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36 Years

Roger Stevens Warehouse
1966 Roger Stevens Drive, Ottawa

Conceptual Stormwater Management Report

Engineering excellence. Planning precision. Inspired landscapes.

CONCEPTUAL STORMWATER MANAGEMENT REPORT

ROGER STEVENS WAREHOUSE
1966 ROGER STEVENS DRIVE
OTTAWA, ONTARIO

Prepared by:

NOVATECH
Suite 200, 240 Michael Cowpland Drive
Kanata, Ontario
K2M 1P6

July 12, 2019

Novatech File: 119018
Ref No. R-2019-127

Engineers, Planners & Landscape Architects

July 12, 2019

City of Ottawa
Planning and Growth Management Department
Infrastructure Approvals Division
110 Laurier Avenue West, 4th Floor
Ottawa, Ontario
K1P 1J1

Attention: Mr. Harry Alvey, P. Eng.

Dear Sir:

Reference: 1966 Roger Stevens Drive, Ottawa
Conceptual Stormwater Management Report
Our File No.: 119018

Enclosed is a 'Conceptual Stormwater Management Report' for the proposed distribution centre located at 1966 Roger Stevens Drive, in the City of Ottawa. This report is submitted in support of Re-zoning and Official Plan Amendment applications. This report should be read in conjunction with the Servicing Options Statement and Conceptual Servicing Report also prepared by Novatech.

Should you have any questions or require additional information, please contact the undersigned.

Yours truly,

NOVATECH

Cara Ruddle, P.Eng.
Senior Project Manager | Land Development

cc: James Beach – Broccolini Development Group

Suite 200, 240 Michael Cowpland Drive, Ottawa ON K2M 1P6 Tel: 613.254.9643 Fax: 613.254.5867 www.novatech-eng.com

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LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

Novatech has been retained by Broccolini Development Group to complete a conceptual stormwater management design for a proposed warehouse located at 1966 Roger Stevens Drive within the City of Ottawa. The proposed development consists of a single large distribution warehouse, trucking access roads, staff parking facility, and two stormwater management facilities. Refer to **Figure 1** – Key Plan for the general site area.

This report addresses the stormwater management approach for the proposed development and is submitted in support of a Zoning By-Law Amendment and Official Plan Amendment.

1.1 Background

The subject property is approximately 49.5 ha is located on the south side of Roger Stevens Drive and is bounded by Highway 416 to the east, Third Line Road South to the west, and undeveloped land to the south. The property, previously known as Jordel Acres, was approved and registered as a four-phase development. Phase 1 included 10-1 acre residential lots fronting on Third Line Road, and three phases of a commercial/industrial park. The approved design included an access to the commercial/industrial park from Roger Stevens Drive and an internal ring road servicing the commercial/industrial lots. Two storm ponds were proposed as part of the approved stormwater management design which provided quality and quantity control of stormwater. A copy of the 'Stormwater Management Report for Pri-Tec International Inc., Jordel Acres Subdivision, Rideau Township' prepared by David McManus Engineering Ltd., dated December 2001 is provided in **Appendix A** for reference. The original Registered Plan of Subdivision is also provided for reference in **Appendix A**. To date only some of the residential lots fronting Third Line Road have been constructed.

1.2 Existing Conditions

Under existing conditions, the site consists of predominantly row crop agricultural lands. An elevated ridge of forested land intersects the property from the southwest to northeast corner. Along the ridge there are several farm buildings and a gravel access road. Refer to **Figure 2** – Existing Conditions Plan.

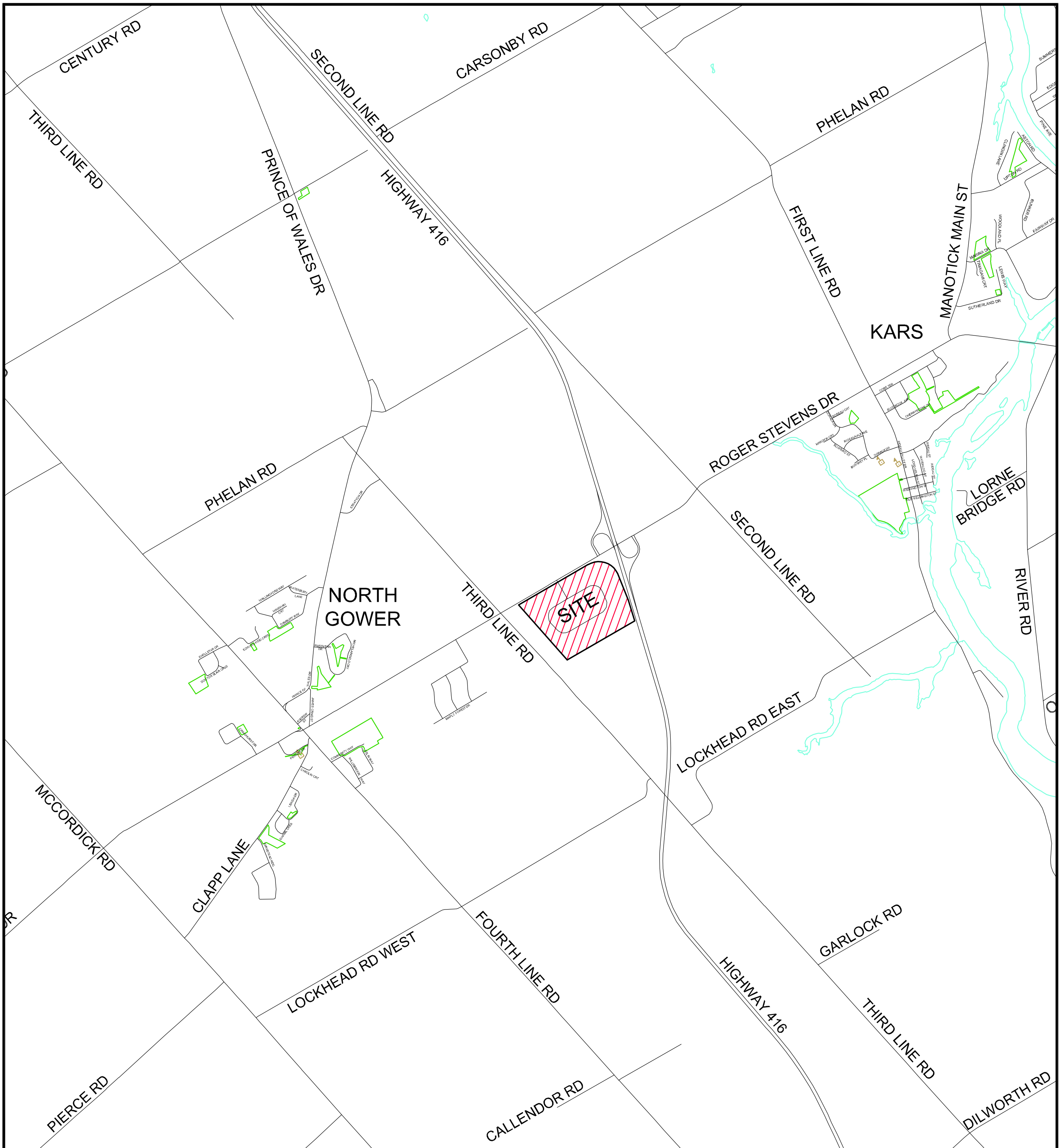
1.3 Topography & Drainage Outlets

The elevated ridge effectively splits the site into two drainage areas. The northwest half of the site slopes towards the northwest corner of the property. Runoff is directed through two culverts; a 500mm dia. culvert at the Roger Stevens Dr./ Third Line Rd. intersection, and a 500mm dia. culvert 520m east of the intersection. The southeast half of the site drains to the Johnston Drain along the southern border of the property, which directs runoff to a box culvert under Highway 416.

The entire site is within the drainage area of Stevens Creek and the Rideau River. Refer to **Figure 5** - Pre-Development Drainage Area Plan for details on the existing drainage patterns.

1.4 Subsurface Conditions

A Hydrogeological Study Report, Jordel Acres Proposed Subdivision, prepared by Sauriol Environmental Inc. (June 1999) has been completed for the subject site. The terrain evaluation concluded that there are three types of soils present on site. The ridge area consists of glacial till



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N

1966 ROGER STEVENS DRIVE

KEYPLAN

SCALE		
N.T.S		
DATE JULY 2019	JOB 119018	FIGURE 1

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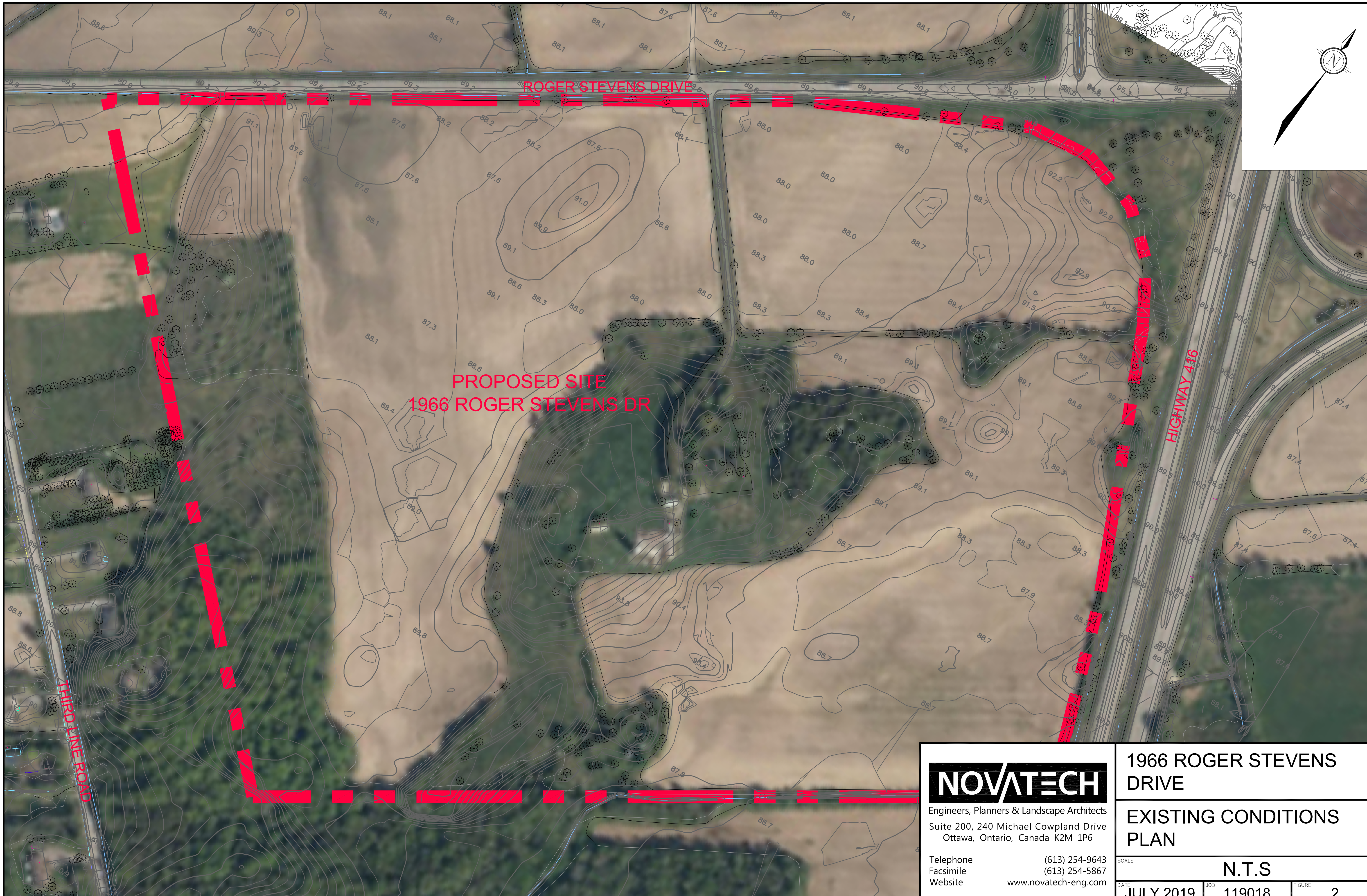

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1966 ROGER STEVENS DRIVE

EXISTING CONDITIONS PLAN

SCALE	N.T.S	
DATE JULY 2019	JOB 119018	FIGURE 2

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and the lower areas consist of a clay material with some parts overlain by a thin sand material. The bedrock is approximately 8 to 13 meters below grade.

2.0 PROPOSED DEVELOPMENT

It is proposed to develop a large distribution warehouse facility which would include one warehouse building approximately 700,000 square foot (+/-) footprint, a large employee parking area and substantial truck parking servicing the multitude of loading docks. Multiple accesses are proposed from Roger Stevens Drive including one specific for truck access. The facility will be services by a private well system and a private sewage treatment plant. The total development area is approximately 49.5 hectares. Refer to **Figure 3** – Conceptual Site Plan for details.

3.0 STORMWATER MANAGEMENT DESIGN

3.1 Stormwater Management Design Criteria and Objectives

The subject site is located within the jurisdiction of the Rideau Valley Conservation Authority (RVCA). As such, the following stormwater management criteria and objectives have been developed through previous consultation with the RVCA and the City of Ottawa Sewer Design Guidelines.

Stormwater Quantity

- Convey post-development peak flows from the site to the proposed stormwater management facility;
- Post-development peak flows are to be controlled to pre-development levels for all storm events, up to and including the 100-year event;
- No ponding within the asphalt parking for storm events up to and including the 2-year event; and,
- The existing 1:100-year floodplain storage will not be adversely affected.

Stormwater Quality

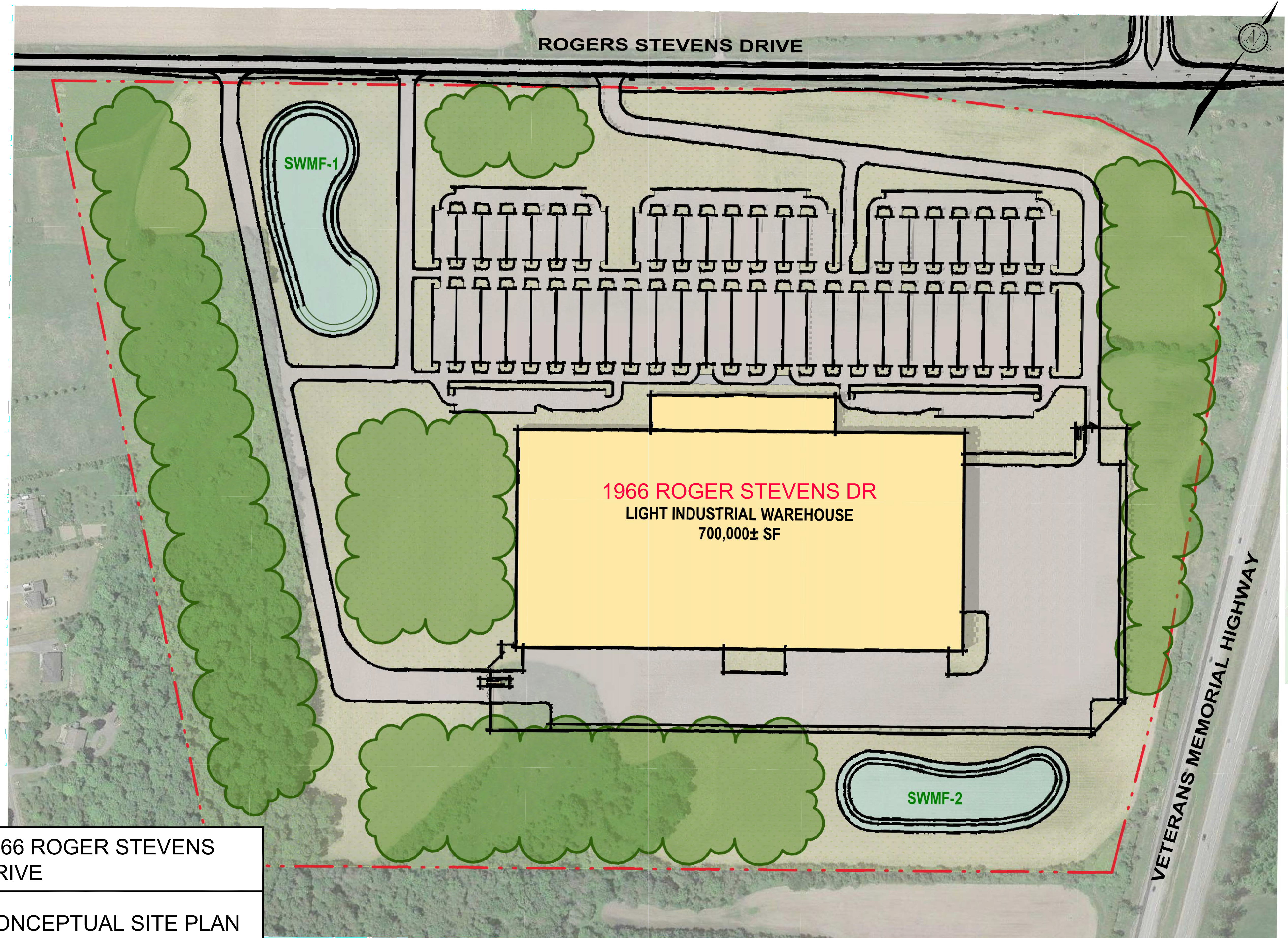
- Provide an 'Enhanced' level of stormwater quality control corresponding to a long-term removal rate of 80% Total Suspended Solids (TSS).

