determined as the average temperature obtained from at least four evenly distributed temperature monitoring points located at a distance of 1 meter from the equipment, at a height halfway up the machine. The temperature sensors must be shielded from airflow, thermal radiation, and rapid temperature fluctuations to prevent display inaccuracies.

MVS6400-LV

MV Turnkey Solution for 1500 Vdc String Inverter SG350HX / SG350HX-20

SAVED INVESTMENT

- Up to 7 MW block design
- Easy transportation due to standard container design
- All pre-assembled for easy set-up and commissioning

SAFETY

- MV and LV isolated, independent control room
- All key components front accessible, no need walk-in operation

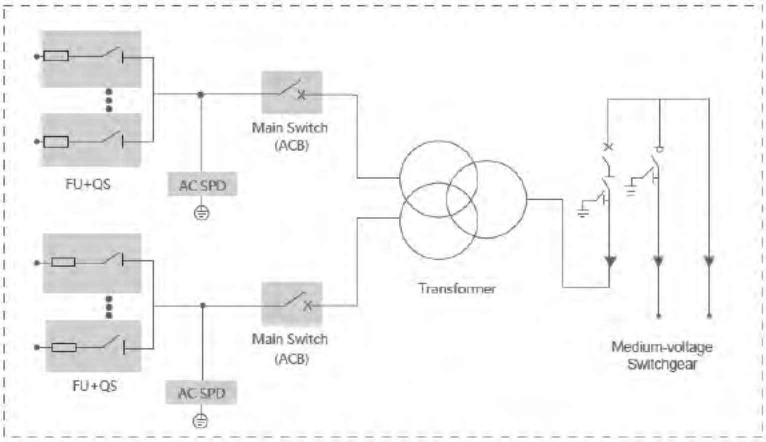
EASY O&M

- Online analysis for fast trouble shooting
- Modular design, main device easy replacement

RELIABLE

- All components type-tested
- Compliance with standards: IEC 60076, IEC 62271, IEC 61439

CIRCUIT DIAGRAM



SUNGROW
Clean power for all

Type designation	MVS6400-LV
Transformer	
Transformer type	Oil immersed
Rated power	6400 kVA @ 40 ℃
Max. power	7040 kVA @ 30 ℃
Vector group	Dy11y11
LV / MV voltage	0.8 kV - 0.8 kV / (10 – 35) kV
Maximum input current at nominal voltage	2540 A * 2
Frequency	50 Hz / 60 Hz
Tapping on HV	0, ± 2 * 2.5 %
Efficiency	≥ 99 %
Cooling method	ONAN (Oil Natural Air Natural)
Impedance	8 % (± 10%)
Oil type	Mineral oil (PCB free)
Winding material	Al / Al
Insulation class	A
MV switchgear	
Insulation type	SF6
Rated voltage range	24 kV – 40.5 kV
Rated current	630 A
Internal arcing fault	IAC AFL 20 kA / 1 s
LV panel	
Main switch specification	4000 A / 800 Vac / 3P, 2 pcs
Disconnector specification	260 A / 800 Vac / 3P, 20 pcs
Fuse specification	400A / 800 Vac / 1P, 60 pcs
Protection	
AC input protection	Fuse+Disconnector
Transformer protection	Oil-temperature, Oil-level, Oil-pressure, Buchholz
Relay protection	50 / 51, 50 N / 51 N
Surge protection	AC Type I + II
General data	
Dimensions (W*H*D)	6058 mm * 2896 mm * 2438 mm
Approximate weight	22 T
Operating ambient temperature range *	-20 ℃ - 60 ℃ (optional: -30 ℃ - 60 ℃)
Auxiliary transformer supply	15 kVA / 400 V (optional: max. 40 kVA)
Degree of protection	IP54
Allowable relative humidity range (non-condensing)	0 % – 95 %
Operating altitude	1000 m (standard) / > 1000 m (optional)
Communication	Standard: RS485, Ethernet, Optical fiber
Compliance	IEC 60076, IEC 62271-200, IEC 62271-202, IEC 61439-1, EN 50588-1

* The ambient temperature is determined as the average temperature obtained from at least four evenly distributed temperature monitoring points located at a distance of 1 meter from the equipment, at a height halfway up the machine. The temperature sensors must be shielded from airflow, thermal radiation, and rapid temperature fluctuations to prevent display inaccuracies.

JA SOLAR | DEEP BLUE 4.0 Pro

Version No. : AUS-EN-20230818A

600W LB Series

- Higher power generation better LCOE
- n-type with very low LID
- Better temperature coefficient
- Better low irradiance response

- 15-year product warranty
- 30-year linear power output warranty

n-type Bifacial Double Glass High Efficiency Mono Module JAM66D45 LB/1500V

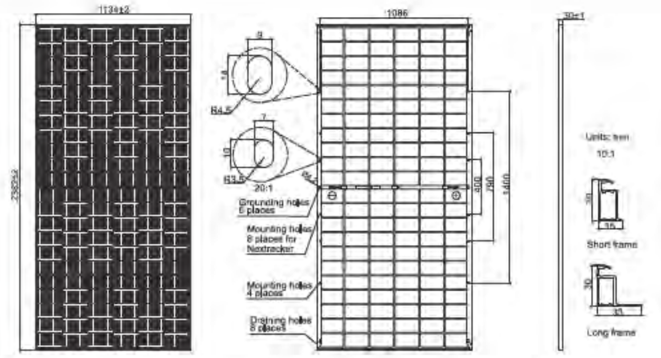
590-615

Comprehensive Certificates

- IEC 61215, IEC 61730
- ISO 9001: 2015 Quality management systems
- ISO 14001: 2015 Environmental management systems
- ISO 45001: 2018 Occupational health and safety management systems

CE

590-615 LB Series
JAM66D45



Remark: customized frame color and cable length available upon request

Cell	Mono-16BB
Weight	33.1kg
Dimensions	2382±2mm×1134±2mm×30±1mm
Cable Cross Section Size	$4mm^2$ (IEC), 12 AWG(UL)
No. of cells	132(6×22)
Junction Box	IP68, 3 diodes
Connector	Stäubli MC4-EVO2A/MC4-EVO2 QC Solar QC 4.10-35I/QC 4.10-35
Cable Length (Including Connector)	Portrait: 300mm(+)/400mm(-); 800mm(+)/800mm(-)(Leapfrog) Landscape: 1500mm(+)/1500mm(-)
Front Glass/Back Glass	2.0mm/2.0mm
Country of Manufacturer	China/Vietnam

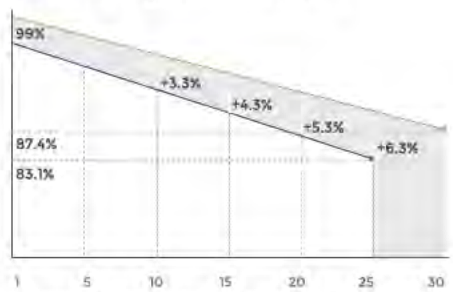
ELECTRICAL PARAMETERS AT STC

TYPE	JAM66D45-590/LB/1500V	JAM66D45-595/LB/1500V	JAM66D45-600/LB/1500V	JAM66D45-605/LB/1500V	JAM66D45-610/LB/1500V	JAM66D45-615/LB/1500V
Rated Maximum Power(Pmax) [W]	590	595	600	605	610	615
Open Circuit Voltage(Voc) [V]	47.30	47.50	47.70	47.90	48.10	48.30
Maximum Power Voltage(Vmp) [V]	39.09	39.27	39.44	39.60	39.77	39.96
Short Circuit Current(Isc) [A]	15.85	15.90	15.95	16.00	16.05	16.10
Maximum Power Current(Imp) [A]	15.09	15.15	15.21	15.28	15.34	15.39
Module Efficiency [%]	21.8	22.0	22.2	22.4	22.6	22.8
Power Tolerance	0~+5W					
Temperature Coefficient of Isc(α_Isc)	+0.046%/℃					
Temperature Coefficient of Voc(β_Voc)	-0.260%/℃					
Temperature Coefficient of Pmax(γ_Pmp)	-0.300%/℃					
STC	Irradiance $1000W/m^2$, cell temperature 25℃, AM1.5G					

Remark: Electrical data in this catalog do not refer to a single module and they are not part of the offer.They only serve for comparison among different module types.
Measurement tolerance at STC: Pmax ±3%, Voc ±3% and Isc ±5%.

Superior Warranty

1% 1st-year Degradation
0.4% Annual Degradation Over 30 years



- n-type Bifacial Double Glass Module Linear Performance Warranty
- Standard Module Linear Performance Warranty

*Subject to the terms and conditions contained in the Limited Warranty Statement. Also this 15-year limited product warranty is available only for products installed and operating on rooftops in certain regions. stalled and operating on rooftops in certain regions.

ELECTRICAL CHARACTERISTICS WITH 10% SOLAR IRRADIATION RATIO

TYPE	JAM66D45-590/LB/1500V	JAM66D45-595/LB/1500V	JAM66D45-600/LB/1500V	JAM66D45-605/LB/1500V	JAM66D45-610/LB/1500V	JAM66D45-615/LB/1500V
Rated Max Power(Pmax) [W]	637	643	648	653	659	664
Open Circuit Voltage(Voc) [V]	47.30	47.50	47.70	47.90	48.10	48.30
Max Power Voltage(Vmp) [V]	39.09	39.27	39.44	39.60	39.77	39.96
Short Circuit Current(Isc) [A]	17.12	17.17	17.23	17.28	17.33	17.39
Max Power Current(Imp) [A]	16.30	16.36	16.43	16.50	16.56	16.62
Irradiation Ratio (rear/front)	10%					

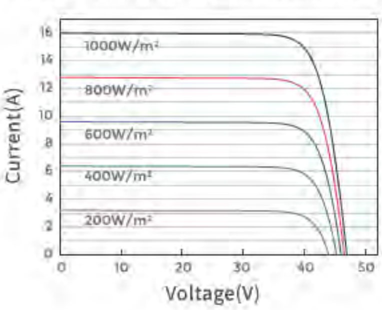
OPERATING CONDITIONS

Maximum System Voltage	1500V DC
Operating Temperature	-40℃~+85℃
Maximum Series Fuse Rating	35A
Maximum Static Load,Front*	3600Pa, 1.5
Maximum Static Load,Back*	1600Pa, 1.5
NOCT	45±2℃
Bifaciality**	80%±10%
Fire Safety Class	Class C

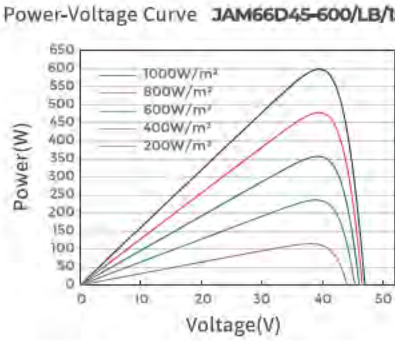
*For Nextracker installations, maximum static load please take compatibility approve letter between JA Solar and Nextracker for reference.
**Bifaciality=Pmax,rear/Rated Pmax,front

CHARACTERISTICS

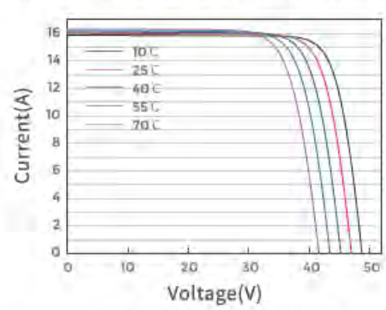
Current-Voltage Curve **JAM66D45-600/LB/1500V**



Power-Voltage Curve **JAM66D45-600/LB/1500V**



Current-Voltage Curve **JAM66D45-600/LB/1500V**



Appendix E – CMW Geotechnical Investigation Report

CMW Geosciences

Great People | Practical Solutions

5 July 2024

PROPOSED RESIDENTIAL SUBDIVISION AND SOLAR FARM

STATION ROAD, MATAMATA

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT

Matamata Development Limited C/O Maven Associates

HAM2023-0124AE Rev 0

HAM2023-0124AE		
Date	**Revision**	**Comments**
02 July 2024	A	Initial draft for internal review
03 July 2024	B	Draft for final review
05 July 2024	0	Report Issue

	Name	Signature	Position
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Authorised by	Sam Gibb	S. Gibb	Principal Geotechnical Engineer CMEngNZ, CPEng

EXECUTIVE SUMMARY

This report presents the results of a preliminary geotechnical investigation and geohazards assessment for a proposed residential subdivision and solar farm along Station Road, Matamata.

Based on the investigation results, the site is underlain by Hinuera Formation and Walton Subgroup soils.

Geotechnical recommendations for the proposed development are summarised as follows:

- Liquefaction analyses indicate the following liquefaction-induced settlement during a ULS event:
    - Between 20mm to 165mm for IL2 structures.
    - Between 100mm to 145mm IL3 structures.
- There is high potential for lateral spread near the swale at all three blocks and near the proposed greenway and the riverbank in the Western Block.
- TC2 foundations will be required for residential development in the Eastern Block and Hybrid TC2/TC3 foundations will be required for residential development in the Western Block. Foundations in the northern block will also be required to be designed to sustain liquefaction-induced settlement and lateral spreading effects.
- Static settlements are anticipated to be low risk due to the presence of dense to very dense sands and stiff to very stiff silts.

Additional investigations, testing and analysis will be required to support subdivision consent application, earthworks and building consent applications.

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1 INTRODUCTION

1.1 Project Brief

CMW Geosciences (CMW) was engaged by Matamata Development Limited C/O Maven Associates to carry out a preliminary geotechnical investigation of a site located at Station Road, Matamata, which is being considered for the construction of a residential subdivision and a solar farm.

The scope of work and associated terms and conditions of our engagement were detailed in our services proposal letter referenced HAM2023-0124AD Rev0, dated 15 May 2024.

This report is to provide a preliminary geotechnical assessment, advice on the geotechnical risks present at the site and possible mitigation measures that are relevant for the proposed residential subdivision and solar farm. At the time of writing this report, the project was in the early stages of planning, and it was anticipated that the geotechnical investigation will inform site suitability and development options. Further investigations and analyses are expected as the project progresses.

1.2 Scope of Work

As detailed in our proposal letter, the agreed scope of work to be conducted by CMW was defined as follows:

- Arrange and perform a geotechnical site investigation (SI).
- Evaluate and develop a ground model.
- Perform a geohazards assessment and provide relevant geotechnical recommendations.
- Compile the above details into a geotechnical report.

2 SITE DESCRIPTION

2.1 Site Location

The site comprises three blocks of land on Station Road as shown on Figure 1 and comprises the following:

- Northern Block – legally described as Lot 2 DP 567678, with an area of approximately 13.5ha.
- Eastern Block, 127 Station Road – legally described as Lot 1 DPS 65481, with an area of approximately 4.2ha.
- Western Block, 243 Station Road – legally described as Lot 1 DP 21055, Lot 2 DP 21055 and Lot 3 DPS 14362, with an area of approximately 73ha.

CMW have previously undertaken a geotechnical investigation on the adjacent site to the east of the Western Block (CMW Ref. HAM2023-0124AB Rev 1, dated 12 Dec 2023).

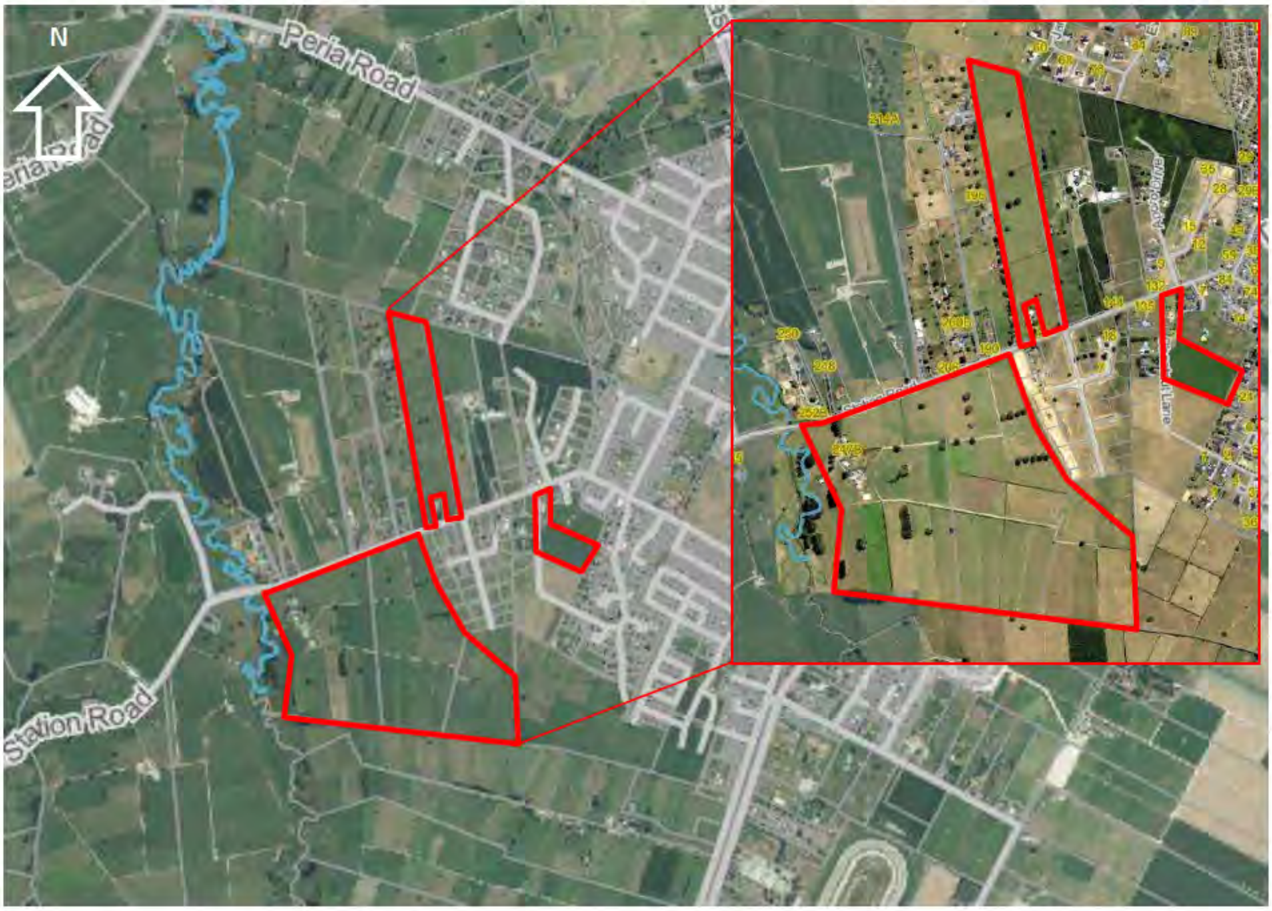

Figure 1: Site Location Plan (Google Maps)

2.2 Landform

The current general landform, together with associated features located within and adjacent to the site is presented on the attached Site Plan as ***Drawing 01.***

- The site can generally be described as near level with existing ground levels ranging from RL68m in the east (Moturiki Datum) to RL72m in the west within the Western Block. Along the western boundary the slope trends in a general east to west direction with a maximum slope gradient of 1(V):5(H) to RL59m towards the Waitoa River which runs parallel to the western site boundary.
- Each Block is adjoined by Station Road (Western and Eastern Blocks to the north, Northern Block to the south). Pastoral land surrounds the Northern Block and the Western Block in all remaining directions.
- Residential development is located to the east of the Eastern Block.
- Early subdivision formation is evident in the land between the Western and Eastern Blocks.
- Each Block is currently utilised as pastoral land and predominantly grass covered with sporadic mature trees. Grazing stock was present during our site visits.
- Landform and use remains consistent within the last 50 years with historical aerial photographs observed (Retrolens images circa 1971)
- Residential dwellings are present on the northern portion of the Eastern Block and the northwestern portion of the Western Block with additional farm buildings noted on the Western Block (milking shed, storage sheds etc).
- A stormwater swale is located along the eastern and northern boundaries of the Western Block and the northern boundary of the Northern Block to a maximum depth of approximately 2m.

- Additional swale drains within each block were observed along edges of paddocks. It is assumed that these drains will be infilled as part of the subdivision earthworks.

3 PROPOSED DEVELOPMENT

A preliminary development plan was provided (prepared by Maven Associates Ref. 289001, dated May 2024) which details the preliminary layout for the proposed development. Residential development (one- and two-storey) is proposed on the Eastern Block and the Western Block with associated roads and infrastructure. It is understood that a greenway is proposed centrally across the Western Block which is to collect stormwater runoff from the subdivision. This greenway is oriented in a general east to west direction and flows towards the river. A copy of this plan is presented within ***Appendix B***. It is understood that a solar farm is proposed on the Northern Block.

As no architectural or engineering design drawings have been supplied to date, we have prepared this report on the basis that a future development will broadly comprise minor cuts and fills to form a near level site supporting residential buildings on the Eastern and Western Blocks.

4 INVESTIGATION SCOPE

4.1 Field Investigation

Following a dial before you dig search, and onsite service location, the field investigation was carried out between 31 May 2024 and 7 June 2024. All fieldwork was carried out under the direction of CMW Geosciences in general accordance with the NZGS specifications[1] and logged in accordance with NZGS guidance[2]. The scope of fieldwork completed was as follows:

- Undertook a walkover survey of the site to assess the general landform, site conditions and adjacent structures / infrastructure;
- An on-site services search was carried out by a specialist contractor to identify the presence of any underground obstructions or hazards prior to the field investigation program commencing;
- 17 no. hand auger boreholes, denoted HA24-09 to HA24-25, were drilled using a 50mm diameter auger to target depths of up to 5.0m below existing ground levels to visually observe the near surface soil profile and to facilitate in-situ permeability / vane shear strength testing. Hand auger HA24-16 met the target depth, with remaining boreholes terminating at depths ranging between 1.7m and 4.6m due to refusal / hole collapse. Engineering logs of the hand auger boreholes, together with peak and remoulded vane shear strengths are presented in ***Appendix C***;
- Dynamic cone (Scala) penetrometer (DCP) tests were carried out adjacent to each hand auger borehole to depths of up to 5m to provide soil density profiles, for use as a comparison with the CPT data and to provide a subgrade CBR value for pavement design purposes. Graphical results of the DCP testing are presented on the borehole logs in ***Appendix C***;
- Eight in-situ falling head permeability tests were completed in the open standpipe piezometers denotated SOA24-05 to SOA24-12 at depths ranging between 1.4m in SOA24-05 to 3.0m in SOA24-08, SOA24-09 and SOA24-12. Results of the permeability tests are presented in ***Appendix D***;

[1] NZ Geotechnical Society (2017) NZ Ground Investigation Specification, Volume 1 – Master Specification.

[2] NZ Geotechnical Society (2005), Field Description of Soil and Rock, Guideline for the field classification and description of soil and rock for engineering purposes.

- Seven Cone Penetrometer Tests (CPT) and four seismic CPT's denoted CPT24-04 to CPT24-10 and SCPT24-01 to SCPT-04 respectively, were pushed to depths ranging between 16.8m to 30m to define the ground model at depths. Results of the tests are presented as traces of tip resistance (q_c), friction resistance (f_s) and friction ratio are presented in ***Appendix E***;

The approximate locations of the respective investigation sites referred to above are shown on the Site Plan presented within ***Appendix A***. Test locations were recorded using handheld GPS.

5 GROUND MODEL

5.1 Published Geology

Published geological maps[3] for the area depict the regional geology as comprising cross-bedded pumice sand, silt and gravel of the Hinuera Formation.

5.2 Stratigraphic Units

The ground conditions encountered and inferred from the investigation were generally consistent with the published geology for the area as well as CMW's aforementioned report (HAM2023-0124AB) and can be generalised according to the following subsurface sequences.

The distribution of the various units encountered is presented in the appended Geological Sections A to C presented as Drawings 2 to 5 within ***Appendix A***.

5.2.1 Summary

The distribution of these units is illustrated on the appended Geological Cross Sections and presented below in Table 1.

Table 1: Summary of Strata Encountered

Unit	Depth to base (m)		Thickness (m)**	
	Min	Max	Min	Max
Topsoil/Fill	0.1	0.5	0.1	0.5
Stiff to Very Stiff Silt (Hinuera Formation)	1.0	1.1	0.5	1
Dense to Very Dense Sand with interbedded Silt (Hinuera Formation)	5.9	17.3	4.9	16.3
Very Stiff to Hard Silt/Clay (Walton Subgroup)	0.1	18.1	9*	18*
Very Dense Silty Sand (Walton Subgroup)	-	-	**	**
Notes: * Strata not encountered within all test locations. **Thickness only recorded were base of strata has been confirmed.				

[3] Edbrooke, S.W (compiler) 2005: Geology of the Waikato area. Institute of Geological & Nuclear Scienced 1:250,000 Geological Map 4.

5.3 Groundwater

During the investigation, which were completed in early winter conditions (May / June 2024), groundwater was encountered within the CPTs and boreholes at the depths provided in Table 2.

Table 2: Groundwater Monitoring Data		
Area	**Minimum Depth (mbgl)**	**Maximum Depth (mbgl)**
Eastern Block	1.4	2.6
Northern Block	1.8	4.2
Western Block	1.6	5+

Groundwater was not encountered within HA24-15 or HA24-16 (located within Western Block). It is noted that groundwater levels do fluctuate seasonally and following periods of heavy / extended periods of rain by up to a metre.

6 GEOHAZARDS ASSESSMENT

6.1 Context

The following sections of this report provide an assessment of the geohazards relevant to this site and provide the basis for the Natural Hazards Risk Assessment presented in ***Appendix F.***

6.2 Seismicity

Reference to NZGS Guidance[4] was made to determine peak horizontal ground acceleration or PGA (a_{max}) values based on a 50-year design life in accordance with the New Zealand Building Code[5] and importance level (IL) 2 for the residential development and IL3 structures for the proposed solar farm. The PGA values for the serviceability limit state (SLS) and ultimate limit state (ULS) earthquake scenarios are as follows:

Table 3: Design Peak Ground Acceleration (PGA) for Various Limit States			
Limit State	**AEP**	**PGA(g)**	**$Magnitude_{eff}$**
IL2 Structures			
SLS	1/25	0.07	5.9
ULS	1/500	0.28	5.9
IL3 Structures			
SLS	1/25	0.07	5.9
ULS	1/1,000	0.36	5.9

[4] NZ Geotechnical Society publication "Earthquake geotechnical engineering practice, Module 1: Overview of the standards", (March 2016).
[5] Ministry of Business, Innovation and Employment (1992) NZ Building Code Handbook, Third Edition, Amendment 13 (effective from 14 February 2014)

6.3 Fault Rupture

The nearest active fault to the site is the Kerepehi Fault. This fault is approximately 5km east of the site. The Kerepehi Fault has a recurrence interval of between 2,000 years to 3,500 years. We consider the site to be low risk, with respect to fault rupture.

6.4 Liquefaction

6.4.1 Methodology

In accordance with MBIE/NZGS guidance[6] the liquefaction susceptibility of the soils at this site was assessed with respect to geological age and compositional (soil fabric and density) criteria, based on the following assumptions:

- Saturated soils below 1.5m to 2.25m depth were modelled as being susceptible to liquefaction. Saturated soils below 3.5m and 4.75m depth were modelled at SCPT24-03 and SCPT24-04.
- A site-specific assessment was carried out using the seismic CPT's to account for soil microstructure in accordance with Robertson[7]. Results in ***Appendix G*** suggest that "no soil microstructure can be justified" and therefore no strength gain factor has been applied.
- Soils are also classified with respect to their grain size and plasticity to assess liquefaction susceptibility. For this project, a cut-off threshold soil behaviour type index value (I_c) of 2.6 was used to distinguish between liquefiable ($I_c<2.6$) and non-liquefiable ($I_c>2.6$) soils.
- Specific liquefaction analyses were undertaken for IL2 and IL3 structures, using the software package CLiq using the Boulanger and Idriss (2014) method. The cyclic stress ratio (CSR), being a function of the earthquake magnitude for the design return period event, was compared to the cyclic resistance ratio (CRR), being a function of the CPT cone resistance (q_c) and friction ratio (F_r).
- Free-field liquefaction induced settlements were determined in accordance with Zhang et al. (2002). With respect to liquefaction response, consideration was given to a 10m cut-off depth to estimate index settlements as per MBIE[8] guidance (foundation technical categories). These were compared to liquefaction settlement estimates over the full depth range of the CPT's with a depth weighting factor ranging from 1 at the ground surface to 0 at 18m depth applied to the volumetric strains (e_v) in accordance with Cetin et al (2009)[9].

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[6] Earthquake Geotechnical Engineering Practice, Module 3: Identification, assessment and mitigation of liquefaction hazards", (November 2021)

[7] P.K. Robertson (2015). Comparing CPT and Vs Liquefaction Triggering Methods, Journal of Geotechnical and Geoenvironmental Engineering.

[8] Repairing and Rebuilding House affected by the Canterbury Earthquakes", (December 2012)

[9] Cetin, K., Bilge, H., Wu, J., Kammerer, A., and Seed, R. (2009). Probabilistic Model for the Assessment of Cyclically Induced Reconsolidation (Volumetric) Settlements, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135(3), pp. 387-398.

6.4.2 Results

Results are presented in ***Appendix G*** and can be summarised as in Table 4:

Table 4: Liquefaction Analyses Results

Development	SLS Settlement (mm)	IL2 Structures		IL3 Structures	
		Total Settlement (mm)	Index Settlement (mm)	Total Settlement (mm)	Index Settlement (mm)
Residential Development in the Eastern and Western Blocks	<5	20 - 165	25 – 185	-	-
Solar Farm in the Northern Block	<5	-	-	100 - 145	85 - 150

Note: All settlements and depths based on existing ground profile.
Index settlements are calculated based on the upper 10m of the soil profile using no depth weighting factor.
Total ULS settlements are based on the full depth of the CPT trace with a depth weighting factor applied.
Index settlements are for assessment of the site against the MBIE site Technical Category guidelines and are not comparable to the total ULS settlements.

The calculations indicate that liquefaction may occur in some soil layers during a ULS earthquake event. In the ULS cases, the liquefaction results indicate a high risk of liquefaction occurring at the site. Recommendations to mitigate effects of liquefaction settlements on the proposed development are provided below in Section 7.

6.5 Lateral Spread

Following the onset of liquefaction, the liquefied soils behave as a very weak undrained material, which can give rise to lateral spreading where a free face is present within the vicinity of the site or where proposed cut and fill batters are proposed over or within liquefied soils.

The boundary swale depths were assumed to be 2m and were considered for all three blocks. Smaller drains between farm paddocks were assumed to be infilled during subdivision formation. The proposed greenway through the Western Block was assumed to be 2m deep. The riverbank along the western boundary of the Western Block is approximately 5m high. Based on the current landform, free face heights of the swales, proposed greenway and the riverbank and depths to liquefiable layers, there is high potential for lateral spread near the swale at all three blocks and near the greenway and the riverbank in the Western Block.

6.6 Slope Stability

The general landform across the site is flat to gently grading, therefore we do not consider slope stability will be problematic on this site.

6.7 Load Induced Settlement

The predominantly stiff to very still nature of the subsoils dictates that the soils encountered across the majority of the site is generally not prone to excessive load-induced or 'static' settlements under typical residential development proposed fill and building loads.

The majority of the site is recorded to be underlain by predominantly dense to very dense sandy soils which will see any settlement built out during construction.

6.8 Sensitive Soils

The Hinuera Formation silt unit present across the site and encountered within the upper 1m is typically considered moderately sensitive to sensitive. These characteristics may make the silt unit challenging to earthwork and will require special consideration to plant movements during the construction period where exposed.

7 RECOMMENDATIONS

7.1 Seismic Site Subsoil Category

The geological units encountered beneath the development areas comprise soil strength materials, which with respect to the seismic site subsoil category defined in Section 3.1.3 of NZS1170.5, is defined as having a UCS < 1MPa. Therefore, the seismic site subsoil category is assessed as being Class D (deep soil site).

7.2 Liquefaction Mitigation

Based on the analysis results presented in Section 6.4, we consider the risk of liquefaction and liquefaction induced settlements to be high for the ULS cases. Foundation recommendations are provided in Section 7.5.

Liquefaction effects can possibly be reduced by performing laboratory testing to assess the fines content and plastic nature of the fine-grained soils at the site and/ or by performing additional investigations and laboratory testing to take in to account the pumiceous sands at the site.

7.3 Lateral Spread Mitigation

Based on the analysis results presented in Section 6.4 and Section 6.5, we consider the risk of lateral spreading to be high for the ULS cases, in areas adjacent to existing swales, proposed greenway and riverbank.

Appropriate setbacks will have to provided at the detailed design stage. Ground improvement in the form of rammed aggregate piers might be required based on the severity of lateral spreading.

Lateral spreading can possibly be reduced by performing laboratory testing to assess the fines content and plastic nature of the fine-grained soils at the site and/ or by performing additional investigations and laboratory testing to take in to account the pumiceous sands at the site.

7.4 Stormwater Soakage

8 no. falling head permeability tests were undertaken across the site to provide soakage rates. Results indicated that the permeability of soils ranged between 2×10^{-6} and 5×10^{-6} m/sec for the silty material and between 7×10^{-6} to 6×10^{-7} m/sec for the sandier material. HAS24-12 has not been considered based on low soakage rate for the insitu sandy soil. Results of testing are presented as ***Appendix D***. The soils at this site are considered suitable to provide rain gardens / attenuation ponds.

7.5 Foundations

On this site, our provisional expectation is that provided earthworks are completed in accordance with the standards, the following will apply:

- A preliminary geotechnical ultimate bearing pressure of 300 kPa should be available in the static case for shallow strip and pad foundations constructed within both the natural cut ground and engineered fill areas. Geotechnical ultimate bearing pressure in the ULS seismic case will be < 300 kPa based on the shallow liquefiable layers.

- There may be areas where localised variations in shear strength within the natural cut ground occur, particularly where the depth of cut varies across the building platforms. Further confirmation of available bearing pressures will be addressed at the time of post earthworks soil testing and will be presented in the Geotechnical Completion Report.
- To accommodate the liquefaction potential on the site, TC2 foundations are anticipated for residential development in the Eastern Block and Hybrid TC2/TC3 foundations will are anticipated for residential development in the Western Block based on liquefaction-induced settlements presented in Section 6.4. Foundations for the solar farm development in the Northern Block should be able to sustain liquefaction-induced settlements as per Section 6.4. Foundations near the swale should also be able to sustain lateral spreading effects.
- As required by section B1/VM4[10] of the New Zealand Building Code Handbook, the following strength reduction factors must be applied to all recommended geotechnical ultimate soil capacities in conjunction with their use in factored design load cases:
  - 0.8 for load combinations involving earthquake overstrength;
  - 0.5 for all other load combinations.

8 SAFETY IN DESIGN

The design landform requires site excavations that may include geotechnical works such as undercuts, temporary excavations, fill batters. Exposure to these works forms a significant safety risk for contractors and inspectors / testers.

In conducting our scope of work, we have considered and addressed Safety in Design (SiD) aspects relevant to our understanding of the proposed design and construction work. SiD must consider the construction, operation, maintenance, and ultimate demolition phases of the relevant works.

It is noted that CMW are focussed on design aspects, and whilst we have attempted to be comprehensive in our assessment, it is the Contractors responsibility to cover construction related risks in a more comprehensive manner (being the competent party in that respect).

Our SiD risk assessment is presented in ***Appendix H***. This risk assessment must be communicated with all affected parties involved with the project and dealt with through specific on-site risk assessment plans.

9 FURTHER WORK

- Additional investigation, laboratory testing and analysis to further define geohazards such as liquefaction, lateral spreading and static settlements below and near housing areas and infrastructure.
- Geotechnical analysis and reporting suitable to support future project stages including a subdivision consent application, and detailed design to support building consent applications.
- Preparation of an earthworks specification, followed by observations, testing, certification and preparation of a Geotechnical Completion Report for the proposed development.

[10] Ministry of Business, Innovation and Employment (2019) *Acceptable Solutions and Verification Methods for NZ Building Code Clause B1 Structure,* B1/VM4, Amendment 19.

10 CLOSURE

Additional important information regarding the use of your CMW report is provided in the '*Using your CMW Report*' document attached to this report.

This report has been prepared for use by Matamata Development Limited C/O Maven Associates in relation to the Proposed Residential Subdivision and Solar Farm Station Road, Matamata project in accordance with the scope, proposed uses and limitations described in the report. Should you have further questions relating to the use of your report please do not hesitate to contact us.

Where a party other than Matamata Development Limited C/O Maven Associates seeks to rely upon or otherwise use this report, the consent of CMW should be sought prior to any such use. CMW can then advise whether the report and its contents are suitable for the intended use by the other party.

USING YOUR CMW GEOTECHNICAL REPORT

Geotechnical reporting relies on interpretation of facts and collected information using experience, professional judgement, and opinion. As such it generally has a level of uncertainty attached to it, which is often far less exact than other engineering design disciplines. The notes below provide general advice on what can be reasonably expected from your report and the inherent limitations of a geotechnical report.

Preparation of your report

Your geotechnical report has been written for your use on your project. The contents of your report may not meet the needs of others who may have different objectives or requirements. The report has been prepared using generally accepted Geotechnical Engineering and Engineering Geology practices and procedures. The opinions and conclusions reached in your report are made in accordance with these accepted principles. Specific items of geotechnical or geological importance are highlighted in the report.

In producing your report, we have relied on the information which is referenced or summarised in the report. If further information becomes available or the nature of your project changes, then the findings in this report may no longer be appropriate. In such cases the report must be reviewed, and any necessary changes must be made by us.

Your geotechnical report is based on your project's requirements

Your geotechnical report has been developed based on your specific project requirements and only applies to the site in this report. Project requirements could include the type of works being undertaken; project locality, size and configuration; the location of any structures on or around the site; the presence of underground utilities; proposed design methodology; the duration or design life of the works; and construction method and/or sequencing.

The information or advice in your geotechnical report should not be applied to any other project given the intrinsic differences between different projects and site locations. Similarly geotechnical information, data and conclusions from other sites and projects may not be relevant or appropriate for your project.

Interpretation of geotechnical data

Site investigations identify subsurface conditions at discrete locations. Additional geotechnical information (e.g. literature and external data source review, laboratory testing etc) are interpreted by Geologists or Engineers to provide an opinion about a site specific ground models, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist due to the variability of geological environments. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. Interpretation of factual data can be influenced by design and/or construction methods. Where these methods change review of the interpretation in the report may be required.

Subsurface conditions can change

Subsurface conditions are created by natural processes and then can be altered anthropically or over time. For example, groundwater levels can vary with time or activities adjacent to your site, fill may be placed on a site, or the consistency of near surface conditions might be susceptible to seasonal changes. The report is based on conditions which existed at the time of investigation. It is important to confirm whether conditions may have changed, particularly when large periods of time have elapsed since the investigations were performed.

Interpretation and use by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical report. To help avoid misinterpretations, it is important to retain the assistance of CMW to work with other project design professionals who are affected by the contents of your report. CMW staff can explain the report implications to design professionals and then review design plans and specifications to see that they have correctly incorporated the findings of this report.

Your report's recommendations require confirmation during construction

Your report is based on site conditions as revealed through selective point sampling. Engineering judgement is then applied to assess how indicative of actual conditions throughout an area the point sampling might be. Any assumptions made cannot be substantiated until construction is complete. For this reason, you should retain geotechnical services throughout the construction stage, to identify variances from previous assumption, conduct additional tests if required and recommend solutions to problems encountered on site.

A Geotechnical Engineer, who is fully familiar with the site and the background information, can assess whether the report's recommendations remain valid and whether changes should be considered as the project develops. An unfamiliar party using this report increases the risk that the report will be misinterpreted.

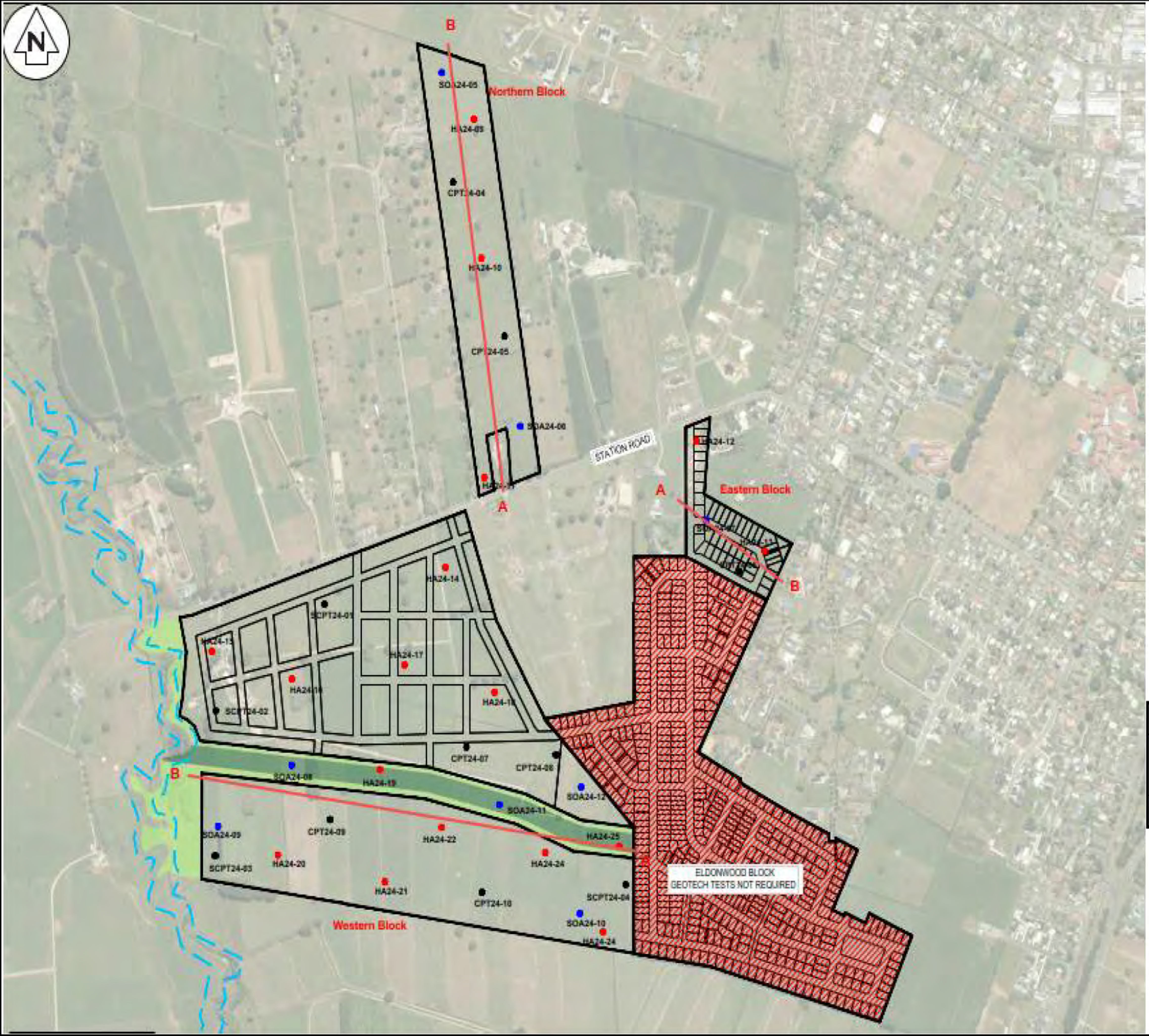
Environmental Matters Are Not Covered

Unless specifically discussed in your report environmental matters are not covered by a CMW Geotechnical Report. Environmental matters might include the level of contaminants present of the site covered by this report, potential uses or treatment of contaminated materials or the disposal of contaminated materials. These matters can be complex and are often governed by specific legislation.

