

Engineering

Land / Site
Development
Municipal
Infrastructure
Environmental /
Water Resources
Traffic /
Transportation
Structural
Recreational

Planning

Land / Site
Development
Planning Application
Management
Municipal Planning
Documents &
Studies
Expert Witness
(OMB)
Wireless Industry

Landscape Architecture

Urban Design &
Streetscapes
Open Space, Parks &
Recreation Planning
Community &
Residential
Developments
Commercial &
Institutional Sites
Environmental
Restoration



Roger Stevens Warehouse 1966 Roger Stevens Drive, Ottawa Conceptual Stormwater Management Report

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

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

TABLE OF CONTENTS

1.0	INTRODUCTION	1
1.1	Background	1
1.2	Existing Conditions	1
1.3	Topography & Drainage Outlets	1
1.4	Subsurface Conditions	1
2.0	PROPOSED DEVELOPMENT	2
3.0	STORMWATER MANAGEMENT DESIGN	2
3.1	Stormwater Management Design Criteria and Objectives	2
3.2	Storm Servicing	2
3.3	Stormwater Management Facilities	3
3.4	SWMF Design Criteria	3
3.5	SWMF Components	3
3.6	SWMF Conceptual Design	3
4.0	EROSION AND SEDIMENT CONTROL	4
4.1	Temporary Measures	4
4.2	Permanent Measures	4
5.0	CONCLUSIONS AND RECOMMENDATIONS	5

LIST OF FIGURES

Figure 1	Key Plan
Figure 2	Existing Conditions Plan
Figure 3	Conceptual Site Plan
Figure 4	Conceptual Servicing Plan
Figure 5	Pre-Development Drainage Area Plan
Figure 6	Post-Development Drainage Area Plan

LIST OF APPENDICES

Appendix A	Jordel Acres Reports
Appendix B	Preliminary Stormwater Management Calculations

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 and 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.

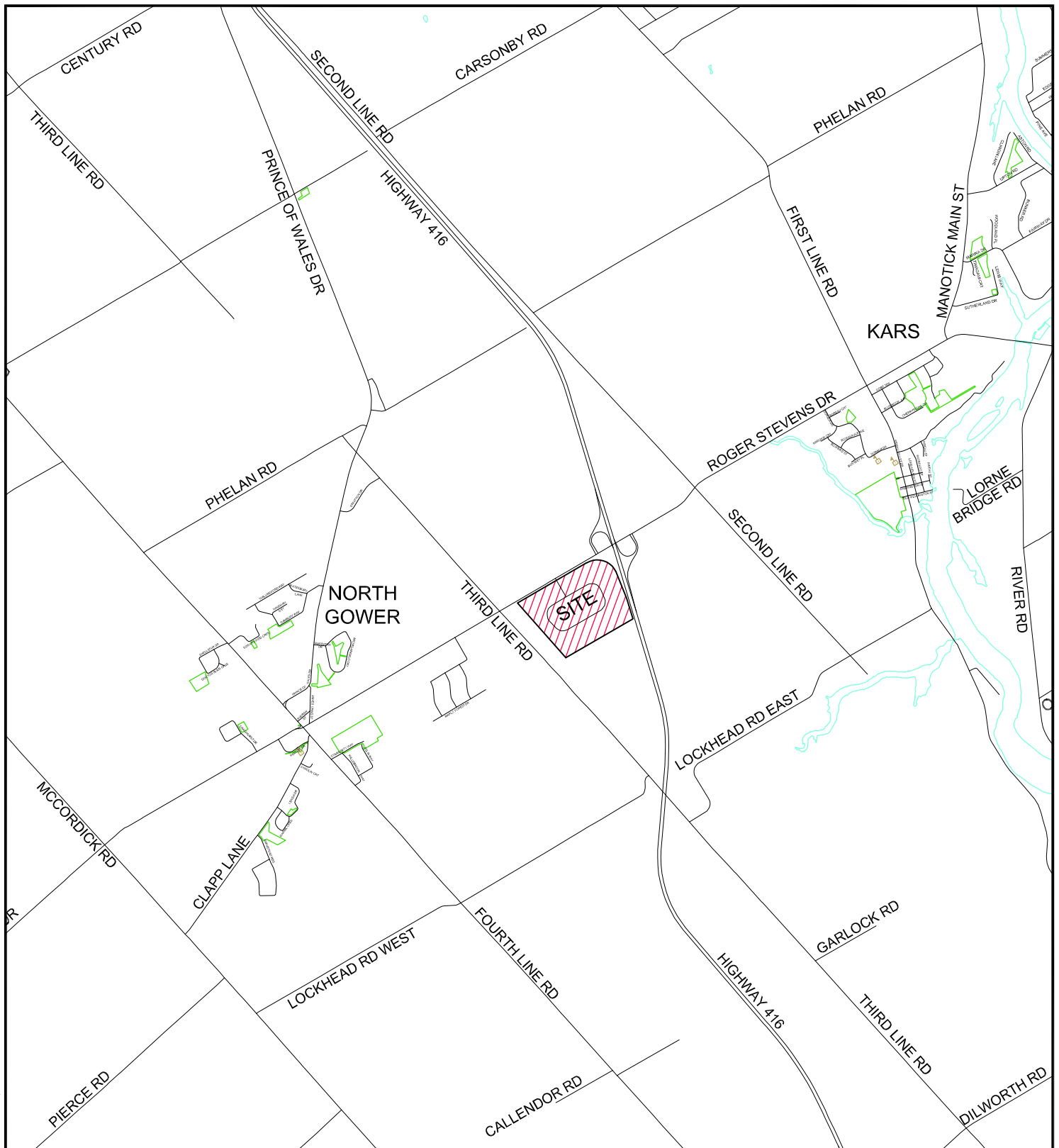
1.3 Topography & Drainage Outlets

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.

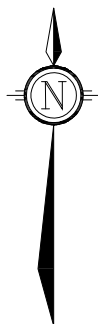
1.4 Subsurface Conditions

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



Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6

Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com



1966 ROGER STEVENS
DRIVE

KEYPLAN

SCALE

N.T.S

DATE

JULY 2019

JOB

119018

FIGURE

1

M:\2019\119018\CAD\Design\Figures\Ex Conditions REPROT FIGURES.dwg, FIG 2 EXC, Jul 10, 2019 - 4:03pm, mhrehorlak



NOVATECH

Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6

Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

**1966 ROGER STEVENS
DRIVE**

**EXISTING CONDITIONS
PLAN**

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

CUT 11V17 DWG 270mm V132mm

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 serviced 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.

M:\2019\119018\CAD\Design\Figures\REPORT FIGURES.dwg, FIG 3 PRS, Jul 03, 2019 - 1:12pm, mhrehorlak



Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6

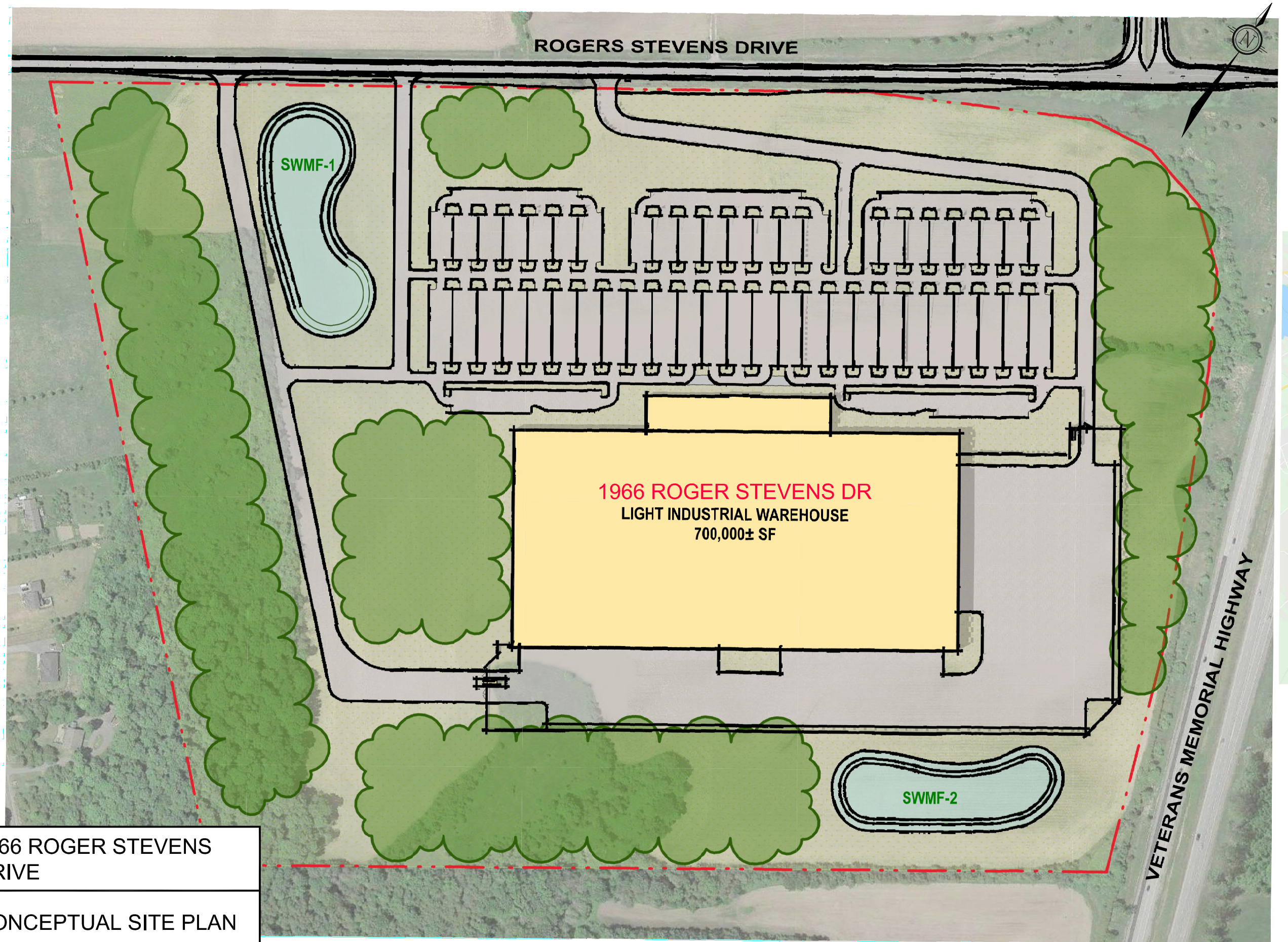
Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

