

Stormwater Management Report

Cork & Vine Subdivision

(formerly Country Club Estates)

Wellington, ON

Revised

November 20, 2025



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Revision Summary

February 2022

- First submission SWM Report issued for detailed design.

September 6, 2022

- Responded to Quinte Conservation and peer review comments
- Added Phases 3A(East), 4 for draft plan of subdivision approval, including increased lot density.

November 20, 2025

- Updated SWM Report authored to support draft plan application.

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1 Introduction and Background

Kaitlin Corp. (Kaitlin) is proposing to develop the Cork & Vine subdivision in Wellington. Cork & Vine is comprised of the former Country Club Estates (CCE) in the east, and Fields of Wellington (FOW) in the west.

The Country Club Estates subdivision comprises approximately 111 ha of land in total, located in Wellington, Prince Edward County (see Figure 1-1). The development is situated north of the Millennium Trail and West of Belleville Street. Kaitlin received draft approval for a plan of subdivision for 460 dwelling units in 2012, which has since undergone a Redline Revision. Additional development is planned beyond the draft approved lots and the stormwater management plan includes provisions for this future development. The total proposed number of units in the former CCE development is now 2126.



Figure 1-1: Development Site Location (Google, Maxar Tech 2018)

West of CCE is another large draft-approved subdivision known as Fields of Wellington (FOW) for 690 residential units. The total proposed number of units in the former FOW development is now 803. Fields of Wellington is planned to be constructed concurrently with County Club Estates.

The stormwater management plan was developed following guidance from the previous Stormwater Management Report (Jewell, September 2022) and the Wellington Master Servicing Plan (RVA, 2021).

SWM Report (2022)

The 2022 SWM report was authored by Jewell Engineering Inc. (Jewell) to update the preliminary SWM report to reflect changes to the conceptual design of the development. The initial concept was a 460-unit subdivision surrounded by multiple golf courses, with stormwater quality and quantity control provided. This was updated to a 1565-unit subdivision with a single country club and golf course area, and only quality control of stormwater was required.

The concept has since been updated and further intensified, with the subdivision now providing 2126 units. The modelling was thus required to be updated to ensure the SWM plan was sufficient.

Master Servicing Plan

Prince Edward County completed the Wellington Master Servicing Plan in 2021, which reviewed the water and sewer infrastructure needs for the village of Wellington. Major upgrades to the sanitary sewage conveyance and treatment are needed to support the planned growth. Water treatment and distribution infrastructure will also be improved. The water and sanitary improvements were scheduled to be started in 2022 and be phased in over several years. Initial phases of the improvements will make accommodation for the Country Club Estates and Fields of Wellington developments.

The Master Servicing Plan (MSP) also considered opportunities for centralized stormwater management facilities and incorporated the preliminary stormwater management designs from each of the two developments. The MSP followed the Schedule B Class Environmental Assessment process and identified a central stormwater management facility should be positioned the Kaitlin lands north of Lane Creek.

Option N1 is the preferred option for the lands north of the Millennium trail, which includes the subject lands. The description of Option N1 from the MSP is included below:

Option N1 – All stormwater ponds (with quality and quantity control) will be directed to drain to Lane Creek via a series of new storm sewers. For development that approaches the northern most point of the Wellington Urban Boundary, the natural drainage of that area discharges to Hubbs Creek instead of Lane Creek. Depending on the development plans for those areas, stormwater pond(s) for quality and quantity control may be required prior to discharge to Hubbs Creek.

This stormwater management plan follows the Option N1 and was commissioned to review the potential drainage impact from the Country Club Estates development and to provide design support for the mitigation measures that must be employed. The proposed central facility for Country Club Estates will outlet directly to Lane Creek, which is considered a safe and preferred outlet for stormwater. With this in mind, Jewell also considered opportunities to receive

drainage from the Fields of Wellington subdivision for treatment in the central SWM facility and discharge safely to Lane Creek.

Lane Creek Flood Risk Mapping

Quinte Conservation and Prince Edward County, with assistance from Jewell Engineering, updated the flood risk mapping for Lane Creek in 2022. The hydrologic model of Lane Creek and draft flood risk maps were made available to guide the detailed design of the stormwater management infrastructure. The 100-yr flood elevation was found to be 84.4m upstream of the Millennium Trail crossing where a significant backwater and storage area is imposed by the undersized culvert crossing under the trail.

Quinte Conservation previously approved a plan to sculpt the land along the perimeter of the floodplain where the stormwater management facility will be located. The SWM facility will intrude into the floodplain, but a cut is proposed to minimize the loss of natural storage.

2 Existing Conditions

The development lands are mostly flat with a gradual ($< 1\%$) slope to the southeast towards Lane Creek (Figure 2-1). Lane Creek has a contributing area of 14.8 km^2 (1480 ha), and the contributing lands extend up to Huff's Corners at the intersection of Scoharie Road (County Road 1) and Hwy 62. The creek crosses Belleville St. and the Millennium Trail at the proposed development site before discharging to Lake Ontario through downtown Wellington (Figure 2-2).



Figure 2-1: Proposed Development Site (Google, 2021)

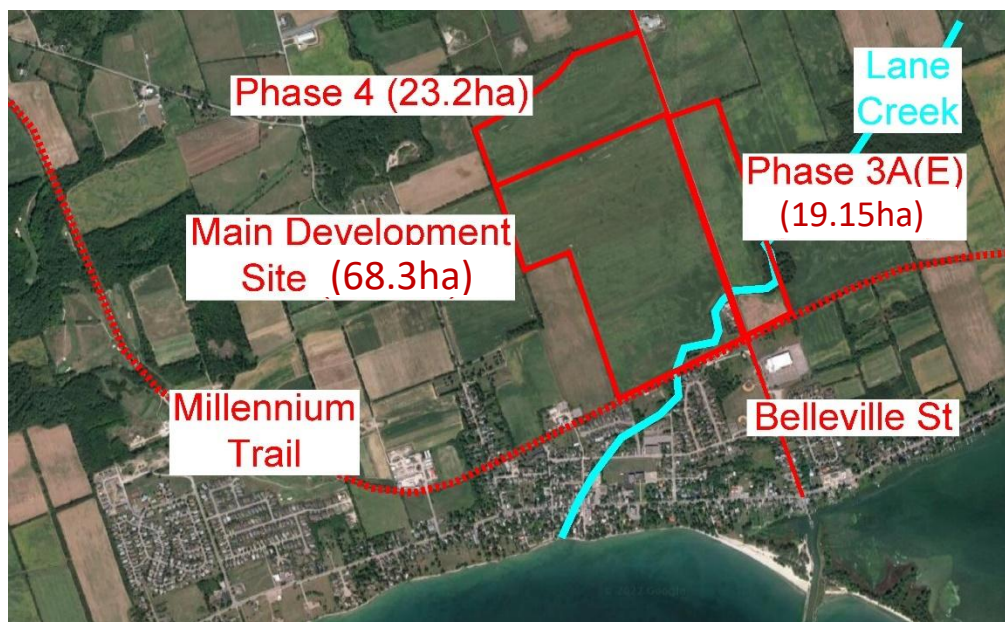


Figure 2-2: Site Overview (Google, Maxar Tech 2015)

2.1 Soils

Soils were reviewed from the Soil Survey of Prince Edward County (Dominion Department of Agriculture and the Ontario Agricultural College, 1948). The development lands fall mostly under hydrologic soils group (HSG) B, with some areas of HSG A and C. Figure 2-3 shows the soil distribution in the Village of Wellington.

Soils found on the site include:

- Brighton Gravelly Sand (Bg)
 - These soils have good natural drainage and the materials are well sorted with few stones. They are subject to erosion and have low fertility. Brighton gravelly sands are classed as hydrologic soil group A.
- Gerow Clay Loam (Gc)
 - The Gerow clay loam soils are poorly drained and shallow with one to three feet of soils over limestone bedrock. They are classed as hydrologic soil group C.
- Waupoos Clay Loam (Wc)
 - These soils vary in drainage, with some areas permitting good drainage and others with limited infiltration potential. These soils are classed as hydrologic soil group B.

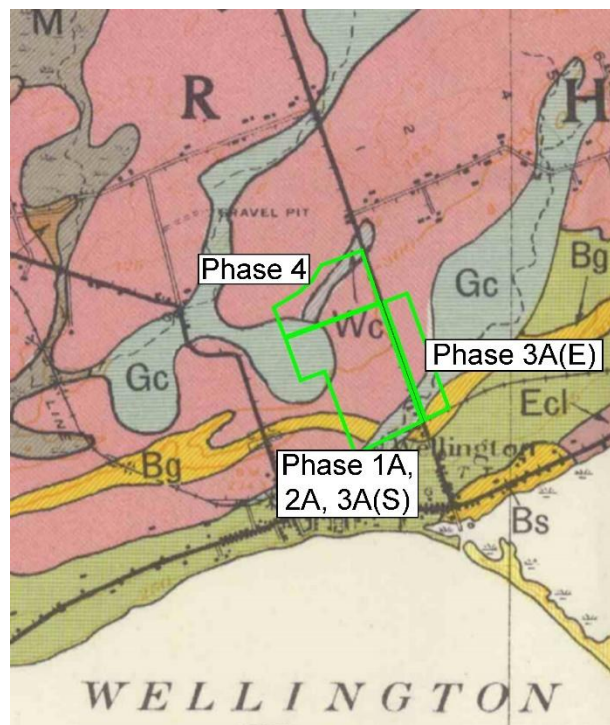


Figure 2-3: Soils Mapping from Soil Survey of Prince Edward County

Figure 2-4 (from the 1997 MTO Drainage Management Manual) outlines the classifications of Hydrologic Soils Groups.

Hydrologic Soil Group	
The hydrologic soil group is used to classify soils into groups of various runoff potential.	
The Soil Conservation Service (SCS) classifies bare thoroughly wet soils into four hydrologic soil groups (A, B, C and D). SCS descriptions of the four groups, modified slightly to suit Ontario conditions, are as follows: (Design Chart 1.09)	
A:	High infiltration and transmission rates when thoroughly wet, eg. deep, well drained to excessively-drained sands and gravels. These soils have a low runoff potential.
B:	Moderate infiltration and transmission rates when thoroughly wet, such as moderately deep to deep open textured loam.
C:	Slow infiltration and transmission rates when thoroughly wet, eg. fine to moderately fine-textured soils such as silty clay loam.
D:	Very slow infiltration and transmission rates when thoroughly wet, eg. clay loams with a high swelling potential. These soils have the highest runoff potential.
In Ontario, soils have been found to lie between the main groups given above, and have therefore been interpolated as AB, BC, CD as appropriate, such as Guelph loam, which is classified as BC.	

Figure 2-4: Soils MTO Drainage Management Manual – Description of Hydrologic Soils Groups

2.2 Targets

The stormwater management targets are developed with reference to provincial, municipal and agency guidance documents. Jewell also consulted with Quinte Conservation to discuss the stormwater management objectives for the development.

The stormwater management plan focusses on three environmental objectives when considering the treatment and conveyance of stormwater runoff. The objectives are to mitigate impacts to flooding, water quality, and erosion of the receiving system.

The development contributes to the lower reach of Lane Creek. When a development is proposed in the lower reaches of a system, the hydrology of the system should be reviewed for the potential of superposition of peak flows. That is, the timing of peak flows from the development should be compared to the timing of the peak flow from Lane Creek.

Traditional stormwater management measures store surplus runoff and release outflows at a lower rate. This results in a delay of peak flows from the development and shifts the time of the peak closer to the time to peak from the watershed. Peak flows are additive and may result in flooding in the creek even though peaks leaving the developing area may be controlled to pre-development levels.

Stormwater management for quantity control in the lower reach of a system can then be counterproductive and result in flooding. In such situations, no quantity control may be appropriate.

The stormwater management plan should ensure there will be no increase to flooding in Lane Creek and that water quality impacts are mitigated. Since Lane Creek has flood plain mapping, Jewell was able to compare modelled water surface elevations using the hydraulic models, and by comparing peak flows.

Consequently, the stormwater management plan for Cork & Vine includes the following objectives:

Quantity Control

- Peak flows in Lane Creek are not increased.
- Water surface elevations are less than or equal to the flood plain mapping.

Quality Control

- Runoff to Lane Creek meets **Enhanced** quality control objectives.

Sediment and Erosion Control

- Minimize the potential for erosion of soils and Lane Creek,
- Mitigate the release of sediment offsite.

2.3 Consultation

Quinte Conservation expressed concerns about the initial peak (produced by uncontrolled runoff from the development) before the peak in Lane Creek. The pre-development peak in the creek is 14.18 m³/s. The uncontrolled peak discharge in the creek from the development lands (the first peak) is below 8 m³/s. The existence of this peak is preferred over providing quantity control for the following reasons:

- Lane Creek can pass the early peak without surcharging the culverts. Potential for damage would be indicated by surcharged culverts. Therefore, the SWM concept is not expected to result in damage.
- The water surface elevation and velocity in a watercourse increases proportionally to the flow through the creek. Quantity control will delay the site's runoff and will *add* to the existing peak. This violates the SWM objectives.
- Adding hydrographs in hydraulic and hydrologic models results in peaks that are less likely to occur in reality. Variation in catchment and channel topography and geometry will result in the skewing of a hydrograph peak over time.

The hydrographs from the development lands and in Lane Creek are shown in Section 4.3.

3 Proposed Conditions

Two stormwater management facilities are proposed. Development of the following lands was considered in the SWM design (refer to development concept plan and catchment area drawing in Appendix B):

- Phase 1A, 2A, and 3A(South) 65.1ha
- Fields of Wellington 10ha
- Phase 3A(East) 12.2ha
- Phase 4 23.2ha

The first facility will be situated near the south edge of the development lands, and was allocated drainage from Phases 1A, 2A, 3A(South), and a 10ha portion from Fields of Wellington, with a total contributing area of 75.1ha. A 3.20ha portion of land south of the creek and north of the Millennium Trail will not be developed and will drain uncontrolled to Lane Creek.

The second facility will be situated east of Belleville Street and will treat runoff from the developed area of Phase 3A(East) to its north, which has a contributing area of 12.2ha (including the 1ha SWM facility). Runoff from the 6.95ha of land in the south of Phase 3A(East) dedicated to parks, natural area and floodplain will drain uncontrolled into Lane Creek, and is accounted for in the lands outside the development contributing to the Belleville St crossing.

Phase 4 runoff drains northwards into Hubbs Creek. These lands are discussed separately in Section 5.

3.1 Drainage Scheme

The drainage scheme follows the major/minor strategy, with minor flows conveyed by storm sewers and major flows routed overland via fully urbanized roadways with curb and gutter. Drainage from rear yard areas will be collected by grassed swales and catch basins and contribute to the minor system, with flows that exceed the minor system capacity spilling to the road network to contribute to the major system. Road grades are established to facilitate the stormwater conveyance ensuring spill points limit potential ponding depths to less than 0.3m.

The minor system will be designed to convey runoff from the 5-year Trenton storm event. The major system will be designed to convey runoff from the 6-hr SCS distribution of the 100-year Trenton storm event that was used for the 2022 flood risk mapping.

The contributing areas for both stormwater management facilities are in excess of 10ha, which indicates either a wet pond or wetland style of facility will be required for quality treatment per Table 4-1 and 4-6 of the Stormwater Management Planning and Design Manual. For quality treatment, the storage volume is sized per Table 3-2 of the guidelines, with the active storage volume sized as the greater of the 40m³/ha or 25mm 4-hr Chicago event for erosion protection. Only the 25mm event is planned to be passed through the facilities for treatment, and runoff in excess of the 25mm event can be bypassed directly into Lane Creek.

Table 3-1 details the nodes of interest of Lane Creek relevant to the drainage scheme in pre-development conditions.

Table 3-1: Lane Creek Nodes of Interest – Pre-Development Drainage

Node	Contributing Area (km ²)	Location
C	13.97	Belleville St crossing, approx. 1.3km upstream of Node A
B	14.56	Millennium Trail crossing, approx. 800m upstream of Node A
A	14.81	Discharge point, on Lake Ontario between West St and Wharf St

3.1.1 East Facility

The east facility receives drainage from the developed area of Phase 3A(East). The major/minor drainage strategy is followed. Major system flows will flow directly to Lane Creek, and flows conveyed by the minor system greater than the 25 mm event will be diverted through a bypass structure to prevent resuspension of solids in the SWMF. The drainage from Phase 3A(E) contributes to Lane Creek at Node C (Belleville St).

The facility has been preliminarily sized to support the current submission. The facility will be fully constructed and operational in Phase 3A of development. Sizing of the facility is discussed further in Section 4.

3.1.2 Main Facility

The main facility receives drainage from Phases 1A, 2A and 3A(south) as well as 10ha of Fields of Wellington lands. Again, the major/minor drainage strategy is followed. The difference between the 5-yr and the 25 mm runoff can bypass the facility via a flow splitter, and major system flows can also bypass in several locations into Lane Creek. Treated flows from the facility are discharged to Lane Creek upstream of Node B.

The detailed design of the facility has been completed and approved, and the facility will be fully constructed and operational in Phase 1A of development. The facility design is discussed further in Section 4.

3.2 Hydrologic Modelling

An OTTHYMO model was prepared to review the hydrologic conditions of the existing lands and compare with the post development conditions to determine the mitigation measures needed. As discussed earlier, the impact to Lane Creek was also reviewed with comparison to the hydrograph of Lane Creek. The hydrologic model was based on the 6-hr SCS 100-yr event that was also used for the 2022 flood risk mapping.

OTTHYMO is a single event hydrologic model that allows a user to generate a runoff hydrograph from simulated catchments using limited input data. The program will add, route, divert, store and delay hydrographs generating several runoff data parameters such as peak flow, time to peak, volume and runoff coefficient.

The user may simulate a catchment by selecting from an urban (Standhyd) or a rural (Nashyd, Wilhyd or SCShyd) unit hydrograph depending on the degree of development measured as imperviousness. The modelling software allows users to simulate catchments using parameters such as:

- catchment area
- imperviousness, total and directly connected
- runoff coefficient or curve number
- length and slope of impervious area
- length of and slope of pervious area
- time to peak of the catchment
- Manning's roughness coefficient
- storage coefficient

Land use information and watershed characteristics were matched to previous floodplain modelling by Crysler & Lathem and current floodplain modelling by Jewell. The parameters were used to develop input parameters for the OTTHYMO hydrologic model.

Precipitation inputs may be provided using precipitation gauge statistics from Intensity Duration Frequency curves, such as those provided in Appendix A, or it may be an historic storm read in from an external file. IDF data was obtained from the Trenton station, and the volume of runoff was increased from 80mm to 84.7 mm to account for climate change (50-year future projection). The hydrologic model estimates how much of the precipitation input is translated to output (runoff) using data supplied about the catchment. The volume of water, measured as depth of water in mm, is conserved throughout the model as per the equation below.

$$\text{Rainfall depth} = \text{Runoff depth} + \text{Losses}$$

Losses include those through interception depression storage (Initial Abstraction) and infiltration determined using one of three infiltration methods; SCS curve number (CN),

proportional loss coefficient (C), or Horton's Infiltration equation. Diversions are another means of subtracting volume from the model output (a loss). These can be imposed by the user with the DUHYD or Divert HYD command. Evaporative losses are ignored.

Peak flows are generated in the Nash unit hydrograph method assuming the runoff is routed through a series of linear reservoirs (the number of which is selected by the user). In the Standard unit hydrograph method, the impervious areas are separated from the pervious and hydrographs are derived for each and added to develop a catchment-wide hydrograph.

Storage, lags or routing through channels do not subtract from the runoff volume, but result in delays of the times to peak and can lead to changes in the peak flow, typically by reducing the peaks. Most stormwater management objectives are focussed on the magnitude of the peak flows, while some may be interested in volume of runoff.

The catchment area inputs to the OTTHYMO model (not including catchments included as part of the floodplain modelling) are summarized in Table 3-2 and Table 3-3. The catchment drawing is included in Appendix B.

Table 3-2: Catchment Area Hydrologic Inputs (Standhyd)

Catchment	Area (ha)	XIMP (%)	TIMP (%)	CN	Length (m)
301	11.2	65	75	73	273
302	1.0	40	50	73	82
201 + FOW	75.1	40%	55%	73	708

Table 3-3: Catchment Area Hydrologic Inputs (Nashyd)

Catchment	Area (ha)	CN	Tp (hr)
303	3.4	73	0.31
202	3.2	73	0.31

The pervious and impervious lengths in the model were calculated using the following formula:

$$L = \sqrt{\frac{10,000 * A}{1.5}}$$

Where:

L = length of pervious and impervious cover in metres

A = area of the contributing lands in hectares.

The time to peak was calculated as 2/3 of the time of concentration, which was calculated using the Airport Method formula below:

$$T_c = \frac{3.26 * (1.1 - C) * L^{0.5}}{S_w^{0.33}}$$

Where:

T_c = Time of concentration in min
 L = Length of Watershed in metres
 C = Runoff Coefficient
 S_w = Watershed Slope in %
 A = Watershed area in hectares

The slope of each drainage area was determined using 85/10 Average Slope Method per Equation B2.2 of the MTO Drainage Manual.

The hydrograph from the Phase 3A(E) lands was shifted by 5 minutes to account for routing through Lane Creek.

The 100-yr peak flow from the development is 7.9 m³/s (runoff from Phase 3A(East), Country Club Estates, and Fields of Wellington contributing to the Node B), and the time to peak is at 3:45. The pre-development peak flow from Lane Creek is 14.18 m³/s between Belleville Street and the Millennium Trail. The time to peak of the greater watershed is 10:35 indicating a very clear opportunity to adopt a 'beat the peak' approach to quantity control. The detailed OTTHYMO model output is provided in Appendix D.

Figure 3-1 shows the relationship of the magnitude and timing of the peak flows from the development with that from Lane Creek. The blue line shows the uncontrolled post-development runoff from the development lands. The orange line shows the hydrograph of Lane Creek at Node B (Millennium Trail crossing).

It is noted that there is a large separation of time between the peaks of the two inputs. Also, the hydrograph from the development has a sharper peak and is of shorter duration. This demonstrates an opportunity for 'beat the peak'.

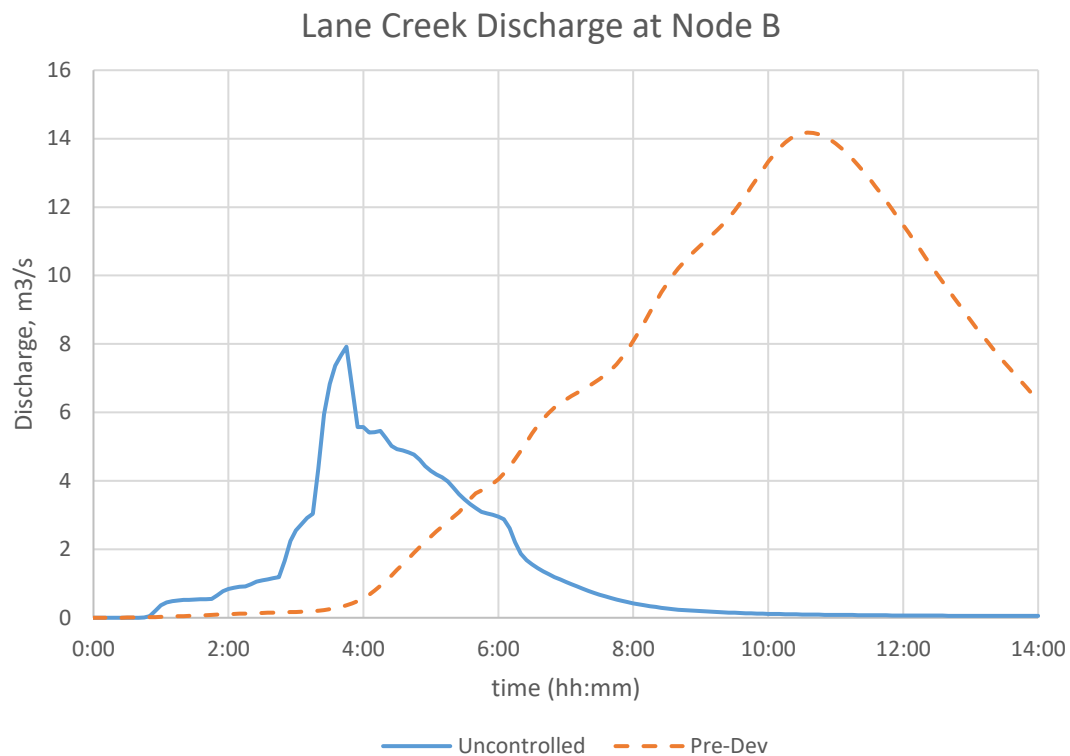


Figure 3-1: Peak Flow Comparison, Pre-Dev vs. Uncontrolled Discharge at Node B

4 Stormwater Management Controls

4.1 East Facility

The wet pond facility was preliminarily sized (see Appendix C) assuming 12.2 ha of developed area would contribute to the Phase 3A(East) facility for treatment – including the 1ha facility itself. The imperviousness of the 1 ha SWM facility was determined as 50%, and the imperviousness of the 11.2 ha of developed land north of the facility was calculated as ~72%, modelled as 75% for conservatism. As the facility design will be finalized in the detailed design of Phase 3A(East), the preliminary sizing is conservative to account for possible changes to the development concept.

The water quality storage requirements specified by Table 3-2 of the Stormwater Management Planning and Design Manual are as follows:

- Permanent pool – 2,266 m³ (185.7 m³/ha)
- Extended detention – 488 m³ (40 m³/ha)
 - Depth of 0.50 m

However, as the required extended detention storage for the 25 mm erosion event was shown to be 1,593 m³ through OTTHYMO, the 25 mm event governed sizing. The facility's sizing is summarized in Table 4-1.

Table 4-1: Phase 3A(E) Preliminary Pond Sizing

Feature	Elevation	Storage (m ³)		
		Required	Provided	Used
Bottom of Pond	84.10	-	-	-
Permanent Pool	85.60	2,266	2,773	-
Extended Detention	86.15	488	1,679	1,593
Top of Berm	86.60	-	3,354	-

The permanent pool elevation was set to 85.60 to reflect the 100Yr flood line elevation upstream of the Belleville St crossing (Node C).

A 0.17m diameter orifice is proposed with invert at the elevation of the permanent pool, which will discharge at a rate of 0.036m³/s at the 25mm event elevation of 86.05m. A 3.5m broad-crested weir is proposed with invert at 86.15m to convey flows exceeding the 25mm event. This structure has a peak flowrate of 1.76m³/s (exceeding the incoming minor flow of 1.04m³/s). In other words, the facility freeboard of 0.3m ensures sufficient capacity to bypass any incoming

flows to the facility from the upstream bypass structure. The facility's controls are summarized as follows.

- | | | |
|-------------|---------------|---------------|
| • Control 1 | 0.17m orifice | Invert 85.6m |
| • Control 2 | 3.5m weir | Invert 86.15m |

A water quality facility must also provide a drawdown time of between 24 hours and 72 hours. The pond drawdown from the 25mm event storage is 26 hours, therefore the pond meets the quality objective of providing **Enhanced** (Level 1) quality treatment to contributing runoff.

4.2 Main Facility

The wet pond facility was sized (see Appendix C) during detailed design assuming Phase 1A, 2A, 3A(South), and 10ha of FOW would contribute to the facility for treatment. The 75.1ha total treatment area was assigned an assumed imperviousness of 55%.

The SWM design remains sufficient as, despite the increased intensification of the development, these initial assumptions remain conservative for the following reasons:

- The ~6.5 ha of land dedicated to parks, natural area and floodplain adjacent to the facility will drain uncontrolled into Lane Creek.
- 9.35 ha of FOW lands will contribute to the facility, whereas 10 ha was modelled.
- A small catchment of the south end of Street A cannot be directed into the facility and instead will be treated through an OGS unit.
- The same 75.1ha contributing area in the revised concept has an impervious cover of ~48%, and accounting for the above non-contributing areas results in an impervious cover of ~53%; both scenarios remain below the design imperviousness of 55%.

The water quality storage requirements specified by Table 3-2 of the Stormwater Management Planning and Design Manual are as follows:

- Permanent pool – 11,265 m³ (150 m³/ha)
- Extended detention – 3004 m³ (40 m³/ha)
- Depth of 0.50 m

However, as the required extended detention storage for the 25 mm erosion event was determined as 4,868 m³ through the OTTHYMO model, the 25 mm event governed sizing. Jewell determined storage requirements for the facility with orifice and weir sizing. The storage volumes required for the 100Yr, 6hr event and those provided in the facility are summarized in Table 4-2.

Table 4-2: Main SWMF Sizing

Feature	Elevation	Storage (m ³)		
		Required	Provided	Used
Bottom of Pond	83.00	-	-	-
Permanent Pool	84.70	11,265	12,214	-
Extended Detention	85.20	3,004	5,188	4,868
Top of Active Storage	85.70	-	11,093	10,443
Top of Berm	86.00	-	14,963	-

The wet pond facility has a forebay that has been sized following the sizing calculations from the design manual. The forebay is separated from the main cell with a submerged weir that permits incoming flows to pass through the forebay into the main cell. For the initial filling and periods of ice cover, two 600mm HDPE culverts in the forebay berm will help to convey flows into them main cell. The pond side slopes are 5:1, which is considered by many jurisdictions to provide sufficient public safety that fencing would not be required.

Two outlets provide the control of discharge. These include a 0.60 m diameter orifice that provides the water quality drawdown for the 25mm event. Flows that exceed the water quality event will pass through a 4.25 m rectangular weir. The controls are as follows:

- Control 1 0.6m orifice Invert 84.7m
- Control 2 4.25m weir Invert 85.2m

A water quality facility must also provide a drawdown time of between 24 hours and 72 hours. The pond drawdown from the 25mm event storage is 25 hours, therefore the pond meets the quality objective of providing **Enhanced** (Level 1) quality treatment to contributing runoff.

The facility will be subject to temperature effects that will tend to warm the water. Some features that help to reduce the warming include a deeper permanent pool, reverse slope outlet pipe that discharges stormwater from the deeper zones, and shading by trees and other vegetation.

A landscape plan has been developed for the site indicating that trees will be planted around the east, west, and south of the SWMF. The landscape plan for the facility has been included in Appendix I.

A perimeter pathway around the facility is also provided for access to maintenance staff.

4.2.1 Flow Splitter Weir

It is not intended for events larger than the 25mm event to be routed through the pond. Therefore, a flow splitter is provided to split off flows larger than the 25mm event from the

storm sewer system and bypass these to Lane Creek. The 25mm run of the OTTHYMO model results in a peak flow to the pond of $3.04 \text{ m}^3/\text{s}$. The Rational Method in the storm sewer design sheet indicates an expected peak flow in the storm sewers of $4.97 \text{ m}^3/\text{s}$ during the 5-yr event. Therefore, a splitter weir is required to bypass portions of flow to the pond, limiting the peak inflow to $3.04 \text{ m}^3/\text{s}$.

The splitter weir is proposed to be constructed by a 2.4m weir inside a manhole structure. The weir will allow $1.93 \text{ m}^3/\text{s}$ of flows to pass over (at a head of 0.62 metres) while a peak of $3.04 \text{ m}^3/\text{s}$ will be directed to the facility. Calculations are included in Appendix C.

4.3 Modelling Results

The OTTHYMO model was completed for the 25mm event and the 100-yr event.

The 25mm event simulated was the 4-hr Chicago distribution event as recommended in MOE 2003. The 100-yr event simulated was the 6-hr SCS Type II with the MTO IDF curves. As discussed in Section 3.2, the 2072 climate change hydrograph was used for the modelling to be consistent with the draft flood risk mapping for Lane Creek. The peak flow results in the 100-yr event are included in Table 4-5. It is found that the uncontrolled, unrouted 100-yr peak flow from the development is $7.9 \text{ m}^3/\text{s}$, which is noticeably less than the peak flow in the Lane Creek Watershed. There is also a 7-hour separation in the times to peak of the development and the watershed.

Prior to development, the peak flow in the creek is $14.18 \text{ m}^3/\text{s}$. With the development removed from the peak, the uncontrolled peak in Lane Creek is reduced slightly to $13.87 \text{ m}^3/\text{s}$. The stormwater runoff from the 25mm event is routed through the pond while flows exceeding are diverted directly to Lane Creek. This is modelled in OTTHYMO using the DUHYD command with the split set to the 25mm peak inflow of $3.038 \text{ m}^3/\text{s}$. The split is accomplished in the design by a combination of a flow splitter upstream of the facility as mentioned in Section 4.3, and by directing the major runoff to the creek through the road network.

The addition of the uncontrolled flows and the pond discharge results in a combined peak flow of $14.17 \text{ m}^3/\text{s}$, which is less than the existing conditions peak flow in the creek. The floodplain limitations are not exceeded and the quantity control target is achieved. See Table 4-3.

Figure 4-1 shows the pre- and post-development hydrographs at the Belleville St crossing, as well as the runoff from Phase 3A(E). Figure 4-2 shows the hydrographs at Node B (the Millennium Trail crossing), as well as the hydrographs from Cork & Vine (former CCE and FOW). As the post-development peak flows are lower than the pre-development flows, the quantity control target is met.