Erosion and Sediment Control

- Minimize the impact on the downstream receiving watercourses by minimizing the potential erosion and volume of sediment entering the watercourses on a temporary basis (during construction) and on a permanent basis.

3.2 Storm Servicing

The proposed development will be serviced by both storm sewers and ditches. Runoff from all storm events will be conveyed by a combination of storm sewers (minor system) and ditches throughout the property (overland flow path). Two stormwater management ponds will be the outlet for both the storm sewers and ditches. Refer to **Figure 4** – Conceptual Servicing Plan.



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1966 ROGER STEVENS DRIVE

CONCEPTUAL SITE PLAN

SCALE	1 : 3000 (0 30 60 90 120)
DATE	JULY 2019
JOB	119018
FIGURE	3

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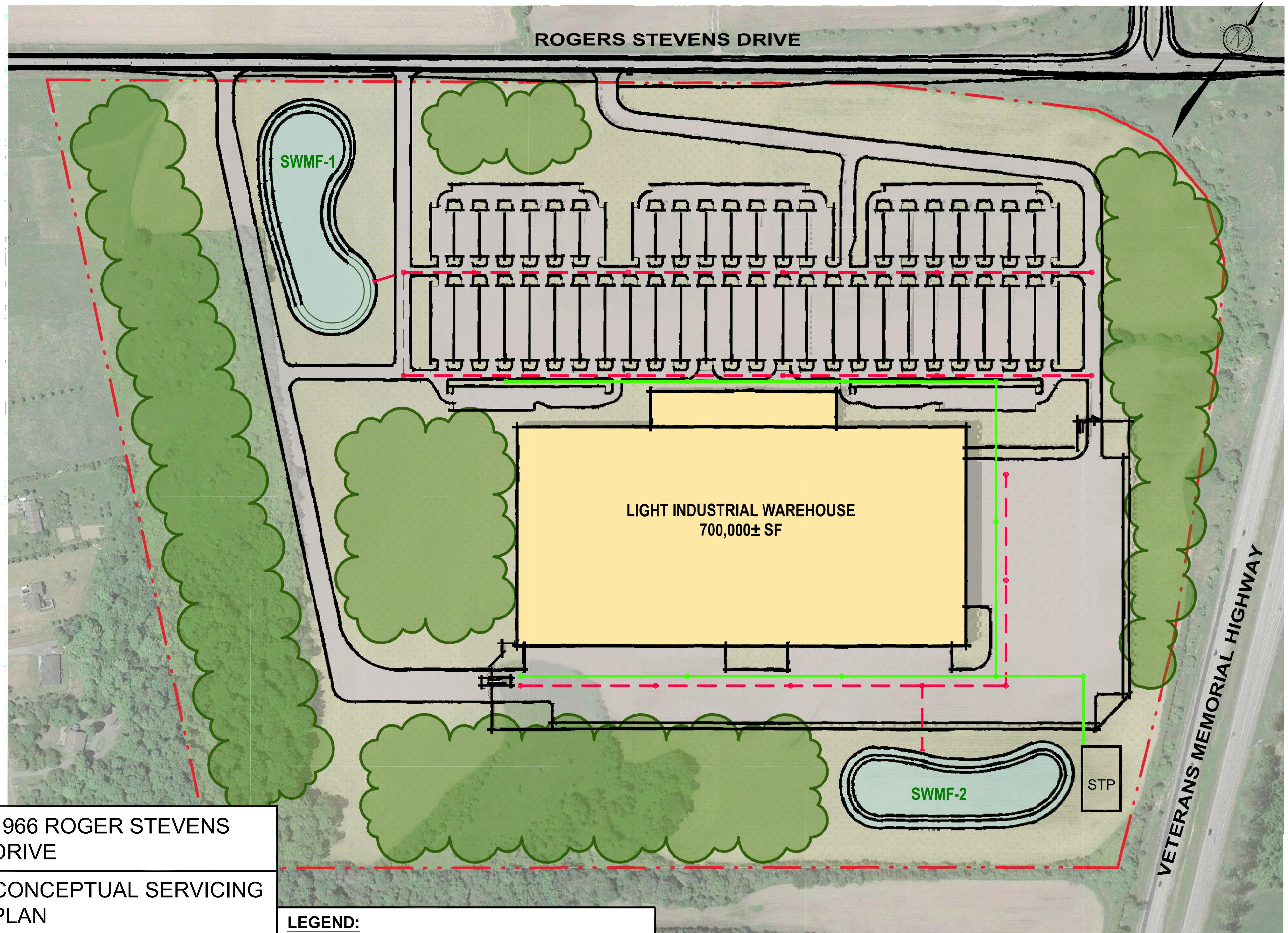

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1966 ROGER STEVENS DRIVE

CONCEPTUAL SERVICING PLAN

SCALE			
1 : 3000			
DATE JULY 2019	JOB 119018	FIGURE 4	

LEGEND:

- CONCEPTUAL STORM SEWER
- CONCEPTUAL SANITARY SEWER

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3.3 Stormwater Management Facilities

Two wet ponds are proposed to meet the design criteria for stormwater management to service the 49.5ha area. One pond (SWMF-1) is located in the northwest corner of the property and outlets to the existing culvert under Roger Stevens Drive. The second pond (SWMF-2) is located in the southeast corner of the property and outlets to the existing watercourse along the southern extent of the property and the Highway 416 culvert.

3.4 SWMF Design Criteria

The proposed SWM facility has been designed to meet the following criteria:

- Provide an *Enhanced* level of water quality control (80% long-term TSS removal);
- Provide quantity control storage to limit post-development peak flows to pre-development levels;
- The SWM facility will have side slopes of 3:1 (H:V) or shallower;
- The forebays have been sized to provide enough storage for 10 years of sediment accumulation;
- A sediment management area has been provided within the SWM block to allow for storage and drying of material removed during maintenance/ cleanout;
- Guardrails conforming to City standards will be installed at the inlet and outlet structures of the SWM facility;
- Infiltration tests are to be performed on the native material during constriction to determine whether a liner will be required.

3.5 SWMF Components

The proposed SWMF will have the components as listed below. These components will be designed in detail and submitted with the Site Plan Application.

- Access pathways to the inlet and outlet structure and sediment storage area.
- Pond inlet structure.
- Sediment Forebay and Permanent Pool. Pond outlet structure
- Overflow spillway

3.6 SWMF Conceptual Design

A conceptual design of the two wet ponds has been completed to confirm there is adequate space within the development area for the two ponds. The preliminary calculations estimate that a total permanent pool volume of 6,980 cubic metres is required to provide an enhanced level of quality control up to 80% TSS removal. The total active storage volume for quantity control was estimated to be 9,050 cubic meters. The total permanent pool and active storage volumes will be divided among the two ponds. The extended detention volume will allow for settling of suspended sediment in the pond prior to releasing stormwater from the site. The extended detention volume will be released over a period of 24 hours.

Flows that exceed the active storage limit will outlet through a concrete weir located at the outlet structure. This outlet structure will control peak flows for all storm events, up to and including the

100-year storm event. Refer to **Figure 6** – Post Development Drainage Area Plan which shows the conceptual stormwater management ponds.

It should be noted that SWMF-1 is located within an existing flood plain area and any storage provided for the proposed development is above the 1:100-year flood plain elevation. A cut/fill permit will be required from the Conservation Authority.

4.0 EROSION AND SEDIMENT CONTROL

4.1 Temporary Measures

The following erosion and sediment control measures will be implemented during construction and are as follows:

- Silt fences are to be installed along the perimeter of the site;
- Straw bales are to be installed in ditches and swales until construction is completed and until vegetation is established;

It is proposed that the SWM facility be used as a temporary sediment control pond during construction. The construction of the temporary sediment pond would include the following:

- Excavate and shape the ultimate pond footprint, including the sediment forebays;
- Construct temporary drainage ditches to convey storm runoff to the pond;
- Construct temporary berms to isolate the areas near the ultimate inlets and outlets;
- Install silt curtains in the forebays to provide additional sediment control during construction;
- Construct the outlet to the 1350mm culvert crossing Highway 417;
- Once the ultimate stormwater management facility has been approved, construct the ultimate inlets and remove the berm protecting the ultimate outlet.

Details and specific locations will be specified during the detailed design stage. The temporary erosion and sediment control measures are to be implemented prior to construction and are to remain in place throughout construction and should be inspected regularly.

4.2 Permanent Measures

Permanent erosion and sediment control measures are to include the following:

- Swales and ditches are to be constructed at minimum grade, where possible;
- Swales and ditches are to be vegetated to provide permanent erosion and sediment control.

The proposed temporary erosion and sediment control measures shall be implemented prior to construction, shall remain in place throughout each phase of construction and shall be inspected regularly. No control measure shall be permanently removed without prior authorization from the Engineer.

LEGEND

DRAINAGE AREA BOUNDARY

FLOW ARROW

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ROGER STEVENS WAREHOUSE BUILDING

PRE-DEVELOPMENT DRAINAGE AREA PLAN

SCALE 1 : 3000 (0, 30, 60, 90, 120)

DATE JULY 2019 | JOB 119018 | FIGURE 5

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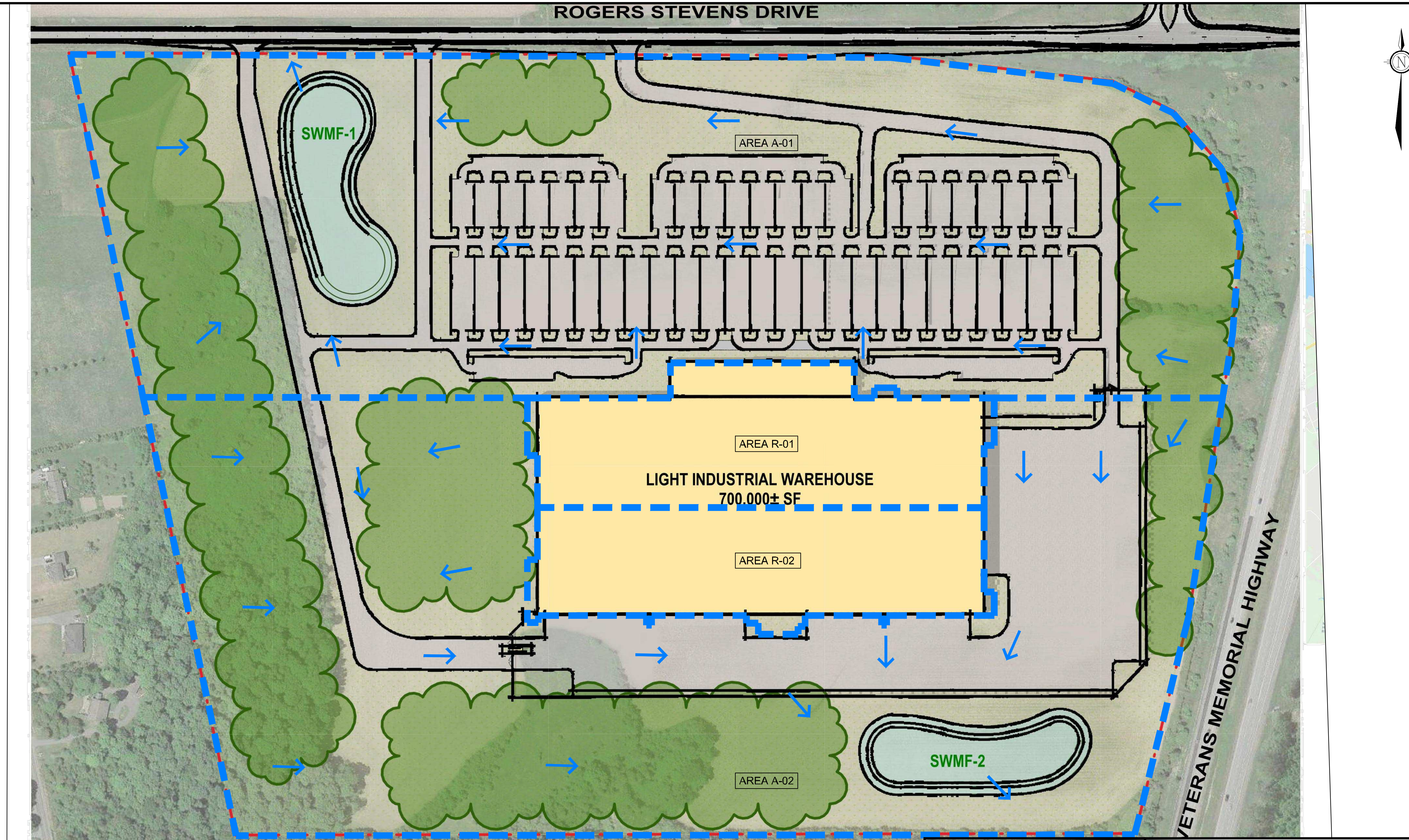

LEGEND

DRAINAGE AREA BOUNDARY

FLOW ARROW

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ROGER STEVENS
WAREHOUSE BUILDING

POST-DEVELOPMENT
DRAINAGE AREA PLAN

SCALE 1 : 3000 (0, 30, 60, 90, 120)		
DATE JULY 2019	JOB 119018	FIGURE 6

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5.0 CONCLUSIONS AND RECOMMENDATIONS

- The proposed development will consist of a single large distribution warehouse, distribution trucking lot, staff parking facility, and associated amenity spaces.
- The proposed development will be graded to direct stormwater runoff towards two proposed water quality and quantity wet ponds which will ultimately discharge to Stevens Creek.
- On-site stormwater quantity control will be provided by two wet ponds equipped with a controlled outlet structure and an approximate total volume of 16,030 m^3, including the permanent pool, which will be required in order to maintain pre-development flow rates for all storms up to and including the 1:100-year event prior to being discharged.
- An Enhanced level of stormwater quality control corresponding to a long-term removal rate of 80% Total Suspended Solids (TSS) will be provided by the proposed wet pond.
- Erosion and sediment controls will be provided both during construction and on a permanent basis.

Prepared by:

Matt Hrehoriak, P.Eng.
Project Engineer
Land Development Engineering

Reviewed by:

Cara Ruddle, P.Eng.
Senior Project Manager
Land Development Engineering

APPENDIX A
Jordel Acres Reports

STORMWATER MANAGEMENT REPORT
FOR
PRI-TEC INTERNATIONAL INC.
JORDEL ACRES SUBDIVISION
RIDEAU TOWNSHIP

Prepared by:

DAVID M[C]MANUS ENGINEERING LTD.

Project No. 1935
(Provincial No. 06T-98004)

December 2001

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1.0 INTRODUCTION

David McManus Engineering Ltd. was retained by Mr. Richard Lalande of Pri-Tec International Inc. to provide engineering services for the site designated as Jordel Acres Subdivision in the Township of Rideau. The site is located on part of Lots 21 and 22, Concession 2, at the southwest corner of Roger Stevens Drive and Highway 416. The development will consist of low-density rural residential lots for single-family homes and a rural commercial/industrial park. The location of the development is shown on Figure 1.

The existing land use of this property is predominantly agricultural (i.e. cash crop and cattle) with an existing home and barns located in the central area of the proposed park. The soil conditions consist of clays and sandy loam.

The proposed development covers approximately 55 ha and will be constructed in four phases. The first phase will consist of ten 0.81ha to 1.0 ha residential lots. Development of the commercial/industrial park will consist of phases 2 to 4. Drawing 1935-PH1 shows the phased development of the project. Construction of the first phase is slated to begin in the spring of 2002.

This report documents the proposed method of attenuating the storm water runoff from the subject site. Items that are addressed include:

Pre-development and post-development runoff.

Determine the location, size and storage volumes of the proposed drainage system components located within the site to address quantity and quality criteria.

Summarize Best Management Practices.

