ARUP

2686570 Ontario Inc. o/a IPCF Baldwin Airport

Baldwin Airport Improvements

Preliminary Stormwater Management Brief

Rev 3 | 1 November 2024

Job number 287943-00

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Document Verification

Project title	Baldwin Airport Improvements
Document title	Preliminary Stormwater Management Brief
Job number	287943-00
Document ref	287943-ARUP-XX-XX-RP-C-XX-001_Preliminary Stormwater Management Brief
File reference	

Revision	**Date**	**Filename**			
Draft VA.0 (issued as Rev 0)	August 1, 2024	**Description**	Preliminary Stormwater Management Brief		
			Prepared by	**Checked by**	**Approved by**
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		Signature			
VA.1 (issued as Rev 0)	August 13, 2024	**Filename**	Preliminary Stormwater Management Brief		
		Description	Incorporates comments from McMillan LLP		
			Prepared by	**Checked by**	**Approved by**
		Name	Michael Tran, Bailey Sadowsky	Joshua Battiston, Jason Do	Joshua Battiston, Jason Do
		Signature			
VA.2	October 11, 2024	**Filename**	Preliminary Stormwater Management Brief		
		Description			
			Prepared by	**Checked by**	**Approved by**
		Name	Michael Tran, Bailey Sadowsky	Joshua Battiston, Jason Do	Joshua Battiston, Jason Do
		Signature			
VA.3	November 1, 2024	**Filename**	Preliminary Stormwater Management Brief		
		Description			

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Issue Document Verification with Document ✓

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November 1, 2024

2686570 Ontario Inc. o/A IPCF Baldwin Airport
c/o McMillan LLP, Brookfield Place
181 Bay Street, Toronto, ON
M5J 2T3
Attn: Kailey Sutton

Baldwin Airport Improvements
Additional Services Scope #1 - Civil Engineering Services

Dear Kailey,

Arup Canada Inc. is pleased to submit this Report in support of your project's site improvements and our agreed scope of work with your team with reference to the above project.

Should you have any questions, please do not hesitate to contact the undersigned.

Yours sincerely,

Arup Canada Ltd.

Senior Planner

George Long	
Name	*Signature*

Civil Engineer

Joshua Battiston, P.Eng.	
Name	*Signature*

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1. Introduction

Arup has been retained to continue supporting the development of improvements at the Baldwin Airport as a site alteration permit is sought with the Town of Georgina. Arup is providing Civil Engineering and Aviation Planning services to prepare this Preliminary Stormwater Management Brief (the "Report") in addition to grading and erosion and sediment control plan drawings (the "Drawings"). The preliminary design outlined in the Report and Drawings are as of November 1st, 2024.

1.1 Project Context

The existing Baldwin aerodrome site, CPB9, is approximately 36 ha and is located 0.5 km east of Baldwin, Ontario, in the Town of Georgina (the "Town"). The site is bordered by Old Homestead Rd to the north, Highway 48 to the west and Smith Blvd to the south.

Proposed improvements at the site for the Baldwin aerodrome that are considered in this Report include:

- Raising the aerodrome by approximately 4.5m to elevate the aviation surfaces from the surrounding wetlands and to better comply with clearance requirements further to Transport Canada TP312 (5th ED).
- Providing aircraft operations as non-instrument, aircraft group number II.
- Providing one (1) runway with designation 01-19.
- Providing one (1) associated parallel taxiway on the east side of the runway.
- Providing a ramp area and space for future buildings such as hangars.
- Providing an access road to the ramp area.

1.2 Purpose

The purpose of this report is to describe the stormwater management methodology and conceptual drainage design. This report outlines the standards and regulations that were considered in the conceptual design of the stormwater management systems, the data used, and the assumptions made.

2. Standards, regulations and guidelines

The primary references for the stormwater management conceptual design are the following:

- Lake Simcoe Region Conservation Authority (LSRCA) Technical Guidelines for Stormwater Management Submissions (April 2022).
- Town of Georgina Department of Operations & Engineering Development Design Criteria (2013).
- City of Barrie Storm Drainage and Stormwater Management Policies and Design Guidelines (May 2022).
- Toronto and Region Conservation Authority Low Impact Development Stormwater Management and Planning Design Guide (2010).
- Erosion and Sediment Control Guide for Urban Construction, TRCA (2019).

The primary references for the grading design are as follows:

- For the aviation surfaces, Transport Canada TP 312 5th Edition and ICAO Aerodrome Design Manual 5th Edition, Part 2 – Taxiways, Aprons and Holding Bays.
- For roadways, Town of Georgina Design Criteria, Ministry of Transportation Ontario (MTO) Supplement to the Transportation Association of Canada Geometric Design Guide for Canadian Roadways, Transportation Association of Canada (TAC) Geometric Design Guide for Canadian Roadways.

3. Data available

The available data that was used to support the stormwater management conceptual design are listed below:

- Topographic survey provided by Geoverra on June 29, 2020. Surface includes the extent of existing gravel surfaces, existing structures and vegetation areas.
  - Horizontal coordinate system is NAD 1983 UTM Zone 17N
  - Vertical coordinate system is CGVD-1928:1978
- Rainfall data taken from Barrie WPCC Station #6110557 per the Town of Georgina Development Design Criteria.
- Pre-development land cover types included within the extents of our site are determined using the provided topographic survey data and aerial imagery (Source: Google Earth, Esri).
- Post-development land cover determined using proposed Baldwin airport layout.
- Copy of the LSRCA hydraulic model (HEC-2) of Black River received on June 26, 2024.
- Copy of the LSRCA hydrologic model (VO2) of Black River watershed and supporting model build report received on June 26, 2024.

4. Stormwater management approach

The proposed stormwater management approach is focused on the management of the water quantity and treatment of water quality. Where feasible, the design seeks to minimize the disturbed area and limit the impact to nearby wetland areas. The goal is to capture and treat runoff before it reaches Black River. The Low-Impact Design (LID) elements proposed includes open swales and a stormwater basin, which are preferred over the use of enclosed pipes and storage tanks.

The treatment train approach is applied by combining practices to manage and treat stormwater runoff. The design looks to minimize impacts through native revegetation to the extent practical, capture runoff in swales, and collect and detain stormwater in a basin. An oil-grit separator is also proposed to provide water quality benefits downstream of the underground storage tank prior to the Black River outfall. Stormwater runoff from the site discharges to Black River under pre- and post-development conditions.

Given the planned grading and infrastructure improvements, erosion and sediment control during and after construction are incorporated into the design.

4.1 Water quantity

A new stormwater management system is designed to accommodate any increase in impervious cover, in accordance with the LSRCA regulations. Any increase in stormwater runoff from the pre-development to post-development condition will need to be detained. The stormwater basins will be sized to manage the post-development runoff volume under a 100-year storm event. Water leaving the stormwater management facilities will ultimately discharge to Black River. Table 1 lists water quantity targets for the project.

Table 1: Water quantity targets

Water Quantity
• Post-development peak flow rates are not to exceed the corresponding pre-development peak flow rates • Minor system to be designed based on the 1:5-year storm event and the Major System to be designed based on the 1:100-year storm event • Maintain existing drainage patterns where feasible • Controlled release of discharge of stormwater runoff into Black River

4.2 Water quality

The goal is to use a combination of natural site features, vegetated open swales, and reduction of impervious cover to meet runoff reduction requirements. Table 2 lists water quality targets for the project.

Table 2: Water quality targets

Water Quality
• Capture/treat the post-development direct runoff volume from 25mm of rainfall on all new / reconstructed impervious surfaces • Treat the volume of water calculated above to achieve a minimum of 80% removal of total suspended solids and a minimum of 80% removal of annual Total Phosphorus load • End of pipe stormwater management facilities to be designed to consider minimizing thermal impacts

4.3 Flood analysis

The LSRCA recommends a flood analysis using the most recent hydraulic model of Black River to demonstrate the proposed airport improvements have no negative impacts to flooding (water surface elevations), erosion (channel and overbank velocities), and conveyance of spills. Arup's initial review of the LSRCA's HEC-RAS model indicated no significant change in water surface elevations in Black River as a result of the proposed design. Therefore, it was determined that a Cut/Fill Compensation Analysis for flood storage was not necessary at this stage of design. Refer to Appendix A for a Technical Note summarizing Arup's review of the existing LSRCA hydraulic model.

4.4 Erosion and sediment control

The proposed work involves an Erosion and Sediment Control (ESC) Plan, prepared for review and approval by the municipality prior to the start of site works. The plan is informed by the LSRCA *Technical Guidelines for Stormwater Management Submissions*, 2022 and the TRCA *Erosion and Sediment Control Guide for Urban Construction, 2019*. Furthermore, the plan will address implementation, phasing, inspection, monitoring, and decommissioning using best management practices.

To minimize local and downstream impacts from erosion, wind-blown dust and sedimentation, a staged approach is proposed to allow the plan to evolve with changes on-site from clearing activities through to re-stabilization. As part of this staged plan, the following best practice measures are recommended for implementation:

- Temporary sediment control fencing should be erected, prior to the commencement of site works, around the perimeter of all grading activities, including double silt fences adjacent to the limits of disturbance;
- Temporary sediment fabric and stone filters should be installed on inlet devices to remain until surface cover has been re-established;
- Temporary rock flow check dams should be installed within drainage cut-off swales;
- All site drainage to be directed to the sediment basins and other check dams via sheet drainage, berms, or swales (as necessary) to facilitate the completion of grading works. The contractor shall construct any additional swales or berms that may be necessary to direct runoff in a controlled manner of suitable quality;
- Temporary drainage cut-off swales to capture flows from exposed earth surfaces and direct them to best management practices for treatment prior to discharge;
- Temporary construction access mud mats to be installed at the construction access, to reduce the amount of material (mud-tracking) that may be transported off site;
- Temporary erosion and sediment control basins are to be constructed, complete with a Hickenbottom outlet control structure and emergency overflow weir. The basins' purpose is to detain runoff long enough to allow most suspended soil particles to settle out of captured runoff;
- Temporary treatment trains are to be provided to treat groundwater discharges from construction dewatering activities, prior to discharge to the Town's sewer network. The treatment train will be designed following completion of the Hydrogeological Assessment, and may include a combination of an Active Treatment System, Dewatering Tanks, and Dewatering Bags as required to meet the appropriate water quality standards;
- Temporary turbidity curtains to be installed within adjacent waterways if applicable to provide a physical barrier isolating the work zone and limiting the movement of suspended sediments released during construction;
- Temporary measures, as described previously on an as required basis, to protect surface and below ground low impact development measures proposed throughout the subject lands, until stabilization is achieved;
- Excess earth and topsoil to be stockpiled away from the water's edge and/or removed from site. Stockpiles shall be seeded or covered with an erosion control blanket if left for periods of greater than 30 days;
- All disturbed areas not under immediate construction for 30 days, or not intended for building activities within a 3-month time period, should be stabilized with seeding;
- Monitor construction during drier months for wind-borne transport of sediment. At the direction of the Engineer, the contractor may be directed to water down exposed earth areas with an aqueous dust suppressing solution;
- Undertake a monitoring program to check all ESC measures as required are in place, not damaged, and generally functioning as intended at the following intervals:
    - On a weekly basis during active construction,
    - Before and after signification rainfall events;
        - An event during which ≥ 15 mm have been received within 24 hours; or,
        - An event with an intensity of ≥ 5 mm/hr and during which 10 mm have been received.

  - After significant snowmelt events,
  - After any extreme weather (e.g. wind storms) which could result in damage to ESC measures,
  - Daily during extended rainfall or snowmelt periods,
  - Monthly during inactive periods (>30 days),
  - During or immediately following any spill event,
  - Before construction is shut down for the winter to ensure the site is ready for freezing conditions and thaws; and,
  - At the end of construction to confirm the site has achieved at least 80% stabilization and that permanent vegetation areas are well-established and effectively preventing erosion.
  - Require the contractor to prepare a Spill Response and Control Plan, which at minimum outlines the following:
    - Description of actions to be taken during a spill, including procedures for responding, reporting, containment and clean up;
    - Description of spills control equipment and materials that should be available on site, including the quantities thereof and locations they are stored in; and,
    - A list of relevant emergency contact numbers, including both project and external contacts.
- Phased removal of temporary sediment basins or control devices during commissioning phase of the development to coincide with upstream stabilization (established vegetation) of the catchment;
- The contractor will be responsible for clean-up, proper off-site disposal of ESC materials, and restoration of disturbed areas.

Through proper planning, implementation and monitoring of erosion and sediment control measures, off site impacts are expected to be minimized during the construction phase of the project.

Primary soil erosion and sediment control items proposed in the project include:

- Unreinforced woven geotextile silt fence (temporary)
- Stockpile stabilization (temporary)
- Sediment basin
- Stabilized construction entrance
- Erosion Control Blanket and Fiber Rolls for stabilization of steep terrain
- Sodding adjacent to aircraft manoeuvring areas to be held down using biodegradable mesh protection and secured with staples

4.5 Airport design requirements

A preliminary grading design for the runway, taxiway, and apron has been developed based on TP 312 5th Edition requirements and forms the basis of the stormwater management analysis.

Transport Canada has guidance on the hazardous wildlife attractants on or near airports (see Transport Canada Aviation Publications TP 13549 and TP 312). The recommendation is for stormwater management systems to be designed and operated so as not to create above-ground standing water. The stormwater detention ponds on site are designed, engineered, constructed, and maintained for a maximum 24–hour detention period after the design storm and to remain completely dry between storms.

5. Conceptual stormwater management design

The hydrologic analysis and conceptual design of stormwater management and drainage features are summarized below.

5.1 Pre-development conditions

5.1.1 Site description

The existing Baldwin aerodrome site, CPB9, is approximately 36 ha and is located 0.5 km east of Baldwin, Ontario. The site is bordered by Old Homestead Rd to the north, Highway 48 to the west and Smith Blvd to the south. The site is generally sloped towards Black River, which flows along the southeast property line.

There are existing structures and a gravel road on the southern portion of the site. Figure 1 shows the overview of the Baldwin aerodrome under the pre-development conditions.

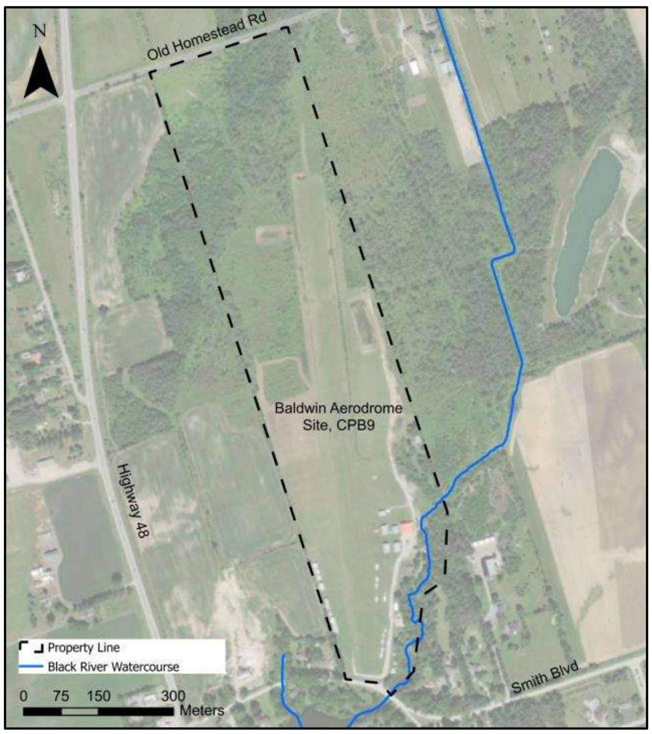


Figure 1: Pre-development site layout

The LSRCA hydraulic model was used to determine the floodplain of Black River. Figure 2 shows the floodplain extent on the southeast portion of the site. Refer to Appendix A for a Technical Note summarizing Arup's review of the existing LSRCA hydraulic and hydrologic models.

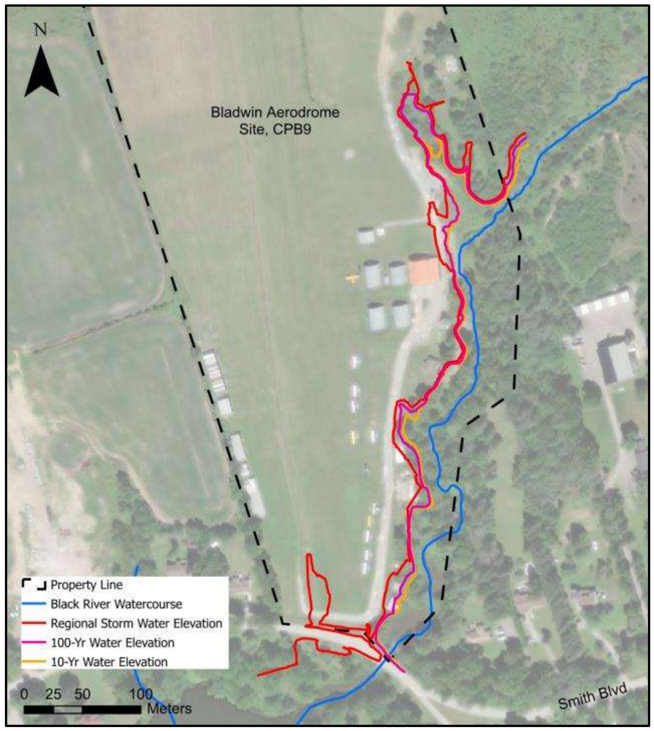


Figure 2: Black River floodplain extent

5.1.2 Hydrology

Based on the available topographical information, the runoff from the Baldwin aerodrome site typically flows overland from northwest to southeast and discharges into Black River. For this project, three drainage catchments have been assumed for the hydrology analysis. The hydrology analysis was undertaken in accordance with the Rational Method. As the design progresses, a runoff software model (e.g. Visual Otthymo) will be used to verify peak discharges.

Based on the available topographical survey data and aerial imagery, the Baldwin aerodrome site catchments were split based on land cover type. Existing vegetated ground cover is assumed to be predominantly lawn (runoff coefficient c, 0.2). Existing homes and roadways are assumed to be impervious cover. Figure 3 depicts identified land cover within the site under the existing conditions. The run-off coefficients for each land cover type were determined per LSRCA Technical Guidelines for Stormwater Management Submissions.

