Stormwater Management Design Brief

Syed, Hyder – 220 Peter Street

For the

Interim Site Condition

June 4, 2024

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1 Objectives/Background

Jewell Engineering Inc. (Jewell) has prepared this *Stormwater Management* (*SWM) Design Brief* to support the interim site plan at 220 Peter Street in Port Hope.

1.1 Background

The site is located approximately 2km east of the Port Hope downtown area, and south of Highway 401. The overall site is relatively large at approximately 8-ha; however, only a portion of the site is proposed to be modified for the interim condition (see Figure 1-1).

In the long term, it is expected that the site will be a fully developed industrial site. With an understanding that this process may take some time, the interim site plan is proposed to provide some utility for the site in the short term. The proposed interim site plan is intended to be a multi-purpose site for storing construction equipment and materials; there is no servicing proposed for the interim condition. The site will include a field office, Quonset, lunch trailer, gravel parking lot, and several laydown areas to store equipment. The remainder of the site will remain undeveloped in the interim conditions.

The proposed development area will include gravel, which is a form of surface hardening that would be expected to increase the runoff rates for the subject site. There may also be total suspended solids (TSS) loading due to vehicle parking. Therefore, quantity and controls are provided with the SWM solution described herein.

1.1.1 Preliminary Planning Report – Clark Consulting

The Preliminary Planning Report prepared by Clark Consulting notes that the south portion of the site is subject to an Exception 19 provision on Schedule A, Sheet 12 of the Zoning By-Law. “The Exception 19 which was approved in 1993 buildings to be located in the Flood Prone Area with a minimum elevation for building openings of 80.22m.” As part of the October 2022 letter (see below or Appendix H), Jewell reviewed the current zoning by-laws and confirmed this exception is still applicable today. However, we note that there are no buildings proposed within the floodplain as part of this interim site plan.

1.1.2 October 2022 Entrance Permit Letter – Jewell Engineering

In October of 2022, Jewell issued a letter to GRCA as part of the entrance permit application to summarize the detailed analysis and modeling that was completed to size the entrance culverts for the subject site. The intent of the entrance was to provide access for field work necessary to prepare the engineering and environmental studies for future full build-out conditions at the site.

The October 2022 letter references the Preliminary Planning Report prepared by Clark Consulting. In anticipation of fill being placed within the special policy area up to 40m from the top of bank on the north side of the creek, a levee was included in the model to confirm no negative impacts upstream. Therefore, as long as fill is placed a minimum distance of 40m from the top of bank for the modeled cross sections immediately upstream of the entrance crossing, there would be no negative impacts to the control of flooding. The interim site plan and proposed SWM controls described in the Interim SWM Design Brief respect this fill limit, and are placed a minimum of 40m from the top of bank.

1.2 SWM Objectives

The proposed SWM solution is based on the technical guidance from:

- 2014 Ganaraska Region Conservation Authority's *Technical and Engineering Guidelines for Stormwater Management Submissions*
- *2003 MOE SWM Planning and Design Manual*
- *1997 MTO Drainage Management Manual*

Based on these guidelines, the **SWM objectives** are:

1. Ensure post-development runoff rates from the subject development area are lesser than or equal to the pre-development runoff rates.
2. Provide quality controls that meet *Enhanced* treatment objectives corresponding to ≥80% TSS removal.
3. Provide general recommendations for sediment and erosion controls, in addition to maintenance requirements for the proposed SWM features.

Figure 1-1: Site Location with Interim Surface Hardening (Red Line) and 0.5m Contour Display

2 Hydrology

This section is intended to identify drainage patterns, summarize hydrology inputs, and assess the need for quantity controls.

2.1 Catchment Areas & Drainage Characteristics

The catchment areas before and after the site improvements are shown in Appendix B. A description of each is provided below.

Existing Conditions:

In existing conditions, Catchment 100 represents the proposed development area. This 2.85-ha area includes the area of proposed surface hardening in addition to the future footprint of the pond and some unimproved areas. The area was delineated using site-specific topographic survey prepared by IBW Surveyors in addition to supplemental detailed LiDAR at 0.5m contour intervals where appropriate.

The runoff coefficient is based on hydrologic soils group (HSG) C type soils and an existing agricultural land use. With a slope between 0 – 5%, MTO Design Chart 1.07 identifies a runoff coefficient of 0.55.

Since the runoff coefficient is greater than 0.4, the Bransby-Williams method is recommended for the time of concentration (Tc) based on the MTO Drainage Manual, producing a Tc of 9.6 minutes. Based on experience, we suspect this method may underestimate the Tc in the pre-development setting and could subsequently over-estimate the pre-development flow target. Therefore, the Airport method was conservatively applied for pre-development conditions only, since it yields a larger Tc of 20 minutes.

Table 2-1: Pre-Development Hydrology Inputs for Interim Condition

Catchment	Area (ha)	RC	Tc (min)
100	2.85	0.55	20 (9.6)
101	2.61	0.55	10

**(9.6) represents Bransby-Williams Tc. 20 represents Airport method Tc.*

Proposed Conditions:

In proposed conditions, Catchment 100A represents the area that drains to the proposed SWM facility and includes the main development area for the interim condition. Catchment 100B represents the gravel driveway that extends north-south across the property to provide an access route. This gravel driveway would drain uncontrolled in the post-development condition and therefore the detention basin that receives Catchment 100A would need to over-control to compensate for the relatively small portion of surface hardening associated with the gravel driveway.

Catchment 101 represents some external drainage to the north. The majority of this north drainage is on the subject property, with a small portion north of the property boundary near the railway that contributes as external drainage. Drainage from Catchment 101 is received at the north limit of the interim development area, where it then drains east to west towards the west property limit. Near the west property limit, the swale turns south to follow the existing grades. The perimeter swale ultimately outlets to the creek at the south end of the property, maintaining the same outlet location and quantity as in existing conditions. The perimeter swale is useful to ensure drainage from the north can continue to be conveyed to the south if there is a net increase in grade elevation with the proposed gravel.

A default Tc of 10 mins was conservatively applied for Catchments 100A and 100B in the interim, post-development condition. Although there is no pavement proposed, and gravel tends to have greater permeability relative to asphalt, a runoff coefficient of 0.95 was applied to all gravel surfaces per the GRCA SWM guidelines.

A summary of post-development hydrology inputs for the interim condition is presented below.

Table 2-2: Post-Development Hydrology Input Summary for Interim Site Plan Condition

Catchment	Area (ha)	RC	Tc (min)
100A	2.43	0.83	10
100B	0.42	0.95	10
101	2.61	0.55	15

2.2 Pre-Development vs. Uncontrolled Post-Development Discharge Rates

Table 2-3 compares the Rational Method peak flows in existing and proposed conditions. With the proposed surface hardening, the interim post-development peak flows exceed the existing runoff rates. Therefore, quantity controls will be required.

Table 2-3: Comparison of Pre-Development and Interim Post-Development Rational Method Peak Flows (m^3/s)

Return Period	Q_{pre}	$Q_{post\ uncontrolled}$			Check:
	100	100A	100B	100A & 100B Total	
	(1)			(2)	(1) ≤ (2)
2	0.183	0.408	0.079	0.487	x
5	0.234	0.522	0.100	0.622	x
10	0.267	0.595	0.115	0.71	x
25	0.309	0.691	0.133	0.824	x
50	0.340	0.759	0.146	0.905	x
100	0.372	0.830	0.160	0.990	x

The SWM solution described in the following section identifies the mitigation measures to provide both quality and quantity controls.

3 SWM Solution

The SWM concept is to use a combination of an enhanced grassed swale, level spreader, vegetative filter strip, and detention basin to achieve both quality and quantity control objectives. The SWM features for the site are summarized in Table 3-1.

Table 3-1: List of SWM Features

SWM Feature	Objective
Perimeter swale (north and west)	Conveyance
Enhanced grassed swale	Quality (Pre-Treatment)
Level spreader & vegetative filter strip	Quality (Secondary & Final)
Detention basin	Quantity

The **perimeter swale** is intended to receive drainage from the north. Majority of this north drainage is also on the subject property, with a small portion north of the property boundary near the railway. This drainage is received at the north limit of the interim development area, where is then drains east to west towards the west property limit. Near the west property limit, the swale turns to a north-south direction to follow the existing grades. The perimeter swale outlets to the creek at the south end of the property, maintaining its same outlet location and quantities as in existing conditions. The perimeter swale is useful to ensure drainage from the north can continue to be conveyed to the south if there is a net increase in grade elevation with the proposed gravel.

The **enhanced grassed swale** begins at the upstream end of the detention basin and is designed with a shallow longitudinal slope and dimensions that meet the criteria in the 2003 MOE SWM Planning & Design Manual (see Subsection 3.1.1). This feature is selected as a reliable method to achieve TSS removal for the runoff from the proposed interim development.

At the downstream end of the enhanced grassed swale, it connects to the **level spreader & vegetative filter strip.** Vegetative filter strips are useful for removing TSS through grassy contact along a wide landscaped surface (see Subsection 3.1.3). The level spreader is included to transfer the concentrated flow from the swale into sheet flow. The level spreader will also act as a sump to collect particulates prior to the filter strip (see Subsection 3.1.2).

The **detention basin** is located at the south limit of the interim development area. This location is within the site's natural low-lying area and allows the drainage pattern to be maintained as much as feasible. The grading plan has runoff entering the detention basin at its northwest corner. Smaller runoff events (25mm quality event and less) will follow the enhanced grassed swale towards the basin outlet. Larger events will overtop a shallow berm within the basin to engage a surplus of quantity storage. The quantity storage is used as part of the stage-storage-discharge relationship to attenuate post-development runoff to pre-development levels or less (see Subsection 3.2).

For the storage calculations used to design the detention basin for flow attenuation purposes, the Modified Rational Method (MRM) was applied. As described in the GRCA SWM guidelines, the MRM is applicable for sites with drainage areas less than 5 ha. The subject SWM plan is within this limitation.

3.1 Quality Control

Best management practices (BMPs) that rely on filtration and infiltration of pollutants are appropriate for the interim site plan condition since the development area is less than 5 ha and does not meet the minimum requirement for a traditional wet pond or dry pond facility. There is also no site servicing proposed for the interim condition, meaning there are no storm sewers and drainage will occur as surface drainage. The quality control component of the SWM solution applies a treatment train approach with a combination of enhanced grassed swale, level spreader / sump, and vegetative filter strips to meet or exceed *Enhanced* treatment objectives. These BMPs were designed based on the guidance in the following documents.