The personnel, equipment, and techniques used to perform an environmental study can differ significantly from those used in this report. For that reason, our report does not provide environmental recommendations. Unanticipated subsurface environmental problems can have large consequences for your site. If you have not obtained your own environmental information about the project site, ask your CMW contact about how to find environmental risk-management guidance.

APPENDIX A: DRAWINGS

Title	Reference No.	Date	Revision
Site Investigation Plan	1	05/07/2024	0
Cross Section A – Eastern Block	2	05/07/2024	0
Cross Section B – Northern Block	3	05/07/2024	0
Cross Section C – Western Block	4	05/07/2024	0



Legend

(thick line)	PR BOUNDARY
(thin line)	PROP LOT BDY
(red dot)	HAND AUGER
(blue dot)	SOAKAGE HOLE
(black dot)	CPT

CMW Geosciences
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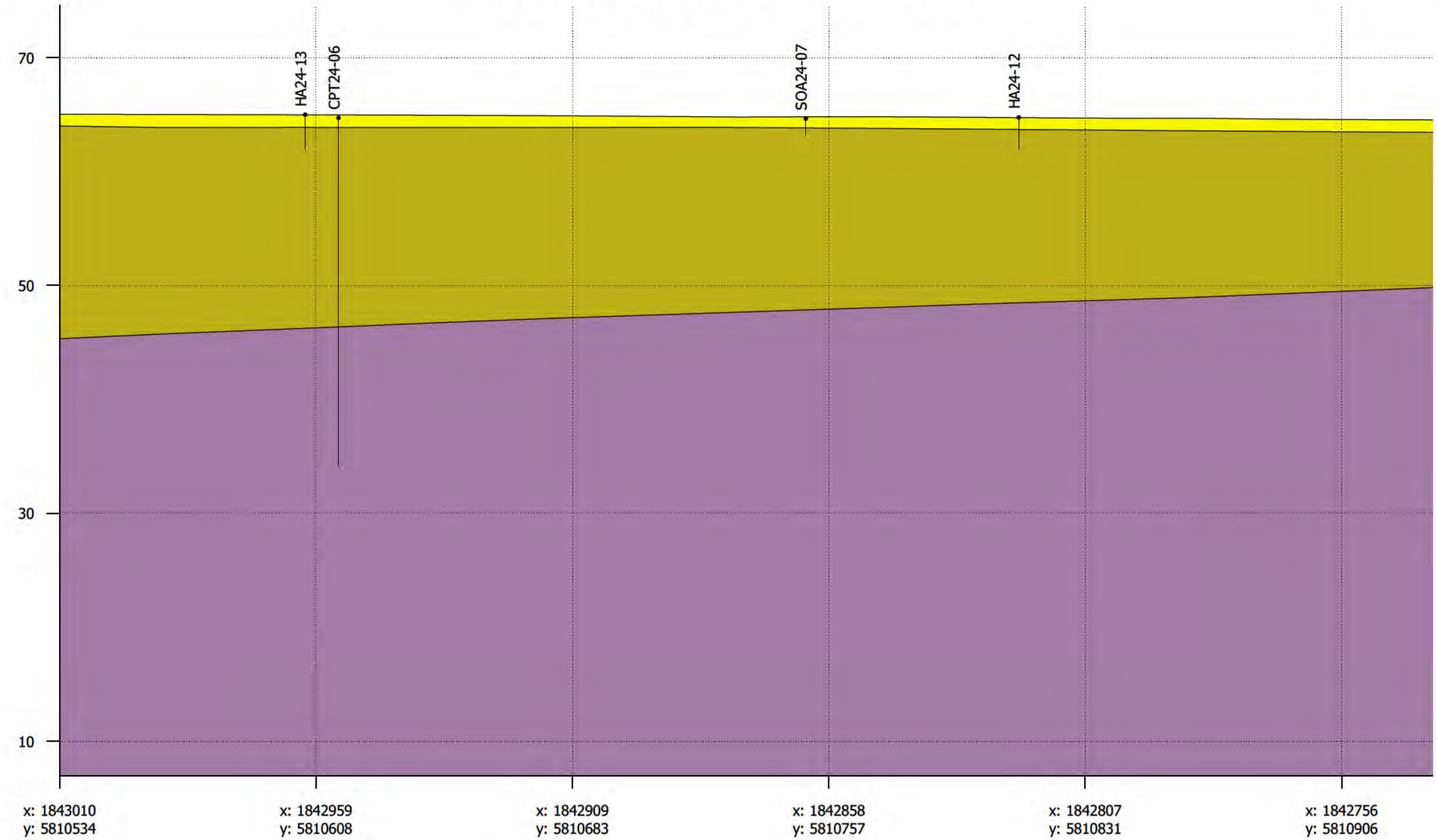
CLIENT:	MATAMATA DEVELOPMENT LTD
PROJECT:	STATION ROAD, MATAMATA
TITLE:	SITE INVESTIGATION PLAN

Cross Section - Eastern Block



Legend

GM

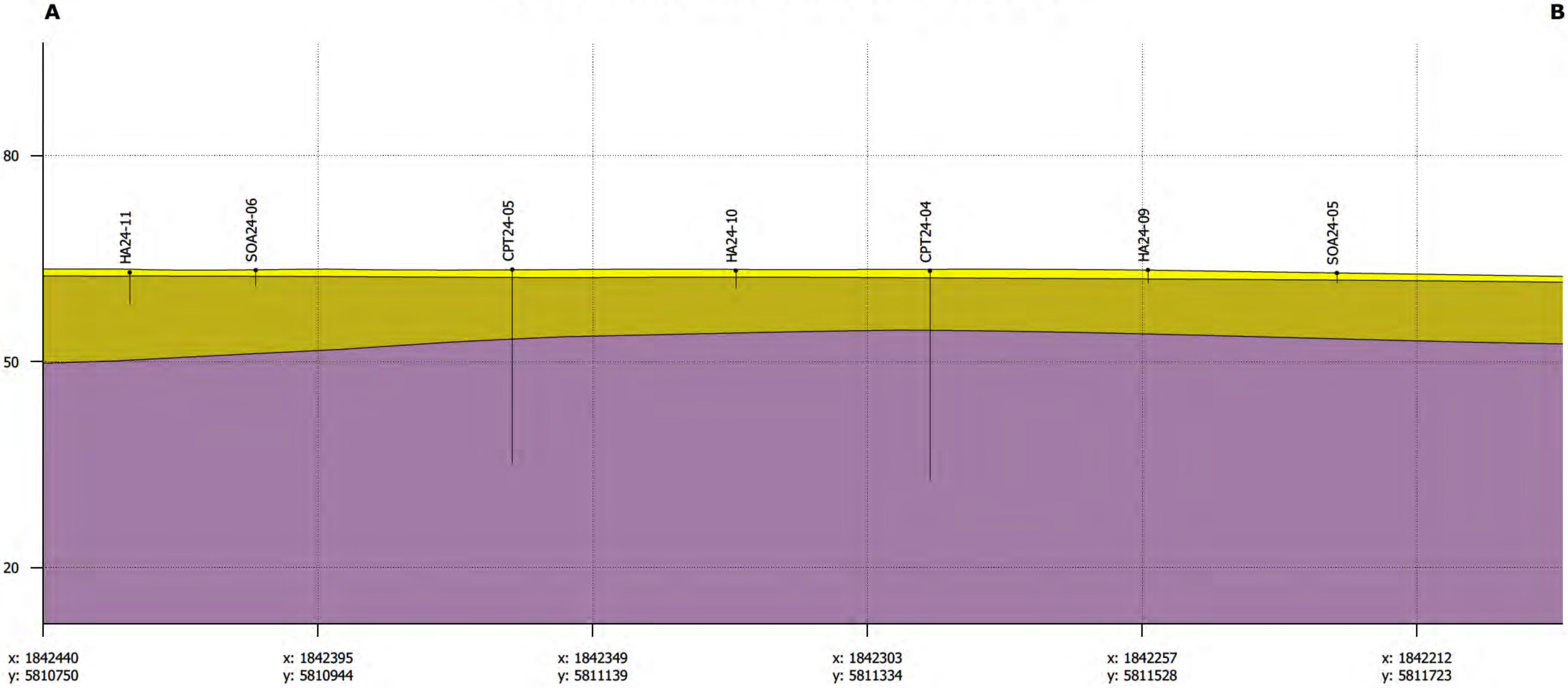
- Stiff to Very Stiff SILT (Hinuera Formation)
- Dense to Very Dense SAND with interbedded SILT (Hinuera Formation)
- Very Dense Silty SAND (Walton Subgroup)

Scale: 1:1,500

Vertical exaggeration: 4x

0m 50m

Cross Section - Northern Block



Legend

GM

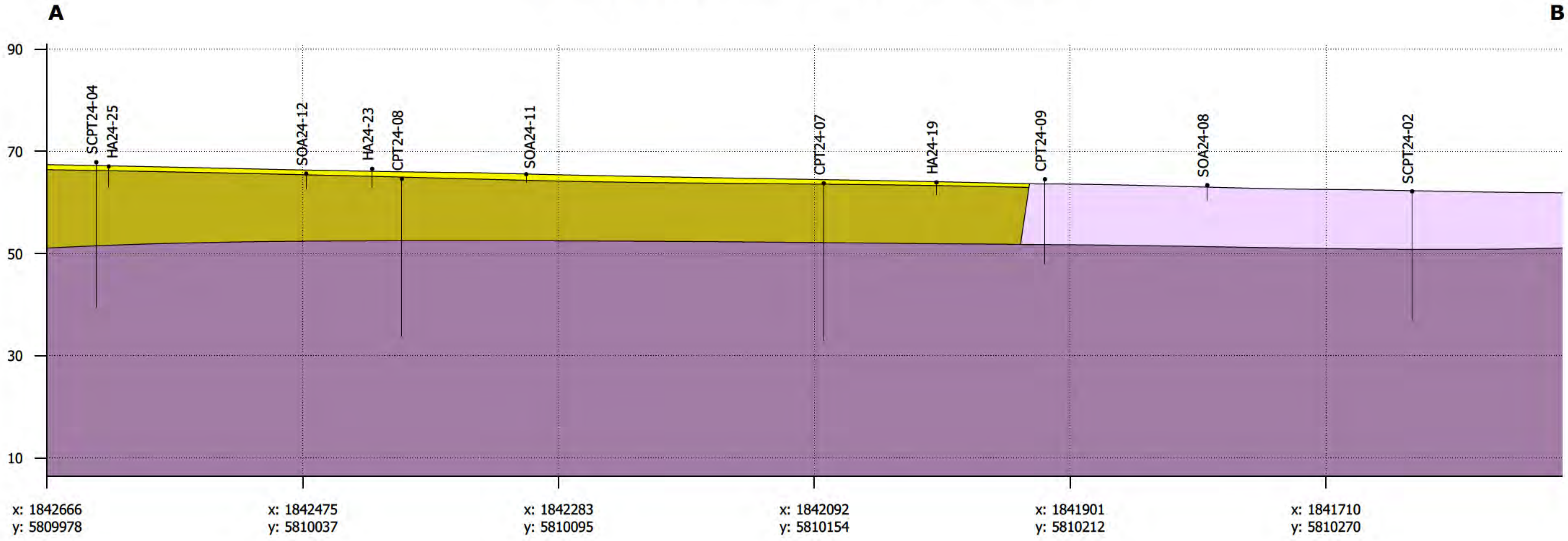
- Stiff to Very Stiff SILT (Hinuera Formation)
- Dense to Very Dense SAND with interbedded SILT (Hinuera Formation)
- Very Stiff to Hard SILT/CLAY (Walton Subgroup)
- Very Dense Silty SAND (Walton Subgroup)

Scale: 1:3,000

Vertical exaggeration: 5x

0m 200m

Cross Section - Western Block



Legend

GM

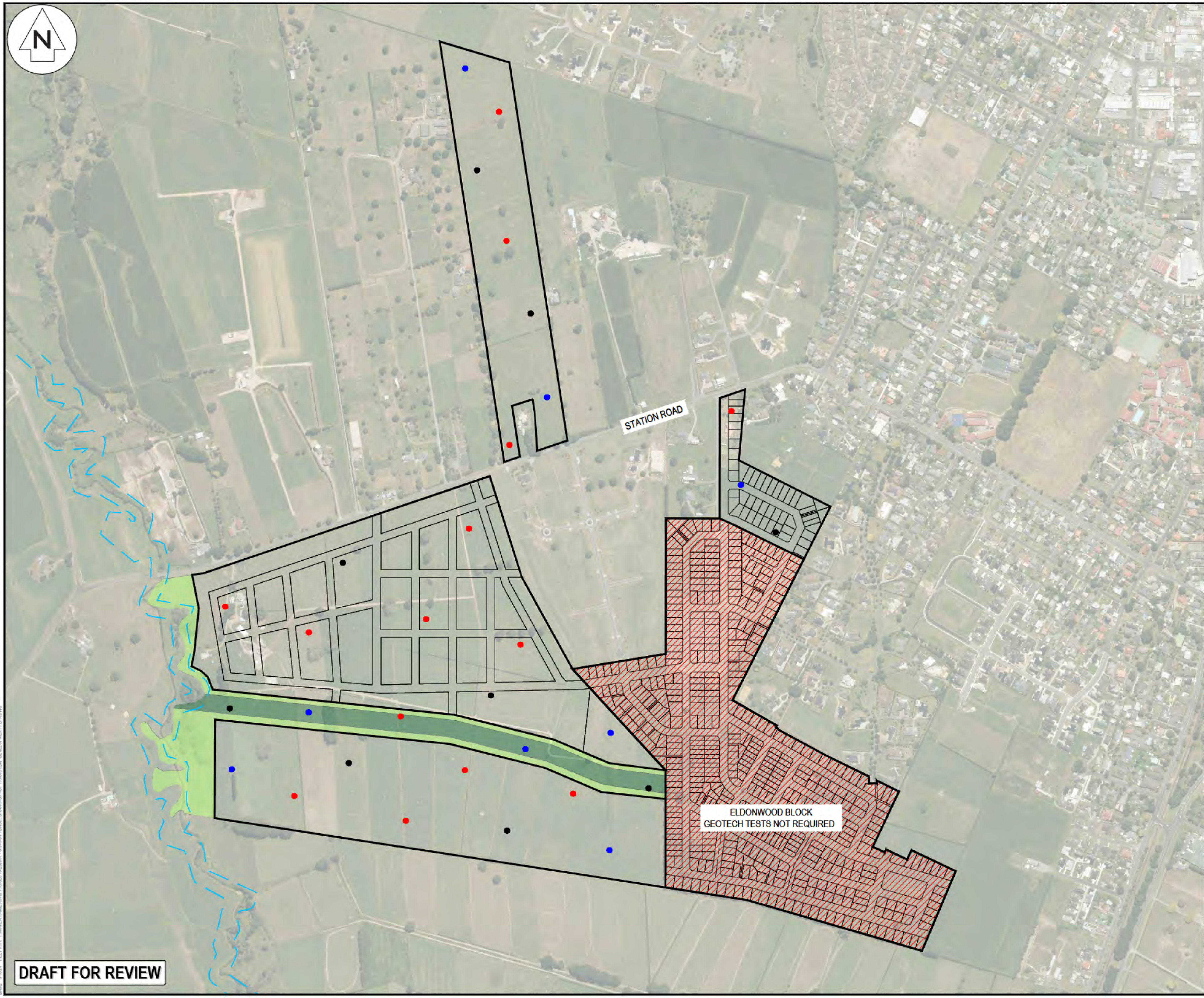
- Stiff to Very Stiff SILT (Hinuera Formation)
- Dense to Very Dense SAND with interbedded SILT (Hinuera Formation)
- Very Stiff to Hard SILT/CLAY (Walton Subgroup)
- Very Dense Silty SAND (Walton Subgroup)

Scale: 1:3,200

Vertical exaggeration: 4x

0m 200m

APPENDIX B: MAVEN ASSOCIATES DEVELOPMENT PLAN



Legend

- PR BOUNDARY
- PROP LOT BDY
- HAND AUGER
- SOAKAGE HOLE
- CPT

Ground investigation locations superseded. Refer to CMW Drawing 01.

Rev	Description	By	Date
B	DRAFT	MKS	05/2024
A	DRAFT	MKS	05/2024

	By	Date
Survey	LINZ	02/2024
Design	MKS	05/2024
Drawn	MKS	05/2024
Checked	DJM	05/2024

MAVEN
Maven Associates
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5 Owens Road, Epsom
Auckland 1023

Project
ELDONWOOD MATAMATA FOR MATAMATA DEVELOPMENTS

Title
PROPOSED GEOTECH WORKS SCOPE

Project no.	289001
Scale	1:8000 @ A3
Cad file	DEVELOPMENT PLAN.DWG
Drawing no.	SCOPE
Rev	B

APPENDIX C: HAND AUGER BOREHOLE LOGS

HAND AUGER BOREHOLE LOG - HA24-09

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 05/06/2024
Borehole Location: Refer to site plan | Logged by: PM | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position:
Projection:
Datum:
Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
				0.0–0.5	OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
	0.6	Peak = UTP		0.5–0.9	ML: SILT: grey mottled orange. Low plasticity. (Hinuera Formation)	M	H
				0.9–1.2	SM: Silty fine to medium SAND: light brownish grey. Poorly graded, sub rounded. (Hinuera Formation)	M to W	D
				1.2–1.7	SP: Fine SAND: light grey streaked orange. Poorly graded, sub rounded. (Hinuera Formation)	M to W / W	MD to D
05-06-2024 (1.8 m)				1.7–1.75	ML: SILT: grey. Low plasticity. (Hinuera Formation)	W	St
				1.75–1.9	SW: Fine to coarse SAND: grey. Well graded, sub rounded. (Hinuera Formation)	S	D

Borehole terminated at 1.9 m

Dynamic Cone Penetrometer (Blows/100mm):

Depth interval	Blows
1	2
2	2
3	2
4	3
5	4
6	5
7	4
8	5
9	6
10	10
11	9
12	13
13	5
14	9
15	13
16	13
17	12
18	9
19	8
20	8
21	7
22	9
23	8
24	5
25	8
26	6
27	7
28	8
29	7
30	6
31	6
32	5
33	4
34	4
35	7
36	12
37	10
38	10
39	13
40	5
41	4
42	4
43	10
44	12
45	10
46	10
47	14
48	10
49	12

Termination Reason: No retrieval.
Shear Vane No: 2087 DCP No: 25
Remarks: Groundwater encountered at 1.8m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-10

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 05/06/2024
Borehole Location: Refer to site plan   Logged by: WD   Checked by: DM   Scale: 1:25   Sheet 1 of 1

CMW Geosciences
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Position: Projection:
Datum: Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Material Description Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/ geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)	Moisture Condition	Consistency/ Relative Density
				0.0–0.4	OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
	0.6	Peak = 178kPa Residual = 39kPa		0.4–0.7	ML: Fine to medium Sandy SILT: light grey streaked orange brown. Low plasticity. Sensitive. (Hinuera Formation)	M	St to VSt
	0.9 1.3	Peak = 192kPa Residual = 36kPa Peak = 92kPa Residual = 44kPa		0.7–1.4	CH: Silty CLAY: with trace fine sand; light grey streaked dark orange brown. High plasticity. Moderately sensitive. (Hinuera Formation) *... at 1.00m, becoming light grey.*	M	St to VSt
				1.4–1.7	SW: Fine to coarse SAND: with minor silt; light grey. Well graded. (Hinuera Formation)	M	MD
05-06-2024 (2.0 m)	1.8	Peak = 111kPa Residual = 42kPa		1.7–2.0	ML: SILT: with some fine to coarse sand; light grey. Low plasticity. Moderately sensitive. Dilatant. (Hinuera Formation)	W	VSt
				2.0–2.5	SW: Fine to coarse SAND: with some silt; grey. Well graded. (Hinuera Formation) *... at 2.20m, becoming greyish brown, poor retention.* *... at 2.40m, becoming brown fine to medium sand.*	S	MD to D
				2.5	Borehole terminated at 2.5 m		

Dynamic Cone Penetrometer (Blows/100mm)

Depth (m)	Blows
0.0–0.1	1
0.1–0.2	2
0.2–0.3	2
0.3–0.4	3
0.4–0.5	3
0.5–0.6	3
0.6–0.7	2
0.7–0.8	1
0.8–0.9	1
0.9–1.0	2
1.0–1.1	5
1.1–1.2	5
1.2–1.3	4
1.3–1.4	4
1.4–1.5	4
1.5–1.6	5
1.6–1.7	4
1.7–1.8	3
1.8–1.9	4
1.9–2.0	5
2.0–2.1	10
2.1–2.2	8
2.2–2.3	7
2.3–2.4	5
2.4–2.5	5
2.5–2.6	3
2.6–2.7	4
2.7–2.8	4
2.8–2.9	4
2.9–3.0	5
3.0–3.1	7
3.1–3.2	5
3.2–3.3	4
3.3–3.4	4
3.4–3.5	10
3.5–3.6	8
3.6–3.7	11
3.7–3.8	10
3.8–3.9	10
3.9–4.0	12
4.0–4.1	12
4.1–4.2	13
4.2–4.3	9
4.3–4.4	8
4.4–4.5	10
4.5–4.6	9
4.6–4.7	10
4.7–4.8	12

Termination Reason: Hole collapse/no retrieval
Shear Vane No: 2560   DCP No: 34
Remarks: Groundwater encountered at 2.0m.

HAND AUGER BOREHOLE LOG - HA24-11

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 05/06/2024
Borehole Location: Refer to site plan   Logged by: WD   Checked by: DM   Scale: 1:25   Sheet 1 of 1

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Position: Projection:
Datum: Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
				0.0	OL: Organic SILT: brown. No plasticity. (Topsoil)	M	
	0.3	Peak = 120kPa Residual = 25kPa			ML: SILT: orange brown. Low plasticity. Moderately sensitive to sensitive. (Hinuera Formation)	M	St to VSt
	0.6	Peak = 78kPa Residual = 17kPa					
	0.9	Peak = 58kPa Residual = 19kPa		1			
					SM: Silty Fine to medium SAND: orange brown. Well graded. (Hinuera Formation)	M to W	
					SW: Fine to coarse SAND: with trace fine gravel; greyish brown. Well graded. Gravel; subangular. (Hinuera Formation)	M	MD to D
					... at 1.40m, thin lens of silty fine sand.		
				2	*... at 2.20m, becoming brownish grey.*		
					... from 2.50m to 2.60m, mottled black.		
					... at 2.80m, becoming dark orange brown.		
				3	*... at 3.10m, becoming greyish brown with grey.*		
					... at 3.80m, with trace silt.	W	
05-06-2024				4	*... at 3.90m, becoming dark brown with brownish grey.*		
					... at 4.40m, becoming dark reddish brown, with some silt.	S	
					Borehole terminated at 4.6 m		
				5			

Dynamic Cone Penetrometer (Blows/100mm)

Depth interval (m)	Blows
0.0–0.1	2
0.1–0.2	3
0.2–0.3	1
0.3–0.4	3
0.4–0.5	2
0.5–0.6	1
0.6–0.7	2
0.7–0.8	1
0.8–0.9	1
0.9–1.0	2
1.0–1.1	3
1.1–1.2	3
1.2–1.3	4
1.3–1.4	3
1.4–1.5	5
1.5–1.6	6
1.6–1.7	4
1.7–1.8	3
1.8–1.9	2
1.9–2.0	6
2.0–2.1	6
2.1–2.2	5
2.2–2.3	6
2.3–2.4	8
2.4–2.5	7
2.5–2.6	8
2.6–2.7	7
2.7–2.8	7
2.8–2.9	5
2.9–3.0	5
3.0–3.1	8
3.1–3.2	8
3.2–3.3	8
3.3–3.4	9
3.4–3.5	9
3.5–3.6	9
3.6–3.7	13
3.7–3.8	15
3.8–3.9	14
3.9–4.0	15
4.0–4.1	20

Termination Reason: Hole collapse
Shear Vane No: 2560   DCP No: 34
Remarks: Groundwater encountered at 4.2m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-12

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 05/06/2024
Borehole Location: Refer site plan. Logged by: PM Checked by: DM Scale: 1:25 Sheet 1 of 1

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Position: Projection:
Datum: Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Material Description Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)	Moisture Condition	Consistency/ Relative Density
					OL: Organic SILT: with some medium gravel; dark brown. No plasticity. (Topsoil)	M	
					SW: Fine to coarse SAND: light brown. Well graded, sub angular. (Possible Fill)	M	MD
	0.4	Peak = 129kPa Residual = 26kPa			ML: SILT: with minor fine sand; brownish orange. Low plasticity. Sensitive. (Hinuera Formation)	M	VSt to H
	0.7	Peak = 167kPa Residual = 29kPa					
	1.0	Peak = UTP		1			
					SM: Silty fine to coarse SAND: light yellowish brown. Well graded, sub rounded. (Hinuera Formation)	M / W	MD
	1.8	Peak = 205kPa			ML: SILT: grey mottled orange. Low plasticity. (Hinuera Formation)	W	H
				2	SW: Fine to coarse SAND: light brown mottled grey and streaked dark red. Well graded, sub rounded. (Hinuera Formation)	W	L to MD
					... *at 2.10m, Becomes grey.*		
05-06-2024						S	
					Borehole terminated at 2.7 m		
				3			
				4			
				5			

Dynamic Cone Penetrometer (Blows/100mm): 4, 7, 7, 6, 2, 2, 2, 2, 2, 2, 4, 5, 7, 5, 5, 3, 3, 3, 2, 3, 2, 5, 5, 5, 4, 3, 3, 4, 3, 4, 5, 5, 5, 6, 6, 8, 9, 9, 10, 12, 13, 13, 13, 10, 13, 10, 10, 9, 10

Termination Reason: Hole Collapse
Shear Vane No: 2087 DCP No: 25
Remarks: Groundwater encountered at 2.6m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-13

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 05/06/2024
Borehole Location: Refer to site plan | Logged by: WD | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position: | Projection: | Datum: | Survey Source: Site Plan

Samples & Insitu Tests

Depth	Type & Results
0.3	Peak = 167kPa Residual = 28kPa
0.6	Peak = 133kPa Residual = 22kPa
0.9	Peak = 111kPa Residual = 39kPa
1.4	Peak = 108kPa Residual = 53kPa

Material Description
Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit)
Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)

Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
0.0 – 0.25	OL: Organic SILT: brown. No plasticity. (Topsoil)	D to M	
0.25 – 0.9	ML: SILT: orange brown. Low plasticity. Moderately sensitive to sensitive. (Hinuera Formation) *... at 0.60m, becoming light orange brown.*	D to M	VSt
0.9 – 1.4	SM: Silty Fine to medium SAND: with trace fine pumiceous gravel; light orange brown. Well graded. (Hinuera Formation) *... at 1.20m, becoming fine sand.*	D to M / M	MD
1.4 – 1.52	CH: Silty CLAY: light grey. High plasticity. Moderately sensitive. (Hinuera Formation)	M	VSt
1.52 – 1.7	SM: Silty Fine SAND: grey. Poorly graded. (Hinuera Formation)	M	MD
1.7 – 2.1	SW: Fine to medium SAND: with some silt and minor fine gravel; brownish grey with grey. Well graded. Gravel; subangular. (Hinuera Formation)	M	MD
2.1 – 3.0	SW: Fine to coarse SAND: with some silt and trace fine gravel; grey. Well graded. Gravel; subangular. (Hinuera Formation) *... at 2.60m, poor retention.*	M / W / S	MD

Borehole terminated at 3.0 m

Groundwater: 05-06-2024 (at 2.6 m)

Dynamic Cone Penetrometer (Blows/100mm)

Depth (m)	Blows/100mm
0.0–0.1	2
0.1–0.2	2
0.2–0.3	2
0.3–0.4	2
0.4–0.5	1
0.5–0.6	2
0.6–0.7	2
0.7–0.8	2
0.8–0.9	3
0.9–1.0	5
1.0–1.1	7
1.1–1.2	7
1.2–1.3	4
1.3–1.4	4
1.4–1.5	4
1.5–1.6	4
1.6–1.7	4
1.7–1.8	4
1.8–1.9	3
1.9–2.0	4
2.0–2.1	6
2.1–2.2	4
2.2–2.3	3
2.3–2.4	3
2.4–2.5	4
2.5–2.6	4
2.6–2.7	4
2.7–2.8	3
2.8–2.9	3
2.9–3.0	4
3.0–3.1	4
3.1–3.2	5
3.2–3.3	4
3.3–3.4	4
3.4–3.5	6
3.5–3.6	7
3.6–3.7	4
3.7–3.8	6
3.8–3.9	5
3.9–4.0	5
4.0–4.1	6
4.1–4.2	6
4.2–4.3	7
4.3–4.4	5
4.4–4.5	7
4.5–4.6	8
4.6–4.7	7
4.7–4.8	5
4.8–4.9	9

Termination Reason: Hole collapse/no retrieval
Shear Vane No: 2560 | DCP No: 34
Remarks: Groundwater encountered at 2.6m.

HAND AUGER BOREHOLE LOG - HA24-14

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 06/06/2024
Borehole Location: Refer to site plan | Logged by: PM | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

CMW Geosciences — Great People | Practical Solutions

Position:
Projection:
Datum:
Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Graphic Log	Material Description	Moisture Condition	Consistency/ Relative Density
						OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
						SM: Silty fine to medium SAND: light grey mottled dark orange. Poorly graded, rub rounded. (Hinuera Formation)	M	L to MD
						SM: Silty fine SAND: brownish orange mottled dark brown. Poorly graded, sub rounded. (Hinuera Formation)	M	MD
				1		SW: Fine to medium SAND: grey. Well graded, sub rounded. (Hinuera Formation)	W	MD to D
06-06-2024						*... at 1.50m, Becomes blueish grey.*	S	MD to D
						Borehole terminated at 1.7 m		

Material Description key: Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/ geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)

Depth scale (m): 1, 2, 3, 4, 5

Dynamic Cone Penetrometer (Blows/100mm), scale 5, 10, 15, from top down:

1, 3, 2, 4, 7, 5, 3, 3, 3, 4, 4, 5, 10, 11, 11, 8, 3, 3, 3, 4, 9, 8, 8, 9, 8, 6, 5, 2, 2, 5, 8, 8, 7, 7, 10, 11, 10, 9, 10, 10, 12, 11, 9, 12, 10, 10, 11, 9, 13

Termination Reason: Hole Collapse
Shear Vane No:
DCP No: 34
Remarks: Groundwater encountered at 1.6m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-15

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 05/06/2024
Borehole Location: Refer to site plan | Logged by: PM | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position: | Projection: | Datum: | Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
				0.0–0.3	OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
	0.4	Peak = 59kPa Residual = 15kPa		0.3–1.1	ML: SILT: light brown. Low plasticity. Moderately sensitive to sensitive. (Hinuera Formation) ... at 0.90m, Becoming a light greyish brown.	M	St to VSt
	0.7	Peak = 59kPa Residual = 15kPa					
	1.0	Peak = 105kPa Residual = 15kPa		1			
				1.1–2.4	SM: Silty fine SAND: light brown. Poorly graded, sub rounded. Dilatant. (Hinuera Formation) ... at 1.40m, Becoming a light grey streaked yellow.	M / W / M to W	MD
	2.5	Peak = 167kPa Residual = 59kPa		2.4–2.5	ML: SILT: grey streaked yellow. Low plasticity. Moderately sensitive. (Hinuera Formation)	M to W	VSt
	2.7	Peak = UTP		2.5–3.0	ML: Clayey SILT: brown. Low plasticity. (Walton Subgroup)	M to W / M	H
	3.0	Peak = UTP		3	Borehole terminated at 3.0 m		

Dynamic Cone Penetrometer (Blows/100mm): 2, 2, 1, 1, 2, 2, 1, 1, 2, 1, 2, 4, 4, 3, 4, 4, 5, 4, 4, 4, 4, 5, 6, 10, 4, 9, 10, 12, 17, 20

Termination Reason: Refusal on hard material.
Shear Vane No: 2087 DCP No: 25
Remarks: Groundwater not encountered.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-16

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 06/06/2024
Borehole Location: Refer to site plan   Logged by: WD   Checked by: DM   Scale: 1:25   Sheet 1 of 1

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Position: Projection:
Datum: Survey Source: Site Plan

Groundwater	Depth	Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density	Dynamic Cone Penetrometer (Blows/100mm)
					OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M		
	0.3	Peak = 100kPa Residual = 23kPa			ML: SILT: orange brown. Low plasticity. Sensitive. (Walton Subgroup) ... at 0.40m, becoming light orange brown, with some clay.	M	VSt to H	
	0.6	Peak = 160kPa Residual = 30kPa			CH: Silty CLAY: light orange brown. High plasticity. Sensitive. (Walton Subgroup)	M	VSt to H	
	0.9	Peak = 123kPa Residual = 28kPa		1	... at 1.00m, becoming light orange grey.	M	VSt to H	
					ML: SILT: light orange grey. Low plasticity. (Walton Subgroup)	M	VSt to H	
	1.3	Peak = UTP			SM: Silty Fine SAND: light brownish grey. Poorly graded. (Walton Subgroup)	M	VSt to H	
	1.6	Peak = 150kPa Residual = 28kPa			CH: Silty CLAY: light brownish grey. High plasticity. Sensitive. (Walton Subgroup)	W	St to VSt	
	1.9	Peak = 85kPa Residual = 20kPa		2	... at 2.00m, becoming light grey, with trace fine sand. ... at 2.10m, becoming light brown grey. ... at 2.20m, becoming light greyish brown.	W	St to VSt	
	2.3	Peak = UTP			ML: SILT: with some clay; greyish brown. Low plasticity. (Walton Subgroup)	D to M	VSt to H	
	2.6	Peak = 175+			CH: Silty CLAY: greyish brown. High plasticity. Moderately sensitive. (Walton Subgroup)	M	VSt to H	
	2.9	Peak = 160kPa Residual = 43kPa		3		M to W	VSt to H	
	3.3	Peak = 158kPa Residual = 45kPa				M to W	VSt to H	
	3.6	Peak = 125kPa Residual = 33kPa				M to W	VSt to H	
	3.9	Peak = 125kPa Residual = 35kPa		4	... at 3.80m, becoming light orange brown.	M to W	VSt to H	
	4.3	Peak = 175+			ML: Clayey SILT: orange brown mottled black. Low plasticity. (Walton Subgroup)	M to W	VSt to H	
	4.6	Peak = 50kPa Residual = 25kPa			CH: Silty CLAY: light greyish brown. High plasticity. Insensitive to moderately sensitive. (Walton Subgroup)	M to W	St	
	4.9	Peak = 60kPa Residual = 50kPa		5	Borehole terminated at 5.0 m	M to W	St	

Termination Reason: Target Depth Reached
Shear Vane No: 3434   DCP No:
Remarks: Groundwater not encountered.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-17

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 06/06/2024
Borehole Location: Refer to site plan | Logged by: WD | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position:
Projection:
Datum:
Survey Source: Site Plan

Groundwater	Samples Depth	Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density	DCP (Blows/100mm)
				0.0–0.4	OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M		2, 2
	0.3	Peak = 63kPa Residual = 20kPa		0.4–2.2	ML: SILT: orange brown. Low plasticity. (Hinuera Formation)	M	St to VSt	1, 2, 2, 2, 2, 1, 1, 3
	0.6	Peak = 108kPa Residual = 25kPa			... at 0.70m, with some clay.			
	0.9	Peak = 160kPa Residual = 35kPa			... at 1.00m, with trace fine sand.			
				1.1–3.0	SW: Fine to coarse SAND: minor fine gravel. orange brown. Well graded. Gravel; black, subangular. (Hinuera Formation)	M	D to VD	6, 8, 13, 17, 24, 22
					... at 1.30m, becoming dark orange brown.			
					... at 1.40m, becoming orange brown with yellow grey.			
					... at 1.60m, becoming dark reddish brown, with some fine gravel; highly iron stained.			
					... at 2.00m, becoming brownish grey with grey.			
					... at 2.10m, becoming dark reddish brown.			
					... at 2.30m, becoming dark reddish dark brown, with trace medium gravel.			
06-06-2024				2.6		W		
					... at 2.70m, with some silt.	S		
	3.1	Peak = 158kPa Residual = 60kPa		3.0–3.8	ML: SILT: light grey. Low plasticity. Moderately sensitive. (Hinuera Formation)	S	VSt to H	
	3.7	Peak = UTP						
				3.8	Borehole terminated at 3.8 m			

Termination Reason: No retrieval
Shear Vane No: 3434 DCP No: 34
Remarks: Groundwater encountered at 2.6m.

HAND AUGER BOREHOLE LOG - HA24-18

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 06/06/2024
Borehole Location: Refer to site plan. Logged by: PM Checked by: DM Scale: 1:25 Sheet 1 of 1

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Position:
Projection:
Datum:
Survey Source: Site Plan

Groundwater	Samples & Insitu Tests – Depth	Samples & Insitu Tests – Type & Results	RL (m)	Depth (m)	Material Description Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)	Moisture Condition	Consistency/ Relative Density
				0.0	OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
	0.3	Peak = 83kPa Residual = 19kPa		0.2	ML: Sandy SILT: greyish brown. Low plasticity. Moderately sensitive. (Hinuera Formation)	M to W	St
	0.6	Peak = 97kPa Residual = 28kPa		0.5	ML: SILT: light grey streaked orange. Low plasticity. Moderately sensitive. (Hinuera Formation)	M to W	St
				0.8	SM: Silty fine SAND: light grey streaked yellow. Poorly graded, sub rounded. (Hinuera Formation)	M to W	MD to D
06-06-2024 (1.6 m)				1.4	SW: Fine to medium SAND: grey. Well graded, sub rounded. (Hinuera Formation)	W / S	D
				1.7	Borehole terminated at 1.7 m		

Dynamic Cone Penetrometer (Blows/100mm): 2, 2, 2, 2, 2, 2, 2, 3, 5, 5, 7, 9, 7, 9, 7, 10, 7, 7, 7, 8, 7, 7, 8, 9, 8, 5, 7, 5, 4, 5, 7, 10, 10, 12, 12, 6, 5, 10, 9, 11, 10, 12, 12, 13, 15, 16, 12, 10, 11, 12

Termination Reason: Hole Collapse
Shear Vane No: 2560 DCP No: 34
Remarks: Groundwater encountered at 1.6m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-19

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 06/06/2024
Borehole Location: Refer to site plan | Logged by: WD | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position:
Projection:
Datum:
Survey Source: Site Plan

Groundwater	Samples & Insitu Tests – Depth	Samples & Insitu Tests – Type & Results	RL (m)	Depth (m)	Graphic Log	Material Description Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)	Moisture Condition	Consistency/ Relative Density
						OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M to W	
						ML: SILT: with some clay; light brown. (Hinuera Formation)		
						SM: Silty Fine SAND: light grey streaked light orange brown. Poorly graded. (Hinuera Formation)		MD
						ML: Fine SAndy SILT: light grey streaked light orange brown. Low plasticity. (Hinuera Formation)		St
						SM: Silty Fine SAND: light grey streaked light orange brown. Poorly graded. (Hinuera Formation)	M to W	MD
						... *at 0.90m, light yellowish grey with brown grey.*		
				1		... *at 1.20m, becoming light grey.*	W	
06-06-2024						ML: SILT: light grey. Low plasticity. Dilatant. (Hinuera Formation)		VSt
						SM: Silty Fine to medium SAND: light grey. Well graded. Pumiceous. (Hinuera Formation)	S	MD to D
				2		... *at 2.00m, poor retention.*		
						ML: SILT: with some fine sand; light grey. Low plasticity. (Hinuera Formation)		H
						SM: Silty Fine SAND: light grey. Poorly graded. (Hinuera Formation)		D
						Borehole terminated at 2.4 m		
				3				
				4				
				5				

Dynamic Cone Penetrometer (Blows/100mm) — 5 10 15

1, 2, 3, 3, 4, 4, 3, 3, 5, 4, 6, 5, 4, 5, 6, 6, 5, 6, 5, 5, 8, 7, 14, 9, 13, 12, 6, 4, 5, 5, 6, 8, 8, 14, 17, 14, 13, 12, 14, 17, 14, 13, 17, 16, 20, 17, 17, 16, 15

Termination Reason: No retrieval
Shear Vane No: | DCP No: 34
Remarks: Groundwater encountered at 1.6m.