The runoff and peak discharge rates were calculated for all storm events using City of Barrie WPCC Intensity-Duration-Frequency (IDF) curves, as specified by the Town of Georgina Development Design Criteria. The IDF curve parameters for Barrie WPCC Station #6110557 are increased by 15% to account for climate change. The Rational Method was used to undertake required hydrology calculations using a 10-minute timestep. Table 3 summarizes the results for runoff and peak discharge rates under the existing conditions. The hydrology calculations for Baldwin aerodrome site are provided in Appendix B.

Table 3: Pre-development hydrology

Catchment	Area (ha)	Weighted runoff coefficient, C	10-Yr peak discharge (L/s)	100-Yr peak discharge (L/s)
C1	10.40	0.20	123.5	225.1
C2	9.39	0.13	53.4	97.6
C3	1.44	0.25	50.8	91.8

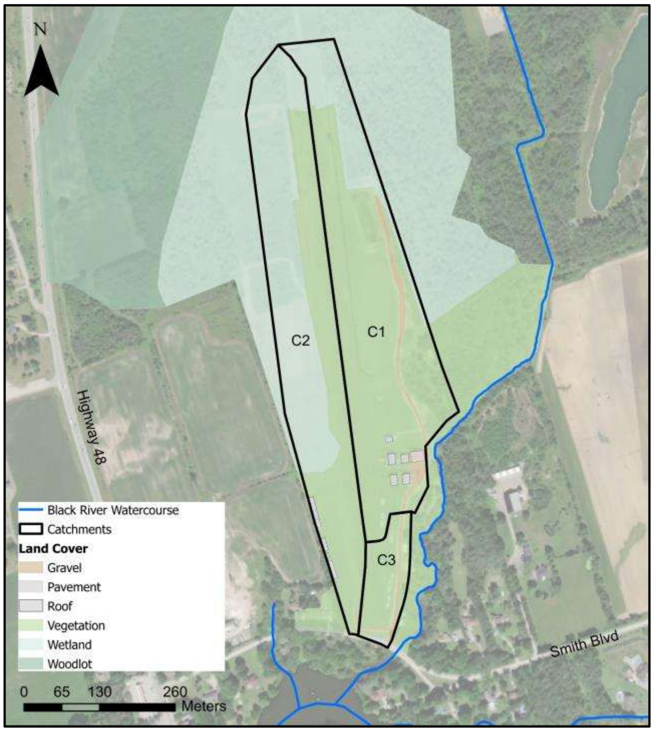


Figure 3: Pre-development land cover

5.2 Post-development conditions

5.2.1 Site description

Under the post-development condition, all existing structures located with the site will be demolished. A new paved runway, taxiway and hangar area will be constructed. Access to the site is proposed to be provided via a paved access road from Smith Blvd. The rest of the site is proposed to be vegetated with native grasses. Figure 4 shows the overview of the Baldwin aerodrome under the post-development conditions.

Existing drainage patterns are maintained where possible. The site continues to slope towards Black River. The taxiway and apron area will discharge to Black River via a new gravity outfall. The runway and access road will have two separate outfalls towards the existing roadside swale along Smith Blvd to the south of the site.

The LSRCA hydraulic model cross-sections through the site were updated to determine the impact that the project may have on the floodplain of Black River. The proposed model results indicate that there is no significant impact on the Black River floodplain. Refer to Appendix A for further information.

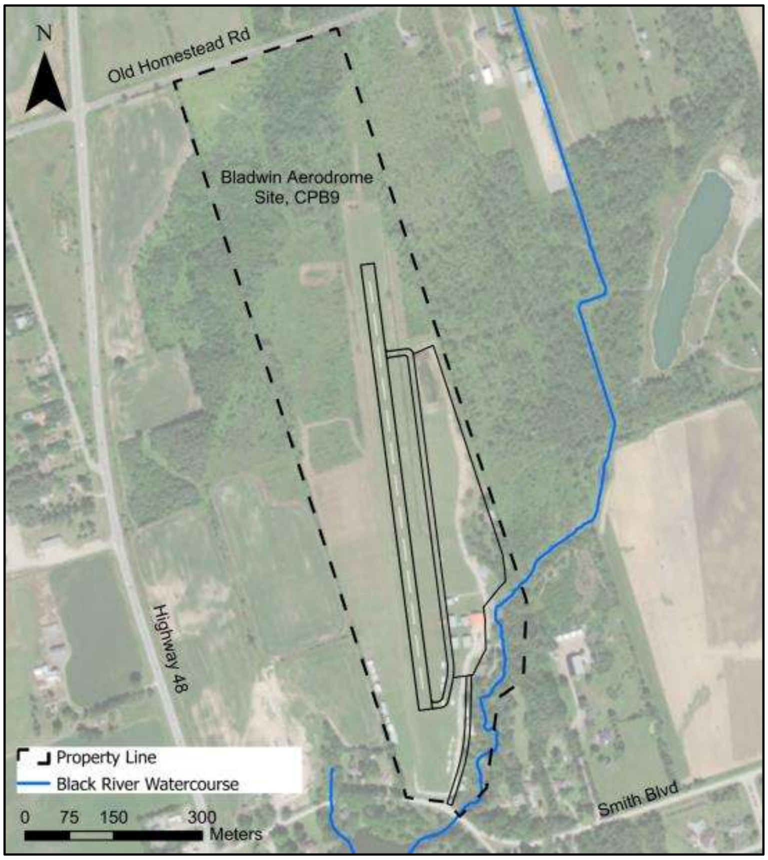


Figure 4: Post-development site layout

5.2.2 Hydrology

Based on the proposed site layout, Baldwin aerodrome site was split on the proposed land cover type. Similar to the pre-development conditions, land cover type was categorized per LSRCA Technical Guidelines for Stormwater Management Submissions. The proposed runway, taxiway, hangar storage and access road were assigned pavement land cover (runoff coefficient c, 0.9), whilst pervious portion of the site is proposed to be lawn (runoff coefficient c, 0.2). Figure 5 depicts assumed land cover within Baldwin aerodrome site under the post-development conditions.

The runoff and peak discharge rates were calculated for all storm events using City of Barrie WPCC Intensity-Duration-Frequency (IDF) curves, as specified by the Town of Georgina Development Design Criteria. The IDF curve parameters for Barrie WPCC Station #6110557 are increased by 10% for the 1:25-year storm, 20% for the 1:50-year storm, and 25% for the 1:100-year storm, per MTO Design Chart 1.07, as suggested in the LSRCA guidelines . The Rational Method was used to undertake required hydrology calculations using a 10-minute timestep. Table 4 summarizes the results for runoff and peak discharge rates under the post-development conditions. The hydrology calculations for Baldwin aerodrome site are provided in Appendix B.

Table 4: Post-development hydrology

Catchment	Area (ha)	Weighted runoff coefficient, C	10-Yr peak discharge (L/s)	100-Yr peak discharge (L/s)
C1	10.40	0.57	742.6	1344.3
C2	9.39	0.27	144.7	263.9
C3	1.44	0.34	87.8	158.2

Table 5: Estimated change in peak discharge rates

Catchment	Δ 10-Yr peak discharge (L/s)	Δ 100-Yr peak discharge (L/s)
C1	+619.1	+1119.2
C2	+91.3	+166.3
C3	+37.0	+66.4

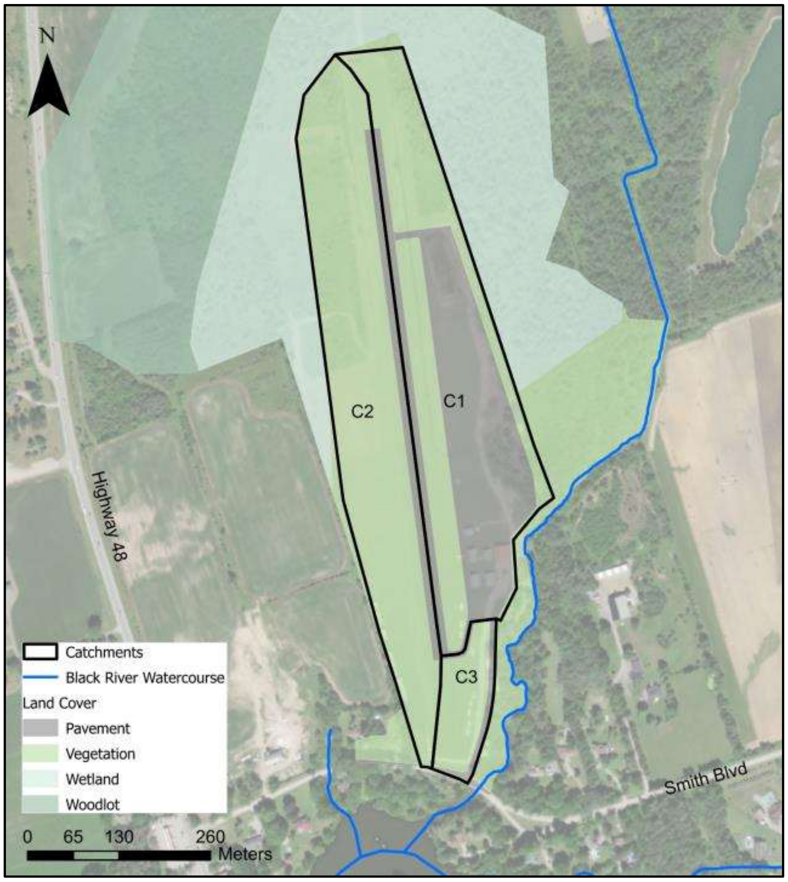


Figure 5: Post-development land cover

5.3 Conceptual design

The site will be divided into 3 catchment areas.

- Catchment C1 includes a portion of the runway, the taxiway and the apron area. Runoff will generally flow from northwest to southeast. A swale is proposed on the east side of the apron to collect runoff from leaving the site. Catchbasins will be used to collect runoff at low points in the swale and conveyed through an underground pipe network towards an underground storage tank. The use of sediment control check dams, installed every 100 m within the swales, will help to lodge sediment and prevent it from entering the catchbasins. The flow is proposed to pass through an Oil-Grit Separator (OGS) before being discharged into Black River.
- Catchment C2 includes the western portion of the runway and the embankment area. A swale is proposed on the west side of the embankment to collect runoff from leaving the site. The swale will convey the runoff towards a dry detention basin, which is used for water quality and quantity control.

Stormwater will be discharged into the roadside ditch along Smith Blvd, thus maintaining existing flow paths.

- Catchment C3 includes the site access road (driveway). A swale is proposed on the west side of the road to collect and store stormwater runoff. Stormwater will be discharged into the roadside ditch along Smith Blvd, thus maintaining existing flow paths.

Finally, any external runoff that would previously flow through the site will be conveyed via swales along the property lines to maintain existing flow patterns. No flow control will be provided for these property line swales.

5.3.1 Swales

The proposed swales are designed as grassed swales and sized to convey a 100-year 24-hour rainfall event. It should be noted that the swales were sized based on the peak discharge rates estimated using the Rational Method.

All swales are proposed to be trapezoidal. Depth of the swales range from 0.5m to 1.0m depending on contributing flow. The minimum longitudinal slope was set to 0.10%. The minimum swales slope is due to constraints for the runway, taxiway and runway end safety area.

Riprap erosion protection is proposed at bends in the swales where the flow changes direction. Swale inlet and outlet riprap protection are also included in the design.

See drainage plan in Appendix C.

5.3.2 Stormwater basin

The proposed stormwater basin is designed in accordance with LSRCA's Technical Guidelines for Stormwater Management Submissions. The water quantity and water quality requirements for the site were compared to determine the required storage capacity of the stormwater basin. The size of the basin is governed by water quantity volume. The basin is designed as a dry detention basin with a storage capacity of approximately 2910.9m^3 at the maximum water level. The basin is proposed to have a permanent storage depth of 1m and active storage depth of 1m, resulting in the total depth of 2m. The stormwater basin is proposed to be constructed with 3H:1V side slope. A 3m wide vegetative buffer is proposed to be provided around the edge of the basin. The basin bottom gradually slopes towards the outlet pipe at 0.30%.

In order to comply with the "water quality" requirements, the stormwater basin is proposed to have an outlet structure containing an orifice sized to detain the water quality volume over at least 24-hours. It should be noted that the stormwater basin is also proposed to have a forebay to aid with water quality pre-treatment and sediment settlement.

An outlet control swale is also proposed to allow stormwater to discharge into the existing roadside ditch along Smith Blvd at the allowable release rate.

See Appendix C for details of the proposed stormwater basin.

5.3.3 Underground drainage network

Stormwater collected from the taxiway and apron areas will be conveyed to an underground storage tank via underground pipes. Discharge from the storage tank will be conveyed through a 250mm diameter underground pipe towards the Black River outfall. Ideally, all storm pipes should have minimum 1.7m cover from proposed grade to pipe obvert. Any pipes with reduced cover need to consider insulation and concrete encasement for additional strength.

5.3.4 Underground storage tank

Due to the flatness of the site and limited elevation difference, it is not possible to convey all stormwater runoff to a single stormwater basin. Therefore, in order to comply with "water quantity" requirements for the apron area, an underground storage tank is proposed on the east side of the site. Stormwater runoff will be collected by catchbasin structures at low points in the swales and directed towards the storage tank. The storage tank is designed with a storage capacity of approximately 4187m^3. The basin is proposed to have an approximate width of 28m, length of 110m and depth of 1.4m.

The storage tank is proposed to have an outlet structure containing an orifice sized to control the discharge to Black River. The proposed outlet structure also includes a weir wall that will allow for emergency overflow out of the storage tank under more severe storm events.

In order to comply with the "water quality" requirements, an Oil-Grit Separator (OGS) is also proposed downstream of the stormwater outlet structure to aid with grit, oil, grease, sediment, and debris separation from stormwater, providing an additional level of water quality treatment for the outgoing flow from storage tank. The unit shall consist of a precast manhole structure, a frame and cover, an inlet pipe, and an outlet pipe. The OGS unit in this case is a polishing step to achieve enhanced level quality control, which is why it is positioned downstream. The proposed storage tank system may be provided with a sediment trap/forebay as pre-treatment device to limit ingress of sediment into storage facility (ex. ADS StormTech Isolator Row or equivalent).

5.3.5 Outfall to Black River

A stabilized riprap outfall is proposed for at the point of water discharge to Black River. The proposed outfall includes a concrete headwall and riprap apron per City of Barrie and Ontario Provincial Standards Drawings (OPSD). From the LSRCA Hydraulic Model the 100-year water level is assumed at an elevation of 228.2m. The proposed outlet is set 0.5m higher than the river's 100-year water level (el. 228.7m) to mitigate the risk backflow as well as minimizing the erosion by accommodating a riprap strip. See Appendix C for details of the proposed outfall to Black River.

5.3.6 Culvert under driveway

In existing conditions, there is a 400mm diameter culvert under the driveway (Asset ID: G-SWCV-5897). The culvert has an upstream invert elevation of 227.668m and a downstream invert elevation of 227.185m (per culvert details provided in the letter response from the Town of Georgina Development Services Department dated October 10, 2023). In the proposed conditions, the culvert is to be replaced in kind to maintain the existing outfall to Black River.

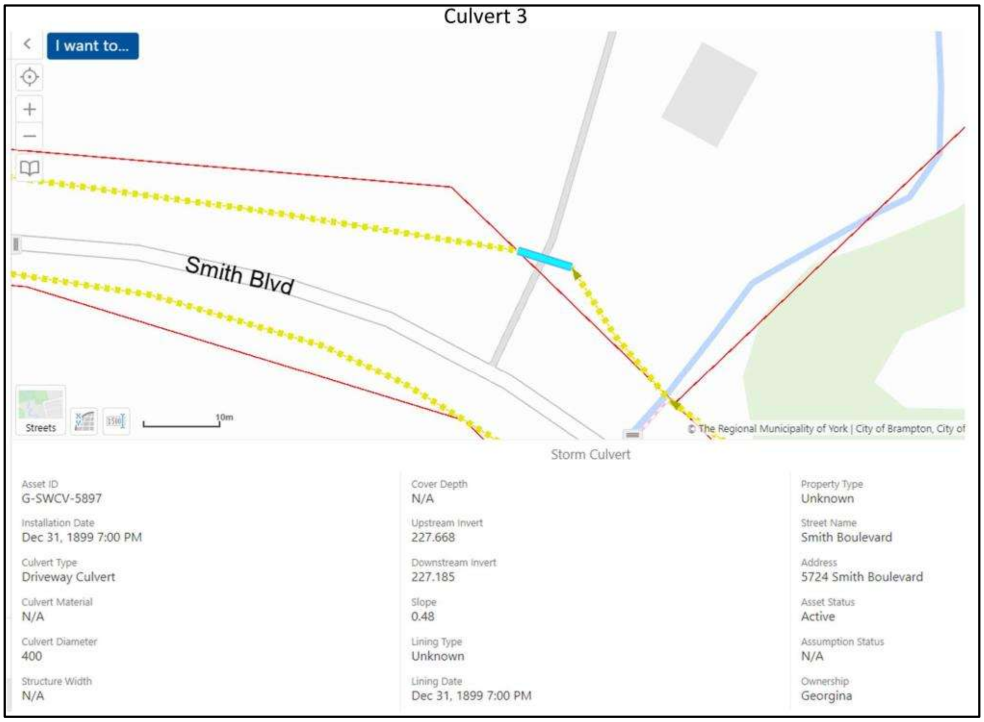


Figure 6: Existing culvert under driveway

6. Conclusions

Arup has undertaken the following actions to develop the stormwater management methodology and conceptual drainage design described in this Report:

- Reviewed the LSRCA hydraulic model of the Black River and validated it for use. Determined the extent of the existing floodplain limits within the site boundaries.
- Updated the LSRCA hydraulic model cross sections to reflect the proposed grading plan. Reviewed the impacts from the proposed grading plan on the existing floodplain. The proposed model results indicated that there is no significant impact on the Black River floodplain. Therefore, it was determined that a cut/fill compensation analysis for flood storage was not necessary at this stage of design.
- Revised the Grading Plan to address comments received from the Town.
- Conducted a hydrologic analysis of the pre and post development conditions at the site. This data was used to prepare a preliminary design for permanent drainage infrastructure and stormwater management facilities to meet quantity and quality control criteria.
- Developed an Erosion and Sediment Control plan for the site summarizing the erosion control measures that should be in place post-construction particularly to allow time for any of the surface materials such as vegetation to establish.