- *2010 Low Impact Development Planning and Design Guide*, prepared by Credit Valley Conservation (CVC) and Toronto and Region Conservation Authority (TRCA)
- *2003 SWM Planning and Design Guide*, prepared by the Ontario Ministry of the Environment
- *2012 Stormwater Management Criteria*, prepared by TRCA

In the *Geotechnical Investigation* report prepared by Nasiruddin Engineering Limited in November of 2022, twelve boreholes numbered 1 through 12 in addition to Boreholes A and B were completed. The results showed that bedrock was as shallow as 1.30m on the subject site. Groundwater was noted to be approximately 2m below ground surface and bedrock may occur at shallower depths than groundwater. With consideration of bedrock and groundwater elevations, the quality controls were kept elevated near existing ground to optimize opportunities for infiltration and in an effort to maintain a separation distance from bedrock and groundwater. We note that each selected treatment type can be applied on any soil type and does not solely rely on infiltration. The filtration components of the mitigation measures will be utilized in addition to infiltration benefits.

3.1.1 Enhanced Grassed Swale (Pre-Treatment)

The low-flow channel within the basin has been designed to meet the enhanced grassed swale criteria from the 2003 MOE manual. It has a 0.75m flat bottom and is sized to convey the water quality event (25mm, 4-hr storm with a Chicago distribution) while reducing velocities to 0.5 m/s or less.

The 2003 MOE Manual indicates that grassed swales are most effective as a best-management practice (BMP) when the bottom width is maximized (≥ 0.75m), longitudinal slope is minimized (≤1 percent), and when the velocity is less than 0.5 m/s. Due to the shallow longitudinal slopes, flow check dams are not required. The grassed swale is located within the SWM basin to allow access for routine maintenance of the low-flow channel by the owner(s) and/or operator(s).

A comparison of design parameters for the enhanced grassed swale to the guidelines in MOE and TRCA documents is provided below. Table 3-2 shows that the proposed swales have been sized in accordance with these guidelines. Enhanced grassed swales can remove approximately 76% of TSS (Credit Valley Conservation and Toronto and Region Conservation Authority, 2010).

Table 3-2: Comparison of Recommended and Proposed Enhanced Grassed Swale Parameters within Detention Basin

Parameter	Recommended	Provided	Check:
Flat Bottom Width (m)	≥ 0.75	0.75	✓
Longitudinal Slope (%)	≤ 1	0.5	✓
Velocity (m/s)	≤ 0.5	0.46	✓

3.1.2 Level Spreader / Sump (Secondary Treatment)

The enhanced grassed swale drains towards a level spreader. The level spreader has two functions as described in the CVC Low Impact Development Design Guidelines:

1) to provide pre-treatment for the downstream filter strip by removing debris and larger sediment particles, and

2) to encourage sheet drainage for the downstream filter strip to reduce flow velocities to maximize filtration / infiltration benefits.

The proposed sump has a 500 mm depth, 15 m length, 1 m width, and is filled with stone.

3.1.3 Vegetative Filter Strip (Final Treatment)

After pre-treatment from the enhanced grassed swales and level spreader, a filter strip is proposed within the SWM facility, with a width of 15m immediately after the level spreader and a minimum flow length of 7m.

Vegetative filter strips are grassed areas that provide filtration of suspended solids. They are generally expected to achieve 20 – 80% TSS removal (Ontario Ministry of the Environment, 2003). There is a wide range of removal efficiencies based on the 2003 MOE Manual and it is reasonable to expect an average of 50% removal from this type of LID. The proposed filter strip has a length and width in excess of 5m, and a longitudinal slope of 0.5% to meet the recommendations in the CVC LID guidelines.

3.1.4 Combined Quality Treatment

Figure 3-1 provides an equation from the *New Jersey Stormwater Best Management Practices Manual* to calculate TSS removal rates for LID/BMPs in series. With this equation, and the information in Table 3-3, the expected TSS removal for the proposed SWM plan is 88%. This is greater than the 80% TSS removal to achieve an *Enhanced* treatment level. Therefore, the proposed controls meet the highest treatment target identified in the *2003 MOE SWM Planning and Design Manual*, and SWM Objective #1 is met.

Table 3-3: Individual TSS Removal for Selected BMPs

BMP	Type	Individual TSS Removal Rate (%)
Enhanced grassed swale	Pre-treatment	76
Detention basin / vegetative filter strip	Secondary	50

A simplified equation for the total TSS removal rate (R) for two BMPs in series is:

$$R = A + B - [(A \times B) / 100] \qquad \text{(Equation 4-1)}$$

Where:

R = Total TSS Removal Rate

A = TSS Removal Rate of the First or Upstream BMP

B = TSS Removal Rate of the Second or Downstream BMP

Figure 3-1: TSS Removal Rate for Consecutive BMPs

For the gravel driveway that drains uncontrolled, it would naturally drain to unimproved surfaces along the perimeter of the driveway. It is recommended that a grassed swale with a 0.75m bottom width and periodic check dams and/or adequate sediment and erosion control devices be provided for the interim to ensure some sediment capture from the gravel driveway.

3.2 Quantity Control

Quantity control is achieved with the detention basin near the south limit of the proposed development area. The basin receives runoff from the surface hardening in the interim site plan condition with the exception of the driveway. The contributing area to the basin is 2.43 ha.

3.2.1 Detention Basin Sizing & Pre- vs. Controlled Post-Development Peak Flows

For the storage calculations used to design the detention basin for flow attenuation purposes, the Modified Rational Method (MRM) was applied. As described in the GRCA SWM guidelines, the MRM is suitable for sites with drainage areas less than 5 ha. The subject SWM plan is within this limitation, as the drainage area to the basin is 2.43 ha, and the total interim development including the driveway that by-passes the facility is 2.85 ha.

The pre-development peak flow rates for each return period were used for the target outflow rates from the combined detention basin outflow plus driveway. Therefore, the allowable release rate from the detention basin is equal to the pre-development peak flow minus the uncontrolled driveway runoff (see Table 3-4).

Table 3-4: Allowable Outflow from Detention Basin (m^3/s)

Return Period	$Q_{pre\ total}$	$Q_{post\ uncontrolled\ 100B}$ (Uncontrolled Driveway)	$Q_{post\ 100A\ Allowable}$ (Detention Basin Outflow)
	(1)	(2)	= (1) - (2)
2	0.183	0.079	0.104
5	0.234	0.100	0.134
10	0.267	0.115	0.152
25	0.309	0.133	0.176
50	0.340	0.146	0.194
100	0.372	0.160	0.212

The stage-storage of the basin operates in tandem with the invert elevation and size of the 450mm outlet pipe to ensure flows are sufficiently routed to meet the pre-development target.

The size of the quantity basin is determined based on a stage-storage-discharge (SSD) relationship. The storage calculations for the facility were prepared using incremental volumes based on a site-specific topographic survey from IBW Surveyors.

The orifice equation is applied to determine outflows at varying elevations and storage volumes. The SSD relationship is used to ensure the storage provided at the target release rate exceeds the storage requirement obtained from the MRM.

Table 3-5 provides a summary of controlled post-development release rates as well as the storage required and provided for each return period event. The storage provided exceeds the storage required to meet the target release rates for each return period event. Therefore, the basin is appropriately sized to meet quantity control objectives, and SWM Objective #2 is met.

The maximum storage requirement is 358 m^3 and the 100-yr pre-development flow rate is maintained with 530 m^3 of storage at elevation 80.08m. The basin includes a 0.30m freeboard with the top of berm at 80.45m.

Based on the guidance in Chapter 8 of the MTO Drainage Manual, runoff coefficients in the post-development condition were increased by 10, 20, and 25% for the 25, 50, and 100-yr events, respectively (see Appendix E). With this adjustment, an upper limit to the runoff coefficient of 0.98 was applied in the post condition, since it is theoretically impossible for a runoff coefficient to exceed 1.0.

Table 3-5: Pre vs. Controlled Post-Development Peak Runoff Rates with Corresponding Storage Volumes and Water Levels

Return Period	$Q_{post\ 100A\ controlled}$	$Q_{post\ 100A\ allowable}$	Storage Required	Storage Provided	Storage Surplus	*Basin Water Level	Basin Depth
	m^3/s		m^3		%	m	m
2	0.104	0.104	176	369	110%	79.94	0.34
5	0.012	0.012	224	407	81%	79.97	0.37
10	0.152	0.152	258	429	66%	79.99	0.39
25	0.176	0.176	318	462	45%	80.02	0.42
50	0.194	0.194	358	488	36%	80.04	0.44
100	0.019	0.019	395	530	34%	80.08	0.48

**Bottom of basin is 79.60m. Invert of overflow spillway is 80.15m.*

A summary of storage basin elevations is provided below.

- Bottom of basin = 79.60m
- 450mm orifice invert = 79.60m
- 100-Yr water level = 80.08m
- Invert of emergency spillway = 80.15m
- Top of freeboard = 80.45m

Note that climate resiliency is demonstrated by providing a minimum 10% surplus of storage in the 100-yr storm to account for potential rainfall increases due to climate change.

3.2.2 Emergency Spillway

The emergency spillway is a 3.5m length earth weir to simulate broad-crested weir flow set at elevation 80.15m. It is sized to independently convey 0.960 m^3/s in an emergency storm event scenario where the outflow pipe is blocked by debris. This is greater than the uncontrolled 100-yr peak flow of 0.830 m^3/s draining to the facility.

Similar to the pond storage volumes, climate resiliency is achieved by ensuring the emergency spillway's capacity is a minimum of 10% greater than the 100-yr uncontrolled peak flow. For added comfort, if the emergency spillway was overtopped in an exceptional rainfall event or construction/maintenance deficiency, the spill would simply drain to the south towards the existing creek as it does in its natural, existing drainage pattern.

3.2.3 Swale Sizing

The perimeter swale that runs along the north and west limits of the interim development area is sized to convey the 100-yr receiving peak flow. Its receiving area is 2.61 ha as shown in the catchment plan in Appendix B. With a runoff coefficient of 0.55 and a time of concentration of 15 minutes, the contributing flow is 0.42 m^3/s. With a 0.75m bottom width, minimum longitudinal slope of 1%, conservative Manning's roughness coefficient of 0.05, 3:1 side slopes, and a 0.4m depth, the swale has a flow capacity of 0.60 m^3/s.

	Swale Capacity	**100-Yr Peak Flow**	**Check:**
	(1)	**(2)**	**(1) > (2)**
North & West Perimeter Swale	0.60	0.42	✓

4 Sediment & Erosion Control

Typical site development will remove much of the vegetated cover. While it is the intention to reduce vegetation removal, exposed soils from the work will be at risk of eroding into the receiving drainage system. Heavy duty silt fences and straw bale check dams are thus recommended for the site and shall be placed in all areas downgradient from the worksite to control sediment runoff. These measures will be required to be put in place to reduce erosion during construction, and for a period of up to one year after construction is completed. Controls should also be placed around stockpiles of topsoil and fill material.