Table 4-3: Pre- and Post-Development Times to Peak

Description of Flow Condition	Peak Discharge (m ³ /s)	Time to Peak (hh:mm)
Node C – Belleville St		
Pre-Development (Node C)	14.04	10:20
Without Development Lands (Node C)	13.93	10:20
Development Only – Phase 3A(East)	1.67	3:45
Post-Development (Node C)	13.95	10:20
Node B – Millennium Trail		
Pre-Development (Node B)	14.18	10:35
Without Development Lands (Node B)	13.87	10:40
Phase 3A(E) SWMF Flows (with routing)	1.67	3:50
CCE + 10.0ha FOW	5.21	3:45
Fields of Wellington	1.77	4:35
Post-Development (Node B)	14.17	10:35

The hydrograph for the Fields of Wellington development was supplied by Forefront Engineering.

Pre-development peak flows from the development are hard to determine as the peak flows were calculated as a watershed. Modeling the pre-development conditions using a NASHYD command would produce inaccurate results that are not an accurate representation of the pre-development conditions.

The initial peak to Node B briefly rises to 6.83 m³/s before remaining below 6.48 m³/s (at which point the pre-development flows exceed the post-development levels). These flows are conveyed through the Millennium Trail crossing at an elevation of 83.50m. This elevation is 0.90m below the 100Yr flood line elevation at the crossing (84.40m). As the highest incremental storage volume occurs near the flood line elevation (where the storage area is the greatest), the initial peak will have no significant impact on the flood storage upstream of the Millennium Trail crossing.

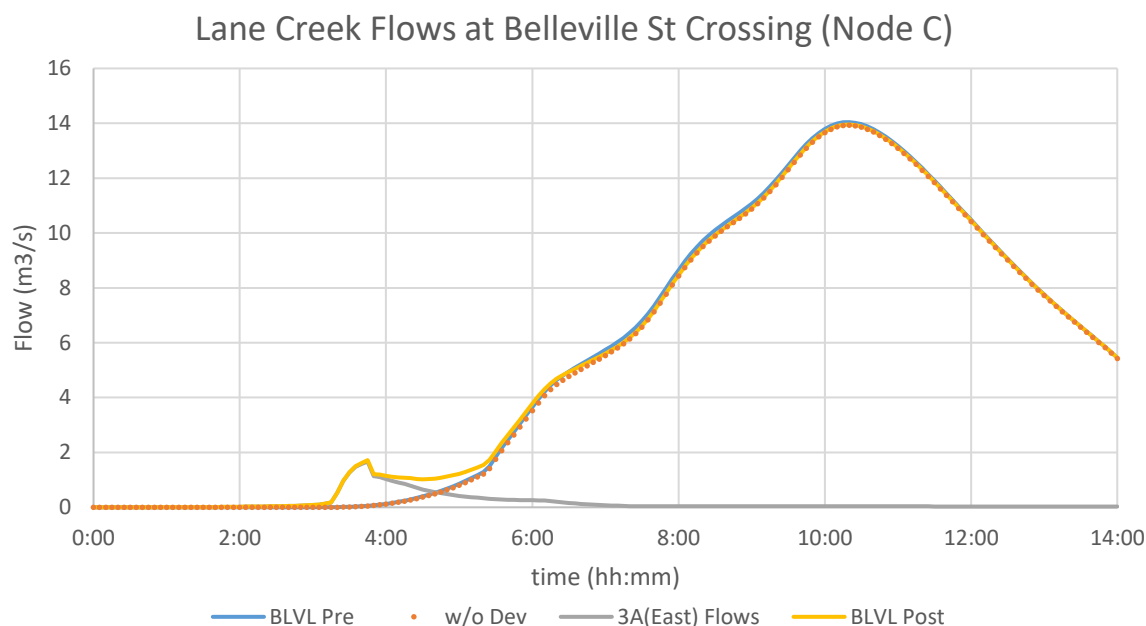


Figure 4-1: Flows at Node C (Belleville St Crossing)

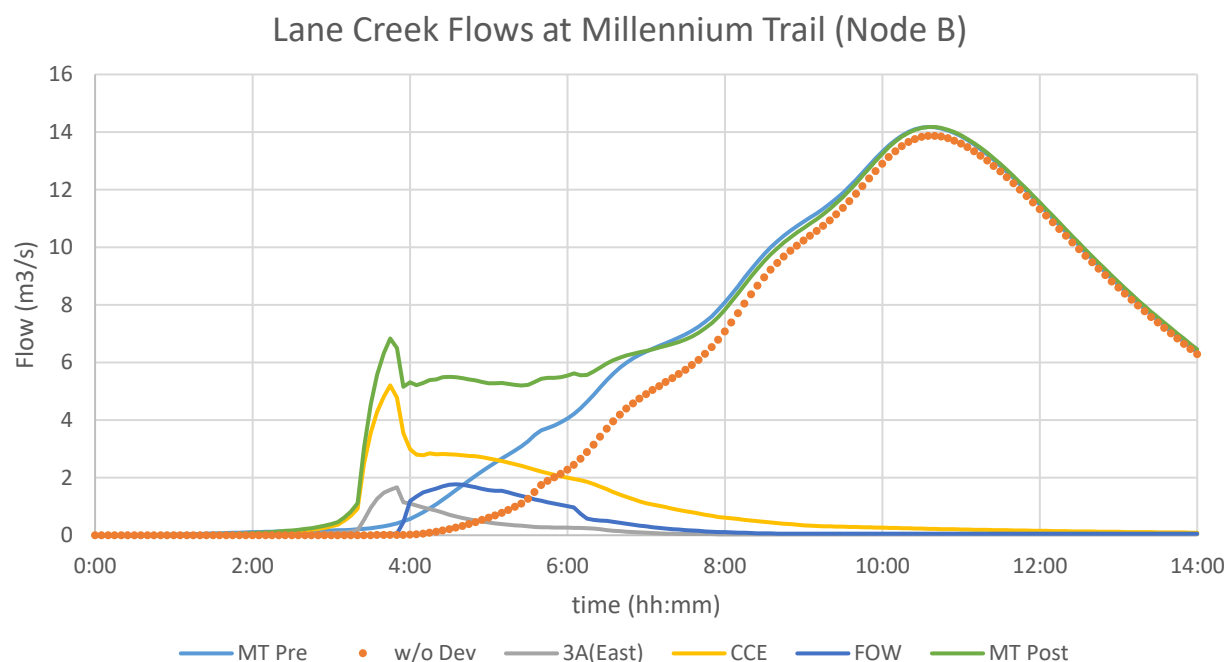


Figure 4-2: Flows at Node B (Millennium Trail Crossing)

The hardening of the development lands shifts the mass of the hydrograph earlier in the storm event, and the selected SWM approach spreads the volume over time.

4.4 Minor System

The storm sewer system was analyzed with a spreadsheet following the Rational Method. The Rational Method is the accepted design tool for sizing storm sewer systems (MOE, 2008). The 5Yr design event was completed using the IDF parameters from Trenton station 6158875. The storm sewer design sheet is included in Appendix E.

The Rational Method equation is shown below. It relies on an estimation of runoff coefficient, flow intensity, and drainage area.

$$Q = \frac{1}{360} CiA$$

Where:

Q = Peak Flow in m³/s

C = Runoff Coefficient

i = Rainfall Intensity in mm/hr

A = Area in hectares

4.5 OGS Sizing and CB Shields

The site grading will result in a 0.80ha catchment along Street A that will not be able to drain to the pond. CB shields and an OGS unit will be connected in series to provide quality treatment to flows. Technologies can be connected in series to provide greater treatment than a single technology alone, and the use of different technologies provides greater opportunity to remove various types of contaminants (for example, hydrocarbon removal in the OGS Unit that the CB Shields can not effectively remove).

Jewell applied a conservative particle size distribution (PSD) in the sizing of the OGS units. This PSD is identified as the NJDEP/ETV distribution and is used for areas targeting high treatment levels. With the NJDEP/ETV PSD, OGS unit sizes typically increase, and finer sediment particles are captured. The benefits of larger OGS units include greater removal rates, greater sediment storage capacities, and greater treatment flow capacities.

Jewell selected an FD-4HC unit to provide 72% TSS removal. The OGS sizing is included in Appendix F.

Table 4-4: OGS Sizing

Catchment	Contributing Area (ha)	R.C.	Unit Selected	TSS Removal	Volume Treated
1	0.80	0.60	FD-4HC	72.0%	>90%

The TSS removal equation, from the NCDENR Stormwater BMP Manual (2007) is as follows:

$$E = A + B - \frac{AB}{100}$$

Where A and B are the TSS removal percentages of the individual technologies. The CB Shields provide a TSS removal of 50%, and the FD-4HC will provide 72% TSS removal with a total volume treated in excess of 90% (see Appendix F for OGS sizing details). The combined TSS removal will be

$$E = 50 + 72 - \frac{50 * 72}{100} = 86\%$$

Therefore, the **Enhanced** quality control objective is met by use of the proposed technologies.

5 Phase 4 SWM Design

Phase 4 of the CCE development consists of approximately 23.2ha of land immediately north of Phase 2A. In the current design, Phase 4 will consist of a club house, golf course, tennis courts, pool, and two hotels, with accompanying parking facilities.

The lands belong to HSG B (Hillier Clay Loam), with some portions in HSG C (Waupoos Clay). The undeveloped lands are currently agricultural fields with a relatively flat slope (0-5%).

5.1 Quantity Controls

Due to the high imperviousness in the east portion of Phase 4, quantity controls are proposed in the form of parking lot, rooftop, and/or underground storage. While approximately 18.2ha (the approximate area of the golf course) remains pervious in the post-development conditions, 5ha to the east will largely be hardened.

As a preliminary example, it is possible that the runoff coefficient increases from 0.37 in the pre-development conditions (as indicated by Jewell's original OTTHYMO model) to 0.98 post-development. The increase in runoff volume during the 6-hour, 100Yr event will be as follows:

$$V = CdA = (0.98 - 0.37) * 72mm * 5.0ha = 0.61 * 0.072m * 50,000m^2 = 2196m^3$$

The volumetric increase of runoff as a result of Phase 4 development is expected to be approximately 2,200 m³ based on the preliminary estimate above. A portion of the increased volume will need to be stored on site to ensure the post-development peak for Phase 4 remains at or below the pre-development target, and this storage will be designed at a future point when Phase 4 proceeds for detailed design.

5.2 Quality Controls

Quality controls for Phase 4 may proceed using some or all of the following technologies:

- Oil and Grit Separator (OGS) unit
- Underground storage (StormTech units)
- Enhanced swales
- Vegetated buffer strips

A treatment train approach may be the optimal choice for quality treatment of Phase 4 as a large portion of the site remains pervious in the post-development conditions.

6 Maintenance

The stormwater management features adopted by this plan include:

- 1) Oil Grit Separators
- 2) CB Shields
- 3) Wet Pond SWMFs

During the first few years of operation, the developer will retain the responsibility of maintenance and will gain the experience of how the technologies perform at Cork & Vine. It will be important for this knowledge to be transferred to the municipality who will be the ultimate owners and operators of the facilities. The OGS water quality treatment device will help to focus sediment removal activities in locations that are accessible by the municipal road allowance. Sediment will collect in underground structures that may be easily removed by vacuum trucks.

6.1 OGS Units

The OGS units will capture sediment and floatable contaminants (oils and debris). The units should be inspected after the first full year of operation. If the inspections reveal that the remaining oil or sediment storage capacity is less than 50% the unit should be cleaned using a vacuum truck. The frequency of inspection should remain once per year until the development is built out and a predictable frequency of sediment or oil removal is reached. Further operations and maintenance details are included in Appendix G.

6.2 CB Shields

CB Shields are simple devices that can be removed and replaced by field staff after lifting the structure lid. The device is lifted out of the catch basin using a pole to facilitate sediment removal and is replaced in a similar fashion. Since the units enhance sediment capture within the catch basin sumps, the operators should anticipate increased sediment collection in the sumps that are fitted with the devices. The units should be inspected annually, and accumulated sediment should be removed by vacuum trucks if the sumps are greater than 50% full. The inspection and removal frequency may be adjusted based on experience.

Further operations and maintenance details can be seen in Appendix H.

6.3 Wet Pond SWMFs

Proper maintenance is important to ensure the proposed SWMF functions as designed. Some simple maintenance practices are recommended that include both surveillance and general cleaning/trash removal. Some maintenance activities, such as sediment removal, take place at very infrequent intervals. The ability of the wet pond to collect sediment will diminish over time as the volume of sediment storage accumulates. Major maintenance of a facility is triggered when the effectiveness of TSS removal is reduced by 5%.

6.3.1 Routine Maintenance

Once per month the pond operators should perform a visual check including observations of:

- trash or debris collecting in the pond
- water level between events (comparing with expected levels)
- signs of leaks or material loss
- presence of public safety measures (ensuring they are still in place)

Pond operators should remove any trash that may be impeding the pond outlet structures. Additionally, grass and weeds should be cut as needed. During and after a large rainfall event the operator should also perform a visual check to see that pond elevations are within expected levels. These are described below in the Pond Features section.

6.3.2 Infrequent Maintenance

The pond will collect sediment in proportion to the construction activity or winter road maintenance of the upstream catchment area. Vehicle access will be provided to the forebay area to allow pumping to remove the water and sediment.

6.3.3 Troubleshooting

Some basic issues that can develop with a pond and the remedies are described below.

Symptom – Pond is not emptying

The outflow pipe may become blocked with debris and should be monitored after every large runoff event. Observe that the pond is not overfilling and that it is emptying out between events. Full storage for the 100-yr event should be:

- 85.65m for the main SWMF
- 86.43m for the Phase 3A (East) SWMF

Orifices have been selected to be as large as possible to allow smaller debris to pass through.

Symptom – Pond does not fill

The orifice size should impose ponding during large runoff events. If the pond does not hold water during large events check to see the orifice plates have not been tampered with or removed.

Symptom – Pond routinely overfills

If the stored water discharges through the emergency spillway, the cause is blockage of the orifice plate. The orifice plate should be checked for blockage by debris and cleaned if necessary. Grating on outlet pipes may also clog with vegetation and can also be a cause of poor outflow. The grating may be cleaned by raking.

7 Erosion and Sediment Control

Typical site development requires removal of some vegetated cover. While it is the intention to limit vegetation removal when possible, exposed soils from the work will be at risk of eroding into the receiving drainage system. Measures will need to be put in place to reduce erosion during construction, and for a period of up to one year after construction is completed. Typical sediment and erosion control measures include:

- Siltation fencing.
- Strawbale check dams.
- Rip-rap check dams.
- Filter sock inserts in catch basins.

Controls are to be placed downstream of all active work areas and upstream of protected receivers. Controls should also be placed around stockpiles of topsoil and fill materials.

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 (with slope 10% or less).

The site developer and contractor should actively maintain the new drainage works to remove accumulations of sediment within catch basin sumps.

A silt fence should be located along the upland perimeter of all sensitive features during the construction process, which should be maintained until the lands have stabilized or as directed by the municipality. There would be benefit in maintaining this silt fence for up to 2 growing seasons.

8 Cut/Fill Analysis

The proposed development will encroach into the floodplain of Lane Creek where some loss of flood storage will occur. This follows previously approved principles for the development. The objective was a 1:1 replacement of lost flood storage with new storage. This is the balanced cut/fill approach.

The cut/fill was supported with calculations based on the 1979 flood lines. Jewell also previously found the proposed Cork & Vine development would not directly impact flooding and peak flows (see Jewell 2020 SWM Brief in Appendix J). This brief was superseded by the floodplain analysis, however it provides confidence to the investigation's results.

During finalization of the Cork & Vine detail engineering, a new floodplain mapping study was commissioned that resulted in a 0.4m increase in flood elevation upstream of Millennium Trail from 84.0m to 84.4m. Owing to the higher flood elevation, a 1:1 restoration of the storage is not feasible. The proposed grading plan incorporates as much cut as was practical given the shallow rock and minimal cut opportunity while maintaining positive drainage.

The Lane Creek floodplain analysis (Jewell 2022) was completed with some conservatism and assumed impacts of development within the urban boundary of Wellington to contribute upstream of Belleville Street. But the Cork & Vine subdivision will contribute flows immediately downstream of Belleville Street and does not benefit from the storage available upstream of the Belleville Street crossing.

Jewell reviewed in more detail the drainage pathways from the proposed development and made adjustments to the hydraulic model.

For the floodplain review, the two objectives for flood protection were maintained. These are:

1. No increase in flood elevation, and
2. No increase in peak flow.

8.1 Model Descriptions

Model 1 – Original 2022 Floodplain Model

This model conservatively assumed inputs from the catchment between Belleville Street and the Millennium Trail to contribute upstream of the Belleville Street crossing. This was a conservative approximation made to simplify modelling effort.

Model 2 – Revised Existing Conditions

Jewell prepared a revised Existing Conditions model that considered the more detailed drainage patterns anticipated from the Cork & Vine Subdivision in preparation for the post-development model (Model 3).

Model 3 – Post-Development with Floodplain Encroachment

This last analysis reviewed the impacts of the Cork & Vine Development and the lost flood storage.

8.2 Model Results

8.2.1 Water Surface Elevation

The revised existing conditions model revealed the 100-yr flood elevation is slightly lower than presented in the new official floodplain mapping (Table 8-1). As an example, the flood elevation would be 84.31m instead of 84.37m at the Millennium Trail crossing. This 6cm difference is minor and suggests the impact of the system storage is not significant.

Table 8-1: Flood Elevations at Points of Interest (m)

Point of Interest	Floodplain Mapping (Model 1)	Pre-Development (Model 2)	Post-Development (Model 3)
Millennium Trail, Upstream	84.37	84.31	84.35
Maple Street, Upstream of Crossing	83.39	83.36	83.38
Maple Street, Downstream of Crossing	83.20	83.14	83.18

Model 3 used the pre-development input hydrographs for the upstream catchments (the same as Model 2), however the Cork & Vine lands were updated to reflect the post-development conditions and the fill that would be placed in the floodplain. As expected, the loss in flood storage results in a minor increase in WSEL and peak flows.

The flood elevation in the post-development condition is 84.35m, which is 4cm higher than the Model 2 results but 2cm lower than the official flood line of 84.37m (Model 1).

8.2.2 Peak Flows

The pre-development conditions in the original model had the flows at the Millennium Trail peaking at 10.40m³/s upstream and 10.23 m³/s downstream at Maple Street, whereas the post-development model peaked at 10.26 m³/s and 10.09 m³/s respectively (Table 8-2).

The peak flows are slightly reduced as a result of the proposed development, compared to the official floodplain study at all crossings.

Table 8-2: Summary of 100-Yr Peak Flows at Millennium Trail and Maple Street Crossings (m³/s)

Point of Interest	April 2022 Model	Pre-Development	Post-Development
Millennium Trail	10.40	9.83	10.26
Maple Street	10.23	9.72	10.09

Summary

The Cork and Vine development includes some fill within the floodplain of Lane Creek. An offsetting cut is proposed within the same reach. Jewell investigated the impacts of the cut/fill using the hydraulic model and found the flood elevations and peak flows are not increased above the values published within the floodplain mapping report.

Water surface elevations and peak flows are not negatively impacted by the proposed development scheme.

9 Conclusions

Kaitlin Corp. is proposing to develop 111 ha of lands just northeast of Wellington in Prince Edward County. Two wet pond facilities are proposed to mitigate water quality impacts. Water quantity impacts were evaluated using hydrologic and hydraulic modelling software with consideration of the watershed contributions.

The proposed beat-the-peak methodology is shown to be effective as the pre-development peaks exceed the post-development peaks at both points of interest: Node C (Belleville St) and Node B (the Millennium Trail crossing) as shown in Table 9-1.

Table 9-1: Summary of Peak Flows (m³/s)

Point of Interest	Pre-Development Peak	Post-Development Peak	Post < Pre?
Node C	14.04	13.95	✓
Node B	14.18	14.17	✓

Note that the above volumes are direct hydrograph outputs and do not including routing. The floodplain storage results in a decrease of peak flows from 14.1m³/s to 10.3m³/s.

The minor flows will be conveyed to the wet pond facilities, and the major flows will bypass and drain directly to the creek.

East Facility

The SWMF for Phase 3A(East) has been sized to provide **Enhanced** quality treatment to 12.2 hectares of runoff (the SWMF and developed lands to its north) during the 25mm event. The sizing for this facility is included in Table 9-2 below.

Table 9-2: Phase 3A(E) SWMF Summary

Feature	Storage (m ³)		
	Required	Provided	Sufficient Storage?
Permanent Pool	2,266	2,773	✓
Erosion Control	1,593	1,679	✓
40m ³ /ha	488		✓

Main Facility

The main development block's SWMF is sized to provide **Enhanced** quality control to 75.1 hectares of runoff (the development lands north of the creek plus 10 additional hectares from

Fields of Wellington) during the 25mm event. The sizing for this facility is included in Table 9-3 below.

Table 9-3: Target Summary

Feature	Elevation (m)	Storage (m ³)		
		Required	Provided	Sufficient Storage?
Bottom of Pond	83.00	-	-	N/A
Permanent Pool	84.70	11,265	12,214	✓
Extended Detention	85.20	3,004	5,188	✓
Active Storage	85.70	-	11,093	✓
Top of Berm	86.00	-	14,963	✓

OGS Unit

A 0.80ha portion of Street A will not contribute to the quality pond, but will be treated by a FD-4HC Oil Grit separator treatment unit that will provide 80% removal of TSS and achieve the *Enhanced* treatment target.

Phase 4

Development of Phase 4 is expected to harden approx. 5ha of land, resulting in approximately 2,200 m³ of additional runoff during the design event (100Yr, 6hr). A portion of this additional runoff will be stored in on-site storage (parking lot and/or rooftop storage) to limit the peak to pre-development levels.

Sediment and erosion controls are proposed to be provided during construction to protect the natural environment. Long term erosion protection is supported using the more stringent 25mm design criteria for the extended detention volume in the wet pond.

Climate change considerations were included by using the 2072 IDF curves from MTO IDF lookup tool as was consistent with the direction from the Lane Creek Flood Risk Mapping draft report.

Prepared by



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Jewell Engineering Inc.

A handwritten signature in blue ink that reads "DSB".

Duncan Burgess, B.A.Sc.
Jewell Engineering Inc.

Reviewed by



Bryon Keene, P.Eng.
Jewell Engineering Inc.

10 References

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APPENDIX A

MTO IDF Curves

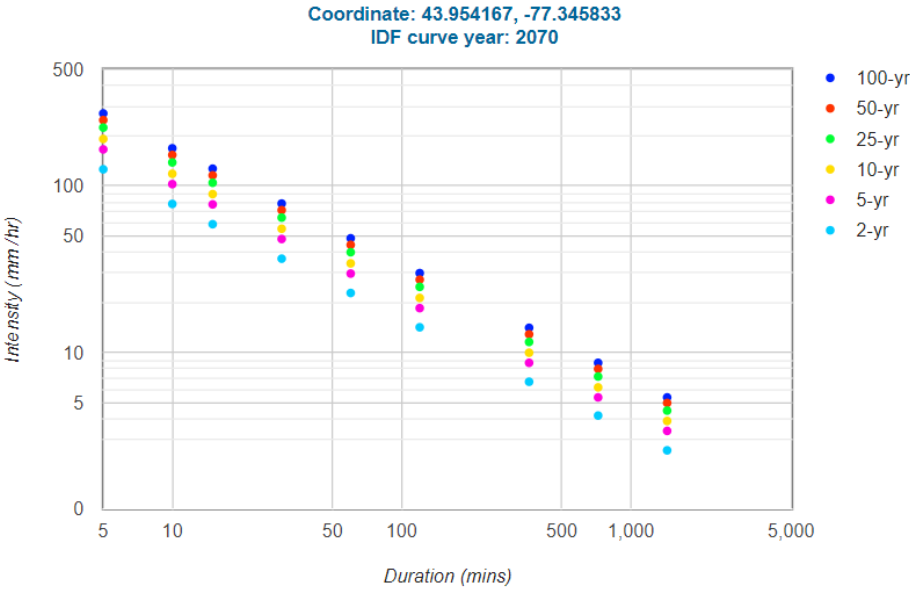
Location summary

These are the locations in the selection.

IDF Curve: 43° 57' 15" N, 77° 20' 44" W (43.954167,-77.345833)

Results

An IDF curve was found.



Coefficient summary

Data year: 2010

IDF curve year: 2070

Click a return period in the table header for more detail.

Statistics

Rainfall intensity (mm hr⁻¹)

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr ⓘ	125.6	77.9	58.9	36.6	22.8	14.2	6.7	4.2	2.6
5-yr ⓘ	165.3	102.4	77.4	48.0	29.8	18.5	8.7	5.4	3.4
10-yr ⓘ	190.9	118.1	89.2	55.3	34.3	21.3	10.0	6.2	3.9
25-yr ⓘ	223.3	138.1	104.3	64.5	40.0	24.8	11.6	7.2	4.5
50-yr ⓘ	247.7	153.1	115.6	71.5	44.3	27.4	12.9	8.0	5.0
100-yr ⓘ	271.5	167.8	126.7	78.3	48.5	30.0	14.1	8.7	5.4

Rainfall depth (mm)

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr ⓘ	10.5	13.0	14.7	18.3	22.8	28.4	40.2	50.4	62.4
5-yr ⓘ	13.8	17.1	19.4	24.0	29.8	37.0	52.2	64.8	81.6
10-yr ⓘ	15.9	19.7	22.3	27.6	34.3	42.6	60.0	74.4	93.6
25-yr ⓘ	18.6	23.0	26.1	32.3	40.0	49.6	69.6	86.4	108.0
50-yr ⓘ	20.6	25.5	28.9	35.8	44.3	54.8	77.4	96.0	120.0
100-yr ⓘ	22.6	28.0	31.7	39.1	48.5	60.0	84.6	104.4	129.6

APPENDIX B

Development Concept Plan

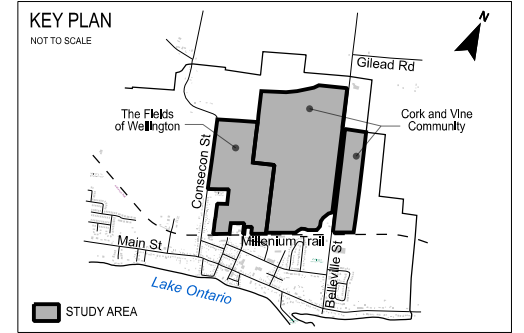
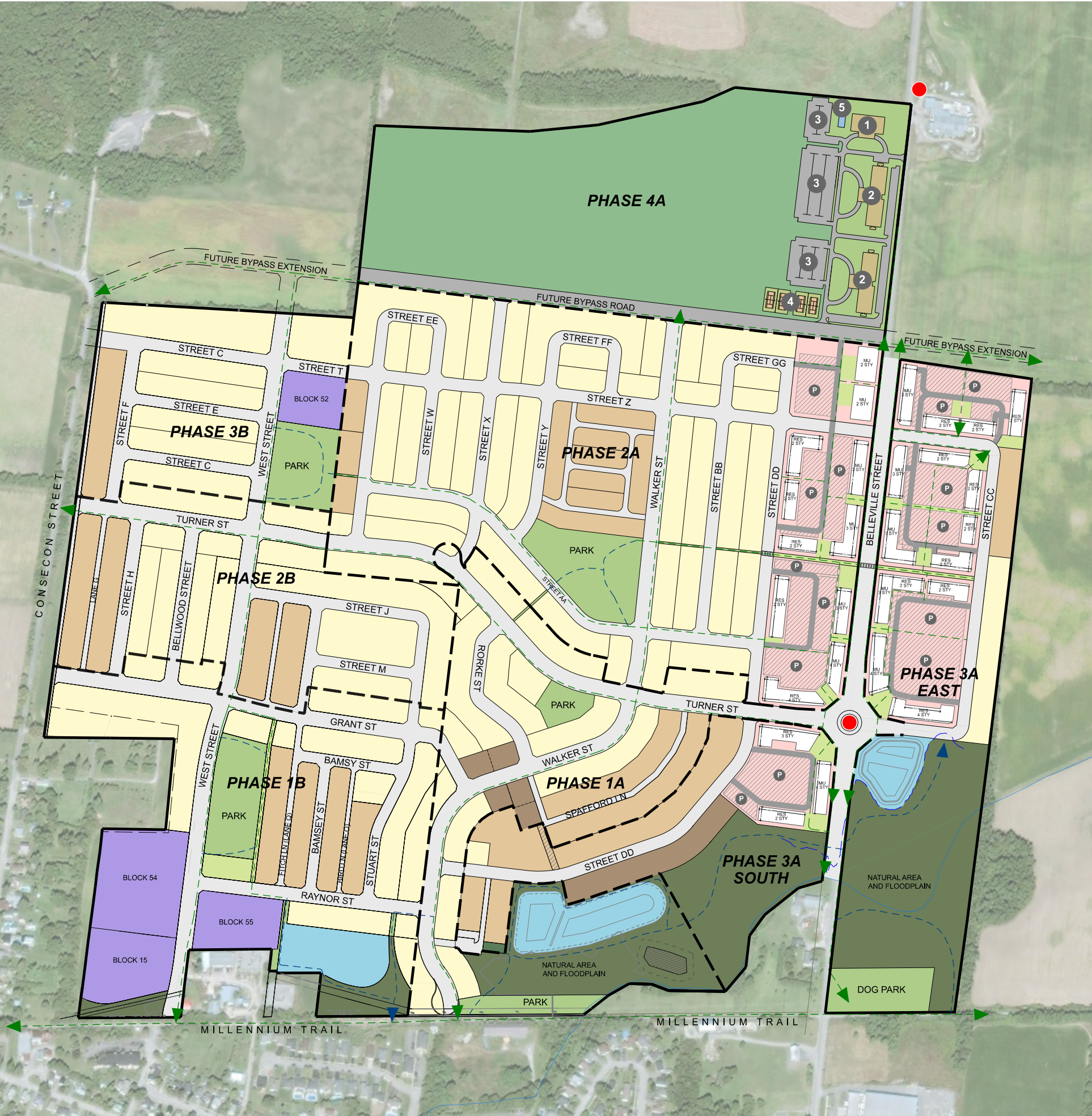
Catchment Area Drawings

NOTES:

- Phase 1A, 1B, 2B and 3B based on proposed Draft Plans.
- Phase 2A, 3A East, and 3A South units are based on working copy Draft Plan, dated 2025.08.22 .
- Average Apartment size @ 70m²/unit based on 85% efficiency of residential GFA.

Notes:

- Property Boundary is approximate based on available mapping and subject to confirmation by survey.
- Air photo from Google Earth, 2019 image.
- Not based on engineering, grading or environmental analysis.
- Units are estimated and subject to change based on detailed design.



- LEGEND**
- | | |
|--|--|
| | Subject Lands |
| | Phasing Boundary |
| | Single-Detached |
| | Standard Condo and Free-hold Townhouses |
| | Bungalow Townhouses |
| | Mid-Rise/Future Development |
| | Gateway Mixed-Use (MU) |
| | Parks and Green Links |
| | Common Greens |
| | Natural Open Space |
| | Storm Pond |
| | Multi-use Pathway/Pedestrian Links |
| | Secondary Pathways |
| | Flood Line 2022 |
| | Water Course |
| | Conceptual Parking Areas |
| | Conceptual Private Roads |
| | Conceptual Buildings in Gateway MU Areas |
| | Major Cross Walk |
| | Major Entry Feature, Signage, Landscaping etc. |

DRAFT

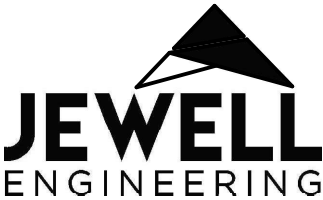
**FOR DISCUSSION
PURPOSES ONLY**

CORK AND VINE COMMUNITY
WELLINGTON,
PRINCE EDWARD COUNTY



WESTON
CONSULTING 

File Number: 10793
Date: 2025-08-28
Drawn By: MH
Planner: RG and MQ
CAD: 10793/Composite Plan/2025/C3_2025-08-28.dgn



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:

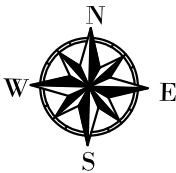
- ALL DIMENSIONS SHOWN ARE IN METRES OR MILLIMETRES, UNLESS OTHERWISE NOTED.