Closure

The work and engineering analysis presented in this Report are based on Arup's current understanding of the project requirements. This Report is subject to the Statement of Qualifications and Limitations presented at the beginning of this Report.

Arup should be retained for a general review of the any developments on the project to verify that this Report, if considered in any of the developments, has been properly interpreted and implemented. Any design changes that impact the analyses in this Report should be assessed by Arup, or alternatively an approved Qualified Person, to determine whether the conclusions of this Report require review and modification.

The hydrology analysis and preliminary stormwater management strategy provided in this Report are based on the available data at the time of design development. The preliminary stormwater management strategy provided respected the aims of the local guidelines of the Town of Georgina and LSRCA as a starting point, as set out in section 2 above. Note that changes in existing condition assumptions may impact the stormwater management strategy, in which case Arup should be provided with the opportunity to review the design strategy.

We trust this information satisfies your requirements at this time. Should you have any questions on the contents of this Report, please do not hesitate to contact the signatories.

Prepared by

Michael Tran	Michael Tran	
		November 1, 2024
Bailey Sadowsky	Bailey Sadowsky	
Name	*Signature*	*Date*

Checked by

George Long		
Joshua Battiston, P.Eng.		November 1, 2024
Name	*Signature*	*Date*

Approved by

Joshua Battiston, P.Eng.		November 1, 2024
Name	*Signature / Seal*	*Date*

Appendix A - LSRCA model review

A.1 Introduction

This technical memorandum has been produced to summarise the review of the Lake Simcoe Region Conservation Authority (LSRCA) models of the Black River for reaches in proximity to the site.

A.2 Models Received

The LSRCA provided the following data to Arup on June 26, 2024:

Table 6: Data received from LSRCA

File Name	File Format
Greenland Hydrology Model (Nov 2003)	Visual OTTHYMO (VO)
Greenland Hydrology Report (Nov 2004)	PDF
H_TRB_B4 Hydraulic Model	HEC-2
H_TRB_B4 Hydraulic Model Output File (Sep 2006)	.OUT

A.3 Hydrologic Model

Greenland Hydrology Model was provided as a Visual OTTHYMO (VO) file, along with the accompanying Greenland Hydrology Report. A screenshot of the model is show in Figure 7 below.

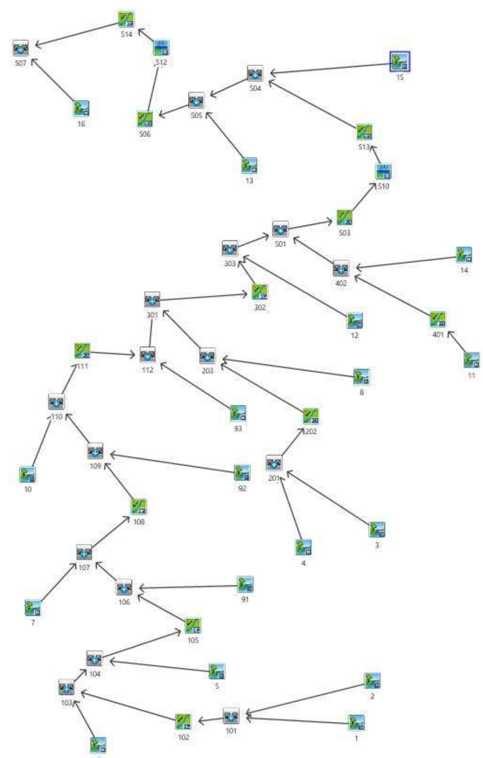


Figure 7: Greenland V06 Hydrology Model

A.3.1 Greenland Hydrology Model Validation

The Greenland Hydrology Model was provided as a Visual Otthymo 2 (V02) file. Since the current Visual Otthymo 6 (V06) software supersedes the V02 software, the data was imported into V06 for review and analysis. Upon review of the provided Greenland Hydrology Report, Catchment 15 captures the location of the proposed site, as seen in Figure 8.

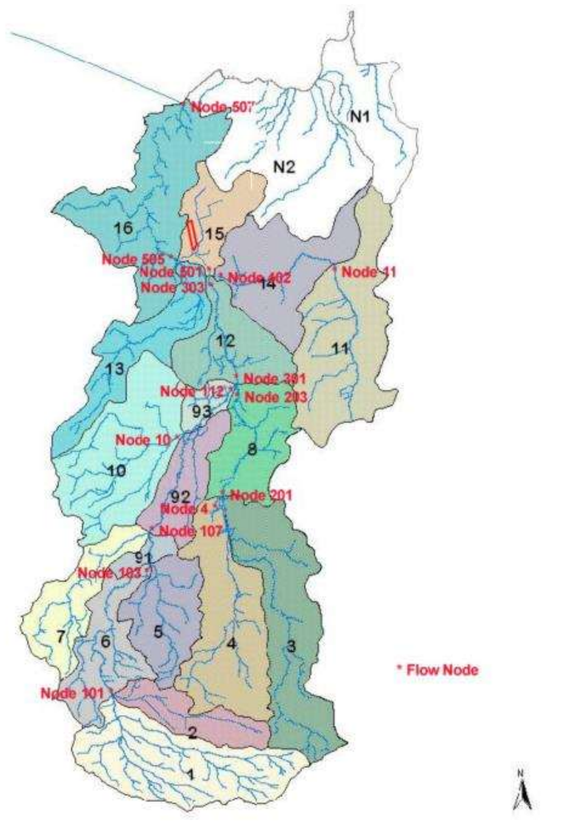


Figure 8: Existing Catchments used in Greenland Hydrology Model

Utilizing the provided Greenland Hydrology Model, a model validation review was conducted. Input parameters for Catchment 15 were cross-checked against those delineated for the current existing conditions for Baldwin Aerodome project, ensuring consistent results. Catchment areas were validated using LSRCA GIS Open Data catchment files, while hydrographs and flow paths in the model were verified against real-world conditions using Google Maps and topography data to accurately reflect any landscape changes since the completion of the existing model. Land use and soil types were verified using open data provided from the Ministry of Agriculture and Food (see Figure 9). The Greenland Hydrology Model was then rerun using the design storm files and parameters provided, yielding results that are acceptably similar to the values reported in the Hydrology Report. These validation processes confirm the reliability of the model.

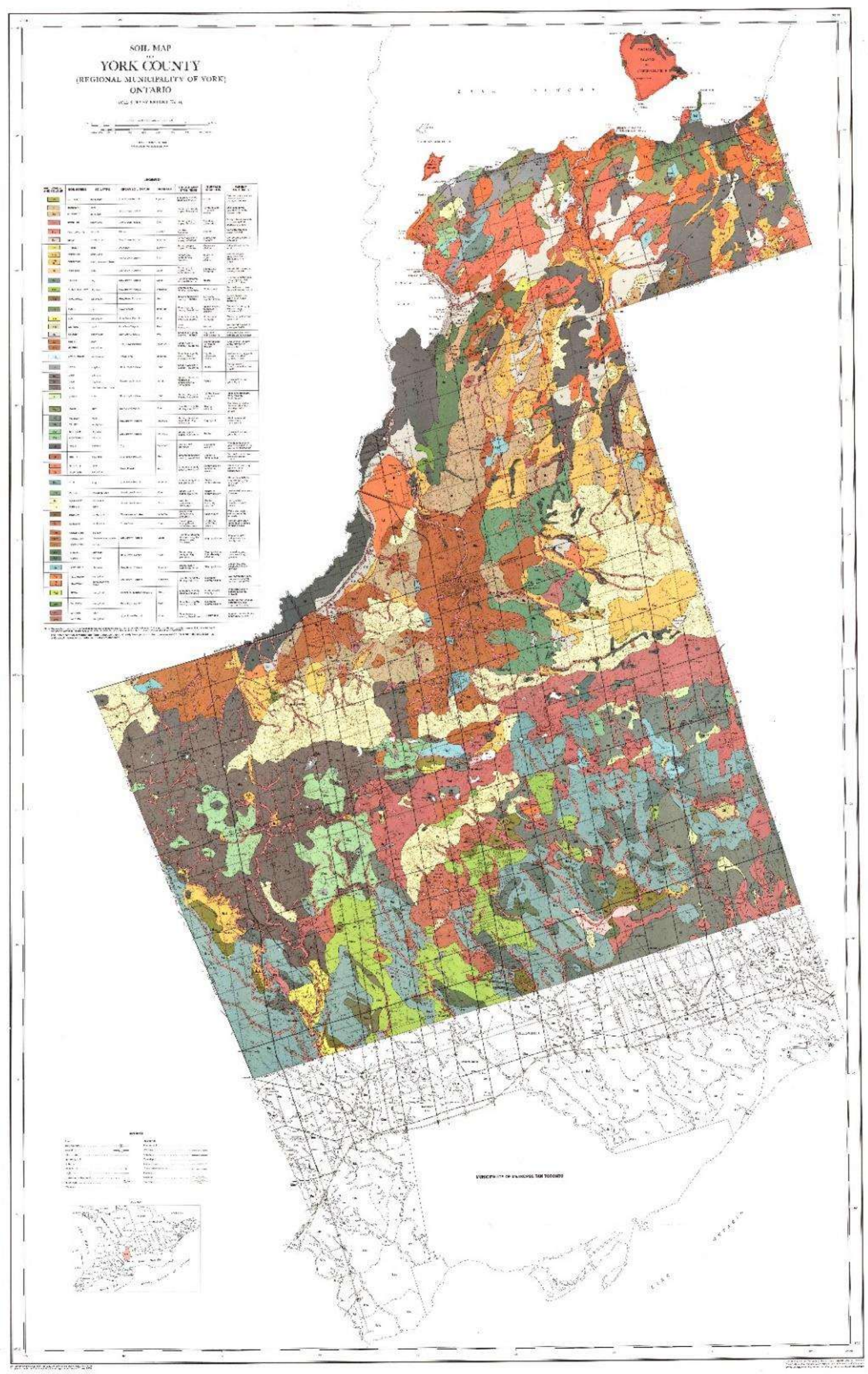


Figure 9: Ministry of Agriculture and Food landuse and soil type map

A.3.2 Next steps - Proposed Conditions Model

Since the provided Greenland Hydrology Model captures a much greater area as compared to the proposed project site, a new V06 Model should be created to capture a smaller extent around the proposed site where proposed peak flow rates may change due to proposed design. The existing Greenland Hydrology Model will be used to verify that both pre- and post-development runoff values computed are reasonable, as well as serve as a baseline to ensure post-development peak flow rates will not exceed pre-development peak flow rates with the proposed stormwater management strategy.

A.4 LSRCA GIS Open Data

Using the GIS Open Data provided online by the LSRCA, it was determined that there are five cross-sections from the hydraulic model that intersect the site. Figure 10 shows the relevant Black River cross-sections. The water elevations for the cross-sections (provided by GIS Open Data) are summarized in Table 7 below.

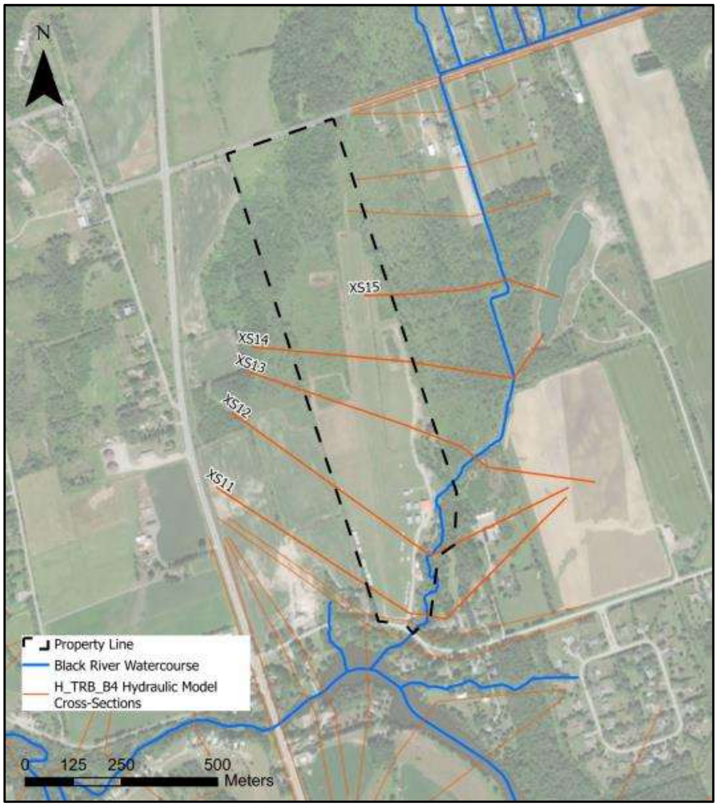


Figure 10: Black River cross-sections

Table 7: Water elevations for Black River cross-sections

River Station	Water Elevation (m)					
	2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
15	229.21	229.55	229.71	229.83	229.98	230.17
14	228.58	228.90	228.94	228.94	228.89	229.20
13	227.24	227.64	227.87	227.99	228.11	228.71
12	227.01	227.78	227.93	228.03	228.13	228.71
11	227.00	227.77	227.93	228.03	228.13	228.70

A portion of the southeast corner of the site is within the Black River floodplain. Using the water elevation data above, the existing floodplain limits within the site boundary are shown in Figure 11 below.

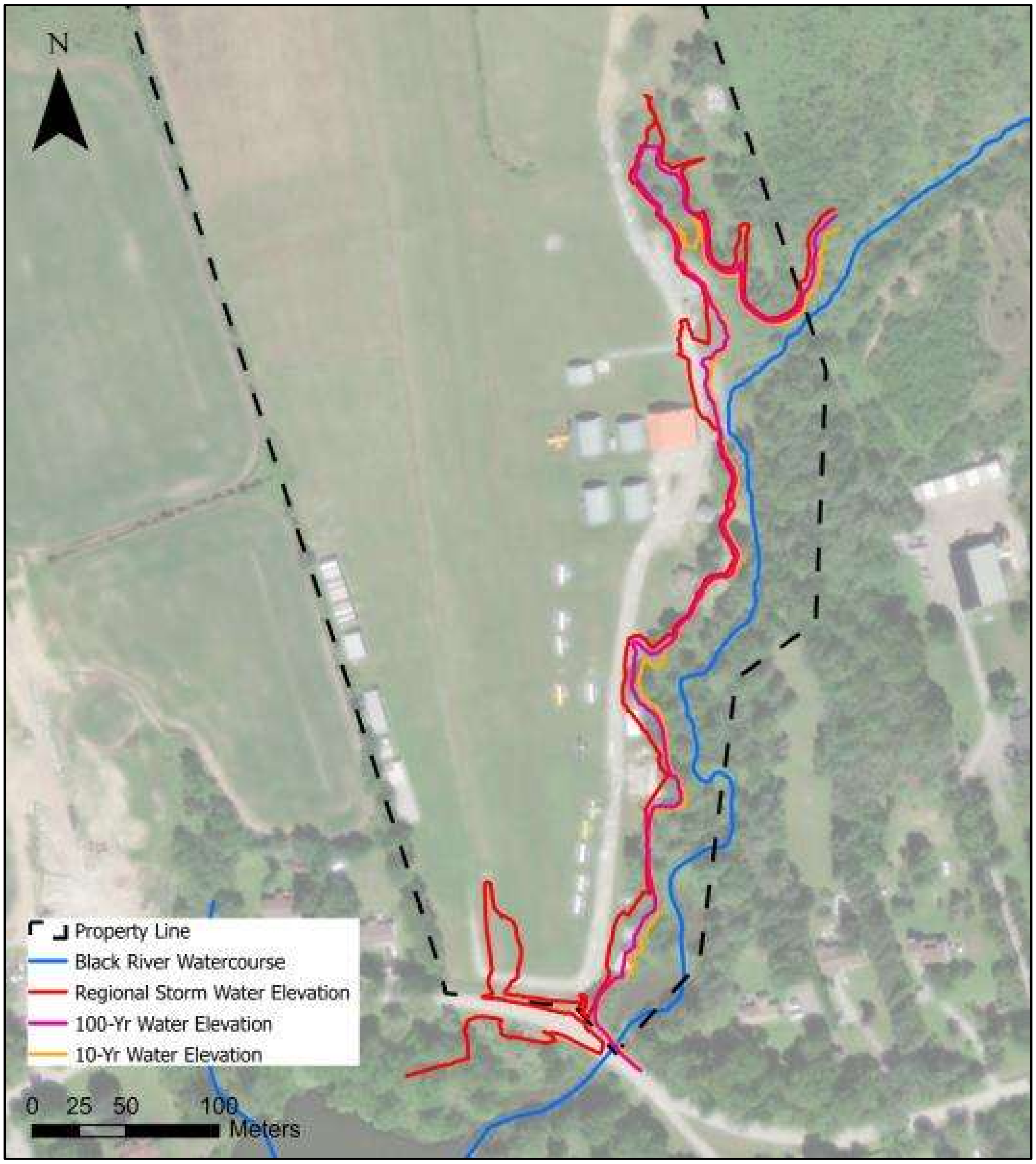


Figure 11: Floodplain limits within site boundary

A.5 H_TRB_B4 hydraulic model

A.5.1 Existing conditions

The *H_TRB_B4* hydraulic model was provided as a HEC-2 file. Since the current HEC-RAS software supersedes the HEC-2 river hydraulics package, the data was imported into HEC-RAS for review and analysis. A screenshot of the imported data is shown in Figure 12 below.