HAND AUGER BOREHOLE LOG - HA24-20

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 06/06/2024

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Borehole Location: Refer to site plan | Logged by: PM | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

Position:
Projection:
Datum:
Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
				0.0–0.3	OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
	0.4	Peak = 97kPa Residual = 19kPa		0.3–1.8	ML: SILT: light brown. Low plasticity. Sensitive. (Hinuera Formation)	M	St to VSt
	0.7	Peak = 89kPa Residual = 19kPa					
	1.0	Peak = 181kPa Residual = 28kPa					
					... at 1.30m, Becomes light greyish brown.	M to W	
	1.5	Peak = 97kPa Residual = 36kPa					
				1.8–2.2	SM: Silty fine to coarse SAND: light greyish yellow. Well graded, sub rounded to sub angular. (Hinuera Formation)		MD
	2.3	Peak = UTP		2.2–2.5	CH: Sandy CLAY: brown. High plasticity. (Walton Subgroup)	M	H
				2.5–3.4	SW: Fiine to coarse SAND: with minor clay and trace fine gravel; Brownish orange mottled black and speckled grey. Well graded sub angular. (Walton Subgroup)		D
				3.4	Borehole terminated at 3.4 m		

Dynamic Cone Penetrometer (Blows/100mm)

Depth interval (m)	Blows
0.0–0.1	2
0.1–0.2	2
0.2–0.3	2
0.3–0.4	1
0.4–0.5	1
0.5–0.6	1
0.6–0.7	2
0.7–0.8	2
0.8–0.9	2
0.9–1.0	3
1.0–1.1	4
1.1–1.2	4
1.2–1.3	2
1.3–1.4	3
1.4–1.5	3
1.5–1.6	3
1.6–1.7	3
1.7–1.8	3
1.8–1.9	3
1.9–2.0	4
2.0–2.1	4
2.1–2.2	5
2.2–2.3	6
2.3–2.4	7
2.4–2.5	7
2.5–2.6	10
2.6–2.7	13
2.7–2.8	12
2.8–2.9	12
2.9–3.0	13
3.0–3.1	14
3.1–3.2	12
3.2–3.3	16
3.3–3.4	15
3.4–3.5	18
3.5–3.6	13
3.6–3.7	17
3.7–3.8	20

Termination Reason: Hard Material.
Shear Vane No: 2560 DCP No: 34
Remarks: Groundwater not encountered.

This report is based on the attached field description for soil and soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-21

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 06/06/2024
Borehole Location: Refer to site plan | Logged by: PM | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position: | Projection:
Datum: | Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Material Description (Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit))	Moisture Condition	Consistency/ Relative Density
				0.0–0.7	OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
06-06-2024 (1.3 m)	0.8	Peak = 97kPa Residual = 14kPa		0.7–1.5	ML: Sandy SILT: light brownish grey. Low plasticity. Sensitive. (Hinuera Formation)	W	St
				1.5–1.8	SM: Silty fine SAND: light grey. Poorly graded, sub rounded. (Hinuera Formation)	S	MD
				1.8	Borehole terminated at 1.8 m		

Dynamic Cone Penetrometer (Blows/100mm)

Depth interval (per 100mm)	Blows
1	2
2	2
3	1
4	1
5	2
6	1
7	1
8	1
9	2
10	2
11	2
12	2
13	3
14	3
15	4
16	3
17	4
18	4
19	3
20	4
21	4
22	4
23	6
24	6
25	4
26	3
27	3
28	2
29	2
30	3
31	3
32	4
33	4
34	6
35	8
36	7
37	8
38	3
39	3
40	3
41	3
42	5
43	5
44	5
45	4
46	3
47	3
48	3
49	3

Termination Reason: Hole collapse
Shear Vane No: 2560 DCP No: 34
Remarks: Groundwater encountered at 1.3m.

This report is based on the attached field description for soil and soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-22

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 06/06/2024
Borehole Location: Refer to site plan | Logged by: WD | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position: | Projection: | Datum: | Survey Source: Site Plan

Samples & Insitu Tests

Depth	Type & Results
0.3	Peak = 108kPa Residual = 20kPa
0.6	Peak = 55kPa Residual = 13kPa
0.9	Peak = 110kPa Residual = 20kPa
2.5	Peak = 65kPa Residual = 38kPa
3.3	Peak = 50kPa Residual = 40kPa
4.0	Peak = 60kPa Residual = 33kPa

Groundwater: 06-06-2024

Material Description
Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit)
Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)

Material Description	Moisture Condition	Consistency/ Relative Density
OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
CH: Silty CLAY: brown. High plasticity. Sensitive. (Hinuera Formation)	M to W	St to VSt
ML: Clayey SILT: light brown. Low plasticity. Sensitive. (Hinuera Formation) ... *at 1.00m, becoming light grey streaked orange brown.*	W	St to VSt
SM: Silty Fine SAND: light grey. Poorly graded. (Hinuera Formation) ... *at 1.60m, poor retention.*	S	MD to D
ML: SILT: with some organic silt; light grey with greyish brown. Low plasticity. Insensitive. Dilatant. (Hinuera Formation)	S	St
SM: Silty Fine SAND: light grey. Poorly graded. (Hinuera Formation) ... *at 2.60m, poor retention.* ... *at 2.70m, with trace fine sand.*	S	MD to D
OL: Organic SILT: with some silt; dark greyish brown with light grey. Low plasticity. Insensitive. (Hinuera Formation)	S	F to St
SM: Silty Fine to medium SAND: dark grey. Well graded. (Hinuera Formation)	S	D
Borehole terminated at 4.3 m		

Dynamic Cone Penetrometer (Blows/100mm): 1, 3, 1, 2, 1, 2, 1, 1, 2, 2, 6, 5, 8, 8, 9, 6, 12, 16, 15, 6, 4, 5, 7, 8, 16, 11, 8, 9, 8, 5, 8, 6, 7, 7, 10, 10, 8, 11, 12, 11, 10, 11, 10, 13, 17, 20

Termination Reason: No retrieval
Shear Vane No: 3434 | DCP No: 34
Remarks: Groundwater encountered at 1.3m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-23

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 06/06/2024
Borehole Location: Refer to site plan | Logged by: PM | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position:
Projection:
Datum:
Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Graphic Log	Material Description Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/ geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)	Moisture Condition	Consistency/ Relative Density
						OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
	0.4	Peak = 153kPa Residual = 33kPa				ML: SILT: light greyish brown. Low plasticity. Sensitive. (Hinuera Formation)	M	St to VSt
	0.7	Peak = 89kPa Residual = 19kPa						
	1.0	Peak = 181kPa Residual = 28kPa		1		ML: Sandy SILT: light grey mottled black and streaked yellow. Low plasticity. Sensitive. (Hinuera Formation)	M	VSt
						SW: Fine to coarse SAND: brown. Well graded, sub rounded. (Hinuera Formation)	M	L to MD
				2		SM: Silty fine SAND: light brownish grey. Poorly graded, sub rounded. (Hinuera Formation) *... from 2.00m to 2.05m, Silt.* *... from 2.80m to 2.90m, Silt*	W	MD
06-06-2024				3		SW: Fine to coarse SAND : light brown. Well graded, sub rounded. (Hinuera Formation)	S	MD
						Borehole terminated at 3.6 m		
				4				
				5				

Dynamic Cone Penetrometer (Blows/100mm) (scale 5, 10, 15):

2, 2, 2, 2, 1, 1, 1, 1, 2, 2, 2, 6, 4, 4, 2, 3, 2, 3, 2, 3, 4, 4, 4, 5, 4, 5, 5, 5, 5, 4, 5, 4, 5, 5, 6, 6, 5, 6, 6, 5, 6, 6, 7, 6, 6, 5, 7, 6, 6

Termination Reason: No Retrieval.
Shear Vane No: 2560 | DCP No: 34
Remarks: Groundwater encountered at 3.5m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-24

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 04/06/2024
Borehole Location: Refer to site plan | Logged by: PM | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position: | Projection: | Datum: | Survey Source: Site Plan

Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
0.0–0.3	OL: Organic SILT: dark brown. No plasticity. (Topsoil)		
0.3–0.5	ML: SILT: brown. Low plasticity. Moderately sensitive. (Hinuera Formation)	M	St
0.5–1.5	ML: SILT: light brown streaked orange. Low plasticity. Sensitive. (Hinuera Formation) ... *at 1.10m, Becoming grey mottled orange.*	M / M to W	VSt to H
1.5–1.7	SP: Fine SAND: with some silt; grey mottled orange. Poorly graded, sub rounded. (Hinuera Formation)	M to W	L
1.7–1.8	ML: SILT: light grey. Low plasticity. Sensitive. (Hinuera Formation)	W	VSt
1.8–2.8	SM: Silty fine to medium SAND: grey. Poorly graded, sub rounded. (Hinuera Formation)	W	L to MD
2.8–3.6	SW: Fine to medium SAND: with minor silt; grey. Well graded, sub rounded. (Hinuera Formation)	W to S / S	MD
3.6	Borehole terminated at 3.6 m		

Samples & Insitu Tests

Depth	Type & Results
0.4	Peak = 74kPa Residual = 18kPa
0.6	Peak = UTP
1.0	Peak = 133kPa Residual = 18kPa
1.2	Peak = 166kPa Residual = 33kPa
1.8	Peak = 178kPa Residual = 33kPa

Groundwater: 04-06-2024 (at approx. 3.3 m)

Dynamic Cone Penetrometer (Blows/100mm): 2, 1, 1, 1, 2, 2, 1, 1, 2, 2, 2, 3, 3, 2, 4, 2, 2, 3, 2, 2, 4, 3, 2, 2, 4, 4, 4, 3, 4, 5, 5, 5, 5, 5, 5, 6, 6, 6, 6, 3, 7, 6, 8, 7, 8, 7, 7, 8, 8

Termination Reason: Hole Collapse.
Shear Vane No: 2955 DCP No: 34
Remarks: Groundwater encountered at 3.3m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - HA24-25

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 04/06/2024
Borehole Location: Refer to site plan | Logged by: WD | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position: | Projection: | Datum: | Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
				0.0	OL: Organic SILT: dark brown. No plasticity. (Topsoil)		
	0.3	Peak = 109kPa Residual = 25kPa		0.2	ML: Clayey SILT: orange brown. Low plasticity. Moderately sensitive to sensitive. (Hinuera Formation)		VSt
	0.6	Peak = 109kPa Residual = 31kPa		0.5	... at 0.50m, becoming light orange brown.		
				0.7	... at 0.70m, with some fine to coarse sand.		
				0.8	SW: Fine to coarse SAND: with minor fine gravel; brownish grey. Well graded. Gravel; subangular, pumiceous and rhyolitic. (Hinuera Formation)	M	MD to D
				1.8	... at 1.80m, becoming light greyish brown.		
				2.2	... at 2.20m, becoming orange brown with trace silt.		
				2.4	... from 2.40m to 2.45m, thin band of grey silty fine sand.		
				2.5	... at 2.50m, becoming greyish brown.		
				3.2	... from 3.20m to 3.40m, becoming grey with greyish brown.		
				3.5		M to W	
04-06-2024				3.6		W	
				3.8	... at 3.80m, becoming medium to coarse; grey with brownish grey.	S	
				4.0	Borehole terminated at 4.0 m		

Material Description: Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/ geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)

Dynamic Cone Penetrometer (Blows/100mm)

Depth (m)	Blows/100mm
0.0–0.1	2
0.1–0.2	3
0.2–0.3	1
0.3–0.4	1
0.4–0.5	2
0.5–0.6	1
0.6–0.7	1
0.7–0.8	2
0.8–0.9	3
0.9–1.0	6
1.0–1.1	6
1.1–1.2	6
1.2–1.3	6
1.3–1.4	5
1.4–1.5	5
1.5–1.6	6
1.6–1.7	5
1.7–1.8	4
1.8–1.9	5
1.9–2.0	5
2.0–2.1	7
2.1–2.2	7
2.2–2.3	5
2.3–2.4	5
2.4–2.5	6
2.5–2.6	7
2.6–2.7	8
2.7–2.8	5
2.8–2.9	4
2.9–3.0	5
3.0–3.1	6
3.1–3.2	7
3.2–3.3	6
3.3–3.4	6
3.4–3.5	5
3.5–3.6	7
3.6–3.7	5
3.7–3.8	5
3.8–3.9	4
3.9–4.0	4
4.0–4.1	7
4.1–4.2	5
4.2–4.3	7
4.3–4.4	5
4.4–4.5	5
4.5–4.6	5
4.6–4.7	6
4.7–4.8	5
4.8–4.9	7

Termination Reason: Hole collapse/no retrieval
Shear Vane No: 2993 DCP No: 34
Remarks: Groundwater encountered at 3.8m.

HAND AUGER BOREHOLE LOG - SOA24-05

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 31/05/2024
Borehole Location: Refer to site plan | Logged by: NK | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position: | Projection: | Datum: | Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Type & Results	RL (m)	Depth (m)	Graphic Log	Material Description (Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit); Rock: Colour; fabric; rock name; additional comments. (origin/geological unit))	Moisture Condition	Consistency/ Relative Density
				0.0–0.3		OL: Organic SILT: dark brown. Low plasticity. (Topsoil)	M	
				0.3–0.7		SM: Silty fine SAND: light grey. Poorly graded. (Hinuera Formation)	M	MD
				0.7–1.0		SM: Silty fine to coarse SAND: with minor fine gravel; brownish grey. Well graded, subrounded. (Hinuera Formation) ... *at 0.90m, thin band of iron staining fine sand.*	M	MD
				1.0–1.2		ML: SILT: light grey. Low plasticity. Dilatant. (Hinuera Formation)	M	F
				1.2–1.4		SM: Silty fine SAND: light grey. Poorly graded. Dilatant. (Hinuera Formation)	M to W	MD
				1.4		Borehole terminated at 1.4 m		

Dynamic Cone Penetrometer (Blows/100mm)

Depth (m)	Blows
0.2–0.3	6
0.3–0.4	7
0.4–0.5	5
0.5–0.6	5
0.6–0.7	5
0.7–0.8	4
0.8–0.9	3
0.9–1.0	5
1.0–1.1	3
1.1–1.2	3
1.2–1.3	3
1.3–1.4	3
1.4–1.5	4
1.5–1.6	5
1.6–1.7	4
1.7–1.8	5
1.8–1.9	3
1.9–2.0	3
2.0–2.1	6
2.1–2.2	6
2.2–2.3	3
2.3–2.4	4
2.4–2.5	4
2.5–2.6	3
2.6–2.7	5
2.7–2.8	5
2.8–2.9	5

Termination Reason: Target Depth Reached
Shear Vane No: | DCP No: 34
Remarks: Groundwater encountered at 1.4m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - SOA24-06

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 31/05/2024
Borehole Location: Refer to site plan | Logged by: NK | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position: | Projection: | Datum: | Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
				0.0–0.2	OL: Organic SILT: dark brown. Low plasticity. (Topsoil)	M	
	0.3	Peak = 121kPa Residual = 41kPa		0.2–0.8	ML: SILT: brown. Low plasticity. Moderately sensitive to sensitive. (Hinuera Formation)	M	VSt
	0.6	Peak = 148kPa Residual = 36kPa					
				0.8–1.3	SM: Silty fine to coarse SAND: with minor fine gravel; brown. Well graded, subrounded. (Hinuera Formation)	M	MD
				1.3–1.7	SM: Silty fine SAND: light grey. Poorly graded. Dilatant. (Hinuera Formation)	M to W	L to MD
	1.8	Peak = UTP		1.7–1.9	ML: SILT: light grey. Low plasticity. (Hinuera Formation)	M	H
				1.9–2.3	SW: Fine to coarse SAND: brown. Well graded, subrounded. (Hinuera Formation)	M / S	MD
				2.3	Borehole terminated at 2.3 m		

Dynamic Cone Penetrometer (Blows/100mm):

Depth interval (per 100mm)	Blows
1	2
2	2
3	1
4	1
5	1
6	1
7	1
8	1
9	3
10	3
11	3
12	3
13	4
14	3
15	3
16	4
17	2
18	4
19	4
20	6
21	6
22	5
23	4
24	6
25	4
26	3
27	2
28	4
29	4

Termination Reason: Target Depth Reached
Shear Vane No: 2955 DCP No: 34
Remarks: Groundwater encountered at 2.2m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - SOA24-07

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 31/05/2024
Borehole Location: Refer to site plan   Logged by: NK   Checked by: DM   Scale: 1:25   Sheet 1 of 1

Position: Projection:
Datum: Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Graphic Log	Material Description Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit)	Moisture Condition	Consistency/ Relative Density
						OL: Organic SILT: dark brown. Low plasticity. (Topsoil)	M	
	0.3	Peak = 193kPa Residual = 44kPa				ML: SILT: light brown mottled orange. Low plasticity. (Hinuera Formation)	M	VSt
						SM: Silty fine to medium SAND: light grey mottled orange. Well graded, subrounded. (Hinuera Formation)	M	L to MD
				1		SW: Fine to coarse SAND: with minor fine pumice gravel; light grey. Well graded, subrounded. (Hinuera Formation)	M / W to S	MD
						Borehole terminated at 1.5 m		

Dynamic Cone Penetrometer (Blows/100mm), scale 5, 10, 15:

Depth (m)	0.0–0.1	0.1–0.2	0.2–0.3	0.3–0.4	0.4–0.5	0.5–0.6	0.6–0.7	0.7–0.8	0.8–0.9	0.9–1.0
Blows	1	1	1	2	2	2	2	3	3	4

Depth (m)	1.0–1.1	1.1–1.2	1.2–1.3	1.3–1.4	1.4–1.5	1.5–1.6	1.6–1.7	1.7–1.8	1.8–1.9	1.9–2.0
Blows	3	3	3	3	4	3	4	4	5	3

Depth (m)	2.0–2.1	2.1–2.2	2.2–2.3	2.3–2.4	2.4–2.5	2.5–2.6	2.6–2.7	2.7–2.8	2.8–2.9
Blows	3	4	13	8	7	9	6	8	9

Depth scale: 1, 2, 3, 4, 5

Termination Reason: Target Depth Reached
Shear Vane No: 2955   DCP No: 34
Remarks: Groundwater encountered at 1.4m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - SOA24-08

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 31/05/2024
Borehole Location: Refer to site plan | Logged by: PM | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position: | Projection:
Datum: | Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density	Dynamic Cone Penetrometer (Blows/100mm) 5 10 15
					OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M		
	0.4	Peak = 89kPa Residual = 30kPa			ML: SILT: with some clay; light brown. Low plasticity. Moderately sensitive. (Walton Subgroup)	M	St	
	0.6	Peak = 44kPa Residual = 21kPa			ML: SILT: with some fine to medium sand; greyish brown. Low plasticity. Moderately sensitive. (Walton Subgroup)	M to W	F	
	1.0	Peak = 133kPa Residual = 44kPa		1			VSt	
	1.5	Peak = 119kPa Residual = 44kPa						
	2.0	Peak = UTP		2	CH: Silty CLAY: dark brown. High plasticity. (Walton Subgroup) *... from 1.90m to 2.00m, Becomes mottled black.*	M	H	
	2.5	Peak = UTP						
	3.0	Peak = 208kPa		3	Borehole terminated at 3.0 m			
				4				
				5				

Termination Reason: Target Depth Reached
Shear Vane No: 2993 | DCP No:
Remarks: Groundwater not encountered.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - SOA24-09

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 31/05/2024
Borehole Location: Refer to site plan | Logged by: NK | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position:
Projection:
Datum:
Survey Source: Site Plan

Groundwater	Samples & Insitu Tests: Depth	Samples & Insitu Tests: Type & Results	RL (m)	Depth (m)	Material Description (Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/ geological unit) Rock: Colour; fabric; rock name; additional comments. (origin/geological unit))	Moisture Condition	Consistency/ Relative Density
				0.0–0.25	OL: Organic SILT: dark brown. Low plasticity. (Topsoil)	M	
	0.3	Peak = 89kPa Residual = 21kPa		0.25–0.8	ML: SILT: brown. Low plasticity. (Walton Subgroup)	M	VSt to St
	0.6	Peak = 160kPa Residual = 30kPa					
	0.9	Peak = 121kPa Residual = 24kPa		0.8–1.2	ML: Sandy SILT: with trace fine pumice gravel. Low plasticity. (Walton Subgroup)	M	VSt to H
	1.2	Peak = UTP		1.2–1.6	SM: Silty fine to coarse SAND: with trace fine gravel; reddish brown. Well graded, subrounded. (Walton Subgroup)	M	
	1.6	Peak = 148kPa Residual = 33kPa		1.6–1.9	ML: Sandy SILT: light yellow mottled orange. Low plasticity. (Walton Subgroup)	M	
	2.0	Peak = 136kPa Residual = 24kPa		1.9–2.6	ML: SILT: light grey. Low plasticity. (Walton Subgroup)	M	
	2.3	Peak = 178kPa Residual = 24kPa					
	2.6	Peak = UTP		2.6–3.0	CH: Silty CLAY: brown. High plasticity. (Walton Subgroup)	M	
	3.0	Peak = 207kPa		3.0	Borehole terminated at 3.0 m		

Dynamic Cone Penetrometer (Blows/100mm)

Depth (m)	Blows/100mm
0.0–0.1	2
0.1–0.2	1
0.2–0.3	1
0.3–0.4	1
0.4–0.5	1
0.5–0.6	1
0.6–0.7	1
0.8–0.9	1
0.9–1.0	1
1.0–1.1	3
1.1–1.2	6
1.2–1.3	13
1.3–1.4	18
1.4–1.5	20

Termination Reason: Target Depth Reached
Shear Vane No: 2955 DCP No: 34
Remarks: Groundwater not encountered.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - SOA24-10

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 04/06/2024
Borehole Location: Refer to site plan | Logged by: WD | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

Position: | Projection: | Datum: | Survey Source: Site Plan

Groundwater	Sample Depth	Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
				0.0–0.3	OL: Organic SILT: dark brown with dark orange brown. No plasticity. (Topsoil)	M	
	0.3	Peak = 101kPa Residual = 15kPa		0.3–0.5	OH: Organic CLAY: dark brown. High plasticity. Sensitive. (Hinuera Formation)	M	St to VSt
	0.6	Peak = 89kPa Residual = 21kPa		0.5–1.3	CH: Silty CLAY: dark brown. High plasticity. Sensitive. (Hinuera Formation) ... at 0.80m, becoming yellowish grey.	M	St to VSt
	1.0	Peak = 127kPa Residual = 30kPa			... from 1.20m to 1.25m, lens of silty fine sand. ... from 1.25m to 1.30m, thin lens of silt.		
				1.3–1.6	SM: Silty Fine to medium SAND: light grey streaked orange brown. Well graded. (Hinuera Formation)	M	MD to D
04-06-2024 (1.8 m)				1.6–1.8	SW: Fine to coarse SAND: with some silt and minor fine pumiceous gravel; light grey. Well graded. (Hinuera Formation) ... at 1.70m, becoming grey.	W	MD to D
				1.8	Borehole terminated at 1.8 m		

Dynamic Cone Penetrometer (Blows/100mm), per 100 mm from surface: 2, 2, 2, 0, 1, 1, 0, 1, 1, 3, 5, 4, 4, 4, 5, 8, 4, 6, 5, 6, 3, 5, 3, 3, 3, 5, 5, 4, 5

Termination Reason: Target Depth Reached
Shear Vane No: 2993 | DCP No: 34
Remarks: Groundwater encountered at 1.8m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - SOA24-11

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 04/06/2024
Borehole Location: Refer to site plan | Logged by: WD | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

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Position:
Projection:
Datum:
Survey Source: Site Plan

Groundwater	Samples & Insitu Tests – Depth	Type & Results	RL (m)	Depth (m)	Graphic Log	Material Description (Soil: Soil symbol; soil type; colour; structure; bedding; plasticity; sensitivity; additional comments. (origin/geological unit); Rock: Colour; fabric; rock name; additional comments. (origin/geological unit))	Moisture Condition	Consistency/ Relative Density
				0.0–0.4		OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
				0.4–1.2		CH: Silty CLAY: brown. High plasticity. (Hinuera Formation) *... at 0.60m, becoming light brownish grey.* *... at 0.70m, becoming light grey streaked light orange brown.*	M	St
04-06-2024 (1.4 m)				1.2–1.6		SM: Silty Fine SAND: light grey. Poorly graded. Pumiceous. (Hinuera Formation) *... from 1.20m to 1.30m, becoming fine to coarse with trace fine pumiceous gravel.*	W (1.2–1.4); S (1.4–1.6)	MD to D
						Borehole terminated at 1.6 m		

Dynamic Cone Penetrometer (Blows/100mm): 1, 3, 2, 1, 2, 1, 3, 2, 2, 2, 5, 7, 7, 5, 6, 8, 6, 8, 9, 5, 3, 4, 5, 5, 7, 6, 5, 6, 7

Termination Reason: Target Depth Reached
Shear Vane No: | DCP No: 34
Remarks: Groundwater encountered at 1.4m.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

HAND AUGER BOREHOLE LOG - SOA24-12

Client: Maven Associates Ltd
Project: Station Road, Matamata
Site Location: 127-247A Station Road, Matamata, 3400
Project No.: HAM2023-0124
Date: 31/05/2024
Borehole Location: Refer to site plan | Logged by: PM | Checked by: DM | Scale: 1:25 | Sheet 1 of 1

Position: | Projection: | Datum: | Survey Source: Site Plan

Groundwater	Sample Depth	Type & Results	RL (m)	Depth (m)	Material Description	Moisture Condition	Consistency/ Relative Density
				0.0–0.4	OL: Organic SILT: dark brown. No plasticity. (Topsoil)	M	
	0.3	Peak = 104kPa Residual = 30kPa		0.4–1.4	ML: SILT: brown. Low plasticity. Moderately sensitive. (Hinuera Formation)	M	St to VSt
	0.6	Peak = 74kPa Residual = 44kPa					
				1.4–3.0	SW: Fine to coarse SAND: light brown. Well graded, sub rounded. (Hinuera Formation)	M	VL; MD to D
				2.5	... at 2.50m, Becoming light grey.	W	
				3.0	Borehole terminated at 3.0 m		

Dynamic Cone Penetrometer (Blows/100mm): 2, 2, 2, 2, 2, 2, 2, 1, 3, 6, 8, 8, 10, 8, 8, 8, 8, 6, 6, 8, 10, 8, 11, 8, 6, 6, 8, 7

Termination Reason: Target Depth Reached
Shear Vane No: 2993 DCP No: 34
Remarks: Groundwater not encountered.

This report is based on the attached field description for soil and rock, CMW Geosciences - Field Logging Guide, Revision 4 - April 2023.

APPENDIX D: IN-SITU PERMEABILITY TESTING RESULTS

CMW Geosciences	CLIENT:	DESIGNER:	NK
	PROJECT: **124 Station Road, Matamata**	CHECKED:	DM
		REVISION:	0
	TITLE: **HAS24-05 Falling Head Permeability Test**	DATE:	10/06/2024
		PROJECT:	HAM2023-0124

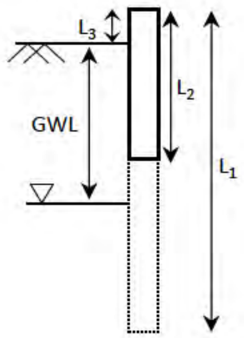
Specifications - Open-Ended Tube

Length L_1: 1.4 m
Diameter: 90 mm
Non-Perm L_2: 0.5 m
Above Gnd L_3: 0 m

Ground Conditions

GWL: 1.4 m BGL (Blank = Bottom of hole)
Permeability Anisotropy
m: 1 $m = \sqrt{k_h/k_v}$

Bottom of Test Hole: 1.40 m BGL



Hydraulic Conductivity (k)

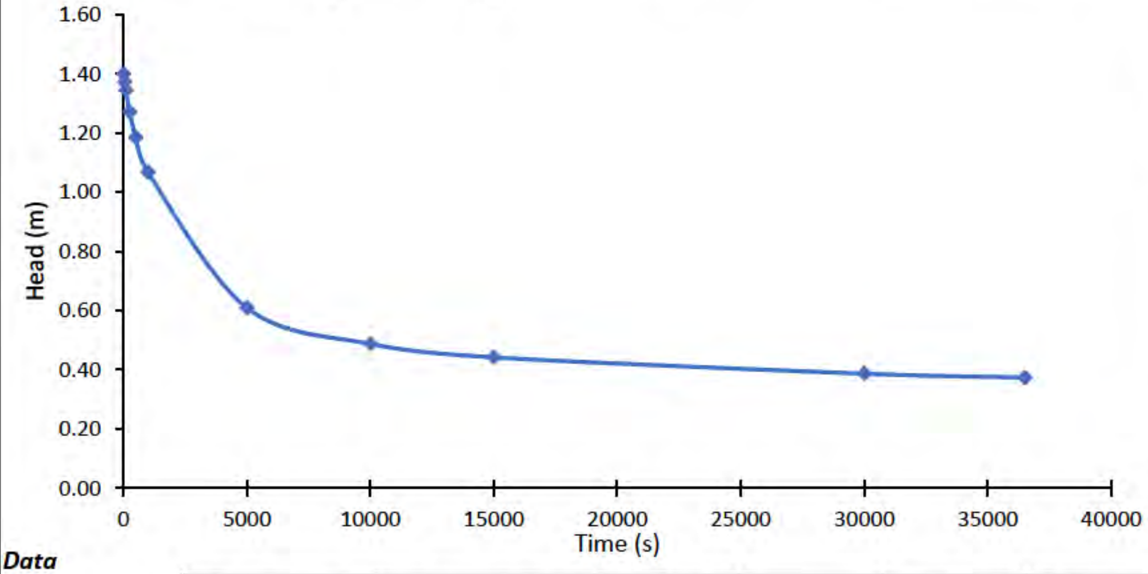
Note: CMW considers the CIRIA 113 value the most appropriate method for most purposes, but also provides the analysis method as outlined by Hvorslev if desired.

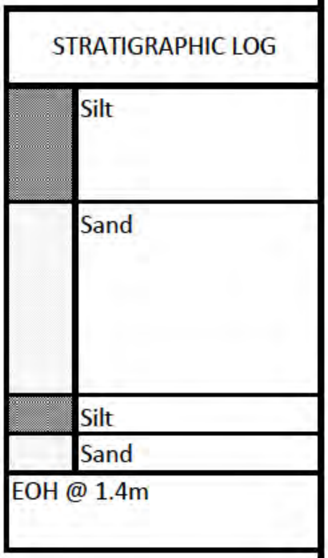
CIRIA 113: Somerville (1986), *Control of groundwater for temporary works,* CIRIA Report 113, Appendix 4

$$k = \left(\log\frac{h_1}{h_2} - \log\frac{2h_1+d}{2h_2+d}\right)\cdot\frac{(h_1+h_2)}{2(t_2-t_1)} =$$ 3.58E-06 ms^{-1} = 0.31 m/day

Hvorslev: Hvorslev (1951) *Time Lag and Soil Permeability in Ground-Water Observations*, Fig 18, p49

$$k = \frac{d^2 \ln\left(\frac{mL}{d} + \sqrt{\left(\frac{mL}{d}\right)^2 + 1}\right)}{8L(t_2-t_1)} \ln\frac{H_1}{H_2} =$$ 6.54E-07 ms^{-1} = 0.06 m/day





Data

Time (s)	Tape Avg (m)	Head (m)	Perm. Length (m)		Hvorslev 'k' Case G (ms^{-1})	CIRIA 113 'k' (ms^{-1})
0	0.000	1.400				
50	0.027	1.373	0.900		1.32E-06	7.40E-06
100	0.056	1.344	0.900		1.42E-06	7.97E-06
250	0.129	1.271	0.900		1.26E-06	7.04E-06
500	0.216	1.184	0.900		9.56E-07	5.35E-06
1000	0.333	1.067	0.900		7.05E-07	3.93E-06
5000	0.791	0.609	0.838		4.95E-07	2.72E-06
10000	0.913	0.487	0.548		2.07E-07	8.13E-07
15000	0.958	0.442	0.465		9.99E-08	3.49E-07
30000	1.013	0.387	0.414		4.83E-08	1.57E-07
36510	1.026	0.374	0.380		3.07E-08	9.39E-08

	CLIENT:	DESIGNER: NK
CMW Geosciences	PROJECT: 124 Station Road, Matamata	CHECKED: DM
		REVISION: 0
	TITLE: HAS24-06 Falling Head Permeability Test	DATE: 10/06/2024
		PROJECT: HAM2023-0124

Specifications - Open-Ended Tube

Length L_1: 2.3 m

Diameter: 90 mm

Non-Perm L_2: 0 m

Above Gnd L_3: 0 m

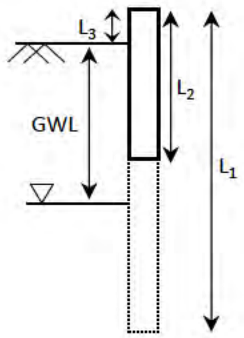
Ground Conditions

GWL: 2.2 m BGL (Blank = Bottom of hole)

Permeability Anisotropy

m: 1 $m = \sqrt{k_h/k_v}$

Bottom of Test Hole: 2.30 m BGL



Hydraulic Conductivity (k)

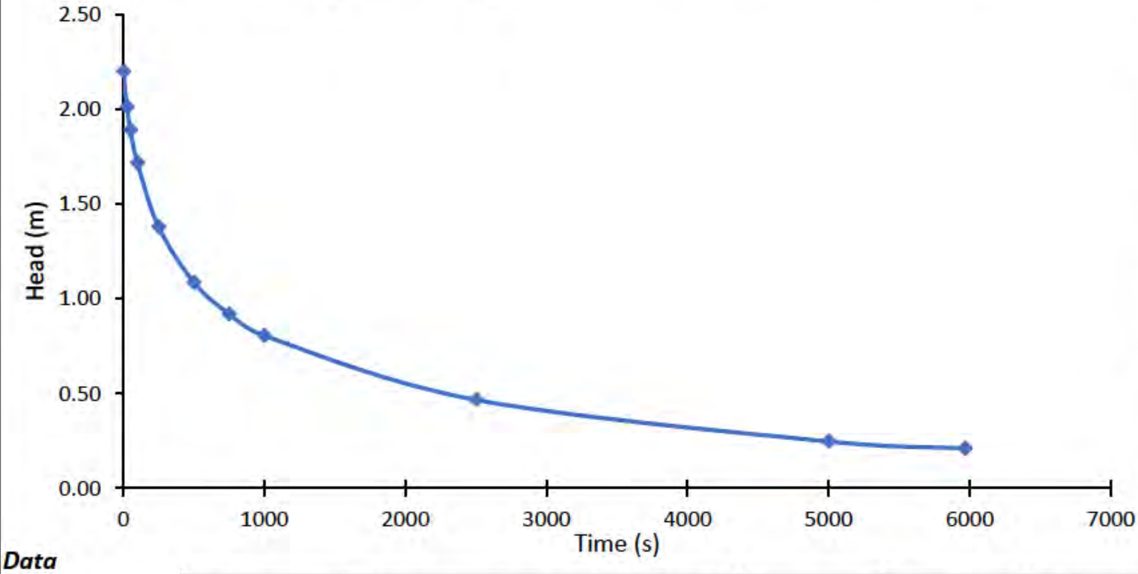
Note: CMW considers the CIRIA 113 value the most appropriate method for most purposes, but also provides the analysis method as outlined by Hvorslev if desired.

CIRIA 113: Somerville (1986), *Control of groundwater for temporary works*, CIRIA Report 113, Appendix 4

$$k = \left(log\frac{h_1}{h_2} - log\frac{2h_1+d}{2h_2+d}\right)\cdot\frac{(h_1+h_2)}{2(t_2-t_1)} = \quad 2.35\text{E-}05\ ms^{-1} = 2.03\ m/day$$

Hvorslev: Hvorslev (1951) *Time Lag and Soil Permeability in Ground-Water Observations*, Fig 18, p49

$$k = \frac{d^2\ ln\left(\frac{mL}{d} + \sqrt{\left(\frac{mL}{d}\right)^2 + 1}\right)}{8L(t_2-t_1)}\ ln\frac{H_1}{H_2} = \quad 2.80\text{E-}06\ ms^{-1} = 0.24\ m/day$$



STRATIGRAPHIC LOG
Silt
Sand
Silt
Sand
EOH @ 2.3m

Data

Time (s)	Tape Avg (m)	Head (m)	Perm. Length (m)		Hvorslev 'k' Case G (ms^{-1})	CIRIA 113 'k' (ms^{-1})
0	0.000	2.200				
25	0.186	2.014	2.207		6.32E-06	6.78E-05
50	0.308	1.892	2.053		4.71E-06	4.77E-05
100	0.483	1.718	1.905		3.85E-06	3.69E-05
250	0.820	1.380	1.649		3.23E-06	2.80E-05
500	1.114	1.086	1.333		2.46E-06	1.82E-05
750	1.280	0.920	1.103		1.96E-06	1.25E-05
1000	1.395	0.805	0.963		1.71E-06	9.90E-06
2500	1.734	0.466	0.736		1.40E-06	6.95E-06
5000	1.954	0.246	0.456		1.32E-06	4.68E-06
5970	1.990	0.210	0.328		1.03E-06	2.72E-06

CMW Geosciences	CLIENT:	DESIGNER:	NK
	PROJECT: 124 Station Road, Matamata	CHECKED:	DM
		REVISION:	0
	TITLE: HAS24-07 Falling Head Permeability Test	DATE:	10/06/2024
		PROJECT:	HAM2023-0124

Specifications - Open-Ended Tube

Length L_1: 1.5 m

Diameter: 90 mm

Non-Perm L_2: 0.5 m

Above Gnd L_3: 0 m

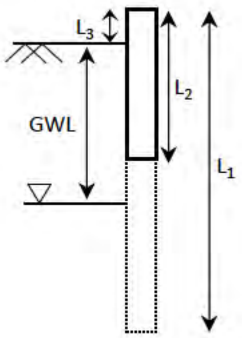
Ground Conditions

GWL: 1.4 m BGL (Blank = Bottom of hole)

Permeability Anisotropy

m: 1 $m = \sqrt{k_h/k_v}$

Bottom of Test Hole: 1.50 m BGL



Hydraulic Conductivity (k)

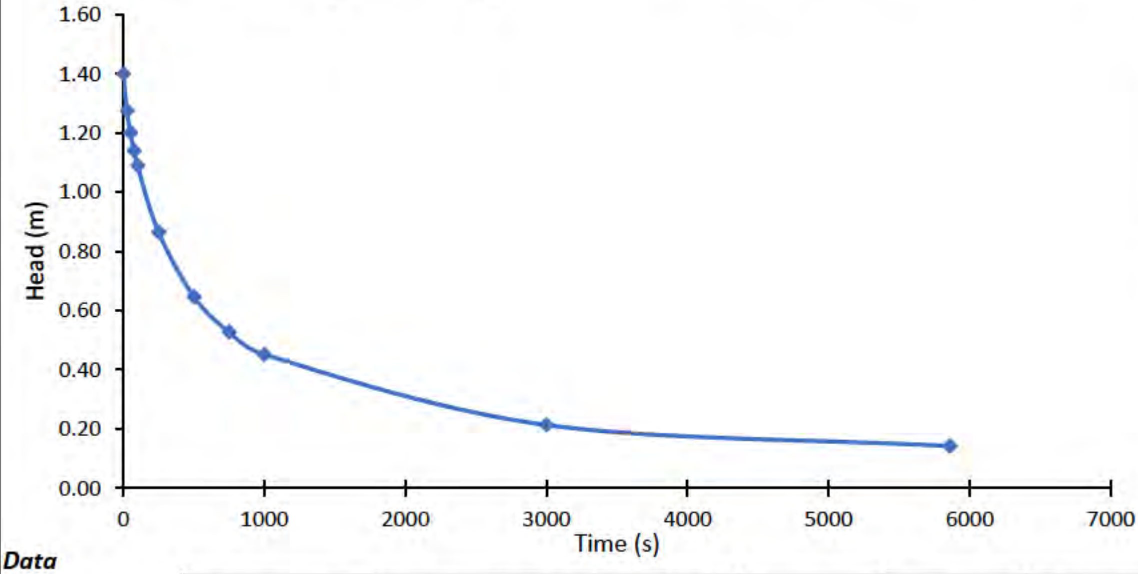
Note: CMW considers the CIRIA 113 value the most appropriate method for most purposes, but also provides the analysis method as outlined by Hvorslev if desired.

CIRIA 113: Somerville (1986), *Control of groundwater for temporary works,* CIRIA Report 113, Appendix 4

$$k = \left(\log\frac{h_1}{h_2} - \log\frac{2h_1+d}{2h_2+d}\right)\cdot\frac{(h_1+h_2)}{2(t_2-t_1)} = \quad 2.76\text{E-}05\ \text{ms}^{-1} = 2.38\ \text{m/day}$$

Hvorslev: Hvorslev (1951) *Time Lag and Soil Permeability in Ground-Water Observations* , Fig 18, p49

$$k = \frac{d^2 \ln\left(\frac{mL}{d} + \sqrt{\left(\frac{mL}{d}\right)^2 + 1}\right)}{8L(t_2-t_1)} \ln\frac{H_1}{H_2} = \quad 4.94\text{E-}06\ \text{ms}^{-1} = 0.43\ \text{m/day}$$



STRATIGRAPHIC LOG
Silt
Sand
EOH @ 1.5m

Data

Time (s)	Tape Avg (m)	Head (m)	Perm. Length (m)		Hvorslev 'k' Case G (ms^{-1})	CIRIA 113 'k' (ms^{-1})
0	0.000	1.400				
25	0.125	1.275	1.000		1.17E-05	7.07E-05
50	0.199	1.201	1.000		7.58E-06	4.55E-05
75	0.260	1.140	1.000		6.49E-06	3.89E-05
100	0.311	1.089	1.000		5.79E-06	3.46E-05
250	0.535	0.865	1.000		4.83E-06	2.90E-05
500	0.753	0.647	0.856		4.04E-06	2.17E-05
750	0.873	0.527	0.687		3.29E-06	1.49E-05
1000	0.949	0.451	0.589		2.76E-06	1.12E-05
3000	1.187	0.213	0.432		1.99E-06	6.94E-06
5860	1.258	0.142	0.278		9.56E-07	2.26E-06

CMW Geosciences	CLIENT:	DESIGNER:	NK
	PROJECT: **124 Station Road, Matamata**	CHECKED:	DM
		REVISION:	0
	TITLE: **HAS24-08 Falling Head Permeability Test**	DATE:	10/06/2024
		PROJECT:	HAM2023-0124

Specifications - Open-Ended Tube

Length L_1: 3 m

Diameter: 90 mm

Non-Perm L_2: 0 m

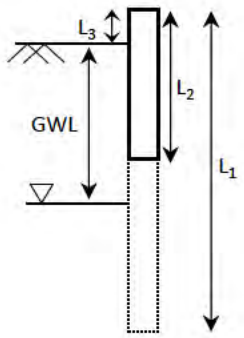
Above Gnd L_3: 0 m

Ground Conditions

GWL: 3 m BGL (Blank = Bottom of hole)

Permeability Anisotropy

m: 1 $m = \sqrt{k_h / k_v}$



Bottom of Test Hole: 3.00 m BGL

Hydraulic Conductivity (k)

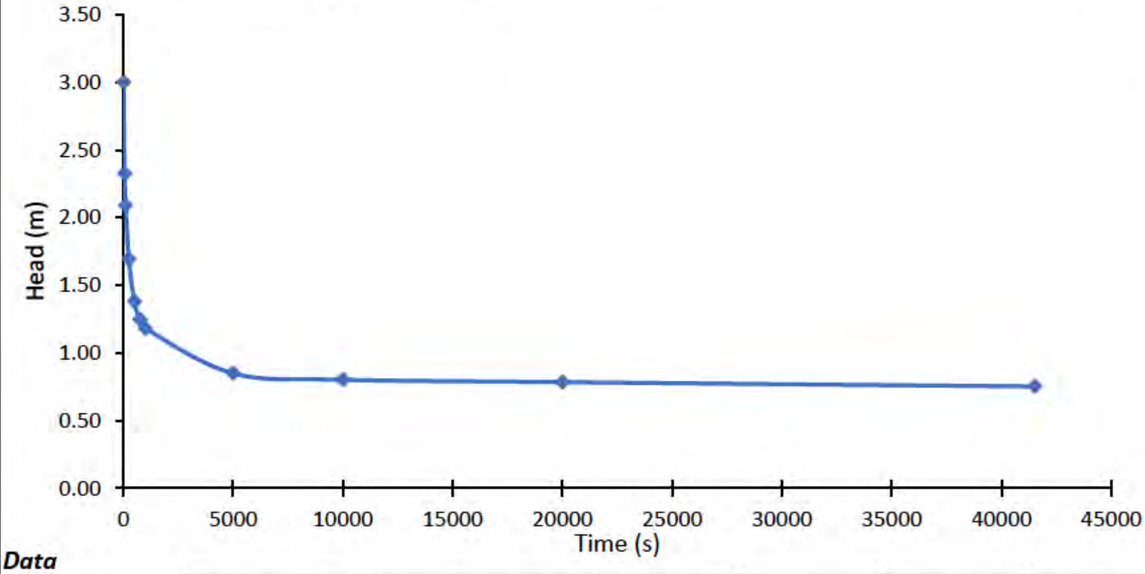
Note: CMW considers the CIRIA 113 value the most appropriate method for most purposes, but also provides the analysis method as outlined by Hvorslev if desired.