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



QUINTE CONSERVATION
LANE CREEK WATERSHED

WELLINGTON, ONTARIO

LANE CREEK EXISTING CONDITION
CATCHMENT DRAWING

DRAWN BY: PROJECT NO:
MB 210-4981

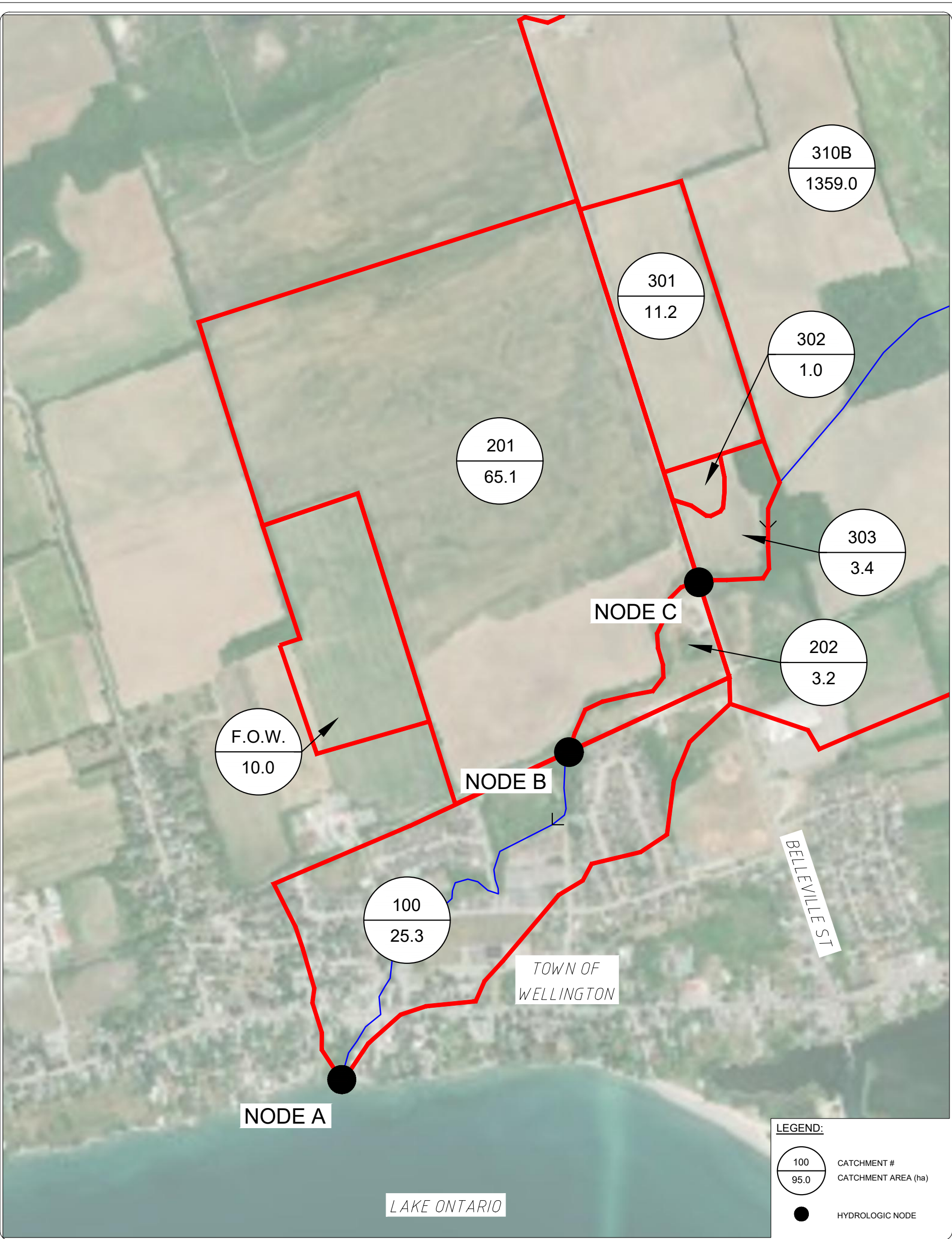
DESIGNED BY: DATE:
EF April 2022

CHECKED BY: CONTRACT NO:
EF

APPROVED BY:

SCALE:
HORIZONTAL - 1:25000
VERTICAL - N/A

DRAWING NO:
A-1





JEWELL
ENGINEERING

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:

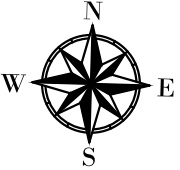
- ALL DIMENSIONS SHOWN ARE IN METRES OR MILLIMETRES, UNLESS OTHERWISE NOTED.

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 - GEOIDETIC MODEL HT2_0, UNLESS DESCRIBED OTHERWISE.

**** DRAWINGS ARE NOT TO BE SCALED ****

REVISIONS			
NO.	DATE	DESCRIPTION	BY



COUNTRY CLUB ESTATES
KAITLIN CORPORATION
WELLINGTON, ONTARIO
PRINCE EDWARD COUNTY

LANE CREEK WATERSHED CATCHMENT

DRAWN BY: JH PROJECT NO: 170-4226-2

DESIGNED BY: DATE: November 2025

CHECKED BY: BK CONTRACT NO:

APPROVED BY:

SCALE:
HORIZONTAL - 1:7500
VERTICAL - N/A

DRAWING NO: B-3

APPENDIX C

Facility Sizing

Andrew Rosenthal, P.Eng.

October 24, 2025

Kaitlin C&V – Phase 3A(East) Preliminary SWMF Sizing

Pond Elevations		Outlet 1		Outlet 2		Outlet 3	
Perm.Pool	85.6	Use Outlet 1?	Yes	Use Outlet 2?	No	Use Outlet 3?	Yes
Max. Elev	86.6	Type	Orifice			Type	Weir
Increment	0.05	Invert	85.6			Invert	86.15
		diam (m)	0.17			Length	3.5
		No. of Outlets	1			No. of Outlets	1

$$Q = 1.67LH^{1.5}$$

Orifice Equation:

$$= (h \leq 2r) 0.6 * \left[\left(\arccos\left(\frac{r-h}{r}\right) \right) r^2 - r(r-h) * \sin\left(\arccos\left(\frac{r-h}{r}\right)\right) \right] * \sqrt{2g * \left(\frac{4r * \sin^3\left(\arccos\left(\frac{r-h}{r}\right)\right)}{3 \left(2 \left(\arccos\left(\frac{r-h}{r}\right) \right) - \sin\left(2 \left(\arccos\left(\frac{r-h}{r}\right) \right) \right) \right)} - (r-h) \right)}$$

$$+ (h > 2r) 0.6 \pi r^2 * \sqrt{2g(h-r)}$$

Elevation m	Incr.Vol m3	Cum.Vol m3	Head on inv., m	Q, m3/s	Head on inv., m	Q, m3/s	Head on inv., m	Q, m3/s	Q.total m3/s
85.60	0	0	0	0.000	-	0.000	-	0.000	0.000
85.65	138	138	0.05	0.002	-	0.000	-	0.000	0.002
85.70	138	276	0.1	0.008	-	0.000	-	0.000	0.008
85.75	144	421	0.15	0.015	-	0.000	-	0.000	0.015
85.80	144	565	0.2	0.020	-	0.000	-	0.000	0.020
85.85	151	716	0.25	0.025	-	0.000	-	0.000	0.025
85.90	151	866	0.3	0.028	-	0.000	-	0.000	0.028
85.95	157	1023	0.35	0.031	-	0.000	-	0.000	0.031
86.00	157	1180	0.4	0.034	-	0.000	-	0.000	0.034
86.05	164	1344	0.45	0.036	-	0.000	-	0.000	0.036
86.10	164	1507	0.5	0.039	-	0.000	-	0.000	0.039
86.15	172	1679	0.55	0.041	-	0.000	-	0.000	0.041
86.20	172	1851	0.6	0.043	-	0.000	0.05	0.065	0.109
86.25	179	2030	0.65	0.045	-	0.000	0.1	0.185	0.230
86.30	179	2210	0.7	0.047	-	0.000	0.15	0.340	0.387
86.35	185	2395	0.75	0.049	-	0.000	0.2	0.523	0.572
86.40	185	2580	0.8	0.051	-	0.000	0.25	0.731	0.782
86.45	191	2770	0.85	0.053	-	0.000	0.3	0.960	1.013
86.50	191	2961	0.9	0.054	-	0.000	0.35	1.210	1.265
86.55	196	3158	0.95	0.056	-	0.000	0.4	1.479	1.535
86.60	196	3354	1	0.058	-	0.000	0.45	1.764	1.822

Andrew Rosenthal, P.Eng.

October 24, 2025

Kaitlin C&V – Main (Central) SWMF Sizing

Pond Elevations		Outlet 1		Outlet 2		Outlet 3	
Perm.Pool	84.7	Use Outlet 1?	Yes	Use Outlet 2?	No	Use Outlet 3?	Yes
Max. Elev	86	Type	Orifice			Type	Weir
Increment	0.1	Invert	84.7			Invert	85.2
		diam (m)	0.6			Length	4.25
		No. of Outlets	1			No. of Outlets	1

$$Q = 1.67LH^{1.5}$$

Orifice Equation:

$$= (h \leq 2r) 0.6 * \left[\left(\arccos\left(\frac{r-h}{r}\right) \right) r^2 - r(r-h) * \sin\left(\arccos\left(\frac{r-h}{r}\right)\right) \right] * 2g * \left(\frac{4r * \sin^3\left(\arccos\left(\frac{r-h}{r}\right)\right)}{3 \left(2 \left(\arccos\left(\frac{r-h}{r}\right) \right) - \sin\left(2 \left(\arccos\left(\frac{r-h}{r}\right) \right) \right) \right)} - (r-h) \right) \\ + (h > 2r) 0.6 \pi r^2 * \sqrt{2g(h-r)}$$

Elevation m	Incr.Vol m3	Cum.Vol m3	Head on inv., m	Q, m3/s	Head on inv., m	Q, m3/s	Head on inv., m	Q, m3/s	Q.total m3/s
84.70	0	0	0	0.000	-	0.000	-	0.000	0.000
84.80	973	973	0.1	0.017	-	0.000	-	0.000	0.017
84.90	1004	1977	0.2	0.063	-	0.000	-	0.000	0.063
85.00	1035	3012	0.3	0.134	-	0.000	-	0.000	0.134
85.10	1074	4087	0.4	0.223	-	0.000	-	0.000	0.223
85.20	1101	5188	0.5	0.321	-	0.000	-	0.000	0.321
85.30	1127	6315	0.6	0.412	-	0.000	0.1	0.224	0.636
85.40	1154	7469	0.7	0.475	-	0.000	0.2	0.635	1.110
85.50	1181	8650	0.8	0.531	-	0.000	0.3	1.166	1.698
85.60	1208	9858	0.9	0.582	-	0.000	0.4	1.796	2.378
85.70	1235	11093	1	0.629	-	0.000	0.5	2.509	3.138
85.80	1263	12356	1.1	0.672	-	0.000	0.6	3.299	3.971
85.90	1290	13646	1.2	0.713	-	0.000	0.7	4.157	4.870
86.00	1318	14963	1.3	0.751	-	0.000	0.8	5.079	5.830

Main Facility Bypass weir and spillway sizing:

$$Q = 1.67LH^{3/2}$$

$$Q = 1.67(2.4)(0.62)^{3/2} = 1.96$$

(note that the bypass structure will engage when the pond has filled to the extended detention storage elevation of 85.20m)

$$Q = 1.67(4.25)(0.80)^{3/2} = 5.08$$

The bypass weir and emergency spillway have a combined capacity of over 7 m³/s, therefore they are sufficiently sized to convey the uncontrolled 100Yr event peak flows (6.06 m³/s).

Forebay Sizing Project Example

Andrew Rosenthal, EIT
2022/07/25

Permanent Pool Dimensions

Length (m)=	43	Side Slope (X:1) =	5	
Width (m)=	32	Depth (m) =	1.5	<i>Average</i>
Bot Width (m) =	17			

Select Forebay Length (m) =

35

Assume 1/3 of Permanent Pool Length

Calculate Settling Length (m) =

14.9

OK

Equation 4.5 (MOE, 2003)

$$Distance = \sqrt{\frac{rQ_p}{V_s}}$$

Where:

r =	1.09	= L:W ratio of forebay
Q _p =	0.061	= Peak Outflow Rate during Quality Event (cms)
V _s =	0.0003	= Settling Velocity for 0.15mm dia particle

Calculate Dispersion Length (m) =

32.4

OK

Equation 4.6 (MOE, 2003)

$$Distance = \frac{8Q}{dV_f}$$

Where:

Q =	3.040	= Inlet Flow Rate (cms)
d =	1.5	= Depth of Permanent Pool in Forebay (m)
V _f =	0.5	= Maximum Velocity in the Forebay (m/s)

Min. Forebay Deep Zone Bottom Width (m) =

4.1

OK

Equation 4.7 (MOE, 2003)

$$Width = \frac{Dist}{8}$$

Where:

Dist =	32.4	= Dispersion Length (m) from Equation 4.6
--------	------	---

Check Average Inflow Velocity < 0.15 (m/s)

0.08

OK

$$V = \frac{Q}{A}$$

Where:

V =		Average Velocity in Forebay (m/s)
Q =	3.04	= Inlet Flow Rate (cms)
A =	36.75	= Cross-sectional Area (m ²)

Phase 3A East SWMF Drawdown Time

Stage, m	Storage, m ³	Discharge, m ³ /s	Incr. Storage, m ³	Avg. Q, m ³ /s	Incr. time, hr	Sum time, hr
86.13	1593	0.037	-	-	-	0
86.1	1507	0.035	86	0.036	1.3	1.3
86.05	1344	0.033	164	0.034	1.4	2.7
86.00	1180	0.030	164	0.031	1.4	4.1
85.95	1023	0.028	157	0.029	1.5	5.6
85.90	866	0.025	157	0.026	1.7	7.3
85.85	716	0.022	151	0.024	1.8	9.1
85.80	565	0.019	151	0.020	2.1	11.1
85.75	421	0.014	144	0.016	2.5	13.6
85.70	276	0.007	144	0.011	3.8	17.4
85.65	138	0.002	138	0.005	8.2	25.6
Total Drawdown Time						<u>25.6</u>

Assume drawdown complete at 5cm.

Main SWMF Drawdown Time

Stage, m	Storage, m ³	Discharge, m ³ /s	Incr. Storage, m ³	Avg. Q, m ³ /s	Incr. time, hr	Sum time, hr
85.17	4868	0.293				
85.1	4087	0.223	781	0.258	0.8	0.8
85.0	3021	0.134	1066	0.179	1.7	2.5
84.9	1977	0.063	1044	0.099	2.9	5.4
84.8	973	0.017	1004	0.040	7.0	12.4
84.75	487	0.004	487	0.011	12.9	25.3
Total Drawdown Time						<u>25.3</u>

Assume drawdown complete at 5cm.

APPENDIX D

OTTHYMO Modeling Output

```

=====
      000   TTTT   TTTT   H   H   Y   Y   M   M   000   I N T E R H Y M O
      0   0   T     T   H   H   Y   Y   M M M M   0   0   * * * 1989a * * *
      0   0   T     T   H H H H H   Y   M M M M   0   0
      0   0   T     T   H   H   Y   M   M   0   0
      000   T     T   H   H   Y   M   M   000           00027

```

Distributed by the INTERHYMO Centre. Copyright (c), 1989. Paul Wisner & Assoc.

Input filename: kait2025.dat
Output filename: kait2025.out
Summary filename: kait2025.sum

DATE: 10-24-2025

TIME: 15:22:24

COMMENTS: _____

```

-----
*
*****
** SIMULATION NUMBER: 1 **
*****
*
*           Country Club Estates (+ Fields of Wellington)
*
*           Original Model Prepared July 28, 2022
*           Andrew Rosenthal, EIT
*
*           October 24, 2025
*           Duncan Burgess, BASc
*           Andrew Rosenthal, P.Eng.
*
*           - Updated for new concept plan imperviousness
*           - Adjustment to Phase 3A(E) CT (decreased area,
*             increased % impervious, 3.4ha undev. bypassing)
*           - 3A(E) SWMF c/w 0.17m orifice, 3.5m weir at 0.55m APP
*           - CCE Main SWMF c/w 0.6m orifice, 4.25m weir at 0.5m APP
*
*           Spillways flow adjusted to broad-crested weir equ'n
*           Partially full orifice flow calculated using Rosenthal
*           equation (2024) - see pond sizing sheets in Appx.
*
*           LGI, LGP from  $L = \sqrt{A/1.5}$ 
*
*           Phase 3A(E), Country Club Estates
*           Pre- and Post-Development flows in Lane Creek
*
*****
*           Phase 3(E) 25mm Event - Post-Development to SWMF
*****

```

```

-----
|   READ STORM   |   Filename: quality.stm
| Ptotal= 25.00 mm |   Comments: 4 Hr Chicago 25mm
-----

```

TIME	RAIN	TIME	RAIN	TIME	RAIN	TIME	RAIN
hrs	mm/hr	hrs	mm/hr	hrs	mm/hr	hrs	mm/hr
.08	1.73	1.08	7.10	2.08	4.61	3.08	2.28
.17	1.83	1.17	10.69	2.17	4.21	3.17	2.19
.25	1.94	1.25	23.51	2.25	3.88	3.25	2.11
.33	2.07	1.33	68.86	2.33	3.61	3.33	2.04
.42	2.23	1.42	29.90	2.42	3.38	3.42	1.97
.50	2.41	1.50	17.13	2.50	3.18	3.50	1.91
.58	2.64	1.58	12.05	2.58	3.00	3.58	1.86
.67	2.92	1.67	9.35	2.67	2.84	3.67	1.80
.75	3.28	1.75	7.68	2.75	2.70	3.75	1.75
.83	3.74	1.83	6.55	2.83	2.58	3.83	1.70
.92	4.41	1.92	5.73	2.92	2.47	3.92	1.66
1.00	5.41	2.00	5.10	3.00	2.37	4.00	1.62

```

-----
*                               Catchment 301
-----

```

```

| CALIB
| STANDHYD (0001) |   Area (ha)= 11.20
| ID= 1 DT= 5.0 min |   Total Imp(%)= 75.00   Dir. Conn.(%)= 65.00
-----

```

	IMPERVIOUS	PERVIOUS (i)	
Surface Area	(ha)= 8.40	2.80	
Dep. Storage	(mm)= 2.00	5.00	
Average Slope	(%)= 2.00	2.00	
Length	(m)= 273.00	273.00	
Mannings n	= .013	.250	
Max.eff.Inten.(mm/hr)=	49.38	3.92	
over (min)	10.00	85.00	
Storage Coeff. (min)=	5.03 (ii)	86.66 (ii)	
Unit Hyd. Tpeak (min)=	5.00	90.00	
Unit Hyd. peak (cms)=	.21	.01	
			TOTALS
PEAK FLOW (cms)=	1.01	.02	1.01 (iii)
TIME TO PEAK (hrs)=	1.58	3.33	1.58
RUNOFF VOLUME (mm)=	23.00	5.19	16.72
TOTAL RAINFALL (mm)=	25.00	25.00	25.00
RUNOFF COEFFICIENT =	.92	.21	.67

- (i) CN PROCEDURE SELECTED FOR RAINFALL LOSSES:
CN* = 73.0 Ia = Dep. Storage (Above)
- (ii) COMPUTATIONAL TIME STEP SHOULD BE SMALL OR EQUAL
THAN THE STORAGE COEFFICIENT.
- (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```

-----
*                               Catchment 302
-----

```

```

| CALIB
| STANDHYD (0001) |   Area (ha)= 1.00
| ID= 2 DT= 5.0 min |   Total Imp(%)= 50.00   Dir. Conn.(%)= 40.00
-----

```

```

-----
                IMPERVIOUS    PERVIOUS (i)
Surface Area    (ha)=         .50         .50
Dep. Storage    (mm)=         2.00         5.00
Average Slope    (%)=         2.00         2.00
Length          (m)=        82.00        82.00
Mannings n      =          .013         .250

Max.eff.Inten.(mm/hr)=      49.38         4.38
                    over (min)      10.00        40.00
Storage Coeff. (min)=        2.44 (ii)      40.39 (ii)
Unit Hyd. Tpeak (min)=        5.00        45.00
Unit Hyd. peak  (cms)=        .30         .03

                                     *TOTALS*
PEAK FLOW        (cms)=         .07         .00         .07 (iii)
TIME TO PEAK     (hrs)=         1.42         2.33         1.42
RUNOFF VOLUME     (mm)=        23.00         4.38        11.60
TOTAL RAINFALL    (mm)=        25.00        25.00        25.00
RUNOFF COEFFICIENT =          .92         .18         .46

```

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

- (i) CN PROCEDURE SELECTED FOR RAINFALL LOSSES:
CN* = 73.0 Ia = Dep. Storage (Above)
- (ii) COMPUTATIONAL TIME STEP SHOULD BE SMALL OR EQUAL
THAN THE STORAGE COEFFICIENT.
- (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```

-----
| ADD HYD (0001) |
| 1 + 2 = 3 |
-----
                AREA      QPEAK      TPEAK      R.V.
                (ha)      (cms)      (hrs)      (mm)
ID1= 1 (0001):  11.20      1.01      1.58      16.72
+ ID2= 2 (0001):  1.00       .07      1.42      11.60
=====
ID = 3 (0001):  12.20      1.04      1.58      16.31

```

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

```

-----
| SAVE HYD (0001) | AREA      (ha)= 12.20
| ID= 3 PCYC=104 | QPEAK      (cms)= 1.04 (i)
| DT= 5.0 min | TPEAK      (hrs)= 1.58
-----
VOLUME      (mm)= 16.31
Filename: K3E00UNC.TXT
Comments:

```

- (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```

-----
*                               Route through SWMF
-----
| RESERVOIR (0001) |
| IN= 3---> OUT= 4 |
| DT= 5.0 min |
-----
                OUTFLOW      STORAGE      OUTFLOW      STORAGE
                (cms)      (ha.m.)      (cms)      (ha.m.)

```

.000	.000	.039	.185
.007	.028	.253	.221
.019	.057	.643	.258
.025	.087	1.146	.296
.030	.118	1.741	.335
.035	.151	.000	.000

	AREA (ha)	QPEAK (cms)	TPEAK (hrs)	R.V. (mm)
INFLOW : ID= 3 (0001)	12.20	1.04	1.58	16.31
OUTFLOW: ID= 4 (0001)	12.20	.04	4.33	16.22

PEAK FLOW REDUCTION [Qout/Qin](%)= 3.44
 TIME SHIFT OF PEAK FLOW (min)=165.00
 MAXIMUM STORAGE USED (ha.m.)= .16

```

-----
| SAVE HYD (0001) | AREA      (ha)= 12.20
| ID= 4  PCYC=521 | QPEAK    (cms)=  .04 (i)
| DT= 5.0 min     | TPEAK    (hrs)=  4.33
-----
|                   | VOLUME   (mm)= 16.22
Filename: K3E25CON.TXT
Comments:
  
```

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```

-----
*****
*               100Yr Existing (pre-dev) Flows to Belleville St Crossing
*****
*               Pre-development flows to Belleville St.
  
```

```

-----
| READ HYD (0001) | AREA      (ha)=1397.00
| ID= 1  PCYC= 1  | QPEAK    (cms)= 14.03
| DT= 5.0 min     | TPEAK    (hrs)= 10.33
-----
|                   | VOLUME   (mm)= 20.17
Filename: BELLEX.TXT
Comments: Flows to Belleville Street (Existing)
  
```

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
hrs	cms	hrs	cms	hrs	cms	hrs	cms	hrs	cms
.00	.00	4.00	.13	8.00	8.65	12.00	10.47	.00	.00
.08	.00	4.08	.16	8.08	8.95	12.08	10.23	.00	.00
.17	.00	4.17	.20	8.17	9.23	12.17	9.99	.00	.00
.25	.00	4.25	.24	8.25	9.48	12.25	9.75	.00	.00
.33	.00	4.33	.29	8.33	9.71	12.33	9.51	.00	.00
.42	.00	4.42	.34	8.42	9.91	12.42	9.28	.00	.00
.50	.00	4.50	.40	8.50	10.10	12.50	9.05	.00	.00
.58	.00	4.58	.46	8.58	10.27	12.58	8.82	.00	.00
.67	.00	4.67	.53	8.67	10.42	12.67	8.60	.00	.00
.75	.00	4.75	.60	8.75	10.58	12.75	8.38	.00	.00
.83	.00	4.83	.68	8.83	10.73	12.83	8.17	.00	.00
.92	.00	4.92	.76	8.92	10.89	12.92	7.96	.00	.00
1.00	.00	5.00	.86	9.00	11.06	13.00	7.75	.00	.00
1.08	.00	5.08	.95	9.08	11.25	13.08	7.55	.00	.00
1.17	.00	5.17	1.06	9.17	11.45	13.17	7.35	.00	.00
1.25	.00	5.25	1.17	9.25	11.68	13.25	7.16	.00	.00
1.33	.00	5.33	1.29	9.33	11.93	13.33	6.97	.00	.00

1.42	.00	5.42	1.50	9.42	12.19	13.42	6.78	.00	.00
1.50	.00	5.50	1.83	9.50	12.46	13.50	6.59	.00	.00
1.58	.00	5.58	2.16	9.58	12.73	13.58	6.41	.00	.00
1.67	.00	5.67	2.46	9.67	12.99	13.67	6.22	.00	.00
1.75	.00	5.75	2.75	9.75	13.23	13.75	6.03	.00	.00
1.83	.00	5.83	3.04	9.83	13.45	13.83	5.84	.00	.00
1.92	.00	5.92	3.35	9.92	13.63	13.92	5.64	.00	.00
2.00	.00	6.00	3.65	10.00	13.79	14.00	5.44	.00	.00
2.08	.00	6.08	3.94	10.08	13.90	.00	-1.00	.00	.00
2.17	.00	6.17	4.21	10.17	13.98	.00	.00	.00	.00
2.25	.00	6.25	4.44	10.25	14.02	.00	.00	.00	.00
2.33	.00	6.33	4.64	10.33	14.03	.00	.00	.00	.00
2.42	.00	6.42	4.80	10.42	14.01	.00	.00	.00	.00
2.50	.00	6.50	4.94	10.50	13.95	.00	.00	.00	.00
2.58	.00	6.58	5.08	10.58	13.87	.00	.00	.00	.00
2.67	.00	6.67	5.21	10.67	13.77	.00	.00	.00	.00
2.75	.00	6.75	5.34	10.75	13.65	.00	.00	.00	.00
2.83	.00	6.83	5.47	10.83	13.50	.00	.00	.00	.00
2.92	.00	6.92	5.60	10.92	13.34	.00	.00	.00	.00
3.00	.00	7.00	5.73	11.00	13.17	.00	.00	.00	.00
3.08	.00	7.08	5.87	11.08	12.98	.00	.00	.00	.00
3.17	.00	7.17	6.02	11.17	12.78	.00	.00	.00	.00
3.25	.00	7.25	6.17	11.25	12.57	.00	.00	.00	.00
3.33	.01	7.33	6.35	11.33	12.36	.00	.00	.00	.00
3.42	.01	7.42	6.55	11.42	12.13	.00	.00	.00	.00
3.50	.02	7.50	6.79	11.50	11.90	.00	.00	.00	.00
3.58	.03	7.58	7.05	11.58	11.67	.00	.00	.00	.00
3.67	.04	7.67	7.34	11.67	11.43	.00	.00	.00	.00
3.75	.05	7.75	7.66	11.75	11.19	.00	.00		
3.83	.07	7.83	7.99	11.83	10.95	.00	.00		
3.92	.10	7.92	8.32	11.92	10.71	.00	.00		

```

-----
| SAVE HYD (0001) | AREA      (ha)=1397.00
| ID= 1  PCYC=169 | QPEAK    (cms)= 14.03 (i)
| DT= 5.0 min     | TPEAK    (hrs)= 10.33
----- VOLUME    (mm)= 20.17

```

Filename: BLVLPRe.TXT
Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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-----
*****
*                               Phase 3(E) 100Yr Climate Change Storm - Post-Development
*****

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| READ STORM      | Filename: 6scs72.stm
| Ptotal= 84.70 mm | Comments: 6 Hr SCS Type 2 - 100Yr
-----

```

TIME	RAIN	TIME	RAIN	TIME	RAIN	TIME	RAIN
hrs	mm/hr	hrs	mm/hr	hrs	mm/hr	hrs	mm/hr
.50	3.39	2.00	8.47	3.50	66.07	5.00	6.78
1.00	5.08	2.50	10.16	4.00	18.63	5.50	5.08
1.50	5.08	3.00	25.41	4.50	8.47	6.00	6.78

*

Catchment 301

CALIB			
STANDHYD (0001)	Area (ha)=	11.20	
ID= 1 DT= 5.0 min	Total Imp(%)=	75.00	Dir. Conn.(%)= 65.00

		IMPERVIOUS	PERVIOUS (i)
Surface Area	(ha)=	8.40	2.80
Dep. Storage	(mm)=	2.00	5.00
Average Slope	(%)=	2.00	2.00
Length	(m)=	273.00	273.00
Mannings n	=	.013	.250

NOTE: RAINFALL WAS TRANSFORMED TO 5.0 MIN. TIME STEP.

		IMPERVIOUS	PERVIOUS (i)	
Max.eff.Inten.(mm/hr)=		66.07	56.51	
over (min)		10.00	30.00	
Storage Coeff. (min)=		4.47 (ii)	32.55 (ii)	
Unit Hyd. Tpeak (min)=		5.00	35.00	
Unit Hyd. peak (cms)=		.23	.03	
				TOTALS
PEAK FLOW (cms)=		1.34	.26	1.51 (iii)
TIME TO PEAK (hrs)=		3.75	4.17	3.75
RUNOFF VOLUME (mm)=		82.13	43.95	68.76
TOTAL RAINFALL (mm)=		84.70	84.70	84.70
RUNOFF COEFFICIENT =		.97	.52	.81

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

- (i) CN PROCEDURE SELECTED FOR RAINFALL LOSSES:
CN* = 73.0 Ia = Dep. Storage (Above)
- (ii) COMPUTATIONAL TIME STEP SHOULD BE SMALL OR EQUAL
THAN THE STORAGE COEFFICIENT.
- (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

*

Catchment 302

CALIB			
STANDHYD (0001)	Area (ha)=	1.00	
ID= 2 DT= 5.0 min	Total Imp(%)=	50.00	Dir. Conn.(%)= 40.00

		IMPERVIOUS	PERVIOUS (i)
Surface Area	(ha)=	.50	.50
Dep. Storage	(mm)=	2.00	5.00
Average Slope	(%)=	2.00	2.00
Length	(m)=	82.00	82.00
Mannings n	=	.013	.250

Max.eff.Inten.(mm/hr)=		66.07	49.36	
over (min)		10.00	15.00	
Storage Coeff. (min)=		2.17 (ii)	16.57 (ii)	
Unit Hyd. Tpeak (min)=		5.00	20.00	
Unit Hyd. peak (cms)=		.31	.06	
				TOTALS
PEAK FLOW (cms)=		.07	.05	.12 (iii)
TIME TO PEAK (hrs)=		3.58	3.67	3.58
RUNOFF VOLUME (mm)=		82.13	40.41	56.99

TOTAL RAINFALL (mm)=	84.70	84.70	84.70
RUNOFF COEFFICIENT =	.97	.48	.67

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

- (i) CN PROCEDURE SELECTED FOR RAINFALL LOSSES:
CN* = 73.0 Ia = Dep. Storage (Above)
- (ii) COMPUTATIONAL TIME STEP SHOULD BE SMALL OR EQUAL
THAN THE STORAGE COEFFICIENT.
- (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

ADD HYD (0001)				
1 + 2 = 3	AREA	QPEAK	TPEAK	R.V.
	(ha)	(cms)	(hrs)	(mm)
ID1= 1 (0001):	11.20	1.51	3.75	68.76
+ ID2= 2 (0001):	1.00	.12	3.58	56.99
=====				
ID = 3 (0001):	12.20	1.58	3.75	67.80

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

SAVE HYD (0001)	AREA	(ha)=	12.20
ID= 3 PCYC=105	QPEAK	(cms)=	1.58 (i)
DT= 5.0 min	TPEAK	(hrs)=	3.75
	VOLUME	(mm)=	67.80

Filename: K3E00UNC.TXT

Comments:

- (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Subtract 25-mm, bypass all other flows

DUHYD (0001)				
Inlet Cap.=1.040				
#of Inlets= 1				
Total(cms)= 1.0	AREA	QPEAK	TPEAK	R.V.
	(ha)	(cms)	(hrs)	(mm)
TOTAL HYD.(ID= 3):	12.20	1.58	3.75	67.80
=====				
MAJOR SYS.(ID= 8):	1.16	.54	3.75	67.80
MINOR SYS.(ID= 7):	11.04	1.04	3.33	67.80

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

* Route through SWMF

RESERVOIR (0001)				
IN= 7---> OUT= 3				
DT= 5.0 min	OUTFLOW	STORAGE	OUTFLOW	STORAGE
	(cms)	(ha.m.)	(cms)	(ha.m.)
	.000	.000	.039	.185
	.007	.028	.253	.221

.019	.057		.643	.258
.025	.087		1.146	.296
.030	.118		1.741	.335
.035	.151		.000	.000

	AREA	QPEAK	TPEAK	R.V.
	(ha)	(cms)	(hrs)	(mm)
INFLOW : ID= 7 (0001)	11.04	1.04	3.33	67.80
OUTFLOW: ID= 3 (0001)	11.04	.93	3.83	67.71

PEAK FLOW REDUCTION [Qout/Qin](%)= 89.03
 TIME SHIFT OF PEAK FLOW (min)= 30.00
 MAXIMUM STORAGE USED (ha.m.)= .28

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-----
| SAVE HYD (0001) | AREA (ha)= 11.04
| ID= 3 PCYC=576 | QPEAK (cms)= .93 (i)
| DT= 5.0 min | TPEAK (hrs)= 3.83
|-----| VOLUME (mm)= 67.71
  
```

Filename: K3E00PQ.TXT
 Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```

-----
| ADD HYD (0001) |
| 3 + 8 = 5 |
|-----|
| ID1= 3 (0001): | AREA (ha)= 11.04 QPEAK (cms)= .93 TPEAK (hrs)= 3.83 R.V. (mm)= 67.71
| + ID2= 8 (0001): | 1.16 .54 3.75 67.80
|=====|
| ID = 5 (0001): | 12.20 1.43 3.75 67.48
  
```

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

* Add CT 303 lands south of pond (bypassing SWMF)