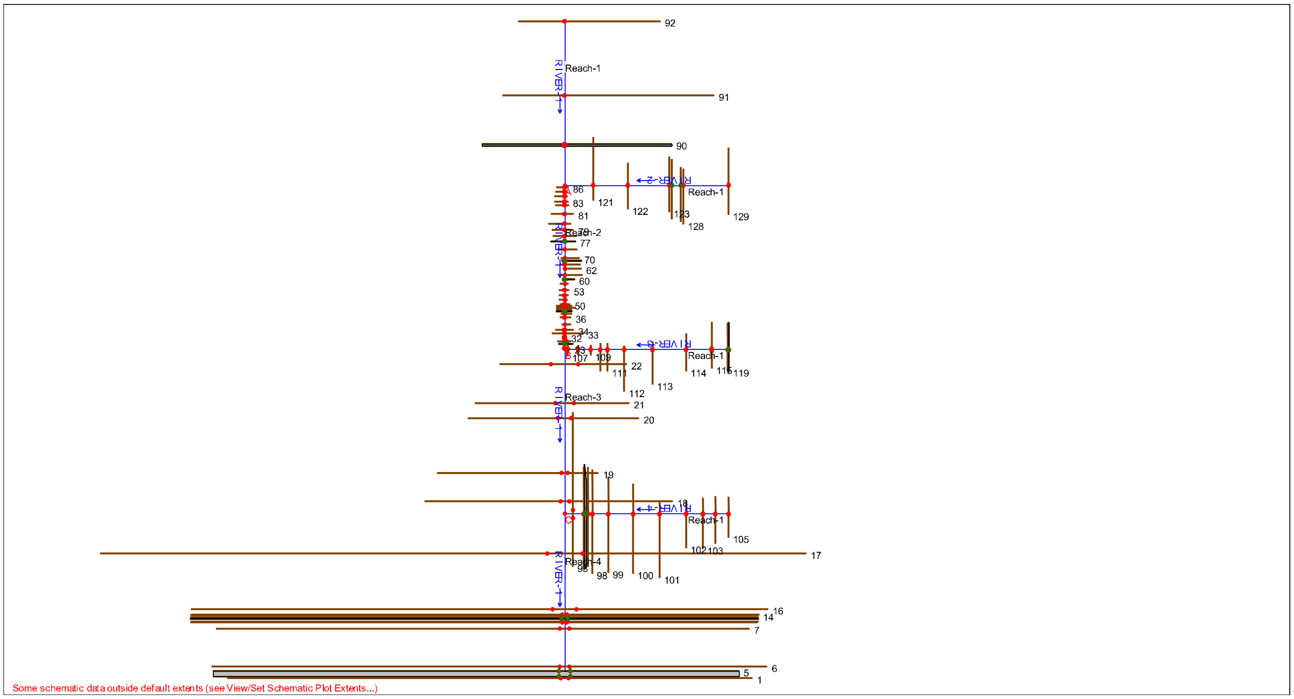


Figure 12: HEC-RAS geometric data view of H_TRB_B4

When the data was imported into HEC-RAS, the cross-sections were automatically renamed. The new names of the relevant cross-sections were determined by reviewing the descriptions included in the model.

The imported HEC-RAS model was also missing key bridge/culvert information which prevented the model from running. Since this information was not available, all bridge and culverts were removed from the hydraulic model. Water elevation checks on the downstream sections immediately following the bridges and culverts were compared between the GIS Open Data and the model run without bridges and culverts. The results showed minimal variations with no significant impact to water elevations. Additionally, a check for overtopping in existing conditions at the culvert located southwest of the site was found not to be overtopped. The roadway elevation above the culvert is 228.67m and the water elevation at the culvert is 228.41m upstream and 228.53m downstream, therefore all bridges and culverts were removed from the model. The results from the imported HEC-RAS model (without bridge/culverts) vary slightly from the water elevation data provided by the GIS Open Data.summarizes the difference in water elevation for each design storm.

Table 8: Difference in Water Elevations at Section 15

River Station		Water Elevation (m)					
		2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
GIS Open Data	15	229.21	229.55	229.71	229.83	229.98	230.17
Imported HEC-RAS model	15	229.21	229.55	229.74	229.88	230.05	230.19
Difference		0.00	0.00	0.03	0.05	0.07	0.02

Table 9: Difference in Water Elevations at Section 14

River Station		Water Elevation (m)					
		2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
GIS Open Data	14	228.58	228.90	228.94	228.94	228.89	229.20
Imported HEC-RAS model	14	228.58	228.90	228.90	228.87	228.84	229.19
Difference		0.00	0.00	0.04	0.07	0.05	0.01

Table 10: Difference in Water Elevations at Section 13

River Station		Water Elevation (m)					
		2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
GIS Open Data	13	227.24	227.64	227.87	227.99	228.11	228.71
Imported HEC-RAS model	13	227.24	227.67	227.93	228.06	228.16	228.72
Difference		0.00	0.03	0.06	0.07	0.05	0.01

Table 11: Difference in Water Elevations at Section 12

River Station		Water Elevation (m)					
		2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
GIS Open Data	12	227.01	227.78	227.93	228.03	228.13	228.71
Imported HEC-RAS model	12	227.01	227.79	227.97	228.08	228.17	228.72
Difference		0.00	0.01	0.04	0.05	0.04	0.01

Table 12: Difference in Water Elevations at Section 11

River Station		Water Elevation (m)					
		2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
GIS Open Data	11	227.00	227.77	227.93	228.03	228.13	228.70
Imported HEC-RAS model	11	227.00	227.79	227.97	228.07	228.17	228.72
Difference		0.01	0.02	0.04	0.04	0.04	0.02

A.5.2 Proposed conditions

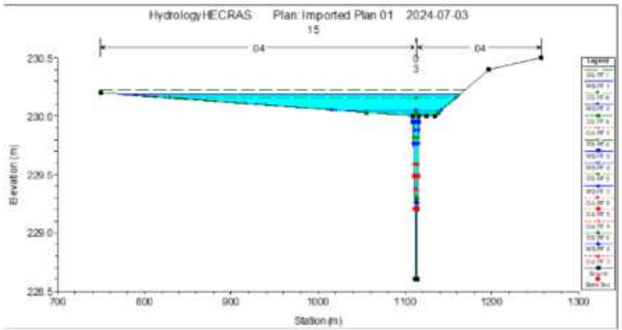
In the proposed condition, the Baldwin Airport runway, taxiway, and apron area are being raised. Therefore, the relevant cross-sections in the imported HEC-RAS model were updated using the proposed surface to determine the potential impact on the Black River floodplain. Figure 13 to Figure 17 show the existing condition and proposed condition cross-sections. The proposed condition water elevations are provided in Table 13 below.

Table 13: Proposed Condition Water Elevation Model Results

River Station	Water Elevation (m)					
	2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
15	229.21	229.54	229.69	229.81	229.94	230.06
14	228.58	228.90	228.90	228.88	228.84	229.2
13	227.24	227.66	227.92	228.06	228.16	228.72
12	227.01	227.79	227.97	228.08	228.17	228.72
11	227.00	227.79	227.97	228.07	228.17	228.72

Cross Section 15

Existing Cross Section



Proposed Cross Section

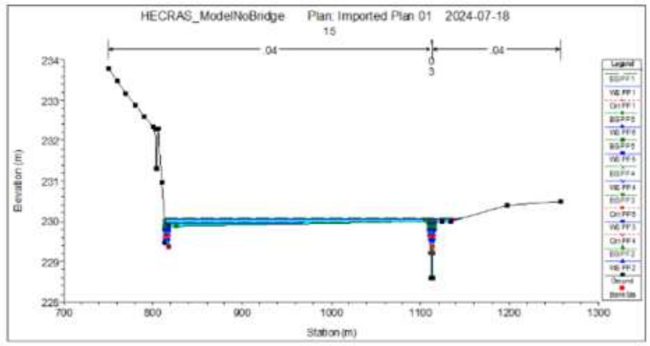
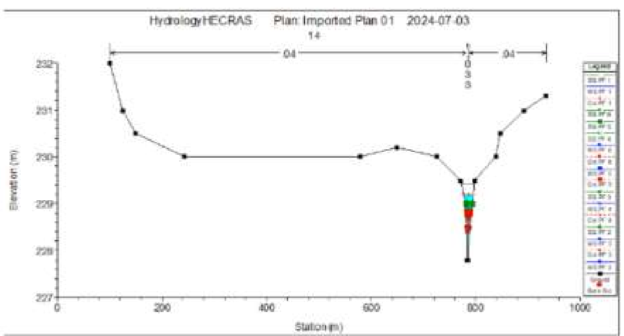


Figure 13: Changes to Cross Section 15

Cross Section 14

Existing Cross Section



Proposed Cross Section

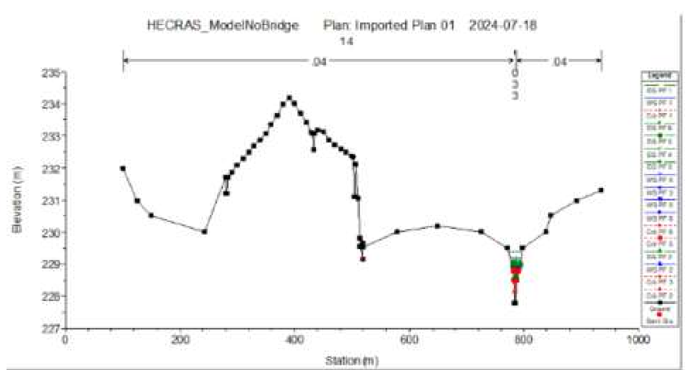


Figure 14: Changes to Cross Section 14

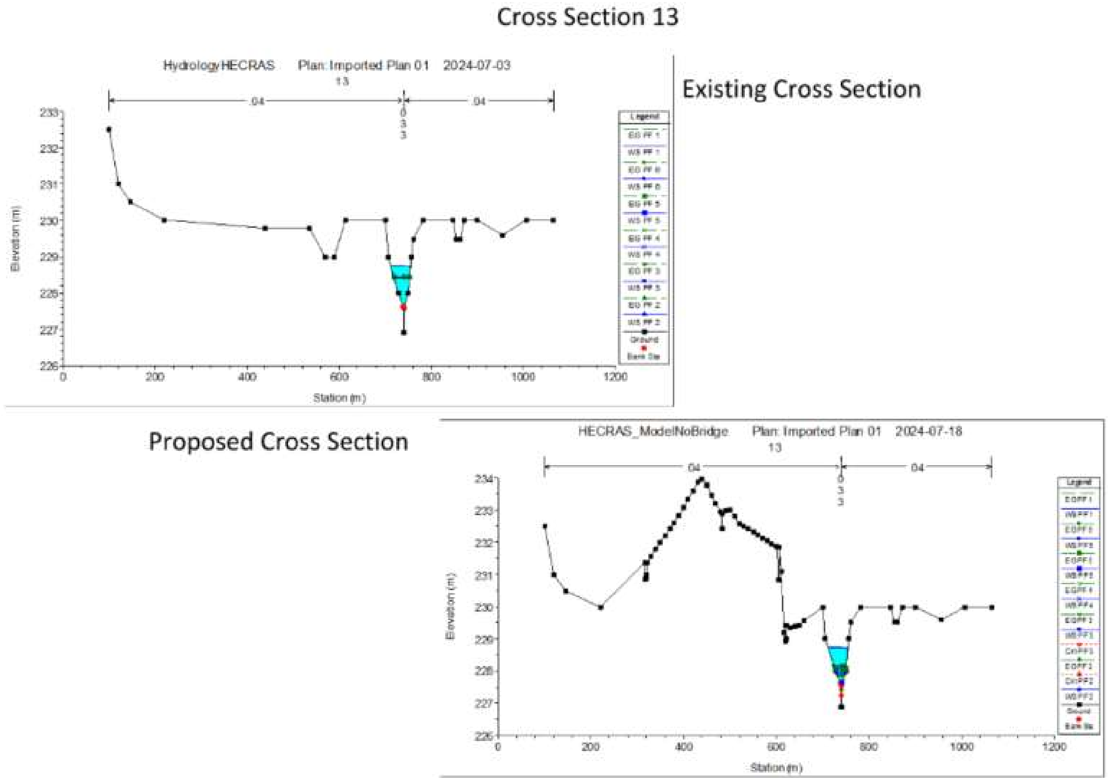


Figure 15: Changes to Cross Section 13

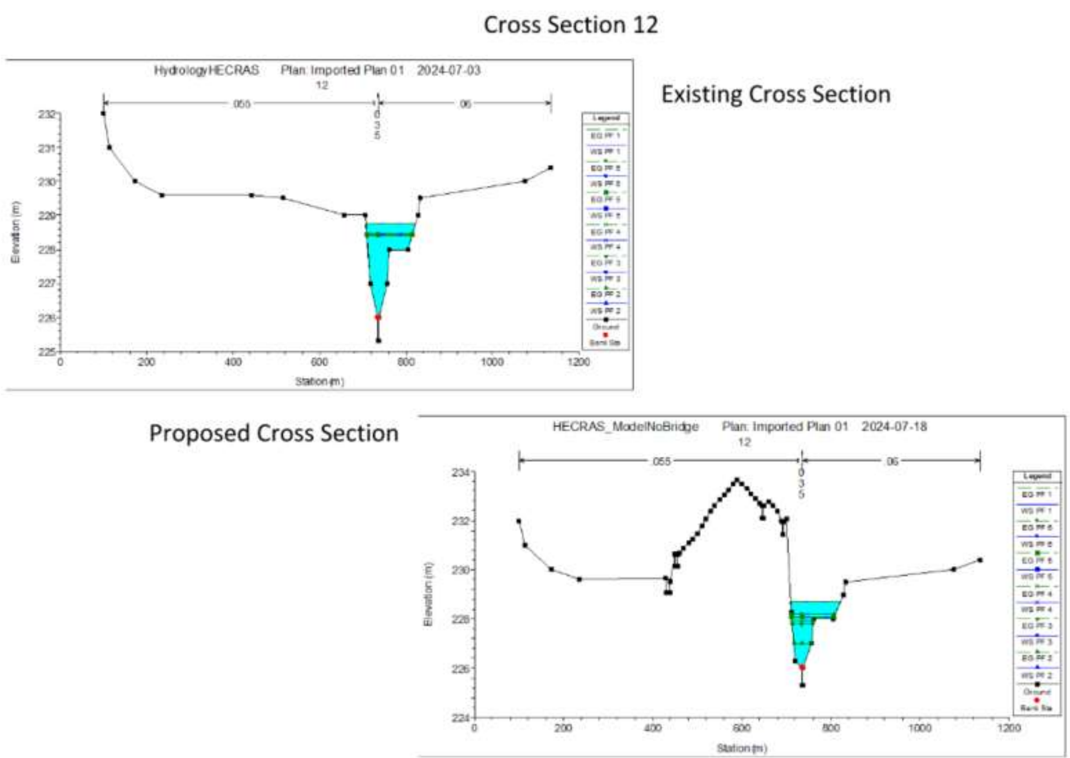


Figure 16: Changes to Cross Section 12

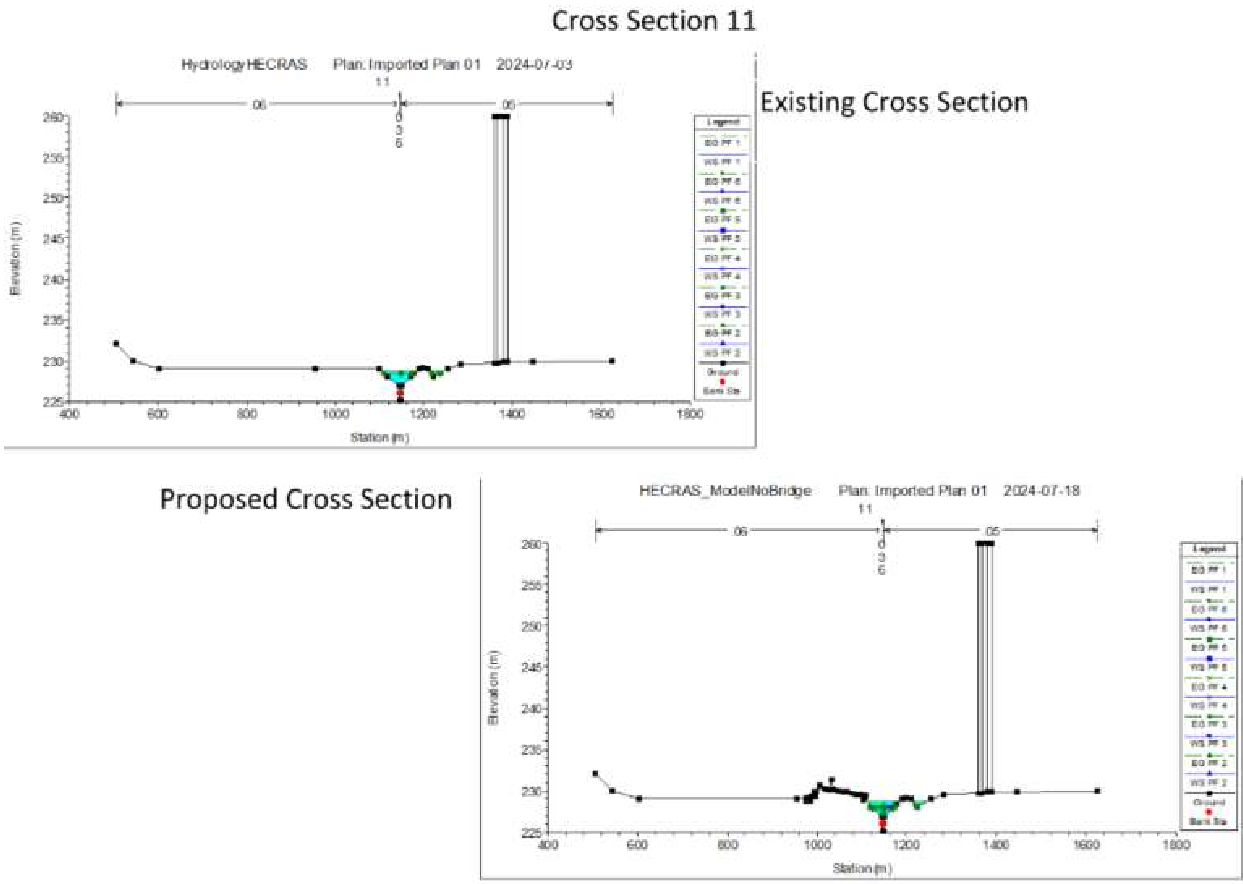


Figure 17: Changes to Cross Section 11

A.5.3 Results analysis

Table 14 to Table 18 summarizes the difference in water elevation for each design storm under the existing and proposed conditions.

Note that the existing condition water elevations from the imported HEC-RAS model were used to determine the relative impact of the proposed design on the Black River floodplain.

