Preliminary Stormwater Management Design Brief

Nautical Lands

Wellings of Picton Phase 2

March 8, 2019

Kingston

4 Cataraqui St.

Unit D

Kingston, ON K7P 1R7

Phone 613-389-7250

Fax 613-389-2754

Belleville

1-71 Millennium Pkwy.

Belleville, ON

K8N 4Z5

Phone 613-969-1111

Fax 613-969-8988

Mississauga

2155 Leanne Blvd.

Suite 200A

Mississauga, ON L5K 2K8

Phone 905-855-1592

Fax 905-855-5428

Executive Summary

Jewell has prepared this stormwater management (SWM) report for the proposed 2.6 ha Phase 2 development at the Wellings of Picton site. The proposed Phase 2 site layout is a residential development that includes townhouses and an access road. The objective this report is to propose a SWM solution that meets the following SWM objectives.

SWM Objectives

- The **quality control** objective is to achieve Enhanced protection. Therefore, a minimum of 80% removal of TSS is recommended.
- The **quantity control** objective is to reduce post-development peak flows to the pre-development peak flows.
- **Sediment and erosion controls** should be provided during construction to minimize the potential for erosion of soils and construction materials. The release of sediment offsite should also be mitigated by sediment and erosion controls.

Site Drainage

Section 4 discusses the site drainage under existing and proposed conditions as well as quality and quantity control measures. For site drainage, pre-development and post-development catchment drawings are provided in Appendix B. Summary tables that correspond to these catchment drawings are provided below. Figure 4-1 shows the grassed swales, storage areas, and outlet locations used to control drainage after the proposed Phase 2 development. Outlet 1 is located at the west end of the site near Millennium Trail and Outlet 2 is located at the east end near County Road 49.

Table 1: Pre-Development Catchment Area Summary

	Catchment	Area (ha)	Drainage Direction
Phase 2 Drainage	200	1.12	Drains west towards Millennium Trail
	201	0.64	Drains east towards ditch at County Rd. 49
	202	1.04	Drains north towards culvert at County Rd. 49 (different outlet than Catchment 201)
	Phase 2 Total=	**2.80**	
External Drainage	300	0.54	External catchment that drains northwest towards Millennium Trail
	301	0.24	External catchment that drains east towards Catchment 201 and County Rd. 49
Total (including external drainage)=		**3.58**	

Table 2: Post-Development Catchment Area Summary

	Catchment	Area (ha)	Drainage Direction
Phase 2 Drainage	200	1.96	Drains west towards Millennium Trail
	201	0.84	Drain eastward towards County Rd. 49
	Phase 2 Total=	**2.80**	
External Drainage	300	0.54	External catchment that drains northwest towards Millennium Trail
	301	0.24	External catchment that drains east towards Catchment 201 and County Rd. 49
Total (including external drainage)=		**3.58**	

Quality Control

Jewell selected two oil-grit separator (OGS) units to provide quality control, one located at the west end of the site that outlets to the Millennium Trail ditch and the other at the east end of the site that outlets to the ditch alongside County Road 49. Each OGS unit is a downstream connection to the lowest outlet structure at each end of the site. Therefore, quality treatment flows are controlled prior to entering the separator units.

Pre-treatment is provided with the storage areas upstream of the OGS units. These temporary storage areas function as dry ponds and are expected to provide significant quality control benefits. Jewell referenced the 2003 MOE *SWM Planning and Design Manual* to design the dry pond facilities. The west SWM facility has sufficient storage volume to achieve 60% removal of total suspended solids (TSS) through pre-treatment (see Table 3). The east SWM facility has approximately 87% of the storage volume required to achieve 60% removal of TSS. Therefore, Jewell conservatively assumed that pre-treatment from this facility will only provide 30% TSS removal (see Table 3). Jewell estimated pre-treatment removal rates using Table 3.2 of the 2003 MOE *SWM Planning and Design Manual.*

The Downstream Defender models are a type of OGS unit that provide further TSS removal. Combined TSS removal rates are shown in Table 3. A weighted calculation in the table below also shows that the theoretical TSS removal rate for the overall site is 87.0 percent. This is greater than the treatment objective of *Enhanced* protection through 80% TSS removal. **Therefore, quality control objectives are met.**

Table 3: Quality Control Summary

Outlet	Outlet Location	Treatment Area (ha)	Thoeretical TSS Removal from Storage Facility Pre-Treatment (%)	Theoretical TSS Removal from OGS Unit After Pre-Treatment (%)	OGS Model	Combined Theoretical TSS Removal (%)	Weighted Theoretical TSS Removal (%) (2)	TSS Removal Objective (%) (3)	Check: (2) > (3)
1	West end near Millennium Trail	1.96	60	27	DD4	**87**	**87**	**80**	✓
2	East end near County Road 49	0.84	30	55	DD4	**85**			

<u>Quantity Control</u>

Jewell completed HEC-HMS simulations to estimate peak flows before and after the development. As mentioned above, the objective is to reduce post-development peak flows to pre-development peak flows. Quantity control is provided with two temporary ponding areas. One facility is located at the west end of the development and another is located near the east end of the development. This quantity control method was selected due to limited land availability and poor infiltration capabilities of underlying soils.

Post-development peak flows are modelled using a storage-discharge relationship for each facility that is supplied to the model. A stage-storage-discharge relationship is developed externally using Excel and iterations are completed until sufficient storage is provided. The 5 to 100-yr return period events were simulated using 3-hr, 12-hr, and 24-hr storm durations. The results for each of these events are shown below. Values highlighted in red exceed pre-development target flows. However, these flow exceedances are minor and do not present a drainage problem. **Aside from these minor exceedances, the quantity control objective is satisfied with the proposed SWM solution.**

Outlet 1 (West End Near Millennium Trail):

Table 4: Post-Development and Controlled Post-Development Peak Flows for 3-Hr Duration Storm Events at Outlet 1

Return Period	Q_{pre}	Q_{post}	Storage Requirement (1)	Storage Provided (2)
	L/s		m^3	
5	27	17	340	812
10	39	23	411	812
25	58	37	485	812
50	74	49	534	812
100	91	62	584	812

Table 5: Post-Development and Controlled Post-Development Peak Flows for 12-Hr Duration Storm Events at Outlet 1

Return Period	Q_{pre}	Q_{post}	Storage Requirement	Storage Provided (2)
	L/s		m^3	
5	46	21	385	812
10	63	31	463	812
25	86	51	540	812
50	104	66	599	812
100	122	84	660	812

Table 6: Post-Development and Controlled Post-Development Peak Flows for 24-Hr Duration Storm Events at Outlet 1

Return Period	Q_{pre}	Q_{post}	Storage Requirement	Storage Provided (2)
	L/s		m^3	
5	52	23	409	812
10	69	37	485	812
25	91	59	572	812
50	109	77	637	812
100	126	96	701	812

Outlet 2 (East End Near County Road 49):

Table 7: Post-Development and Controlled Post-Development Peak Flows for 3-Hr Duration Storm Events at Outlet 2

Return Period	Q_{pre}	Q_{post}	Storage Requirement (1)	Storage Provided (2)
	L/s		m^3	
5	20	19	110	213
10	28	33	126	213
25	39	45	144	213
50	48	54	160	213
100	58	61	178	213

There are small exceedances for the short duration 3-hr storm events. Jewell investigated the drainage path downstream of Outlet 2. There is a short storm sewer network that connects to a drainage ditch that outlets into the Bay of Quinte. Storage in the temporary ponding facility upstream of Outlet 2 has been maximized given land availability constraints and outlet control structures have been sized to minimize the potential impact to downstream property owners. **The drainage path from Outlet 2 to the receiving water body is relatively short and the slight exceedances shown above do not present a drainage problem.**

Table 8: Post-Development and Controlled Post-Development Peak Flows for 12-Hr Duration Storm Events at Outlet 2

Return Period	Q_{pre}	Q_{post}	Storage Requirement (1)	Storage Provided (2)
	L/s		m^3	
5	30	27	119	213
10	42	39	134	213
25	57	54	160	213
50	69	61	178	213
100	81	68	199	213

Table 9: Post-Development and Controlled Post-Development Peak Flows for 24-Hr Duration Storm Events at Outlet 2

Return Period	Q_{pre}	Q_{post}	Storage Requirement (1)	Storage Provided (2)
	L/s		m^3	
5	33	33	125	213
10	43	44	142	213
25	56	57	166	213
50	65	64	186	213
100	75	71	209	213

The 1 L/s increases in the table above are negligible and do not present a drainage problem.

Outflows were estimated using equations for orifice flow and weir flow. The tables below summarize the size, invert, and primary function for each outlet structure.

Table 10: Summary of Quantity Control Structures at Outlet 1 (West End of Development)

Type and Size	Invert (m)	Primary Function
100mm diameter pipe	92.80	Control minor flows
0.5m length, 260mm deep weir	93.35	Control major flows
1.1m length, 290mm deep weir	93.61	Convey emergency overflows

Table 11: Summary of Quantity Control Structures at Outlet 2 (East End of Development)

Type and Size	Invert (m)	Primary Function
100mm diameter pipe	90.00	Control minor flows
200mm diameter pipe	90.75	Control major flows
2m length, 300mm deep weir	91.20	Convey emergency overflows

On-site runoff is conveyed through grassed swales and a relatively short storm sewer network that is proposed within Catchment 201. The dimensions, purpose, and drainage direction of each swale are summarized in Table 12.