Construction of the proposed development involves the movement and exposure of soils and is typically protracted in time. There is a risk of erosion leading to sediment deposition into the downstream system. Typical sediment and erosion control measures include:

- Siltation fencing
- Strawbale check dams
- Rip-rap check dams

Typical OPSDs provide good instruction on the correct placement and construction of the controls. The controls provide some protection if they are properly maintained, but they should be considered last-resort measures. The most effective means of control are those which prevent or reduce erosion at the source. This would include diligent stabilization of exposed areas immediately after grading is completed. Stabilization measures include sod, erosion blankets or rip-rap and filter cloth on steep slopes as well as topsoil and hydroseed on gently sloped areas (<10%).

A silt fence should be located along the west and south property lines during the construction at a minimum and be maintained until the lands have stabilized or as directed by the municipality.

The sediment and erosion control plan has been prepared by others. It is recommended the sediment and erosion control plan follow the above standard measures.

5 Maintenance

During the first 2 years of operation, the owner should complete a visual inspection of the complete stormwater management system after each significant storm event (approx. 4 times per year) with annual spring inspections thereafter. The owner is responsible for all inspection and maintenance requirements. The points below are a guide for inspection and maintenance practices. Maintenance of the quality control facilities should be in accordance with CVC and TRCA's *2010 Low Impact Development Stormwater Management Planning and Design Guide.*

- **Vegetation Condition** – annual weed control including the removal of invasive species.

- **Obstruction Occurrences** – obstructions and garbage should be cleaned from the SWM basin, swales, and outlet structures.

- **Swales** – all swales should be inspected for signs of erosion. Areas of erosion should be infilled and vegetated immediately.

- **Outlet Structure** – the outlet configuration should be inspected for blockages and outlet erosion. All blockages and outlet erosion should be repaired immediately to ensure proper function of the outlet structure.

- **Sediment** may accumulate in the SWM basin over time. It is recommended that sediment be removed from the basin if it exceeds 10% of the overall storage volume or becomes noticeably elevated above the invert of the outlet pipe.

6 Conclusions / Recommendations

Jewell has prepared this *SWM Design Brief* to support the interim development area for the subject site at 220 Peter Street in Port Hope.

In the long term, it is expected that the site will be a fully developed industrial site. With an understanding that this process may take some time, the interim site plan is proposed to provide some utility for the site in the short term. The proposed interim site plan is intended to be a multi-purpose site for storing construction equipment and materials; there is no servicing proposed for the interim condition. The site will include a field office, Quonset, lunch trailer, gravel parking lot, and several laydown areas to store equipment. The remainder of the site will remain undeveloped in the interim conditions.

The SWM concept is to use a combination of an enhanced grassed swale, level spreader, vegetative filter strip, and detention basin to achieve both quality and quantity control objectives.

The combination of best management practices will be applied to achieve the quality treatment objectives in Section 1. A detention basin is included in the grading plan and has a stage-storage-discharge relationship that is sufficient to attenuate post-development peak flows to the pre-development condition for both post-development Catchments 100A and 100B. The basin is designed to over-control for the uncontrolled gravel driveway in Catchment 100B. Section 3.1 concludes that the basin has surplus storage to achieve these objectives while also providing resiliency measures for climate change.

The proposed interim condition does not include asphalt surfaces. However, a runoff coefficient of 0.95 was applied for the post-development condition based on the GRCA SWM guidelines. The emergency spillway for the basin is also sized to convey the uncontrolled 100-yr peak flow from Catchment 100A.

Authored by:

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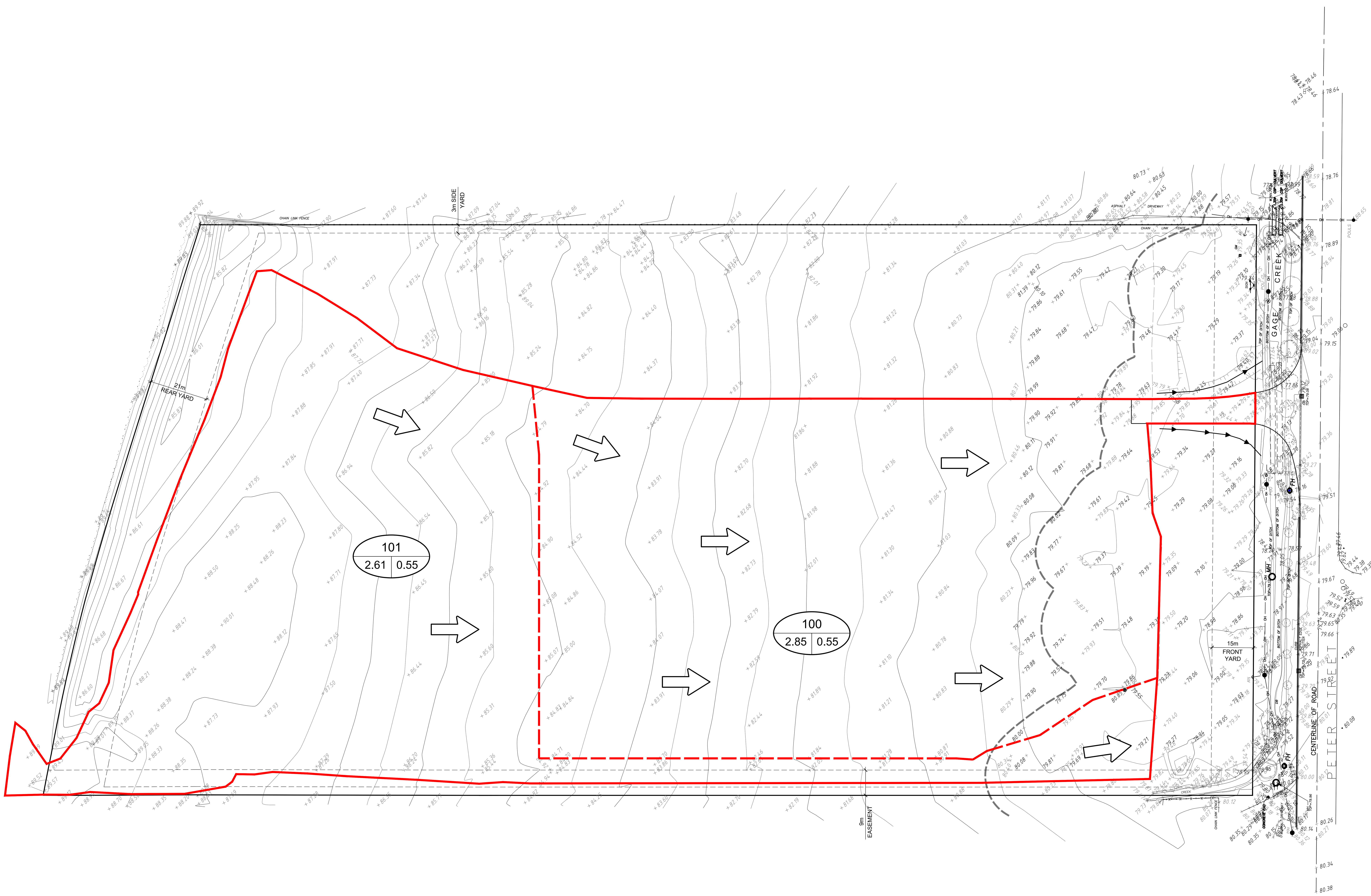
Jewell Engineering Inc.

Appendix A: Site Plan - see Attachment

Appendix B:
Catchment Plans



GENERAL NOTES

- ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK. DISCREPANCIES ARE TO BE REPORTED TO THE CONSULTANT IMMEDIATELY.
- ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM LOCATIONS ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.
- EXCLUDING THE BENCHMARK AND DESCRIPTION PROVIDED FOR THIS PROJECT, NO OTHER ELEVATIONS ARE TO BE USED AS A REFERENCE ELEVATION FOR ANY PURPOSE.
- ALL DIMENSIONS SHOWN ARE IN METRES OR MILLIMETRES, UNLESS OTHERWISE NOTED.

GEOMETRIC NOTE

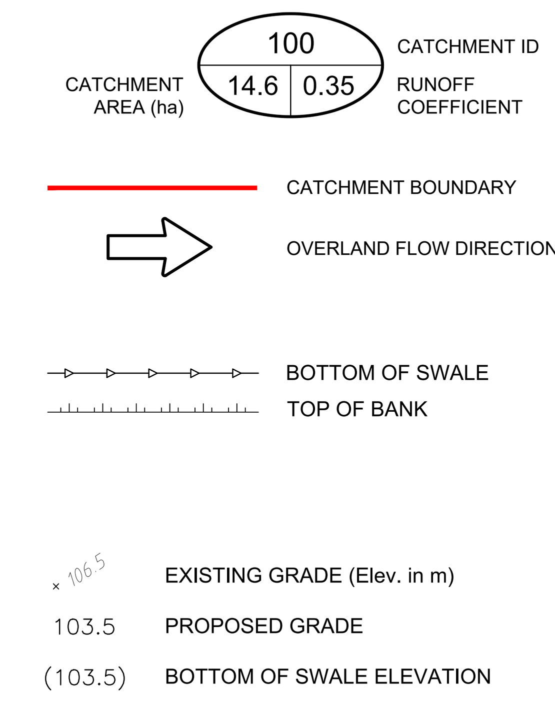
- ALL SURVEY DATA SHOWN ON THIS DRAWING WAS RECORDED USING REAL-TIME KINETIC (RTK) GPS OBSERVATIONS IN REFERENCE TO UTM 18 NORTH COORDINATE SYSTEM.
- ALL ELEVATIONS ARE IN REFERENCE TO LOCAL DATUM NAD83 - GEODETIC MODEL HT2_0, UNLESS DESCRIBED OTHERWISE.