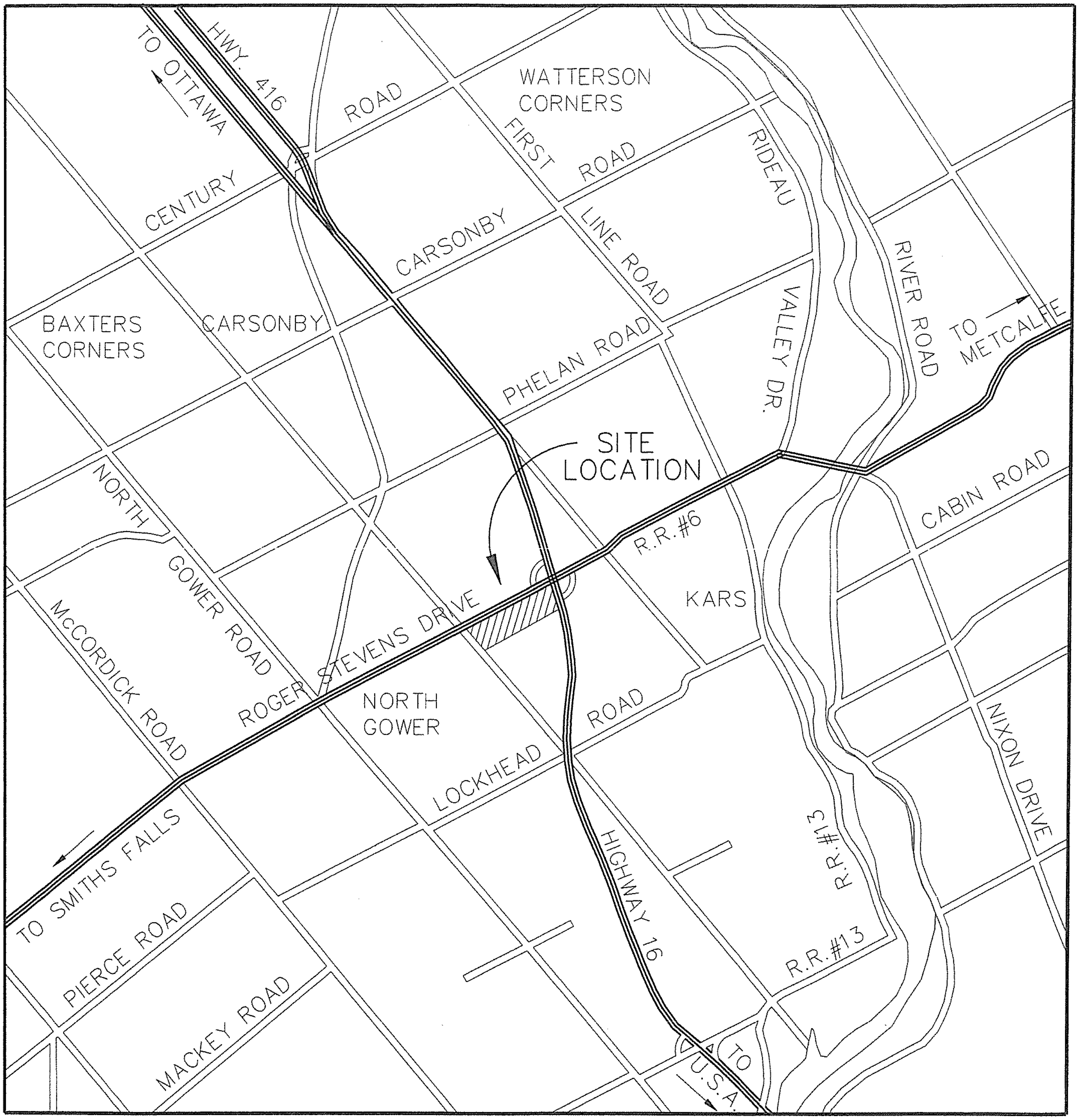
2.0 PRE DEVELOPMENT CONDITIONS

2.1 Existing Drainage

The existing drainage pattern is basically split into two drainage areas that are divided diagonally from the corner of Roger Stevens Dr. and Highway 416 to the southwest corner of the property at Third Line Rd. The northwest half drains in a northerly direction toward Roger Stevens Dr. via existing ditches to two separate outlets.

The first is an existing 500mm dia. culvert at the intersection of Roger Stevens Dr. and Third Line Road. The second is an existing 500mm dia. culvert approximately 520 meters east of the intersection on Roger Stevens Drive. Drainage continues north easterly through an existing ditch system that outlets to Stevens Creek and eventually outlets to the Rideau River.

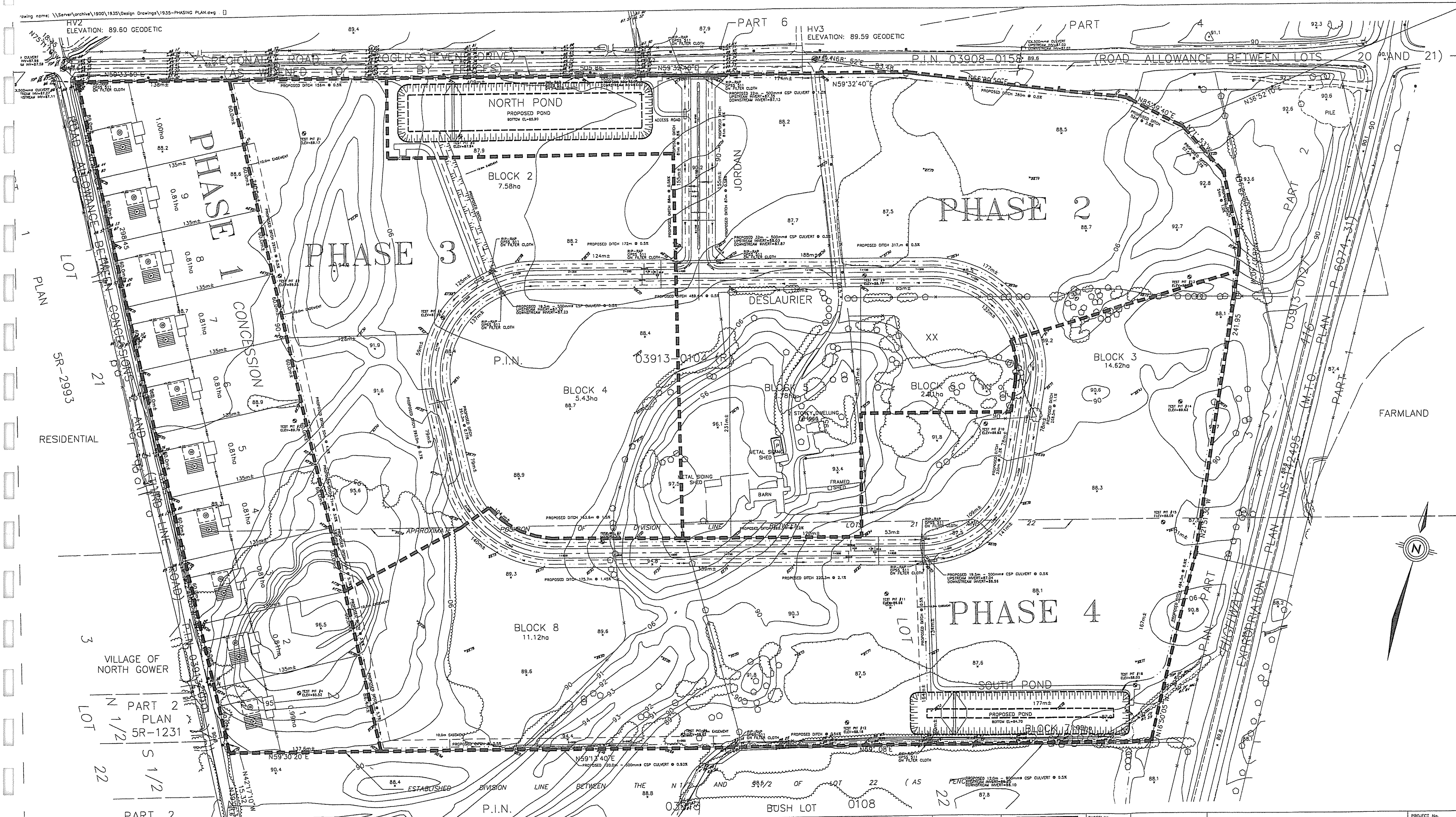
The south half drains via existing ditches to an existing box culvert under Highway 416 eventually out-letting to Stevens Creek and the Rideau River.



Rideau Township	DME Ltd. David McManus Engineering Ltd. 200 - 39 Camelot Drive Nepean Ontario, K2H 5W6 E-mail: mcmanus@dmel.on.ca Ph. 225-1929 Fax 225-7330	Pri-Tec International Inc. Jordel Acres Development	
		DATE: December 2001	KEY PLAN	Poject No. 1935

Date of plot: Dec 11, 2001 - 4:42pm

Drawing name: \\Server\archive\1900\1935\Design Drawings\1935-PHASING PLAN.dwg



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THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

PRELIMINARY

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No.	REVISION	DATE	BY

REGISTERED PROFESSIONAL ENGINEER J. D. McMANUS PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER UDO BOEHME PROVINCE OF ONTARIO

DME
David McManus Engineering Ltd.
200 - 39 Camelot Drive
Nepean Ontario, K2H 5W6
E-mail: mcmanus@dmel.on.ca
Ph. 225-1929 Fax 225-7330

BASEPLAN	DME
DESIGN	UB/MJG
CHECKED	UB
CAD	GSC
PROJ. MGR.	UB
APPROVED	JDM

SCALE N.T.S.

PRI-TEC INTERNATIONAL INC.
JORDEL ACRES
TOWNSHIP OF RIDEAU

PHASING PLAN

PROJECT No.	1935
SURVEY BY	DME
DATE	DECEMBER 2001
DRAWING No.	1935-PH1

Last saved by:

2.2 Water Quality

It has been determined by the RVCA that the receiving waters of Stevens Creek is a Type 2 fish habitat. The outlet for Stevens Creek is the Rideau River which is considered a Type 1 fish Habitat.

2.3 Water Quantity

Stormwater runoff is to be controlled to pre-development conditions. Pre-development flows have been determined using SWMHYMO modelling software. The summary output from the model showing the pre-development flows is attached in Appendix B.

2.4 Floodplain Infill

The Rideau Valley Conservation Authority has identified issues regarding floodplain infill and loss of storage. Portions of the development are situated within the 1:100 year floodplain of Stevens Creek. Lands below the existing elevation of 88.34 within lots 8 and 9 and Block 1 and lands below the existing elevation 88.07 within Blocks 2, 3, and 8 are affected. The affected areas are shown on Drawing 1935-GR1.

3.0 POST DEVELOPMENT CONDITIONS

3.1 Development Criteria

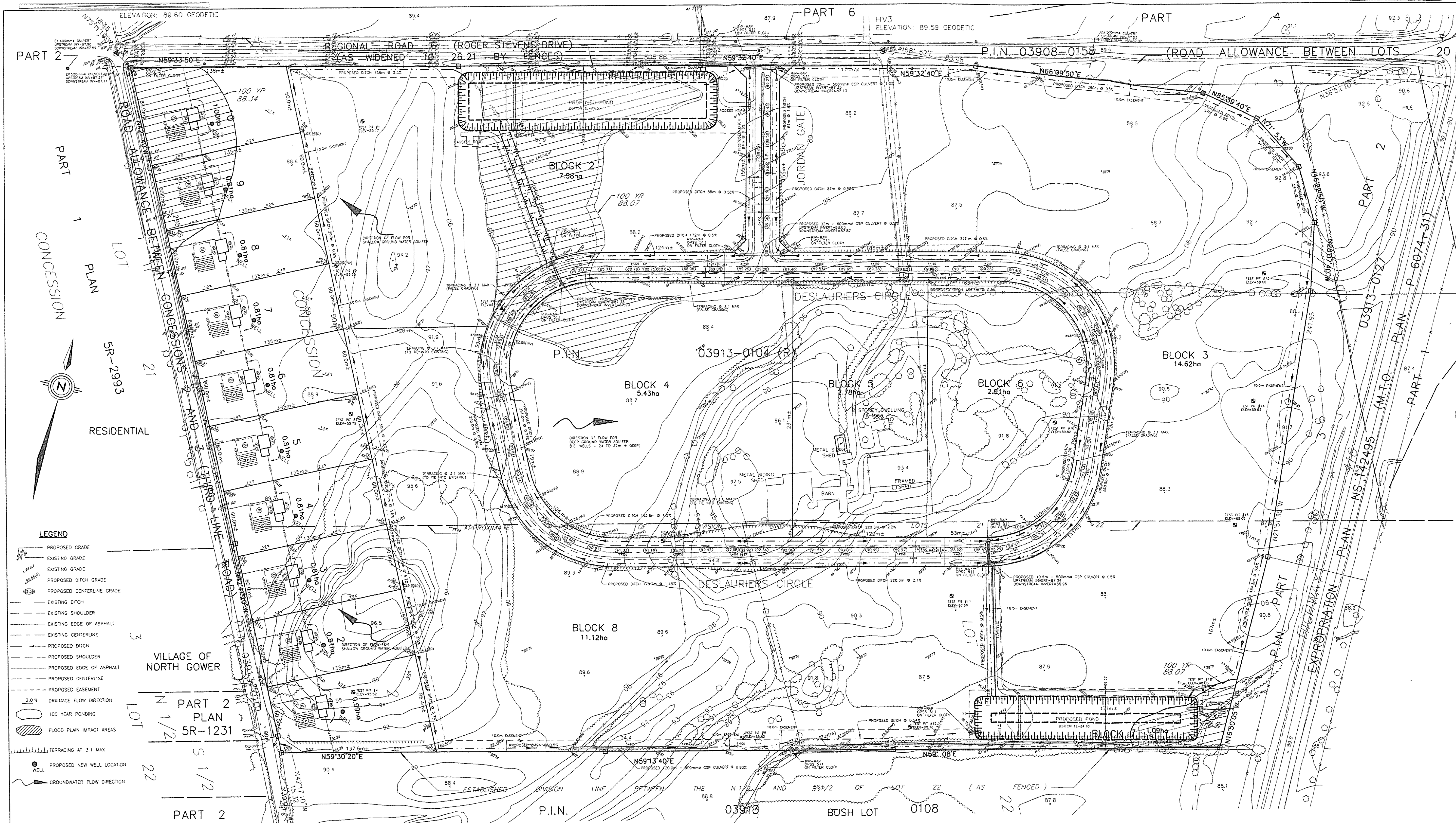
The proposed development may impact surface water, fish habitat and floodplain loss of storage.

Stormwater management facilities will be constructed to provide both quality and quantity control for the commercial/industrial park. Two separate facilities will be required to service the development. The ponds are referred to as the North Pond, which will accommodate phases 2 and 3 and the South Pond, which will accommodate Phase 4.

The development of the residential lots in phase 1 were not included as part of the stormwater management design. The proposed site drainage is meant to improve the quality of the storm runoff. This will be achieved by means of minimum grade changes over the lots, which promotes water infiltration. The storm runoff in the ditches will be subject to a natural filtration process caused by the residential grassed areas, and the use of minimum longitudinal grades acceptable to the Township. This will minimize erosion and enhance runoff quality. Therefore any degradation of downstream watercourses will be naturally miniscule.

3.2 Water Quality

Despite requiring only level 2 treatment, as indicated by the type 2 habitat of Stevens Creek, the ponds are design to treat runoff to level 1. Water quality impacts from the development will be mitigated by appropriate design elements within the storm ponds. The extended detention



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2.	ISSUED FOR CITY COMMENT	JAN. 03/02	MJG
1.	ISSUED FOR COMMENT	DEC. 12/01	MJG

REGISTERED PROFESSIONAL ENGINEER J. D. McMANUS PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER UDO BOEHME PROVINCE OF ONTARIO

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David McManus Engineering Ltd.
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Nepean Ontario, K2H 5W6
E-mail: mcmanus@dmel.on.ca
Ph. 225-1929 Fax 225-7330

BASEPLAN	DME
DESIGN	UB/MJG
CHECKED	UB
CAD	GSC
PROJ MGR	UB
APPROVED	JDM

SCALE 1:1500
0 7.5 15 22.5 30
HORIZONTAL

PRI-TEC INTERNATIONAL INC.
JORDEL ACRES
TOWNSHIP OF RIDEAU

AGRI-INDUSTRIAL LOTS - GRADING PLAN

PROJECT No	1935
SURVEY BY	DME
DATE	JANUARY 2001
DRAWING No	1935-GR1

wet ponds with fore bays will reduce suspended solids concentrations to a level that can be released to a type 1 fish habitat.

Due to grading and location constraints, runoff from drainage area #4 will enter at the east end of the north pond without entering the fore bay. This represents approximately 4.3ha or 14% of the total 29.9ha area tributary to the North pond. The same scenario occurs at the south pond. Drainage area #2, which represents 3.5ha or 18% of the total 19.9ha tributary area to the south pond, enters the at the east end of the pond bypassing the fore bay. See drawing 1935-STM.

Since level 1protection design criteria is being used for the remaining tributary areas, which represents a 66% increase from the level 2 protection, the quality of the outflow from the pond will be well above the level 2 protection required.