```

| CALIB
| NASHYD (0001) | Area (ha)= 3.40 Curve Number (CN)= 73.0
| ID= 6 DT= 5.0 min | Ia (mm)= 5.00 # of Linear Res.(N)= 3.00
|-----| U.H. Tp(hrs)= .31
  
```

Unit Hyd Qpeak (cms)= .42

PEAK FLOW (cms)= .25 (i)
 TIME TO PEAK (hrs)= 3.58
 RUNOFF VOLUME (mm)= 36.15
 TOTAL RAINFALL (mm)= 84.70
 RUNOFF COEFFICIENT = .43

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

ADD HYD (0001)				
5 + 6 = 1	AREA	QPEAK	TPEAK	R.V.
	(ha)	(cms)	(hrs)	(mm)
ID1= 5 (0001):	12.20	1.43	3.75	67.48
+ ID2= 6 (0001):	3.40	.25	3.58	36.15
=====				
ID = 1 (0001):	15.60	1.67	3.75	60.65

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

SAVE HYD (0001)	AREA	(ha)= 15.60
ID= 1 PCYC=576	QPEAK	(cms)= 1.67 (i)
DT= 5.0 min	TPEAK	(hrs)= 3.75
	VOLUME	(mm)= 60.65

Filename: K3E00PQR.TXT

Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Add flows to Belleville St from undeveloped lands (310B)

READ HYD (0001)	AREA	(ha)=1359.00
ID= 2 PCYC= 1	QPEAK	(cms)= 13.92
DT= 5.0 min	TPEAK	(hrs)= 10.33
	VOLUME	(mm)= 22.39

Filename: BELL.TXT

Comments: Flows to Belleville St. Crossing (Without Phase 3 East)

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
hrs	cms	hrs	cms	hrs	cms	hrs	cms	hrs	cms
.00	.00	4.92	.72	9.83	13.31	14.75	3.29	19.67	.23
.08	.00	5.00	.81	9.92	13.50	14.83	2.86	19.75	.23
.17	.00	5.08	.90	10.00	13.66	14.92	2.28	19.83	.22
.25	.00	5.17	1.00	10.08	13.78	15.00	1.68	19.92	.21
.33	.00	5.25	1.11	10.17	13.86	15.08	1.48	20.00	.21
.42	.00	5.33	1.22	10.25	13.91	15.17	1.39	20.08	.20
.50	.00	5.42	1.42	10.33	13.92	15.25	1.33	20.17	.19
.58	.00	5.50	1.75	10.42	13.90	15.33	1.28	20.25	.19
.67	.00	5.58	2.06	10.50	13.85	15.42	1.24	20.33	.18
.75	.00	5.67	2.35	10.58	13.77	15.50	1.20	20.42	.18
.83	.00	5.75	2.64	10.67	13.67	15.58	1.15	20.50	.17
.92	.00	5.83	2.93	10.75	13.55	15.67	1.12	20.58	.17
1.00	.00	5.92	3.22	10.83	13.41	15.75	1.08	20.67	.16
1.08	.00	6.00	3.52	10.92	13.26	15.83	1.04	20.75	.16
1.17	.00	6.08	3.81	11.00	13.08	15.92	1.01	20.83	.15
1.25	.00	6.17	4.07	11.08	12.90	16.00	.97	20.92	.15
1.33	.00	6.25	4.29	11.17	12.70	16.08	.94	21.00	.14
1.42	.00	6.33	4.48	11.25	12.50	16.17	.91	21.08	.14
1.50	.00	6.42	4.64	11.33	12.28	16.25	.88	21.17	.13
1.58	.00	6.50	4.77	11.42	12.06	16.33	.85	21.25	.13
1.67	.00	6.58	4.91	11.50	11.84	16.42	.82	21.33	.12
1.75	.00	6.67	5.03	11.58	11.61	16.50	.79	21.42	.12
1.83	.00	6.75	5.16	11.67	11.37	16.58	.77	21.50	.12
1.92	.00	6.83	5.28	11.75	11.13	16.67	.74	21.58	.11
2.00	.00	6.92	5.41	11.83	10.89	16.75	.72	21.67	.11
2.08	.00	7.00	5.53	11.92	10.65	16.83	.69	21.75	.10

2.17	.00	7.08	5.67	12.00	10.42	16.92	.67	21.83	.10
2.25	.00	7.17	5.81	12.08	10.18	17.00	.65	21.92	.10
2.33	.00	7.25	5.96	12.17	9.94	17.08	.63	22.00	.09
2.42	.00	7.33	6.14	12.25	9.70	17.17	.61	22.08	.09
2.50	.00	7.42	6.34	12.33	9.47	17.25	.59	22.17	.09
2.58	.00	7.50	6.57	12.42	9.24	17.33	.57	22.25	.09
2.67	.00	7.58	6.83	12.50	9.01	17.42	.55	22.33	.08
2.75	.00	7.67	7.12	12.58	8.78	17.50	.53	22.42	.08
2.83	.00	7.75	7.44	12.67	8.56	17.58	.51	22.50	.08
2.92	.00	7.83	7.77	12.75	8.35	17.67	.50	22.58	.07
3.00	.00	7.92	8.11	12.83	8.13	17.75	.48	22.67	.07
3.08	.00	8.00	8.43	12.92	7.92	17.83	.47	22.75	.07
3.17	.00	8.08	8.73	13.00	7.72	17.92	.45	22.83	.07
3.25	.00	8.17	9.02	13.08	7.52	18.00	.44	22.92	.07
3.33	.01	8.25	9.27	13.17	7.32	18.08	.42	23.00	.06
3.42	.01	8.33	9.50	13.25	7.13	18.17	.41	23.08	.06
3.50	.02	8.42	9.70	13.33	6.94	18.25	.40	23.17	.06
3.58	.02	8.50	9.89	13.42	6.75	18.33	.38	23.25	.06
3.67	.04	8.58	10.06	13.50	6.57	18.42	.37	23.33	.05
3.75	.05	8.67	10.22	13.58	6.38	18.50	.36	23.42	.05
3.83	.07	8.75	10.38	13.67	6.20	18.58	.35	23.50	.05
3.92	.09	8.83	10.54	13.75	6.01	18.67	.34	23.58	.05
4.00	.12	8.92	10.70	13.83	5.82	18.75	.33	23.67	.05
4.08	.15	9.00	10.87	13.92	5.62	18.83	.32	23.75	.04
4.17	.19	9.08	11.06	14.00	5.42	18.92	.31	23.83	.04
4.25	.23	9.17	11.28	14.08	5.22	19.00	.30	23.92	.04
4.33	.27	9.25	11.51	14.17	5.01	19.08	.29	24.00	.04
4.42	.32	9.33	11.76	14.25	4.80	19.17	.28	24.08	.04
4.50	.37	9.42	12.03	14.33	4.58	19.25	.27	24.17	.03
4.58	.43	9.50	12.30	14.42	4.36	19.33	.26	24.25	.03
4.67	.50	9.58	12.57	14.50	4.14	19.42	.25		
4.75	.56	9.67	12.84	14.58	3.90	19.50	.25		
4.83	.64	9.75	13.09	14.67	3.62	19.58	.24		

```

-----
| ADD HYD (0001) |
| 1 + 2 = 3 |
-----
          AREA      QPEAK      TPEAK      R.V.
          (ha)      (cms)      (hrs)      (mm)
ID1= 1 (0001):   15.60       1.67       3.75    60.65
+ ID2= 2 (0001): 1359.00     13.92     10.33    22.39
=====
ID = 3 (0001):  1374.60     13.95     10.33    22.82

```

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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-----
| SAVE HYD (0001) | AREA      (ha)=1374.60
| ID= 3 PCYC=576 | QPEAK      (cms)= 13.95 (i)
| DT= 5.0 min | TPEAK      (hrs)= 10.33
-----
VOLUME      (mm)= 22.82

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Filename: BLVLPost.TXT

Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* 25mm Event - Post-Development CCE, FOW to SWMF

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-----
| READ STORM | Filename: quality.stm
| Ptotal= 25.00 mm | Comments: 4 Hr Chicago 25mm
-----

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TIME	RAIN	TIME	RAIN	TIME	RAIN	TIME	RAIN
hrs	mm/hr	hrs	mm/hr	hrs	mm/hr	hrs	mm/hr
.08	1.73	1.08	7.10	2.08	4.61	3.08	2.28
.17	1.83	1.17	10.69	2.17	4.21	3.17	2.19
.25	1.94	1.25	23.51	2.25	3.88	3.25	2.11
.33	2.07	1.33	68.86	2.33	3.61	3.33	2.04
.42	2.23	1.42	29.90	2.42	3.38	3.42	1.97
.50	2.41	1.50	17.13	2.50	3.18	3.50	1.91
.58	2.64	1.58	12.05	2.58	3.00	3.58	1.86
.67	2.92	1.67	9.35	2.67	2.84	3.67	1.80
.75	3.28	1.75	7.68	2.75	2.70	3.75	1.75
.83	3.74	1.83	6.55	2.83	2.58	3.83	1.70
.92	4.41	1.92	5.73	2.92	2.47	3.92	1.66
1.00	5.41	2.00	5.10	3.00	2.37	4.00	1.62

* Post-Development (CT 201 + FOW)

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-----
| CALIB |
| STANDHYD (0001) | Area (ha)= 75.10
| ID= 1 DT= 5.0 min | Total Imp(%)= 55.00 Dir. Conn.(%)= 40.00
-----

```

	IMPERVIOUS	PERVIOUS (i)	
Surface Area	(ha)= 41.31	33.79	
Dep. Storage	(mm)= 2.00	5.00	
Average Slope	(%)= 2.00	2.00	
Length	(m)= 708.00	708.00	
Mannings n	= .013	.250	
Max.eff.Inten.(mm/hr)=	49.38	2.13	
over (min)	10.00	185.00	
Storage Coeff. (min)=	8.90 (ii)	193.48 (ii)	
Unit Hyd. Tpeak (min)=	10.00	195.00	
Unit Hyd. peak (cms)=	.12	.01	
			TOTALS
PEAK FLOW (cms)=	3.04	.10	3.04 (iii)
TIME TO PEAK (hrs)=	1.75	5.33	1.75
RUNOFF VOLUME (mm)=	23.00	4.92	12.14
TOTAL RAINFALL (mm)=	25.00	25.00	25.00
RUNOFF COEFFICIENT =	.92	.20	.49

- (i) CN PROCEDURE SELECTED FOR RAINFALL LOSSES:
 CN* = 73.0 Ia = Dep. Storage (Above)
 (ii) COMPUTATIONAL TIME STEP SHOULD BE SMALL OR EQUAL
 THAN THE STORAGE COEFFICIENT.
 (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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-----
| SAVE HYD (0001) | AREA (ha)= 75.10

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| ID= 1 PCYC=256 | QPEAK (cms)= 3.04 (i)
| DT= 5.0 min | TPEAK (hrs)= 1.75
----- VOLUME (mm)= 12.14

Filename: CCE25IN.TXT
Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Route through SWMF

| RESERVOIR (0001) |
| IN= 1---> OUT= 2 |
DT= 5.0 min

OUTFLOW (cms)	STORAGE (ha.m.)	OUTFLOW (cms)	STORAGE (ha.m.)
.000	.000	.223	.409
.004	.049	.321	.519
.017	.097	.636	.632
.036	.148	1.110	.747
.063	.198	2.378	.986
.096	.250	3.971	1.254
.134	.301	5.830	1.496

	AREA (ha)	QPEAK (cms)	TPEAK (hrs)	R.V. (mm)
INFLOW : ID= 1 (0001)	75.10	3.04	1.75	12.14
OUTFLOW: ID= 2 (0001)	75.10	.29	3.08	11.97

PEAK FLOW REDUCTION [Qout/Qin](%)= 9.66
TIME SHIFT OF PEAK FLOW (min)= 80.00
MAXIMUM STORAGE USED (ha.m.)= .49

| SAVE HYD (0001) | AREA (ha)= 75.10
| ID= 2 PCYC=*** | QPEAK (cms)= .29 (i)
| DT= 5.0 min | TPEAK (hrs)= 3.08
----- VOLUME (mm)= 11.97

Filename: CCE25OUT.TXT
Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* 100Yr Lane Creek Flows to Millennium Trail (pre-dev)

* Flows to Node B (Millennium Trail Crossing)

| READ HYD (0001) | AREA (ha)=1456.00
| ID= 1 PCYC= 1 | QPEAK (cms)= 14.17
| DT= 5.0 min | TPEAK (hrs)= 10.58
----- VOLUME (mm)= 22.75

Filename: PREDEV.TXT
Comments: Flows to Millennium Trail (Existing)

TIME hrs	FLOW cms	TIME hrs	FLOW cms	TIME hrs	FLOW cms	TIME hrs	FLOW cms	TIME hrs	FLOW cms
.00	.00	5.67	3.64	11.33	13.21	17.00	.72	.00	.00

.08	.00	5.75	3.72	11.42	13.02	17.08	.69	.00	.00
.17	.00	5.83	3.81	11.50	12.82	17.17	.67	.00	.00
.25	.00	5.92	3.92	11.58	12.61	17.25	.65	.00	.00
.33	.00	6.00	4.05	11.67	12.39	17.33	.63	.00	.00
.42	.00	6.08	4.21	11.75	12.16	17.42	.61	.00	.00
.50	.00	6.17	4.41	11.83	11.93	17.50	.59	.00	.00
.58	.01	6.25	4.63	11.92	11.70	17.58	.57	.00	.00
.67	.01	6.33	4.88	12.00	11.47	17.67	.55	.00	.00
.75	.01	6.42	5.13	12.08	11.23	17.75	.53	.00	.00
.83	.01	6.50	5.38	12.17	10.99	17.83	.52	.00	.00
.92	.02	6.58	5.61	12.25	10.75	17.92	.50	.00	.00
1.00	.02	6.67	5.81	12.33	10.52	18.00	.49	.00	.00
1.08	.03	6.75	5.99	12.42	10.28	.00	-1.00	.00	.00
1.17	.04	6.83	6.14	12.50	10.04	.00	.00	.00	.00
1.25	.04	6.92	6.26	12.58	9.81	.00	.00	.00	.00
1.33	.05	7.00	6.38	12.67	9.58	.00	.00	.00	.00
1.42	.06	7.08	6.48	12.75	9.35	.00	.00	.00	.00
1.50	.06	7.17	6.57	12.83	9.12	.00	.00	.00	.00
1.58	.07	7.25	6.67	12.92	8.90	.00	.00	.00	.00
1.67	.08	7.33	6.76	13.00	8.68	.00	.00	.00	.00
1.75	.08	7.42	6.86	13.08	8.47	.00	.00	.00	.00
1.83	.09	7.50	6.97	13.17	8.26	.00	.00	.00	.00
1.92	.10	7.58	7.09	13.25	8.05	.00	.00	.00	.00
2.00	.11	7.67	7.23	13.33	7.85	.00	.00	.00	.00
2.08	.11	7.75	7.40	13.42	7.65	.00	.00	.00	.00
2.17	.12	7.83	7.60	13.50	7.46	.00	.00	.00	.00
2.25	.12	7.92	7.82	13.58	7.26	.00	.00	.00	.00
2.33	.13	8.00	8.08	13.67	7.08	.00	.00	.00	.00
2.42	.14	8.08	8.35	13.75	6.89	.00	.00	.00	.00
2.50	.14	8.17	8.64	13.83	6.71	.00	.00	.00	.00
2.58	.15	8.25	8.94	13.92	6.53	.00	.00	.00	.00
2.67	.15	8.33	9.22	14.00	6.34	.00	.00	.00	.00
2.75	.15	8.42	9.50	14.08	6.16	.00	.00	.00	.00
2.83	.16	8.50	9.75	14.17	5.97	.00	.00	.00	.00
2.92	.16	8.58	9.99	14.25	5.78	.00	.00	.00	.00
3.00	.17	8.67	10.20	14.33	5.59	.00	.00	.00	.00
3.08	.17	8.75	10.39	14.42	5.40	.00	.00	.00	.00
3.17	.18	8.83	10.57	14.50	5.20	.00	.00	.00	.00
3.25	.19	8.92	10.73	14.58	5.00	.00	.00	.00	.00
3.33	.20	9.00	10.88	14.67	4.79	.00	.00	.00	.00
3.42	.22	9.08	11.02	14.75	4.59	.00	.00	.00	.00
3.50	.25	9.17	11.17	14.83	4.37	.00	.00	.00	.00
3.58	.28	9.25	11.33	14.92	4.14	.00	.00	.00	.00
3.67	.31	9.33	11.50	15.00	3.87	.00	.00	.00	.00
3.75	.36	9.42	11.68	15.08	3.57	.00	.00	.00	.00
3.83	.42	9.50	11.88	15.17	3.22	.00	.00	.00	.00
3.92	.49	9.58	12.10	15.25	2.80	.00	.00	.00	.00
4.00	.57	9.67	12.34	15.33	2.28	.00	.00	.00	.00
4.08	.67	9.75	12.58	15.42	1.60	.00	.00	.00	.00
4.17	.79	9.83	12.83	15.50	1.38	.00	.00	.00	.00
4.25	.93	9.92	13.08	15.58	1.29	.00	.00	.00	.00
4.33	1.08	10.00	13.31	15.67	1.23	.00	.00	.00	.00
4.42	1.24	10.08	13.52	15.75	1.19	.00	.00	.00	.00
4.50	1.40	10.17	13.71	15.83	1.15	.00	.00	.00	.00
4.58	1.56	10.25	13.87	15.92	1.11	.00	.00	.00	.00
4.67	1.73	10.33	14.00	16.00	1.07	.00	.00	.00	.00
4.75	1.90	10.42	14.09	16.08	1.04	.00	.00	.00	.00
4.83	2.06	10.50	14.14	16.17	1.00	.00	.00	.00	.00
4.92	2.22	10.58	14.17	16.25	.97	.00	.00	.00	.00

5.00	2.37	10.67	14.16	16.33	.94	.00	.00	.00	.00
5.08	2.52	10.75	14.12	16.42	.91	.00	.00	.00	.00
5.17	2.66	10.83	14.05	16.50	.88	.00	.00	.00	.00
5.25	2.80	10.92	13.96	16.58	.85	.00	.00	.00	.00
5.33	2.94	11.00	13.84	16.67	.82	.00	.00		
5.42	3.08	11.08	13.71	16.75	.79	.00	.00		
5.50	3.26	11.17	13.56	16.83	.77	.00	.00		
5.58	3.48	11.25	13.39	16.92	.74	.00	.00		

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-----
| SAVE HYD (0001) | AREA      (ha)=1456.00
| ID= 1  PCYC=217 | QPEAK    (cms)= 14.17 (i)
| DT= 5.0 min     | TPEAK    (hrs)= 10.58
-----
                        VOLUME  (mm)= 22.75

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Filename: TRAILPRE.TXT

Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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*****
*                               Lane Creek Flows to Millennium Trail (post-dev)
*****
*                               Flows to Node B (Millennium Trail Crossing)

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| READ HYD (0001) | AREA      (ha)=1359.00
| ID= 9  PCYC= 1  | QPEAK    (cms)= 13.86
| DT= 5.0 min     | TPEAK    (hrs)= 10.67
-----
                        VOLUME  (mm)= 21.70

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Filename: TRAIL.TXT

Comments: Flows to Millennium Trail (no development lands)

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
hrs	cms	hrs	cms	hrs	cms	hrs	cms	hrs	cms
.00	.00	3.67	.01	7.33	5.45	11.00	13.59	14.67	4.75
.08	.00	3.75	.01	7.42	5.59	11.08	13.47	14.75	4.54
.17	.00	3.83	.01	7.50	5.74	11.17	13.33	14.83	4.33
.25	.00	3.92	.02	7.58	5.91	11.25	13.17	14.92	4.10
.33	.00	4.00	.02	7.67	6.09	11.33	13.00	15.00	3.84
.42	.00	4.08	.03	7.75	6.29	11.42	12.82	15.08	3.53
.50	.00	4.17	.06	7.83	6.53	11.50	12.62	15.17	3.16
.58	.00	4.25	.09	7.92	6.79	11.58	12.42	15.25	2.70
.67	.00	4.33	.12	8.00	7.07	11.67	12.21	15.33	2.16
.75	.00	4.42	.17	8.08	7.38	11.75	11.99	15.42	1.54
.83	.00	4.50	.22	8.17	7.71	11.83	11.77	15.50	1.34
.92	.00	4.58	.27	8.25	8.03	11.92	11.55	15.58	1.26
1.00	.00	4.67	.33	8.33	8.35	12.00	11.32	15.67	1.20
1.08	.00	4.75	.39	8.42	8.66	12.08	11.09	15.75	1.16
1.17	.00	4.83	.45	8.50	8.95	12.17	10.85	15.83	1.12
1.25	.00	4.92	.53	8.58	9.21	12.25	10.62	15.92	1.08
1.33	.00	5.00	.61	8.67	9.45	12.33	10.39	16.00	1.05
1.42	.00	5.08	.69	8.75	9.67	12.42	10.16	.00	-1.00
1.50	.00	5.17	.78	8.83	9.86	12.50	9.92	.00	.00
1.58	.00	5.25	.88	8.92	10.05	12.58	9.70	.00	.00
1.67	.00	5.33	.98	9.00	10.22	12.67	9.47	.00	.00
1.75	.00	5.42	1.10	9.08	10.39	12.75	9.24	.00	.00
1.83	.00	5.50	1.27	9.17	10.56	12.83	9.02	.00	.00
1.92	.00	5.58	1.50	9.25	10.74	12.92	8.80	.00	.00
2.00	.00	5.67	1.74	9.33	10.93	13.00	8.59	.00	.00

2.08	.00	5.75	1.89	9.42	11.13	13.08	8.38	.00	.00
2.17	.00	5.83	2.00	9.50	11.35	13.17	8.17	.00	.00
2.25	.00	5.92	2.13	9.58	11.59	13.25	7.97	.00	.00
2.33	.00	6.00	2.27	9.67	11.85	13.33	7.77	.00	.00
2.42	.00	6.08	2.44	9.75	12.11	13.42	7.57	.00	.00
2.50	.00	6.17	2.65	9.83	12.38	13.50	7.38	.00	.00
2.58	.00	6.25	2.89	9.92	12.64	13.58	7.19	.00	.00
2.67	.00	6.33	3.15	10.00	12.89	13.67	7.01	.00	.00
2.75	.00	6.42	3.42	10.08	13.12	13.75	6.83	.00	.00
2.83	.00	6.50	3.69	10.17	13.33	13.83	6.64	.00	.00
2.92	.00	6.58	3.95	10.25	13.50	13.92	6.46	.00	.00
3.00	.00	6.67	4.18	10.33	13.64	14.00	6.28	.00	.00
3.08	.00	6.75	4.39	10.42	13.75	14.08	6.10	.00	.00
3.17	.00	6.83	4.58	10.50	13.82	14.17	5.92	.00	.00
3.25	.00	6.92	4.74	10.58	13.85	14.25	5.73	.00	.00
3.33	.00	7.00	4.90	10.67	13.86	14.33	5.54	.00	.00
3.42	.00	7.08	5.04	10.75	13.83	14.42	5.35		
3.50	.00	7.17	5.18	10.83	13.77	14.50	5.15		
3.58	.00	7.25	5.31	10.92	13.69	14.58	4.95		

* Phase 3(E) SWMF flows

READ HYD (0001)	AREA	(ha)=	15.60
ID= 1 PCYC= 1	QPEAK	(cms)=	1.66
DT= 5.0 min	TPEAK	(hrs)=	3.75
	VOLUME	(mm)=	60.58

Filename: K3E00PQR.TXT

Comments:

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
hrs	cms	hrs	cms	hrs	cms	hrs	cms	hrs	cms
.00	.00	3.67	1.57	7.33	.04	11.00	.03	14.67	.03
.08	.00	3.75	1.66	7.42	.04	11.08	.03	14.75	.03
.17	.00	3.83	1.14	7.50	.04	11.17	.03	14.83	.03
.25	.00	3.92	1.10	7.58	.04	11.25	.03	14.92	.03
.33	.00	4.00	1.03	7.67	.04	11.33	.03	15.00	.03
.42	.00	4.08	.96	7.75	.04	11.42	.03	15.08	.03
.50	.00	4.17	.90	7.83	.04	11.50	.03	15.17	.03
.58	.00	4.25	.86	7.92	.04	11.58	.03	15.25	.03
.67	.00	4.33	.79	8.00	.04	11.67	.03	15.33	.03
.75	.00	4.42	.71	8.08	.04	11.75	.03	15.42	.03
.83	.00	4.50	.65	8.17	.04	11.83	.03	15.50	.03
.92	.00	4.58	.60	8.25	.04	11.92	.03	15.58	.03
1.00	.00	4.67	.55	8.33	.04	12.00	.03	15.67	.03
1.08	.00	4.75	.51	8.42	.04	12.08	.03	15.75	.03
1.17	.00	4.83	.48	8.50	.04	12.17	.03	15.83	.02
1.25	.00	4.92	.44	8.58	.04	12.25	.03	15.92	.02
1.33	.00	5.00	.41	8.67	.04	12.33	.03	16.00	.02
1.42	.00	5.08	.39	8.75	.04	12.42	.03	16.08	.02
1.50	.01	5.17	.37	8.83	.04	12.50	.03	16.17	.02
1.58	.01	5.25	.35	8.92	.04	12.58	.03	16.25	.02
1.67	.01	5.33	.33	9.00	.04	12.67	.03	16.33	.02
1.75	.01	5.42	.31	9.08	.04	12.75	.03	16.42	.02
1.83	.01	5.50	.29	9.17	.04	12.83	.03	16.50	.02
1.92	.02	5.58	.28	9.25	.04	12.92	.03	16.58	.02
2.00	.02	5.67	.27	9.33	.04	13.00	.03	16.67	.02
2.08	.02	5.75	.27	9.42	.04	13.08	.03	16.75	.02
2.17	.02	5.83	.26	9.50	.04	13.17	.03	16.83	.02
2.25	.03	5.92	.26	9.58	.04	13.25	.03	16.92	.02

2.33	.03	6.00	.26	9.67	.04	13.33	.03	17.00	.02
2.42	.03	6.08	.25	9.75	.04	13.42	.03	17.08	.02
2.50	.04	6.17	.25	9.83	.03	13.50	.03	17.17	.02
2.58	.04	6.25	.23	9.92	.03	13.58	.03	17.25	.02
2.67	.05	6.33	.21	10.00	.03	13.67	.03	17.33	.02
2.75	.05	6.42	.18	10.08	.03	13.75	.03	17.42	.02
2.83	.06	6.50	.16	10.17	.03	13.83	.03	17.50	.02
2.92	.07	6.58	.14	10.25	.03	13.92	.03	17.58	.02
3.00	.08	6.67	.12	10.33	.03	14.00	.03	17.67	.02
3.08	.10	6.75	.11	10.42	.03	14.08	.03	17.75	.02
3.17	.13	6.83	.10	10.50	.03	14.17	.03	17.83	.02
3.25	.18	6.92	.08	10.58	.03	14.25	.03	17.92	.02
3.33	.53	7.00	.07	10.67	.03	14.33	.03	18.00	.02
3.42	.96	7.08	.06	10.75	.03	14.42	.03		
3.50	1.26	7.17	.06	10.83	.03	14.50	.03		
3.58	1.47	7.25	.05	10.92	.03	14.58	.03		

* Shift flows from Phase 3(E) to account for routing

SHIFT HYD (0001)				
IN= 1---> OUT= 8				
SHIFT= 5.0 min				
	AREA	QPEAK	TPEAK	R.V.
	(ha)	(cms)	(hrs)	(mm)
ID= 1 (0001):	15.60	1.66	3.75	60.58
SHIFT ID= 8 (0001):	15.60	1.66	3.83	60.58

* ***** Flows from CCE SWMF *****

READ STORM		Filename: 6scs72.stm					
Ptotal= 84.70 mm		Comments: 6 Hr SCS Type 2 - 100Yr					
TIME	RAIN	TIME	RAIN	TIME	RAIN	TIME	RAIN
hrs	mm/hr	hrs	mm/hr	hrs	mm/hr	hrs	mm/hr
.50	3.39	2.00	8.47	3.50	66.07	5.00	6.78
1.00	5.08	2.50	10.16	4.00	18.63	5.50	5.08
1.50	5.08	3.00	25.41	4.50	8.47	6.00	6.78

CALIB			
STANDHYD (0001)		Area (ha)= 75.10	
ID= 1 DT= 5.0 min		Total Imp(%)= 55.00	Dir. Conn.(%)= 40.00

	IMPERVIOUS	PERVIOUS (i)
Surface Area (ha)=	41.31	33.79
Dep. Storage (mm)=	2.00	5.00
Average Slope (%)=	2.00	2.00
Length (m)=	708.00	708.00
Mannings n =	.013	.250

NOTE: RAINFALL WAS TRANSFORMED TO 5.0 MIN. TIME STEP.

	IMPERVIOUS	PERVIOUS (i)
Max.eff.Inten.(mm/hr)=	66.07	33.62
over (min)	10.00	65.00
Storage Coeff. (min)=	7.93 (ii)	69.13 (ii)

Unit Hyd. Tpeak (min)=	10.00	70.00	
Unit Hyd. peak (cms)=	.13	.02	
			TOTALS
PEAK FLOW (cms)=	5.39	1.89	6.06 (iii)
TIME TO PEAK (hrs)=	3.75	4.75	3.75
RUNOFF VOLUME (mm)=	82.13	42.84	58.55
TOTAL RAINFALL (mm)=	84.70	84.70	84.70
RUNOFF COEFFICIENT =	.97	.51	.69

- (i) CN PROCEDURE SELECTED FOR RAINFALL LOSSES:
 CN* = 73.0 Ia = Dep. Storage (Above)
- (ii) COMPUTATIONAL TIME STEP SHOULD BE SMALL OR EQUAL
 THAN THE STORAGE COEFFICIENT.
- (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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-----
| SAVE HYD (0001) | AREA (ha)= 75.10
| ID= 1 PCYC=176 | QPEAK (cms)= 6.06 (i)
| DT= 5.0 min | TPEAK (hrs)= 3.75
|-----| VOLUME (mm)= 58.55
Filename: CCEUNC.TXT
Comments:

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- (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Subtract 25-mm, bypass all other flows

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-----
| DUHYD (0001) |
| Inlet Cap.=3.038 |
| #of Inlets= 1 |
| Total(cms)= 3.0 |
|-----|
| AREA QPEAK TPEAK R.V. |
| (ha) (cms) (hrs) (mm) |
|-----|
TOTAL HYD.(ID= 1): 75.10 6.06 3.75 58.55
=====
MAJOR SYS.(ID= 2): 8.18 3.02 3.75 58.55
MINOR SYS.(ID= 3): 66.92 3.04 3.42 58.55

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NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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-----
| SAVE HYD (0001) | AREA (ha)= 8.18
| ID= 2 PCYC= 52 | QPEAK (cms)= 3.02 (i)
| DT= 5.0 min | TPEAK (hrs)= 3.75
|-----| VOLUME (mm)= 58.55
Filename: CCEMAJ.TXT
Comments:

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- (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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-----
| SAVE HYD (0001) | AREA (ha)= 66.92
| ID= 3 PCYC=176 | QPEAK (cms)= 3.04 (i)
| DT= 5.0 min | TPEAK (hrs)= 3.42
|-----| VOLUME (mm)= 58.55

```

Filename: CCEQIN.TXT

Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Route through SWMF				

RESERVOIR (0001)				
IN= 3---> OUT= 1				
DT= 5.0 min				

	OUTFLOW	STORAGE	OUTFLOW	STORAGE
	(cms)	(ha.m.)	(cms)	(ha.m.)
	.000	.000	.223	.409
	.004	.049	.321	.519
	.017	.097	.636	.632
	.036	.148	1.110	.747
	.063	.198	2.378	.986
	.096	.250	3.971	1.254
	.134	.301	5.830	1.496
	AREA	QPEAK	TPEAK	R.V.
	(ha)	(cms)	(hrs)	(mm)
INFLOW : ID= 3 (0001)	66.92	3.04	3.42	58.55
OUTFLOW: ID= 1 (0001)	66.92	2.74	4.42	58.35
	PEAK FLOW REDUCTION [Qout/Qin](%)= 90.27			
	TIME SHIFT OF PEAK FLOW (min)= 60.00			
	MAXIMUM STORAGE USED (ha.m.)= 1.05			

SAVE HYD (0001)				
ID= 1 PCYC=***				
DT= 5.0 min				

	AREA	(ha)=	66.92	
	QPEAK	(cms)=	2.74 (i)	
	TPEAK	(hrs)=	4.42	
	VOLUME	(mm)=	58.35	

Filename: CCEQOUT.TXT				
Comments:				

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

ADD HYD (0001)				
1 + 2 = 3				

	AREA	QPEAK	TPEAK	R.V.
	(ha)	(cms)	(hrs)	(mm)
ID1= 1 (0001):	66.92	2.74	4.42	58.35
+ ID2= 2 (0001):	8.18	3.02	3.75	58.55
=====				
ID = 3 (0001):	75.10	4.98	3.75	58.38

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

CALIB				
NASHYD (0001)				
ID= 2 DT= 5.0 min				

	Area	(ha)=	3.20	Curve Number (CN)= 73.0
	Ia	(mm)=	5.00	# of Linear Res.(N)= 3.00
	U.H. Tp	(hrs)=	.31	

Unit Hyd Qpeak (cms)= .39

PEAK FLOW (cms)= .24 (i)

TIME TO PEAK (hrs)= 3.58

RUNOFF VOLUME (mm)= 36.14

TOTAL RAINFALL (mm)= 84.70

RUNOFF COEFFICIENT = .43

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```
-----
| ADD HYD (0001) |
| 2 + 3 = 1 |
-----
                AREA      QPEAK      TPEAK      R.V.
                (ha)      (cms)      (hrs)      (mm)
ID1= 2 (0001):  3.20      .24      3.58      36.14
+ ID2= 3 (0001): 75.10      4.98      3.75      58.38
=====
ID = 1 (0001):  78.30      5.21      3.75      57.47
```