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY

LEGEND



LICENSED PROFESSIONAL ENGINEER
E.J.W. FLEDDERUS
100502141
June 4, 2024
PROVINCE OF ONTARIO

JEWELL ENGINEERING

WONDER INDUSTRIAL PARK

Peter Street
Port Hope, Ontario

INTERIM PRE-DEVELOPMENT CATCHMENT AREAS

DRAWN BY:	PROJECT NO:	
JO	220-5146	
DESIGNED BY: EF	DATE: 4 June 2024	
CHECKED BY: EF	SCALE: 1 : 750	
APPROVED BY:	CONTRACT NO:	DRAWING NO: iCA-1

06/04/2024

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06/04/2024

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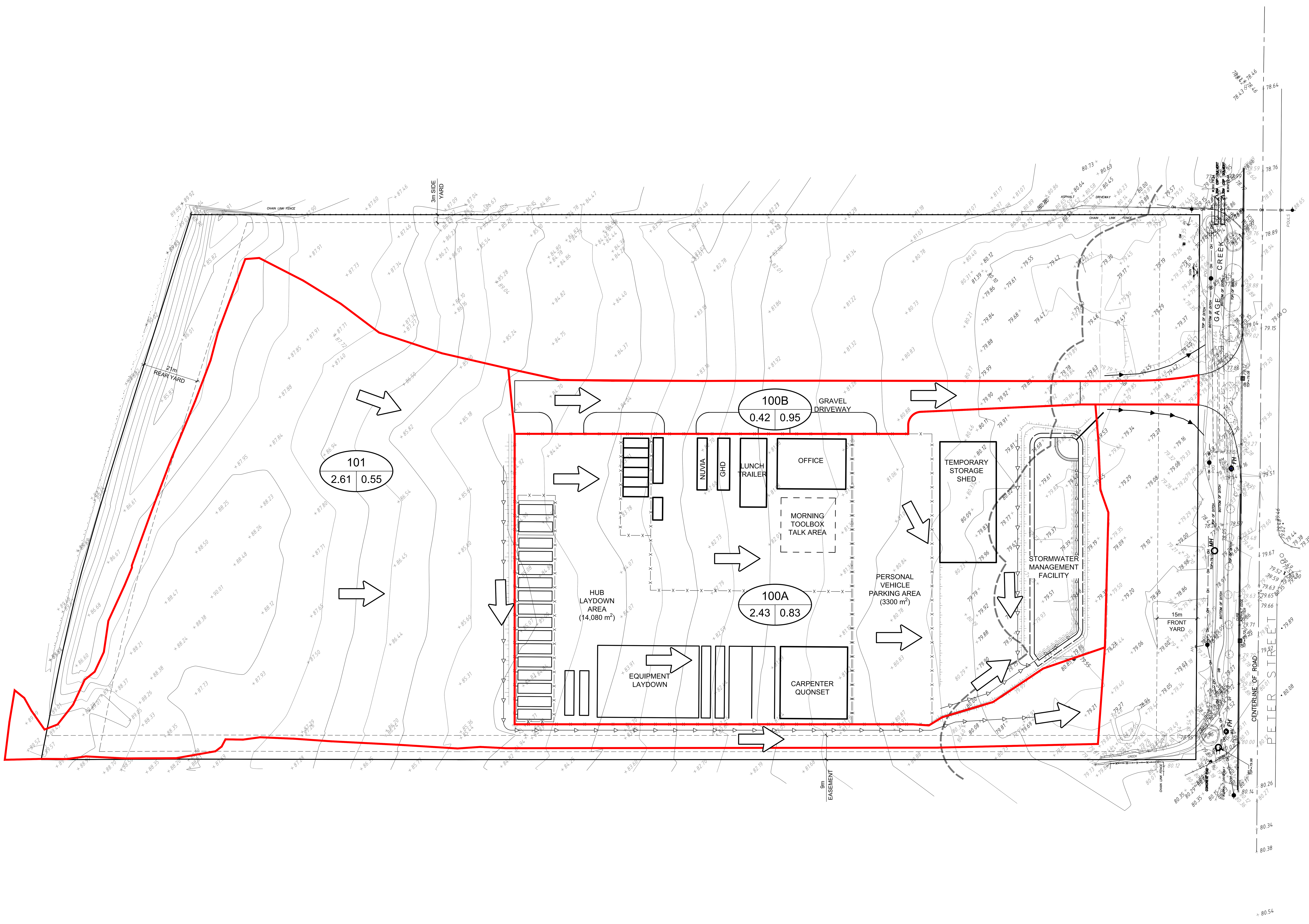

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REVISIONS

NO.	DATE	DESCRIPTION	BY

LEGEND

CATCHMENT AREA (ha) — 100 / 14.6 | 0.35 — CATCHMENT ID, RUNOFF COEFFICIENT

CATCHMENT BOUNDARY

OVERLAND FLOW DIRECTION

BOTTOM OF SWALE

TOP OF BANK

106.5 EXISTING GRADE (Elev. in m)

103.5 PROPOSED GRADE

(103.5) BOTTOM OF SWALE ELEVATION

LICENSED PROFESSIONAL ENGINEER
E.J.W. FLEDDERUS
100502141
June 4, 2024
PROVINCE OF ONTARIO

JEWELL ENGINEERING

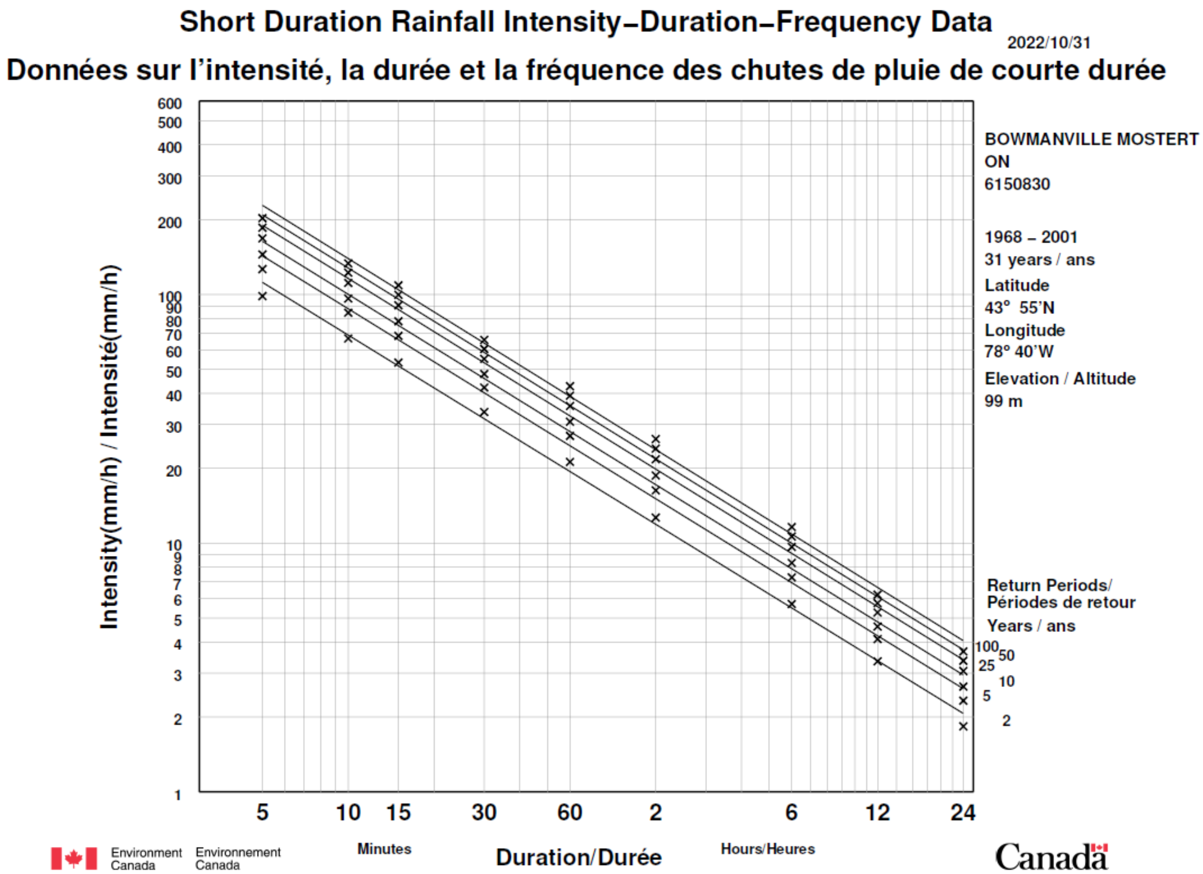
WONDER INDUSTRIAL PARK

Peter Street
Port Hope, Ontario

INTERIM POST-DEVELOPMENT CATCHMENT AREAS

DRAWN BY: JO	PROJECT NO: 220-5146	
DESIGNED BY: EF	DATE: 4 June 2024	
CHECKED BY: EF	SCALE: 1 : 750	
APPROVED BY:	CONTRACT NO:	DRAWING NO: iCA-2

Appendix C:
Environment Canada IDF Curves for Bowmanville


Short Duration Rainfall Intensity–Duration–Frequency Data
2022/10/31
Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée
BOWMANVILLE MOSTERT
ON
6150830
1968 – 2001
31 years / ans
Latitude
43° 55'N
Longitude
78° 40'W
Elevation / Altitude
99 m
Return Periods/
Périodes de retour
Years / ans
100
50
25
10
5
2
Intensity(mm/h) / Intensité(mm/h)
600
500
400
300
200
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
5
10
15
30
60
2
6
12
24
Minutes
Duration/Durée
Hours/Heures
Environment Canada
Environnement Canada
Canada

Appendix D:
Stage-Storage-Discharge Relationship for Modified Rational Method

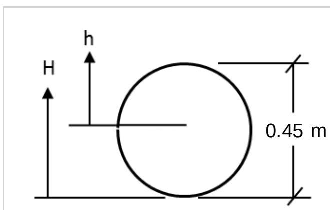
Determine Stage - Storage - Discharge Relationship

Active Storage Calculations

Full Storage Elevation (m)	**80.15**	
Depth of Active Storage (m)	**0.55**	
Bottom of Active Storage (m)	**79.6**	
Active Volume (cu.m)	**617**	*(approx)*

Select Storage Value Method User Defined

Select Stage Increment (m) 0.05
(not less than 0.01 m)



	Outlet 1	Outlet 2	Outlet 3
Use Outlet?	Use Outlet 1 ? **Yes**	Use Outlet 2 ? **No**	Use Outlet 3 ? **Yes**
Type	**Orifice**	**Orifice**	**Broad Crested Weir**
Formula	$Q = CA_o(2gh)^{0.5}$	Q = CAo(2gh)^(0.5)	Q = 1.67LH13/2
Invert =	79.60 m	79.60 m	80.15 m
Coeff =	0.60	0.60	
Length			3.5 m
Orifice Dia =	0.45 m	0.400 m	
Circular?	**Yes** *(Select Yes or No)*		*(No End Contractions)*
Area =	0.159 m^2	0.126	
Obvert =	80.05 m	80.00	