Pond	Feature	Quantity	Comment
North	**Area**	1.2 Ha	Pond and forebay
	Volume Permanent Pool Extended Storage	 8280m^3 (6279m^3 required) 1215 m^3 (1196m^3 required)	 elev. 87.10 elev. 87.23
	Sideslopes	5:1 4:1	-Adjacent to pond -Within pond (sideslopes not maintained)
	Outlet Orifice (quality) Orifice (quantity)	 180 mm dia. 500 mm dia.	 -Controlled to 25 L/s -Designed to attenuate the 5 year runoff to a coefficient of 0.4
	Pond depth	1.2 m	
South	**Area**	0.67 Ha	Pond and forebay
	Volume Permanent Pool Extended Storage	 4200m^3 (4179m^3 required) 1290 m^3 (796 m^3 required)	 elev. 86.20 elev. 86.50
	Sideslopes	5:1 4:1	-Adjacent to pond -Within pond (sideslopes not maintained)
	Outlet Orifice (quality) Orifice (quantity)	 150 mm dia. 450 mm dia.	 -Controlled to 28 L/s -Designed to attenuate the 5 year runoff to a coefficient of 0.45
	Pond depth	1.5 m	

-Table 1- SWM Pond Design Elements and Assumptions

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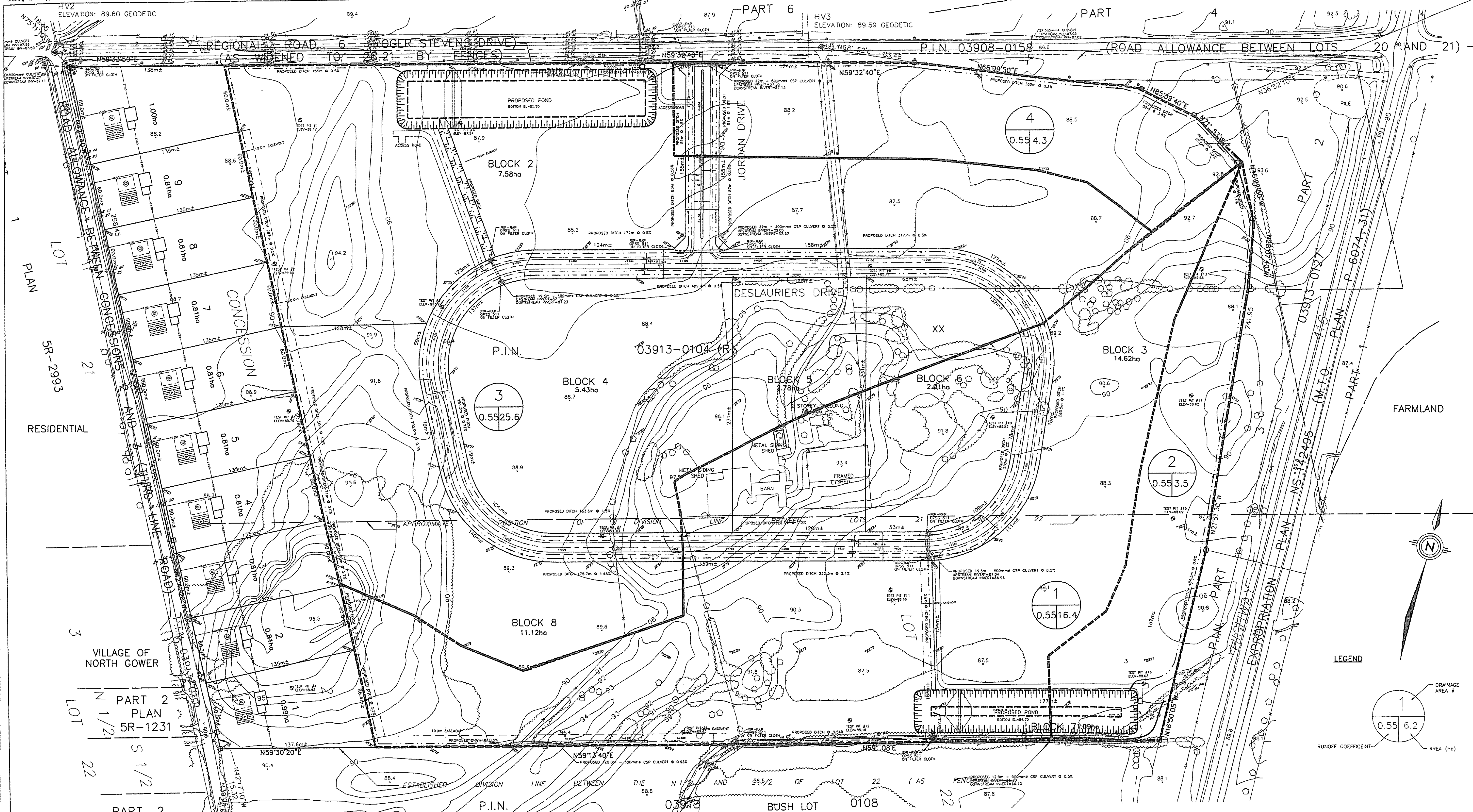

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No.	REVISION	DATE	BY

REGISTERED PROFESSIONAL ENGINEER
J. D. McMANUS
PROVINCE OF ONTARIO

DME Ltd.
David McManus Engineering Ltd.
200 - 39 Camelot Drive
Nepean Ontario, K2H 5W6
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Ph. 225-1929 Fax 225-7330

BASEPLAN	DME
DESIGN	JLS
CHECKED	JDM
CAD	LMV
PROJ. MGR.	JLS
APPROVED	JDM

SCALE 1:1500
0 15 30 45 60
HORIZONTAL

PRI-TEC INTERNATIONAL INC.
JORDEL ACRES
TOWNSHIP OF RIDEAU

STORM DRAINAGE AREA PLAN

PROJECT No.	1935
SURVEY BY	DME
DATE	JANUARY 2001
DRAWING No.	1935-STM

Last saved by:

3.3 Water Quantity

Attenuating flows to pre-development levels will mitigate impacts from increased run-off due to development. This will require the construction of two stormwater management wet pond facilities. Both ponds will provide storage for the 5 and 100-year events.

Design of the north pond assumes that development of the tributary areas can achieve a post development runoff coefficient of 0.4 with on site stormwater management. Design of the south pond assumes a post development runoff coefficient of 0.45 with on site stormwater management. The reasoning for the varying runoff coefficients is the allowable minimum lot size based on the zoning. Areas tributary to the north and south ponds have a minimum allowable lot size of 1.0ha and 1.5ha respectively.

The following Table 2 summarizes the five and one hundred year design storm flows and storage requirements.

Design Storm (return period)	Pre-development flow rate (cms)	Post-Development flow rate (cms)	Approximate Storage Required (m^3)
North Pond			
5 year	0.22	2.6	5,368
100 year	1.0	5.7	11,040
South Pond			
5 year	0.2	2.1	3,989
100 year	1.0	4.5	7,866

-Table 2 (see SWMHYMO output files in Appendix B)

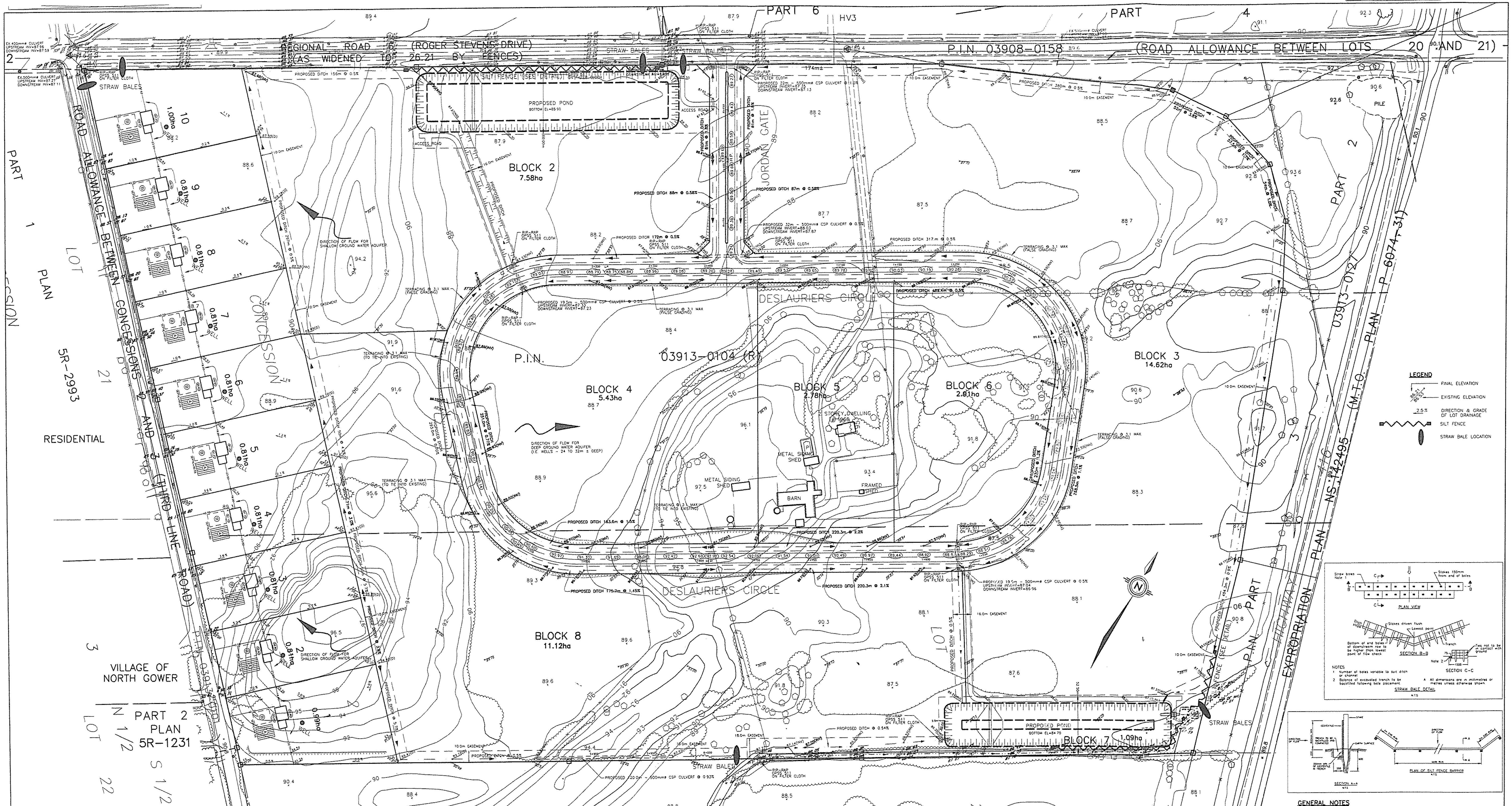
3.4 Floodplain Infill/Impacts

The issue of floodplain infill and loss of storage has been discussed with RVCA. The discussions have centred on the fact that some of the areas that fall within the floodplain elevations are not directly connected to an affected watercourse. Discussions that impact on loss of storage within the floodplain are on going with RVCA.

Design considerations with respect to the floodplain impact have been addressed by ensuring that finished floor elevations within the development are a minimum of 0.3m above the established floodplain elevation. Flap gates will be installed at the outlet control manholes in order to minimize flooding impact on the storm ponds. Details on the flap gates are included in Appendix D.

4.0 BEST MANAGEMENT PRACTICES

In order to follow the guidelines set by the Ministry of Environment, for Stormwater Best Management Practices, the following controls have been implemented in the design of the subdivision:



GENERAL NOTES

1. THE CONTRACTOR SHALL INSPECT AND MAINTAIN THE SEDIMENT CONTROL MEASURES ON A DAILY BASIS DURING CONSTRUCTION
2. THE CONTRACTOR SHALL STOCKPILE MATERIALS IN AREAS AS DIRECTED BY THE ENGINEER.

NOTE
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

PRELIMINARY

NOT FOR CONSTRUCTION

No.	REVISION	DATE	BY
2.	ISSUED FOR CITY COMMENT	JAN. 03/02	MJG
1.	ISSUED FOR COMMENT	DEC. 12/01	MJG

REGISTERED PROFESSIONAL ENGINEER — J. D. McMANUS — PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER — UDO BOEHME — PROVINCE OF ONTARIO

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BASEPLAN	DME
DESIGN	UB/MJG
CHECKED	UB
CAD	GSC
PROJ. MGR.	UB
APPROVED	JDM

SCALE 1:1500
0 15 30 45 60

PRI-TEC INTERNATIONAL INC.
JORDEL ACRES
TOWNSHIP OF RIDEAU

EROSION AND SEDIMENT CONTROL PLAN

PROJECT No	1935
SURVEY BY	DME
DATE	JANUARY 2001
DRAWING No	1935-SC

- Down spouts from roof areas will outlet to grassed areas
- Sediment and erosion control devices are to be deployed, as illustrated in Drawing No. 1935-SC, and maintained through all Phases of construction.
- Side slope areas around the SWM pond are not to be maintained to maximize the effectiveness of vegetative filtration.

4.1 Maintenance and Operation

The operation and maintenance will depend ultimately upon the actual conditions that arise during the operation of the facility. However, based on preliminary determinations shown in Appendix C, the forebay of the facility will have to be maintained every 10+ years by removal of the 0.50 metres of sediment that is anticipated over that time period. The pond will be drained by pumping. The developer shall maintain the pond during the construction period and the Township of Rideau will assume maintenance responsibilities after the one-year warranty period has expired.

5.0 CONCLUSIONS

The proposed development has been designed to control storm flows to pre-development conditions. The cost effective implementation of quantity and quality controls will affect the success of the development. As mentioned, the development of the land will occur in four separate phases. The required quantity and quality controls for the phased development will be attended at each Phase of construction. This will be achieved with the construction of a Stormwater Management Facility to attain the desired level of service (type 1 fish habitat) for all three phases.

This report satisfactorily addresses the method by which this site will meet the overall storm water detention requirements

Prepared by
David McManus Engineering Ltd.

Reviewed by

Michael J Green B.Sc

Udo Boehme, P.Eng.

LICENSED PROFESSIONAL ENGINEER
UDO BOEHME
PROVINCE OF ONTARIO

APPENDIX A

Design elements for Stormwater Management ponds
Design details

December 14, 2002

Memo

To: Mike Green, DMEL
From: Paul Frigon, PSR

Re: Jordel SWM

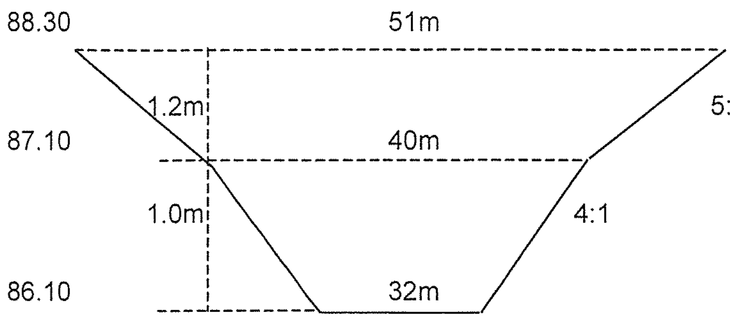
Revised North Pond – changes in bold italics

1. SWM for the North Pond contains the following elements:
   a) use existing land use draining 29.9 ha to the pond has hydrologic characteristics as indicated on attached sheets
   b) proposed land use of 85% impervious. We assume a ***27%*** impervious (equivalent C=0.4) achieved by onsite SWM
   c) runoff from proposed land use constrained by ***500***mmdia CSP under Roger Stevens. (600 l/s at h=***1.2*** +/-). Post development flow attenuated to this capacity
   d) SWM modeled using SWMHYMO
   e) Water quality SWM for 85% impervious and TYPE 1 fish habitat
   f) proposed SWM pond with characteristics as indicated in on attached sheets including a top-width of ***51***m at elevation ***88.3*** and a length of ***230***m (forebay length of 50m).

2. SWM for the South Pond contains the following elements:
   a) use existing land use draining 19.9ha to the pond has hydrologic characteristics as indicated on attached sheets
   b) proposed land use of 85% impervious. We assume a 35% impervious (equivalent C=0.45) achieved by onsite SWM
   c) runoff from proposed land use attenuated to predevelopment levels
   d) SWM modeled using SWMHYMO
   e) Water quality SWM for 85% impervious and TYPE 1 fish habitat
   f) proposed SWM pond with characteristics as indicated in on attached sheets including a top-width of *38*m at elevation *88.0* and a length of *200*m (forebay length of 40m).

POND DETAILS

North Pond

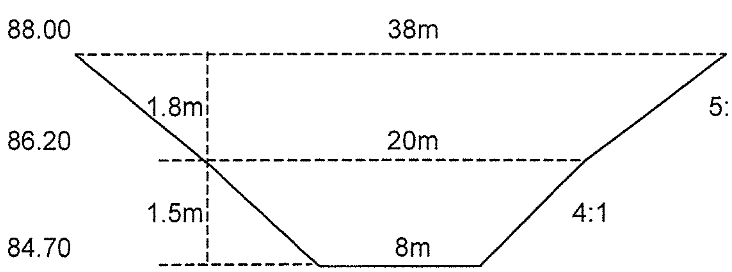


5:1 Forebay 50m long
Pond (including forebay) 260m long

Permanent Pool (PP) storage = 36x230x1 = 8280m^3
Extended Detention (ED) storage = 1215m^3
Quantity Storage = 45.5x230x1.2 = 12558m^3

DA = 29.9ha
Existing hydrology - SCSHYD
CN = 70 Tp = 0.6hr
Developed hydrology - STANDHYD
%imp = 27 Slope = 0.5%
C = 0.40

South Pond



5:1 Forebay 40m long
Pond (including forebay) 200m long

Permanent Pool (PP) storage = 14x200x1.5 = 4200m^3
Extended Detention (ED) storage = 1290m^3
Quantity Storage = 29x200x1.8 = 10440m^3

DA = 19.9ha
Existing hydrology - SCSHYD
CN = 70 Tp = 0.3hr
Developed hydrology - STANDHYD
%imp = 35 Slope = 0.75%
C = 0.45