1966 ROGER STEVENS
DRIVE

CONCEPTUAL SITE PLAN

SCALE 1 : 3000

DATE JULY 2019 JOB 119018 FIGURE 3



M:\2019\119018\CAD\Design\119018-GS.dwg, Fig 4, Jun 27, 2019 - 4:49pm, mhrehorlak



Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6

Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

1966 ROGER STEVENS
DRIVE

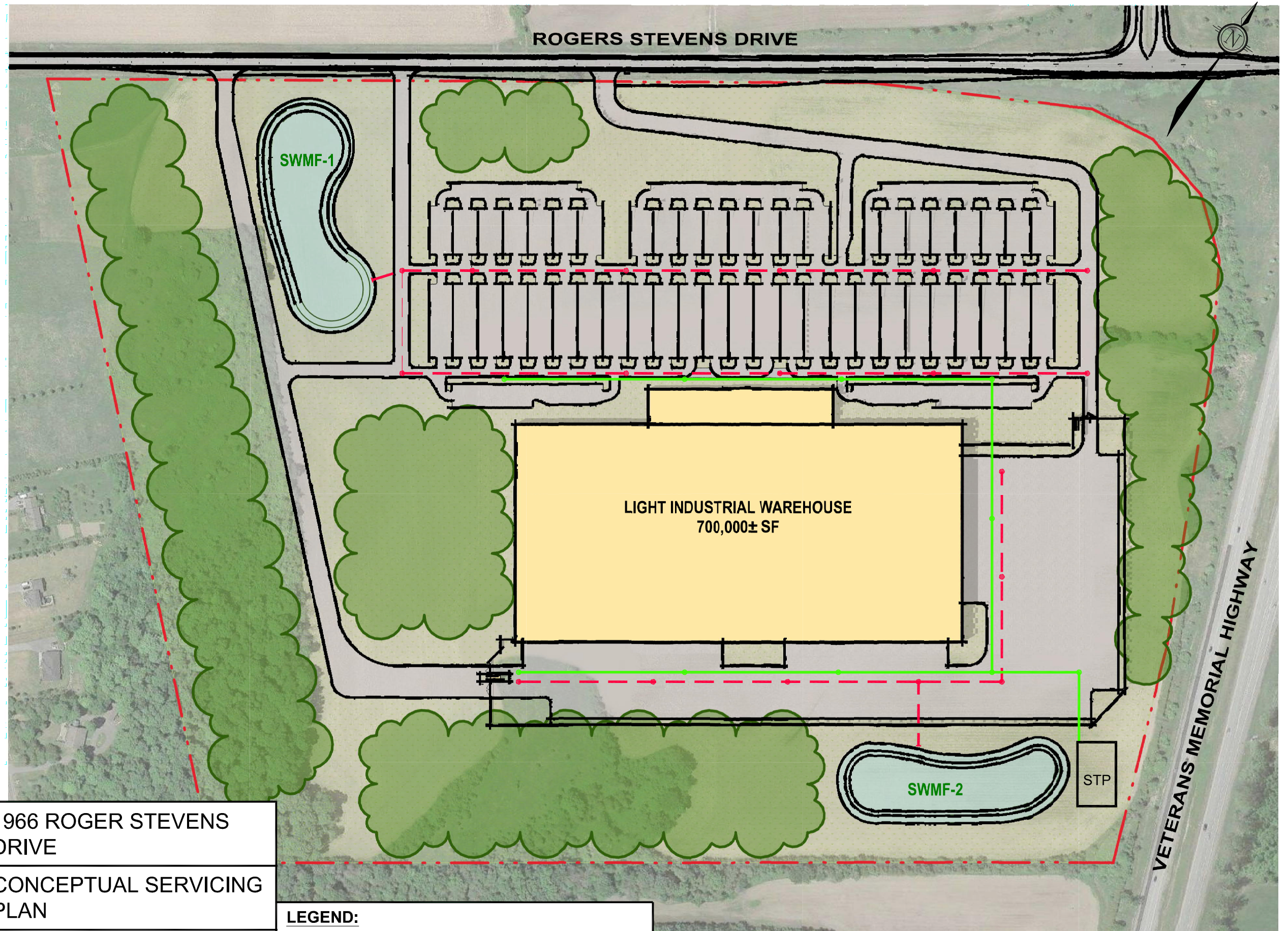
CONCEPTUAL SERVICING PLAN

SCALE 1 : 3000

DATE JULY 2019 JOB 119018 FIGURE 4

LEGEND:

- CONCEPTUAL STORM SEWER
- CONCEPTUAL SANITARY SEWER



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/ cleanup;
- 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 construction 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.

M:\2019\119018\CAD\Design\Figures\SWM\119018-SWM new.dwg, PRE, Jul 02, 2019 - 4:46pm, mihnehorciak



LEGEND

-  DRAINAGE AREA BOUNDARY
-  FLOW ARROW

NOVATECH

Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6

Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

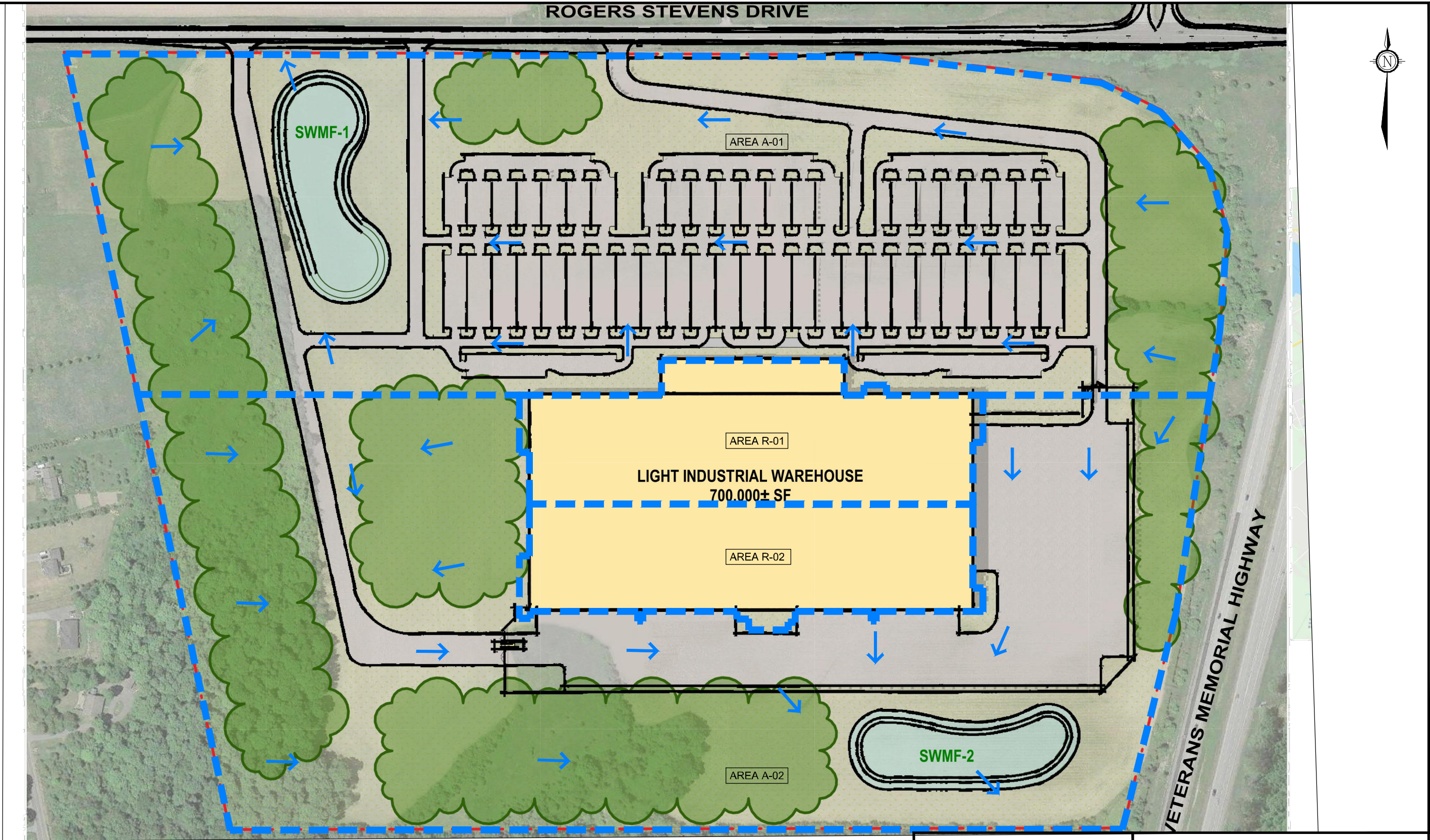
**ROGER STEVENS
WAREHOUSE BUILDING**

**PRE-DEVELOPMENT
DRAINAGE AREA PLAN**

SCALE 1 : 3000 

DATE JULY 2019 JOB 119018 FIGURE 5

M:\2019\119018\CAD\Design\Figures\SWM\119018-SWM new.dwg, POST, Jul 03, 2019 - 10:27am, mthreboriak



LEGEND

- DRAINAGE AREA BOUNDARY
→ FLOW ARROW

NOVATECH

Engineers, Planners & Landscape Architects
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6

Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

**ROGER STEVENS
WAREHOUSE BUILDING**

**POST-DEVELOPMENT
DRAINAGE AREA PLAN**

SCALE 1 : 3000 0 30 60 90 120

DATE JULY 2019 JOB 119018 FIGURE 6

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 m³, 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 Ruddell, 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^CMANUS ENGINEERING LTD.