Elevation	Length	Width	Incr Vol	Cum vol	Low Flow Outlet (Orifice) Weir (H)	Low Flow Outlet (Orifice) Head (h)	Low Flow Outlet (Orifice) Flow (Q)	Orifice Head (H)	Orifice Head (h)	Orifice Flow (Q)	Emergency Spillway Head (H)	Emergency Spillway Flow (Q)	Total Discharge
m	m	m	m3	m3	m	m	cms	m	m	cms	m	cms	cms
79.60				0	0.000	-0.225	0.000	0.000	-0.200	0.000	0.000	0.000	0.0000
79.65				53	0.050	-0.175	0.001	0.050	-0.150	0.001	0.000	0.000	0.0008
79.70				106	0.100	-0.125	0.004	0.100	-0.100	0.004	0.000	0.000	0.0044
79.75				160	0.150	-0.075	0.012	0.150	-0.050	0.012	0.000	0.000	0.0120
79.80				214	0.200	-0.025	0.025	0.200	0.000	0.025	0.000	0.000	0.0247
79.85				270	0.250	0.025	0.043	0.250	0.050	0.045	0.000	0.000	0.0426
79.90				326	0.300	0.075	0.073	0.300	0.100	0.076	0.000	0.000	0.0727
79.95				383	0.350	0.125	0.114	0.350	0.150	0.115	0.000	0.000	0.1137
80.00				440	0.400	0.175	0.160	0.400	0.200	0.149	0.000	0.000	0.1602
80.05				498	0.450	0.225	0.200	0.450	0.250	0.167	0.000	0.000	0.2005
80.10				557	0.500	0.275	0.222	0.500	0.300	0.183	0.000	0.000	0.2217
80.15				617	0.550	0.325	0.241	0.550	0.350	0.198	0.000	0.000	0.2410
80.20				617	0.600	0.375	0.259	0.600	0.400	0.211	0.050	0.065	0.3242
80.25				617	0.650	0.425	0.276	0.650	0.450	0.224	0.100	0.185	0.4604
80.30				617	0.700	0.475	0.291	0.700	0.500	0.236	0.150	0.340	0.6309
80.35				617	0.750	0.525	0.306	0.750	0.550	0.248	0.200	0.523	0.8291
80.40				617	0.800	0.575	0.321	0.800	0.600	0.259	0.250	0.731	1.0511

2-Yr:

Area = 2.43 ha Required Release Rate = 0.104 cms
C = 0.83
Q = 1/360 x CiA

Time (min)	i	Qi	Qallow	Qdiff	Storage
	mm/hr	cms	cms	cms	
5	98.6	0.554	0.104	0.450	135.0
10	66.6	0.374	0.104	0.270	162.2
15	53.3	0.300	0.104	0.196	176.0
30	33.7	0.189	0.104	0.085	153.7
60	21.2	0.119	0.104	0.015	54.5

5-Yr:

Area = 2.43 ha Required Release Rate = 0.134 cms
C = 0.83
Q = 1/360 x CiA

Time (min)	i	Qi	Qallow	Qdiff	Storage
	mm/hr	cms	cms	cms	
5	126.5	0.711	0.134	0.577	173.1
10	84.5	0.475	0.134	0.341	204.5
15	68.2	0.383	0.134	0.249	224.4
30	42.3	0.238	0.134	0.104	186.7
60	27	0.152	0.134	0.018	63.9

10-Yr:

Area = 2.43 ha Required Release Rate = 0.152 cms
C = 0.83
Q = 1/360 x CiA

Time (min)	i	Qi	Qallow	Qdiff	Storage
	mm/hr	cms	cms	cms	
5	145	0.815	0.152	0.663	198.9
10	96.4	0.542	0.152	0.390	233.9
15	78.1	0.439	0.152	0.287	258.2
30	47.9	0.269	0.152	0.117	211.0
60	30.8	0.173	0.152	0.021	76

25-Yr:

Area = 2.43 ha Required Release Rate = 0.176 cms
C = 0.87
Q = 1/360 x CiA

Time (min)	i	Qi	Qallow	Qdiff	Storage
	mm/hr	cms	cms	cms	
5	168.4	0.985	0.176	0.809	242.7
10	111.4	0.652	0.176	0.476	285.4
15	90.5	0.529	0.176	0.353	318.1
30	55.1	0.322	0.176	0.146	263.4
60	35.6	0.208	0.176	0.032	116

50-Yr:

Area = 2.43 ha Required Release Rate = 0.194 cms
C = 0.88
Q = 1/360 x CiA

Time (min)	i	Qi	Qallow	Qdiff	Storage
	mm/hr	cms	cms	cms	
5	185.7	1.100	0.194	0.906	271.9
10	122.5	0.726	0.194	0.532	319.2
15	99.8	0.591	0.194	0.397	357.7
30	60.4	0.358	0.194	0.164	295.1
60	39.2	0.232	0.194	0.038	138

100-Yr:

Area = 2.43 ha Required Release Rate = 0.212 cms
C = 0.88
Q = 1/360 x CiA

Time (min)	i	Qi	Qallow	Qdiff	Storage
	mm/hr	cms	cms	cms	
5	202.9	1.211	0.212	0.999	299.6
10	133.5	0.797	0.212	0.585	350.8
15	109	0.650	0.212	0.438	394.6
30	65.7	0.392	0.212	0.180	324.1
60	42.8	0.255	0.212	0.043	156.2

Appendix E:
Adjusted Post-Development Runoff Coefficients for Major Storm Events

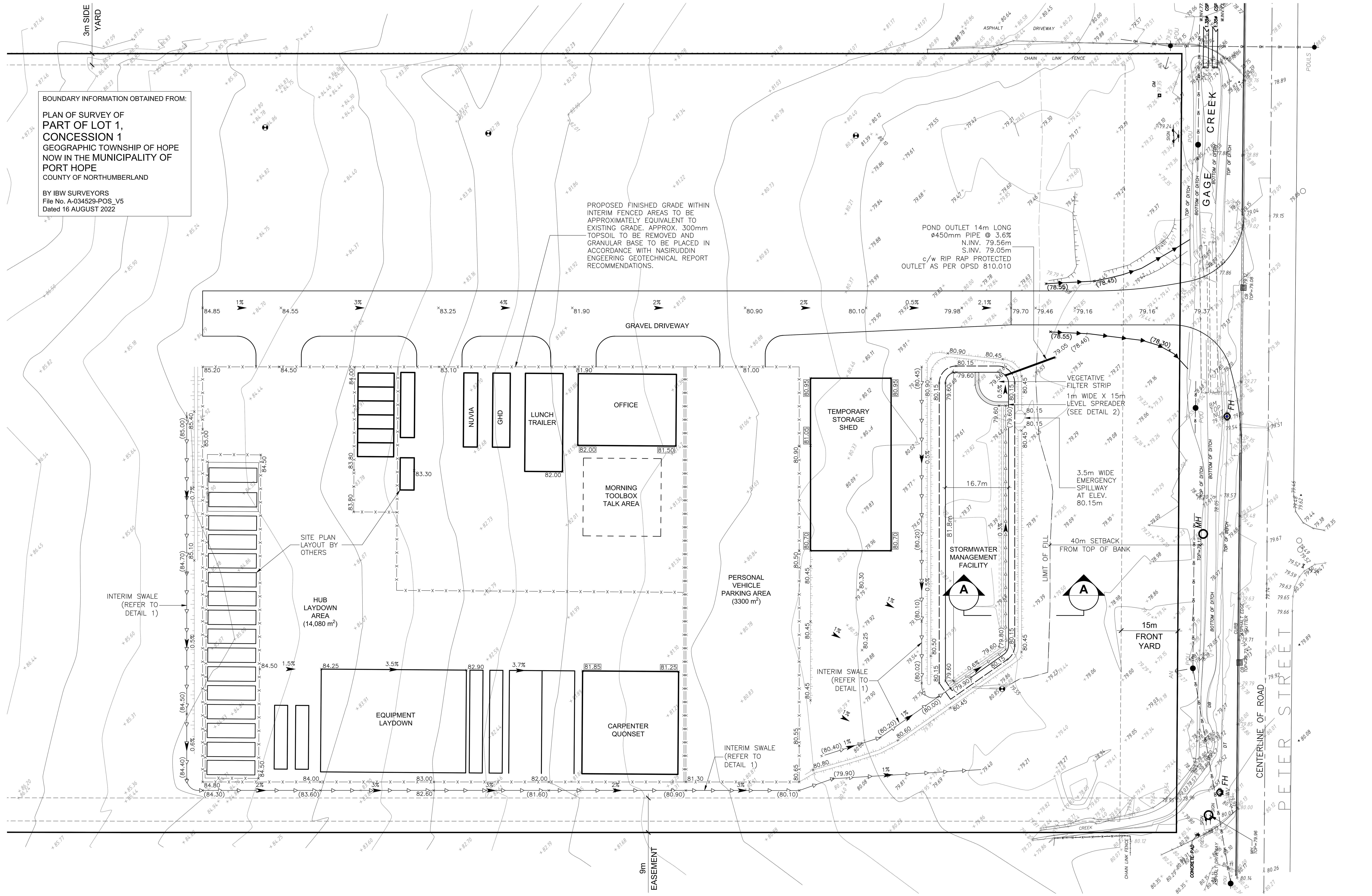
Minor Events - No Adjustment			
Land Cover Description	**Area (ha)**	**RC**	**A x RC**
Catchment 100A: Uncontrolled Gravel Driveway			
Driveway by-passing basin	0.42	0.95	0.40
	0.42		0.40
Weighted RC		**0.95**	
Catchment 100B: Gravel Area to Proposed SWM Facility			
Temporary storage shed	0.10	0.95	0.10
Main gravel area	1.74	0.95	1.65
Unimproved	0.42	0.55	0.23
Grass/Landscaped Pond Area	0.16	0.25	0.04
	2.43		2.02
Weighted RC		**0.83**	

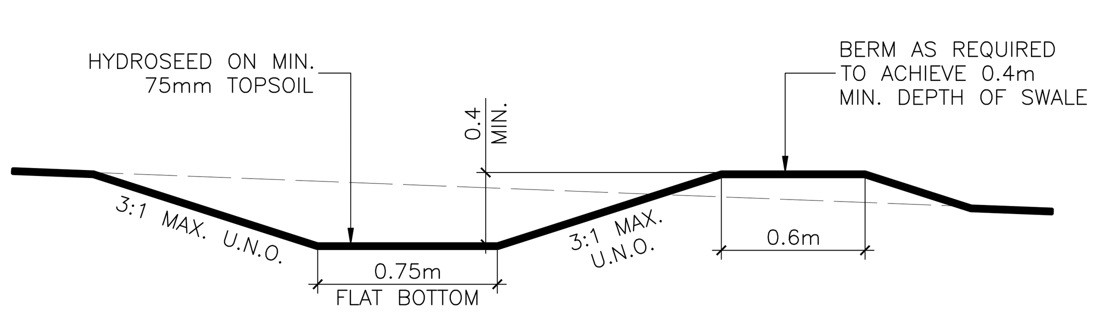
25-Yr - 10% Adj. up to 0.98			
Land Cover Description	**Area (ha)**	**RC**	**A x RC**
Catchment 100A: Uncontrolled Gravel Driveway			
Driveway by-passing basin	0.42	0.98	0.41
	0.42		0.41
Weighted RC		**0.98**	
Catchment 100B: Gravel Area to Proposed SWM Facility			
Temporary storage shed	0.10	0.98	0.10
Main gravel area	1.74	0.98	1.71
Unimproved	0.42	0.61	0.26
Grass/Landscaped Pond Area	0.16	0.28	0.05
	2.43		2.11
Weighted RC		**0.87**	