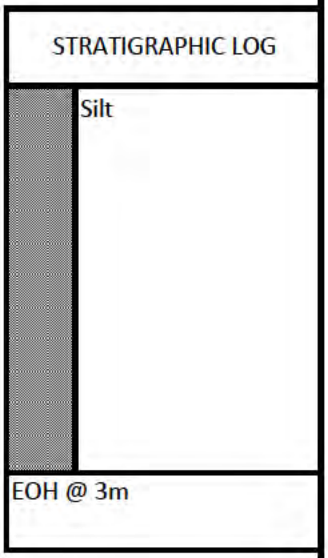
CIRIA 113: Somerville (1986), *Control of groundwater for temporary works,* CIRIA Report 113, Appendix 4

$$k = \left(log\frac{h_1}{h_2} - log\frac{2h_1 + d}{2h_2 + d} \right) \cdot \frac{(h_1 + h_2)}{2(t_2 - t_1)} =$$ 1.96E-05 ms^{-1} = 1.69 m/day

Hvorslev: Hvorslev (1951) *Time Lag and Soil Permeability in Ground-Water Observations*, Fig 18, p49

$$k = \frac{d^2 \ln\left(\frac{mL}{d} + \sqrt{\left(\frac{mL}{d}\right)^2 + 1} \right)}{8L(t_2 - t_1)} \ln\frac{H_1}{H_2} =$$ 1.84E-06 ms^{-1} = 0.16 m/day





Data

Time (s)	Tape Avg (m)	Head (m)	Perm. Length (m)		Hvorslev 'k' Case G (ms^{-1})	CIRIA 113 'k' (ms^{-1})
0	0.000	3.000				
50	0.676	2.325	2.662		7.92E-06	9.91E-05
100	0.909	2.091	2.208		3.78E-06	4.06E-05
250	1.307	1.693	1.892		2.82E-06	2.71E-05
500	1.621	1.379	1.536		1.91E-06	1.57E-05
750	1.754	1.246	1.312		1.06E-06	7.70E-06
1000	1.819	1.181	1.213		5.84E-07	4.00E-06
5000	2.151	0.849	1.015		2.57E-07	1.57E-06
10000	2.199	0.801	0.825		4.12E-08	2.14E-07
20000	2.216	0.784	0.792		8.10E-09	4.08E-08
41530	2.249	0.751	0.768		7.32E-09	3.61E-08

CLIENT:		DESIGNER:	NK
PROJECT:	124 Station Road, Matamata	CHECKED:	DM
		REVISION:	0
TITLE:	HAS24-09 Falling Head Permeability Test	DATE:	10/06/2024
		PROJECT:	HAM2023-0124

Specifications - Open-Ended Tube

Length L_1: 3 m

Diameter: 90 mm

Non-Perm L_2: 0 m

Above Gnd L_3: 0 m

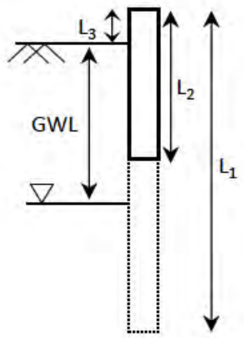
Ground Conditions

GWL: 3 m BGL (Blank = Bottom of hole)

Permeability Anisotropy

m: 1 $m = \sqrt{k_h/k_v}$

Bottom of Test Hole: 3.00 m BGL



Hydraulic Conductivity (k)

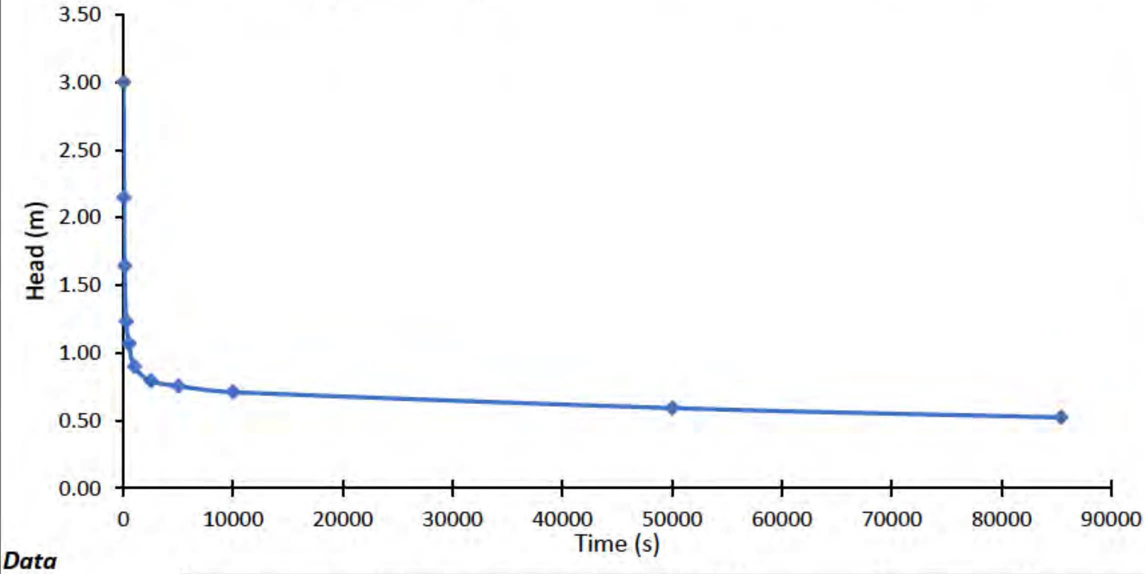
Note: CMW considers the CIRIA 113 value the most appropriate method for most purposes, but also provides the analysis method as outlined by Hvorslev if desired.

CIRIA 113: Somerville (1986), *Control of groundwater for temporary works,* CIRIA Report 113, Appendix 4

$$k = \left(log\frac{h_1}{h_2} - log\frac{2h_1+d}{2h_2+d}\right).\frac{(h_1+h_2)}{2(t_2-t_1)} =$$ 2.91E-05 ms^{-1} = 2.51 m/day

Hvorslev: Hvorslev (1951) *Time Lag and Soil Permeability in Ground-Water Observations*, Fig 18, p49

$$k = \frac{d^2\, ln\left(\frac{mL}{d} + \sqrt{\left(\frac{mL}{d}\right)^2 + 1}\right)}{8L(t_2-t_1)}\, ln\frac{H_1}{H_2} =$$ 2.92E-06 ms^{-1} = 0.25 m/day



STRATIGRAPHIC LOG
Silt
Sand
Silt
EOH @ 3m

Data

Time (s)	Tape Avg (m)	Head (m)	Perm. Length (m)		Hvorslev 'k' Case G (ms^{-1})	CIRIA 113 'k' (ms^{-1})
0	0.000	3.000				
50	0.853	2.147	2.573		1.07E-05	1.31E-04
100	1.357	1.644	1.895		1.07E-05	1.03E-04
250	1.767	1.233	1.438		4.68E-06	3.68E-05
500	1.934	1.066	1.149		1.66E-06	1.10E-05
1000	2.102	0.898	0.982		1.09E-06	6.43E-06
2500	2.207	0.793	0.846		2.91E-07	1.54E-06
5000	2.245	0.756	0.774		7.29E-08	3.62E-07
10000	2.290	0.710	0.733		4.80E-08	2.29E-07
50000	2.408	0.592	0.651		1.89E-08	8.35E-08
85440	2.477	0.523	0.557		1.61E-08	6.38E-08

CMW Geosciences	CLIENT:		DESIGNER:	NK
	PROJECT:	124 Station Road, Matamata	CHECKED:	DM
			REVISION:	0
	TITLE:	HAS24-10 Falling Head Permeability Test	DATE:	10/06/2024
			PROJECT:	HAM2023-0124

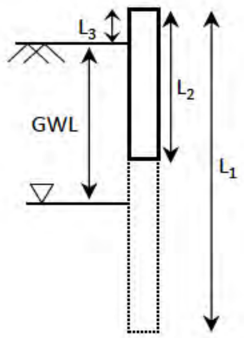
Specifications - Open-Ended Tube

Length L_1: 1.8 m
Diameter: 90 mm
Non-Perm L_2: 0 m
Above Gnd L_3: 0 m

Ground Conditions

GWL: 1.8 m BGL (Blank = Bottom of hole)
Permeability Anisotropy
m: 1 $m = \sqrt{k_h/k_v}$

Bottom of Test Hole: 1.80 m BGL



Hydraulic Conductivity (k)

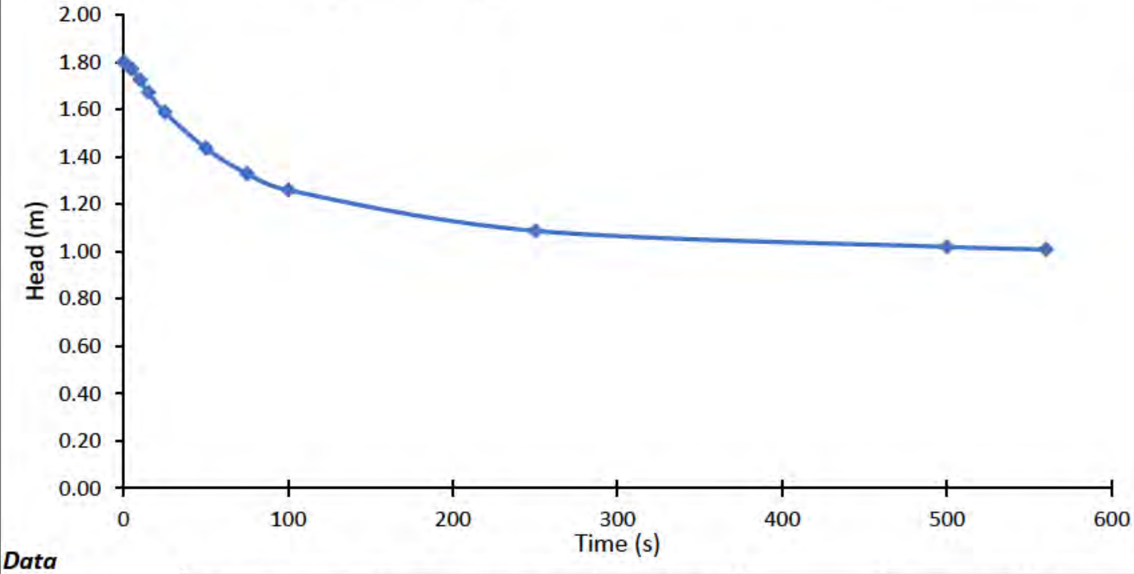
Note: CMW considers the CIRIA 113 value the most appropriate method for most purposes, but also provides the analysis method as outlined by Hvorslev if desired.

CIRIA 113: Somerville (1986), *Control of groundwater for temporary works,* CIRIA Report 113, Appendix 4

$$k = \left(\log\frac{h_1}{h_2} - \log\frac{2h_1+d}{2h_2+d}\right)\cdot\frac{(h_1+h_2)}{2(t_2-t_1)} =$$ 5.82E-05 ms^{-1} = 5.03 m/day

Hvorslev: Hvorslev (1951) *Time Lag and Soil Permeability in Ground-Water Observations*, Fig 18, p49

$$k = \frac{d^2 \ln\left(\frac{mL}{d} + \sqrt{\left(\frac{mL}{d}\right)^2+1}\right)}{8L(t_2-t_1)} \ln\frac{H_1}{H_2} =$$ 7.00E-06 ms^{-1} = 0.60 m/day



STRATIGRAPHIC LOG
Silt
Sand
EOH @ 1.8m

Data

Time (s)	Tape Avg (m)	Head (m)	Perm. Length (m)		Hvorslev 'k' Case G (ms^{-1})	CIRIA 113 'k' (ms^{-1})
0	0.000	1.800				
5	0.031	1.769	1.785		7.19E-06	6.56E-05
10	0.077	1.723	1.746		1.12E-05	1.00E-04
15	0.127	1.673	1.698		1.30E-05	1.14E-04
25	0.213	1.587	1.630		1.17E-05	9.97E-05
50	0.364	1.436	1.511		9.45E-06	7.63E-05
75	0.472	1.328	1.382		7.81E-06	5.89E-05
100	0.541	1.259	1.293		5.67E-06	4.07E-05
250	0.713	1.087	1.173		2.75E-06	1.85E-05
500	0.781	1.019	1.053		7.83E-07	4.84E-06
560	0.792	1.008	1.013		5.74E-07	3.45E-06

CMW Geosciences	CLIENT:	DESIGNER:	NK
	PROJECT: 124 Station Road, Matamata	CHECKED:	DM
		REVISION:	0
	TITLE: HAS24-11 Falling Head Permeability Test	DATE:	10/06/2024
		PROJECT:	HAM2023-0124

Specifications - Open-Ended Tube

Length L_1:	1.6	m
Diameter:	90	mm
Non-Perm L_2:	0	m
Above Gnd L_3:	0	m

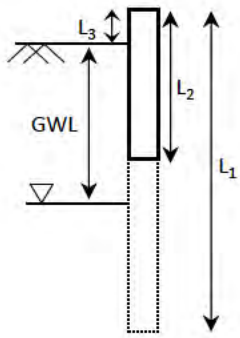
Ground Conditions

GWL: 1.4 m BGL (Blank = Bottom of hole)

Permeability Anisotropy

m: 1 $m = \sqrt{k_h / k_v}$

Bottom of Test Hole: 1.60 m BGL



Hydraulic Conductivity (k)

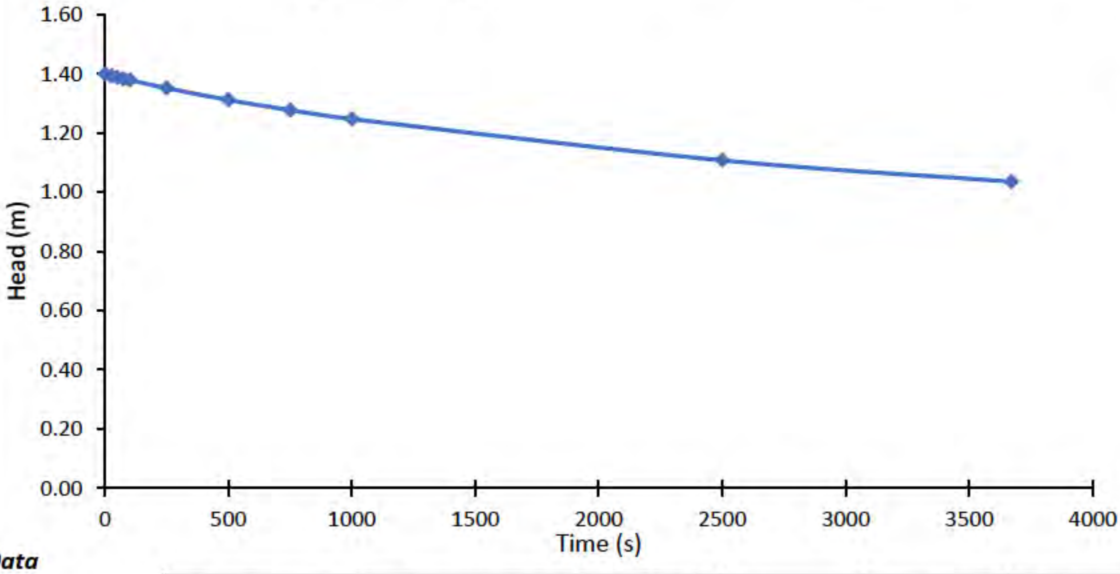
Note: CMW considers the CIRIA 113 value the most appropriate method for most purposes, but also provides the analysis method as outlined by Hvorslev if desired.

CIRIA 113: Somerville (1986), *Control of groundwater for temporary works,* CIRIA Report 113, Appendix 4

$$k = \left(\log\frac{h_1}{h_2} - \log\frac{2h_1+d}{2h_2+d}\right)\cdot\frac{(h_1+h_2)}{2(t_2-t_1)} = \quad 2.27\text{E-}06\ \text{ms}^{-1} = 0.20\ \text{m/day}$$

Hvorslev: Hvorslev (1951) *Time Lag and Soil Permeability in Ground-Water Observations*, Fig 18, p49

$$k = \frac{d^2 \ln\left(\frac{mL}{d} + \sqrt{\left(\frac{mL}{d}\right)^2 + 1}\right)}{8L(t_2-t_1)} \ln\frac{H_1}{H_2} = \quad 2.80\text{E-}07\ \text{ms}^{-1} = 0.02\ \text{m/day}$$



STRATIGRAPHIC LOG
Sand
EOH @ 1.6m

Data

Time (s)	Tape Avg (m)	Head (m)	Perm. Length (m)		Hvorslev 'k' Case G (ms^{-1})	CIRIA 113 'k' (ms^{-1})
0	0.000	1.400				
25	0.007	1.393	1.597		4.41E-07	3.69E-06
50	0.013	1.387	1.590		3.99E-07	3.32E-06
75	0.018	1.383	1.585		3.02E-07	2.52E-06
100	0.021	1.379	1.581		2.44E-07	2.03E-06
250	0.048	1.352	1.565		3.03E-07	2.49E-06
500	0.088	1.312	1.532		2.82E-07	2.28E-06
750	0.122	1.278	1.495		2.49E-07	1.98E-06
1000	0.153	1.247	1.462		2.32E-07	1.82E-06
2500	0.292	1.108	1.377		1.99E-07	1.49E-06
3670	0.364	1.036	1.272		1.53E-07	1.08E-06

CLIENT:		DESIGNER:	NK
PROJECT:	**124 Station Road, Matamata**	CHECKED:	DM
		REVISION:	0
TITLE:	**HAS24-12 Falling Head Permeability Test**	DATE:	10/06/2024
		PROJECT:	HAM2023-0124

Specifications - Open-Ended Tube

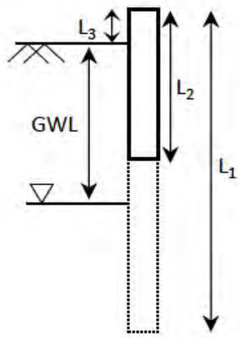
Length L_1:	3	m
Diameter:	90	mm
Non-Perm L_2:	0	m
Above Gnd L_3:	0	m

Ground Conditions

GWL: 3 m BGL (Blank = Bottom of hole)

Permeability Anisotropy

m: 1 $m = \sqrt{k_h / k_v}$



Bottom of Test Hole: 3.00 m BGL

Hydraulic Conductivity (k)

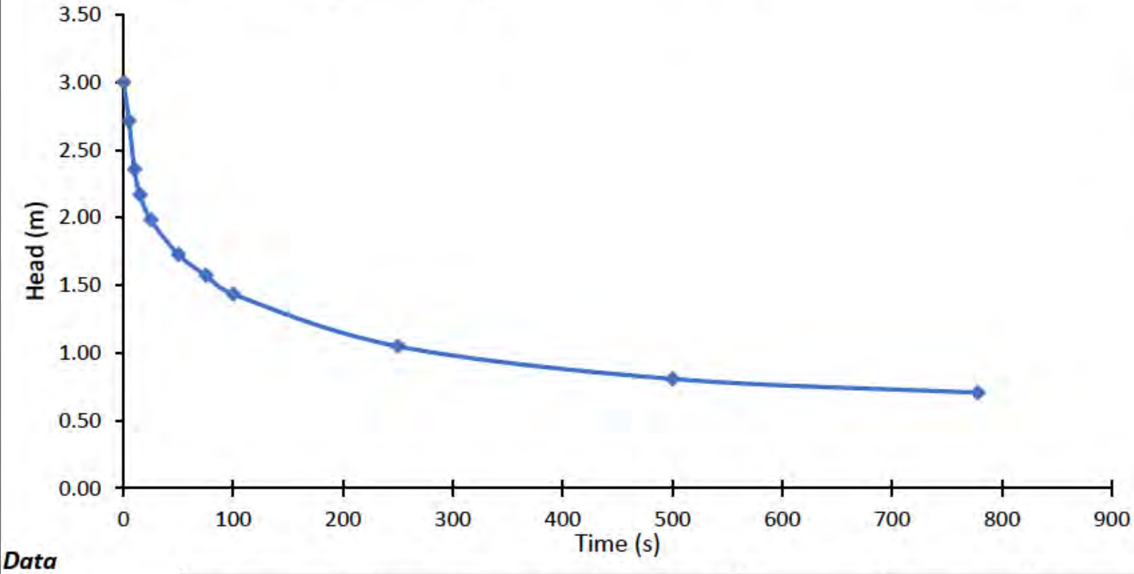
Note: CMW considers the CIRIA 113 value the most appropriate method for most purposes, but also provides the analysis method as outlined by Hvorslev if desired.

CIRIA 113: Somerville (1986), *Control of groundwater for temporary works,* CIRIA Report 113, Appendix 4

$$k = \left(log\frac{h_1}{h_2} - log\frac{2h_1+d}{2h_2+d}\right)\cdot\frac{(h_1+h_2)}{2(t_2-t_1)} = \quad 1.74E\text{-}04\ ms^{-1} = 15.00\ m/day$$

Hvorslev: Hvorslev (1951) *Time Lag and Soil Permeability in Ground-Water Observations*, Fig 18, p49

$$k = \frac{d^2\, ln\left(\frac{mL}{d} + \sqrt{\left(\frac{mL}{d}\right)^2 + 1}\right)}{8L(t_2-t_1)}\, ln\frac{H_1}{H_2} = \quad 1.60E\text{-}05\ ms^{-1} = 1.38\ m/day$$



STRATIGRAPHIC LOG
Silt
Sand
EOH @ 3m

Data

Time (s)	Tape Avg (m)	Head (m)	Perm. Length (m)		Hvorslev 'k' Case G (ms^{-1})	CIRIA 113 'k' (ms^{-1})
0	0.000	3.000				
5	0.287	2.713	2.857		2.96E-05	3.87E-04
10	0.643	2.357	2.535		4.53E-05	5.42E-04
15	0.832	2.168	2.262		2.93E-05	3.21E-04
25	1.016	1.984	2.076		1.65E-05	1.70E-04
50	1.273	1.727	1.855		1.13E-05	1.06E-04
75	1.427	1.574	1.650		8.22E-06	7.08E-05
100	1.566	1.434	1.504		8.78E-06	7.06E-05
250	1.951	1.049	1.241		5.65E-06	4.00E-05
500	2.193	0.807	0.928		3.45E-06	1.97E-05
778	2.295	0.705	0.756		1.84E-06	9.01E-06

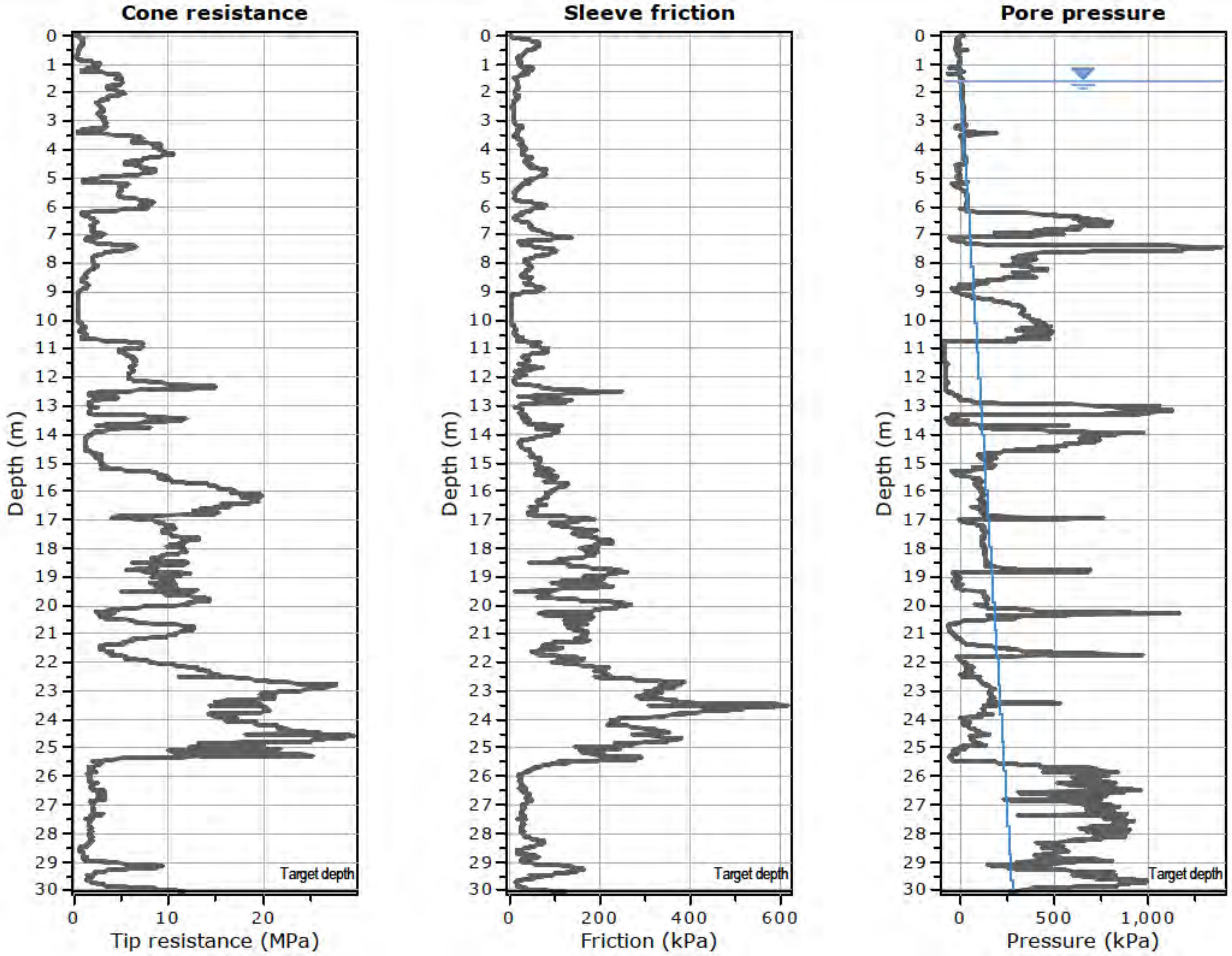
APPENDIX E: CPT INVESTIGATION RESULTS

Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

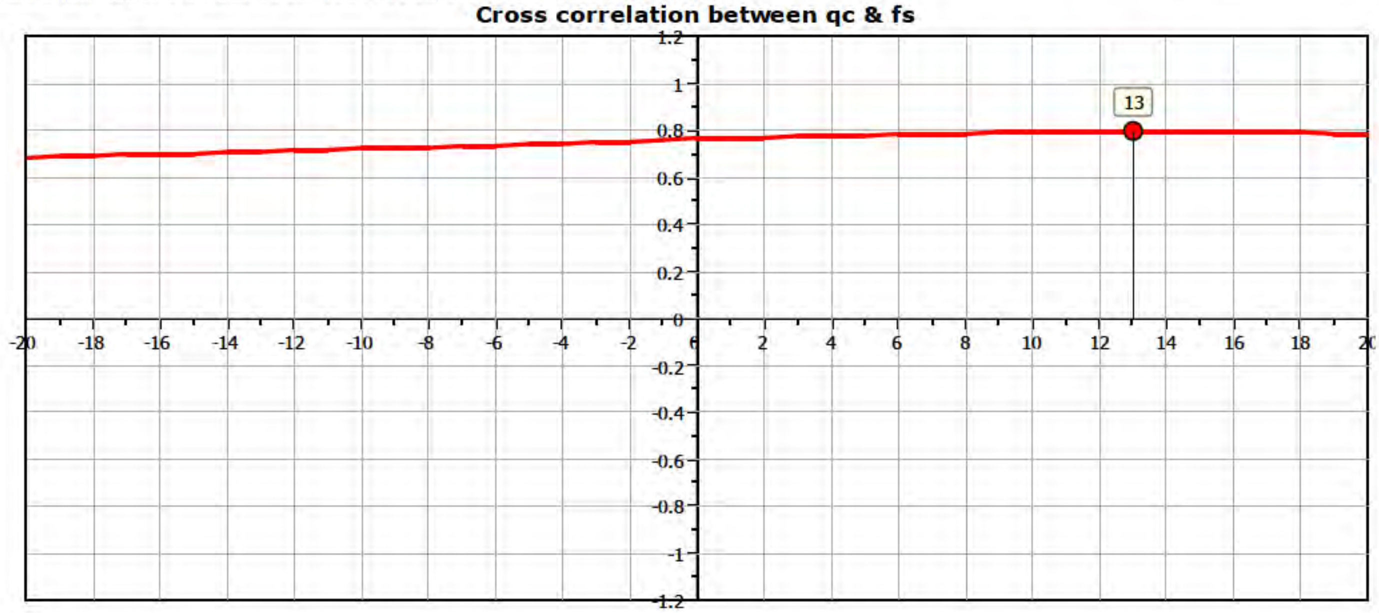
CPT: 24-04

Total depth: 30.04 m, Date: 4/06/2024
Coords: lat -37.812787° lon 175.751857°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



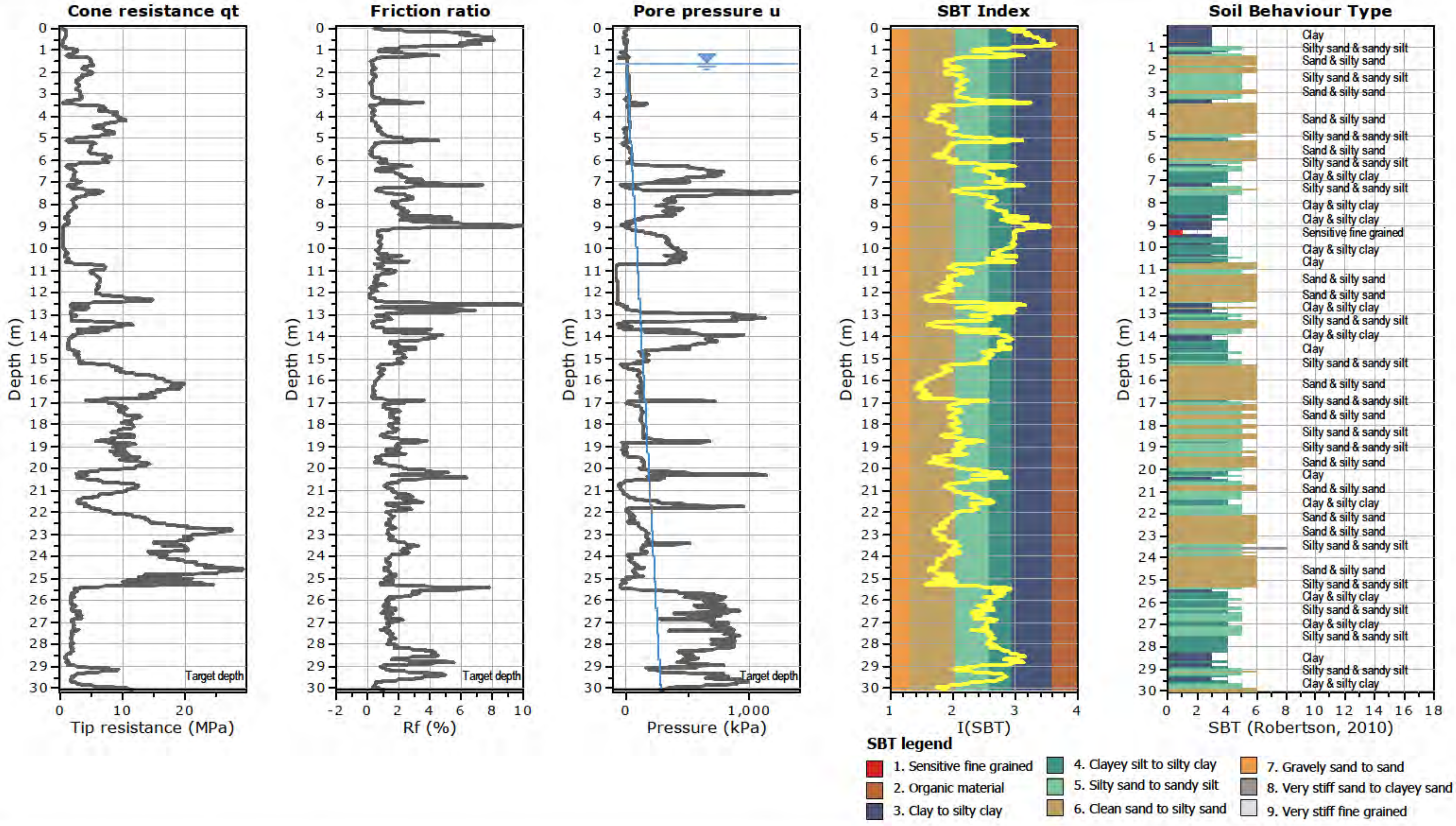
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: 24-04
Total depth: 30.04 m, Date: 4/06/2024
Coords: lat -37.812787° lon 175.751857°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter

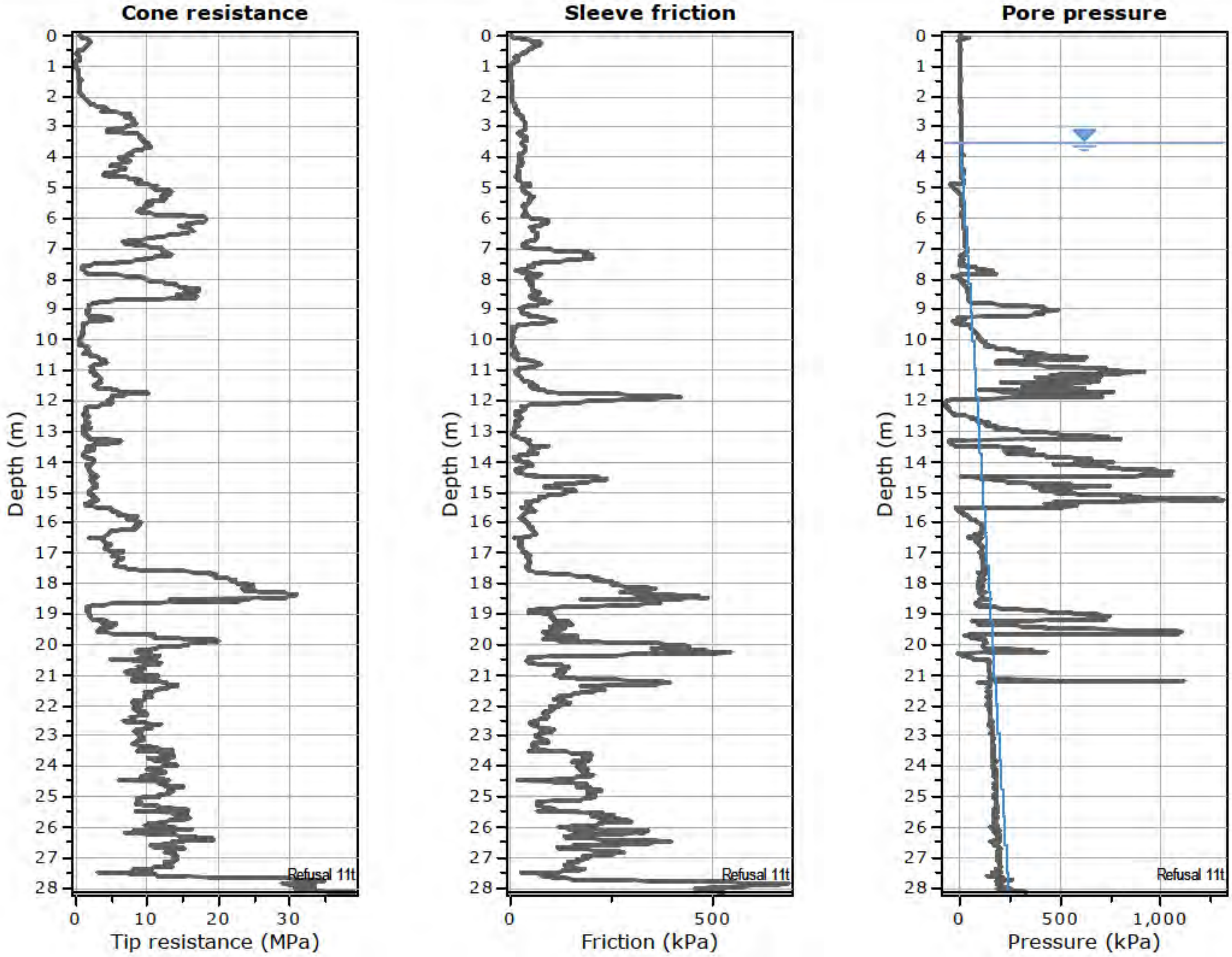


Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: 24-05

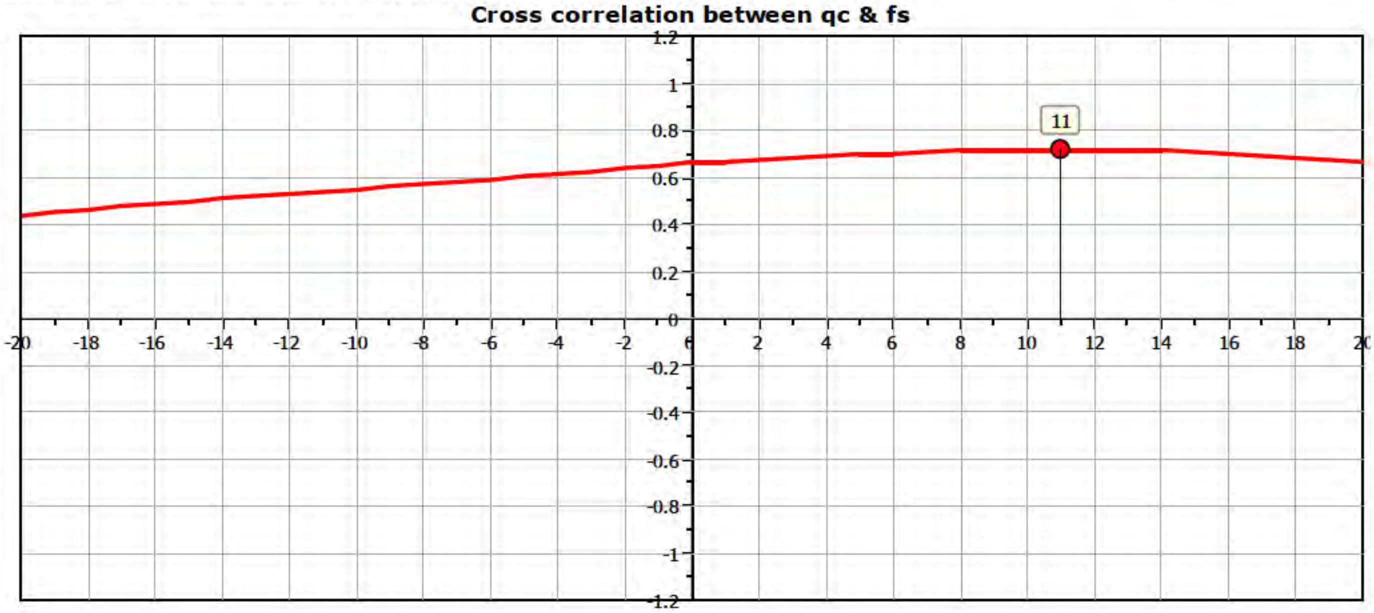
Total depth: 28.14 m, Date: 4/06/2024
Coords: lat -37.815987° lon 175.753168°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