Table 11: Summary of Dimensions, Purpose, and Drainage Direction of Proposed Grassed Swales

Swale	Dimensions	Purpose
A	V-shape, 3:1 side slopes	Receive drainage from north portion of Catchment 200 and drain towards Outlet 1
B	Varying bottom width, 3:1 side slopes	Receive drainage from Catchment 300 (external), Swale A, and the remainder of Catchment 200 while providing temporary storage for quantity control at Outlet 1
C	V-shape, 3:1 side slopes	Receive drainage from rear-yards of lots at west portion of Catchment 201 as well as Catchment 301 (external) and drains towards Outlet 2
D	V-shape, 3:1 side slopes	Receive drainage from rear yards of lots at north portion of Catchment 201 and drains towards Outlet 2
E	Varying bottom width, 3:1 side slopes	Receive drainage from Swale C, Swale D, and the remainder of Catchment 201 while providing temporary storage for quantity control at Outlet 2

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1 Background

The 2.6 ha site is located along the west side of County Road 49 in Prince Edward County, approximately 2 km north of Picton. The site is bound by the Millennium Trail to the west, Picton Golf and Country Club to the north, Picton Bay to the east, and the completed Phase 1 development to the south (see Figure 1-1). Phase 1 of the development was completed in 2014 and stormwater management was implemented following the approval of the stormwater management report prepared by WaterPlan Associates in August 2014. The purpose of this SWM design brief is to describe the SWM solution for Phase 2 of the development to address the quality and quantity treatment objectives described below.

1.1 Review of Relevant Documents

The following documents were reviewed to assist with the stormwater management solution proposed for the Wellings of Picton - Phase 2 development. Previous reports were reviewed to understand previous work that has been completed at the site. Guidance documents were referenced to develop SWM to meet the treatment objectives identified in Section 2.

- **HYDROLOGY & HYDRAULICS REPORT** prepared by WaterPlan Associates for the Wellings of Picton Retirement Residence Complex (Phase 1) and submitted to the County of Prince Edward in August of 2014.
- **Stormwater Management Report** prepared by Jewell Engineering Inc. for the Wellings of Picton Retirement Residence Complex – Phase 2 in April of 2016. *Note: The site layout for the proposed Phase 2 development has changed since submission of Jewell's 2016 report. This updated report has been completed to address the updated site layout as of February 2019.*
- **Quinte Conservation Stormwater Management Submission Guidelines** prepared by Quinte Conservation dated May of 2012.
- **Bay of Quinte Implementation Area Stormwater Management Design Guidelines** revised in March of 2006**.**
- **2003 Stormwater Management Planning and Design Manual** prepared by the Ministry of the Environment.
- **1997 Drainage Management Manual** prepared by the Ministry of Transportation.

1.2 Overview of Phase 1 Development (Completed)

The Hydrology & Hydraulics Report prepared by WaterPlan Associates in 2014 for Phase 1 of the Wellings of Picton development addressed stormwater management for this phase of development. Phase 1 included the development of a retirement building, parking areas, and vehicle assess routes within a 1.62 ha limit.

The 2014 report confirms that the site is outside of the Hospital Creek watershed and states that Level 1 (Enhanced) quality treatment objectives were required by Quinte Conservation. On-site quantity control was also required by the conservation authority.

Due to land availability constraints, the SWM solution in the 2014 report proposed oil-grit separator (OGS) units for quality control and an underground superpipe storm network system for quantity control.

1.3 Overview of Phase 2 Development (Proposed)

Jewell completed a SWM report in 2016 for an original site plan for Phase 2. In the 2016 report, Jewell proposed parking lot storage to control post-development peak flows to the pre-development condition. An STC 3000 model oil-grit separator (OGS) unit by Stormceptor was proposed to provide 81% particulate removal at a 12-month cleanout frequency. Since the 2016 report, the site layout for Phase 2 has been modified. Therefore, the SWM controls are revised in this report since the parking lot storage that was utilized in the 2016 report is not available with the proposed changes.

The updated site layout for Phase 2 includes a roadway that connects Highway 49 to a turn-around near the Millennium Trail. Townhouses are located along this roadway throughout the entire Phase 2 boundary.

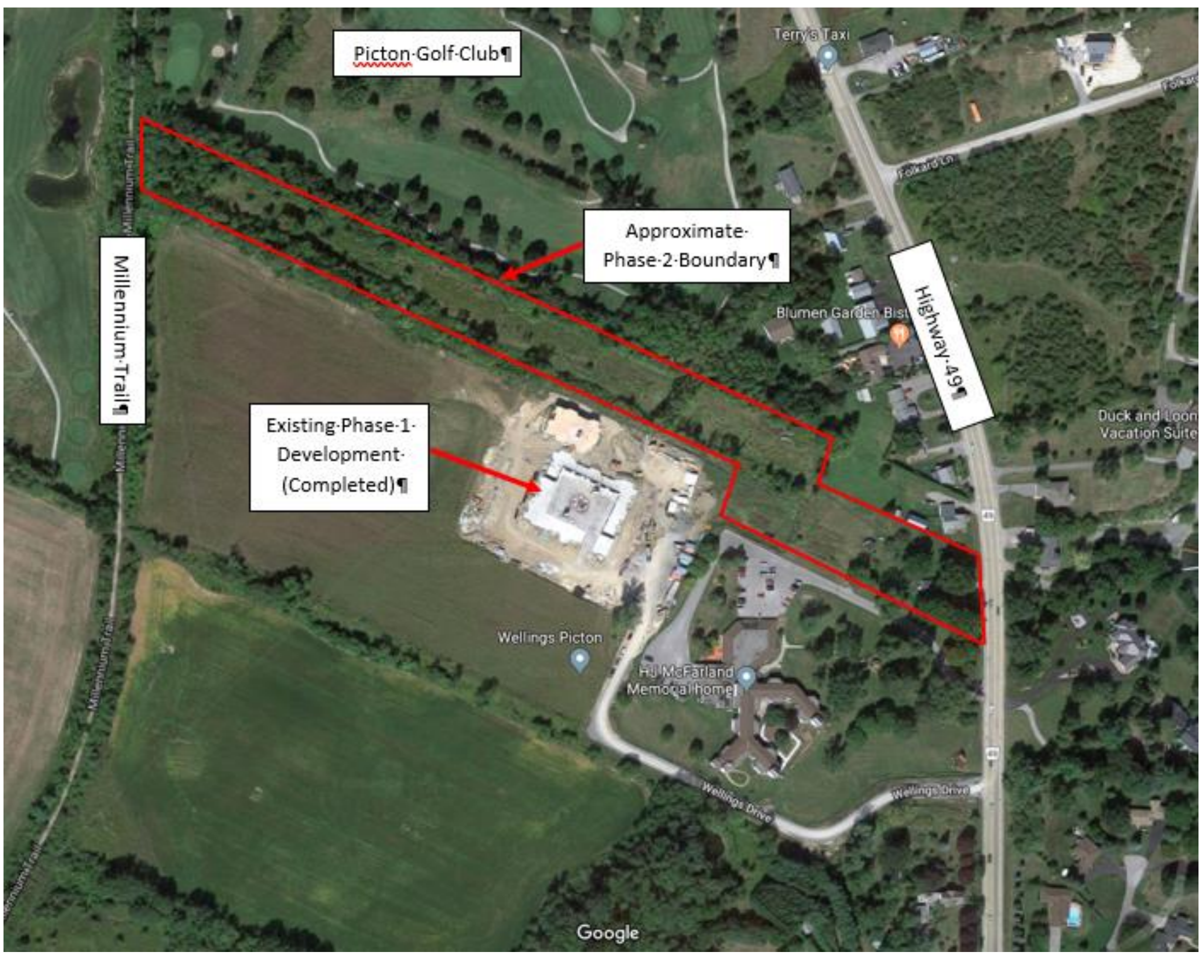

Figure 1-1: Site Location

2 SWM Objectives

By meeting quantity control objectives and providing SWM practices that have water quality benefits, the potential impact on stormwater runoff can be effectively mitigated with the proposed SWM solution. Jewell met with Quinte Conservation for assistance in identifying the treatment objectives for the site. The stormwater objectives are summarized below.

Quality Control Objectives

Quality control should be provided to meet Enhanced treatment objectives. Therefore, a minimum of 80% of total suspended solids should be removed from site runoff.

Quantity Control Objectives

For quantity control, post-development peak flows are to be controlled to pre-development conditions. Jewell completed hydrologic simulations with varying return periods and storm durations to ensure quantity control is effectively managed in the proposed SWM solution.

Sediment and Erosion Control during Construction

- Minimize the potential for erosion of soils and construction materials
- Mitigate the release of sediment offsite

3 Methodology

The stormwater management solution was prepared with understanding of:

- the site hydrology to estimate the peak flows as well as
- hydraulic calculations to determine storage requirements, swale dimensions, and outlet structures.