50-Yr - 20% Adj. up to 0.98			
Land Cover Description	**Area (ha)**	**RC**	**A x RC**
Catchment 100A: Uncontrolled Gravel Driveway			
Driveway by-passing basin	0.42	0.98	0.41
	0.42		0.41
Weighted RC		**0.98**	
Catchment 100B: Gravel Area to Proposed SWM Facility			
Temporary storage shed	0.10	0.98	0.10
Main gravel area	1.74	0.98	1.71
Unimproved	0.42	0.66	0.28
Grass/Landscaped Pond Area	0.16	0.30	0.05
	2.43		2.13
Weighted RC		**0.88**	

100-Yr - 25% Adj. up to 0.98			
Land Cover Description	**Area (ha)**	**RC**	**A x RC**
Catchment 100A: Uncontrolled Gravel Driveway			
Driveway by-passing basin	0.42	0.98	0.41
	0.42		0.41
Weighted RC		**0.98**	
Catchment 100B: Gravel Area to Proposed SWM Facility			
Temporary storage shed	0.10	0.98	0.10
Main gravel area	1.74	0.98	1.71
Unimproved	0.42	0.69	0.29
Grass/Landscaped Pond Area	0.16	0.31	0.05
	2.43		2.15
Weighted RC		**0.88**	

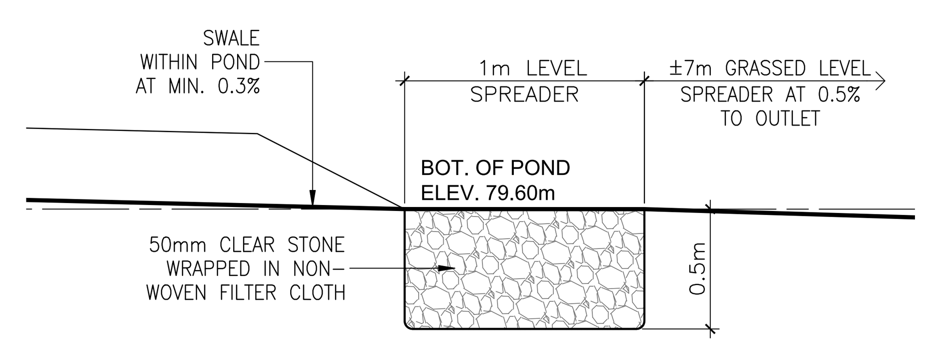
Appendix F:
Grading Plan with Detention Basin & Swale Details





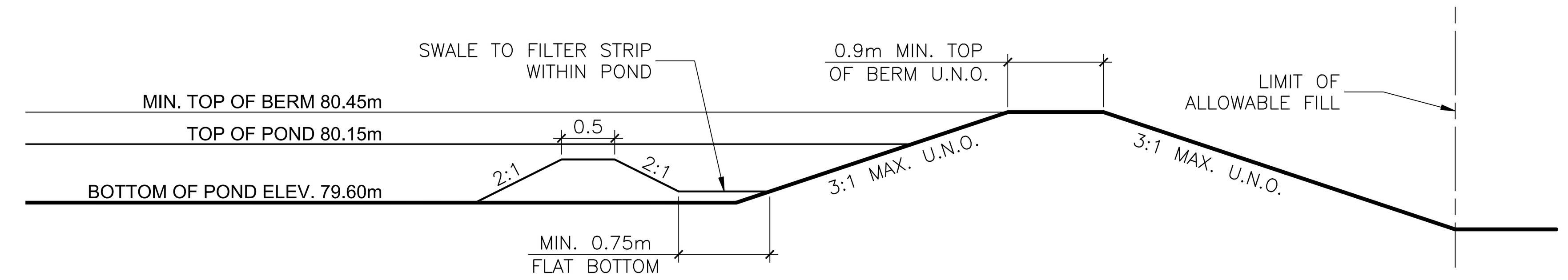
INTERIM SWALE DETAIL 1

Not to scale



LEVEL SPREADER DETAIL 2

Scale 1:25



SECTION A

Scale 1:50

GENERAL NOTES

- ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK. DISCREPANCIES ARE TO BE REPORTED TO THE CONSULTANT IMMEDIATELY.
- ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM LOCATIONS ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.
- EXCLUDING THE BENCHMARK AND DESCRIPTION PROVIDED FOR THIS PROJECT, NO OTHER ELEVATIONS ARE TO BE USED AS A REFERENCE ELEVATION FOR ANY PURPOSE.
- ALL DIMENSIONS SHOWN ARE IN METRES OR MILLIMETRES, UNLESS OTHERWISE NOTED.

GEOMETRIC NOTE

- ALL SURVEY DATA SHOWN ON THIS DRAWING WAS RECORDED USING REAL-TIME KINETIC (RTK) GPS OBSERVATIONS IN REFERENCE TO UTM 18 NORTH COORDINATE SYSTEM.
- ALL ELEVATIONS ARE IN REFERENCE TO LOCAL DATUM NAD83 - GEODETIC MODEL HT2_0, UNLESS DESCRIBED OTHERWISE.

* * DRAWINGS ARE NOT TO BE SCALED * *

REVISIONS

NO.	DATE	DESCRIPTION	BY

LEGEND

106.5	EXISTING GRADE (Elev. in m)
103.5	PROPOSED GRADE
(103.5)	BOTTOM OF SWALE ELEVATION
	TOP OF BANK
	TOE OF SLOPE
	BOTTOM OF SWALE
	FENCE

LICENSED PROFESSIONAL ENGINEER
E.J.W. FLEDDERUS
100502141
June 5, 2024
PROVINCE OF ONTARIO

JEWELL ENGINEERING

WONDER INDUSTRIAL PARK

Peter Street
Port Hope, Ontario

INTERIM GRADING PLAN

DRAWN BY: JO	PROJECT NO: 220-5146
DESIGNED BY: EF/AMR	DATE: 5 June 2024
CHECKED BY: EF	SCALE: 1 : 500
APPROVED BY:	CONTRACT NO:
	DRAWING NO: iGP-1

F:\FTP\PROJECT FILES\1 CIVIL 3D PROJECT FILES\2205146 - SYED, HYDER - PETER STREET SITE\3 - SHEETS\5146 - IGRADING

06/05/2024

Appendix G:
Swale Sizing Open Channel Flow

North & West Perimeter Swale – 100-Yr Event:

Mannings - Open channel flow for Trapezoidal Swale

$Q = 1/n\ AR^{2/3}S^{1/2}$

Desired Flow Capacity = **0.42** **cms** <=== Largest 100-yr peak flow contributing to rear-yard swale from Rational Method with 02.61 ha, 0.55 runoff coefficient, and 15 minute time of concentration.

Channel Configuration

Bottom Width	0.75	m
Side Slopes	3	:1
Slope	0.01	m/m
Roughness	0.05	<=== conservatively applied for grass channel
Channel Depth	0.4	m

R = Hydraulic Radius = Area / Wetted Perimeter (m)
P = Wetted Perimeter (m)
A= Area (m^2)

<u>Assume Full Flow</u>

A = 0.78
P = 3.279822
R = 0.237818
V = Channel Velocity (m/s) = 0.768
Q = Channel Flow Capacity = **0.60** **cms**

Enhanced Grassed Swale – 25mm Event:

$Q = 1/n\ AR^{2/3}S^{1/2}$

Desired Flow Capacity = **0.23** **cms**

<=== 25mm Quality Peak Flow as per Equation 4.8 of 2003 MOE Manual

Channel Configuration

Bottom Width	0.75	m
Side Slopes	3	:1
Slope	0.005	m/m
Roughness	0.05	
Channel Depth	0.30	m

R = Hydraulic Radius = Area / Wetted Perimeter (m)
P = Wetted Perimeter (m)
A= Area (m^2)

<u>Assume Full Flow</u>

A =	0.495
P =	2.647367
R =	0.186978
V = Channel Velocity (m/s) =	0.46
Q = Channel Flow Capacity =	**0.23** **cms**

Appendix H:
October 2022 Entrance Sizing Letter – Jewell Engineering

JEWELL ENGINEERING

October 18, 2022

Ganaraska Conservation Authority
2216 County Road 28
Port Hope, ON
L1A 3V8

Attention: Ms. Leslie Benson, Water Resources Engineer

RE: Proposed Entrance Permit – Peter St, Port Hope
Jewell Engineering Inc. File No.: 220-5146

Dear Ms. Benson:

We have completed the sizing of an entrance culvert for the proposed future industrial site along Highway 2 (Peter Street) near the east end of Port Hope.

Description of the Purpose of the Work:

The purpose of the proposed entrance is to ultimately provide access to a future industrial park.

For the interim, the purpose of the entrance construction is to allow equipment to access the site to facilitate the studies required to prepare a site plan submission to the Municipality of Port Hope. The remainder of the site construction would not take place until site plan approval is received from the municipality.

The access will allow equipment to cross West Gage Creek from Highway 2.

Summary of Entrance Design and Analysis:

A summary of the entrance location, road profile, flood limits, and proposed culverts along the subject property is provided in Figure 1.

The Preliminary Planning Report prepared by Clark Consulting notes that the south portion of the site is subject to an exception 19 provision on Schedule A, Sheet 12 of the Zoning By-law. "The Exception 19 which was approved in 1993 buildings to be located in the Flood Prone Area with a minimum elevation for building openings of 80.22m." We reviewed the

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current zoning by-laws and confirmed this exception is still applicable today. However, this permit application does not request or include any buildings with the floodplain, and we do not anticipate future buildings will be located within the floodplain at this time.

We reviewed the existing West Gage Creek model provided by GRCA. Per GRCA's request, we updated the model with our proposed crossing configuration to assess:

- potential impacts to the upstream property owner,
- the relief flow depth over Highway 2 in the regulatory event,
- the relief flow depth at the proposed entrance in the regulatory event.

In a Teams Meeting on August 30, 2022, we presented the findings from our review of the existing West Gage Creek model within the vicinity of our subject site. We also presented some preliminary findings from our proposed analysis.

The existing model is described in a 2011 letter prepared by previous GRCA staff. This letter references an updated 'Master' file as the existing model applied by GRCA.

In our review of the existing model and the 2011 letter prepared by others, we noted that the 100-yr peak flow governs as the regulatory storm event. We also noted that a lateral structure was applied to model the spill over Highway 2. However, the 'Flow Optimization' component of the lateral structure was turned off. Due to this, the flows do not effectively spill over the road, and the entire 25 m^3/s regulatory flow unrealistically stays within the ditch system along the north side of Highway 2. We also noticed that the lateral structure did not extend far enough to the east to capture the road sag of Highway 2.

The result is a relief flow depth over the road in the existing 'Master' file that is artificially high at 0.67m.

An excerpt from the HEC-RAS manual that describes the flow optimization toggle is provided below for reference.

> **Optimization**. This option is for steady flow modeling only. When modeling in a steady flow mode, the user can have the software figure out how much flow will leave through the lateral structure, and how much will continue on downstream. This calculation requires an iterative solution. Pressing the Optimization button brings up an editor that allows the user to turn the optimization option on. When optimization is not turned on, the program will assume all of the water is still going downstream, though it will calculate what could have gone out the lateral weir based on the computed water surface. When optimization is turned on, the software calculates the flow out of the lateral structure, reduces the flow in the main river, and then recalculates the profile in the main river. This operation continues until there is a balance between the calculated and assumed flows for the main river.