JORDEL ACRES

SWM Pond Design Elements

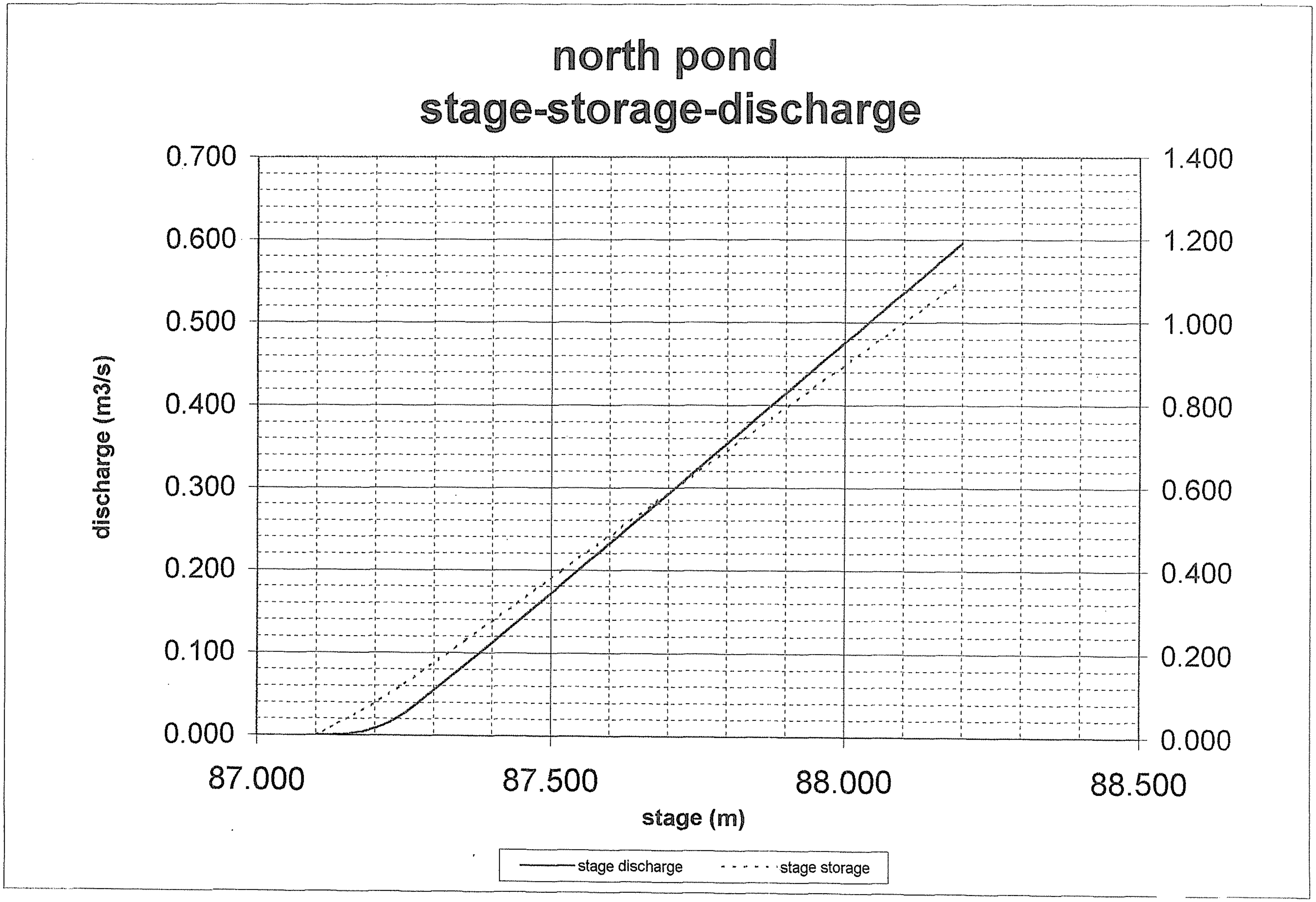
POND LEVELS (m)				
	CONDITION			
	Normal wl	ED	5 year	100 year
North Pond	87.10	87.23	87.60	88.30
South Pond	86.20	86.50	87.00	87.55

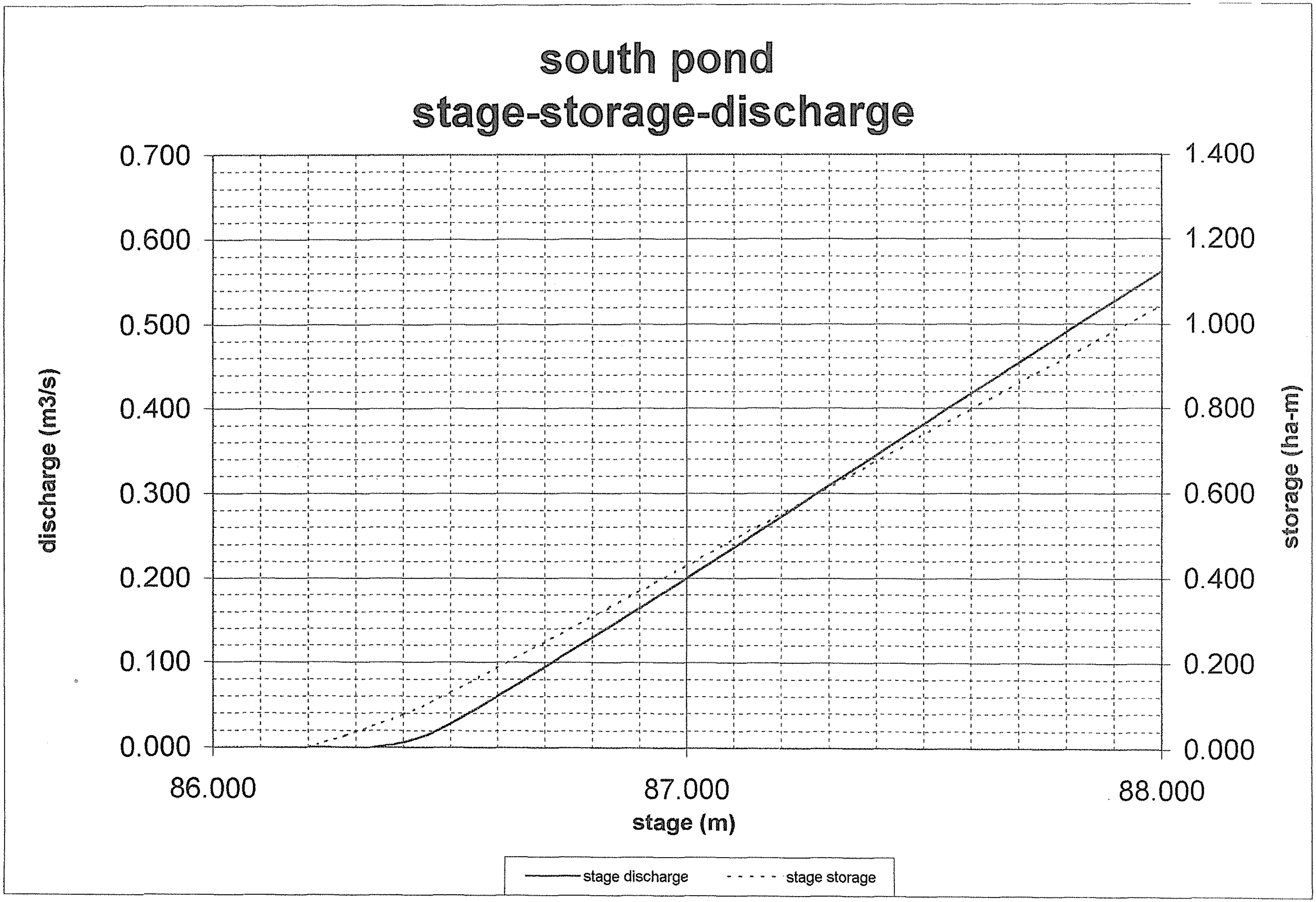
FLOWS (m^3/S)			
		Return Period	
Location	**Condition**	**5 year**	**100 year**
North	Pre-development	0.22	1.00
Pond	Post-development in	2.60	5.70
	Post-development out	0.27	0.60
South	Pre-development	0.20	1.00
Pond	Post-development in	2.10	4.50
	Post-development out	0.20	0.40

JORDEL ACRES
SWM Pond Design Elements

OUTLET DETAILS	$\frac{\text{(quality)}}{\text{(quantity)}}$			
	d (m)	Q (m^3/S)	A_o (m^2)	ϕ (m)
North	0.130	0.027	0.025	0.180
Pond	1.200	0.596	0.196	0.500
South	0.300	0.028	0.018	0.150
Pond	1.800	0.562	0.150	0.450

STAGE AREA (for drawdown)			
	Stage (m)	d (m)	Area (m^2)
North	87.10	0.00	9 200
Pond	87.70	0.60	10 580
	88.30	1.20	11 960
South	86.20	0.00	4 000
Pond	87.10	0.90	5 800
	88.00	1.80	7 600


north pond
stage-storage-discharge
0.700
0.600
0.500
0.400
0.300
0.200
0.100
0.000
discharge (m3/s)
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
87.000
87.500
88.000
88.500
stage (m)
stage discharge
stage storage


south pond
stage-storage-discharge
0.700
0.600
0.500
0.400
0.300
0.200
0.100
0.000
discharge (m3/s)
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
storage (ha-m)
86.000
87.000
88.000
stage (m)
stage discharge
stage storage

APPENDIX B
SWMHYMO input and output files

```
2
*jordel acres - Steven Creek
*Existing and future drainage conditons
*PSR Group Ltd. - october 2001
START                RAINFALL BEGINNING @ 0.0 HRS
*
*#
*# 100 YEAR 4hour chicago rainfall
*#
*24 years data - Ottawa International airport
*
READ STORM              "100co.stm"
*
*
*#
*#  NORTH POND - 100 year
*#
DESIGN SCSHYD             1              116 5       29.9ha 0.0  70 0.6Tp       -1
DESIGN STANDHYD           1              112 5       29.9ha .27 .27 0.0dwf 1horton 0.5%slope -1
ROUTE RESERVOIR     idout=4 nhyd="north pond" idin=1  dt=2min
                     flow (cms)    storage (ha-m)
                       0              0
                        .027           .125
                        .596          1.104
                       -1              -1
*SAVE HYD                 1 1 -1 "Nin" "100 year flows - into north pond"
*SAVE HYD                 4 1 -1 "Nout" "100 year flows - out of north pond"
*#
*#  SOUTH POND - 100 year
*#
DESIGN SCSHYD             1              117 5       19.9ha 0.0  70 0.3Tp       -1
DESIGN STANDHYD           1              113 5       19.9ha .35 .35 0.0dwf 1hortonloss 0.75%slope
-1
ROUTE RESERVOIR     idout=4 nhyd="south pond" idin=1  dt=2min
                     flow (cms)    storage (ha-m)
                        0              0
                       .028           .129
                       .562          1.044
                       -1              -1
*
*24 years data - Ottawa International airport
*#
*#5 YEAR 4 hour Chicago Rainfall
*#
READ STORM                "5co.stm"
*#
*#  NORTH POND - 5 year
*#
DESIGN SCSHYD             1              116 5       29.9ha 0.0  70 0.6Tp       -1
DESIGN STANDHYD           1              112 5       29.9ha .27 .27 0.0dwf 1hortonloss 0.75%slope
-1
ROUTE RESERVOIR     idout=4 nhyd="north pond" idin=1  dt=2min
                     flow (cms)    storage (ha-m)
                        0              0
                        .027           .125
                        .596          1.104
                        -1              -1
*#
*#  SOUTH POND - 5 year
*#
DESIGN SCSHYD             1              117 5       19.9ha 0.0  70 0.3Tp       -1
DESIGN STANDHYD           1              113 5       19.9ha .35 .35 0.0dwf 1hortonloss 0.75%slope
-1
ROUTE RESERVOIR     idout=4 nhyd="south pond" idin=1  dt=2min
                     flow (cms)    storage (ha-m)
                        0              0
                        .028           .129
                        .562          1.044
                        -1              -1
FINISH
```