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

December 2001

TABLE OF CONTENTS

	Page
1.0 INTRODUCTION.....	1
2.0 PRE-DEVELOPMENT CONDITIONS	1
2.1) Existing Drainage.....	1
2.2) Water Quality.....	2
2.3) Water Quantity.....	2
2.4) Floodplain Infill.....	2
3.0 POST-DEVELOPMENT CONDITIONS.....	2
3.1) Development Criteria.....	2
3.2) Water Quality.....	2
3.3) Water Quantity.....	4
3.4) Floodplain Infill/Impacts.....	4
4.0 BEST MANAGEMENT PRACTICES.....	4
4.1) Maintenance and Operation.....	5
5.0 CONCLUSIONS.....	5
<i>Appendix A: Design elements for Stormwater Management ponds</i> <i>Design details</i>	
<i>Appendix B: SWMHYMO input and output files</i>	
<i>Appendix C: Calculations for Extended Detention Wet Pond and Forebay</i> <i>(Source: SWMP Planning and Design Manual)</i>	
<i>Appendix D: Armtec Flap Gate Model 10C Details</i>	
<i>Drawings: Stormwater Management Pond Construction Details</i> <i>Drawings: 1935-D1</i> <i>1935-D2</i>	

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.

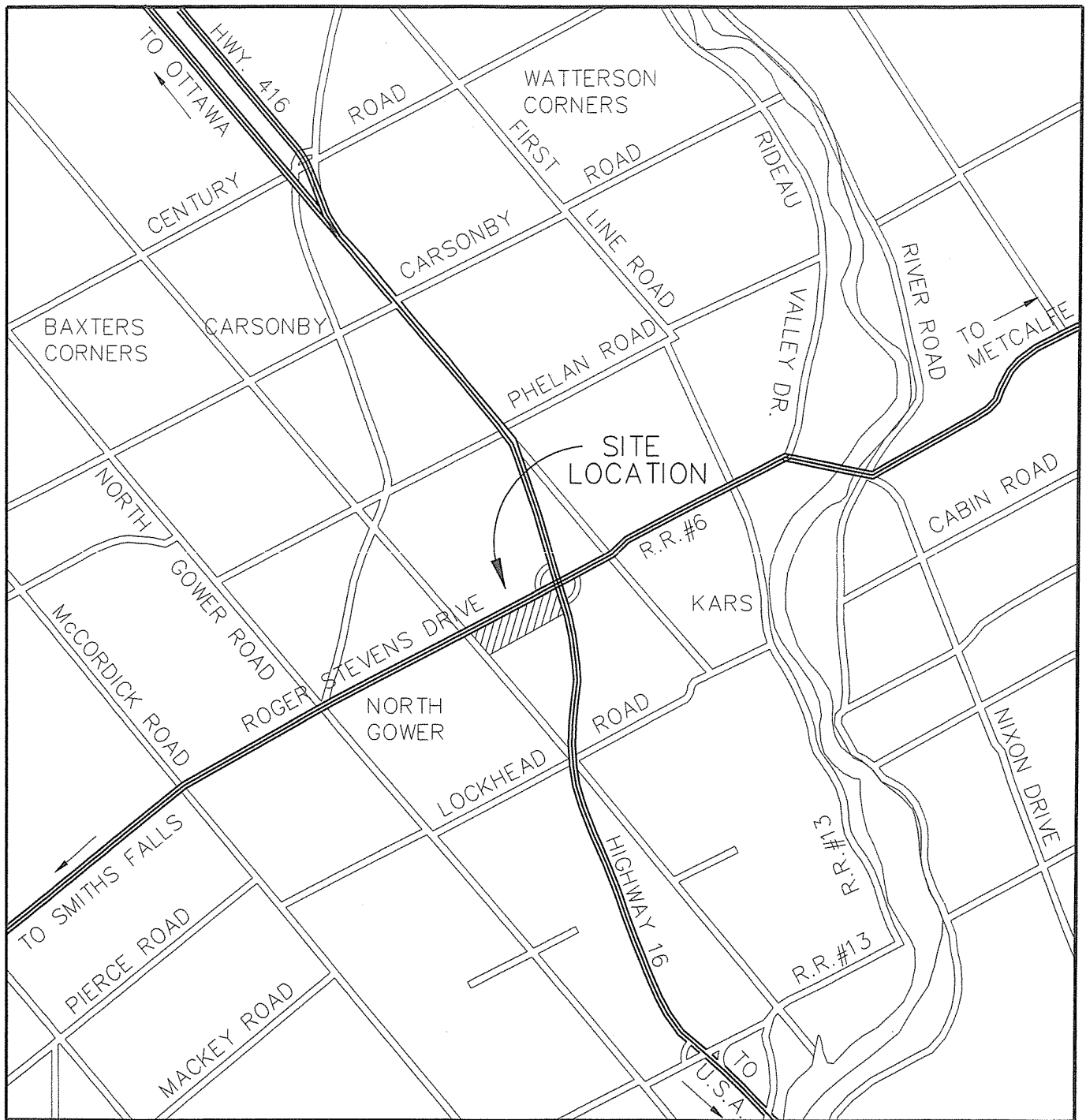
2.0 PRE DEVELOPMENT CONDITIONS

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



David McManus
Engineering Ltd.

200 - 39 Camelot Drive
Nepean Ontario, K2H 5W6
E-mail: mcmanus@dme.on.ca
Ph. 225-1929 Fax 225-7330

Pri-Tec International Inc.
Jordel Acres Development

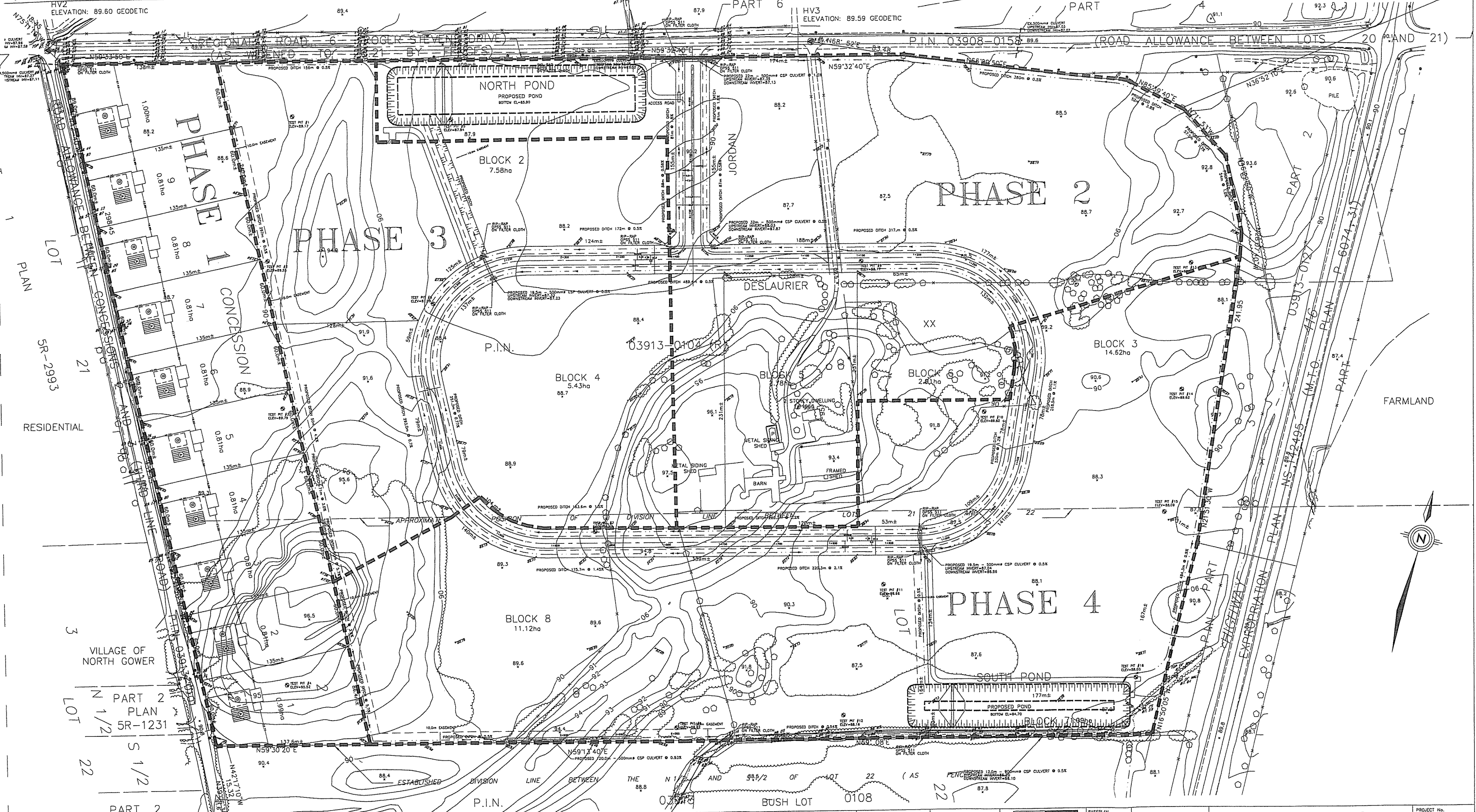
DATE:

December 2001

KEY PLAN

Project No. 1935

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



NOTE
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS,
SEWERS AND OTHER UNDERGROUND AND OVERGROUND
UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN
IN 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