To achieve a more realistic modelling result, we updated this portion of the existing conditions model with the flow optimization turned on. We also completed detailed topographic survey at the subject site to get detailed creek cross-sections, centerline elevations along Highway 2, the two driveways on the property immediately downstream (east) of the subject site, as well as the overbank areas on the subject property. This detailed survey data is reflected in the added cross-sections and updated lateral weir in the Geometry Editor of the 2022 HEC-RAS model (2022 updated model files attached).

The proposed entrance was designed to maximize flow efficiency. The entrance is designed with a 25m flat bottom that simulates a weir to provide relief flow in the regulatory event.

The entrance weir is intentionally set to elevation 79.16m to match the existing edge of pavement at Highway 2; this allows the entrance to engage relief flow at the same time as Highway 2 and ensures there is no increase in flow depth over the road. The long, 25m entrance ‘weir’ is also used to limit the relief flow over the <u>entrance</u> in the regulatory (100-yr) event. This allows mid-size and emergency vehicles to access the site at all times, while also ensuring there are no increases in water surfaces elevations at the upstream property limit.

The culvert sizes include two, 1.84m span x 1.26m rise CSP arch culverts. The culverts were also sized to maximize flow efficiency. The existing creek width ranges from 5 – 7m across the front of the site based on our topographic survey. The two CSPA culverts, including 600mm of spacing (fill material) between them, have a combined span of 4.3m. This utilizes close to the full width of the creek while respecting the existing creek dimensions for limited disturbance (see Figure 2).

The culverts have 420mm of cover to accommodate a reasonable gradient (approx. 3%) immediately off of Highway 2. This cover is slightly greater than the minimum cover requirement of 300mm for the selected culverts based on the *Handbook of Steel Drainage & Highway Construction Products* published by the Corrugated Steel Pipe Institute. As expected, the proposed culverts are similar in size to the existing entrance culverts immediately downstream (east) of the subject site.

Table 1 below summarizes the relief flow depths at Highway 2 and the entrances as well as a comparison of water surface elevations at the upstream (west) property limit. A comparison of WSELs between existing and proposed conditions is shown in Figure 3.

Parameter	**Unit**	**Updated Existing Conditions**	**With Proposed Entrance**	**Check:**
		(1)	**(2)**	**(2) ≤ (1)**
Max. Relief Flow Depth at Hwy. 2	m	0.44	0.42	✓
Max. Relief Flow Depth at Downstream Crossing	m	0.37	0.34	✓
Max. Relief Flow Depth at Proposed Crossing	m	-	0.39	-
*WSEL at Upstream (West) Property Limit	m	79.9	79.9	✓

Based on the *2002 Ontario MNR Technical Guide for Flood Hazard Limits*, there is no defined flood depth for safe access; rather, it is a range that depends on the type of vehicle. For example, emergency vehicles can generally travel roads with flood depths up to 0.4 – 0.5m. Firetrucks can generally travel roads with up to 0.9m of depth. Small family vehicles are generally limited to shallower flood depths in the range of 0.3 – 0.4m.

The subject site is intended to be an industrial park with predominantly mid to large-sized vehicles. Mid to large size vehicles and emergency vehicles will have access to the site in all return period events. The 100-yr event is a return period event that has a statistical probably of 1% of occurrence in a given year. Small family vehicles may temporarily not have access for approximately 3-5 hours once every 100 years. This presents an unlikely inconvenience rather than a safety concern. Given the site's proposed land use, the slight improvement (reduction) in relief flow depth at Highway 2 and the downstream private crossing, and maximum flow efficiency with the proposed entrance, we recommend the selected crossing configuration described in this letter and attached drawing.

Additional notes:

- The 2011 letter noted a downstream boundary condition of 'Known WSEL' was set to 78.0m. There is no further information available for the adjacent tributary to set an alternative WSEL for this boundary condition. So, we tested the sensitivity of the model to this downstream WSEL. In a test simulation with the 'Known WSEL' raised by 1m to elevation 79.0m, there were no appreciable impacts to the relief flow depth over Highway 2 as it increased by 2cm.

- Following the initial submission of this letter on September 30, 2022, GRCA requested further commentary on the increase in WSEL that occurs immediately upstream of the proposed crossing. A floodline for the south side of the creek was also requested to show where the limit of spill over Highway 2 occurs. This has been added to the attached entrance drawing. GRCA and JE agreed it was not necessary to show the south floodline beyond Highway 2 since the spill would continue all the way down to the main branch of Gage Creek south of Highway 2 and a survey of these downstream lands was not available.

    - All new crossings present an increase in WSELs on their immediate upstream side. **Objective #1** is to ensure this increase dissipates before the upstream (west) property limit to avoid negative impacts to upstream property owners. **Objective #2** is to ensure the increase does not create flow depths greater than 0.3m over Highway 2 since it is a significant arterial road.

        - The HEC-RAS calculated WSELs show that the increase in WSEL is dissipated before the upstream property limit (see table and drawing on following pages). This is represented by Cross Section 1439. The RAS results actually show a 2cm reduction, but this minimal discrepancy is due to RAS oscillations; in reality, the WSELs would be equivalent at this cross section. **Objective #1 is satisfied.**

- The largest increase in flood elevation is 0.13m and occurs at Cross Section 1335 immediately upstream of the proposed crossing. The depth over the road is 0.06m in existing conditions and increased to 0.19m. The increases in flood elevation upstream of the crossing are limited to 0.13m or less with a resulting relief flow depth of 0.19m or less (see Cross Sections 1335 to 1439). This is less than 0.3m and does not present a drainage concern since all motor vehicles have safe passage at these depths. Therefore, **Objective #2 is satisfied**.

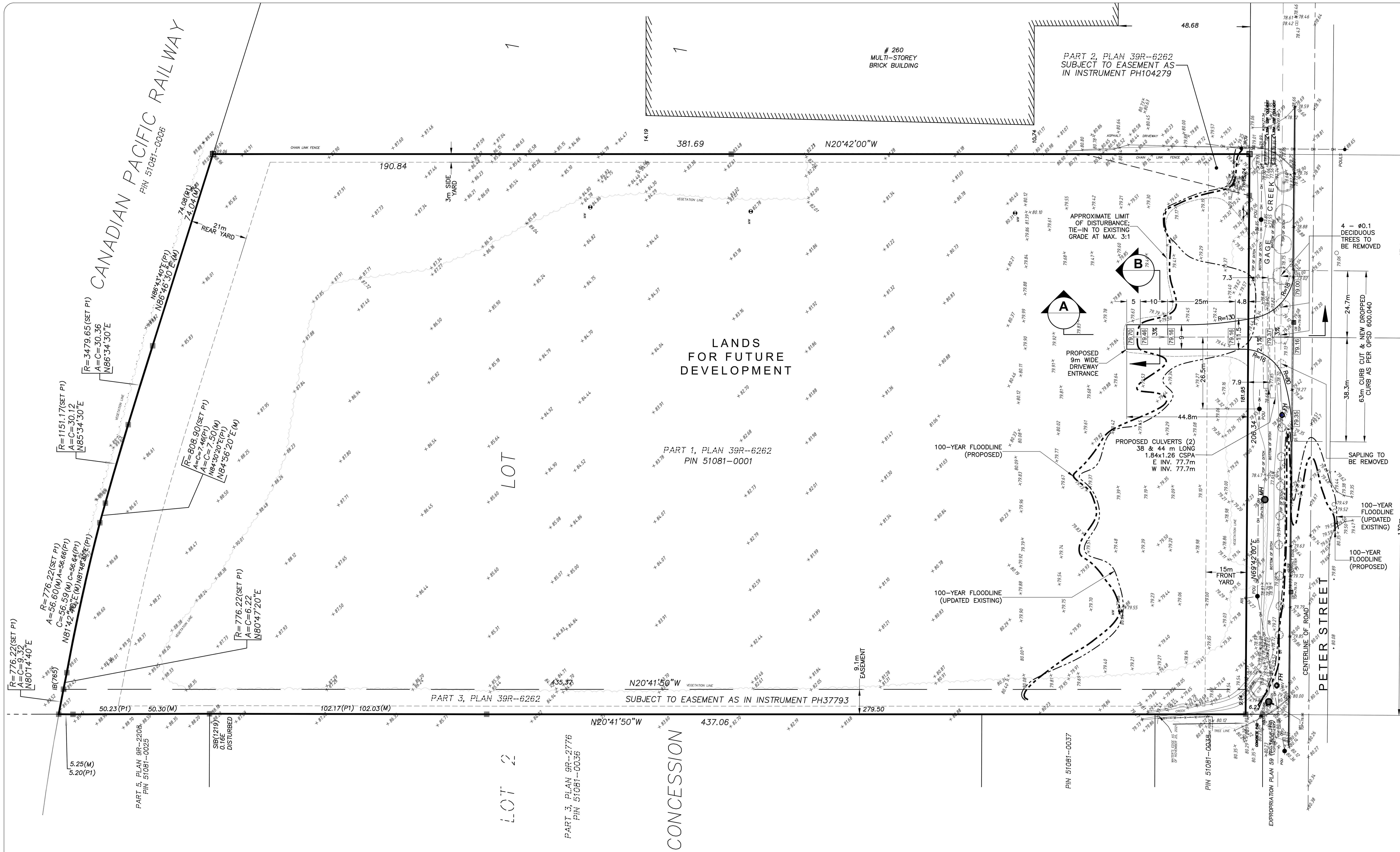
    - We re-iterate that there is no increase in maximum relief flow depth along Highway 2 with the proposed crossing. The maximum relief flow depth is a result of the existing downstream crossings that are not part of the subject application. As indicated in the table on P. 3 of this letter, the proposed crossing offers a slight improvement in the maximum relief flow depth at Highway 2 by reducing this depth from 0.44m to 0.42m.

I am happy to discuss should further clarification be required.