INPUT

output

```
RUN:COMMAND#
001:0001------------------------------------------------------------------------
    START
     [TZERO =    .00 hrs on        0]
     [METOUT=   2    (1=imperial, 2=metric output)]
     [NSTORM=   0 ]
     [NRUN  =   1 ]
#
# 100 YEAR 4hour chicago rainfall
#
001:0002------------------------------------------------------------------------
    READ STORM
     Filename = 100co.stm
     Comment  = 100 year Chicago Storm - ottawa
     [SDT=10.00:SDUR=   4.00:PTOT=  76.13]
#
#  NORTH POND - 100 year
#
001:0003---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    DESIGN SCSHYD      01:000116    29.90    1.036 No_date    2:05   18.11 .238
     [CN= 70.0: N= 5.00]
     [Tp=  .60:DT= 5.00]
001:0004---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    DESIGN STANDHYD    01:000112    29.90    5.736 No_date    1:25   46.23 .607
     [XIMP=.27:TIMP=.27]
     [SLP= .50:DT= 5.00]
     [LOSS= 1 : HORTONS]
001:0005---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    ROUTE RESERVOIR -> 01:000112    29.90    5.736 No_date    1:25   46.23  n/a
     [RDT= 1.67] out<- 04:north     29.90     .596 No_date    2:17   46.23  n/a
    {MxStoUsed=.1104E+01}
#
#  SOUTH POND - 100 year
#
001:0006---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    DESIGN SCSHYD      01:000117    19.90    1.021 No_date    1:40   18.11 .238
     [CN= 70.0: N= 5.00]
     [Tp=  .30:DT= 5.00]
001:0007---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
 *  DESIGN STANDHYD    01:000113    19.90    4.542 No_date    1:20   49.42 .649
     [XIMP=.35:TIMP=.35]
     [SLP= .75:DT= 5.00]
     [LOSS= 1 : HORTONS]
001:0008---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    ROUTE RESERVOIR -> 01:000113    19.90    4.542 No_date    1:20   49.42  n/a
     [RDT= 1.67] out<- 04:south     19.90     .412 No_date    2:13   49.42  n/a
    {MxStoUsed=.7866E+00}
#
#5 YEAR 4 hour Chicago Rainfall
#
001:0009------------------------------------------------------------------------
    READ STORM
     Filename = 5co.stm
     Comment  = 5year Chicago Storm - ottawa
     [SDT=10.00:SDUR=   4.00:PTOT=  48.06]
#
#  NORTH POND - 5 year
#
001:0010---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    DESIGN SCSHYD      01:000116    29.90     .221 No_date    2:15    5.11 .106
     [CN= 70.0: N= 5.00]
     [Tp=  .60:DT= 5.00]
001:0011---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    DESIGN STANDHYD    01:000112    29.90    2.579 No_date    1:20   23.00 .479
     [XIMP=.27:TIMP=.27]
     [SLP= .75:DT= 5.00]
     [LOSS= 1 : HORTONS]
001:0012---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    ROUTE RESERVOIR -> 01:000112    29.90    2.579 No_date    1:20   23.00  n/a
     [RDT= 1.67] out<- 04:north     29.90     .266 No_date    2:25   23.00  n/a
    {MxStoUsed=.5368E+00}
#
#  SOUTH POND - 5 year
#
001:0013---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    DESIGN SCSHYD      01:000117    19.90     .192 No_date    1:50    5.11 .106
     [CN= 70.0: N= 5.00]
     [Tp=  .30:DT= 5.00]
001:0014---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    DESIGN STANDHYD    01:000113    19.90    2.147 No_date    1:20   25.66 .534
     [XIMP=.35:TIMP=.35]
     [SLP= .75:DT= 5.00]
     [LOSS= 1 : HORTONS]
001:0015---------------ID:NHYD-------AREA----QPEAK-TpeakDate_hh:mm----R.V.-R.C.-
    ROUTE RESERVOIR -> 01:000113    19.90    2.147 No_date    1:20   25.66  n/a
```

```
     [RDT= 1.67] out<- 04:south      19.90      .185 No_date    2:25   25.66  n/a
     {MxStoUsed=.3989E+00}
001:0016-----------------------------------------------------------------------
     FINISH
```

```
--------------------------------------------------------------------------------
001:0001------------------------------------------------------------------------
*jordel acres - Steven Creek
*Existing and future drainage conditons
*PSR Group Ltd. - october 2001
--------------------
| START             |  Project  dir.: c:\WATER\SWMHYMO\PROJECTS\JORDEL\
--------------------  Rainfall dir.: c:\WATER\SWMHYMO\PROJECTS\JORDEL\
    TZERO =    .00 hrs on          0
    METOUT=   2 (output = METRIC)
    NRUN  = 001
    NSTORM=   0
--------------------------------------------------------------------------------
001:0002------------------------------------------------------------------------
*
*#
*# 100 YEAR 4hour chicago rainfall
*#
*24 years data - Ottawa International airport
*
--------------------
| READ STORM        |     Filename: c:\WATER\SWMHYMO\PROJECTS\JORDEL\100co.s
| Ptotal=  76.13 mm|    Comments: 100 year Chicago Storm - ottawa
--------------------
              TIME    RAIN |   TIME     RAIN |   TIME    RAIN |   TIME    RAIN
               hrs   mm/hr |    hrs    mm/hr |    hrs   mm/hr |    hrs   mm/hr
               .17   3.950 |   1.17   42.930 |   2.17  10.740 |   3.17   4.820
               .33   4.620 |   1.33  179.790 |   2.33   8.890 |   3.33   4.430
               .50   5.580 |   1.50   57.530 |   2.50   7.580 |   3.50   4.090
               .67   7.090 |   1.67   28.390 |   2.67   6.620 |   3.67   3.810
               .83   9.800 |   1.83   18.450 |   2.83   5.880 |   3.83   3.570
              1.00  16.000 |   2.00   13.590 |   3.00   5.300 |   4.00   3.350

--------------------------------------------------------------------------------
001:0003------------------------------------------------------------------------
*
*
*#
*#  NORTH POND - 100 year
*#
----------------------
| DESIGN SCSHYD       |   Area     (ha)=   29.90   Curve Number   (CN)=70.00
| 01:000116 DT= 5.00 |   Ia       (mm)=  21.771   # of Linear Res.(N)= 5.00
----------------------   U.H. Tp(hrs)=    .600

     Ia as  0.2xS     (mm)=   21.771
     Unit Hyd Qpeak  (cms)=    2.748

     PEAK FLOW       (cms)=    1.036 (i)
     TIME TO PEAK    (hrs)=    2.083
     RUNOFF VOLUME    (mm)=   18.106
     TOTAL RAINFALL   (mm)=   76.133
     RUNOFF COEFFICIENT   =     .238

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

--------------------------------------------------------------------------------
001:0004------------------------------------------------------------------------
----------------------
| DESIGN STANDHYD     |   Area     (ha)=   29.90
| 01:000112 DT= 5.00 |   Total Imp(%)=   27.00   Dir. Conn.(%)=   27.00
----------------------

                              IMPERVIOUS    PERVIOUS (i)
     Surface Area      (ha)=       8.07        21.83
     Dep. Storage      (mm)=        .80         1.50
     Average Slope      (%)=        .50          .50
     Length             (m)=     446.47        40.00
     Mannings n            =       .013         .250

     Max.eff.Inten.(mm/hr)=     179.79       113.52
                over (min)        5.00        15.00
     Storage Coeff.  (min)=       6.10 (ii)   16.27 (ii)
     Unit Hyd. Tpeak (min)=       5.00        15.00
     Unit Hyd. peak  (cms)=        .19          .07
                                                          *TOTALS*
     PEAK FLOW       (cms)=       3.41         3.92        5.736 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.50        1.417
     RUNOFF VOLUME    (mm)=      75.33        35.46       46.229
     TOTAL RAINFALL   (mm)=      76.13        76.13       76.133
     RUNOFF COEFFICIENT   =        .99          .47         .607

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 50.00          K  (1/hr)=   2.00
            Fc    (mm/hr)=  7.50     Cum.Inf.   (mm)=    .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
```

```
        THAN THE STORAGE COEFFICIENT.
  (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
001:0005----------------------------------------------------------------------
--------------------
| ROUTE RESERVOIR    |     Requested routing time step =  2.0 min.
|  IN>01:(000112)    |
| OUT<04:(north )    |     =========  OUTLFOW STORAGE TABLE  =========
--------------------       OUTFLOW    STORAGE   | OUTFLOW    STORAGE
                            (cms)     (ha.m.)   |  (cms)     (ha.m.)
                             .000  .0000E+00    |    .596  .1104E+01
                             .027  .1250E+00    |    .000  .0000E+00

     ROUTING RESULTS              AREA      QPEAK      TPEAK        R.V.
     --------------------         (ha)      (cms)      (hrs)        (mm)
     INFLOW >01: (000112)        29.90      5.736      1.417      46.229
     OUTFLOW<04: (north )        29.90       .596      2.278      46.228

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=   10.386
                   TIME SHIFT OF PEAK FLOW         (min)=    51.67
                   MAXIMUM  STORAGE   USED       (ha.m.)=.1104E+01

------------------------------------------------------------------------------
001:0006----------------------------------------------------------------------
*SAVE HYD                  1 1 -1 "Nin" "100 year flows - into north pond"
*SAVE HYD                  4 1 -1 "Nout" "100 year flows - out of north pond"
*#
*#  SOUTH POND - 100 year
*#
---------------------
| DESIGN SCSHYD       |    Area     (ha)=    19.90   Curve Number   (CN)=70.00
| 01:000117 DT= 5.00 |    Ia       (mm)=   21.771   # of Linear Res.(N)= 5.00
---------------------     U.H. Tp(hrs)=     .300

     Ia as  0.2xS     (mm)=   21.771
     Unit Hyd Qpeak  (cms)=    3.657

     PEAK FLOW       (cms)=    1.021 (i)
     TIME TO PEAK    (hrs)=    1.667
     RUNOFF VOLUME    (mm)=   18.106
     TOTAL RAINFALL   (mm)=   76.133
     RUNOFF COEFFICIENT   =     .238

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

------------------------------------------------------------------------------
001:0007----------------------------------------------------------------------
---------------------
| DESIGN STANDHYD     |    Area     (ha)=    19.90
| 01:000113 DT= 5.00 |    Total Imp(%)=    35.00   Dir. Conn.(%)=    35.00
---------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        6.96        12.93
     Dep. Storage     (mm)=         .80         1.50
     Average Slope     (%)=         .75          .75
     Length            (m)=      364.23        40.00
     Mannings n           =        .013         .250

     Max.eff.Inten.(mm/hr)=      179.79       113.52
                over (min)         5.00        15.00
     Storage Coeff.  (min)=        4.78 (ii)   13.79 (ii)
     Unit Hyd. Tpeak (min)=        5.00        15.00
     Unit Hyd. peak  (cms)=         .22          .08
                                                          *TOTALS*
     PEAK FLOW       (cms)=        3.14         2.53        4.542 (iii)
     TIME TO PEAK    (hrs)=        1.33         1.50        1.333
     RUNOFF VOLUME    (mm)=       75.33        35.46       49.419
     TOTAL RAINFALL   (mm)=       76.13        76.13       76.133
     RUNOFF COEFFICIENT   =         .99          .47         .649
      *** WARNING: Storage Coefficient is smaller than DT!
                   Use a smaller DT or a larger area.

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 50.00           K  (1/hr)=  2.00
            Fc    (mm/hr)=  7.50     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
001:0008----------------------------------------------------------------------
--------------------
| ROUTE RESERVOIR    |     Requested routing time step =  2.0 min.
|  IN>01:(000113)    |
```

```
| OUT<04:(south )    |      =========  OUTLFOW STORAGE TABLE  =========
 ----------------------      OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
                               (cms)     (ha.m.)   |   (cms)     (ha.m.)
                                .000  .0000E+00   |     .562  .1044E+01
                                .028  .1290E+00   |     .000  .0000E+00

     ROUTING RESULTS               AREA      QPEAK      TPEAK        R.V.
     --------------------          (ha)      (cms)      (hrs)        (mm)
     INFLOW >01: (000113)         19.90      4.542      1.333      49.419
     OUTFLOW<04: (south )         19.90       .412      2.222      49.418

                    PEAK   FLOW   REDUCTION [Qout/Qin](%)=     9.066
                    TIME SHIFT OF PEAK FLOW          (min)=    53.33
                    MAXIMUM  STORAGE   USED         (ha.m.)=.7866E+00

--------------------------------------------------------------------------------
001:0009------------------------------------------------------------------------
*
*24 years data - Ottawa International airport
*#
*#5 YEAR 4 hour Chicago Rainfall
*#
--------------------
| READ STORM       |     Filename: c:\WATER\SWMHYMO\PROJECTS\JORDEL\5co.stm
| Ptotal=  48.06 mm|     Comments: 5year Chicago Storm - ottawa
--------------------
               TIME     RAIN |    TIME     RAIN |    TIME     RAIN |    TIME     RAIN
                hrs    mm/hr |     hrs    mm/hr |     hrs    mm/hr |     hrs    mm/hr
                .17    3.080 |    1.17   24.970 |    2.17    7.350 |    3.17    3.670
                .33    3.540 |    1.33  108.650 |    2.33    6.240 |    3.33    3.410
                .50    4.170 |    1.50   32.830 |    2.50    5.440 |    3.50    3.180
                .67    5.130 |    1.67   17.140 |    2.67    4.840 |    3.67    2.990
                .83    6.790 |    1.83   11.730 |    2.83    4.370 |    3.83    2.820
               1.00   10.360 |    2.00    9.000 |    3.00    3.990 |    4.00    2.670

--------------------------------------------------------------------------------
001:0010------------------------------------------------------------------------
*#
*#  NORTH POND - 5 year
*#
----------------------
| DESIGN SCSHYD      |    Area     (ha)=    29.90    Curve Number    (CN)=70.00
| 01:000116 DT= 5.00 |    Ia       (mm)=   21.771    # of Linear Res.(N)= 5.00
----------------------    U.H. Tp(hrs)=     .600

     Ia as  0.2xS      (mm)=    21.771
     Unit Hyd Qpeak   (cms)=     2.748

     PEAK FLOW        (cms)=      .221 (i)
     TIME TO PEAK     (hrs)=     2.250
     RUNOFF VOLUME     (mm)=     5.114
     TOTAL RAINFALL    (mm)=    48.060
     RUNOFF COEFFICIENT    =      .106

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

--------------------------------------------------------------------------------
001:0011------------------------------------------------------------------------
----------------------
| DESIGN STANDHYD    |    Area     (ha)=    29.90
| 01:000112 DT= 5.00 |    Total Imp(%)=    27.00    Dir. Conn.(%)=    27.00
----------------------
                               IMPERVIOUS     PERVIOUS (i)
     Surface Area      (ha)=        8.07          21.83
     Dep. Storage      (mm)=         .80           1.50
     Average Slope      (%)=         .75            .75
     Length             (m)=      446.47          40.00
     Mannings n            =        .013           .250

     Max.eff.Inten.(mm/hr)=       108.65          42.10
                over (min)          5.00          20.00
     Storage Coeff.  (min)=         6.61 (ii)     20.00 (ii)
     Unit Hyd. Tpeak (min)=         5.00          20.00
     Unit Hyd. peak  (cms)=          .18            .06
                                                              *TOTALS*
     PEAK FLOW        (cms)=        2.01           1.47          2.579 (iii)
     TIME TO PEAK     (hrs)=        1.33           1.58          1.333
     RUNOFF VOLUME     (mm)=       47.26          14.03         23.005
     TOTAL RAINFALL    (mm)=       48.06          48.06         48.060
     RUNOFF COEFFICIENT    =         .98            .29           .479

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 50.00          K  (1/hr)=  2.00
            Fc    (mm/hr)=  7.50     Cum.Inf.   (mm)=   .00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
```

```
            THAN THE STORAGE COEFFICIENT.
      (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

 --------------------------------------------------------------------------------
 001:0012------------------------------------------------------------------------
 ---------------------
 | ROUTE RESERVOIR    |    Requested routing time step =  2.0 min.
 |  IN>01:(000112)    |
 | OUT<04:(north )    |    =========  OUTLFOW STORAGE TABLE  =========
 ---------------------     OUTFLOW    STORAGE   | OUTFLOW    STORAGE
                             (cms)     (ha.m.)  |   (cms)     (ha.m.)
                              .000  .0000E+00   |     .596  .1104E+01
                              .027  .1250E+00   |     .000  .0000E+00

      ROUTING RESULTS              AREA     QPEAK      TPEAK       R.V.
      --------------------         (ha)     (cms)      (hrs)       (mm)
      INFLOW >01: (000112)        29.90     2.579      1.333     23.005
      OUTFLOW<04: (north )        29.90      .266      2.417     23.004

                     PEAK   FLOW   REDUCTION [Qout/Qin](%)=   10.327
                     TIME SHIFT OF PEAK FLOW         (min)=    65.00
                     MAXIMUM  STORAGE   USED       (ha.m.)=.5368E+00

 --------------------------------------------------------------------------------
 001:0013------------------------------------------------------------------------
 *#
 *#  SOUTH POND - 5 year
 *#
 ----------------------
 | DESIGN SCSHYD       |   Area      (ha)=   19.90   Curve Number    (CN)=70.00
 | 01:000117 DT= 5.00  |   Ia        (mm)=  21.771   # of Linear Res.(N)= 5.00
 ----------------------    U.H. Tp(hrs)=      .300

      Ia as  0.2xS      (mm)=    21.771
      Unit Hyd Qpeak   (cms)=     3.657

      PEAK FLOW        (cms)=      .192 (i)
      TIME TO PEAK     (hrs)=     1.833
      RUNOFF VOLUME     (mm)=     5.114
      TOTAL RAINFALL    (mm)=    48.060
      RUNOFF COEFFICIENT   =       .106

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

 --------------------------------------------------------------------------------
 001:0014------------------------------------------------------------------------
 ----------------------
 | DESIGN STANDHYD     |   Area    (ha)=   19.90
 | 01:000113 DT= 5.00  |   Total Imp(%)=   35.00   Dir. Conn.(%)=   35.00
 ----------------------
                              IMPERVIOUS    PERVIOUS (i)
      Surface Area      (ha)=       6.96        12.93
      Dep. Storage      (mm)=        .80         1.50
      Average Slope      (%)=        .75          .75
      Length             (m)=     364.23        40.00
      Mannings n            =       .013         .250

      Max.eff.Inten.(mm/hr)=      108.65        42.10
                 over (min)        5.00        20.00
      Storage Coeff.  (min)=       5.85 (ii)   19.24 (ii)
      Unit Hyd. Tpeak (min)=       5.00        20.00
      Unit Hyd. peak  (cms)=        .20          .06
                                                             *TOTALS*
      PEAK FLOW        (cms)=       1.80          .89          2.147 (iii)
      TIME TO PEAK     (hrs)=       1.33         1.58          1.333
      RUNOFF VOLUME     (mm)=      47.26        14.03         25.663
      TOTAL RAINFALL    (mm)=      48.06        48.06         48.060
      RUNOFF COEFFICIENT   =         .98          .29           .534

        (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo    (mm/hr)= 50.00         K  (1/hr)=  2.00
            Fc    (mm/hr)=  7.50      Cum.Inf.   (mm)=   .00
       (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
            THAN THE STORAGE COEFFICIENT.
      (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

 --------------------------------------------------------------------------------
 001:0015------------------------------------------------------------------------
 ---------------------
 | ROUTE RESERVOIR    |    Requested routing time step =  2.0 min.
 |  IN>01:(000113)    |
 | OUT<04:(south )    |    =========  OUTLFOW STORAGE TABLE  =========
 ---------------------     OUTFLOW    STORAGE   | OUTFLOW    STORAGE
                             (cms)     (ha.m.)  |   (cms)     (ha.m.)
                              .000  .0000E+00   |     .562  .1044E+01
```

```
                 .028  .1290E+00   |     .000  .0000E+00

ROUTING RESULTS              AREA     QPEAK      TPEAK       R.V.
--------------------         (ha)     (cms)      (hrs)       (mm)
INFLOW >01: (000113)        19.90     2.147      1.333     25.663
OUTFLOW<04: (south )        19.90      .185      2.417     25.662

              PEAK   FLOW   REDUCTION [Qout/Qin](%)=     8.638
              TIME SHIFT OF PEAK FLOW          (min)=    65.00
              MAXIMUM  STORAGE   USED       (ha.m.)=.3989E+00

--------------------------------------------------------------------------------
001:0016------------------------------------------------------------------------
      FINISH
```

APPENDIX C

Calculations for Extended Detention Wet Pond and Forebay
(Source: SWMP Planning and Design Manual)

APPENDIX C1 - Stormwater Management - Water Quality North Pond

Calculation For Extended Detention Wet Pond and Forebay

Source: SWMP Planning and Design Manual

1. Water quality storage requirements. Level 1 protection Wet Pond.

Light Industrial/Commercial 85% impervious @ 250 m^3/ha

$A = 29.9$ ha

$\therefore$ Permanent Pool (PP)storage = (250-40) m^3 x 29.9 ha = 6279 m^3

Extended Detention (ED)= 40 m^3 x 29.9 ha = 1,196 m^3

As designed, PP = 8280 m^3 > 6279

ED = 1218m^3 > 1196

$\therefore$ OK

2. Drawdown Time

The relationship between the pond surface area and pond depth can be established as follows: $A = C_2h + C_3$ can be determined where:

C_2 = slope coefficient from area-depth linear regression

C_3 = intercept from the area-depth linear regression

h = maximum elevation above centreline of orifice

A_1 Area	= 9,200 m^2	@	$h_1 = 0.0$ m
A_2 Area	= 10,580 m^2	@	$h_1 = 0.6$ m
A_3 Area	= 11,960 m^2	@	$h_1 = 1.2$ m

$\therefore C_3 = 9{,}200$

from $A = C_2h + C_3$

$10{,}580 = C_2\,(.45) + 9{,}200$

$\therefore C_2 = 3{,}067$

$$h = \frac{\text{Volume}}{\text{area}} = \frac{1200}{9{,}200} = 0.13 \text{ m}$$

Drawdown time can now be calculated using

$$t = \frac{0.66\, C_2\, h^{1.5} + 2C_3\, h^{0.5}}{2.75\, Ao} \qquad \frac{Ao = 0.025m^2 \text{ (assumed)}}{h = 0.13}$$

$$= 0.66 \times \frac{3{,}067\,(0.13)^{1.5}}{2.75 \times 0.025} + 2 \times 9{,}200(0.13)^{0.5}$$

= 97,877.7 sec or 27 hrs. > 24 hrs. $\therefore$ OK

3. **Orifice Flow**

$Q = VA$ where

or $h = 0.13$

$Q = CAo\,(2gh)^{0.5}$ $g = 9.81$

$Q = 0.63 \times 0.025\,(2 \times 9.81 \times 0.13)^{0.5}$ $C = 0.63$

$= 0.025\ m^3/sec$ $Ao = 0.025$

4. **Forebay Settling Length**

Where r = length/width=50/40=1.25

$D = \frac{(r\,Qp)^{0.5}}{(vs)^{0.5}} = \frac{(1.25 \times 0.025)^{0.5}}{(0.0003)^{0.5}} = 10.2\ m$ Q_p = peak flowrate from pond

$\therefore$ Required width = 10.2 m < 40 m $\therefore$OK

Designed Forebay is $\therefore$ OK

5. **Area Calculation**

Forebay Area 40 x 50 = 2000 m²

Pond Area 40 x 230 = 9,200 m²

$\frac{\text{Forebay Area}}{\text{Pond Area}}$ = . 22 < .33 $\therefore$OK

6. **Forebay Dispersion Length**

$D = \frac{8Q}{dxV_f}$ where Q =3.11 (for 5 yr event)

d = perm. pool depth(1.0m)

V_f= 0.5 (desired velocity at berm

$D = \frac{8x(3.11)}{(1.0)(0.5)}$

= 49.8 m >10.2

(settling length)

7. **Minimum Forebay Bottom Width**

Equation 3.5

$$\text{Min width} = \frac{D}{8}$$

$$= \frac{49.8}{8} = 6.2\text{ m}$$

6.2 m< 32m ∴ OK

8. **Clean Out Frequency**

<u>Annual Sediment Load</u>

(85%) Commercial 3.8 m^3/ha x 29.9 = 113.62 m^3

<u>Removal Efficiency</u>

80% of 113.62 m^3 = 90.9 m^3/year

TOTAL CAPACITY for sediment = (36x1.0x50) = 1600

∴ clean out frequency $\frac{1600}{90.9}$ = 18 years > 10 yrs ∴ OK

S

APPENDIX C1 - Stormwater Management - Water Quality South Pond

Calculation For Extended Detention Wet Pond and Forebay

Source: SWMP Planning and Design Manual

1. Water quality storage requirements. Level 1 protection Wet Pond.

Light Industrial/Commercial 85% impervious @ 190 m^3/ha

A = 19.9 ha

∴ Permanent Pool (PP)storage = (250-40) m^3 x 19.9 ha = 4,179 m^3

Extended Detention (ED)= 40 m^3 x 19.9 ha = 796 m^3

As designed, PP = 4200 m^3

ED = 1290m^3

∴ OK

2. Drawdown Time

The relationship between the pond surface area and pond depth can be established as follows: $A = C_2h + C_3$ can be determined where:

C_2 = slope coefficient from area-depth linear regression

C_3 = intercept from the area-depth linear regression

h = maximum elevation above centreline of orifice

A_1 Area	= 4,000 m^2	@	$h_1 = 0.0$ m
A_2 Area	= 5,800 m^2	@	$h_1 = 0.9$ m
A_3 Area	= 7,600 m^2	@	$h_1 = 1.8$ m

$\therefore C_3 = 4{,}000$

from $A = C_2h + C_3$

$7{,}600 = C_2\,(.45) + 4{,}000$

$\therefore C_2 = 2{,}000$

$$h = \frac{\text{Volume}}{\text{area}} = \frac{1200}{10{,}400} = 0.12 \text{ m}$$

Drawdown time can now be calculated using

$$t = \frac{0.66\, C_2\, h^{1.5} + 2C_3\, h^{0.5}}{2.75\, Ao} \qquad \frac{Ao = 0.018m^2 \text{ (assumed)}}{h = 0.3}$$

$$= 0.66 \times \frac{2000\,(0.3)^{1.5} + 2 \times 175\,(0.3)^{0.5}}{2.75 \times 0.018}$$

= 92,903 sec or 26 hrs.

3. Orifice Flow

$Q = VA$

or

$Q = CA_o (2gh)^{0.5}$

$Q = 0.63 \times 0.018 (2 \times 9.81 \times 0.3)^{0.5}$

$= 0.028\ m^3/sec$

where

$h = 0.3$

$g = 9.81$

$C = 0.63$

$A_o = 0.018$

4. Forebay Settling Length

Equation 3.3

$$D = \frac{(r\,Qp)^{0.5}}{(vs)^{0.5}} = \frac{(2 \times 0.009)^{0.5}}{(0.0003)^{0.5}} = 7.7m$$

where r = length/width =40/20=2:1

$\therefore$ Required width = 3.9 m

Designed Forebay is 40m x 20m $\therefore$ OK

5. Area Calculation

Forebay Area 20 x 40 = 800 m²

Pond Area 20 x 200 = 4,000 m²

$$\frac{\text{Forebay Area}}{\text{Pond Area}} = .02 < .33 \quad \therefore OK$$

6. Forebay Dispersion Length

$$D = \frac{8Q}{dxV_f}$$

where $Q = 2.1 m^3/s$

d = perm. pool depth= 1.5m

V_f= 0.5m/s (desired velocity berm)

$$D = \frac{8x(2.1)}{(1.5)(0.5)}$$

$= 22.4\ m > 7.7m$

7. Minimum Forebay Bottom Width

$$\text{Min width} = \frac{D}{8}$$

$$= \frac{22.4}{8} \quad = 2.8 < 8.0$$

$\therefore$ OK

8. Clean Out Frequency

<u>Annual Sediment Load</u>

(85%) Commercial 3.8 m^3/ha x 19.9 = 75.6 m^3

<u>Removal Efficiency</u>

80% of 75.6 m^3 = 60.5 m^3/year

TOTAL CAPACITY for sediment = (14x1.5x40) = 840m^3

$\therefore$ clean out frequency $\frac{840}{60.5}$ = 14 years > 10 yrs $\therefore$ OK

APPENDIX D

Armtec Flap Gate Model 10C
Details

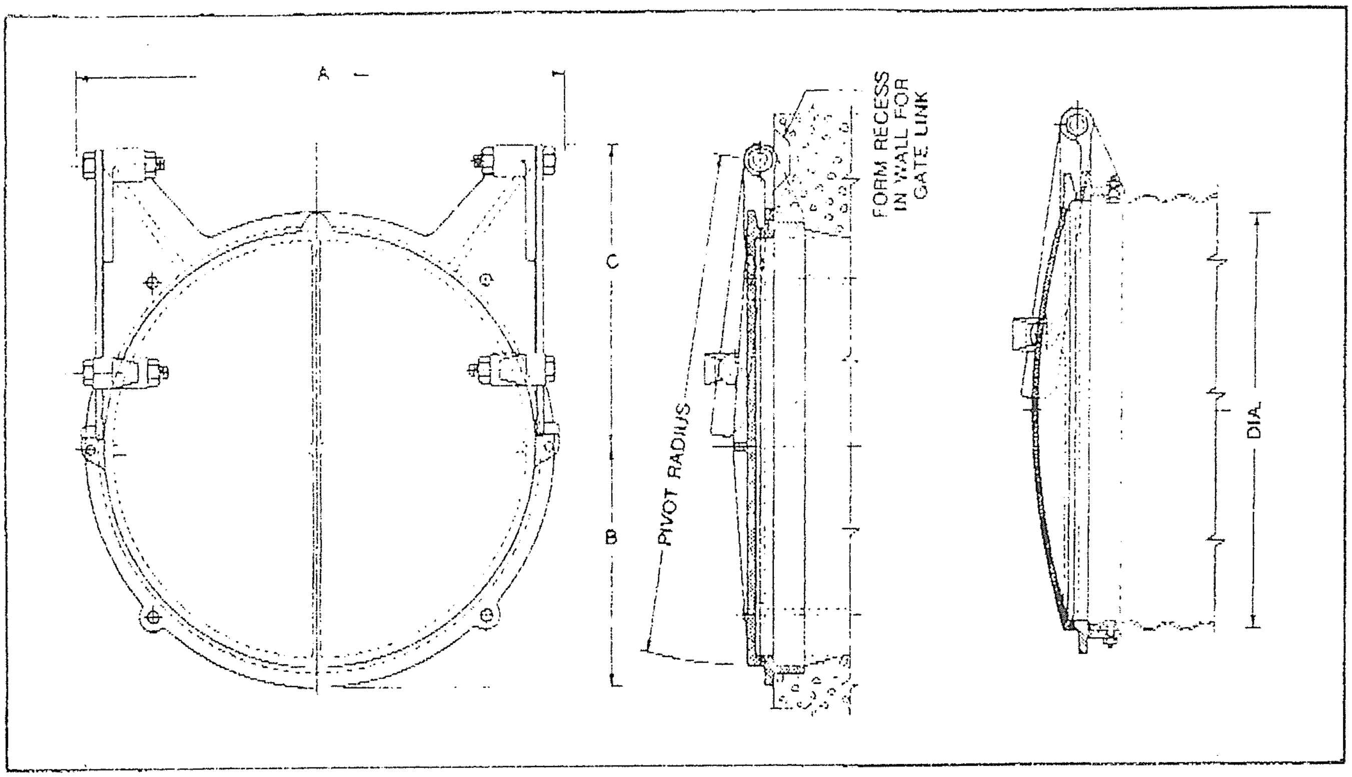
MODEL 10C FLAP GATE

- For seating heads to 3 metres
- Round opening
- Spigot back
- Cast iron seating surfaces

The Model 10C flap gate consists of the simplest possible design with double hinge action for heads to 3 metres. Pivot points are stationary. Ring and flap are of cast iron with galvanized steel hinge arms and assembly bolts and bronze bushings. Extension of the cast iron bosses of the flap over the top of the pivot arms limits the double hinge action, and prevents the bottom of the flap from folding inside the ring and wedging the gate in the open position.

This gate is made only in spigot back and normally attaches to corrugated steel pipe. The spigot back gate may be embedded in the concrete at the time it is poured; however, a flat back gate anchored and then grouted to the wall is recommended. See "Model 20C Gates" for this application.

This gate opens under a minimum head differential, yet is positive closing under a few mm of water on the face of the gate. A lifting eye is cast integrally with the flap to permit manual operation.

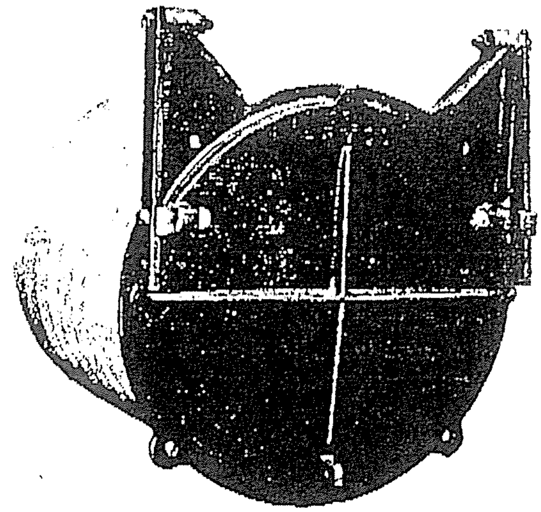


(All Dimensions in mm)

SIZE	A *	B	C	PIVOT RADIUS
152	222	95	121	191
203	273	127	165	260
254	324	159	203	318
305	387	184	235	381
381	476	229	292	470
457	552	266	349	565
533	629	311	406	660
610	705	349	457	756
762	870	432	552	921
914	1054	521	667	1111
1067	1207	597	787	1308
1219	1372	686	902	1492

*Maximum width of gate may occur at top or on horizontal center line. "A" dimension is shown for maximum horizontal width of gate.

SPECIFICATIONS FOR MODEL 10C FLAP GATE



General
Flap gates shall be Armco Model 10C or approved equal. Similar installations shall have operated successfully for five years or more. All component parts shall be of the type material shown in the Materials section of this specification.

Seat
The spigot back seat shall be one-piece cast iron with a raised section around the perimeter of the waterway to provide the seating face. The seat shall be shaped to provide two pivot bosses extended above the top of the waterway opening.

Cover
The cover shall be one-piece cast iron with pivot point bosses, a lifting eye and a reinforced section around the perimeter of the waterway opening. Pivot bosses shall be designed to limit the double hinge action, preventing the cover from rotating sufficiently to become wedged in the open position.

Seating Faces
The cast iron seating faces of the seat and cover shall be machined to a plane with a minimum 1600 nm finish.

Links
The links connecting the cover and the upper pivot bosses shall be one-piece galvanized steel and of sufficient section to safely withstand the normal forces encountered during gate operation. Each link shall be provided with a commercial grade bronze bushing at the pivot points.

Fasteners
All anchor bolts, assembly bolts and nuts shall be galvanized steel and of ample section to safely withstand forces created by operation of the gate under the heads shown in the Gate Schedule. Quantity and size of the fasteners shall be as recommended by the manufacturer. Anchor bolts shall be furnished with two nuts each to install gates attached to concrete.

Painting
Exposed machined or bearing surfaces shall be coated with a water-resistant rust preventive compound. All assembled units shall be shop painted in accordance with the manufacturer's standard practice.

Installation
Installation of the flap gates shall be done by the contractor in a workmanlike manner in accordance with the manufacturer's instructions.

Materials
Materials shall conform to the requirements of the following ASTM Standards.

CAST IRON	A 48, Class 30
GALVANIZED STEEL (Fasteners)	A 307 (Bolts) A 164 (Galvanized Coating)
GALVANIZED STEEL	A 36 or A 306 (Carbon Steel) A 123 (Galvanized Coating)

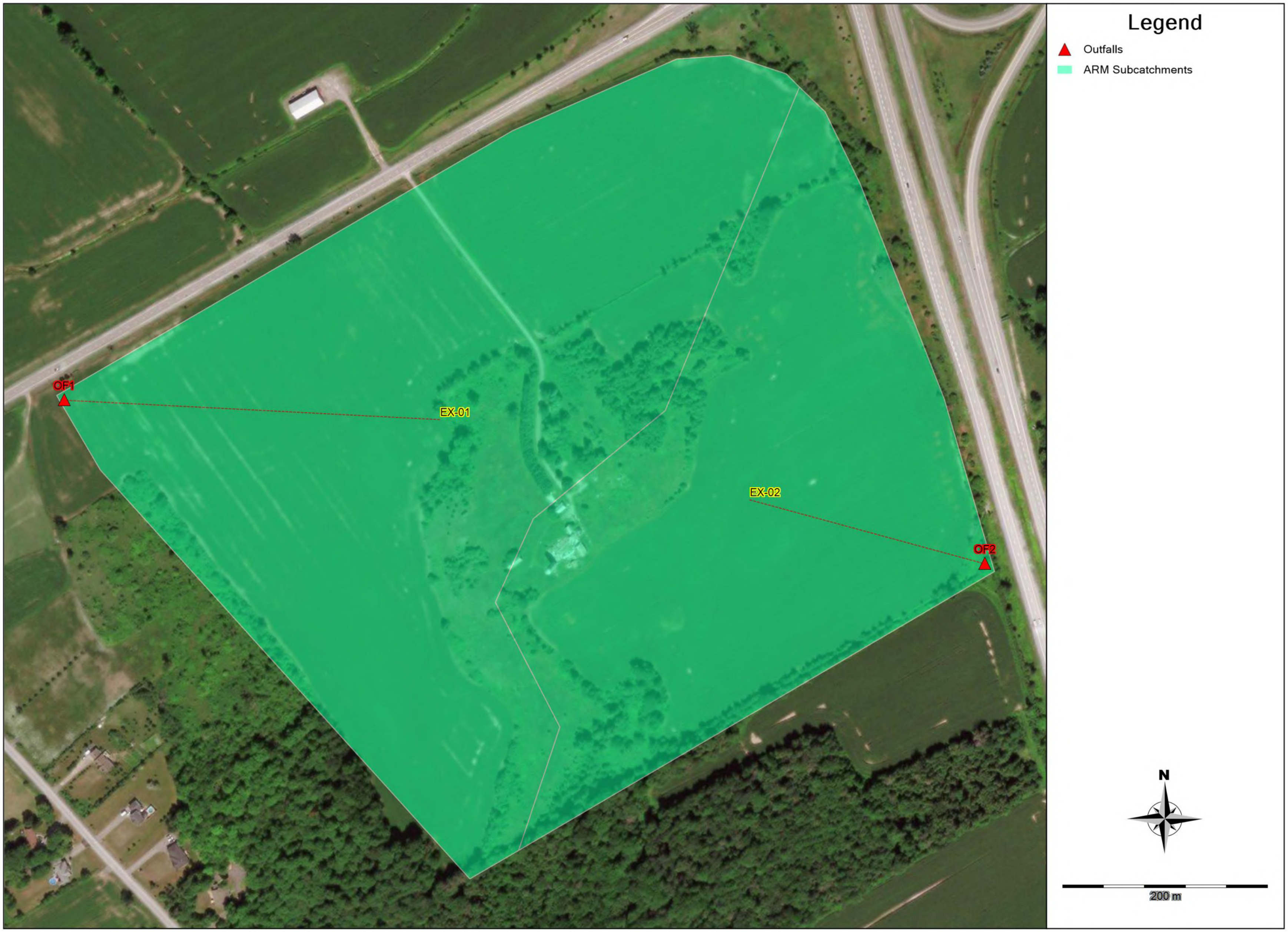
GATE SCHEDULE

QUANTITY REQUIRED	SIZE OPENING	SEATING HEAD	REMARKS

Revised 8/82

APPENDIX B
Stormwater Management Calculations

1966 Roger Stevens Drive (119018)
PCSWMM Model Schematic - Existing Conditions



Date: 2019/07/10
M:\2019\119018\DATA\Calculations\Sewer Calcs\SWM\PCSWMM\Ex_Model_Schematic.pdf

ALTERNATIVE RUNOFF METHOD (ARM) - PCSWMM BETA VERSION 7.2.2780

This is a *BETA* version of ARM - your feedback and suggestions are solicited.
Create a ticket, post on the PCSWMM feature request forum, or email us directly!

Simulation start time:	02/20/2019 00:00:00
Simulation end time:	02/21/2019 00:00:00
Runoff wet weather time steps:	300 seconds
Report time steps:	60 seconds
Number of data points:	1441

****
Unit Hydrographs Runoff Method
****

Subcatchment	Runoff Method	Raingage	Area (ha)	Time of Concentration (min)	Time to Peak (min)	Time after Pe (min)
EX-01	Nash IUH	Raingage	25.98	61	40.67	264.33
EX-02	Nash IUH	Raingage	16.75	29	19.33	130.67

****
ARM Runoff Summary
****

Subcatchment	Total Precip (mm)	Total Losses (mm)	Total Runoff (mm)	Total Runoff 10^6 ltr	Peak Runoff LPS	Runoff Coeff (fraction)
EX-01	106.73	43.22	61.663	16.02	1540.709	0.578
EX-02	106.73	48.774	56.669	9.492	1256.958	0.531

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.013)

PCSWMM Model for Proposed Roger Stevens Drive Warehouse

****
Element Count
****
Number of rain gages 1
Number of subcatchments ... 0
Number of nodes 2
Number of links 0
Number of pollutants 0
Number of land uses 0

****
Raingage Summary
****

Name	Data Source	Data Type	Recording Interval
Raingage	S24hr-100yr	INTENSITY	60 min.

****
Node Summary
****

Name	Type	Invert Elev.	Max. Depth	Ponded Area	External Inflow
OF1	OUTFALL	88.00	0.00	0.0	
OF2	OUTFALL	87.50	0.00	0.0	

****
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
****

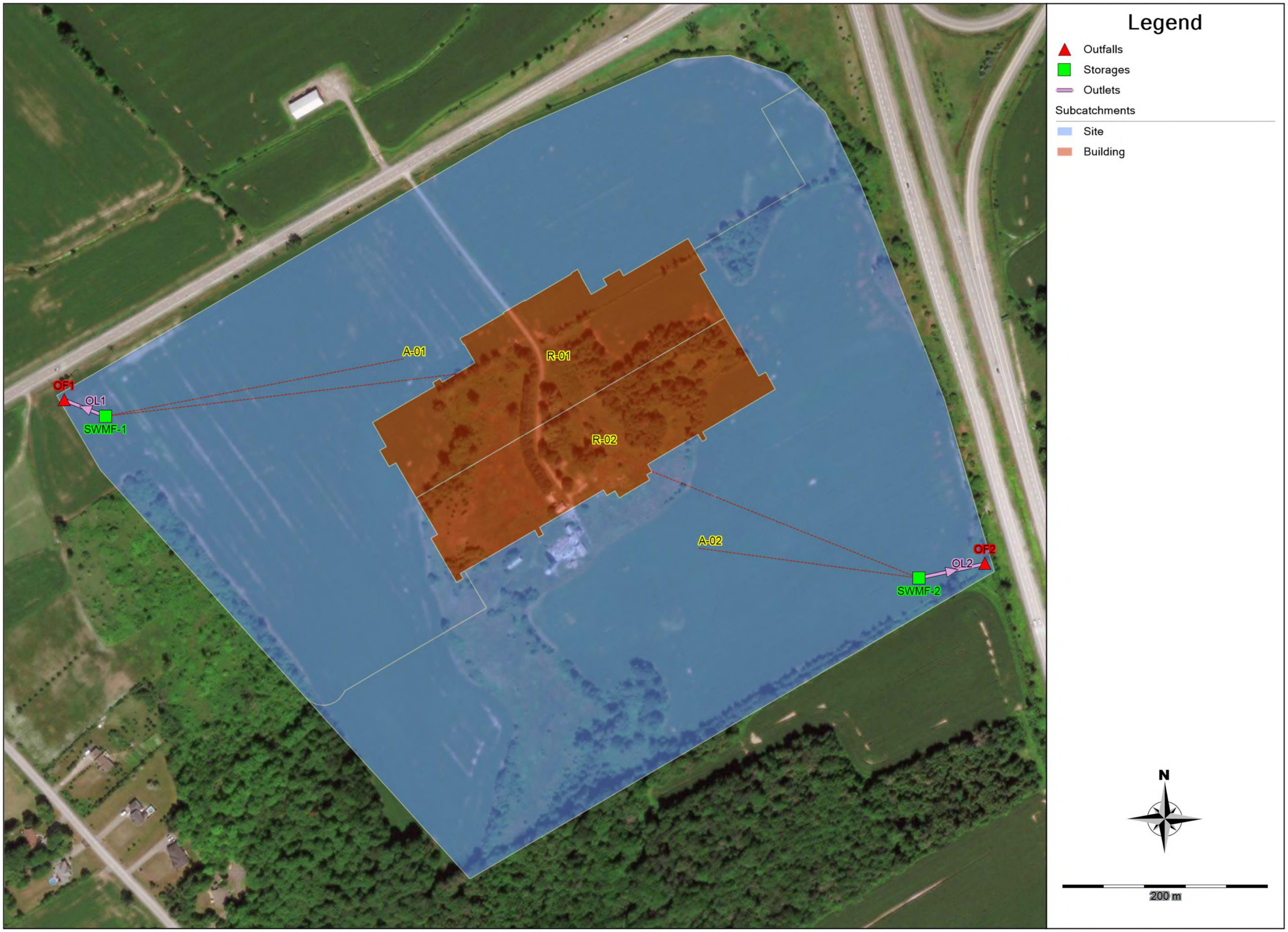
****
Analysis Options
****
Flow Units .... LPS
Process Models:
  Rainfall/Runoff .... YES
  RDII .... NO
  Snowmelt .... NO

```
  Groundwater ............ NO
  Flow Routing ........... NO
  Water Quality .......... NO
Surcharge Method ......... EXTRAN
Starting Date ............ 02/20/2019 00:00:00
Ending Date .............. 02/21/2019 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:01:00


**************************        Volume         Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------      ---------
Dry Weather Inflow .......         0.000          0.000
Wet Weather Inflow .......         0.000          0.000
Groundwater Inflow .......         0.000          0.000
RDII Inflow ..............         0.000          0.000
External Inflow ..........         2.549         25.493
External Outflow .........         2.549         25.493
Flooding Loss ............         0.000          0.000
Evaporation Loss .........         0.000          0.000
Exfiltration Loss ........         0.000          0.000
Initial Stored Volume ....         0.000          0.000
Final Stored Volume ......         0.000          0.000
Continuity Error (%) .....         0.000


Analysis begun on:  Wed Jul 10 10:29:24 2019
Analysis ended on:  Wed Jul 10 10:29:24 2019
Total elapsed time: < 1 sec
```

1966 Roger Stevens Drive (119018)
PCSWMM Model Schematic - Proposed Conditions



Date: 2019/07/10
M:\2019\119018\DATA\Calculations\Sewer Calcs\SWM\PCSWMM\Model_Schematic.pdf

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.013)
--------------------------------------------------------------

PCSWMM Model for Proposed Roger Stevens Drive Warehouse


*************
Element Count
*************
Number of rain gages ...... 1
Number of subcatchments ... 4
Number of nodes ........... 4
Number of links ........... 2
Number of pollutants ...... 0
Number of land uses ....... 0


****************
Raingage Summary
****************
                                                    Data       Recording
Name                 Data Source                    Type       Interval
------------------------------------------------------------------------
Raingage             C3hr-100yr                     INTENSITY   10 min.


********************
Subcatchment Summary
********************
Name                       Area     Width   %Imperv    %Slope Rain Gage            Outlet
-----------------------------------------------------------------------------------------------------------
A-01                      17.00   1700.00     50.00    1.5000 Raingage             SWMF-1
A-02                      19.24   1924.00     21.43    1.5000 Raingage             SWMF-2
R-01                       3.39    678.00    100.00    2.0000 Raingage             SWMF-1
R-02                       3.10    620.00    100.00    2.0000 Raingage             SWMF-2


************
Node Summary
************
                                           Invert      Max.    Ponded    External
Name                 Type                   Elev.     Depth      Area    Inflow
-------------------------------------------------------------------------------
OF1                  OUTFALL                88.00      0.00       0.0
OF2                  OUTFALL                87.50      0.00       0.0
SWMF-1               STORAGE                88.10      1.50       0.0
SWMF-2               STORAGE                87.60      1.50       0.0


************
Link Summary
************
Name             From Node        To Node          Type            Length    %Slope Roughness
---------------------------------------------------------------------------------------------
OL1              SWMF-1           OF1              OUTLET
OL2              SWMF-2           OF2              OUTLET


*********************
Cross Section Summary
*********************
                                        Full     Full     Hyd.     Max.   No. of     Full
Conduit          Shape                 Depth     Area     Rad.    Width  Barrels     Flow
---------------------------------------------------------------------------------------


*********************************************************
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
*********************************************************

****************
Analysis Options
****************
Flow Units ............... LPS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... NO
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Surcharge Method ......... EXTRAN
Starting Date ............ 02/20/2019 00:00:00
Ending Date .............. 02/21/2019 00:00:00
Antecedent Dry Days ...... 0.0
```

```
Report Time Step ......... 00:01:00
Wet Time Step ............ 00:05:00
Dry Time Step ............ 00:05:00
Routing Time Step ........ 2.00 sec
Variable Time Step ....... YES
Maximum Trials ........... 8
Number of Threads ........ 1
Head Tolerance ........... 0.001500 m

**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......         3.062        71.667
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         1.170        27.391
Surface Runoff ...........         1.888        44.186
Final Storage ............         0.020         0.464
Continuity Error (%) .....        -0.523

**************************        Volume        Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         1.888        18.881
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         1.888        18.881
Flooding Loss ............         0.000         0.000
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.000         0.000
Continuity Error (%) .....        -0.000

***************************
Time-Step Critical Elements
***************************
None

********************************
Highest Flow Instability Indexes
********************************
All links are stable.

*************************
Routing Time Step Summary
*************************
Minimum Time Step           :     1.50 sec
Average Time Step           :     2.00 sec
Maximum Time Step           :     2.00 sec
Percent in Steady State     :     0.00
Average Iterations per Step :     2.00
Percent Not Converging      :     0.00

***************************
Subcatchment Runoff Summary
***************************
```

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff LPS	Runoff Coeff
A-01	71.67	0.00	0.00	24.19	35.31	11.84	47.16	8.02	5241.92	0.658
A-02	71.67	0.00	0.00	39.46	15.08	17.05	32.13	6.18	3298.78	0.448
R-01	71.67	0.00	0.00	0.00	72.15	0.00	72.15	2.45	1680.41	1.007
R-02	71.67	0.00	0.00	0.00	72.15	0.00	72.15	2.24	1536.66	1.007

```
******************
Node Depth Summary
******************
```

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters
OF1	OUTFALL	0.00	0.00	88.00	0 00:00	0.00
OF2	OUTFALL	0.00	0.00	87.50	0 00:00	0.00
SWMF-1	STORAGE	0.13	1.50	89.60	0 01:29	1.50
SWMF-2	STORAGE	0.12	1.49	89.09	0 01:34	1.49

```
*******************
```

Node Inflow Summary

Node	Type	Maximum Lateral Inflow LPS	Maximum Total Inflow LPS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
OF1	OUTFALL	0.00	1554.84	0 01:29	0	10.5	0.000
OF2	OUTFALL	0.00	1237.82	0 01:34	0	8.42	0.000
SWMF-1	STORAGE	6922.33	6922.33	0 01:10	10.5	10.5	-0.000
SWMF-2	STORAGE	4835.43	4835.43	0 01:10	8.42	8.42	-0.000

Node Surcharge Summary

No nodes were surcharged.

Node Flooding Summary

No nodes were flooded.

Storage Volume Summary

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow LPS
SWMF-1	0.472	9	0	0	5.490	100	0 01:29	1554.84
SWMF-2	0.318	8	0	0	4.033	100	0 01:34	1237.82

Outfall Loading Summary

Outfall Node	Flow Freq Pcnt	Avg Flow LPS	Max Flow LPS	Total Volume 10^6 ltr
OF1	63.84	189.68	1554.84	10.462
OF2	50.36	193.47	1237.82	8.418
System	57.10	383.15	1237.82	18.880

Link Flow Summary

Link	Type	Maximum |Flow| LPS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
OL1	DUMMY	1554.84	0 01:29			
OL2	DUMMY	1237.82	0 01:34			

Flow Classification Summary

Conduit	Adjusted /Actual Length	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl

(---------- Fraction of Time in Flow Class ----------)

Conduit Surcharge Summary

No conduits were surcharged.

Analysis begun on: Wed Jul 10 09:45:05 2019
Analysis ended on: Wed Jul 10 09:45:06 2019
Total elapsed time: 00:00:01

1966 ROGER STEVENS DRIVE (119018)
EXISTING CONDITIONS SWM CALCULATIONS

NOVATECH
Engineers, Planners & Landscape Architects

Area ID	Area ha	TIMP %	XIMP %	Soil Type % B	Soil Type % D	Land Use % Row Crop	Land Use % Meadow	Soil CN	IA
EX-01	25.98	0.01	0.00	43%	57%	77%	23%	81	6
EX-02	16.75	0.00	0.00	59%	41%	73%	27%	78	7

		Land Use	
		Row Crop	Meadow
Soil Type	Type B	78	58
	Type D	89	78
Uplands	V (m/s)	0.15	0.20

Area ID	Length (m)	Elevation U/S (m)	Elevation D/S (m)	Overland Flow Slope (%)	Velocity (m/s)	Time of Concentration (min)	Time to Peak (hrs)
EX-01	550	90.00	88.00	0.4%	0.15	61	0.68
EX-02	350	90.00	87.50	0.7%	0.20	29	0.33

1966 ROGER STEVENS DRIVE (119018)
PROPOSED CONDITIONS SWM CALCULATIONS

NOVATECH
Engineers, Planners & Landscape Architects

Pond ID	Total Area (ha)	C	Allowable Release Rate (L/s)		Uncontrolled Release Rate (L/s)		Required Storage Volume (m^3)
			5-Year	100-Year	5-Year	100-Year	100-Year
SWMF-1 (North)	20.39	0.61	680	1540	3550	6470	5150
SWMF-2 (South)	22.34	0.43	540	1260	2260	4140	3900

Pond ID	Permanent Pool Volume (m^3)	Required Storage Volume (m^3)	Required Storage Volume (m^3)
SWMF-1 (North)	3610	5150	8760
SWMF-2 (South)	3370	3900	7270

Date: 7/10/2019
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1966 ROGER STEVENS DRIVE (119018)
CONCEPTUAL SWM FACILITY DESIGN

NOVATECH
Engineers, Planners & Landscape Architects

Required Storage Volumes (Quality)
SWMF-1

Drainage Area	**20.4**	ha
% Impervious:	**60%**	
Enhanced protection (80% TSS removal):		
Treatment Volume	217	m3/ha
Extended Detention Storage.	40	m3/ha required
	816	m3 required
	850	m3 provided
	41.7	m3/ha provided
Perm Pool:	177	m3/ha required
	3611	m3 required
	3610	m3 provided
	177.0	m3/ha provided
Extended Detention:	18.89	L/s average
	47.22	L/s max (2.5 x avg)

(% impervious was calculated as the average imperviousness
for the drainage areas tributary to the SWM facility)

Required Forebay Length and Width

Parameters:

Length to width ratio of forebay, r =	4.0:1	
Peak outflow rate during 25 mm storm, (	0.047	m^3/s (24hr ext. det)
Target particle size =	150	µm
Settling velocity, V_s =	0.0003	m/s

Forebay Settling Length, Dist

$$Dist = \sqrt{\frac{rQ_P}{V_S}}$$

= 25 m

Check Dispersion Length, Dist$_2$

Desired velocity in forebay, V_f =	0.15	m/s
Inlet flow rate , Q_{2yr} =	0.619	m^3/s
Depth in forebay, d =	1.3	m

$$Dist_2 = \frac{8Q}{dV_f}$$

= 25 m

Therefore, the dispersion length of 25 m governs the design.

Required Length	**= 25 m**
Provided Length	**= 25 m**

Minimum Forebay width:

Length of Forebay, L =	25 m
Minimum width, W =	L/4
W =	6.3 m

Required Width	**= 6.3 m**
Provided Width	**= 14.0 m**

1966 ROGER STEVENS DRIVE (119018)
CONCEPTUAL SWM FACILITY DESIGN

NOVATECH
Engineers, Planners & Landscape Architects

Required Storage Volumes (Quality)
SWMF-2

Drainage Area	**22.3**	ha
% Impervious:	**34%**	
Enhanced protection (80% TSS removal):		
Treatment Volume	191	m3/ha
Extended Detention Storage.	40	m3/ha required
	892	m3 required
	900	m3 provided
	40.4	m3/ha provided
Perm Pool:	151	m3/ha required
	3367	m3 required
	3370	m3 provided
	151.1	m3/ha provided
Extended Detention:	20.65	L/s average
	51.62	L/s max (2.5 x avg)

(% impervious was calculated as the average imperviousness
for the drainage areas tributary to the SWM facility)

Required Forebay Length and Width

Parameters:

Length to width ratio of forebay, r =	4.0:1	
Peak outflow rate during 25 mm storm, Q	0.052	m^3/s (24hr ext. det)
Target particle size =	150	µm
Settling velocity, V_s =	0.0003	m/s

Forebay Settling Length, Dist

$$Dist = \sqrt{\frac{rQ_P}{V_S}}$$

= 26 m

Check Dispersion Length, $Dist_2$

Desired velocity in forebay, V_f =	0.15	m/s
Inlet flow rate , Q_{2yr} =	0.619	m^3/s
Depth in forebay, d =	1.3	m

$$Dist_2 = \frac{8Q}{dV_f}$$

= 25 m

Therefore, the settling length of 26 m governs the design

Required Length	**= 26 m**
Provided Length	**= 25 m**

Minimum Forebay width:

Length of Forebay, L =	26 m
Minimum width, W =	L/4
W =	6.6 m

Required Width	**= 6.6 m**
Provided Width	**= 14.0 m**