The methodologies followed for each of the hydrology and hydraulic calculations are provided in this section. The analysis is provided in Section 4.

3.1 Hydrology

A hydrologic model is a mathematical simplification of various parameters that contribute to runoff during rainfall events. Parameters include:

- Precipitation – intensity, duration and frequency as well as distribution
- Catchment area
- Percent imperviousness – runoff volume, time to peak and peak flow increase with percent imperviousness
- Soil conditions – these determine how much and how quickly water will be removed from runoff through infiltration. This may be expressed as a curve number, or by a runoff coefficient or using an infiltration model such as Horton's Infiltration
- Slope – peak flows increase with slope
- Initial abstraction – depth of precipitation input that is subtracted from the model and does not contribute to runoff. This value is different for impervious and pervious areas and is expressed as two values.
- Manning's n – frictional coefficient that affects the time to peak. This value is different for impervious and pervious areas and is expressed as two values.
- Basin lag or time to peak.

Further discussion on these model inputs is described in the following subsections.

The hydrologic analysis was completed using HEC-HMS version 4.2.1. This hydrologic modeling software is developed by the U.S. Army Corps of Engineers and distributed freely.

Existing conditions were modeled first to determine pre-development peak flows. A post-development model was then completed that includes measures to reduce peak flows to pre-development conditions.

3.1.1 Precipitation

Precipitation inputs are supplied as hyetographs to the precipitation gages in HEC-HMS located in the *Time-Series Data Manager*. These inputs were derived using precipitation gauge statistics from Belleville Intensity-Duration-Frequency curves (see Appendix A). Short duration storms have high instantaneous intensities, but low runoff volumes. Longer duration storms have lower instantaneous intensities, but higher runoff accumulations.

The intensity is a resultant of the time of concentration calculated as a summation of the overland flow time and time of flow in pipe or swale. As the time of concentration is increased the peak intensity is decreased. Jewell uses a standard storm sewer design sheet in Excel that varies the intensity with duration by Equation 1. The duration is set equal to the time of concentration that is discussed further in Section 3.1.7.

Equation1: Relationship between Intensity and Time of Concentration

$$i = A \, x \, T^{B}$$

Where:

i = rainfall intensity in mm/hr
A = dimensionless coefficient provided in IDF curves
T = time of concentration in minutes
B = dimensionless exponent provided in IDF curves

3.1.2 Catchment Area

Catchment areas (A) are determined using topographic data obtained from Jewell survey data and LiDAR information. Catchment areas are specified areas of land that drain to a designated location based on land topography. The land use within the catchment areas define runoff coefficient values. Catchment areas are shown in Appendix B and summarized in Tables 3-1 and 3-2.

In pre-development conditions, runoff drains from the site in three directions before ultimately discharging to the Bay of Quinte. Catchment 200 drains westward towards the Millennium Trail and receives runoff from the external catchment labelled Catchment 300. Catchment 201 drains eastward towards the existing roadside ditch along County Road 49 and receives runoff from an external catchment labelled Catchment 301. Catchment 202 drains northeast towards an existing 800mm circular pipe that crosses County Road 49.

The post-development drainage pattern is discussed in Section 4. With the exception of Catchment 202, the overall drainage pattern is generally maintained with the proposed Phase 2 development. Catchment 202 in the pre-development drawing is directed westward and becomes part of Catchment 200 in the post-development catchment drawing.

Table 3-1: Pre-Development Catchment Areas

	Catchment	Area (ha)	Drainage Direction
Phase 2 Drainage	200	1.12	Drains west towards Millennium Trail
	201	0.64	Drains east towards ditch at County Rd. 49
	202	1.04	Drains north towards culvert at County Rd. 49 (different outlet than Catchment 201)
	Phase 2 Total=	**2.80**	
External Drainage	300	0.54	External catchment that drains northwest towards Millennium Trail
	301	0.24	External catchment that drains east towards Catchment 201 and County Rd. 49
Total (including external drainage)=		**3.58**	

Note: For simplicity, a small portion (0.17 ha) of the Phase 1 development area is included in Catchment 201. The total site area in the Phase 2 development boundary is 2.63 ha.

Table 3-2: Post-Development Catchment Areas

	Catchment	Area (ha)	Drainage Direction
Phase 2 Drainage	200	1.96	Drains west towards Millennium Trail
	201	0.84	Drain eastward towards County Rd. 49
	Phase 2 Total=	**2.80**	
External Drainage	300	0.54	External catchment that drains northwest towards Millennium Trail
	301	0.24	External catchment that drains east towards Catchment 201 and County Rd. 49
Total (including external drainage)=		**3.58**	

Note: For simplicity, a small portion (0.17 ha) of the Phase 1 development area is included in Catchment 201. The total site area in the Phase 2 development boundary is 2.63 ha.

3.1.3 Imperviousness

Imperviousness is a measure of how much surface hardening has occurred within the catchment. It is expressed as a percentage of the catchment area that is hardened by asphalt, roof, concrete, etc. that restricts water infiltration into the ground and results in higher runoff. The imperviousness for individual basins used in the hydrologic modelling is summarized in Tables 3-5 and 3-6.

3.1.4 Curve Number

Curve numbers are used in the Soil Conservation Service (now known as the *National Resources Conservation Service*) methodology for estimating the proportion of precipitation that will runoff the lands and the 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 are representative of the pervious portion of the watershed.

Jewell calculated a CN of 81 using *Design Charts 1.08 and 1.09* of the *MTO Drainage Management Manual* (see Table 4-3). The *Geotechnical Investigation* report prepared by Inspec-Sol in 2014 provided soil information for boreholes within the vicinity of the Phase 1 boundaries. Since Phase 1 is adjacent to Phase 2, Jewell used this information to investigate soil characteristics.

Borehole information from the 2014 Inspec-Sol report show that the soils are generally comprised of sandy clay, silty clay, or clayey silt. Several boreholes were noted to have practical auger refusal between depths of 1-3m. *Design Chart 1.08* of the MTO DMM suggests that these soil types would be categorized as Hydrologic Soils Group (HSG) C or D. Additionally, Jewell's 2016 SWM report noted that the land cover at the site is undeveloped although it would have been in agricultural use in the not too distant past. With this information, Jewell used *Design Chart 1.09* of MTO's DMM by assuming HSG CD and a CN between that provided for crops and other improved lands (84) and the CN for unimproved land (79). Therefore, a CN of 81 is used in the hydrologic model.

3.1.5 Initial Abstraction

Initial abstraction is the depth of precipitation input that is subtracted from the model and does not contribute to runoff. It may include interception by vegetation and infiltration or depression storage. Initial abstraction is typically expressed with separate values for each of the impervious and pervious areas. The HEC-HMS model represents the initial abstraction for the pervious lands as 0.2 x the storativity (S). This is related to the curve number. Equation 2 shows how storativity is calculated from the curve number and Equation 3 the initial abstraction from storativity.

Equation 2: Storativity Derived from Curve Number

$$S = \frac{1000}{CN} - 10$$

Where:

S = Storativity (dimensionless)
CN = SCS Curve Number (dimensionless)

Equation 3: Initial Abstraction

$$I_a = 0.2S$$

Where:

Ia = Initial Abstraction
S = Storativity (dimensionless)

3.1.6 Runoff Coefficient

Runoff coefficients (C) are determined by a ratio of the depth of runoff from a drainage basin to the depth of rainfall, and indicating the runoff potential of particular topography/soil type/ land use combination (MTC).

The time of concentration varies with runoff coefficient. The runoff coefficient is calculated using a weighted average and the values provided in MTO *Design Chart 1.07* (Ontario Ministry of Transportation, 1997). Equation 4 is used to calculate the weighted runoff coefficients shown in Tables 3-5 and 3-6.

Equation 4: Weighted Runoff Coefficient

$$C_w = \frac{C_1 \, x \, A_1 + \; C_2 \, x \, A_2 + \; \ldots \; C_n \, x \, A_n}{A_1 + \; A_2 + \; \ldots \; A_n}$$

Where:
C_i = Runoff Coefficient for A_i
A_i = Land Area (ha) of Cover Type

3.1.7 Basin Lag

Basin lag time has been defined as the time from the centroid of rainfall excess to the centroid of the corresponding runoff hydrograph (USGS, 2012). This is an important measure in estimating the time of peak runoff.

The Natural Resources Conservation Service completed studies of basin lag time and it is generally accepted that the lag time can be estimated as 60 percent of the time of concentration (Hydrologic Modeling System HEC-HMS User's Manual, 2013).

The times of concentration were obtained using the summation of overland flow time and time of flow in pipe or swale. For the overland flow calculations, Jewell selected to use the Airport Method and Bransby-Williams Method depending on the corresponding runoff coefficient (Ontario Ministry of the Environment, 2003).

A summary of time of concentration and basin lag times used in HEC-HMS model is provided in Tables 3-3 and 3-4.