Table 14: Cross Section 15 Existing vs Proposed Water Elevations

	Water Elevation (m)					
	2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
Existing condition	229.21	229.55	229.74	229.88	230.05	230.19
Proposed condition	229.21	229.54	229.69	229.81	229.94	230.06
Difference	0.00	-0.01	-0.05	-0.07	-0.11	-0.13

Table 15: Cross Section 14 Existing vs Proposed Water Elevations

	Water Elevation (m)					
	2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
Existing condition	228.58	228.90	228.90	228.87	228.84	229.19
Proposed condition	228.58	228.90	228.90	228.88	228.84	229.20
Difference	0.00	0.00	0.00	0.01	0.00	+0.01

Table 16: Cross Section 13 Existing vs Proposed Water Elevations

	Water Elevation (m)					
	2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
Existing condition	227.24	227.67	227.93	228.06	228.16	228.72
Proposed condition	227.24	227.66	227.92	228.06	228.16	228.72
Difference	0.00	-0.01	-0.01	0.00	0.00	0.00

Table 17: Cross Section 12 Existing vs Proposed Water Elevations

	Water Elevation (m)					
	2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
Existing condition	227.01	227.79	227.97	228.08	228.17	228.72
Proposed condition	227.01	227.79	227.97	228.08	228.17	228.72
Difference	0.00	0.00	0.00	0.00	0.00	0.00

Table 18: Cross Section 11 Existing vs Proposed Water Elevations

	Water Elevation (m)					
	2-Year Storm Event	10-Year Storm Event	25-Year Storm Event	50-Year Storm Event	100-Year Storm Event	Regional Storm Event
Existing condition	227.00	227.79	227.97	228.07	228.17	228.72
Proposed condition	227.00	227.79	227.97	228.07	228.17	228.72
Difference	0.00	0.00	0.00	0.00	0.00	0.00

There appears to be no significant change between the proposed condition and existing condition water elevations. Generally, in the proposed conditions, the water elevations decrease slightly, except for Cross Section 14 where the water elevation increases by 0.01m during the 50-year and Regional Storm event. Also, the change in water elevation only occurs in cross sections upstream of the proposed site. There are no changes in water elevation for cross sections downstream of the site.

The updated *H_TRB_B4* hydraulic model and extracted water elevation data are used to inform the preliminary site layout and stormwater management design.

Appendix B - Calculations

ARUP

JOB TITLE	Baldwin Aerodome
JOB NUMBER	287943-00
MADE BY	MT
CHECKED BY	BS
DATE	10-10-24
Description of spreadsheet	Modified Rational Method for stormwater management facility sizing
Member/Location	Toronto
Report Reference	
Filename	\\global.arup.com\americas\Jobs\TOR\280000\287943-00\3 Design\3-06 Calcs\SWM Calcs\[Release Rates and Storage - 3

CONTENTS OF SPREADSHEET

Sheet	Description
1	SWM Drainage Area Calculations
2	SWM Drainage Calculations Summary
3	Weighted C and Tc Calculator
4	C1 - SWM Allowable Release Rate and Required Storage
5	C1 - Tank Outlet structure
6	C1 - ESC Pond Outlet structure
7	C1a - SWM Allowable Release Rate and Required Storage
8	C1b - SWM Allowable Release Rate and Required Storage
9	C2 - SWM Allowable Release Rate and Required Storage
10	C2 - Outlet structure
11	C3 - SWM Allowable Release Rate and Required Storage
12	C3 - Outlet structure

AUTHORISATION OF LATEST VERSION

Type and method of check		
Signatures & dates:	Made by	MT
	Checked	BS

REVISIONS Current Revision 1

Rev.	Date	Made by	Checked	Description
1	08-01-2024	MT	BS	
2	10-10-2024	MT	BS	

ARUP	Job No. 287943-00	Sheet 1	Rev. 1
	Member/Location	Toronto	
Job Title: Baldwin Aerodome	Drg. Ref.		
Calculation: SWM Drainage Area Calculations	Made by MT	Date 10-10-24	Chd. BS

Existing Controlled/Unncontrolled Area

Catchment ID	Total Area (ha)	Impervious Area (ha)	Semi_imperv (ha)	Wetlands (ha)	Woodlot (ha)	Pervious Area (ha)
C1	10.40	0.15	0.41	1.81	0	8.02
C2	9.39	0.11	0.01	5.15	0	4.12
C3	1.44	0.04	0.15	0	0	1.25
C1a	3.086	0.00	0.00	0.00	0.00	3.09
C1b	7.31	0.15	0.41	1.81	0.00	4.94

Proposed Controlled/Unncontrolled Area

Controlled/ Uncontrolled	Total Area (ha)	Impervious Area (ha)	Semi_imperv (ha)	Wetlands (ha)	Woodlot (ha)	Pervious Area (ha)
C1	10.40	5.55	0	0	0	4.85
C2	9.39	0.88	0	0	0	8.51
C3	1.44	0.29	0	0	0	1.15
C1a	3.086	1.05	0.00	0.00	0.00	2.03
C1b	7.31	4.50	0.00	0.00	0.00	2.81

volume required to treat 25mm runoff (Existing)

Total Impervious Area (whole Site)	0.68	ha
Target Volume Control Required	169.75	m3
Flow Area, **A** (m^2)		
Wetted perimeter, **Wp** (m)		
Hydraulic Radius, **Rh** (m)	5.55	ha
Full flow capacity, **Qcapacity**, m3/s	1387.75	m3

volume required to treat 25mm runoff (Proposed Catchment 2)

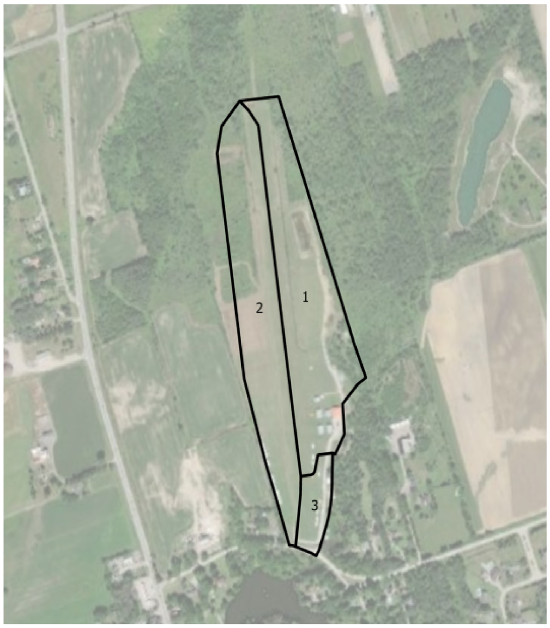
Total Impervious Area (whole Site)	0.88	ha
Target Volume Control Required	220.25	m3

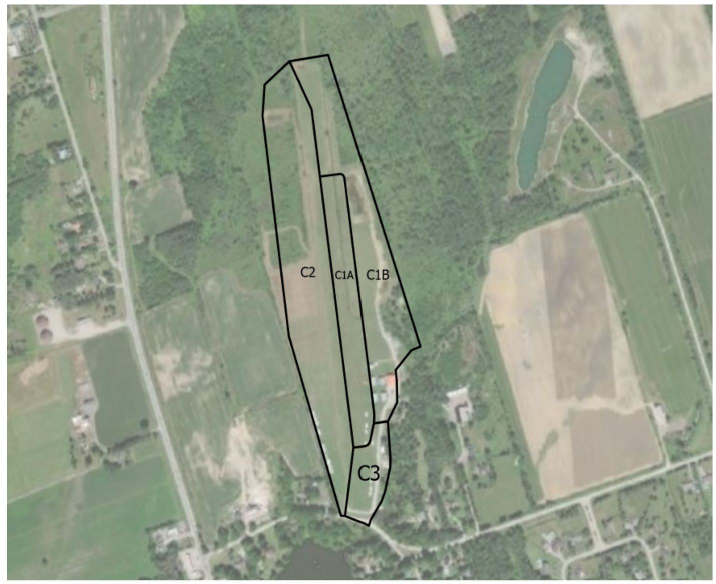
volume required to treat 25mm runoff (Proposed Catchment 3)

Total Impervious Area (whole Site)	0.29	ha
Target Volume Control Required	72.00	m3

ARUP	Job No. 287943-00	Sheet 2	Rev. 1
	Member/Location	Toronto	
Job Title: Baldwin Aerodome	Drg. Ref.		
Calculation: SWM Drainage Calculations Summary	Made by MT	Date 10-10-24	Chd. BS

Catchment ID	Water Quantity Volume (m3)	Water Quality Volume (m3)	Storage Volume Required (m3)
C1	3806.26	1387.75	3806.26
C2	1570.34	220.25	1570.34
C3	140.67	72.00	140.67





ARUP					
	Job No.	287943-00	3	Rev.	0
	Member/Location	Toronto			
Job Title	Baldwin Aerodome	Drg. Ref.	XXX		
Calculation	Weighted C and Tc Calculator	Made by	M.T.	Date	10-10-2024
		Chd.	B.S.		

Existing Condition

Catchment ID	Area (ha)	Land Cover																Weighted Runoff 'C'	%Impervio
		Tree		Grass		Gravel Road		Wetlands		Building		Runway		Other		Shrub			
		Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C		
Ex_C1	10.40	0.00	0.42	8.02	0.2	0.4	0.5	1.8140	0.05	0.00	0.95	0.15	0.9	0.00	0.95	0.00	0.35	0.20	5.39
Ex_C2	9.39	0.00	0.42	4.12	0.2	0.0	0.5	5.1480	0.05	0.00	0.95	0.11	0.9	0.00	0.95	0.00	0.35	0.13	1.27
Ex_C3	1.44	0.00	0.42	1.25	0.2	0.2	0.5	0.0000	0.05	0.00	0.95	0.04	0.9	0.00	0.95	0.00	0.35	0.25	13.54
Ex_C1a	3.086	0.00	0.42	3.09	0.2	0.0	0.5	0.0000	0.05	0.00	0.95	0.00	0.9	0.00	0.95	0.00	0.35	0.20	0.00
Ex_C1b	7.31	0.00	0.42	4.94	0.2	0.4	0.5	1.8140	0.05	0.00	0.95	0.15	0.9	0.00	0.95	0.00	0.35	0.19	7.66

Proposed Condition

Catchment ID	Area (ha)	Land Cover																Weighted Runoff 'C'	%Impervio
		Tree		Grass		Gravel Road		Wetlands		Building		Runway		Other		Shrub			
		Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C	Area (ha)	C		
Pr_C1	10.40	0.00	0.42	4.85	0.2	0.00	0.5	0.0000	0.05	0.00	0.95	5.55	0.9	0.00	0.95	0.00	0.35	0.57	53.39
Pr_C2	9.39	0.00	0.42	8.51	0.2	0.00	0.5	0.0000	0.05	0.00	0.95	0.88	0.9	0.00	0.95	0.00	0.35	0.27	9.38
Pr_C3	1.44	0.00	0.42	1.15	0.2	0.00	0.5	0.0000	0.05	0.00	0.95	0.29	0.9	0.00	0.95	0.00	0.35	0.34	20.00
Pr_C1a	3.086	0.00	0.42	2.03	0.2	0.00	0.5	0.0000	0.05	0.00	0.95	1.05	0.9	0.00	0.95	0.00	0.35	0.44	34.12
Pr_C1b	7.31	0.00	0.42	2.81	0.2	0.00	0.5	0.0000	0.05	0.00	0.95	4.50	0.9	0.00	0.95	0.00	0.35	0.63	61.53

Time of Concentration Calculator

Existing Condition

Catchment ID	Input	Catchment Max El (m) (85)	Catchment Min El (m)(10)	Catchment Flow Length (m)	Avg. Slope (%)	Imperviousness (%)	C				Time of Concentration [min]			Tp
	Area (ha)						5 Yr	25 Yr	50 Yr	100 Yr	Airport Method (C<0.4)	Bransby-Williams Method (C>0.4)	Selected Tc	
Ex_C1	10.40	228.926	227.924	761.5335	0.18	5.39	0.20	0.22	0.24	0.24	144.461	48.64581	144.461	96.30735
Ex_C2	9.39	229.191	228.503	972.7889	0.09	1.27	0.13	0.14	0.15	0.16	215.8455	71.07636	215.8455	143.897
Ex_C3	1.44	228.808	227.464	230.1446	0.78	13.54	0.25	0.28	0.30	0.32	45.50283	13.29756	45.50283	30.33522
Ex_C1a	3.09	229.624	228.866	581.793	0.17	0.00	0.20	0.22	0.24	0.25	126.0948	42.04708	126.0948	84.06323
Ex_C1b	7.31	229.015	227.887	752.281	0.20	7.66	0.19	0.21	0.23	0.24	137.7884	48.49418	137.7884	91.85896

Proposed Condition

Catchment ID	Input	Catchment Max El (m) (85)	Catchment Min El (m)(10)	Catchment Flow Length (m)	Avg. Slope (%)	Imperviousness (%)	C				Time of Concentration [min]			Tp
	Area (ha)						5 Yr	25 Yr	50 Yr	100 Yr	Airport Method (C<0.4)	Bransby-Williams Method (C>0.4)	Selected Tc	
Pr_C1	10.40	231.31	230.384	817.1834	0.15	53.39	0.57	0.63	0.69	0.72	91.50975	53.78403	53.78403	35.85602
Pr_C2	9.39	230.695	229.335	1018.793	0.18	9.38	0.27	0.29	0.32	0.33	153.4497	65.55655	153.4497	102.2998
Pr_C3	1.44	231.278	228.703	216.6375	1.58	20.00	0.34	0.37	0.41	0.43	31.32592	10.85865	31.32592	20.88394
Pr_C1a	3.09	232.515	232.014	556.486	0.12	34.12	0.44	0.48	0.53	0.55	102.345	43.30371	43.30371	28.86914
Pr_C1b	7.31	231.351	230.559	674.61	0.16	61.53	0.63	0.69	0.76	0.79	73.27397	45.66818	45.66818	30.44545

Utilized MTO 85/10 method for watershed slope

ARUP	Job No. 287943-00	Sheet 4	Rev. 1
	Member/Location	Toronto	
Job Title: Baldwin Aerodome	Drg. Ref.		
Calculation: C1 - SWM Allowable Release Rate and Required Storage	Made by MT	Date 10-10-24	Chd. BS

Runoff Coefficients

Impervious Area	0.9
Pervious Area	0.2
Semi Impervious Area	0.5
Wetlands Area	0.05
Woodlots Area	0.42

Pre-Development Conditions

Catchment Area	10.40	ha
Impervious Area	0.15	ha
Pervious Area	8.02	ha
Semi Impervious (Gravel Road)	0.41	ha
Wetlands Area	1.81	ha
Woodlots Area	0.00	ha
Composite C Coeff.	0.20	

Post-Development Conditions

Catchment Area	10.40	ha
Impervious Area	5.55	ha
Pervious Area	4.85	ha
Semi Impervious (Gravel Road)	0.00	ha
Wetlands Area	0.00	ha
Woodlots Area	0.00	ha
Composite C Coeff.	0.57	

Development Conditions Comparison

Catchment Area	-0.002 ha	0%
Impervious Area	5.3990 ha	52%
Pervious Area	-3.179 ha	-31%

ALLOWABLE PEAK FLOW RATES

Pre-Development Catchment Area	10.40	ha
Pre-Development Time of Concentration (t)	144	min

Pre-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Bottom Width, **Wb** (m)	675.586	843.019	976.898	1133.123	1251.473	1383.628
B	4.681	4.582	4.745	4.734	4.847	4.905
C	0.78	0.763	0.76	0.756	0.753	0.754
i (mm/hr)	13.7	18.6	21.8	25.8	28.9	31.8

**Values taken from Town of Georgina Development Design Criteria and Standards*
https://www.georgina.ca/sites/default/files/page_assets

Release Rates (Q, in L/s)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Ca Value	1.00	1.00	1.00	1.10	1.20	1.25
Rational Method Release Rates (Q = 2.78 CaCiA)	77.3	105.1	123.5	160.8	196.5	225.1
Regulated Allowable Release Rates	NA	NA	NA	NA	NA	NA
Design Allowable Release Rates	77.3	105.1	123.5	160.8	196.5	225.1

Flow Area, **A** (m^2)
Wetted perimeter, **Wp** (m)
Hydraulic Radius, **Rh** (m)
Full flow capacity, **Qcapacity**, m3/s

Post-Development Catchment Area	10.40	ha
Post-Development Runoff Coefficient	0.57	
Post-Development Time of Concentration (t)	53	min

Post-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
i (mm/hr)	28.6	38.3	44.8	52.8	58.9	64.9

Required Storage Volume

Release Flow Volume Calculation Method	Allowable Release Rate x Storm Duration	
Initial Storm Duration	10	min
Time Step	1	min

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Critical Duration (min)	98	104	106	107	109	109
Uncontrolled Peak Flow (L/s)	473.9	634.4	742.6	963.1	1172.9	1344.3
Volume (m^3)	1322.7	1784.3	2093.7	2721.2	3321.7	3806.3

Summary of Calculations

100 Year Peak Flow

Storm Duration (min)	Intensity (i) (mm/hr)	Peak Flow (Q) (L/s)	Runoff Volume (m^3)	Release Flow Volume (m^3)	Required Storage Volume (m^3)	
100	41.43	858.81	5152.86	1350.81	3802.05	
101	41.14	852.69	5167.29	1364.32	3802.98	
102	40.85	846.67	5181.60	1377.82	3803.78	
103	40.56	840.74	5195.80	1391.33	3804.47	
104	40.28	834.92	5209.88	1404.84	3805.04	
105	40.00	829.18	5223.85	1418.35	3805.50	
106	39.73	823.54	5237.71	1431.86	3805.85	
107	39.46	817.98	5251.46	1445.36	3806.09	
108	39.20	812.52	5265.10	1458.87	3806.23	
109	38.94	807.13	5278.64	1472.38	3806.26	MAX
110	38.68	801.83	5292.07	1485.89	3806.18	
111	38.43	796.61	5305.40	1499.40	3806.01	
112	38.18	791.46	5318.64	1512.90	3805.73	
113	37.94	786.40	5331.77	1526.41	3805.36	
114	37.70	781.41	5344.81	1539.92	3804.89	
115	37.46	776.49	5357.76	1553.43	3804.33	