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

```
-----
| SAVE HYD (0001) | AREA      (ha)= 78.30
| ID= 1 PCYC=*** | QPEAK      (cms)= 5.21 (i)
| DT= 5.0 min | TPEAK      (hrs)= 3.75
-----
                VOLUME      (mm)= 57.47
```

Filename: CCEQTOTL.TXT

Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Flows from Fields of Wellington Development

```
-----
| READ HYD (0001) | AREA      (ha)= 30.75
| ID= 2 PCYC= 1 | QPEAK      (cms)= 1.77
| DT= 5.0 min | TPEAK      (hrs)= 4.58
-----
                VOLUME      (mm)= 49.47
```

Filename: FOWPOST.TXT

Comments:

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
hrs	cms	hrs	cms	hrs	cms	hrs	cms	hrs	cms
.00	.00	6.00	1.01	12.00	.06	.00	.00	.00	.00
.08	.00	6.08	.97	12.08	.06	.00	.00	.00	.00
.17	.00	6.17	.75	12.17	.06	.00	.00	.00	.00
.25	.00	6.25	.58	12.25	.06	.00	.00	.00	.00
.33	.00	6.33	.53	12.33	.05	.00	.00	.00	.00
.42	.00	6.42	.51	12.42	.05	.00	.00	.00	.00
.50	.00	6.50	.49	12.50	.05	.00	.00	.00	.00
.58	.00	6.58	.47	12.58	.05	.00	.00	.00	.00
.67	.00	6.67	.43	12.67	.05	.00	.00	.00	.00
.75	.00	6.75	.40	12.75	.05	.00	.00	.00	.00
.83	.00	6.83	.36	12.83	.05	.00	.00	.00	.00
.92	.00	6.92	.33	12.92	.05	.00	.00	.00	.00

1.00	.00	7.00	.30	13.00	.05	.00	.00	.00	.00
1.08	.00	7.08	.28	13.08	.05	.00	.00	.00	.00
1.17	.00	7.17	.25	13.17	.05	.00	.00	.00	.00
1.25	.00	7.25	.23	13.25	.05	.00	.00	.00	.00
1.33	.00	7.33	.21	13.33	.05	.00	.00	.00	.00
1.42	.00	7.42	.19	13.42	.05	.00	.00	.00	.00
1.50	.00	7.50	.18	13.50	.05	.00	.00	.00	.00
1.58	.00	7.58	.16	13.58	.05	.00	.00	.00	.00
1.67	.01	7.67	.15	13.67	.05	.00	.00	.00	.00
1.75	.01	7.75	.14	13.75	.05	.00	.00	.00	.00
1.83	.01	7.83	.13	13.83	.05	.00	.00	.00	.00
1.92	.01	7.92	.12	13.92	.05	.00	.00	.00	.00
2.00	.01	8.00	.11	14.00	.05	.00	.00	.00	.00
2.08	.01	8.08	.10	14.08	.05	.00	.00	.00	.00
2.17	.01	8.17	.09	14.17	.05	.00	.00	.00	.00
2.25	.01	8.25	.08	14.25	.05	.00	.00	.00	.00
2.33	.01	8.33	.08	14.33	.05	.00	.00	.00	.00
2.42	.01	8.42	.07	14.42	.05	.00	.00	.00	.00
2.50	.01	8.50	.06	14.50	.05	.00	.00	.00	.00
2.58	.01	8.58	.06	14.58	.05	.00	.00	.00	.00
2.67	.01	8.67	.06	14.67	.05	.00	.00	.00	.00
2.75	.01	8.75	.06	14.75	.05	.00	.00	.00	.00
2.83	.01	8.83	.06	14.83	.05	.00	.00	.00	.00
2.92	.01	8.92	.06	14.92	.05	.00	.00	.00	.00
3.00	.01	9.00	.06	15.00	.05	.00	.00	.00	.00
3.08	.01	9.08	.06	15.08	.05	.00	.00	.00	.00
3.17	.01	9.17	.06	15.17	.05	.00	.00	.00	.00
3.25	.01	9.25	.06	15.25	.05	.00	.00	.00	.00
3.33	.01	9.33	.06	15.33	.05	.00	.00	.00	.00
3.42	.01	9.42	.06	15.42	.05	.00	.00	.00	.00
3.50	.02	9.50	.06	15.50	.05	.00	.00	.00	.00
3.58	.02	9.58	.06	15.58	.05	.00	.00	.00	.00
3.67	.03	9.67	.06	15.67	.05	.00	.00	.00	.00
3.75	.04	9.75	.06	15.75	.05	.00	.00	.00	.00
3.83	.05	9.83	.06	15.83	.05	.00	.00	.00	.00
3.92	.45	9.92	.06	15.92	.05	.00	.00	.00	.00
4.00	1.19	10.00	.06	16.00	.05	.00	.00	.00	.00
4.08	1.35	10.08	.06	.00	-1.00	.00	.00	.00	.00
4.17	1.49	10.17	.06	.00	.00	.00	.00	.00	.00
4.25	1.55	10.25	.06	.00	.00	.00	.00	.00	.00
4.33	1.62	10.33	.06	.00	.00	.00	.00	.00	.00
4.42	1.70	10.42	.06	.00	.00	.00	.00	.00	.00
4.50	1.75	10.50	.06	.00	.00	.00	.00	.00	.00
4.58	1.77	10.58	.06	.00	.00	.00	.00	.00	.00
4.67	1.75	10.67	.06	.00	.00	.00	.00	.00	.00
4.75	1.71	10.75	.06	.00	.00	.00	.00	.00	.00
4.83	1.67	10.83	.06	.00	.00	.00	.00	.00	.00
4.92	1.61	10.92	.06	.00	.00	.00	.00	.00	.00
5.00	1.56	11.00	.06	.00	.00	.00	.00	.00	.00
5.08	1.55	11.08	.06	.00	.00	.00	.00	.00	.00
5.17	1.54	11.17	.06	.00	.00	.00	.00	.00	.00
5.25	1.48	11.25	.06	.00	.00	.00	.00	.00	.00
5.33	1.42	11.33	.06	.00	.00	.00	.00	.00	.00
5.42	1.37	11.42	.06	.00	.00	.00	.00	.00	.00
5.50	1.30	11.50	.06	.00	.00	.00	.00	.00	.00
5.58	1.25	11.58	.06	.00	.00	.00	.00	.00	.00
5.67	1.20	11.67	.06	.00	.00	.00	.00		
5.75	1.15	11.75	.06	.00	.00	.00	.00		
5.83	1.10	11.83	.06	.00	.00	.00	.00		

5.92 1.05 | 11.92 .06 | .00 .00 | .00 .00 |

* Add undeveloped, Ph3(E), CCE, FOW Flows

```
-----
| ADD HYD (0001) |
| 1 + 2 = 3 |
-----
          AREA      QPEAK      TPEAK      R.V.
          (ha)      (cms)      (hrs)      (mm)
ID1= 1 (0001):  78.30      5.21      3.75      57.47
+ ID2= 2 (0001):  30.75      1.77      4.58      49.47
=====
ID = 3 (0001):  109.05      5.25      3.75      55.21
```

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

```
-----
| ADD HYD (0001) |
| 3 + 8 = 7 |
-----
          AREA      QPEAK      TPEAK      R.V.
          (ha)      (cms)      (hrs)      (mm)
ID1= 3 (0001):  109.05      5.25      3.75      55.21
+ ID2= 8 (0001):  15.60      1.66      3.83      60.58
=====
ID = 7 (0001):  124.65      6.82      3.75      55.89
```

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

```
-----
| ADD HYD (0001) |
| 7 + 9 = 1 |
-----
          AREA      QPEAK      TPEAK      R.V.
          (ha)      (cms)      (hrs)      (mm)
ID1= 7 (0001):  124.65      6.82      3.75      55.89
+ ID2= 9 (0001): 1359.00     13.86     10.67     21.70
=====
ID = 1 (0001):  1483.65     14.17     10.58     24.58
```

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

```
-----
| SAVE HYD (0001) | AREA      (ha)=1483.65
| ID= 1 PCYC=*** | QPEAK      (cms)= 14.17 (i)
| DT= 5.0 min | TPEAK      (hrs)= 10.58
----- VOLUME      (mm)= 24.58
```

Filename: TRAILPST.TXT

Comments:

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

FINISH

APPENDIX E

Storm Sewer Design Sheet

STORM SEWER DESIGN SHEET																					
Peak Runoff Estimate by Rational Method $Q = \frac{1}{360} C i A$ Where: Q = Peak Flow in cms C = Runoff Coefficient i = Rainfall Intensity in mm/hr A = Area in hectares												Pipe Capacity by Manning's Equation $Q = \frac{1}{n} A R^{2/3} S^{1/2}$ Where: A = area of pipe in m² R = Hydraulic radius = A / P P = Wetted perimeter S = Slope (m/m) n = Manning's friction coef. Check $q \leq Q$ $V \leq 6 \text{ m/s}$									
Intensity for: Trenton 2020 IDF Station: 6158875 5-Year Parameters A = 25.2 B = -0.664 Manning's Coef CSP 0.024 RCP/PVC 0.013																					
$i = A * T_c^B$ Where: i = Rainfall Intensity in mm/hr T_c = Time of Concentration in minutes																					
LOCATION			PEAK FLOW CALCULATION									PROPOSED SEWER									
CATCHMENT	FROM	TO	CATCHMENT AREAS (ha)				C x A	C x A CUM.	TIME OF CONCENTRATION	INTENSITY <i>i</i>	PEAK FLOW Q	PIPE SIZE	LENGTH	PIPE MATERIAL	GRADE USE m/m	CAPACITY n = 0.013 (m ³ /s)	FULL FLOW VELOCITY (m/s)	TIME OF FLOW (min)	ACTUAL VELOCITY AT Q _d (m/s)	CHECK CAPACITY	q/Q
			RUNOFF COEFFICIENT																		
			0.4	0.5	0.6	0.7															
Lane																					
21	ST38	ST37			0.37		0.22	0.22	15.00	63.3	0.039	300	17.5	PVC	0.75%	0.084	1.18	0.25	1.16	OK	0.47
	ST37	ST36					0.00	0.22	15.25	62.6	0.039	300	44.8	PVC	0.75%	0.084	1.18	0.63	1.16	OK	0.46
	ST36	ST35					0.00	0.22	15.88	60.9	0.038	300	35.9	PVC	0.75%	0.084	1.18	0.51	1.15	OK	0.45
20	ST35	ST34			0.44		0.26	0.49	16.38	59.7	0.081	375	89.8	PVC	0.75%	0.152	1.37	1.09	1.39	OK	0.53
Easement																					
19	ST34	ST33			0.58		0.35	0.83	17.47	57.2	0.133	375	12.9	PVC	0.75%	0.152	1.37	0.16	1.55	OK	0.87
	ST33	ST32					0.00	0.83	17.63	56.8	0.132	375	77.9	PVC	1.00%	0.175	1.59	0.82	1.75	OK	0.75
Street G																					
Outlet C & 24	ST32	ST1			20.36		12.22	13.05	20.19	51.9	1.884	1200	30.9	RCP	0.30%	2.135	1.89	0.27	2.13	OK	0.88
Street D																					
18	ST31	ST30			0.52		0.31	0.31	15.00	63.3	0.055	375	43.7	PVC	1.00%	0.175	1.59	0.46	1.40	OK	0.31
	ST30	ST16					0.00	0.31	15.46	62.0	0.054	375	15.5	PVC	1.00%	0.175	1.59	0.16	1.40	OK	0.31
Street D																					
Future - West	ST29	ST28			11.10		6.66	6.66	20.00	52.3	0.968	825	100.6	RCP	0.75%	1.243	2.33	0.72	2.58	OK	0.78
Street C																					
17	ST28	ST27			0.67		0.40	7.06	20.72	51.1	1.002	825	6.8	RCP	0.75%	1.243	2.33	0.05	2.61	OK	0.81
	ST27	ST26					0.00	7.06	20.77	51.0	1.001	825	26.0	RCP	0.75%	1.243	2.33	0.19	2.61	OK	0.80
	ST26	ST25					0.00	7.06	20.96	50.7	0.995	825	65.5	RCP	0.75%	1.243	2.33	0.47	2.60	OK	0.80
16	ST25	ST24			0.58		0.35	7.41	21.43	49.9	1.029	825	16.1	RCP	0.75%	1.243	2.33	0.12	2.61	OK	0.83
	ST24	ST23					0.00	7.41	21.54	49.8	1.025	825	45.4	RCP	0.75%	1.243	2.33	0.33	2.61	OK	0.82
	ST23	ST22					0.00	7.41	21.87	49.3	1.015	825	17.5	RCP	0.75%	1.243	2.33	0.13	2.61	OK	0.82
13 & 14 & 15	ST22	ST21			1.24		0.74	8.15	21.99	49.1	1.112	900	43.5	RCP	0.75%	1.568	2.46	0.29	2.67	OK	0.71
12	ST21	ST11			0.16		0.10	8.25	22.29	48.6	1.116	900	15.3	RCP	0.75%	1.568	2.46	0.10	2.67	OK	0.71
Street B																					
Future - Fields & 24	ST20	ST19			4.75		2.85	2.85	17.00	58.2	0.461	675	36.9	RCP	0.50%	0.594	1.66	0.37	1.84	OK	0.78
6	ST19	ST18			0.48		0.29	3.14	17.37	57.4	0.501	675	34.6	RCP	0.50%	0.594	1.66	0.35	1.87	OK	0.84
	ST18	ST9			0.24		0.14	3.28	17.72	56.6	0.517	600	15.3	RCP	0.50%	0.434	1.54	0.17	1.54	Surch	1.19

Peak Runoff Estimate by Rational Method

$$Q = \frac{1}{360} C i A$$

Where:

Q =

Peak Flow in cms

C =

Runoff Coefficient

i =

Rainfall Intensity in mm/hr

A =

Area in hectares

Intensity for:

Trenton

2020 IDF

Station:

6158875

$$i = A * T_c^B$$

Where:

i =

Rainfall Intensity in mm/hr

T_c =

Time of Concentration in minutes

5-Year Parameters

A = 25.2

B = -0.664

Manning's Coef

CSP

0.024

RCP/PVC

0.013

Pipe Capacity by Manning's Equation

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

Where:

A = area of pipe in m²

R = Hydraulic radius = A / P

P = Wetted perimeter

S = Slope (m/m)

n = Manning's friction coef.

Check

$$q \leq Q$$
$$V \leq 6 \text{ m/s}$$

LOCATION			PEAK FLOW CALCULATION									PROPOSED SEWER									
CATCHMENT	FROM	TO	CATCHMENT AREAS (ha)				C x A	C x A CUM.	TIME OF CONCENTRATION	INTENSITY i	PEAK FLOW Q	PIPE SIZE	LENGTH	PIPE MATERIAL	GRADE USE m/m	CAPACITY n = 0.013	FULL FLOW VELOCITY	TIME OF FLOW	ACTUAL VELOCITY AT Q _d	CHECK CAPACITY	q/Q
			RUNOFF COEFFICIENT																		
			0.4	0.5	0.6	0.7	(ha)	(ha)	(min)	(mm/hr)	(m ³ /s)	(mm)	(m)	(%)	(m ³ /s)	(m/s)	(min)	(m/s)			
Street A																					
Future - Middle	ST17	ST16			12.10		7.26	7.26	20.00	52.3	1.055	900	15.2	RCP	0.50%	1.280	2.01	0.13	2.26	OK	0.82
11	ST16	ST15			0.39		0.23	7.81	20.13	52.0	1.129	900	17.0	RCP	0.50%	1.280	2.01	0.14	2.27	OK	0.88
	ST15	ST14					0.00	7.81	20.27	51.8	1.124	900	37.2	RCP	0.50%	1.280	2.01	0.31	2.27	OK	0.88
	ST14	ST13					0.00	7.81	20.57	51.3	1.113	900	24.2	RCP	0.50%	1.280	2.01	0.20	2.27	OK	0.87
8 & 10	ST13	ST12			0.77		0.46	8.27	20.78	51.0	1.171	975	94.8	RCP	0.50%	1.585	2.12	0.74	2.33	OK	0.74
7 & 9	ST12	ST11			0.84		0.50	8.77	21.52	49.8	1.214	975	10.9	RCP	0.50%	1.585	2.12	0.09	2.35	OK	0.77
	ST11	ST10					0.00	17.02	22.39	48.5	2.295	1350	66.5	RCP	0.30%	2.923	2.04	0.54	2.28	OK	0.78
	ST10	ST9					0.00	17.02	22.93	47.7	2.258	1350	23.0	RCP	0.30%	2.923	2.04	0.19	2.27	OK	0.77
	ST9	ST8					0.00	20.30	23.12	47.5	2.679	1350	12.5	RCP	0.30%	2.923	2.04	0.10	2.31	OK	0.92
4	ST8	ST7			0.56		0.34	20.64	23.22	47.3	2.716	1350	41.0	RCP	0.30%	2.923	2.04	0.33	2.32	OK	0.93
3	ST7	ST6			0.29		0.17	20.81	23.56	46.9	2.713	1350	31.9	RCP	0.30%	2.923	2.04	0.26	2.32	OK	0.93
	ST6	ST5					0.00	20.81	23.82	46.5	2.693	1350	17.4	RCP	0.30%	2.923	2.04	0.14	2.32	OK	0.92
2	ST5	ST4			0.52		0.31	21.13	23.96	46.4	2.723	1350	14.3	RCP	0.30%	2.923	2.04	0.12	2.32	OK	0.93
Future - Fields	CAP 1	ST47			4.91		2.95	2.95	17.00	58.2	0.477	750	14.6	RCP	0.20%	0.498	1.13	0.22	1.28	OK	0.96
	ST47	ST4					0.00	2.95	17.22	57.7	0.473	750	54.4	RCP	0.20%	0.498	1.13	0.80	1.28	OK	0.95
Street G	ST4	ST3					0.00	24.07	24.08	46.2	3.092	1500	6.3	RCP	0.30%	3.872	2.19	0.05	2.45	OK	0.80
	ST3	ST2					0.00	24.07	24.12	46.1	3.088	1500	114.1	RCP	0.30%	3.872	2.19	0.87	2.45	OK	0.80
									Total 5-year flow		4.972										
									Bypass flow		1.932										
	ST2-Bypass	ST46					0.00	0.00	24.99	45.1	1.932	1500	28.9	RCP	0.40%	4.471	2.53	0.19	2.43	OK	0.43
	ST46	Outlet B					0.00	0.00	25.18	44.9	1.932	1500	81.8	RCP	0.20%	3.161	1.79	0.76	1.87	OK	0.61
									25mm Peak to pond from OTTHYMO		3.040										
	ST2	ST1					0.00	24.07	24.99	45.1	3.040	1500	16.7	RCP	0.30%	3.872	2.19	0.13	2.44	OK	0.79
	ST1	Pond Inlet					0.00	37.12	25.12	44.9	3.040	1500	31.7	RCP	0.30%	3.872	2.19	0.24	2.44	OK	0.79
									Peak outflow from pond (OTTHYMO)		0.590	@85.63m Storage Elevation									
	Pond Outlet	ST43									0.590	750	13.2	RCP	5.65%	2.646	5.99	0.04	4.81	OK	0.22
	ST43	Outlet A									0.590	750	44.3	RCP	0.50%	0.787	1.78	0.41	1.96	OK	0.75
						Total=	37.12														



Jewell Engineering Inc

1-71 Millennium Parkway

Belleville, ON, K8N 4Z5

Tel: Ph. 613-969-1111

Fax: Fx. 613-969-8988

Website: www.jewelleng.ca

LICENSED PROFESSIONAL ENGINEER

B. C. KEENE

Aug 29/22

PROVINCE OF ONTARIO

Designed: Julie Humphries, C.E.T.

Checked: Bryon Keene, P.Eng.

Date: August 29, 2022

Project:

Cork and Vine Community

Storm Sewer Calculations

Wellington



APPENDIX F

OGS Sizing

Hydro First Defense® - HC



Rev. 11.3

Project Name: **Cork & Vine** Report Date: **2022/08/18** Paste
 Street: **Street A** City: **Wellington**
 Province: **ON** Country: **Canada**
 Designer: **Andrew Rosenthal, EIT** email: **arosenthal@jewelleng.ca**

Treatment Parameters:

Structure ID:
 TSS Goal: **80 % Removal**
 TSS Particle Size: **NJDEP / ETV**
 Area: **0.8 ha**
 Percent Impervious:
 Rational C value: **0.60** Calc. Cn
 Rainfall Station: **Belleville, ONT** MAP
 Peak Storm Flow: **L/s**

RESULTS SUMMARY

Model	TSS	Volume
FD-3HC	63.0%	>90%
FD-4HC	72.0%	>90%
FD-5HC	75.0%	>90%
FD-6HC	79.0%	>90%
FD-8HC	85.0%	>90%
FD-10HC	89.0%	>90%

Model Specification:

Model: **FD-4HC**
 Diameter: **1200 mm**
 Peak Flow Capacity: **510.00 L/s**
 Sediment Storage: **0.54 m³**
 Oil Storage: **723.00 L**

Installation Configuration:

Placement: **Online**
 Outlet Pipe Size: **375 mm OK**
 Inlet Pipe 1 Size: **375 mm OK**
 Inlet Pipe 2 Size: **375 mm OK**
 Inlet Pipe 3 Size: **mm OK**

Rim Level: **85.130 m** Calc Invs.
 Outlet Pipe Invert: **83.940 m OK**
 Invert Pipe 1: **83.970 m OK**
 Invert Pipe 2: **84.120 m OK**
 Invert Pipe 3: **m**

Designer Notes:

Net Annual Removal Model: FD-4HC

Intensity ⁽¹⁾	Fraction of Rainfall ⁽¹⁾	FD-4HC Removal Efficiency ⁽²⁾	Weighted Net Annual Efficiency
(mm/hr)	(%)	(%)	(%)
0.50	0.4%	97.5%	0.4%
1.00	13.2%	88.9%	11.8%
1.50	14.0%	83.9%	11.7%
2.00	14.0%	80.3%	11.2%
2.50	3.6%	77.6%	2.8%
3.00	2.5%	75.3%	1.9%
3.50	8.4%	73.4%	6.2%
4.00	5.1%	71.7%	3.6%
4.50	1.6%	70.3%	1.1%
5.00	5.1%	69.0%	3.5%
6.00	4.8%	66.7%	3.2%
7.00	4.5%	64.8%	2.9%
8.00	3.5%	63.1%	2.2%
9.00	2.4%	61.7%	1.5%
10.00	2.5%	60.4%	1.5%
20.00	9.7%	51.8%	5.0%
30.00	2.8%	46.7%	1.3%
40.00	0.9%	43.2%	0.4%
50.00	0.4%	40.4%	0.2%
100.00	0.6%	0.0%	0.0%
150.00	0.1%	0.0%	0.0%
200.00	0.0%	0.0%	0.0%

Total Net Annual Removal Efficiency: 72.0%

Total Annual Runoff Volume Treated: >90%

1. Rainfall Data: 1960:2007, HLY03, Belleville, ONT, 6150700 & 6150689.

2. Based in NJDEP / ETV PSD, NJDEP Test Protocols 2013.

3. Rainfall adjusted to 5 min peak intensity based on hourly average.

Hydro
International

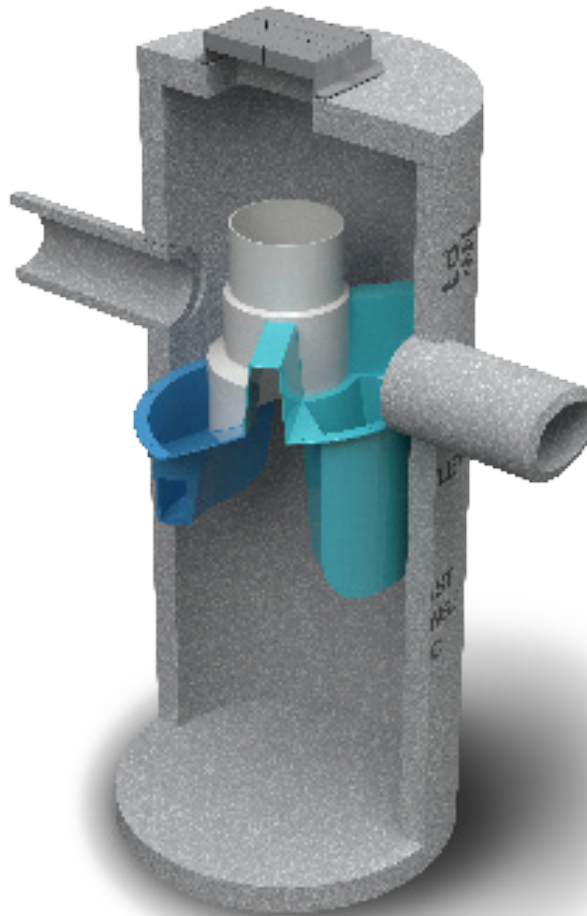


1	Vortex Chamber Diameter	1200 mm
2	Inlet Pipe Diameter	375 mm
3	Oil Storage Capacity	723.00 L
4	Min. Provided Sediment Storage Capacity	0.54 m ³
5	Outlet Pipe Diameter	375 mm
6	Height(Final Grade to Outlet Invert)	1190 mm
7	Sump Depth(Outlet Invert to Sump)	1800 mm
	Total Depth	2990 mm

[illegible]

APPENDIX G

Operations and Maintenance – OGS



Operation and Maintenance Manual

First Defense® High Capacity and First Defense® Optimum

Vortex Separator for Stormwater Treatment

Table of Contents

3	FIRST DEFENSE® BY HYDRO INTERNATIONAL - INTRODUCTION - OPERATION - POLLUTANT CAPTURE AND RETENTION
4	MODEL SIZES & CONFIGURATIONS - FIRST DEFENSE® COMPONENTS
5	MAINTENANCE - OVERVIEW - MAINTENANCE EQUIPMENT CONSIDERATIONS - DETERMINING YOUR MAINTENANCE SCHEDULE
6	MAINTENANCE PROCEDURES - INSPECTION - FLOATABLES AND SEDIMENT CLEAN OUT
8	FIRST DEFENSE® INSTALLATION LOG
9	FIRST DEFENSE® INSPECTION AND MAINTENANCE LOG

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DISCLAIMER: Information and data contained in this manual is exclusively for the purpose of assisting in the operation and maintenance of Hydro International plc's First Defense®. No warranty is given nor can liability be accepted for use of this information for any other purpose. Hydro International plc has a policy of continuous product development and reserves the right to amend specifications without notice.

I. First Defense® by Hydro International

Introduction

The First Defense® is an enhanced vortex separator that combines an effective and economical stormwater treatment chamber with an integral peak flow bypass. It efficiently removes total suspended solids (TSS), trash and hydrocarbons from stormwater runoff without washing out previously captured pollutants. The First Defense® is available in several model configurations to accommodate a wide range of pipe sizes, peak flows and depth constraints.

The two product models described in this guide are the First Defense® High Capacity and the First Defense® Optimum; they are inspected and maintained identically.

Operation

The First Defense® operates on simple fluid hydraulics. It is self-activating, has no moving parts, no external power requirement and is fabricated with durable non-corrosive components. No manual procedures are required to operate the unit and maintenance is limited to monitoring accumulations of stored pollutants and periodic clean-outs. The First Defense® has been designed to allow for easy and safe access for inspection, monitoring and clean-out procedures. Neither entry into the unit nor removal of the internal components is necessary for maintenance, thus safety concerns related to confined-space-entry are avoided.

Pollutant Capture and Retention

The internal components of the First Defense® have been designed to optimize pollutant capture. Sediment is captured and retained in the base of the unit, while oil and floatables are stored on the water surface in the inner volume (Fig. 1).

The pollutant storage volumes are isolated from the built-in bypass chamber to prevent washout during high-flow storm events. The sump of the First Defense® retains a standing water level between storm events. This ensures a quiescent flow regime at the onset of a storm, preventing resuspension and washout of pollutants captured during previous events.

Accessories such as oil absorbent pads are available for enhanced oil removal and storage. Due to the separation of the oil and floatable storage volume from the outlet, the potential for washout of stored pollutants between clean-outs is minimized.

Applications

- Stormwater treatment at the point of entry into the drainage line
- Sites constrained by space, topography or drainage profiles with limited slope and depth of cover
- Retrofit installations where stormwater treatment is placed on or tied into an existing storm drain line
- Pretreatment for filters, infiltration and storage

Advantages

- Inlet options include surface grate or multiple inlet pipes
- Integral high capacity bypass conveys large peak flows without the need for "offline" arrangements using separate junction manholes
- Long flow path through the device ensures a long residence time within the treatment chamber, enhancing pollutant settling
- Delivered to site pre-assembled and ready for installation

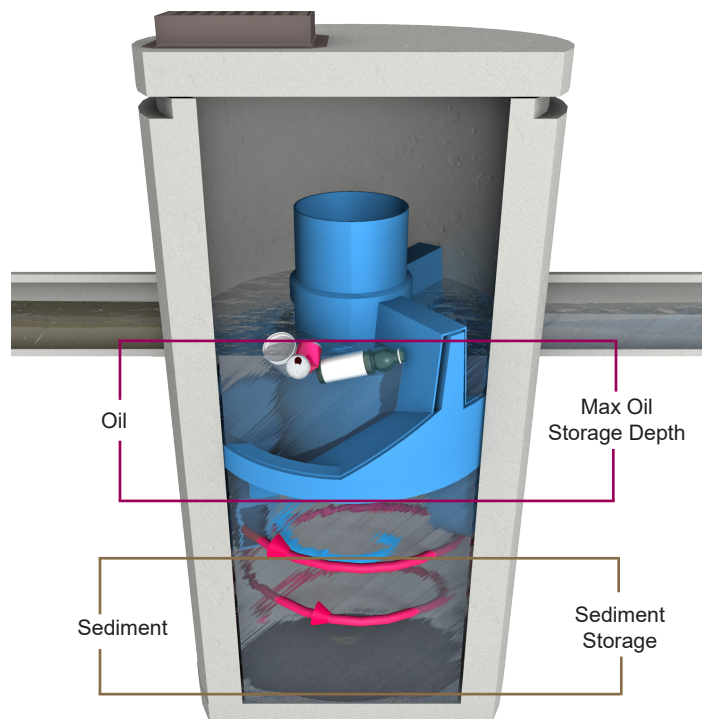


Fig. 1 Pollutant storage volumes in the First Defense®.

II. Model Sizes & Configurations

The First Defense® inlet and internal bypass arrangements are available in several model sizes and configurations. The components have modified geometries allowing greater design flexibility to accommodate various site constraints.

All First Defense® models include the internal components that are designed to remove and retain total suspended solids (TSS), gross solids, floatable trash and hydrocarbons (Fig.2). First Defense® model sizes (diameter) are shown in Table 1.

III. Maintenance

First Defense® Components

1. Built-In Bypass

2. Inlet Pipe

3. Inlet Chute
4. Floatables Draw-off Port

5. Outlet Pipe

6. Floatables Storage
7. Sediment Storage

8. Inlet Grate or Cover

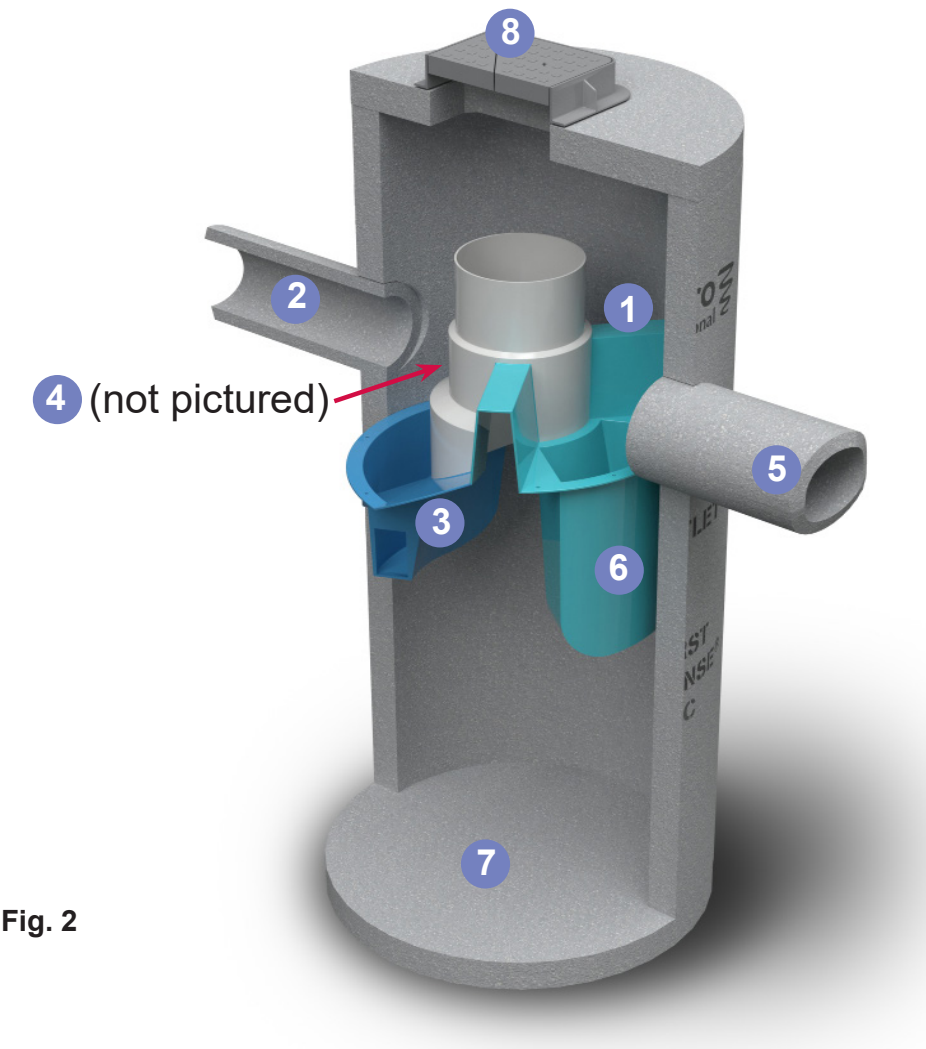


Table 1

First Defense® Model Sizes
(ft / m) diameter
3 / 0.9
4 / 1.2
5 / 1.5
6 / 1.8
7 / 2.1
8 / 2.4
10 / 3.0

Overview

The First Defense® protects the environment by removing a wide range of pollutants from stormwater runoff. Periodic removal of these captured pollutants is essential to the continuous, long-term functioning of the First Defense®. The First Defense® will capture and retain sediment and oil until the sediment and oil storage volumes are full to capacity. When sediment and oil storage capacities are reached, the First Defense® will no longer be able to store removed sediment and oil.