Cross correlation between qc & fs



Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: 24-05

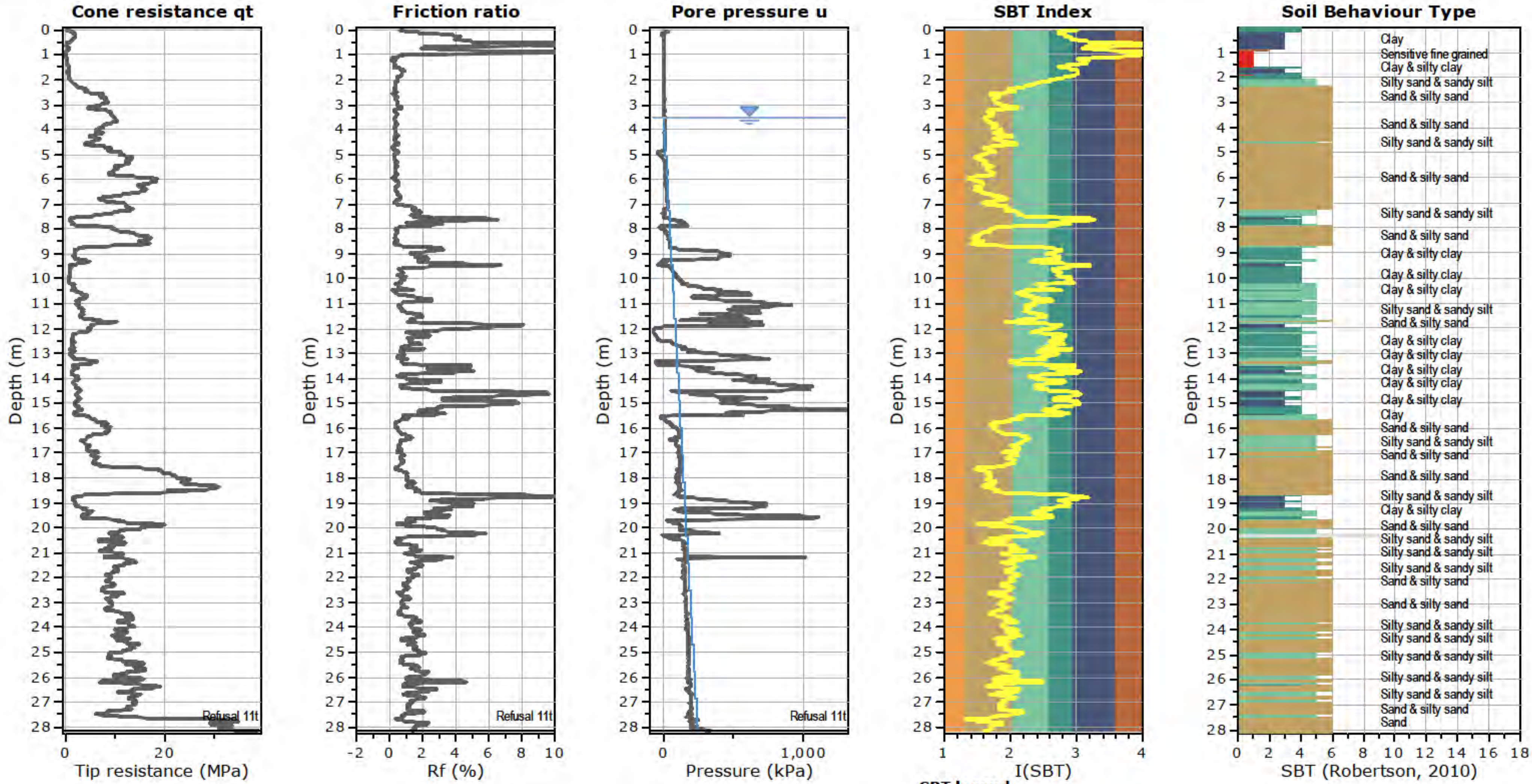
Total depth: 28.14 m, Date: 4/06/2024

Coords: lat -37.815987° lon 175.753168°

Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd

Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



SBT legend

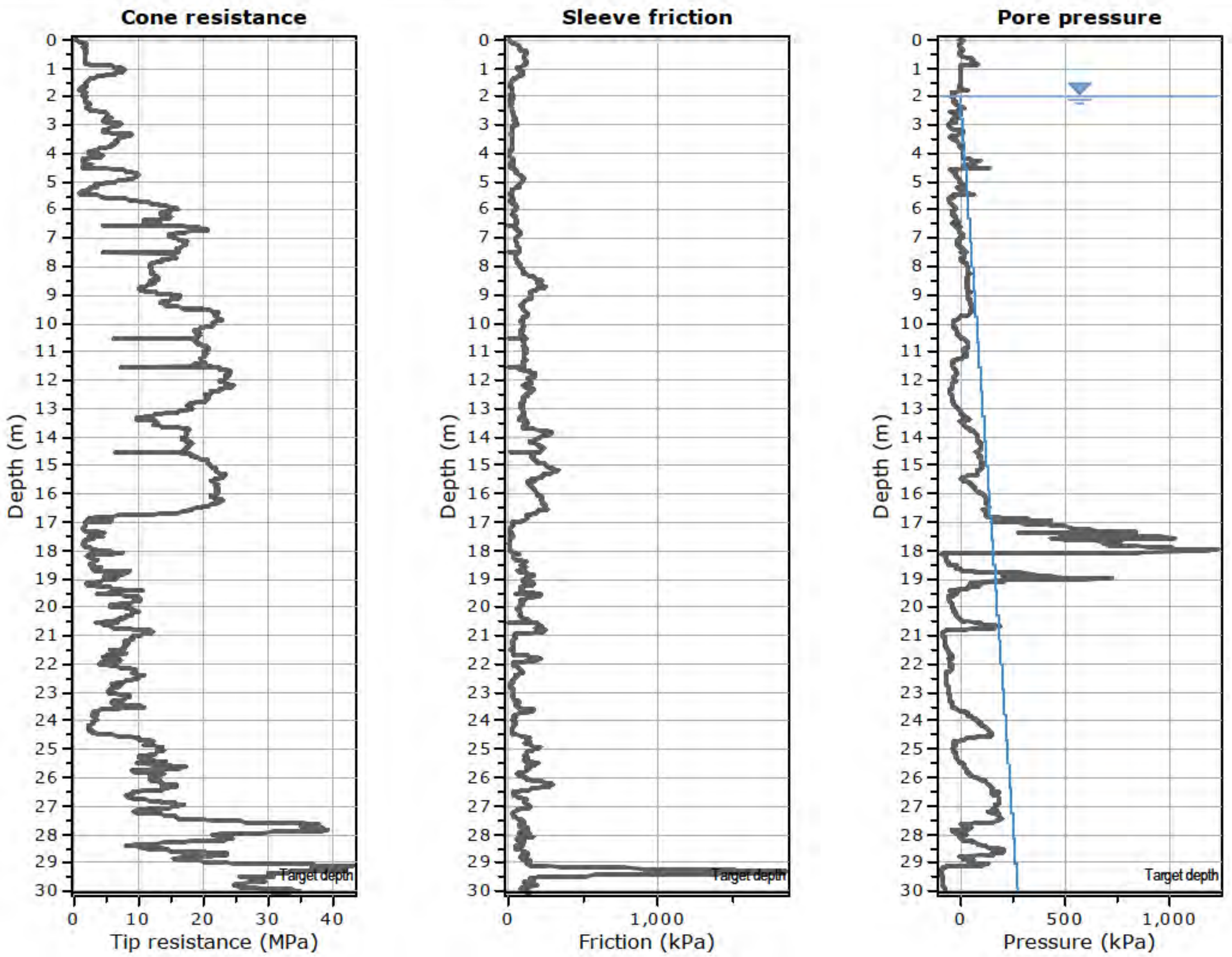
- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

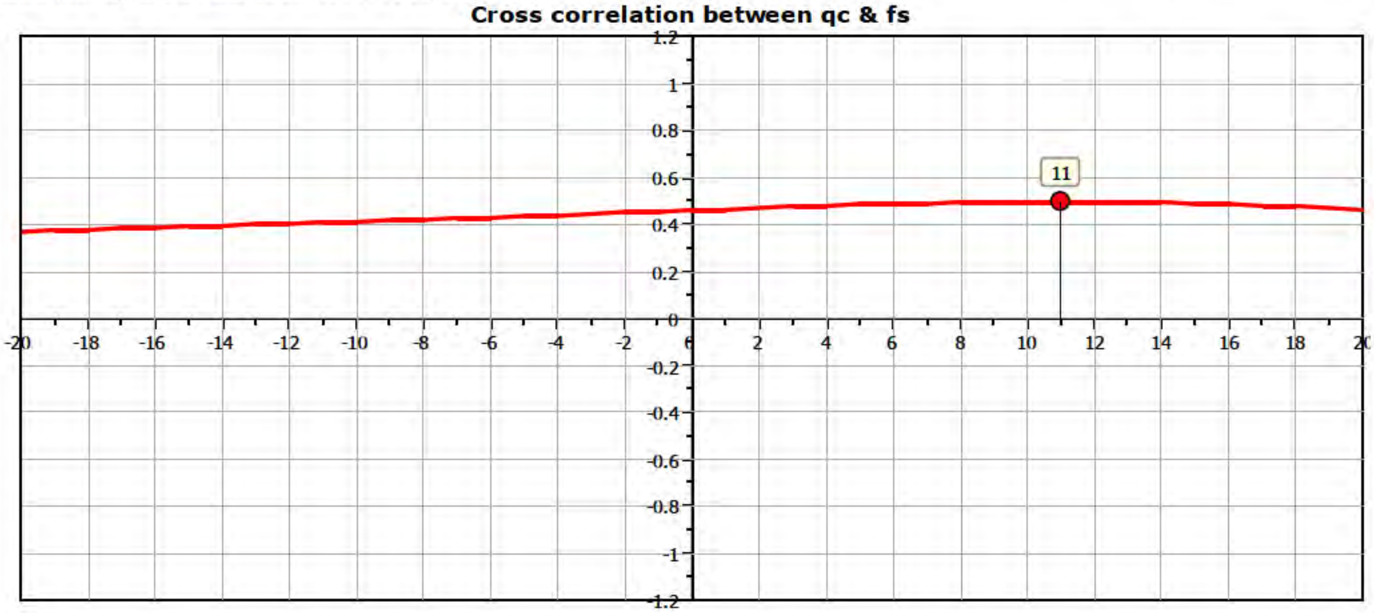
CPT: 24-06

Total depth: 30.06 m, Date: 4/06/2024
Coords: lat -37.820042° lon 175.759802°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: 24-06

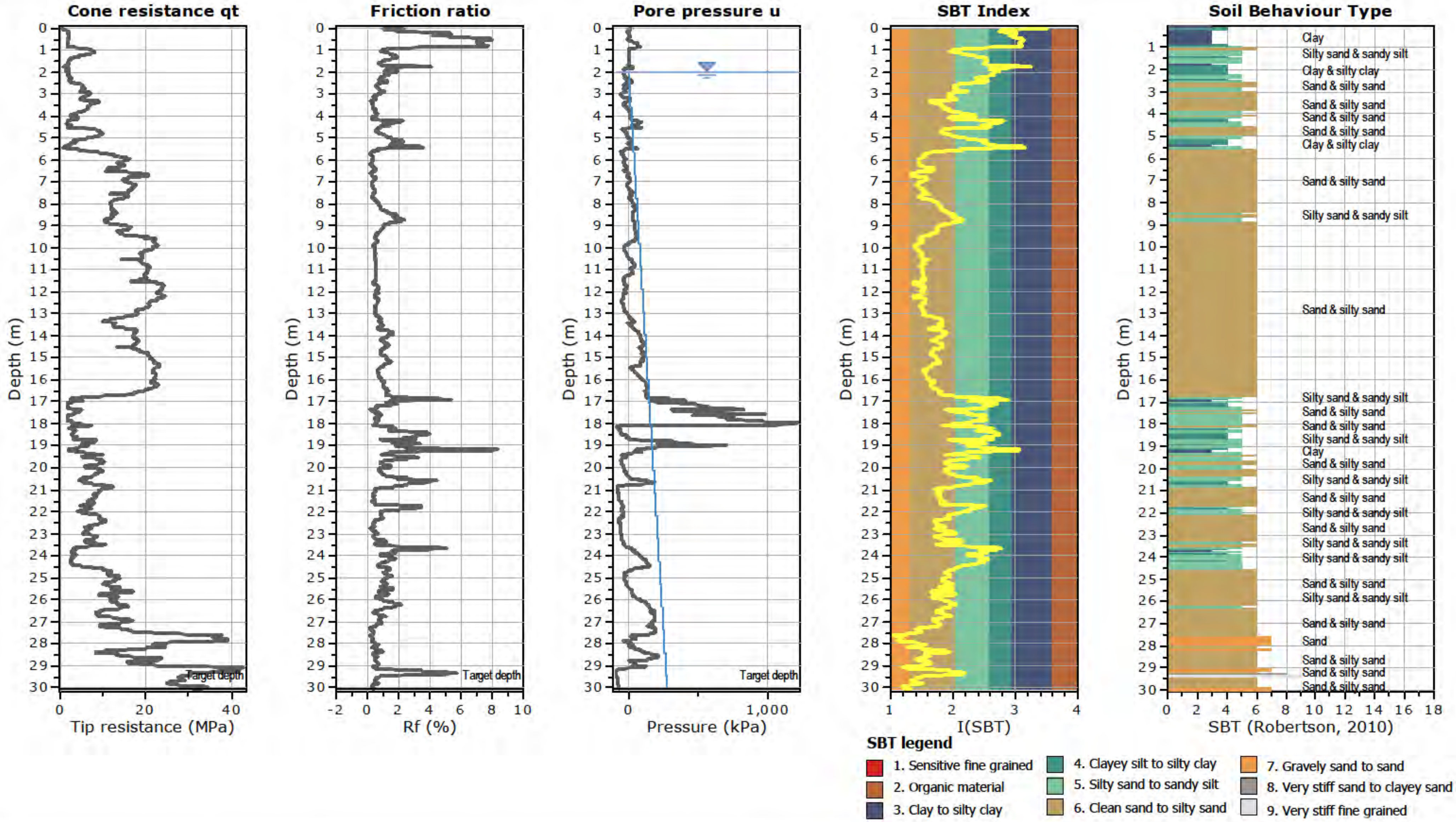
Total depth: 30.06 m, Date: 4/06/2024

Coords: lat -37.820042° lon 175.759802°

Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd

Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter

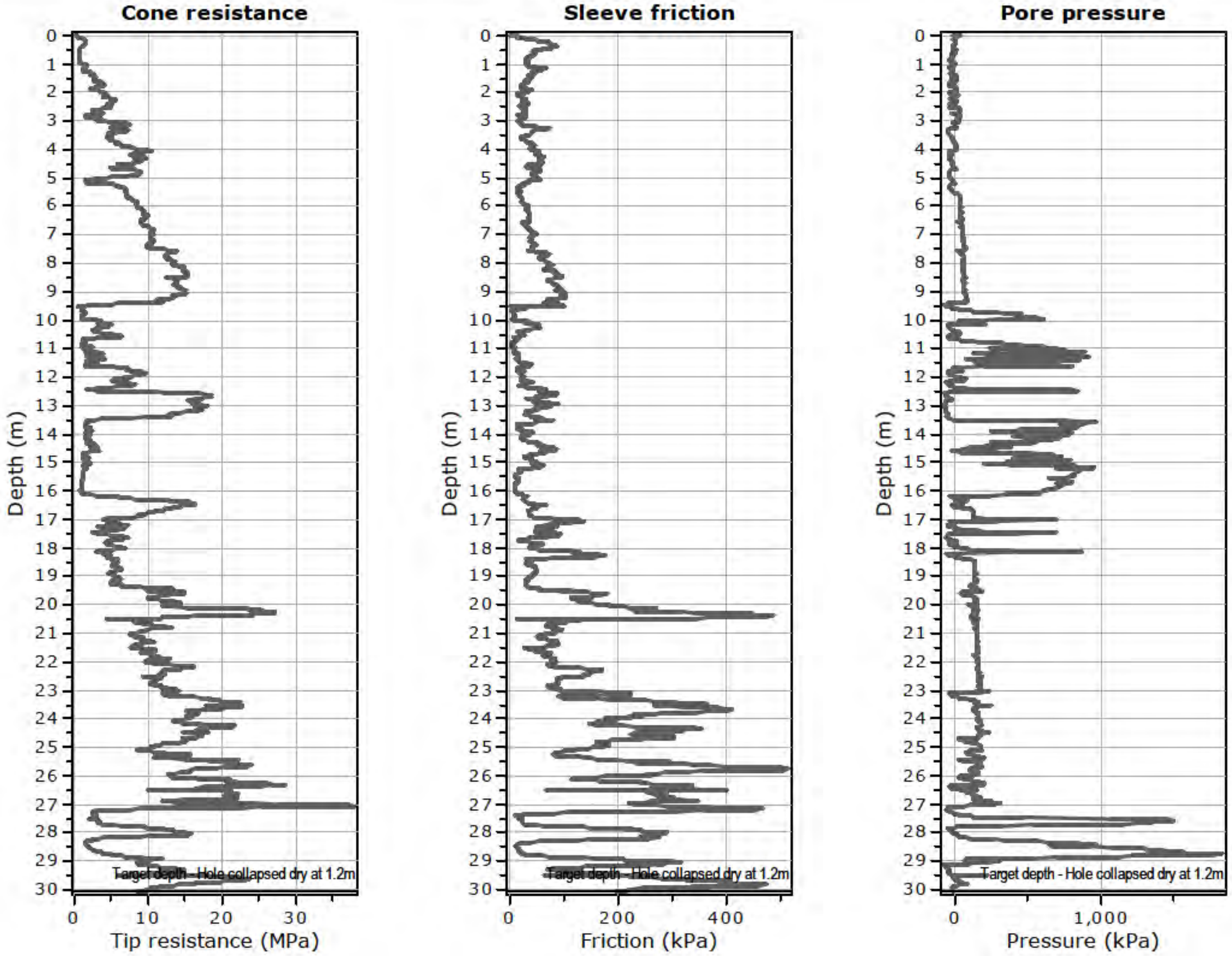


Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

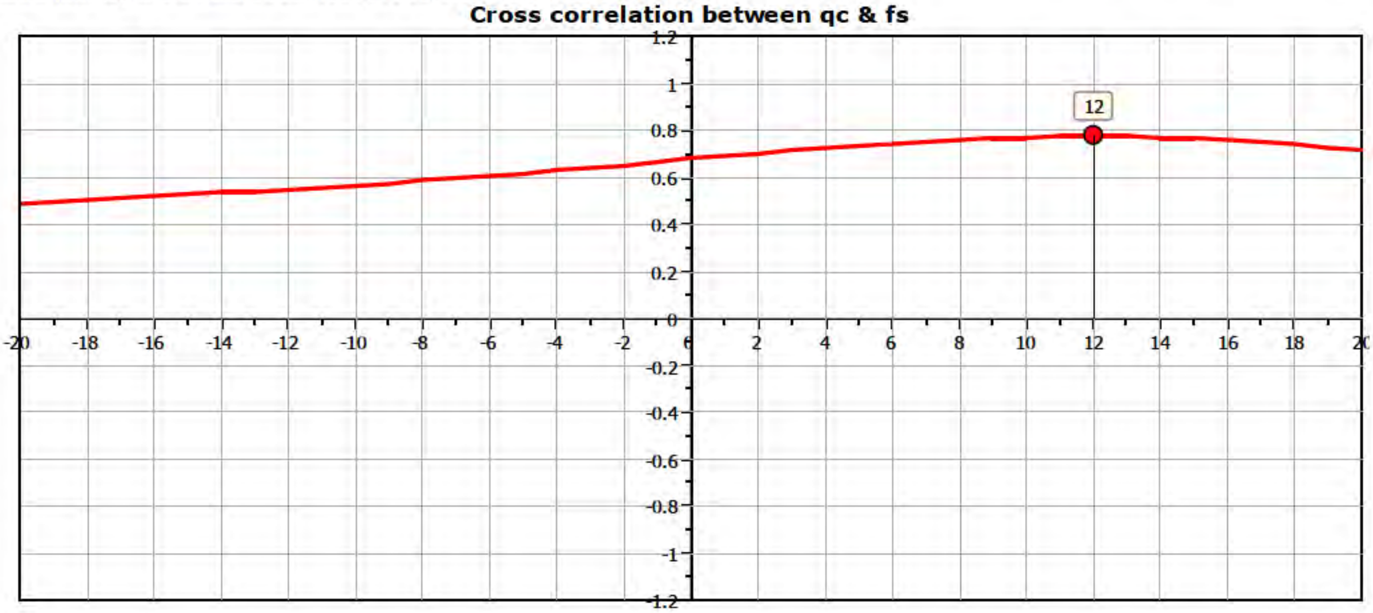
CPT: 24-07

Total depth: 30.07 m, Date: 4/06/2024
Coords: lat -37.82305° lon 175.750916°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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Email: Josh@gdsnz.co. nz
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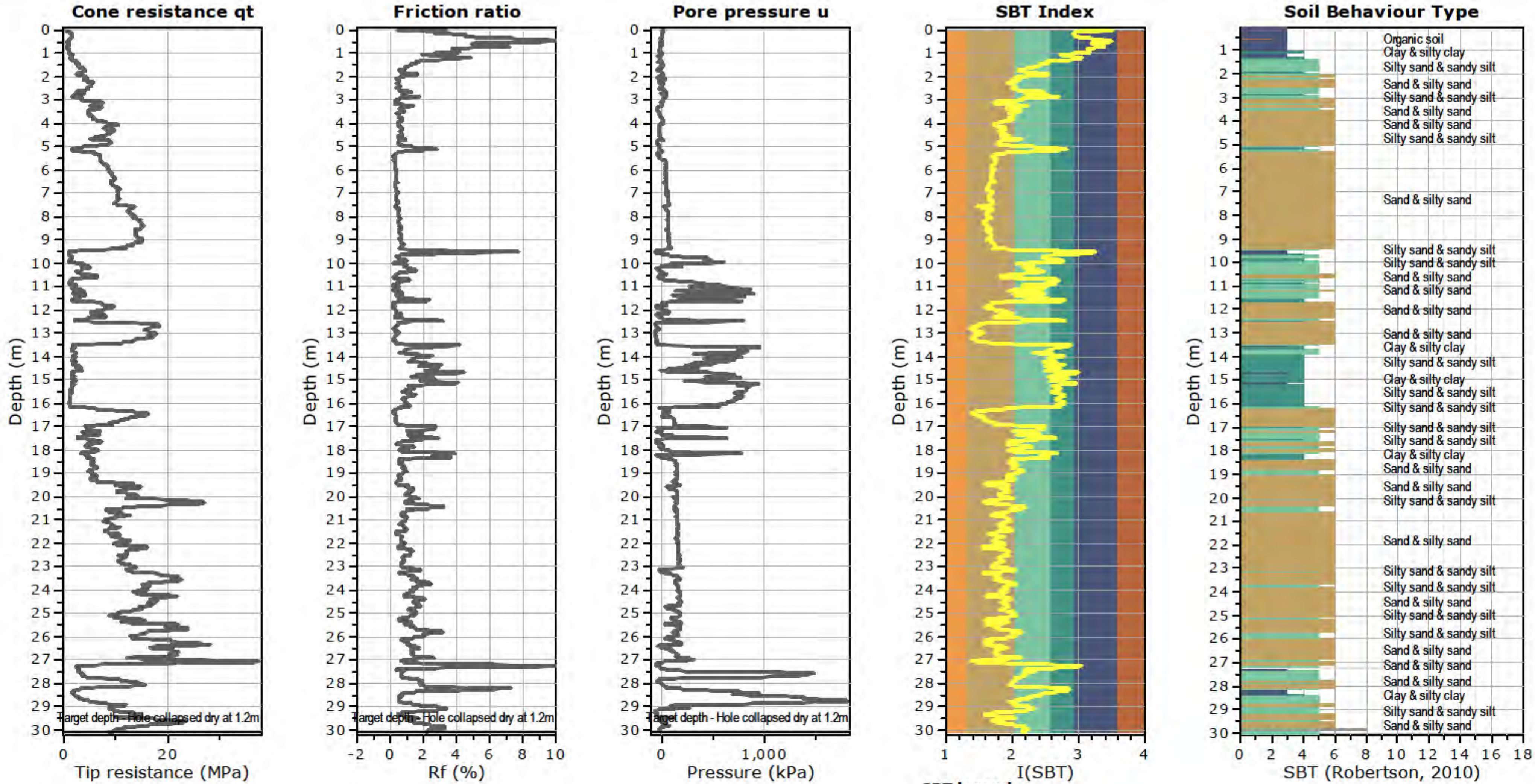
CPT: 24-07

Total depth: 30.07 m, Date: 4/06/2024
Coords: lat -37.82305° lon 175.750916°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



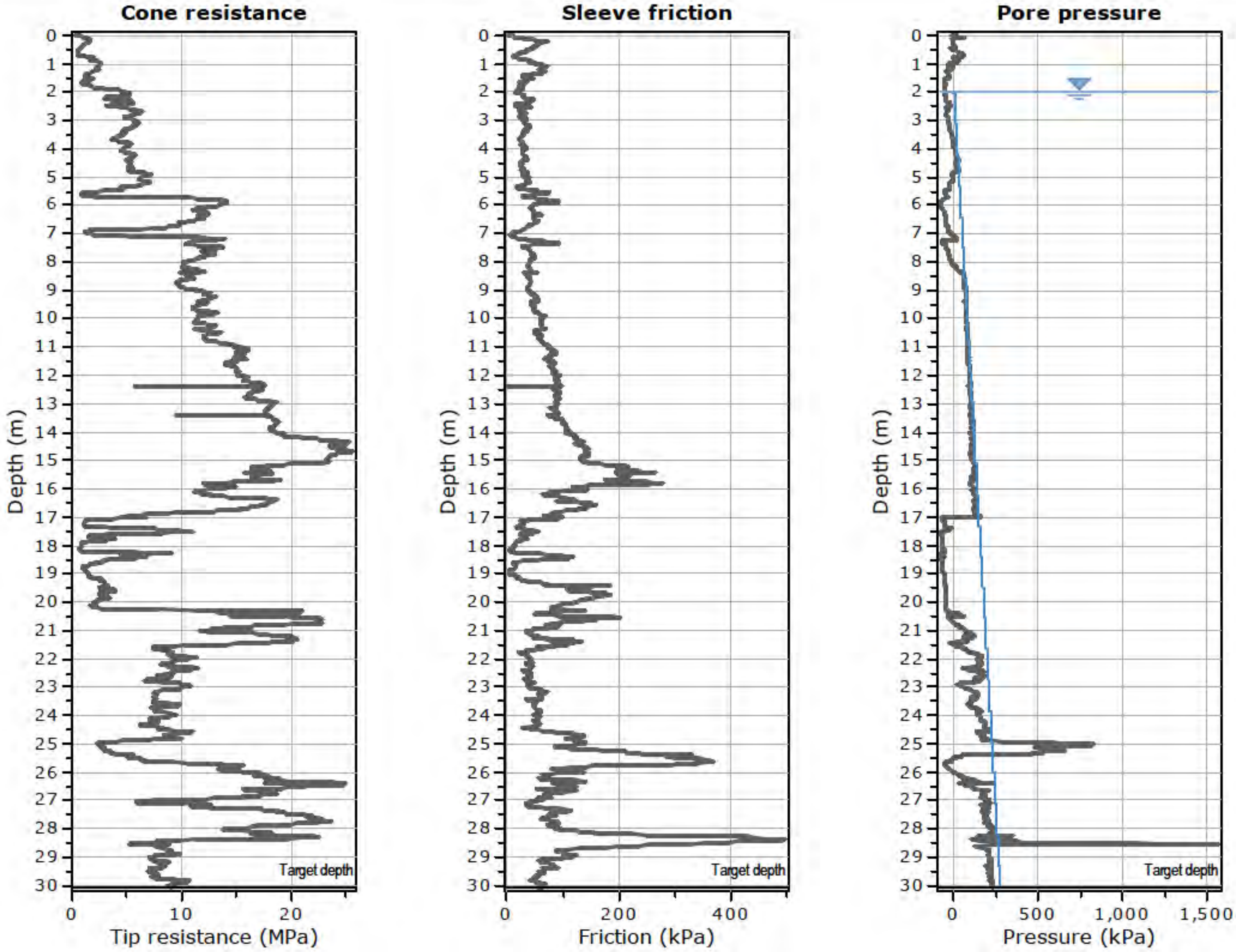
SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

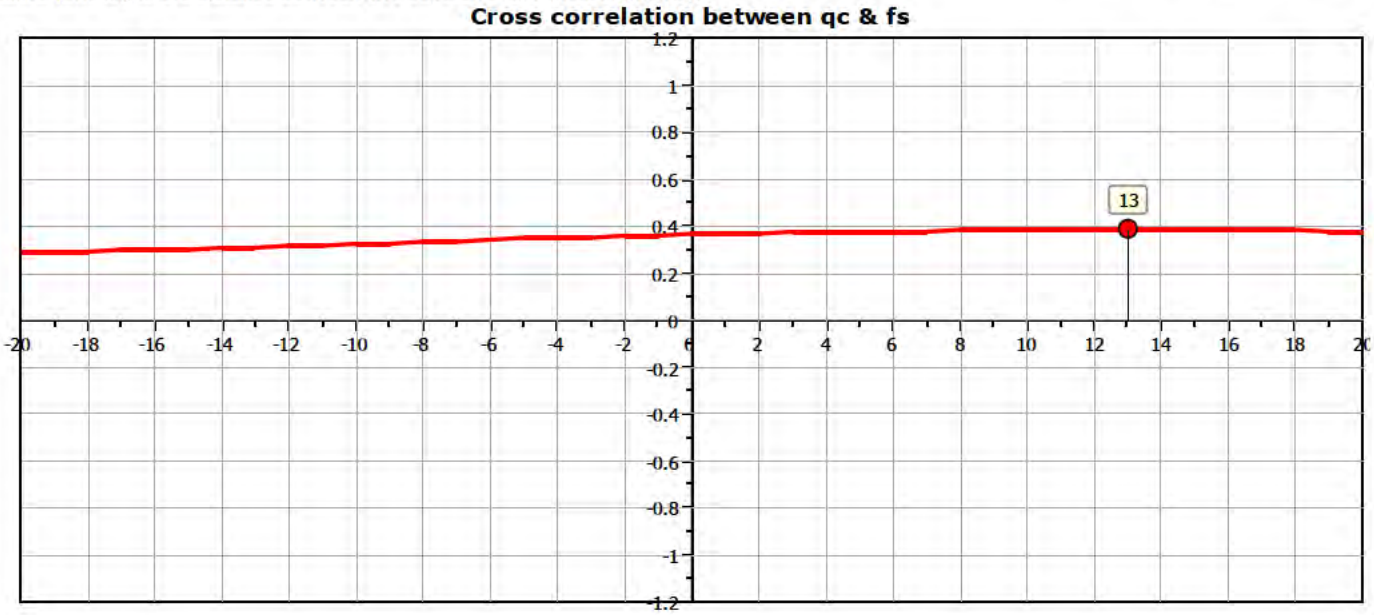
Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: 24-08
Total depth: 30.08 m, Date: 4/06/2024
Coords: lat -37.82359° lon 175.754635°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



Geo Data Solutions (GDS) NZ Ltd.
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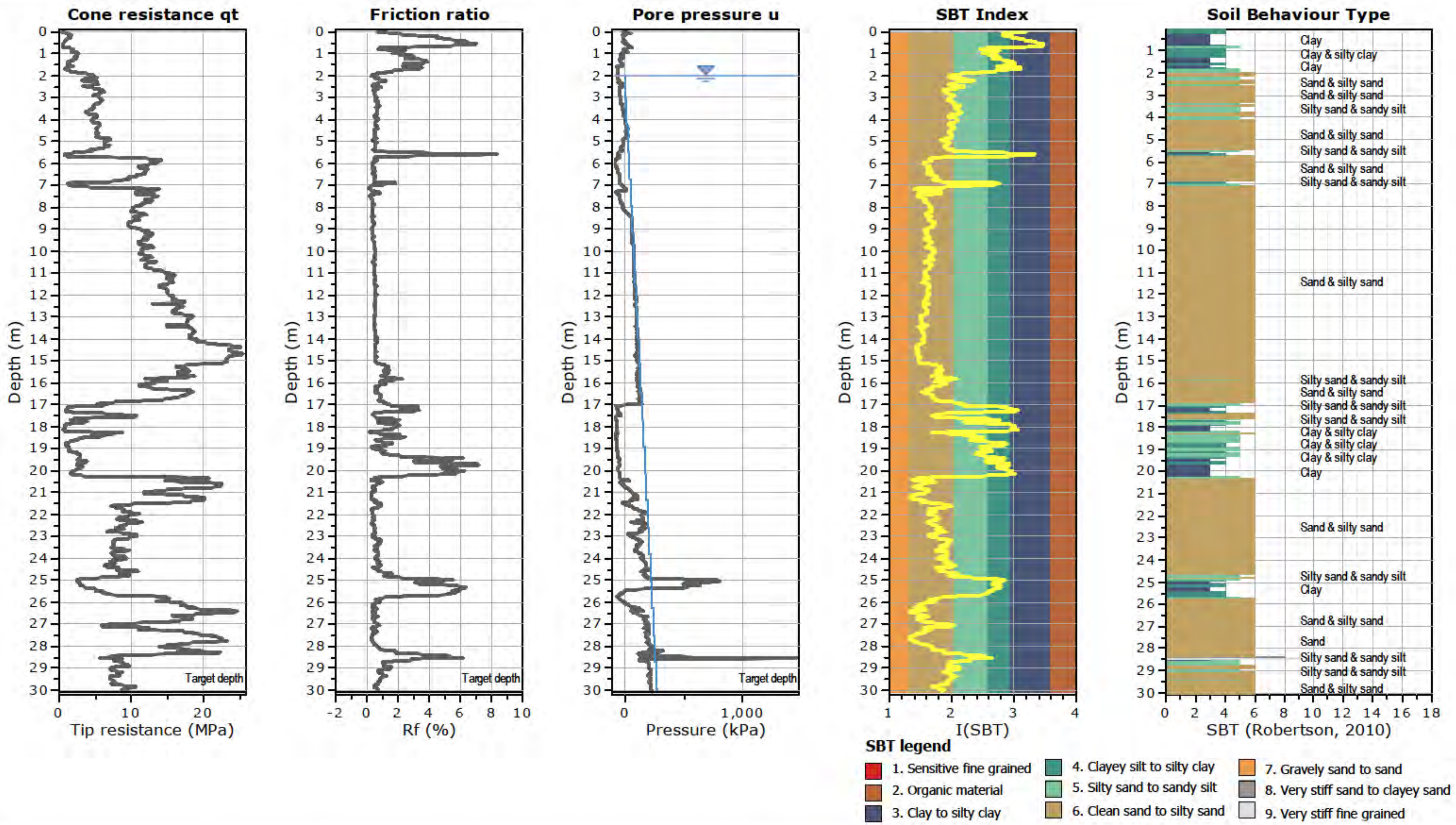
CPT: 24-08

Total depth: 30.08 m, Date: 4/06/2024
Coords: lat -37.82359° lon 175.754635°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter

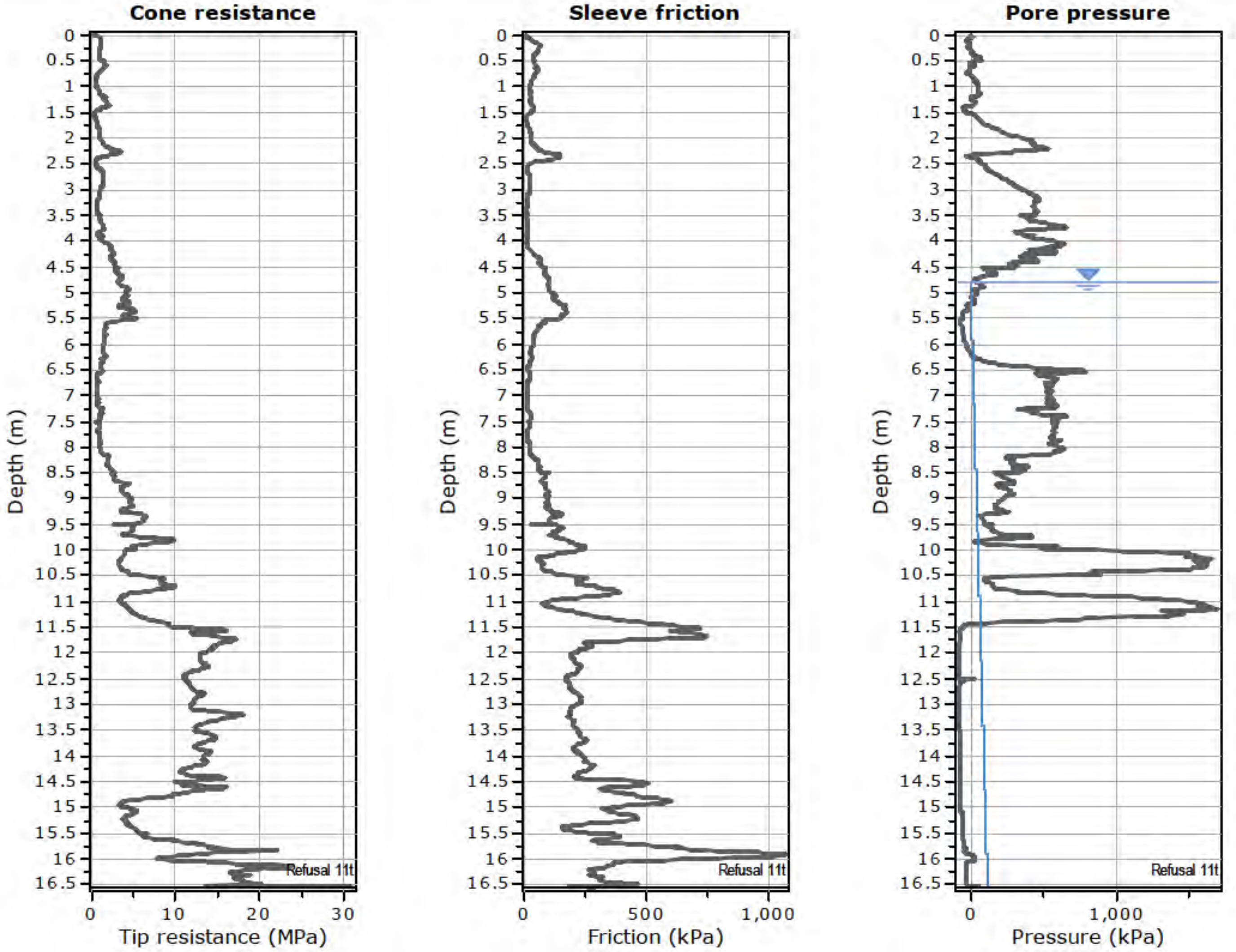


Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

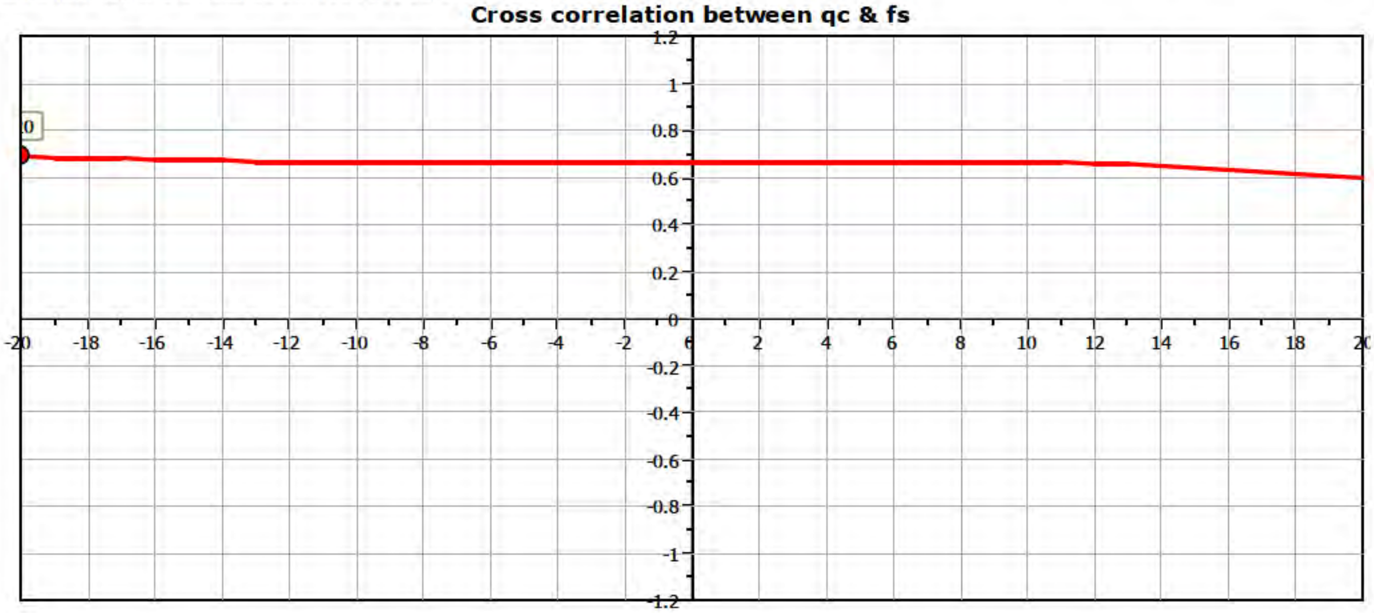
CPT: 24-09

Total depth: 16.56 m, Date: 4/06/2024
Coords: lat -37.824685° lon 175.748286°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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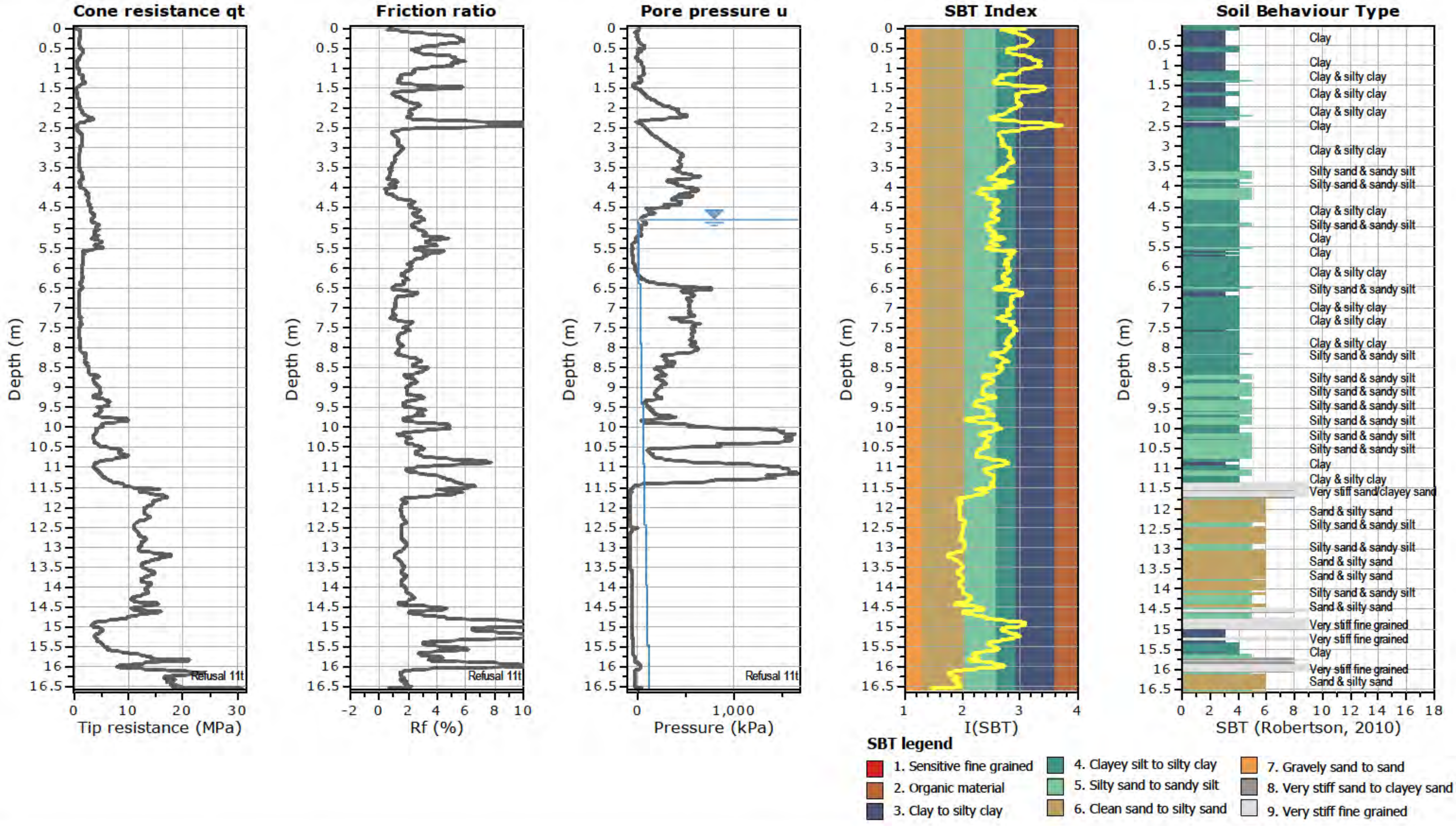
CPT: 24-09