Storage Calculations			
Required Volume (+10%)	4186.9	m3	Concrete Decast STM tank
Depth	1.4	m	
Width	28	m	
Length	110	m	
Storage Volume	4312	m3	

ARUP	Job No.	Sheet	Rev.
	287943-00	5	1
	Member/Location	Toronto	
Job Title: Baldwin Aerodome	Drg. Ref.		
Calculation: C1 - Tank Outlet Structure	Made by: MT	Date: 10-10-24	Chd.: BS

		Orifice No. 1		Emergency Overflow Weir		
	Elevation (m)	Depth Above Orifice Centroid (m)	Orifice No. 1 Flow (m^3/s)	Depth Above Overflow Weir (m)	Overflow Weir Flow (m^3/s)	Total Flow (m3/s)
	230.00	0.00	0.000	0.00	0.000	0.000
	230.05	0.00	0.000	0.00	0.000	0.000
	230.10	0.00	0.000	0.00	0.000	0.000
	230.15	0.03	0.022	0.00	0.000	0.022
	230.20	0.08	0.038	0.00	0.000	0.038
	230.25	0.13	0.048	0.00	0.000	0.048
	230.30	0.18	0.057	0.00	0.000	0.057
	230.35	0.23	0.065	0.00	0.000	0.065
	230.40	0.28	0.072	0.00	0.000	0.072
2 Year	230.45	0.33	0.078	0.00	0.000	0.078
	230.50	0.38	0.084	0.00	0.000	0.084
5 year	230.55	0.43	0.089	0.00	0.000	0.089
	230.60	0.48	0.094	0.00	0.000	0.094
10 year	230.65	0.53	0.099	0.00	0.000	0.099
	230.70	0.58	0.104	0.00	0.000	0.104
	230.75	0.63	0.108	0.00	0.000	0.108
	230.80	0.68	0.113	0.00	0.000	0.113
25 year	230.85	0.73	0.117	0.00	0.000	0.117
	230.90	0.78	0.121	0.00	0.000	0.121
	230.95	0.83	0.124	0.00	0.000	0.124
	231.00	0.88	0.128	0.00	0.000	0.128
50 year	231.05	0.93	0.132	0.00	0.000	0.132
	231.10	0.98	0.135	0.00	0.000	0.135
	231.15	1.03	0.139	0.01	0.001	0.140
	231.20	1.08	0.142	0.06	0.024	0.166
100 year	231.25	1.13	0.145	0.11	0.059	0.205
	231.30	1.18	0.148	0.16	0.105	0.253
	231.35	1.23	0.152	0.21	0.158	0.310
Top of Tank	231.40	1.28	0.155	0.26	0.218	0.373

	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
Storage Volume (m3)	1322.67634	1784.292752	2093.692	2721.233	3321.733	3806.25656
Depth (m)	0.42944037	0.579315829	0.67977	0.883517	1.078485	1.235797585
Release Rate (m3/s)						**0.225134636**

Tank Volume	
4312	m3
Tank Depth	
1.4	m

CONTROL CHAMBER - ORIFICE CONTROL

Orifice No. 1

- Orifice diameter (m)=	**0.250**
- Area (m^2) =	**0.049086**
- Orifice C =	**0.63**
- Invert (m)=	**230.00**
- Orifice Centroid (m) =	**230.13**

Submerged Orifice Equation:
Q = CxAx(2gH)^0.5

where;
Q = flow rate (m^3)
C = constant
A = area of opening(m^2)
H = net head on the orifice
g = Acceleration due to gravity

Orifice Centroid):
Q_w = 1.65([(pi*(D^2)/4)(2*$\cos^{-1}$[(((D/2)-d)/(D/2))*180/pi)]/360)-((D/2-d)(Dd-d^2)$^{0.5}$)]/d)$d^{1.5}$

where;
Q = flow rate (m^3/s)
D=orifice diameter (m)
d=depth of flow above invert (m)

OVERFLOW WEIR

Overflow Weir

- Length of Weir(m)	**0.9**	Choose 1.2m MH	
- Elevation of Weir Crest P (m)	**231.14**	- Height of Weir Crest	1.2
Discharge Coefficient, C	**1.837**		

$Q = (C)(L)(H)^{3/2}$

Sharp-Crested Weir

A sharp-crested weir (see figure 8.18) has a "sharp" upstream corner. The flow over a sharp-crested rectangular weir is defined as:

$$Q = CLH^{3/2} \qquad (36)$$

where: **Q** - discharge, cfs
C - discharge coefficient
L - effective length of the weir, ft.
H - head, difference in feet between the crest of the weir and the upstream water energy gradient.

5.0 Typical Weir and Orifice Coefficients

The following table identifies commonly used coefficients (C) for orifice and weir analysis.

Table 1 – Typical C Values for Weir and Orifice Calculations

Application	Typical C Values
Orifice	0.63
Orifice Tube	0.80
Sharp Crested Weir	1.837
Broad Crested Weir (SWM Facility and Dam Spillway)	1.7
Broad Crested Weir (Road Crossing)	1.5

ARUP	Job	Sheet No.	Rev
	287943-00	6	1
	Member/Location	Toronto	
Job Title: Baldwin Airport	Drg.		
Calculation: C1 - ESC Pond Outlet Structure	Made by: BS	Date: 08-01-24	Chd.

		Orifice No. 1		Emergency Overflow Weir		
	Elevation (m)	Depth Above Orifice Centroid (m)	Orifice No. 1 Flow (m^3/s)	Depth Above Overflow Weir (m)	Overflow Weir Flow (m^3/s)	Total Flow (m3/s)
	228.00	0.00	0.000	0.00	0.000	0.000
	228.05	0.00	0.000	0.00	0.000	0.000
	228.10	0.00	0.000	0.00	0.000	0.000
	228.15	0.00	0.000	0.00	0.000	0.000
	228.20	0.00	0.000	0.00	0.000	0.000
	228.25	0.00	0.000	0.00	0.000	0.000
	228.30	0.00	0.000	0.00	0.000	0.000
	228.35	0.00	0.000	0.00	0.000	0.000
	228.40	0.00	0.000	0.00	0.000	0.000
	228.45	0.00	0.000	0.00	0.000	0.000
	228.50	0.00	0.000	0.00	0.000	0.000
	228.55	0.00	0.000	0.00	0.000	0.000
	228.60	0.04	0.007	0.00	0.000	0.007
	228.65	0.09	0.010	0.00	0.000	0.010
	228.70	0.14	0.013	0.00	0.000	0.013
	228.75	0.19	0.015	0.00	0.000	0.015
	228.80	0.24	0.017	0.00	0.000	0.017
2 Year	228.85	0.29	0.018	0.00	0.000	0.018
	228.90	0.34	0.020	0.00	0.000	0.020
	228.95	0.39	0.021	0.00	0.000	0.021
	229.00	0.44	0.023	0.00	0.000	0.023
	229.05	0.49	0.024	0.00	0.000	0.024
	229.10	0.54	0.025	0.00	0.000	0.025
5 year	229.15	0.59	0.026	0.00	0.000	0.026
	229.20	0.64	0.027	0.00	0.000	0.027
	229.25	0.69	0.028	0.00	0.000	0.028
10 year	229.30	0.74	0.029	0.00	0.000	0.029
	229.35	0.79	0.030	0.00	0.000	0.030
	229.40	0.84	0.031	0.00	0.000	0.031
	229.45	0.89	0.032	0.00	0.000	0.032
	229.50	0.94	0.033	0.00	0.000	0.033
	229.55	0.99	0.034	0.00	0.000	0.034
	229.60	1.04	0.035	0.00	0.000	0.035
	229.65	1.09	0.036	0.00	0.000	0.036
25 year	229.70	1.14	0.037	0.00	0.000	0.037
	229.75	1.19	0.037	0.00	0.000	0.037
	229.80	1.24	0.038	0.04	0.012	0.050
	229.85	1.29	0.039	0.09	0.041	0.080
	229.90	1.34	0.040	0.14	0.080	0.120
	229.95	1.39	0.040	0.19	0.127	0.167
Top of Pond	230.00	1.44	0.041	0.24	0.180	0.221

	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
Storage Volume (m3)	1322.67634	1784.292752	2093.692	2721.233	3321.733	3806.25656
Depth (m)	0.82051882	1.10688136	1.298816	1.68811	2.060628	2.36120134
Release Rate (m3/s)						0.225134636

Pond Dimensions and Volume		
Pond Storage Volume	3224	m3
Pond Depth	2	m
bottom elev (perm)	228.01	m
top elev (perm)	229.01	m
Top elec (active)	230.01	m

CONTROL CHAMBER - ORIFICE CONTROL

Orifice No. 1

- Orifice diameter (m)=	**0.125**
- Area (m^2) =	**0.012271**
- Orifice C =	**0.63**
- Invert (m)=	**228.50**
- Orifice Centroid (m) =	**228.56**

Submerged Orifice Equation:
Q = CxAx(2gH)^0.5

where;
Q = flow rate (m^3)
C = constant
A = area of opening(m^2)
H = net head on the orifice
g = Acceleration due to gravity

Orifice Centroid):
Q_w = 1.65([(pi*(D^2)/4)(2*$\cos^{-1}$[(((D/2)-d)/(D/2))*180/pi)]/360)-((D/2-d)(Dd-$d^2)^{0.5}$)]/d)$d^{1.5}$

where;
Q = flow rate (m^3/s)
D=orifice diameter (m)
d=depth of flow above invert (m)

OVERFLOW WEIR

Overflow Weir

$Q = (C)(L)(H)^{3/2}$

- Length of Weir(m)	**0.9**	Choose 1.2m MH	
- Elevation of Weir Crest P (m)	**229.76**	- Height of Weir Crest	1.75
Discharge Coefficient, C	**1.7**		

Broad-Crested Weir

A broad-crested weir differs from a sharp-crested weir in that the weir is wide enough to support the water as it flows over the weir (see figure 8.21). The discharge over a broad - crested weir is determined using:

$Q = CLH^{3/2}$

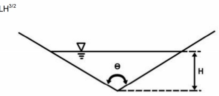


Figure 8.21 - Broad-Crested Weir

5.0 Typical Weir and Orifice Coefficients

The following table identifies commonly used coefficients (C) for orifice and weir analysis.

Table 1 – Typical C Values for Weir and Orifice Calculations

Application	Typical C Values
Orifice	0.63
Orifice Tube	0.80
Sharp Crested Weir	1.837
Broad Crested Weir (SWM Facility and Dam Spillway)	1.7
Broad Crested Weir (Road Crossing)	1.5

ARUP	Job No. 287943-00	Sheet 7	Rev. 1
	Member/Location	Toronto	
Job Title: Baldwin Aerodome	Drg. Ref.		
Calculation: C1a - SWM Allowable Release Rate and Required Storage	Made by MT	Date 10-10-24	Chd. BS

Runoff Coefficients

Impervious Area	0.9
Pervious Area	0.2
Semi Impervious Area	0.5
Wetlands Area	0.05
Woodlots Area	0.42

Pre-Development Conditions

Catchment Area	3.09	ha
Impervious Area	0.00	ha
Pervious Area	3.09	ha
Semi Impervious (Gravel Road)	0.00	ha
Wetlands Area	0.00	ha
Woodlots Area	0.00	ha
Composite C Coeff.	0.20	

Post-Development Conditions

Catchment Area	3.09	ha
Impervious Area	1.05	ha
Pervious Area	2.03	ha
Semi Impervious (Gravel Road)	0.00	ha
Wetlands Area	0.00	ha
Woodlots Area	0.00	ha
Composite C Coeff.	0.44	

Development Conditions Comparison

Catchment Area	0.000	ha	0%
Impervious Area	1.0530	ha	34%
Pervious Area	-1.053	ha	-34%

ALLOWABLE PEAK FLOW RATES

Pre-Development Catchment Area	3.09	ha
Pre-Development Time of Concentration (t)	126	min

Pre-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Bottom Width, **Wb** (m)	675.586	843.019	976.898	1133.123	1251.473	1383.628
B	4.681	4.582	4.745	4.734	4.847	4.905
C	0.78	0.763	0.76	0.756	0.753	0.754
i (mm/hr)	15.1	20.5	24.1	28.5	31.9	35.1

**Values taken from Town of Georgina Development Design Criteria and Standards*
https://www.georgina.ca/sites/default/files/page_assets

Release Rates (Q, in L/s)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Ca Value	1.00	1.00	1.00	1.10	1.20	1.25
Rational Method Release Rates (Q = 2.78 CaCiA)	25.9	35.1	41.3	53.7	65.6	75.2
Regulated Allowable Release Rates	NA	NA	NA	NA	NA	NA
Design Allowable Release Rates	25.9	35.1	41.3	53.7	65.6	75.2
Flow Area, **A** (m^2)						
Wetted perimeter, **Wp** (m)						
Hydraulic Radius, **Rh** (m)						
Full flow capacity, **Qcapacity**, m3/s						

Post-Development Catchment Area	3.09	ha
Post-Development Runoff Coefficient	0.44	
Post-Development Time of Concentration (t)	43	min

Post-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
i (mm/hr)	33.2	44.3	51.7	61.0	68.0	74.8

Required Storage Volume

Release Flow Volume Calculation Method	Allowable Release Rate x Storm Duration	
Initial Storm Duration	10	min
Time Step	1	min

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Critical Duration (min)	63	66	67	68	69	69
Uncontrolled Peak Flow (L/s)	124.8	166.6	194.8	252.5	307.2	352.1
Volume (m^3)	261.1	349.2	408.7	530.0	645.6	739.9

Summary of Calculations

100 Year Peak Flow

Storm Duration (min)	Intensity (i) (mm/hr)	Peak Flow (Q) (L/s)	Runoff Volume (m^3)	Release Flow Volume (m^3)	Required Storage Volume (m^3)	
60	59.51	280.05	1008.17	270.72	737.45	
61	58.82	276.84	1013.22	275.23	737.99	
62	58.16	273.71	1018.21	279.74	738.47	
63	57.51	270.67	1023.12	284.25	738.87	
64	56.88	267.70	1027.97	288.77	739.20	
65	56.27	264.81	1032.75	293.28	739.47	
66	55.67	261.99	1037.47	297.79	739.68	
67	55.08	259.23	1042.12	302.30	739.82	
68	54.51	256.55	1046.72	306.81	739.91	
69	53.96	253.93	1051.26	311.33	739.93	**MAX**
70	53.41	251.37	1055.74	315.84	739.90	
71	52.88	248.87	1060.17	320.35	739.82	
72	52.36	246.42	1064.54	324.86	739.68	
73	51.85	244.03	1068.87	329.37	739.49	
74	51.36	241.70	1073.14	333.89	739.25	
75	50.87	239.41	1077.36	338.40	738.96	

ARUP	Job No. 287943-00	Sheet 8	Rev. 1
	Member/Location	Toronto	
Job Title: Baldwin Aerodome	Drg. Ref.		
Calculation: C1b - SWM Allowable Release Rate and Required Storage	Made by MT	Date 10-10-24	Chd. BS

Runoff Coefficients

Impervious Area	0.9
Pervious Area	0.2
Semi Impervious Area	0.5
Wetlands Area	0.05
Woodlots Area	0.42

Pre-Development Conditions

Catchment Area	7.31 ha
Impervious Area	0.15 ha
Pervious Area	4.94 ha
Semi Impervious (Gravel Road)	0.41 ha
Wetlands Area	1.81 ha
Woodlots Area	0.00 ha
Composite C Coeff.	0.19

Post-Development Conditions

Catchment Area	7.31 ha
Impervious Area	4.50 ha
Pervious Area	2.81 ha
Semi Impervious (Gravel Road)	0.00 ha
Wetlands Area	0.00 ha
Woodlots Area	0.00 ha
Composite C Coeff.	0.63

Development Conditions Comparison

Catchment Area	-0.002 ha	0%
Impervious Area	4.3460 ha	59%
Pervious Area	-2.126 ha	-29%

ALLOWABLE PEAK FLOW RATES

Pre-Development Catchment Area	7.31 ha
Pre-Development Time of Concentration (t)	137 min

Pre-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Bottom Width, **Wb** (m)	675.586	843.019	976.898	1133.123	1251.473	1383.628
B	4.681	4.582	4.745	4.734	4.847	4.905
C	0.78	0.763	0.76	0.756	0.753	0.754
i (mm/hr)	14.2	19.3	22.6	26.8	30.0	33.0

**Values taken from Town of Georgina Development Design Criteria and Standards*
https://www.georgina.ca/sites/default/files/page_assets

Release Rates (Q, in L/s)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Ca Value	1.00	1.00	1.00	1.10	1.20	1.25
Rational Method Release Rates (Q = 2.78 CaCiA)	55.9	76.0	89.3	116.2	142.0	162.7
Regulated Allowable Release Rates	NA	NA	NA	NA	NA	NA
Design Allowable Release Rates	55.9	76.0	89.3	116.2	142.0	162.7
Flow Area, **A** (m^2)						
Wetted perimeter, **Wp** (m)						
Hydraulic Radius, **Rh** (m)						
Full flow capacity, **Qcapacity**, m3/s						

Post-Development Catchment Area	7.31 ha
Post-Development Runoff Coefficient	0.63
Post-Development Time of Concentration (t)	45 min

Post-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
i (mm/hr)	32.1	42.9	50.2	59.1	65.9	72.5

Required Storage Volume

Release Flow Volume Calculation Method	Allowable Release Rate x Storm Duration
Initial Storm Duration	10 min
Time Step	1 min

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Critical Duration (min)	106	113	115	117	118	118
Uncontrolled Peak Flow (L/s)	411.6	549.7	642.9	833.3	1014.1	1162.3
Volume (m^3)	1045.6	1413.3	1659.2	2157.6	2634.9	3019.0