The First Defense® allows for easy and safe inspection, monitoring and clean-out procedures. A commercially or municipally owned sump-vac is used to remove captured sediment and floatables. Access ports are located in the top of the manhole.

Maintenance events may include Inspection, Oil & Floatables Removal, and Sediment Removal. Maintenance events do not require entry into the First Defense®, nor do they require the internal components of the First Defense® to be removed. In the case of inspection and floatables removal, a vactor truck is not required. However, a vactor truck is required if the maintenance event is to include oil removal and/or sediment removal.

Maintenance Equipment Considerations

The internal components of the First Defense® have a centrally located circular shaft through which the sediment storage sump can be accessed with a sump vac hose. The open diameter of this access shaft is 15 inches in diameter (Fig.3). Therefore, the nozzle fitting of any vactor hose used for maintenance should be less than 15 inches in diameter.

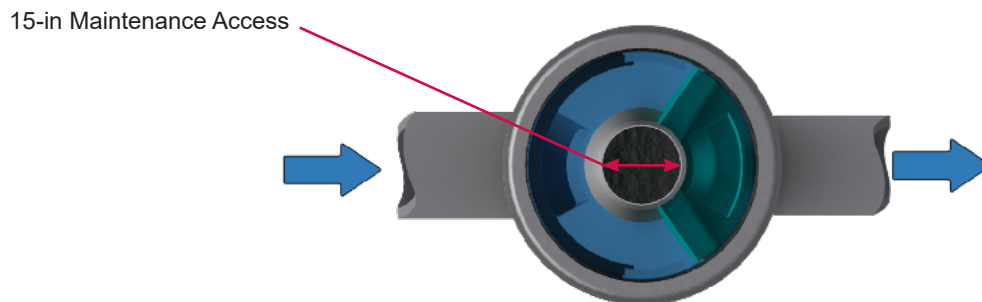


Fig.3 The central opening to the sump of the First Defense® is 15 inches in diameter.

Determining Your Maintenance Schedule

The frequency of clean out is determined in the field after installation. During the first year of operation, the unit should be inspected every six months to determine the rate of sediment and floatables accumulation. A simple probe such as a Sludge-Judge® can be used to determine the level of accumulated solids stored in the sump. This information can be recorded in the maintenance log (see page 9) to establish a routine maintenance schedule.

The vactor procedure, including both sediment and oil / floatables removal, for First Defense® typically takes less than 30 minutes and removes a combined water/oil volume of about 765 gallons.

Inspection Procedures

1. Set up any necessary safety equipment around the access port or grate of the First Defense® as stipulated by local ordinances. Safety equipment should notify passing pedestrian and road traffic that work is being done.
2. Remove the grate or lid to the manhole.
3. Without entering the vessel, look down into the chamber to inspect the inside. Make note of any irregularities. Fig.4 shows the standing water level that should be observed.
4. Without entering the vessel, use the pole with the skimmer net to remove floatables and loose debris from the components and water surface.
5. Using a sediment probe such as a Sludge Judge®, measure the depth of sediment that has collected in the sump of the vessel.
6. On the Maintenance Log (see page 9), record the date, unit location, estimated volume of floatables and gross debris removed, and the depth of sediment measured. Also note any apparent irregularities such as damaged components or blockages.
7. Securely replace the grate or lid.
8. Take down safety equipment.
9. Notify Hydro International of any irregularities noted during inspection.

Floatables and Sediment Clean Out

Floatables clean out is typically done in conjunction with sediment removal. A commercially or municipally owned sump-vac is used to remove captured sediment and floatables (Fig.4).

Floatables and loose debris can also be netted with a skimmer and pole. The access port located at the top of the manhole provides unobstructed access for a vactor hose to be lowered to the base of the sump.

Scheduling

- Floatables and sump clean out are typically conducted once a year during any season.
- Floatables and sump clean out should occur as soon as possible following a spill in the contributing drainage area.



Fig.4 Floatables are removed with a vactor hose

Recommended Equipment

- Safety Equipment (traffic cones, etc)
- Crow bar or other tool to remove grate or lid
- Pole with skimmer or net (if only floatables are being removed)
- Sediment probe (such as a Sludge Judge®)
- Vactor truck (flexible hose recommended)
- First Defense® Maintenance Log

Floatables and Sediment Clean Out Procedures

1. Set up any necessary safety equipment around the access port or grate of the First Defense® as stipulated by local ordinances. Safety equipment should notify passing pedestrian and road traffic that work is being done.
2. Remove the grate or lid to the manhole.
3. Without entering the vessel, look down into the chamber to inspect the inside. Make note of any irregularities.
4. Remove oil and floatables stored on the surface of the water with the vactor hose or with the skimmer or net
5. Using a sediment probe such as a Sludge Judge®, measure the depth of sediment that has collected in the sump of the vessel and record it in the Maintenance Log (page 9).
6. Once all floatables have been removed, drop the vactor hose to the base of the sump. Vactor out the sediment and gross debris off the sump floor
7. Retract the vactor hose from the vessel.
8. On the Maintenance Log provided by Hydro International, record the date, unit location, estimated volume of floatables and gross debris removed, and the depth of sediment measured. Also note any apparent irregularities such as damaged components, blockages, or irregularly high or low water levels.
9. Securely replace the grate or lid.

Maintenance at a Glance

Inspection	- Regularly during first year of installation - Every 6 months after the first year of installation
Oil and Floatables Removal	- Once per year, with sediment removal - Following a spill in the drainage area
Sediment Removal	- Once per year or as needed - Following a spill in the drainage area

NOTE: For most clean outs the entire volume of liquid does not need to be removed from the manhole. Only remove the first few inches of oils and floatables from the water surface to reduce the total volume of liquid removed during a clean out.



First Defense® Installation Log

HYDRO INTERNATIONAL REFERENCE NUMBER:	
SITE NAME:	
SITE LOCATION:	
OWNER:	CONTRACTOR:
CONTACT NAME:	CONTACT NAME:
COMPANY NAME:	COMPANY NAME:
ADDRESS:	ADDRESS:
TELEPHONE:	TELEPHONE:
FAX:	FAX:

INSTALLATION DATE: / /

MODEL SIZE (CIRCLE ONE): [3-FT] [4-FT] [5-FT] [6-FT] [7-FT] [8-FT] [10-FT]

INLET (CIRCLE ALL THAT APPLY): GRATED INLET (CATCH BASIN) INLET PIPE (FLOW THROUGH)

First Defense® Inspection and Maintenance Log

[illegible]

Stormwater Solutions

94 Hutchins Drive
Portland, ME 04102

Tel: (207) 756-6200

Fax: (207) 756-6212

stormwaterinquiry@hydro-int.com

www.hydro-int.com

Turning Water Around...®

FD_O+M_K_2105

APPENDIX H

Operations and Maintenance – CB Shields

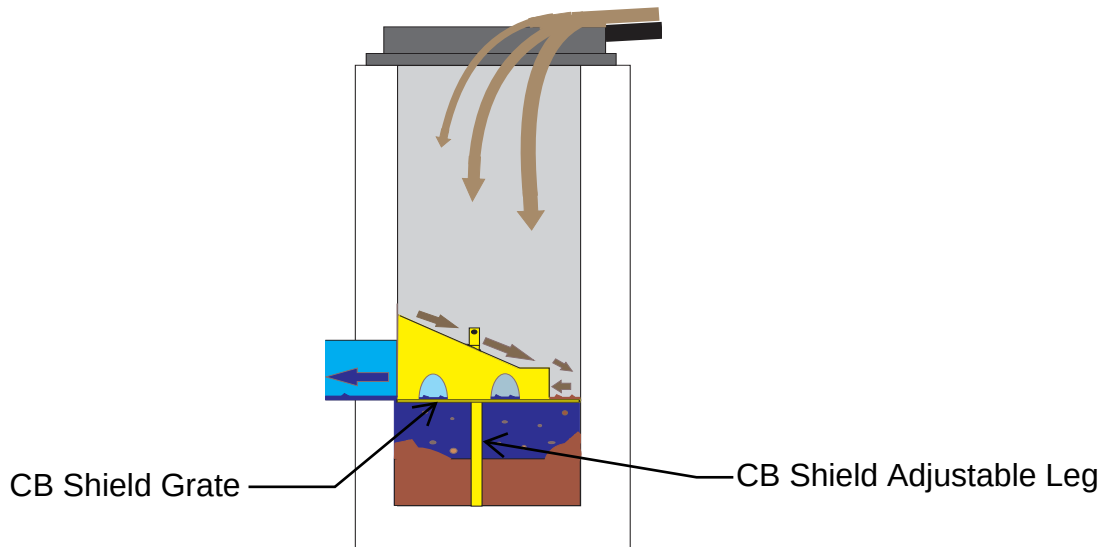
CB Shield Operations Manual

Installing CB Shield

It is important the catch basin frame and cover is aligned properly with the catch basin below

If it is misaligned it may be difficult to install the CB Shield insert

Determine the depth of the sump (i.e. the distance from the invert of the outlet pipe to the bottom of the catch basin). If the catch basin is in service the sump depth will be the depth of the water. The grate section of the CB Shield insert should be the same elevation as the water depth in the sump.



Adjust the leg of the CB Shield to achieve the appropriate elevation

The CB Shield is lowered into place with the rope attached to the top of the leg. The high side of the sloped plate should face the wall with the outlet pipe. (The incoming water should be directed to the wall furthest from the outlet)

The flexible plastic skirt around the outer edges of the CB Shield insert may interfere with some misaligned frame and grates. If so a slice can be cut into the skirt with a utility knife at the point of interference.

Make sure the grate is at the desired level or remove CB Shield and re-adjust the leg length.

Inspecting a CB Shield Enhanced Catch Basin

Open grate

A lifting rope is attached to the top of the centered leg of the CB Shield insert. Lift and remove the insert. Inspect CB Shield for any possible damage. Quite often leaves will accumulate on the grate. This can actually improve the Shield's ability to capture sediment and assist in preventing leave litter from being washed down stream.

Use a Sludge Judge to measure the sediment depth in 4 - 6 locations of the sump.

If the sediment depth is 300mm – 600mm deep it is recommended that the unit be cleaned.

Cleaning a CB Shield Enhanced Catch Basin

Open grate and remove CB Shield with lift rope.

Clean catch basin as usual with a Vacuum truck.

Clean CB Shield (if needed) and re-install into catch basin.

If there is any significant damage to a CB Shield please send a picture and its location to CB Shield Inc. (info@cbshield.com).

APPENDIX I

Landscape Plan – SWMF

1. Drawing is in metric scale. All dimensions are in millimeters. Do not scale drawings.	9. Landscape Architect is not responsible for the accuracy of survey, engineering or architectural drawings. Contractor to verify existing grading/topographic information prior to installing granular bases.
2. Plus/Minus (+/-) dimensions are for general verification purposes only and are NOT to be used to establish layout of any feature(s), ever.	10. Construction must conform to all applicable codes and regulations of all authorities having jurisdiction.
3. The contractor shall verify all field dimensions onsite during the quotation period. No extras will be allowed for discrepancies between the drawings and the actual site conditions unless reported in writing during the quotation period.	11. Ensure positive drainage for finished surfaces leaving no pockets or low points for standing water.
4. It is the contractor's responsibility to locate and have staked the exact utility location with all companies involved before starting any work. Hand dig within two (2) meters of all electrical lines.	12. All drawings, specifications and related documents are the copyright of the Landscape Architect noted on the drawings. Reproduction of any part is forbidden without the Landscape Architect's permission.
5. Keep the area outside the immediate construction zone clean, safe and usable by the public at all times. Contractor to coordinate all deliveries of materials to ensure minimal construction delays to the approval of the Landscape Architect and/or client's representative.	13. Wherever any material is specified by name and/or number thereof, such specifications shall be deemed to be used for the purpose of facilitating a description of the materials and establishing quality, and shall be deemed and construed to be followed by the words "or approved equal".
6. Obtain approval of layout from the Landscape Architect and/or client's representative prior to commencement.	14. No substitutions will be permitted which have not been submitted for prior approval by the Landscape Architect or client's representative. All materials shall be new and without flaws or defects and shall be the best of their class and kind. Sufficient descriptive literature and/or samples must be submitted as "Equal" substitutes.
7. All work to be guaranteed for a period of One (1) Year from the date of substantial performance.	15. These notes apply to all parts of this contract.
8. Make good all damage resulting from work carried out under the contract, at no extra cost. This includes restoration of any/all construction access routes - to as new condition, to the satisfaction of the Landscape Architect and/or client's representative.	

ID	Quantity	Common Name	Latin Name	Scheduled Size
TREES				
AsGM	19	Green Mountain(R) Sugar Maple	Acer saccharum 'Green Mountain'	60mm CT/WB
AxFc	16	Celebration(R) Maple	Acer x freemanii 'Celzam'	60mm CT/WB
Am	12	Robin Hill Serviceberry	Amelanchier x grandiflora 'Robin Hill'	60mm CT/WB
Co	21	Hackberry	Celtis occidentalis	60mm CT/WB
Gb	18	Autumn Gold Maidenhair Tree	Ginkgo biloba 'Autumn Gold'	60mm CT/WB
Gt	14	Shademaster locust	Gladiolia triacanthos 'Shademaster'	60mm CT/WB
Lit	13	Tulip Tree	Liriodendron tulipifera	60mm CT/WB
Pa-1	20	London Planetree	Platanus acerifolia	60mm CT/WB
Qm	20	Burr Oak	Quercus macrocarpa	60mm CT/WB
Qr	14	Northern Red Oak	Quercus rubra	60mm CT/WB
Ta	12	Basswood	Tilia americana	60mm CT/WB
Tc	10	Summer Sprite Linden	Tilia cordata 'Halka'	60mm CT/WB

ITEM	OFFSET (m)
Light Poles	3m
Underground Utilities	1.5m
Hydrants	3m
Hydro Transformers	3m
Driveways	1.5m
Sidewalk Edge	1m
Curb Face	1m

-
- Key Plan:
- BELLEVILLE ST
- SUBJECT SITE
- CONCESSION ST
- CLEMONS ST
- MAIN ST (LOYALIST HWY)
- LAKE ONTARIO
- CONCESSION RD

 PROPERTY LINE
 PROPOSED TREE- SEE PLANT LIST
 PLANT LIST/QUANTITY

[illegible]

NOT FOR CONSTRUCTION

Country Club Estates
Kaitlin Corporation

Wellington, Ontario
Prince Edward County

Landscape Plan

Designed By: GB	Project #: 22.17
Drawn By: HC	
Approved By: GB	Drawing #:
Date: 22/03/14	L 1.1



1.	Drawing is in metric scale. All dimensions are in millimeters. Do not scale drawings.	9.	Landscape Architect is not responsible for the accuracy of survey, engineering or architectural drawings. Contractor to verify existing grading/topographic information prior to installing granular bases.
2.	Plus/Minus (+/-) dimensions are for general verification purposes only and are NOT to be used to establish layout of any feature(s), ever.	10.	Construction must conform to all applicable codes and regulations of all authorities having jurisdiction.
3.	The contractor shall verify all field dimensions onsite during the quotation period. No extras will be allowed for discrepancies between the drawings and the actual site conditions unless reported in writing during the quotation period.	11.	Ensure positive drainage for finished surfaces leaving no pockets or low points for standing water.
4.	It is the contractor's responsibility to locate and have staked the exact utility location with all companies involved before starting any work. Hand dig within two (2) meters of all electrical lines.	12.	All drawings, specifications and related documents are the copyright of the Landscape Architect noted on the drawings. Reproduction of any part is forbidden without the Landscape Architect's permission.
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7.	All work to be guaranteed for a period of One (1) Year from the date of substantial performance.	15.	These notes apply to all parts of this contract.
8.	Make good all damage resulting from work carried out under the contract, at no extra cost. This includes restoration of any/all construction access routes - to as new condition, to the satisfaction of the Landscape Architect and/or client's representative.		

ID	Quantity	Common Name	Latin Name	Scheduled Size
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Co	21	Hackberry	Celtis occidentalis	60mm CT/WB
Gb	18	Autumn Gold Maidenhair Tree	Ginkgo biloba 'Autumn Gold'	60mm CT/WB
Gt	14	Shademaster locust	Gleditsia triacanthos 'Shademaster'	60mm CT/WB
Lit	13	Tulip Tree	Liriodendron tulipifera	60mm CT/WB
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ITEM	OFFSET (m)
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Underground Utilities	1.5m
Hydrants	3m
Hydro Transformers	3m
Driveways	1.5m
Sidewalk Edge	1m
Curb Face	1m

- Total Trees: 189

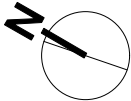


Map showing the location of the subject site in Belleville, Ontario. The site is bounded by Belleville St to the north, Consequon St to the south, and Clemmison St to the east. The site is located west of Lake Ontario and south of Main St/Loyalist Hwy.

 PROPERTY LINE
 PROPOSED TREE- SEE PLANT LIST
 PLANT LIST/QUANTITY

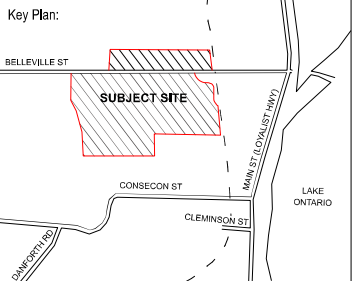
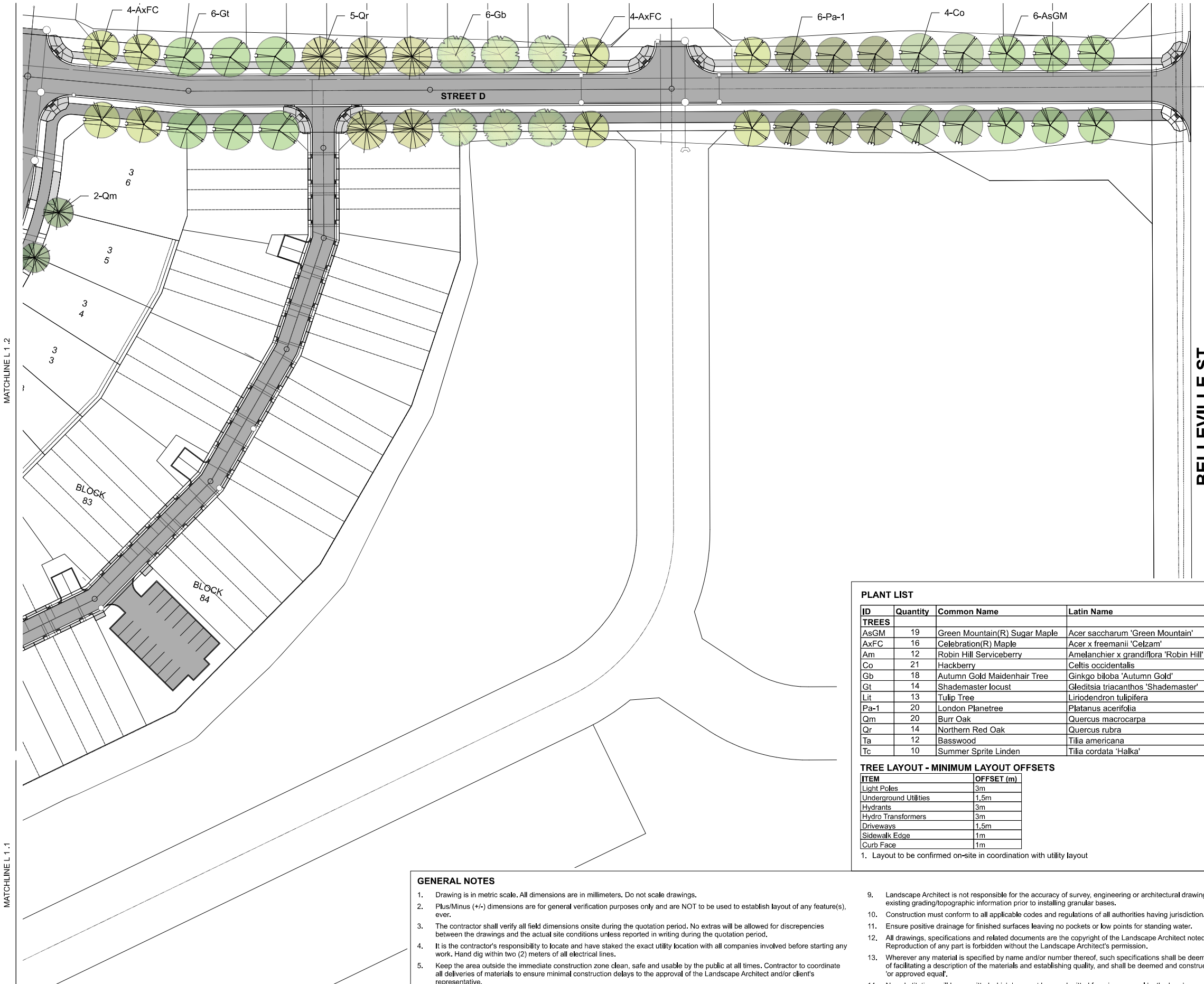
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1	Issued for Review	22.07.25	GB
No.	Description	Date	By
Issue / Revision Schedule			



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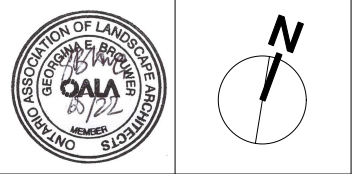
L 1.2



Legend:

- PROPERTY LINE
- PROPOSED TREE- SEE PLANT LIST
- PLANT LIST/QUANTITY

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NOT FOR CONSTRUCTION

Client:
Country Club Estates
Kaitlin Corporation

Project:
Wellington, Ontario
Prince Edward County

Drawing Title:
Landscape Plan

Designed By: GB	Project #: 22.17
Drawn By: HC	Drawing #:
Approved By: GB	L 1.3
Date: 22/03/14	

PLANT LIST

ID	Quantity	Common Name	Latin Name	Scheduled Size
TREES				
AsGM	19	Green Mountain(R) Sugar Maple	Acer saccharum 'Green Mountain'	60mm CT/WB
AxFC	16	Celebration(R) Maple	Acer x freemanii 'Celzam'	60mm CT/WB
Am	12	Robin Hill Serviceberry	Amelanchier x grandiflora 'Robin Hill'	60mm CT/WB
Co	21	Hackberry	Celtis occidentalis	60mm CT/WB
Gb	18	Autumn Gold Maidenhair Tree	Ginkgo biloba 'Autumn Gold'	60mm CT/WB
Gt	14	Shademaster locust	Gleditsia triacanthos 'Shademaster'	60mm CT/WB
Lit	13	Tulip Tree	Liriodendron tulipifera	60mm CT/WB
Pa-1	20	London Planetree	Platanus acerifolia	60mm CT/WB
Qm	20	Burr Oak	Quercus macrocarpa	60mm CT/WB
Qr	14	Northern Red Oak	Quercus rubra	60mm CT/WB
Ta	12	Basswood	Tilia americana	60mm CT/WB
Tc	10	Summer Sprite Linden	Tilia cordata 'Halka'	60mm CT/WB

Total Trees: 189

TREE LAYOUT - MINIMUM LAYOUT OFFSETS

ITEM	OFFSET (m)
Light Poles	3m
Underground Utilities	1.5m
Hydrants	3m
Hydro Transformers	3m
Driveways	1.5m
Sidewalk Edge	1m
Curb Face	1m

1. Layout to be confirmed on-site in coordination with utility layout

GENERAL NOTES

- Drawing is in metric scale. All dimensions are in millimeters. Do not scale drawings.
- Plus/Minus (+/-) dimensions are for general verification purposes only and are NOT to be used to establish layout of any feature(s), ever.
- The contractor shall verify all field dimensions onsite during the quotation period. No extras will be allowed for discrepancies between the drawings and the actual site conditions unless reported in writing during the quotation period.
- It is the contractor's responsibility to locate and have staked the exact utility location with all companies involved before starting any work. Hand dig within two (2) meters of all electrical lines.
- Keep the area outside the immediate construction zone clean, safe and usable by the public at all times. Contractor to coordinate all deliveries of materials to ensure minimal construction delays to the approval of the Landscape Architect and/or client's representative.
- Obtain approval of layout from the Landscape Architect and/or client's representative prior to commencement.
- All work to be guaranteed for a period of One (1) Year from the date of substantial performance.
- Make good all damage resulting from work carried out under the contract, at no extra cost. This includes restoration of any/all construction access routes - to as new condition, to the satisfaction of the Landscape Architect and/or client's representative.

- Landscape Architect is not responsible for the accuracy of survey, engineering or architectural drawings. Contractor to verify existing grading/topographic information prior to installing granular bases.
- Construction must conform to all applicable codes and regulations of all authorities having jurisdiction.
- Ensure positive drainage for finished surfaces leaving no pockets or low points for standing water.
- All drawings, specifications and related documents are the copyright of the Landscape Architect noted on the drawings. Reproduction of any part is forbidden without the Landscape Architect's permission.
- Wherever any material is specified by name and/or number thereof, such specifications shall be deemed to be used for the purpose of facilitating a description of the materials and establishing quality, and shall be deemed and construed to be followed by the words 'or approved equal'.
- No substitutions will be permitted which have not been submitted for prior approval by the Landscape Architect or client's representative. All materials shall be new and without flaws or defects and shall be the best of their class and kind. Sufficient descriptive literature and/or samples must be submitted as 'Equal' substitutes.
- These notes apply to all parts of this contract.

GENERAL NOTES

1. Drawing is in metric scale. All dimensions are in millimeters. Do not scale drawings.

2. Plus/Minus (+/-) dimensions are for general verification purposes only and are NOT to be used to establish layout of any feature(s), ever.

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6. Obtain approval of layout from the Landscape Architect and/or client's representative prior to commencement.

7. All work to be guaranteed for a period of One (1) Year from the date of substantial performance.

8. Make good all damage resulting from work carried out under the contract, at no extra cost. This includes restoration of any/all construction access routes - to as new condition, to the satisfaction of the Landscape Architect and/or client's representative.
9. Landscape Architect is not responsible for the accuracy of survey, engineering or architectural drawings. Contractor to verify existing grading/topographic information prior to installing granular bases.

10. Construction must conform to all applicable codes and regulations of all authorities having jurisdiction.

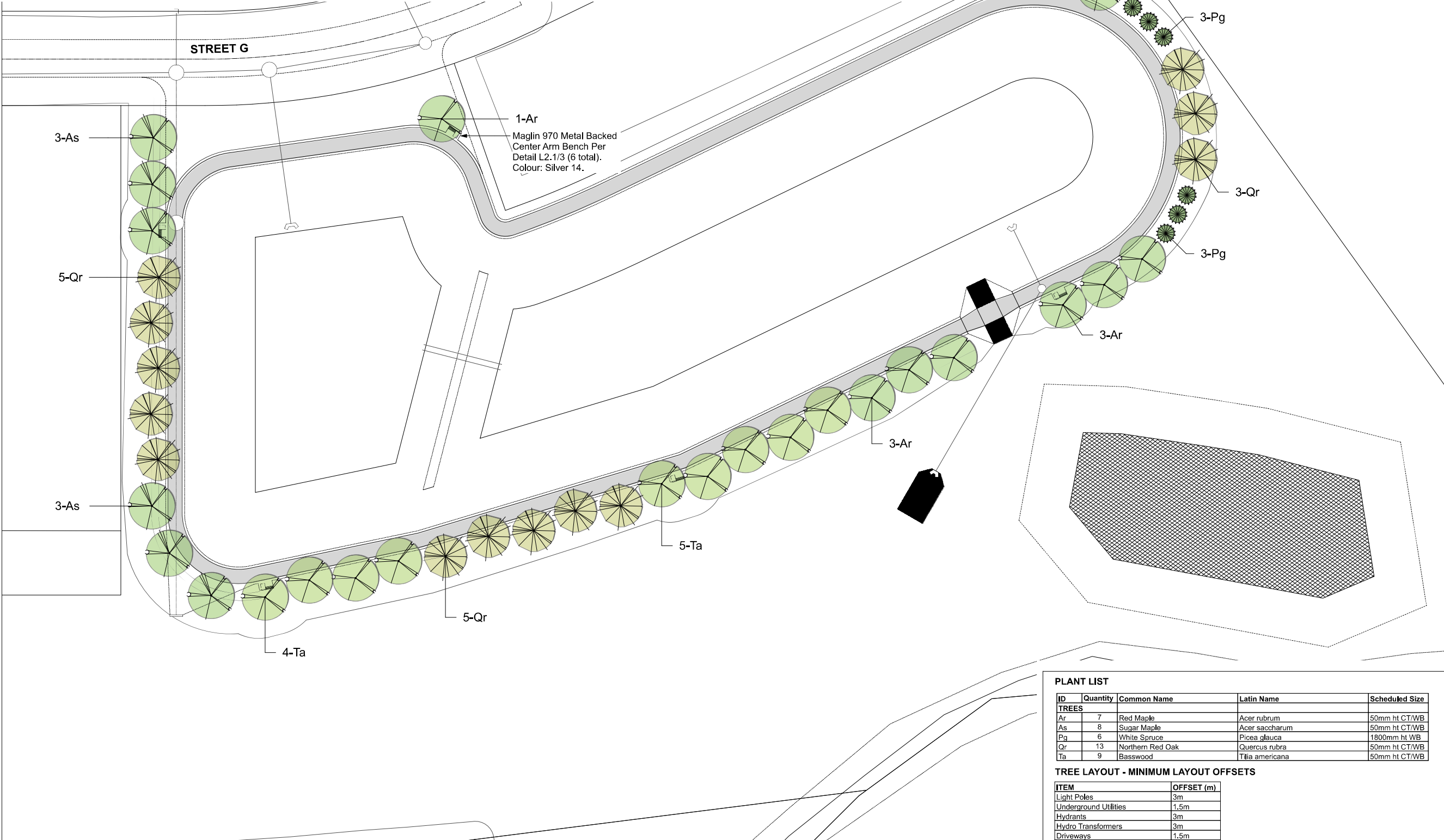
11. Ensure positive drainage for finished surfaces leaving no pockets or low points for standing water.

12. All drawings, specifications and related documents are the copyright of the Landscape Architect noted on the drawings. Reproduction of any part is forbidden without the Landscape Architect's permission.

13. Wherever any material is specified by name and/or number thereof, such specifications shall be deemed to be used for the purpose of facilitating a description of the materials and establishing quality, and shall be deemed and construed to be followed by the words 'or approved equal'.

14. No substitutions will be permitted which have not been submitted for prior approval by the Landscape Architect or client's representative. All materials shall be new and without flaws or defects and shall be the best of their class and kind. Sufficient descriptive literature and/or samples must be submitted as 'Equal' substitutes.

15. These notes apply to all parts of this contract.



Key Plan:

Legend:

Proposed Trees- See plant list for additional details

Maglin 970 Center Arm Bench

1
L 2.1

2
L 2.1

3
L 1.4

1

No.