Total depth: 16.56 m, Date: 4/06/2024
Coords: lat -37.824685° lon 175.748286°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter

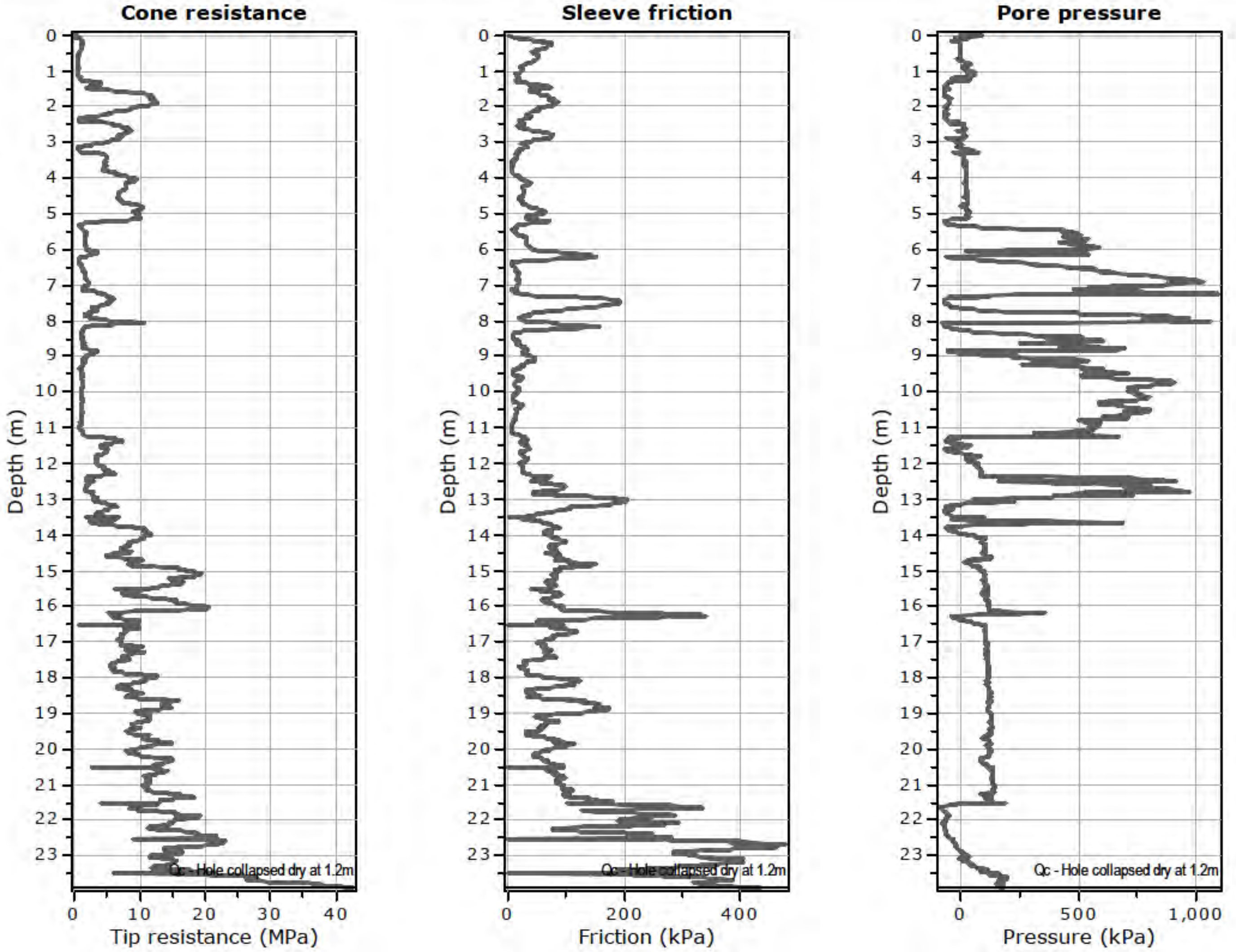


Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: 24-10

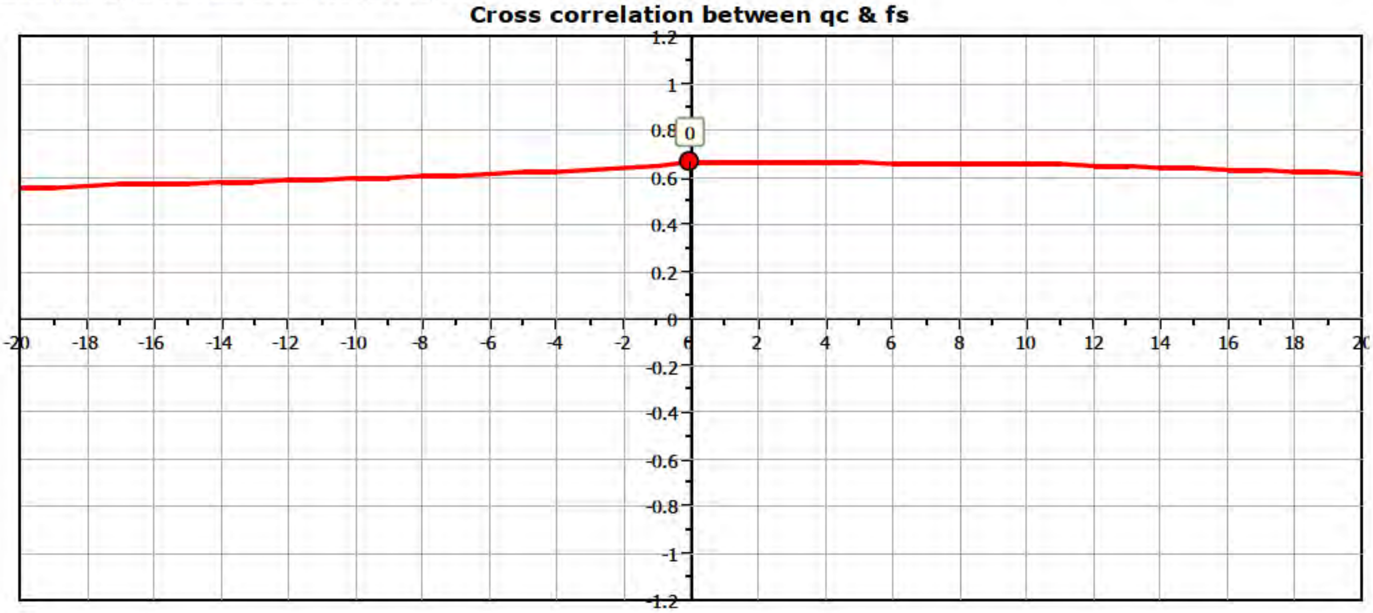
Total depth: 23.89 m, Date: 4/06/2024
Coords: lat -37.82614° lon 175.752408°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

Cross correlation between qc & fs



Geo Data Solutions (GDS) NZ Ltd.
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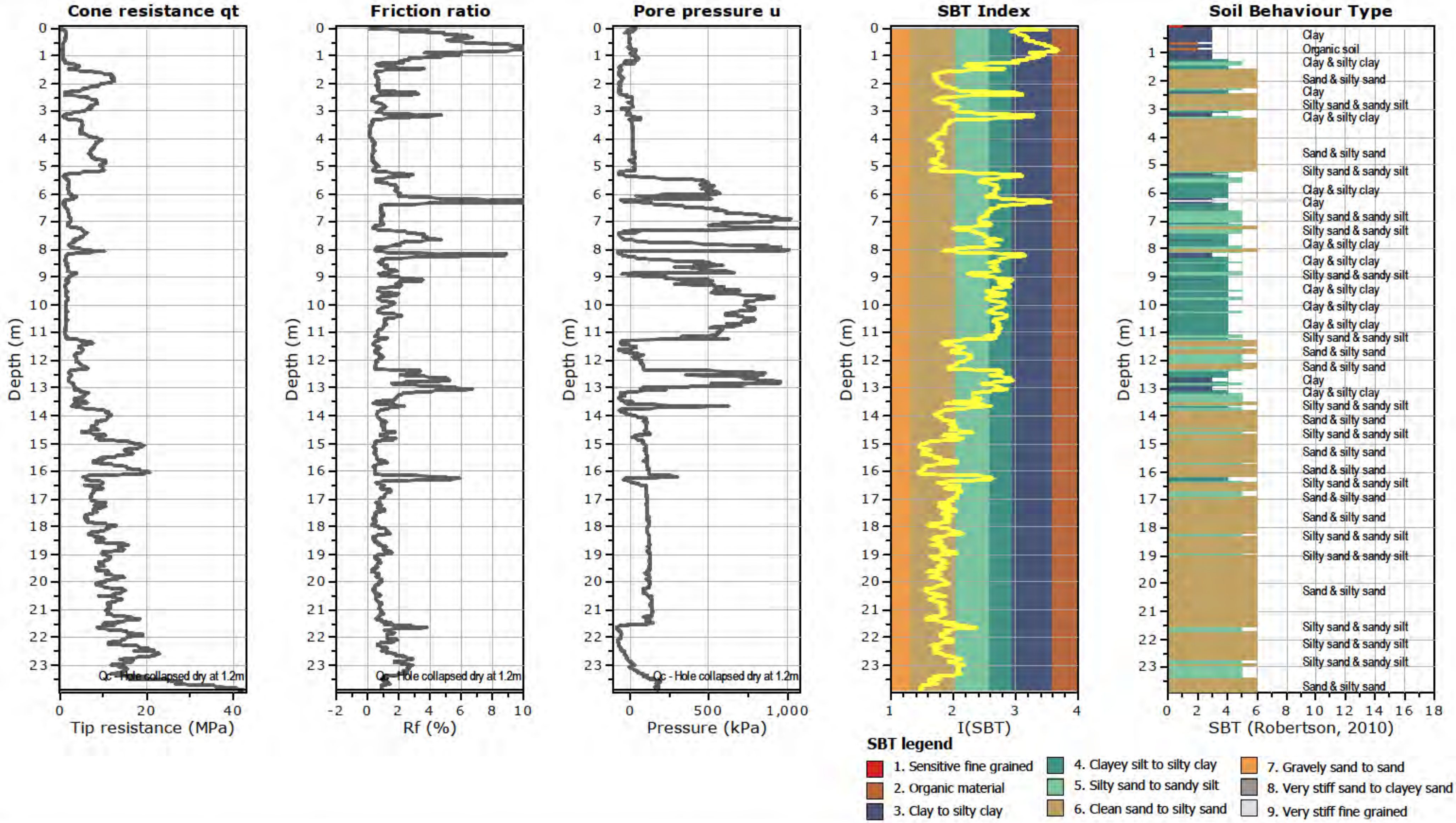
CPT: 24-10

Total depth: 23.89 m, Date: 4/06/2024
Coords: lat -37.82614° lon 175.752408°
Cone Type: DC10

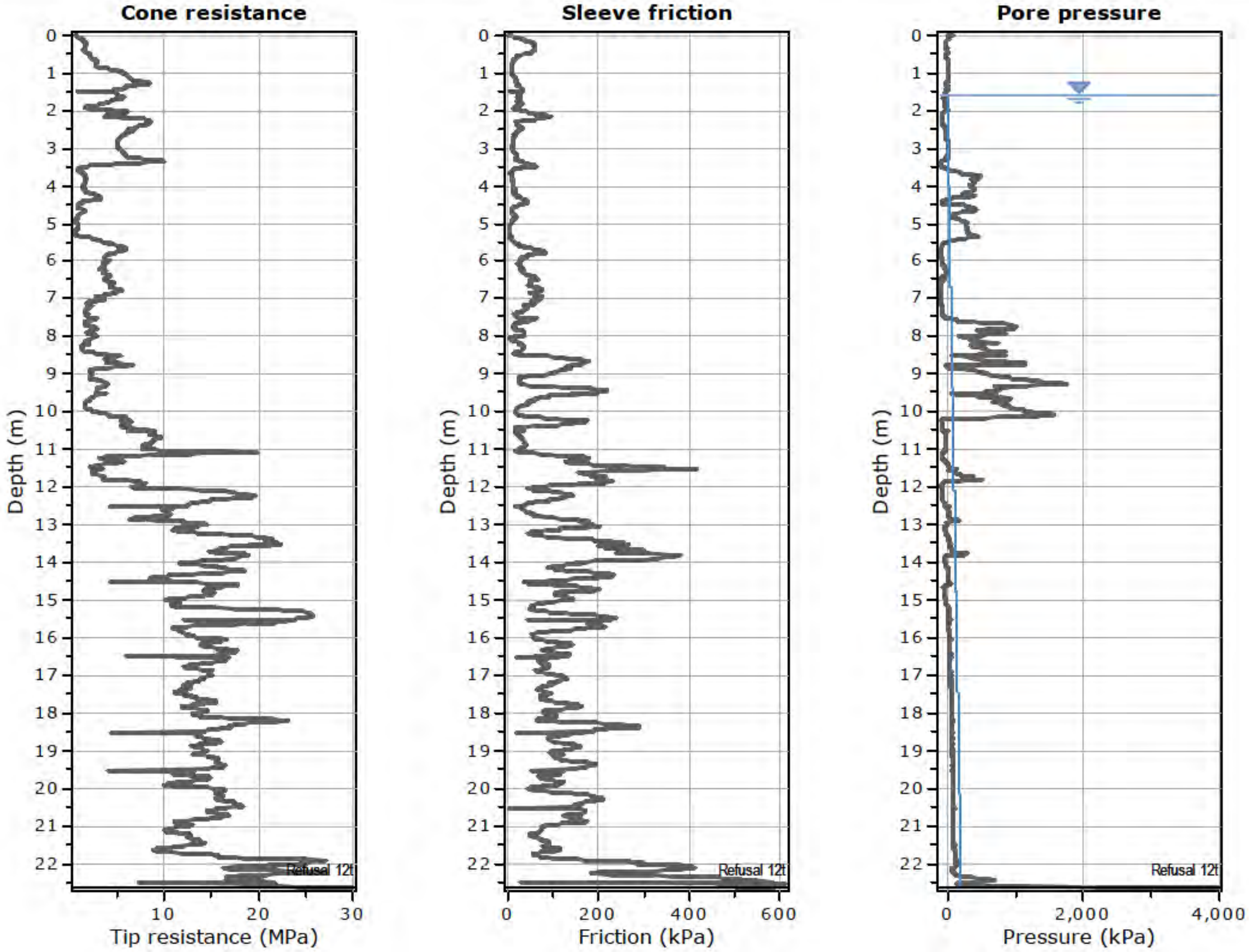
Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



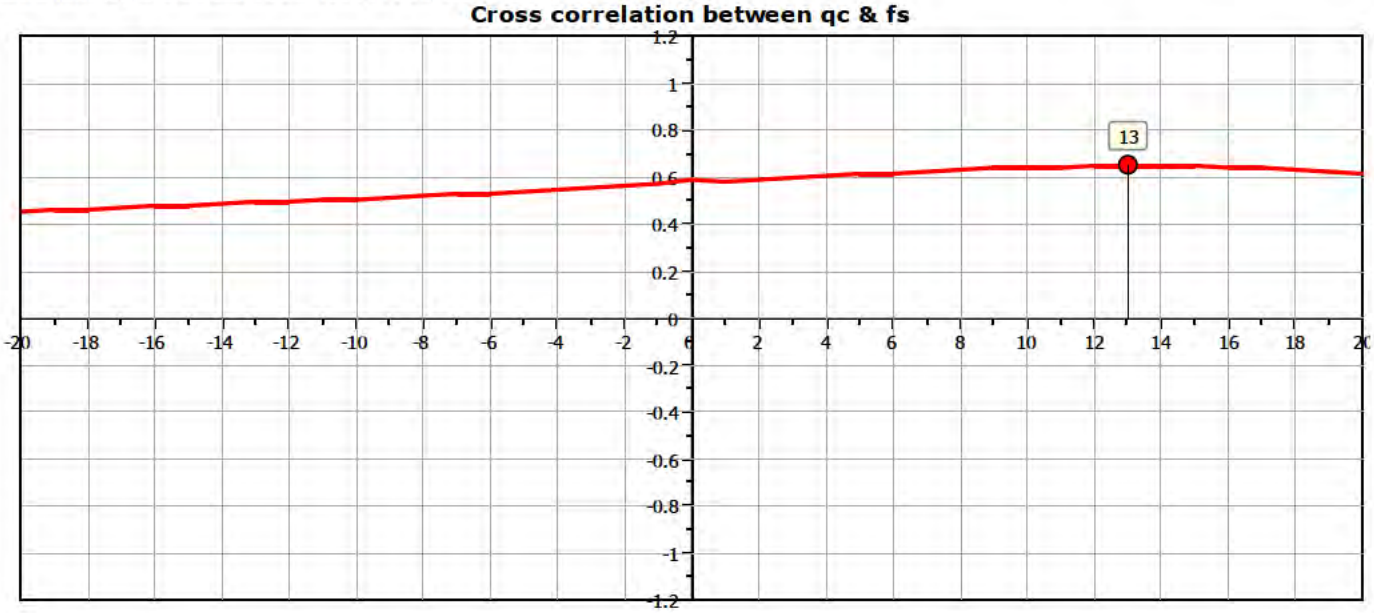
Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: SCPT24-01
Total depth: 22.64 m, Date: 4/06/2024
Coords: lat -37.820766° lon 175.747979°
Cone Type: DC10

Project: **CMW Geosciences | HAM2023-0124 | GDS NZ Ltd**
Location: **127 Station Road, Matamata | Holes dipped onsite using Dipmeter**



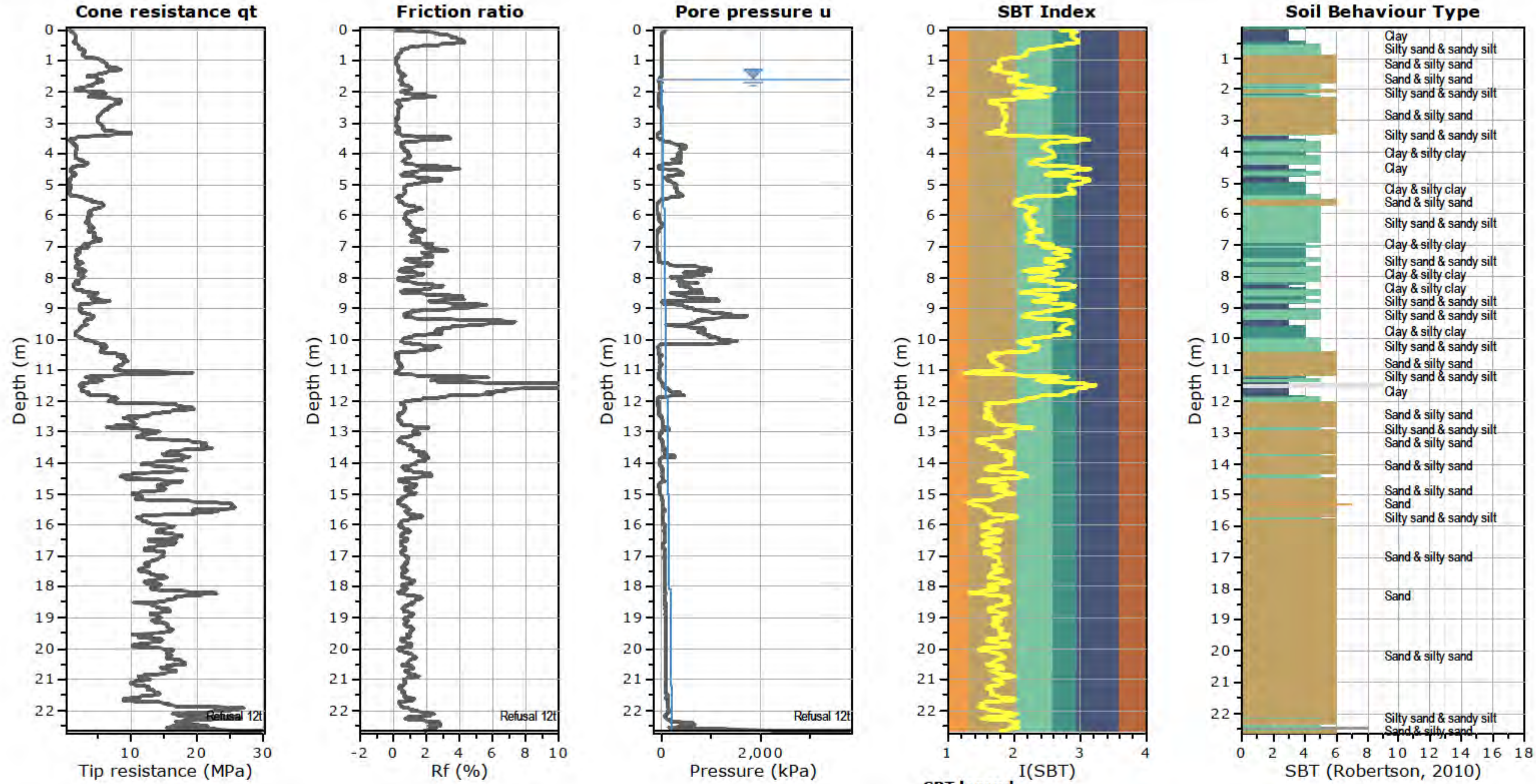
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: SCPT24-01
Total depth: 22.64 m, Date: 4/06/2024
Coords: lat -37.820766° lon 175.747979°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Geo Data Solutions (GDS) NZ Ltd.
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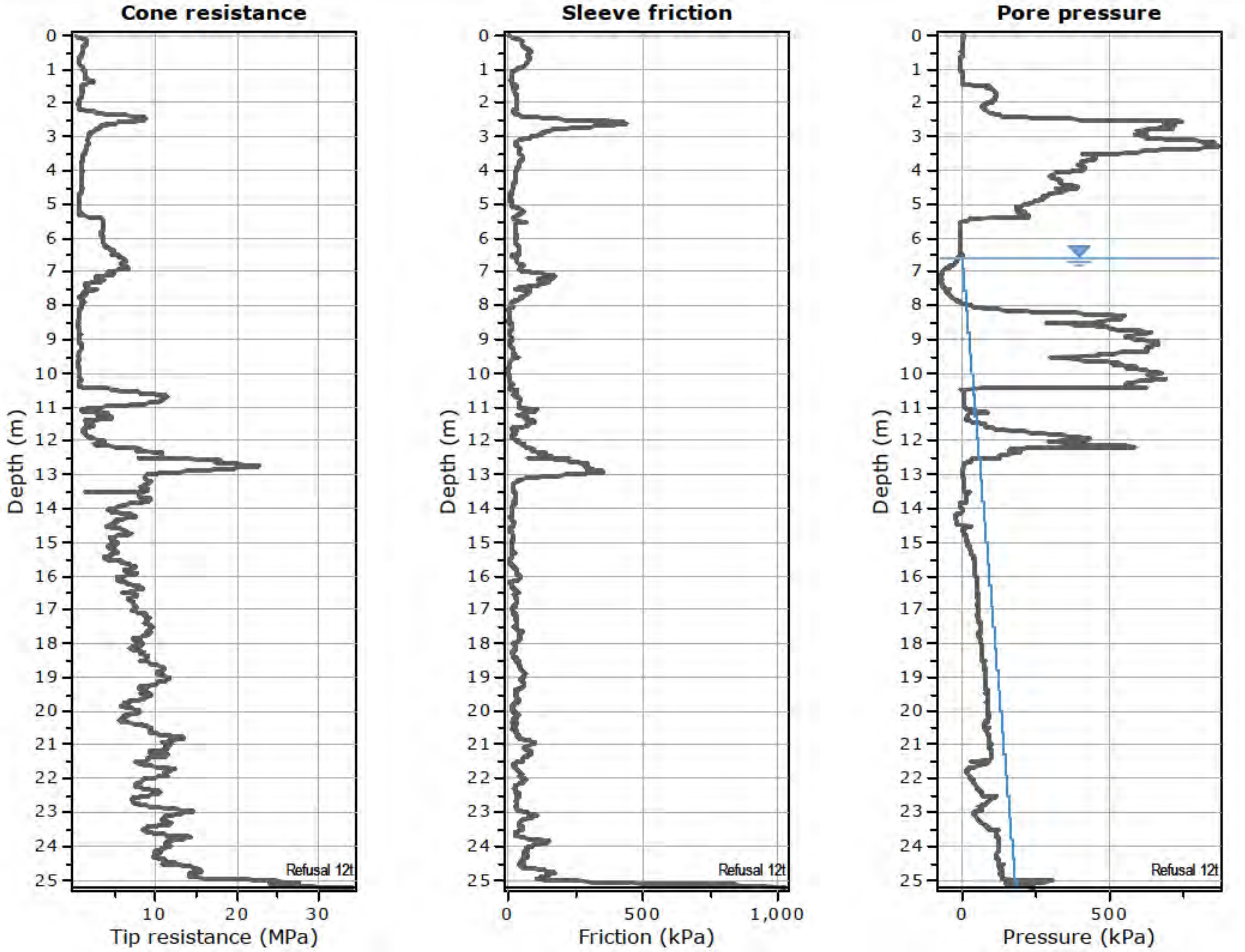
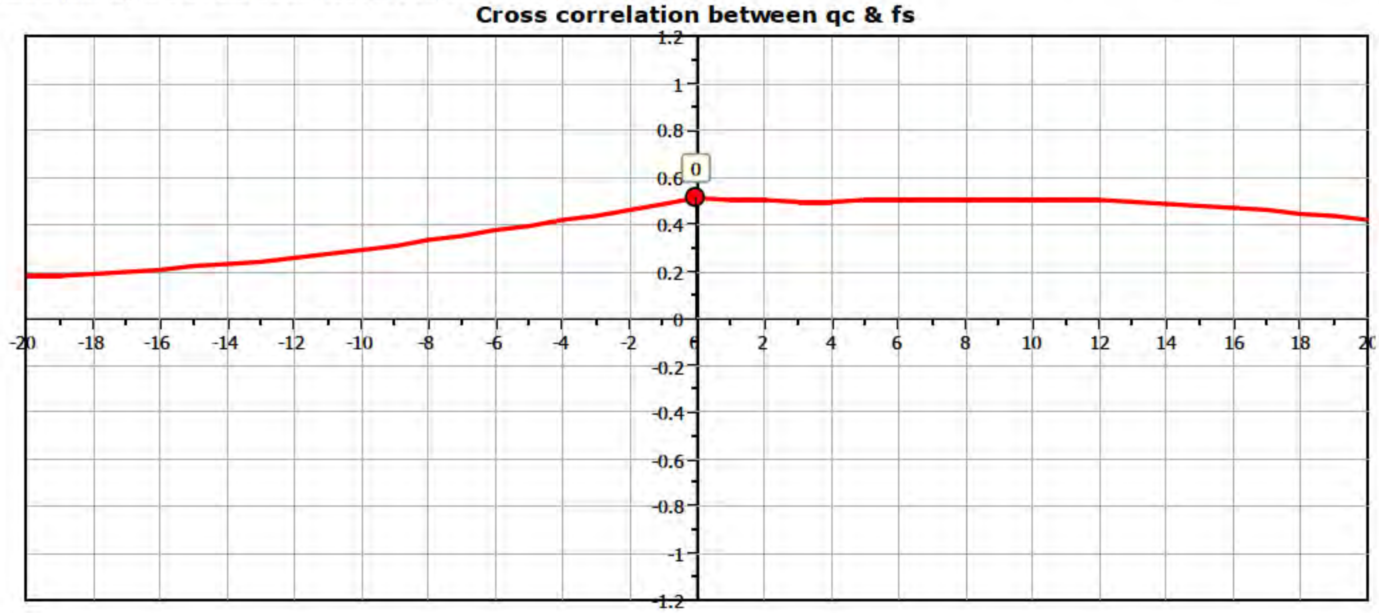
CPT: SCPT24-02

Total depth: 25.23 m, Date: 4/06/2024
Coords: lat -37.822827° lon 175.745404°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



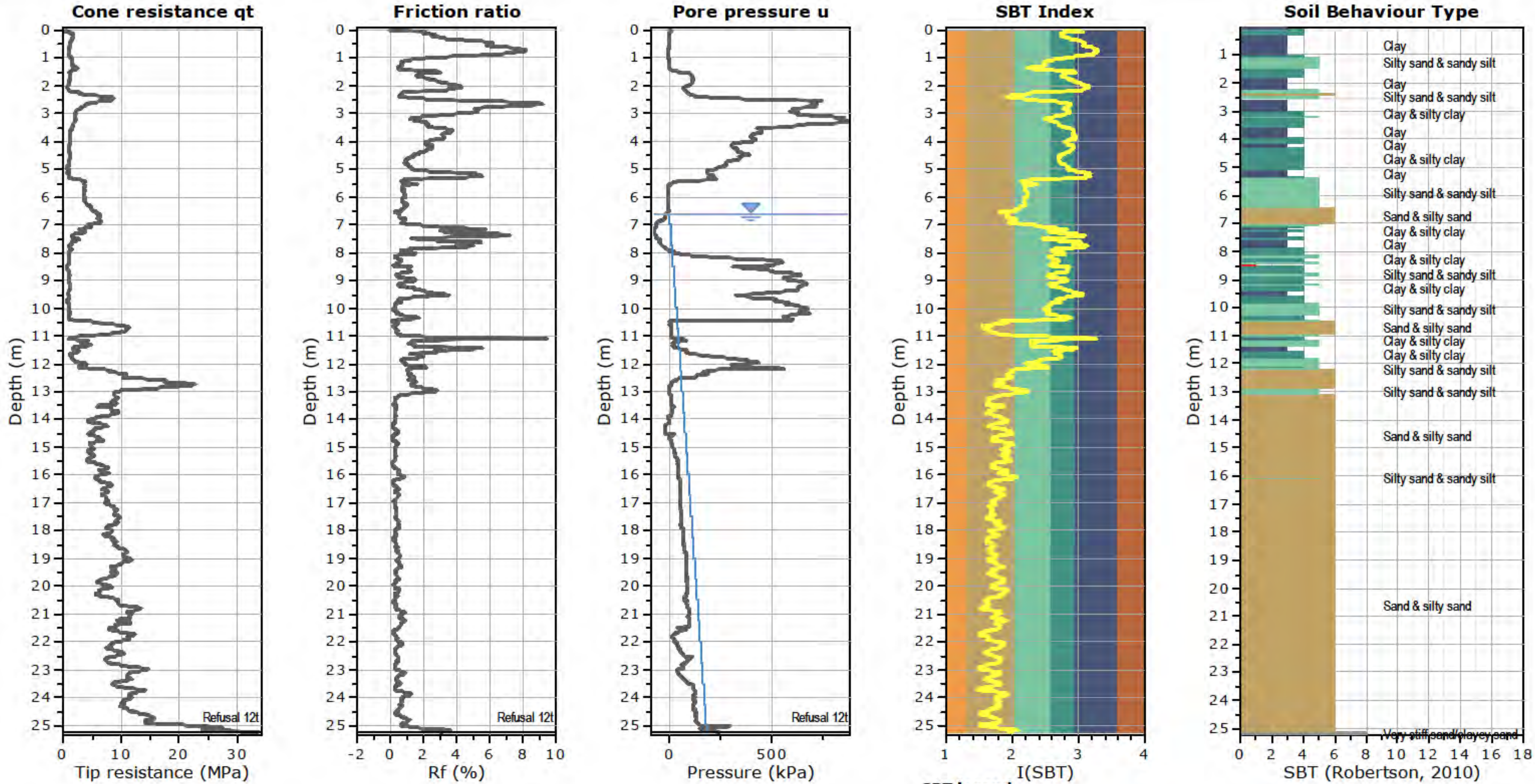
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: SCPT24-02
Total depth: 25.23 m, Date: 4/06/2024
Coords: lat -37.822827° lon 175.745404°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



SBT legend

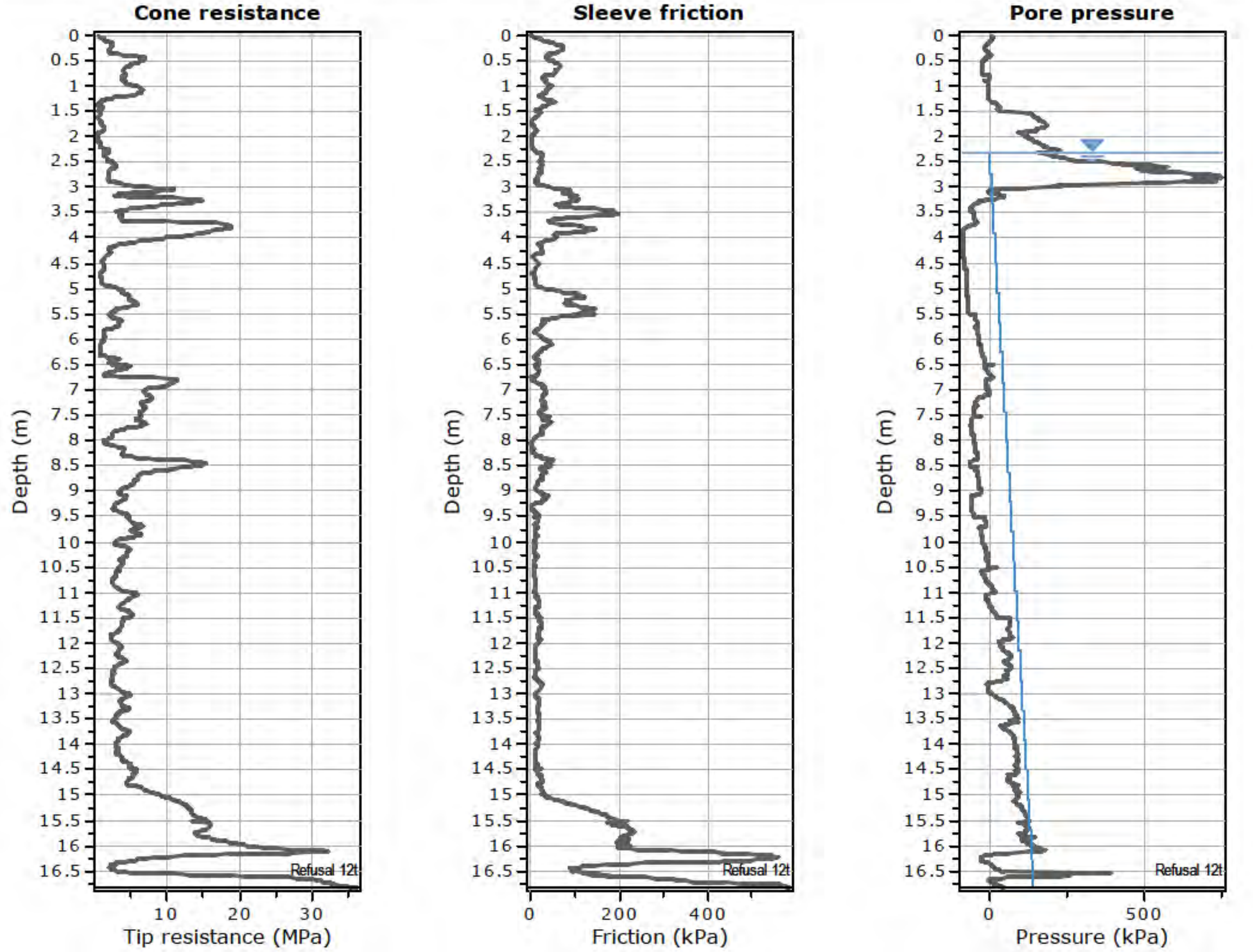
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

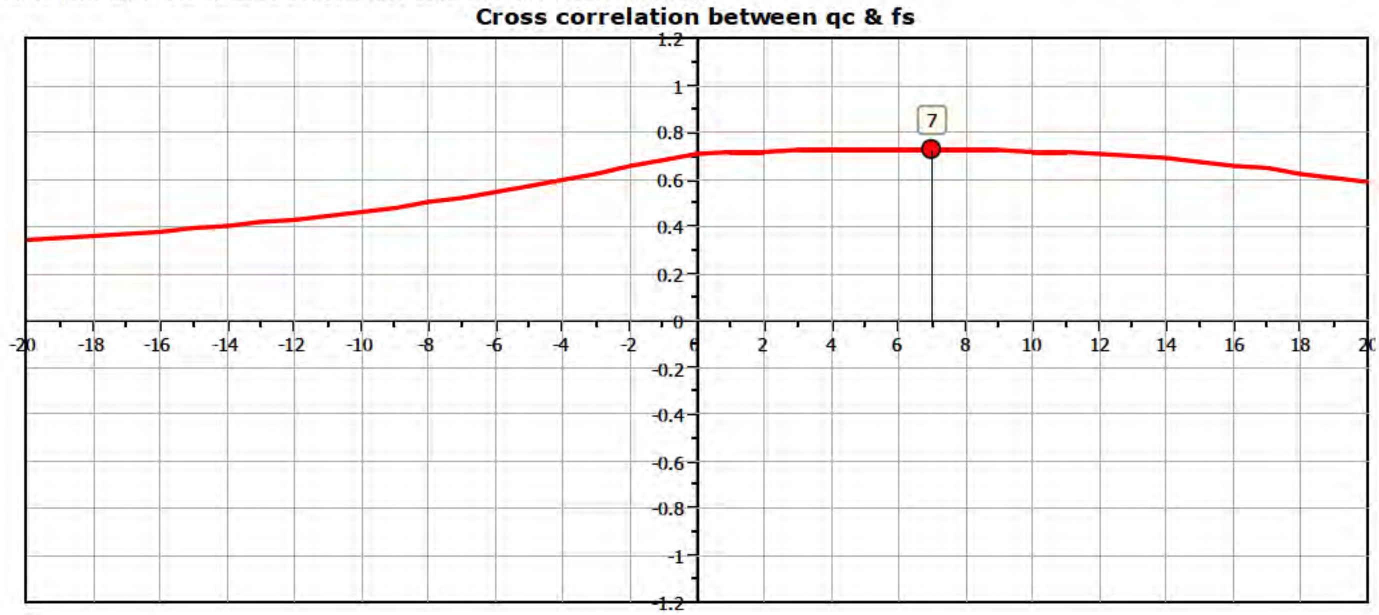
CPT: SCPT24-03

Total depth: 16.83 m, Date: 4/06/2024
Coords: lat -37.825528° lon 175.745502°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

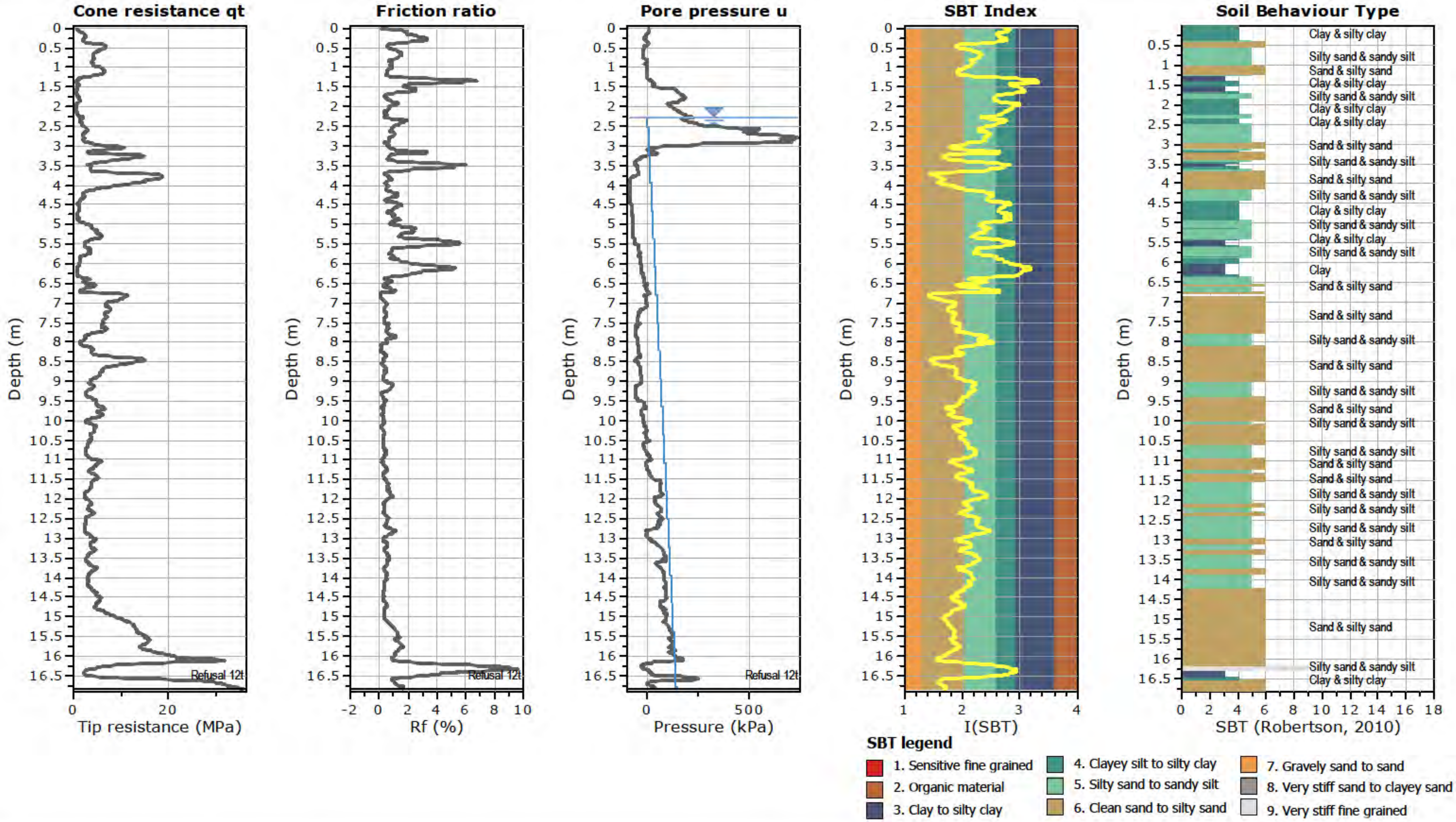


Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: SCPT24-03

Total depth: 16.83 m, Date: 4/06/2024
Coords: lat -37.825528° lon 175.745502°
Cone Type: DC10