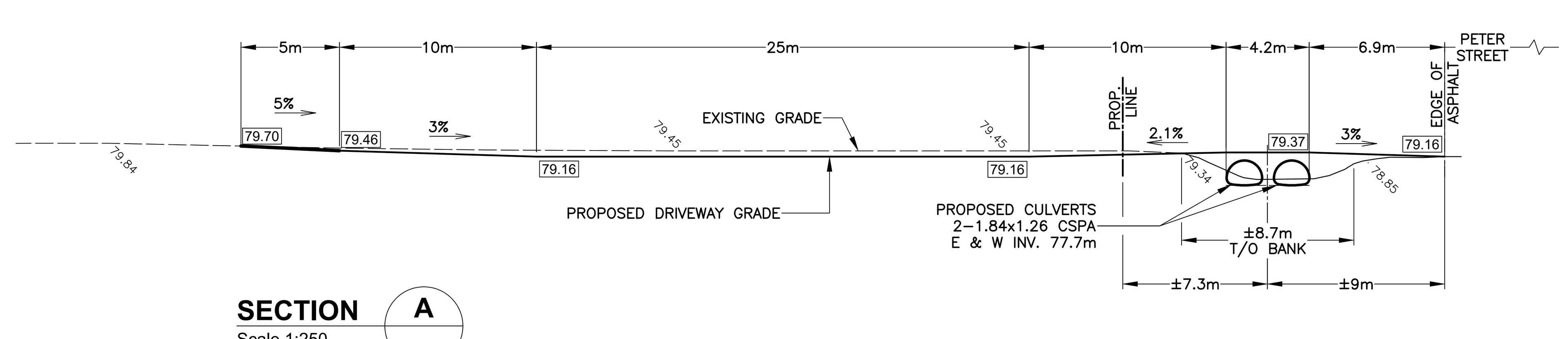
Sincerely,

Elliott Fledderus, P.Eng.
Jewell Engineering Inc.

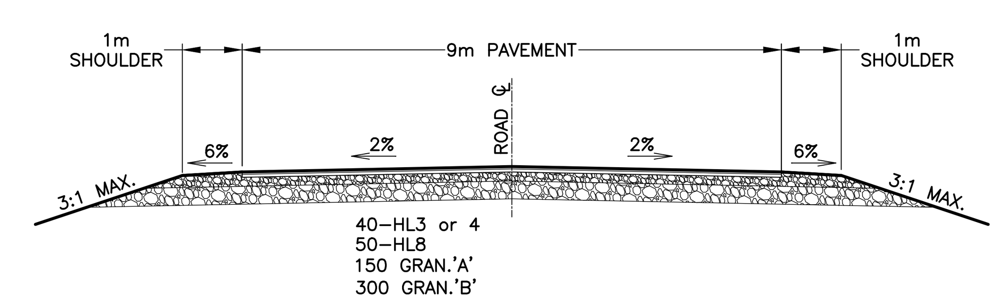
Figure 1: Driveway Entrance Sketch (see following page)



PROPOSED DRIVEWAY ENTRANCE
Scale 1:750



SECTION A
Scale 1:250



SECTION B
Scale 1:100

GENERAL NOTES:

- ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK. ANY DISCREPANCIES ARE TO BE REPORTED TO THE CONSULTANT IMMEDIATELY.
- ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. THE CONTRACTOR SHALL CONFIRM THE LOCATION ON SITE AND ASSUME ALL LIABILITY FOR DAMAGE TO ALL UTILITIES.
- EXCLUDING THE BENCHMARK AND DESCRIPTION PROVIDED FOR THIS PROJECT, NO OTHER ELEVATIONS ARE TO BE USED AS A REFERENCE ELEVATION FOR ANY PURPOSE.

METRIC NOTE:

- ALL DIMENSIONS SHOWN ARE IN METRES OR MILLIMETRES, UNLESS OTHERWISE NOTED.

GEOMETRIC NOTE:

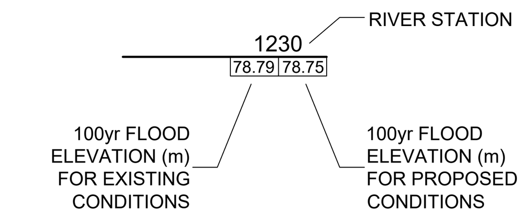
- ALL SURVEY DATA SHOWN ON THIS DRAWING WAS RECORDED USING REAL-TIME KINETIC (RTK) GPS OBSERVATIONS IN REFERENCE TO UTM 18 NORTH COORDINATE SYSTEM.
- ALL ELEVATIONS ARE IN REFERENCE TO LOCAL DATUM NAD83 - GEODETIC MODEL HT2_0, UNLESS DESCRIBED OTHERWISE.

* * DRAWINGS ARE NOT TO BE SCALED * *

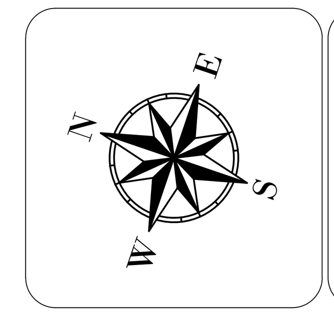
REVISIONS

NO.	DATE	DESCRIPTION	BY
0	30Sep22	ISSUED FOR ENTRANCE PERMIT	JO
1	11Oct22	TREE REMOVAL & CURB CUT NOTES ADDED	JO
2	12Oct22	FLOODLINES REVISED & CULV. INFO ADDED	JO

LEGEND



NOTE:
100-YEAR STORM REPRESENTS REGULATORY EVENT BASED ON THE WEST GAGE CREEK INFORMATION RECEIVED FROM GRCA.



JEWELL
ENGINEERING

WONDER
INDUSTRIAL PARK
PETER STREET
PORT HOPE

PROPOSED
DRIVEWAY ENTRANCE

DRAWN BY: JO	PROJECT NO: 220-5146
DESIGNED BY: EF/AMR	DATE: October 2022
CHECKED BY: AMR	SCALE: HORIZONTAL - 1:750 VERTICAL -
APPROVED BY:	CONTRACT NO:
DRAWING NO: EPA-1	

Figure 2: Cross Section with 100-Yr WSEL at Proposed Entrance

Note: The vertical pink line represents a levee used to limit the spill area to 40m west of the creek bank. This levee was added as a conservative measure and it is expected the future site plan design will ensure all flow obstructions (i.e. buildings, raised lands) are placed outside (north) of the levees applied in this model. No development beyond the site entrance is proposed at this time since the entrance will be used to facilitate the necessary studies to complete a site plan submission package.

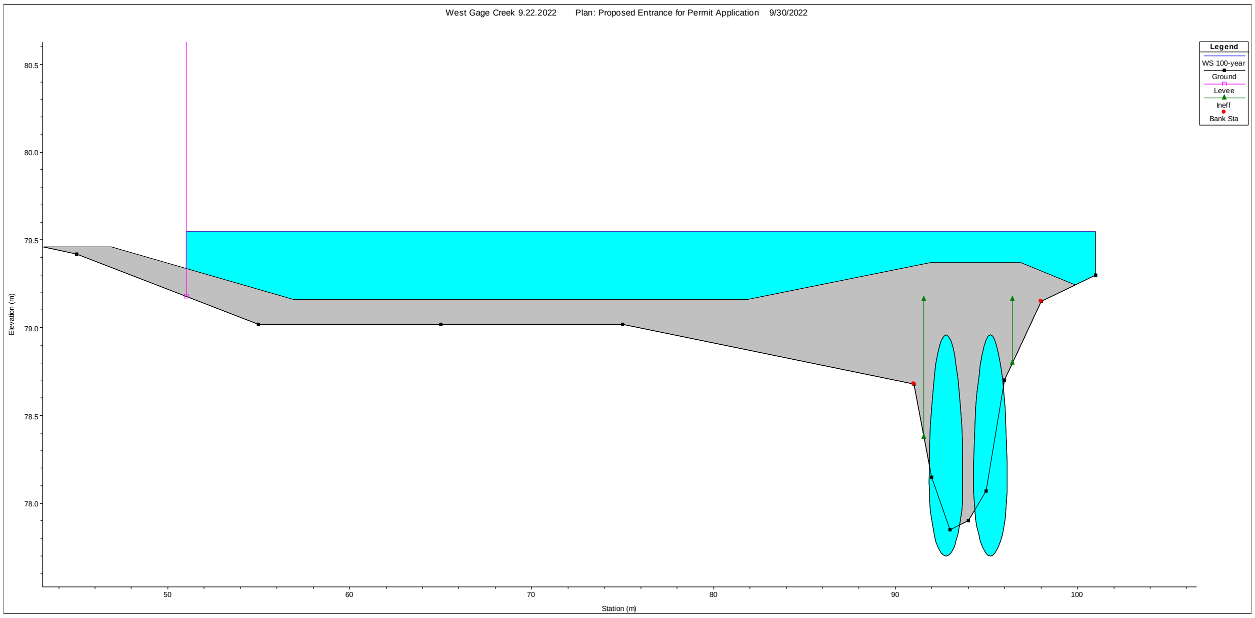


Figure 3: Comparison of 100-Yr WSELs for Existing vs. Proposed Conditions

Short ID Plan Descriptions for Table Below:

Updated existing conditions – short ID is **'NoAdded'** for no added culverts (i.e. existing conditions).
Proposed conditions – short ID is **'Pr.Ent.Permit'**.
Note: The River Station for the upstream (west) property limit is 1439. All WSELs upstream of RS 1439 remain unchanged.

River Sta	Profile	Plan	Q Total	W.S. Elev
			(m3/s)	(m)
1439	100-year	UpdatedEx.	24.63	79.91
1439	100-year	Pr.Ent.Permit	24.63	79.89
1431	100-year	UpdatedEx.	24.63	79.57
1431	100-year	Pr.Ent.Permit	24.63	79.61
1417			Lat Struct	
1403	100-year	UpdatedEx.	24.63	79.53
1403	100-year	Pr.Ent.Permit	24.28	79.59
1391	100-year	UpdatedEx.	24.63	79.52
1391	100-year	Pr.Ent.Permit	24.28	79.58
1359	100-year	UpdatedEx.	24.63	79.45
1359	100-year	Pr.Ent.Permit	23.81	79.55
1335	100-year	UpdatedEx.	24.63	79.42
1335	100-year	Pr.Ent.Permit	20.83	79.55
1314	100-year	UpdatedEx.	24.23	79.38
1295	100-year	UpdatedEx.	22.68	79.35
1295	100-year	Pr.Ent.Permit	20.83	79.32
1279	100-year	UpdatedEx.	19.81	79.34
1279	100-year	Pr.Ent.Permit	18.61	79.31
1255	100-year	UpdatedEx.	12.45	79.33
1255	100-year	Pr.Ent.Permit	11.44	79.30
1242			Culvert	
1230	100-year	UpdatedEx.	12.45	78.79
1230	100-year	Pr.Ent.Permit	11.44	78.77
1228			Lat Struct	
1210	100-year	UpdatedEx.	9.58	78.77
1210	100-year	Pr.Ent.Permit	9.17	78.75
1136	100-year	UpdatedEx.	8.69	78.72
1136	100-year	Pr.Ent.Permit	8.58	78.70
1125			Culvert	
1113	100-year	UpdatedEx.	8.69	78.48
1113	100-year	Pr.Ent.Permit	8.58	78.46
1090	100-year	UpdatedEx.	8.69	78.48
1090	100-year	Pr.Ent.Permit	8.58	78.46
1067			Culvert	
1048	100-year	UpdatedEx.	8.69	78.00
1048	100-year	Pr.Ent.Permit	8.58	78.00