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Description

22.07.25

Date

GB

By

Issue / Revision Schedule

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NOT FOR CONSTRUCTION

Client:

Country Club Estates

Kaitlin Corporation

Project:

Wellington, Ontario

Prince Edward County

Drawing Title:

Stormwater Pond

Designed By: GB

Drawn By: HC

Approved By: GB

Date: 22/03/14

Project #: 22.17

Drawing #:

L 1.4

PLANT LIST

ID	Quantity	Common Name	Latin Name	Scheduled Size
TREES				
Ar	7	Red Maple	Acer rubrum	50mm ht CT/WB
As	8	Sugar Maple	Acer saccharum	50mm ht CT/WB
Pg	6	White Spruce	Picea glauca	1800mm ht WB
Qr	13	Northern Red Oak	Quercus rubra	50mm ht CT/WB
Ta	9	Basswood	Tilia americana	50mm ht CT/WB

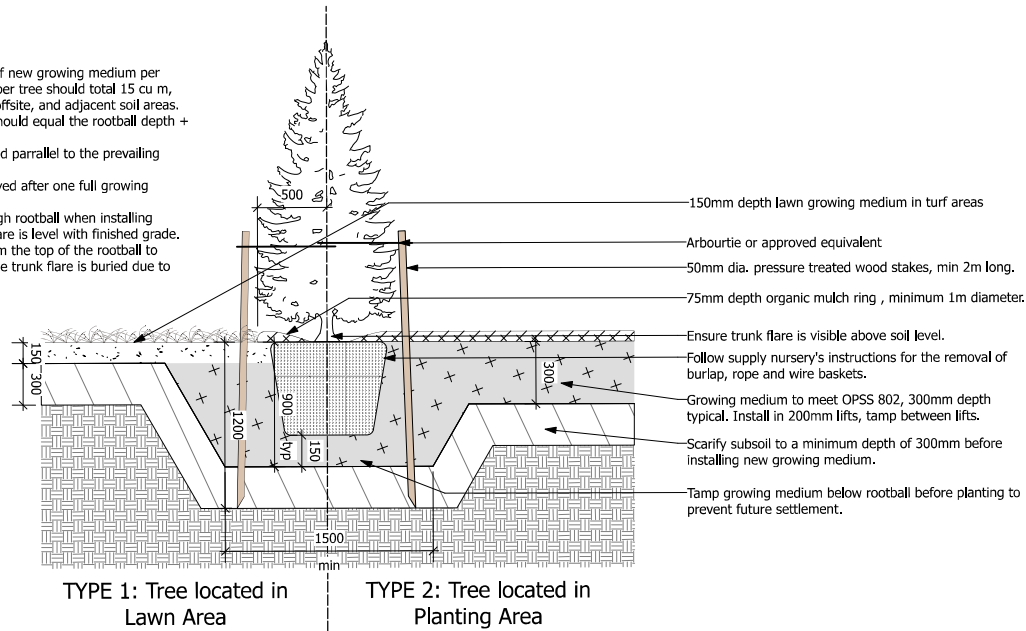
TREE LAYOUT - MINIMUM LAYOUT OFFSETS

ITEM	OFFSET (m)
Light Poles	3m
Underground Utilities	1.5m
Hydrants	3m
Hydro Transformers	3m
Driveways	1.5m
Sidewalk Edge	1m
Curb Face	1m

1. Layout to be confirmed on-site in coordination with utility layout

- ### General Notes

1. Install a minimum 5 cu m of new growing medium per tree. Available soil volume per tree should total 15 cu m, which can include shared, offsite, and adjacent soil areas.
2. Max depth of planting pit should equal the rootball depth + 150mm
3. Tree stakes should be placed parallel to the prevailing wind
4. Wooden stakes to be removed after one full growing season
5. DO NOT drive stakes through rootball when installing
6. Set tree so base of trunk flare is level with finished grade. Soil should be removed from the top of the rootball to expose trunk flare (often the trunk flare is buried due to typical nursery practices)



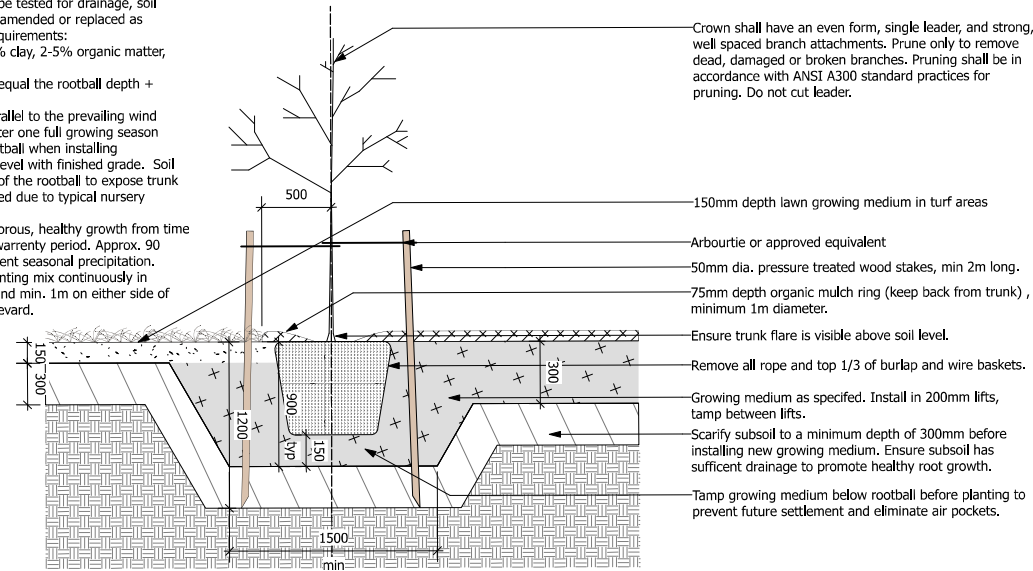
1

Coniferous Tree Detail

Scale: 1:25

- ### General Notes

1. Planting mix: 2/3 native soil from the tree pit excavation or stockpile and 1/3 commercially prepared triple mix. Add approved mycorrhizal inoculant in quantities recommended by supplier
2. All proposed planting areas shall be tested for drainage, soil quality and pH. Poor soil shall be amended or replaced as required to meet the following requirements:
50-60% sand, 20-40% silt, 6-10% clay, 2-5% organic matter, and pH between 6.5-7.5.
3. Max depth of planting pit should equal the rootball depth + 150mm
4. Tree stakes should be placed parallel to the prevailing wind
5. Wooden stakes to be removed after one full growing season
6. DO NOT drive stakes through rootball when installing
7. Soil on trunk flare is level with finished grade. Soil should be removed from the top of the rootball to expose trunk flare (often the trunk flare is buried due to typical nursery practices)
8. Water thoroughly to maintain vigorous, healthy growth from time of delivery installation to end of warranty period. Approx. 90 litres every 7-10 days to supplement seasonal precipitation.
9. Excavate and prepare tree pit planting mix continuously in boulevards less than 1.5m wide and min. 1m on either side of root ball along the length of boulevard.



2

Deciduous Tree Planting

Scale: 1:25

Key Plan:

Legend:

[illegible]

1	Issued for Review	22.07.25	GB
No.	Description	Date	By
Issue / Revision Schedule			



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NOT FOR CONSTRUCTION

Client:

Country Club Estates

Kaitlin Corporation

Project:

Wellington, Ontario

Prince Edward County

Drawing Title:

Landscape Details

Designed By: GB	Project #: 22.17 Drawing #: L 2.1
Drawn By: HC	
Approved By: GB	
Date: 22/03/14	

APPENDIX J

Jewell 2020 SWM Brief

KAITLIN CORP – COUNTRY CLUB ESTATES

Floodplain Delineation

Lane Creek Watershed

Technical Memorandum

April 8, 2020

Prepared by Bryon Keene, P.Eng.



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Appendix B	Figures
Appendix C	OTTHYMO Model Output

1 Introduction

Kaitlin Corporation is seeking to develop a large residential subdivision in the Village of Wellington in Prince Edward County (PEC) and commissioned this report to support the application.

A secondary plan for the development was previously approved by PEC and provided the basis for a draft plan of subdivision approval that since lapsed. The application was resubmitted to PEC in January of 2020. As part of the previous and current submissions, some modifications are proposed adjacent to the floodplain of Lane Creek.

This memorandum provides the supporting documentation that explains how the floodplain delineation was completed. Jewell referenced the following technical studies in the preparation of this memorandum.

- Crysler and Latham. (1979). *Lane Creek Village of Wellington Water Management Study*.
- Jewell Engineering. (2018). *Preliminary Stormwater Management Report*

Quinte Conservation regulates Lane Creek and any fill placement near the creek must be reviewed and approved by them under their Regulation 319/09.

The Lane Creek watershed encompasses an area of 14.56km² upstream of the Millennium Trail. It is largely rural with a mix of mostly agricultural use and some bush. The creek flows through the Village of Wellington and discharges into Lake Ontario. The village is susceptible to flooding and no flood increase would be tolerable.

The proposed residential development is situated at the lower reaches of the study area.

2 Hydrology

Crysler and Latham (CCL) reported the peak flow at the Millennium Trail crossing (Jewell Node A) was 12.3cms. Jewell reviewed the hydrology using Environment Canada IDF curves (available in 2018) to compare with the peak flows from the previous investigation. A hydrologic model was prepared for the investigation of peak flows using OTTHYMO and Jewell determined that a peak flow of 14.65cms should be used. Model inputs and results are discussed below.

2.1 Model Inputs

Hydrologic inputs were assembled from the 2018 SWM report and the Chrysler and Latham (CCL) study.

A summary of inputs is included in Table 2-1. The interested reader is directed to the content following the table to understand how the model inputs were derived.

Table 2-1: OTTHYMO Model Inputs

Catchment	Area (ha)	Ia (mm)	CN	Tp (hr)
101	508.05	8	70.3	2.44
102	143.10	8	68.4	2.3
103	110.00	8	68.4	1.9
104	58.29	8	68.4	1.6
105	269.70	8	69.4	1.98
106	125.10	8	69.6	1.77
107	90.21	5	71.7	1.26
108	93.91	8	69.6	1.32
109	57.58	8	69.1	0.87
Total	1,455.94			

2.1.1 Precipitation

100-Yr Storm

The floodplain mapping investigation used a 6-hr SCS Type II hyetograph for the hydrologic model. The SCS distribution was based on a total depth of 3" (76.2mm) of precipitation. The current investigation employed the total precipitation depths from the Belleville IDF curves - station Environment Canada station 6150689 (See Appendix A). The 6-hr 100-yr storm used in the current investigation has a total depth of 80.5mm.

Table 2-2: Precipitation Statistics used in Jewell Study

Return Period (yrs)	Distribution	Duration	Depth (mm)
100	SCS Type II	6-Hr	80.5

The 6-hr SCS distribution of the rainfall was referenced from Design Chart 104 (Ontario Ministry of Transportation, 1997, p. 19) – see Table 2-3.

Table 2-3: 6 –Hr SCS Distribution

Time Step (hr)	0.5	1	1.5	2	2.5	2.75	3	3.5	4	4.5	5	6
Depth (%)	2	3	3	5	6	15	39	11	5	4	3	4

2.1.2 Catchment Area

Catchments were developed from the LiDAR supplied by Quinte Conservation and enhanced with topographic survey completed by Jewell Engineering.

The watershed and subcatchments were delineated by hand at points of interest that were typically positioned at road crossings or tributary junctions.

2.1.3 Time to Peak

The OTTHYMO model requires a time to peak (T_p) for rural areas modelled using the Nash unit hydrograph. Time to peak is derived from the time of concentration by Equation 1.

Equation 1

$$T_p = 0.67 \times T_c$$

Where:

T_p = Time to peak (min)

T_c = Time of concentration (min)

Time of concentration is determined following the Airport Method that is presented in the Ministry of Transportation Drainage Manual (Ontario Ministry of Transportation, 1997).

The Airport Method Formula is given by Equation 2.

Equation 2: Airport Method of Determining Time of Concentration

$$T_c = \frac{3.26 \times (1.1 - C) \times L^{0.5}}{S_w^{0.33}}$$

Where:

T_c = Time of concentration (min)

C = runoff coefficient (dimensionless)

L = Length of the flow path (m)

S_w = Slope of the watershed (%)

2.1.4 Curve Number

Curve numbers are used in the Soil Conservation Services (now known as the *National Resources Conservation Service*) methodology for estimating the proportion of precipitation that will runoff the lands from that portion that will infiltrate. Curve numbers are a function of soil type, land cover, slope, and land use. The higher the curve number – the greater the proportion of precipitation that is expected to runoff the lands.

Curve numbers were selected from the CCL study and compared to soils mapping from the Soil Survey of Prince Edward County (Dominion Department of Agriculture and the Ontario Agricultural College, 1948).

Jewell found the resulting runoff coefficients were higher than expected for a low slope rural watershed and reviewed the curve numbers (CN). CN values were adjusted slightly lower to produce runoff coefficients in the range of 0.34 to 0.36 that, although high, are reasonable for spring runoff conditions.

2.1.5 Linear Reservoirs

OTTHYMO provides the user the opportunity to adjust the number of linear reservoirs that results in a change in the shape of the runoff hydrographs. Typically, the number of linear reservoirs used is 3. Jewell found that with n = 3, peak flows were high. Jewell adjusted the number of linear reservoirs to 2. This resulted in slightly reduced peak runoff and agreed better with peak flows from OFAT III and CCL.

2.2 Nodes

Points of interest are placed within the model to reveal peak flows at road crossings or at junctions of tributaries. See Table 2-4.

A model schematic is provided in Figure 2-1.

Table 2-4: Points of Interest (Nodes)

Node	Location	Peak Flow (cms)
A	Millennium Trail Crossing	14.65
B	Belleville Street Crossing	14.31
C	Junction	13.48
D	Gilead Road	11.6
E	Junction	8.75
F	Junction	7.36

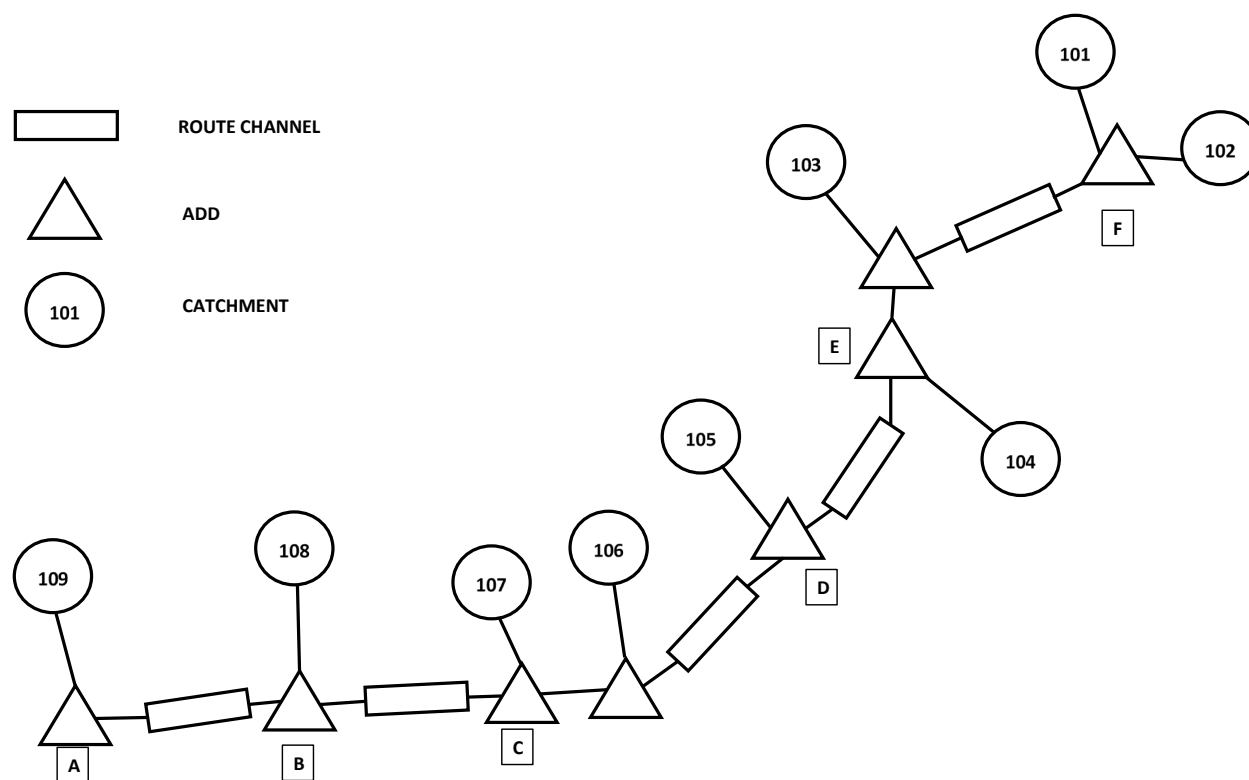


Figure 2-1: Watershed Schematic

2.3 Model Results

One hundred-year peak flows were compared from various sources and the OTTHYMO model results from the current study (14.65cms) were recommended for use in the subsequent hydraulic modelling investigation. A summary of peak flows that were reviewed is listed in Table 2-5.

Table 2-5: 100-Year Peak Flows

Source	Peak Flow (cms)
CCL	12.3
OFAT iii (Index Flood)	8.02
OFAT iii (Multiple Regression)	11.1
OTTHYMO (Jewell)	14.65

3 Hydraulics

Jewell prepared a hydraulic model of the Lane Creek watershed from the Millennium Trail upstream beyond Belleville Street. The model was prepared using HEC-RAS version 5.0.7 with RAS Mapper. Elevations were derived from LiDAR data supplied by Quinte Conservation and supplemented by topographic survey completed by Jewell Engineering.

Two models were developed. An existing conditions model was prepared to show flooding that would occur from the 100-year event assuming no alterations are made to the existing grades. A second model was prepared to review the impacts from proposed alterations within the floodplain of Lane Creek to accommodate the necessary stormwater management facilities and a small park block at the Millennium Trail. The portions to be filled have very shallow flooding and do not contribute to the flow path and it was found that water surface elevations are not affected. See Table 3-1 for results.

Table 3-1: HEC-RAS Water Surface Elevations (m)

X-Section	WSEL		
	Existing	Proposed	Difference
4339	85.69	85.68	-0.01
3893	85.68	85.68	0
3538	85.68	85.67	-0.01
3043	85.67	85.67	0
2840	85.67	85.67	0
2738	85.67	85.66	-0.01
Belleville Street Crossing - Twin 1600mm CSP			
2646	84.42	84.4	-0.02
2570	84.59	84.57	-0.02
2408	84.55	84.52	-0.03
2302	84.5	84.47	-0.03
2011	84.37	84.37	0
1733	83.93	83.93	0
1461	83.94	83.94	0
1328	83.94	83.94	0
1124	83.63	83.63	0
Millennium Trail Crossing - 3.5m Span Concrete Rigid Framed Bridge Structure			
2646	84.42	84.4	-0.02

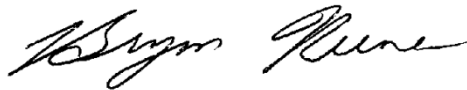
4 Conclusions

Hydrologic and hydraulic modelling were completed to identify the floodplain elevations from a statistical 100-year flood throughout the proposed Kaitlin (Country Club Estates) subdivision lands. The 100-year peak flow is increased from the 12.3cms used in the former floodplain mapping by CCL in 1979 to 14.65cms used in the current study. The reason for this increase is largely due to the increased rainfall depth at the Belleville precipitation station.

The development proposes slight alterations to the perimeter of the floodplain for the stormwater management facilities and a small park block, but the water surface elevations are not increased.

As a result of this investigation, Jewell recommends that the 100-year floodlines be established as presented in Drawing 1.

Prepared and Submitted by:



Bryon Keene, P.Eng.

Jewell Engineering Ltd.



5 References

- Crysler & Lathem Ltd. (1979). *Lane Creek Village of Wellington Water Management Study*.
- Dominion Department of Agriculture and the Ontario Agricultural College. (1948). *Soil Survey of Prince Edward County*. Guelph.
- Ontario Ministry of the Environment. (2003). *Stormwater Management Planning and Design Manual*. Queen's Printer for Ontario.
- Ontario Ministry of Transportation. (1997). *Drainage Management Manual*.
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- USGS. (2012). *Estimating Basin Lagtime and Hydrograph-Timing Indexes Used to Characterize Stormflows for Runoff-Quality Analysis*.

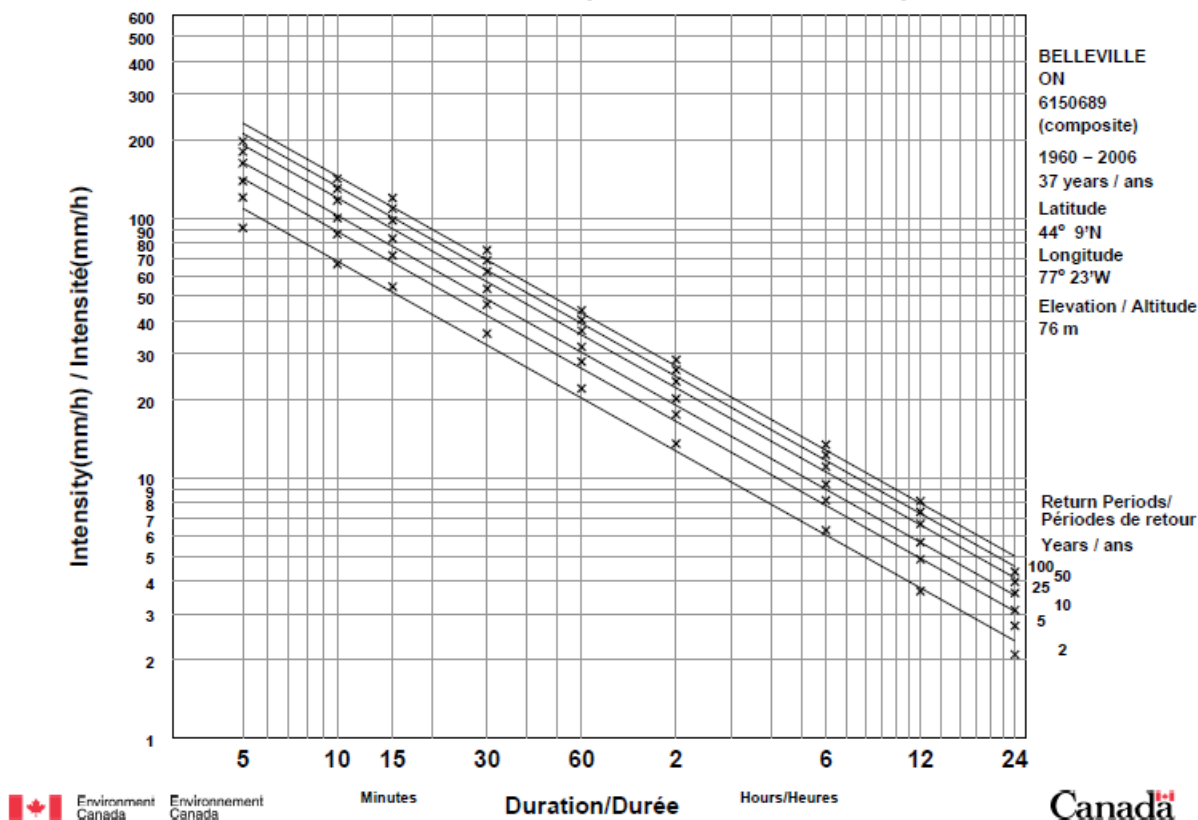
APPENDIX A

Belleville, Ontario

Station 6150689

Short Duration Rainfall Intensity–Duration–Frequency Data Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée

2012/02/09



Environment Canada/Environnement Canada

Short Duration Rainfall Intensity-Duration-Frequency Data
Données sur l'intensité, la durée et la fréquence des chutes
de pluie de courte durée

Gumbel - Method of moments/Méthode des moments

2012/02/09

=====

BELLEVILLE ON 6150689
(composite)
Latitude: 44 9'N Longitude: 77 23'W Elevation/Altitude: 76 m
Years/Années : 1960 - 2006 # Years/Années : 37

=====

Table 1 : Annual Maximum (mm)/Maximum annuel (mm)

Year Année	5 min	10 min	15 min	30 min	1 h	2 h	6 h	12 h	24 h
1960	6.3	9.1	12.4	23.4	25.4	35.1	53.8	55.1	55.9
1961	6.1	7.9	8.9	12.2	18.0	18.8	23.9	34.0	36.3
1963	12.4	19.0	23.1	28.4	30.7	31.0	31.0	31.7	44.4
1964	4.3	5.6	7.4	12.2	12.4	20.6	45.2	45.7	45.7
1966	6.3	8.9	10.4	11.9	13.2	16.0	32.8	37.8	38.1
1967	7.4	9.9	10.4	10.4	11.9	13.2	26.4	42.4	58.7
1968	7.9	11.9	13.7	18.5	21.8	27.2	43.9	57.1	57.1
1969	5.8	9.7	13.0	17.5	24.4	31.0	37.8	43.2	62.2
1971	7.4	11.4	13.0	23.9	25.1	25.4	25.7	25.7	32.5
1972	9.4	10.7	11.7	12.4	14.7	20.6	28.2	33.5	50.5
1973	7.4	10.7	11.9	18.0	21.3	21.8	37.3	45.5	48.0
1974	10.9	15.2	17.8	25.4	25.4	25.4	34.3	42.7	42.7
1977	4.8	8.1	9.9	14.7	25.1	30.2	60.5	66.0	66.0
1980	13.2	16.9	19.0	20.5	20.5	34.6	46.9	47.6	59.6
1981	-99.9	-99.9	13.3	25.5	29.4	34.6	46.2	49.2	57.4
1982	4.6	8.5	10.1	14.2	18.3	24.7	39.8	45.0	45.0
1983	6.5	8.9	10.5	18.4	22.2	30.7	39.6	39.6	50.3
1984	5.1	8.1	10.1	11.3	19.7	23.7	33.4	51.4	55.1
1985	10.5	16.2	20.0	27.0	27.4	42.3	42.3	44.5	44.5
1986	9.1	14.4	16.4	23.2	25.2	35.0	59.2	68.8	78.9
1987	4.3	6.6	9.3	14.2	24.7	37.1	39.2	39.2	39.2
1988	3.7	6.2	7.4	8.6	9.2	10.6	20.8	22.2	28.2
1989	14.5	16.7	17.9	18.4	24.2	24.2	27.7	27.7	37.7
1990	6.9	8.3	10.0	12.3	13.6	20.8	29.7	34.8	38.7

KAITLIN CORP – COUNTRY CLUB ESTATES
Floodplain Delineation

1991	8.5	13.8	18.5	18.8	32.0	32.4	32.4	32.4	32.7
1992	6.3	7.6	8.4	13.2	18.8	21.9	38.2	48.3	50.1
1993	8.2	15.8	23.6	28.3	28.3	28.3	-99.9	-99.9	69.6
1994	8.8	10.2	14.5	18.3	23.6	25.5	38.2	49.2	52.8
1995	8.0	12.9	14.9	19.3	27.5	31.5	48.5	58.5	67.3
1996	6.9	10.4	13.4	19.2	25.1	41.3	41.5	53.8	53.8
1997	10.3	16.8	20.9	25.5	42.8	50.0	56.0	56.0	56.0
1998	9.5	12.1	15.1	22.1	25.0	32.6	38.6	38.6	50.2
1999	9.6	13.1	17.9	23.2	29.4	36.9	42.8	72.7	72.7
2000	10.4	13.4	14.7	16.8	29.0	39.8	52.0	52.4	53.0
2001	7.4	10.1	11.0	11.8	16.7	17.4	21.2	31.6	39.8
2002	7.1	9.4	14.0	21.0	22.4	26.0	39.4	44.2	49.8
2003	7.6	13.5	20.1	26.2	27.0	27.0	31.1	-99.9	56.2
2004	14.4	22.1	28.8	33.3	33.3	49.0	89.9	114.4	124.5
2006	9.0	14.7	18.8	19.5	19.5	19.5	37.3	42.7	59.8

# Yrs.	38	38	39	39	39	39	38	37	39
Années									
Mean	8.1	11.7	14.4	18.9	23.2	28.6	39.8	46.6	52.8
Moyenne									
Std. Dev.	2.7	3.8	4.9	6.0	6.7	9.0	13.0	16.3	16.4
Écart-type									
Skew.	0.69	0.66	0.83	0.29	0.20	0.37	1.60	2.08	2.19
Dissymétrie									
Kurtosis	3.44	3.23	3.63	2.58	4.05	3.21	7.90	10.48	11.56

*-99.9 Indicates Missing Data/Données manquantes

Warning: annual maximum amount greater than 100-yr return period amount

Avertissement : la quantité maximale annuelle excède la quantité
pour une période de retour de 100 ans

Year/Année	Duration/Durée	Data/Données	100-yr/ans
2004	6 h	89.9	80.5
2004	12 h	114.4	97.6
2004	24 h	124.5	104.4

Table 2a : Return Period Rainfall Amounts (mm)
Quantité de pluie (mm) par période de retour

Duration/Durée	2	5	10	25	50	100	#Years
	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	Années
5 min	7.6	10.0	11.6	13.5	15.0	16.5	38
10 min	11.1	14.4	16.7	19.5	21.6	23.6	38
15 min	13.6	18.0	20.8	24.5	27.2	29.9	39
30 min	18.0	23.2	26.7	31.1	34.4	37.6	39
1 h	22.1	28.0	31.9	36.8	40.5	44.1	39
2 h	27.1	35.1	40.4	47.0	52.0	56.9	39
6 h	37.7	49.1	56.7	66.3	73.4	80.5	38
12 h	44.0	58.3	67.8	79.9	88.8	97.6	37
24 h	50.1	64.7	74.3	86.5	95.5	104.4	39

Table 2b :

Return Period Rainfall Rates (mm/h) - 95% Confidence limits
Intensité de la pluie (mm/h) par période de retour - Limites de confiance de 95%

Duration/Durée	2	5	10	25	50	100	#Years
	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	Années
5 min	91.6	120.0	138.8	162.6	180.2	197.7	38
	+/- 9.4	+/- 15.8	+/- 21.3	+/- 28.8	+/- 34.4	+/- 40.1	38
10 min	66.5	86.6	100.0	116.9	129.4	141.8	38
	+/- 6.7	+/- 11.2	+/- 15.1	+/- 20.4	+/- 24.4	+/- 28.5	38
15 min	54.4	71.8	83.4	97.9	108.7	119.5	39
	+/- 5.7	+/- 9.6	+/- 12.9	+/- 17.4	+/- 20.8	+/- 24.3	39
30 min	35.9	46.5	53.4	62.2	68.8	75.3	39
	+/- 3.4	+/- 5.8	+/- 7.8	+/- 10.5	+/- 12.6	+/- 14.7	39
1 h	22.1	28.0	31.9	36.8	40.5	44.1	39
	+/- 1.9	+/- 3.2	+/- 4.4	+/- 5.9	+/- 7.1	+/- 8.2	39
2 h	13.5	17.5	20.2	23.5	26.0	28.5	39
	+/- 1.3	+/- 2.2	+/- 3.0	+/- 4.0	+/- 4.8	+/- 5.6	39
6 h	6.3	8.2	9.5	11.1	12.2	13.4	38
	+/- 0.6	+/- 1.1	+/- 1.4	+/- 1.9	+/- 2.3	+/- 2.7	38
12 h	3.7	4.9	5.7	6.7	7.4	8.1	37
	+/- 0.4	+/- 0.7	+/- 0.9	+/- 1.2	+/- 1.5	+/- 1.7	37
24 h	2.1	2.7	3.1	3.6	4.0	4.4	39
	+/- 0.2	+/- 0.3	+/- 0.4	+/- 0.6	+/- 0.7	+/- 0.8	39

Table 3 : Interpolation Equation / Équation d'interpolation: $R = A \cdot T^B$

R = Interpolated Rainfall rate (mm/h)/Intensité interpolée de la pluie (mm/h)

RR = Rainfall rate (mm/h) / Intensité de la pluie (mm/h)

T = Rainfall duration (h) / Durée de la pluie (h)

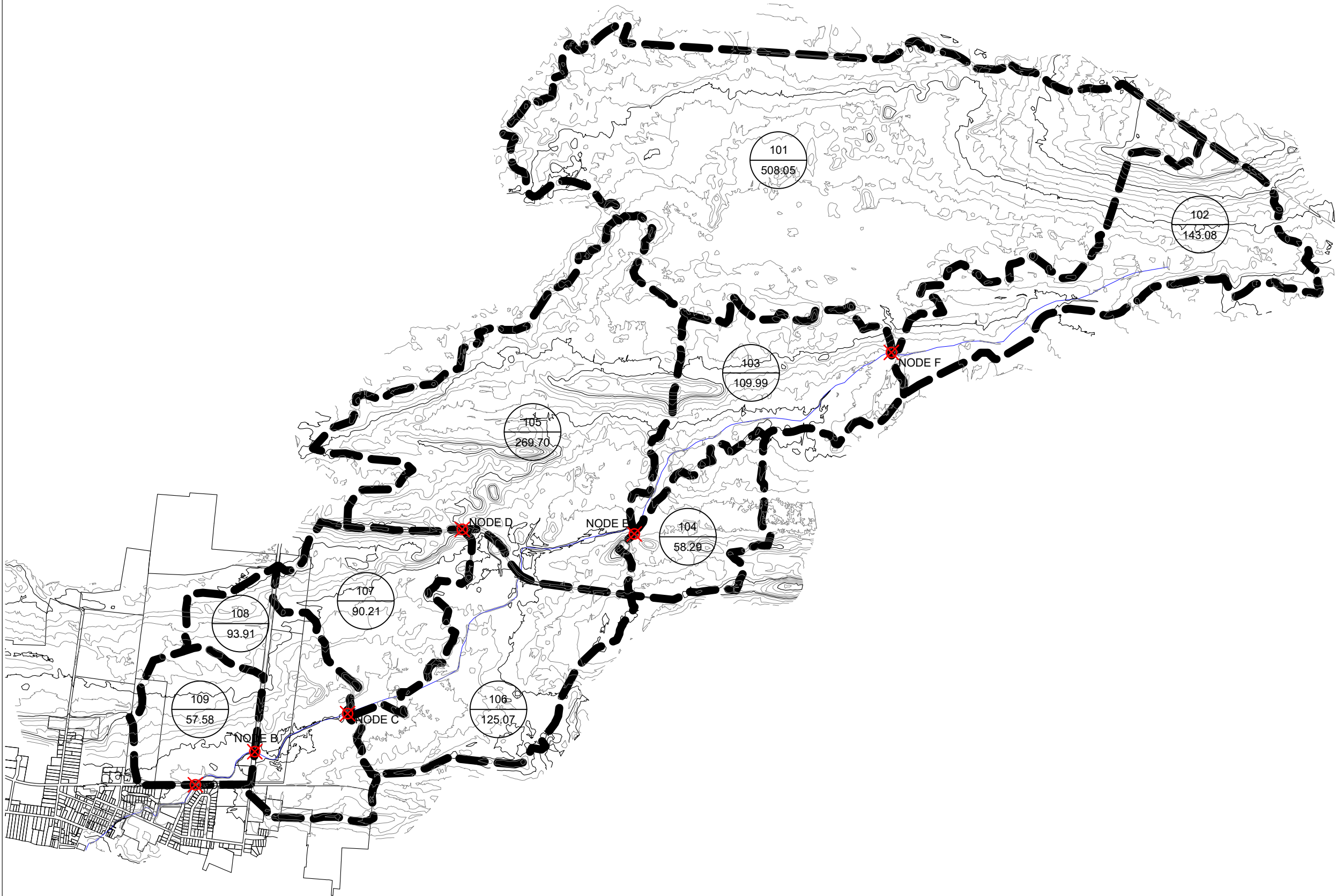
Statistics/Statistiques	2	5	10	25	50	100
	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans	yr/ans
Mean of RR/Moyenne de RR	32.9	42.9	49.5	57.9	64.1	70.3
Std. Dev. /Écart-type (RR)	31.7	41.6	48.1	56.4	62.5	68.6
Std. Error/Erreur-type	6.8	8.7	9.9	11.4	12.6	13.7
Coefficient (A)	20.3	26.4	30.4	35.5	39.3	43.0
Exponent/Exposant (B)	-0.677	-0.677	-0.678	-0.678	-0.678	-0.678
Mean % Error/% erreur moyenne	7.8	7.4	7.2	7.2	7.2	7.2