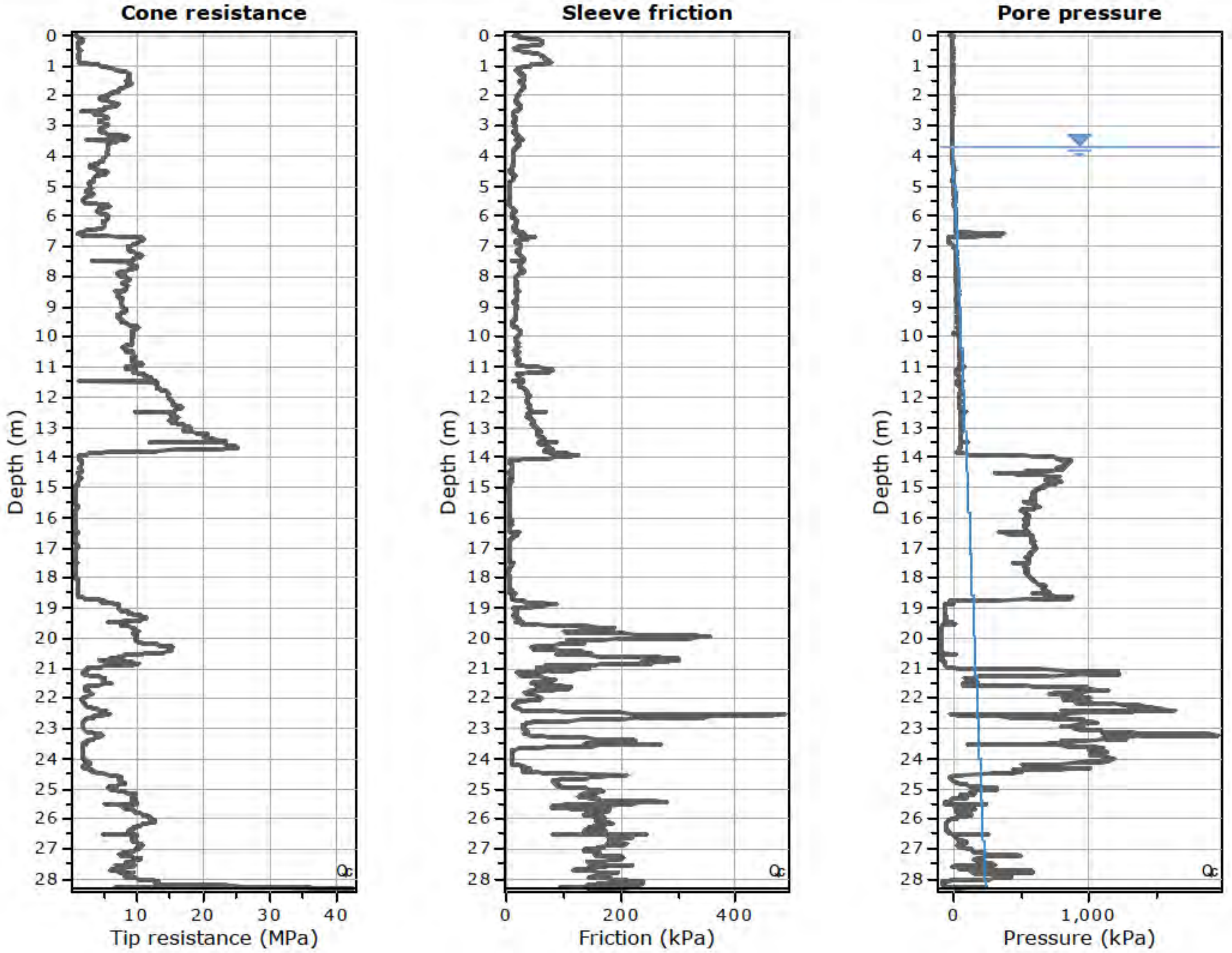
Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



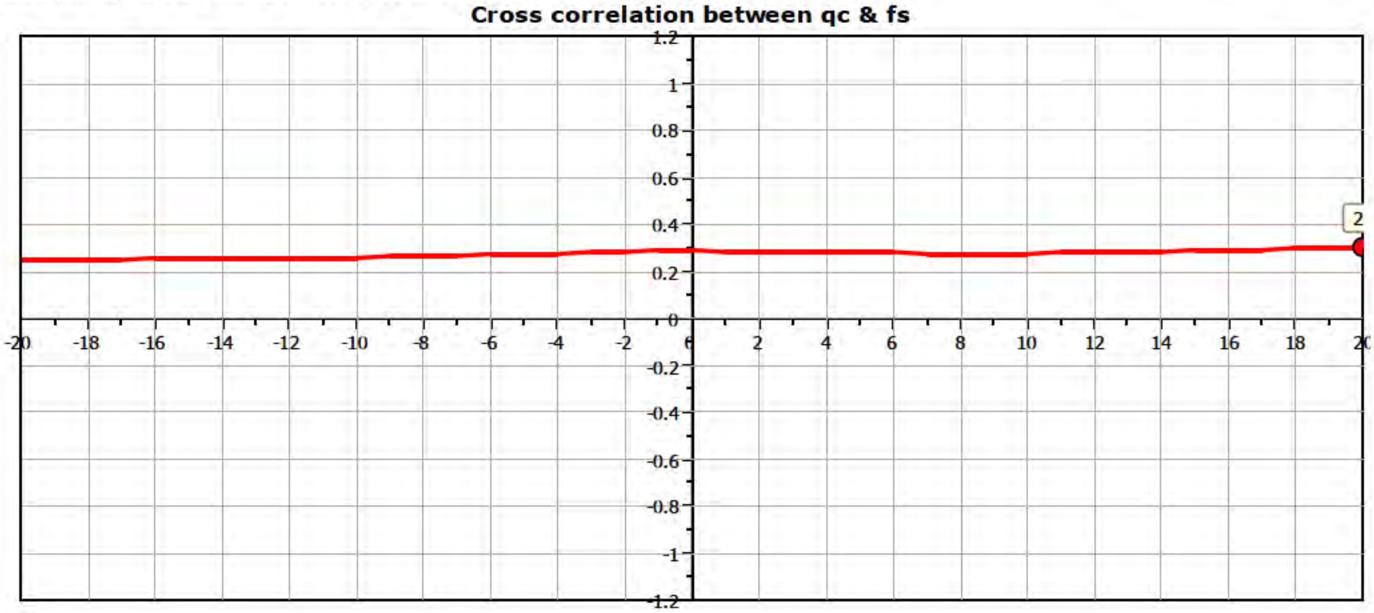
Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: SCPT24-04
Total depth: 28.31 m, Date: 4/06/2024
Coords: lat -37.825977° lon 175.756443°
Cone Type: DC10

Project: **CMW Geosciences | HAM2023-0124 | GDS NZ Ltd**
Location: **127 Station Road, Matamata | Holes dipped onsite using Dipmeter**



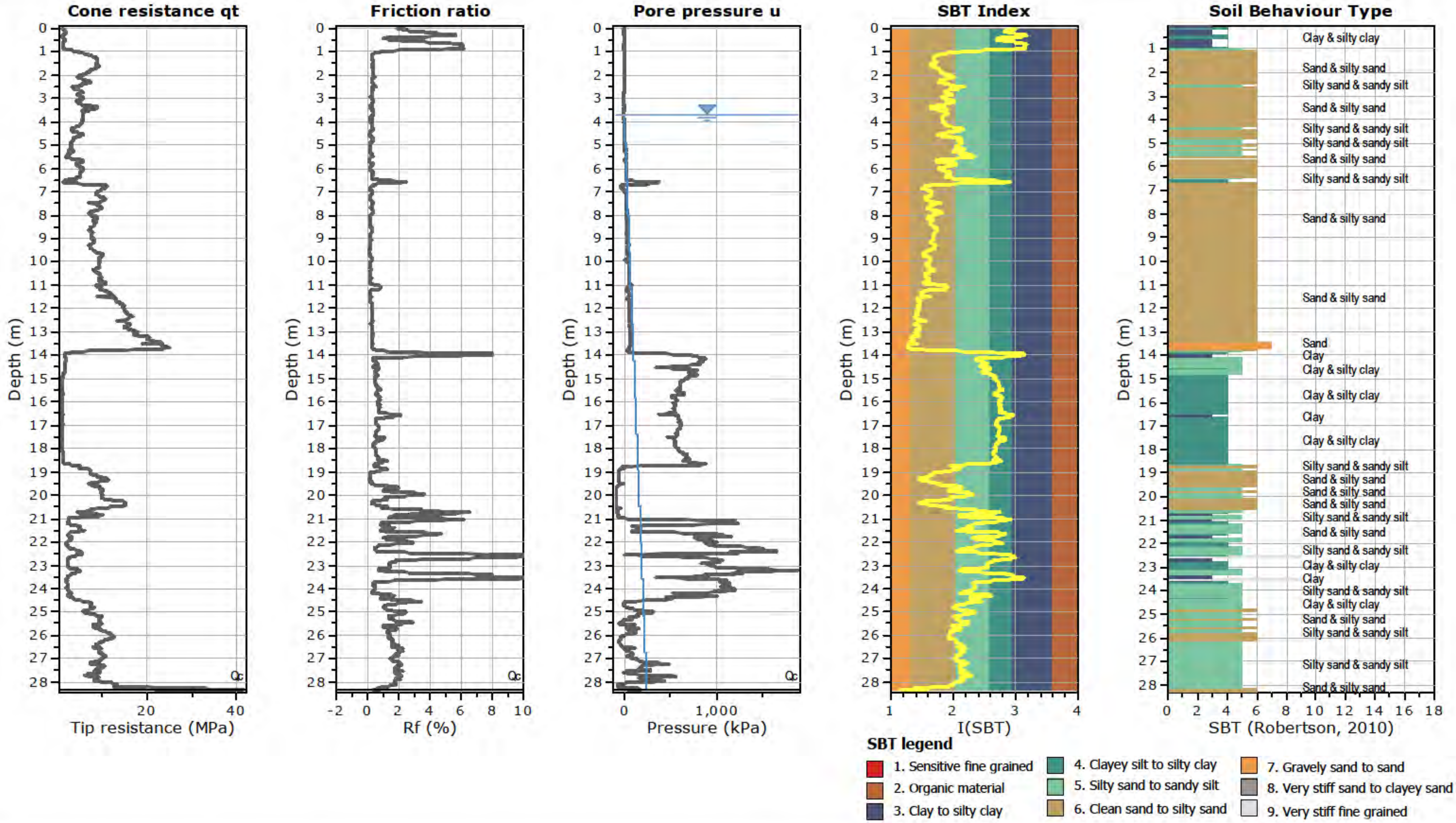
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



Geo Data Solutions (GDS) NZ Ltd.
Email: Josh@gdsnz.co. nz
www.gdsnz.co.nz

CPT: SCPT24-04
Total depth: 28.31 m, Date: 4/06/2024
Coords: lat -37.825977° lon 175.756443°
Cone Type: DC10

Project: CMW Geosciences | HAM2023-0124 | GDS NZ Ltd
Location: 127 Station Road, Matamata | Holes dipped onsite using Dipmeter



APPENDIX F: GEOHAZARDS ASSESSMENT TABLE

Appendix F - Geohazard Assessment Summary												
Item	Geotechnical Hazard	Description	Area Affected	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
					Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
1	Earthquake	Seismicity	Entire Site	Site subsoil class = Class D due to less than dense/ stiff soil profile Importance Level = 2 (Residential Subdivision) Importance Level = 3 (Solar Farm)	1	5	5	None	1	5	5
		Fault Rupture	Entire Site	Nearest active fault (Kerpehi fault) is approximately 5km from the site. (Refer to section 6.2)	1	5	5	Mitigation not required	1	5	5
		Liquefaction	Whole site	Refer to section 6.3	3	4	12	TC2 or TC2-TC3 hybrid foundations required.	3	2	6
		Cyclic Softening	Entire Site	Not anticipated.	1	4	4	Mitigation not required.	1	4	4
		Lateral Spread	Swale drains, proposed greenway and riverbanks	Refer to Section 6.5	3	4	12	Lateral spreading risk should be considered in the design, further investigation and analysis will be required.	3	2	6

Appendix F - Geohazard Assessment Summary											
Item	Geotechnical Hazard	Description	Area Affected	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
					Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
2	Volcanic Activity	Ash and Pyroclastic Falls	Entire Site	Nearest active volcano is the Rotorua Caldera. Currently at alert level 0.	1	5	5	Mitigation not required.	1	5	5
3	Slope Instability / Landslide	Global Instability	Entire site	Due to the landform being generally near level to gently sloping, slope instability is not anticipated.	1	4	4	Mitigation not required.	1	4	4
4	Problematic Soils	Expansive Soils	Entire Site	Experience in similar soils indicate that the soils on site are non-expansive.	2	3	6	Mitigation not required.	2	3	6
5	Settlement	Compressible Soils	Entire Site	No compressible soils were encountered on site.	1	4	4	Mitigation not required.	1	4	4
		Fill Induced Settlement	Entire Site	No earthwork plans are available at the time of writing report. Site is primarily underlain with dense to very dense sand and stiff to very stiff silt. It is anticipated any fill induced settlement would occur immediately and be built out during construction.	2	4	8	Mitigation not required based on current details. Will have to be assessed once cut/ fill plans are available.	1	4	4

Appendix F - Geohazard Assessment Summary											
Item	Geotechnical Hazard	Description	Area Affected	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
					Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
6	Bearing Capacity	Bearing Capacity Failure	Building Platform	Refer to Section 7.4 of the report	1	5	5	A preliminary geotechnical ultimate bearing capacity (GUBC) of 300kPa should be available in the static case. Low ultimate bearing capacity is anticipated in the seismic case.	1	5	5
7	Construction Risks	Excavatability	Building Platform	Given the density of the soil units that will be encountered, excavation is expected to be readily achieved with normal earthworks plant. However, excavations may require temporary support due to high expected groundwater and granular soils leading to 'running sands'.	3	3	9	Consideration to be given to sensitive silts and shallow groundwater table.	3	2	6
		Sediment Retention Ponds	Building Platform	Sediment retention ponds will require geotechnical input at design stage to ensure batter stability is achievable.	2	3	6	Consideration to be given to batter stability of proposed ponds at design and construction phase.	2	2	4
		Stockpile locations	Building Platform	Stockpiles to be away from the river bank.	1	2	2	Stockpiles to be away from the river bank.	1	2	2

Appendix F - Geohazard Assessment Summary											
Item	Geotechnical Hazard	Description	Area Affected	Assessment Outcome	Existing Risk of Damage to Land / Structures			Mitigation Measure	Residual Risk of Damage to Land / Structures		
					Likelihood	Consequence	Risk Rating		Likelihood	Consequence	Risk Rating
		Subgrade Preparation	Building Platforms and Road Alignment	Topsoil and existing vegetation within the building footprints and road alignments will be cleared as part of the proposed development earthworks.	1	2	2	Mitigation not required.	1	2	2
		Service Trenches (trench collapse / long term settlement)	Building Platform and Road Alignment	Trench collapse may occur in surficial soils / if proposed service trenches extend below GW level.	3	3	9	Mitigation should be considered in the form of: - trench support - temporary dewatering, in the form of regularly spaced pumps	3	1	3

GEOHAZARD ASSESSMENT FOR LAND SUBDIVISION

Station Road, Matamata

The occurrence of natural hazards and their potential impacts on the proposed subdivision development is assessed in terms of risk significance, which is based on likelihood and consequence factors. A risk table is used to help assess the likelihood and consequence factors, the form of which used by CMW for this project is presented in Table B1.

Table B1: Natural Hazard Risk Classification

Risk Matrix		Consequence				
		Insignificant 1	Minor 2	Moderate 3	Major 4	Catastrophic 5
Likelihood	Almost Certain 5	Medium 5	High 10	Very high 15	Extreme 20	Extreme 25
	Likely 4	Low 4	Medium 8	High 12	Very high 16	Extreme 20
	Moderate 3	Low 3	Medium 6	Medium 9	High 12	Very high 15
	Unlikely 2	Very low 2	Low 4	Medium 6	Medium 8	High 10
	Rare 1	Very low 1	Very low 2	Low 3	Low 4	Medium 5

1.1 Likelihood

With respect to assessing the likelihood or chance of the risk occurring, the qualitative definitions used by CMW for this project are provided in Table B2 for each likelihood classification.

Table B2: Qualitative Natural Hazard Likelihood Definitions

1	Rare	The natural hazard is not expected to occur during the design life of the project
2	Unlikely	The natural hazard is unlikely, but may occur during the design life
3	Moderate	The natural hazard will probably occur at some time during the life of the project
4	Likely	The natural hazard is expected to occur during the design life of the project
5	Almost Certain	The natural hazard will almost definitely occur during the design life of the project

1.2 Consequence

In terms of determining the consequence or severity of the natural hazard occurring, the qualitative definitions used by CMW for this project are provided in Table B3 for each consequence classification.

Table B3: Qualitative Natural Hazard Consequence Definitions		
1	Insignificant	Very minor to no damage, not requiring any repair, no people at risk, no economic effect to landowners.
2	Minor	Minor damage to land only, any repairs can be considered normal property maintenance no people at risk, very minor economic effect.
3	Moderate	Some damage to land requiring repair to reinstate within few months, minor cosmetic damage to buildings being within relevant code tolerances, does not require immediate repair, no people at risk, minor economic effect.
4	Major	Significant damage to land requiring immediate repair, damage to buildings beyond serviceable limits requiring repair, no collapse of structures, perceptible effect to people, no risk to life, considerable economic effect.
5	Catastrophic	Major damage to land and buildings, possible structure collapse requiring replacement, risk to life, major economic effect, or possible site abandonment.

1.3 Risk Acceptance

It is recognised that the natural hazard risk assessment provided herein is qualitative and, due to the wide range of possible geohazards that could occur, is somewhat subjective. Other methods are available to quantitatively assess an acceptable level of geotechnical related natural hazard risk, such as defining an acceptable factor of safety with respect to slope stability or acceptable differential ground settlements with respect to recommended building code limits.

Therefore, to give this qualitative natural hazard risk assessment some relevance to more commonly adopted numerical or quantitative geotechnical assessment techniques, a residual risk rating of very low to medium (risk value = 1 to 9 inclusive) is considered an acceptable result for the proposed subdivision development.

A risk rating of high to extreme (risk value ≥ 10) is considered an unacceptable result for the proposed subdivision development.

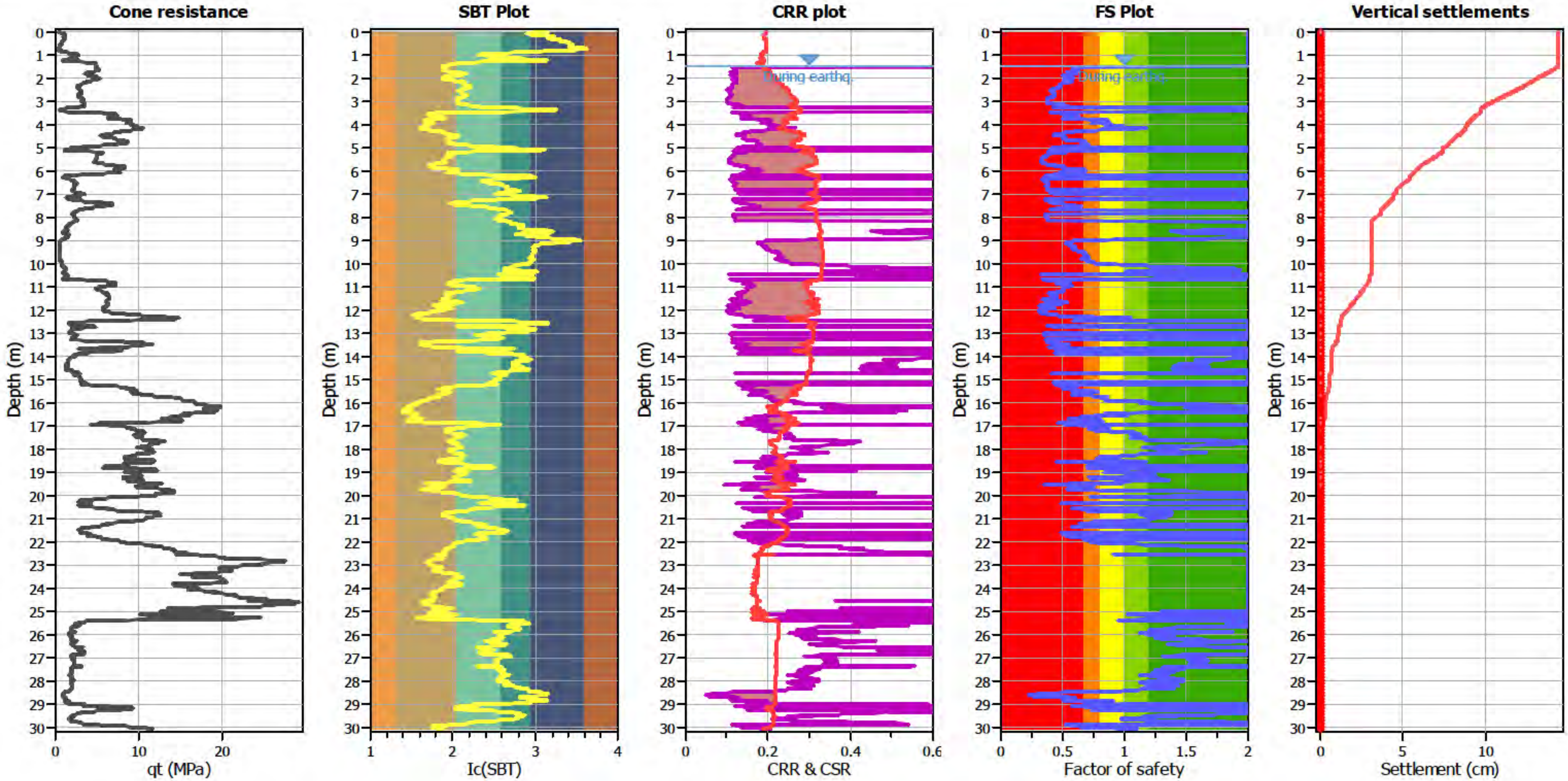
1.4 Geohazards Assessment Summary

The table below is a summary of critical geohazards to this project and is based on information available to date.

APPENDIX G: LIQUEFACTION RESULTS

Project: **Station Road Proposed Subdivision**
Location: **Station Road, Matamata**

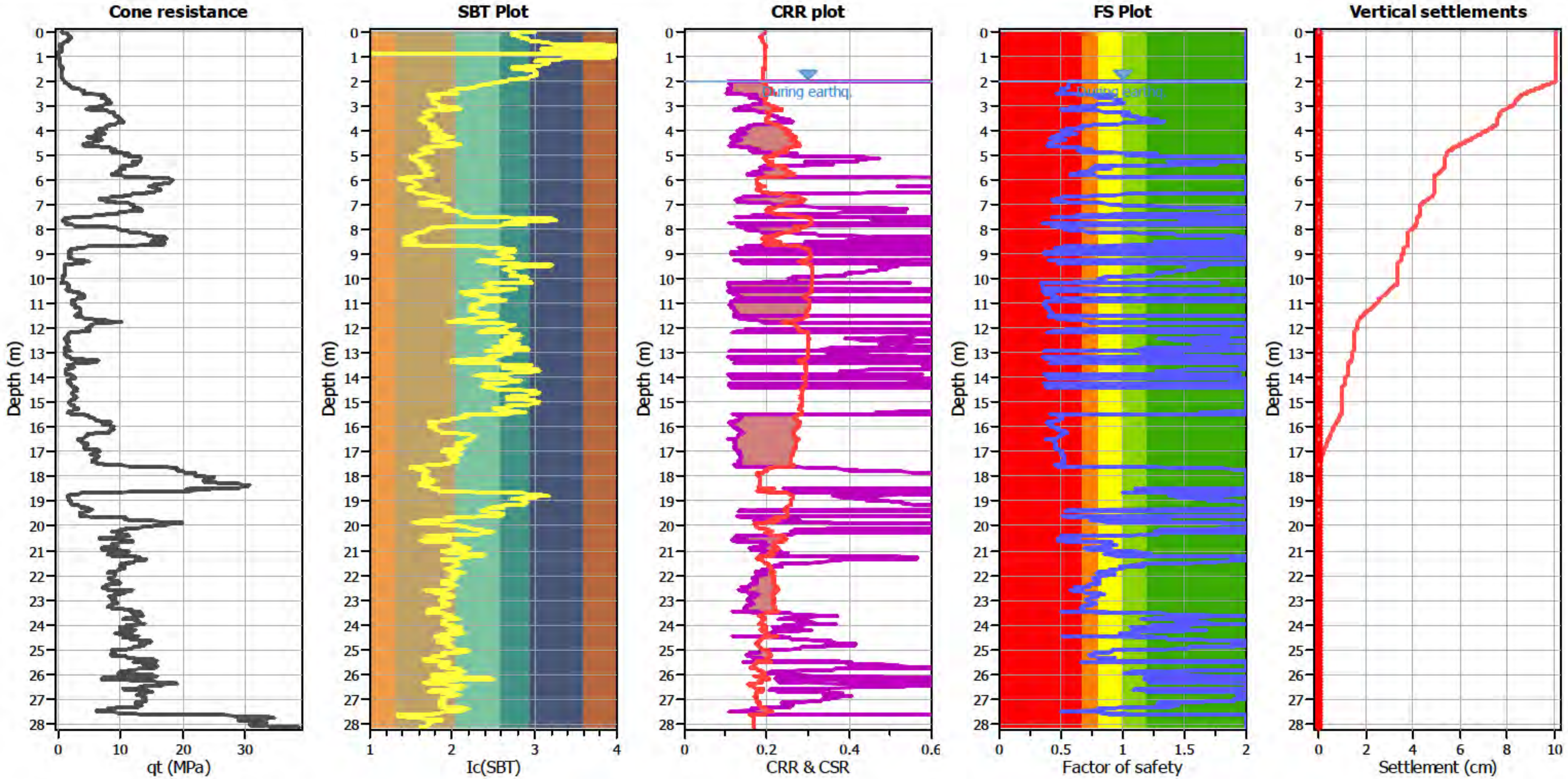
CPT: CPT24-04
Total depth: 30.04 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	-
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **Station Road Proposed Subdivision**
Location: **Station Road, Matamata**

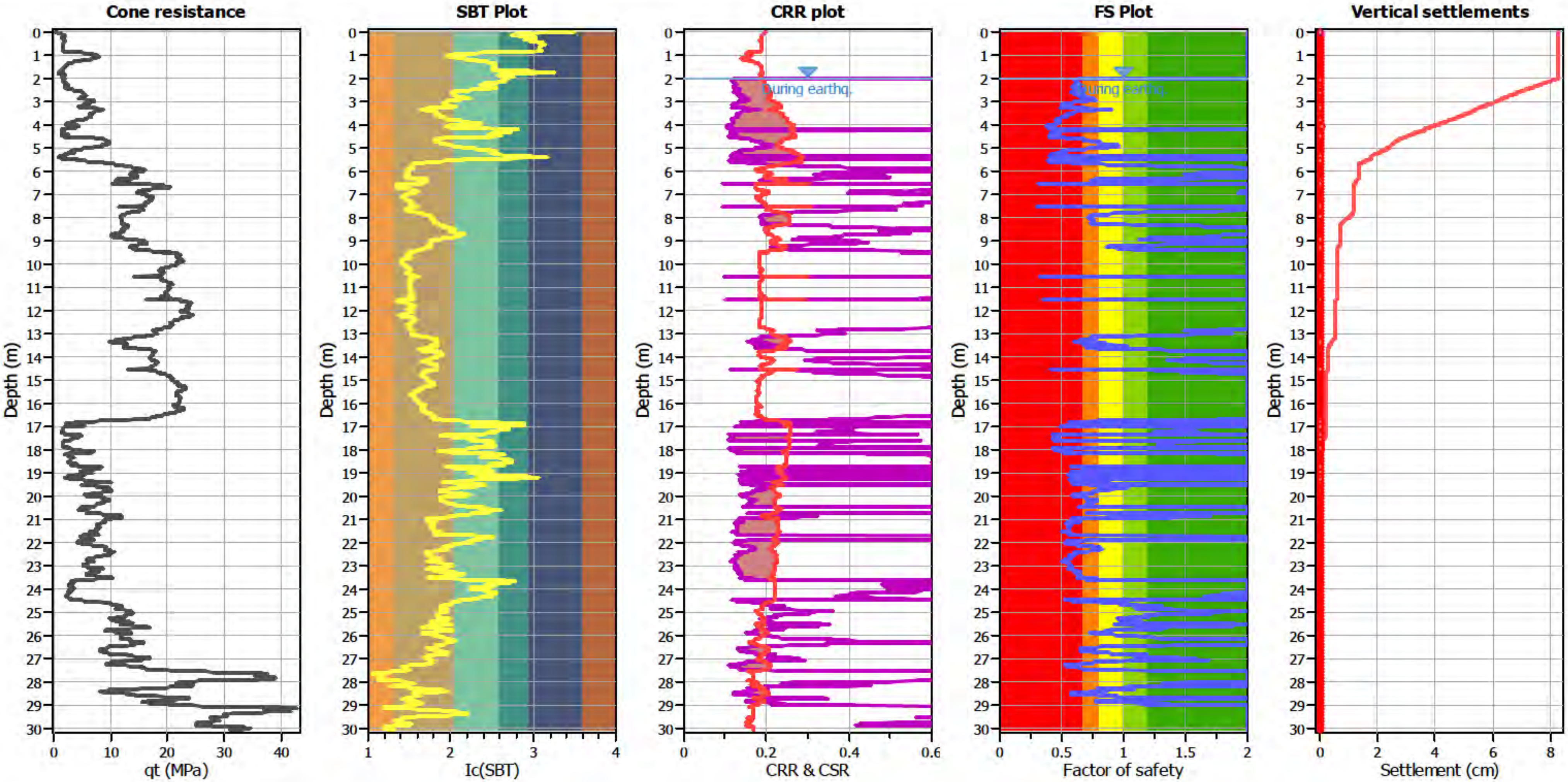
CPT: CPT24-05
Total depth: 28.14 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: Station Road Proposed Subdivision
Location: Station Road, Matamata

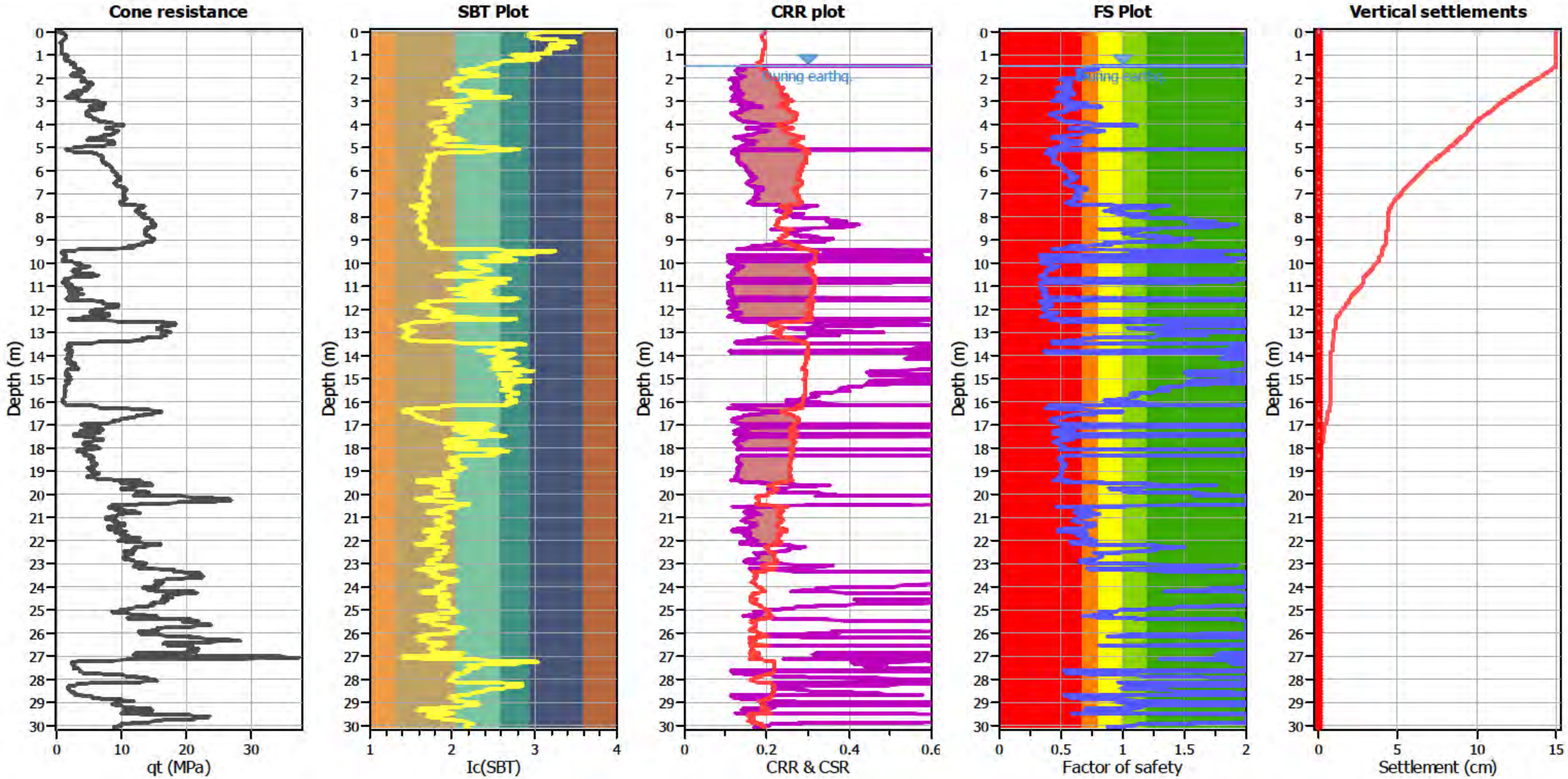
CPT: CPT24-06
Total depth: 30.06 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: Station Road Proposed Subdivision
Location: Station Road, Matamata

CPT: CPT24-07
Total depth: 30.07 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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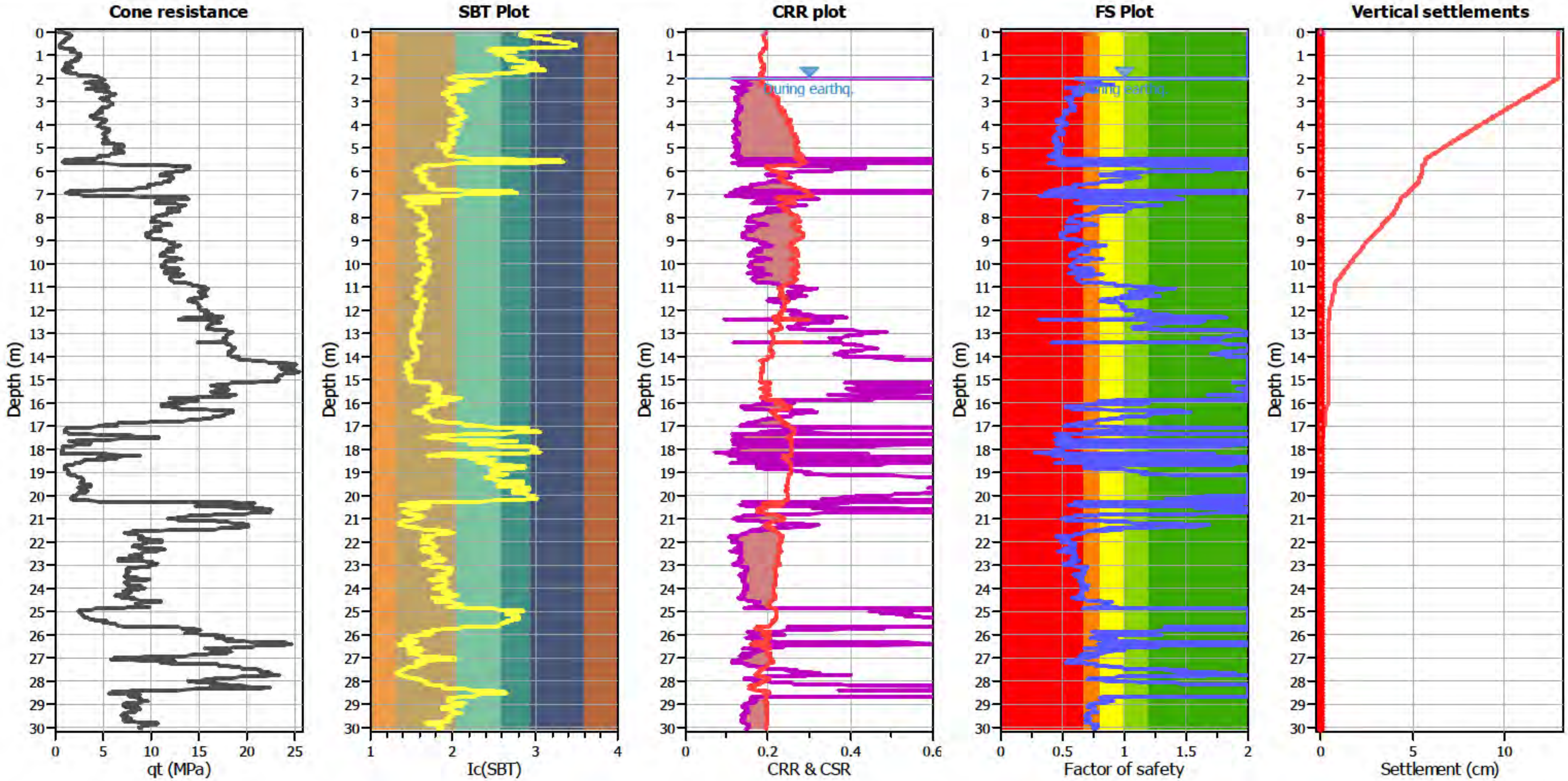
Project: Station Road Proposed Subdivision
Location: Station Road, Matamata

CPT: CPT24-08
Total depth: 30.08 m

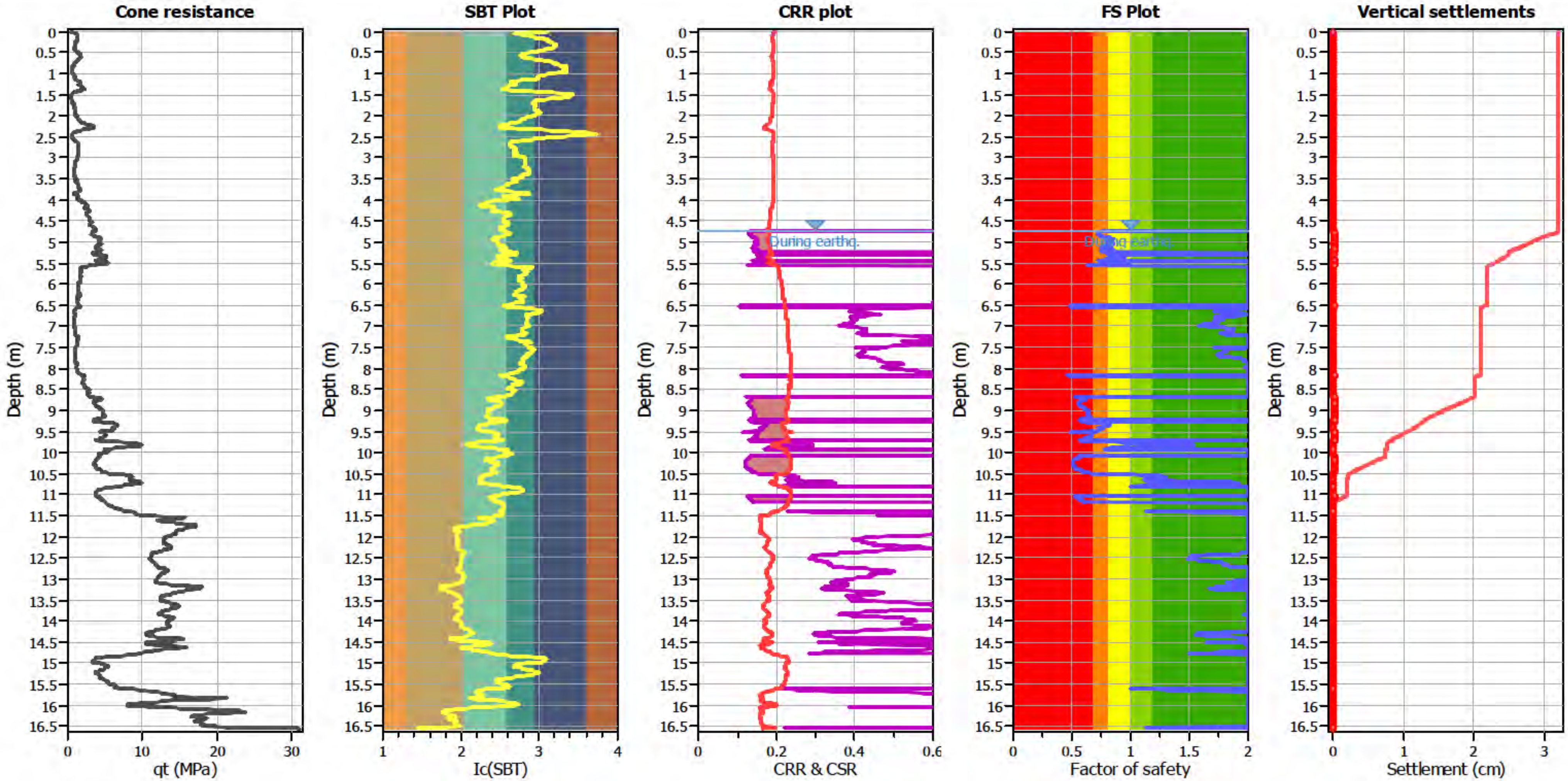


Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.00 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