Summary of Calculations

100 Year Peak Flow

Storm Duration (min)	Intensity (i) (mm/hr)	Peak Flow (Q) (L/s)	Runoff Volume (m^3)	Release Flow Volume (m^3)	Required Storage Volume (m^3)	
110	38.68	619.78	4090.53	1073.80	3016.73	
111	38.43	615.74	4100.84	1083.57	3017.27	
112	38.18	611.77	4111.07	1093.33	3017.74	
113	37.94	607.85	4121.22	1103.09	3018.13	
114	37.70	603.99	4131.30	1112.85	3018.45	
115	37.46	600.19	4141.31	1122.61	3018.69	
116	37.23	596.44	4151.24	1132.37	3018.87	
117	37.00	592.75	4161.10	1142.14	3018.97	
118	36.77	589.11	4170.90	1151.90	3019.00	**MAX**
119	36.55	585.52	4180.62	1161.66	3018.96	
120	36.32	581.98	4190.28	1171.42	3018.86	
121	36.11	578.49	4199.87	1181.18	3018.69	
122	35.89	575.05	4209.40	1190.95	3018.45	
123	35.68	571.66	4218.86	1200.71	3018.15	
124	35.47	568.31	4228.25	1210.47	3017.79	
125	35.27	565.01	4237.59	1220.23	3017.36	

ARUP	Job No. 287943-00	Sheet 9	Rev. 1
	Member/Location	Toronto	
Job Title: Baldwin Aerodome	Drg. Ref.		
Calculation: C2 - SWM Allowable Release Rate and Required Storage	Made by MT	Date 10-10-24	Chd. BS

Runoff Coefficients

Impervious Area	0.9
Pervious Area	0.2
Semi Impervious Area	0.5
Wetlands Area	0.05
Woodlots Area	0.42

Pre-Development Conditions		*Post-Development Conditions*		Development Conditions Comparison		
Catchment Area	9.39 ha	Catchment Area	9.39 ha	Catchment Area	0.00 ha	0%
Impervious Area	0.11 ha	Impervious Area	0.88 ha	Impervious Area	0.77 ha	8%
Pervious Area	4.12 ha	Pervious Area	8.51 ha	Pervious Area	4.39 ha	47%
Semi Impervious (Gravel Road)	0.01 ha	Semi Impervious (Gravel Road)	0.00 ha			
Wetlands Area	5.15 ha	Wetlands Area	0.00 ha			
Woodlots Area	0.00 ha	Woodlots Area	0.00 ha			
Composite C Coeff.	0.13	Composite C Coeff.	0.27			

ALLOWABLE PEAK FLOW RATES

Pre-Development Catchment Area 9.39 ha
Pre-Development Time of Concentration (t) 215 min

Pre-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	
A	675.586	843.019	976.898	1133.123	1251.473	1383.628	*Values taken from Town of Georgina Development
Bottom Width, **Wb** (m)	4.681	4.582	4.745	4.734	4.847	4.905	Design Criteria and Standards
C	0.78	0.763	0.76	0.756	0.753	0.754	https://www.georgina.ca/sites/default/files/page_assets
i (mm/hr)	10.1	13.8	16.2	19.2	21.6	23.7	

Release Rates (Q, in L/s)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Ca Value	1.00	1.00	1.00	1.10	1.20	1.25
Rational Method Release Rates (Q = 2.78 CaCiA)	33.2	45.4	53.4	69.6	85.2	97.6
Regulated Allowable Release Rates	NA	NA	NA	NA	NA	NA
Design Allowable Release Rates	33.2	45.4	53.4	69.6	85.2	97.6

Flow Area, **A** (m^2)
Wetted perimeter, **Wp** (m)
Hydraulic Radius, **Rh** (m)
Full flow capacity, **Qcapacity**, m3/s 9.39 ha
Post-Development Runoff Coefficient 0.27
Post-Development Time of Concentration (t) 153 min

Post-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
i (mm/hr)	13.0	17.7	20.9	24.7	27.7	30.4

Required Storage Volume

Release Flow Volume Calculation Method: Allowable Release Rate x Storm Duration
Initial Storm Duration 10 min
Time Step 1 min

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Critical Duration (min)	95	100	102	103	104	104
Uncontrolled Peak Flow (L/s)	90.5	123.1	144.7	188.4	230.4	263.9
Volume (m^3)	548.5	737.7	865.0	1123.4	1370.2	1570.3

Summary of Calculations

100 Year Peak Flow

Storm Duration (min)	Intensity (i) (mm/hr)	Peak Flow (Q) (L/s)	Runoff Volume (m^3)	Release Flow Volume (m^3)	Required Storage Volume (m^3)	
100	41.43	359.24	2155.43	585.52	1569.91	
101	41.14	356.68	2161.46	591.38	1570.09	
102	40.85	354.16	2167.45	597.23	1570.22	
103	40.56	351.68	2173.39	603.09	1570.30	
104	40.28	349.24	2179.28	608.94	1570.34	**MAX**
105	40.00	346.84	2185.12	614.80	1570.32	
106	39.73	344.48	2190.92	620.65	1570.27	
107	39.46	342.16	2196.67	626.51	1570.16	
108	39.20	339.87	2202.38	632.36	1570.01	
109	38.94	337.62	2208.04	638.22	1569.82	

ARUP		Job	Sheet No.	Rev
		287943-00	10	1
		Member/Location	Toronto	
Job Title	Baldwin Airport	Drg.		
Calculation	C2 - Outlet Structure	Made by BS	Date 08-01-24	Chd.

1. Orifice and Weir

		Orifice No. 1		Emergency Overflow Weir		
	Elevation (m)	Depth Above Orifice Centroid (m)	Orifice No. 1 Flow (m^3/s)	Depth Above Overflow Weir (m)	Overflow Weir Flow (m^3/s)	Total Flow (m3/s)
	228.50	0.00	0.000	0.00	0.000	0.000
	228.55	0.00	0.000	0.00	0.000	0.000
	228.60	0.04	0.007	0.00	0.000	0.007
	228.65	0.09	0.010	0.00	0.000	0.010
	228.70	0.14	0.013	0.00	0.000	0.013
	228.75	0.19	0.015	0.00	0.000	0.015
	228.80	0.24	0.017	0.00	0.000	0.017
	228.85	0.29	0.018	0.00	0.000	0.018
2 Year	228.90	0.34	0.020	0.00	0.000	0.020
	228.95	0.39	0.021	0.00	0.000	0.021
5 year	229.00	0.44	0.023	0.00	0.000	0.023
	229.05	0.49	0.024	0.00	0.000	0.024
10 year	229.10	0.54	0.025	0.00	0.000	0.025
	229.15	0.59	0.026	0.00	0.000	0.026
	229.20	0.64	0.027	0.00	0.000	0.027
	229.25	0.69	0.028	0.00	0.000	0.028
25 year	229.30	0.74	0.029	0.00	0.000	0.029
	229.35	0.79	0.030	0.00	0.000	0.030
	229.40	0.84	0.031	0.00	0.000	0.031
50 year	229.45	0.89	0.032	0.00	0.000	0.032
	229.50	0.94	0.033	0.01	0.002	0.035
	229.55	0.99	0.034	0.06	0.022	0.057
100 year	229.60	1.04	0.035	0.11	0.056	0.091
	229.65	1.09	0.036	0.16	0.098	0.134
	229.70	1.14	0.037	0.21	0.147	0.184
	229.75	1.19	0.037	0.26	0.203	0.240
	229.80	1.24	0.038	0.31	0.264	0.302
	229.85	1.29	0.039	0.36	0.330	0.369
	229.90	1.34	0.040	0.41	0.402	0.441
	229.95	1.39	0.040	0.46	0.477	0.518
	230.00	1.44	0.041	0.51	0.557	0.598
	230.05	1.49	0.042	0.56	0.641	0.683
	230.10	1.54	0.042	0.61	0.729	0.771
	230.15	1.59	0.043	0.66	0.820	0.864
	230.20	1.64	0.044	0.71	0.915	0.959
	230.25	1.69	0.044	0.76	1.014	1.058
	230.30	1.74	0.045	0.81	1.115	1.161
	230.35	1.79	0.046	0.86	1.220	1.266
	230.40	1.84	0.046	0.91	1.328	1.375
	230.45	1.89	0.047	0.96	1.439	1.486
Top of Pond	230.50	1.94	0.048	1.01	1.553	1.601

CONTROL CHAMBER - ORIFICE CONTROL

Orifice No. 1

- Orifice diameter (m)=	**0.125**
- Area (m^2) =	**0.012271**
- Orifice C =	**0.63**
- Invert (m)=	**228.50**
- Orifice Centroid (m) =	**228.56**

Submerged Orifice Equation:
Q = CxAx(2gH)^0.5

where;
Q = flow rate (m^3)
C = constant
A = area of opening(m^2)
H = net head on the orifice
g = Acceleration due to gravity

Submerged Orifice Equation (Flow Below

where;
Q = flow rate (m^3/s)
D=orifice diameter (m)
d=depth of flow above invert (m)

OVERFLOW WEIR

Overflow Weir

- Length of Weir(m)	**0.9**	Choose 1.2m MH		$Q = (C)(L)(H)^{3/2}$
- Elevation of Weir Crest P (m)	**229.49**	- Height of Weir Crest	1	
Discharge Coefficient, C	**1.7**			

Broad-Crested Weir

A broad-crested weir differs from a sharp-crested weir in that the weir is wide enough to support the water as it flows over the weir (see figure 8.21). The discharge over a broad - crested weir is determined using:

$Q = CLH^{3/2}$

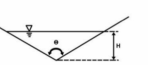


Figure 8.21 - Broad-Crested Weir

5.0 Typical Weir and Orifice Coefficients

The following table identifies commonly used coefficients (C) for orifice and weir analysis.

Table 1 – Typical C Values for Weir and Orifice Calculations

Application	Typical C Values
Orifice	0.63
Orifice Tube	0.80
Sharp Crested Weir	1.837
Broad Crested Weir (SWM Facility and Dam Spillway)	1.7
Broad Crested Weir (Road Crossing)	1.5

	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
Storage Volume (m3)	548.513641	737.71344	864.986331	1123.355	1370.229	1570.337053
Depth (m)	0.37686876	0.506862785	0.594308517	0.771827	0.941447	1.078935761
Release Rate (m3/s)						**0.097586695**

Pond Dimensions and Volume		
Pond Storage Volume	2910.9	m3
Pond Depth	2	m
	North Side	South Side
bottom elev (perm)	228.64	228.5
top elev (perm)	229.64	229.5
Top elec (active)	230.64	230.5

2. Swale geometry

Total Swale Depth, **D** (m)	0.280	
Bottom Width, **Wb** (m)	6.00	
Side Slope, **z** (m/m)	1.00	
Top Width, **Wt** (m)	6.56	

3. Full flow capacity of swale (Manning's Equation)

Composite Manning's roughness, **n**	0.03	Horton Method
Manning's roughness for grass base, **nb_g**	0.030	Grass with some weeds per MTO Design Chart 2.01
Manning's roughness for grass sides, **ns_g**	0.030	Grass with some weeds per MTO Design Chart 2.01
Base perimeter, **P_base**	6.00	Base width
Side perimeter, **P_side**	0.40	assumed trapezoidal shape
Longitudinal Slope, **So** (m/m)	0.45%	
Flow Area, **A** (m^2)	1.76	
Wetted perimeter, **Wp** (m)	6.79	
Hydraulic Radius, **Rh** (m)	0.26	
Full flow capacity, **Qcapacity**, m3/s	1.60	Allowable discharge during 100-yr storm event is 0.0976m3/s

ARUP	Job No. 287943-00	Sheet 11	Rev. 1
	Member/Location	Toronto	
Job Title: Baldwin Aerodome	Drg. Ref.		
Calculation: C3 - SWM Allowable Release Rate and Required Storage	Made by MT	Date 10-10-24	Chd. BS

Runoff Coefficients

Impervious Area	0.9
Pervious Area	0.2
Semi Impervious Area	0.5
Wetlands Area	0.05
Woodlots Area	0.42

Pre-Development Conditions		*Post-Development Conditions*		Development Conditions Comparison		
Catchment Area	1.44 ha	Catchment Area	1.44 ha	Catchment Area	0.00 ha	0%
Impervious Area	0.04 ha	Impervious Area	0.29 ha	Impervious Area	0.24 ha	17%
Pervious Area	1.25 ha	Pervious Area	1.15 ha	Pervious Area	-0.09 ha	-6%
Semi Impervious (Gravel Road)	0.15 ha	Semi Impervious (Gravel Road)	0.00 ha			
Wetlands Area	0.00 ha	Wetlands Area	0.00 ha			
Woodlots Area	0.00 ha	Woodlots Area	0.00 ha			
Composite C Coeff.	0.25	Composite C Coeff.	0.34			

ALLOWABLE PEAK FLOW RATES

Pre-Development Catchment Area	1.44 ha
Pre-Development Time of Concentration (t)	45 min

Pre-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	
A	675.586	843.019	976.898	1133.123	1251.473	1383.628	*Values taken from Town of Georgina Development
Bottom Width, **Wb** (m)	4.681	4.582	4.745	4.734	4.847	4.905	Design Criteria and Standards
C	0.78	0.763	0.76	0.756	0.753	0.754	https://www.georgina.ca/sites/default/files/page_assets
i (mm/hr)	32.1	42.9	50.2	59.1	65.9	72.5	

Release Rates (Q, in L/s)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Ca Value	1.00	1.00	1.00	1.10	1.20	1.25
Rational Method Release Rates (Q = 2.78 CaCiA)	32.5	43.4	50.8	65.8	80.1	91.8
Regulated Allowable Release Rates	NA	NA	NA	NA	NA	NA
Design Allowable Release Rates	32.5	43.4	50.8	65.8	80.1	91.8

Flow Area, **A** (m^2)	
Wetted perimeter, **Wp** (m)	
Hydraulic Radius, **Rh** (m)	
Full flow capacity, **Qcapacity**, m3/s	1.44 ha
Post-Development Runoff Coefficient	0.34
Post-Development Time of Concentration (t)	31 min

Post-Development Rainfall Intensity (i = A / (t+B)^C)

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
i (mm/hr)	41.6	55.2	64.5	75.9	84.5	93.0

Required Storage Volume

Release Flow Volume Calculation Method	Allowable Release Rate x Storm Duration
Initial Storm Duration	10 min
Time Step	1 min

Storm Event	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR
Critical Duration (min)	17	17	17	18	18	18
Uncontrolled Peak Flow (L/s)	56.6	75.2	87.8	113.6	138.0	158.2
Volume (m^3)	52.0	68.0	78.8	101.6	122.8	140.7

Summary of Calculations

100 Year Peak Flow

Storm Duration (min)	Intensity (i) (mm/hr)	Peak Flow (Q) (L/s)	Runoff Volume (m^3)	Release Flow Volume (m^3)	Required Storage Volume (m^3)	
10	180.437595	306.9893059	184.1935836	55.07385276	129.1197308	
11	171.815742	292.320431	192.9314845	60.58123804	132.3502464	
12	164.095209	279.1850239	201.0132172	66.08862331	134.9245939	
13	157.136364	267.3455249	208.5295094	71.59600859	136.9335008	
14	150.827497	256.6118704	215.5539711	77.10339386	138.4505773	
15	145.078099	246.8300749	222.1470674	82.61077914	139.5362883	
16	139.814017	237.8739753	228.3590163	88.11816442	140.2408519	
17	134.973878	229.639157	234.2319402	93.62554969	140.6063905	
18	130.506427	222.0384142	239.8014873	99.13293497	140.6685523	**MAX**
19	126.368499	214.998309	245.0980723	104.6403202	140.457752	

ARUP	Job	Sheet No.	Rev
	287943-00	12	1
	Member/Location	Toronto	
Job Title: Baldwin Airport	Drg.		
Calculation: C3 - Outlet Structure	Made by: BS	Date: 08-01-24	Chd.

		Orifice No. 1		Emergency Overflow Weir		
	Elevation (m)	Depth Above Orifice Centroid (m)	Orifice No. 1 Flow (m^3/s)	Depth Above Overflow Weir (m)	Overflow Weir Flow (m^3/s)	Total Flow (m3/s)
	227.50	0.00	0.000	0.00	0.000	0.000
	227.55	0.00	0.000	0.00	0.000	0.000
	227.60	0.00	0.000	0.00	0.000	0.000
	227.65	0.00	0.000	0.00	0.000	0.000
	227.70	0.00	0.000	0.00	0.000	0.000
	227.75	0.00	0.000	0.00	0.000	0.000
	227.80	0.00	0.000	0.00	0.000	0.000
	227.85	0.00	0.000	0.00	0.000	0.000
	227.90	0.00	0.000	0.00	0.000	0.000
	227.95	0.00	0.000	0.00	0.000	0.000
	228.00	0.00	0.000	0.00	0.000	0.000
	228.05	0.00	0.000	0.00	0.000	0.000
	228.10	0.00	0.000	0.00	0.000	0.000
	228.15	0.00	0.000	0.00	0.000	0.000
	228.20	0.00	0.000	0.00	0.000	0.000
	228.25	0.00	0.000	0.00	0.000	0.000
	228.30	0.00	0.000	0.00	0.000	0.000
	228.35	0.00	0.000	0.00	0.000	0.000
	228.40	0.00	0.000	0.00	0.000	0.000
	228.45	0.00	0.000	0.00	0.000	0.000
	228.50	0.01	0.012	0.00	0.000	0.012
	228.55	0.06	0.033	0.00	0.000	0.033
	228.60	0.11	0.045	0.00	0.000	0.045
	228.65	0.16	0.054	0.02	0.004	0.058
100 Year	228.70	0.21	0.062	0.07	0.025	0.087
	228.75	0.26	0.070	0.12	0.056	0.126
	228.80	0.31	0.076	0.17	0.095	0.171
	228.85	0.36	0.082	0.22	0.139	0.221
Top of Road	228.90	0.41	0.087	0.27	0.189	0.277

	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
Storage Volume (m3)	51.9661161	68.03203803	78.81701	101.6456	122.7828	140.6685523
Depth (m)	0.11727853	0.153536534	0.177876	0.229397	0.277099	0.317464573
Release Rate (m3/s)						**0.091789755**

Swale Dimensions		
Depth	0.6	m
Bottom Width	1.5	m
Side slope (H:V)	1	
Longitudinal slope	0.1	
Cross Area (m2)	1.26	m2
Length	211	m
Storage Volume	265.86	m3

CONTROL CHAMBER - ORIFICE CONTROL

Orifice No. 1

- Orifice diameter (m)=	**0.250**
- Area (m^2) =	**0.049086**
- Orifice C =	**0.63**
- Invert (m)=	**228.37**
- Orifice Centroid (m) =	**228.49**

Submerged Orifice Equation:
Q = CxAx(2gH)^0.5

where;
Q = flow rate (m^3)
C = constant
A = area of opening(m^2)
H = net head on the orifice
g = Acceleration due to gravity

Un-Submerged Orifice Equation (Flow Below Orifice Centroid):
Q_w = 1.65([(pi*(D^2)/4)(2*cos^{-1}[(((D/2)-

where;
Q = flow rate (m^3/s)
D=orifice diameter (m)
d=depth of flow above invert (m)

OVERFLOW WEIR

Overflow Weir

- Length of Weir(m)	**0.9**	Choose 1.2m MH	
- Elevation of Weir Crest P (m)	**228.63**	- Height of Weir Crest P (m)	0.3
Discharge Coefficient, C	**1.5**		

Broad-Crested Weir

A broad-crested weir differs from a sharp-crested weir in that the weir is wide enough to support the water as it flows over the weir (see figure 8.21). The discharge over a broad - crested weir is determined using:

$Q = CLH^{3/2}$

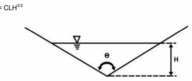


Figure 8.21 - Broad-Crested Weir

5.0 Typical Weir and Orifice Coefficients

The following table identifies commonly used coefficients (C) for orifice and weir analysis.