APPENDIX B

Figures

Watershed

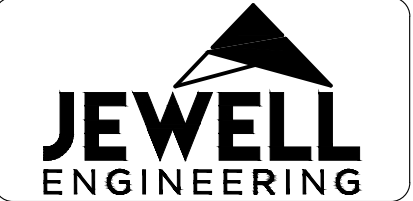
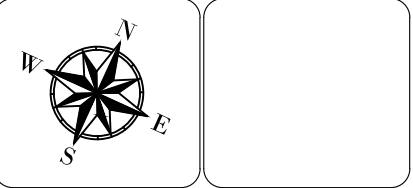
Floodplain Drawing



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.
** DRAWINGS ARE NOT TO BE SCALED **

REVISIONS			
NO.	DATE	DESCRIPTION	BY

- LEGEND**
- 101 CATCHMENT #
 - 508.05 CATCHMENT AREA (ha)
 - X HYDROLOGIC NODE



COUNTRY CLUB ESTATES
KAITLIN CORPORATION

WELLINGTON, ONTARIO

LANE CREEK CATCHMENT AREA

DRAWN BY:	JH	PROJECT NO:	170-4226
DESIGNED BY:	JH	DATE:	April 2020
CHECKED BY:	BK	SCALE:	HORIZONTAL -1:25,000 VERTICAL -N/A
APPROVED BY:	BK	CONTRACT NO:	DRAWING NO: CTCH-01



REGULATORY FLOODLINE ELEVATIONS		
RIVER STATION (ft)	EXISTING ELEV. (m)	PROPOSED ELEV. (m)
4+339	85.69	85.69
3+893	85.68	85.69
3+538	85.68	85.68
3+043	85.67	85.67
2+840	85.67	85.67
2+738	85.66	85.67
BELLEVILLE ST CROSSING		
2+646	84.55	84.55
2+570	84.61	84.60
2+408	84.58	84.57
2+302	84.00	84.52
2+011	84.38	84.36
1+733	83.94	83.94
1+461	83.95	83.95
1+328	83.94	83.94
MILLENNIUM TRAIL CROSSING		
1+124	83.63	83.63

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.
** DRAWINGS ARE NOT TO BE SCALED **

REVISIONS			
NO.	DATE	DESCRIPTION	BY

LEGEND

85.68

85.68

3+043

EXISTING REGULATORY FLOODLINE ELEVATION
PROPOSED REGULATORY FLOODLINE ELEVATION
RIVER STATION (ft)

EXISTING REGULATORY FLOODLINE
PROPOSED REGULATORY FLOODLINE
FILL LINE

N

W

E

S

LICENSED PROFESSIONAL ENGINEER

B. C. KEENE

APR 6/2020

PROVINCE OF ONTARIO



COUNTRY CLUB ESTATES
KAITLIN CORPORATION

WELLINGTON, ONTARIO

REGULATORY FLOODLINE OF LANE CREEK

DRAWN BY:
JH

DESIGNED BY:
JH

CHECKED BY:
BK

APPROVED BY:
BK

PROJECT NO:
170-4226

DATE:
April 2020

SCALE:
HORIZONTAL - 1:2,000
VERTICAL - N/A

CONTRACT NO:

DRAWING NO:
FLD-01

APPENDIX C

OTTHYMO Model Output

```

=====
      OOO      TTTTT  TTTTT  H   H   Y   Y   M   M   OOO      I N T E R H Y M O
      O   O      T      T      H   H       Y Y      MM MM  O   O      * * * 1989a * * *
      O   O      T      T      HHHHH       Y      M M M  O   O
      O   O      T      T      H   H       Y      M   M  O   O
      OOO      T      T      H   H       Y      M   M  OOO      _____ 01643

```

Distributed by the INTERHYMO Centre. Copyright (c), 1989. Paul Wisner & Assoc.

Input filename: kaitlin.dat
Output filename: kaitlin.out
Summary filename: kaitlin.sum

DATE: 01-30-2019

TIME: 15:04:31

COMMENTS: _____

```

-----
*
*****
** SIMULATION NUMBER:    1 **
*****

*
*      Kaitlin Corp - Country Club Estates
*      Belleville Road, Wellington, ON
*      Belleville Rainfall from AES December 2014 Curves
*      6Hr SCS Type II Rainfall Distribution
*      (6Hr was used in Lane Creek Water Management Study, 1979)
*      February 18, 2020
*      Patrick Quinn, E.I.T.
*
*      Preliminary Pond Sizing
*      Peak Flow in Lane Creek to compare with 1979
*      1979 by Crysler and Latham was 12.3cms
*
*
*****
*      2 Year Storm
*****
*      Pre-Development
*CHICAGO STORM      IUNITS=2 TD=1.0 HRS R=0.333 STD=5 MIN ICASE =2
*      (TIME, INTENSITY)= 5,89.5 10,63.6 15,49.6 30,31.2
*      60,19.1 120,11.8 360,5.7 720,3.3 1440,1.9 END=-1

```

KAITLIN CORP – COUNTRY CLUB ESTATES
Floodplain Delineation

```
*****
*
*           5 Year Storm
*****
*
*           Pre-Development
*CHICAGO STORM      IUNITS=2 TD=1.0 HRS R=0.333 STD=5 MIN ICASE =2
*                   (TIME, INTENSITY)= 5,120 10,86.6 15,71.8 30,46.5
*                   60,28 120,17.5 360,8.2 720,4.9 1440,2.7 END=-1
*****
*
*           10 Year Storm
*****
*
*           Pre-Development
*CHICAGO STORM      IUNITS=2 TD=1.0 HRS R=0.333 STD=5 MIN ICASE =2
*                   (TIME, INTENSITY)= 5,138.8 10,100 15,83.4 30,53.4
*                   60,31.9 120,20.2 360,9.5 720,5.7 1440,3.1 END=-1
*****
*
*           25 Year Storm
*****
*
*           Pre-Development
*CHICAGO STORM      IUNITS=2 TD=1.0 HRS R=0.333 STD=5 MIN ICASE =2
*                   (TIME, INTENSITY)= 5,162.6 10,116.9 15,97.9 30,62.2
*                   60,36.8 120,23.5 360,11.1 720,6.7 1440,3.6 END=-1
*****
*
*           50 Year Storm
*****
*
*           Pre-Development
*CHICAGO STORM      IUNITS=2 TD=1.0 HRS R=0.333 STD=5 MIN ICASE =2
*                   (TIME, INTENSITY)= 5,180.2 10,129.4 15,108.7 30,68.8
*                   60,40.5 120,26 360,12.2 720,7.4 1440,4 END=-1
*****
*
*           100 Year Storm
*****
*
*           Pre-Development
```

```
-----
|   READ STORM   |   Filename: 6SCS100.STM
| Ptotal= 80.50 mm |   Comments: 06 Hr SCS Type 2 - 100Yr
-----
```

TIME	RAIN	TIME	RAIN	TIME	RAIN	TIME	RAIN
hrs	mm/hr	hrs	mm/hr	hrs	mm/hr	hrs	mm/hr
.25	3.22	1.75	8.05	3.25	17.71	4.75	4.83
.50	3.22	2.00	8.05	3.50	17.71	5.00	4.83
.75	4.83	2.25	9.66	3.75	8.05	5.25	3.22
1.00	4.83	2.50	9.66	4.00	8.05	5.50	3.22
1.25	4.83	2.75	48.30	4.25	6.44	5.75	3.22
1.50	4.83	3.00	125.58	4.50	6.44	6.00	3.22

```
-----
*CHICAGO STORM      IUNITS=2 TD=6.0 HRS R=0.333 STD=5 MIN ICASE =2
*                   (TIME, INTENSITY)= 5,197.7 10,141.8 15,119.5 30,75.3
*                   60,44.1 120,28.5 360,13.4 720,8.1 1440,4.4 END=-1
*                   Catchment 101
```



```
-----
| CALIB                               |
| NASHYD      (0101) | Area      (ha)= 508.05   Curve Number   (CN)= 70.3
| ID= 2 DT= 5.0 min | Ia        (mm)=   8.00   # of Linear Res. (N)= 2.00
-----
                        U.H. Tp(hrs)=   2.44
```

NOTE: RAINFALL WAS TRANSFORMED TO 5.0 MIN. TIME STEP.

Unit Hyd Qpeak (cms)= 5.40

PEAK FLOW (cms)= 5.75 (i)
TIME TO PEAK (hrs)= 6.08
RUNOFF VOLUME (mm)= 29.06
TOTAL RAINFALL (mm)= 80.50
RUNOFF COEFFICIENT = .36

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```
-----
| SAVE HYD (0101) | AREA      (ha)= 508.05
| ID= 2 PCYC=400 | QPEAK     (cms)=   5.75 (i)
| DT= 5.0 min    | TPEAK     (hrs)=   6.08
-----
                        VOLUME      (mm)=  29.06
```

Filename: 101pre.TXT

Comments: Catchment 101 100-Yr

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```
-----
*                               Catchment 102
-----
| CALIB                               |
| NASHYD      (0102) | Area      (ha)= 143.10   Curve Number   (CN)= 68.4
| ID= 3 DT= 5.0 min | Ia        (mm)=   8.00   # of Linear Res. (N)= 2.00
-----
                        U.H. Tp(hrs)=   2.30
```

Unit Hyd Qpeak (cms)= 1.61

PEAK FLOW (cms)= 1.61 (i)
TIME TO PEAK (hrs)= 6.00
RUNOFF VOLUME (mm)= 27.51
TOTAL RAINFALL (mm)= 80.50
RUNOFF COEFFICIENT = .34

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```
-----
| SAVE HYD (0102) | AREA      (ha)= 143.10
| ID= 3 PCYC=341 | QPEAK     (cms)=   1.61 (i)
| DT= 5.0 min    | TPEAK     (hrs)=   6.00
-----
```

----- VOLUME (mm) = 27.51
Filename: 102pre.TXT
Comments: 100-Yr Pre

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Total of all Lands to Node F

| ADD HYD (0300) |
2 + 3 = 4
AREA QPEAK TPEAK R.V.
(ha) (cms) (hrs) (mm)
ID1= 2 (0101): 508.05 5.75 6.08 29.06
+ ID2= 3 (0102): 143.10 1.61 6.00 27.51
=====

ID = 4 (0300): 651.15 7.36 6.08 28.72

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

| SAVE HYD (0300) | AREA (ha) = 651.15
| ID= 4 PCYC=400 | QPEAK (cms) = 7.36 (i)
| DT= 5.0 min | TPEAK (hrs) = 6.08

VOLUME (mm) = 28.72
Filename: Fpre.TXT
Comments: 100-Yr Pre Node F

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Route Flows from Node F to Node E

| ROUTE CHN (0201) |
| IN= 4--> OUT= 1 | Routing time step (min) = 5.00

SECTION No =***** No. of Segments = 3
Channel Slp % = .28 Floodplain Slp % = .33

<----- CROSS SECTION DATA ----->

Distance	Elevation	Manning	
.00	101.00	.1000	
10.00	100.00	.1000 / .0500	Main Channel
11.00	99.40	.0500	Main Channel
12.50	99.40	.0500	Main Channel
13.50	100.00	.0500 / .1000	Main Channel
23.50	101.00	.1000	

Valley Sections = 1

Channel Length = 2287.0
Channel Slope % = .280

<----- TRAVEL TIME TABLE ----->

DEPTH	ELEV	VOLUME	FLOW RATE	VELOCITY	TRAV.TIME
(m)	(m)	(cu.m.)	(cms)	(m/s)	(min)
.08	99.48	.279E+03	.0	.18	216.12
.15	99.55	.600E+03	.1	.27	143.28
.23	99.63	.965E+03	.1	.33	113.96
.30	99.70	.137E+04	.2	.39	97.34
.38	99.78	.182E+04	.4	.44	86.34
.45	99.85	.232E+04	.5	.49	78.36
.53	99.93	.285E+04	.7	.53	72.22
.60	100.00	.343E+04	.8	.57	67.31
.69	100.09	.435E+04	1.2	.62	61.58
.78	100.18	.564E+04	1.6	.64	59.90
.87	100.27	.731E+04	2.0	.64	59.67
.96	100.36	.937E+04	2.6	.64	59.85
1.05	100.45	.118E+05	3.3	.64	60.02
1.15	100.55	.146E+05	4.1	.63	60.03
1.24	100.64	.178E+05	5.0	.64	59.85
1.33	100.73	.213E+05	6.0	.64	59.48
1.42	100.82	.253E+05	7.1	.65	58.96
1.51	100.91	.296E+05	8.5	.65	58.33
1.60	101.00	.343E+05	9.9	.66	57.60

			<---- hydrograph ---->			<--- channel --->	
	AREA		QPEAK	TPEAK	R.V.	MAX DEPTH	MAX VEL
	(ha)		(cms)	(hrs)	(mm)	(m)	(m/s)
INFLOW : ID= 4 (0300)	651.15		7.36	6.08	28.72	1.43	.65
OUTFLOW: ID= 1 (0201)	651.15		6.83	7.08	28.71	1.39	.64

* Catchment 103

CALIB			
NASHYD (0103)		Area (ha)= 110.00	Curve Number (CN)= 68.4
ID= 2 DT= 5.0 min		Ia (mm)= 8.00	# of Linear Res.(N)= 2.00
		U.H. Tp(hrs)= 1.90	

Unit Hyd Qpeak (cms)= 1.50

PEAK FLOW (cms)= 1.44 (i)
TIME TO PEAK (hrs)= 5.50
RUNOFF VOLUME (mm)= 27.51
TOTAL RAINFALL (mm)= 80.50
RUNOFF COEFFICIENT = .34

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

KAITLIN CORP – COUNTRY CLUB ESTATES
Floodplain Delineation

* Catchment 104

CALIB				
NASHYD (0100)		Area (ha)=	58.29	Curve Number (CN)= 68.4
ID= 3 DT= 5.0 min		Ia (mm)=	8.00	# of Linear Res. (N)= 2.00
		U.H. Tp(hrs)=	1.60	

Unit Hyd Qpeak (cms)= .95

PEAK FLOW (cms)= .87 (i)
 TIME TO PEAK (hrs)= 5.08
 RUNOFF VOLUME (mm)= 27.50
 TOTAL RAINFALL (mm)= 80.50
 RUNOFF COEFFICIENT = .34

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Total of all Lands to Node E

ADD HYD (0301)					
2 + 3 = 4		AREA	QPEAK	TPEAK	R.V.
		(ha)	(cms)	(hrs)	(mm)
ID1= 2 (0103):		110.00	1.44	5.50	27.51
+ ID2= 3 (0100):		58.29	.87	5.08	27.50
		=====			
ID = 4 (0301):		168.29	2.31	5.33	27.51

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

ADD HYD (0302)					
1 + 4 = 5		AREA	QPEAK	TPEAK	R.V.
		(ha)	(cms)	(hrs)	(mm)
ID1= 1 (0201):		651.15	6.83	7.08	28.71
+ ID2= 4 (0301):		168.29	2.31	5.33	27.51
		=====			
ID = 5 (0302):		819.44	8.75	6.58	28.47

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

SAVE HYD (0302)		AREA	(ha)=	819.44
ID= 5 PCYC=491		QPEAK	(cms)=	8.75 (i)
DT= 5.0 min		TPEAK	(hrs)=	6.58
		VOLUME	(mm)=	28.47

Filename: Epre.TXT

Comments: 100-Yr Pre at Node E

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```

-----
*                               Route Flows from Node E to Node D
-----
| ROUTE CHN (0202) |
| IN= 5---> OUT= 1 |      Routing time step (min)'=  5.00
-----

```

```

SECTION No      =*****      No. of Segments   =    3
Channel Slp %   =   .11      Floodplain Slp %   =   .11

```

```

<----- CROSS SECTION DATA ----->
Distance      Elevation      Manning
    .00         101.00         .1000
   10.00         100.00      .1000 / .0500  Main Channel
   11.00          99.40         .0500      Main Channel
   12.50          99.40         .0500      Main Channel
   13.50         100.00      .0500 / .1000  Main Channel
   23.50         101.00         .1000

```

```

Valley Sections =    1
Channel Length  =  864.0
Channel Slope % =   .110

```

```

<----- TRAVEL TIME TABLE ----->
DEPTH      ELEV      VOLUME      FLOW RATE      VELOCITY      TRAV.TIME
(m)         (m)      (cu.m.)      (cms)         (m/s)         (min)
.08      99.48      .105E+03      .0             .11          130.26
.15      99.55      .227E+03      .0             .17           86.36
.23      99.63      .365E+03      .1             .21           68.69
.30      99.70      .518E+03      .1             .25           58.67
.38      99.78      .689E+03      .2             .28           52.04
.45      99.85      .875E+03      .3             .30           47.23
.53      99.93      .108E+04      .4             .33           43.53
.60     100.00      .130E+04      .5             .35           40.57
.69     100.09      .164E+04      .7             .39           37.13
.78     100.18      .213E+04      1.0            .40           36.17
.87     100.27      .276E+04      1.3            .40           36.12
.96     100.36      .354E+04      1.6            .40           36.34
1.05     100.45      .446E+04      2.0            .39           36.56
1.15     100.55      .552E+04      2.5            .39           36.69
1.24     100.64      .672E+04      3.1            .39           36.70
1.33     100.73      .807E+04      3.7            .39           36.60
1.42     100.82      .955E+04      4.4            .40           36.39
1.51     100.91     .112E+05      5.2            .40           36.10
1.60     101.00     .130E+05      6.0            .40           35.74

```

**** WARNING: TRAVEL TIME TABLE EXCEEDED

<---- hydrograph ----> <--- channel --->

	AREA (ha)	QPEAK (cms)	TPEAK (hrs)	R.V. (mm)	MAX DEPTH (m)	MAX VEL (m/s)
INFLOW : ID= 5 (0302)	819.44	8.75	6.58	28.47	1.60	.40
OUTFLOW: ID= 1 (0202)	819.44	8.56	7.25	28.47	1.60	.40

* Catchment 105

CALIB						
NASHYD (0105)		Area (ha)= 269.70		Curve Number (CN)= 69.4		
ID= 2 DT= 5.0 min		Ia (mm)= 8.00		# of Linear Res. (N)= 2.00		
		U.H. Tp (hrs)= 1.98				

Unit Hyd Qpeak (cms)= 3.54

PEAK FLOW (cms)= 3.52 (i)
 TIME TO PEAK (hrs)= 5.58
 RUNOFF VOLUME (mm)= 28.31
 TOTAL RAINFALL (mm)= 80.50
 RUNOFF COEFFICIENT = .35

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Total of all Lands to Node D (Gilead Road)

ADD HYD (0303)					
1 + 2 = 4		AREA (ha)	QPEAK (cms)	TPEAK (hrs)	R.V. (mm)
ID1= 1 (0202):		819.44	8.56	7.25	28.47
+ ID2= 2 (0105):		269.70	3.52	5.58	28.31
=====					
ID = 4 (0303):		1089.14	11.60	6.75	28.43

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

SAVE HYD (0303)		AREA (ha)=1089.14
ID= 4 PCYC=513		QPEAK (cms)= 11.60 (i)
DT= 5.0 min		TPEAK (hrs)= 6.75
		VOLUME (mm)= 28.43

Filename: Dpre.TXT
 Comments: 100-Yr Pre at Node D

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Route Flows from Node D to Node C

| ROUTE CHN (0203) |
| IN= 4---> OUT= 3 | Routing time step (min)'= 5.00

SECTION No =***** No. of Segments = 3
Channel Slp % = .26 Floodplain Slp % = .24

<----- CROSS SECTION DATA ----->

Distance	Elevation	Manning	
.00	101.00	.1000	
10.00	100.00	.1000 / .0500	Main Channel
11.00	99.40	.0500	Main Channel
12.50	99.40	.0500	Main Channel
13.50	100.00	.0500 / .1000	Main Channel
23.50	101.00	.1000	

Valley Sections = 1
Channel Length = 1515.0
Channel Slope % = .260

<----- TRAVEL TIME TABLE ----->

DEPTH (m)	ELEV (m)	VOLUME (cu.m.)	FLOW RATE (cms)	VELOCITY (m/s)	TRAV.TIME (min)
.08	99.48	.185E+03	.0	.17	148.57
.15	99.55	.398E+03	.1	.26	98.50
.23	99.63	.639E+03	.1	.32	78.34
.30	99.70	.909E+03	.2	.38	66.92
.38	99.78	.121E+04	.3	.43	59.35
.45	99.85	.153E+04	.5	.47	53.87
.53	99.93	.189E+04	.6	.51	49.65
.60	100.00	.227E+04	.8	.55	46.27
.69	100.09	.288E+04	1.1	.60	42.36
.78	100.18	.374E+04	1.5	.61	41.30
.87	100.27	.485E+04	2.0	.61	41.28
.96	100.36	.620E+04	2.5	.61	41.59
1.05	100.45	.781E+04	3.1	.60	41.91
1.15	100.55	.967E+04	3.8	.60	42.12
1.24	100.64	.118E+05	4.7	.60	42.20
1.33	100.73	.141E+05	5.6	.60	42.14
1.42	100.82	.168E+05	6.7	.60	41.96
1.51	100.91	.196E+05	7.8	.61	41.68
1.60	101.00	.227E+05	9.2	.61	41.33

**** WARNING: TRAVEL TIME TABLE EXCEEDED

	AREA (ha)	QPEAK (cms)	TPEAK (hrs)	R.V. (mm)	MAX DEPTH (m)	MAX VEL (m/s)
INFLOW : ID= 4 (0303)	1089.14	11.60	6.75	28.43	1.59	.61
OUTFLOW: ID= 3 (0203)	1089.14	11.22	7.50	28.43	1.60	.61

* Catchment 106

CALIB			
NASHYD (0106)		Area (ha)= 125.10	Curve Number (CN)= 69.6
ID= 1 DT= 5.0 min		Ia (mm)= 8.00	# of Linear Res. (N)= 2.00
-----		U.H. Tp(hrs)= 1.77	

Unit Hyd Qpeak (cms)= 1.83

PEAK FLOW (cms)= 1.80 (i)
 TIME TO PEAK (hrs)= 5.25
 RUNOFF VOLUME (mm)= 28.47
 TOTAL RAINFALL (mm)= 80.50
 RUNOFF COEFFICIENT = .35

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Catchment 107

CALIB			
NASHYD (0107)		Area (ha)= 90.21	Curve Number (CN)= 71.7
ID= 2 DT= 5.0 min		Ia (mm)= 5.00	# of Linear Res. (N)= 2.00
-----		U.H. Tp(hrs)= 1.26	

Unit Hyd Qpeak (cms)= 1.86

PEAK FLOW (cms)= 1.92 (i)
 TIME TO PEAK (hrs)= 4.58
 RUNOFF VOLUME (mm)= 32.23
 TOTAL RAINFALL (mm)= 80.50
 RUNOFF COEFFICIENT = .40

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Total of all Lands to Node C (Confluence of Tribs 1&2)

ADD HYD (0304)					
1 + 2 = 4		AREA	QPEAK	TPEAK	R.V.
-----		(ha)	(cms)	(hrs)	(mm)
ID1= 1 (0106):		125.10	1.80	5.25	28.47
+ ID2= 2 (0107):		90.21	1.92	4.58	32.23
=====					
ID = 4 (0304):		215.31	3.65	4.83	30.05

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

```

-----
| ADD HYD   (0305) |
|  3 +  4 =  5   |
-----
                        AREA      QPEAK      TPEAK      R.V.
                        (ha)      (cms)      (hrs)      (mm)
      ID1= 3 (0203):  1089.14    11.22      7.50      28.43
+ ID2= 4 (0304):    215.31      3.65      4.83      30.05
=====
      ID = 5 (0305):  1304.45    13.48      7.00      28.69

```

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

```

-----
| SAVE HYD   (0305) |   AREA      (ha)=1304.45
| ID= 5   PCYC=548 |   QPEAK      (cms)= 13.48 (i)
| DT= 5.0 min      |   TPEAK      (hrs)=  7.00
-----
                        VOLUME      (mm)= 28.69

```

Filename: Cpre.TXT

Comments: 100-Yr Pre at Node C

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```

-----
*                               Route Flows from Node C to Node B

```

```

-----
| ROUTE CHN (0204) |
| IN= 5---> OUT= 1 |   Routing time step (min)'= 5.00
-----

```

SECTION No =***** No. of Segments = 3
Channel Slp % = .14 Floodplain Slp % = .14

<----- CROSS SECTION DATA ----->

Distance	Elevation	Manning	
.00	101.00	.1000	
10.00	100.00	.1000 / .0500	Main Channel
12.00	99.20	.0500	Main Channel
14.00	99.20	.0500	Main Channel
16.00	100.00	.0500 / .1000	Main Channel
26.00	101.00	.1000	

Valley Sections = 1
Channel Length = 700.0
Channel Slope % = .140

<----- TRAVEL TIME TABLE ----->

DEPTH	ELEV	VOLUME	FLOW RATE	VELOCITY	TRAV.TIME
(m)	(m)	(cu.m.)	(cms)	(m/s)	(min)
.09	99.29	.138E+03	.0	.14	84.18
.18	99.38	.304E+03	.1	.21	55.98
.27	99.47	.498E+03	.2	.26	44.56

KAITLIN CORP – COUNTRY CLUB ESTATES
Floodplain Delineation

.36	99.56	.719E+03	.3	.31	38.04
.44	99.64	.968E+03	.5	.35	33.69
.53	99.73	.124E+04	.7	.38	30.53
.62	99.82	.155E+04	.9	.42	28.09
.71	99.91	.188E+04	1.2	.45	26.13
.80	100.00	.224E+04	1.5	.48	24.51
.90	100.10	.273E+04	2.0	.52	22.38
1.00	100.20	.336E+04	2.6	.55	21.36
1.10	100.30	.413E+04	3.3	.56	20.86
1.20	100.40	.504E+04	4.1	.57	20.61
1.30	100.50	.609E+04	5.0	.57	20.48
1.40	100.60	.728E+04	6.0	.57	20.39
1.50	100.70	.861E+04	7.1	.57	20.30
1.60	100.80	.101E+05	8.3	.58	20.22
1.70	100.90	.117E+05	9.7	.58	20.11
1.80	101.00	.134E+05	11.2	.58	19.99

**** WARNING: TRAVEL TIME TABLE EXCEEDED

			<---- hydrograph ---->			<--- channel --->	
		AREA	QPEAK	TPEAK	R.V.	MAX DEPTH	MAX VEL
		(ha)	(cms)	(hrs)	(mm)	(m)	(m/s)
INFLOW :	ID= 5 (0305)	1304.45	13.48	7.00	28.69	1.79	.58
OUTFLOW:	ID= 1 (0204)	1304.45	13.39	7.33	28.69	1.80	.58

* Catchment 108

CALIB							
NASHYD (0108)		Area	(ha)=	93.91	Curve Number	(CN)=	69.6
ID= 2 DT= 5.0 min		Ia	(mm)=	8.00	# of Linear Res. (N)=	2.00	
		U.H. Tp	(hrs)=	1.32			

Unit Hyd Qpeak (cms)= 1.85

PEAK FLOW (cms)= 1.69 (i)
 TIME TO PEAK (hrs)= 4.67
 RUNOFF VOLUME (mm)= 28.47
 TOTAL RAINFALL (mm)= 80.50
 RUNOFF COEFFICIENT = .35

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Total of all Lands to Node B (Belleville Rd)

ADD HYD (0306)							
1 + 2 = 3		AREA	QPEAK	TPEAK	R.V.		
		(ha)	(cms)	(hrs)	(mm)		
ID1= 1 (0204):		1304.45	13.39	7.33	28.69		
+ ID2= 2 (0108):		93.91	1.69	4.67	28.47		

=====

ID = 3 (0306): 1398.36 14.31 7.08 28.68

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

SAVE HYD (0306)	AREA	(ha)=1398.36
ID= 3 PCYC=559	QPEAK	(cms)= 14.31 (i)
DT= 5.0 min	TPEAK	(hrs)= 7.08
-----	VOLUME	(mm)= 28.68

Filename: Bpre.TXT

Comments: 100-Yr Pre at Node B

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Route Flows from Node B to Node A

ROUTE CHN (0203)	
IN= 3---> OUT= 1	Routing time step (min)'= 5.00

SECTION No =***** No. of Segments = 3
Channel Slp % = .36 Floodplain Slp % = .38

<----- CROSS SECTION DATA ----->

Distance	Elevation	Manning	
.00	101.00	.1000	
10.00	100.00	.1000 / .0500	Main Channel
12.00	99.20	.0500	Main Channel
14.00	99.20	.0500	Main Channel
16.00	100.00	.0500 / .1000	Main Channel
26.00	101.00	.1000	

Valley Sections = 1
Channel Length = 464.0
Channel Slope % = .360

<----- TRAVEL TIME TABLE ----->

DEPTH	ELEV	VOLUME	FLOW RATE	VELOCITY	TRAV.TIME
(m)	(m)	(cu.m.)	(cms)	(m/s)	(min)
.09	99.29	.917E+02	.0	.22	34.80
.18	99.38	.202E+03	.1	.33	23.14
.27	99.47	.330E+03	.3	.42	18.42
.36	99.56	.477E+03	.5	.49	15.72
.44	99.64	.642E+03	.8	.56	13.93
.53	99.73	.825E+03	1.1	.61	12.62
.62	99.82	.103E+04	1.5	.67	11.61
.71	99.91	.125E+04	1.9	.72	10.80
.80	100.00	.148E+04	2.4	.76	10.13

KAITLIN CORP – COUNTRY CLUB ESTATES
Floodplain Delineation

.90	100.10	.181E+04	3.3	.84	9.25
1.00	100.20	.223E+04	4.2	.88	8.83
1.10	100.30	.274E+04	5.3	.90	8.62
1.20	100.40	.334E+04	6.5	.91	8.51
1.30	100.50	.404E+04	8.0	.92	8.45
1.40	100.60	.483E+04	9.6	.92	8.40
1.50	100.70	.571E+04	11.4	.92	8.36
1.60	100.80	.668E+04	13.4	.93	8.32
1.70	100.90	.775E+04	15.6	.93	8.27
1.80	101.00	.891E+04	18.1	.94	8.22

				<---- hydrograph ---->			<--- channel --->	
				AREA	QPEAK	TPEAK	R.V.	MAX DEPTH
				(ha)	(cms)	(hrs)	(mm)	(m)
								MAX VEL
								(m/s)
INFLOW : ID= 3 (0306)				1398.36	14.31	7.08	28.68	1.64
OUTFLOW: ID= 1 (0203)				1398.36	14.29	7.17	28.68	.93

* Catchment 109

```

| CALIB |
| NASHYD (0109) | Area (ha)= 57.58 Curve Number (CN)= 69.1
| ID= 2 DT= 5.0 min | Ia (mm)= 8.00 # of Linear Res. (N)= 2.00
----- U.H. Tp(hrs)= .87

```

Unit Hyd Qpeak (cms)= 1.72

PEAK FLOW (cms)= 1.41 (i)
TIME TO PEAK (hrs)= 4.00
RUNOFF VOLUME (mm)= 28.05
TOTAL RAINFALL (mm)= 80.50
RUNOFF COEFFICIENT = .35

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

* Total of all Lands to Node A (Millennium Trail)

```

| ADD HYD (0307) |
| 1 + 2 = 3 |
-----
ID1= 1 (0203): 1398.36 14.29 7.17 28.68
+ ID2= 2 (0109): 57.58 1.41 4.00 28.05
=====
ID = 3 (0307): 1455.94 14.65 7.00 28.65

```

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

SAVE HYD (0307)	AREA	(ha)=1455.94
ID= 3 PCYC=562	QPEAK	(cms)= 14.65 (i)
DT= 5.0 min	TPEAK	(hrs)= 7.00
-----	VOLUME	(mm)= 28.65

Filename: Apre.TXT

Comments: 100-Yr at Node A

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

FINISH

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