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

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

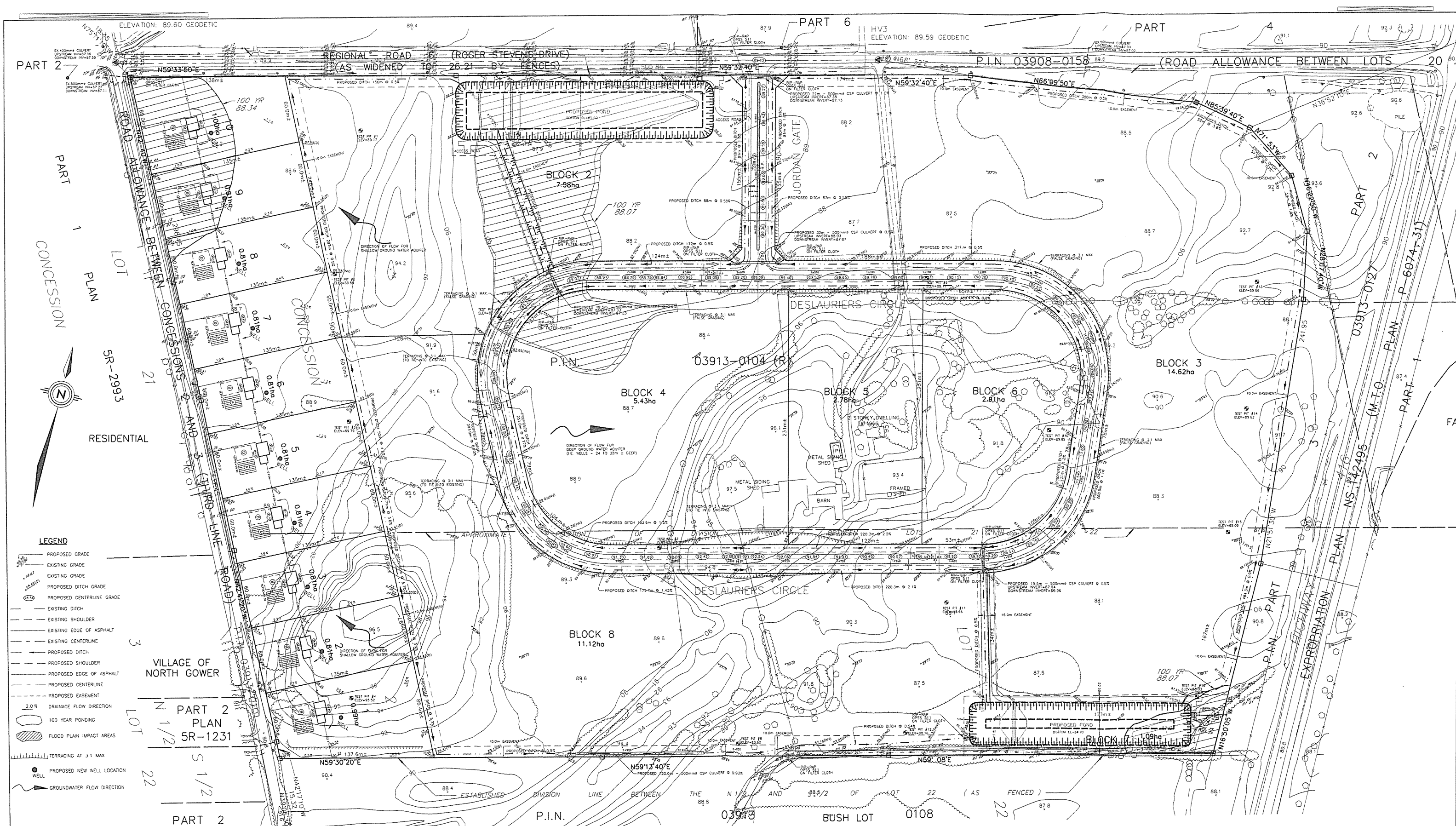
The proposed development may impact surface water, fish habitat and floodplain loss of storage.

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

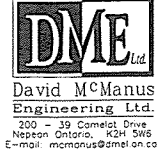
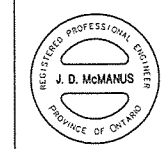


- LEGEND**
- PROPOSED GRADE
 - EXISTING GRADE
 - PROPOSED DITCH GRADE
 - EXISTING DITCH GRADE
 - EXISTING SHOULDER
 - EXISTING EDGE OF ASPHALT
 - EXISTING CENTERLINE
 - PROPOSED DITCH
 - PROPOSED SHOULDER
 - PROPOSED EDGE OF ASPHALT
 - PROPOSED CENTERLINE
 - PROPOSED EASEMENT
 - 2.0% GRAINAGE FLOW DIRECTION
 - 100 YEAR PONDING
 - FLOOD PLAIN IMPACT AREAS
 - TERRACING AT 3.1 MAX
 - PROPOSED NEW WELL LOCATION
 - GROUNDWATER FLOW DIRECTION

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



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

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

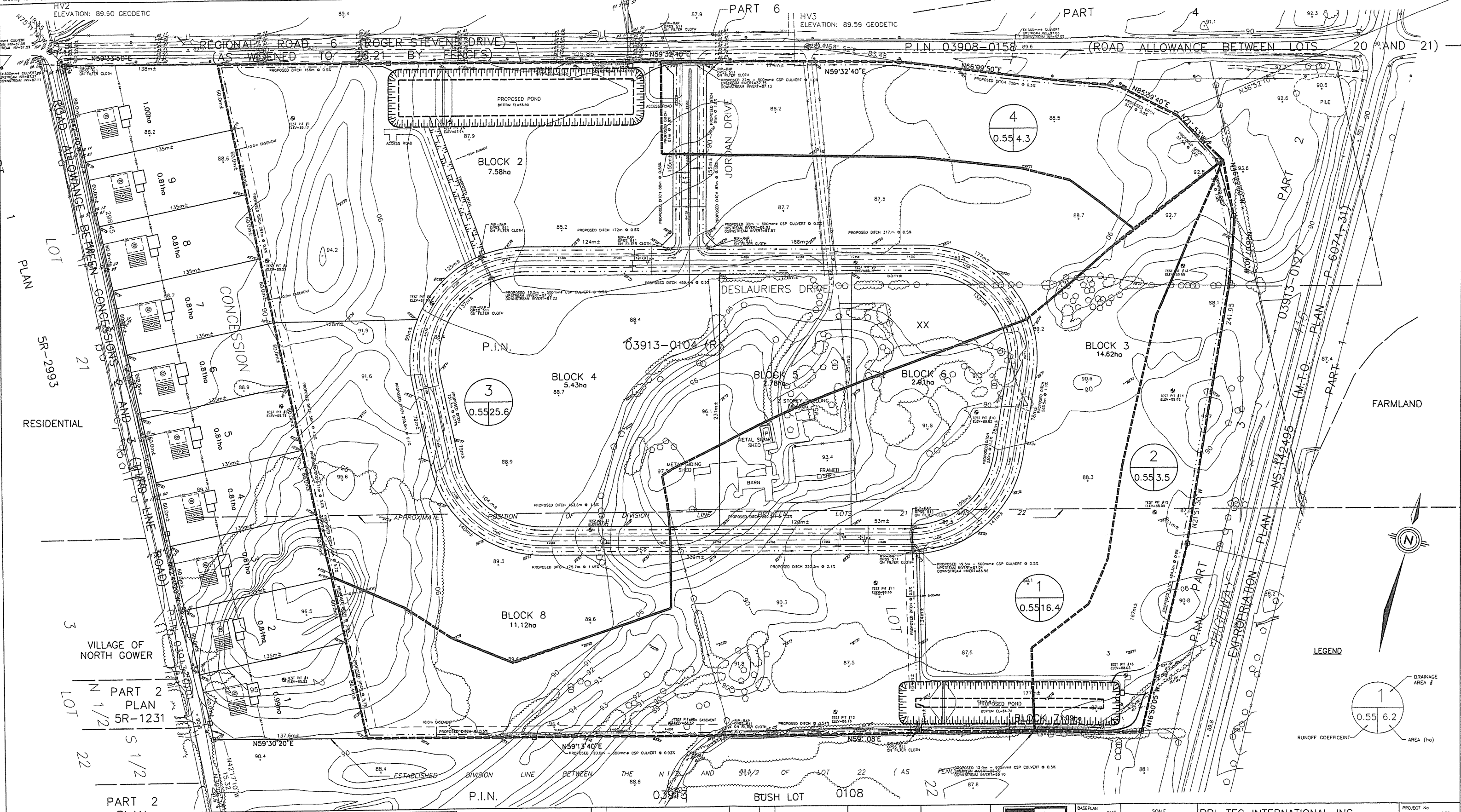
wet ponds with fore bays will reduce suspended solids concentrations to a level that can be released to a type 1 fish habitat.

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 1 protection 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	8280m ³ (6279m ³ required)	elev. 87.10
	Extended Storage	1215 m ³ (1196m ³ required)	elev. 87.23
	Sideslopes	5:1 4:1	-Adjacent to pond -Within pond (sideslopes not maintained)
	Outlet		
	Orifice (quality)	180 mm dia.	-Controlled to 25 L/s
	Orifice (quantity)	500 mm dia.	-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	4200m ³ (4179m ³ required)	elev. 86.20
	Extended Storage	1290 m ³ (796 m ³ required)	elev. 86.50
	Sideslopes	5:1 4:1	-Adjacent to pond -Within pond (sideslopes not maintained)
	Outlet		
	Orifice (quality)	150 mm dia.	-Controlled to 28 L/s
	Orifice (quantity)	450 mm dia.	-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

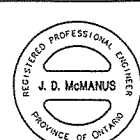


NOTE
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS,
SEWERS AND OTHER UNDERGROUND AND OVERGROUND
UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN
ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE
ACCURACY OF THE POSITION OF SUCH UTILITIES AND
STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK,
DETERMINE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



DME Ltd.
David McManus
Engineering Ltd.
200 - 39 Camelot Drive
Nepean Ontario, K2H 3W6
E-mail: mcmanus@dme.on.ca
Ph: 225-1929 Fax: 225-7331

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

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)

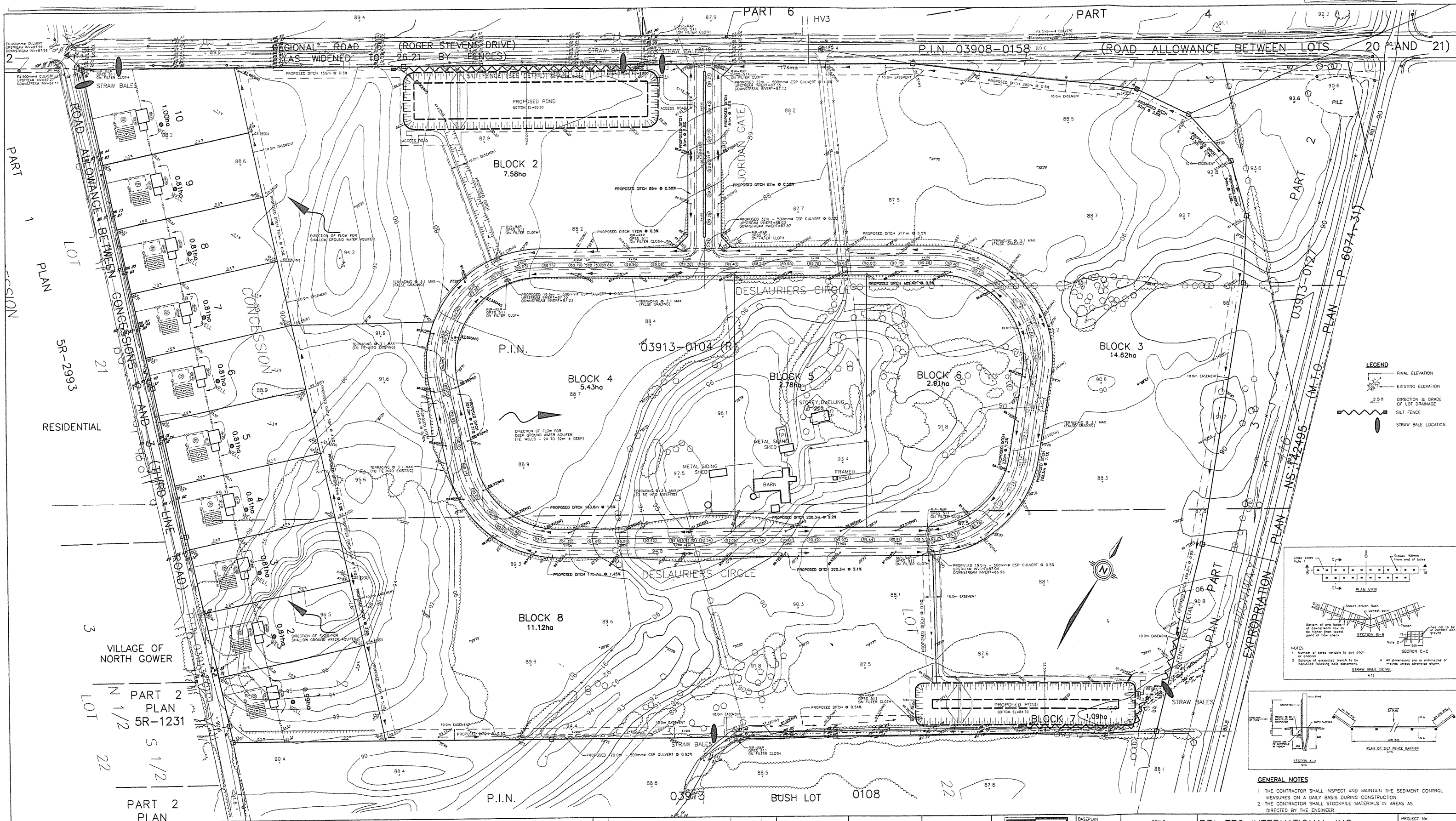
3.4 Floodplain Infill/Impacts

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.

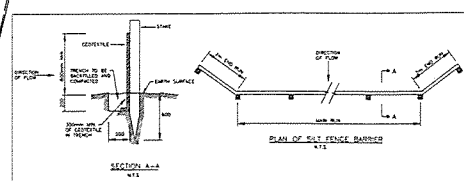
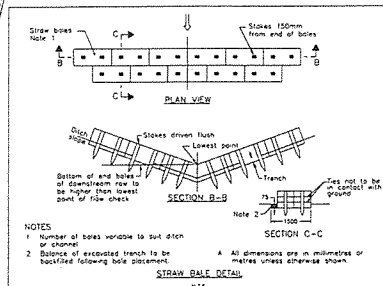
4.0 BEST MANAGEMENT PRACTICES

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:



LEGEND

- FINAL ELEVATION
- EXISTING ELEVATION
- DIRECTION & GRADE OF LOT DRAINAGE
- SILT FENCE
- STRAW BALE LOCATION



GENERAL NOTES

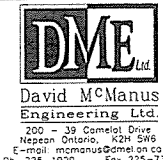
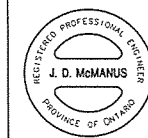
- THE CONTRACTOR SHALL INSPECT AND MAINTAIN THE SEDIMENT CONTROL MEASURES ON A DAILY BASIS DURING CONSTRUCTION
- THE CONTRACTOR SHALL STOCKPILE MATERIALS IN AREAS AS DIRECTED BY THE ENGINEER

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

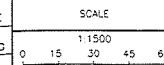
PRELIMINARY

NOT FOR CONSTRUCTION

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



BASEPLAN	DME
DESIGN	UB/MJS
CHECKED	UB
CAD	QSC
PROJ. MGR	UB
APPROVED	JDM



PRI-TEC INTERNATIONAL INC.	PROJECT No	1935
JORDEL ACRES	SURVEY BY	DME
TOWNSHIP OF RIDEAU	DATE	JANUARY 2001
EROSION AND SEDIMENT CONTROL PLAN	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.


Michael J Green B.Sc



APPENDIX A

Design elements for Stormwater Management ponds
Design details

December 14, 2002

Memo

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

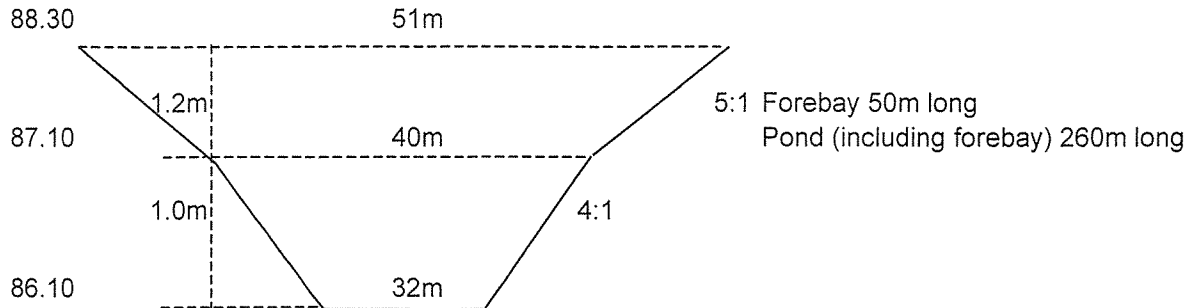
Re: Jordel SWM

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 **500mm** dia 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 **51m** at elevation **88.3** and a length of **230m** (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 38m at elevation 88.0 and a length of 200m (forebay length of 40m).

POND DETAILS

North Pond



Permanent Pool (PP) storage = $36 \times 230 \times 1 = 8280\text{m}^3$

Extended Detention (ED) storage = 1215m^3

Quantity Storage = $45.5 \times 230 \times 1.2 = 12558\text{m}^3$

DA = 29.9ha

Existing hydrology - SCSHYD

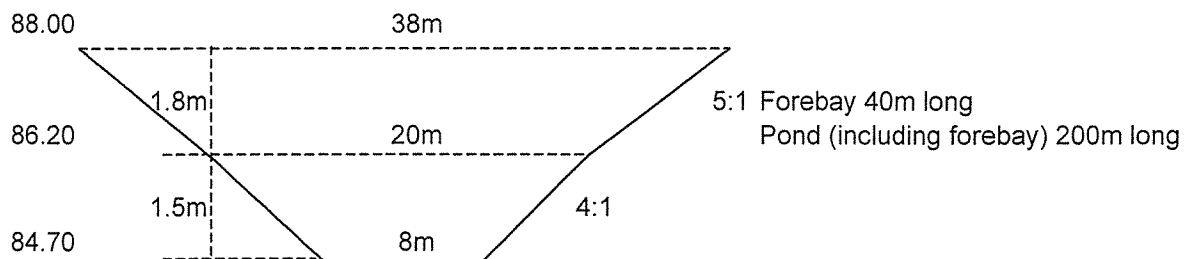
CN = 70 $T_p = 0.6\text{hr}$

Developed hydrology - STANDHYD

%imp = 27 Slope = 0.5%

C = 0.40

South Pond



Permanent Pool (PP) storage = $14 \times 200 \times 1.5 = 4200\text{m}^3$

Extended Detention (ED) storage = 1290m^3

Quantity Storage = $29 \times 200 \times 1.8 = 10440\text{m}^3$

DA = 19.9ha

Existing hydrology - SCSHYD

CN = 70 $T_p = 0.3\text{hr}$

Developed hydrology - STANDHYD

%imp = 35 Slope = 0.75%

C = 0.45

JORDEL ACRES
SWM Pond Design Elements

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

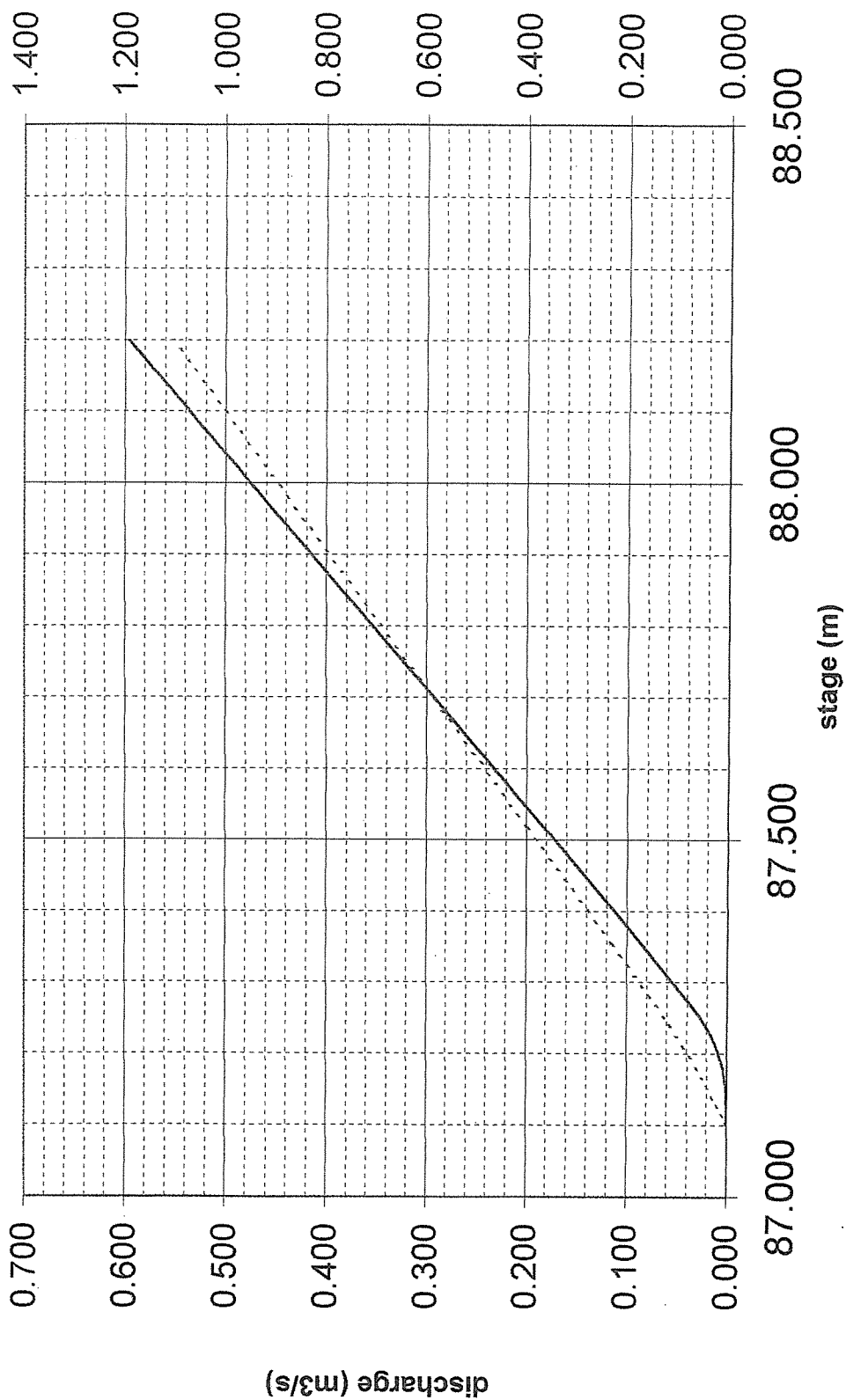
FLOWS (m ³ /S)			
Location	Condition	Return Period	
		5 year	100 year
North Pond	Pre-development	0.22	1.00
	Post-development in	2.60	5.70
	Post-development out	0.27	0.60
South Pond	Pre-development	0.20	1.00
	Post-development in	2.10	4.50
	Post-development out	0.20	0.40

JORDEL ACRES
SWM Pond Design Elements

OUTLET (quality) DETAILS (quantity)				
	d (m)	Q (m ³ /S)	A _o (m ²)	φ (m)
North	<u>0.130</u>	<u>0.027</u>	<u>0.025</u>	<u>0.180</u>
Pond	1.200	0.596	0.196	0.500
South	<u>0.300</u>	<u>0.028</u>	<u>0.018</u>	<u>0.150</u>
Pond	1.800	0.562	0.150	0.450

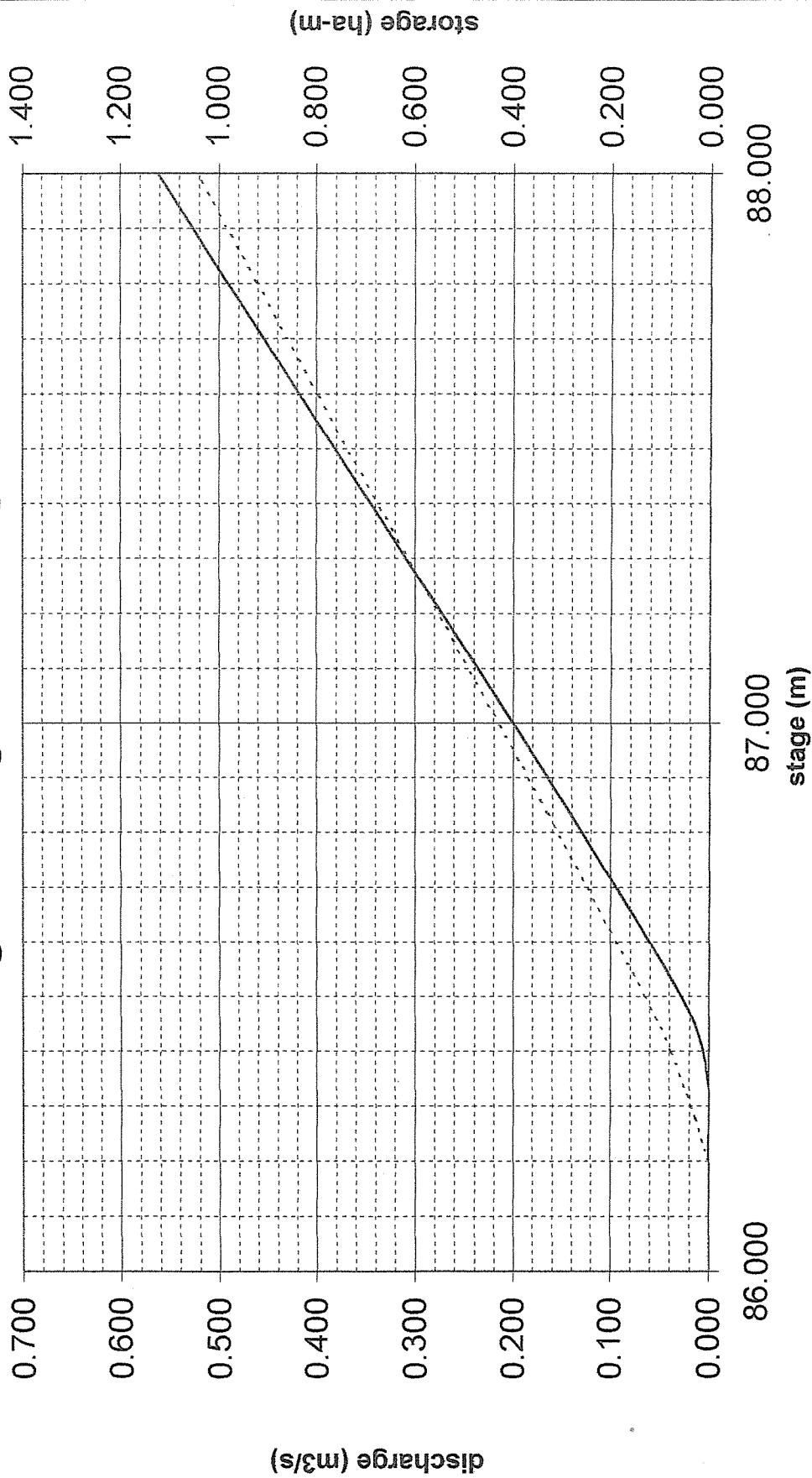
STAGE AREA (for drawdown)			
	Stage (m)	d (m)	Area (m ²)
North	87.10	0.00	9 200
Pond	87.70	0.60	10 580
	88.30	1.20	11 960
South	86.20	0.00	4 000
Pond	87.10	0.90	5 800
	88.00	1.80	7 600

north pond stage-storage-discharge



— stage discharge - - - stage storage

south pond stage-storage-discharge



— stage discharge - - - - stage storage

APPENDIX B
SWMHYMO input and output files

input

```

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

```


output

```
RUN:COMMAND#
001:0001-----
START
  [TZERO = .00 hrs on 0]
  [METCUT= 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	
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

TOTALS

(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)	===== OUTFLOW 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
TIME TO PEAK	(hrs)=	1.33	1.50
RUNOFF VOLUME	(mm)=	75.33	35.46
TOTAL RAINFALL	(mm)=	76.13	76.13
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)		===== OUTFLOW 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 (ha)	QPEAK (cms)	TPEAK (hrs)	R.V. (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	
hrs	mm/hr	hrs	mm/hr	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) |
===== OUTFLOW 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.
-----
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) |
===== OUTFLOW 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 (ha)	QPEAK (cms)	TPEAK (hrs)	R.V. (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 m³/ha

$$A = 29.9 \text{ ha}$$

$$\therefore \text{Permanent Pool (PP) storage} = (250-40) \text{ m}^3 \times 29.9 \text{ ha} = 6279 \text{ m}^3$$

$$\text{Extended Detention (ED)} = 40 \text{ m}^3 \times 29.9 \text{ ha} = 1,196 \text{ m}^3$$

$$\text{As designed, PP} = 8280 \text{ m}^3 > 6279$$

$$\text{ED} = 1218 \text{ m}^3 > 1196$$

$\therefore$ 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

$$A_1 \text{ Area} = 9,200 \text{ m}^2 \quad @ \quad h_1 = 0.0 \text{ m}$$

$$A_2 \text{ Area} = 10,580 \text{ m}^2 \quad @ \quad h_1 = 0.6 \text{ m}$$

$$A_3 \text{ Area} = 11,960 \text{ m}^2 \quad @ \quad h_1 = 1.2 \text{ m}$$

$$\therefore C_3 = 9,200$$

$$\text{from } A = C_2h + C_3$$

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

$$\therefore C_2 = 3,067$$

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

Drawdown time can now be calculated using

$$t = \frac{0.66 C_2 h^{1.5} + 2C_3 h^{0.5}}{2.75 A_o} \quad \begin{matrix} A_o = 0.025 \text{ m}^2 \text{ (assumed)} \\ h = 0.13 \end{matrix}$$

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

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

3. Orifice Flow

$$Q = VA$$

or

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

$$Q = 0.63 \times 0.025 (2 \times 9.81 \times 0.13)^{0.5} \\ = 0.025 \text{ m}^3/\text{sec}$$

where

$$h = 0.13$$

$$g = 9.81$$

$$C = 0.63$$

$$A_o = 0.025$$

4. Forebay Settling Length

$$D = \frac{(r Q_p)^{0.5}}{(v_s)^{0.5}} = \frac{(1.25 \times 0.025)^{0.5}}{(0.0003)^{0.5}} = 10.2 \text{ m}$$

Where $r = \text{length/width} = 50/40 = 1.25$
 $Q_p = \text{peak flowrate from pond}$

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

Designed Forebay is $\therefore$ OK

5. Area Calculation

$$\text{Forebay Area} \quad 40 \times 50 = 2000 \text{ m}^2$$

$$\text{Pond Area} \quad 40 \times 230 = 9,200 \text{ m}^2$$

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

6. Forebay Dispersion Length

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

where $Q = 3.11$ (for 5 yr event)

$d = \text{perm. pool depth}(1.0\text{m})$

$V_f = 0.5$ (desired velocity at berm)

$$D = \frac{8 \times (3.11)}{(1.0)(0.5)} \\ = 49.8 \text{ m} > 10.2 \\ (\text{settling length})$$

7. Minimum Forebay Bottom Width

Equation 3.5

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

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

$$6.2 \text{ m} < 32 \text{ m} \therefore \text{OK}$$

8. Clean Out Frequency

Annual Sediment Load

$$(85\%) \text{ Commercial} \quad 3.8 \text{ m}^3/\text{ha} \times 29.9 = 113.62 \text{ m}^3$$

Removal Efficiency

$$80\% \text{ of } 113.62 \text{ m}^3 = 90.9 \text{ m}^3/\text{year}$$

$$\text{TOTAL CAPACITY for sediment} = (36 \times 1.0 \times 50) = 1600$$

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

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 m³/ha

A = 19.9 ha

∴ Permanent Pool (PP) storage = (250-40) m³ x 19.9 ha = 4,179 m³

Extended Detention (ED) = 40 m³ x 19.9 ha = 796 m³

As designed, PP = 4200 m³

ED = 1290 m³

∴ 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

A₁ Area = 4,000 m² @ $h_1 = 0.0$ m

A₂ Area = 5,800 m² @ $h_1 = 0.9$ m

A₃ Area = 7,600 m² @ $h_1 = 1.8$ m

∴ $C_3 = 4,000$

from $A = C_2h + C_3$

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

∴ $C_2 = 2,000$

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

Drawdown time can now be calculated using

$$t = \frac{0.66 C_2 h^{1.5} + 2C_3 h^{0.5}}{2.75 A_o} \quad \begin{matrix} A_o = 0.018\text{m}^2 \text{ (assumed)} \\ h = 0.3 \end{matrix}$$

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

$$= 92,903 \text{ sec or } 26 \text{ hrs.}$$

3. Orifice Flow

$$Q = VA$$

or

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

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

$$= 0.028 \text{ m}^3/\text{sec}$$

where

$$h = 0.3$$

$$g = 9.81$$

$$C = 0.63$$

$$A_o = 0.018$$

4. Forebay Settling Length

Equation 3.3

$$D = \frac{(r Q_p)^{0.5}}{(v_s)^{0.5}} = \frac{(2 \times 0.009)^{0.5}}{(0.0003)^{0.5}} = 7.7 \text{ m}$$

where $r = \text{length/width} = 40/20 = 2:1$

$\therefore$ Required width = 3.9 m

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

5. Area Calculation

$$\text{Forebay Area} \quad 20 \times 40 = 800 \text{ m}^2$$

$$\text{Pond Area} \quad 20 \times 200 = 4,000 \text{ m}^2$$

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

6. Forebay Dispersion Length

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

$$\text{where } Q = 2.1 \text{ m}^3/\text{s}$$

$$d = \text{perm. pool depth} = 1.5 \text{ m}$$

$$V_f = 0.5 \text{ m/s (desired velocity berm)}$$

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

$$= 22.4 \text{ m} > 7.7 \text{ m}$$

S

7. Minimum Forebay Bottom Width

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

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

∴ OK

8. Clean Out Frequency

Annual Sediment Load

$$(85\%) \text{ Commercial} \quad 3.8 \text{ m}^3/\text{ha} \times 19.9 = 75.6 \text{ m}^3$$

Removal Efficiency

$$80\% \text{ of } 75.6 \text{ m}^3 = 60.5 \text{ m}^3/\text{year}$$

$$\text{TOTAL CAPACITY for sediment} = (14 \times 1.5 \times 40) = 840 \text{ m}^3$$

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

APPENDIX D
Armtec Flap Gate Model 10C
Details

MODEL 10C FLAP GATE

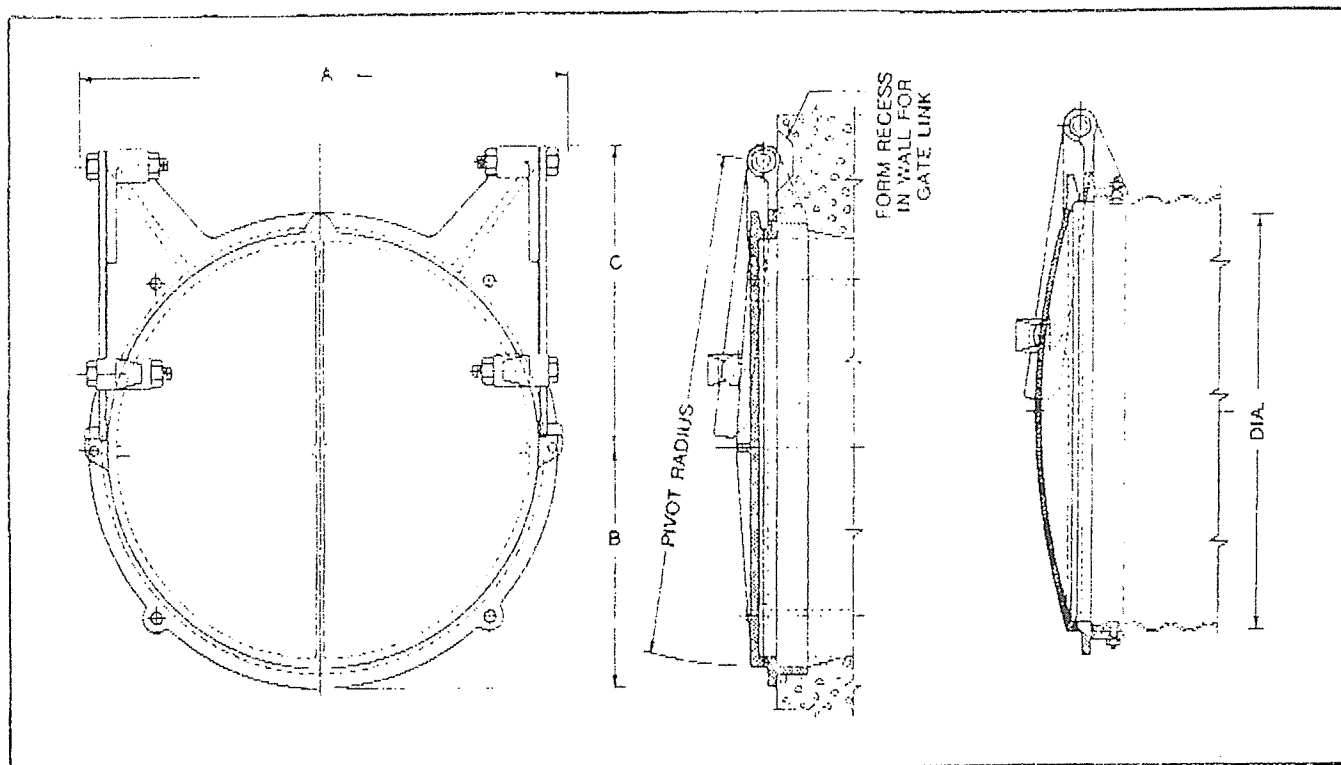
- For seating heads to 3 metres
- Round opening
- Spigot back
- Cast iron seating surfaces

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	562	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

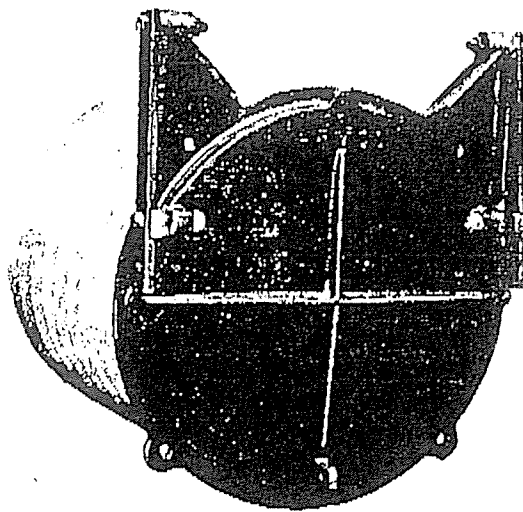
The cast iron seating faces of the seat and cover shall be machined to a plane with a minimum 1600 nm finish.

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.

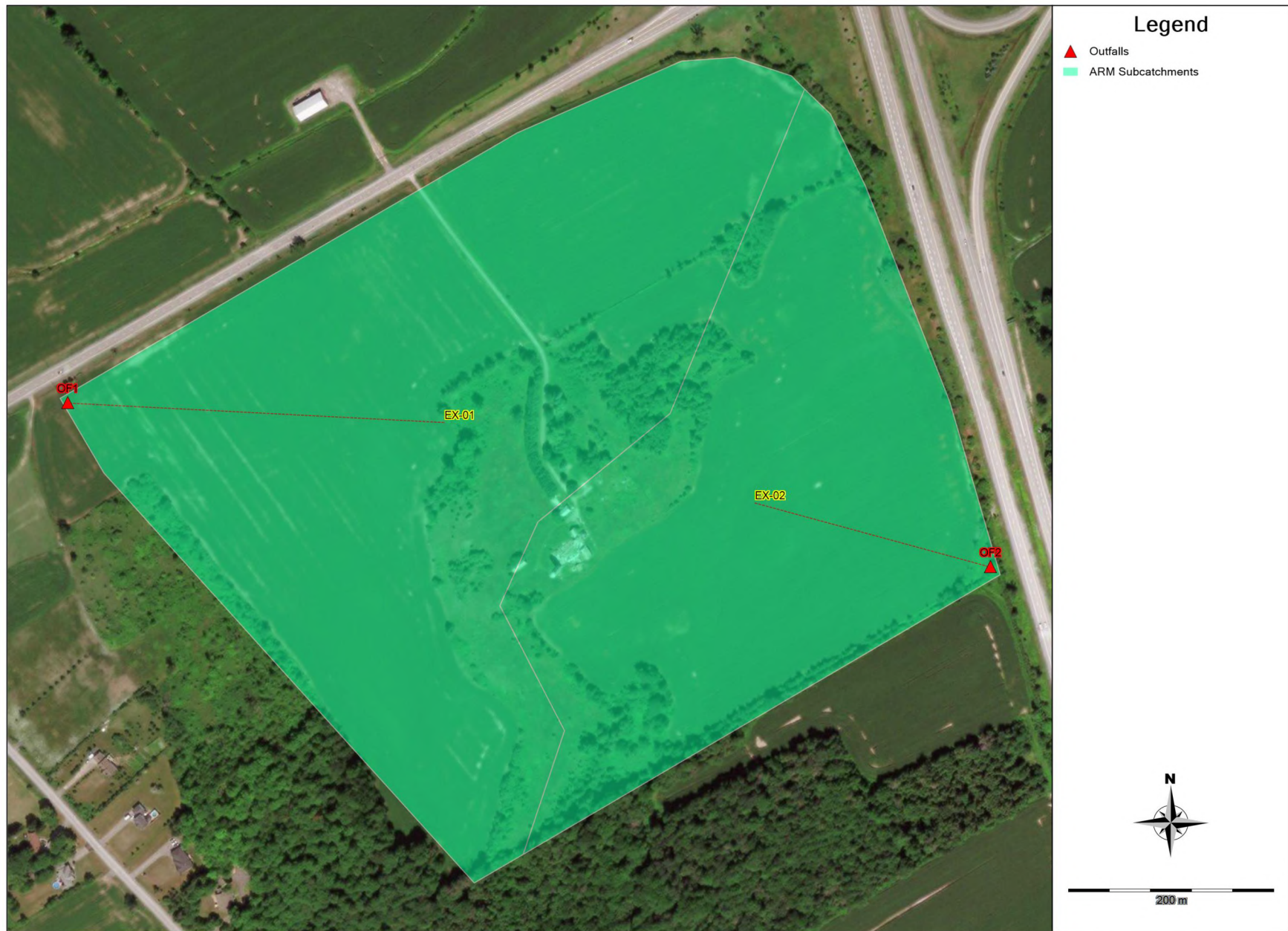
CAST IRON	A 48, Class 30
GALVANIZED STEEL (Fasteners)	A 307 (Bolts)
	A 164 (Galvanized Coating)
GALVANIZED STEEL	A 36 or A 306 (Carbon Steel)
	A 123 (Galvanized Coating)

GATE SCHEDULE			
QUANTITY REQUIRED	SIZE OPENING	SEATING HEAD	REMARKS

APPENDIX B

Stormwater Management Calculations

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



1966 Roger Stevens Drive (119018)

PCSWMM Model Output - Existing Conditions (100-Year, 24 Hour SCS Type II)

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 Peak (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

Date: 2019/07/10

M:\2019\119018\DATA\Calculations\Sewer Calcs\SWM\PCSWMM\Ex_Model_Output.pdf

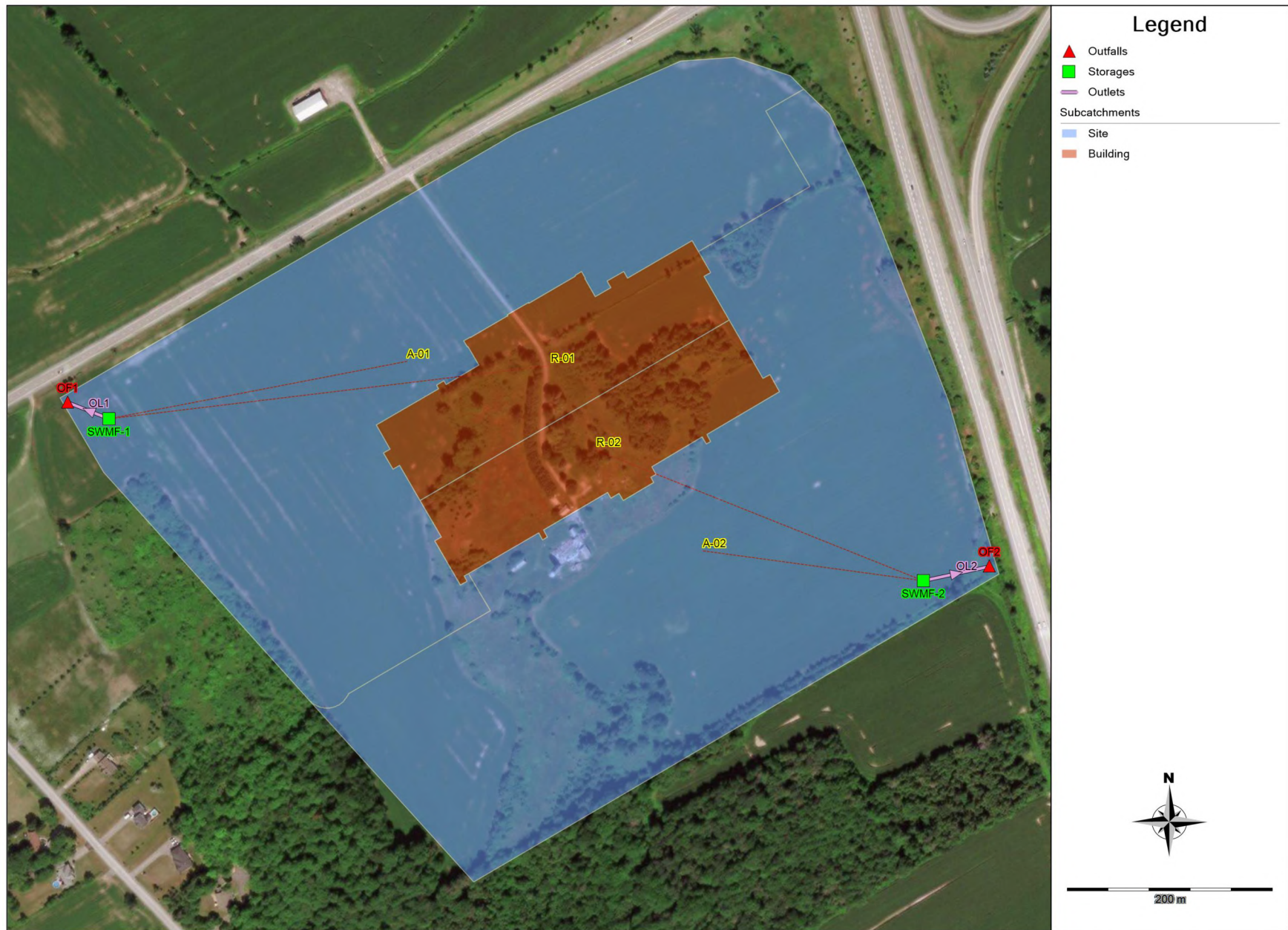
1966 Roger Stevens Drive (119018)
PCSWMM Model Output - Existing Conditions (100-Year, 24 Hour SCS Type II)

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



1966 Roger Stevens Drive (119018) PCSWMM Model Output (100-Year, 3 Hour Chicago) - Proposed Conditions

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

Name	Data Source	Data Type	Recording 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

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	

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

Conduit	Shape	Full Depth	Full Area	Hyd. Rad.	Max. Width	No. of Barrels	Full 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

Date: 2019/07/10

M:\2019\119018\DATA\Calculations\Sewer Calcs\SWM\PCSWMM\Model_Output.pdf

1966 Roger Stevens Drive (119018) **PCSWMM Model Output (100-Year, 3 Hour Chicago) - Proposed Conditions**

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

1966 Roger Stevens Drive (119018) PCSWMM Model Output (100-Year, 3 Hour Chicago) - Proposed Conditions

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	Up Dry	Down Dry	Sub Dry	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl
---------	-------------------------------	-----------	-------------	------------	-------------	------------	--------------	-------------	---------------

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

Date: 2019/07/10

M:\2019\119018\DATA\Calculations\Sewer Calcs\SWMM\PCSWMM\Model_Output.pdf

1966 ROGER STEVENS DRIVE (119018)
EXISTING CONDITIONS SWM CALCULATIONS

Area ID	Area ha	TIMP %	XIMP %	Soil Type % B	Soil Type % D	% 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

Overland Flow							
Area ID	Length (m)	Elevation U/S (m)	Elevation D/S (m)	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

Pond ID	Total Area (ha)	C	Allowable Release Rate		Uncontrolled Release Rate		Required Storage Volume (m ³)
			5-Year	100-Year	5-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	Required Storage Volume (m ³)	Required Storage Volume (m ³)
SWMF-1 (North)	3610	5150	8760
SWMF-2 (South)	3370	3900	7270

1966 ROGER STEVENS DRIVE (119018) CONCEPTUAL SWM FACILITY DESIGN

Required Storage Volumes (Quality) SWMF-1

Drainage Area	20.4	ha
% Impervious:	60%	
<i>Enhanced protection (80% TSS removal):</i>		
<i>Treatment Volume</i>	217	m3/ha
<i>Extended Detention Storage:</i>		
	40	m3/ha required
	816	m3 required
	850	m3 provided
	41.7	m3/ha provided
<i>Perm Pool:</i>		
	177	m3/ha required
	3611	m3 required
	3610	m3 provided
	177.0	m3/ha provided
<i>Extended Detention:</i>		
	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, C	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

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

$$= 25 \text{ m}$$

Check Dispersion Length, Dist₂

Desired velocity in forebay, V_f =	0.15 m/s
Inlet flow rate, Q_{2yr} =	0.619 m³/s
Depth in forebay, d =	1.3 m

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

$$= 25 \text{ m}$$

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

Required Storage Volumes (Quality) SWMF-2

Drainage Area	22.3	ha
% Impervious:	34%	
<i>Enhanced protection (80% TSS removal):</i>		
<i>Treatment Volume</i>	191	m3/ha
<i>Extended Detention Storage:</i>	40	m3/ha required
	892	m3 required
	900	m3 provided
	40.4	m3/ha provided
<i>Perm Pool:</i>	151	m3/ha required
	3367	m3 required
	3370	m3 provided
	151.1	m3/ha provided
<i>Extended Detention:</i>	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, C	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

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

$$= 26 \text{ m}$$

Check Dispersion Length, Dist₂

Desired velocity in forebay, V_f =	0.15 m/s
Inlet flow rate, Q_{2yr} =	0.619 m³/s
Depth in forebay, d =	1.3 m

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

$$= 25 \text{ m}$$

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