Table 3-3: Pre-Development Time of Concentration and Lag Time Summary

Catchment	Area (ha)	Watershed Length (m)	Elevation at 85% (m)	Elevation at 10% (m)	Slope (%)	RC	T_c (min)	Lag Time (min)
200	1.12	175	95.00	93.00	1.5	0.25	31.8	19.1
201	0.64	125	94.50	91.00	3.7	0.25	20.0	12.0
202	1.04	185	95.50	94.00	1.1	0.25	36.7	22.0
300	0.54	118	96.25	94.60	1.9	0.30	23.1	13.9
301	0.24	105	94.50	93.25	1.6	0.33	22.1	13.3

Note: Airport method used for T_c calculations pre-development since runoff coefficients are less than 0.4.

Table 3-4: Post-Development Time of Concentration and Lag Time Summary

Catchment	Area (ha)	Watershed Length (m)	Elevation at 85% (m)	Elevation at 10% (m)	Slope (%)	RC	Airport T_c (min)	Bransby-Williams T_c (min)	Pipe or Channel Flow T_c (min)	Total T_c (min)	Lag Time (min)
200	1.96	20	-	-	2.0	0.50	-	0.9	15.3	16.3	9.8
201	0.84	20	-	-	2.0	0.58	-	1.0	5.0	6.0	3.6
300	0.54	118	96.25	94.60	1.9	0.30	23.1	-	-	23.1	13.9
301	0.24	105	94.50	93.25	1.6	0.33	22.1	-	-	22.1	13.3

Note: Bransby-Williams method used for T_c calculations for catchments with runoff coefficients greater than 0.4.

3.1.8 Reservoir Storage

A reservoir simulation relies on a storage:discharge curve that represents the storage and outflow relationship for a proposed SWM facility. A curve is supplied by the user to the model and tests the storage-discharge relationship by routing the inflow hydrograph through the facility with a trial and error approach. The curve is developed externally in Excel and is adjusted until sufficient storage and outflow are obtained. The storage for each scenario was developed by this trial and error approach and entered into the 'Paired Data' component of the HEC-HMS model.

3.1.9 Hydrology Input Summary

The hydrology inputs discussed in this section are summarized in Table 3-5 for pre-development conditions and Table 3-6 for post-development conditions.

Table 3-5: Pre-Development Hydrology Input Summary

Catchment	Area (ha)	% Imp.	RC	T_c (min)	Lag Time (min)	CN
200	1.12	0	0.25	31.8	19.1	81
201	0.64	0	0.25	20.0	12.0	81
202	1.04	0	0.25	36.7	22.0	81
300	0.54	0	0.30	23.1	13.9	81
301	0.24	5	0.33	22.1	13.3	81

Table 3-6: Post-Development Hydrology Input Summary

Catchment	Area (ha)	% Imp.	RC	T_c (min)	Lag Time (min)	CN
200	1.96	43	0.56	16.3	9.8	81
201	0.84	50	0.58	6.0	3.6	81
300	0.54	0	0.30	23.1	13.9	81
301	0.24	5	0.33	22.1	13.3	81

3.2 Hydraulics

Each equation in this section is provided in Chapter 8 of the MTO Drainage Management Manual (Ontario Ministry of Transportation, 1997). The orifice and weir flow equations are used to determine outflows at various elevations to complete the stage-storage-discharge relationship used in the sizing of the temporary storage areas.

Orifice Flow

Jewell used the orifice flow equation in the sizing of the low-flow outlet structures (see Equation 5). The outflow from the low-flow control pipe is connected to a downstream oil-grit separator unit that provides quality control. In the case of the east storage facility as described in Section 4, the orifice equation is also used for sizing of the outlet to control. This is due to the small size of the site that yields small pre-development peak flows and therefore requires a small post-development peak flow. The orifice equation is suitable for the major outflow structure at the east facility since it can provide smaller outflows whereas weirs are more suitable for conveyance of high peak flows.

Equation 5: Orifice Equation

$$Q = CA\sqrt{2gH}$$

Where:

Q = discharge (m^3/s)
C = discharge coefficient (dimensionless)
A = area of opening (m^2)
H = hydraulic head (m)
g = acceleration due to gravity (m/s^2)

Sharp-Crested Weir Flow

Jewell selected to use a sharp-crested weir to control outflows at the west end of the site. A calculation of rectangular sharp-crested weir flow is completed using Equation 6.

Equation 6: Sharp-Crested Weir Formula

$$Q = 1.84LH^{3/2}$$

Where:

Q = Discharge (m^3/s)
L = Length of Weir (m)
H = Depth of flow (m)

Note: Jewell reduced the length by 0.2H to account for end contractions.

Broad-Crested Weir Flow

It is common to use a broad-crested weir in a SWM facility to provide an emergency spillway for storm events exceeding the 100-yr return period (see Equation 7).

Equation 7: Broad-Crested Weir Formula

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

Where:

Q = Flow over the road in cms
L = Length of Weir (m)
H = Depth of flow (m)

For conveyance of runoff through grassed swales, open channel flow calculations were completed to determine flow capacity. Open channel flow calculations for each of the swales proposed in Section 4.3 are provided in Appendix F.

4 Stormwater Management Solution

The following sections describe the proposed SWM solution for Phase 2 of the Wellings of Picton development.

4.1 Overview

The proposed SWM solution utilizes grassed swales, two storage facilities, and two OGS units (see Figure 4-1). Grassed swales are used to receive runoff and provide conveyance to the storage facilities. The storage facilities are controlled by outlet structures with simulated outflows using the equations in Section 3.2. A relatively small storm sewer pipe network is used to convey a portion of Catchment 201 to the east SWM facility (see Appendix C).

The purpose of the storage facilities is to provide **quantity control** by reducing post-development peak flows to the pre-development condition. Two OGS units, one for each storage facility, are proposed to provide **quality control**. The size of OGS unit required to achieve *Enhanced* protection is determined based on TSS removal capabilities. Sections 4.1-4.4 provide detail regarding the SWM solution for Phase 2 of the Wellings of Picton development.

The proposed SWM solution was selected after consideration of other SWM alternatives. Common SWM technologies include wet ponds, dry ponds, and infiltration facilities. Since this is a 2.6 ha site, a traditional wet pond or dry pond is not recommended based on MOE guidance and land availability constraints.

Infiltration is often a preferred SWM technique for small sites. However, borehole results from the *Geotechnical Investigation* report prepared by Inspec-Sol in 2014 suggests bedrock exists at shallow depths. Since infiltration techniques require a minimum 1m buffer from the bottom of facility to the top of bedrock, infiltration should not be selected as the primary treatment method for this site. Therefore, Jewell has proposed temporary ponding areas for quantity control and oil-grit separator (OGS) units for quality control due to the limited land availability and poor infiltration capabilities.

4.2 Site Drainage (Existing vs. Proposed)

The site drainage in existing and proposed conditions is described below.

Site drainage in existing conditions is summarized in the pre-development catchment area drawing (see Appendix B). As described in Section 3.1.2, runoff drains from the site in three directions before ultimately discharging to the Bay of Quinte. Catchment 200 drains westward towards the Millennium Trail and receives runoff from the external catchment labelled Catchment 300. Catchment 201 drains eastward towards the existing roadside ditch along County Road 49 and receives runoff from an external catchment labelled Catchment 301. Catchment 202 drains northeast towards an existing 800mm circular pipe that crosses County Road 49.

After the development, runoff will drain to the two outlet locations as shown in Figure 4-1. Outlet 1 is located at the west end of the site and drains to the ditch alongside the existing Millennium Trail. Outlet 2 is located at the east end of the site and drains to the roadside ditch at County Road 49.

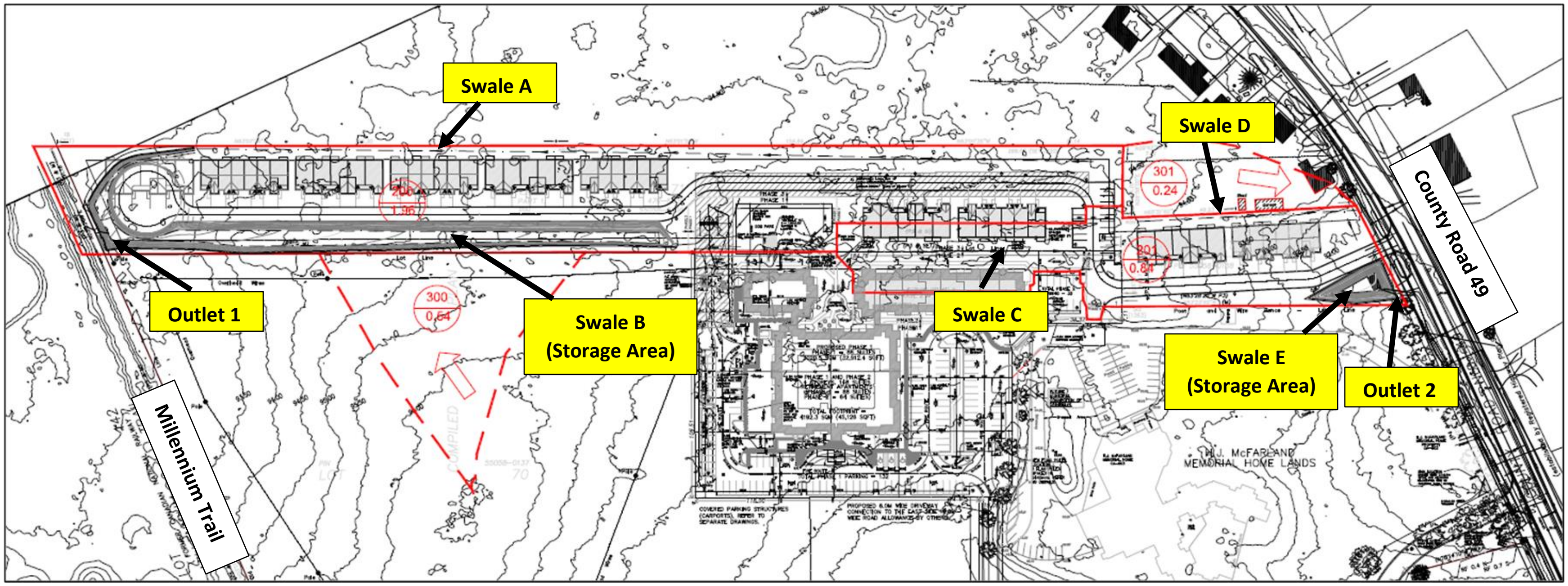

Figure 4-1: Summary of Grassed Swale and Outlet Locations with Proposed Phase 2 Development