Table 1 – Typical C Values for Weir and Orifice Calculations

Application	Typical C Values
Orifice	0.63
Orifice Tube	0.80
Sharp Crested Weir	1.837
Broad Crested Weir (SWM Facility and Dam Spillway)	1.7
Broad Crested Weir (Road Crossing)	1.5

ARUP

JOB TITLE	Baldwin Aerodome
JOB NUMBER	287943-00
MADE BY	ME
CHECKED BY	JB
DATE	10/10/24
Description of spreadsheet	Swale hydraulics and stormwater basin sizing
Member/Location	Calgary
Report Reference	
Filename	J:\TOR\280000\287943-00\3 Design\3-06 Calcs\SWM Calcs\[Swale Hydraulics and Stormwater Basin Sizing.xlsx]

CONTENTS OF SPREADSHEET

Sheet	Description
1	Swale hydraulics FlowMaster results
2	Swale capacity comparisons with 100-year release rates
3	Erosion control blanket options
4	C2 - Stormwater basin sizing
5	C1 - Temporary ESC pond sizing

AUTHORISATION OF LATEST VERSION

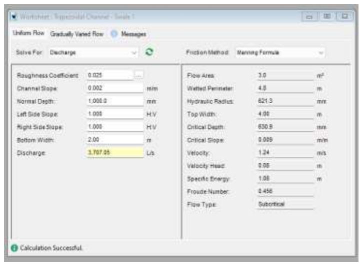
Type and method of check		
Signatures & dates:	Made by	ME
	Checked	JB

REVISIONS Current Revision 1

Rev.	Date	Made by	Checked	Description
1	10/10/24	ME	JB	

Swale 1

Slope (%)	0.18
Slope (m/m)	0.0018
Bottom Width (m)	2
Minimum Depth (m)	1
Normal Depth (mm)	1000
Roughness Coefficient	0.025
Left Side Slope (H:V)	1
Right Side Slope (H:V)	1
Results:	
Discharge (L/s)	3,707.05
Velocity (m/s)	1.24
Check dam needed?	Maybe



23.2.2 Swales and Open Channels

Swales and open channels can play an important role in both the major overland flow systems and the minor systems. They are to be designed to be aesthetically pleasing, safe, resistant to erosion and easy to maintain. Design velocities are to be calculated using Manning's equation and need to consider critical depth.

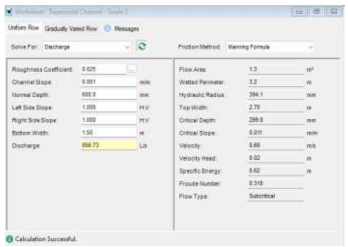
The following table provides acceptable values for Manning's "n":

Grass Channel (>0.5 m deep)	0.025
Grass Swale (<0.5 m deep)	0.030
Rip-Rap Channel (>1 m deep)	0.035
Rip-Rap Channel (<1 m deep)	0.040

Generally, grassed surfaces are adequate for velocities up to 1.5 m/s and more robust erosion protection is required for velocities beyond this range.

Swale 2

Slope (%)	0.1
Slope (m/m)	0.001
Bottom Width (m)	1.5
Minimum Depth (m)	0.6
Normal Depth (mm)	600
Roughness Coefficient	0.025
Left Side Slope (H:V)	1
Right Side Slope (H:V)	1
Results:	
Discharge (L/s)	856.73
Velocity (m/s)	0.68
Check dam needed?	No

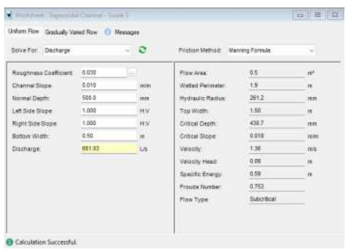


SWALE DIMENSIONS

SWALE NO.	LONG. SLOPE (%)	BOTTOM WIDTH (m)	MINIMUM DEPTH (m)
1	0.18	2.00	1.00
2	0.10	1.50	0.60
3	1.00	0.50	0.50
4	0.16	2.75	0.50
5	0.15 - ASSUMED CONSISTENT SLOPE	5.50	0.50
6	0.15 - ASSUMED CONSISTENT SLOPE	3.50	0.50

Swale 3

Slope (%)	1
Slope (m/m)	0.01
Bottom Width (m)	0.5
Minimum Depth (m)	0.5
Normal Depth (mm)	500
Roughness Coefficient	0.03
Left Side Slope (H:V)	1
Right Side Slope (H:V)	1
Results:	
Discharge (L/s)	681.03
Velocity (m/s)	1.36
Check dam needed?	Maybe



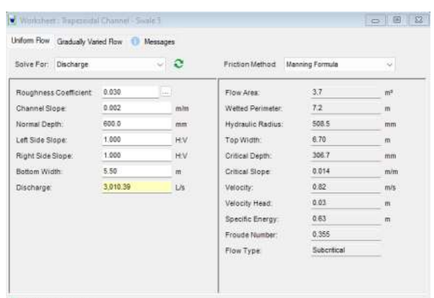
Swale 4

Slope (%)	0.16
Slope (m/m)	0.0016
Bottom Width (m)	2.75
Minimum Depth (m)	0.5
Normal Depth (mm)	500
Roughness Coefficient	0.03
Left Side Slope (H:V)	1
Right Side Slope (H:V)	1
Results:	
Discharge (L/s)	1,157.02
Velocity (m/s)	0.71
Check dam needed?	No

Swale 5

Slope (%)	0.15
Slope (m/m)	0.0015
Bottom Width (m)	5.5
Minimum Depth (m)	0.6
Normal Depth (mm)	600
Roughness Coefficient	0.025
Left Side Slope (H:V)	1
Right Side Slope (H:V)	1
Results:	
Discharge (L/s)	3,010.39
Velocity (m/s)	0.82
Check dam needed?	No

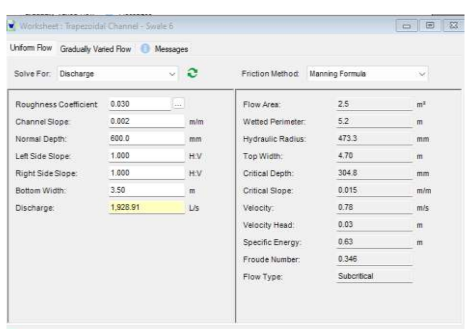
*updated to 0.6m to increase capacity



Swale 6

Slope (%)	0.15
Slope (m/m)	0.0015
Bottom Width (m)	3.5
Minimum Depth (m)	0.6
Normal Depth (mm)	600
Roughness Coefficient	0.025
Left Side Slope (H:V)	1
Right Side Slope (H:V)	1
Results:	
Discharge (L/s)	1,928.91
Velocity (m/s)	0.78
Check dam needed?	No

*updated to 0.6m to increase capacity



Swale #	1	2	3	4	5	6
Catchment #	1	1	3	2	external	external
Capacity (L/s)	3,707.05	856.73	681.03	1,157.02	3,010.39	1,928.91
100-Year Storm Release Rate (L/s)	1162.3	352.1	158.2	263.9	2860	1640
Ok?	Yes	Yes	Yes	Yes	Yes	Yes

Post-Development

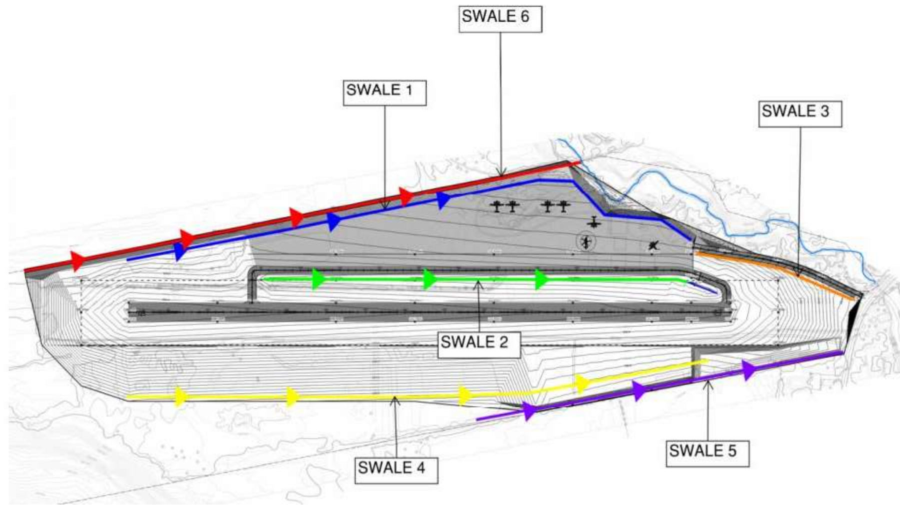
C1b Release Rate 100-Yr Storm (L/s)	1162.3
C1a Release Rate 100-Yr Storm (L/s)	352.1
C2 Release Rate 100-Yr Storm (L/s)	263.9
C3 Release Rate 100-Yr Storm (L/s)	158.2

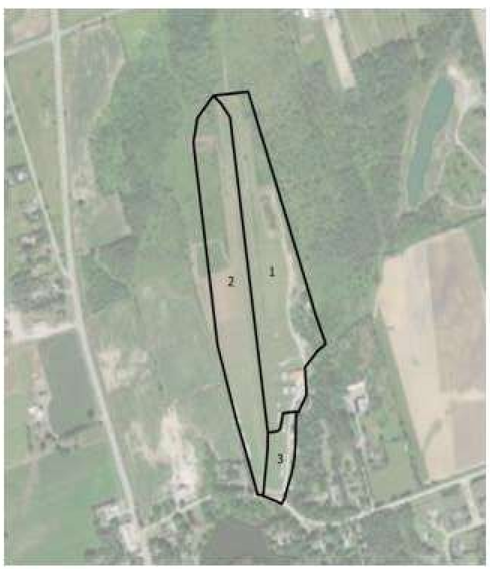
(from release rates and storage excel)

External Catchments

C5 100-Yr Storm (L/s)	2860
C6 100-Yr Storm (L/s)	1640

(from swale calculations excel)





Material	Max Flow Velocity (m/s)	Swale 1?	Swale 2?	Swale 3?	Swale 4?	Swale 5?	Swale 6?	Functional Longevity	Notes
FM200	3.8	Yes	Yes	Yes	Yes	Yes	Yes	not specified	
S100 Straw Single Net	1.4	Yes	Yes	Yes	Yes	Yes	Yes	12 mos	close to 1.36 m/s for swale 3
S200 Straw Double Net	2.1	Yes	Yes	Yes	Yes	Yes	Yes	12 mos	
SC200 Straw Coconut Double Net	2.6	Yes	Yes	Yes	Yes	Yes	Yes	24 mos	
C200 Coconut Double Net	2.7	Yes	Yes	Yes	Yes	Yes	Yes	36 mos	Selected on the basis of suitable max flow velocity and functional longevity
Wood Netfree	0.9	No	Yes	No	Yes	Yes	Yes	18 mos	Not suitable
W100 G Wood Single Net Green Fibre	2.1	Yes	Yes	Yes	Yes	Yes	Yes	18 mos	
W200 G Wood Double Net Green Fibre	Not specified								
W300 G Wood Double Net	Not specified								

Model Specifications Source: Erosion Control Blankets - Terrafix Geosynthetics Inc.

LSRCA Temporary ESC Pond Reqs

Permanent Pool Storage (m3/ha)	185
Active Storage (m3/ha)	125
Permanent Pool Depth Max (mm)	1000
Active Storage Depth (mm)	1000
Side Slopes (H:V)	3 to 1
L:W	4 to 1

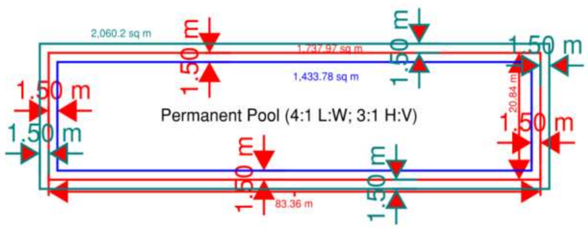
	C1	C2	C3
Permanent Pool Volume (m3)	1924	1737.15	266.4
Active Storage Volume (m3)	1300	1173.75	180

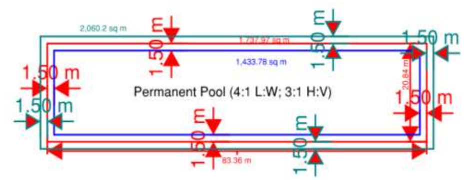
Catchment Areas

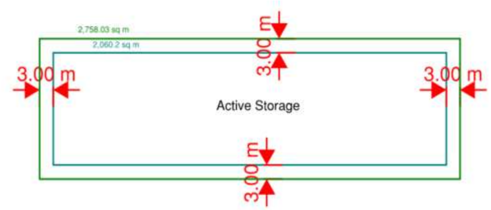
C1 (ha)	10.4
C2 (ha)	9.39
C3 (ha)	1.44

Permanent Pool	Area (m2)	Elevation (m)
Average Area Perm Pool (m2)	1737.15	229.14
Bottom Area (m2)	1433.78	228.64
Top Area (m2)	2060.2	229.64

*elevations determined by taking the base elevation provided on the drawings, then adding the corresponding height of each section of the pond, per LSRCA guidelines

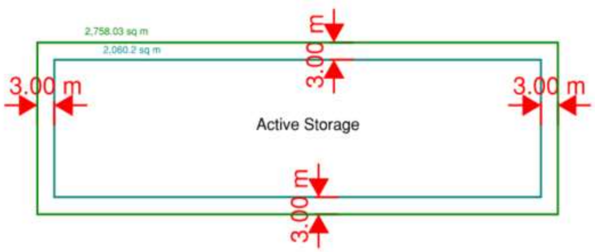






Active Storage

	Area (m2)	Elevtion (m)
Bottom Area (m2)	2060.2	229.64
Top Area (m2)	2758.03	230.64



LSRCA Temporary ESC Pond Reqs

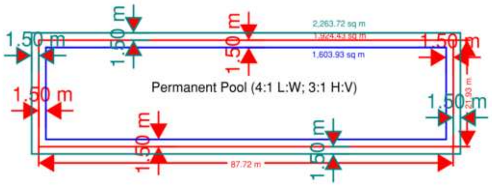
Permanent Pool Storage (m3/ha)	185
Active Storage (m3/ha)	125
Permanent Pool Depth Max (mm)	1000
Active Storage Depth (mm)	1000
Side Slopes (H:V)	3 to 1
L:W	4 to 1

	C1	C2	C3
Permanent Pool Volume (m3)	1924	1737.15	266.4
Active Storage Volume (m3)	1300	1173.75	180

Catchment Areas

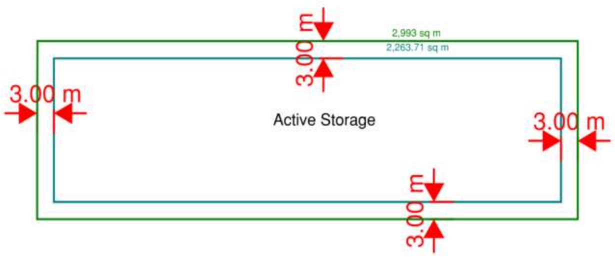
C1 (ha)	10.4
C2 (ha)	9.39
C3 (ha)	1.44

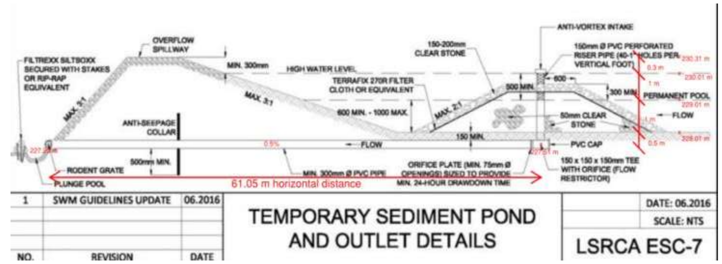
Permanent Pool	Area (m2)	Elevation (m)
Average Area Perm Pool (m2)	1924	228.51
Bottom Area (m2)	1603.93	228.01
Top Area (m2)	2263.72	229.01



Active Storage

	Area (m2)	Elevtion (m)
Bottom Area (m2)	2263.72	229.01
Top Area (m2)	2993	230.01





Elevation Methodology:
- Identify existing contour where we'd be placing the pond: 229.20 m
- Identify distance from the pond to the creek (placing the inlet in accordance with where the LSRCA suggests on their diagram): 61.05 m
- Based on the existing 229.20 m elevation, I then determined the elevations at each point on the LSRCA cross section, deducting 0.3 m, 1 m, 1m, and 0.5 m to get to the inlet elevation of the pipe (resulting in 226.40 m). Then, assuming the 0.5% down slope of the pipe and 61.05 m horizontal distance, the outlet elevation should be around 226.09 m. However, based on the existing contours, the elevation right before the creek is a bit higher at 226.20 m.
- Based on this, it was decided that the bottom elevation would be raised by 1m. As a result, the elevations would be back-calculated from a creek elevation taken at 227.20 m. Following the above steps(in reverse), the pond elevations were determined.

Catchment 1 ESC Pond: