



**Village A - Base 31  
Picton, Ontario**

**Functional Servicing and Stormwater  
Management Report**

**October 2024**

**Revised August 2025**

**Submitted by:**

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## Submission History

Submission	Date	In Support Of	Distributed To
1 <sup>st</sup>	October 24, 2024	Draft Plan Approval	Prince Edward County
2 <sup>nd</sup>	August 29, 2025	Draft Plan Approval	Prince Edward County



## 1.0 Introduction

SCS Consulting Group Ltd. has been retained by PEC Community Partners Inc. to prepare a Functional Servicing and Stormwater Management Report (FSSR) for a proposed residential development within the Base31 Development Area, located in the Community of Picton, in Prince Edward County.

### 1.1 Purpose of the Report

This FSSR has been prepared in support of Draft Plan Approval applications for the proposed development, known throughout this report as “Village A”. The Draft Plan of Subdivision is provided in **Appendix A**. The proposed development consists of the following land uses:

- low density residential,
- medium density residential,
- parks,
- open space,
- SWM pond blocks, and
- proposed roads.

The purpose of this report is to demonstrate that the development can be graded and serviced in accordance with Prince Edward County, Quinte Conservation Authority and the Ministry of Environment, Conservation and Parks (MECP) design criteria.

This report has been prepared in collaboration with DLW Engineering Ltd. with regard to the mechanical aspects of water and wastewater facilities.

Prince Edward County is currently undergoing a Master Servicing Plan for the Community of Picton. This servicing plan will assess the existing servicing infrastructure and will outline the ultimate servicing upgrades anticipated to accommodate the growth projections. This application is subsequent to the Master Servicing Concept submission, prepared by SCS dated November 2023, in support of Official Plan Amendment (OPA) Applications for the subject site.



## 1.2 Study Area

The study area is approximately 29 ha in size and is bound by County Road #22 to the west, Kingsley Road to the south and an existing residential community known as Macaulay Village to the north. The existing Village A development area is primarily vacant. The proposed development is located within the Picton subwatershed, which is ultimately tributary to Lake Ontario. Refer to **Figure 1.1** below for the subject site in the context of the surroundings.



**Figure 1.1: Site Location Plan**

The study area is the first phase of a larger, 340 ha area currently owned by the applicant, known in its entirety as Base31. It is noted that the high-level servicing and

grading designs for the ultimate Base31 development have been previously presented within the Master Servicing Concept Application (SCS Consulting Group Ltd., November 2023), submitted in support of the Official Plan Amendment.

It is noted that sanitary and watermain servicing within the Base31 area within the Revitalization District (RD), immediately south of Kingsley Road, was constructed in 2024.

As per the Draft Plan of Subdivision, provided by The Biglieri Group Ltd., dated August 12, 2025, the following unit counts and breakdown are anticipated:

**Table 1.1: Draft Plan of Subdivision Site Statistics, The Biglieri Group Ltd., Aug 12, 2025**

Description	Unit Count	Area (ha)
Single Detached	257	10.06
Townhouses	201	3.97
Medium Density		4.02
Stormwater Management Pond		0.90
Park		2.08
Walkway		0.18
Open Space		0.19
Road Widening		0.35
Right of Way (Public)		9.07
Total Site	458	30.82

It is noted that Village A is anticipated to be constructed in phases based on market demand. A copy of the anticipated phasing plan is shown on the Draft Plan included in **Appendix A**.

### 1.3 Background Information

A pre-consultation with Prince Edward County was completed for this application in May 2024, and comments were provided to the applicant via email. Refer to **Appendix A** for the pre-consultation meeting summary.

In preparation of the servicing and stormwater management (SWM) strategies, the following design guidelines and standards were used:

- Sanitary, Storm and Water Systems Design Manual, prepared by the County of Prince Edward, March 2023;
- Grading and Drainage Manual, prepared by the County of Prince Edward, March 2023;
- Transportation Design Manual, prepared by the County of Prince Edward, March 2023;
- Ministry of Environment, Conservation, and Parks (MECP) Environmental Compliance Approval for a Municipal Stormwater Management System (April 2022).
- Quinte Conservation Stormwater Management Submission Guidelines, Quinte Conservation (May 2012);
- Bay of Quinte Remedial Action Plan (BQRAP) Implementation Area Stormwater Management Design Guidelines (Revised March 2006); and
- Stormwater Management Planning and Design Manual, prepared by Ministry of Environment (March 2003)

The servicing and SWM strategies in this report are based on the following reports:

- Wetland Stormwater Management Facility Information, prepared by SCS Consulting Group Ltd., July 10, 2025;
- Draft Preliminary Hydrogeological Investigation Report, Village A – Base 31 Residential Subdivision, prepared by Gemtec, May 13, 2025;
- Functional Servicing Assessment, Base31 – Draft Plan of Subdivision, Picton, prepared by SCS Consulting Group Ltd., March 13, 2025;
- Preliminary Geotechnical Site Investigation, Village A – Base31 Residential Subdivision, prepared by Gemtec, dated September 20, 2024;
- Master Servicing Concept Submission, prepared by SCS Consulting Group Ltd., dated November 2023;
- Scoped Hydrogeological Assessment – Base31, prepared by Palmer, November 1, 2023;
- Preliminary Geotechnical Investigation prepared by Palmer TM, July 9, 2021;
- Hydrogeological Assessment Draft Memorandum to support planning for Groundwater and Septic Servicing by Palmer Environmental Consulting Group, November 24, 2021;



- Wellington Master Servicing Plan for Water, Wastewater and Stormwater, prepared by R.V. Anderson (RVA), dated May 6, 2021;
- Vineridge Boutique Towns Conceptual Stormwater Management Report prepared by Greck and Associates Limited, September 1, 2020 (Not approved); and
- Prince Edward County 2017 Development Charges Background Study, prepared by Watson & Associates, dated December 1, 2017.

Relevant excerpts from the above listed documents are included in **Appendix B**.

## 2.0 Geotechnical and Hydrogeological Conditions

### 2.1 Soils

Gemtec has been retained to pursue a geotechnical investigation for the subject site. Borehole logs and test pit information has been provided to support this Draft Plan submission, and have been included in **Appendix B.1**. This is in addition to a Preliminary Geotechnical Investigation, prepared by Palmer, dated July 2021, which has also been included in **Appendix B.1**.

The site is located within the physiographic region of southern Ontario, known as the Prince Edward Peninsula. This region consists of clays, sandy/silty sands and till deposits and is underlain, specifically in the subject site, by Palaeozoic bedrock.

Field work has been carried out in July to August 2024 to advance 18 boreholes and 47 test pits throughout the subject site. Monitoring wells were installed on 7 of the 18 boreholes to monitor groundwater levels.

As per the recorded information on-site, the subsurface conditions at the site generally consist of topsoil (50 mm to 510 mm), overlying silty sand to silt sand and gravel, overlying the limestone bedrock. Bedrock was encountered at depths ranging from 0.30 mbgs to 0.8 mbgs.

As noted in the Draft Preliminary Hydrogeological Investigation Report, prepared by Gemtec, May 13, 2025 (**Appendix B.1**) groundwater levels have been recorded in the seven monitoring wells, with depths ranging from 0.6 to 3.4 mbgs (137.6- 145.4m). Groundwater levels at the site are anticipated to vary between and beyond borehole locations, and fluctuate on a seasonal basis.



## 3.0 Topography and Grading

### 3.1 Existing Conditions

#### 3.1.1 Topography

An existing topographic survey has been completed by Schaeffer Dzaldov, dated May 1<sup>st</sup>, 2023. A copy of this survey has been included in **Appendix B.2** for reference.

The existing topography of the subject site has slopes in the range of 0.50% to 8.00% - with higher slopes noted adjacent to County Road #22. It is noted that a natural ridge/high point exists separating the site such that approximately half the site drains northwest towards County Road #22 and the remainder of the site slopes southeast towards Kingsley Road. The ground surface elevations through the study area range from approximately 146.50 m at the natural ridge, to approximately 140.50 m adjacent to County Road #22 and 145.50 m in the southeast corner.

### 3.2 Proposed Conditions

#### 3.2.1 Site Grading

In general, the site will be graded in a manner which will satisfy the following criteria:

- Satisfy lot and road grading best practices including:
  - Minimum Road Grade: 0.5%
  - Maximum Road Grade: 5.0% (6% for Collector and 8% for local residential)
  - Minimum Lot Grade: 2%
  - Maximum Lot Grade: 5%
- Provide continuous road grades for overland flow conveyance;
- Minimize the need for retaining walls;
- Minimize the volume of earth to be moved and minimize cut/fill differential;
- Minimize the need for rear lot catchbasins;
- Achieve the stormwater management objectives required for the site; and
- Meet all applicable Accessibility for Ontarians with Disabilities Act, 'AODA' guidelines and standards.

A preliminary grading plan is shown on **Figure 3.1 – Preliminary Grading Plan**. The natural ridge is to be modified slightly to maximize the use of two stormwater management ponds: SWM Pond 1 located adjacent to County Road #22 at the lowest point of the site, and an interim SWM Pond 2 located southeast of the subject site, outside of the Village A boundary within lands owned by the applicant. At the detailed design stage, the preliminary grading shown on **Figure 3.1** will be subject to a more in-depth analysis in an attempt to balance the cut and fill volumes and minimize sloping, retaining walls and rear yard catchbasins. It is noted that due to the bedrock present on

the subject site, an emphasis has been made to minimize the required on-site excavations. Previously, basements were avoided to minimize the cost associated with rock excavation. To mitigate the cost of rock excavations, the site grading has been raised and the earthworks cost analyzed. From this analysis, it was concluded that the addition of basements would be beneficial from a marketing perspective, and financially viable. The homes within Village A will therefore be equipped with basements.



## 4.0 Rights-of-Way and Sidewalks

The road network for the proposed development will be comprised of a series of major/minor collector roads, local residential roads, as well as lane ways. The road cross sections have been prepared based in principle on the following guidelines and documents:

- Transportation Systems Design Manual, prepared by Prince Edward County, dated March 2023 (not approved).
- Prince Edward County drawings:
  - PSD-019 Local Road 18.0 m Residential
  - PSD- Local Road 20.0 m Industrial
  - PSD-022 – Arterial 27.0 m
  - PSD-023 – Collector 26.0 m

In collaboration with TYLin the transportation consultant, and NAK Design Group the landscape consultant, road cross sections have been prepared for the Base31 development. Generally, modifications have been made on the Prince Edward County (PEC) standard drawings to include for designated bike lanes and accommodating on-street parking, where required. Furthermore, NAK Design Group prepared a Right of Way Concept Design with specific objectives related to cycling routes and pedestrian routes. This design concept is outlined in **Appendix C.1**.

The existing roads surrounding the site, County Road #22 and Kingsley Road, may be fully urbanized during the ultimate build out, the extents of which will be determined through detailed design. As per the Official Plan Review, only County Road #22 is to be designated as an “Inter-Centre Collector” road, and therefore will need to be widened to 26.0 m (all existing right of ways are approximately 20.0 m). Kingsley Road will be designated as urban collector road, also at 26.0 m wide. However, for the purpose of Draft Plan approval for Village A, partial urbanization of Kingsley Road and County Road #22 will be completed pending ultimate build out of Base31 as follows and shown on figures provided in **Appendix C.2**.

- Figure C.1 - 26.0 m – Urban Collector Road (Kingsley Road)
- Figure C.2 - 26.0 m - Inter-Centre Road (County Road #22)

Additional cross sections have been developed for all new public roads proposed in this development, as outlined below and shown on figures provided in **Appendix C.2**:

- Figure C.3.1 - 18.0 m – Local Residential Road (Single Loaded)
- Figure C.3.2 – 18.0 m – Local Residential Road (Double Loaded)
- Figure C.3.3 – 18.0 m – Local Residential Road (Enhanced)
- Figure C.4.1 - 23.5 m – Urban Collector Road
- Figure C.4.2 – 23.5 m – Urban Collector Road (with Parking Lane)
- Figure C.5 – 8.5 m Municipal Rear Lane



Public 8.5 m laneways are proposed along the rear of all townhouse units and select single detached units. The laneways will not contain sanitary, water or utilities aside from the cables required to operate the street lights. Refer to Figure C.5 provided in **Appendix C.2** for the 8.5 m Municipal Laneway.

These cross sections along with NAK’s Concept Design have been reviewed by the County and were accepted with the sole comment being to shift the watermain from under the walkway or bike path within the boulevard, to the road. There was also a comment related to the need for a single sidewalk as opposed to a sidewalk on both sides of the right-of-way. This will be explored further at detailed design. Refer to **Appendix C.3** for the PEC and Stantec comments.

Refer to **Figure 4.1** for the proposed sidewalk plan. PEC Municipal standards require sidewalks on one (1) side of all urban local streets. However, dual sidewalks are proposed in certain locations due to high pedestrian activity and building typologies. Sidewalk locations will be further investigated through detailed design.

## 5.0 Storm Drainage and Servicing

### 5.1 Existing Conditions

#### 5.1.1 Existing Storm Drainage

The existing high level storm drainage pattern is shown on **Figure 5.1**. The site is divided into two separate catchment areas, as described below:

##### Marsh Creek

As illustrated, runoff from the northwest portion of the study area, Catchment 101 (19.20 ha, **Figure 5.1**) and external flows from an existing residential development north of the study area (McCauley Village), Catchment EXT-1 (1.18 ha, **Figure 5.1**), drain west via overland flow to Marsh Creek within the Picton subwatershed. Flows from Marsh Creek ultimately discharge to the Picton Harbour in Lake Ontario.

##### East Lake

As illustrated, runoff from the southeastern portion of the study area, Catchment 102 (33.05 ha, **Figure 5.1**) and external flows from the future Vineridge development north of the study area, Catchment EXT-2 (15.86 ha, **Figure 5.1**), drain south via overland flow to an existing watercourse in the east portion of the study area. The watercourse continues to convey flows south and ultimately discharges to East Lake, southwest of the study area, which then outlets to Lake Ontario. Refer to Figure 2.1 included in **Appendix B3** for the total external drainage areas draining to East Lake.

#### 5.1.2 Existing Storm Servicing

The existing site does not contain any stormwater management facilities, structures or controls. The vacant and vegetated areas drain solely by gravity to existing ditches along the east boulevard of County Road #22 and the northern boulevard of Kingsley Road. County Road #22 and Kingsley Road are currently rural roads with roadside ditches and culverts.

### 5.2 Proposed Conditions

#### 5.2.1 Proposed Storm Drainage

The proposed major and minor system flow patterns and storm drainage areas are shown on **Figures 5.2** and **5.3**. As illustrated, drainage from the proposed development will be conveyed to both Marsh Creek and East Lake.

The proposed minor and major system flows from the western portion of Village A, Catchment 201 (19.50 ha, **Figure 5.2**) will be captured via the proposed internal storm



sewer system and conveyed to proposed SWM Pond 1, which ultimately discharges to Marsh Creek.

Drainage from Catchment 202 (0.21 ha, **Figure 5.2**) will drain uncontrolled via overland flow to Country Road #22, ultimately discharging to Marsh Creek.

The proposed minor and major system flows from the eastern portion of Village A, Catchment 203 (17.16 ha, **Figure 5.2**) and external Catchment EXT-1 (1.18 ha, **Figure 5.2**) will be captured via the proposed internal storm sewer system and conveyed to proposed interim SWM Pond 2, which ultimately discharges to East Lake.

Drainage from Catchment 204 (0.96 ha, **Figure 5.2**) will drain uncontrolled to Kingsley Road, ultimately discharging to East Lake. Drainage from Catchment 205 (14.50 ha, **Figure 5.2**) and external Catchment EXT-2 (15.86 ha, **Figure 5.1**) will drain uncontrolled to East Lake. Ultimately, Catchment 205 will be developed as future Village C, and serviced through proposed SWM Facility 2, which will be extended and sized accordingly at that time.



## 5.2.2 Proposed Storm Servicing

### 5.2.2.1 Minor System

The minor storm sewer system will be designed for the 5 year return storm as per PEC standards. The storm sewer system will generally be designed with gradients between 0.5% and 2%. In some cases, to avoid excessively deep sewers, or to follow the road profile, it may be required that the sewer slope falls out of this range; however, it will be confirmed through detailed design that the minimum and maximum velocities outlined in the County’s criteria are respected. The preliminary layout for the proposed storm sewer within the proposed development is provided on **Figure 5.3**.

The storm drainage system will be designed in accordance with the PEC, MECP, and OBC guidelines (where applicable), including the following:

- Pipes to be sized to accommodate runoff from a 5 year storm event,
- Minimum Pipe Size: 300 mm diameter,
- Maximum Flow Velocity: 6.0 m/s under full flow conditions,
- Minimum Flow Velocity: 1.0 m/s under full flow conditions, and
- Minimum Pipe Depth: 1.2 m to obvert.

The rainfall intensity will be calculated in accordance with the PEC design guidelines based on using the City of Belleville 2012 IDF curves provided by Quinte Conservation and a minimum time to concentration of 15 minutes. The rainfall intensity will be calculated as follows, where ‘i’ is the rainfall intensity (mm/hour) and A and C are as per **Table 5.1**:

$$i = A * T_c^C$$

**Table 5.1: Rainfall Intensity Parameters**

Return Period Storm	A	C
2 Year	20.3	0.677
5 Year	26.4	0.677
10 Year	30.4	0.678
25 Year	35.5	0.678
50 Year	39.3	0.678
100 Year	43.0	0.678

As per PEC standards, there are no individual storm connections required for residential units. As such, all rooftops are anticipated to drain by downspouts to grade, and all foundation drainage, if required, will be discharged to grade via sump pumps.



### 5.2.2.2 Major System

Major system flows (greater than the 5 year up to the 100 year storm event) will be conveyed within the road rights-of-way to either SWM Pond 1 or SWM Pond 2.

The proposed right-of-way cross-sections outlined in **Section 4.0**, have been designed in accordance with County criteria. Overland flow and rightof-way capacity calculations have been included in **Appendix D**.



## 6.0 Stormwater Management

### 6.1 Stormwater Management Criteria

The following stormwater management criteria have been established based on the greatest requirements of each of the design guidelines and standards listed in **Section 1.3** and discussion with Quinte Conservation. The stormwater management criteria are summarized below in **Table 6.1**:

**Table 6.1: Stormwater Management Criteria**

Criteria	Control Measure
Quality Control	At a minimum, Enhanced Level Protection (80% Total Suspended Solids (TSS) Removal) through conventional stormwater management (Per BQRAP).
Erosion Control	Detention of the 25 mm rainfall runoff for a minimum of 24 hours (Per MOE).
Quantity Control	Control to existing peak flows for the 2 through 100 year storm events (Per BQRAP).
Water Balance	Based on conversations with Quinte Conservation Authority, due to the high bed-rock present on site, no water balance targets are required.

### 6.2 Proposed Stormwater Management Plan

In accordance with the Ministry of Environment (MOE) Stormwater Management Planning and Design Manual (2003), a review of stormwater management best practices was completed using a treatment train approach, which evaluated lot-level, conveyance system and end-of-pipe alternatives.

The following study area characteristics and constraints were taken into consideration:

- The topography follows a natural ridge separating the site such that approximate half the site drains north-west towards County Road #22 and the remainder slopes south-east towards Kingsley Road;
- Based on the Geotechnical investigation, the study area soils consisted of clays, sandy/silty sands and till deposits soils. Shallow bedrock was observed across the study area at depths ranging from 0.30 mbgs to 0.8 mbgs;
- Within the installed site wells, groundwater was observed at depths ranging from 0.6 to 3.4 mbgs (137.6- 145.4 m);



- ➔ The proposed Village A development is approximately 28.71 ha and consists of a SWM block, two high-density blocks, and a combination of single-family and townhouse units; and
- ➔ The study area drains northwest to Marsh Creek and southeast to East Lake according to the aforementioned natural ridge.

The feasibility of at-source, conveyance and end-of-pipe SWM controls were evaluated for use in the proposed development to achieve the design criteria provided in **Section 6.1**. Refer to **Sections 6.3** and **6.4** for a review of Low Impact Development (LID) measures and end-of-pipe SWM facilities, respectively.

### **6.3 Low Impact Development Measures**

Low Impact Development (LID) measures provide reduction of runoff and promote infiltration at the source, where possible. Each type of LID measure has the potential to provide water quality control including Total Suspended Solids (TSS) removal, erosion control, and/or water budget benefits.

Due to the shallow bedrock throughout the proposed development, retention via infiltration type LID measures is not feasible. Retention via water re-use may be possible on proposed residential lots through rain barrels. Water re-use may also be feasible within the high-density areas of the proposed development, if underground SWM facilities are proposed. Water re-use opportunities can be explored at the detailed design phases of these areas of the proposed development.

LID measures that provide filtration are feasible within the proposed development, where there is sufficient depth to the storm sewer system. However, filtration LIDs require frequent maintenance to ensure proper operation. Considering the rural nature of the surrounding area, and the additional staffing and financial resources required for maintaining these facilities, LID filtration is not recommended as the primary quality control measure for the proposed development. LID filtration as part of a treatment train approach can be explored at the detailed design phases.

It is therefore recommended that conventional stormwater management methods be used as the primary quality control measure within the proposed development. As listed in **Table 6.1**, per the BQRAP SWM Guidelines, Enhanced Level Protection (80% TSS Removal) is required, which can be achieved via permanent pools within the proposed SWM ponds.

### **6.4 End-of-Pipe Stormwater Management Facilities**

Two SWM facilities (SWM Pond 1 and interim SWM Pond 2) are proposed to provide quality, erosion and quantity control for Village A. Proposed SWM Pond 1 is located adjacent to County Road #22 at the lowest point of the site and proposed interim SWM Pond 2 is located southeast of the subject site, as shown on **Figures 6.1** and **6.2**. SWM



Pond 1 and interim SWM Pond 2 will be sized for total drainage areas of 19.50 ha (62% imperviousness) and 18.34 ha (41% imperviousness), respectively. Refer to **Appendix D** for the drainage area summary, and **Appendix E** for impervious area calculations.

Interim SWM Pond 2 is proposed as an interim condition to service Phase 2 of Village A, in addition to the drainage from the future development lands (Village C), which will remain undeveloped, and continue to drain to interim SWM Pond 2. The ultimate SWM Pond is not proposed to be built out at this time, as the future land uses and servicing for these lands are not known, and due to the expensive nature of building the SWM facility within bedrock, it is recommended to wait until the ultimate SWM Facility sizing required to service the future lands is known, to avoid unnecessary costs associated with digging into the bedrock. When development of Village C proceeds, interim SWM Pond 2 can be expanded easterly in the ultimate condition. This will include the addition of a second eastern forebay while maintaining the existing outfall with modifications to the outlet control structure based on the additional drainage from Village C. The bottom, top and permanent pool elevations of the SWM facility will remain the same.

Due to the presence of bedrock, the wet pond design presented in the first submission of the Functional Servicing and Stormwater Management Report (SCS Consulting Group Ltd., October 2024) was reevaluated. To minimize the amount of bedrock to be excavated to construct the SWM facilities, the grading and servicing of the Village A lands was raised, and the type of SWM facilities was changed from a wet pond to a constructed wetland design. Refer to **Appendix B.4** for the correspondence with PEC staff discussing the change to the proposed wetland SWM facility.

As outlined below in **Sections 6.4.1** through **6.4.3**, both wetland SWM facilities will provide the required quality, erosion and quantity control to achieve the SWM criteria in **Section 6.1**.

#### **6.4.1 Quality Control**

The function of the permanent pool is to provide quality control by removing sediment from the storm runoff conveyed to the SWM facilities. The proposed SWM facilities will be designed to provide permanent pool storage based on an Enhanced level of protection for a wetland facility (MOE, 2003). A summary of permanent pool storage for SWM Pond 1 and interim SWM Pond 2 is provided in **Table 6.2** below, refer to calculations in **Appendix D**. The available permanent pool storage exceeds the required storage as the facility grading must meet the minimum depth and sloping standards in accordance with the MOE SWMP Manual (2003). The grading design will be optimized at detailed design to reduce available storage to meet required storage.



**Table 6.2: SWM Facility Permanent Pool Summary**

SWM Facility	SWM Pond 1	Interim SWM Pond 2
Required Permanent Pool (m <sup>3</sup> )	1,408	869
Provided Permanent Pool Volume (m <sup>3</sup> )	2,232	1,932

#### 6.4.2 Erosion Control

The MOE SWMP Manual (2003) states that the extended detention storage is based on the greater of 40 m<sup>3</sup>/ha or the storage volume required to retain a 25 mm storm for 24 to 48 hours. As outlined in **Section 6.1** above, a detention time of 24 hours is required for the site. A Visual Otthymo 6.2 hydrologic model was used to establish the post-development runoff from the proposed development during a 25mm, 4-hour Chicago rainfall event. The required extended detention storage for SWM Pond 1 and interim SWM Pond 2 based on the runoff volume detained for 24 hours was determined to be approximately 780 m<sup>3</sup> and 734 m<sup>3</sup>.

The modelled runoff yielded a larger storage volume than the extended detention volume based on 40 m<sup>3</sup>/ha, therefore the erosion control storage volumes for SWM Pond 1 and interim SWM Pond 2 are 2,410 m<sup>3</sup> and 2,074 m<sup>3</sup> respectively. A summary of extended detention volumes for SWM Pond 1 and interim SWM Pond 2 is provided in **Table 6.3** below.

**Table 6.3: SWM Facility Extended Detention Summary**

SWM Facility	SWM Pond 1	Interim SWM Pond 2
25mm – 4 hour Chicago Storm Volume Requirement (m <sup>3</sup> )	2,410	1,520
MOE 40 m <sup>3</sup> /ha Volume Requirement (m <sup>3</sup> )	780	734
Governing Volume (m <sup>3</sup> )	2,410	1,520

The calculations for the extended detention volumes for the proposed stormwater management facilities are provided in **Appendix D**, with hydrologic modelling files and outputs provided in **Appendix E**. Orifice sizing calculations and details will be provided at detailed design.

#### 6.4.3 Quantity Control

As outlined in **Section 6.1**, control of the post-development flows to pre-development levels is required for the 2 through 100 year storm events. The pre-development flow rates for the 2 through 100 year storm events have been established utilizing the Visual Otthymo Version 6.2 software (VO6) based on the 4-hour Chicago and 6-hour, 12-hour, and 24-hour SCS Type II Distribution methods.



The study area is located within the Picton subwatershed; therefore, the City of Belleville 2012 IDF curves provided by Quinte Conservation were utilized to determine the existing peak flows to outlet locations. The existing flows from the study area to the Marsh Creek and East Lake are summarized in **Table 6.4** and **Table 6.5**.

**Table 6.4: Summary of Existing Flows – Marsh Creek Outfall (Outlet 1)**

Return Period Storm	4 Hour Chicago (m <sup>3</sup> /s)	6 Hour SCS (m <sup>3</sup> /s)	12 Hour SCS (m <sup>3</sup> /s)	24 Hour SCS (m <sup>3</sup> /s)
2 Year	0.038	0.053	0.076	0.106
5 Year	0.068	0.100	0.137	0.187
10 Year	0.094	0.136	0.184	0.248
25 Year	0.133	0.189	0.253	0.335
50 Year	0.165	0.233	0.310	0.406
100 Year	0.199	0.280	0.369	0.480

**Table 6.5: Summary of Existing Flows – East Lake Outfall (Outlet 2)**

Return Period Storm	4 Hour Chicago (m <sup>3</sup> /s)	6 Hour SCS (m <sup>3</sup> /s)	12 Hour SCS (m <sup>3</sup> /s)	24 Hour SCS (m <sup>3</sup> /s)
2 Year	0.154	0.267	0.401	0.588
5 Year	0.304	0.496	0.720	1.020
10 Year	0.426	0.676	0.962	1.339
25 Year	0.608	0.939	1.313	1.798
50 Year	0.760	1.156	1.598	2.167
100 Year	0.922	1.383	1.894	2.548

A summary of modelling parameters and the existing conditions VO6.2 schematic are provided in **Appendix E** with digital modelling files included with the submission of this report. Refer to **Section 6.4.4** for pond sizing.

The proposed SWM facilities will control future flows from the proposed development to existing flow rates provided in **Tables 6.4** and **6.5** for the 2 to 100 year storm events discharging to the Marsh Creek and East Lake outlets. Both SWM facilities will provide over-control for uncontrolled areas. Proposed hydrologic modelling was completed using the Visual Otthymo model (Version 6.2) to determine the required SWM facility volume. Refer to the File Safe Cloud Link provided in **Appendix E** to download the VO hydrology model files.

The 6-hour, 12-hour, and 24-hour SCS Type II design storms and the 4-hour Chicago Storm distribution per PEC requirements were modelled for the proposed conditions



hydrology model. While the 4-hour Chicago storm produced the smallest allowable peak flows, setting the target release rates for the SWM Facilities, the 24-hour SCS Type II design storm generated the largest quantity control volumes within the SWM Facility. The results of the 24-hour SCS Type II design storm will therefore be referenced for the below storage requirements, refer to **Table 6.6**. Storage summary tables for the other storm distributions can be found in **Appendix E**.

**Table 6.6: SWM Facility Storage Requirements (24-hour SCS Type II)**

Storm Facility	SWM Pond 1		Interim SWM Pond 2	
	Discharge (m <sup>3</sup> /s)	Storage (ha*m)	Discharge (m <sup>3</sup> /s)	Storage (ha*m)
2 Year	0.086	0.429	0.061	0.295
5 Year	0.157	0.565	0.120	0.399
10 Year	0.214	0.654	0.160	0.469
25 Year	0.286	0.775	0.215	0.567
50 Year	0.342	0.868	0.257	0.644
100 Year	0.397	0.960	0.300	0.721

**Table 6.7** and **Table 6.8** below, provide a comparison for the allowable and total controlled release rates from the development for the 24-hour, SCS Type II design storms. The resulting post-development peak flows to Marsh Creek and East Lake for all storm events are summarized in **Appendix D** where they are compared to the pre-development allowable targets, showing that the pre-development peak flows are not exceeded for all storm events. The stormwater management strategy for the proposed development is therefore effective, and meets the quantity control criteria outlined in **Table 6.1**.

**Table 6.7: Summary of Proposed Release Rates to Marsh Creek (24-hour SCS Type II)**

Return Period Storm	Allowable Release Rate (m <sup>3</sup> /s)	Controlled Release Rate - SWM Pond 1 (m <sup>3</sup> /s)	Uncontrolled Release Rate - Catchment 202 (m <sup>3</sup> /s)	Total Proposed Release Rate (m <sup>3</sup> /s)
2 Year	0.106	0.086	0.028	0.087
5 Year	0.187	0.157	0.038	0.159
10 Year	0.248	0.214	0.044	0.216
25 Year	0.335	0.286	0.053	0.289
50 Year	0.406	0.342	0.059	0.345
100 Year	0.480	0.397	0.066	0.401

**Table 6.8: Summary of Proposed Release Rates to East Lake (24-hour SCS Type II)**

Return Period Storm	Allowable Release Rate (m <sup>3</sup> /s)	Controlled Release Rate - SWM Pond 2 (m <sup>3</sup> /s)	Uncontrolled Release Rate - Catchments 204, 205, EXT-2 (m <sup>3</sup> /s)	Total Proposed Release Rate (m <sup>3</sup> /s)
2 Year	0.588	0.061	0.512	0.549
5 Year	1.020	0.120	0.867	0.949
10 Year	1.339	0.160	1.127	1.217
25 Year	1.798	0.215	1.500	1.678
50 Year	2.167	0.257	1.799	2.017
100 Year	2.548	0.300	2.107	2.365

To summarize, the proposed SWM facilities have been designed to control the proposed development runoff to the allowable release rates to Marsh Creek and East Lake for all storm events, taking into consideration the uncontrolled drainage to each outlet. The SWM facility outlet control structures consisting of orifice controls and/or weirs will be optimized during detailed design to minimize the volume requirements while still achieving the allowable release rates at the outfalls.



#### 6.4.4 General SWM Facility Design Criteria

Preliminary SWM facility grading is provided on **Figures 6.1** and **6.2**. The wetland pond block size was established based on the following general criteria in accordance with MOE SWMP Guidelines (2003):

- A 4 m wide maintenance access road will be provided from a proposed municipal road with a maximum longitudinal slope of 8% and a crossfall of 2% (max). It will be used to facilitate machinery to access the forebay during scheduled maintenance as well as to access the outlet structure for maintenance purposes;
- A maximum slope of 3:1 from the pond bottom to 0.5 m below the normal water level will be provided;
- A maximum slope of 5:1 from 0.5 m below and above the normal water level will be provided; and
- A maximum slope of 3:1 will be provided from 0.5 m above the normal water level to the pond grading limits.

#### 6.4.5 Wetland Maintenance and Operations

Through discussion with PEC staff, further information regarding the maintenance and operations of a constructed wetland facility was requested, particularly with respect to how it may differ from a wet pond. A letter was prepared by SCS Consulting Group Ltd. re: Wetland Stormwater Management Facility Information, dated July 10, 2025, included in **Appendix B.4**. This letter outlines the general maintenance and rationale of the wetland facility, which was shared with the County, reviewed and accepted.

As discussed in the letter, the maintenance and operations for a wetland facility are similar to that of a wet pond, with the major difference being the total volume of water within the permanent pool of a wetland being less than a wet pond, with similar 10 year sediment removal volumes.

An Operations and Maintenance Manual will be provided through the detailed design of the wetland facilities, which will provide direction on inspections, maintenance activities including garbage removal and plantings, and sediment removal frequency and methodology. The general sediment removal procedure will be as follows:

1. Install erosion and sediment control measures to ensure the maintenance activities will not cause downstream erosion problems or sediment/dust problems with adjacent residences.
2. Inspect all inlets, outlets, control structure, overland flow route, spillway etc. If there are any deficiencies, maintain the infrastructure as required.
3. Drain the wetland facility. (As we work through the design details of the wetland facilities, we will look to incorporate gravity drains for the permanent pool; however, if this is not feasible, the permanent pool will need to be pumped.)



4. Remove sediment (and vegetation) via long reach backhoes. (Dependent on the frequency of maintenance and the amount of sediment accumulation, removal of sediment can be isolated to the forebay; however, it is recommended to maintain the main cell if vegetation grows to the point of impacting the available flood storage).
5. Restore vegetation as per the approved Landscape Restoration Plan.

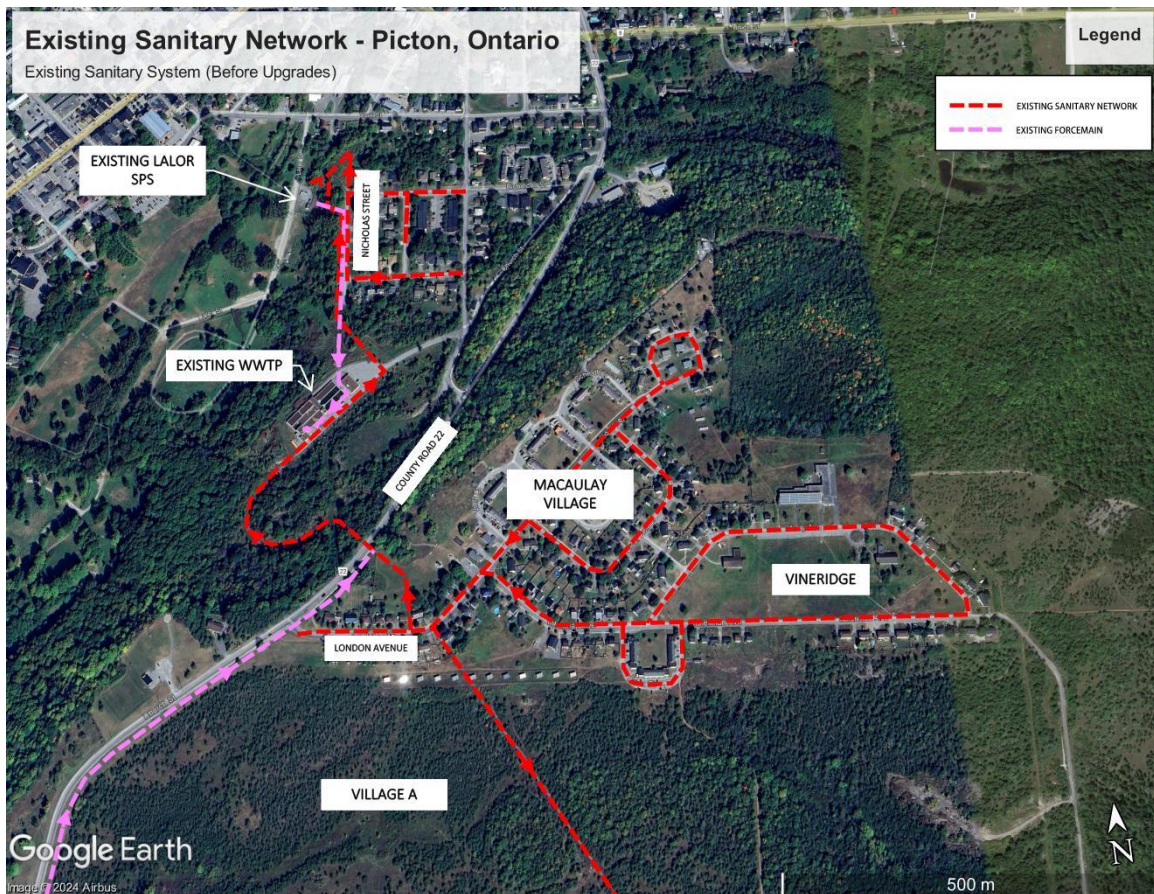


## 7.0 Sanitary Servicing

### 7.1 Existing Sanitary Sewer System

All existing sanitary flows from the residential area north of the Base31 lands are conveyed by a 200 mm diameter sanitary sewer across County Road #22 to the Lalor Sanitary Pumping Station (SPS). This area is referred to throughout this report as the “Heights”, and includes the existing Macaulay Village and a future residential site, noted as Vineridge. The Lalor SPS is located just west of Nicholas Street, north-west of the Base31 subject site.

From the Lalor SPS, the sanitary flow from all of Picton is pumped to the Picton Water Pollution Control Plant (WWTP), located south of the Lalor SPS. The existing sanitary network is shown on **Figure 7.1** below:



**Figure 7.1: Existing Sanitary System**

Based on input from County staff and a review of the background information, all three components of this existing system – the 200 mm diameter gravity sewer, the Lalor SPS and the WWTP are anticipated to be upgraded in the coming years and are approaching their rated capacity.



The following summarizes the existing systems:

Sanitary Gravity Sewer:

- The existing 200 mm diameter sanitary gravity sewer does not have the hydraulic capacity to service significant additional flow. Several pipes are currently running in excess of their hydraulic capacity and hydraulic grade line issues occur when adding additional flow. Refer to **Section 7.3** for details.

The Lalor Sanitary Pumping Station (SPS):

- The Lalor SPS has a rated design capacity of 44,000 m<sup>3</sup>/day however, with only 2 pumps functioning at 175 L/s each, the firm capacity is 30,240 m<sup>3</sup>/day.
- Operators report that the headworks at the WWTP are overwhelmed when the larger flow pumps are operating.
- It was confirmed by CIMA+ in their comment letter dated March 18, 2025 that the existing pumps within the SPS are not variable frequency drive (VFD) driven – as such, when the SPS experiences a peaked event, the high-flow pumps automatically start – causing an instantaneous, and overwhelming discharge to the headworks. Additional investigations are required to confirm.

In regard to the capacity of the Lalor SPS, we have considered information provided by the County (see **Appendix F.1**) which notes the following:

- Flow to the Lalor SPS is 364 L/s according to RVA's Report
- WWW Operations has noted that the SPS does not surcharge and stays within the wet well
- The Lalor SPS sustained a flow rate of 313 L/s in 2017 which was considered a wet year

In regards to RVA's assertion that the flow to the Lalor Street SPS is 364 L/s, we suspect that this may be overly conservative. It is higher than the measured flow which may be the result of a factor of safety being applied to the multiple catchments which drain to the Lalor SPS and as such, the overall cumulative flow is exceeding what is being observed in reality. As noted by WWW Operations, if the SPS does not surcharge and stays within the wet well, this would suggest an incoming flow of 350 L/s or less. Based on this, we have revised the PCSWMM model to exclude a surcharge at the Lalor SPS which was part of the original model.

As noted by the County, if during a wet year such as 2017, the Lalor SPS sustained a flow rate of 313 L/s over a sustained hour, this would be considered a worst-case scenario and a conservative approach to assessing the available capacity of the SPS. If the



inclusion of the third pump increases flows to the extent that upgrades are required at the WWTP headworks, the capacity of the third pump would be ignored and the use of only the two pumps which have a capacity of 175 L/s each for a total flow of 350 L/s would be considered.

If the Lalor SPS capacity can be considered as 350 L/s, and the worst-case flow event reaches 313 L/s based on historical data, it could be comfortably concluded that the station has a remaining capacity of 37 L/s.

The Picton Water Pollution Control Plant (WWTP)

- The WWTP has been designed with an average day flow (ADF) capacity of 6,000 m<sup>3</sup>/day.
- Based on Annual Operating Results, the WWTP is currently running at an average rate of 2,924 m<sup>3</sup>/day (2019-2023).
- Allocation of the remaining capacity within the WWTP is reserved to the discretion of the County.
- It is noted that a future expansion of the WWTP is anticipated based on the original design, and details regarding timing and approach will be provided within the County MSP and subsequent reports.

Upgrades to the three components of this system will be primarily dictated by the on-going Picton Master Servicing Plan (MSP) based on overall growth in Picton. While the details of these upgrades are dictated based on the on-going MSP, as well as on-going discussions with County Staff, it is to be determined by Staff if there is the potential for partial development of Village A prior to all upgrades.

The existing lands occupied by Village A currently do not contribute to this sanitary network. An existing abandoned sanitary sewer and watermain bisects the subject site, which previously serviced 204 Kingsley Road. However, this sewer and watermain are to be removed as part of this application. A subsurface utility investigation (SUE) was completed for the entire Village A area, including County Road #22 and the outfall to the WWTP. The SUE is found in **Appendix B.2**.

Two private sanitary forcemain service connections (75 mm and 200 mm diameter) have been constructed within the applicant's overall land holdings as part of an approved Interim Servicing Agreement between the applicant and the municipality. While it is intended that these sewers remain operational throughout construction, the intention is that they can be eliminated north of Kingsley Road, and transferred to discharge to the sanitary extensions proposed under this application as described below.

#### **Existing Sewers East of County Road #22 to Lalor SPS (Surcharging Runs under Existing Conditions)**



With respect to the gravity sewer from County Road #22 to the Lalor SPS, an HGL analysis has been completed to confirm its capacity using the original model provided by the County (CIMA+) in December 2023. This model has been revised to include sewer runs which were missing from the original model. This analysis requires an understanding of the existing flows from MacCauley Village which are well documented in the RVA Report dated March 9, 2023 entitled “Picton Sanitary Model – VineRidge Boutique Towns” (see **Appendix F.2**). This report provides findings based on 4 years of flow data (years 2019, 2020, 2021 and 2022) and concludes the following:

- The average daily flow was approximately 1 L/s.
- There were 10 days with recorded peak flows greater than 10 L/s.
- In 2022, the system experienced more events with peak flows above 15 L/s when compared to other years (i.e. 2019, 2020 and 2021).
- In 2022, there were 3 events with a peak flow equal to or greater than 20 L/s.

The RVA report notes a total flow from McCauley Village of 30.3 L/s. Upon review of the original PCSWMM model received from the County in December 2023, it was observed that a flow rate of 34 L/s was utilized to represent the flow from existing McCauley Village. This slightly exceeds what is in the RVA report however, a flow of 34 L/s has been utilized as the flow from McCauley Village in the updated PCSWMM model to be conservative.

Using the PCSWMM model provided by the County which includes a flow rate of 34 L/s for McCauley Village, and noting that the model has been modified to include sewer runs missing from the original model, under existing conditions, 4 sewer runs are surcharging. See **Figures 7.1.1** and **7.1.2** for the plan and profile views respectively which illustrate that there are 4 surcharging runs between County Road #22 and the Lalor SPS.

## **7.2 Proposed Internal Sanitary Sewer System**

The preliminary layout for the proposed sanitary sewer within the subject lands is provided on **Figure 7.2**.

The sanitary sewers within the proposed development will have slopes ranging between 0.5% and 2% (typically) and will be provided at 3.3 m to 5 m deep.

The sanitary sewer system will be designed in accordance with Prince Edward County and MECP criteria, including but not limited to:

- Residential Sanitary Generation Rate: 320 L/c/day (excluding extraneous flows)
- Residential Population Density:
  - Low Density - Single detached, semi-detached or duplexes- 25 units/ha or 3.0 persons per unit (PPU)



- Medium Density – Triplexes, fourplexes, townhouses – 26-53 units/ha or 2.5 PPU.
- High Density – apartment Dwellings – 54-300 units/ha at the following PPU:
  - Bachelor/1 Bedroom – 1.4 PPU
  - 2 Bedroom – 2.1 PPU
  - 3 Bedroom – 2.4 PPU
  - 4 Bedroom – 3.4 PPU
  - Bedroom Unknown – 1.7 PPU
- Peaking Factor: Harmon (Min 2.0)
- Infiltration Rate: 0.28 L/s/ha
- Industrial and Commercial (ICI) Flow Rates
  - Commercial Flow Rate – 28 m<sup>3</sup>/day/floor ha (excludes extraneous flows)
    - Total floor area cannot exceed total site area
    - Peaking Factor – 2, generally
  - Industrial Flow Rate – 55m<sup>3</sup>/day/floor ha
    - Total floor area cannot exceed total site area
    - Peaking Factor – 2-4
- Minimum pipe size: 200 mm diameter in residential areas
- Minimum pipe cover: 2.5 below centerline road elevation to obvert, (sewers will be designed slightly deeper than the minimum to allow for gravity drainage from basements)
- Maximum velocity: 3.0 m/s at full flow
- Minimum actual velocity: 0.6 m/s
- Minimum slope (first run): 1.0%

Using the above criteria, as well as the site statistics provided in the Draft Plan of Subdivision, Village A (Phases 1 and 2) has an anticipated serviced population of 1,270 persons. Based on this population, as well as the proposed developable limits, the total peaked flow rate (including extraneous flow) from the site is 24.39 L/s. Refer to **Appendix F.4** for calculations.

The proposed internal sewers have been illustrated on **Figure 7.2**. The main sanitary sewer, located within Street A, will be sized to accommodate future phases, beyond Village A. It is noted that these phases will likely discharge to this gravity main via a future, twinned forcemain from a future sanitary pumping station located at the low point of the ultimate Base31 development area (at the southern end in Village H). An estimate of the total peaked sanitary flow rate from the future Villages is approximated at 230 L/s, based on the anticipated populations presented in the Master Servicing Concept submission. Refer to **Appendix F.4** for design sheets and **Appendix B.3** for the conceptual sanitary servicing plan for Base31 in its ultimate buildout.



The internal sanitary network will discharge to a new sanitary sewer installed along County Road #22, extending north to the existing 200 mm diameter sewer crossing the road, north of London Avenue, and south, extending to Kingsley Road. This main is to be sized to capture flows from any future anticipated development, as well as to provide an ultimate outlet for the Base31 Revitalization District, south of Village A.

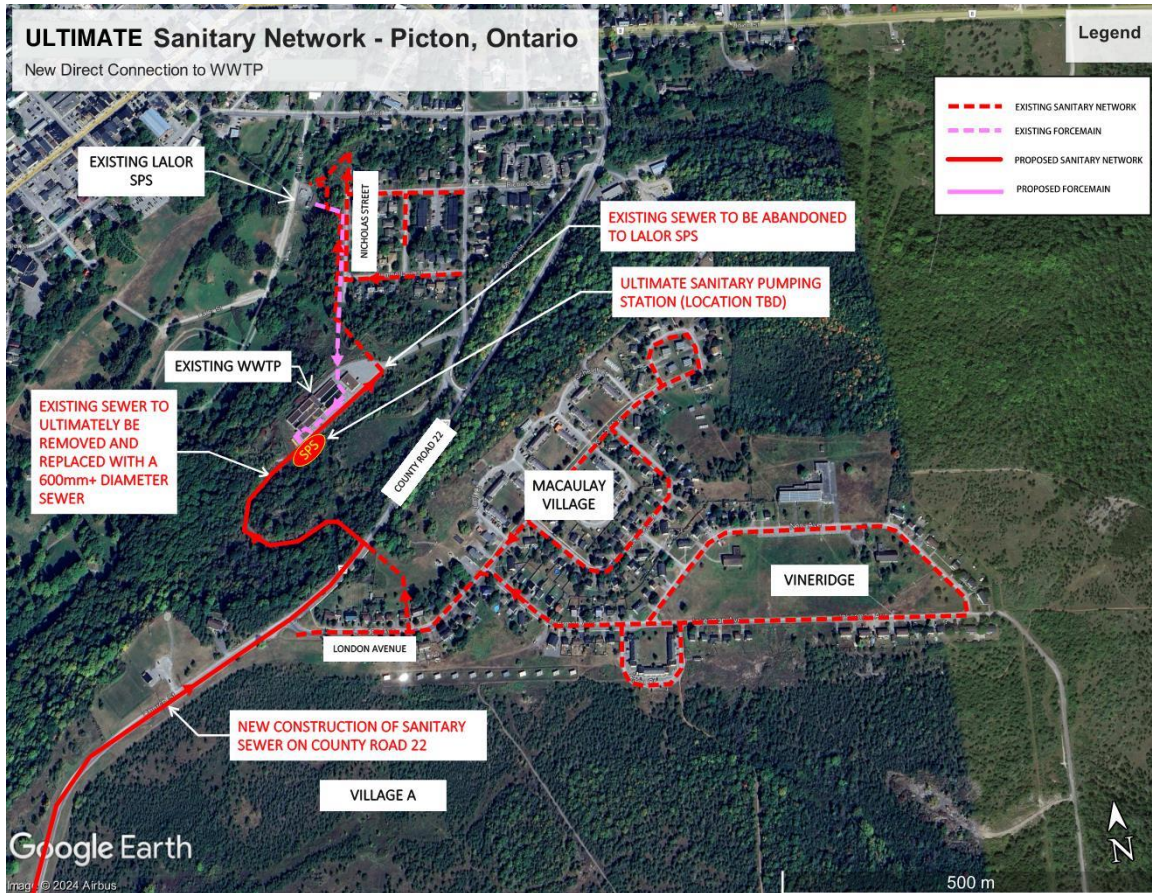
Each residential unit is to be supplied with a 125 mm diameter sanitary service lateral, as per Prince Edward County criteria. As noted above, the homes within Village A will be equipped with basements.

### **7.3 Proposed External Sanitary Sewer System**

As aforementioned, upgrades to the existing sanitary network, north of London Avenue will be required. It is understood that the County, per their MSP process, will dictate future expansion requirements to the WWTP. In order to utilize the remaining capacity within the WWTP, upgrades to the tributary system (i.e. the Lalor SPS and/or the gravity sewer), will ultimately be required. Upgrades to the WWTP will be considered in the future.

In the fullness of time, the County's preference to date is to install a new sanitary pumping station adjacent to the WWTP. Thereby, eliminating reliance on the Lalor SPS completely and providing a second inlet to the WWTP headworks. This pumping station will be ultimately sized to accommodate the entirety of Base31, Macaulay Village and all other anticipated development on the Heights. While the majority of the upfront installation efforts will be proposed within the wet well, components of this pumping station may be made modular, to expand with the growing population of the tributary population.

Beyond the installation of the new pumping station, the existing gravity sewer between County Road #22 and the new pumping station is to be ultimately upgraded when necessary to accommodate the overall future Base 31 and Height's growth. Refer to **Figure 7.3** for schematic of the ultimate solution.



**Figure 7.3: Ultimate Direct Connection to WWTP via New Sanitary Pumping Station**

While the ultimate solution will become necessary as additional flow is brought forth, interim solutions have been explored to bring on Village A, as well as the Rental Building (Building A), before triggering the requirement of the pumping station installation.

To service the immediate requirements for Village A, as well as the Rental Building (Building A) within the revitalization district, a phased approach to these upgrades has been explored which would allow the new pumping station installation to be phased to accommodate additional growth in the Heights. This would involve selective upgrades to a portion of the eighteen (18) existing sewer segments between MH25A on County Road #22 and the Lalor SPS. This could allow for the delayed installation of the sanitary pumping station until such time that the WWTP expansions are brought forth and/or additional flow rates warrant it.

**Existing Sewers East of County Road #22 to Lalor SPS (Surcharging Runs with Phase 1 of Village A)**

Using the above noted criteria and the site statics provided by the Draft Plan, Village A Phase 1 has an anticipated population of 580 people and a flow rate of 12.07 L/s (see **Appendix F.3**). To confirm the impact on the sewers east of County Road #22, the flow



of 12.07 L/s must be combined with the Phase 2 flow from the Revitalization District which is 5.78 L/s based on the upgraded pump within the private SPS which is required to accommodate the Rental Building. Therefore, the flow directed to the Lalor SPS will be increased by the sum of 12.07 and 5.78 L/s for a total of 17.85 L/s. This flow is less than the Lalor SPS capacity of 37 L/s as noted in **Section 7.1** above. Furthermore, in regard to surcharging, the model was run by adding the proposed flow of 17.85 L/s to the existing 34.0 L/s for a total of 51.85 L/s and it was confirmed that, in addition to the 4 runs surcharging under existing conditions, one additional run would surcharge. Therefore, the existing 200 mm diameter sewers are proposed to be replaced with 600 mm diameter sewers, where possible and as previously proposed, a 600 mm diameter sewer is proposed to be added above the existing 200 mm diameter sewer where the existing sewer is adjacent to the existing Rental Building, to avoid the need to excavate or break rock close to or below the building foundation. See **Figures 7.3.1** and **7.3.2** for the plan and profile views respectively which illustrate that there are 5 surcharging runs between County Road #22 and the Lalor SPS.

#### **Existing Sewers East of CR 22 to Lalor SPS (Surcharging Runs with all of Village A)**

In the ultimate scenario when all of Village A is developed, the total population will be 1,270 people which generates a flow rate of 24.39 L/s (see **Appendix F.4**). To confirm the impact on the sewers east of County Road #22, the flow of 24.39 L/s must be combined with the ultimate flow from the Revitalization District which is 10 L/s. Therefore, the flow directed to the Lalor SPS will be increased by the sum of 24.39 and 10 L/s for a total of 34.39 L/s. This flow is less than the Lalor SPS capacity of 37 L/s as noted in **Section 7.1** above. Furthermore, in regard to surcharging, the model was run by adding the proposed flow of 34.39 L/s to the existing 34.0 L/s for a total of 68.39 L/s and it was confirmed that, in addition to the 5 runs surcharging under the Phase 1 scenario, one additional run would surcharge. The existing 200 mm diameter sewers are proposed to be replaced with 600 mm diameter sewers, where possible and as previously proposed, a 600 mm diameter sewer is proposed to be added above the existing 200 mm diameter sewer where the existing sewer is adjacent to the existing Rental Building, to avoid the need to excavate or break rock close to or below the building foundation. See **Figures 7.3.3** and **7.3.4** for the plan and profile views respectively which illustrate that there are 6 surcharging runs between County Road #22 and the Lalor SPS.

Additional interim servicing scenarios with phased growth on the Heights can be assessed at the detail design stage to optimize and phase the proposed interim gravity sewer modifications.

As additional flow is brought on-stream, further segments of the sewer will need to be upsized. However, due to the imminent design and construction of the sanitary pumping station, they have been deferred at this time.



Village A and the Rental Building (Building A) will continue to rely on the Lalor SPS and ultimately be treated in the WWTP. Until such time that the WWTP is expanded, it is understood that wastewater servicing allocation will be distributed based on the remaining capacity, at the discretion of the County.



## 8.0 Water Supply and Distribution

### 8.1 Existing Water Distribution

The subject lands are within the Picton Heights/Macaulay Village pressure zone, which is supplied by the Picton Reservoir and Macaulay Village Booster Pumping Station. The lands north of the subject site (i.e. the Heights/Macaulay Village), as well as the initial servicing of the Base31 Revitalization District are currently serviced from this pressure zone. The entire Picton area, and the community of Bloomfield are serviced by the Picton Water Treatment Plant (WTP) located within the Picton Bay. The Picton Reservoir also services the remaining Picton community by gravity, which is at a lower pressure zone than the Heights.

From the WTP, there is an existing 350 mm diameter feeder main that supplies water to the Picton Heights Reservoir (constructed in 2019) and Macaulay Village Booster Pumping Station (BPS). The Heights BPS, which is located on the west side of County Road #22, north of Kingsley Road, currently feeds the existing Macaulay Village. In addition, a new 300 mm diameter feed has been recently installed from this BPS to service the Base31 Revitalization District. Refer to **Appendix B.2** for detailed subsurface investigation of the existing watermain as well as design drawings for the revitalization district.

The existing Picton Heights Reservoir has a total volume of 4,550 m<sup>3</sup>. Additional storage facilities in the Picton Water System include a 1,345 m<sup>3</sup> reservoir at the Picton WTP. Therefore, a total of 5,895 m<sup>3</sup> of storage is available. The existing BPS has four 12 L/s pumps and one 82 L/s fire pump. Refer to the design report for the existing BPS in **Appendix G.1**.

As per the Environmental Study Report, Municipal Class Environmental Assessment prepared by CIMA+, dated June 2024, as well per as the August 27, 2024 Council Meeting, a Regional Water Treatment Plant has been approved in the community of Wellington. This Regional plant is intended to service both the growing communities of Wellington as well as Picton and Bloomfield. Phase 1 of the expansion is expected to reach a service capacity of 22,000 m<sup>3</sup>/day (or 29,800 people) by 2047 – including 21,200 people from the communities of Picton/Bloomfield. A transmission main will be required to supply water to the Picton/Bloomfield the service area. It is understood the preferred alignment for the transmission main will be identified as part of the upcoming Picton Master Servicing Plan report. Once ultimately constructed, the Wellington Plant is intended to provide service for the ultimate Base31 site. However, until such time, reliance to the Picton WTP is required.

Provision of water servicing allocation based on the remaining existing water system capacity is at the discretion of PEC.



The area occupied by Village A does not currently have an active watermain connection. An existing abandoned watermain bisects the site. As noted in **Section 7.1**, this watermain is to be removed. In addition, a watermain currently servicing Macaulay Village currently traverses the subject site at the north-west corner. It is understood that this watermain will be relocated to County Road #22 prior to the proposed Village A works. A subsurface utility investigation (SUE) was completed for the entire Village A area, including areas surrounding the BPS. The SUE is found in **Appendix B.2**.

## 8.2 Proposed Water System

Village A will be initially supplied by the existing Picton WTP via a connection to the existing BPS. A connection will be made to the recently constructed 300 mm diameter watermain installed for the Revitalization District. The proposed layout for the proposed watermain system is provided on **Figure 8.1**.

In accordance with the current version of the Prince Edward County and MECP design guidelines, the water distribution system will be designed based on the following criteria:

- Average Day Demand: 320 L/c/d
- Residential Population Density:
  - Low Density - Single detached, semi-detached or duplexes- 25 units/ha or 3.0 persons per unit (PPU)
  - Medium Density – Triplexes, fourplexes, townhouses – 26-53 units/ha or 2.5 PPU.
  - High Density – apartment Dwellings – 54-300 units/ha at the following PPU:
    - Bachelor/1 Bedroom – 1.4 PPU
    - 2 Bedroom – 2.1 PPU
    - 3 Bedroom – 2.4 PPU
    - 4 Bedroom – 3.4 PPU
    - Bedroom Unknown – 1.7 PPU
- Fire Flow Demands – Fire Underwriter’s Survey Document Water Supply for Public Fire Protection (FUS)

Development applications are currently on-going north of the subject site (i.e. Vineridge) as well as within the Base31 site itself (Building A – a 120-unit rental building within the Revitalization District). Detailed design of Building A is proceeding concurrently with this application, under separate cover.

DLW Engineering Services Ltd., has been retained to analyze the existing BPS and storage requirements for the Base31 works – including both Village A, as well as the rental unit in the revitalization district. Refer to the “Water Servicing Memorandum”, prepared by DLW Engineering Services Ltd., dated October 29, 2024, in **Appendix G.2**.



As detailed in the report, the existing serviced population of Picton is 4,576 people. Combined with current completed/under construction development, this increases the population by an additional 2,514 persons (as per email confirmation from the County February 2024). Therefore, combined with the current development within Base31, which includes Phase 1 of the Revitalization District, Building A in the revitalization district, as well as Village A, a combined population of 9,524 people is anticipated.

Based on this combined 9,524-person population, the total water demands for Picton, based on MECP design guidelines, are as follows:

- Maximum Day Demand (Maximum Day Factor of 2.0): 6,760 m<sup>3</sup>/day (77.3 L/s)
- Fire Flow Demand (Based on 3 hours): 184 L/s

Therefore, the required storage for the total development within the Heights is 4,598 m<sup>3</sup>. We acknowledge the County's preference to consider the capacity to be limited to 4,550 m<sup>3</sup> since it is used for chlorine contact purposes however, the shortfall is insignificant and should be able to be absorbed given that the reservoir was designed with emergency storage volume included. Refer to DLW report for detailed calculations. This is within the total storage available of 5,895 m<sup>3</sup>. Therefore, there is sufficient storage to satisfy the water demands of Village A and the rental unit. As Base31 continues to develop and densify, additional storage, by the use of an above ground or in-ground reservoir will ultimately be required.

Further detailed in the DLW report are the upgrades required at the Picton Heights BPS to facilitate Village A and the rental unit. As outlined in the report, no modifications are required to the four low-flow pumps at this stage, as the maximum day design flows will be 23.2 L/s. However, modifications are required to the fire pump to achieve fire demands. Subject to principals of cost sharing with the neighboring development VineRidge, the following upgrades are recommended to the existing Picton heights BPS:

- Replace the existing 82 L/s fire pump with one new 131.1 L/s fire pump or a 163.1 L/s Maximum Day plus Fire Flow pump.

Upgrading this station will require a new stand-by generator to meet the new electrical loads. These upgrades may result in the replacement of the pump starters as well as the inclusion of pressure tanks to maintain pressure in the system during extreme low flows, when pump operation is not possible.

Water modeling, which confirms the watermain size requirements throughout Village A, as well as the pressure impacts to the existing system, can be found in **Appendix G.3**. Furthermore, DLW's memorandum confirming there is sufficient capacity within the existing infrastructure, pending modifications outlined above to the BPS is located in **Appendix G.2**.



### 8.2.1 Modeling of Additional Scenarios

Through discussions and consultation with the County, it was determined that it would be beneficial to model additional scenarios to improve the understanding of the existing system and how the increased demands would impact the existing system. Based on this, the following scenarios were analyzed:

- Scenario 0 - MacCauley Village development
- Scenario 1 - MacCauley Village + New Interim Historic District + Proposed Rental Unit
- Scenario 2 - MacCauley Village + New Interim Historic District + Proposed Rental Unit + Proposed Village A
- Scenario 3 - MacCauley Village + New Interim Historic District + Proposed Rental Unit + Proposed Village A + Proposed VineRidge development

The results of these different modelling scenarios are outlined in a memo prepared by DLW Engineering dated June 6, 2025, which summarizes the more detailed modelling output produced by RA Engineering. This memo and supporting model outputs are provided in **Appendix G.2**, and have been reviewed by the County. In summary, the analyses concluded that when fire flow demands were placed within Base31, there were no appreciable impacts to pressures within MacCauley Village.

Following the County’s review of the various water modelling scenarios prepared by DLW Engineering and RA Engineering, interest was expressed in obtaining field data to confirm the findings of the modelling. A Flow Test Procedure (see **Appendix G.4**) was developed and agreed to by all parties, and field tests were executed on July 8, 2025. The purpose of the flow test was to test the capability of the Picton Booster Station flow output to the water system with the new Base31 private water system, to determine the limits that are possible with the current arrangement. The results of the Flow Test Procedure are summarized in a memo prepared by DLW Engineering dated August 19, 2025 (see **Appendix G.5**). In summary, the onsite field testing confirmed the results determined by the modeling work previously done, in that when fire flow demands are placed within Base31, there were no appreciable impacts to pressures within MacCaulay Village. As previously mentioned, allocation at the WTP is determined by the municipality in accordance with EV 800.

## 9.0 Utility Considerations

Callidus Engineering, the utility Engineers on file, have prepared the following report in support of the Master Servicing Concept, previously submitted by SCS Consulting Group Ltd.:

- Base31: Electrical Needs Study, prepared by Callidus Engineering, dated August 4, 2023.

The purpose of this report is to provide a high-level analysis of the electrical service required for the site. Currently, Callidus is in conversation with Hydro One and other utilities (gas, bell and cable) to determine a servicing plan for this development.

It is understood that all new utilities (gas, bell and cable) shall be buried to protect against winter conditions and freezing, as shown on the cross sections provided in **Appendix H**. Existing utilities shall remain in their current state.

## **10.0 Erosion and Sediment Control During Construction**

During the detailed design stage, erosion and sediment control measures will be designed with a focus on erosion control practices (such as stabilization, track walking, staged earthworks, etc.) as well as sediment controls (such as fencing, mud mats, catchbasin sediment control devices, check dams and temporary sediment control ponds). These measures will be designed and constructed as per the “Erosion and Sediment Control Guideline for Urban Construction” document (December 2019). A detailed erosion and sediment control plan will be prepared for review and approval by the Municipality and Conservation Authority prior to any site grading being undertaken. This plan will address phasing, inspection and monitoring aspects of erosion and sediment control. All reasonable measures will be taken to ensure sediment loading to the adjacent properties and drainage features are minimized both during and following construction.

## 11.0 Summary

This Functional Servicing and Stormwater Management Report has been prepared in support of the Draft Plan of Subdivision applications for Village A of the Base31 development in the community of Picton, Prince Edward County. This report outlines the means by which the proposed development can be graded and serviced in accordance with the Prince Edward County, Quinte Conservation Authority, MTO, and the Ministry of Environment, Conservation and Parks design criteria and policies.

### Grading

- The proposed development grading has been developed to match to the existing surrounding grades, and provide conveyance of stormwater runoff, including external drainage; and
- The site grading will be subject to further grading design at the detailed design stage.

### Rights-of-Way and Sidewalks

- Right-of-way cross sections have been developed between SCS, TYLin, Biglieri Group and NAK Design Group
- These sections were reviewed by the County and the enclosed cross sections reflect the County's comments.

### Storm Drainage and Servicing

- Storm Servicing:
  - Storm runoff will be conveyed by storm sewers designed in accordance with Municipality and MECP criteria;
  - Storm sewers will generally be designed for the 5 year storm event; and
  - Adequate 100 year overland flow routes will be provided.
- Existing external drainage will be accommodated through the proposed development via a municipal storm sewer.

### Stormwater Management

- Quality Control: MOE (2003) Enhanced water quality protection can be provided through the use of two wetland SWM ponds;
- Erosion Control: The runoff volume from a 25 mm rainfall event will be detained over 24 hours by two wetland SWM ponds;
- Quantity Control: Quantity control will be provided via two wetland SWM ponds to control proposed runoff rates in the 2 through 100 year storm events;
- Water Budget: Infiltration is not proposed due to the bedrock on-site.



### **Sanitary Servicing**

- Sanitary connection will ultimately be provided to the existing Wastewater Treatment Plant, northwest of the subject site.
- Modifications to the existing 200 mm diameter tributary sewer are required for Village A to proceed.
- Sanitary allocation at the WWTP is required by the County.

### **Water Supply and Distribution**

- Water supply will be provided through a 300 mm diameter connection to the existing Booster Pumping Station.
- DLW Engineering has prepared an assessment of the existing system and has determined that there is sufficient storage provided in the Height's reservoir and storage facilities at the Picton WTP.
- It is recommended that fire pump modifications be implemented within the Booster Pumping Station to achieve maximum day and fire flow demands.
- Water supply allocation is required from the County from the Picton WTP.

### **Utility Considerations**

- Callidus Engineering is currently undertaking an assessment of the electrical requirements for Village A. Coordination with HydroOne is on-going to confirm capacity.

### **Erosion and Sediment Control during Construction**

- An erosion and sediment control plan will be prepared at the detailed engineering stage, in accordance with the "Erosion and Sediment Control Guideline for Urban Construction" document (December 2019).

**Respectfully Submitted:**

**SCS Consulting Group Ltd.**



Ryan Brockie, P.Eng,  
rbrockie@scsconsultinggroup.com



Jill Stark for,

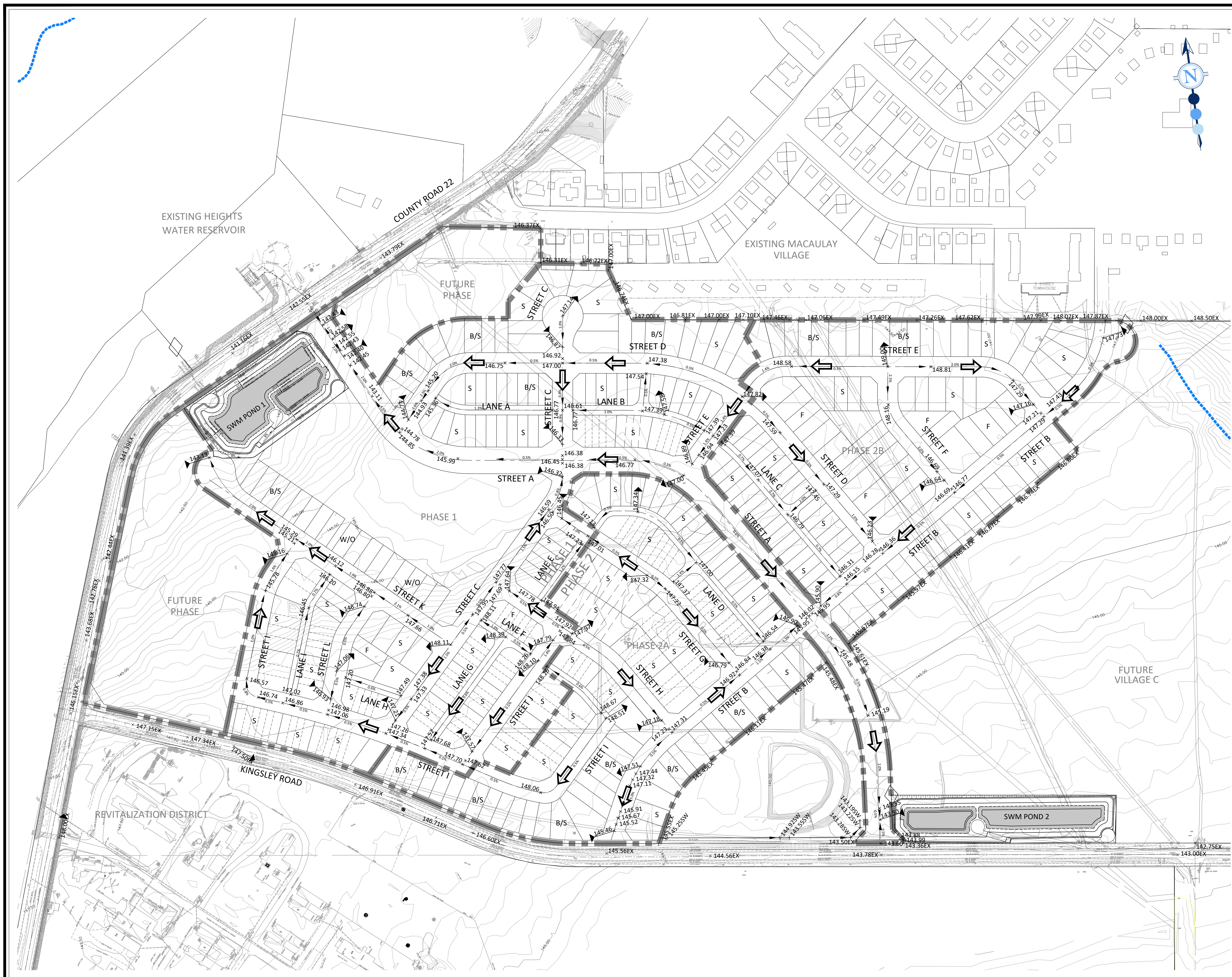
Cosimo Costa, P.Eng  
ccosta@scsconsultinggroup.com

P:\2365 Picton Airport - Tercot\Design\Reports\FSSR\Village A\2365 - Village A FSSR.docx

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## Figures

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**LEGEND:**

	LIMIT OF DEVELOPMENT
	LIMIT OF PHASE
	DRAINAGE FEATURE (PALMER, 2023)
	PROPOSED ELEVATION
	PROPOSED HIGH POINT
	PROPOSED LOW POINT
	PROPOSED SLOPE
	OVERLAND FLOW
	PROPOSED LOT TYPE (FRONT DRAIN)
	PROPOSED LOT TYPE (SPLIT DRAIN)
	PROPOSED LOT TYPE (BACK SPLIT)
	EXISTING ELEVATION
	EXISTING CONTOUR AND ELEVATION

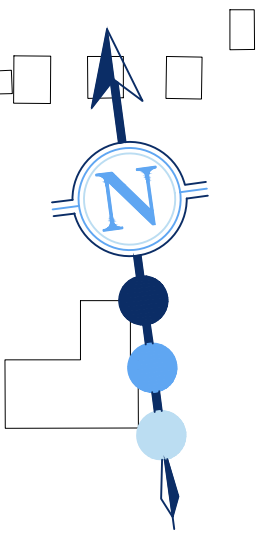
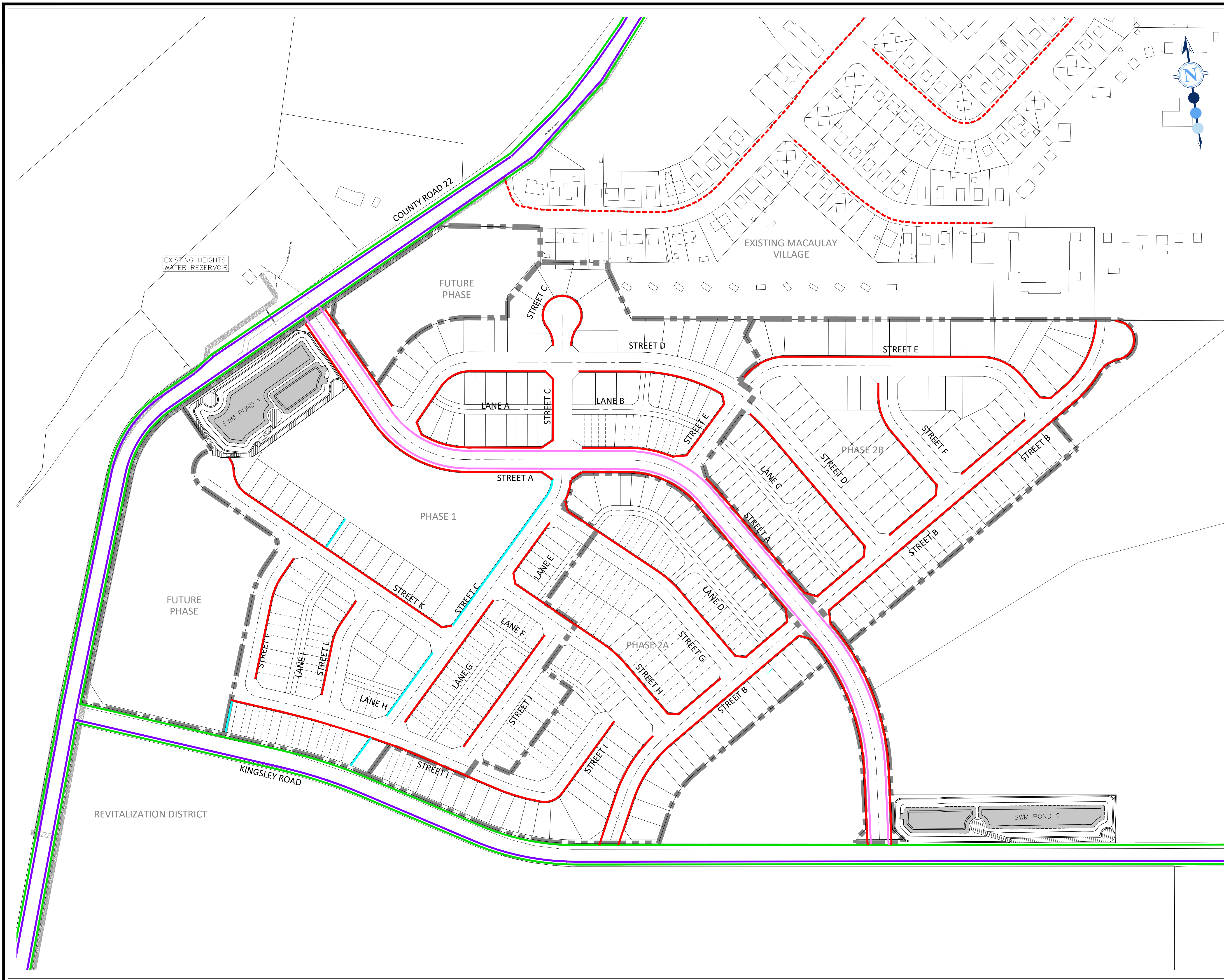
\*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

**SGS consulting group ltd**  
 30 CENTURIAN DRIVE, SUITE 100  
 MARKHAM, ONTARIO L3R 8B8  
 TEL: (905) 475-1900  
 FAX: (905) 475-8335

**PICTON AIRPORT - PRINCE EDWARD COUNTY  
 COMMUNITY PARTNERS INC.**

**PRELIMINARY GRADING PLAN**

DESIGNED BY: J.S.S.	CHECKED BY: C.C.
SCALE: 1:1500	DATE: AUGUST 2025
PROJECT No: 2365	FIGURE No: 3.1

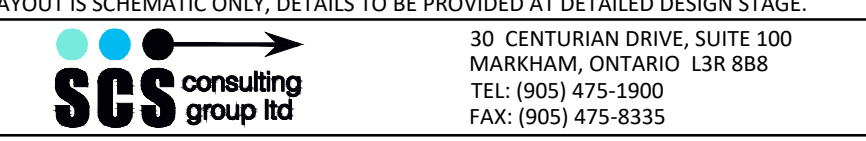


**LEGEND:**

	LIMIT OF DEVELOPMENT
	LIMIT OF PHASE
	EXISTING SIDEWALK
	PROPOSED 1.5m SIDEWALK
	PROPOSED 1.5m ASPHALT BIKE LANE
	PROPOSED 3.0m ASPHALT MULTI-USE PATH
	FUTURE SIDEWALK
	FUTURE 1.5m ASPHALT BIKE LANE

\*NOTE: BASED ON COMMENTS ON CROSS SECTIONS DATED JULY 29, 2025 (SEE APPENDIX C.3), SIDEWALKS ARE NOT REQUIRED ON BOTH SIDES OF THE ROW AND AROUND CUL-DE-SACS. THIS WILL BE CONFIRMED AT DETAILED DESIGN.

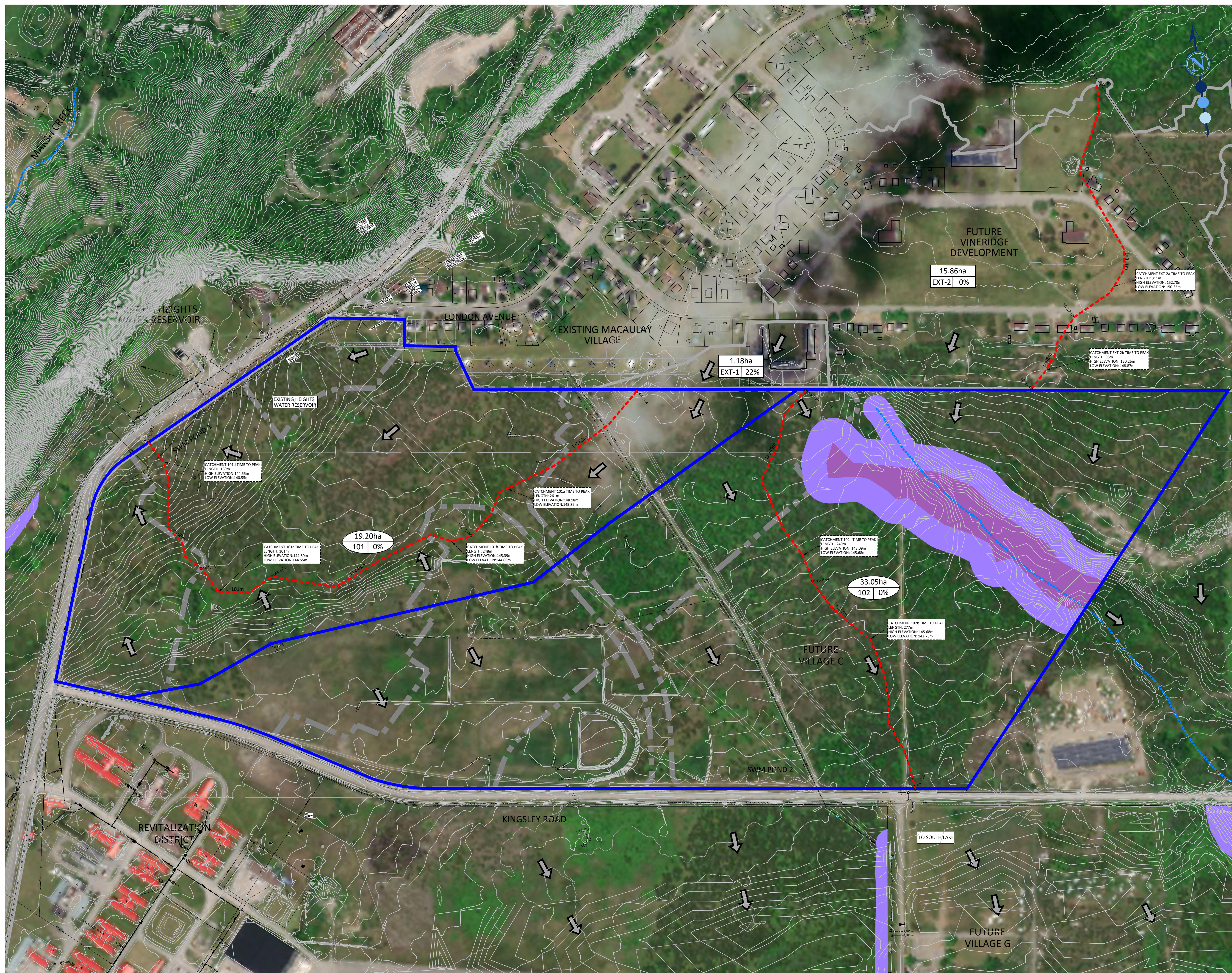
\*NOTE: LAYOUT IS SCHEMATIC ONLY. DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.



**PICTON AIRPORT - PRINCE EDWARD COUNTY COMMUNITY PARTNERS INC.**

**SIDEWALK PLAN**

DESIGNED BY: J.S.S.	CHECKED BY: C.C.
SCALE: 1:1500	DATE: AUGUST 2025
PROJECT No: 2365	FIGURE No: 4.1



**LEGEND:**

- LIMIT OF DEVELOPMENT
- DRAINAGE FEATURE (PALMER, 2023)
- HIGH CONSTRAINT AREA (PALMER ENGINEERING, 2023)
- HIGH CONSTRAINT SETBACK (PALMER ENGINEERING, 2023)
- OVERLAND FLOW DIRECTION
- STORM DRAINAGE BOUNDARY
- EXTERNAL STORM DRAINAGE BOUNDARY
- L=XXXm TIME OF PEAK FLOW PATH AND LENGTH

15.86ha EXT-2   22%	EXTERNAL STORM DRAINAGE AREA (HECTARES)
	PERCENT (%) IMPERVIOUS
	CATCHMENT ID

19.20ha 101   0%	DRAINAGE AREA (HECTARES)
	PERCENT (%) IMPERVIOUS
	CATCHMENT ID

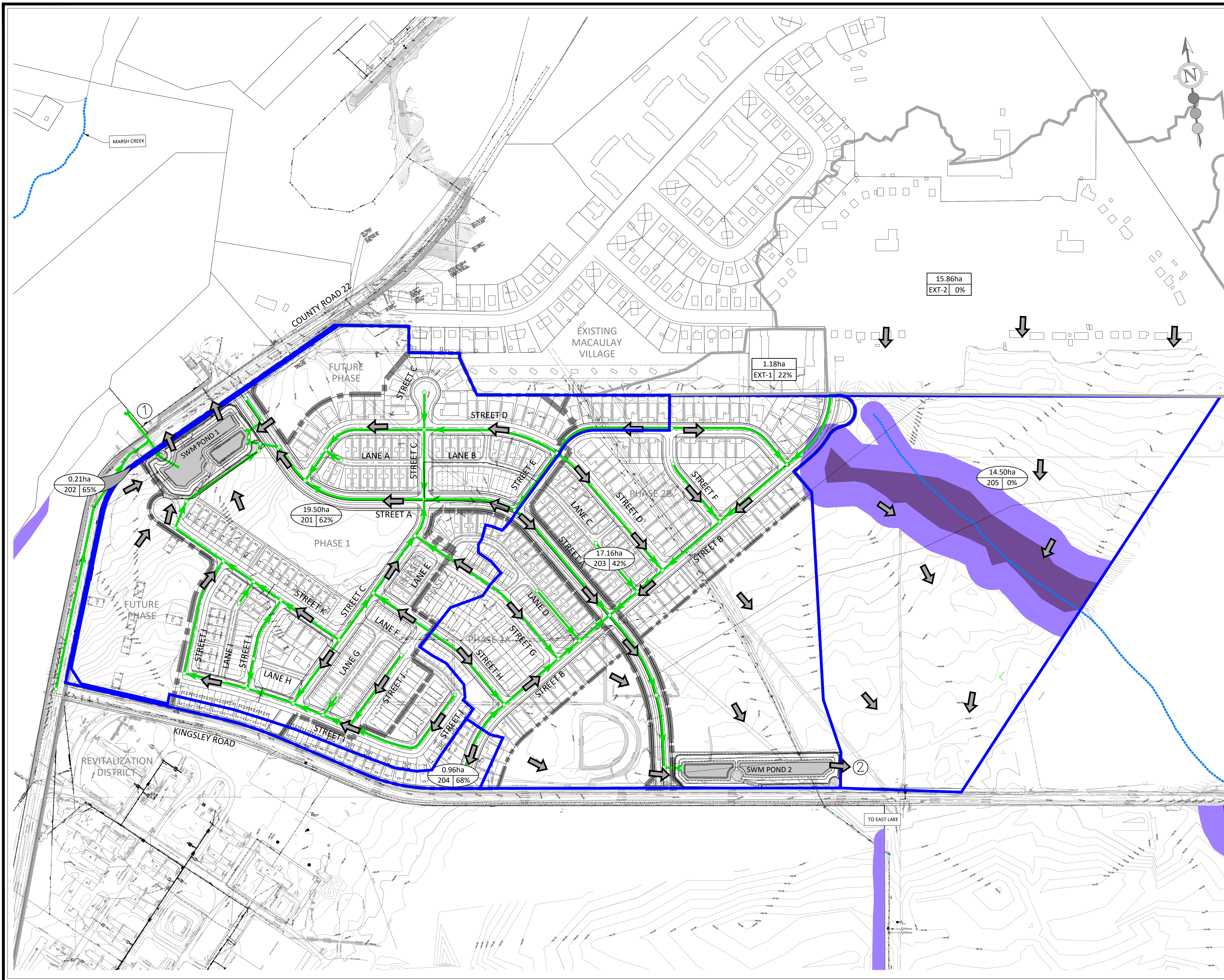
**SGS consulting group ltd**

30 CENTURIAN DRIVE, SUITE 100  
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 FAX: (905) 475-8335

**PICTON AIRPORT - PRINCE EDWARD COUNTY COMMUNITY PARTNERS INC.**

**EXISTING STORM DRAINAGE PLAN**

DESIGNED BY: R.B.	CHECKED BY: A.R.K.
SCALE: 1:2000	DATE: AUGUST 2025
PROJECT No: 2365	FIGURE No: 5.1



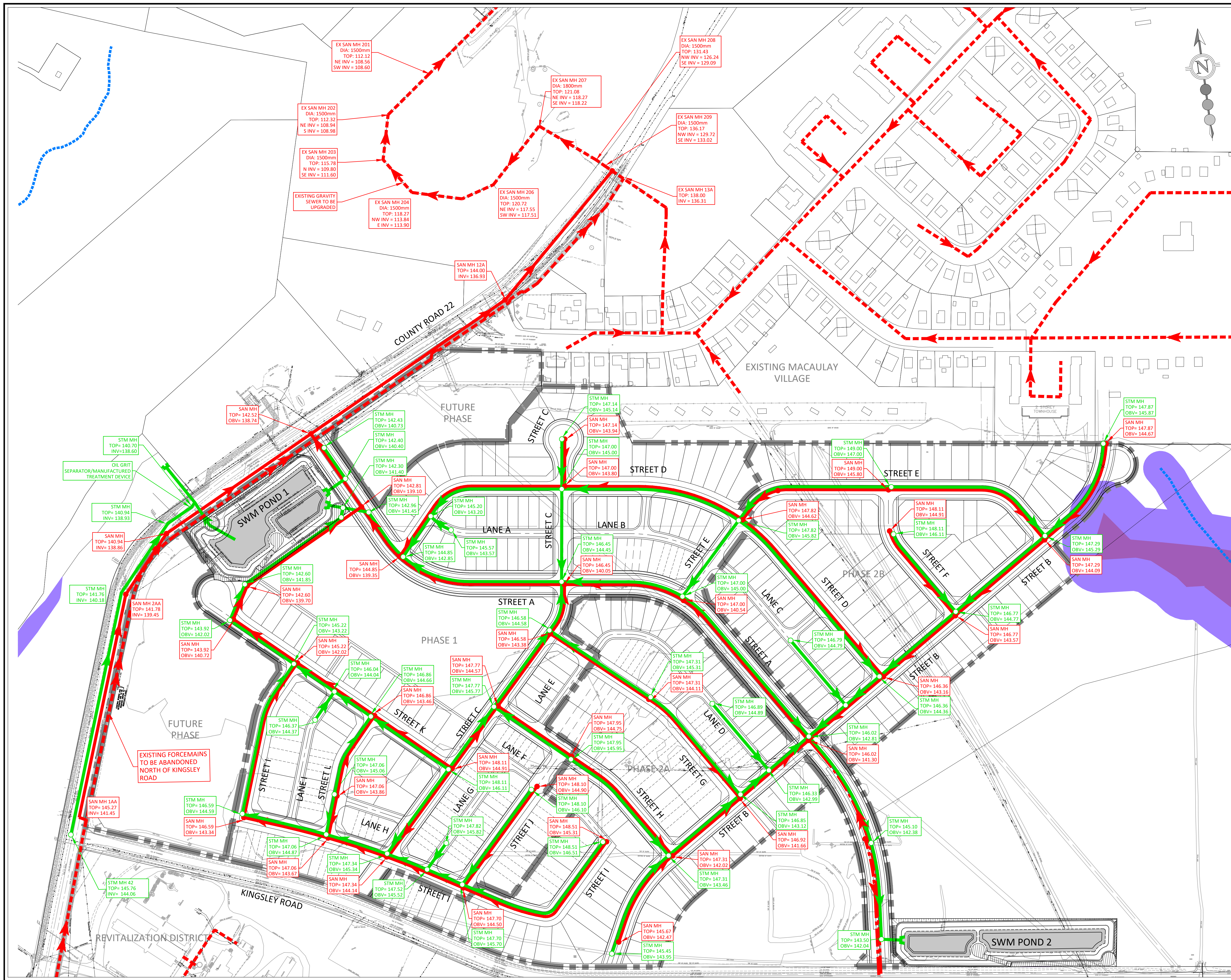
**LEGEND:**

- LIMIT OF DEVELOPMENT
- LIMIT OF PHASE
- DRAINAGE FEATURE (PALMER, 2023)
- HIGH CONSTRAINT AREA (PALMER ENGINEERING, 2023)
- HIGH CONSTRAINT SETBACK (PALMER ENGINEERING, 2023)
- PROPOSED DRAINAGE BOUNDARY
- EXTERNAL DRAINAGE BOUNDARY
- PROPOSED STORM SEWER AND MAINTENANCE HOLE
- EXISTING STORM SEWER AND MAINTENANCE HOLE
- OVERLAND FLOW DIRECTION
- DRAINAGE AREA (HECTARES)  
PERCENT (%) IMPERVIOUS  
CATCHMENT ID
- EXTERNAL STORM DRAINAGE AREA (HECTARES)  
PERCENT (%) IMPERVIOUS  
CATCHMENT ID
- OUTLET ID

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 30 CENTURIAN DRIVE, SUITE 100  
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**PICTON AIRPORT - PRINCE EDWARD COUNTY  
 COMMUNITY PARTNERS INC.**  
**PROPOSED STORM DRAINAGE PLAN**

DESIGNED BY: R.B.	CHECKED BY: A.R.K.
SCALE: 1:2000	DATE: AUGUST 2025
PROJECT No: 2365	FIGURE No: 5.2



- LEGEND:**
- LIMIT OF DEVELOPMENT
  - LIMIT OF PHASE
  - DRAINAGE FEATURE (PALMER, 2023)
  - HIGH CONSTRAINT AREA (PALMER ENGINEERING, 2023)
  - HIGH CONSTRAINT SETBACK (PALMER ENGINEERING, 2023)
  - PROPOSED STORM SEWER AND MAINTENANCE HOLE
  - EXISTING STORM SEWER AND MAINTENANCE HOLE
  - PROPOSED SANITARY SEWER AND MAINTENANCE HOLE
  - EXISTING SANITARY SEWER
  - EXISTING SANITARY FORCEMAIN
  - FUTURE SANITARY FORCEMAIN
  - FUTURE SANITARY SEWER

\*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

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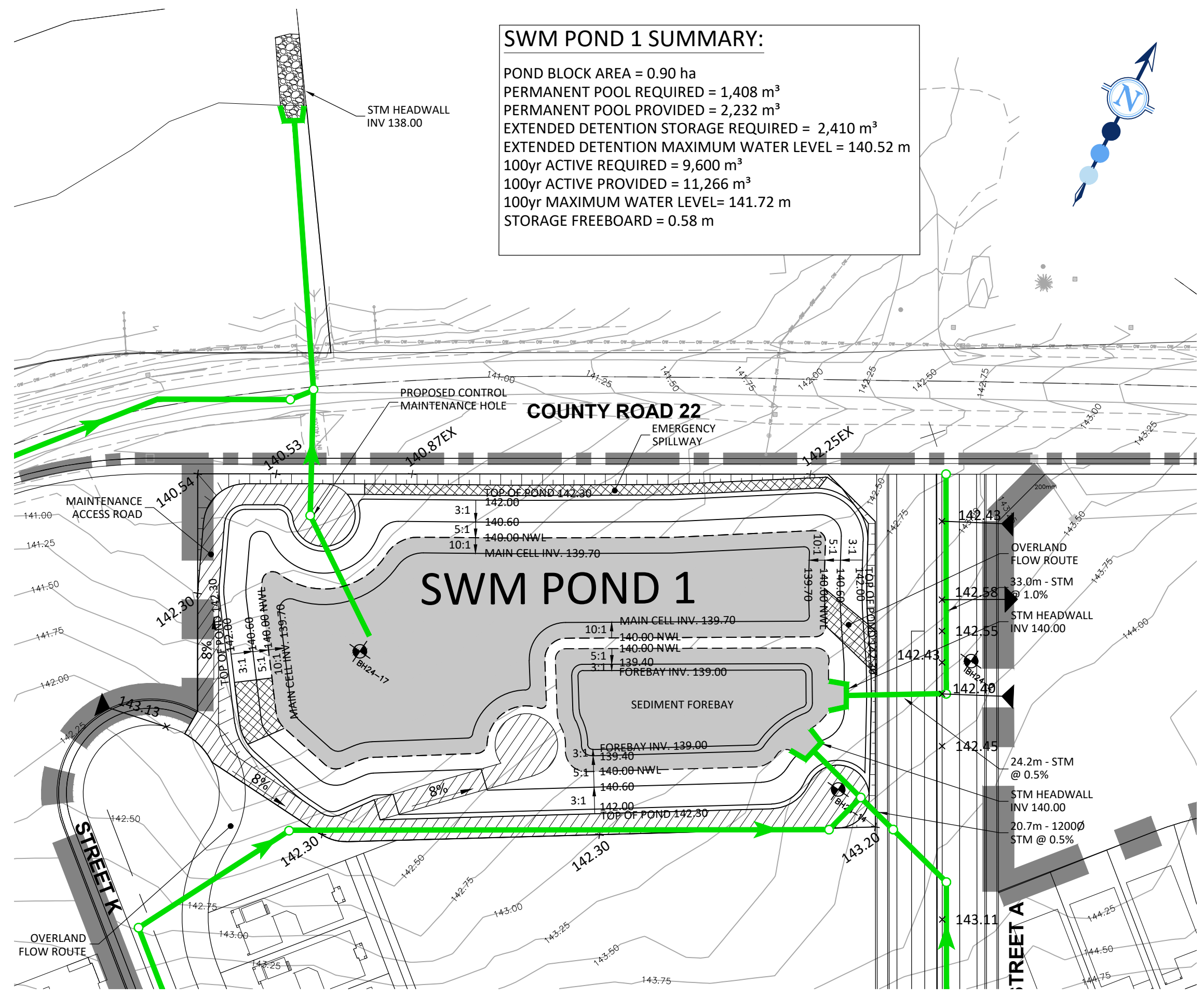
**PICTON AIRPORT - PRINCE EDWARD COUNTY COMMUNITY PARTNERS INC.**

**PRELIMINARY SERVICING PLAN**



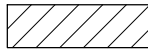


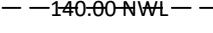
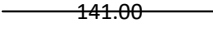
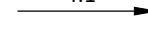


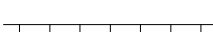

DESIGNED BY: J.S.S.	CHECKED BY: C.C.
SCALE: 1:1500	DATE: AUGUST 2025
PROJECT No: 2365	FIGURE No: 5.3

**SWM POND 1 SUMMARY:**

POND BLOCK AREA = 0.90 ha  
 PERMANENT POOL REQUIRED = 1,408 m<sup>3</sup>  
 PERMANENT POOL PROVIDED = 2,232 m<sup>3</sup>  
 EXTENDED DETENTION STORAGE REQUIRED = 2,410 m<sup>3</sup>  
 EXTENDED DETENTION MAXIMUM WATER LEVEL = 140.52 m  
 100yr ACTIVE REQUIRED = 9,600 m<sup>3</sup>  
 100yr ACTIVE PROVIDED = 11,266 m<sup>3</sup>  
 100yr MAXIMUM WATER LEVEL = 141.72 m  
 STORAGE FREEBOARD = 0.58 m



**LEGEND:**

-  LIMIT OF DEVELOPMENT
-  PROPOSED STORM SEWER AND MAINTENANCE HOLE
-  MAINTENANCE ACCESS ROAD
-  NORMAL WATER LEVEL
-  EMERGENCY SPILLWAY/ OVERLAND FLOW ROUTE
-  140.00-NWL
-  141.00
-  4:1
-  x 142.43
-  x 142.25EX
-  142.00
-  3:1 SLOPING

\*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

 30 CENTURIAN DRIVE, SUITE 100  
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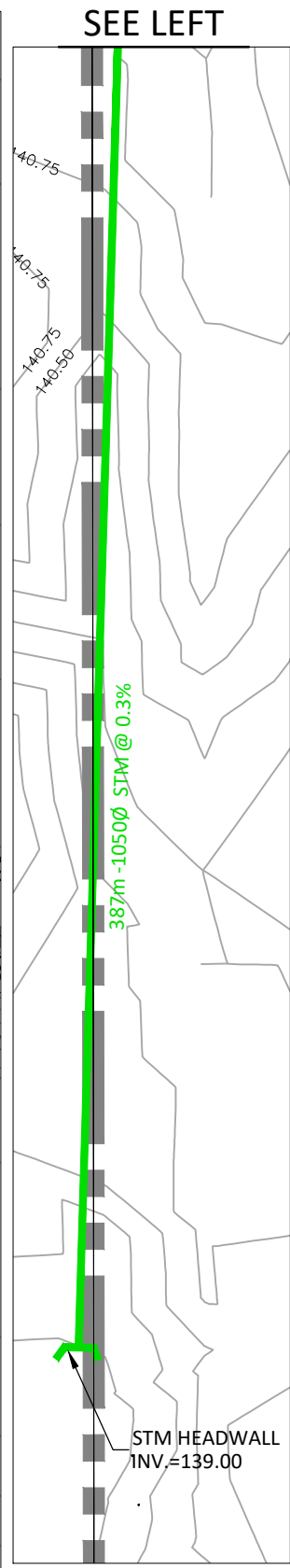
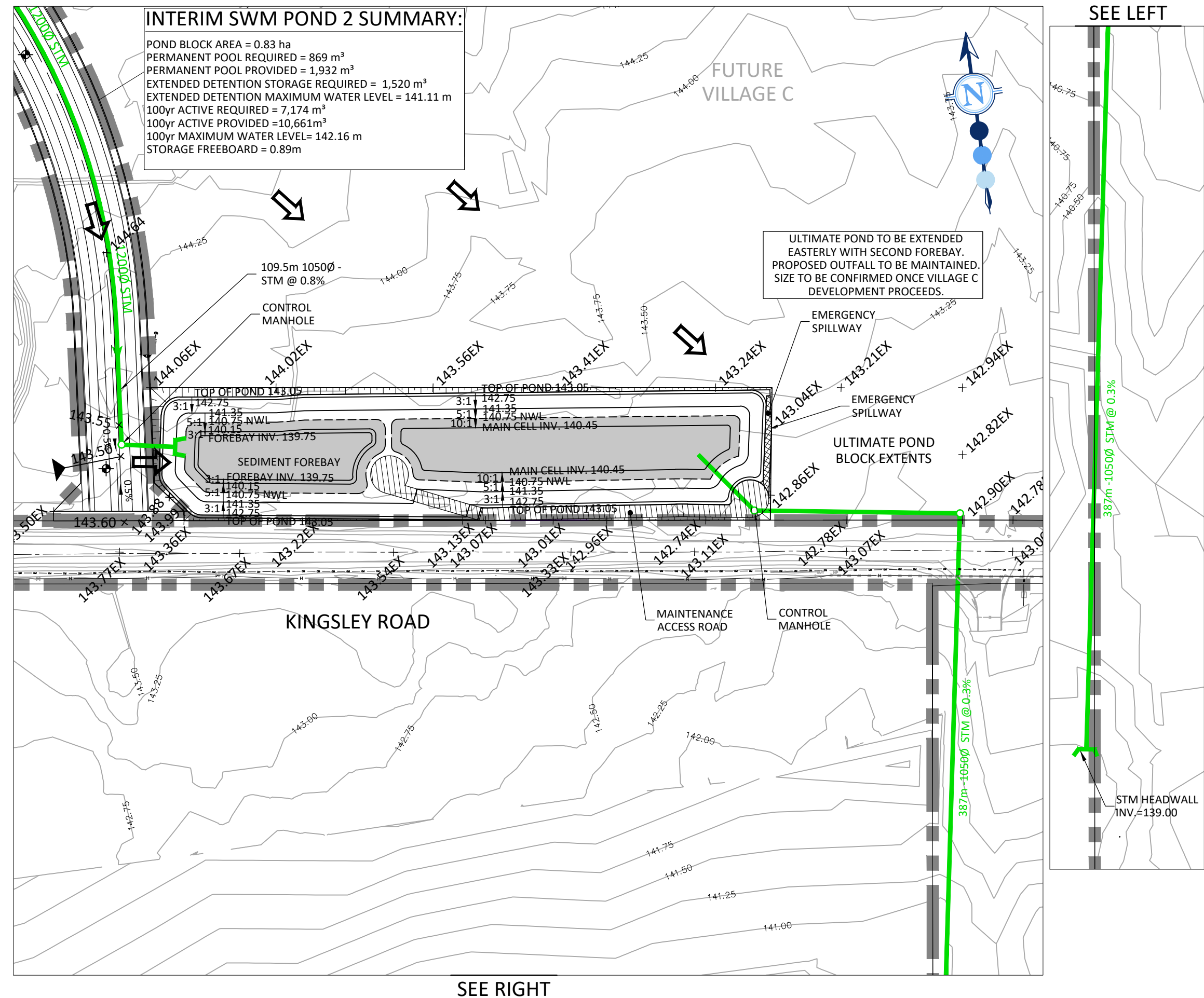
**PROPOSED SWM POND 1**

DESIGNED BY:	R.B.	CHECKED BY:	A.R.K.
SCALE:	1:750	DATE:	AUGUST 2025
PROJECT No:	<b>2365A</b>	FIGURE No:	<b>6.1</b>

**INTERIM SWM POND 2 SUMMARY:**

POND BLOCK AREA = 0.83 ha  
 PERMANENT POOL REQUIRED = 869 m<sup>3</sup>  
 PERMANENT POOL PROVIDED = 1,932 m<sup>3</sup>  
 EXTENDED DETENTION STORAGE REQUIRED = 1,520 m<sup>3</sup>  
 EXTENDED DETENTION MAXIMUM WATER LEVEL = 141.11 m  
 100yr ACTIVE REQUIRED = 7,174 m<sup>3</sup>  
 100yr ACTIVE PROVIDED = 10,661 m<sup>3</sup>  
 100yr MAXIMUM WATER LEVEL = 142.16 m  
 STORAGE FREEBOARD = 0.89m

ULTIMATE POND TO BE EXTENDED  
 EASTERLY WITH SECOND FOREBAY.  
 PROPOSED OUTFALL TO BE MAINTAINED.  
 SIZE TO BE CONFIRMED ONCE VILLAGE C  
 DEVELOPMENT PROCEEDS.



**LEGEND:**

- LIMIT OF DEVELOPMENT
- LIMIT OF PHASE
- PROPOSED STORM SEWER AND MAINTENANCE HOLE
- MAINTENANCE ACCESS ROAD
- NORMAL WATER LEVEL
- EMERGENCY SPILLWAY/OVERLAND FLOW ROUTE
- NORMAL WATER LEVEL CONTOUR AND ELEVATION
- PROPOSED ULTIMATE CONTOUR AND ELEVATION
- PROPOSED INTERIM CONTOUR AND ELEVATION
- PROPOSED POND SLOPE
- EXISTING CONTOUR AND ELEVATION
- 3:1 SLOPING
- MAJOR SYSTEM - OVERLAND FLOW

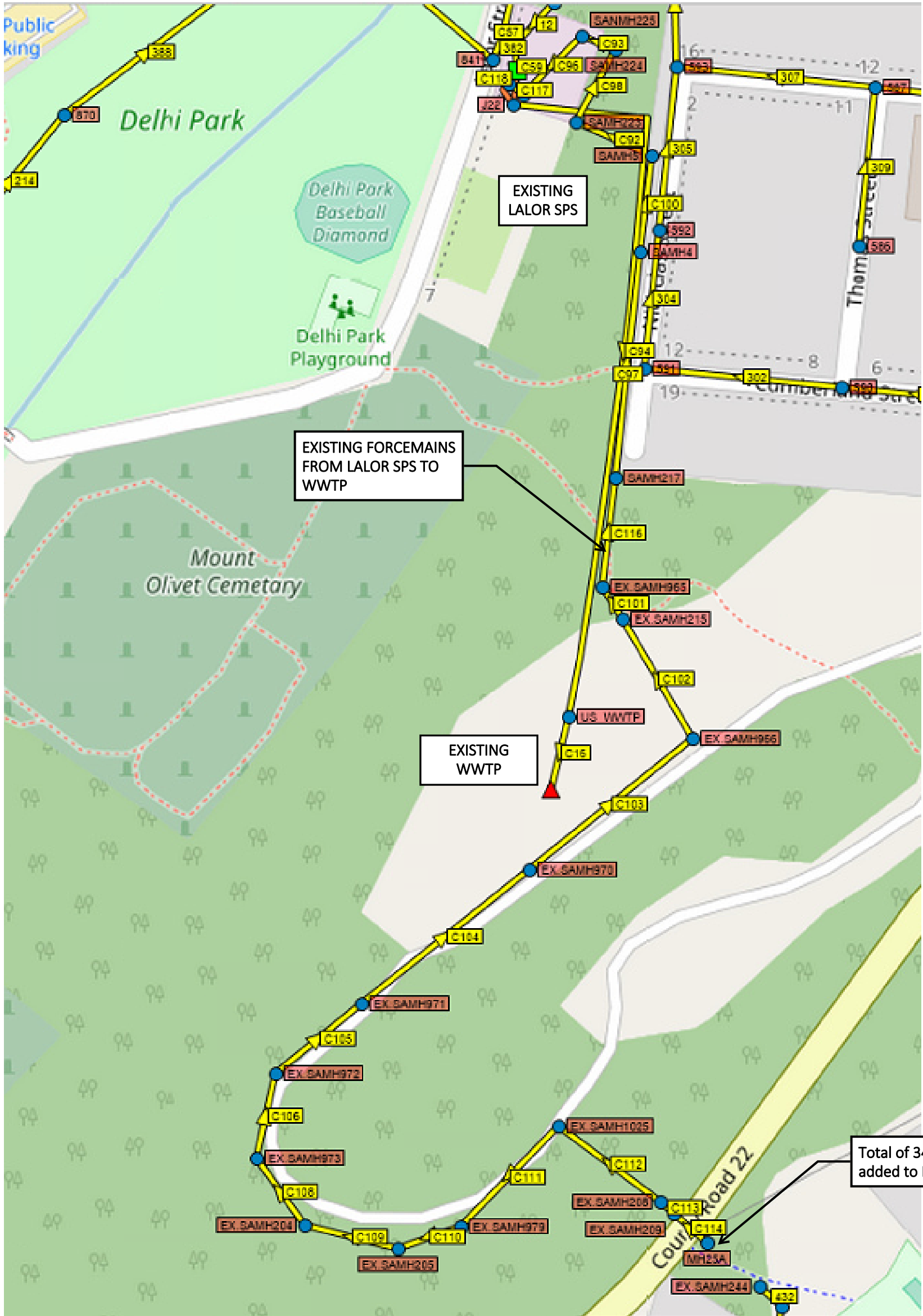
\*NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE.

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**PICTON AIRPORT - PRINCE EDWARD COUNTY COMMUNITY PARTNERS INC.**

**PROPOSED INTERIM SWM POND 2**

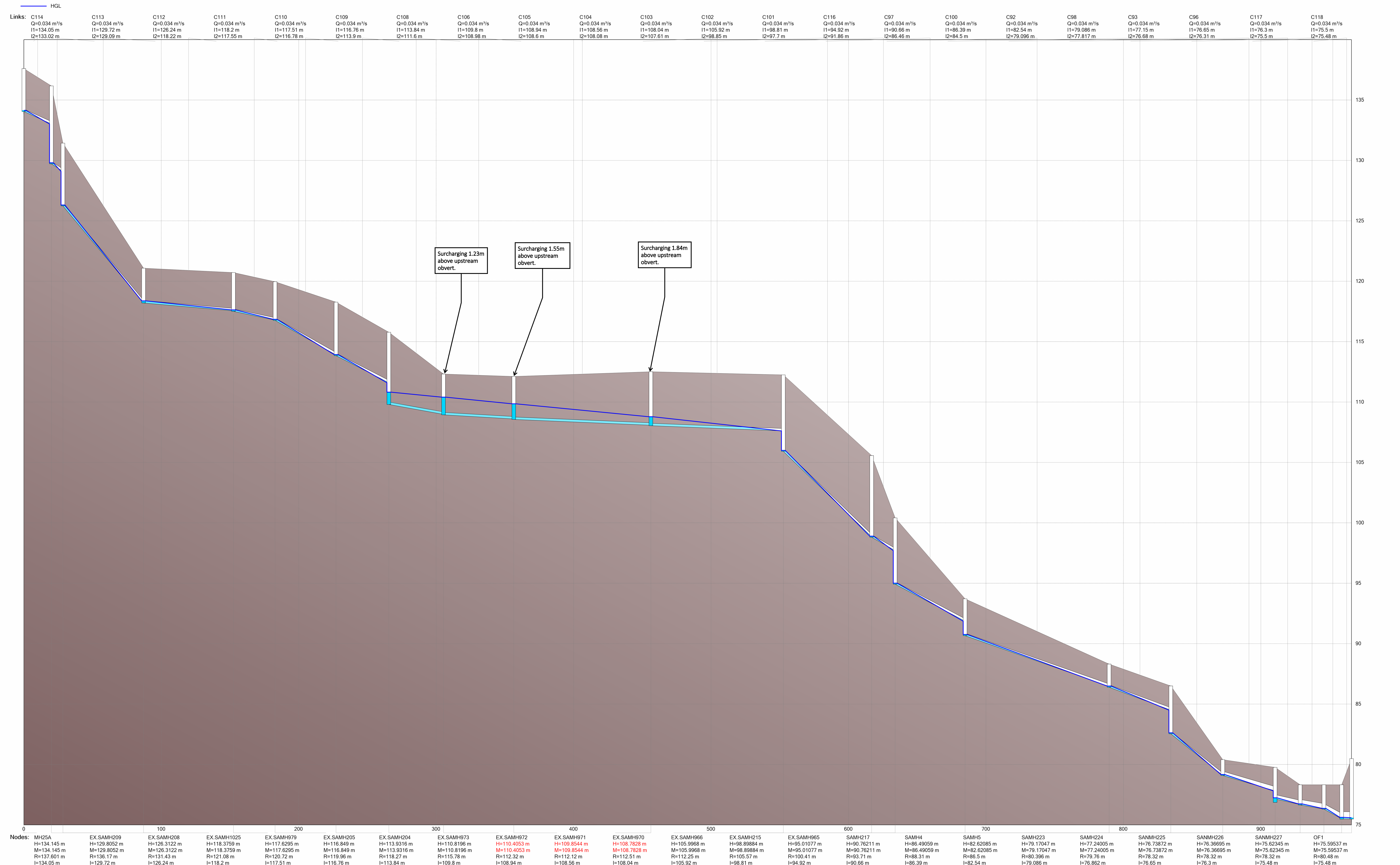
DESIGNED BY: R.B.	CHECKED BY: A.R.K.
SCALE: 1:1250	DATE: AUGUST 2025
PROJECT No: 2365A	FIGURE No: 6.2



**LEGEND:**

- PROPOSED 600mm DIAMETER SEWER
- EXISTING SEWER
- EXISTING MAINTENANCE HOLE

PEC COMMUNITY PARTNERS INC	BASE31 - VILLAGE A	PCSWM MODEL - PLAN VIEW EXISTING MCCAULEY VILLAGE		
30 CENTURIAN DRIVE, SUITE 100 MARKHAM, ONTARIO L3R 8B8 TEL: (905) 475-1900 FAX: (905) 475-8335	DESIGNED BY: MP	CHECKED BY: SH	PROJECT No:	FIGURE No:
	SCALE: NTS	DATE: JULY 2025	2365A	7.1.1



PEC COMMUNITY PARTNERS INC

**LEGEND:**



30 CENTURIAN DRIVE, SUITE 100  
 MARKHAM, ONTARIO L3R 8B8  
 TEL: (905) 475-1900  
 FAX: (905) 475-8335

BASE31- VILLAGE A

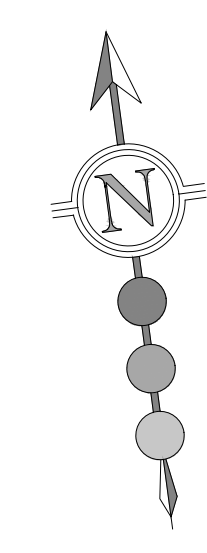
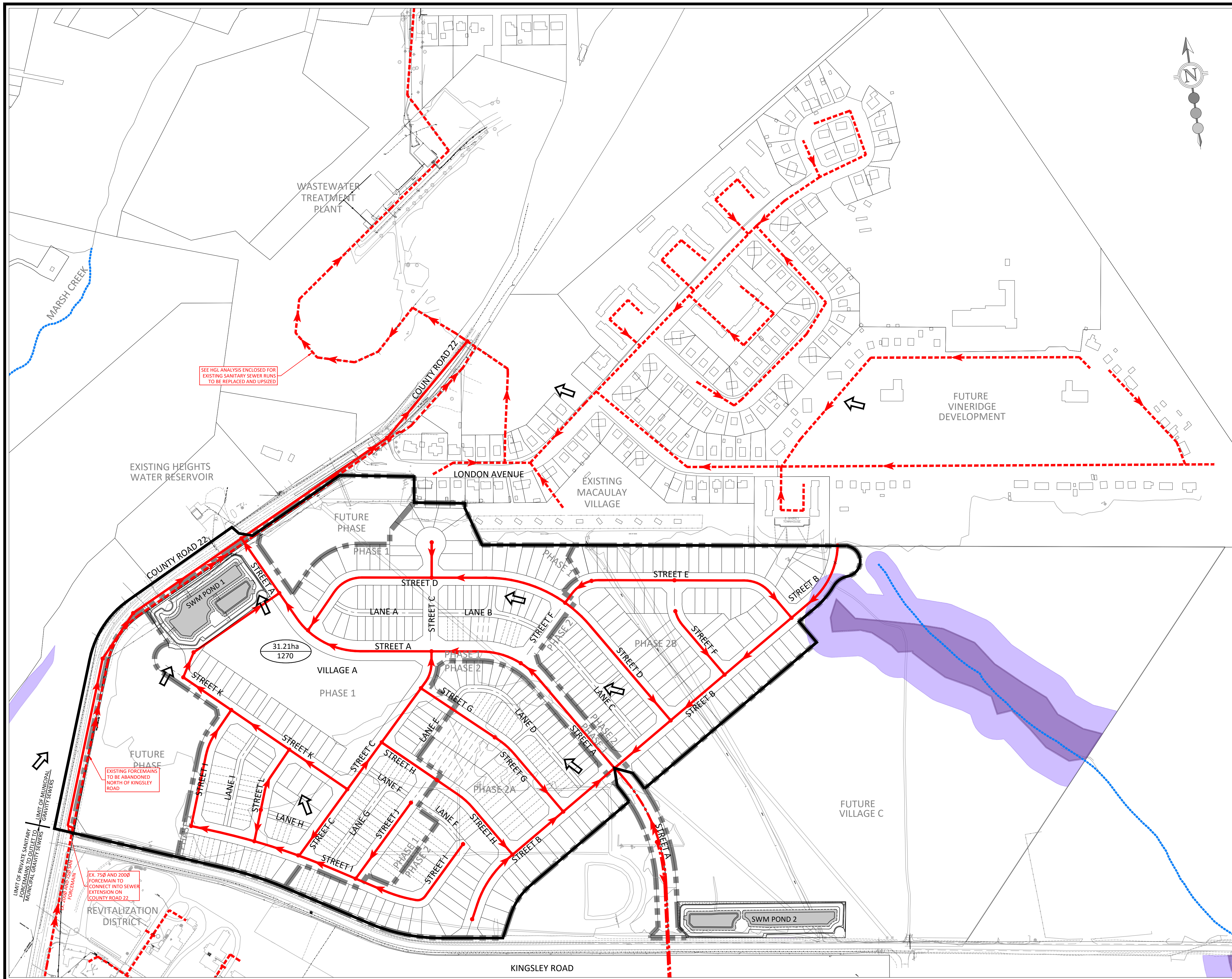
PCSWMM MODEL - PROFILE  
 EXISTING MCCAULEY VILLAGE

DESIGNED BY: MP  
 CHECKED BY: SH  
 SCALE: NTS

DATE: AUGUST 2025

PROJECT No:  
**2365A**

FIGURE No:  
**7.1.2**

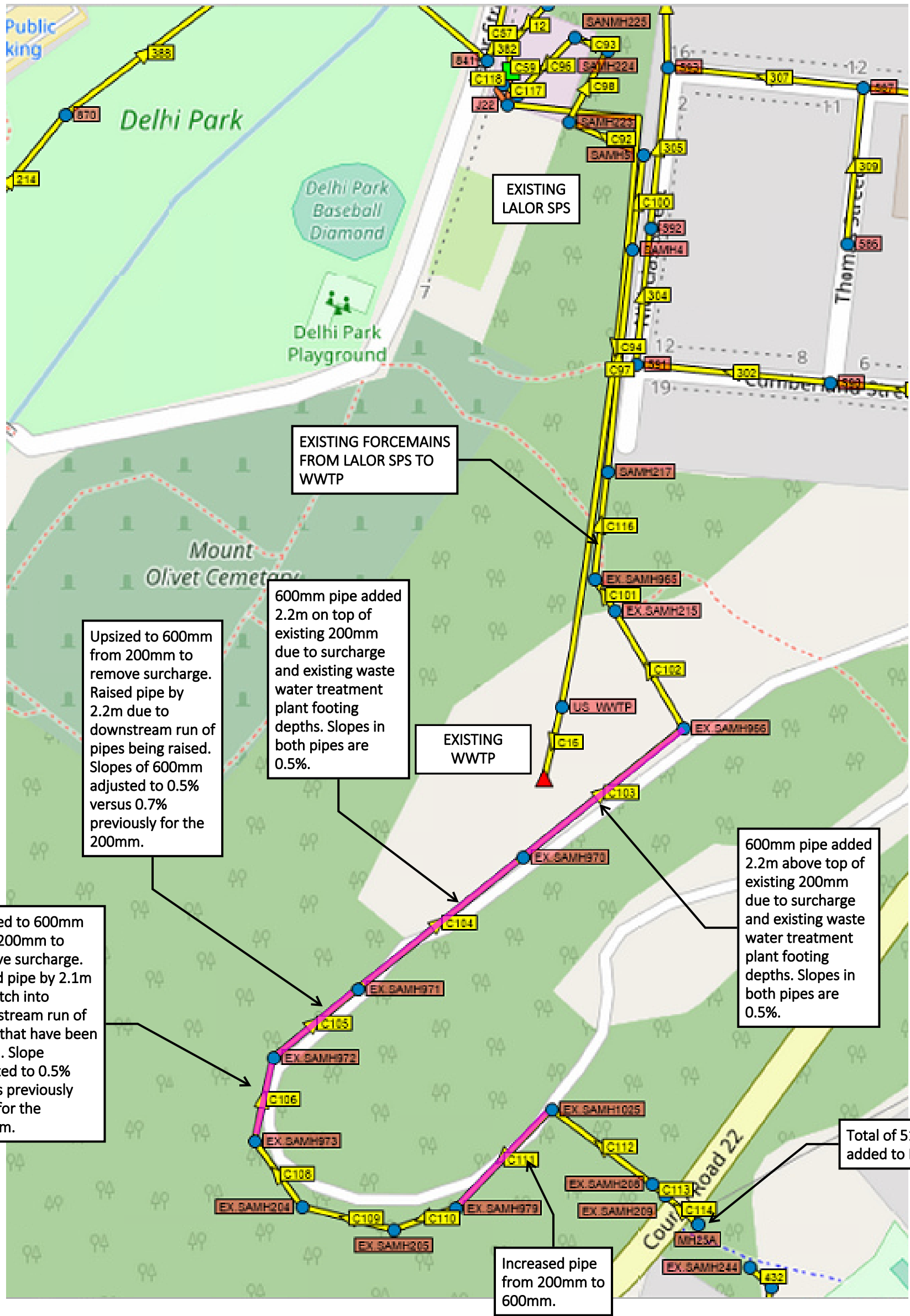


- LEGEND:**
- LIMIT OF DEVELOPMENT
  - LIMIT OF PHASE
  - DRAINAGE FEATURE (PALMER, 2023)
  - HIGH CONSTRAINT AREA (PALMER ENGINEERING, 2023)
  - HIGH CONSTRAINT SETBACK (PALMER ENGINEERING, 2023)
  - EXISTING SANITARY SEWER
  - EXISTING SANITARY FORCEMAIN
  - FUTURE SANITARY FORCEMAIN
  - FUTURE SANITARY SEWER
  - SANITARY DRAINAGE BOUNDARY
  - PROPOSED SANITARY SEWER AND MAINTENANCE HOLE
  - SANITARY DRAINAGE FLOW DIRECTION
  - 29.06ha  
2450
  - DRAINAGE AREA (HECTARES)
  - POPULATION

**SGS** consulting group ltd  
 30 CENTURIAN DRIVE, SUITE 100  
 MARKHAM, ONTARIO L3R 8B8  
 TEL: (905) 475-1900  
 FAX: (905) 475-8335

**PICTON AIRPORT - PRINCE EDWARD COUNTY COMMUNITY PARTNERS INC.**  
**PROPOSED SANITARY DRAINAGE PLAN**

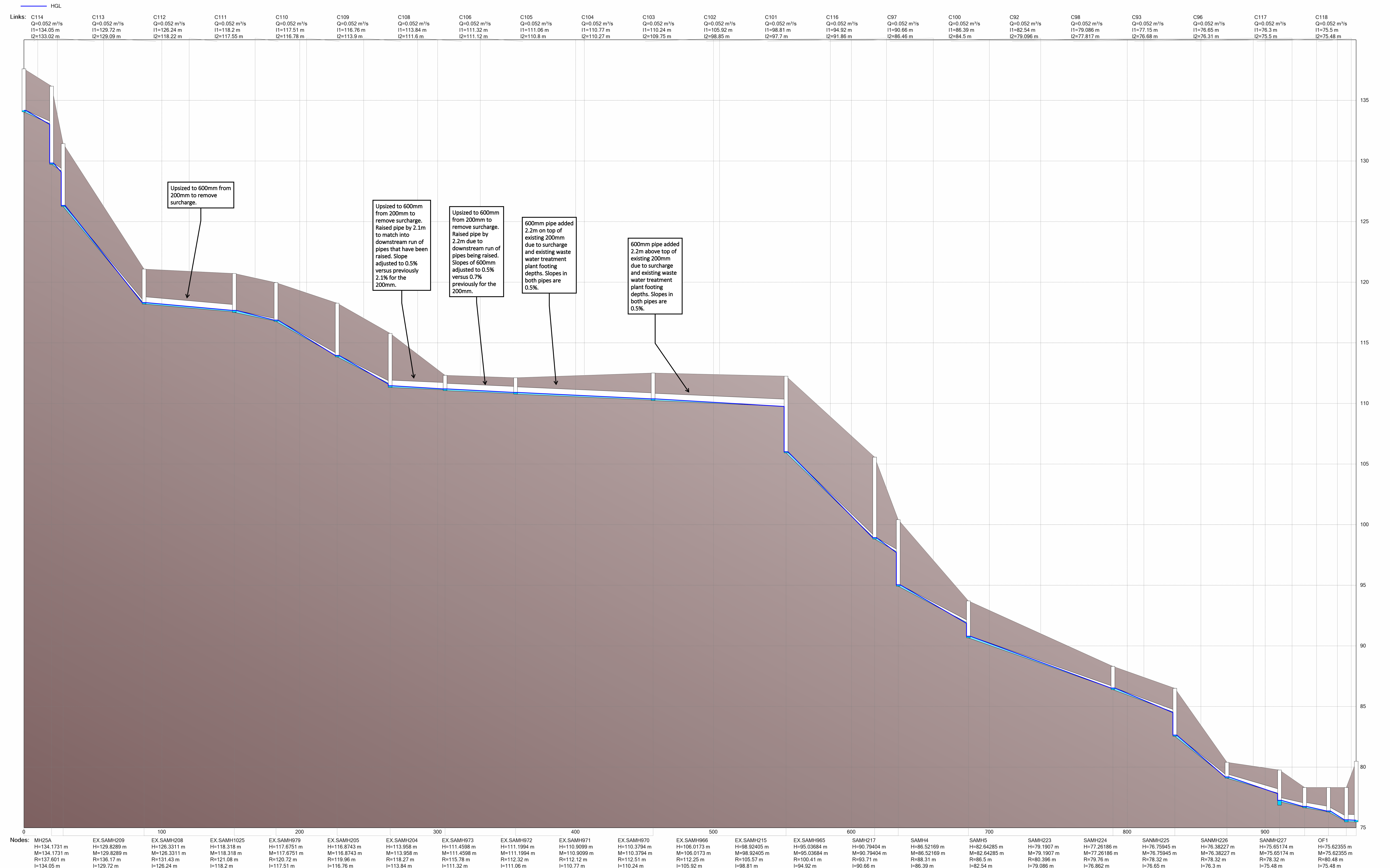
DESIGNED BY: J.S.S.	CHECKED BY: C.C.
SCALE: 1:2000	DATE: AUGUST 2025
PROJECT No: 2365	FIGURE No: 7.2



**LEGEND:**

- PROPOSED 600mm DIAMETER SEWER
- EXISTING SEWER
- EXISTING MAINTENANCE HOLE

PEC COMMUNITY PARTNERS INC	<b>BASE31 - VILLAGE A</b>	PCSWMM MODEL - PLAN VIEW EXISTING MCCAULEY VILLAGE + RD PH2 INTERIM + VILLAGE A PH1
30 CENTURIAN DRIVE, SUITE 100 MARKHAM, ONTARIO L3R 8B8 TEL: (905) 475-1900 FAX: (905) 475-8335	DESIGNED BY: MP SCALE: NTS	CHECKED BY: SH DATE: AUGUST 2025
		PROJECT No: <span style="font-size: 24pt; font-weight: bold;">2365A</span>
		FIGURE No: <span style="font-size: 24pt; font-weight: bold;">7.3.1</span>