4.3 Quality Control

Jewell selected two oil-grit separator (OGS) units to provide quality control, one located at the west end of the site at Outlet 1 and the other located near the east end of the site at Outlet 2.

Pre-treatment is provided with the storage areas upstream of the OGS units. These temporary storage areas function as dry ponds and are expected to provide significant quality control benefits. Jewell referenced the 2003 MOE *SWM Planning and Design Manual* to design the dry pond facilities. The west SWM facility has sufficient storage volume to achieve 60% removal of total suspended solids (TSS) through pre-treatment (see Table 4-1). The east SWM facility has approximately 87% of the storage volume required to achieve 60% removal of TSS. Therefore, Jewell conservatively assumed that pre-treatment from this facility will only provide 30% TSS removal (see Table 4-1). Jewell estimated pre-treatment removal rates using Table 3.2 of the 2003 MOE *SWM Planning and Design Manual.*

Table 3.2 of the 2003 MOE guidelines was used to estimate 223 m^3 of storage required for the west SWM facility. The volume in this facility controlled only by the 100mm pipe is 371 m^3. This 100mm diameter outlet structure is the recommended minimum size in MOE guidelines and therefore it does not require a 24-hr drawdown time as any smaller size could present clogging problems (Ontario Ministry of the Environment, 2003). However, Jewell reviewed the hydrologic simulations to estimate a drawdown time of approximately 12 hours. This is within the acceptable range identified in the guidelines for minimum sizes of outflow control pipes for dry ponds.

For the east SWM facility, 113 m^3 of storage is required based on Table 3.2 of the MOE guidelines. While the volume is maximized within the land area available for the east SWM facility, the 100mm diameter outflow structure solely controls the first 99 m^3 of storage in this facility. This is approximately 87 percent of the storage requirement to achieve 60% TSS removal with a dry pond facility. Therefore, Jewell conservatively applied only 30% TSS removal for pre-treatment from the east SWM facility (see Table 4-1).

The Downstream Defender is a type of OGS unit capable of achieving further removal of total suspended solids (TSS). The units are sized using the online sizing tool supplied by Hydro-International. With this sizing tool, a particle size distribution is selected as well as a treatment area and runoff coefficient. Jewell determined appropriate Downstream Defender sizes with input from the manufacturer, Aqua Q. For the west portion of the site (Catchment 200), a DD4 model is recommended and for the east portion of the site (Catchment 201), a DD4 model is also recommended.

The DD4 model provides a removal efficiency of 68.6% at the west SWM facility and 78.0% at the east SWM facility based on the Hydro International sizing tool (see Appendix D). Since a significant portion of sediment will be removed with the pre-treatment provided from the dry pond facilities, the OGS units will provide a smaller percentage of overall TSS removal as shown in Table 4-1. The estimated TSS removal of 27% and 55% from the OGS units in Table 4-1 were estimated assuming that the OGS units remove 68.6% and 78.0% of **remaining** sediment contributing to their respective SWM facility.

A weighted calculation in the table below shows that the theoretical TSS removal rate for the overall site is 87.0 percent. This is greater than the treatment objective of *Enhanced* protection through 80% TSS removal. **Therefore, quality control objectives are met.**

Table 4-1: Comparison of TSS Removal from OGS Units to Quality Control Target

Outlet	Outlet Location	Treatment Area (ha)	Thoeretical TSS Removal from Storage Facility Pre-Treatment (%)	Theoretical TSS Removal from OGS Unit After Pre-Treatment (%)	OGS Model	Combined Theoretical TSS Removal (%)	Weighted Theoretical TSS Removal (%) (2)	TSS Removal Objective (%) (3)	Check: (2) > (3)
1	West end near Millennium Trail	1.96	60	27	DD4	**87**	**87**	**80**	✓
2	East end near County Road 49	0.84	30	55	DD4	**85**			

Note: A fine particle size distribution was selected for OGS unit sizing.

4.4 Quantity Control

The quantity objective is to control post-development peak flows to the pre-development condition. Pre-development peak flows were calculated using HEC-HMS and the pre-development input parameters in Table 3-5. Post-development peak flows are calculated using the input parameters in Table 3-6 and the storage-discharge relationship for each storage facility (see Appendix E). The stage-storage-discharge relationship is developed externally using Excel and iterations are completed until sufficient storage is provided.

Jewell varied storm durations and return periods in HEC-HMS to determine the need for quantity controls. Jewell believes an investigation of the 3-hr, 12-hr, and 24-hr storm events are appropriate for this site.

4.4.1 Uncontrolled Peak Flows

Tables 4-2 and 4-3 show pre-development and uncontrolled post-development peak flows. <u>Evidently, quantity controls are triggered.</u>

Table 4-2: Pre-Development and Uncontrolled Post-Development Peak Flows at Outlet 1 with Varying Storm Durations

Return Period	3-Hr Duration		12-Hr Duration		24-Hr	
	Q_{pre}	$Q_{post\ (uncontrolled)}$	Q_{pre}	$Q_{post\ (uncontrolled)}$	Q_{pre}	$Q_{post\ (uncontrolled)}$
	L/s		L/s		L/s	
5	27	138	46	155	52	128
10	39	169	63	192	69	157
25	58	211	86	241	91	193
50	74	244	104	278	109	221
100	91	278	122	317	126	249

Table 4-3: Pre-Development and Uncontrolled Post-Development Peak Flows at Outlet 1 with Varying Storm Durations

Return Period	3-Hr Duration		12-Hr Duration		24-Hr Duration	
	Q_{pre}	$Q_{post\ (uncontrolled)}$	Q_{pre}	$Q_{post\ (uncontrolled)}$	Q_{pre}	$Q_{post\ (uncontrolled)}$
	L/s		L/s		L/s	
5	20	76	30	76	33	59
10	28	93	42	94	43	72
25	39	114	57	116	56	88
50	48	131	69	133	65	100
100	58	148	81	150	75	112

4.4.2 Controlled Peak Flows

There is a total of five swales that are proposed to convey on-site runoff. These swales are shown in Figure 4-1 and their purpose is summarized in Table 4-4. Swale sizing calculations are provided in Appendix F.

Table 4-4: Summary of Swale Dimensions and Purpose

Swale	Dimensions	Purpose
A	V-shape, 3:1 side slopes	Receive drainage from north portion of Catchment 200 and drain towards Outlet 1
B	Varying bottom width, 3:1 side slopes	Receive drainage from Catchment 300 (external), Swale A, and the remainder of Catchment 200 while providing temporary storage for quantity control at Outlet 1
C	V-shape, 3:1 side slopes	Receive drainage from rear-yards of lots at west portion of Catchment 201 as well as Catchment 301 (external) and drains towards Outlet 2
D	V-shape, 3:1 side slopes	Receive drainage from rear yards of lots at north portion of Catchment 201 and drains towards Outlet 2
E	Varying bottom width, 3:1 side slopes	Receive drainage from Swale C, Swale D, and the remainder of Catchment 201 while providing temporary storage for quantity control at Outlet 2

Swale B functions as a SWM facility that provides quantity control at Outlet 1 by allowing temporary ponding of water during storm events. The depth of ponding is based on the sizing of the outlet structures and the amount of storage provided.

Swale B receives runoff from Swale A as well as sheet flow runoff from Catchment 300 (external) and a portion of Catchment 200. The outlet structures at the downstream end of the swale convey runoff to the existing ditch that runs alongside the Millennium Trail.

Similarly, Swale E functions as a SWM facility that provides quantity control at Outlet 2. Swale E receives runoff from Swale C and a portion of Catchment 201 through a storm sewer system. Runoff from Catchment 301 (external) and the remainder of Catchment 201 is received by Swale D that connects to Swale E through a 300mm pipe that crosses the site's driveway entrance.