CMW Geosciences
Great People | Practical Solutions

Project: Station Road Proposed Subdivision
Location: Station Road, Matamata

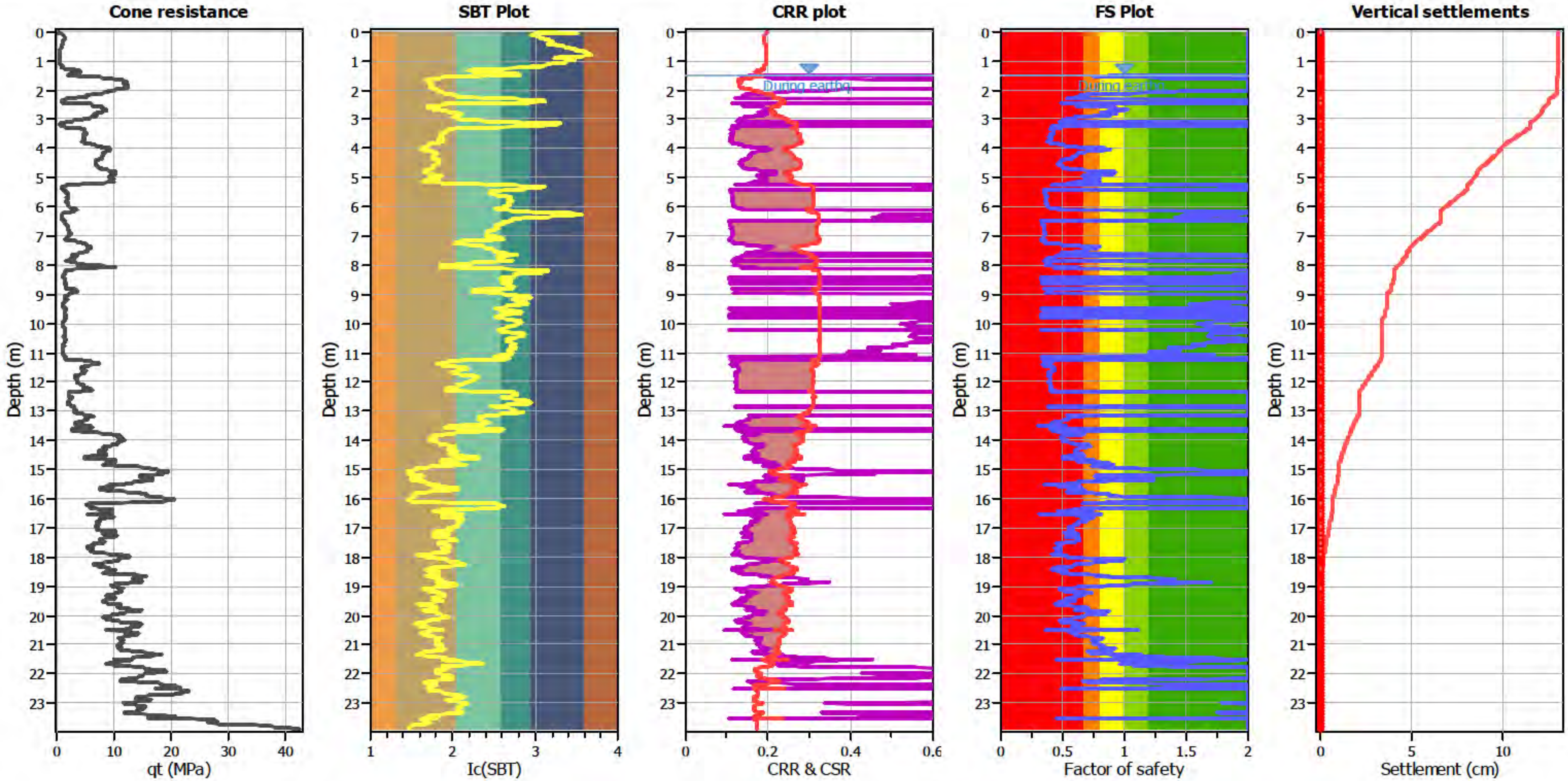
CPT: CPT24-09
Total depth: 16.56 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.75 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	4.75 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **Station Road Proposed Subdivision**
Location: **Station Road, Matamata**

CPT: CPT24-10
Total depth: 23.89 m



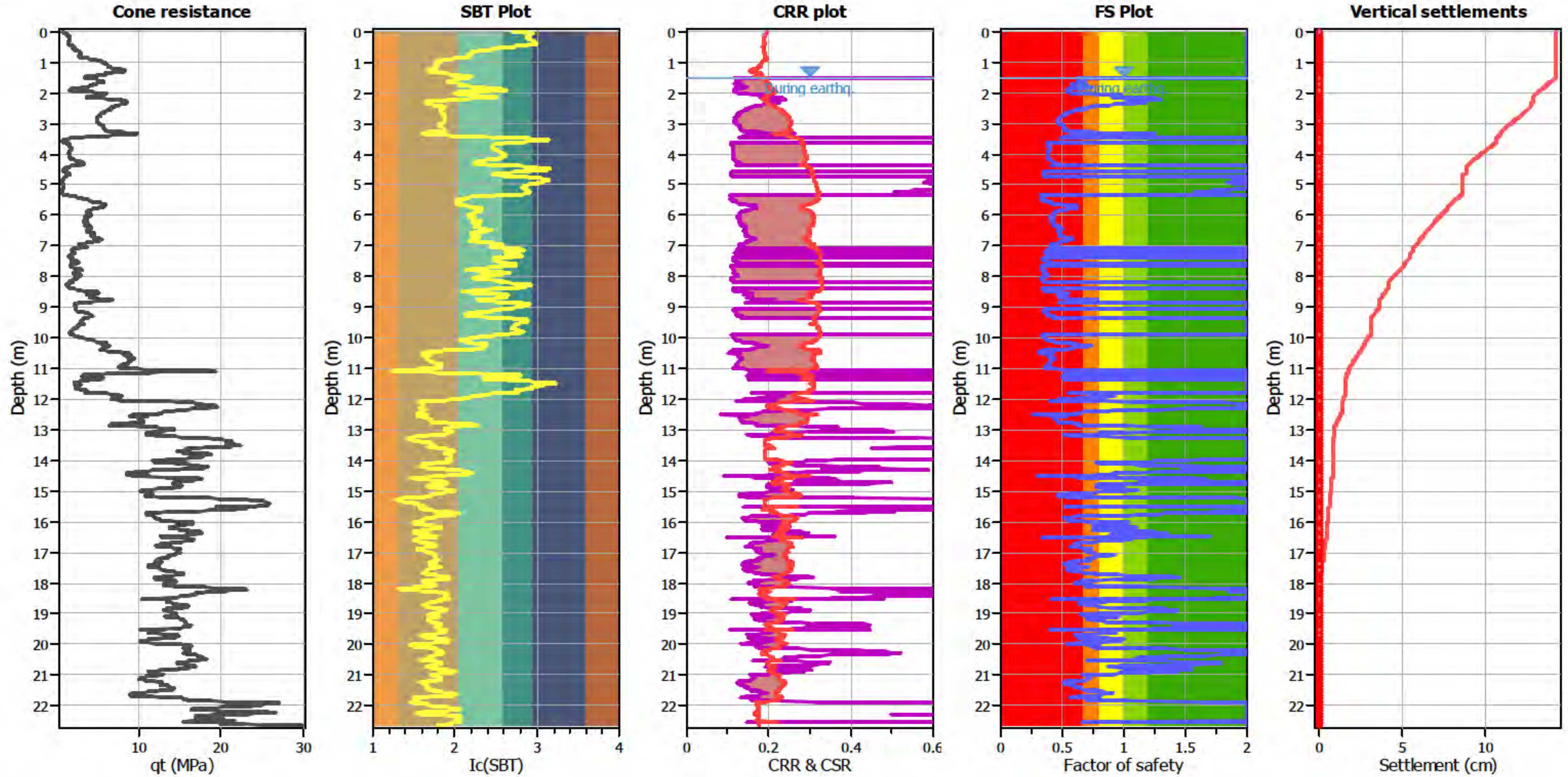
Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: Station Road Proposed Subdivision

Location: Station Road, Matamata

CPT: SCPT24-01

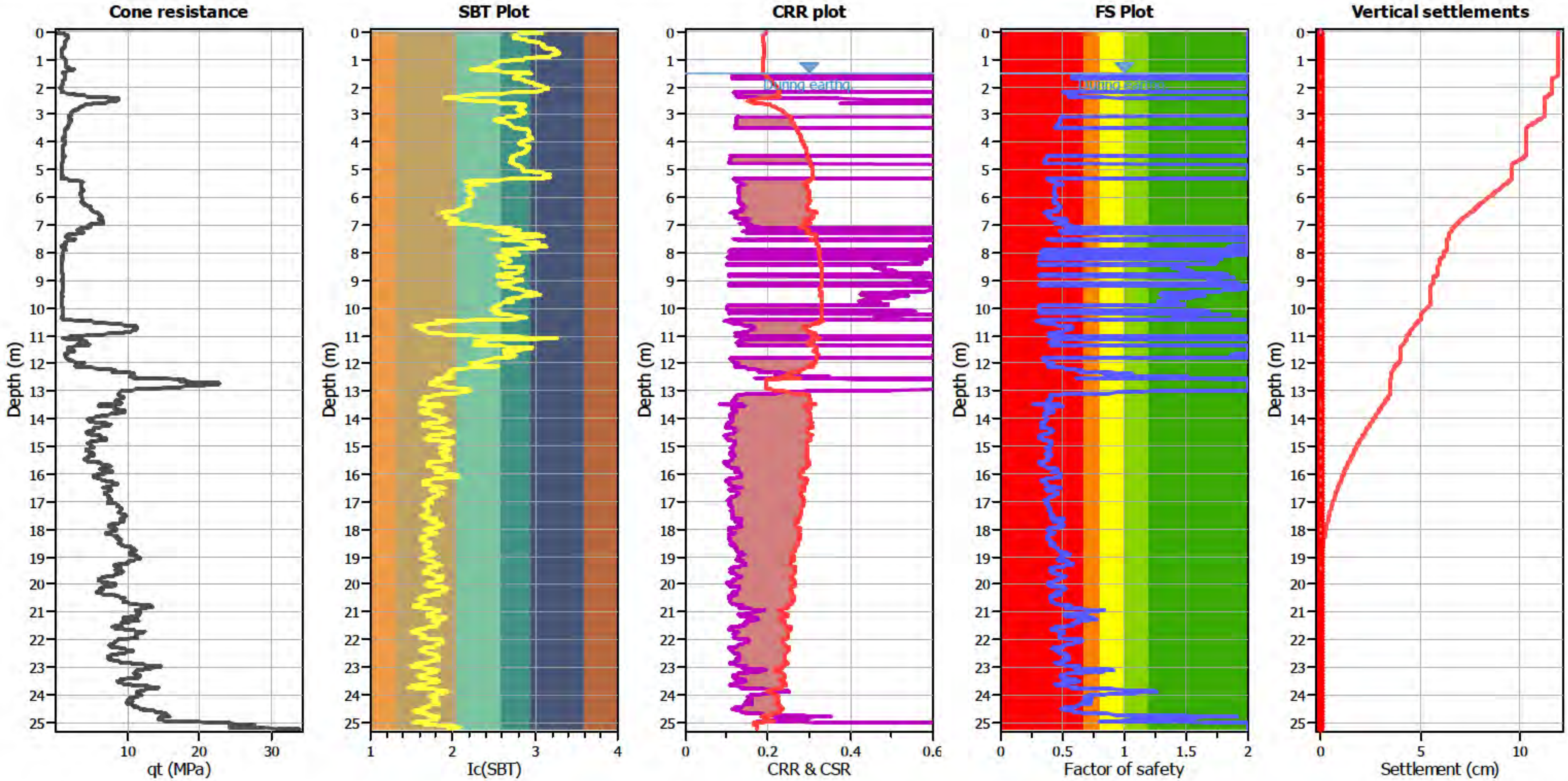
Total depth: 22.64 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: Station Road Proposed Subdivision
Location: Station Road, Matamata

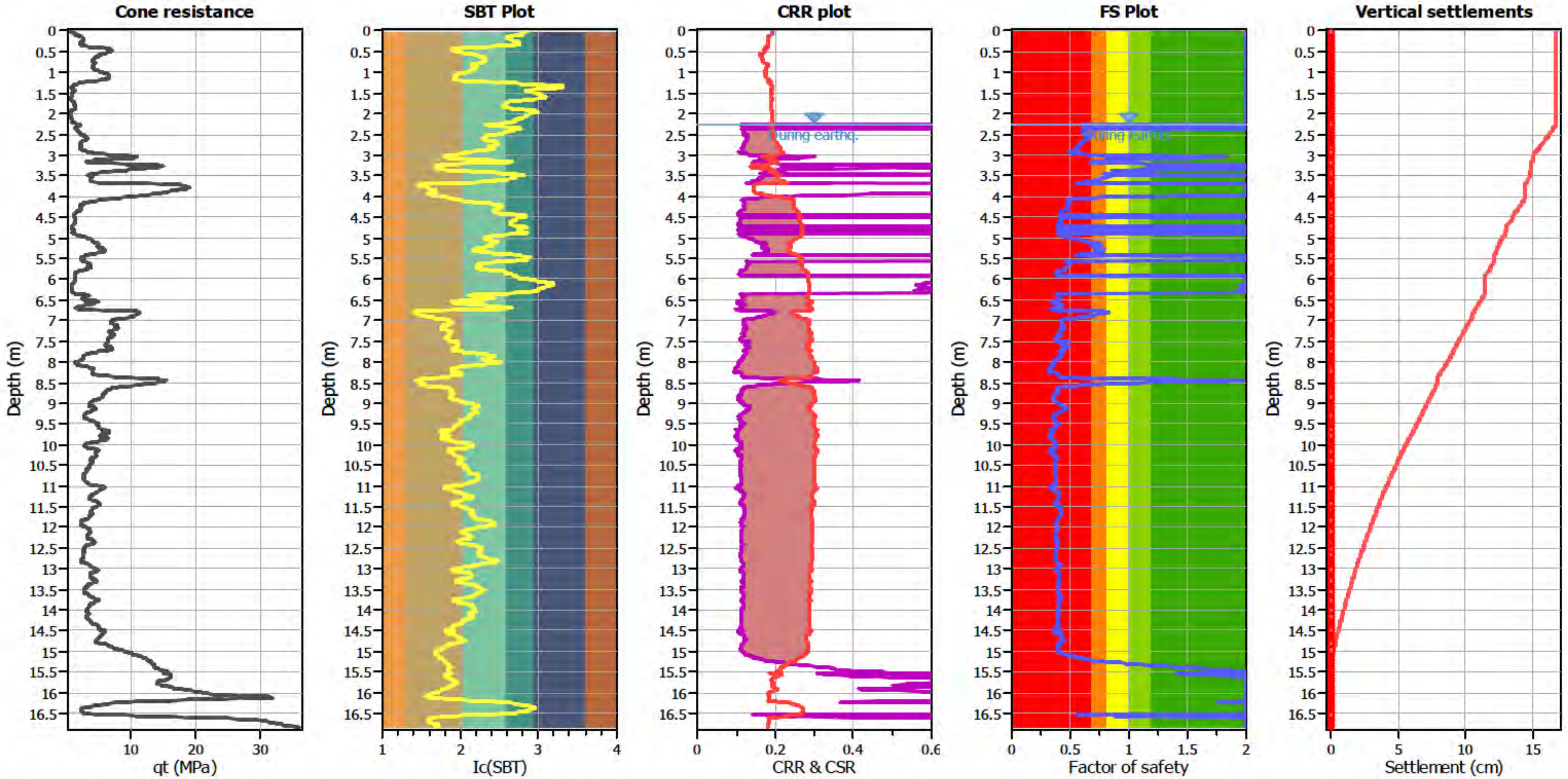
CPT: SCPT24-02
Total depth: 25.23 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	1.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: Station Road Proposed Subdivision
Location: Station Road, Matamata

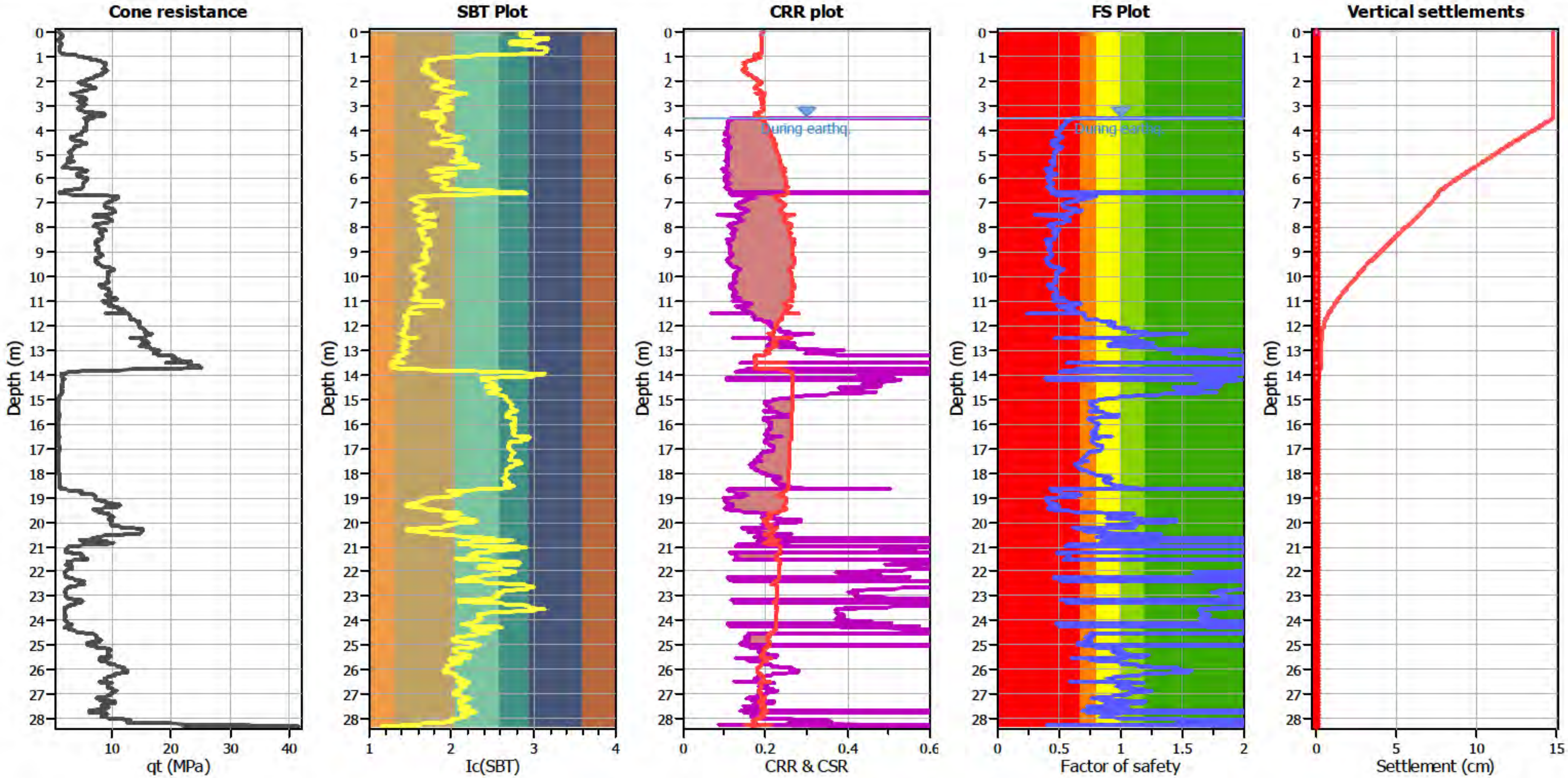
CPT: SCPT24-03
Total depth: 16.83 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	2.25 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.25 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: Station Road Proposed Subdivision
Location: Station Road, Matamata

CPT: SCPT24-04
Total depth: 28.31 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	3.50 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	3.50 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	5.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.36	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

APPENDIX H: SAFETY IN DESIGN RISK ASSESSMENT

CMW Safety in Design Risk Assessment

HAM2023-0124 - Station Road Proposed Subdivison and Solar Farm

Design Element	Hazard	Description	Assessed Risk: Consequence	Assessed Risk: Likelihood	Assessed Risk: Risk Rating	Controls Incorporated in Design	Residual Risk: Consequence	Residual Risk: Likelihood	Residual Risk: Risk Rating
Retaining Walls (If required)	Falling from height	Injury to construction staff while constructing or public once wall is constructed.	4	3	12	Temporary barrier fence to avoid public climbing, permanent fencing to be considered to prevent falls access.	4	1	4
	Striking underground services	Injury to construction staff if live services are struck.	4	2	8	All sites cleared for services prior to construction requiring digging or boring into the ground.	4	1	4
	Moving Machinery	Lifting and swing area of machinery may cause injury to construction staff.	4	3	12	Separate moving machinery from light vehicles and person movements with fencing and/or safe distances from exposed construction staff operations.	4	1	4
	Working at edges of excavations	Injury to construction staff or public by falling into excavations.	4	3	12	Site to be made safe if excavations are to be left open and public can access, excavations to be filled or securely covered on same day of excavation, safe distances from excavations to be maintained and demarked with boundary fence.	4	1	4
	Excavation collapse	Injury to construction staff or persons able to access the excavation after hours.	4	3	12	Staged excavation to be undertaken where able, boundary fence where excavations are under construction or other means of separation for staff or public from potential collapse.	4	1	4
	Retaining wall failure	Exceed specified loading conditions, wall drainage blockage.	4	2	8	Appropriate construction and permanent loading conditions allowed for, design adequate drainage measures, assess impact of blocked drainage on design.	4	1	4
	Falling objects from above	Injury to construction staff or persons under the proposed wall.	4	3	12	Hard hats to be worn at all times as the wall is constructed and where lifting is undertaken, safe distance from any lifting or movements above when being undertaken.	4	1	4
Earthworks	Falling from height	Injury to construction staff while constructing steep temporary or permanent earthworks cut or fill faces.	4	3	12	Temporary barrier fence or other means to be used to ensure persons cannot access to the edge of steep excavations	4	1	4
	Striking underground services	Injury to construction staff if live services are struck.	4	2	8	All sites cleared for services prior to site investigations and earthworks construction	4	1	4
	Moving Machinery	Injury to construction staff.	4	3	12	Separate moving machinery from light vehicles and person movements with fencing and/or safe distances from exposed construction staff operations.	4	1	4
	Working at edges of excavations	Injury to construction staff.	4	3	12	Install safety barriers, exclusion zones, signage as necessary to warn of hazard.	4	1	4
	Trench excavation collapse	Injury to construction staff or persons due to crushing/impact injury.	4	3	12	Follow Worksafe requirements, trench shields or benching of excavations to be used. No staff to enter the trench without appropriate and approved measures already in place.	4	1	4
	Cut / fill batter collapse	Injury to construction staff during construction.	3	2	6	Safe distances and appropriate temporary slope gradients and heights to be assessed prior to construction and monitored during to confirm as appropriate, safe distances and barrier fencing to be used on site where deemed necessary.	3	1	3
	Excessive noise during construction	Damage to hearing of construction staff or persons adjacent to the site.	3	2	6	Comply with appropriate allowances for noise on site, ear protection to be worn where appropriate, setback distances from adjacent sites or notified working hours to avoid conflict with adjacent property inhabitants.	3	1	3
	Machinery rollover	Machinery trafficability over soft, wet or uneven ground.	4	2	8	Appropriate construction of temporary haul roads, implement drainage and geofabrics, appropriate driver training.	4	1	4
	Contaminated Soils	Airborne or in-ground contaminants affecting construction staff.	4	1	4	Perform an environmental assessment of the site prior to construction.	4	1	4
Plant Platform	Moving Machinery	Injury to construction staff.	4	3	12	Separate moving machinery from light vehicles and person movements with fencing and/or safe distances from exposed construction staff operations.	4	1	4
	Plant platform instability	Injury to construction staff or persons due to crushing/impact injury.	4	3	12	Design to incorporate adequate factor of safety, prepare lift management plans to ensure adequate separation between plant and persons.	4	1	4
	Excessive plant settlement	Plant / equipment damage, injury to construction staff or persons due to sudden plant / load movements.	4	3	12	Undertake trial lift with adequate separation of plant and load from persons, monitor settlements during lift.	4	1	4

NOTE: It is the Contractors responsibility to cover construction related risks in a more comprehensive manner (being the competent party in that respect).

Safety in Design Assessment Framework

	Risk Matrix	Consequence: Insignificant 1	Minor 2	Moderate 3	Major 4	Catastrophic 5
Likelihood	Event Will Occur 5	Medium 5	High 10	High 15	Extreme 20	Extreme 25
	Event Almost Certain to Occur 4	Low 4	Medium 8	High 12	Extreme 16	Extreme 20
	Event May Occur 3	Low 3	Medium 6	High 9	High 12	High 15
	Event Not Likely to Occur 2	Low 2	Low 4	Medium 6	Medium 8	High 10
	Event Rarely Occurs 1	Low 1	Low 2	Low 3	Low 4	Medium 5

Appendix F – Service Provisions

Tim Hawke

From: Resource Consents [redacted]
Sent: Friday, 6 June 2025 11:07 am
To: Tim Hawke
Subject: Electricity Supply To: Lot 2 DP 567678 / Northern Solar farm
Attachments: C350-RD.pdf; C700-Serv.pdf

Our privacy policy is here. It tells you how we may collect, hold, use and share personal information.

Hi Tim,

Electricity Supply To: Lot 2 DP 567678 / Northern Solar farm

An upgrade will be required to provide a suitable connection point for all lots of this development.
There will be a cost to complete this work.

Please contact a Powerco Approved Contractor for a price and design. Conditions may apply. These conditions will be advised as part of the quotation from the Contractor.
Standard connection fees will apply once this upgrade work has been completed.

Please be advised the information contained herein is current as of the date of this letter but could be subject to change over time.

Many thanks,

Zoe Kerwin ***(Huygen)***
Project Manager Support

www.powerco.co.nz

From: Tim Hawke [redacted]
Sent: Thursday, 29 May 2025 12:11 pm
To: Resource Consents [redacted]
Cc: Dean Morris [redacted]
Subject: Ashbourne - Northern Solar Farm

Tim Hawke

From: Resource Consents [redacted]
Sent: Friday, 6 June 2025 12:10 pm
To: Tim Hawke
Subject: Electricity Supply To: 247A Station Road Matamata / Southern Solar Farm
Attachments: Powerco residential power provision letter.pdf; C700-Serv.pdf

Hi Tim,

Electricity Supply To: 247A Station Road Matamata / Southern Solar Farm

An upgrade will be required to provide a suitable connection point for the Souther Solar Farm of this development. There will be a cost to complete this work.

Please contact a Powerco Approved Contractor for a price and design. Conditions may apply. These conditions will be advised as part of the quotation from the Contractor.
Standard connection fees will apply once this upgrade work has been completed.

Please be advised the information contained herein is current as of the date of this letter but could be subject to change over time.

Many thanks,

Zoe Kerwin ***(Huygen)***
Project Manager Support

www.powerco.co.nz

POWERCO

From: Tim Hawke [redacted]
Sent: Thursday, 29 May 2025 12:43 pm
To: Resource Consents [redacted]
Cc: Navdeep Kaur [redacted]
Subject: Ashbourne - Southern Solar Farm

To Whom in Concern,

We are currently preparing a resource consent package for a new southern solar farm. The southern solar farm will connect to the proposed Ashbourne – Residential subdivision and the new roads will provide the access to the southern solar farm site. We requested service provision letter for power to service the residential development, and we received confirmation letter back from Powerco on 12th November last year, which is attached.

The site address is 247A Station Road Matamata. We are proposing to extend a general power connection from the proposed residential subdivision to the southern solar farm site. The southern solar farm will require a general house power connection for the purpose of onsite monitoring equipment for the solar farm. Refer to the attached C700 drawing for our preliminary services plan for the southern solar farm.

We would like a power provision letter to confirm that the southern solar farm can be serviced for power. We will require this for our resource consent submission to council.

Tim Hawke
Senior Engineer
NZDE (Civil) MENZ

MAVEN WAIKATO LIMITED
07 242 0616 | 027 298 7762
Timh@maven.co.nz
www.maven.co.nz
Level 1, 286 Victoria Street, Hamilton Central

Tuatahi First Fibre

PO Box 27050
Garnett Avenue 3257
0800 Fibre LTD
tuatahifibre.co.nz

30 May 2025

CONDITIONAL ACCEPTANCE BY TUATAHI FIRST FIBRE LIMITED AS TELECOMMUNICATIONS OPERATOR

Development: Ashbourne Northern Solar Farm
Legal Description: LOT 3 DPS 14362, LOT 2 DP 21055, PTL 1 DP 21055, LOT 2 DP 567678

1. Tuatahi First Fibre Limited (TFF) confirms that a TFF telecommunications connection will be made available for each site in the development, **providing the developer was to sign a TFF Installation Agreement.** Upon approval of this agreement, TFF will undertake to become the telecommunications operator of the telecommunications reticulation in the proposed development and any proposed public roads at Station Road (the "**Subdivision**"), to provide network connections to all lots / units in the Subdivision (the "**Reticulation**") as per the below plan.

2. The Reticulation will be installed in accordance with:

   (a) the requirements and standards set by the Matamata-Piako District Council and advised to TFF via the Council's website; and

   (b) the requirements of the Telecommunications Act 2001 and all other applicable laws, regulations and codes (as amended).

3. The Reticulation will be installed by our preferred provider to TFF's satisfaction.

4. TFF will be the owner, operator and maintainer of the Reticulation.

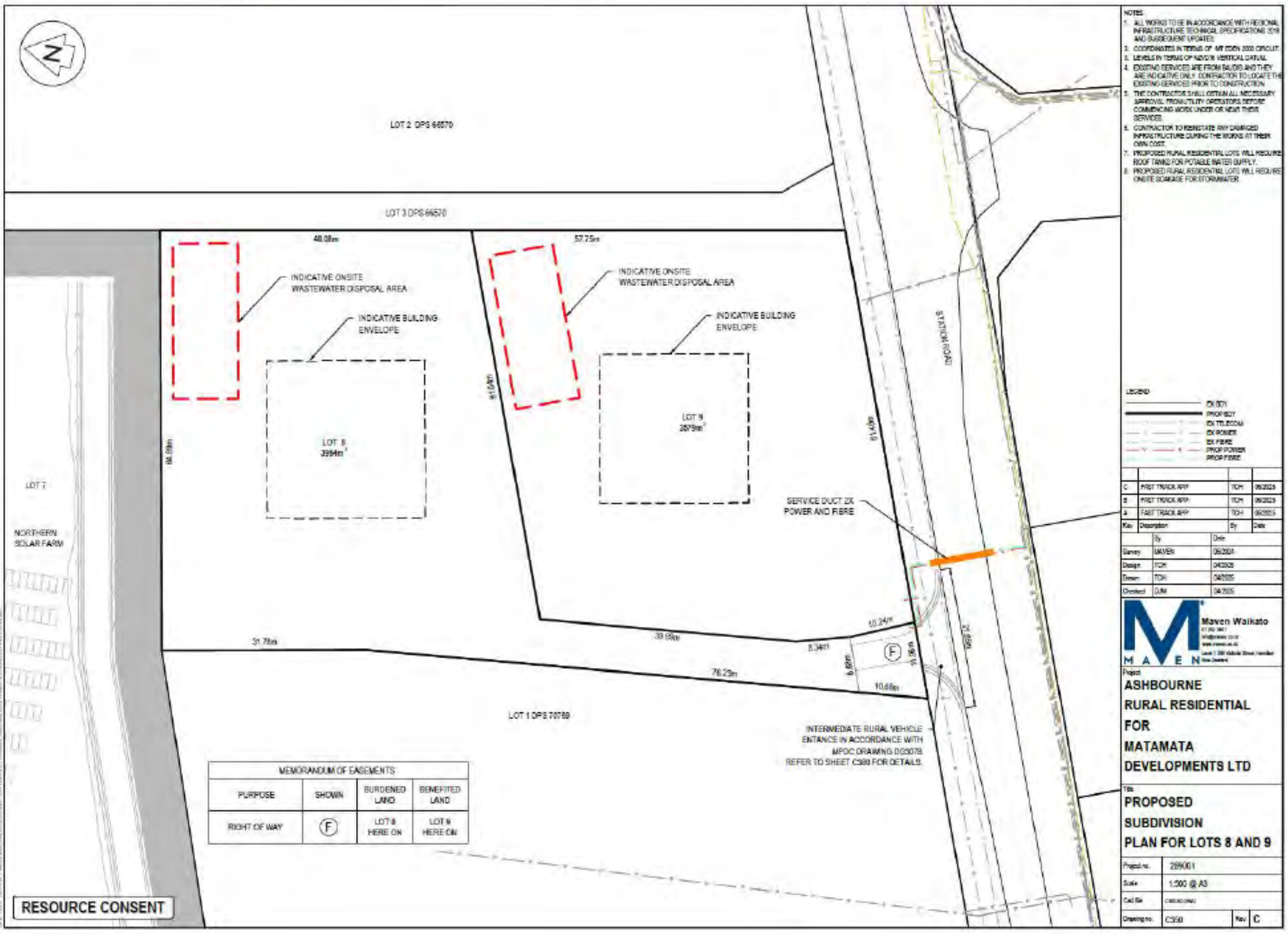
5. One or more retail service providers will be available to supply telecommunications services over the completed Reticulation when service is available, provided that TFF shall not be responsible if the retail service provider's offer to supply such telecommunications services or the number of such providers varies from time to time.

SIGNED for and on behalf of **TUATAHI FIRST FIBRE LIMITED** by:

Signature:

Name: Dan Fenwick

Tuatahi First Fibre



Tuatahi
First Fibre

PO Box 27050
Garnett Avenue 3257
0800 Fibre LTD
tuatahifibre.co.nz

30 May 2025

CONDITIONAL ACCEPTANCE BY TUATAHI FIRST FIBRE LIMITED AS TELECOMMUNICATIONS OPERATOR

Development: Ashbourne Southern Solar Farm
Legal Description: LOT 3 DPS 14362, LOT 2 DP 21055, PTL 1 DP 21055, LOT 2 DP 567678

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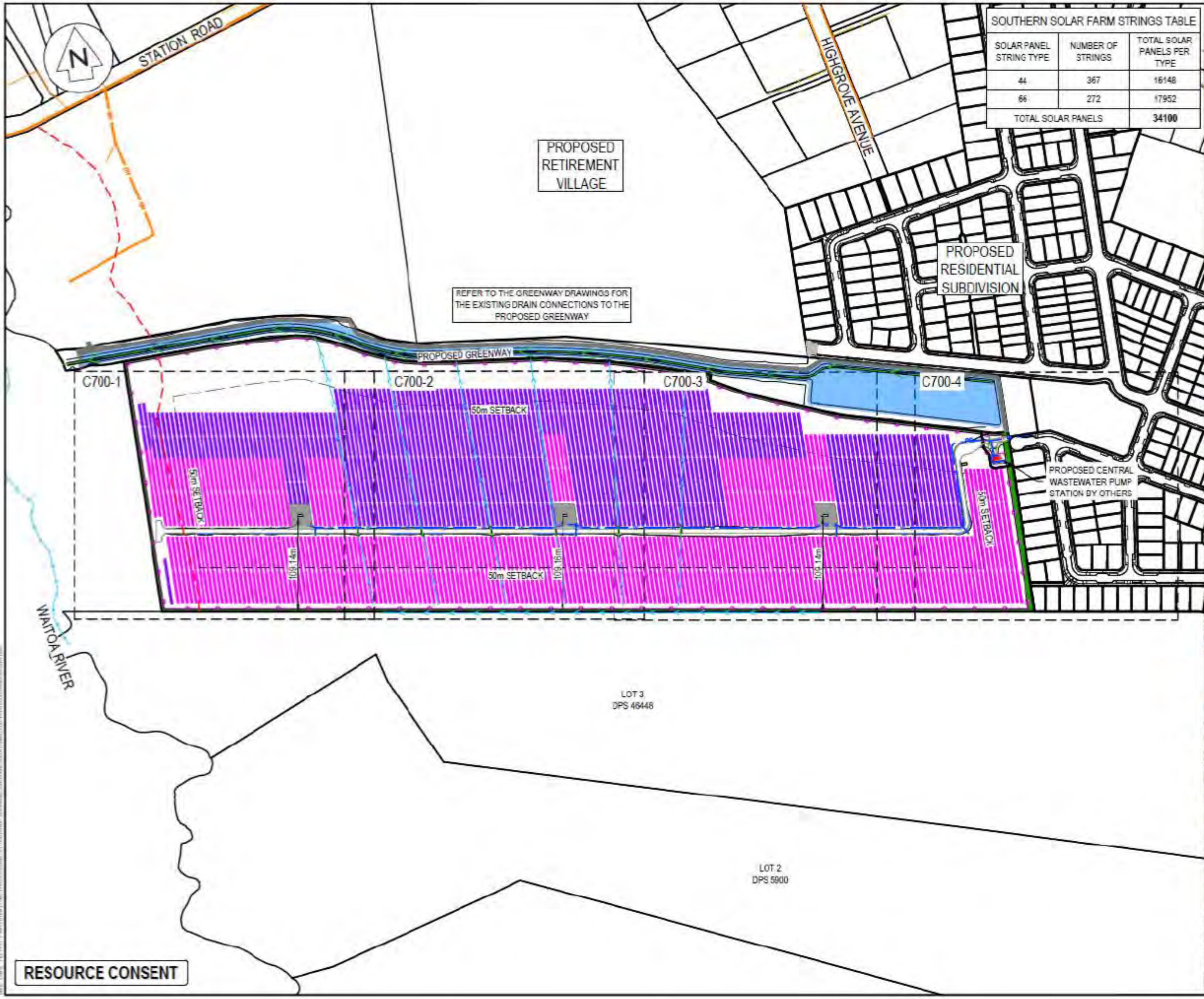
SIGNED for and on behalf of **TUATAHI FIRST FIBRE LIMITED** by:

Signature:

Name: Dan Fenwick



RESOURCE CONSENT

SOUTHERN SOLAR FARM STRINGS TABLE

SOLAR PANEL STRING TYPE	NUMBER OF STRINGS	TOTAL SOLAR PANELS PER TYPE
44	367	16148
66	272	17952
TOTAL SOLAR PANELS		**34100**

NOTES

1. ALL WORKS TO BE IN ACCORDANCE WITH REGIONAL INFRASTRUCTURE TECHNICAL SPECIFICATIONS 2018 AND SUBSEQUENT UPDATES.
2. COORDINATES IN TERMS OF MT EDEN 2000 CIRCUIT.
3. LEVELS IN TERMS OF NZVD16 VERTICAL DATUM.
4. EXISTING SERVICES ARE FROM BAUDIG AND THEY ARE INDICATIVE ONLY. CONTRACTOR TO LOCATE THE EXISTING SERVICES PRIOR TO CONSTRUCTION.
5. THE CONTRACTOR SHALL OBTAIN ALL NECESSARY APPROVAL FROM UTILITY OPERATORS BEFORE COMMENCING WORK UNDER OR NEAR THEIR SERVICES.
6. CONTRACTOR TO REINSTATE ANY DAMAGED INFRASTRUCTURE DURING THE WORKS AT THEIR OWN COST.
7. REFER TO C710 SERIES DRAWINGS FOR THE SERVICE DETAILS.
8. SERVICES TO BE INSTALLED IN ACCORDANCE WITH THE SERVICE PROVIDERS SPECIFICATIONS.
9. 21.09MW EXPECTED POWER GENERATION PER DAY.
10. FOR THE POWER TRANSFORMER MV STATION FOUNDATION DETAILS & ELECTRICAL TRENCHING DETAILS, REFER TO THE LIGHTYEARS SOLAR DRAWINGS FOR FURTHER DETAILS.
11. REFER TO RESIDENTIAL SUBDIVISION DRAWINGS FOR FURTHER DETAILS.
12. REFER TO RETIREMENT VILLAGE DRAWINGS FOR FURTHER DETAILS.

LEGEND

- EX BDY
- PR BDY
- EX TELECOM
- EX POWER
- PROP POWER
- EX FIBRE
- PROP FIBRE
- EX WATERMAIN
- PR WATERMAIN
- PR BOUNDARY SETBACK
- ECOLOGY 100m SETBACK
- POWER TRANSFORMER
- 66 x SOLAR PANEL STRING
- 44 x SOLAR PANEL STRING
- PR WATER TROUGH

Rev	Description	By	Date
D	FAST TRACK APP	TCH	05/2025
C	FAST TRACK APP	TCH	05/2025
B	FAST TRACK APP	TCH	05/2025
A	FAST TRACK APP	TCH	04/2025

	By	Date
Survey	MAVEN	05/2024
Design	TCH	04/2025
Drawn	TCH	04/2025
Checked	DJM	04/2025

MAVEN
Maven Waikato
07 242 0601
info@maven.co.nz
www.maven.co.nz
Level 1 286 Victoria Street, Hamilton
New Zealand

Project
ASHBOURNE SOUTHERN SOLAR FARM FOR MATAMATA DEVELOPMENTS LTD

Title
PROPOSED SERVICES KEY PLAN

Project no.	289001
Scale	1:4000 @ A3
Cad file	C700-SERV.DWG
Drawing no.	C700
Rev	D

Tuatahi First Fibre