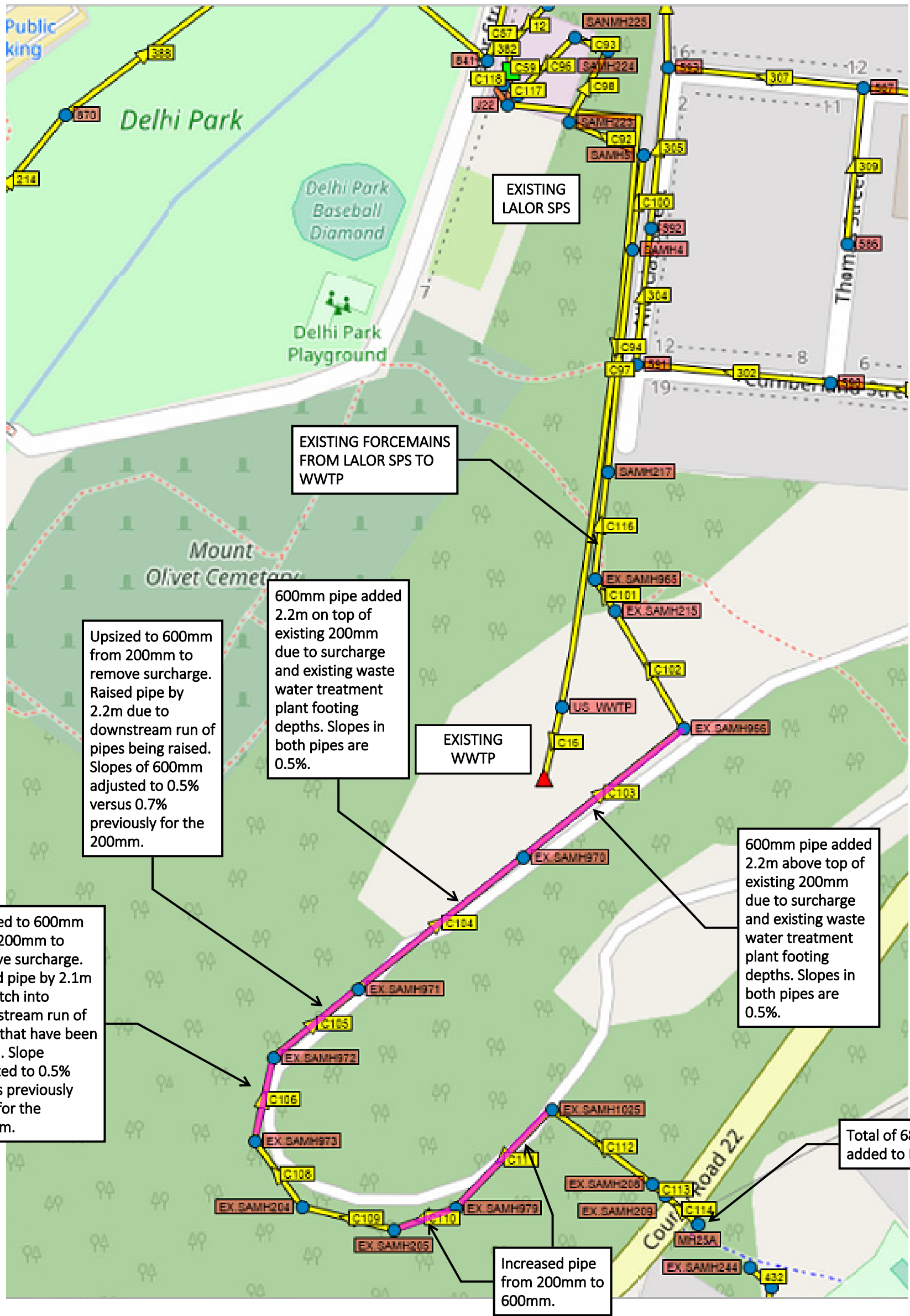
PEC COMMUNITY PARTNERS INC

**LEGEND:**



30 CENTURIAN DRIVE, SUITE 100  
 MARKHAM, ONTARIO L3R 8B8  
 TEL: (905) 475-1900  
 FAX: (905) 475-8335

BASE31- VILLAGE A		PCSWMM MODEL - PROFILE EXISTING MCCAULEY VILLAGE + RD PH2 INTERIM + VILLAGE A PH1	
DESIGNED BY: MP	CHECKED BY: SH	PROJECT No:	FIGURE No:
SCALE: NTS	DATE: AUGUST 2025	<b>2365A</b>	<b>7.3.2</b>

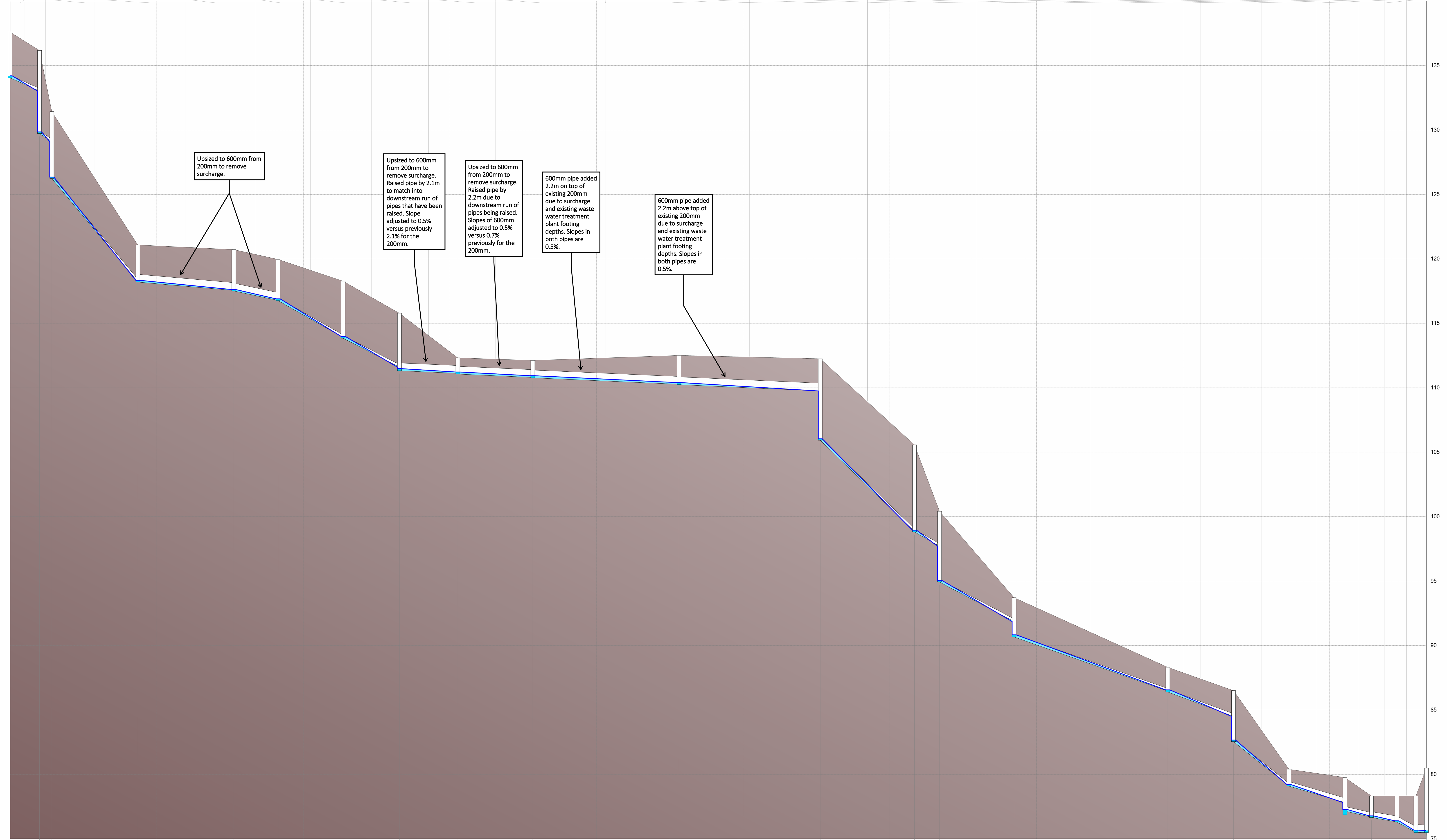


**LEGEND:**

- PROPOSED 600mm DIAMETER SEWER
- EXISTING SEWER
- EXISTING MAINTENANCE HOLE

PEC COMMUNITY PARTNERS INC 	<b>BASE31 - VILLAGE A</b>		PCSWMM MODEL - PLAN VIEW EXISTING MCCAULEY VILLAGE + RD PH2 ULTIMATE + VILLAGE A PH1, PH2A, PH2B	
	DESIGNED BY: MP SCALE: NTS	CHECKED BY: SH DATE: AUGUST 2025	PROJECT No: <h1 style="margin: 0;">2365A</h1>	FIGURE No: <h1 style="margin: 0;">7.3.3</h1>

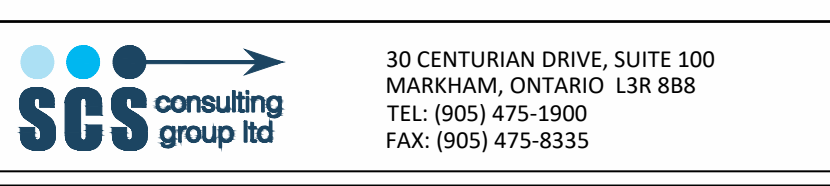
Links:	C114	C113	C112	C111	C110	C109	C108	C106	C105	C104	C103	C102	C101	C116	C97	C100	C92	C98	C93	C96	C117	C118
	Q=0.068 m <sup>3</sup> /s I1=134.05 m I2=133.02 m	Q=0.068 m <sup>3</sup> /s I1=129.72 m I2=129.09 m	Q=0.068 m <sup>3</sup> /s I1=126.24 m I2=118.22 m	Q=0.068 m <sup>3</sup> /s I1=118.2 m I2=117.55 m	Q=0.068 m <sup>3</sup> /s I1=117.51 m I2=116.76 m	Q=0.068 m <sup>3</sup> /s I1=116.76 m I2=113.9 m	Q=0.068 m <sup>3</sup> /s I1=113.84 m I2=111.6 m	Q=0.068 m <sup>3</sup> /s I1=111.32 m I2=111.12 m	Q=0.068 m <sup>3</sup> /s I1=111.06 m I2=110.8 m	Q=0.068 m <sup>3</sup> /s I1=110.77 m I2=110.27 m	Q=0.068 m <sup>3</sup> /s I1=110.24 m I2=109.75 m	Q=0.068 m <sup>3</sup> /s I1=105.92 m I2=98.85 m	Q=0.068 m <sup>3</sup> /s I1=98.81 m I2=97.7 m	Q=0.068 m <sup>3</sup> /s I1=94.92 m I2=91.86 m	Q=0.068 m <sup>3</sup> /s I1=90.66 m I2=86.46 m	Q=0.068 m <sup>3</sup> /s I1=86.39 m I2=84.5 m	Q=0.068 m <sup>3</sup> /s I1=82.54 m I2=79.096 m	Q=0.068 m <sup>3</sup> /s I1=79.086 m I2=77.817 m	Q=0.069 m <sup>3</sup> /s I1=77.15 m I2=76.66 m	Q=0.069 m <sup>3</sup> /s I1=76.65 m I2=76.31 m	Q=0.069 m <sup>3</sup> /s I1=76.3 m I2=75.5 m	Q=0.069 m <sup>3</sup> /s I1=75.5 m I2=75.48 m



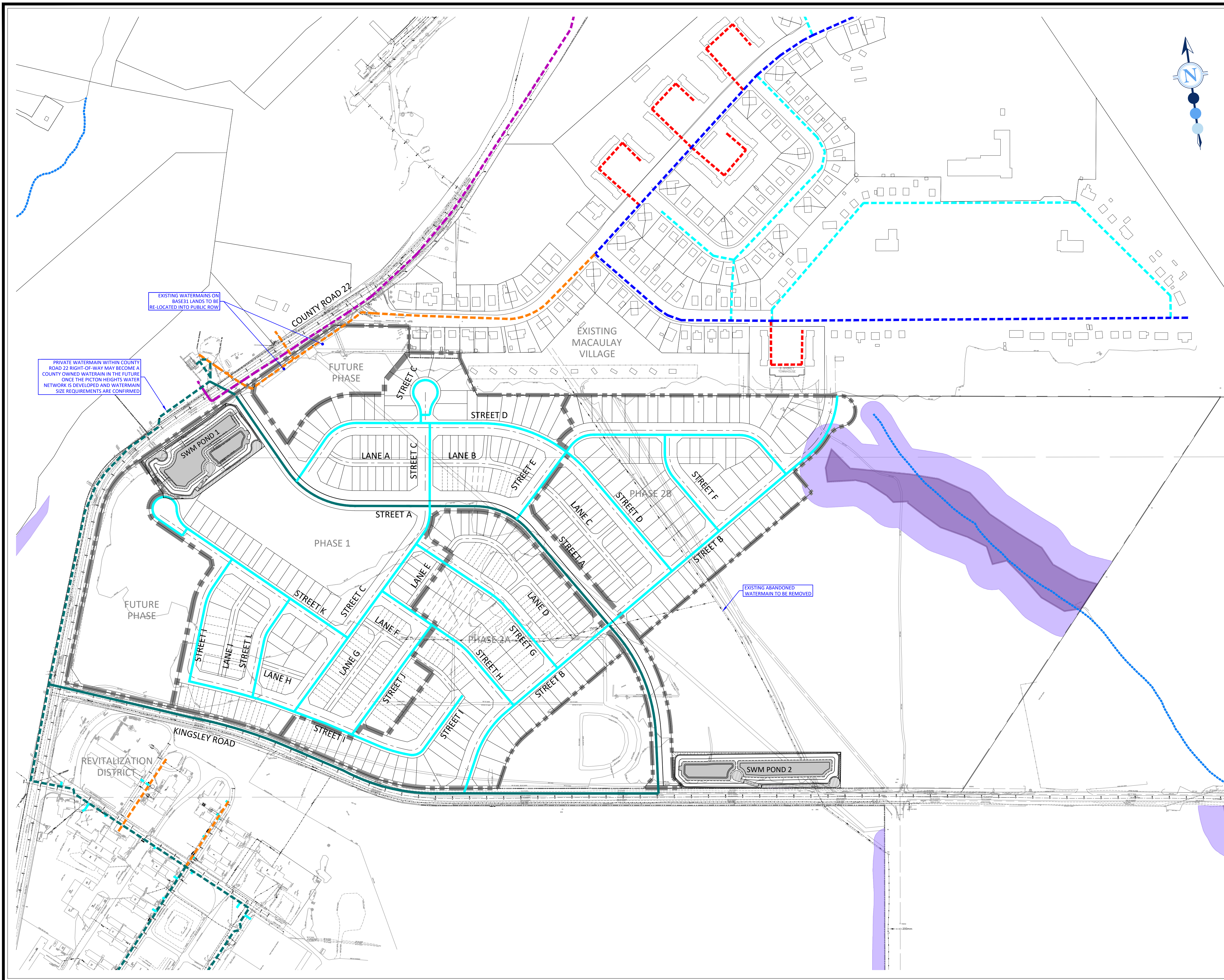
Nodes:	MH25A	EX.SAMH209	EX.SAMH208	EX.SAMH1025	EX.SAMH979	EX.SAMH205	EX.SAMH204	EX.SAMH973	EX.SAMH972	EX.SAMH971	EX.SAMH970	EX.SAMH966	EX.SAMH215	EX.SAMH965	SAMH217	SAMH4	SAMH5	SAMH223	SAMH224	SANMH225	SANMH226	SANMH227	OF1
H=	134.2012 m	129.8505 m	126.3472 m	118.3354 m	117.6188 m	116.898 m	113.9835 m	111.4808 m	111.2203 m	110.9309 m	110.4003 m	106.035 m	98.94762 m	95.06173 m	90.82455 m	86.55444 m	82.66233 m	79.20699 m	77.27953 m	76.77636 m	76.39478 m	75.67496 m	75.64658 m
M=	134.2012 m	129.8505 m	126.3472 m	118.3354 m	117.6188 m	116.898 m	113.9835 m	111.4808 m	111.2203 m	110.9309 m	110.4003 m	106.035 m	98.94762 m	95.06173 m	90.82455 m	86.55444 m	82.66233 m	79.20699 m	77.27953 m	76.77636 m	76.39478 m	75.67496 m	75.64658 m
R=	137.601 m	136.17 m	131.43 m	121.08 m	120.72 m	119.96 m	118.27 m	115.78 m	112.32 m	112.12 m	112.51 m	112.25 m	105.57 m	100.41 m	93.71 m	88.31 m	86.5 m	80.396 m	79.76 m	78.32 m	78.32 m	78.32 m	80.48 m
I=	134.05 m	129.72 m	126.24 m	118.2 m	117.51 m	116.76 m	113.84 m	111.32 m	111.06 m	110.77 m	110.24 m	105.92 m	98.81 m	94.92 m	90.66 m	86.39 m	82.54 m	79.086 m	79.086 m	76.65 m	76.3 m	75.48 m	75.48 m

PEC COMMUNITY PARTNERS INC

LEGEND:



BASE31- VILLAGE A		PCSWMM MODEL - PROFILE EXISTING MCCAULEY VILLAGE + RD PH2 ULTIMATE + VILLAGE A PH1, PH2A, PH2B	
DESIGNED BY: MP	CHECKED BY: SH	PROJECT No:	FIGURE No:
SCALE: NTS	DATE: AUGUST 2025	<b>2365A</b>	<b>7.3.4</b>



EXISTING WATERMANS ON BASE31 LANDS TO BE RE-LOCATED INTO PUBLIC ROW

PRIVATE WATERMAIN WITHIN COUNTY ROAD 22 RIGHT-OF-WAY MAY BECOME A COUNTY OWNED WATERMAIN IN THE FUTURE ONCE THE PICTON HEIGHTS WATER NETWORK IS DEVELOPED AND WATERMAIN SIZE REQUIREMENTS ARE CONFIRMED

EXISTING ABANDONED WATERMAIN TO BE REMOVED

**LEGEND:**

	LIMIT OF DEVELOPMENT
	LIMIT OF PHASE
	DRAINAGE FEATURE (PALMER, 2023)
	HIGH CONSTRAINT AREA (PALMER ENGINEERING, 2023)
	HIGH CONSTRAINT SETBACK (PALMER ENGINEERING, 2023)
	EXISTING 100Ø WATERMAIN
	EXISTING 150Ø WATERMAIN
	EXISTING 200Ø WATERMAIN
	EXISTING 250Ø WATERMAIN
	EXISTING 300Ø WATERMAIN
	EXISTING 350Ø WATERMAIN
	PROPOSED 300Ø WATERMAIN
	PROPOSED 150Ø WATERMAIN
	PROPOSED 150Ø WATERMAIN PLUG

NOTE: LAYOUT IS SCHEMATIC ONLY, DETAILS TO BE PROVIDED AT DETAILED DESIGN STAGE

**SGS consulting group ltd**  
 30 CENTURIAN DRIVE, SUITE 100  
 MARKHAM, ONTARIO L3R 8B8  
 TEL: (905) 475-1900  
 FAX: (905) 475-8335

**PICTON AIRPORT - PRINCE EDWARD COUNTY  
 COMMUNITY PARTNERS INC.**

**WATER DISTRIBUTION SYSTEM PLAN**

DESIGNED BY: J.S.S.	CHECKED BY: C.C.
SCALE: 1:2000	DATE: AUGUST 2025
PROJECT No: 2365	FIGURE No: 8.1