Tables 4-4 to 4-9 show a comparison of pre-development to controlled post-development peak flows. Evidently, post-development peak flows are effectively managed for the return period events shown in Tables 4-5 to 4-8. There are minor increases in peak outflows for some return period events as highlighted in red. However, these flow exceedances are minor and do not present a drainage problem. **Aside from these minor exceedances, the quantity control objective is satisfied with the proposed SWM solution.**

Summary tables for temporary storage area near east end of site (Outlet 1):

Table 4-5: Post-Development and Controlled Post-Development Peak Flows for 3-Hr Duration Storm Events at Outlet 1

Return Period	Q_{pre}	Q_{post}	Storage Requirement (1)	Storage Provided (2)
	L/s		m^3	
5	27	17	340	812
10	39	23	411	812
25	58	37	485	812
50	74	49	534	812
100	91	62	584	812

Table 4-6: Post-Development and Controlled Post-Development Peak Flows for 12-Hr Duration Storm Events at Outlet 1

Return Period	Q_{pre}	Q_{post}	Storage Requirement	Storage Provided (2)
	L/s		m^3	
5	46	21	385	812
10	63	31	463	812
25	86	51	540	812
50	104	66	599	812
100	122	84	660	812

Table 4-7: Post-Development and Controlled Post-Development Peak Flows for 24-Hr Duration Storm Events at Outlet 1

Return Period	Q_{pre}	Q_{post}	Storage Requirement	Storage Provided (2)
	L/s		m^3	
5	52	23	409	812
10	69	37	485	812
25	91	59	572	812
50	109	77	637	812
100	126	96	701	812

Summary tables for temporary storage area near east end of site (Outlet 2):

Table 4-8: Post-Development and Controlled Post-Development Peak Flows for 3-Hr Duration Storm Events at Outlet 2

Return Period	Q_{pre}	Q_{post}	Storage Requirement (1)	Storage Provided (2)
	L/s		m^3	
5	20	19	110	213
10	28	33	126	213
25	39	45	144	213
50	48	54	160	213
100	58	61	178	213

There are small exceedances for the short duration 3-hr storm events. Jewell investigated the drainage path downstream of Outlet 2. There is a short storm sewer network that connects to a drainage ditch that outlets into the Bay of Quinte. Storage in the temporary ponding facility upstream of Outlet 2 has been maximized given land availability constraints and outlet control structures have been sized to minimize the potential impact to downstream property owners. **The drainage path from Outlet 2 to the receiving water body is relatively short and the slight exceedances shown above do not present a drainage problem.**

Table 4-9: Pre-Development and Controlled Post-Development Peak Flows for 12-Hr Duration Storm Events at Outlet 2

Return Period	Q_{pre}	Q_{post}	Storage Requirement (1)	Storage Provided (2)
	L/s		m^3	
5	30	27	119	213
10	42	39	134	213
25	57	54	160	213
50	69	61	178	213
100	81	68	199	213

Table 4-10: Pre-Development and Controlled Post-Development Peak Flows for 24-Hr Duration Storm Events at Outlet 2

Return Period	Q_{pre}	Q_{post}	Storage Requirement (1)	Storage Provided (2)
	L/s		m^3	
5	33	33	125	213
10	43	44	142	213
25	56	57	166	213
50	65	64	186	213
100	75	71	209	213

The 1 L/s increases in the table above are negligible and do not present a drainage problem.

4.4.3 Outlet Structures

Jewell designed the SWM facility using calculations of orifice flow and weir flow as described in Section 3.2. These calculations are used in the selection and sizing of quantity control structures. The type, size, invert, and primary function of each quantity control structure is summarized below for each storage facility.

Table 4-11: Summary of Quantity Control Structures at Outlet 1 (West End of Development)

Type and Size	Invert (m)	Primary Function
100mm diameter pipe	92.80	Control minor flows
0.5m length, 260mm deep weir	93.35	Control major flows
1.1m length, 290mm deep weir	93.61	Convey emergency overflows

Table 4-12: Summary of Quantity Control Structures at Outlet 2 (East End of Development)

Type and Size	Invert (m)	Primary Function
100mm diameter pipe	90.00	Control minor flows
200mm diameter pipe	90.75	Control major flows
2m length, 300mm deep weir	91.20	Convey emergency overflows

5 Sediment and Erosion Control

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

6 Maintenance Plan

The following sections provide basic instructions on good maintenance for the east and west SWM facilities.

6.1 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 and compare with expected levels
- Evidence of erosion

Pond operators should remove any trash that may be impeding the pond outlet structures. During and after a large rainfall event the operator should also perform a visual check to see that pond elevations are within expected levels.

Routine maintenance is required for the DD4 oil-grit separator units to ensure their functionality. The *Downstream Defender Operation and Maintenance Manual* prepared by Hydro-International provides guidance for maintenance schedules for these units. It describes that the frequency of cleanout should be determined in the field after installation, with inspections completed every six months during the first year of operation. Inspection procedures are also provided in the Hydro International document. Sediment removal is not required unless the sediment depths exceed 75% of the maximum cleanout depth of 18 inches (Hydro International). Floatables and sump cleanouts are typically conducted once a year during any season with a commercially or municipally owned sump-vac (Hydro International).

6.2 Infrequent Maintenance

The dry pond facilities will collect sediment in proportion to the construction activity or winter road maintenance of the upstream catchment area. It is recommended the site owners remove accumulated sediment from the dry ponds as sediment begins to accumulate near the outlet structure, or if an average depth of sediment exceeds 0.2m. Sediment removal may be completed with an excavator or back hoe.

The dry pond facilities should be restored to their original configuration ensuring the basic elevations are maintained as given in the design drawings and summarized below.

West SWM Facility Pond Features:

- Bottom of Pond: 92.80m
- Top of Berm: 93.90m
- Top of 100-Yr: 93.55m
- Invert of Concrete Weir: 93.35m
- Invert of Emergency Spillway: 93.61m
- Side slopes: 3:1

East SWM Facility Pond Features:

- Bottom of Pond: 90.0m
- Top of Berm: 91.50m
- Top of 100-Yr: 91.20m
- Invert of Concrete Weir: 90.75m
- Invert of Emergency Spillway: 91.20m
- Side slopes: 3:1

6.3 Troubleshooting

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

6.3.1 Symptom – Pond is not emptying

The 100mm pipes at the pond outlet provides the outflow control for frequent storm events and 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 also emptying out between events. Full storage for the 100-yr event should be should not be spilling over the emergency weir. The 100mm pipe size has been selected to be as large as possible to allow smaller debris to pass through.

6.3.2 Symptom – Pond does not fill

The 100mm pipes at the outlet of each facility should impose ponding during large runoff events. If the pond does not hold water during large events, check to see that the pipe has not been tampered with.

6.3.3 Symptom – Pond routinely overfills

If the stored water discharges through the emergency spillway the cause is blockage of the 100mm pipe. It requires cleanout.

7 Conclusions

Jewell has prepared this stormwater management (SWM) report for the proposed 2.6 ha Phase 2 development at the Wellings of Picton site. The proposed Phase 2 site layout is a residential development that includes townhouses and an access road. The objective this report is to propose a SWM solution that meets the following SWM objectives.

SWM Objectives

- The **quality control** objective is to achieve Enhanced protection. Therefore, a minimum of 80% removal of TSS is recommended.
- The **quantity control** objective is to reduce post-development peak flows to the pre-development peak flows.
- **Sediment and erosion controls** should be provided during construction to minimize the potential for erosion of soils and construction materials. The release of sediment offsite should also be mitigated by sediment and erosion controls.

Site Drainage

Section 4 discussed the site drainage under existing and proposed conditions as well as quality and quantity control measures. For site drainage, pre-development and post-development catchment drawings are provided in Appendix B. Summary tables that correspond to these catchment drawings are provided below. Figure 4-1 shows the grassed swales, storage areas, and outlet locations used to control drainage after the proposed Phase 2 development. Outlet 1 is located at the west end of the site near Millennium Trail and Outlet 2 is located at the east end near County Road 49.

Table 7-1: Pre-Development Catchment Area Summary

	Catchment	Area (ha)	Drainage Direction
Phase 2 Drainage	200	1.12	Drains west towards Millennium Trail
	201	0.64	Drains east towards ditch at County Rd. 49
	202	1.04	Drains north towards culvert at County Rd. 49 (different outlet than Catchment 201)
	Phase 2 Total=	**2.80**	
External Drainage	300	0.54	External catchment that drains northwest towards Millennium Trail
	301	0.24	External catchment that drains east towards Catchment 201 and County Rd. 49
Total (including external drainage)=		**3.58**	

Table 7-2: Post-Development Catchment Area Summary

	Catchment	Area (ha)	Drainage Direction
Phase 2 Drainage	200	1.96	Drains west towards Millennium Trail
	201	0.84	Drain eastward towards County Rd. 49
	Phase 2 Total=	**2.80**	
External Drainage	300	0.54	External catchment that drains northwest towards Millennium Trail
	301	0.24	External catchment that drains east towards Catchment 201 and County Rd. 49
Total (including external drainage)=		**3.58**	

Quality Control

Jewell selected two oil-grit separator (OGS) units to provide quality control, one located at the west end of the site that outlets to the Millennium Trail ditch and the other at the east end of the site that outlets to the ditch alongside County Road 49. Each OGS unit is the downstream connection to the lowest outlet structure. Therefore, quality treatment flows are controlled prior to entering the separator units.

The Downstream Defender models are a type of OGS unit that can achieve 80% TSS removal at the site. A weighted calculation in the table below shows that the theoretical TSS removal for the overall site is 82.0 percent. **Therefore, quality control objectives are met.**

Table 7-3: Quality Control Summary

Outlet	Outlet Location	Treatment Area (ha)	Thoeretical TSS Removal from Storage Facility Pre-Treatment (%)	Theoretical TSS Removal from OGS Unit After Pre-Treatment (%)	OGS Model	Combined Theoretical TSS Removal (%)	Weighted Theoretical TSS Removal (%) (2)	TSS Removal Objective (%) (3)	Check: (2) > (3)
1	West end near Millennium Trail	1.96	60	27	DD4	**87**	**87**	**80**	✓
2	East end near County Road 49	0.84	30	55	DD4	**85**			

Quantity Control

Jewell completed HEC-HMS simulations to estimate peak flows before and after the development. As mentioned above, the objective is to reduce post-development peak flows to pre-development peak flows. Quantity control is provided with two temporary ponding areas. One facility is located at the west end of the development and another is located near the east end of the development. This quantity control method was selected due to limited land availability and poor infiltration capabilities of underlying soils.

Post-development peak flows are modelled using a storage-discharge relationship for each facility that is supplied to the model (see Appendix E). A stage-storage-discharge relationship is developed externally using Excel and iterations are completed until sufficient storage is provided. The 5-100-yr return period events were simulated using 3-hr, 12-hr, and 24-hr storm durations. The results for each of these events are shown Tables 4-5 to 4-10. Values highlighted in red exceed pre-development target flows. However, these flow exceedances are minor and do not present a drainage problem. **Aside from these minor exceedances, the quantity control objective is satisfied with the proposed SWM solution.**

Outflows were estimated using equations for orifice flow and weir flow. The tables below summarize the size, invert, and primary function for each outlet structure.

Table 7-4: Summary of Quantity Control Structures at Outlet 1 (West End of Development)

Type and Size	Invert (m)	Primary Function
100mm diameter pipe	92.80	Control minor flows
0.5m length, 260mm deep weir	93.35	Control major flows
1.1m length, 290mm deep weir	93.61	Convey emergency overflows

Table 7-5: Summary of Quantity Control Structures at Outlet 2 (East End of Development)

Type and Size	Invert (m)	Primary Function
100mm diameter pipe	90.00	Control minor flows
200mm diameter pipe	90.75	Control major flows
2m length, 300mm deep weir	91.20	Convey emergency overflows

On-site runoff is conveyed through grassed swales and a relatively short storm sewer network that is proposed within Catchment 201. The dimensions, purpose, and drainage direction of each swale are summarized in Table 4-4.

The proposed combination of OGS units, temporary storage facilities, and outlet structures provide a SWM solution that satisfies the objectives outlined in Section 2.

Prepared and Submitted by:

LICENSED PROFESSIONAL ENGINEER
B. C. KEENE
Mar. 8/19
PROVINCE OF ONTARIO

Elliott Fledderus, E.I.T.
Jewell Engineering Inc.

Bryon Keene, P.Eng.
Jewell Engineering Inc.

8 References

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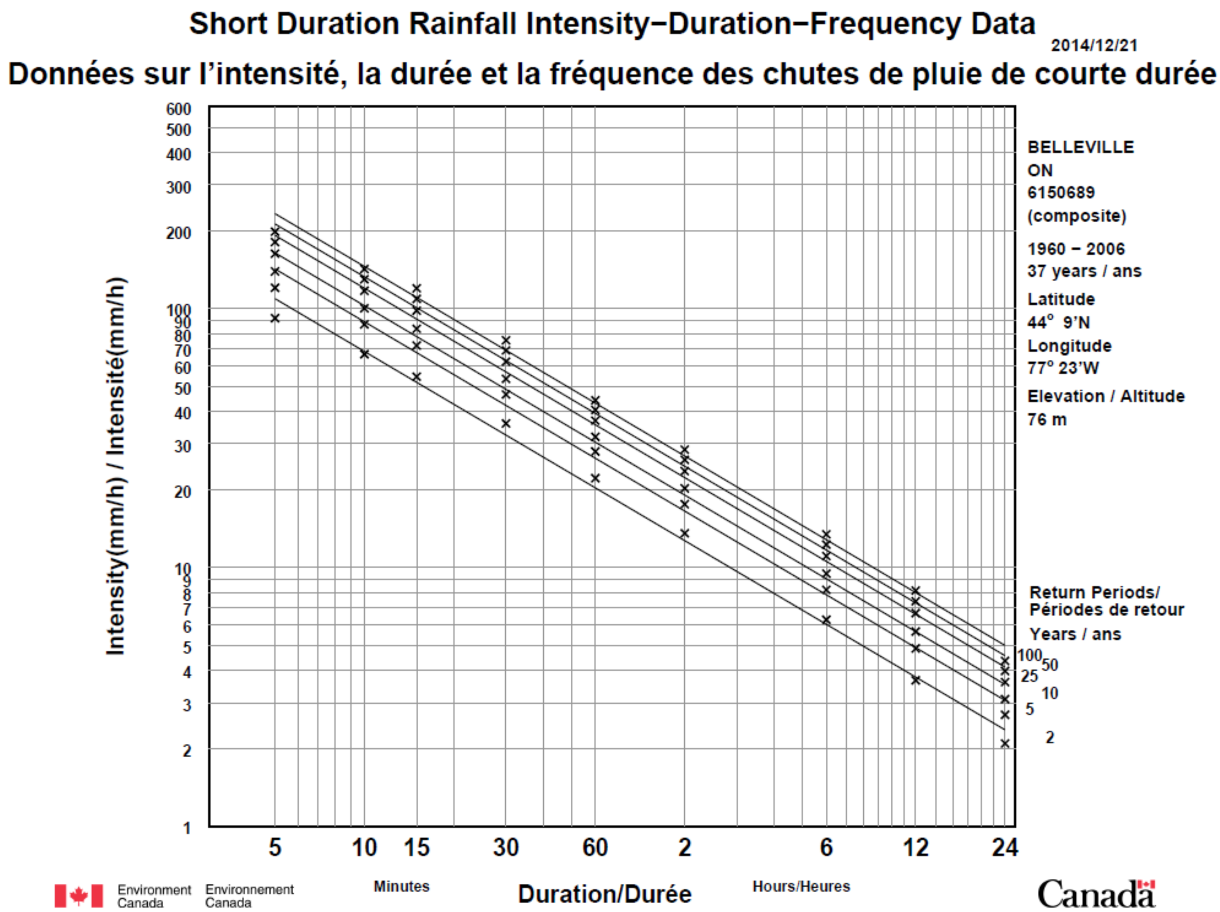
Appendix A

Belleville IDF Curves


Short Duration Rainfall Intensity−Duration−Frequency Data
2014/12/21
Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée
Intensity(mm/h) / Intensité(mm/h)
BELLEVILLE
ON
6150689
(composite)
1960 − 2006
37 years / ans
Latitude
44° 9'N
Longitude
77° 23'W
Elevation / Altitude
76 m
Return Periods/
Périodes de retour
Years / ans
100
50
25
10
5
2
5
10
15
30
60
2
6
12
24
Minutes
Duration/Durée
Hours/Heures
Environment Canada
Environnement Canada
Canada

Appendix B

Catchment Area Drawings

(see attached)

Appendix C

Storm Sewer Design Sheet for Pipe Network to East SWM Facility

STORM SEWER DESIGN SHEET FOR NETWORK TO EAST SWM FACILITY - NAUTICAL LANDS PHASE 2

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 Belleville

$$i = A \cdot t^{B}$$

Where:

i = Rainfall Intensity in mm/hr
t = Time of Concentration in hours

5-Year Parameters

A = 26.4
B = -0.677

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 m^2
R = Hydraulic radius = A / P
P = Wetted perimeter
S = Slope (m/m)
n = Manning's friction coef.

Check

$q \leq Q$
$V \leq 6\ m/s$

LOCATION			PEAK FLOW CALCULATION									PROPOSED SEWER								
STREET	FROM	TO	CATCHMENT AREAS RUNOFF COEFFICIENT 0.35	0.45	0.6	0.9	R.C. x A (ha)	CUM. R.C x A (ha)	TIME OF CONCENTRATION (min)	INTENSITY (mm/hr)	PEAK FLOW (m^3/s)	Pipe Size (mm)	Length (m)	Type of Pipe	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
Phase 2 Access Road																				
	CB1	CBMH3			0.29		0.17	0.2	4.5	152.5	0.07	300	26.70	RCP	1.79%	0.13	1.83	0.24	1.88	OK
	CB2	CBMH3				0.03	0.03	0.0	3.0	200.6	0.01	300	14.60	RCP	1.00%	0.10	1.37	0.18	0.97	OK
	CBMH3	CBMH4					0.00	0.2	4.7	147.1	0.08	300	84.00	RCP	1.00%	0.10	1.37	1.02	1.54	OK
	CBM4	East Pond Inlet			0.20		0.12	0.5	5.8	128.9	0.19	300	3.10	RCP	6.43%	0.25	3.47	0.01	3.83	OK

Jewell Engineering Inc	Ph. 613-969-1111	**Designed:** Elliott Fledderus, EIT	**Project:**
1-71 Millennium Parkway	Fx. 613-969-8988	**Checked:** Bryon Keene, P. Eng.	Nautical Lands - Wellings of Picton - Phase 2
Belleville, ON, K8N 4Z5	www.jewelleng.ca	**Date:** Friday, March 08, 2019	Prince Edward County

Appendix D

Hydro International Sizing Tool Used for Sizing of Downstream Defender Units

DD4 Unit for West SWM Facility:

Hydro Downstream Defender®
Net Annual Water Quality Worksheet

Project Name:	Wellings of Picton Inc.	Report Date:	2/15/2019
Street:		City:	Prince Edward County
Province:	Ontario	Country:	
Designer:		email:	

Treatment Parameters

Structure ID:	
TSS Goal:	80 % Removal
TSS Particle Size:	Fine
Area:	1.96 ha
Percent Impervious:	60%
Rational C value:	0.56
Rainfall Station:	Belleville, ON
Peak Storm Flow:	60 L/s
Peak Storm Return:	100 yrs

RESULTS SUMMARY

Model	TSS	Volume
DD4	**68.6%**	**101.5%**
DD6	78.0%	101.5%
DD8	84.0%	101.5%
DD10	87.8%	101.5%
DD12	90.3%	101.5%

DD4 Unit for East SWM Facility:

Hydro Downstream Defender®
Net Annual Water Quality Worksheet

Project Name:	Wellings of Picton Inc.	Report Date:	2/21/2019
Street:		City:	Prince Edward County
Province:	Ontario	Country:	
Designer:		email:	

Treatment Parameters

Structure ID:	
TSS Goal:	80 % Removal
TSS Particle Size:	Fine
Area:	0.84 ha
Percent Impervious:	50%
Rational C value:	0.58
Rainfall Station:	Belleville, ON
Peak Storm Flow:	60 L/s
Peak Storm Return:	100 yrs

RESULTS SUMMARY

Model	TSS	Volume
DD4	**78.0%**	**101.5%**
DD6	86.1%	101.5%
DD8	90.3%	101.5%
DD10	92.4%	101.5%
DD12	93.5%	101.5%

Appendix E

Stage-Discharge-Storage Calculations

Outlet 1 (West Towards Millennium Trail) Stage-Discharge and Stage-Discharge Calculations

Pipe 1 Invert (m)	92.8
Stage Increment (m)	0.1
Pipe 1 Diameter (m)	0.1
Orifice Coefficient	0.6
Area of Pipe (m2)	0.01
Pipe 2 Invert (m) Not Used)	1000000.00
Pipe 2 Diameter (m)	0.20
Orifice Coefficient	0.60
Area of Pipe (m2)	0.03
Weir Invert (m)	93.35
Weir Length (m)	0.5

Stage-Discharge Summary:

Stage (m)	$h_{orifice}$ (m)	Q_{pipe} (cms)	$h_{orifice}$ (m)	Q_{pipe} (cms)	h_{weir} (m)	Q_{weir} (cms)	Q_{total} (cms)
92.80	-0.05	#NUM!	0.00	0.00	0.00	0.00	#NUM!
92.90	0.05	0.00	0.00	0.00	0.00	0.00	0.00
93.00	0.15	0.01	0.00	0.00	0.00	0.00	0.01
93.10	0.25	0.01	0.00	0.00	0.00	0.00	0.01
93.20	0.35	0.01	0.00	0.00	0.00	0.00	0.01
93.30	0.45	0.01	0.00	0.00	0.00	0.00	0.01
93.40	0.55	0.02	0.00	0.00	0.05	0.01	0.03
93.50	0.65	0.02	0.00	0.00	0.15	0.05	0.07
93.60	0.75	0.02	0.00	0.00	0.25	0.11	0.13

Stage-Storage Summary:

Stage (m)	Area (m^2)	Incremental Volume (m^3)	Cumulative Volume (m^3)
92.8	0	0	0
92.9	175	9	9
93	459	32	40
93.1	740	60	100
93.2	1003	87	187
93.3	1270	114	301
93.4	1535	140	441
93.5	1869	170	612
93.6	2132	200	812

Outlet 2 (East Towards County Road 49) Stage-Discharge and Stage-Discharge Calculations

Pipe 1 Invert (m)	90.0
Stage Increment (m)	0.1
Pipe 1 Diameter (m)	0.1
Orifice Coefficient	0.6
Area of Pipe (m2)	0.01
Pipe 2 Invert (m)	90.85
Pipe 2 Diameter (m)	0.20
Orifice Coefficient	0.60
Area of Pipe (m2)	0.03

Stage-Discharge Summary:

Stage (m)	$h_{orifice}$ (m)	$Q_{pipe\ (1)}$ (cms)	$h_{orifice}$ (m)	$Q_{pipe\ (2)}$ (cms)	h_{weir} (m)	Q_{weir} (cms)	Q_{total} (cms)
90.00	0.00	0.000	0.000	0.000	0.00	0.00	0.0000
90.10	0.05	0.005	0.000	0.000	0.00	0.00	0.0047
90.20	0.15	0.008	0.000	0.000	0.00	0.00	0.0081
90.30	0.25	0.010	0.000	0.000	0.00	0.00	0.0104
90.40	0.35	0.012	0.000	0.000	0.00	0.00	0.0123
90.50	0.45	0.014	0.000	0.000	0.00	0.00	0.0140
90.60	0.55	0.015	0.000	0.000	0.00	0.00	0.0155
90.70	0.65	0.017	0.000	0.000	0.00	0.00	0.0168
90.80	0.75	0.018	0.000	0.000	0.00	0.00	0.0181
90.90	0.85	0.019	0.000	0.000	0.00	0.00	0.0192
91.00	0.95	0.020	0.050	0.019	0.00	0.00	0.0390
91.10	1.05	0.021	0.150	0.032	0.00	0.00	0.0537
91.20	1.15	0.022	0.250	0.042	0.00	0.00	0.0641

Stage-Storage Summary:

Stage (m)	Area (m^2)	Incremental Volume (m^3)	Cumulative Volume (m^3)
90.0	76	0	0
90.1	84	8	8
90.2	99	9	17
90.3	116	11	28
90.4	133	12	40
90.5	151	14	55
90.6	171	16	71
90.7	191	18	89
90.8	213	20	109
90.9	235	22	131
91.0	259	25	156
91.1	284	27	183
91.2	303	29	213

Appendix F

Swale Sizing Calculations

Note: *Swale B and Swale E as shown in Figure 4-1 are not included in this section as these swales are considered temporary ponding areas that function as the east and west SWM facilities. The conveyance from these facilities governs the post-development peak flows at Outlets 1 and 2 as discussed in Section 4. The swale sizing calculations provided below are for the rear-yard swales referred to as Swale A, C, and D in Figure 4-1.*

Swale A Sizing Calculation:

Mannings - Open channel flow

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

Desired Flow Capacity = **0.17** **<=== From Rational Method for maximum catchment contributing to Swale A**

Channel Configuration

Bottom Width	0	m
Side Slopes	3	:1
Slope	0.005	m/m
Roughness	0.025	(grass = 0.025, stone = 0.03)
Channel Depth	0.3	m

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

Assume Full Flow

A =	0.27	
P =	1.897367	
R =	0.142302	
V = Channel Velocity (m/s) =	0.77	
Q = Channel Flow Capacity =	**0.21**	**cms**

Check: **Capacity > Desired** **OK**

Swale C Sizing Calculations:

Mannings - Open channel flow

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

Desired Flow Capacity = **0.12** **<=== From Rational Method for maximum catchment contributing to Swale C**

Channel Configuration

Bottom Width	0	m
Side Slopes	3	:1
Slope	0.003	m/m
Roughness	0.025	(grass = 0.025, stone = 0.03)
Channel Depth	0.3	m

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

Assume Full Flow

A =	0.27	
P =	1.897367	
R =	0.142302	
V = Channel Velocity (m/s) =	0.60	
Q = Channel Flow Capacity =	**0.16**	**cms**

Check: **Capacity > Desired** **OK**

Swale D Sizing Calculations:

Mannings - Open channel flow

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

Desired Flow Capacity = **0.11** **<=== From Rational Method for maximum catchment contributing to Swale D**

Channel Configuration

Bottom Width	0	m
Side Slopes	3	:1
Slope	0.03	m/m
Roughness	0.025	(grass = 0.025, stone = 0.03)
Channel Depth	0.3	m

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

<u>Assume Full Flow</u>

A =	0.27	
P =	1.897367	
R =	0.142302	
V = Channel Velocity (m/s) =	1.89	
Q = Channel Flow Capacity =	**0.51**	**cms**

Check: **Capacity > Desired** **OK**