FUNCTIONAL SERVICING STUDY

15544 McLAUGHLIN ROAD
VILLAGE OF INGELWOOD

TOWN OF CALEDON

May 2024

REGISTERED PROFESSIONAL ENGINEER
S D. LANG
MAY 28, 2024
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
S.SINGH
100514247
May,28,2024
PROVINCE OF ONTARIO

PROJECT NO: W22002

CANDEVCON GROUP INC.
CONSULTING ENGINEERS & PLANNERS

GTA WEST OFFICE
9358 GOREWAY DRIVE
BRAMPTON, ONTARIO L6P 0M7
T: (905) 794-0600 F: (905) 794-0611

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1. INTRODUCTION

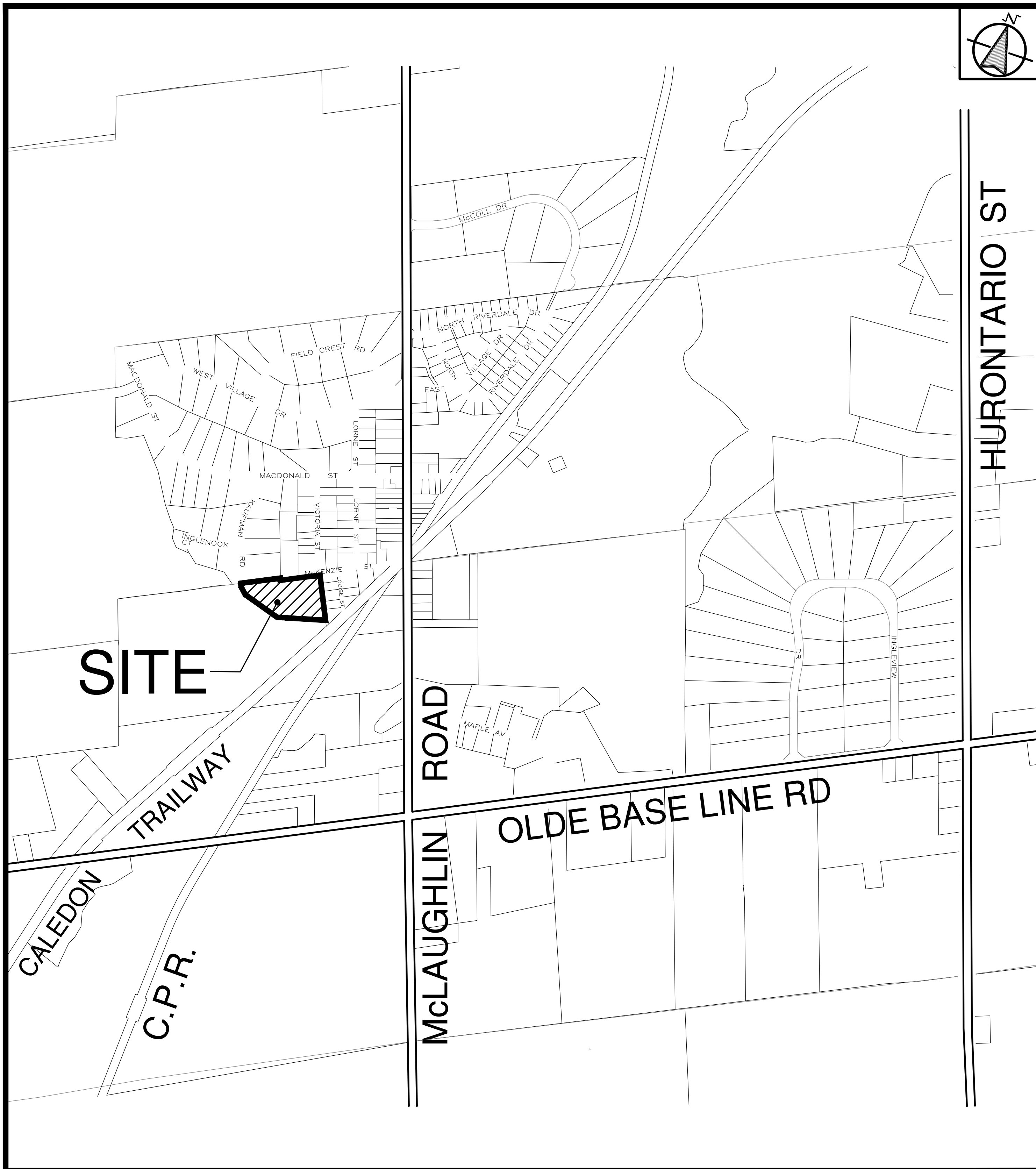
This Study has been prepared as a technical document in support of the Draft Plan application for the subject subdivision located at 15544 Mclaughlin Road and addresses sanitary, storm and water servicing and stormwater management.

The proposed subdivision is located in the Village of Inglewood, in the Town of Caledon and is bounded by agricultural lands to the south and west and existing residential lands to the north and east.

The subdivision, as illustrated on the Draft Plan (copy attached) comprises an area of 4.02 ha and includes:

- Thirteen (13) Detached Residential Lots;
- One (1) Parkette Block;
- Two (2) Open Space Blocks;
- One (1) SWM Block;
- One (1) storm easement Block;

The report describes the existing site conditions, and the proposed sanitary, storm and water systems, as well as the stormwater management infrastructure. The report includes preliminary grading information and outlines the required Erosion and Sediment Control Measures.



2868577 ONTARIO INC.
PROPOSED RESIDENTIAL SUBDIVISION
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD

TOWN FILE No. PRE-2029-0197 REGION FILE No.

LOCATION PLAN

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS

9358 GOREWAY DRIVE, BRAMPTON, ONTARIO L6P 0M7
TEL. (905) 794-0600 FAX (905) 794-0611

DRAWN BY: Y.D.N.	JOB No. W22002
SCALE: N.T.S.	SKETCH No.
DATE: AUGUST 15th., 2023	**FIG-1**

2. BACKGROUND TECHNICAL STUDIES

The subject subdivision is located in the Village of Inglewood in the Town of Caledon. The following applicable Studies were reviewed which are relevant to the development servicing of the subject subdivision.

2.1 Inglewood Village Water and Wastewater Servicing Plan[1]

The Town of Caledon and the Region of Peel commissioned a study of the water and wastewater capacity in the village of Inglewood in 1999. The report reviewed both existing and proposed sanitary and water demands for the area. The report included the subject lands in their analysis. The report recommended the following;

WATER

- Connect McDonald Street watermain with Balmer Heights area
- New permit to take water to increase capacity from existing wells
- New pressure reducing valve at West Village drive and Mclaughlin road
- Expansion of the existing reservoir

WASTEWATER

- New sanitary sewers on Mclaughlin Road, McDonald street, McKenzie Street, Louise Street Victoria Street and Lorne Street
- New wastewater treatment plant

[1] Inglewood Village Water and Wastewater Servicing Plans, prepared for The Town of Caledon, Credit Valley Conservation, Region of Peel, dated June 1999, prepared by XCG

3. EXISTING CONDITIONS

3.1 General

The subject lands are currently agricultural lands.

As part of the Planning process for the subject subdivision the following Studies were completed:

- Preliminary Geotechnical Investigation[2]
- Preliminary Hydrogeological Investigation[3]

3.2 Topography, Drainage and Natural Features

As noted above, the lands are currently agricultural lands and generally drain from north to south toward an existing creek located south of the subject lands.

3.3 Physiography and Geotechnical Conditions

The preliminary Geotechnical Investigation (copy of report included in Appendix "D") indicated topsoil depths of 0.15 to 0.30m overlaying earth fill to depths up to 2.2m. The earth fill overlays a layer of silt, sandy silt and silty sand. The boreholes also encountered sand and gravel in the eastern portion of the site.

Falling Head tests were conducted to determine soil conductivity (K) values of the soils. Based on the testing the average K value was calculated to be between 6×10^{-7} to 1.7×10^{-6} m/s. A copy of the hydrogeological report is included in Appendix "E".

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2 Report to 2868577 Ontario Inc., Geotechnical Investigation for Proposed Residential Development, 15544 McLaughlin Road, Town of Caledon, Prepared by Soil Engineers reference number 2301-S042, Dated March 2024

3 Report to 2868577 Ontario Inc., Hydrogeological Assessment for Proposed Residential Development, 15544 McLaughlin Road, Town of Caledon, Prepared by Soil Engineers reference number 2301-W042, Dated August 2023, Revised on April 9, 2024

4. SANITARY AND WATER SERVICING

4.1 Sanitary

4.1.1 Existing Sanitary Sewers

The existing sewers in the vicinity of the subdivision consists of a 250mm sanitary sewer on MacKenzie Street.

4.1.2 Proposed Sanitary Sewer System

The subject subdivision is proposed to drain to the existing sanitary sewer on MacKenzie Street via a proposed 250mm sanitary sewer.

The drainage areas for the sanitary sewer system are shown on Drawing SA-1, and the Sanitary Sewer Design Sheets are included in Appendix "A".

The sewer system will be designed in accordance with the Region of Peel Criteria and Standards. A copy of the Region of Peel Connection Multi-Use Demand Table for the subdivision is included in Appendix "A". The sanitary design population/ha for different land uses is provided in Table 4.1.

TABLE 4.1 SANITARY DESIGN POPULATIONS PER HECTARE

Density	**Pop. /Hectare**
Single family (greater than 10m frontage)	50 persons/hectare

The anticipated sanitary flows from the development, based on actual unit types and number of units are given in Table 4.3.

TABLE 4.2 SANITARY DESIGN POPULATIONS PER UNIT TYPE

Unit Type		Population/Unit
Single detached		4.20

TABLE 4.3 SANITARY FLOWS BASED ON UNIT TYPE

Developable Area(ha)	1.24		
Unit Type	Units	Pop/Unit	Population
Single Detached	13	4.2	54.6
	total population		54.6
Average Day (L/s)	0.183		
Peak Factor	4.307		
Infiltration	0.322		
Total Peak Sanitary Flow (Ls)	1.112		

A copy of the Region of Peel Connection Multi-Use Demand Table for the subdivision is included in Appendix “A” and is based on the values shown in Table 4.2.

The downstream sanitary sewers were reviewed to confirm capacity exists for the proposed development, based on a sanitary design prepared by RJ Burnside for Inglewood Village. The review shows that there is capacity within the existing sewers for the proposed development. A copy of the calculation sheet is included in Appendix “A”

4.2 Water

4.2.1 Existing and Planned Watermains

The proposed subdivision is located within Region of Peel water pressure Zone IN-7. There are existing 150mm watermains on Kaufman Road and MacKenzie Street adjacent to the site.

4.2.2 Proposed Watermain System

It is proposed that the subdivision be serviced via a 150m watermain that will connect to the existing watermains located on Kaufman Road and MacKenzie Street.

The anticipated water demands for the subject lands are summarized in Tables 4.4.

TABLE 4.4 WATER DEMAND BASED ON UNIT TYPE

Unit Type	Units	Pop/Unit	Population
Single detached	13	4.2	54.6
	Total Population		54.6
Average Day (L/s)	0.177		
Max Day (L/s)	0.354		
Peak Hour (L/s)	0.531		
Max Day Plus Fire (L/s)	133.690		

5. STORM DRAINAGE and STORM WATER MANAGEMENT

5.1 General–Related Background Studies and Design Guidelines

The following reports and documentation were consulted to establish stormwater management design criteria for the proposed development.

- *Inglewood Village Community Design Guidelines, Town of Caledon, Credit Valley Conservation & Regional of Peel, dated July 12th, 1999.*
- *Tributary Study, Village of Inglewood, Town of Caledon, dated May 1999.*
- *Inglewood Village Environmental Management Plan, Town of Caledon, Credit Valley Conservation & Regional of Peel, dated June 1999.*
- *Characterization Report, East Credit Sub Watershed Study (Sub watershed 13), Town of Caledon, Credit Valley Conservation & Regional of Peel, dated July 2007.*
- *Stormwater Management Guideline, Credit Valley Conservation, dated July 2022.*
- *Low Impact Development Stormwater Management Planning and Design Guide, Credit Valley Conservation Authority (2010).*
- *Ministry of Environment (MOE), Stormwater Management Planning and Design Manual, March 2003.*
- *Town of Caledon, Development Standards Manual, Version 5.0, 2019.*
- *Environmental Compliance Approval for a Municipal Stormwater System, ECA Number: 324-S701.*

5.2 Storm Sewer design

Storm sewers shall be designed in accordance with the current Town of Caledon Development Standards Manual. Storm sewers will be designed to accommodate a 10-year storm with footing drains connected to the storm sewer. The proposed storm sewer design is shown on drawing STMDR -1 and PS-1. In the event of a storm greater than the 10-year event, the proposed storm sewer will surcharge, forcing stormwater to the surface. The site will be graded so that the major storm runoff will be conveyed via overland flow route (roads) and will enter the dry pond.

5.3 Stormwater Management Design Criteria

The proposed development will meet the design standards set out in the MOE Stormwater Management Planning, Inglewood Studies by Town of Caledon and CVC engineering standards. A brief summary of the design criteria is provided below;

- For proposed development, the return frequency for design shall be 10-year for the Minor System (Storm sewers) and 100-year for the Major System.
- The Town of Caledon Rainfall Intensity Curves is to be used for the design of the Minor System.
- Water Quantity Control: Post-development flows to pre-development levels.
- Water Quality Control: Level 1 (Enhanced level protection – 80% TSS Removal) and Phosphorous Removal is required.
- The Rainfall data used for VO modelling is Toronto Pearson Int Airport IDF values for SCS Type II storm.

5.4 Proposed Stormwater Measures

5.4.1 Water Quantity Control

The release of stormwater from the development assumes that "post-development flows will be equal to or less than the pre-development flows". For this development, the predevelopment runoff was estimated through Visual Otthymo Modelling. The Toronto Pearson Int'l Airport parameters were used to determine the storm intensity values and the following 2, 5, 10, 25, 50 and 100-year pre- and post-development release rates have been calculated. The allowable peak flows for the proposed development will be determined using the pre-development flows as shown in Table I. VO modelling results can be found in Appendix A.

TABLE I

Pre-Development Peak Flows

Drainage Area = 1.76 ha (including 0.13 ha External Drainage from North) **Composite Runoff C =** 0.25						
Storm Event	**2-Year Storm**	**5-Year Storm**	**10-Year Storm**	**25-Year Storm**	**50-Year Storm**	**100-Year Storm**
VO Flows (m^3/s)	0.108	0.166	0.211	0.266	0.314	0.362

The post-development drainage replicates the existing drainage patterns and will convey the flows from the site to the proposed Dry Pond for quantity control before release. The proposed storm drainage system is shown on Drawing STMDR -1. Table II below summarizes post-development peak flows and demonstrates that the post-development flows for all storm events are equal to or less than the pre-development peak flows.

TABLE II
Post-Development Peak Flows

POST - DEVELOPMENT SCENARIO						
Total Site Development Area	1.76 ha					
Controlled Site Area =	1.60 ha					
External Drainage Area =	0.13 ha					
Un-Controlled Area =	0.03 ha (Open Space draining directly to the Valley in the South)					
Total Area to Dry Pond =	1.73 ha					
Time of Concentration (min) =	10 min					
Composite Runoff C =	0.55					
VO Uncontrolled Peak Flows (Area = 0.03 Ha)						
Storm Event	**2-Year Storm**	**5-Year Storm**	**10-Year Storm**	**25-Year Storm**	**50-Year Storm**	**100-Year Storm**
VO Flows (m^3/s) – NHYD 5	0.002	0.003	0.004	0.005	0.005	0.006
Peak Flows from Dry Pond (Area = 1.73 Ha)						
VO Flows (m^3/s) - NHYD 4	0.100	0.117	0.126	0.139	0.145	0.151
Comparison between Pre and Post Flows from Site						
Pre	*0.108*	*0.166*	*0.211*	*0.266*	*0.314*	*0.362*
Post (Total Flows from Site) -NHYD 6	0.101	0.119	0.129	0.142	0.149	0.156

<u>**Dry Pond Design**</u>

The proposed dry pond is designed with 4:1 side slope and a 100-year storage capacity of 579 m^3 at an elevation of 276.69m within the pond. The top of the berm elevation is 276.75m. A 245 mm diameter orifice plate located within a Hickenbottom outlet structure of the pond will control outflow from the pond and reduce it to pre-development values. The dry pond has been designed to provide quantity control for all storm events up to and including the 100-year storm event. The following table summarizes the dry pond stage storage discharge (detailed calculations located in Appendix "C").

TABLE II

Dry Pond – Stage-Storage-Discharge

Orifice Plate Diameter (mm) =			245 mm (Flow Control), Invert = 275.35m			
Storm Event	**2 - Year**	**5 - Year**	**10 - Year**	**25 - Year**	**50 - Year**	**100 – Year**
VO Storage Required (m^3)	193	265	330	436	503	572
Storage provided in Dry Pond (m^3) *	203	269	349	438	552	579
Discharge (m^3/s) *	0.106	0.118	0.129	0.139	0.150	0.152
Elevation (m)	276.00	276.15	276.31	276.47	276.65	276.69
** Refer to Dry Pond Orifice Calcs*						

5.4.2 Water Quality Control

In accordance with the MOE SWM planning and Design Manual for this site, stormwater should be treated to an Enhanced Protection Level (Level 1) which is 80% TSS (Total Suspended Solids) Removal.

Major storm event stormwater flows from the site will be conveyed via overland flow into the dry pond. The Dry Pond will provide Basic Level (60%) suspended solid removal. Following the discharge from the (SWM) dry pond, the stormwater will pass through a filtration unit before outletting the headwall. Based on the manufacturer's modelling software, Jellyfish unit JF6-6-1 has been designed to provide the removal of approximately 90% of the Total Suspended Solids. For a detailed sizing report of the JellyFish unit refer to Appendix "C".

5.4.3 Phosphorus Budget

Inglewood Village Studies has identified the importance of reducing the phosphorus level in the watercourses in these areas. The MOE Phosphorus Loading Development tool[4] has been utilized to determine Pre and Post Development Phosphorus conditions.

i. Pre-Development Phosphorus Loading

[4] Hutchinson Environmental Sciences Ltd., Phosphorus Budget Tool in Support of Sustainable Development for the Lake Simcoe Watershed, prepared for Ontario Ministry of the Environment, Dated: March 30th, 2012, Version 2.

The Pre-Development phosphorus loading was calculated based on the existing conditions from the site survey. The Pre-Development land use consists of vacant agricultural fields. The Pre-Development phosphorous loading for the 1.76 ha area was calculated to be 0.21 Kg/year with no Best Management Practices (BMPs).

ii. Post-Development Phosphorus Loading

The Post-Development land use for the subject site consists of residential development [Refer to Draft Plan prepared by Candevcon Ltd.]. The Post-Development phosphorus loading, with no BMPs, was calculated to be 3.08 Kg/year.

To minimize the amount of phosphorus being discharged from the site, a treatment train approach is proposed. Runoff from the site will be treated by dry pond and Jellyfish unit prior to release. The following Table IV represents the anticipated phosphorus loading and treatment process results.

TABLE IV

Pre and Post Phosphorus Loading and Mitigation

Pre-development phosphorus loading =				0.21 Kg/Year
Post-development phosphorus loading (with No BMP) =				2.32 Kg/Year
Treatment Train Approach through LIDs				
Treatment ID	**Run-off From**	**LID Type**	**% Phosphorus Removal as per MOE**	**Phosphorus Removed (Kg/Year)**
1	Site (Residential)	Dry Pond	10%	0.23
2	Site (Residential)	Filtration Unit (Jellyfish)	77.5%	2.28
Total Post-development phosphorus loading =				**0.33 Kg/Year (86% P removal)**

5.4.4 Water Balance

Since the post-development condition will increase the imperviousness of the site the pre and post infiltration volumes were calculated to assess the infiltration deficit and the proposed mitigation measures were evaluated to demonstrate the pre-development infiltration can be matched in the post-development condition.

Site climatic conditions were calculated using the Thornthwaite method utilizing meteorological data obtained from Environmental Canada historical weather data for Toronto Lester B. Pearson, ON (Climate Id: 6158733) weather station. Monthly precipitation averages were obtained over the period of 1981 to 2010.

Site conditions have been summarized as follows:

- Precipitation 786 mm/yr
- Evapotranspiration 625.1 mm/yr
- Water Surplus 160.9 mm/yr
- Infiltration 96.6 mm/yr
- Run-off 64.4 mm/yr

Infiltration rates were determined based on infiltration guideline sub-factors provided within the "MOEE Hydrogeological Technical Information Requirements for Land Development Application", (MOEE, 1995).

Based on the noted climate and soil conditions of the Subject Site it is expected that the increase in impervious areas will result in a groundwater infiltration deficit following development. The Water Balance calculations are provided in Appendix "C". As shown in the calculations, the proposed development without mitigation would result in an infiltration deficit of 895m3/year. To balance this infiltration deficit, StormTech underground infiltration chambers will be utilized to meet the volume requirements. The sizing and design specifications of infiltration chambers will be finalized during detailed design stage.

5.4.6 CLI ECA Performance Criteria

The stormwater management design of the site aligns with the standards set forth in the CLI ECA Stormwater Management criteria. Below is a detailed description of the criteria and how they are met:

TABLE V
SUMMARY OF CLI ECA STORMWATER MANAGEMENT PERFORMANCE

Control	Criteria	SWM Strategy	Check
Water Quality	Enhanced Level Suspended Solids Removal Phosphorus Removal	1. Rear yard and roof will drain to Infiltration galleries proposed in ROW. 2. Dry Pond will provide basic 60% TSS Removal. 3. Jellyfish filtration unit will further provide 89% TSS removal and 77.5% Phosphorus Removal	*Criteria is Met*
Erosion Control	25mm Storm Event	Dry Pond is designed for 25mm rainfall event volume and further erosion control will be provided with engineered low flow channel at outlet	*Criteria is Met*
Water Quantity	Stormwater volumes generated from the geographically specific 90th percentile rainfall event on an annual average basis from all surfaces on the entire site is targeted for control. Control is in the following hierarchical order, with each step exhausted before proceeding to the next: 1) retention (infiltration, reuse, or evapotranspiration), 2) LID filtration, and 3) conventional Stormwater management. Step 3, conventional Stormwater management, should proceed only once Maximum Extent Possible* has been attained for Steps 1 and 2 for retention and filtration.	Building roof will drain to Infiltration galleries proposed in ROW which will promote storage and support infiltration. 2 to 100-Year Storm Event storage will be provided in Dry Pond. ** Dry Pond volume includes roof drainage.*	*Criteria is Met*
Flood Control	Manage peak flow control as per watershed/sub watershed plans, municipal criteria being a minimum 100-year return storm	Post Development Flows controlled to Pre-Development Flows for 2 to 100-Year Storm Events	*Criteria is Met*
Water Balance	Control the recharge to meet Pre-Development conditions	Roofs downspout to connect to Infiltration chambers in ROW to achieve infiltration deficit volume	*Criteria is Met*

** Maximum Extent Possible means maximum achievable Stormwater volume control through retention and LID filtration engineered/landscaped/technical Stormwater practices, given the site constraints*

6 EROSION AND SEDIMENT CONTROL

Erosion and sedimentation are naturally occurring processes that involve particle detachment, sediment transport and deposition of soil particles. Construction activities commonly alter the landscapes where they are located, exacerbating these natural processes. One of the most significant alterations encountered during construction is the removal of the vegetation that stabilizes the subsoil. In the absence of the vegetation, the underlying soils are fully or partially exposed to various natural forces such as rain, flowing water, wind, and gravity.[5]

The discharge of high sediment loads to natural watercourses has significant impacts on receiving waters and aquatic habitat. Some specific examples include:

- Degradation of water quality;
- Damage or destruction of fish habitat;
- Increased flooding.

In consideration of the above, it is necessary as part of the Final Design and implementation of infrastructure and development servicing to incorporate a comprehensive Erosion and Sediment Control Plan. The objectives are:

(i) Minimize wherever possible the extent of vegetation removal;
(ii) Provide appropriate sediment control measures to minimize the off-site transport of sediment;
(iii) Minimize the extent of time that sites are devoid of stabilizing vegetation;
(iv) Provide interim erosion control measures where permanent restoration is not feasible.
(v) Provide permanent restoration to eliminate future erosion.

[5] *Erosion and Sediment Control Guidelines for Urban Construction,* December 2006, Greater Horseshoe Conservation Authorities.

The Erosion and Sediment Control Plan should consider the specific characteristics of each development site and address the requirements relating to the following typical construction stages:

- Topsoil Stripping and Site Pre-Grading
- Infrastructure Servicing
- Building Construction

A "treatment train" approach is recommended in the development of an appropriate Erosion and Sediment Control Plan in compliance with the *Erosion and Sediment Control Guidelines for Urban Construction.* Typical sediment control measures include:

- Installation of double silt fencing along the boundary of work areas adjacent to the NHS;
- Construction of vegetated cut off swales including sediment traps and rock check dams;
- Stabilization of temporary sediment traps and provision of vegetated filter strips adjacent to the NHS;
- Provision of catch basin sediment controls.

Inherent in the Erosion and Sediment Control Plan is a monitoring program with an Action Plan to implement remedial measures in a timely manner where required.

As part of the final engineering design, the Sediment and Erosion Control Plan will be prepared including sizing of temporary sedimentation ponds and sediment traps.

7 SUMMARY AND COMPLIANCE DECLARATION

7.1 Summary

Based on the findings of this report, the conclusions and recommendations are as follows:

(i) Sanitary sewer servicing can be achieved by connecting to the existing Sanitary sewers on Mackenzie Street.

(ii) Water supply can be achieved by connecting to the existing watermains on Kaufman Road and MacKenzie Street.

(iii) Storm water management will be provided by a dry pond to be located on Block 3 as well as by a filtration system.

(iv) It should be noted that the details of the stormwater management systems will be finalized during the detailed design stage of the Subdivision;

(v) Erosion and sediment control measures will be installed as recommended.

7.2 Compliance Declaration

The undersigned hereby confirm that:

(i) The Functional Servicing/Stormwater Management Study complies with the Town of Caledon current edition of the Subdivision Design Manual

(ii) The drainage of the adjacent lands will not be adversely affected by the proposed stormwater management provisions.

Scott Lang, P. Eng

Shuchi Singh, P.Eng.

REFERENCE DRAWINGS

DRAWING S-155-13/P5	Kaufman Road, STN 0+000 to STN 0+256.554
DRAWING 06-2720	McKenzie Street, STA 0+180 to STA 0+320

DRAWINGS

DRAWING PL-1	Draft Plan
DRAWING PS-1	Preliminary Servicing Plan
DRAWING PG-1	Preliminary Grading Plan
DRAWING STMDR -1	Storm Drainage Area Plan
DRAWING SANDR-1	Sanitary Drainage Area Plan
DRAWING WM-1	Water Distribution Plan
DRAWING SWM-G5	Storm Water Management Pond Plan and Sections

APPENDIX A

SANITARY SEWER CALCULATIONS AND MULTI USE TABLES

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS
9358 GOREWAY DRIVE — TEL (905) 794-0600
BRAMPTON ON. L6P-0M7 — FAX (905) 794-0611

Subdivision:	**15544 MCIAUGHLIN ROAD**	**CITY OF BRAMPTON**	**Project No.:**	**W22002**
File No.:	**Pre 2020-0106**	**SANITARY DRAINAGE**	**Date:**	**April 30, 2024**
Consultant:	**Candevcon Limited**		**Prepared By:**	**YDN**
Drainage Area Plan:	**SA-1**		**Checked By:**	**SDL**

LOCATION					**SECTION AREA (Ha)**									**POPULATION**					**FLOWS**										**REMARKS**
STREET		AREA ID	MAINTANANCE HOLES Upstream	Downstream	Executive Residential	Low Density	Low / Medium Density	Medium Density	High Density	Commercial /Retail	Junior School	Senior School	High School	Residential	Commercial	School	TOTAL POP	ACCUM. POP.	AREA (ha)	ACCUM. AREA (ha)	PK. DAY FLOW (m^3/s)	INFILT. (m^3/s)	**TOTAL FLOW (m^3/s)**	SIZE (mm)	SLOPE (%)	**CAPACITY** (m^3/s)	**VELOCITY** FULL FLOW (m/s)	ACT. FLOW (m/s)	DESIGN FLOW / FULL FLOW %
1		2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	20	21	22	23	24	25	26	27	28	29	30
		1	MH1A	MH2A		0.917								46	0.00	0.00	46	46	0.92	0.92	0.013	0.000	**0.013**	250	5.40%	**0.138**	**2.82**	1.08	9
		2	MH2A	MH3A		0.164								8	0.00	0.00	8	54	0.16	1.08	0.013	0.000	**0.013**	250	2.50%	**0.094**	**1.92**	0.90	14
		3	MH3A	OUTLET		0.000								0	0.00	0.00	0	54	0.00	1.08	0.013	0.000	**0.013**	250	2.10%	**0.086**	**1.76**	0.90	15

REGION OF PEEL DESIGN CRITERIA

equivalent populations	**people/ha**
Executive residential	10
low density (greater than 10m frontage)	50
Low density (less than 10m frontage)	70
Low/medium density	123
Medium density	175
High density	475
Junior school	pop/3
Senior School	pop/2
High School	(pop x 2)/3
Commercial/retail	50

Average Day Flow: 302.8 Lpcd

Peaking Factor: 1+14/(4+(P)^0.5) P = Pop. in 1000's

Infiltration: Pipes: 0.0002 $m^3/s/ha$

0.000028 $m^3/s/m$

Manning's Co-eff.: n = 0.013

15544 MCLAUGHLIN ROAD

Connection Multi Use Demand Table

WATER CONNECTION

Connection point [3]				
Pressure zone of connection point			5	
Total equivalent population to be serviced [1]			0	
Total lands to be serviced			1.24	
Hydrant flow test				
	Hydrant flow test location			
		Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure				
Maximum water pressure				
No.	**Water demands**			
	Demand type	**Demand (in l/s)**		
		Use 1 [5]	**Use 2 [5]**	**Total**
1	Average day flow	0.18		0.18
2	Maximum day flow	0.35		0.35
3	Peak hour flow	0.53		0.53
4	Fire flow [2]	100.00		100.00
Analysis				
5	Maximum day plus fire flow	100.35		100.35

WASTEWATER CONNECTION

				Total
Connection point [4]				
Total equivalent population to be serviced [1]		0		0
Total lands to be serviced		1.24		1.240
6	Wastewater sewer effluent (in l/s)	1.1		1.1

1) The calculations should be based on the development estimated population (employment and/or residential).

2) fire flow based on Table 8.1, Fire Flow requirements, MOE Design Guidelines For Drinking Water Systems

3) Please specify the connection point ID

4) Please specify the connection point (wastewater line or manhole ID)

Also, the "total equivalent popopulation to be serviced" and the "total lands

to be serviced" should reference the connection point. (The FSR should contain one copy of Site Servicing Plan)

5) Please complete as many uses are necessary for the development. (Please specify the use)

Population

area (Ha)	1.24				
land use	residential				
total population	54.6				

Unit Count Sanitary Flows Based on Unit Type

unit type	units	pop/unit	pop/sm		population
single detached	13	4.2			54.6
semi detached		4.2			0
townhouse		3.4			0
large apartment (greater than 1 bedroom)		3.1			
small apartment (less than or equal to 1 bedroom)		1.7			
Medium density (ha)		175			0
reserve (ha)		50			0
commercial/industrial (ha)		50			0
				residential population	54.6
				non-residential population	0
				total population	54.6

Water Demand

demand type	factor	demand		
			Residential per capita sanitary demand(lpcd)	280
ave day	1	0.177	Non-Residential per capita sanitary demand(lpcd)	300
max day	2	0.354		
peak hour	3	0.531		

Sanitary Demand

			Residential per capita sanitary demand(lpcd)	290
Average day (l/s)	0.183		Non-Residential per capita sanitary demand(lpcd)	270
peak factor	4.307	*calculated using emiprical formula		
infiltration	0.322		infiltration (l/sec/ha)	0.26
total peak sanitary flow (l/s)	1.112			

Fire Flow

Unit Type	Required flow (L/s)	Suggested flow (L/min)	6000	SINGLE DETACHED
Residential	100.000			

F (220CA^.5)/60

F the required fire flow in litres per secound

C = coefficient related to the type of construction.

= 1.5 for wood frame construction (structure essentially all combustible).

= 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior).

= 0.8 for non-combustible construction (unprotected metal structural components, masonry or metal walls).

= 0.6 for fire-resistive construction (fully protected frame, floors, roof).

A The total floor area in square metres (including all storeys, but excluding basements at least 50 percent below grade) in the building being considered.

For fire-resistive buildings, consider the two largest adjoining floors plus 50 percent of each of any floors immediately above them up to eight, when the vertical openings are inadequately protected. If the vertical openings and exterior vertical communications are properly protected (one hour rating), consider only the area of the largest floor plus 25 percent of each of the two immediately adjoining floors.

1. To the value obtained above a percentage should be added for structures exposed within 45 metres by the fire area under consideration. This percentage shall depend upon the height, area, and construction of the building(s) being exposed, the separation, openings in the exposed building(s), the length and height of exposure, the provision of automatic sprinklers and/or outside sprinklers in the building(s) exposed, the occupancy of the exposed building(s), and the effect of hillside locations on the possible spread of fire.

The charge for any one side generally should not exceed the following limits for the separation:

Separation Charge	Separation Charge
0 to 3m 25%	20.1 to 30 m 10%
3.1 to 10m 20%	30.1 to 45m 5%

The total percentage shall be the sum of the percentage for all sides, but shall not exceed 75 %.
The fire flow shall not exceed 45,000 L/min nor be less than 2,000 L/min.

Note E:	Fire Walls: - In determining floor areas, a fire wall that meets or exceeds the requirements of the current edition of the National Building Code of Canada (provided this necessitates a fire resistance rating of 2 or more hours) may be deemed to subdivide the building into more than one area or may, as a party wall, separate the building from an adjoining building. Normally any unpierced party wall considered to form a boundary when determining floor areas may warrant up to a 10% exposure charge.
Note F:	High one storey buildings: When a building is stated as 1=2, or more storeys, the number of storeys to be used in the formula depends upon the use being made of the building. For example, consider a 1=3 storey building. If the building is being used for high piled stock, or for rack storage, the building would probably be considered as 3 storeys and, in addition, an occupancy percentage increase may be warranted. However, if the building is being used for steel fabrication and the extra height is provided only to facilitate movement of objects by a crane, the building would probably be considered as a one storey building and an occupancy credit percentage may be warranted.
Note G:	If a building is exposed within 45 metres, normally some surcharge for exposure will be made.
Note H:	Where wood shingle or shake roofs could contribute to spreading fires, add 2,000 L/min to 4,000 L/min in accordance with extent and condition.
Note I:	Any non-combustible building is considered to warrant a 0.8 coefficient.
Note J:	Dwellings: For groupings of detached one family and small two family dwellings not exceeding 2 stories in height, the following short method may be used. (For other residential buildings, the regular method should be used.)

	sugested fire flow	
Exposure distances		Masonry or Brick
Less than 3m	See Note "D" 4,000 L/min	6,000 L/min
3 to 10m	3,000 L/min	4,000 L/min
10.1 to 30m	2,000 L/min	3,000 L/min
Over 30m		2,000 L/min

If the buildings are contiguous, use a minimum of 8,000 L/min. Also consider Note H.

OUTLINE OF PROCEDURE

A.	Determine the type of construction.
B.	Determine the ground floor area.
C.	Determine the height in storeys. Using the fire flow formula, determine the required fire flow to the nearest 1,000 L/min.
E.	Determine the increase or decrease for occupancy and apply to the value obtained in D above. Do not round off the answer.
F.	Determine the decrease, if any, for automatic sprinkler protection. Do not round off the value.
G.	Determine the total increase for exposures, Do not round off the value.
H.	To the answer obtained in E, subtract the value obtained in F and add the value obtained in G. The final figure is customarily rounded off to the nearest 1,000 L/min.

APPENDIX B

STORM SEWER CALCULATIONS

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS

9358 GOREWAY DRIVE
BRAMPTON ON. L6P-0M7
TEL (905) 794-0600
FAX (905) 794-0611

Subdivision: 15544 MCLAUGHLIN
File No.: REGION:21T-XX / CITY: PRE-2029-0197
Consultant: Candevcon Limited
Drainage Area Plan: STM 1

TOWN OF CALEDON
STORM DRAINAGE

FILE NUMBEI W22002
DATE: April 30, 2024
PREPARED BY: SDL
CHECKED BY:

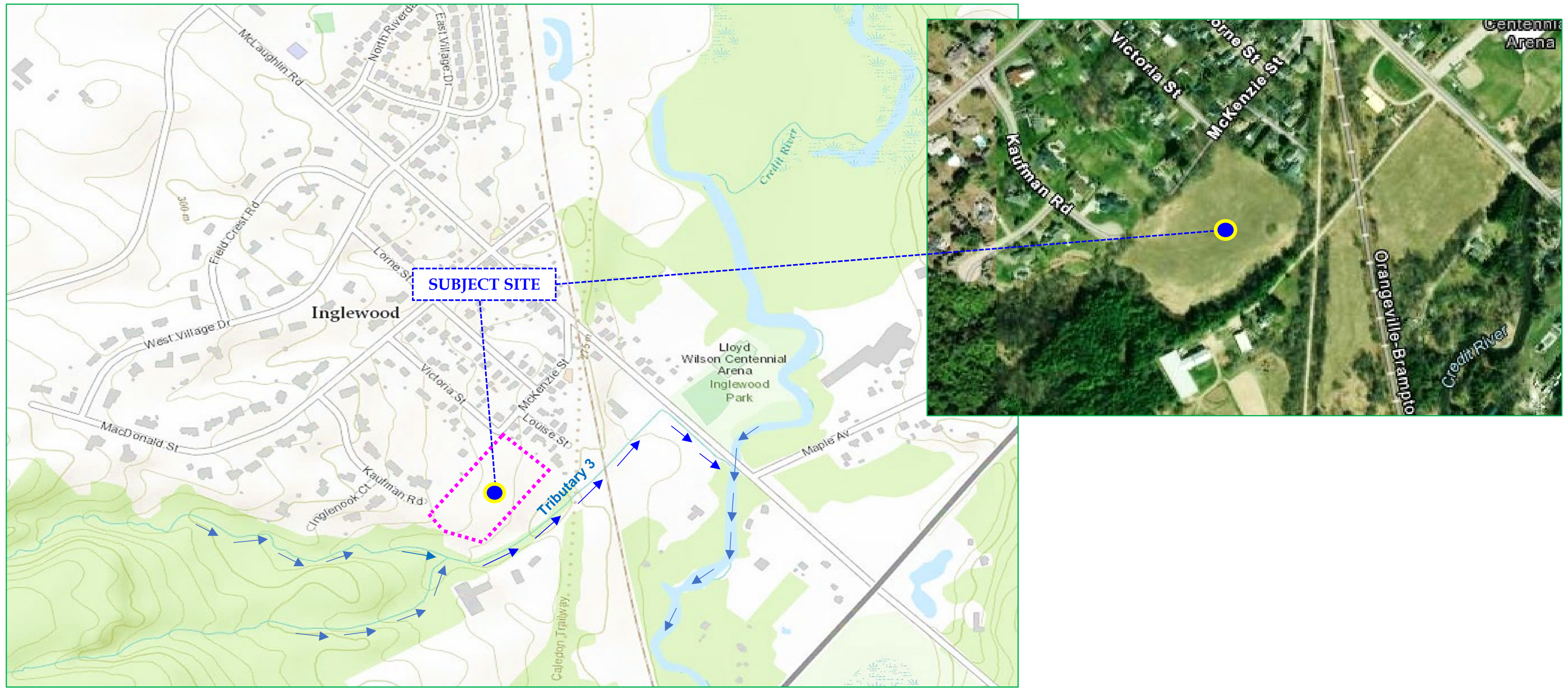
Parks	0.25
Single/Semi	0.50
Multiple/Inst.	0.75
Industrial/Com.	0.90
Roads	0.90

		A	B	C
For 2-yr storm I_2	=	1070	0.8759	7.85
For 5-yr storm I_5	=	1593	0.8789	11
For 10-yr storm I_{10}	=	2221	0.908	12
For 100-yr storm I_{100}	=	4688	0.9624	17

Core System	Area No.	Up-stream	Down-stream	Contributing Area (ha)			Breakdown of Areas				Area x Storm Co-eff				C	Total	Cummulative	Time (min)		I_2	I_5	I_{10}	FLOW Q=2.78AC I/1000	PIPE						
		Node	Node	In Area	Control	Total	0.25	0.50	0.75	0.90	0.25	0.50	0.75	0.90		A x C	AxC	In Area	Total				Q_{design}	Length (m)	Size (mm)	Grade (%)	Capacity (m^3/sec)	Velocity (m/s)	Time (min)	% Full
POND G5																														
	EXT-1	EXT1	CB	0.05	0.00	0.05	0.05				0.01	0.00	0.00	0.00	0.25	0.01	0.01													
	1	CB	MH1	0.25	0.05	0.30		0.25			0.00	0.13	0.00	0.00	0.50	0.13	0.14	10.00	10.0	85.6	109.5	133.9	**0.042**	4.0	250	2.00	**0.084**	1.71	0.04	50%
	EXT-2	EXT2	CB	0.05	0.00	0.05	0.05				0.01	0.00	0.00	0.00	0.25	0.01	0.01													
	2	MH1	CB	0.41	0.35	0.76		0.41			0.00	0.21	0.00	0.00	0.50	0.21	0.36	10.04	10.4	84.1	107.9	132.0	**0.107**	66.0	300	5.30	**0.223**	3.15	0.35	48%
	3	CB	MH2	0.12	0.76	0.88		0.12			0.00	0.06	0.00	0.00	0.50	0.06	0.42	10.39	10.5	83.6	107.4	131.4	**0.124**	24.0	300	5.30	**0.223**	3.15	0.13	56%
	EXT-3	EXT3	CB	0.03	0.00	0.03	0.03				0.01	0.00	0.00	0.00	0.25	0.01	0.01													
	4	MH2	CBMH3	0.39	0.92	1.31		0.39			0.00	0.20	0.00	0.00	0.50	0.20	0.62	10.52	10.7	82.9	106.6	130.5	**0.183**	38.0	375	5.20	**0.400**	3.62	0.17	46%
	5	CBMH3	MH5	0.03	1.31	1.34		0.03			0.00	0.02	0.00	0.00	0.50	0.02	0.63	10.69	10.8	82.6	106.2	130.0	**0.187**	10.0	450	1.00	**0.285**	1.79	0.09	66%
	6	MH5	POND	0.40	1.34	1.74		0.40			0.00	0.20	0.00	0.00	0.50	0.20	0.83	10.78	10.8	82.4	106.0	129.7	**0.245**	6.0	525	1.00	**0.430**	1.99	0.05	57%
			TOTAL	1.74																										

APPENDIX "C"
STORMWATER MANAGEMENT CALCULATIONS

Existing Site Conditions for Pre-Development Model



Total Site Area = **1.64 Ha**

Current Site Condition = farmland, where the surrounding land use includes; a water course flowing south of the site, wooded areas, situated immediate to the south-west, and existing residential properties to the north, north-east and north-west of the subject site

Current Drainage Condition = At present, the entire site drains from the north to the south-east corner of the site towards existing Ditch

Stormwater Management Design Criteria applicable for Subject Site ;

- *SWM Measures for Quality, Quantity and Erosion Control will be provided as per Town of Caledon & CVC recommendations*
- *As per Pre-consultation meeting comments, proposed SWM Design should be in accordance with the CVC's Stormwater Management Guideline (July 2022)*
- *The Storm Servicing Criteria is to address the previously completed Inglewood Village Studies.*

Stormwater Management Recommendations from Inglewood Village Studies ;

Tributary 3 of Credit River is located south of subject site

Water Quality Control :
- *Level 1 (Enhanced- 80% TSS Removal) as per MOEE Guidelines for Storage Requirement*
- *Phosphorous Control is required*

Erosion Control :
- *25mm Rainfall event stored over 24 hours**
- **Releasing over longer duration is recommended to prevent further erosion damage*

Water Quantity Control :
- *Post Development Flows to Pre-Development Levels*
- *Regional Storm - Hurricane Hazel*

Rainfall/ IDF Data Used :
- *Toronto Pearson International Airport*
- *24hours SCS Type II for Pond Design*
- *Chicago Storm for Culvert, Sewers and Channel Design*

Infiltration & Water Balance :
- *Structural IT measures are suited due to till soil present throughout the area*
- *Lot Level & Conveyance Controls recommended*

CVC Water Quantity/Flood Control Criteria (Ref: Stormwater Management Guidelines. 2022)

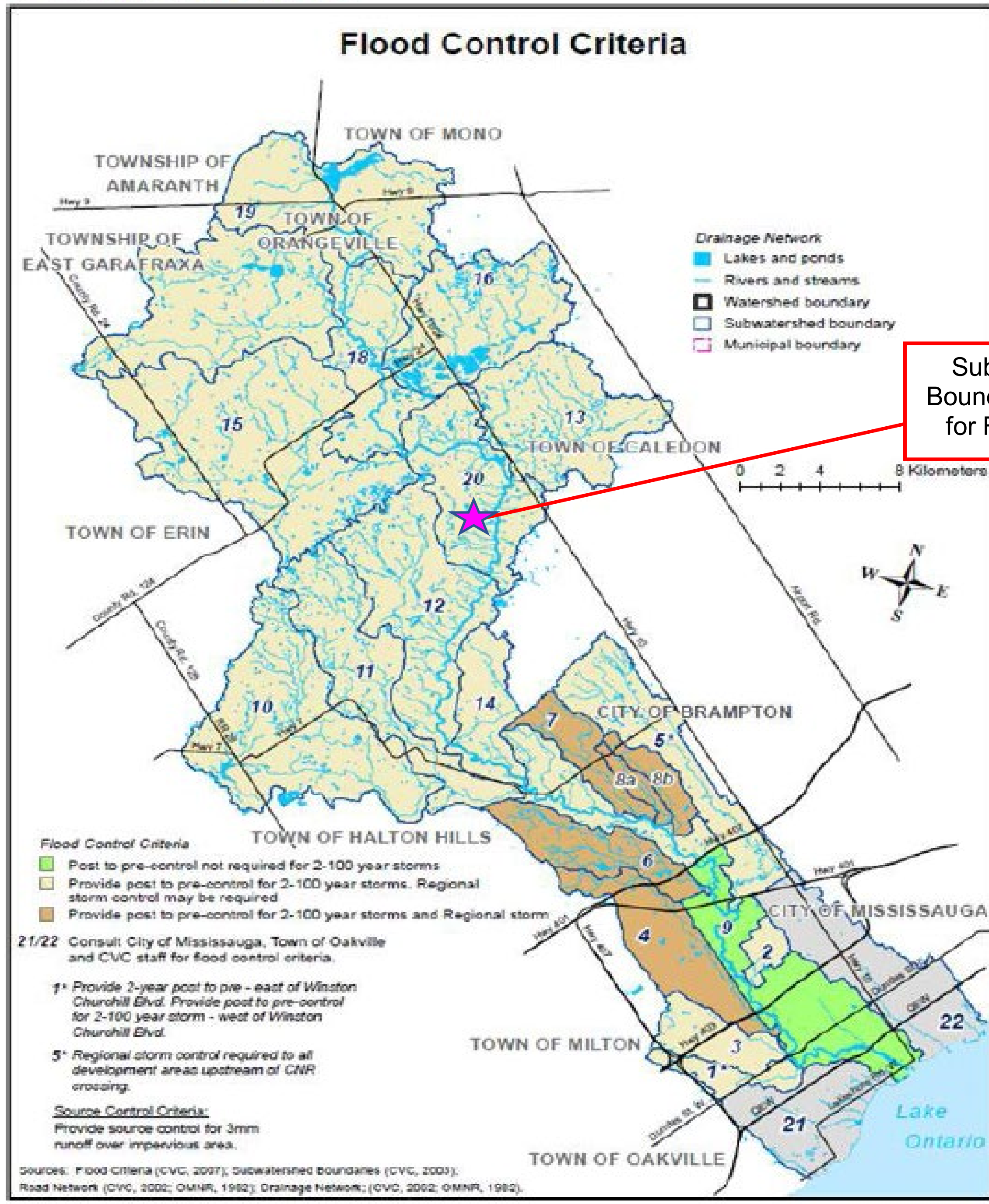


Figure 4-2 CVC Flood Control Criteria

Subject Site Watershed Boundary belongs to **No. 20** for Flood Control Criteria

Table 4-1: CVC Flood Control Criteria for Credit River and its Tributaries

#	Subwatershed Name	Flood Control Criteria	References & Notes
18	Credit River - Melville to Forks of the Credit	Provide post to pre control for all storms (i.e. 2, 5, 10, 25, 50 & 100 year)	Caledon Creek and Credit River Subwatershed Study by CVC, Blackport Hydrogeologic, EWRG, Water System Analysis & Parish Geomorphic
19	Orangeville (Mill Creek, Monora Creek)	Provide post to pre control for all storms (i.e. 2, 5, 10, 25, 50 & 100 year). See Recommendations 1.1 to 1.13 of the SWS	Headwaters Subwatershed Study (CVC, 2020) Update to hydrologic modelling in progress (CVC, 2021)
20	Credit River - Forks of the Credit to Cheltenham	Provide post to pre control for all storms (i.e. 2, 5, 10, 25, 50 & 100 year)	Hydrologic Model: GAWSER Model- Return period peak flows based on 24-hour SCS Type II distribution

Rainfall Data & IDF Curves Information

The IDF equations parameters are based on MTO IDF Curve Lookup Tool

Ontario.ca | Français

Ontario IDF CURVE LOOKUP

Terms and Conditions | Coordinate Selection | About

Active coordinate

43° 40' 45" N, 79° 37' 44" W (43.679167,-79.629167) Modify selection

Retrieved: Thu, 11 May 2023 15:04:45 GMT

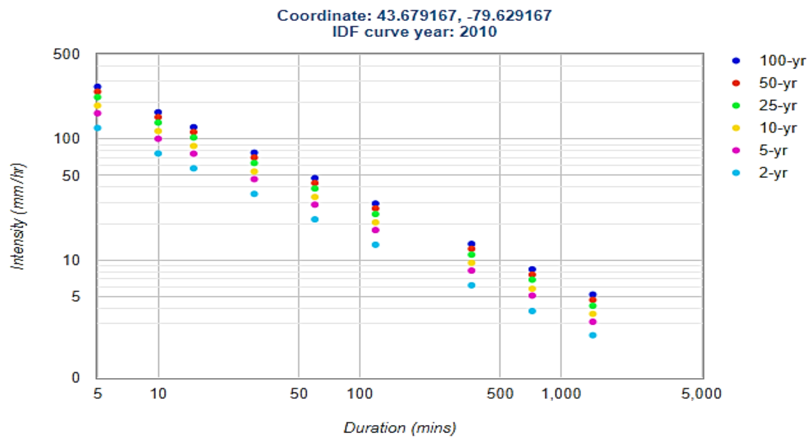
Map options: Modify selection | Show/hide gauging stations | Re-center selection

Location summary

These are the locations in the selection.

IDF Curve: 43° 40' 45" N, 79° 37' 44" W (43.679167,-79.629167)

An IDF curve was found.



Switch variable: Intensity or Depth

Coefficient summary

Data year: 2010
IDF curve year: 2010

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

Return period	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
A	21.7	28.7	33.2	39.0	43.3	47.5
B	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699

Statistics

Rainfall intensity (mm hr^{-1})

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	123.3	75.9	57.2	35.2	21.7	13.4	6.2	3.8	2.4
5-yr	163.0	100.4	75.6	46.6	28.7	17.7	8.2	5.1	3.1
10-yr	188.6	116.2	87.5	53.9	33.2	20.5	9.5	5.8	3.6
25-yr	221.5	136.5	102.8	63.3	39.0	24.0	11.1	6.9	4.2
50-yr	245.9	151.5	114.1	70.3	43.3	26.7	12.4	7.6	4.7
100-yr	269.8	166.2	125.2	77.1	47.5	29.3	13.6	8.4	5.2

Rainfall depth (mm)

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	10.3	12.7	14.3	17.6	21.7	26.7	37.2	45.8	56.5
5-yr	13.6	16.7	18.9	23.3	28.7	35.4	49.2	60.6	74.7
10-yr	15.7	19.4	21.9	26.9	33.2	40.9	56.9	70.1	86.4
25-yr	18.5	22.7	25.7	31.7	39.0	48.0	66.9	82.4	101.5
50-yr	20.5	25.3	28.5	35.1	43.3	53.3	74.3	91.5	112.7
100-yr	22.5	27.7	31.3	38.6	47.5	58.5	81.5	100.4	123.6

Pre-Development VO Model Scenario

For VO Model, the storm files for 2 to 100-year are generated using 24-Hour SCS type II design storm distribution

Pre-Development Model Parameters ;

Total Site Area = 1.76 Ha (Site Area + External Drainage to Site from North)

TABLE I (PRE-DEVELOPMENT VO MODEL PARAMETERS)

	NHYD	DT (min)	Area (Ha)	CN	IA (mm)	N	Tp (hr)
Farm Land/ Green	1	10	1.76	80	5	3	0.2

Pre-Development (Hydrograph Results) ;

TABLE II (PRE-DEVEVELOPMENT VO OUTPUT RESULTS)

	NHYD	Flow Type	DT (Hr)	Area (Ha)	PKFW (m^3/s)	TP (hr)	RV (mm)
2yr 24hr 10min SCS Type II (MTO)	1	Outflow	0.167	1.76	***0.108***	12.167	23.211
5yr 24hr 10min SCS Type II (MTO)	1	Outflow	0.167	1.76	***0.166***	12.167	35.299
10yr 24hr 10min SCS Type II (MTO)	1	Outflow	0.167	1.76	***0.211***	12.167	44.539
25yr 24hr 10min SCS Type II (MTO)	1	Outflow	0.167	1.76	***0.266***	12.167	56.115
50yr 24hr 10min SCS Type II (MTO)	1	Outflow	0.167	1.76	***0.314***	12.167	66.076
100yr 24hr 10min SCS Type II (MTO)	1	Outflow	0.167	1.76	***0.362***	12.167	76.263

Post-Development VO Model Scenario

TABLE III (POST-DEVELOPMENT DRAINAGE AREA VS IMPERVIOUSNESS)				
Catchment Type	Typical Imperviousness	Typical Run-Off "C"	Area (Ha)	
Area - 1	60%	0.50	0.25	
Area - 2	60%	0.50	0.41	
Area - 3	60%	0.50	0.12	
Area - 4	80%	0.75	0.39	
Area - 5	60%	0.50	0.03	
Area - 6	60%	0.50	0.40	
External Drainage from North (EXT 1 to 3)	10%	0.25	0.13	
		Total Area to Dry Pond =	**1.73**	Ha
		Composite Imperviousness =	**61%**	
		Composite Runoff C =	**0.54**	
Uncontrolled Drainage from Site (101)	10%	0.25	0.03	
		Total Drainage Area =	**1.76**	Ha

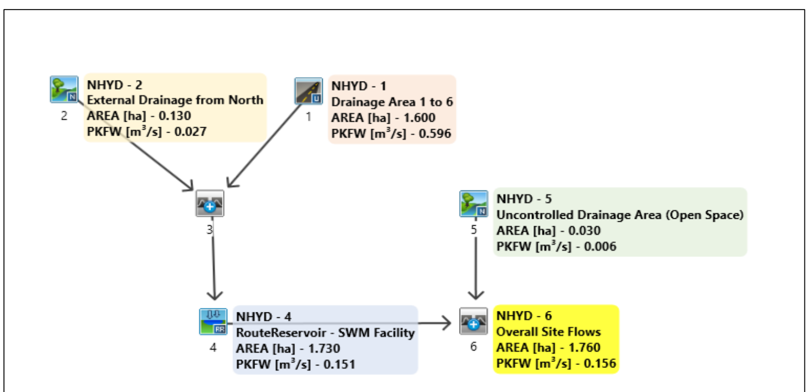
Total Area draining to Proposed Dry Pond = 1.73 Ha *(Includes External drainage from North)

** (Excludes Uncontrolled drainage area)

Flows Generated by Uncontrolled Drainage Area ;

TABLE IV (POST-DEVELOPMENT - VO FLOWS GENERATED BY UNCONTROLLED DRAINAGE AREA)							
	NHYD	Flow Type	DT (Hr)	Area (Ha)	PKFW (m^3/s)	TP (hr)	RV (mm)
2yr 24hr 10min SCS Type II (MTO)	5	Outflow	0.167	0.030	*0.002*	12.167	23.186
5yr 24hr 10min SCS Type II (MTO)	5	Outflow	0.167	0.030	*0.003*	12.167	35.288
10yr 24hr 10min SCS Type II (MTO)	5	Outflow	0.167	0.030	*0.004*	12.167	44.526
25yr 24hr 10min SCS Type II (MTO)	5	Outflow	0.167	0.030	*0.005*	12.167	56.099
50yr 24hr 10min SCS Type II (MTO)	5	Outflow	0.167	0.030	*0.005*	12.167	66.057
100yr 24hr 10min SCS Type II (MTO)	5	Outflow	0.167	0.030	*0.006*	12.167	76.242

TABLE V DRY POND (SWM) RELEASE RATE TARGETS			
Storm Event	Pre-Development Release Rates (m^3/s)	Uncontrolled Release Rates (m^3/s)	Dry Pond Release Rate Targets (m^3/s)
2-Year	0.108	0.002	**0.106**
5-Year	0.166	0.003	**0.163**
10-Year	0.211	0.004	**0.207**
25-Year	0.266	0.005	**0.261**
50-Year	0.314	0.005	**0.309**
100-Year	0.362	0.006	**0.356**



VO MODEL Schematic - Post Development

TABLE VI SUMMARY OF FLOWS FROM SITE							
Storm Event	Flow Targets for Dry Pond	Flows from Dry pond Orifice	Uncontrolled Flows (NHYD 5)	Total Flows from Site (NHYD 6)	Storage Required in Facility	Post flow comparison with Pre	
	m^3/s	m^3/s	m^3/s	m^3/s	(m^3)		
2-Year	*0.106*	0.100	0.002	0.101	197	4.72	% (less than Pre)
5-Year	*0.163*	0.117	0.003	0.119	265	26.99	% (less than Pre)
10-Year	*0.207*	0.126	0.004	0.129	330	37.68	% (less than Pre)
25-Year	*0.261*	0.139	0.005	0.142	436	45.59	% (less than Pre)
50-Year	*0.309*	0.145	0.005	0.149	503	51.78	% (less than Pre)
100-Year	*0.356*	0.151	0.006	0.156	572	56.18	% (less than Pre)

Total Storage available in Dry Detention Pond = 621 m^3 (at Elv = 276.75m)

Orifice Control No. 1

Orifice Plate Diameter =	0.245	m
	245	**mm**
Opening Area =	0.0471	m^2
Orifice Coeff. (C)=	0.63	
Invert =	275.35	m

Submerged Orifice Equation = **$Qo = 0.63 \times A \times [2 \times g \times H]^{1/2}$**

Where,

Q = Flow rate (m^3/s)
C = Constant
A = Area of opening (m^2)
H = Net head above the orifice (m)
g = Acceleration due to gravity (m/s)

TABLE VII VO RATING CURVE DESIGN (STAGE VS STORAGE VS FLOWS)

Stage (m): 0.10

Elevation	Depth above orifice (m)	Orifice No.1 Flow (m^3/s)	Storage in Dry Pond (m^3)	Total Flow (m^3/s)	
275.25	0	0	0	0.000	
275.35	0	0	15	0.000	
275.45	0.10	0.042	33	0.042	
275.55	0.20	0.059	56	0.059	
275.60	0.25	0.066	57	0.066	
275.65	0.30	0.072	82	0.072	
275.75	0.40	0.083	112	0.083	
275.85	0.50	0.093	145	0.093	
275.95	0.60	0.102	183	0.102	
276.00	**0.65**	**0.106**	**203**	**0.106**	**2-Year**
276.05	0.70	0.110	224	0.110	
276.10	0.75	0.114	246	0.114	
276.15	**0.80**	**0.118**	**269**	**0.118**	**5-Year**
276.20	**0.85**	**0.121**	**293**	**0.121**	**25mm**
276.25	0.90	0.125	317	0.125	
276.30	0.95	0.128	343	0.128	
276.31	**0.96**	**0.129**	**349**	**0.129**	**10-Yr**
276.35	1.00	0.132	370	0.132	
276.45	1.10	0.138	426	0.138	
276.47	**1.12**	**0.139**	**438**	**0.139**	**25-Yr**
276.55	1.20	0.144	487	0.144	
276.65	**1.30**	**0.150**	**552**	**0.150**	**50-Yr**
276.69	**1.34**	**0.152**	**579**	**0.152**	**100-Yr**
276.75	1.40	0.156	621	0.156	

Water Quality and 25mm Erosion Volume Design ;

Contributing Drainage Area (ha) =	**1.73**	**Ha**	
As per MOE Table 3.2 (Basic 60% TSS Removal) =	170.00	(m^3/ha)	
Storage Required =	294.10	m^3	
Storage Provided =	621.00	m^3	
25mm 4Hr Chicago Post Development Runoff Volume in Depth =	16.705	mm	(Refer to 25mm VO Results)
(R. V x Drainage Area)			
25mm 4Hr Chicago Post Development Storage Required in Pond =	**289**	**m^3**	
Storage Provided =	**293**	**m^3 @ Elv = 276.20m**	

TABLE VIII Summary of Visual Otthymo Results for 2 to 100-Year , 24hr SCS Type II Storm (Toronto Pearson IDF) - Dry Pond

Storm Event	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
Target Rate (m^3/s)	*0.106*	*0.163*	*0.207*	*0.261*	*0.309*	*0.356*
Peak Flows Generated (m^3/s)	0.223	0.314	0.373	0.487	0.555	0.623
VO Release Rate (m^3/s)	0.100	0.117	0.126	0.138	0.145	0.151

VISUAL OTTHYMO
MODELLING RESULTS

PRE-DEVELOPMENT VO LAYOUT

```
==================================================================================================================

       V    V   I    SSSSS  U   U    A    L             (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
```



```
                   ***** D E T A I L E D   O U T P U T *****
------------------------------------------------------------------------------------------------------------------
  ************************************************
  ** SIMULATION : 2yr 24hr 10min SCS Type II (M **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              873ef3e2-bf5d-4277-89d5-
b3dc59f526f6\9ca73161
| Ptotal= 57.60 mm |    Comments: 2yr 24hr 10min SCS Type II (MTO)-Toronto
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
                 0.00    0.00 |  6.17    1.04 | 12.33    8.29 | 18.50
1.04
                 0.17    0.63 |  6.33    1.04 | 12.50    8.29 | 18.67
1.04
                 0.33    0.63 |  6.50    1.04 | 12.67    4.26 | 18.83
1.04
                 0.50    0.63 |  6.67    1.04 | 12.83    4.26 | 19.00
1.04
                 0.67    0.63 |  6.83    1.04 | 13.00    4.26 | 19.17
1.04
                 0.83    0.63 |  7.00    1.04 | 13.17    3.11 | 19.33
1.04
                 1.00    0.63 |  7.17    1.27 | 13.33    3.11 | 19.50
1.04
                 1.17    0.63 |  7.33    1.27 | 13.50    3.11 | 19.67
1.04
                 1.33    0.63 |  7.50    1.27 | 13.67    2.42 | 19.83
1.04
```

1.50	0.63	7.67	1.27	13.83	2.42	20.00	1.04
1.67	0.63	7.83	1.27	14.00	2.42	20.17	0.69
1.83	0.63	8.00	1.27	14.17	1.73	20.33	0.69
2.00	0.63	8.17	1.50	14.33	1.73	20.50	0.69
2.17	0.75	8.33	1.50	14.50	1.73	20.67	0.69
2.33	0.75	8.50	1.50	14.67	1.73	20.83	0.69
2.50	0.75	8.67	1.61	14.83	1.73	21.00	0.69
2.67	0.75	8.83	1.61	15.00	1.73	21.17	0.69
2.83	0.75	9.00	1.61	15.17	1.73	21.33	0.69
3.00	0.75	9.17	1.84	15.33	1.73	21.50	0.69
3.17	0.75	9.33	1.84	15.50	1.73	21.67	0.69
3.33	0.75	9.50	1.84	15.67	1.73	21.83	0.69
3.50	0.75	9.67	2.07	15.83	1.73	22.00	0.69
3.67	0.75	9.83	2.07	16.00	1.73	22.17	0.69
3.83	0.75	10.00	2.07	16.17	1.04	22.33	0.69
4.00	0.75	10.17	2.65	16.33	1.04	22.50	0.69
4.17	0.92	10.33	2.65	16.50	1.04	22.67	0.69
4.33	0.92	10.50	2.65	16.67	1.04	22.83	0.69
4.50	0.92	10.67	3.57	16.83	1.04	23.00	0.69
4.67	0.92	10.83	3.57	17.00	1.04	23.17	0.69
4.83	0.92	11.00	3.57	17.17	1.04	23.33	0.69
5.00	0.92	11.17	5.53	17.33	1.04	23.50	0.69
5.17	0.92	11.33	5.53	17.50	1.04	23.67	0.69
5.33	0.92	11.50	5.53	17.67	1.04	23.83	0.69
5.50	0.92	11.67	17.05	17.83	1.04	24.00	0.69
5.67	0.92	11.83	43.78	18.00	1.04		
5.83	0.92	12.00	70.50	18.17	1.04		
6.00	0.92	12.17	8.29	18.33	1.04		

```
-----------------------------------------------------------------------------
-----

--------------------
| CALIB             |
| NASHYD   (  0001)|   Area     (ha)=   1.76   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia       (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.336

     PEAK FLOW       (cms)=   0.108 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  23.211
     TOTAL RAINFALL   (mm)=  57.600
     RUNOFF COEFFICIENT   =   0.403

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

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 FINISH
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==================================================================================================================

      V    V   I    SSSSS  U   U    A    L              (v 6.2.2015)
      V    V   I    SS     U   U   A A   L
       V  V    I     SS    U   U  AAAAA  L
       V  V    I      SS   U   U  A   A  L
        VV     I    SSSSS  UUUUU  A   A  LLLLL

       OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
      O   O    T      T    H   H   Y Y   MM MM  O   O
      O   O    T      T    H   H    Y    M   M  O   O
       OOO     T      T    H   H    Y    M   M   OOO
```

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                   *****  D E T A I L E D   O U T P U T *****
------------------------------------------------------------------------------------------------------------------
  ************************************************
  ** SIMULATION : 5yr 24hr 10min SCS Type II (M **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              873ef3e2-bf5d-4277-89d5-
b3dc59f526f6\955bcef0
| Ptotal= 74.40 mm |    Comments: 5yr 24hr 10min SCS Type II (MTO)-Toronto
--------------------
```

TIME	RAIN	TIME	RAIN	TIME	RAIN	TIME	RAIN
hrs	mm/hr	hrs	mm/hr	hrs	mm/hr	hrs	mm/hr
0.00	0.00	6.17	1.34	12.33	10.71	18.50	1.34
0.17	0.82	6.33	1.34	12.50	10.71	18.67	1.34
0.33	0.82	6.50	1.34	12.67	5.51	18.83	1.34
0.50	0.82	6.67	1.34	12.83	5.51	19.00	1.34
0.67	0.82	6.83	1.34	13.00	5.51	19.17	1.34
0.83	0.82	7.00	1.34	13.17	4.02	19.33	1.34
1.00	0.82	7.17	1.64	13.33	4.02	19.50	1.34
1.17	0.82	7.33	1.64	13.50	4.02	19.67	1.34

1.33	0.82	7.50	1.64	13.67	3.12	19.83	1.34
1.50	0.82	7.67	1.64	13.83	3.12	20.00	1.34
1.67	0.82	7.83	1.64	14.00	3.12	20.17	0.89
1.83	0.82	8.00	1.64	14.17	2.23	20.33	0.89
2.00	0.82	8.17	1.93	14.33	2.23	20.50	0.89
2.17	0.97	8.33	1.93	14.50	2.23	20.67	0.89
2.33	0.97	8.50	1.93	14.67	2.23	20.83	0.89
2.50	0.97	8.67	2.08	14.83	2.23	21.00	0.89
2.67	0.97	8.83	2.08	15.00	2.23	21.17	0.89
2.83	0.97	9.00	2.08	15.17	2.23	21.33	0.89
3.00	0.97	9.17	2.38	15.33	2.23	21.50	0.89
3.17	0.97	9.33	2.38	15.50	2.23	21.67	0.89
3.33	0.97	9.50	2.38	15.67	2.23	21.83	0.89
3.50	0.97	9.67	2.68	15.83	2.23	22.00	0.89
3.67	0.97	9.83	2.68	16.00	2.23	22.17	0.89
3.83	0.97	10.00	2.68	16.17	1.34	22.33	0.89
4.00	0.97	10.17	3.42	16.33	1.34	22.50	0.89
4.17	1.19	10.33	3.42	16.50	1.34	22.67	0.89
4.33	1.19	10.50	3.42	16.67	1.34	22.83	0.89
4.50	1.19	10.67	4.61	16.83	1.34	23.00	0.89
4.67	1.19	10.83	4.61	17.00	1.34	23.17	0.89
4.83	1.19	11.00	4.61	17.17	1.34	23.33	0.89
5.00	1.19	11.17	7.14	17.33	1.34	23.50	0.89
5.17	1.19	11.33	7.14	17.50	1.34	23.67	0.89
5.33	1.19	11.50	7.14	17.67	1.34	23.83	0.89
5.50	1.19	11.67	22.02	17.83	1.34	24.00	0.89
5.67	1.19	11.83	56.54	18.00	1.34		
5.83	1.19	12.00	91.07	18.17	1.34		

```
                  6.00    1.19 | 12.17   10.71 | 18.33    1.34 |

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--------------------
| CALIB            |
| NASHYD   (  0001)|   Area    (ha)=   1.76   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.336

     PEAK FLOW       (cms)=   0.166 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  35.299
     TOTAL RAINFALL   (mm)=  74.400
     RUNOFF COEFFICIENT   =   0.474

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

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       V    V   I    SSSSS  U   U    A    L               (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
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                   *****  D E T A I L E D   O U T P U T *****
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-------------------------------------
  ************************************************
  ** SIMULATION : 10yr 24hr 10min SCS Type II ( **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              873ef3e2-bf5d-4277-89d5-
b3dc59f526f6\ca7e1ce8
| Ptotal= 86.40 mm |    Comments: 10yr 24hr 10min SCS Type II (MTO)-Toront
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
                 0.00    0.00 |  6.17    1.56 | 12.33   12.44 | 18.50
1.56
                 0.17    0.95 |  6.33    1.56 | 12.50   12.44 | 18.67
1.56
                 0.33    0.95 |  6.50    1.56 | 12.67    6.39 | 18.83
1.56
                 0.50    0.95 |  6.67    1.56 | 12.83    6.39 | 19.00
1.56
                 0.67    0.95 |  6.83    1.56 | 13.00    6.39 | 19.17
1.56
                 0.83    0.95 |  7.00    1.56 | 13.17    4.67 | 19.33
1.56
                 1.00    0.95 |  7.17    1.90 | 13.33    4.67 | 19.50
1.56
                 1.17    0.95 |  7.33    1.90 | 13.50    4.67 | 19.67
1.56
```

```
                1.33    0.95 |  7.50    1.90 | 13.67    3.63 | 19.83
1.56
                1.50    0.95 |  7.67    1.90 | 13.83    3.63 | 20.00
1.56
                1.67    0.95 |  7.83    1.90 | 14.00    3.63 | 20.17
1.04
                1.83    0.95 |  8.00    1.90 | 14.17    2.59 | 20.33
1.04
                2.00    0.95 |  8.17    2.25 | 14.33    2.59 | 20.50
1.04
                2.17    1.12 |  8.33    2.25 | 14.50    2.59 | 20.67
1.04
                2.33    1.12 |  8.50    2.25 | 14.67    2.59 | 20.83
1.04
                2.50    1.12 |  8.67    2.42 | 14.83    2.59 | 21.00
1.04
                2.67    1.12 |  8.83    2.42 | 15.00    2.59 | 21.17
1.04
                2.83    1.12 |  9.00    2.42 | 15.17    2.59 | 21.33
1.04
                3.00    1.12 |  9.17    2.76 | 15.33    2.59 | 21.50
1.04
                3.17    1.12 |  9.33    2.76 | 15.50    2.59 | 21.67
1.04
                3.33    1.12 |  9.50    2.76 | 15.67    2.59 | 21.83
1.04
                3.50    1.12 |  9.67    3.11 | 15.83    2.59 | 22.00
1.04
                3.67    1.12 |  9.83    3.11 | 16.00    2.59 | 22.17
1.04
                3.83    1.12 | 10.00    3.11 | 16.17    1.56 | 22.33
1.04
                4.00    1.12 | 10.17    3.97 | 16.33    1.56 | 22.50
1.04
                4.17    1.38 | 10.33    3.97 | 16.50    1.56 | 22.67
1.04
                4.33    1.38 | 10.50    3.97 | 16.67    1.56 | 22.83
1.04
                4.50    1.38 | 10.67    5.36 | 16.83    1.56 | 23.00
1.04
                4.67    1.38 | 10.83    5.36 | 17.00    1.56 | 23.17
1.04
                4.83    1.38 | 11.00    5.36 | 17.17    1.56 | 23.33
1.04
                5.00    1.38 | 11.17    8.29 | 17.33    1.56 | 23.50
1.04
                5.17    1.38 | 11.33    8.29 | 17.50    1.56 | 23.67
1.04
                5.33    1.38 | 11.50    8.29 | 17.67    1.56 | 23.83
1.04
                5.50    1.38 | 11.67   25.57 | 17.83    1.56 | 24.00
1.04
                5.67    1.38 | 11.83   65.66 | 18.00    1.56 |
                5.83    1.38 | 12.00  105.75 | 18.17    1.56 |
```

```
                  6.00    1.38 | 12.17   12.44 | 18.33    1.56 |

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--------------------
| CALIB             |
| NASHYD   (  0001)|   Area    (ha)=   1.76   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.336

     PEAK FLOW       (cms)=   0.211 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  44.539
     TOTAL RAINFALL   (mm)=  86.400
     RUNOFF COEFFICIENT   =   0.516

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

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       V    V   I    SSSSS  U   U    A    L             (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
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                  *****  D E T A I L E D   O U T P U T *****
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---------------------------------
  ************************************************
  ** SIMULATION : 25yr 24hr 10min SCS Type II ( **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              873ef3e2-bf5d-4277-89d5-
b3dc59f526f6\0d9dafb1
| Ptotal=100.80 mm |    Comments: 25yr 24hr 10min SCS Type II (MTO)-Toront
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
                 0.00    0.00 |  6.17    1.81 | 12.33   14.52 | 18.50
1.81
                 0.17    1.11 |  6.33    1.81 | 12.50   14.52 | 18.67
1.81
                 0.33    1.11 |  6.50    1.81 | 12.67    7.46 | 18.83
1.81
                 0.50    1.11 |  6.67    1.81 | 12.83    7.46 | 19.00
1.81
                 0.67    1.11 |  6.83    1.81 | 13.00    7.46 | 19.17
1.81
                 0.83    1.11 |  7.00    1.81 | 13.17    5.44 | 19.33
1.81
                 1.00    1.11 |  7.17    2.22 | 13.33    5.44 | 19.50
1.81
                 1.17    1.11 |  7.33    2.22 | 13.50    5.44 | 19.67
1.81
```

1.81	1.33	1.11	7.50	2.22	13.67	4.23	19.83
1.81	1.50	1.11	7.67	2.22	13.83	4.23	20.00
1.21	1.67	1.11	7.83	2.22	14.00	4.23	20.17
1.21	1.83	1.11	8.00	2.22	14.17	3.02	20.33
1.21	2.00	1.11	8.17	2.62	14.33	3.02	20.50
1.21	2.17	1.31	8.33	2.62	14.50	3.02	20.67
1.21	2.33	1.31	8.50	2.62	14.67	3.02	20.83
1.21	2.50	1.31	8.67	2.82	14.83	3.02	21.00
1.21	2.67	1.31	8.83	2.82	15.00	3.02	21.17
1.21	2.83	1.31	9.00	2.82	15.17	3.02	21.33
1.21	3.00	1.31	9.17	3.23	15.33	3.02	21.50
1.21	3.17	1.31	9.33	3.23	15.50	3.02	21.67
1.21	3.33	1.31	9.50	3.23	15.67	3.02	21.83
1.21	3.50	1.31	9.67	3.63	15.83	3.02	22.00
1.21	3.67	1.31	9.83	3.63	16.00	3.02	22.17
1.21	3.83	1.31	10.00	3.63	16.17	1.81	22.33
1.21	4.00	1.31	10.17	4.64	16.33	1.81	22.50
1.21	4.17	1.61	10.33	4.64	16.50	1.81	22.67
1.21	4.33	1.61	10.50	4.64	16.67	1.81	22.83
1.21	4.50	1.61	10.67	6.25	16.83	1.81	23.00
1.21	4.67	1.61	10.83	6.25	17.00	1.81	23.17
1.21	4.83	1.61	11.00	6.25	17.17	1.81	23.33
1.21	5.00	1.61	11.17	9.68	17.33	1.81	23.50
1.21	5.17	1.61	11.33	9.68	17.50	1.81	23.67
1.21	5.33	1.61	11.50	9.68	17.67	1.81	23.83
1.21	5.50	1.61	11.67	29.84	17.83	1.81	24.00
	5.67	1.61	11.83	76.61	18.00	1.81	
	5.83	1.61	12.00	123.38	18.17	1.81	

```
                  6.00    1.61 | 12.17   14.52 | 18.33    1.81 |

---------------------------------------------------------------------------
-----

--------------------
| CALIB            |
| NASHYD   (  0001)|   Area    (ha)=   1.76   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.336

     PEAK FLOW       (cms)=   0.266 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  56.115
     TOTAL RAINFALL   (mm)= 100.800
     RUNOFF COEFFICIENT   =   0.557

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

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==================================

      V    V   I    SSSSS  U   U    A    L             (v 6.2.2015)
      V    V   I    SS     U   U   A A   L
       V  V    I     SS    U   U  AAAAA  L
       V  V    I      SS   U   U  A   A  L
        VV     I    SSSSS  UUUUU  A   A  LLLLL

       OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
      O   O    T      T    H   H   Y Y   MM MM  O   O
      O   O    T      T    H   H    Y    M   M  O   O
       OOO     T      T    H   H    Y    M   M   OOO
```

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                  *****  D E T A I L E D   O U T P U T *****
-------------------------------------------------------------------------------------
---------------------------------
  ************************************************
  ** SIMULATION : 50yr 24hr 10min SCS Type II ( **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              873ef3e2-bf5d-4277-89d5-
b3dc59f526f6\c5ab3b65
| Ptotal=112.80 mm |    Comments: 50yr 24hr 10min SCS Type II (MTO)-Toront
--------------------
```

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
0.00	0.00	6.17	2.03	12.33	16.24	18.50	2.03
0.17	1.24	6.33	2.03	12.50	16.24	18.67	2.03
0.33	1.24	6.50	2.03	12.67	8.35	18.83	2.03
0.50	1.24	6.67	2.03	12.83	8.35	19.00	2.03
0.67	1.24	6.83	2.03	13.00	8.35	19.17	2.03
0.83	1.24	7.00	2.03	13.17	6.09	19.33	2.03
1.00	1.24	7.17	2.48	13.33	6.09	19.50	2.03
1.17	1.24	7.33	2.48	13.50	6.09	19.67	2.03

1.33	1.24	7.50	2.48	13.67	4.74	19.83	2.03
1.50	1.24	7.67	2.48	13.83	4.74	20.00	2.03
1.67	1.24	7.83	2.48	14.00	4.74	20.17	1.35
1.83	1.24	8.00	2.48	14.17	3.38	20.33	1.35
2.00	1.24	8.17	2.93	14.33	3.38	20.50	1.35
2.17	1.47	8.33	2.93	14.50	3.38	20.67	1.35
2.33	1.47	8.50	2.93	14.67	3.38	20.83	1.35
2.50	1.47	8.67	3.16	14.83	3.38	21.00	1.35
2.67	1.47	8.83	3.16	15.00	3.38	21.17	1.35
2.83	1.47	9.00	3.16	15.17	3.38	21.33	1.35
3.00	1.47	9.17	3.61	15.33	3.38	21.50	1.35
3.17	1.47	9.33	3.61	15.50	3.38	21.67	1.35
3.33	1.47	9.50	3.61	15.67	3.38	21.83	1.35
3.50	1.47	9.67	4.06	15.83	3.38	22.00	1.35
3.67	1.47	9.83	4.06	16.00	3.38	22.17	1.35
3.83	1.47	10.00	4.06	16.17	2.03	22.33	1.35
4.00	1.47	10.17	5.19	16.33	2.03	22.50	1.35
4.17	1.80	10.33	5.19	16.50	2.03	22.67	1.35
4.33	1.80	10.50	5.19	16.67	2.03	22.83	1.35
4.50	1.80	10.67	6.99	16.83	2.03	23.00	1.35
4.67	1.80	10.83	6.99	17.00	2.03	23.17	1.35
4.83	1.80	11.00	6.99	17.17	2.03	23.33	1.35
5.00	1.80	11.17	10.83	17.33	2.03	23.50	1.35
5.17	1.80	11.33	10.83	17.50	2.03	23.67	1.35
5.33	1.80	11.50	10.83	17.67	2.03	23.83	1.35
5.50	1.80	11.67	33.39	17.83	2.03	24.00	1.35
5.67	1.80	11.83	85.73	18.00	2.03		
5.83	1.80	12.00	138.07	18.17	2.03		

```
                  6.00    1.80 | 12.17   16.24 | 18.33    2.03 |

----------------------------------------------------------------------------
-----

--------------------
| CALIB            |
| NASHYD   (  0001)|   Area    (ha)=   1.76   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.336

     PEAK FLOW       (cms)=   0.314 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  66.076
     TOTAL RAINFALL   (mm)= 112.800
     RUNOFF COEFFICIENT   =   0.586

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

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       V    V   I    SSSSS  U   U    A    L            (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
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                   *****  D E T A I L E D   O U T P U T *****
--------------------------------------------------------------------------------
----------------------------------
  ************************************************
  ** SIMULATION : 100yr 24hr 10min SCS Type II  **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              873ef3e2-bf5d-4277-89d5-
b3dc59f526f6\40ec4445
| Ptotal=124.80 mm |    Comments: 100yr 24hr 10min SCS Type II (MTO)-Toron
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
                 0.00    0.00 |  6.17    2.25 | 12.33   17.97 | 18.50
2.25
                 0.17    1.37 |  6.33    2.25 | 12.50   17.97 | 18.67
2.25
                 0.33    1.37 |  6.50    2.25 | 12.67    9.24 | 18.83
2.25
                 0.50    1.37 |  6.67    2.25 | 12.83    9.24 | 19.00
2.25
                 0.67    1.37 |  6.83    2.25 | 13.00    9.24 | 19.17
2.25
                 0.83    1.37 |  7.00    2.25 | 13.17    6.74 | 19.33
2.25
                 1.00    1.37 |  7.17    2.75 | 13.33    6.74 | 19.50
2.25
                 1.17    1.37 |  7.33    2.75 | 13.50    6.74 | 19.67
2.25
```

```
                1.33    1.37 |  7.50    2.75 | 13.67    5.24 | 19.83
2.25
                1.50    1.37 |  7.67    2.75 | 13.83    5.24 | 20.00
2.25
                1.67    1.37 |  7.83    2.75 | 14.00    5.24 | 20.17
1.50
                1.83    1.37 |  8.00    2.75 | 14.17    3.74 | 20.33
1.50
                2.00    1.37 |  8.17    3.24 | 14.33    3.74 | 20.50
1.50
                2.17    1.62 |  8.33    3.24 | 14.50    3.74 | 20.67
1.50
                2.33    1.62 |  8.50    3.24 | 14.67    3.74 | 20.83
1.50
                2.50    1.62 |  8.67    3.49 | 14.83    3.74 | 21.00
1.50
                2.67    1.62 |  8.83    3.49 | 15.00    3.74 | 21.17
1.50
                2.83    1.62 |  9.00    3.49 | 15.17    3.74 | 21.33
1.50
                3.00    1.62 |  9.17    3.99 | 15.33    3.74 | 21.50
1.50
                3.17    1.62 |  9.33    3.99 | 15.50    3.74 | 21.67
1.50
                3.33    1.62 |  9.50    3.99 | 15.67    3.74 | 21.83
1.50
                3.50    1.62 |  9.67    4.49 | 15.83    3.74 | 22.00
1.50
                3.67    1.62 |  9.83    4.49 | 16.00    3.74 | 22.17
1.50
                3.83    1.62 | 10.00    4.49 | 16.17    2.25 | 22.33
1.50
                4.00    1.62 | 10.17    5.74 | 16.33    2.25 | 22.50
1.50
                4.17    2.00 | 10.33    5.74 | 16.50    2.25 | 22.67
1.50
                4.33    2.00 | 10.50    5.74 | 16.67    2.25 | 22.83
1.50
                4.50    2.00 | 10.67    7.74 | 16.83    2.25 | 23.00
1.50
                4.67    2.00 | 10.83    7.74 | 17.00    2.25 | 23.17
1.50
                4.83    2.00 | 11.00    7.74 | 17.17    2.25 | 23.33
1.50
                5.00    2.00 | 11.17   11.98 | 17.33    2.25 | 23.50
1.50
                5.17    2.00 | 11.33   11.98 | 17.50    2.25 | 23.67
1.50
                5.33    2.00 | 11.50   11.98 | 17.67    2.25 | 23.83
1.50
                5.50    2.00 | 11.67   36.94 | 17.83    2.25 | 24.00
1.50
                5.67    2.00 | 11.83   94.85 | 18.00    2.25 |
                5.83    2.00 | 12.00  152.76 | 18.17    2.25 |
```

```
                  6.00    2.00 | 12.17   17.97 | 18.33    2.25 |

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--------------------
| CALIB             |
| NASHYD   (  0001)|   Area    (ha)=   1.76   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.336

     PEAK FLOW       (cms)=   0.362 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  76.263
     TOTAL RAINFALL   (mm)= 124.800
     RUNOFF COEFFICIENT   =   0.611

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

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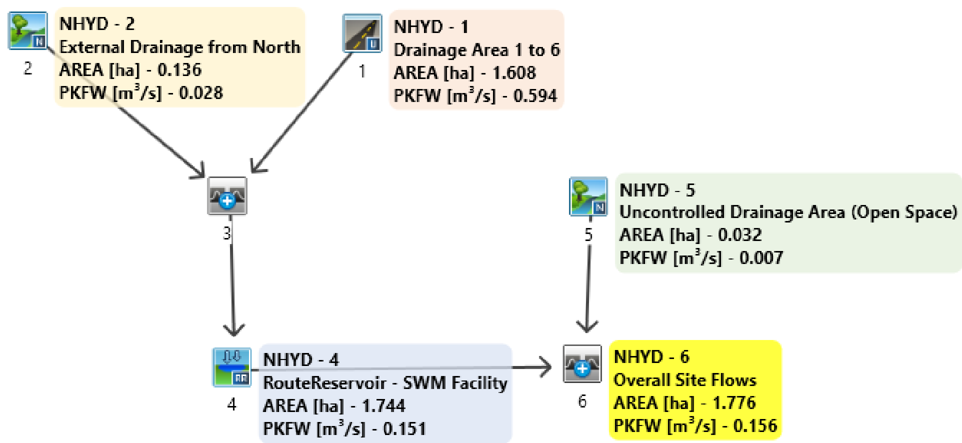
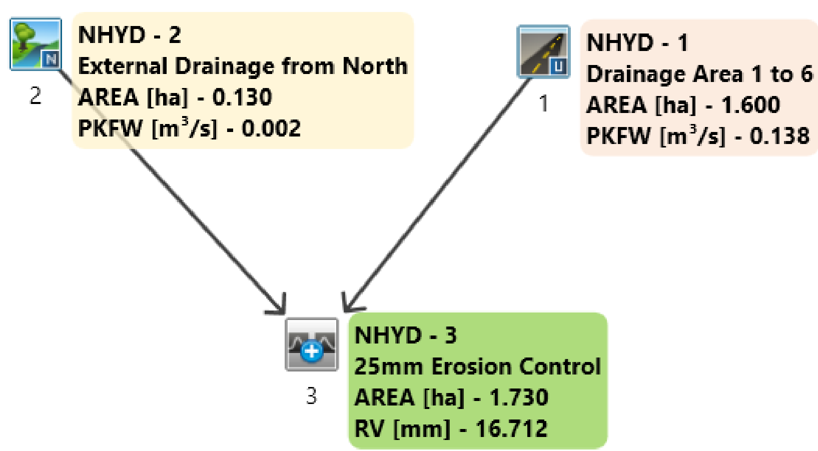
POST-DEVELOPMENT VO RESULTS



2 to 100-Year Storm Events VO Layout



25mm Storm Event VO Layout

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================================================================================
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       V    V   I    SSSSS  U   U    A    L              (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
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                   *****  D E T A I L E D   O U T P U T *****
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---------------------------------
  ************************************************
  ** SIMULATION : 25mm Chicago Storm            **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              fa713a13-5efc-404a-8020-
e3da1b5550e1\35b5a328
| Ptotal= 25.00 mm |    Comments: 25mm Chicago Storm
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
                 0.00    1.43 |  1.00    6.03 |  2.00    4.18 |  3.00
1.97
                 0.08    1.52 |  1.08    8.92 |  2.08    3.80 |  3.08
1.89
                 0.17    1.62 |  1.17   17.69 |  2.17    3.48 |  3.17
1.82
                 0.25    1.73 |  1.25   87.18 |  2.25    3.22 |  3.25
1.75
                 0.33    1.87 |  1.33   34.64 |  2.33    3.00 |  3.33
1.69
                 0.42    2.03 |  1.42   17.96 |  2.42    2.80 |  3.42
1.64
                 0.50    2.22 |  1.50   12.02 |  2.50    2.64 |  3.50
1.59
                 0.58    2.46 |  1.58    9.05 |  2.58    2.49 |  3.58
1.54
```

```
                 0.67    2.77 |  1.67    7.28 |  2.67    2.36 |  3.67
1.49
                 0.75    3.18 |  1.75    6.11 |  2.75    2.25 |  3.75
1.45
                 0.83    3.74 |  1.83    5.28 |  2.83    2.15 |  3.83
1.41
                 0.92    4.60 |  1.92    4.66 |  2.92    2.05 |  3.92
1.37

---------------------------------------------------------------------------
-----

--------------------
| CALIB             |
| NASHYD   (  0002)|   Area    (ha)=   0.13   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

         NOTE:  RAINFALL WAS TRANSFORMED TO  10.0 MIN. TIME STEP.


                               ---- TRANSFORMED HYETOGRAPH ----
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
                0.167    1.47 | 1.167    7.48 | 2.167    3.99 |  3.17
1.93
                0.333    1.67 | 1.333   52.44 | 2.333    3.35 |  3.33
1.79
                0.500    1.95 | 1.500   26.30 | 2.500    2.90 |  3.50
1.66
                0.667    2.34 | 1.667   10.54 | 2.667    2.57 |  3.67
1.57
                0.833    2.98 | 1.833    6.69 | 2.833    2.31 |  3.83
1.47
                1.000    4.17 | 2.000    4.97 | 3.000    2.10 |  4.00
1.39

     Unit Hyd Qpeak  (cms)=   0.025

     PEAK FLOW       (cms)=   0.002 (i)
     TIME TO PEAK    (hrs)=   1.500
     RUNOFF VOLUME    (mm)=   4.664
     TOTAL RAINFALL   (mm)=  25.002
     RUNOFF COEFFICIENT   =   0.187

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

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--------------------
| CALIB             |
| STANDHYD (  0001)|   Area    (ha)=   1.60
```

```
|ID= 1 DT=10.0 min |   Total Imp(%)=  61.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.98         0.62
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=     103.28        40.00
     Mannings n           =      0.013        0.250

         NOTE:  RAINFALL WAS TRANSFORMED TO  10.0 MIN. TIME STEP.


                               ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                 hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
               0.167    1.47 | 1.167    7.48 | 2.167    3.99 |  3.17
1.93
               0.333    1.67 | 1.333   52.44 | 2.333    3.35 |  3.33
1.79
               0.500    1.95 | 1.500   26.30 | 2.500    2.90 |  3.50
1.66
               0.667    2.34 | 1.667   10.54 | 2.667    2.57 |  3.67
1.57
               0.833    2.98 | 1.833    6.69 | 2.833    2.31 |  3.83
1.47
               1.000    4.17 | 2.000    4.97 | 3.000    2.10 |  4.00
1.39

     Max.Eff.Inten.(mm/hr)=      52.44        11.93
                over (min)      10.00        20.00
     Storage Coeff.  (min)=       3.37 (ii)   19.89 (ii)
     Unit Hyd. Tpeak (min)=      10.00        20.00
     Unit Hyd. peak  (cms)=       0.16         0.06
                                                          *  TOTALS*
     PEAK FLOW       (cms)=       0.13         0.01          0.138 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.67           1.33
     RUNOFF VOLUME    (mm)=      24.00         8.24          17.69
     TOTAL RAINFALL   (mm)=      25.00        25.00          25.00
     RUNOFF COEFFICIENT   =       0.96         0.33           0.71

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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```
| ADD HYD  (  0003)|
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0001):      1.60   0.138     1.33    17.69
      + ID2= 2 (  0002):      0.13   0.002     1.50     4.66
        ====================================================
        ID = 3 (  0003):      1.73   0.138     1.33    16.71

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
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 FINISH
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       V    V   I    SSSSS  U   U    A    L             (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
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                   *****  D E T A I L E D   O U T P U T *****
-------------------------------------------------------------------------------------
---------------------------------
  ************************************************
  ** SIMULATION : 2yr 24hr 10min SCS Type II (M **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              a0fd72d0-0518-4658-b701-
41444193f4ef\9fa91462
| Ptotal= 57.60 mm |    Comments: 2yr 24hr 10min SCS Type II (MTO)-Toronto
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
                 0.00    0.00 |  6.17    1.04 | 12.33    8.29 | 18.50
1.04
                 0.17    0.63 |  6.33    1.04 | 12.50    8.29 | 18.67
1.04
                 0.33    0.63 |  6.50    1.04 | 12.67    4.26 | 18.83
1.04
                 0.50    0.63 |  6.67    1.04 | 12.83    4.26 | 19.00
1.04
                 0.67    0.63 |  6.83    1.04 | 13.00    4.26 | 19.17
1.04
                 0.83    0.63 |  7.00    1.04 | 13.17    3.11 | 19.33
1.04
                 1.00    0.63 |  7.17    1.27 | 13.33    3.11 | 19.50
1.04
                 1.17    0.63 |  7.33    1.27 | 13.50    3.11 | 19.67
1.04
```

1.04	1.33	0.63	7.50	1.27	13.67	2.42	19.83
1.04	1.50	0.63	7.67	1.27	13.83	2.42	20.00
0.69	1.67	0.63	7.83	1.27	14.00	2.42	20.17
0.69	1.83	0.63	8.00	1.27	14.17	1.73	20.33
0.69	2.00	0.63	8.17	1.50	14.33	1.73	20.50
0.69	2.17	0.75	8.33	1.50	14.50	1.73	20.67
0.69	2.33	0.75	8.50	1.50	14.67	1.73	20.83
0.69	2.50	0.75	8.67	1.61	14.83	1.73	21.00
0.69	2.67	0.75	8.83	1.61	15.00	1.73	21.17
0.69	2.83	0.75	9.00	1.61	15.17	1.73	21.33
0.69	3.00	0.75	9.17	1.84	15.33	1.73	21.50
0.69	3.17	0.75	9.33	1.84	15.50	1.73	21.67
0.69	3.33	0.75	9.50	1.84	15.67	1.73	21.83
0.69	3.50	0.75	9.67	2.07	15.83	1.73	22.00
0.69	3.67	0.75	9.83	2.07	16.00	1.73	22.17
0.69	3.83	0.75	10.00	2.07	16.17	1.04	22.33
0.69	4.00	0.75	10.17	2.65	16.33	1.04	22.50
0.69	4.17	0.92	10.33	2.65	16.50	1.04	22.67
0.69	4.33	0.92	10.50	2.65	16.67	1.04	22.83
0.69	4.50	0.92	10.67	3.57	16.83	1.04	23.00
0.69	4.67	0.92	10.83	3.57	17.00	1.04	23.17
0.69	4.83	0.92	11.00	3.57	17.17	1.04	23.33
0.69	5.00	0.92	11.17	5.53	17.33	1.04	23.50
0.69	5.17	0.92	11.33	5.53	17.50	1.04	23.67
0.69	5.33	0.92	11.50	5.53	17.67	1.04	23.83
0.69	5.50	0.92	11.67	17.05	17.83	1.04	24.00
	5.67	0.92	11.83	43.78	18.00	1.04	
	5.83	0.92	12.00	70.50	18.17	1.04	

```
                6.00    0.92 | 12.17    8.29 | 18.33    1.04 |

----------------------------------------------------------------------------
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--------------------
| CALIB            |
| NASHYD   (  0005)|   Area    (ha)=   0.03   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.006

     PEAK FLOW       (cms)=   0.002 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  23.186
     TOTAL RAINFALL   (mm)=  57.600
     RUNOFF COEFFICIENT   =   0.403

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

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-----
--------------------
| CALIB            |
| NASHYD   (  0002)|   Area    (ha)=   0.13   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.025

     PEAK FLOW       (cms)=   0.008 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  23.209
     TOTAL RAINFALL   (mm)=  57.600
     RUNOFF COEFFICIENT   =   0.403

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

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--------------------
| CALIB            |
| STANDHYD (  0001)|   Area    (ha)=   1.60
|ID= 1 DT=10.0 min |   Total Imp(%)=  61.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.98         0.62
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=     103.28        40.00
     Mannings n           =      0.013        0.250

     Max.Eff.Inten.(mm/hr)=      70.50        47.31
```

```
               over (min)       10.00        20.00
     Storage Coeff.  (min)=      3.00 (ii)   12.52 (ii)
     Unit Hyd. Tpeak (min)=     10.00        20.00
     Unit Hyd. peak  (cms)=      0.16         0.07
                                                          *  TOTALS*
     PEAK FLOW       (cms)=      0.19         0.05          0.225 (iii)
     TIME TO PEAK    (hrs)=     12.17        12.33          12.17
     RUNOFF VOLUME    (mm)=     56.60        31.56          46.58
     TOTAL RAINFALL   (mm)=     57.60        57.60          57.60
     RUNOFF COEFFICIENT   =      0.98         0.55           0.81

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

---------------------------------------------------------------------------
-----

--------------------
| ADD HYD  (  0003)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0001):     1.60   0.225    12.17    46.58
      + ID2= 2 (  0002):     0.13   0.008    12.17    23.21
        =======================================================
        ID = 3 (  0003):     1.73   0.233    12.17    44.82

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
---------------------------------------------------------------------------
-----
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT= 10.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.1390      0.0432
                          0.1060     0.0203   |   0.1500      0.0552
                          0.1180     0.0269   |   0.1520      0.0579
                          0.1290     0.0349   |   0.0000      0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0003)      1.730      0.233     12.17       44.82
   OUTFLOW: ID= 1 (  0004)      1.730      0.100     12.33       44.80

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 42.96
                   TIME SHIFT OF PEAK FLOW         (min)= 10.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0197
```

```
--------------------------------------------------------------------------
-----

--------------------
| ADD HYD  (  0006)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0004):     1.73   0.100    12.33    44.80
      + ID2= 2 (  0005):     0.03   0.002    12.17    23.19
        ====================================================
        ID = 3 (  0006):     1.76   0.101    12.33    44.43

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
--------------------------------------------------------------------------
-----
```

```
=====================================================================================
=====================================

       V    V   I    SSSSS  U   U    A    L             (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
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                   *****  D E T A I L E D   O U T P U T *****
-------------------------------------------------------------------------------------
-------------------------------------
  ************************************************
  ** SIMULATION : 5yr 24hr 10min SCS Type II (M **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              a0fd72d0-0518-4658-b701-
41444193f4ef\c5c83801
| Ptotal= 74.40 mm |    Comments: 5yr 24hr 10min SCS Type II (MTO)-Toronto
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
                 0.00    0.00 |  6.17    1.34 | 12.33   10.71 | 18.50
1.34
                 0.17    0.82 |  6.33    1.34 | 12.50   10.71 | 18.67
1.34
                 0.33    0.82 |  6.50    1.34 | 12.67    5.51 | 18.83
1.34
                 0.50    0.82 |  6.67    1.34 | 12.83    5.51 | 19.00
1.34
                 0.67    0.82 |  6.83    1.34 | 13.00    5.51 | 19.17
1.34
                 0.83    0.82 |  7.00    1.34 | 13.17    4.02 | 19.33
1.34
                 1.00    0.82 |  7.17    1.64 | 13.33    4.02 | 19.50
1.34
                 1.17    0.82 |  7.33    1.64 | 13.50    4.02 | 19.67
1.34
```

1.33	0.82	7.50	1.64	13.67	3.12	19.83	1.34
1.50	0.82	7.67	1.64	13.83	3.12	20.00	1.34
1.67	0.82	7.83	1.64	14.00	3.12	20.17	0.89
1.83	0.82	8.00	1.64	14.17	2.23	20.33	0.89
2.00	0.82	8.17	1.93	14.33	2.23	20.50	0.89
2.17	0.97	8.33	1.93	14.50	2.23	20.67	0.89
2.33	0.97	8.50	1.93	14.67	2.23	20.83	0.89
2.50	0.97	8.67	2.08	14.83	2.23	21.00	0.89
2.67	0.97	8.83	2.08	15.00	2.23	21.17	0.89
2.83	0.97	9.00	2.08	15.17	2.23	21.33	0.89
3.00	0.97	9.17	2.38	15.33	2.23	21.50	0.89
3.17	0.97	9.33	2.38	15.50	2.23	21.67	0.89
3.33	0.97	9.50	2.38	15.67	2.23	21.83	0.89
3.50	0.97	9.67	2.68	15.83	2.23	22.00	0.89
3.67	0.97	9.83	2.68	16.00	2.23	22.17	0.89
3.83	0.97	10.00	2.68	16.17	1.34	22.33	0.89
4.00	0.97	10.17	3.42	16.33	1.34	22.50	0.89
4.17	1.19	10.33	3.42	16.50	1.34	22.67	0.89
4.33	1.19	10.50	3.42	16.67	1.34	22.83	0.89
4.50	1.19	10.67	4.61	16.83	1.34	23.00	0.89
4.67	1.19	10.83	4.61	17.00	1.34	23.17	0.89
4.83	1.19	11.00	4.61	17.17	1.34	23.33	0.89
5.00	1.19	11.17	7.14	17.33	1.34	23.50	0.89
5.17	1.19	11.33	7.14	17.50	1.34	23.67	0.89
5.33	1.19	11.50	7.14	17.67	1.34	23.83	0.89
5.50	1.19	11.67	22.02	17.83	1.34	24.00	0.89
5.67	1.19	11.83	56.54	18.00	1.34		
5.83	1.19	12.00	91.07	18.17	1.34		

```
                 6.00     1.19 | 12.17   10.71 | 18.33    1.34 |

----------------------------------------------------------------------------
-----

--------------------
| CALIB            |
| NASHYD   (  0005)|   Area    (ha)=   0.03   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.006

     PEAK FLOW       (cms)=   0.003 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  35.288
     TOTAL RAINFALL   (mm)=  74.400
     RUNOFF COEFFICIENT   =   0.474

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

----------------------------------------------------------------------------
-----
--------------------
| CALIB            |
| NASHYD   (  0002)|   Area    (ha)=   0.13   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.025

     PEAK FLOW       (cms)=   0.012 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  35.296
     TOTAL RAINFALL   (mm)=  74.400
     RUNOFF COEFFICIENT   =   0.474

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

----------------------------------------------------------------------------
-----
--------------------
| CALIB            |
| STANDHYD (  0001)|   Area    (ha)=   1.60
|ID= 1 DT=10.0 min |   Total Imp(%)=  61.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.98         0.62
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=     103.28        40.00
     Mannings n           =      0.013        0.250

     Max.Eff.Inten.(mm/hr)=      91.07        67.90
```

```
              over (min)        10.00        20.00
     Storage Coeff.  (min)=      2.70 (ii)   10.94 (ii)
     Unit Hyd. Tpeak (min)=     10.00        20.00
     Unit Hyd. peak  (cms)=      0.17         0.08
                                                          *  TOTALS*
     PEAK FLOW       (cms)=      0.24         0.07          0.301 (iii)
     TIME TO PEAK    (hrs)=     12.17        12.33          12.17
     RUNOFF VOLUME    (mm)=     73.40        45.61          62.28
     TOTAL RAINFALL   (mm)=     74.40        74.40          74.40
     RUNOFF COEFFICIENT   =      0.99         0.61           0.84

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

---------------------------------------------------------------------------
-----

--------------------
| ADD HYD  (  0003)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0001):     1.60   0.301    12.17    62.28
      + ID2= 2 (  0002):     0.13   0.012    12.17    35.30
        =======================================================
        ID = 3 (  0003):     1.73   0.314    12.17    60.25

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
---------------------------------------------------------------------------
-----
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT= 10.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.1390      0.0432
                          0.1060     0.0203   |   0.1500      0.0552
                          0.1180     0.0269   |   0.1520      0.0579
                          0.1290     0.0349   |   0.0000      0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0003)      1.730      0.314     12.17      60.25
   OUTFLOW: ID= 1 (  0004)      1.730      0.117     12.33      60.23

                    PEAK   FLOW   REDUCTION [Qout/Qin](%)= 37.36
                    TIME SHIFT OF PEAK FLOW         (min)= 10.00
                    MAXIMUM  STORAGE   USED       (ha.m.)=  0.0265
```

```
--------------------------------------------------------------------------------
-----

--------------------
| ADD HYD  (  0006)|
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0004):      1.73   0.117    12.33    60.23
      + ID2= 2 (  0005):      0.03   0.003    12.17    35.29
        ====================================================
        ID = 3 (  0006):      1.76   0.119    12.33    59.80

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
--------------------------------------------------------------------------------
-----
```

```
==================================================================================
=================================

      V    V   I    SSSSS  U   U    A    L              (v 6.2.2015)
      V    V   I    SS     U   U   A A   L
       V  V    I     SS    U   U  AAAAA  L
       V  V    I      SS   U   U  A   A  L
        VV     I    SSSSS  UUUUU  A   A  LLLLL

       OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
      O   O    T      T    H   H   Y Y   MM MM  O   O
      O   O    T      T    H   H    Y    M   M  O   O
       OOO     T      T    H   H    Y    M   M   OOO
```



```
                   *****  D E T A I L E D   O U T P U T *****

----------------------------------------------------------------------------------
---------------------------------
  ************************************************
  ** SIMULATION : 10yr 24hr 10min SCS Type II ( **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              a0fd72d0-0518-4658-b701-
41444193f4ef\664e8d69
| Ptotal= 86.40 mm |    Comments: 10yr 24hr 10min SCS Type II (MTO)-Toront
--------------------
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME
RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs
mm/hr
                 0.00    0.00 |  6.17    1.56 | 12.33   12.44 | 18.50
1.56
                 0.17    0.95 |  6.33    1.56 | 12.50   12.44 | 18.67
1.56
                 0.33    0.95 |  6.50    1.56 | 12.67    6.39 | 18.83
1.56
                 0.50    0.95 |  6.67    1.56 | 12.83    6.39 | 19.00
1.56
                 0.67    0.95 |  6.83    1.56 | 13.00    6.39 | 19.17
1.56
                 0.83    0.95 |  7.00    1.56 | 13.17    4.67 | 19.33
1.56
                 1.00    0.95 |  7.17    1.90 | 13.33    4.67 | 19.50
1.56
                 1.17    0.95 |  7.33    1.90 | 13.50    4.67 | 19.67
1.56
```

1.33 0.95 | 7.50 1.90 | 13.67 3.63 | 19.83
1.56
1.50 0.95 | 7.67 1.90 | 13.83 3.63 | 20.00
1.56
1.67 0.95 | 7.83 1.90 | 14.00 3.63 | 20.17
1.04
1.83 0.95 | 8.00 1.90 | 14.17 2.59 | 20.33
1.04
2.00 0.95 | 8.17 2.25 | 14.33 2.59 | 20.50
1.04
2.17 1.12 | 8.33 2.25 | 14.50 2.59 | 20.67
1.04
2.33 1.12 | 8.50 2.25 | 14.67 2.59 | 20.83
1.04
2.50 1.12 | 8.67 2.42 | 14.83 2.59 | 21.00
1.04
2.67 1.12 | 8.83 2.42 | 15.00 2.59 | 21.17
1.04
2.83 1.12 | 9.00 2.42 | 15.17 2.59 | 21.33
1.04
3.00 1.12 | 9.17 2.76 | 15.33 2.59 | 21.50
1.04
3.17 1.12 | 9.33 2.76 | 15.50 2.59 | 21.67
1.04
3.33 1.12 | 9.50 2.76 | 15.67 2.59 | 21.83
1.04
3.50 1.12 | 9.67 3.11 | 15.83 2.59 | 22.00
1.04
3.67 1.12 | 9.83 3.11 | 16.00 2.59 | 22.17
1.04
3.83 1.12 | 10.00 3.11 | 16.17 1.56 | 22.33
1.04
4.00 1.12 | 10.17 3.97 | 16.33 1.56 | 22.50
1.04
4.17 1.38 | 10.33 3.97 | 16.50 1.56 | 22.67
1.04
4.33 1.38 | 10.50 3.97 | 16.67 1.56 | 22.83
1.04
4.50 1.38 | 10.67 5.36 | 16.83 1.56 | 23.00
1.04
4.67 1.38 | 10.83 5.36 | 17.00 1.56 | 23.17
1.04
4.83 1.38 | 11.00 5.36 | 17.17 1.56 | 23.33
1.04
5.00 1.38 | 11.17 8.29 | 17.33 1.56 | 23.50
1.04
5.17 1.38 | 11.33 8.29 | 17.50 1.56 | 23.67
1.04
5.33 1.38 | 11.50 8.29 | 17.67 1.56 | 23.83
1.04
5.50 1.38 | 11.67 25.57 | 17.83 1.56 | 24.00
1.04
5.67 1.38 | 11.83 65.66 | 18.00 1.56 |
5.83 1.38 | 12.00 105.75 | 18.17 1.56 |

```
                  6.00    1.38 | 12.17   12.44 | 18.33    1.56 |

----------------------------------------------------------------------------
-----

--------------------
| CALIB            |
| NASHYD   (  0005)|   Area    (ha)=   0.03   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.006

     PEAK FLOW       (cms)=   0.004 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  44.526
     TOTAL RAINFALL   (mm)=  86.400
     RUNOFF COEFFICIENT   =   0.515

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

----------------------------------------------------------------------------
-----
--------------------
| CALIB            |
| NASHYD   (  0002)|   Area    (ha)=   0.13   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.025

     PEAK FLOW       (cms)=   0.016 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  44.536
     TOTAL RAINFALL   (mm)=  86.400
     RUNOFF COEFFICIENT   =   0.515

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

----------------------------------------------------------------------------
-----
--------------------
| CALIB            |
| STANDHYD (  0001)|   Area    (ha)=   1.60
|ID= 1 DT=10.0 min |   Total Imp(%)=  61.00   Dir. Conn.(%)=  60.00
--------------------
                             IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.98         0.62
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=     103.28        40.00
     Mannings n           =      0.013        0.250

     Max.Eff.Inten.(mm/hr)=     105.75        83.05
```

```
                over (min)        10.00         20.00
     Storage Coeff.  (min)=        2.55 (ii)   10.15 (ii)
     Unit Hyd. Tpeak (min)=       10.00         20.00
     Unit Hyd. peak  (cms)=        0.17          0.08
                                                            *  TOTALS*
     PEAK FLOW       (cms)=        0.28          0.09          0.357 (iii)
     TIME TO PEAK    (hrs)=       12.17         12.33          12.17
     RUNOFF VOLUME    (mm)=       85.40         56.08          73.67
     TOTAL RAINFALL   (mm)=       86.40         86.40          86.40
     RUNOFF COEFFICIENT   =        0.99          0.65           0.85

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

---------------------------------------------------------------------------
-----

--------------------
| ADD HYD  (  0003)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0001):     1.60   0.357    12.17    73.67
      + ID2= 2 (  0002):     0.13   0.016    12.17    44.54
        =====================================================
        ID = 3 (  0003):     1.73   0.373    12.17    71.48

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
---------------------------------------------------------------------------
-----
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT= 10.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.1390      0.0432
                          0.1060     0.0203   |   0.1500      0.0552
                          0.1180     0.0269   |   0.1520      0.0579
                          0.1290     0.0349   |   0.0000      0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0003)      1.730      0.373     12.17       71.48
   OUTFLOW: ID= 1 (  0004)      1.730      0.126     12.33       71.46

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 33.77
                   TIME SHIFT OF PEAK FLOW         (min)= 10.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0330
```

```
----------------------------------------------------------------------------
-----

--------------------
| ADD HYD  (  0006)|
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0004):      1.73   0.126    12.33    71.46
      + ID2= 2 (  0005):      0.03   0.004    12.17    44.53
        ====================================================
        ID = 3 (  0006):      1.76   0.129    12.33    71.00

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
----------------------------------------------------------------------------
-----
```

```
==================================================================================================================

       V    V   I    SSSSS  U   U    A    L              (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO     TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
```

Developed and Distributed by Smart City Water Inc


```
                   *****  D E T A I L E D   O U T P U T *****
------------------------------------------------------------------------------------------------------------------
  ************************************************
  ** SIMULATION : 25yr 24hr 10min SCS Type II ( **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              a0fd72d0-0518-4658-b701-
41444193f4ef\6fd937b5
| Ptotal=100.80 mm |    Comments: 25yr 24hr 10min SCS Type II (MTO)-Toront
--------------------
```

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
0.00	0.00	6.17	1.81	12.33	14.52	18.50	1.81
0.17	1.11	6.33	1.81	12.50	14.52	18.67	1.81
0.33	1.11	6.50	1.81	12.67	7.46	18.83	1.81
0.50	1.11	6.67	1.81	12.83	7.46	19.00	1.81
0.67	1.11	6.83	1.81	13.00	7.46	19.17	1.81
0.83	1.11	7.00	1.81	13.17	5.44	19.33	1.81
1.00	1.11	7.17	2.22	13.33	5.44	19.50	1.81
1.17	1.11	7.33	2.22	13.50	5.44	19.67	1.81

1.33 1.11 | 7.50 2.22 | 13.67 4.23 | 19.83
1.81
1.50 1.11 | 7.67 2.22 | 13.83 4.23 | 20.00
1.81
1.67 1.11 | 7.83 2.22 | 14.00 4.23 | 20.17
1.21
1.83 1.11 | 8.00 2.22 | 14.17 3.02 | 20.33
1.21
2.00 1.11 | 8.17 2.62 | 14.33 3.02 | 20.50
1.21
2.17 1.31 | 8.33 2.62 | 14.50 3.02 | 20.67
1.21
2.33 1.31 | 8.50 2.62 | 14.67 3.02 | 20.83
1.21
2.50 1.31 | 8.67 2.82 | 14.83 3.02 | 21.00
1.21
2.67 1.31 | 8.83 2.82 | 15.00 3.02 | 21.17
1.21
2.83 1.31 | 9.00 2.82 | 15.17 3.02 | 21.33
1.21
3.00 1.31 | 9.17 3.23 | 15.33 3.02 | 21.50
1.21
3.17 1.31 | 9.33 3.23 | 15.50 3.02 | 21.67
1.21
3.33 1.31 | 9.50 3.23 | 15.67 3.02 | 21.83
1.21
3.50 1.31 | 9.67 3.63 | 15.83 3.02 | 22.00
1.21
3.67 1.31 | 9.83 3.63 | 16.00 3.02 | 22.17
1.21
3.83 1.31 | 10.00 3.63 | 16.17 1.81 | 22.33
1.21
4.00 1.31 | 10.17 4.64 | 16.33 1.81 | 22.50
1.21
4.17 1.61 | 10.33 4.64 | 16.50 1.81 | 22.67
1.21
4.33 1.61 | 10.50 4.64 | 16.67 1.81 | 22.83
1.21
4.50 1.61 | 10.67 6.25 | 16.83 1.81 | 23.00
1.21
4.67 1.61 | 10.83 6.25 | 17.00 1.81 | 23.17
1.21
4.83 1.61 | 11.00 6.25 | 17.17 1.81 | 23.33
1.21
5.00 1.61 | 11.17 9.68 | 17.33 1.81 | 23.50
1.21
5.17 1.61 | 11.33 9.68 | 17.50 1.81 | 23.67
1.21
5.33 1.61 | 11.50 9.68 | 17.67 1.81 | 23.83
1.21
5.50 1.61 | 11.67 29.84 | 17.83 1.81 | 24.00
1.21
5.67 1.61 | 11.83 76.61 | 18.00 1.81 |
5.83 1.61 | 12.00 123.38 | 18.17 1.81 |

```
                  6.00    1.61 | 12.17   14.52 | 18.33    1.81 |

-----------------------------------------------------------------------------
-----

--------------------
| CALIB             |
| NASHYD   (  0005)|   Area    (ha)=   0.03   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.006

     PEAK FLOW       (cms)=   0.005 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  56.099
     TOTAL RAINFALL   (mm)= 100.800
     RUNOFF COEFFICIENT   =   0.557

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

-----------------------------------------------------------------------------
-----
--------------------
| CALIB             |
| NASHYD   (  0002)|   Area    (ha)=   0.13   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.025

     PEAK FLOW       (cms)=   0.020 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  56.111
     TOTAL RAINFALL   (mm)= 100.800
     RUNOFF COEFFICIENT   =   0.557

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

-----------------------------------------------------------------------------
-----
--------------------
| CALIB             |
| STANDHYD (  0001)|   Area    (ha)=   1.60
|ID= 1 DT=10.0 min |   Total Imp(%)=  61.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.98         0.62
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=     103.28        40.00
     Mannings n           =      0.013        0.250

     Max.Eff.Inten.(mm/hr)=     123.38       101.51
```

```
              over (min)       10.00        10.00
     Storage Coeff.  (min)=       2.39 (ii)    9.41 (ii)
     Unit Hyd. Tpeak (min)=      10.00        10.00
     Unit Hyd. peak  (cms)=       0.17         0.11
                                                          *  TOTALS*
     PEAK FLOW       (cms)=       0.33         0.14          0.467 (iii)
     TIME TO PEAK    (hrs)=      12.17        12.17          12.17
     RUNOFF VOLUME    (mm)=      99.80        68.99          87.47
     TOTAL RAINFALL   (mm)=     100.80       100.80         100.80
     RUNOFF COEFFICIENT   =       0.99         0.68           0.87

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| ADD HYD  (  0003)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0001):     1.60   0.467    12.17    87.47
      + ID2= 2 (  0002):     0.13   0.020    12.17    56.11
        ===================================================
        ID = 3 (  0003):     1.73   0.487    12.17    85.11

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
---------------------------------------------------------------------------
-----
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT= 10.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.1390      0.0432
                          0.1060     0.0203   |   0.1500      0.0552
                          0.1180     0.0269   |   0.1520      0.0579
                          0.1290     0.0349   |   0.0000      0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0003)      1.730      0.487     12.17       85.11
   OUTFLOW: ID= 1 (  0004)      1.730      0.139     12.33       85.09

                    PEAK   FLOW   REDUCTION [Qout/Qin](%)= 28.49
                    TIME SHIFT OF PEAK FLOW         (min)= 10.00
                    MAXIMUM  STORAGE   USED       (ha.m.)=  0.0436
```

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---------------------------------------------------------------------------
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--------------------
| ADD HYD  (  0006)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0004):     1.73   0.139    12.33    85.09
      + ID2= 2 (  0005):     0.03   0.005    12.17    56.10
        =====================================================
        ID = 3 (  0006):     1.76   0.142    12.33    84.60

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
---------------------------------------------------------------------------
-----
```

```
==========================================================================================================================
       V    V   I    SSSSS  U   U    A    L             (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
```


```
                   *****  D E T A I L E D   O U T P U T *****
--------------------------------------------------------------------------------------------------------------------------
  ************************************************
  ** SIMULATION : 50yr 24hr 10min SCS Type II ( **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              a0fd72d0-0518-4658-b701-
41444193f4ef\afcb7b4a
| Ptotal=112.80 mm |    Comments: 50yr 24hr 10min SCS Type II (MTO)-Toront
--------------------
```

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
0.00	0.00	6.17	2.03	12.33	16.24	18.50	2.03
0.17	1.24	6.33	2.03	12.50	16.24	18.67	2.03
0.33	1.24	6.50	2.03	12.67	8.35	18.83	2.03
0.50	1.24	6.67	2.03	12.83	8.35	19.00	2.03
0.67	1.24	6.83	2.03	13.00	8.35	19.17	2.03
0.83	1.24	7.00	2.03	13.17	6.09	19.33	2.03
1.00	1.24	7.17	2.48	13.33	6.09	19.50	2.03
1.17	1.24	7.33	2.48	13.50	6.09	19.67	2.03

1.33	1.24	7.50	2.48	13.67	4.74	19.83	2.03
1.50	1.24	7.67	2.48	13.83	4.74	20.00	2.03
1.67	1.24	7.83	2.48	14.00	4.74	20.17	1.35
1.83	1.24	8.00	2.48	14.17	3.38	20.33	1.35
2.00	1.24	8.17	2.93	14.33	3.38	20.50	1.35
2.17	1.47	8.33	2.93	14.50	3.38	20.67	1.35
2.33	1.47	8.50	2.93	14.67	3.38	20.83	1.35
2.50	1.47	8.67	3.16	14.83	3.38	21.00	1.35
2.67	1.47	8.83	3.16	15.00	3.38	21.17	1.35
2.83	1.47	9.00	3.16	15.17	3.38	21.33	1.35
3.00	1.47	9.17	3.61	15.33	3.38	21.50	1.35
3.17	1.47	9.33	3.61	15.50	3.38	21.67	1.35
3.33	1.47	9.50	3.61	15.67	3.38	21.83	1.35
3.50	1.47	9.67	4.06	15.83	3.38	22.00	1.35
3.67	1.47	9.83	4.06	16.00	3.38	22.17	1.35
3.83	1.47	10.00	4.06	16.17	2.03	22.33	1.35
4.00	1.47	10.17	5.19	16.33	2.03	22.50	1.35
4.17	1.80	10.33	5.19	16.50	2.03	22.67	1.35
4.33	1.80	10.50	5.19	16.67	2.03	22.83	1.35
4.50	1.80	10.67	6.99	16.83	2.03	23.00	1.35
4.67	1.80	10.83	6.99	17.00	2.03	23.17	1.35
4.83	1.80	11.00	6.99	17.17	2.03	23.33	1.35
5.00	1.80	11.17	10.83	17.33	2.03	23.50	1.35
5.17	1.80	11.33	10.83	17.50	2.03	23.67	1.35
5.33	1.80	11.50	10.83	17.67	2.03	23.83	1.35
5.50	1.80	11.67	33.39	17.83	2.03	24.00	1.35
5.67	1.80	11.83	85.73	18.00	2.03		
5.83	1.80	12.00	138.07	18.17	2.03		

```
                  6.00    1.80 | 12.17   16.24 | 18.33    2.03 |

---------------------------------------------------------------------------
-----

--------------------
| CALIB            |
| NASHYD   (  0005)|   Area    (ha)=   0.03   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.006

     PEAK FLOW       (cms)=   0.005 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  66.057
     TOTAL RAINFALL   (mm)= 112.800
     RUNOFF COEFFICIENT   =   0.586

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

---------------------------------------------------------------------------
-----
--------------------
| CALIB            |
| NASHYD   (  0002)|   Area    (ha)=   0.13   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.025

     PEAK FLOW       (cms)=   0.023 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  66.071
     TOTAL RAINFALL   (mm)= 112.800
     RUNOFF COEFFICIENT   =   0.586

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

---------------------------------------------------------------------------
-----
--------------------
| CALIB            |
| STANDHYD (  0001)|   Area    (ha)=   1.60
|ID= 1 DT=10.0 min |   Total Imp(%)=  61.00   Dir. Conn.(%)=  60.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.98         0.62
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=     103.28        40.00
     Mannings n          =      0.013        0.250

     Max.Eff.Inten.(mm/hr)=     138.07       117.03
```

```
                over (min)       10.00        10.00
     Storage Coeff.  (min)=       2.29 (ii)    8.92 (ii)
     Unit Hyd. Tpeak (min)=      10.00        10.00
     Unit Hyd. peak  (cms)=       0.17         0.11
                                                          *  TOTALS*
     PEAK FLOW       (cms)=       0.37         0.16          0.531 (iii)
     TIME TO PEAK    (hrs)=      12.17        12.17          12.17
     RUNOFF VOLUME    (mm)=     111.80        79.95          99.06
     TOTAL RAINFALL   (mm)=     112.80       112.80         112.80
     RUNOFF COEFFICIENT   =       0.99         0.71           0.88

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

---------------------------------------------------------------------------
-----

--------------------
| ADD HYD  (  0003)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0001):     1.60   0.531    12.17    99.06
      + ID2= 2 (  0002):     0.13   0.023    12.17    66.07
        =======================================================
        ID = 3 (  0003):     1.73   0.555    12.17    96.58

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
---------------------------------------------------------------------------
-----
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT= 10.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.1390      0.0432
                          0.1060     0.0203   |   0.1500      0.0552
                          0.1180     0.0269   |   0.1520      0.0579
                          0.1290     0.0349   |   0.0000      0.0000

                               AREA     QPEAK     TPEAK       R.V.
                               (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0003)     1.730      0.555     12.17       96.58
   OUTFLOW: ID= 1 (  0004)     1.730      0.145     12.33       96.56

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 26.19
                  TIME SHIFT OF PEAK FLOW         (min)= 10.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=  0.0503
```

```
----------------------------------------------------------------------------
-----

--------------------
| ADD HYD  (  0006)|
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0004):      1.73   0.145    12.33    96.56
      + ID2= 2 (  0005):      0.03   0.005    12.17    66.06
        ====================================================
        ID = 3 (  0006):      1.76   0.149    12.33    96.04

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
----------------------------------------------------------------------------
-----
```

```
=========================================================================================================

       V    V   I    SSSSS  U   U    A    L              (v 6.2.2015)
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO     TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO
```



```
                   *****  D E T A I L E D   O U T P U T *****
---------------------------------------------------------------------------------------------------------
  ************************************************
  ** SIMULATION : 100yr 24hr 10min SCS Type II  **
  ************************************************


--------------------
|    READ STORM    |    Filename: C:\Users\shuchi\AppD
|                  |              ata\Local\Temp\
|                  |              a0fd72d0-0518-4658-b701-
41444193f4ef\d268efaa
| Ptotal=124.80 mm |    Comments: 100yr 24hr 10min SCS Type II (MTO)-Toron
--------------------
```

TIME	RAIN	TIME	RAIN	TIME	RAIN	TIME	RAIN
hrs	mm/hr	hrs	mm/hr	hrs	mm/hr	hrs	mm/hr
0.00	0.00	6.17	2.25	12.33	17.97	18.50	2.25
0.17	1.37	6.33	2.25	12.50	17.97	18.67	2.25
0.33	1.37	6.50	2.25	12.67	9.24	18.83	2.25
0.50	1.37	6.67	2.25	12.83	9.24	19.00	2.25
0.67	1.37	6.83	2.25	13.00	9.24	19.17	2.25
0.83	1.37	7.00	2.25	13.17	6.74	19.33	2.25
1.00	1.37	7.17	2.75	13.33	6.74	19.50	2.25
1.17	1.37	7.33	2.75	13.50	6.74	19.67	2.25

```
                 1.33    1.37 |  7.50    2.75 | 13.67    5.24 | 19.83
 2.25
                 1.50    1.37 |  7.67    2.75 | 13.83    5.24 | 20.00
 2.25
                 1.67    1.37 |  7.83    2.75 | 14.00    5.24 | 20.17
 1.50
                 1.83    1.37 |  8.00    2.75 | 14.17    3.74 | 20.33
 1.50
                 2.00    1.37 |  8.17    3.24 | 14.33    3.74 | 20.50
 1.50
                 2.17    1.62 |  8.33    3.24 | 14.50    3.74 | 20.67
 1.50
                 2.33    1.62 |  8.50    3.24 | 14.67    3.74 | 20.83
 1.50
                 2.50    1.62 |  8.67    3.49 | 14.83    3.74 | 21.00
 1.50
                 2.67    1.62 |  8.83    3.49 | 15.00    3.74 | 21.17
 1.50
                 2.83    1.62 |  9.00    3.49 | 15.17    3.74 | 21.33
 1.50
                 3.00    1.62 |  9.17    3.99 | 15.33    3.74 | 21.50
 1.50
                 3.17    1.62 |  9.33    3.99 | 15.50    3.74 | 21.67
 1.50
                 3.33    1.62 |  9.50    3.99 | 15.67    3.74 | 21.83
 1.50
                 3.50    1.62 |  9.67    4.49 | 15.83    3.74 | 22.00
 1.50
                 3.67    1.62 |  9.83    4.49 | 16.00    3.74 | 22.17
 1.50
                 3.83    1.62 | 10.00    4.49 | 16.17    2.25 | 22.33
 1.50
                 4.00    1.62 | 10.17    5.74 | 16.33    2.25 | 22.50
 1.50
                 4.17    2.00 | 10.33    5.74 | 16.50    2.25 | 22.67
 1.50
                 4.33    2.00 | 10.50    5.74 | 16.67    2.25 | 22.83
 1.50
                 4.50    2.00 | 10.67    7.74 | 16.83    2.25 | 23.00
 1.50
                 4.67    2.00 | 10.83    7.74 | 17.00    2.25 | 23.17
 1.50
                 4.83    2.00 | 11.00    7.74 | 17.17    2.25 | 23.33
 1.50
                 5.00    2.00 | 11.17   11.98 | 17.33    2.25 | 23.50
 1.50
                 5.17    2.00 | 11.33   11.98 | 17.50    2.25 | 23.67
 1.50
                 5.33    2.00 | 11.50   11.98 | 17.67    2.25 | 23.83
 1.50
                 5.50    2.00 | 11.67   36.94 | 17.83    2.25 | 24.00
 1.50
                 5.67    2.00 | 11.83   94.85 | 18.00    2.25 |
                 5.83    2.00 | 12.00  152.76 | 18.17    2.25 |
```

```
                6.00    2.00 | 12.17   17.97 | 18.33    2.25 |

----------------------------------------------------------------------------
-----

--------------------
| CALIB            |
| NASHYD   (  0005)|   Area    (ha)=   0.03   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.006

     PEAK FLOW       (cms)=   0.006 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  76.242
     TOTAL RAINFALL   (mm)= 124.800
     RUNOFF COEFFICIENT   =   0.611

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

----------------------------------------------------------------------------
-----
--------------------
| CALIB            |
| NASHYD   (  0002)|   Area    (ha)=   0.13   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   0.025

     PEAK FLOW       (cms)=   0.027 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)=  76.258
     TOTAL RAINFALL   (mm)= 124.800
     RUNOFF COEFFICIENT   =   0.611

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

----------------------------------------------------------------------------
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--------------------
| CALIB            |
| STANDHYD (  0001)|   Area    (ha)=   1.60
|ID= 1 DT=10.0 min |   Total Imp(%)=  61.00   Dir. Conn.(%)=  60.00
--------------------
                               IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.98          0.62
     Dep. Storage     (mm)=       1.00          1.50
     Average Slope     (%)=       1.00          2.00
     Length            (m)=     103.28         40.00
     Mannings n           =      0.013         0.250

     Max.Eff.Inten.(mm/hr)=     152.76        132.62
```

```
               over (min)       10.00        10.00
     Storage Coeff.  (min)=      2.20 (ii)    8.50 (ii)
     Unit Hyd. Tpeak (min)=     10.00        10.00
     Unit Hyd. peak  (cms)=      0.17         0.12
                                                       *  TOTALS*
     PEAK FLOW       (cms)=      0.41         0.19         0.596 (iii)
     TIME TO PEAK    (hrs)=     12.17        12.17         12.17
     RUNOFF VOLUME    (mm)=    123.80        91.07        110.70
     TOTAL RAINFALL   (mm)=    124.80       124.80        124.80
     RUNOFF COEFFICIENT   =      0.99         0.73          0.89

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

---------------------------------------------------------------------------
-----

--------------------
| ADD HYD  (  0003)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0001):     1.60   0.596    12.17   110.70
      + ID2= 2 (  0002):     0.13   0.027    12.17    76.26
        ===================================================
        ID = 3 (  0003):     1.73   0.623    12.17   108.12

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
---------------------------------------------------------------------------
-----
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT= 10.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.1390      0.0432
                          0.1060     0.0203   |   0.1500      0.0552
                          0.1180     0.0269   |   0.1520      0.0579
                          0.1290     0.0349   |   0.0000      0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0003)      1.730      0.623     12.17      108.12
   OUTFLOW: ID= 1 (  0004)      1.730      0.151     12.33      108.10

                       PEAK   FLOW   REDUCTION [Qout/Qin](%)= 24.32
                       TIME SHIFT OF PEAK FLOW         (min)= 10.00
                       MAXIMUM  STORAGE   USED       (ha.m.)=  0.0572
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---------------------------------------------------------------------------
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--------------------
| ADD HYD  (  0006)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0004):     1.73   0.151    12.33   108.10
      + ID2= 2 (  0005):     0.03   0.006    12.17    76.24
        ====================================================
        ID = 3 (  0006):     1.76   0.156    12.33   107.55

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
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WATER QUALITY - JELLYFISH FILTRATION
TREATMENT DESIGN REPORT

STANDARD OFFLINE
Jellyfish Filter Sizing Report

Project Information

Date	Friday, May 24, 2024
Project Name	Kaufman Rd.
Project Number	W22002
Location	W22002

Jellyfish Filter Design Overview

This report provides information for the sizing and specification of the Jellyfish Filter. When designed properly in accordance to the guidelines detailed in the Jellyfish Filter Technical Manual, the Jellyfish Filter will exceed the performance and longevity of conventional horizontal bed and granular media filters.

Please see ***www.ImbriumSystems.com*** *for more information.*

Jellyfish Filter System Recommendation

The Jellyfish Filter model JF8-6-2 is recommended to meet the water quality objective by treating a flow of 35.3 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 398 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF8-6-2	6	2	2.4	35.3	398

The Jellyfish Filter System

The patented Jellyfish Filter is an engineered stormwater quality treatment technology featuring unique membrane filtration in a compact stand-alone treatment system that removes a high level and wide variety of stormwater pollutants. Exceptional pollutant removal is achieved at high treatment flow rates with minimal head loss and low maintenance costs. Each lightweight Jellyfish Filter cartridge contains an extraordinarily large amount of membrane surface area, resulting in superior flow capacity and pollutant removal capacity.

Maintenance

Regular scheduled inspections and maintenance is necessary to assure proper functioning of the Jellyfish Filter. The maintenance interval is designed to be a minimum of 12 months, but this will vary depending on site loading conditions and upstream pretreatment measures. Quarterly inspections and inspections after all storms beyond the 5-year event are recommended until enough historical performance data has been logged to comfortably initiate an alternative inspection interval.

Please see ***www.ImbriumSystems.com*** *for more information.*

Thank you for the opportunity to present this information to you and your client.

Jellyfish® Filter

Performance

Jellyfish efficiently captures a high level of Stormwater pollutants, including:

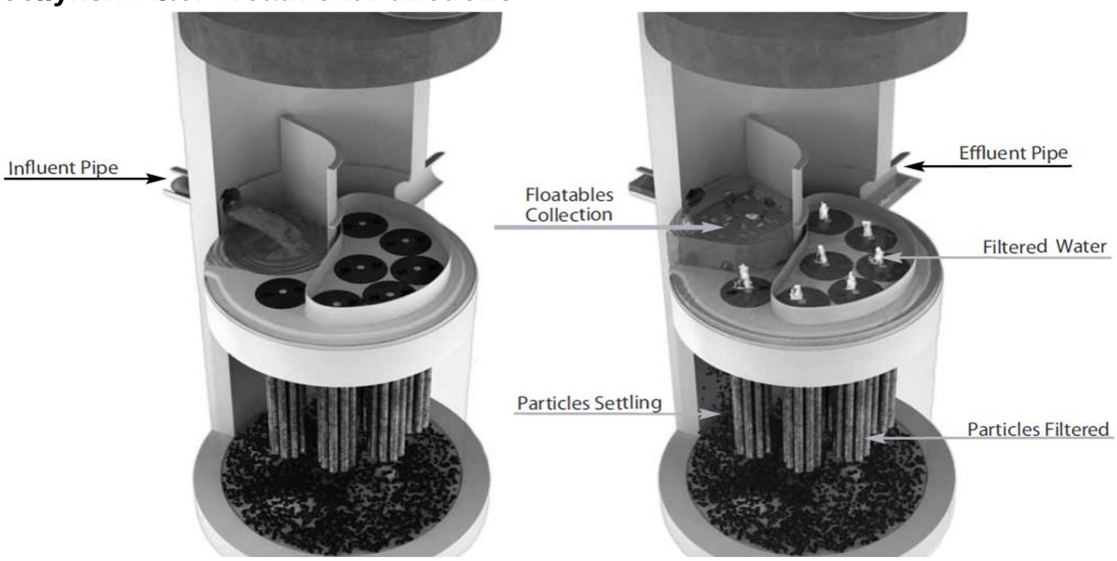
- ☑ 89% of the total suspended solids (TSS) load, including particles less than 5 microns
- ☑ 77% TP removal & 51% TN removal
- ☑ 90% Total Copper, 81% Total Lead, 70% Total Zinc
- ☑ Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons and bacteria
- ☑ Free oil, Floatable trash and debris

Field Proven Peformance

The Jellyfish filter has been field-tested on an urban site with 25 TAPE qualifying rain events and field monitored according to the TAPE field test protocol, demonstrating:

- A median TSS removal efficiency of 90%, and a median SSC removal of 99%;
- The ability to capture fine particles as indicated by an effluent d50 median of 3 microns for all monitotred storm events, and a median effluent turbidity of 5 NTUs;
- A median Total Phosphorus removal of 77%, and a median Total Nitrogen removal of 51%.

Jellyfish Filter Treatment Functions



Pre-treatment and Membrane Filtration

Jellyfish® Filter

Project Information

Date:	Friday, May 24, 2024
Project Name:	Kaufman Rd.
Project Number:	W22002
Location:	W22002

Designer Information

Company:	Candevcon Ltd.
Contact:	Shuchi Singh
Phone #:	

Notes

Rainfall

Name:	TORONTO CENTRAL
State:	ON
ID:	100
Record:	1982 to 1999
Co-ords:	45°30'N, 90°30'W

Drainage Area

Total Area:	1.73 ha
Imperviousness:	61%

Upstream Detention

Peak Release Rate:	n/a
Pretreatment Credit:	n/a

Design System Requirements

Flow Loading	90% of the Average Annual Runoff based on 18 years of TORONTO CENTRAL rainfall data:	**29.2 L/s**
Sediment Loading	Treating 90% of the average annual runoff volume, 6311 m³, with a suspended sediment concentration of 60 mg/L.	**379 kg**

Recommendation

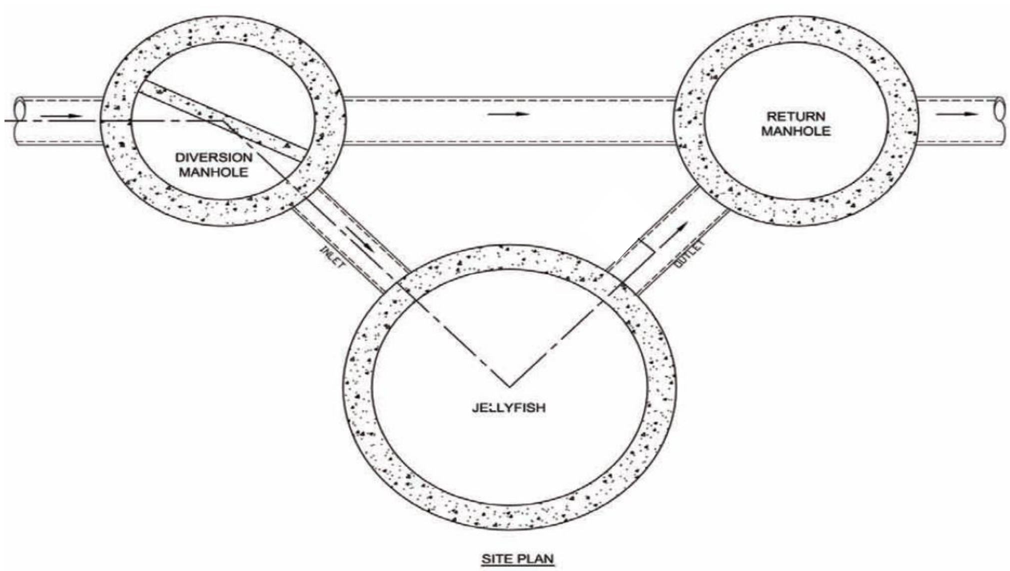
The Jellyfish Filter model JF8-6-2 is recommended to meet the water quality objective by treating a flow of 35.3 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 398 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Wet Vol Below Deck (L)	Sump Storage (m³)	Oil Capacity (L)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF4-1-1	1	1	1.2	2313	0.34	379	7.6	85
JF4-2-1	2	1	1.2	2313	0.34	379	12.6	142
JF6-3-1	3	1	1.8	5205	0.79	848	17.7	199
JF6-4-1	4	1	1.8	5205	0.79	848	22.7	256
JF6-5-1	5	1	1.8	5205	0.79	848	27.8	313
JF6-6-1	6	1	1.8	5205	0.79	848	28.6	370
JF8-6-2	**6**	**2**	**2.4**	**9252**	**1.42**	**1469**	**35.3**	**398**
JF8-7-2	7	2	2.4	9252	1.42	1469	40.4	455
JF8-8-2	8	2	2.4	9252	1.42	1469	45.4	512
JF8-9-2	9	2	2.4	9252	1.42	1469	50.5	569
JF8-10-2	10	2	2.4	9252	1.42	1469	50.5	626
JF10-11-3	11	3	3.0	14456	2.21	2302	63.1	711
JF10-12-3	12	3	3.0	14456	2.21	2302	68.2	768
JF10-12-4	12	4	3.0	14456	2.21	2302	70.7	796
JF10-13-4	13	4	3.0	14456	2.21	2302	75.7	853
JF10-14-4	14	4	3.0	14456	2.21	2302	78.9	910
JF10-15-4	15	4	3.0	14456	2.21	2302	78.9	967
JF10-16-4	16	4	3.0	14456	2.21	2302	78.9	1024
JF10-17-4	17	4	3.0	14456	2.21	2302	78.9	1081
JF10-18-4	18	4	3.0	14456	2.21	2302	78.9	1138
JF10-19-4	19	4	3.0	14456	2.21	2302	78.9	1195
JF12-20-5	20	5	3.6	20820	3.2	2771	113.6	1280
JF12-21-5	21	5	3.6	20820	3.2	2771	113.7	1337
JF12-22-5	22	5	3.6	20820	3.2	2771	113.7	1394
JF12-23-5	23	5	3.6	20820	3.2	2771	113.7	1451
JF12-24-5	24	5	3.6	20820	3.2	2771	113.7	1508
JF12-25-5	25	5	3.6	20820	3.2	2771	113.7	1565
JF12-26-5	26	5	3.6	20820	3.2	2771	113.7	1622
JF12-27-5	27	5	3.6	20820	3.2	2771	113.7	1679

Jellyfish® Filter

Jellyfish Filter Design Notes

- Typically the Jellyfish Filter is designed in an offline configuration, as all stormwater filter systems will perform for a longer duration between required maintenance services when designed and applied in off-line configurations. Depending on the design parameters, an optional internal bypass may be incorporated into the Jellyfish Filter, however note the inspection and maintenance frequency should be expected to increase above that of an off-line system. Speak to your local representative for more information.



Jellyfish Filter Typical Layout

- Typically, 18 inches (457 mm) of driving head is designed into the system, calculated as the difference in elevation between the top of the diversion structure weir and the invert of the Jellyfish Filter outlet pipe. Alternative driving head values can be designed as 12 to 24 inches (305 to 610mm) depending on specific site requirements, requiring additional sizing and design assistance.
- Typically, the Jellyfish Filter is designed with the inlet pipe configured 6 inches (150 mm) above the outlet invert elevation. However, depending on site parameters this can vary to an optional configuration of the inlet pipe entering the unit below the outlet invert elevation.
- The Jellyfish Filter can accommodate multiple inlet pipes within certain restrictions.
- While the optional inlet below deck configuration offers 0 to 360 degree flexibility between the inlet and outlet pipe, typical systems conform to the following:

Model Diameter (m)	Minimum Angle Inlet / Outlet Pipes	Minimum Inlet Pipe Diameter (mm)	Minimum Outlet Pipe Diameter (mm)
1.2	62º	150	200
1.8	59º	200	250
2.4	**52º**	**250**	**300**
3.0	48º	300	450
3.6	40º	300	450

- The Jellyfish Filter can be built at all depths of cover generally associated with conventional stormwater conveyance systems. For sites that require minimal depth of cover for the stormwater infrastructure, the Jellyfish Filter can be applied in a shallow application using a hatch cover. The general minimum depth of cover is 36 inches (915 mm) from top of the underslab to outlet invert.
- If driving head caclulations account for water elevation during submerged conditions the Jellyfish Filter will function effectively under submerged condtions.
- Jellyfish Filter systems may incorporate grated inlets depending on system configuration.
- For sites with water quality treatment flow rates or mass loadings that exceed the design flow rate of the largest standard Jellyfish Filter manhole models, systems can be designed that hydraulically connect multiple Jellyfish Filters in series or alternatively Jellyfish Vault units can be designed.

STANDARD SPECIFICATION
STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

Specifies requirements for construction and performance of an underground stormwater quality membrane filtration treatment device that removes pollutants from stormwater runoff through the unit operations of sedimentation, floatation, and membrane filtration.

1.2 REFERENCE STANDARDS

ASTM C 891: Specification for Installation of Underground Precast Concrete Utility Structures
ASTM C 478: Specification for Precast Reinforced Concrete Manhole Sections
ASTM C 443: Specification for Joints for Concrete Pipe and Manholes, Using Rubber Gaskets
ASTM D 4101: Specification for Copolymer steps construction

CAN/CSA-A257.4-M92
Joints for Circular Concrete Sewer and Culvert Pipe, Manhole Sections and Fittings Using Rubber Gaskets

CAN/CSA-A257.4-M92
Precast Reinforced Circular Concrete Manhole Sections, Catch Basins and Fittings

Canadian Highway Bridge Design Code

1.3 SHOP DRAWINGS

Shop drawings for the structure and performance are to be submitted with each order to the contractor. Contractor shall forward shop drawing submittal to the consulting engineer for approval. Shop drawings are to detail the structure's precast concrete and call out or note the fiberglass (FRP) internals/components.

1.4 PRODUCT SUBSTITUTIONS

No product substitutions shall be accepted unless submitted 10 days prior to project bid date, or as directed by the engineer of record. Submissions for substitutions require review and approval by the Engineer of Record, for hydraulic performance, impact to project designs, equivalent treatment performance, and any required project plan and report (hydrology/hydraulic, water quality, stormwater pollution) modifications that would be required by the approving jurisdictions/agencies. Contractor to coordinate with the Engineer of Record any applicable modifications to the project estimates of cost, bonding amount determinations, plan check fees for changes to approved documents, and/or any other regulatory requirements resulting from the product substitution.

1.5 HANDLING AND STORAGE

Prevent damage to materials during storage and handling.

PART 2 – PRODUCTS

2.1 GENERAL

2.1.1 The device shall be a cylindrical or rectangular, all concrete structure (including risers), constructed from precast concrete riser and slab components or monolithic precast structure(s), installed to conform to ASTM C 891 and to any required state highway, municipal or local specifications; whichever is more stringent. The device shall be watertight.

2.1.2 Cartridge Deck The cylindrical concrete device shall include a fiberglass deck. The rectangular concrete device shall include a coated aluminum deck. In either instance, the insert shall be bolted and sealed watertight inside the precast concrete chamber. The deck shall serve as: (a) a horizontal divider between the lower treatment zone and the upper treated effluent zone; (b) a deck for attachment of filter cartridges such that the membrane filter elements of each cartridge extend into the lower treatment zone; (c) a platform for maintenance workers to service the filter cartridges (maximum manned weight = 450 pounds (204 kg)); (d) a conduit for conveyance of treated water to the effluent pipe.

2.1.3 Membrane Filter Cartridges Filter cartridges shall be comprised of reusable cylindrical membrane filter elements connected to a perforated head plate. The number of membrane filter elements per cartridge shall be a minimum of eleven 2.75-inch (70-mm) diameter elements. The length of each filter element shall be a minimum 15 inches (381 mm). Each cartridge shall be fitted into the cartridge deck by insertion into a cartridge receptacle that is permanently mounted into the cartridge deck. Each cartridge shall be secured by a cartridge lid that is threaded onto the receptacle, or similar mechanism to secure the cartridge into the deck. The maximum treatment flow rate of a filter cartridge shall be controlled by an orifice in the cartridge lid, or on the individual cartridge itself, and based on a design flux rate (surface loading rate) determined by the maximum treatment flow rate per unit of filtration membrane surface area. The maximum design flux rate shall be 0.21 gpm/ft^2 (0.142 lps/m^2).

Each membrane filter cartridge shall allow for manual installation and removal. Each filter cartridge shall have filtration membrane surface area and dry installation weight as follows (if length of filter cartridge is between those listed below, the surface area and weight shall be proportionate to the next length shorter and next length longer as shown below):

Filter Cartridge Length (in / mm)	Minimum Filtration Membrane Surface Area (ft2 / m2)	Maximum Filter Cartridge Dry Weight (lbs / kg)
15	106 / 9.8	10.5 / 4.8
27	190 / 17.7	15.0 / 6.8
40	282 / 26.2	20.5 / 9.3
54	381 / 35.4	25.5 / 11.6

2.1.4 Backwashing Cartridges The filter device shall have a weir extending above the cartridge deck, or other mechanism, that encloses the high flow rate filter cartridges when placed in their respective cartridge receptacles within the cartridge deck. The weir, or other mechanism, shall collect a pool of filtered water during inflow events that backwashes the high flow rate cartridges when the inflow event subsides. All filter cartridges and membranes shall be reusable and allow for the use of filtration membrane rinsing procedures to restore flow capacity and sediment capacity; extending cartridge service life.

2.1.5 Maintenance Access to Captured Pollutants The filter device shall contain an opening(s) that provides maintenance access for removal of accumulated floatable pollutants and sediment, removal of and replacement of filter cartridges, cleaning of the sump, and rinsing of the deck. Access shall have a minimum clear vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

2.1.6 Bend Structure The device shall be able to be used as a bend structure with minimum angles between inlet and outlet pipes of 90-degrees or less in the stormwater conveyance system.

2.1.7 Double-Wall Containment of Hydrocarbons The cylindrical precast concrete device shall provide double-wall containment for hydrocarbon spill capture by a combined means of an inner wall of fiberglass, to a minimum depth of 12 inches (305 mm) below the cartridge deck, and the precast vessel wall.

2.1.8 Baffle The filter device shall provide a baffle that extends from the underside of the cartridge deck to a minimum length equal to the length of the membrane filter elements. The baffle shall serve to protect the membrane filter elements from contamination by floatables and coarse sediment. The baffle shall be flexible and continuous in cylindrical configurations, and shall be a straight concrete or aluminum wall in rectangular configurations.

2.1.9 Sump The device shall include a minimum 24 inches (610 mm) of sump below the bottom of the cartridges for sediment accumulation, unless otherwise specified by the design engineer. Depths less than 24 inches may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

2.2 PRECAST CONCRETE SECTIONS

All precast concrete components shall be manufactured to a minimum live load of HS-20 truck loading or greater based on local regulatory specifications, unless otherwise modified or specified by the design engineer, and shall be watertight.

2.3 JOINTS All precast concrete manhole configuration joints shall use nitrile rubber gaskets and shall meet the requirements of ASTM C443, Specification C1619, Class D or engineer approved equal to ensure oil resistance. Mastic sealants or butyl tape are not an acceptable alternative.

2.4 GASKETS Only profile neoprene or nitrile rubber gaskets in accordance to CSA A257.3-M92 will be accepted. Mastic sealants, butyl tape or Conseal CS-101 are not acceptable gasket materials.

2.5 FRAME AND COVER Frame and covers must be manufactured from cast-iron or other composite material tested to withstand H-20 or greater design loads, and as approved by the local regulatory body. Frames and covers must be embossed with the name of the device manufacturer or the device brand name.

2.6 DOORS AND HATCHES If provided shall meet designated loading requirements or at a minimum for incidental vehicular traffic.

2.7 CONCRETE All concrete components shall be manufactured according to local specifications and shall meet the requirements of ASTM C 478.

2.8 FIBERGLASS The fiberglass portion of the filter device shall be constructed in accordance with the following standard: ASTM D-4097: Contact Molded Glass Fiber Reinforced Chemical Resistant Tanks.

2.9 STEPS Steps shall be constructed according to ASTM D4101 of copolymer polypropylene, and be driven into preformed or pre-drilled holes after the concrete has cured, installed to conform to applicable sections of state, provincial and municipal building codes, highway, municipal or local specifications for the construction of such devices.

2.10 INSPECTION All precast concrete sections shall be inspected to ensure that dimensions, appearance and quality of the product meet local municipal specifications and ASTM C 478.

PART 3 – PERFORMANCE

3.1 GENERAL

3.1.1 Verification – The stormwater quality filter must be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV).

3.1.2 Function - The stormwater quality filter treatment device shall function to remove pollutants by the following unit treatment processes; sedimentation, floatation, and membrane filtration.

3.1.3 Pollutants - The stormwater quality filter treatment device shall remove oil, debris, trash, coarse and fine particulates, particulate-bound pollutants, metals and nutrients from stormwater during runoff events.

3.1.4 Bypass - The stormwater quality filter treatment device shall typically utilize an external bypass to divert excessive flows. Internal bypass systems shall be equipped with a floatables baffle, and must avoid passage through the sump and/or cartridge filtration zone.

3.1.5 Treatment Flux Rate (Surface Loading Rate) – The stormwater quality filter treatment device shall treat 100% of the required water quality treatment flow based on a maximum design treatment flux rate (surface loading rate) across the membrane filter cartridges of 0.21 gpm/ft² (0.142 lps/m²).

3.2 FIELD TEST PERFORMANCE

At a minimum, the stormwater quality filter device shall have been field tested and verified with a minimum 25 TARP qualifying storm events and field monitoring shall have been conducted according to the TARP 2009 NJDEP TARP field test protocol, and have received NJCAT verification.

3.2.1 Suspended Solids Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median TSS removal efficiency of 85% and a minimum median SSC removal efficiency of 95%.

3.2.2 Runoff Volume – The stormwater quality filter treatment device shall be engineered, designed, and sized to treat a minimum of 90 percent of the annual runoff volume determined from use of a minimum 15-year rainfall data set.

3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, an effluent d_{50} of 15 microns or lower for all monitored storm events.

3.2.4 Turbidity Reduction - The stormwater quality filter treatment device shall have demonstrated the ability to reduce the turbidity from influent from a range of 5 to 171 NTU to an effluent turbidity of 15 NTU or lower.

3.2.5 Nutrient (Total Phosphorus & Total Nitrogen) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Phosphorus removal of 55%, and a minimum median Total Nitrogen removal of 50%.

3.2.6 Metals (Total Zinc & Total Copper) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Zinc removal of 55%, and a minimum median Total Copper removal of 85%.

3.3 INSPECTION and MAINTENANCE

The stormwater quality filter device shall have the following features:

3.3.1 Durability of membranes are subject to good handling practices during inspection and maintenance (removal, rinsing, and reinsertion) events, and site specific conditions that may have heavier or lighter loading onto the cartridges, and pollutant variability that may impact the membrane structural integrity. Membrane maintenance and replacement shall be in accordance with manufacturer's recommendations.

3.3.2 Inspection which includes trash and floatables collection, sediment depth determination, and visible determination of backwash pool depth shall be easily conducted from grade (outside the structure).

3.3.3 Manual rinsing of the reusable filter cartridges shall promote restoration of the flow capacity and sediment capacity of the filter cartridges, extending cartridge service life.

3.3.4 The filter device shall have a minimum 12 inches (305 mm) of sediment storage depth, and a minimum of 12 inches between the top of the sediment storage and bottom of the filter cartridge tentacles, unless otherwise specified by the design engineer. Variances may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

3.3.5 Sediment removal from the filter treatment device shall be able to be conducted using a standard maintenance truck and vacuum apparatus, and a minimum one point of entry to the sump that is unobstructed by filter cartridges.

3.3.6 Maintenance access shall have a minimum clear height that provides suitable vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

3.3.7 Filter cartridges shall be able to be maintained without the requirement of additional lifting equipment.

PART 4 – EXECUTION

4.1 INSTALLATION

4.1.1 PRECAST DEVICE CONSTRUCTION SEQUENCE

The installation of a watertight precast concrete device should conform to ASTM C 891 and to any state highway, municipal or local specifications for the construction of manholes, whichever is more stringent. Selected sections of a general specification that are applicable are summarized below.

4.1.1.1 The watertight precast concrete device is installed in sections in the following sequence:

- aggregate base
- base slab
- treatment chamber and cartridge deck riser section(s)
- bypass section
- connect inlet and outlet pipes
- concrete riser section(s) and/or transition slab (if required)
- maintenance riser section(s) (if required)
- frame and access cover

4.1.2 The precast base should be placed level at the specified grade. The entire base should be in contact with the underlying compacted granular material. Subsequent sections, complete with joint seals, should be installed in accordance with the precast concrete manufacturer's recommendations.

4.1.3 Adjustment of the stormwater quality treatment device can be performed by lifting the upper sections free of the excavated area, re-leveling the base, and re-installing the sections. Damaged sections and gaskets should be repaired or replaced as necessary to restore original condition and watertight seals. Once the stormwater quality treatment device has been constructed, any/all lift holes must be plugged watertight with mortar or non-shrink grout.

4.1.4 Inlet and Outlet Pipes Inlet and outlet pipes should be securely set into the device using approved pipe seals (flexible boot connections, where applicable) so that the structure is watertight, and such that any pipe intrusion into the device does not impact the device functionality.

4.1.5 Frame and Cover Installation Adjustment units (e.g. grade rings) should be installed to set the frame and cover at the required elevation. The adjustment units should be laid in a full bed of mortar with successive units being joined using sealant recommended by the manufacturer. Frames for the cover should be set in a full bed of mortar at the elevation specified.

4.2 MAINTENANCE ACCESS WALL
In some instances the Maintenance Access Wall, if provided, shall require an extension attachment and sealing to the precast wall and cartridge deck at the job site, rather than at the precast facility. In this instance, installation of these components shall be performed according to instructions provided by the manufacturer.

4.3 FILTER CARTRIDGE INSTALLATION Filter cartridges shall be installed in the cartridge deck only after the construction site is fully stabilized and in accordance with the manufacturer's guidelines and recommendations. Contractor to contact the manufacturer to schedule cartridge delivery and review procedures/requirements to be completed to the device prior to installation of the cartridges and activation of the system.

PART 5 – QUALITY ASSURANCE

5.1 FILTER CARTRIDGE INSTALLATION Manufacturer shall coordinate delivery of filter cartridges and other internal components with contractor. Filter cartridges shall be delivered and installed complete after site is stabilized and unit is ready to accept cartridges. Unit is ready to accept cartridges after is has been cleaned out and any standing water, debris, and other materials have been removed. Contractor shall take appropriate action to protect the filter cartridge receptacles and filter cartridges from damage during construction, and in accordance with the manufacturer's recommendations and guidance. For systems with cartridges installed prior to full site stabilization and prior to system activation, the contractor can plug inlet and outlet pipes to prevent stormwater and other influent from entering the device. Plugs must be removed during the activation process.

5.2 INSPECTION AND MAINTENANCE

5.2.1 The manufacturer shall provide an Owner's Manual upon request.

5.2.2 After construction and installation, and during operation, the device shall be inspected and cleaned as necessary based on the manufacturer's recommended inspection and maintenance guidelines and the local regulatory agency/body.

5.3 REPLACEMENT FILTER CARTRIDGES When replacement membrane filter elements and/or other parts are required, only membrane filter elements and parts approved by the manufacturer for use with the stormwater quality filter device shall be installed.

END OF SECTION

STANDARD PERFORMANCE SPECIFICATION
STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

This section specifies requirements for selecting, sizing, and designing an underground stormwater quality membrane filtration treatment device that removes pollutants from stormwater runoff through the unit operations of sedimentation, floatation, and membrane filtration.

1.2 REFERENCE STANDARDS & PROCEDURES

ISO 14034:2016 Environmental Management – Environmental Technology Verification (ETV)

1.3 SUBMITTALS

1.3.1 All submittals, including sizing reports & shop drawings, shall be submitted upon request with each order to the contractor then forwarded to the Engineer of Record for review and acceptance. Shop drawings shall detail all OGS components, elevations, and sequence of construction.

1.3.2 Alternative devices shall have features identical to or greater than the specified device, including: filtration surface area, treatment chamber diameter, treatment chamber wet volume, sediment storage volume, and oil storage volume.

1.3.3 Unless directed otherwise by the Engineer of Record, filtration treatment device product substitutions or alternatives submitted within ten days prior to project bid shall not be accepted. All alternatives or substitutions submitted shall be signed and sealed by a local registered Professional Engineer, based on the exact same criteria detailed in Section 3, in entirety, subject to review and approval by the Engineer of Record.

PART 2 – PRODUCTS

2.1 GENERAL

2.1.1 Maintenance Access to Captured Pollutants The filter device shall contain an opening(s) that provides maintenance access for removal of accumulated floatable pollutants and sediment, removal of and replacement of filter cartridges, cleaning of the sump, and rinsing of the internal components. Access shall have a minimum clear vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of their installed placement for the entire length of the cartridge.

2.1.2 Pollutant Storage: The Filter device shall include a sump for sediment storage, and a protected volume for the capture and storage of petroleum hydrocarbons and buoyant gross pollutants.

PART 3 – PERFORMANCE

3.1 GENERAL

3.1.1 Verification – The stormwater quality filter treatment device shall have been field tested in accordance with either TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements – Amendments to TARP Tier II Protocol (NJDEP, 2009) or Washington State Technology Assessment Protocol – Ecology (TAPE), 2011 or later version. The field test shall have been verified in accordance with ISO 14034:2016 Environmental Management – Environmental Technology Verification (ETV). See Section 3.2 of this specification for field test performance requirements.

3.2 FIELD TEST PERFORMANCE

The field test (as specified in section 3.1.1)shall have monitored a minimum of twenty (20) TARP or TAPE qualifying storm events, and report at **minimum** the following results:

3.2.1 Suspended Solids Removal - The stormwater quality filter treatment device shall have ISO 14034 ETV verified load based median TSS removal efficiency of at least 85% and load based median SSC removal efficiency of at least 98%.

3.2.2 Runoff Volume – The stormwater quality filter treatment device shall be engineered, designed, and sized to treat a minimum of 90 percent of the annual runoff volume determined from use of a minimum 15-year rainfall data set.

3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, and an effluent d_{50} of 15 microns or lower for all monitored storm events.

3.2.4 Turbidity Reduction - The stormwater quality filter treatment device shall have demonstrated the ability to reduce turbidity such that effluent turbidity is 15 NTU or lower.

3.2.5 Nutrients & Metals – The stormwater quality filter treatment device shall have ISO 14034 ETV Verified minimum load based removal efficiencies for the following:

3.2.5.1 Total Phosphorus (TP) Removal - Median TP removal efficiency of at least 49%.

3.2.5.2 Total Nitrogen (TN) Removal - Median TN removal efficiency of at least 39%.

3.2.5.3 Total Zinc (Zn) Removal - Median Zn removal efficiency of at least 69%.

3.2.5.4 Total Copper (Cu) Removal - Median Cu removal efficiency of at least 91%.

END OF SECTION

STANDARD PERFORMANCE SPECIFICATION
STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

This section specifies requirements for selecting, sizing, and designing an underground stormwater quality membrane filtration treatment device that removes pollutants from stormwater runoff through the unit operations of sedimentation, floatation, and membrane filtration.

1.2 REFERENCE STANDARDS & PROCEDURES

ISO 14034:2016 Environmental Management – Environmental Technology Verification (ETV)

1.3 SUBMITTALS

1.3.1 All submittals, including sizing reports & shop drawings, shall be submitted upon request with each order to the contractor then forwarded to the Engineer of Record for review and acceptance. Shop drawings shall detail all OGS components, elevations, and sequence of construction.

1.3.2 Alternative devices shall have features identical to or greater than the specified device, including: filtration surface area, treatment chamber diameter, treatment chamber wet volume, sediment storage volume, and oil storage volume.

1.3.3 Unless directed otherwise by the Engineer of Record, filtration treatment device product substitutions or alternatives submitted within ten days prior to project bid shall not be accepted. All alternatives or substitutions submitted shall be signed and sealed by a local registered Professional Engineer, based on the exact same criteria detailed in Section 3, in entirety, subject to review and approval by the Engineer of Record.

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PART 3 – PERFORMANCE

3.1 GENERAL

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3.2 FIELD TEST PERFORMANCE

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3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, and an effluent d_{50} of 15 microns or lower for all monitored storm events.

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3.2.5.3 Total Zinc (Zn) Removal - Median Zn removal efficiency of at least 69%.

3.2.5.4 Total Copper (Cu) Removal - Median Cu removal efficiency of at least 91%.

END OF SECTION

PHOSPHORUS LOADING
&
WATER BALANCE
CALCULATIONS

Complete Report is attached in Appendix"F"

through a consideration of channel roughness (substrate material), the critical depth and velocity required to entrain substrate materials, and the average cross-sectional shape of the channel.

In Table 2.1 it is evident that although Tributary 4 flows into Tributary 3, the discharge of the former is larger than that of the latter. Such an observation is counter-intuitive. A closer examination of the field data and of the physical setting, however, reveals that it is warranted. The gradient and substrate materials in reach 4 are greater than in reach 3. Further, some water is diverted from Tributary 3 at its upstream end. Similarly, although the cross-sectional dimensions of reaches 2 and 4 are very similar, any quantitative measure or estimate of discharge at these sites differs substantially. Again, the observed trend may be accounted for by properties of the substrate and channel gradient.

2.7 Water Quality

Water quality data examined in Phases I and II of the Inglewood Village Study indicate that for most parameters, the water quality of the Credit River in the vicinity of Inglewood is quite good. The river however is classified as Policy 2 (Provincial Water Quality Objectives are currently exceeded and no further degradation will be permitted) on the basis of Total Phosphorus and bacteria. The bacteria objective is based upon body-contact recreation. The phosphorus objective is actually a guideline and should be assessed using other data sources where possible (eg. dissolved oxygen variation, fish and benthic communities). Diurnal oxygen studies, completed as part of the Inglewood Village Study, indicate no significant dissolved oxygen depression during the night. This, together with the presence of spawning redds in the area suggest that phosphorus is probably not having a major impact on the Credit River locally. Never the less, frequency of phosphorus exceedences should be taken as a warning and all practical efforts should be made to limit phosphorus inputs.

Table 2. Land-Use Specific Phosphorus Export Coefficients (kg/ha/yr) for Lake Simcoe Subwatersheds

Subwatershed	Phosphorus Export (kg/ha/yr)											
	Cropland	Hay-Pasture	Sod Farm/Golf Course	High Intensity Development: Commercial /Industrial	High Intensity Development: Residential	Low Intensity Development	Quarry	Unpaved Road	Forest	Transition	Wetland	Open Water
Monitored Subwatersheds												
Beaver River	0.22	0.04	0.01	1.82	1.32	0.19	0.06	0.83	0.02	0.04	0.02	0.26
Black River	0.23	0.08	0.02	1.82	1.32	0.17	0.15	0.83	0.05	0.06	0.04	0.26
East Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hawkestone Creek	0.19	0.10	0.06	1.82	1.32	0.09	0.10	0.83	0.03	0.04	0.03	0.26
Lovers Creek	0.16	0.07	0.17	1.82	1.32	0.07	0.06	0.83	0.06	0.06	0.05	0.26
Pefferlaw/Uxbridge Brook	0.11	0.06	0.02	1.82	1.32	0.13	0.04	0.83	0.03	0.04	0.04	0.26
Whites Creek	0.23	0.10	0.42	1.82	1.32	0.15	0.08	0.83	0.10	0.11	0.09	0.26
Unmonitored Subwatersheds												
Barrie Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
GeorginaCreeks	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hewitts Creek	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Innisfil Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Maskinonge River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Oro Creeks North	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Oro Creeks South	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Ramara Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Talbot/Upper Talbot River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
West Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26

Table 3. Phosphorus Removal Efficiencies for Major Classes of BMPs Using the Decision Tree (Figure 5).

BMP Class	Reference IDs[1]	Reported Phosphorus Removal Efficiency (%)		Relevant to Ontario?	Range <40%?	Are Non-Ontario values acceptable?	Possible design criteria?	Median % Removal Efficiency
		Min	Max					
Post-development BMPs								
Bioretention Systems	8-10, 12,13, 34-38, 40	-1552	80	no	no	no	No	none
Constructed Wetlands	104, 106, 109	72	87	yes	yes			77
Dry Detention Ponds	104, 109	0	20	no	yes	yes		10
Dry Swales	24, 26-32	-216	94	no	no	no	possible	none
Enhanced Grass/Water Quality Swales	21, 104	34	55	no	yes	no	No	none
Flow Balancing Systems	106	77		no	?	yes	Min data	77
Green Roofs	2	-248		no	no	no	No	none
Hydrodynamic Devices	109	-8		no	?	yes		none
Perforated Pipe Infiltration/Exfiltration Systems	7, 4	81	93	yes	yes			87
Sand or Media Filters	104, 109	30	59	no	yes	yes		45
Soakaways - Infiltration Trenches	6, 104	50	70	no	yes	yes		60
Sorbtive Media Interceptors	111	78	80	no	yes	yes		79
Underground Storage	106	25		no	?	yes	Min data	25
Vegetated Filter Strips/Stream Buffers	6, 42, 104	60	70	no	yes	yes	Yes	65
Wet Detention Ponds	104-106, 109	42	85	yes	yes			63

Notes: [1]References associated with IDs are provided in Appendix 7.

CANDEVCON LIMITED
CONSULTING ENGINEERS & PLANNERS

GTA WEST OFFICE (CORPORATE)
9358 GOREWAY DRIVE
BRAMPTON, ONTARIO L6P 0M7
T: (905) 794-0600 F: (905) 794-0611

PROJECT NO. : W22002
PROJECT NAME : 15544 McLaughlin, Caledon
PREPARED BY : S.S
CHECKED BY : DKH
DATE : 5/22/2024

Phosphorus Budget Tool in Support of Sustainable Development

NOTE : In the absence of a references available from CVC for phosphorus removal calculations, analysis is conducted using Phosphorus Budget Tool in Support of Sustainable Development for the **Lake Simcoe Watershed (Prepared for Ontario Ministry of the Environment)** . Attached reference documents are provided for review purposes.

(Pre-Development Scenario)

Existing Site Condition:

- At present the site is vacant and covered with grass.
- Refer to Pre-Development Plan attached for Existing Site Condition

Pre-Development Condition (Overall Site)

	Residential	Commercial /Industrial /Institutional	Hay & Pasture Fields	Forest
Phosphorus Export (kg/ha/year)	**1.32**	**1.82**	**0.12**	**0.10**
Area (Ha)	0.00	0.000	1.76	0
Total P (kg/Year)	0.00	0.000	0.21	0
	Total Pre-Development P (kg/yr) =		**0.21**	

Phosphorous Export values are taken from Table 2 of the reference report.

(Post-Development Scenario)

Proposed Site Condition:

- Under the proposed condition, the subject site will be built with Single Family Residential Lots
- The land use of the proposed development will fall under residential for P Coefficient
- Refer to Storm Drainage Area Plan for the proposed development plan

Post-Development Condition (Catchment Area) with NO BMP

	Residential	Commercial /Industrial /Institutional	Hay & Pasture Fields	Forest
Phosphorus Export (kg/ha/year)	**1.32**	**1.82**	**0.12**	**0.10**
Untreated Area (Ha)	1.73	0.00	0.00	0.00
Total (P) Kg/yr.	2.28	0.00	0.00	0.00

Total Post-Development (without BMPs) P = 2.28 Kg/Yr

Post-Development Condition (Uncontrolled Runoff)

Untreated Areas (Draining directly to Valley)

	Residential	Commercial/Industrial /Institutional	Hay & Pasture Fields	Forest
Phosphorus Export (kg/ha/year)	**1.32**	**1.82**	**0.12**	**0.10**
Untreated Area (Ha)	0.03	0.00	0.00	0.00
Total (P) Kg/yr.	0.04	0.00	0.00	0.00

Total Post-Development (without BMPs) P = 0.04 Kg/Yr

Post-Development Condition (With BMPS)

1. Areas draining through Filtration Treatment (Jellyfish Unit) :

Areas to (Jellyfish Unit)

	Residential	Institutional	Pasture	Forest
Export (kg/ha/year)	**1.32**	**1.82**	**0.12**	**0.10**
Area (ha)	**1.73**	0.00	0.0	0.0
Total (P) Kg/yr.	**2.28**	0.00	0.0	0.0

(Jellyfish)Treatment

Jellyfish Proficiency (%):	77.5	(As per ETV Statement)
P Removed (kg):	1.77	
P Remaining (kg):	**0.51**	(2.28 - 1.77)

2. Areas draining through Dry Pond :

Areas to (Dry Pond)

	Residential	Institutional	Pasture	Forest
Export (kg/ha/year)	**1.32**	**1.82**	**0.12**	**0.10**
Area (ha)	**1.73**	0.00	0.0	0.0
Total (P) Kg/yr.	**2.28**	0.00	0.0	0.0

(Dry Pond)Treatment

Dry Pond Proficiency (%):	10	(As per LSRCA Reference Doc)
P Removed (kg):	0.23	
P Remaining (kg):	**0.29**	(0.51 - 0.23)

Total Pre-Development P Load =	**0.21**	**Kg/Yr**
Total Post-Development Load (without BMPs) =	**2.28**	**Kg/Yr**
Total Post-Development Load (with BMPs) =	**0.29**	**Kg/Yr**
Uncontrolled Runoff Load =	0.04	Kg/Yr
Total P Load from Site =	**0.33**	**Kg/Yr (0.29 + 0.06)**

*Phosphorus Rate reduces up to **86%** with the use of BMPs.*

WATER BALANCE CALCULATION

THORNTHWAITE WATER BALANCE MODEL

Location :	TORONTO LESTER B. PEARSON INT'L A * ONTARIO	**Elevation :**	173.40 m
Latitude :	43°40'38.000" N	**Climate Id:**	6158733
Longitude :	79°37'50.000" W	**WMO Id:**	71624

CLIMATE NORMAL 1981 - 2010 CANADIAN CLIMATE NORMALS STATION DATA (TORONTO)

Site Climatic conditions were calculated using the "Thornthwaite Method" utilizing meterological data available from Environmental Canada historical weather data for Toronto Lester B. Pearson Int'l A. Pearson Station is within the closest proximity of Site

Month	Mean Temperature (°C)	Heat Index* (i)	Potential Evapotranspiration (PET) (mm)	Daylight Correction Value for Toronto	Adjusted Evapotranspiration (mm)	Total Precipitation (mm)	Water Surplus (mm)
January	-5.5	0	0	0.81	0.0	51.8	51.8
February	-4.5	0	0	0.81	0.0	47.7	47.7
March	0.1	0	0	1.02	0.0	49.8	49.8
April	7.1	1.7	30.9	1.12	34.6	68.5	33.9
May	13.1	4.2	61.4	1.27	78.0	74.3	0.0
June	18.6	7.2	91.0	1.29	117.3	71.5	0.0
July	21.5	8.9	107.0	1.3	139.1	75.7	0.0
August	20.6	8.4	102.0	1.2	122.4	78.1	0.0
September	16.2	5.8	77.9	1.04	81.0	74.5	0.0
October	9.5	2.6	42.8	0.95	40.7	61.1	20.4
November	3.7	0.6	14.9	0.8	11.9	75.1	63.2
December	-2.2	0	0	0.74	0	57.9	57.9
	Overall (I) =	**39.47**			**625.1**	**786**	**324.7**
	a =	**1.12**		**Total Water Surplus =**	**160.9**	**mm/year**	

NOTE : PET estimation is obtained for each month, considering a month is 30 days long and there are 12 theoretical sunshine hours per day.

Where,

Potential evapotranspiration, PET (cm/month) = $1.6\ (L/12)\ (10T_a/I)^{a}$

Where: L is the average day length

T_a is the average daily temperature

$I = \Sigma\ (T_a/5)^{1.5}$ **(HEAT INDEX)**

$a = (6.75 \times 10^{-7})\ I^3 - (7.71 \times 10^{-5})\ I^2 + (1.792 \times 10^{-2})\ I + 0.49$

1. Climate Information :

Precipitation =	786 mm/yr
Evapotranspiration =	625.1 mm/yr
Water Surplus =	160.9 mm/yr

2. Infiltration Rates :

" Infiltration Rates were determined based on infiltration guidelines sub-factors provided within the "MOEE Hydrogeological Technical Information required for Land Development Application" - Refer to MOEE, 1995 - Chapter-4,Table 2 & 3 (Page - 4-63)

Table 2 : Infiltration Factors (from MOEE)

Description of Area/Development of Site	Value of Infiltration Factor
TOPOGRAPHY	
Flat land, average slope not exceeding 0.6m per km	0.30
Rolling land, average slope of 2.8m to 3.8m to 3.8m per km	0.20
Hilly land, average slope of 28m to 47m per km	0.10
SOIL	
Tight impervious clay	0.10
Medium Combination of Clay and Loam	0.20
Open and Sandy Loam	0.40
COVER	
Cultivated Lands	0.10
Woodland	0.20
Factor Total =	**0.60**

The Values for Infiltration Factor are selected based on the results from Geotechnical Report prepared by Soil Eng. The report describes existing Site and Soil Characterstics.

Infiltration per year =	(0.60 x 160.9) =	**96.6**	**mm/yr**
Runoff per year =	(160.9 - 96.6) =	**64.4**	**mm/yr**

WATER BUDGET - PRE-DEVELOPMENT CONDITION

Total Area of Development = **1.63 Hectares (Excludes External Drainage)**

NOTE : Subject Site consisted of Weed-covered lot and treelines along east and north boundaries

Catchment Designation (Land Use)	Site Grassed Areas	Site Paved Surfaces	Site Total	
Pervious Areas (m^2)	16,300	0	16,300	(Site is 100% pervious)
Impervious Areas (m^2)	0	0	0	
Infiltration Factors				
Topography Infiltration Factor	0.30	0.30		(From MOE Table 3.1 for Flat lands)
Soil Infiltration Factor	0.20	0.20		(From MOE Table 3.1 for Medium combinations of clay and loam)
Land Cover Infiltration Factor	0.10	0.00		
MOE Infiltration Factor	0.60	0		
Actual Infiltration Factor	0.60	0.0		
Run-off Coefficient	0.40	1.0		
Run-off from Impervious Surfaces*	0	0.80		* Evaporation from impervious areas was assumed to be 20% of precipitation
Inputs (per Unit Area)				
Precipitation (mm/yr)	786.0	786.0	786.0	
Run-On (mm/yr)	0	0	0	
Other Inputs (mm/yr)	0	0	0	
Total Inputs (mm/yr)	**786.0**	**786.0**	**786.0**	
Outputs (per Unit Area)				
Precipitation Surplus (mm/yr)	160.9	628.8	160.9	
Net Surplus (mm/yr)	160.9	628.8	160.9	
Evapotranspiration (mm/yr)	625.1	157.2	625.1	
Infiltration (mm/yr)	96.6	0	96.6	
Roof Infiltration (mm/yr)	0	0	0	
Total Infiltration (mm/yr)	96.6	0	96.6	
Runoff Pervious Areas	64.4	0	64.4	
Runoff Impervious Areas	0	628.8	0.0	
Total Runoff (mm/yr)	64.4	628.8	64.4	
Total Output (mm/yr)	**786.0**	**786.0**	**786.0**	
Difference (Input - Output) =	**0.0**	**0.0**	**0.0**	
Input (Volumes)				
Precipitation (m^3/yr)	12811.8	0.0	12811.8	
Run-on (m^3/yr)	0	0	0	
Other Inputs (m^3/yr)	**12811.8**	**0.0**	**12811.8**	
Output (Volumes)				
Precipitation Surplus (m^3/yr)	2623.2	0.0	2623.2	
Net Surplus (m^3/yr)	2623.2	0.0	2623.2	
Evapotranspiration (m^3/yr)	10188.6	0.0	10188.6	
Infiltration (m^3/yr)	1573.9	0	1573.9	
Roof Infiltration (m^3/yr)	0	0	0	
Total Infiltration (m^3/yr)	1573.9	0.0	1573.9	
Runoff Pervious Areas (m^3/yr)	1049.3	0	1049.3	
Runoff Impervious Areas (m^3/yr)	0	0.0	0.0	
Total Runoff (m^3/yr)	1049.3	0.0	1049.3	
Total Output (m^3/yr)	**12811.8**	**0.0**	**12811.8**	
Difference (Input - Output) =	0.0	0.0	0.0	

WATER BUDGET - POST-DEVELOPMENT CONDITION (WITH NO INFILTRATION)

Total Area of Subdivision Development = **1.63 Hectares**
(Refer to Storm Drainage Area Plans prepared by Candevcon)

Catchment Designation (Land Use)	Site		
	Parks/Green Spaces	Paved Surface	Total
Pervious Areas (m^2)	7,030	0	7,030
Impervious Areas (m^2)	0	9,279	9,279
Infiltration Factors			
Topography Infiltration Factor	0.30	0.30	
Soil Infiltration Factor	0.20	0.20	
Land Cover Infiltration Factor	0.10	0.00	
MOE Infiltration Factor	0.60	0	
Actual Infiltration Factor	0.60	0.0	
Run-off Coefficient	0.40	1.0	
Run-off from Impervious Surfaces*	0	0.80	
Inputs (per Unit Area)			
Precipitation (mm/yr)	786.0	786.0	786.0
Run-On (mm/yr)	0	0	0
Other Inputs (mm/yr)	0	0	0
Total Inputs (mm/yr)	**786.0**	**786.0**	**786.0**
Outputs (per Unit Area)			
Precipitation Surplus (mm/yr)	160.9	628.8	394.9
Net Surplus (mm/yr)	160.9	628.8	394.9
Evapotranspiration (mm/yr)	625.1	157.2	391.1
Infiltration (mm/yr)	96.6	0	48.3
Roof Infiltration (mm/yr)	0	0	0
Total Infiltration (mm/yr)	96.6	0	48.3
Runoff Pervious Areas	64.4	0	32.2
Runoff Impervious Areas	0	628.8	314.4
Total Runoff (mm/yr)	64.4	628.8	346.6
Total Output (mm/yr)	**786.0**	**786.0**	**786.0**
Difference (Input - Output) =	**0.0**	**0.0**	**0.0**
Input (Volumes)			
Precipitation (m^3/yr)	5525.5	5525.5	11050.9
Run-on (m^3/yr)	0	0	0
Other Inputs (m^3/yr)	**5525.5**	**5525.5**	**11050.9**
Output (Volumes)			
Precipitation Surplus (m^3/yr)	1131.3	5834.7	6966.0
Net Surplus (m^3/yr)	1131.3	5834.7	6966.0
Evapotranspiration (m^3/yr)	4394.1	1458.7	5852.8
Infiltration (m^3/yr)	678.8	0	678.8
Roof Infiltration (m^3/yr)	0	0	0
Total Infiltration (m^3/yr)	678.8	0.0	678.8
Runoff Pervious Areas (m^3/yr)	452.5	0	452.5
Runoff Impervious Areas (m^3/yr)	0	5834.7	5834.7
Total Runoff (m^3/yr)	452.5	5834.7	6287.2
Total Output (m^3/yr)	**5525.5**	**7293.3**	**12818.8**
Difference (Input - Output) =	0.0	1767.9	1767.9

* Evaporation from impervious areas was assumed to be 20% of precipitation

Comparison Between Pre- Development to Post-Development

	Precipitation (m^3/yr)	Evapotranspiration (m^3/yr)	Infiltration (m^3/yr)	Run-off (m^3/yr)
Pre-Development	2623.19	10188.61	1573.9	1049.28
Post-Development	1131.33	4394.13	678.8	6287.2

Estimated Post-Development Infiltration Deficit = **895 m^3/yr**
56.87 %

Roof Top Infiltration to Infiltration Trenches :

- Assume infiltration chambers to collect runoff from roofs
- Assume infiltration media = clear stone with 40% void ratio
- To follow CLI ECA Criteria the chamber will be provided in ROW

Total Roof Area draining to Chambers = 0.5993 Ha

Rainfall data for below calculations is extracted from Environmental Canada historical weather data for Toronto Lester B. Pearson Int'L Station Id : 6158733

1971 to 2000 Canadian Climate Normals station data

Days with Precipitation

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Code
>= 0.2 mm	14.9	11.6	13.1	12.1	11.9	11.0	10.1	10.8	10.7	11.5	13.2	14.6	145.5	A
>= 5 mm	3.5	2.7	3.5	4.5	4.6	5.2	3.9	4.3	4.3	4.0	4.3	4.3	49.2	A
>= 10 mm	1.2	0.97	1.8	2.4	2.4	2.6	2.4	2.4	2.5	2.1	2.2	1.7	24.8	A
>= 25 mm	0.13	0.17	0.23	0.27	0.27	0.43	0.77	0.83	0.60	0.27	0.40	0.23	4.6	A

Storm Events	To	Storm Events	Average Depth of Events	No. of Events (1971-2000)	Trench Infiltration Depth per event	Trench Infiltration Volume per event	Cumulative Infiltration Volume per event	Cumulative Infiltration Volume per Year	Cumulative Infiltration Volume per Year	Cumulative Swale overflow
mm		mm	(mm)		mm	(m^3)	(m^3)	(m^3)	(m^3)	mm
0.2	-	5	2.6	145.5	2.60	15.58	15.58	2267	2,267	0
5.0	-	10	7.5	49.2	**7.50**	**44.94**	**60.52**	**2211**	**4,478**	**0**
10.0	-	25	17.5	24.8	17.50	104.87	165.39	1835	6,313	0
25.0	-	50	37.5	4.6	37.50	224.72	390.11	1034	7,347	0

Satisfies Infiltration Deficit Targets of 895 m^3/year

Average Depth of 10mm of rainfall is required to achieve the Infiltration targets of 895m^3/year. In case of rainfall events higher than 10mm the Chamber will overflow to the proposed Dry Pond. Dry Pond is designed for volume including Roof Areas

Annual Rainfall Depth Required

Required Rainfall Depth = 10 mm (From Post Development Water Balance)

Storage volume required for rainfall events of 10mm to Rooftop Infiltration Gallery

Roof Top Area = 5992.5 m^2
Rainfall Depth = 10 mm
Storage Volume Required in Chambers = A x D
= 5992.5 x 10
= 59.925 m^3

The governing 10mm Storm Event over the Site Area as per Water Balance Calculations :

Site Area (A) = 1.73 Ha
Rainfall Depth (R) = 10 mm

Volume Required = A x R
= 173 m^3

Design Infiltration Rate :

Infiltration Rate at the bottom of the BMP = 52 mm/hr (In-situ infiltration rates to be confimed prior to construction)

Infiltration Rate at the bottom of the BMP = 52 mm/hr

Ratio of Mean Measured Infiltration Rates = 1.0

Based on CVC SWM Guide, 2022

STORMWATER MANAGEMENT GUIDELINE – APPENDIX A: WATER BALANCE AND RECHARGE 25

Table A-3: Safety correction factors for calculating design infiltration rates

Ratio of Mean Measured Infiltration Rates[1]	Safety Correction Factor[2]
≤ 1	2.5
1.1 to 4.0	3.5
4.1 to 8.0	4.5
8.1 to 16.0	6.5
16.1 or greater	8.5

Source: Wisconsin Department of Natural Resources. 2004. Conservation Practice Standards. Site Evaluation for Stormwater Infiltration (1002). Madison, WI.

Safety Factor = 2.5

Design Infiltration Rate (i) = $\frac{52}{2.5}$

20.8 mm/Hr

Calculate Maximum Allowable Infiltration Chamber Depth :

Design Infiltration Rate (i) = 20.8 mm/Hr
Drawdown Time (T) = 48 Hr
Stone Porosity/Void Ratio (Vr) = 0.40 (Clear Stone)

Maximum Depth Required (Dr_{max}) = $\frac{i \times t_s}{V_r}$

= 2496 mm

Depth Provided = 1250 mm

Therefore, StormTech Infiltration chambers for the site will achieve infiltration targets and meet the requirements for a drawdown time of 48 hours.

APPENDIX D

GEOTECHNICAL REPORT

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE 100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 542-2769

A REPORT TO
2868577 ONTARIO INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED RESIDENTIAL DEVELOPMENT

15544 MCLAUGHLIN ROAD

TOWN OF CALEDON

REFERENCE NO. 2301-S042

MARCH 2023

DISTRIBUTION

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TABLES

ENCLOSURES

1.0 INTRODUCTION

In accordance with written authorization dated January 3, 2023, from Mr. Manoj Sharma of 2868577 Ontario Inc., a geotechnical investigation was carried out on a vacant area within 15544 McLaughlin Road in the Town of Caledon.

The purpose of this investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the design and construction of a proposed residential development. The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The Town of Caledon is situated on Halton-Peel till plain where the drift dominates the soil stratigraphy. In places, lacustrine sand, silt and clay, which has been reworked by the water action of Peel Ponding (glacial lake), have modified the drift stratigraphy.

The investigated area, being part of 15544 McLaughlin Road, is located approximately 200 m west of McLaughlin Road and approximately 470 m north of Old Base Line Road, at the terminus of Kaufman Road. The area is currently a vacant lot. The site gradient generally slopes towards east with a grade difference of almost 9 m.

Based on the preliminary development plan prepared by Candevcon Limited, the area will be developed into 13 single detached dwelling lots and a parkette. Access to the lots will be provided by extension of Kaufman Road, and will be connected to Victoria Street and McKenzie Street.

3.0 FIELD WORK

The field work, consisting of five (5) sampled boreholes extending to depths of 6.2 to 6.6 m, was performed on January 24, 2023. Upon the completion of borehole drilling and sampling, monitoring wells were installed in all boreholes to facilitate groundwater monitoring. Details of the monitoring wells are included in the corresponding borehole logs. The locations of the boreholes and monitoring wells are shown on the Borehole and Monitoring Well Location Plan, Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by a track-mounted machine using solid stem auger, and equipped with split spoon sampler for soil sampling. Split-spoon samples were recovered for soil classification and laboratory testing. Standard

Penetration Tests, using the procedures described on the enclosed "List of Abbreviations and Terms," were performed at the sampling depths. The test results are recorded as the Standard Penetration Resistance (or 'N' values) of the subsoil. The relative density of the non-cohesive strata and the consistency of the cohesive strata are inferred from the 'N' values. The field work was supervised and the findings were recorded by a Geotechnical Technician.

The ground elevation at each borehole location was determined using a hand-held Global Navigation Satellite System (GNSS) equipment.

4.0 SUBSURFACE CONDITIONS

The investigation revealed that beneath a layer of topsoil veneer, and a layer of earth fill or weathered soil, the site is underlain by strata of silt, silty sand, sandy silt, silty sand till, sandy silt till and silty clay till. Weathered shale was observed in one of the boreholes at deeper elevation.

Detailed descriptions of the encountered subsurface conditions from the boreholes are presented on the Borehole Logs, comprising Figures 1 to 5, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

4.1 **Topsoil** (All boreholes)

All boreholes were carried out on the vacant field where the ground surface is covered with a layer of topsoil, approximately 15 to 30 cm in thickness. Thicker topsoil may be encountered beyond the borehole locations.

4.2 **Earth Fill** (Boreholes 3, 4 and 5)

Beneath the topsoil layer, a layer of earth fill, consisting of a mixture of sand, silt and clay, was encountered in three of the boreholes, extending to depths of 1.5 to 2.2 m below the prevailing ground surface.

The obtained 'N' values of the earth fill range from 3 to 6 blows per 30 cm of penetration, indicating that the fill is loosely placed with nominal compaction.

The natural water content of the earth fill was determined at a range from 3% to 24%, with a median of 16%, indicating generally moist conditions.

4.3 **Silt, Sandy Silt and Silty Sand** (Boreholes 1, 2, 3 and 5)

The silt, sandy silt and silty sand deposits were contacted in Borehole 1, 2, 3 and 5. It is very fine grained in texture, and interbedded with occasional gravel and silty clay layers. Grain size analyses were performed on representative samples of the silt, sandy silt and silty sand, and the results are plotted on Figures 6, 7 and 8.

The natural water content of the samples ranged from 7% to 33%, with a median of 18%, showing that the soil samples are moist to wet, generally in wet conditions.

The obtained 'N' values range from 4 to over 100, with a median of 15 blows per 30 cm of penetration, indicating that the silt, sandy silt and silty sand are loose to very dense, generally compact in relative density.

The engineering properties of the deposits are given below:

- High frost susceptibility, with high soil-adfreezing potential.
- High water erodibility, the fine particles are susceptible to migration through small opening under seepage pressure.
- The soils have high capillarity and water retention capacity.
- The shear strength is density dependent. The wet soils are susceptible to impact disturbance, while will result in soil dilation and reduction in shear strength.
- In excavation, the sand and silt will slough and run slowly with seepage from the cut face. It will boil with a piezometric head of 0.4 m.

4.4 **Sand and Gravel** (Boreholes 4 and 5)

The sand and gravel deposit was contacted in Boreholes 4 and 5 in the eastern portion of the investigated area. Grain size analysis was performed on one representative sample of the sand and gravel, and the result is plotted on Figure 9.

The natural water contents of the samples ranged from 3% to 18%, with a median of 6%, indicating the sand and gravel deposit is generally in moist condition with a wet silt layer embedded in the deposit found on Borehole 4.

The obtained 'N' values range from 34 to 46 blows per 30 cm of penetration, indicating that the deposit is dense in relative density.

The engineering properties of the sand and gravel deposit are given below:

- Low frost to relatively high in susceptibility, depending on its silt content.
- High water erodibility.
- In steep cuts, the sand and gravel will slough to its angle of repose, run with water seepage, and boil with a piezometric head of 0.4 m.

4.5 **Sandy Silt Till/Silty Sand Till** (Boreholes 1, 2 and 4)

The sandy silt till and/or silty sand till were contacted at various depths in Boreholes 1, 2 and 4. It consists of a random mixture of particle sizes ranging from clay to gravel, with sand and silt being the dominant fraction.

The natural water contents of the samples ranged from 10% to 36%, with a median of 11%, indicating the till deposit is generally in moist conditions. The high moisture is contacted near ground surface, likely due to the presence of topsoil and other organic materials.

The obtained 'N' values range from 3 to over 100, with a median of 22 blows per 30 cm of penetration, indicating that the till deposit is very loose to very dense, generally compact in relative density. The low 'N' values are generally contacted near the ground surface, likely being disturbed/weakened from weathering.

The engineering properties of the till deposit are given below:

- High frost susceptibility and low water erodibility.
- The till will be relatively stable in steep excavation; however, the sand and silt seams or layers in the till deposit may slough after prolonged exposure.

4.6 **Silty Clay Till** (Boreholes 3, 4 and 5)

The silty clay till deposit was encountered at the lower stratigraphy in Boreholes 3, 4 and 5. It consists of a random mixture of particle sizes ranging from clay to gravel, with clay being the dominant fraction.

The natural water content of the clay samples ranged from 9% to 16%, with a median of 10%, indicating generally moist conditions.

The obtained 'N' values range from 45 to over 100 blows per 30 cm of penetration, indicating hard in consistency.

The engineering properties of the silty clay till are listed below:

- High frost susceptibility and high soil-adfreezing potential.
- Low water erodibility.
- In excavation, the clay till will generally be stable in relatively steep slope, but will slough under prolonged exposure.

4.7 Shale

Weathered shale was contacted near the termination depth of Borehole 5. Occasional shale fragments were also observed in the silty clay till deposit in Boreholes 3 and 4, showing the presence of shale bedrock at deeper depth in the borehole locations.

The contacted shale deposit was fragmented and clay-shale reversion was identified, showing that the shale is weathered near the interface.

4.8 Compaction Characteristics of the Revealed Soils

The obtainable degree of compaction is primarily dependent on the soil moisture and, to a lesser extent, on the type of compactor used and the effort applied. As a general guide, the typical water content values of the revealed soils for Standard Proctor compaction are presented in Table 1.

Table 1 - Estimated Water Content for Compaction

Soil Type	Determined Natural Water Content (%)	Water Content (%) for Standard Proctor Compaction	
		100% (optimum)	Range for 95% or +
Existing Earth Fill	3 to 24 (median 16)	10	8 to 12
Silt, Silty Sand and Sandy Silt	11 to 33 (median 18)	12	9 to 14
Sand and Gravel	3 to 18 (median 7)	6	4 to 9
Sandy Silt Till and Silty Sand Till	10 to 36 (median 11)	12	9 to 16
Silty Clay Till	9 to 16 (median 10)	17	15 to 20

The above values showed that the till deposits are suitable for structural compaction; the other subsoils were found on the wet side of the optimum and will require aeration prior to structural compaction. Aeration can be achieved by spreading them thinly on the ground in the dry and warm weather.

The existing earth fill should be subexcavated and inspected, sorted free of organics and other deleterious material, before reusing for structural backfill.

5.0 GROUNDWATER CONDITION

The boreholes were checked for the presence of groundwater and cave-in upon the completion of drilling. The records are plotted on the Borehole Logs and summarized in Table 2.

Table 2 - Groundwater Levels (Upon Completion of Drilling)

Borehole No.	Ground Elevation (m)	Well Depth (m)	Measured Groundwater Level Upon Completion	
			Depth (m)	El. (m)
1	285.8	6.3	Dry	
2	281.7	6.4	5.8	275.9
3	282.8	6.2	5.5	277.3
4	277.2	6.6	Dry	
5	287.6	6.2	Dry	

Upon completion of drilling, groundwater was recorded at 5.5 to 5.8 m, or at El. 275.9 m and El. 277.3 m, in Boreholes 2 and 3 respectively. The remaining boreholes were dry upon completion. Detailed groundwater condition within the investigated area will be discussed in the hydrogeological report, under separate cover.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation revealed that beneath the topsoil, and a layer of earth fill or weathered soil, the site is underlain by strata of silt, silty sand, sandy silt, silty sand till, sandy silt till and silty clay till. Weathered shale was observed in one of the boreholes at deeper elevation.

Groundwater was recorded upon completion of drilling at 5.5 to 5.8 m, or at El. 275.9 m and El. 277.3 m, in Boreholes 2 and 3 respectively. The remaining boreholes were dry upon completion.

Based on the preliminary development plan prepared by Candevcon Limited, the area will be developed into 13 single detached dwelling lots and a parkette. The geotechnical findings which warrant special consideration are presented below:

1. Prior to construction, all vegetation and topsoil must be removed for site development. They can only be reused for landscape purpose. Any surplus must be removed off site.
2. The existing earth fill is not suitable for supporting the proposed structure at its current state. The earth fill should be subexcavated, sorted free of organics and/or deleterious materials, prior to be reused for structural backfill or engineered fill constructions.
3. Where additional fill is required for site grading, the earth fill can be constructed in accordance with the engineering fill specifications for supporting the foundation, underground services, and pavement construction.
4. The proposed structures can be constructed on conventional spread and strip footings founded on the undisturbed native soils or on engineered fill below the frost penetration depth. The foundation subgrade must be inspected by the geotechnical engineer or a senior geotechnical technician to ensure that the revealed conditions are compatible with the foundation design requirements.

The recommendations appropriate for the project described in Section 2.0 are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should this become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

6.1 Site Preparation

Where additional fill is required for site grading, the earth fill can be constructed in accordance with the engineering fill specifications for supporting the foundation, underground services, and pavement construction. The engineering requirements for a certifiable fill are presented below:

1. The topsoil and vegetation must be stripped. The existing earth fill and weathered soil should also be removed and examined. Any topsoil and deleterious material must be segregated and removed before reuse for structural backfill. The exposed soil subgrade must be inspected and proof-rolled prior to any fill placement. Any loose material identified during proof-rolling must be further subexcavated and backfilled with organic free material, compacted to engineered fill specifications.
2. Inorganic soils must be used for the engineered fill construction, and they must be uniformly compacted in lifts of 20 cm thick to at least 98% of their maximum Standard Proctor dry density (SPDD) with its moisture content controlled near its optimum. If the foundations are to be built soon after the fill placement, the densification process for the engineered fill must be increased to 100% SPDD.
3. If the engineered fill is compacted with the moisture content on the wet side of the optimum, the underground services and pavement construction should not begin until

the pore pressure within the fill mantle has completely dissipated. This must be further assessed at the time of the engineered fill construction.

4. If imported fill is to be used, it should be inorganic soils, free of any deleterious material with environmental issue (contamination). Any potential imported earth fill from off-site must be reviewed for geotechnical and environmental quality by the appropriate personnel as authorized by the developer or agency, before it is hauled to the site.
5. The engineered fill must not be placed during the period where freezing ambient temperatures occur either persistently or intermittently. This is to ensure that the fill is free of frozen soils, ice and snow. If the fill is to be left over the winter months, adequate earth cover, or equivalent, must be provided for protection against frost action.
6. The fill operation must be supervised and inspected on a full-time basis by a geotechnical technician under the direction of a geotechnical engineer.
7. The engineered fill envelope and finished elevations must be clearly and accurately defined in the field, and they must be precisely documented.
8. Foundations founded on engineered fill must be properly reinforced. It should be designed by a structural engineer to allow distribution of stress induced by the abrupt differential settlement (about 20 mm) in engineered fill.
9. The footing, slab-on-grade and underground services subgrade must be inspected by the geotechnical consulting firm which supervised the engineered fill placement. This is to ensure that the foundations and service pipes are placed within the engineered fill envelope, and the integrity of the fill has not been compromised by interim construction, environmental degradation and/or disturbance by any excavation.
10. Any excavation carried out in certified engineered fill must be reported to the geotechnical consultant who supervised the fill placement in order to document the locations of excavation and/or to supervise reinstatement of the excavated areas to engineered fill status. If construction on the engineered fill does not commence within a period of 2 years from the date of certification, the condition of the engineered fill must be assessed for re-certification.

6.2 Foundations

The proposed structures can be supported on conventional spread and strip footings, founded on engineered fill or competent native soil, at a depth at least 1.2 m below the proposed finished grade. The recommended soil bearing pressures for the design of conventional footings are presented below:

- Maximum Soil Bearing Pressure, at Serviceability Limit State (SLS) = 150 kPa

- Factored Ultimate Bearing Pressure, at Ultimate Limit State (ULS) = 200 kPa

The total and differential settlements of the conventional spread and strip footings, designed for the bearing pressure at SLS, are estimated to be 25 mm and 20 mm, respectively.

The foundation subgrade must be inspected by either a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the revealed conditions are compatible with the design of the foundation.

It should be noted that if water seepage is encountered during footing excavations, or where the foundation subgrade is found to be wet, the subgrade should be protected by a concrete mud-slab immediately after exposure. This will prevent construction disturbance and costly rectification.

Footings exposed to weathering or in unheated areas should have at least 1.2 m of earth cover for protection against frost action.

The building foundation must meet the requirements specified in the latest Ontario Building Code. As a guide, the structure should be designed to resist an earthquake force using Site Classification 'D' (stiff soil).

6.3 Basement Structure and Slab-On-Grade Construction

For structures with a basement, the perimeter walls should be designed to sustain a lateral earth pressure calculated using the soil parameters stated in Section 6.7. Any applicable surcharge loads adjacent to the basement must also be considered in the wall design.

The basement structure should be damp-proofed and provided with a drainage system (Drawing No. 3) at the wall base. The subdrains should be encased in a fabric filter to protect them against blockage by silting.

The floor subgrade should consist of sound native soil or well compacted inorganic earth fill. It must be inspected and proof-rolled prior to the placement of granular bedding. Any weak spots must be subexcavated and replaced with inorganic earth fill compacted to at least 98% SPDD in lifts no more than 20 cm in thickness. The floor slab should be constructed on granular bedding, at least 15 cm thick, consisting of 19-mm Crusher-Run Limestone (CRL), or equivalent, compacted to 100% SPDD.

The existing grade around the building structure must be such that it directs runoff away from the structure.

6.4 Underground Services

The subgrade for the underground services should be found on sound native soils or properly compacted, inorganic earth fill. A Class 'B' bedding, consisting of compacted 19-mm CRL, or equivalent, is recommended for the underground service construction.

The pipe joints connecting into the manholes and catch basins must be leak-proof to prevent the migration of fines through the joints. Openings to subdrains and catch basins should be shielded with a fabric filter to prevent blockage by silting.

A soil cover having a thickness equal to the diameter of the pipe, should be in place at all times after pipe installation to prevent pipe floatation when the trench is deluged with water derived from precipitation.

The on-site soil is corrosive to ductile iron pipes and metal fittings; therefore, they should be protected against soil corrosion. For estimation for the anode weight requirements, the electrical resistivities of the disclosed soils can be used. The proposed anode weight must meet the minimum requirements as specified by the municipality standard.

6.5 Backfilling in Trenches and Excavated Areas

Some of the on-site inorganic soils are suitable for trench backfill; however, any wet soil will require aeration prior to its use as structural backfill. The backfill material should be inorganic soils, free of boulders or oversized rock pieces (over 15 cm in size), compacted to at least 95% SPDD in lifts no more than 20 cm in thickness, or the thickness should be determined by test strips.

In the zone within 1.0 m below the pavement subgrade or slab-on-grade, the backfill should be compacted to at least 98% SPDD, with the water content at 2% to 3% drier than the optimum.

The narrow trenches should be cut at 1 vertical:2 or + horizontal so that the backfill can be effectively compacted. Otherwise, soil arching will prevent the achievement of proper compaction.

In normal sewer construction practice, the problem areas of ground settlement largely occur adjacent to manholes, catch basins and services crossings, foundation walls and columns. In areas which are inaccessible to a heavy compactor, sand backfill should be used and compacted with lighter equipment.

6.6 Pavement Design

The recommended pavement design for local residential road is presented in Table 3.

Table 3 - Pavement Design for Local Road

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL3
Asphalt Binder	65	HL8
Granular Base	150	Granular 'A' or equivalent
Granular Sub-base	300	Granular 'B' or equivalent

In preparation of pavement subgrade, any topsoil and compressible material should be removed. The final subgrade must be proof-rolled and inspected. Any soft spot identified must be rectified by subexcavation and replacing with selected inorganic material. In zone within 1.0 m below the pavement subgrade must be compacted to at least 98% SPDD, with the water content at 2% to 3% drier than its optimum. All the granular bases should be compacted to 100% SPDD.

The pavement subgrade will suffer a strength regression if water is allowed to saturate the mantle. The following measures should, therefore, be incorporated in the construction procedures and pavement design:

- The subgrade should be properly crowned and smooth-rolled to allow interim precipitation to be properly drained prior to pavement construction.
- Areas adjacent to the pavement should be properly graded to prevent water ponding. Otherwise, the water will seep into the subgrade mantle and induce a regression of the subgrade strength, with costly consequences for the pavement construction.
- Fabric filter-encased curb subdrains connecting to a positive outlet of catch basin, will be required at the edge of the pavement.
- If the pavement is to be constructed during wet seasons, wet or soft subgrade may occur and should be properly rectified. Alternatively, the granular sub-base can be thickened to compensate for the inadequate strength of the subgrade. This can be assessed during construction.

6.7 Soil Parameters

The recommended soil parameters for the project design are given in Table 4.

Table 4 - Soil Parameters

Unit Weight and Bulk Factor	**Unit Weight γ (kN/m^3)**		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Existing Earth Fill	20.5	10.5	12.5	1.00
Silt, Silty Sand, Sandy Silt	20.5	10.5	1.20	1.00
Sand and Gravel	20.0	10.0	1.20	0.98
Sandy Silt Till, Silty Sand Till	22.5	12.5	1.33	1.00
Silty Clay Till	22.0	12.0	1.30	1.03

Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_0**	**Passive K_p**
Compacted Earth Fill	0.36	0.53	2.77
Silt, Silty Sand, Sandy Silt	0.32	0.48	3.12
Sand and Gravel	0.29	0.46	3.39
Sandy Silt Till, Silty Sand Till, Silty Clay Till	0.33	0.50	3.00

Estimated Coefficient of Permeability (K) and Percolation Time (T)	**K (cm/sec)**	**T (min/cm)**
Silt, Silty Sand, Sandy Silt	10^{-4} to 10^{-5}	12 to 20
Sand and Gravel	10^{-3}	8
Sandy Silt Till, Silty Sand Till	10^{-5} to 10^{-6}	20 to 50
Silty Clay Till	10^{-7}	Over 80

Coefficients of Friction	
Between Concrete and Granular Base	0.50
Between Concrete and Sound Native Soils	0.35

6.8 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. The types of soils are classified in Table 5.

Table 5 - Classification of Soils for Excavation

Material	Type
Sound Till, Weathered Shale	2
Earth Fill/Drained Soils, Sand and Gravel	3
Saturated soils	4

Water seepage can be expected during excavation into the sand and silt deposits. Depending on the rate and quantity, extensive dewatering may be required. This should be confirmed with the hydrogeologist.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of 2868577 Ontario Inc., and for review by its designated consultants and government agencies. Use of this report is subject to the conditions and limitations of the contractual agreement.

The material in the report reflects the judgement of Poh Fung (Derek) Kwok, M.Sc., and Kin Fung Li, P.Eng., in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, is the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

SOIL ENGINEERS LTD.

LICENSED PROFESSIONAL ENGINEER
March 7, 2023
K. F. LI
100169280
PROVINCE OF ONTARIO

Poh Fung (Derek) Kwok, M.Sc. Kin Fung Li, P.Eng.

PFK/KFL

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS Auger sample
CS Chunk sample
DO Drive open (split spoon)
DS Denison type sample
FS Foil sample
RC Rock core (with size and percentage recovery)
ST Slotted tube
TO Thin-walled, open
TP Thin-walled, piston
WS Wash sample

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows for each foot of penetration of a 2-inch diameter, 90° point cone driven by a 140-pound hammer falling 30 inches.

Plotted as '—•—'

Standard Penetration Resistance or 'N' Value:

The number of blows of a 140-pound hammer falling 30 inches required to advance a 2-inch O.D. drive open sampler one foot into undisturbed soil.

Plotted as '○'

WH Sampler advanced by static weight
PH Sampler advanced by hydraulic pressure
PM Sampler advanced by manual pressure
NP No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/ft)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

Cohesive Soils:

Undrained Shear Strength (ksf)	'N' (blows/ft)	Consistency
less than 0.25	0 to 2	very soft
0.25 to 0.50	2 to 4	soft
0.50 to 1.0	4 to 8	firm
1.0 to 2.0	8 to 16	stiff
2.0 to 4.0	16 to 32	very stiff
over 4.0	over 32	hard

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0 Field vane test in borehole; the number denotes the sensitivity to remoulding

△ Laboratory vane test

□ Compression test in laboratory

For a saturated cohesive soil, the undrained shear strength is taken as one half of the undrained compressive strength

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres
1lb = 0.454 kg
1 inch = 25.4 mm
1ksf = 47.88 kPa

JOB NO.: 2301-S042

LOG OF BOREHOLE: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value	Depth Scale (m)	Penetration Resistance (blows/30 cm)	Moisture Content (%)	WATER LEVEL
285.8	Ground Surface							
0.0	**30 cm Topsoil**	1	DO	4	0	4	33	Dry upon Completion
	Brown, loose to compact **SANDY SILT** a trace of clay occ. gravel — weathered	2	DO	12	1	12	16	
		3	DO	11		11	18	
283.6 / 2.2	Brown, compact to very dense **SANDY SILT TILL / SILTY SAND TILL** a trace of clay some gravel to gravelly	4	DO	13	2	13	10	
		5	DO	22	3	22	13	
		6	DO	73	4	73	10	
279.5 / 6.3	**END OF BOREHOLE**	7	DO	50/15	6	50/15	10	

Legend: ● Dynamic Cone (blows/30 cm) 10 30 50 70 90; × Shear Strength (kN/m²) 50 100 150 200; ○ Penetration Resistance (blows/30 cm) 10 30 50 70 90; Atterberg Limits PL — LL; ● Moisture Content (%) 10 20 30 40

Installed 50 mm Ø monitoring well to 6.1 m
completed with 3 m PVC slotted screen
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a monument casing

Soil Engineers Ltd.

JOB NO.: 2301-S042

LOG OF BOREHOLE: 2

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

Legend: ● Dynamic Cone (blows/30 cm) · ✕ Shear Strength (kN/m²) · ○ Penetration Resistance (blows/30 cm) · Atterberg Limits PL – LL · ● Moisture Content (%)

El. (m) / Depth (m)	Soil Description	Sample Number	Type	N-Value	Moisture Content (%)
281.7	Ground Surface				
0.0	**25 cm Topsoil**	1	DO	3	36
	Brown, very loose to compact **SANDY SILT TILL** traces of clay and gravel (weathered)	2	DO	15	11
280.0 / 1.7	Brown, compact to very dense **SILT** very moist to wet, a trace to some sand	3	DO	18	17
		4	DO	15	22
		5	DO	21	20
		6	DO	28	17
	occ. cobbles, a trace of gravel	7	DO	50/15	17
275.3 / 6.4	**END OF BOREHOLE**				

Installed 50 mm Ø monitoring well to 6.1 m
completed with 3 m PVC slotted screen
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a monument casing

WATER LEVEL: W.L. @ El. 275.9 m upon completion

Soil Engineers Ltd.

JOB NO.: 2301-S042

LOG OF BOREHOLE: 3

FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed Residential Development

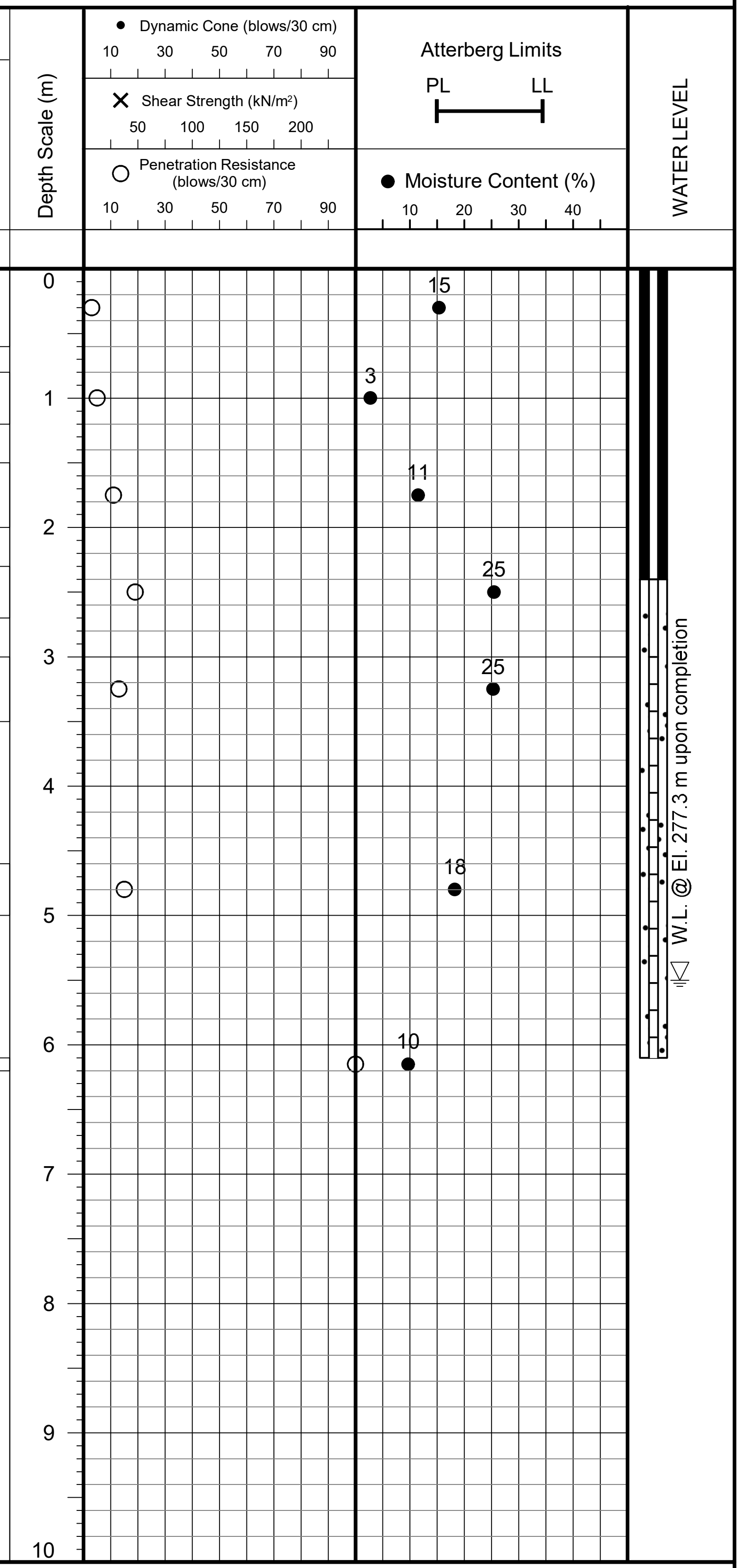
METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
282.8	Ground Surface			
0.0	**20 cm Topsoil**			
	Dark brown **EARTH FILL** sand, some silt occ. topsoil inclusion occ. organics and rootlets	1	DO	3
		2	DO	5
281.3 / 1.5	Brown, compact **SILTY SAND** occ. silty clay layers	3	DO	11
280.5 / 2.3	Brown, compact, wet **SILT** traces of clay and gravel	4	DO	19
		5	DO	13
278.8 / 4.0	Brown, compact **SANDY SILT** a trace of clay occ. gravel	6	DO	15
277.2 / 5.6	Brown, hard **SILTY CLAY TILL** occ. shale fragments	7	DO	50/3
276.6 / 6.2	**END OF BOREHOLE**			

Installed 50 mm Ø monitoring well to 6.1 m
completed with 3 m PVC slotted screen
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a monument casing



JOB NO.: 2301-S042

LOG OF BOREHOLE: 4

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	WATER LEVEL
277.2	Ground Surface					
0.0	**20 cm Topsoil**	1	DO	5	0	
	Dark brown **EARTH FILL** mixture of sand, silt and clay, a trace of gravel, occ. topsoil inclusion, occ. organics and rootlets	2	DO	6	1	
		3	DO	3	2	
275.0 / 2.2	Brown, dense **SAND AND GRAVEL** a trace to some silt (silty)	4	DO	37		
		5	DO	35	3	
					4	
272.4 / 4.8	Brown, dense **SANDY SILT TILL** traces of clay and gravel	6	DO	34	5	Dry upon Completion
270.9 / 6.3	Brown, hard	7A	DO	45	6	
270.6 / 6.6	**SILTY CLAY TILL**	7B				
	END OF BOREHOLE				7	
	Installed 50 mm Ø monitoring well to 6.1 m completed with 3 m PVC slotted screen. Sand backfill from 2.4 to 6.1 m. Bentonite seal from 0.0 m to 2.4 m. Provided with a monument casing				8, 9, 10	

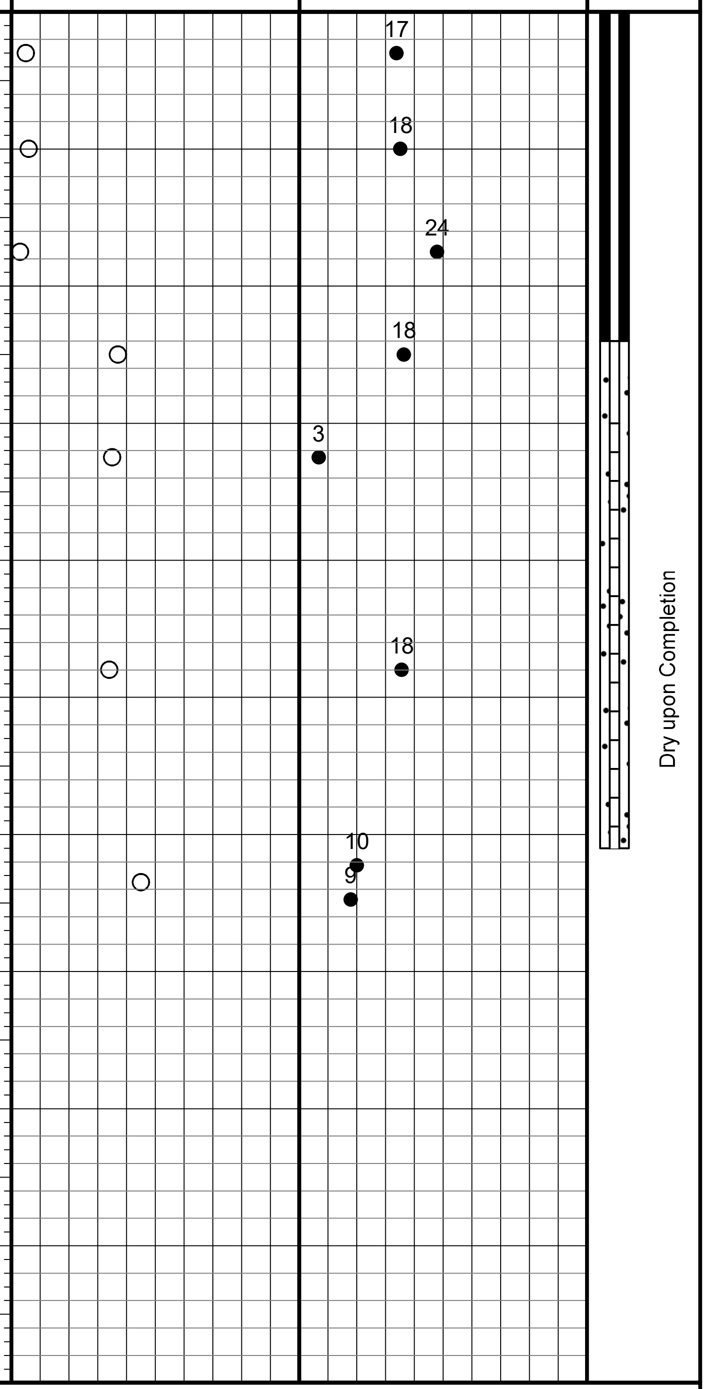
● Dynamic Cone (blows/30 cm) 10 30 50 70 90

× Shear Strength (kN/m²) 50 100 150 200

○ Penetration Resistance (blows/30 cm) 10 30 50 70 90

Atterberg Limits PL LL

● Moisture Content (%) 10 20 30 40



Soil Engineers Ltd.

JOB NO.: 2301-S042

LOG OF BOREHOLE: 5

FIGURE NO.: 5

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Penetration Resistance (blows/30 cm)	Moisture Content (%)	WATER LEVEL
278.6	Ground Surface						
0.0	**15 cm Topsoil**						
	Dark brown **EARTH FILL** sand, some silt to silty occ. topsoil inclusion occ. organics and rootlets	1	DO	5			
		2	DO	6		15	
277.1 / 1.5	Brown, compact **SILTY SAND** a trace of clay	3	DO	16		7	
276.3 / 1.5	Brown, dense **SAND AND GRAVEL** a trace to some silt	4	DO	42		6	
		5	DO	46		5	
274.6 / 4.0	Brown, hard **SILTY CLAY TILL** a trace of gravel	6	DO	47		16	
272.9 / 5.7	Grey, weathered **SHALE** water seepage	7	DO	50/3			
272.4 / 6.2	**END OF BOREHOLE**						Dry upon Completion

Chart scales: Dynamic Cone (blows/30 cm) 10, 30, 50, 70, 90; Shear Strength (kN/m²) 50, 100, 150, 200; Penetration Resistance (blows/30 cm) 10, 30, 50, 70, 90; Atterberg Limits PL, LL; Moisture Content (%) 10, 20, 30, 40; Depth Scale (m) 0–10.

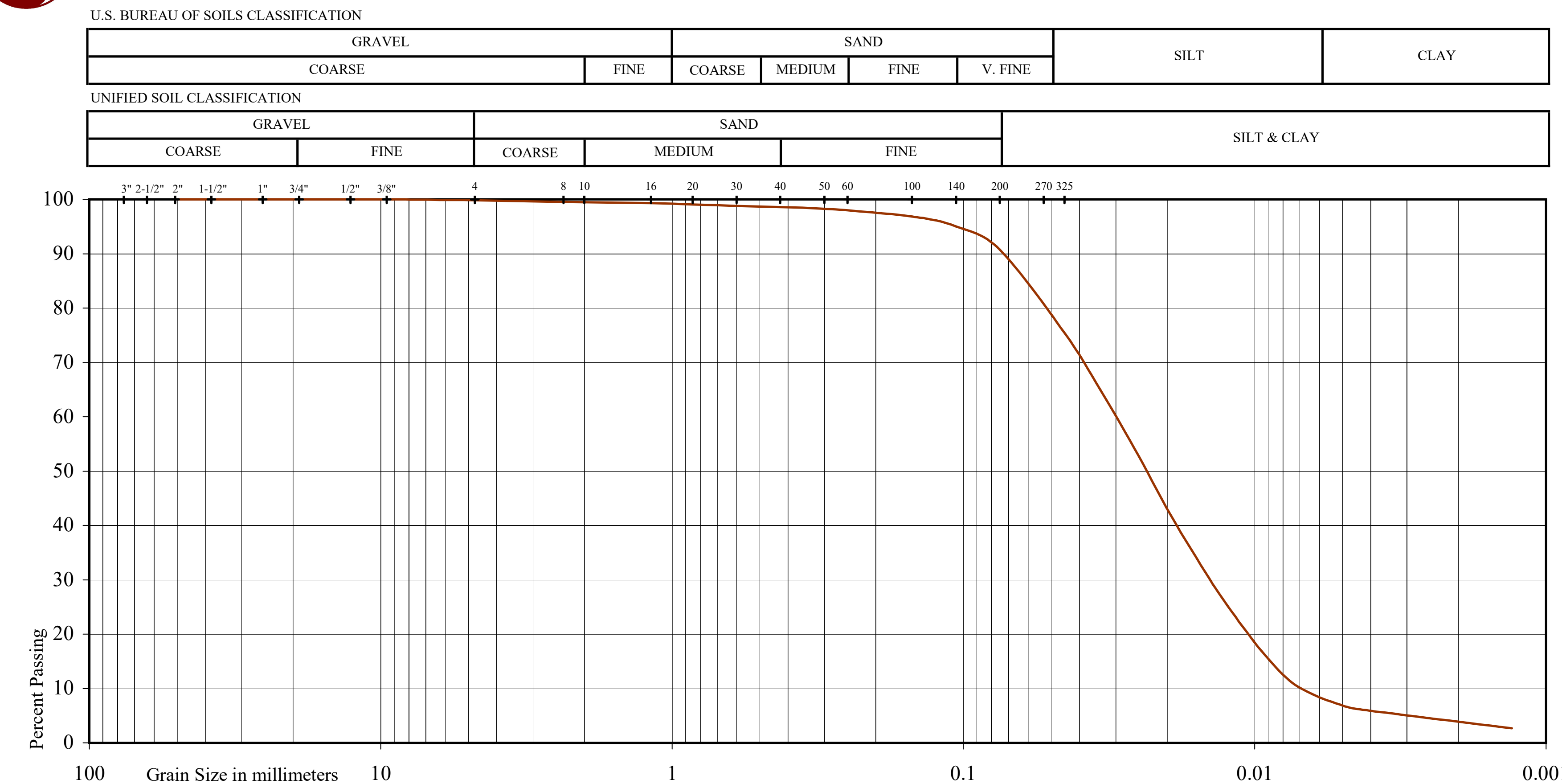
Installed 50 mm Ø monitoring well to 6.1 m
completed with 3 m PVC slotted screen
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a monument casing

Soil Engineers Ltd.

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 2301-S042



Project: Proposed Residential Development

Location: 15544 McLaughlin Road, Town of Caledon

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 20

Estimated Permeability (cm./sec.) = 10^{-4}

Borehole No: 2

Sample No: 5

Depth (m): 3.3

Elevation (m): 278.5

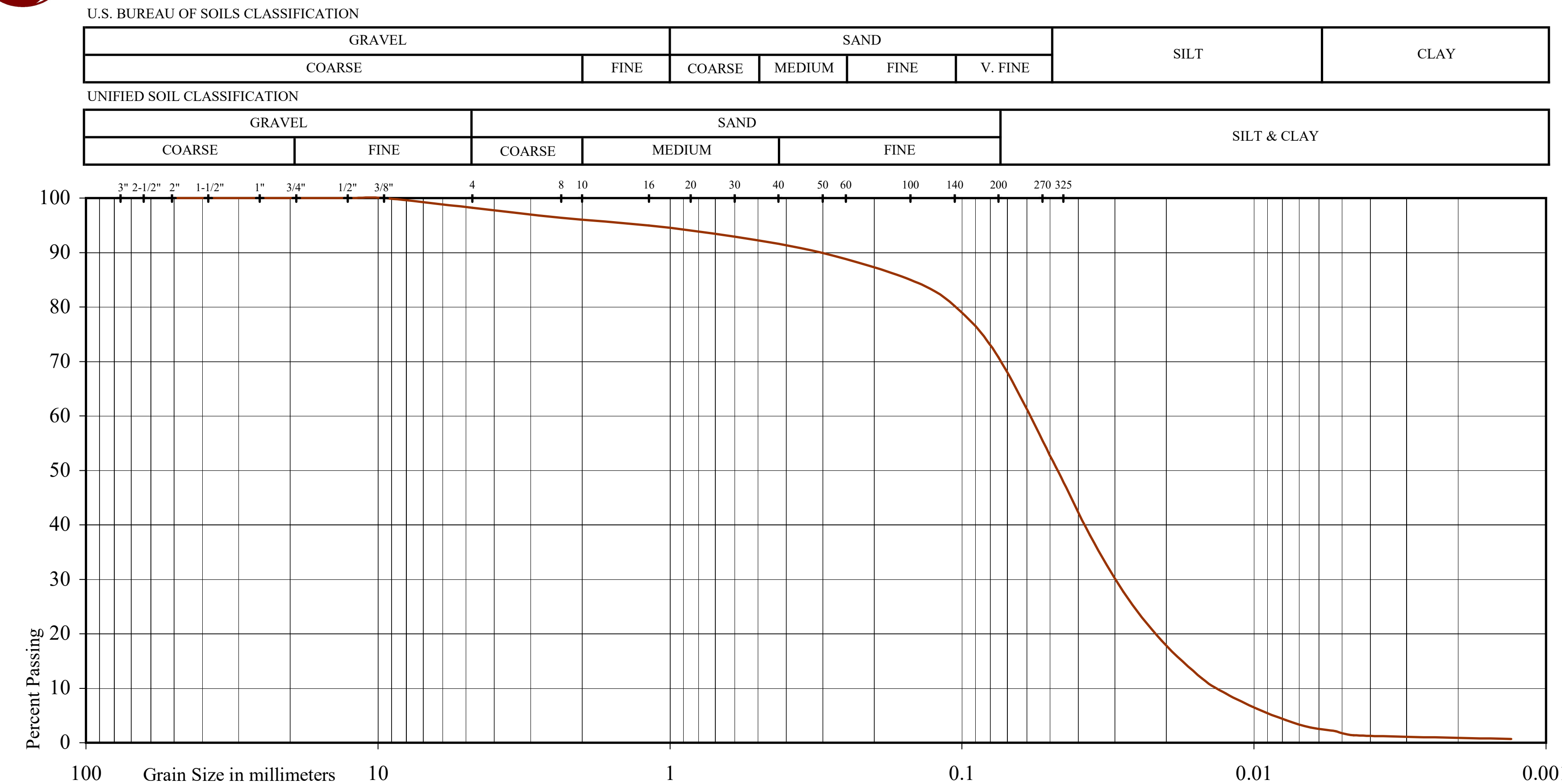
Classification of Sample [& Group Symbol]: SILT
traces of clay and sand

Figure: 6

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 2301-S042



Project: Proposed Residential Development

Location: 15544 McLaughlin Road, Town of Caledon

Liquid Limit (%) = -

Plastic Limit (%) = -

Borehole No: 3

Plasticity Index (%) = -

Sample No: 6

Moisture Content (%) = 18

Depth (m): 4.8

Estimated Permeability

Elevation (m): 278.0

(cm./sec.) = 10^{-4}

Classification of Sample [& Group Symbol]: SANDY SILT

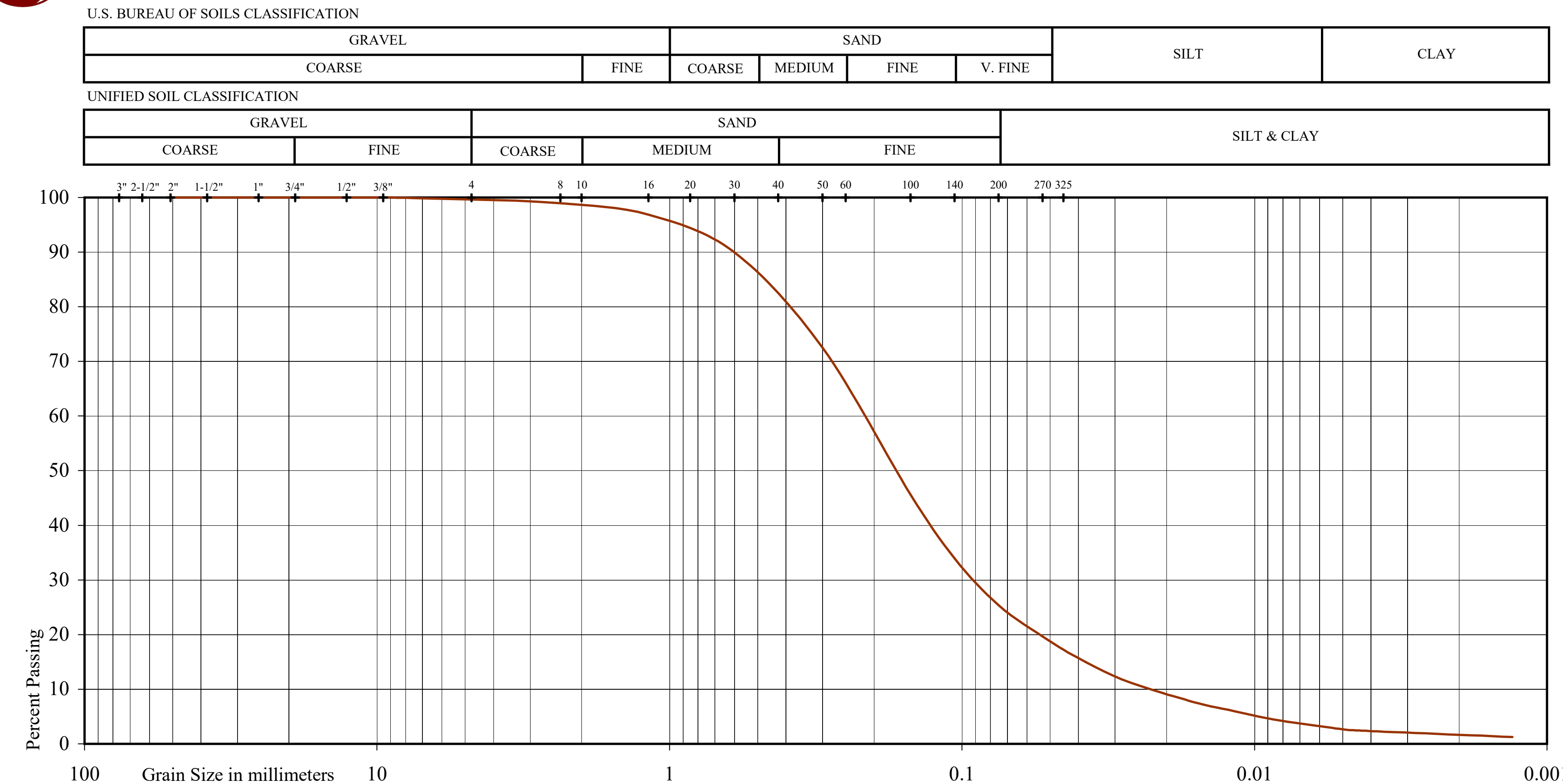
a trace of clay and occ. gravel

Figure: 7

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 2301-S042



Project:	Proposed Residential Development		
Location:	15544 McLaughlin Road, Town of Caledon	Liquid Limit (%) =	-
		Plastic Limit (%) =	-
Borehole No:	5	Plasticity Index (%) =	-
Sample No:	3	Moisture Content (%) =	7
Depth (m):	1.8	Estimated Permeability	
Elevation (m):	276.9	(cm./sec.) =	10^{-3}

Classification of Sample [& Group Symbol]: SILTY SAND
a trace of clay

Figure: 8

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

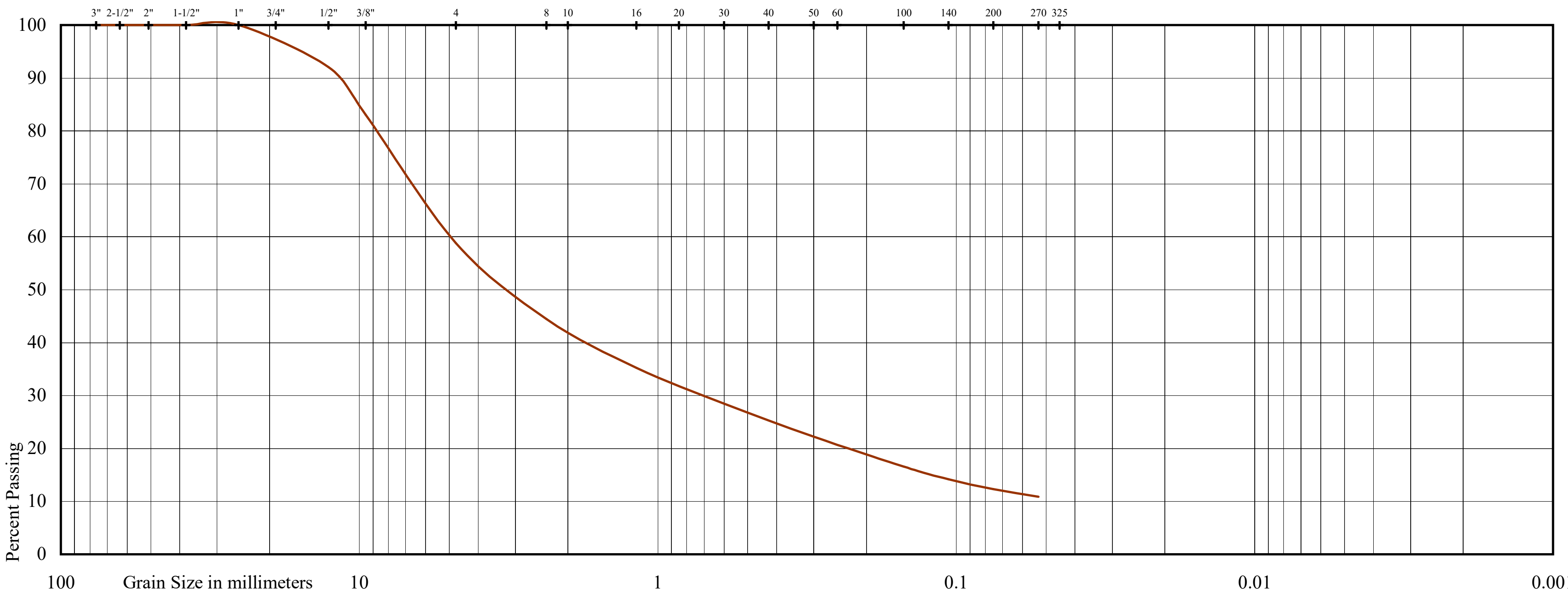
Reference No: 2301-S042

U.S. BUREAU OF SOILS CLASSIFICATION

GRAVEL		SAND				SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	V. FINE		

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND			SILT & CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Project: Proposed Residential Development

Location: 15544 McLaughlin Road, Town of Caledon

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 3

Estimated Permeability (cm./sec.) = 10^{-3}

Borehole No: 4

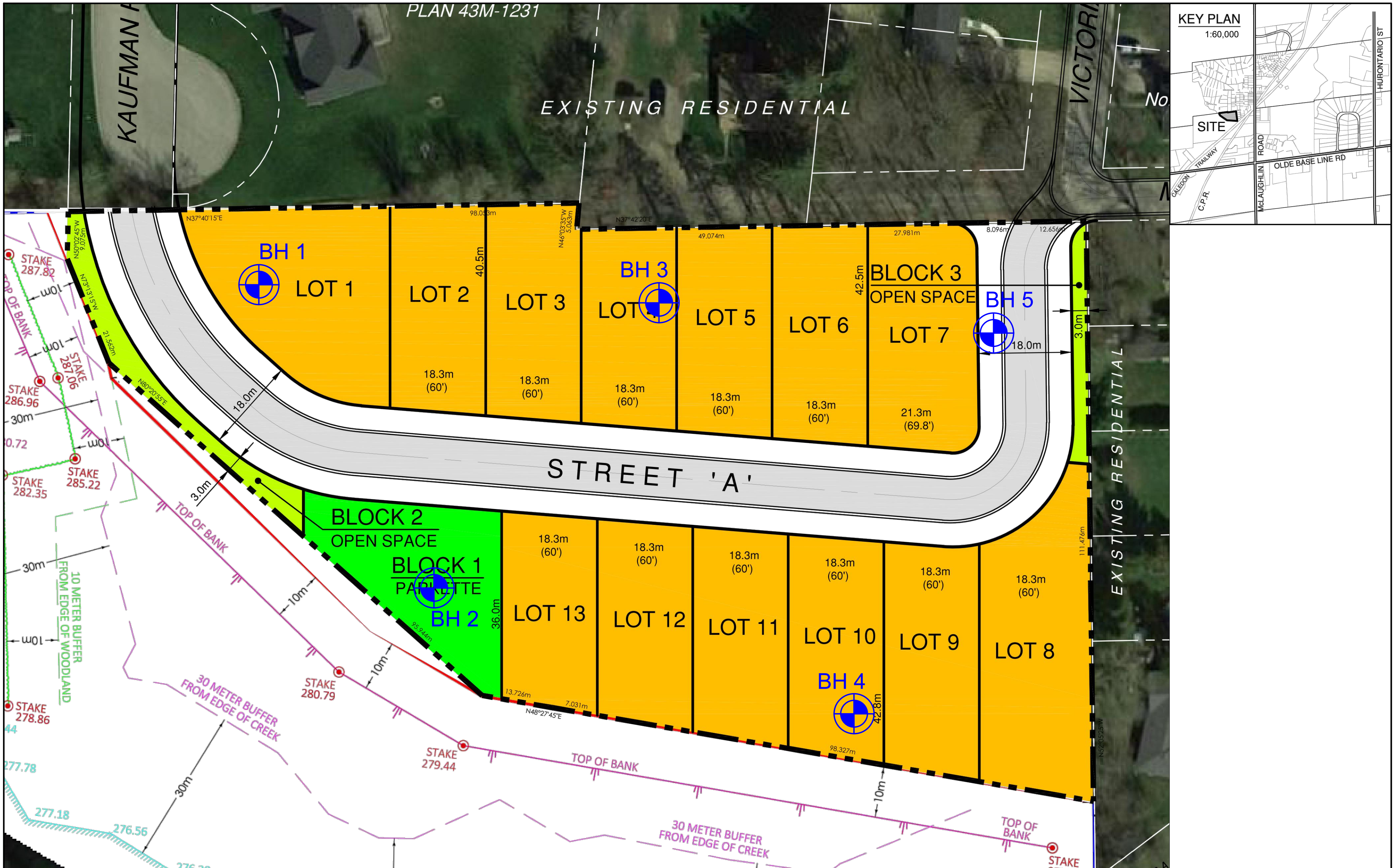
Sample No: 5

Depth (m): 3.3

Elevation (m): 274.0

Classification of Sample [& Group Symbol]: SAND AND GRAVEL
some silt

Figure: 9



Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

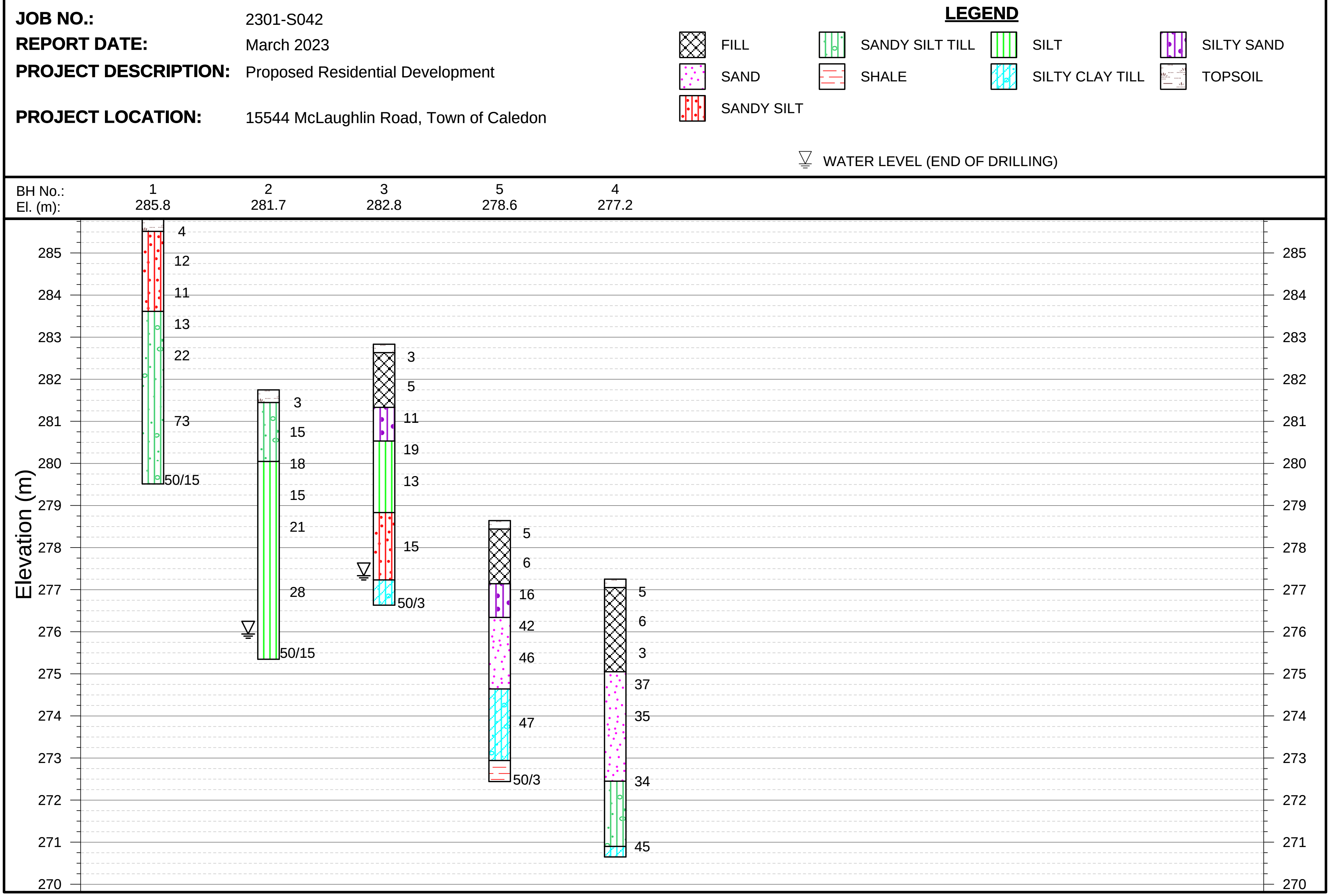
BOREHOLE AND MONITORING WELL LOCATION PLAN

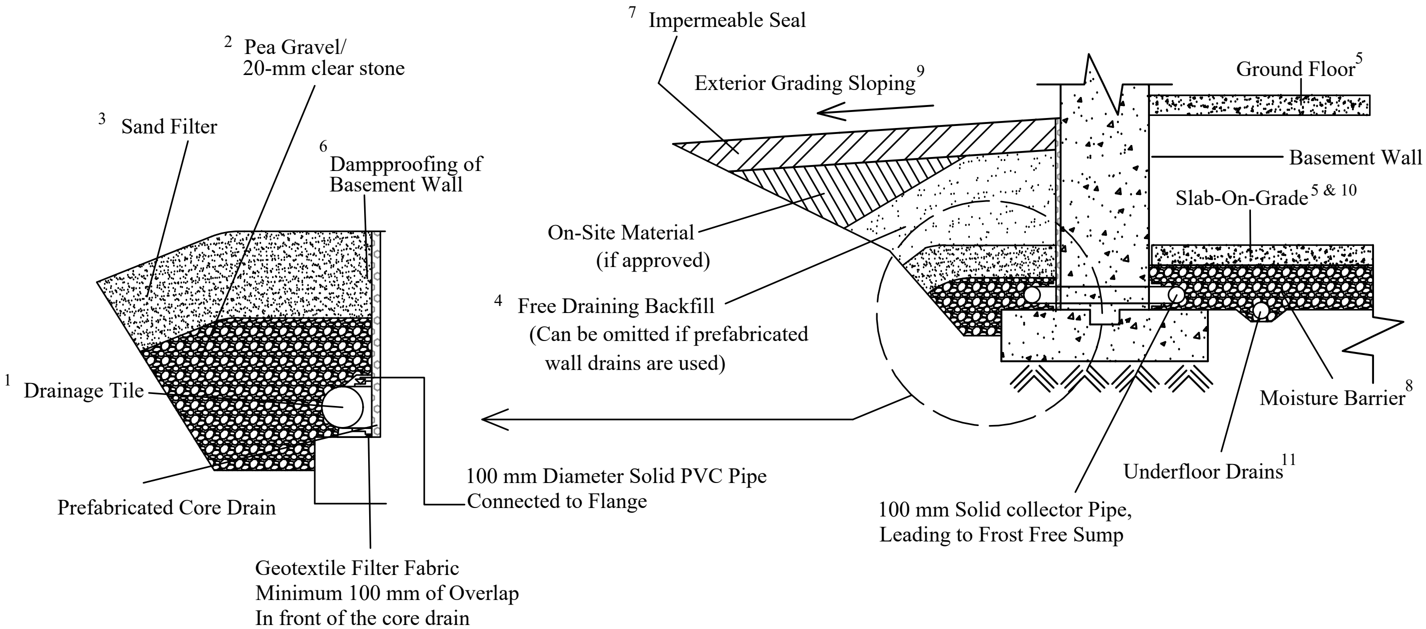
SITE: 15544 MCLAUGHLIN ROAD, TOWN OF CALEDON		
DESIGNED BY: D.K.	CHECKED BY: K.L.	DWG NO.: 1
SCALE: 1:1,000	REF. NO.: 2301-S042	DATE: MARCH 2023	REV -

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE

SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.: 2301-S042
REPORT DATE: March 2023
PROJECT DESCRIPTION: Proposed Residential Development
PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon





NOTES:

1. **Drainage tile**: consists of 100 mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. Invert to be at minimum of 150 mm (6") below underside of basement floor slab.

2. **Pea gravel**: at 150 mm (6") on the top and sides of drain. If drain is not placed on concrete footing, provide 100 mm (4") of pea gravel below drain. The pea gravel may be replaced by 20 mm clear stone provided that the drain is covered by a porous geotextile membrane of Terrafix 270R or equivalent.

3. **Filter material**: consists of C.S.A. fine concrete aggregate. A minimum of 300 mm (12") on the top and sides of gravel. This may be replaced by an approved porous geotextile membrane of Terrafix 270R or equivalent.

4. **Free-draining backfill**: OPSS Granular 'B' or equivalent, compacted to 95% to 98% (maximum) Standard Proctor dry density. Do not compact closer than 1.8 m (6') from wall with heavy equipment. This may be replaced by on-site material if prefabricated wall drains (Miradrain) extending from the finished grade to the bottom of the basement wall are used.

5. **Do not backfill** until the wall is supported by the basement floor slab and ground floor framing, or adequate bracing.

6. **Dampproofing** of the basement wall is required before backfilling

7. **Impermeable backfill seal** of compacted clay, clayey silt or equivalent. If the original soil in the vicinity is a free-draining sand, the seal may be omitted.

8. **Moisture barrier**: 19-mm CRL or compacted OPSS Granular 'A', or equivalent. The thickness of this layer should be 150 mm (6") minimum.

9. **Exterior Grade**: slope away from basement wall on all the sides of the building.

10. **Slab-On-Grade** should not be structurally connected to walls or foundations.

11. **Underfloor drains*** should be placed in parallel rows at 6 to 8 m (20'-25') centre, on 100 mm (4") of pea gravel with 150 mm (6") of pea gravel on top and sides. The spacing should be at least 300 mm (12") between the underside of the floor slab and the top of the pipe. The drains should be connected to positive sumps or outlets. Do not connect the underfloor drains to the perimeter drains.

*Underfloor drains can be deleted where not required.

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90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

Details of Perimeter Drainage System

SITE: 15544 McLaughlin Road, Town of Caledon

DESIGNED BY: K.L.	**CHECKED BY:** B.L.	**DWG NO.:** 3	
SCALE: N.T.S.	**REF. NO.:** 2301-S042	**DATE:** March 2023	**REV** -

APPENDIX E

HYDROGEOLOGICAL REPORT

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE 100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	MUSKOKA	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 542-2769

A REPORT TO
2868577 ONTARIO INC.

HYDROGEOLOGICAL ASSESSMENT FOR
PROPOSED RESIDENTIAL DEVELOPMENT

15544 MCLAUGHLIN ROAD

TOWN OF CALEDON

REFERENCE NO. 2301-W042

AUGUST 2023
(REVISION OF REPORT DATED JULY 2023)

DISTRIBUTION

Digital Copy - 2868577 ONTARIO INC.
1 Copy - Soil Engineers Ltd. (Richmond Hill)

LIMITATIONS OF LIABILITY

This report was prepared by Soil Engineers Ltd. (SEL) for the account of 2868577 Ontario Inc. and for review by their designated agents, financial institutions and government agencies, and can be used for development approval purposes by the Town of Caledon and the Ontario Ministry of the Environment, Conservation and Parks who may rely on the results of the report. The material in it reflects the judgement of Gurkaranbir Singh, M.Eng., Bhawandeep Singh Brar, B.Sc., and Gavin O'Brien, M.Sc., P.Geo. Any use which a Third Party makes of this report and/or any reliance on decisions to be made based on it is the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

One must understand that the mandate of Soil Engineers Ltd. is to obtain readily available current and past information pertinent to the subject site for a Hydrogeological Assessment only. No other warranty or representation expressed or implied, as to the accuracy of the information is included or intended by this assessment. Site conditions are not static and this report documents site conditions observed at the time of the site reconnaissance.

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ENCLOSURES

APPENDICES

1.0 EXECUTIVE SUMMARY

Soil Engineers Ltd. has conducted a hydrogeological assessment for a proposed residential development site, located at 15544 McLaughlin Road in the Town of Caledon. The subject site is currently a farmland, where the surrounding land use includes; a water course flowing south of the site, wooded areas, situated immediate to the south-west, and existing residential properties to the north, north-east and north-west of the subject site.

The subject site lies within the Physiographic Region of Southern Ontario, known as the Niagara Escarpment, on the Spillways Physiographic feature. The site is underlain, partially by the Halton Till Unit deposits, and is situated partially on the bedrock deposits.

The subject site is located within the Credit Valley Watershed, and Credit River-Forks of the Credit to Churchville Sub-watershed. Records review shows that a tributary of the Credit River is located to the south, along with wooded areas that are located southwest of the subject site.

A review of the local topography map for the area, and from the review of the ground surface elevations recorded at the borehole and monitoring well locations indicates that the total elevation relief across the site is about 9 m.

This study has revealed that beneath a layer of topsoil veneer, and a layer of earth fill or weathered soil, the site is underlain by strata of native soils comprising silt, silty sand, sandy silt, silty sand till, sandy silt till and silty clay till. Weathered shale was also observed in some of the BH/MWs at deeper elevation.

The findings of this study confirm that the measured groundwater level elevations ranged from 272.32 to 284.68 masl on average.

The shallow groundwater is interpreted to flow in a eastern direction, beneath the site towards the low relief portion of the property.

The single well response tests yielded estimated hydraulic conductivity (K) values that range from 6.0×10^{-7} to 4.0×10^{-6} m/sec for the sandy silt till/silty sand till, silt, sandy silt, sand and gravel, and silty clay till subsoils at the depths of the monitoring well screen intervals. These results suggest that moderate groundwater seepage rates can be anticipated into open excavations below the shallow groundwater table.

Based on the follow up test pit investigation, performed at the anticipated depths for the housing basement foundations structures and proposed underground services indicates that the minor groundwater seepage rates within the open test pits excavations occurred at depths of 1.6 mbgs and <5.0 mbgs, or at elevations, ranging between 273.6 to 282.5 masl. Limited groundwater seepage was observed within test pit excavations, after the test pits remained opened for up to 6.0 hours. Review of the groundwater level elevations recorded at the test pits when compared to the concurrent groundwater level elevations within the monitoring wells indicates that the groundwater levels were about 0.44 to 3.39 m higher that the corresponding levels observed within the open test pits.

Given that limited un-sustained groundwater seepage is anticipated during excavations for the proposed underground housing basement structures, and for the installation of the underground service. It is not anticipated that the groundwater seepage will be sustained within the open excavations for the development areas, and occasional sump pit pumping should be adequate to remove any occasional limited water that may accumulate within the open excavations. Pumping rates for any anticipated occasional sump pit pumping are expected to be below the 50,000 L/day threshold limit for requiring an approval for any proposed construction related groundwater takings, which will not require any registration or filing with the MECP.

The long-term foundation drainage rate needs from a perimeter footing, drainage network for a conventionally side sloped foundation for each of the proposed housing basement structures for the proposed residential development areas, range from between 134.36 L/day to 488.58 L/day. By applying a safety factor of three (3), the drainage rates could reach maximums, ranging between 403.07 L/day to 1,465.74 L/day. The drainage estimates, above are considered very conservative and are unlikely to come to fruition give the low permeability and slow seepage rates for groundwater within the test pits as revealed from the recent test pit investigation. Any occasional seepage drainage to housing basements is likely to be un-sustained and may occur during spring thaw and following heavy rainfall events.

The shallow groundwater levels were measured at depths, ranging from 0.66 to 3.42 m below the prevailing ground surface. As such, low impact development (LID) infrastructure may be considered for implementation beneath certain portions of the site. If the shallow soils remain unsaturated, proposed Low Impact Development (LID) infrastructure should be considered for implementation in areas where the shallow groundwater is deeper than 1.0 m below the ground surface, and where it is possible to maintain a minimum 1.0 m separation between the bases for any proposed LID stormwater management infiltration infrastructure and the high groundwater table to address future stormwater management planning.

2.0 INTRODUCTION

2.1 Project Description

In accordance with authorization from Mr. Manoj Sharma of 2868577 Ontario Inc., Soil Engineers Ltd. (SEL) has conducted a hydrogeological assessment for a proposed residential development, for a site, located at 15544 McLaughlin Road in the Town of Caledon. The location of the subject site is shown on Drawing No. 1.

The subject site is currently a vacant land, located approximately 200 m west of McLaughlin Road and approximately 470 m north of Old Base Line Road, at the terminus of Kaufman Road. The subject site is surrounded by existing residential developments. The site slopes with its southwest portion being at higher elevations compared to its northeast portion. As per Drawing No. 1, a water course flows 70 m to the south, 50 m east and 325 m north of the site where it further contributes to the Credit River.

Based on the preliminary development plan, prepared by Candevcon Limited, the area will be developed into 13 single detached dwelling lots and a parkette.

This report summarizes the findings of the field study and the associated groundwater monitoring and testing programs and provides a description and characterization of the interpreted hydro-geo-stratigraphy for the subject site and the local surrounding area. The current study provides preliminary recommendations for any dewatering needs for construction, including an estimation for the construction dewatering flow rates and the associated zones of influence, prior to the detailed design. Furthermore, the report provides a recommendation for any need to acquire an Environmental Activity and Sector Registry (EASR), or to acquire aa Permit-To-Take Water (PTTW) as approvals to facilitate temporary groundwater taking for construction dewatering program, if required.

2.2 Project Objectives

The major objectives of this Hydrogeological Study Report are as follows:

1. Establish the local hydrogeological setting for the subject site, and the local surrounding area;
2. Interpretation of the shallow groundwater flow and runoff patterns;
3. Characterizing the hydraulic conductivity (K) for the groundwater-bearing shallow subsoil strata;

4. Estimate the anticipated, dewatering flows that may be required to lower the groundwater table to facilitate earthworks for the construction and for installation of underground services for proposed residential development, and assessment for any long-term foundation drainage needs following the site development, if required;
5. Identify zones of higher groundwater yield as potential sources for any ongoing shallow groundwater seepage;
6. Prepare an interpreted hydro-geo-stratigraphic cross-section across the subject site;
7. Evaluate potential impacts to nearby groundwater receptors within the anticipated zone of influence for construction dewatering;
8. Determine the groundwater function of the subject site, and assessment of potential impacts to nearby groundwater receptors relative to the proposed development;
9. Assess the shallow groundwater quality in advance of any construction dewatering, or for any anticipated long-term foundation drainage needs, after development, to assess disposal management options for use of the Region of Peel sewer system for any generated dewatering or drainage effluent;
10. Providing comments regarding any need to file for an Environmental Activity and Sector Registry (EASR) approval, or to acquire a Permit-To-Take Water (PTTW) approval to facilitate a temporary construction dewatering program.
11. Determine the feasibility of the subject site for the implementation of any Low Impact Development (LID) infrastructure to address future stormwater management planning and design for the proposed development.

2.3 **Scope of Work**

The scope of work for the Hydrogeological Study is summarized below:

1. Clearance of underground services, borehole drilling and installation of five (5) monitoring wells within the site’s development footprint.
2. Monitoring well development and groundwater level measurements at the five (5) installed monitoring wells.
3. Performance of Single Well Response Tests (SWRTs) at the installed monitoring wells to estimate the hydraulic conductivity (K) for the groundwater-bearing subsoil strata at the depths of the monitoring well screens.
4. Describing the geological and hydrogeological setting for the subject site, and the local surround area.
5. Review of the Ministry of the Environment, Conservation, and Parks (MECP) water well records within 500 m of the proposed development site.

6. Assessing the shallow groundwater quality to evaluate, disposal management options in advance of any dewatering effluent disposal management to the Region of Peel Storm and Sanitary system.
7. Review of available engineering development plans and profiles for the proposed development; assessing preliminary dewatering needs, and estimation of any anticipated dewatering flows to lower the groundwater levels to facilitate construction and earth works, or for any anticipated long-term foundation drainage needs following site development.
8. Providing comments, regarding any need to register any proposed groundwater-taking through an Environmental Activity and Sector Registry (EASR), or to apply for a Permit-To-Take Water (PTTW) as groundwater taking approvals.
9. Commenting on the suitability of the subsurface condition for implementing a LID infrastructure at the proposed developed site to address future stormwater management planning and design for the developed site.

3.0 METHODOLOGY

3.1 Borehole Advancement and Monitoring Well Installation

Borehole drilling and monitoring well construction were conducted on January 24, 2023. The program consisted of the drilling of five (5) boreholes (BHs) and the installation of five (5) monitoring wells (MW), one within each of five (5) boreholes drilled for the soil investigation report. The locations of the boreholes/monitoring wells are shown on Drawing No. 2.

The borehole drilling and monitoring well construction were completed by licensed water well contractor, DBW Drilling, under the full-time supervision of a field technician from SEL, who also logged the subsoil strata, encountered during borehole advancement, collected representative subsoil samples for textural classification, and supervised the monitoring well installations. The boreholes were drilled, using a continuous-flight, power auger machine, equipped with solid-stem augers. Selected subsoil samples, retrieved during the drilling program underwent laboratory grain size analysis to confirm the subsoil textures. Detailed descriptions of the encountered subsurface soil and groundwater conditions are presented on the borehole and monitoring well logs, Figures 1 to 5, inclusive.

The monitoring wells were constructed, using 50-mm diameter PVC riser pipes and screens, which were installed in each of the boreholes in accordance with Ontario Regulation (O. Reg.) 903. All of the monitoring wells were provided with steel, monument protective casings at the ground surface. Details for the monitoring well construction are provided on the enclosed Borehole Logs (Figures 1 to 5).

The ground surface elevations and horizontal coordinates at the monitoring well locations were determined at the time of the investigation, using a handheld Global Navigation Satellite System survey equipment (Trimble Geoexplorer unit TSC3) which has an accuracy of ±0.05 m. The UTM coordinates and ground surface elevations at the borehole/monitoring well locations, together with the summary of the monitoring well installation details, are provided in Table 3-1.

Table 3-1 - Monitoring Well Installation Details

Well ID	Installation Date	UTM Coordinates		Ground El. (masl)	Borehole Depth (mbgs)	Well Screen Interval (mbgs)	Well Casing Dia. (mm)
		East (m)	North (m)				
BH/MW 1	January 24, 2023	585730.94	4849365.40	285.81	6.3	3.1-6.1	50
BH/MW 2	January 24, 2023	585793.89	4849351.95	281.75	6.4	3.1-6.1	50
BH/MW 3	January 24, 2023	585781.60	4849417.52	282.83	6.2	3.2-6.2	50
BH/MW 4	January 24, 2023	585862.87	4849395.19	277.25	6.6	3.1-6.1	50
BH/MW 5	January 24, 2023	585827.02	4849464.92	278.64	6.2	3.2-6.2	50

Notes: mbgs -- metres below ground surface masl -- metres above sea level

3.2 Groundwater Monitoring

The groundwater levels within the monitoring wells were manually measured, on January 31, March 2 and on April 3, 2023 to record the fluctuation of the shallow groundwater table beneath the subject site, with the details discussed in the section 6.3 of this report.

3.3 Mapping of Ontario Water Well Records

SEL reviewed the Ministry of the Environment, Conservation and Parks (MECP) Water Well Records (WWRs) for the registered wells, located on the subject site and within 500 m of the subject site boundaries (study area). The water well records indicate that seventy-four (74) wells are located within the 500 m zone of influence study area relative to the subject site. The well record locations are marked, and presented in Drawing No. 3, and related WWRs review information is summarized in Section 6.2, with details of the reviewed records being provided in Appendix ‘A’.

3.4 Monitoring Well Development and Single Well Response Tests

The monitoring wells underwent development in preparation for single well response tests (SWRT) to estimate the hydraulic conductivity (K) for saturated subsoil strata at the depths of the monitoring well screens. Well development involved the purging and removal of several well casing volumes of groundwater from each monitoring well to remove remnants of clay, silt and other debris introduced into the monitoring wells during construction, and to induce the flow of formation groundwater through the monitoring well screens, thereby improving the transmissivity of the subsoil strata formation at the monitoring well screen depths.

The test results from SWRT's are used to estimate the hydraulic conductivity (K) for groundwater-bearing subsoil strata at the depths of the monitoring well screens. The K values, estimated from the SWRTs provide an indication of the yield capacity for the groundwater-bearing subsoil strata, and can be used to estimate the flow of groundwater through the groundwater-bearing subsoil strata.

The SWRT involves the placement of a slug of known volume into the well, below the groundwater table, to displace the groundwater level upward. The rate at which the groundwater level recovers to static conditions (falling head) was tracked using a data logger/pressure transducer that was set to record water level data at 5 second recording intervals. An electronic water level tape was also used to manually record the groundwater levels to verify the data logger measurements.

The rate at which the groundwater table recovers to static conditions is used to estimate the K values for the groundwater-bearing subsoil strata formation at the monitoring well screen depths. The Bower Rice formula was used to interpret the SWRTs. The BH/MWs 1, 2 and 3 underwent SWRTs on March 2, 2023, whereas SWRTs on BH/MWs 4 and 5 were performed on April 3, 2023. The detailed test results are provided in Appendix 'B', with a summary of the findings, being provided in Table 6-2.

3.5 **Review Summary of Concurrent Report**

The following, concurrent geotechnical report, prepared by SEL was reviewed in preparation of this hydrogeological study:

"A Report to 2868577 Ontario Inc., a Geotechnical Investigation for Proposed Residential Development, 15544 McLaughlin Road, Town of Caledon", Reference No. 2301-S042 dated March 2023.

3.6 **Groundwater Quality Assessment**

The monitoring well location at the BH/MW 1 underwent sampling for analysis to characterize the shallow groundwater quality for comparison evaluation of the testing results against the Region of Peel Storm and Sanitary Sewer Use By-Law standards. This was performed to assess whether any anticipated dewatering effluent, generated from any construction dewatering, or from any long-term foundation drainage needs can be disposed of into the Region of Peel sewer system. Based on the results, recommendations for any pre-treatment of the dewatering effluent can be developed, if required.

BH/MW 1 was developed and purged in accordance with best management practices with a minimum of 3 well casing volumes of groundwater purged, prior to sample collection. In accordance with Region of Peel Storm and Sanitary Sewer Use By-Law sampling protocol, one set of groundwater samples was not filtered prior to placement in the laboratory sample bottles. Upon sampling, all of the bottles were placed in ice and packed in a cooler for shipment to the analytical laboratory. Sample analysis was performed by SGS Laboratories, which is accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA). Results of the analysis are provided in Appendix ‘C’, with a discussion of the findings, provided in Section 7.6.

4.0 REGIONAL AND LOCAL SETTING

4.1 Regional Geology

The subject site lies within the Physiographic Region of Southern Ontario, known as the Niagara Escarpment. The Niagara Escarpment extends from the Niagara River to the northern tip of the Bruce Peninsula and continues through the Manitoulin Islands. It consists of an association of landforms, not found anywhere else in Ontario. Vertical cliffs along the brow of the escarpment often outlines the edge of the dolostone of the Lockport and Amabel Formations while the slopes below are carved in red shale. For some distance back from the brow, the dip-slope of the cuesta in many places has been stripped of soil and over-burden. Flanked by landscapes of glacial origin, this rock-hewn topography stands in striking contrast, and its steep-sided valleys are strongly suggestive of non-glaciated regions. While the escarpment stands out boldly in the Niagara Peninsula, and along the shore of Georgian Bay, there is an intervening area in which the slopes are mantled by morainic posits, particularly in Mono and Mulmur Townships, and the Town of Caledon, with long stretches of area being almost completely hidden.

The Dundas Valley is the most notable break in the southern part of the escarpment, extending inland eight miles from the west end of Lake Ontario. The rim is sharply outlined by rock bluffs but within the valley there is deep drift, the surface of which is deeply cut by many gullies. Worthy of note is the occurrence of beds of sand and silty clay in alternate layers (Chapman and Putnam, 1984).

Under the Niagara Escarpment, the physiographic description for the project site is Spillways. These are usually occupied by streams, and are basically a broad trough, floored wholly or in part by gravel beds at one or more levels. It sometimes shows a peculiar disregard for existing grades, since it flowed along an ice front. It is common to find a spillway that now is unoccupied by any stream. On the upland west of the Niagara Escarpment the spillways mostly, but not always, run along the front of the moraines (Chapman and Putnam, 1984).

Review of the surface geological map of Ontario shows that the subject site is located, partially on the Halton Till Unit deposits, consisting predominantly of silt to silty clay matrix which is high in calcium carbonate content, and is clast poor, which was deposited, partially on the bedrock deposits, consisting of undifferentiated igneous and metamorphic rocks, or carbonate and classic sedimentary rocks, being exposed at the surface or covered by a discontinuous, thin layer of drift. Drawing No. 4, as reproduced from Ontario Geological Survey (OGS) mapping, illustrates the Quaternary surface soil geology for the site and surrounding area.

The underlying bedrock is comprised, mainly of shale, limestone, dolostone and siltstone of the Georgian Bay formation, Blue Mountain Formation, Billings Formation, and both the Collingwood and Eastview Member, which were deposited during the Upper Ordovician Epoch (Bedrock Geology of Ontario, 1993). The approximate elevations for the top of the bedrock beneath the site approximately ranges between 267 to 278 masl (metres above sea level).

4.2 Physical Topography

A review of the topography shows that the subject site and surrounding area is sloping in nature, exhibiting a decline in elevation relief towards the east from west, towards the Credit River. Based on review of the topographic map, and from the review of the ground surface elevations at borehole and monitoring well locations, the total elevation relief across the subject site is about 9.0 m. Drawing No. 5 shows the mapped topographical contours for the subject site, and the surrounding area.

4.3 Watershed Setting

The subject site is located within the Credit Valley Watershed, and Credit River-Forks of the Credit to Churchville Sub-watershed as shown, mapped on Drawing No. 6. The Credit River watershed is comprised of twenty-three (23) sub-watersheds and covers an area of about 1,000 km^2. The Credit River is approximately 90 km long and meanders through nine (9) municipalities. Its headwaters, or upper reaches, are located in Orangeville, Erin and in the Town of Mono. It flows south where it empties into Lake Ontario at Port Credit, Mississauga (Credit Valley Conservation Authority, 2009).

4.4 Local Surface Water and Natural Features

Records review show that a tributary of Credit River, and its associated wooded areas and a watercourse are located, immediately south and south-west of the site. This tributary is shown to flow south-easterly, before bending east where it then joins the Credit River, located approximately 50 m south of the subject site. Another small tributary, flowing north joins the Credit River, approximately 300 m north of the site.

Immediately south-west of the site lies a wooded area, and a further 30 m southwest of the site lies an area of natural and scientific interest (ANSI). Apart from these, there are a lot of wooded areas present around the site. The locations of the site and the noted natural features are shown on Drawing No. 7.

5.0 SOIL LITHOLOGY

The investigation revealed that beneath a layer of topsoil veneer, and a layer of earth fill or weathered soil, the site is underlain by native strata of silt, silty sand, sandy silt, silty sand till, sandy silt till and silty clay till. Weathered shale was also observed in some of the BH/MWs at deeper elevations.

Detailed descriptions of the encountered subsurface conditions from the BH/MWs are presented on the BH/MW Logs, comprising Figures 1 to 5, inclusive. A Key Plan and the interpreted geological cross-sections, along the delineated southwest to northeast and southwest to southeast transects across the site are presented on Drawing Nos. 8-1 and 8-2.

5.1 **Topsoil** (All BH/MWs)

All BH/MWs were completed on the vacant field where the ground surface is covered with a layer of topsoil, approximately 15 to 30 cm in thickness. Thicker topsoil deposits may be encountered beyond the BH/MW locations.

5.2 **Earth Fill** (BH/MWs 3, 4 and 5)

Earth fill, approximately 0.2 to 2.2 m thickness, was observed beneath the topsoil layer at BH/MWs 3, 4 and 5 locations. The fill unit consists of mixture of sand, silt, clay and contains organic inclusions.

5.3 **Silt, Sandy Silt and Silty Sand** (BH/MWs 1, 2, 3 and 5)

The silt, sandy silt and silty sand deposits were encountered in BH/MWs 1, 2, 3 and 5. It has trace of clay and occasional gravel. It is brown in colour, is very loose to compact in consistency. The moisture contents for the retrieved subsoil samples ranges from 7% to 33%, indicating moist to wet conditions. The estimated permeability of this layers at the depth of 3.3 mbgs, 4.8 mbgs and 1.8 mbgs ranges from 10^{-4} to 10^{-3} cm/sec. Grain size analyses were performed on three (3) subsoil samples, and the gradation are plotted on Figures 6, 7 and 8.

5.4 **Sandy Silt Till/Silty Sand Till** (BH/MWs 1, 2 and 4)

The sandy silt till and/or silty sand till were contacted in the upper stratigraphy in BH/MWs 1 and 2 at depths of 0.3 to 2.2 m below the prevailing ground surface. With an approximate thickness ranging from 1.4 to 1.9 m. While at BH/MW 4, sandy silt till layer was encountered at a depth of 4.8 m below the prevailing ground surface. It is brown in colour, is very loose to very dense in consistency, having trace of clay and gravel. The moisture contents for the retrieved subsoil samples ranges from 10 to 36%, indicating moist to saturated conditions.

5.5 **Sand and Gravel** (BH/MWs 4 and 5)

The sand and gravel deposits were encountered in BH/MWs 4 and 5 beneath the eastern portion of the investigated area, at the approximate depth of 2.2 m below the prevailing ground surface. Having an approximate thickness of 1.7 to 2.6 m. This subsoil unit is brown in colour, is dense in consistency, having a trace to some silt. The moisture contents for the retrieved subsoil samples ranges from 3% to 18%, indicating moist condition. The estimated permeability of this layer at the depth of 3.3 mbgs is 10^{-3} cm/sec. Grain size analysis was performed on one representative subsoil sample of the sand and gravel, and the soil gradation is plotted on Figure 9.

5.6 **Silty Clay Till** (BH/MWs 3, 4 and 5)

The silty clay till deposit was encountered at the lower stratigraphy in BH/MWs 3, 4 and 5, at depths, ranging from 4.0 to 6.3 m below prevailing ground surface. It has a trace of gravel and occasional shale fragments. It is brown in colour, hard in consistency, where it extends to the maximum investigation depth at BH/MWs 3 and 4. The moisture contents for the retrieved subsoil samples ranges from 9 to 16% indicating moist conditions.

5.7 **Shale** (BH/MW 5)

Shale bedrock was encountered at the depth of 5.7 m below the prevailing ground surface, at the BH/MW 55 location. It is grey in colour, it is weathered. It extends to the termination depth of investigation of 6.2 mbgs. The permeability of the underlying upper shale unit is anticipated to vary depending on the extent of fracturing, and presence of bedding planes.

6.0 GROUNDWATER STUDY

6.1 Review Summary of Concurrent Report

A review of the findings from the concurrent geotechnical soil investigation report (SEL Reference No. 2301-S042) has disclosed that beneath the topsoil horizon, and a layer of earth fill or weathered subsoil, the subject site is underlain by native strata of silt, silty sand, sandy silt, silty sand till, sandy silt till and silty clay till. Weathered shale was observed in one of the boreholes at deeper elevation.

6.2 Review of Ontario Water Well Records

The Ministry of the Environment, Conservation and Parks (MECP) water well records for the subject site and for the properties within a 500 m radius of the boundaries of the subject site (study area) were reviewed.

The records indicate that seventy-four (74) well records are located within the study area relative to the subject site. The locations of these well records, based on the UTM coordinates provided by the records, are shown on Drawing No. 3. Details for the MECP water well records that were reviewed are provided in Appendix 'A'.

A review of the final status of the well records within the study area reveals that thirty-four (34) are registered as water supply wells, twenty-four (24) are abandoned – other wells, seven (7) are observation wells, five (5) wells have an unidentified status, two (2) are test hole wells, one (1) is an abandoned-supply well, and one (1) dewatering well.

A review of the first usage of the well records reveals that thirty-one (31) are domestic wells, twenty-three (23) wells have an unidentified status, five (5) are monitoring wells, five (5) are dewatering wells, three (3) wells are not being used, two (2) wells are used for livestock, one (1) of each is registered as a test hole well, public, municipal, industrial, and other use well, respectively.

Should there be any water supply wells discovered during the future site grading operations, we recommend that they be properly decommissioned in accordance with the Ontario Water resources Act, Regulation 903.

6.3 Groundwater Monitoring

The groundwater levels within the monitoring wells were measured, manually on three occasions over the study period, on the following dates; January 31, March 2, and on April 3, 2023, to record the fluctuation of the shallow groundwater table beneath the subject site. The groundwater levels and their corresponding elevations are given below in Table 6-1.

Table 6-1 - Groundwater Level Measurements

Well ID		January 31, 2023	March 02, 2023	April 03, 2023	Average (m)	Fluctuation (m)
BH/MW 1	mbgs	3.04	2.14	1.13	2.10	1.91
	masl	282.77	283.67	284.68	283.71	
BH/MW 2	mbgs	3.52	2.20	0.66	2.13	2.86
	masl	278.23	279.55	281.09	279.62	
BH/MW 3	mbgs	3.56	2.78	2.11	2.82	1.45
	masl	279.27	280.05	280.72	280.01	
BH/MW 4	mbgs	4.93	4.17	3.42	4.17	1.51
	masl	272.32	273.08	273.83	273.08	
BH/MW 5	mbgs	2.07	1.39	0.93	1.46	1.14
	masl	276.57	277.25	277.71	277.18	

Notes: mbgs -- metres below ground surface masl -- metres above sea level

As shown above, the groundwater levels within all of the BH/MW locations generally increased over the monitoring period. As shown above the groundwater levels at the BH/MWs range from the depths of between 0.66 to 3.56 m below ground surface. The greatest fluctuation was recorded at BH/MW 2, where a 2.86 m difference in groundwater elevation level was documented during the monitoring period.

6.4 Shallow Groundwater Flow Pattern

The shallow groundwater flow pattern beneath the subject site was interpreted, based on the highest shallow groundwater levels measured at all the BH/MWs, suggesting that it flows in an eastern direction, beneath the site, towards the low relief portions of the property. The flow pattern interpretation was completed within the proposed development footprint area. The interpreted shallow groundwater flow pattern beneath the subject site is illustrated on Drawing No. 9.

6.5 Single Well Response Test Analysis

All of the BH/MWs underwent a single well response test (SWRT) to assess the hydraulic conductivity (K) for saturated aquifer subsoils at the depths of the monitoring well screens. The results for the SWRTs are presented in Appendix 'B', with a summary of the findings shown in Table 6-2.

Table 6-2 - Summary of SWRT Results

Well ID	Ground El. (masl)	Monitoring Well Depth (mbgs)	Borehole Depth (mbgs)	Well Screen Interval (mbgs)	Screened Sub Soil Strata	Hydraulic Conductivity (K) (m/sec)
BH/MW 1	285.81	6.1	6.3	3.1-6.1	Sandy Silt Till/ Silty Sand Till	4.0×10^{-6}
BH/MW 2	281.75	6.1	6.4	3.1-6.1	Silt	1.7×10^{-6}
BH/MW 3	282.83	6.2	6.2	3.2-6.2	Sandy Silt/ Silty Clay Till	1.1×10^{-6}
BH/MW 4	277.25	6.0	6.0	3.1-6.1	Sandy Silt/ Silty Clay Till	6.0×10^{-7}
BH/MW 5	278.64	6.2	6.2	3.2-6.2	Sand and Gravel/ Silty Clay Till	3.5×10^{-6}

Notes: mbgs -- metres below ground surface masl -- metres above sea level

As shown above, the K estimates for the silt, silty sand till, silty clay unit ranges from 6.0×10^{-7} to 4.0×10^{-6} m/sec. The results of the SWRT's provide an indication of the yield capacity for the groundwater-bearing subsoil strata at the depths of the monitoring well screens. The above results suggest that the hydraulic conductivity (K) for the groundwater-bearing subsoils at the depths for the monitoring well screens ranges from low to moderate, with correspondingly low to moderate anticipated groundwater seepage rates being anticipated into open excavations, below the groundwater table.

6.6 Follow Up Test Pit Investigation

On May 30, 2023, a Soil Engineers Ltd. representative performed a site visit to witness a test pit investigation program. Test pit excavations were completed for the subject, at the locations, shown on Drawing No. 2. For the test pit investigation, a backhoe sub-contractor excavated to the target depths, at the indicated test pit locations that were provided in advance by Candevcon Limited. Detailed findings of the test pit investigation are provided in Appendix 'D'.

Based on the test pit observations, no groundwater seepage was observed in one (1) of the test pits, while minimal seepage was observed within three (3) open test pits excavations, with only low to moderate groundwater seepage being observed within one (1) of the open test pit excavations, along with only minimal accumulation of groundwater within the open test pits after about the test pits remained open for about ±4 to 6 hours. This indicates that there is likely to be only limited, low to minor, un-stained groundwater seepage within open excavations at the anticipated depths for the proposed underground services and proposed housing basement structures, with only minimal, occasional groundwater control being anticipated, if that. Any groundwater control can likely be accomplished with occasional pumping from sump pits if required with no approval for any temporary groundwater taking being anticipated in advance of construction.

7.0 GROUNDWATER CONTROL DURING CONSTRUCTION

The estimated hydraulic conductivity (K) for the sandy silt till, silty clay and silty clay till units suggest that groundwater seepage rates into open excavations below the groundwater table will range from moderate to low. To provide safe, dry and stable conditions for earthworks excavations for construction of the proposed underground housing foundation structures and associated underground services, the groundwater table should be lowered in advance of, or, during construction. Preliminary estimates for construction dewatering flows required to locally lower the shallow groundwater table, based on the SWRT, K test estimates, are discussed in the following sections.

7.1 Groundwater Construction Dewatering Flow Rates

A proposed preliminary development plan, prepared by Candevcon Limited, Project No. W22002, dated May 16, 2022 was reviewed for this preliminary dewatering needs assessment. Since the finished floor elevations (FFE) were not available for review at the time of preparation of this report. The BH/MW location elevations, and existing ground elevation contours were considered as the grade elevations and were used to prepare the dewatering needs assessment. Based on review of the plan, the proposed development will comprise 13 single detached dwelling lots and a parkette, along with associated roads and municipal services and infrastructure, meeting urban standards. It is assumed that all of the proposed residential units are anticipated to have basement structures.

7.1.1 Groundwater Construction Dewatering Rates for the Construction of Proposed Detached Dwellings with Basement Structures

Based on the provided conceptual development plan, the dimensions for each detached dwelling were provided as 18.3 m wide and 42.8 m in length. As such, for the current dewatering needs assessment, the anticipated excavation areas for the construction of underground housing basement structures were considered at 20 x 10 m in dimensions, with total anticipated excavated area for each lot being about 200.0 m^2.

Dewatering Assessment for the Construction of Underground Basement for the Detached House Development in the Vicinity if Lot 1 (Test Pit 1), at a Grade Elevation of approximately 285.8 masl:

For the proposed detached house basement structure, having a dimension of 20.0 m x 10.0 m within the southwestern part of the site, at an estimated grading plan elevation of 285.8 masl which was considered with an assumed excavation depth of up to ±3.0 m for the underground basement structures. The estimated depth elevation for the construction of the detached housing basement structure base was estimated at 282.8 masl. The subsoil comprises topsoil, sandy silt, and sandy silt till/silty sand till extending to the maximum proposed depths for excavation. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the groundwater table be lowered to an elevation of 281.8 masl, which is about 1 m below the lowest proposed excavation depth. Minimal groundwater seepage was observed at the TP 1 at an elevation of about 282.50 masl. There was only a slight accumulation of groundwater seepage observed within the test pit after it had been left open for a duration of ±5.0 hours. Given the low permeability of the underlying, compact to very dense, sandy silt till/silty sand till, having a trace of clay unit, and the limited un-sustained groundwater seepage as observed within the test pit, it is anticipated that occasional sump pit pumping may be required to remove any limited groundwater seepage within open excavation within this portion of the subdivision, where any occasional encountered groundwater seepage is likely to be un-sustained.

Dewatering Assessment for the Construction of Underground Basement for the Detached House Development in the Vicinity if Lot 13 (Test Pit 2), at a Grade Elevation of approximately 281.7 masl:

For the proposed detached house basement structure having a dimension of 20.0 m x 10.0 m within the southwestern part of the site, at an estimated grading plan elevation of 281.7 masl was considered with an assumed excavation depth of up to ±3.0 m for the underground basement structure. The estimated depth elevation for the construction of the detached housing basement structure base was estimated at 278.7 masl. The subsoil comprises topsoil, sandy silt till, and silt extending to the maximum proposed depths for excavation. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the groundwater table be lowered to an elevation of 277.7 masl, which is about 1 m below the lowest proposed excavation depth.

No accumulation of groundwater seepage was observed within the test pit 2 after it had been left open for a duration of ±4.0 hours. Given that the underlying, compact to very dense, silt unit, exhibiting limited, un-sustained groundwater seepage, it is anticipated that occasional sump pit pumping may be required to remove any limited groundwater seepage within open excavations, if required, where only occasional groundwater seepage is likely to be un-sustained.

Dewatering Assessment for the Construction of Underground Basement for the Detached House Development in the Vicinity if Lot 10 (Test Pit 3), at a Grade Elevation of approximately 282.8 masl:

For the proposed detached house basement structure having a dimension of 20.0 m x 10.0 m within the southwestern part of the site, at an estimated grading plan elevation of 282.8 masl was considered with an assumed excavation depth of up to ±3.0 m for the underground basement structure. The estimated depth elevation for the construction of the detached housing basement structure base was estimated at 279.8 masl. The subsoil comprises topsoil, earth fill, silty sand, silt, sandy silt and silty clay till extending to the maximum proposed depths for excavation. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the groundwater table be lowered to an elevation of 278.8 masl, which is about 1 m below the lowest proposed excavation depth. Minimal groundwater seepage was observed at the TP 3, at an elevation of about 281.4 masl. There was only a slight accumulation of groundwater seepage observed within the test pit after it had been left open for a duration of ±6.0 hours. Given the low permeability of the underlying, compact, silt, sandy silt, having a trace of clay, which exhibited limited un-sustained groundwater seepage within the open test pit, it is anticipated that occasional sump pit pumping may be required to remove any limited groundwater seepage within open excavations, where any occasional encountered groundwater seepage is likely to be un-sustained.

Dewatering Assessment for the Construction of Underground Basement for the Detached House Development in the Vicinity if Lot 4 (Test Pit 4), at a Grade Elevation of approximately 277.2 masl:

For the proposed detached house basement structure having a dimension of 20.0 m x 10.0 m within the southwestern part of the site, at an estimated grading plan elevation of 277.2 masl was considered with an assumed excavation depth of up to ±3.0 m for the underground housing basement structure. The estimated depth elevation for the construction of the detached housing basement structure base was estimated at 274.2 masl. The subsoil comprises topsoil, earth fill, sand and gravel, sandy silt till and silty clay till extending to the maximum proposed depths for excavation. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the groundwater table be lowered to an elevation of 273.2 masl, which is about 1 m below the lowest proposed excavation depth.

Medium to minor groundwater seepage was observed at the TP 4, at an elevation of about 273.8 masl. There was a limited medium to minor accumulation of groundwater observed within the test pit after it had been left open for a duration of ±4.0 hours. Given the underlying, dense, sand with gravel and trace do some silt unit, which exhibited limited un-sustained groundwater seepage as observed within the test pit, it is anticipated that occasional sump pit pumping may be required to remove any limited groundwater seepage within open excavations, where any occasional encountered groundwater seepage is likely to be un-sustained.

<u>Dewatering Assessment for the Construction of Underground Basement for the Detached House Development in the Vicinity if Lot 7 (Test Pit 5), at a Grade Elevation of approximately 278.6 masl:</u>

For the proposed detached house basement structure having a dimension of 20.0 m x 10.0 m within the southwestern part of the site, at an estimated grading plan elevation of 278.6 masl was considered with an assumed excavation depth of up to ±3.0 m for the underground basement structure. The estimated depth elevation for the construction of the detached housing basement structure base was estimated at 275.6 masl. The subsoil comprises topsoil, sandy silt till, and silt extending to the maximum proposed depths for the excavation. To facilitate excavation and construction in dry and stable subsoil conditions, it is proposed that the groundwater table be lowered to an elevation of 274.6 masl, which is about 1 m below the lowest proposed excavation depth.

Limited groundwater seepage was observed at TP 5 at an elevation of about 273.65 masl. There was a minor accumulation of groundwater seepage observed within the test pit after it had been left open for a duration of ±4.0 hours. Given the underlying, dense to hard, sand and silty clay till unit, exhibiting limited un-sustained groundwater seepage as observed within the open test pit, it is anticipated that occasional sump pit pumping may be required to remove any limited groundwater seepage within the open excavations, where any occasional encountered groundwater seepage is likely to be un-sustained.

7.1.2 <u>Groundwater Construction Dewatering Rates for the Installation of Proposed Underground Services</u>

The proposed excavation depths were not available for review at the time of preparation of this current report. As such, the bases for proposed installation of services have been considered at depths of 5.0± m beneath the existing grade surface elevations.

Installation of Underground Services beneath the Southwestern Portion of the Site, at an Estimated Grade Elevation of 285.8 masl at Test Pit 1 location:

The dewatering needs assessment was based on the lowest proposed servicing invert depths, considered at a maximum depth of about 5.0 m below the estimated grade elevation of 285.8 masl. An estimated services installation excavation depth elevation of 280.8 masl was utilized for this portion of the site. Minimal groundwater seepage was observed at the TP 1 at an elevation of about 282.50 masl. There was only a slight accumulation of groundwater seepage observed within the test pit after it had been left open for a duration of ±5.0 hours. Given the low permeability of the underlying, compact to very dense, sandy silt till/silty sand till having a trace of clay unit, which exhibited, limited un-sustained groundwater seepage as observed within the open test pit, it is anticipated that occasional sump pit pumping may be required to remove any limited groundwater seepage within open services excavations, where any occasional encountered seepage is likely to be un-sustained.

Installation of Underground Services beneath the Center Portion of the Site, at an Estimated Grade Elevation of 282.8 masl at Test Pit 3 location:

The dewatering needs assessment was based on the lowest proposed servicing invert depths, considered at a maximum depth of about 5.0 m below the estimated grade elevation of 282.8 masl. The estimated installation excavation depth elevation of 277.8 masl was utilized for this portion of the site. Minimal ground seepage was observed at the TP 3, at an elevation of about 281.4 masl. There was only a slight accumulation of groundwater observed within the open test pit after it had been left open for a duration of ±6.0 hours. Given the low permeability of the underlying, compact, silt, sandy silt having trace of clay, and the limited un-sustained groundwater seepage as observed within the test pit, it is anticipated that occasional sump pit pumping may be required to remove any limited groundwater seepage within open service excavations, where any occasional encountered seepage is likely to be un-sustained.

Installation of Underground Services beneath the Northern Portion of the Site, at an Estimated Grade Elevation of 278.6 masl at Test Pit 5 location:

The dewatering needs assessment was based on the lowest proposed servicing invert depths, considered at a maximum depth of about 5.0 m below the estimated grade elevation of 278.6 masl. The estimated installation excavation depth elevation of 273.6 masl was utilized for this portion of the site. Limited groundwater seepage was observed at TP 5 at an elevation of about 273.65 masl. There was only a minor accumulation of groundwater observed within the test pit after it had been left open for a duration of ±4.0 hours. Given the underlying, dense to hard, sand and silty clay till unit, exhibiting limited un-sustained groundwater seepage as observed within the open test pit, it is anticipated that occasional sump pit pumping may be required to remove any limited groundwater seepage within open services excavations, where only occasional encountered groundwater seepage is likely to be un-sustained.

The groundwater seepage rates are anticipated to be below the 50,000 L/day threshold limit for requiring an approval for any proposed construction related groundwater takings and which will not require any registration or filing with the MECP.

7.2 Groundwater Control Methodology

The groundwater seepage rates into open excavations within sandy silt, sandy silt till/silty sand till, silt, silty sand, and sand and gravel subsoils below the groundwater levels are expected to range from low to moderate. Pumping from the sump pits may be adequate to control local groundwater seepage into excavations. Well points could be considered if e encountered groundwater seepage and stable subsoil conditions cannot be controlled by localized sump pit dewatering within the excavation footprints or underground servicing trenches. The final design for any temporary construction dewatering system will be the responsibility of the construction contractors.

7.3 Permanent Foundation Drainage for Underground Structures

The proposed development plan indicates that it is anticipated to construct a residential subdivision, consisting of thirteen (13) single detached housing units, for the proposed development. Each house unit is anticipated to be completed with underground basement structure. It is anticipated that limited, occasional long-term foundation drainage needs may be required for each of the proposed underground housing basement structures, following site development.

Conventional perimeter footings drains can be included for the design for each of the house footings to address any long-term groundwater seepage to the excavation and completed underground housing basement structures, to address the limited, occasional groundwater seepage to the proposed housing basement structures.

In order to estimate the long-term foundation dewatering needs associated with a perimeter foundation drainage network at the subject site, Darcy's Equation was used, as described below:

Perimeter Drainage for a Single-Detached House Lot Size (10 m x 20 m) in the Vicinity of BH/MW 1 at an estimated grade elevation of 285.80 mal

$$Q = KiA$$

Where:

Q = Estimated seepage, drainage rate (m^3/day)

K = 4.0×10^{-6} m/sec (highest hydraulic conductivity (K) assessed for the Sandy Silt Till/Silty Sand Till unit encountered during the study)

A = 18.85 m^2 for the surface area of weepers around the perimeter of foundation footings

ih = 0.075 [unitless], Horizontal Hydraulic Gradient for groundwater considered for the perimeter footing drainage system.

The drainage estimates, above are considered very conservative and are unlikely to come to fruition give the low permeability and slow seepage rates for groundwater within the test pits within the Sandy Silt Till/Silty Sand Till rich subsoil as revealed from the recent test pit investigation. Any occasional seepage drainage to housing basement structures is likely to be un-sustained and may occur during spring thaw, and following heavy rainfall events.

Based on the proposed underground basement structure, the long-term seepage drainage rate for the perimeter drainage network for a conventionally side-sloped excavation is 488.58 L/day. By applying a safety factor of three (3), the drainage rate could reach a maximum of about 1,465.74 L/day.

Perimeter Drainage for a Single-Detached House Lot Size (10 m x 20 m) in the Vicinity of BH/MW 2 at an estimated grade elevation of 281.70 masl

$$Q = KiA$$

Where:

Q = Estimated groundwater seepage, drainage rate (m^3/day)

K = 1.7×10^{-6} m/sec (highest hydraulic conductivity (K) assessed for the silt unit encountered during the study)

A = 18.85 m^2 for the surface area of weepers around the perimeter of foundation footings

ih = 0.075 [unitless], Horizontal Hydraulic Gradient for groundwater considered for the perimeter footing drainage system.

Based on the proposed underground basement structure, the long-term groundwater seepage drainage rate for the perimeter drainage network for a conventionally side-sloped excavation is 207.65 L/day. By applying a safety factor of three (3), the drainage rates could reach maximum 622.94 L/day.

The drainage estimates, above are considered very conservative and are unlikely to come to fruition give the low permeability and slow groundwater seepage rate within the test pits for the silt rich subsoil as revealed from the recent test pit investigation. Any occasional seepage drainage to housing basements is likely to be un-sustained and may occur only during spring thaw and following heavy rainfall events.

Perimeter Drainage for a Single Detached House Lot Size (10 m x 20m) at an estimated grade elevation and in the Vicinity of BH/MW 3 (282.80 masl).

$$Q = KiA$$

Where:

Q = Estimated seepage, drainage rate (m^3/day)

K = 1.1×10^{-6} m/sec (highest hydraulic conductivity (K) assessed for the Sandy Silt/Silty Clay Till unit encountered during the study)

A = 18.85 m^2 for the surface area of weepers around the perimeter of foundation footings

ih = 0.075 [unitless], Horizontal Hydraulic Gradient for groundwater considered for the perimeter footing drainage system.

Based on the proposed underground basement structure, the long-term seepage drainage rate for the perimeter drainage network for a conventionally side-sloped excavation is 134.36 L/day. By applying a safety factor of three (3), the drainage rates could reach maximum 403.07 L/day.

The drainage estimates, above are considered very conservative and are unlikely to come to fruition give the low permeability and slow seepage rates for groundwater within the open test pits having sandy silt/silty clay rich till subsoil as revealed from the recent test pit investigation. Any occasional seepage drainage to housing basements is likely to be un-sustained and may occur during spring thaw and following heavy rainfall events.

The long-term foundation drainage estimates for complete housing are considered conservative based on findings of the recent test pit investigation which suggests minimal occasional foundation drainage for complete housing basements.

Perimeter Drainage for a Single Detached House Lot Size (10 m x 20m) at an estimated grade elevation and in the Vicinity of BH/MW 4 (277.2 masl)

Based on the proposed underground basement structure, no long-term groundwater seepage drainage is required in the vicinity of BH/MW 4.

Perimeter Drainage for a Single Detached House Lot Size (10 m x 20m) at an estimated grade elevation and in the Vicinity of BH/MW 5 (278.60 masl).

$$Q = KiA$$

Where:

Q	=	Estimated seepage, drainage rate (m^3/day)
K	=	3.5×10^{-6} m/sec (highest hydraulic conductivity (K) assessed for the Sand and Gravel/Silty Clay Till unit encountered during the study)
A	=	18.85 m^2 for the surface area of weepers around the perimeter of foundation footings
ih	=	0.075 [unitless], Horizontal Hydraulic Gradient for groundwater considered for the perimeter footing drainage system.

Based on the proposed underground basement structure, the long-term groundwater seepage drainage rate for the perimeter drainage network for a conventionally side-sloped excavation is 427.51 L/day. By applying a safety factor of three (3), the drainage rates could reach maximum 1,282.5 L/day.

The drainage estimates, above are considered very conservative and are unlikely to come to fruition give the low permeability and slow seepage rates of groundwater within the test pits within the Sand and Gravel/Silty Clay Till rich subsoil as revealed from the recent test pit investigation. Any occasional seepage drainage to housing basements is likely to be un-sustained and may occur during spring thaw and following heavy rainfall events.

The long-term foundation drainage estimates for complete housing are considered conservative based on findings of the recent test pit investigation which suggests minimal occasional foundation drainage for complete housing basements.

7.4 Mitigation of Potential Impacts Associated with Dewatering

There is a record of one domestic water supply well and one abandoned supply well, located on the property. These well are identified as Well ID. Nos. 10 and 34, on MECP Well Location Plan, Drawing No. 3 and are listed in Appendix 'A'. It is recommended that the two wells that are located within the site be decommissioned in advance of construction should it still exist. Records review indicate that a tributary of Credit River and its associated wooded areas are located, about 50 m south of the subject site.

There should be no anticipated concerns associated with potential ground settlement to any existing nearby structures, infrastructure or natural heritage features. It is recommended that a geotechnical engineer should be consulted to review potential ground settlement concerns to nearby structures prior to construction.

7.5 **Groundwater Function for the Subject Site**

The proposed development will consist of a residential housing development along with associated underground services and utilities and a park. Any occasional sump pumping will be temporary with no potential impacts to groundwater receptors including any nearby supply wells being used in the area.

The subject site is currently comprised of a vacant land. Surrounding land uses includes residential development, Kaufman Road, Victoria Street and McKenzie Street. Furthermore, there is a tributary of Credit River, located about 50 m south of the site, along with wooded area. As such, the local shallow groundwater flow pattern for the area may be locally impacted on temporary basis from the proposed development.

Any construction dewatering will be temporary with low anticipated dewatering flow rates, and any long-term foundation drainage rates for the completed housing basement structures is anticipated to be only occasional, low and un-sustained.

7.6 **Ground Settlement**

It is recommended that the potential ground settlement concerns associated with any temporary construction dewatering should be assessed by a geotechnical engineer, prior to earthworks and construction.

7.7 **Groundwater Quality**

One set of groundwater samples were collected for analysis from the monitoring well at BH/MW 1, on April 3, 2023 using a dedicated sampling bailer. The monitoring well was purged of three (3) well casing volumes of groundwater prior to sample collection. Upon sampling, all of the sample bottles were placed in ice and packed in a cooler at about 4^{o} C for shipment to the analytical laboratory. The groundwater sample was submitted for analysis for comparison evaluation of the results against the Peel Region storm and sanitary sewer use by-law standards, and the Provincial Water Quality Objectives (PWQO) standards. Sample analysis was performed by SGS Environmental Services, which is accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA). Results of the analysis are provided in Appendix ‘C’, with a discussion of the findings provided below. The submitted samples consisted of unfiltered groundwater, with results presented as totals for various parameters analyzed. The chain of custody number for the submitted samples that underwent analysis is 029455 (SGS Group).

The results of the analysis for the unfiltered groundwater indicate one (1) exceedance when evaluated against the Peel Region Storm and Sanitary Sewer Use By-Law standards. The exceedance, together with the storm and sanitary standards criteria, is presented in Table 7-3.

Table 7-3 - Groundwater Quality Results

Parameter	BH/MW 3 – Groundwater Quality Results (Unfiltered Groundwater) (mg/L)	Peel Region Sanitary Sewer Use Limits (mg/L)	Peel Region Storm Sewer Use Limits (mg/L)
Phosphorus (total)	0.879	10	0.4

As shown above, the concentration for Phosphorous exceeded the Peel Region Storm Sewer Use By-Law standards for the sample obtained from BH/MW 1. However, it meets the limits for the Peel Region Sanitary Sewer Use By-Law standards.

The results suggest that any short-term, construction dewatering effluent, and or any long-term foundation drainage effluent should be acceptable for disposal to the Region of Peel Sanitary Sewer system, and that it should be acceptable for disposal to the Region of Peel Storm Sewer system after minimal pre-treatment has been implemented to lower Phosphorus to meet applicable storm sewer standards prior to its disposal.

The final design for any construction dewatering effluent pre-treatment system is the responsibility of contractors responsible for construction. The final design for any long-term foundation drainage systems effluent pre-treatment system will be the responsibility of the mechanical engineer responsible for the design of the long-term foundation drainage system network.

7.8 Low Impact Development (LIDs)

The shallow groundwater levels were measured at depths, ranging from 0.66 to 3.42 m below the prevailing ground surface. The existing shallow subsoil unit beneath the subject site consists of sandy silt, sandy silt till/silty sand till, silt, silty sand, and sand and gravel layers could facilitate some infiltration of precipitation revived at the developed site to the subsurface to recharge the shallow groundwater table. If the shallow soils remain unsaturated, proposed Low Impact Development (LID) infrastructure should be considered for implementation in areas where the shallow groundwater is deeper than 1.0 m below the ground surface, and where it is possible to maintain a minimum 1.0 m separation between the bases for any proposed LID stormwater management infiltration infrastructure and the high groundwater table to address future stormwater management planning and design. Any proposed LID infrastructure should be designed by the stormwater engineer for the project.

8.0 CONCLUSION

1. The subject site lies within the Physiographic Region of Southern Ontario, known as the Niagara Escarpment on the spillways Plain Physiographic Feature.
2. Based on review of the surface geological map of Ontario, the subject site is located on the Halton Till Unit, native mineral soil deposits, consisting predominantly of silt to silty clay being high in matrix calcium carbonate content which is considered as being clast poor, comprised mainly of silt and clay.
3. Based on the review of the local topography map for the area, and from the review of the ground surface elevation based on the borehole and monitoring well locations the total elevation relief across the site is about 9 m.
4. The subject site is located within the Credit Valley Watershed. Records review shows that a tributary of the Credit River its associated wooded area is located about 50 m south of the subject site.
5. This study has disclosed that beneath layer of topsoil veneer, and a layer of earth fill or weathered soil, the site is underlain by native subsoil strata, comprised of silt, silty sand, sandy silt, silty sand till, sandy silt till and silty clay till, extending to the maximum depth of investigation.
6. The findings of this study confirm that the measured groundwater level elevations ranged from 272.32 to 284.68 masl, and that shallow groundwater is interpreted to flow in north -westerly directions, beneath the site towards the low relief portion of the property.
7. The single well response tests yielded estimated hydraulic conductivity (K) values that range from 6.0×10^{-7} to 4.0×10^{-6} m/sec for the sandy silt till/silty sand till, silt, sandy silt, sand and gravel, and silty clay till subsoils at the depths of the monitoring well screen intervals. These results suggest that low to moderate groundwater seepage rates can be anticipated into open excavations below the shallow groundwater table.
8. Based on the test pit investigations at the anticipated depths for the housing basement foundations structures and proposed underground services indicate that the minor groundwater seepages within test pits excavations occurred at depths of 1.6 mbgs and <5.0 mbgs or at elevations, ranging between 273.6 to 282.5 masl. Limited seepage was observed within test pit excavations, after the test pits remained opened for up to 6.0 hours. Review of the groundwater level elevations recorded at the test pits when compared to the concurrent groundwater level elevations in the monitoring wells indicate that the groundwater levels were 0.44 to 3.39 m higher that the levels observed within the test pits.
9. Given that only limited un-sustained groundwater seepage rates are anticipated during excavations for the proposed underground housing basement structures, and

for the installation of the underground service. It is not anticipated that the groundwater seepage will be sustained within the open excavations, where occasional sump pit pumping should be adequate to remove any occasional limited groundwater seepage that may accumulate within the open excavations. Pumping rates for the anticipated occasional sump pit pumping are expected to be below the 50,000 L/day threshold limit for requiring an approval for any proposed construction related groundwater takings, which will not require any registration or filing with the MECP.

10. The long-term foundation drainage needs from a perimeter footing, drainage network for a conventionally side sloped foundation for each of the proposed housing basement structures within the proposed residential development areas, range from between 134.36 to 488.58 L/day. By applying a safety factor of three (3), the drainage rates could reach maximums, ranging between 403.07 to 1,465.74 L/day. The drainage estimates, above are considered very conservative and are unlikely to come to fruition give the low permeability and slow seepage rates for groundwater within the test pits as revealed from the recent test pit investigation. Any occasional, groundwater seepage drainage to housing basements is likely to be un-sustained and may occur during spring thaw, and following heavy rainfall events.

11. The shallow groundwater levels were measured at depths ranging from 0.66 to 3.42 m below the prevailing ground surface. As such, low impact development (LID) infrastructure may be considered for implementation beneath certain portions of the site. If the shallow soils remain unsaturated, proposed Low Impact Development (LID) infrastructure should be considered for implementation in areas where the shallow groundwater is deeper than 1.0 m below the ground surface, and where it is possible to maintain a minimum 1.0 m separation between the bases for any proposed LID stormwater management infiltration infrastructure and the high groundwater table to address future stormwater management planning.

SOIL ENGINEERS LTD.

Bhawandeep Singh Brar

Bhawandeep Singh Brar, B.Sc.

GR O'Brien

PROFESSIONAL GEOSCIENTISTS ONTARIO
GAVIN R. O'BRIEN
PRACTISING MEMBER
1267
Aug 11, 2023

Gavin O'Brien, M.Sc., P.Geo.
BB/GO

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4. Credit Valley Conservation Authority, 2009.

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FIGURES 1 to 14

BOREHOLE LOGS/MONITORING WELL LOGS
GRAIN SIZE DISTRIBUTION GRAPHS,
AND
TEST PIT LOGS

REFERENCE NO. 2110-W007

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS Auger sample
CS Chunk sample
DO Drive open (split spoon)
DS Denison type sample
FS Foil sample
RC Rock core (with size and percentage recovery)
ST Slotted tube
TO Thin-walled, open
TP Thin-walled, piston
WS Wash sample

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows for each foot of penetration of a 2-inch diameter, 90° point cone driven by a 140-pound hammer falling 30 inches.

Plotted as '—•—'

Standard Penetration Resistance or 'N' Value:

The number of blows of a 140-pound hammer falling 30 inches required to advance a 2-inch O.D. drive open sampler one foot into undisturbed soil.

Plotted as '○'

WH Sampler advanced by static weight
PH Sampler advanced by hydraulic pressure
PM Sampler advanced by manual pressure
NP No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/ft)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

Cohesive Soils:

Undrained Shear Strength (ksf)	'N' (blows/ft)	Consistency
less than 0.25	0 to 2	very soft
0.25 to 0.50	2 to 4	soft
0.50 to 1.0	4 to 8	firm
1.0 to 2.0	8 to 16	stiff
2.0 to 4.0	16 to 32	very stiff
over 4.0	over 32	hard

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0 Field vane test in borehole; the number denotes the sensitivity to remoulding

△ Laboratory vane test

□ Compression test in laboratory

For a saturated cohesive soil, the undrained shear strength is taken as one half of the undrained compressive strength

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres
1lb = 0.454 kg
1 inch = 25.4 mm
1ksf = 47.88 kPa

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JOB NO.: 2301-W042

LOG OF BOREHOLE: BH/MW 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Residential Development

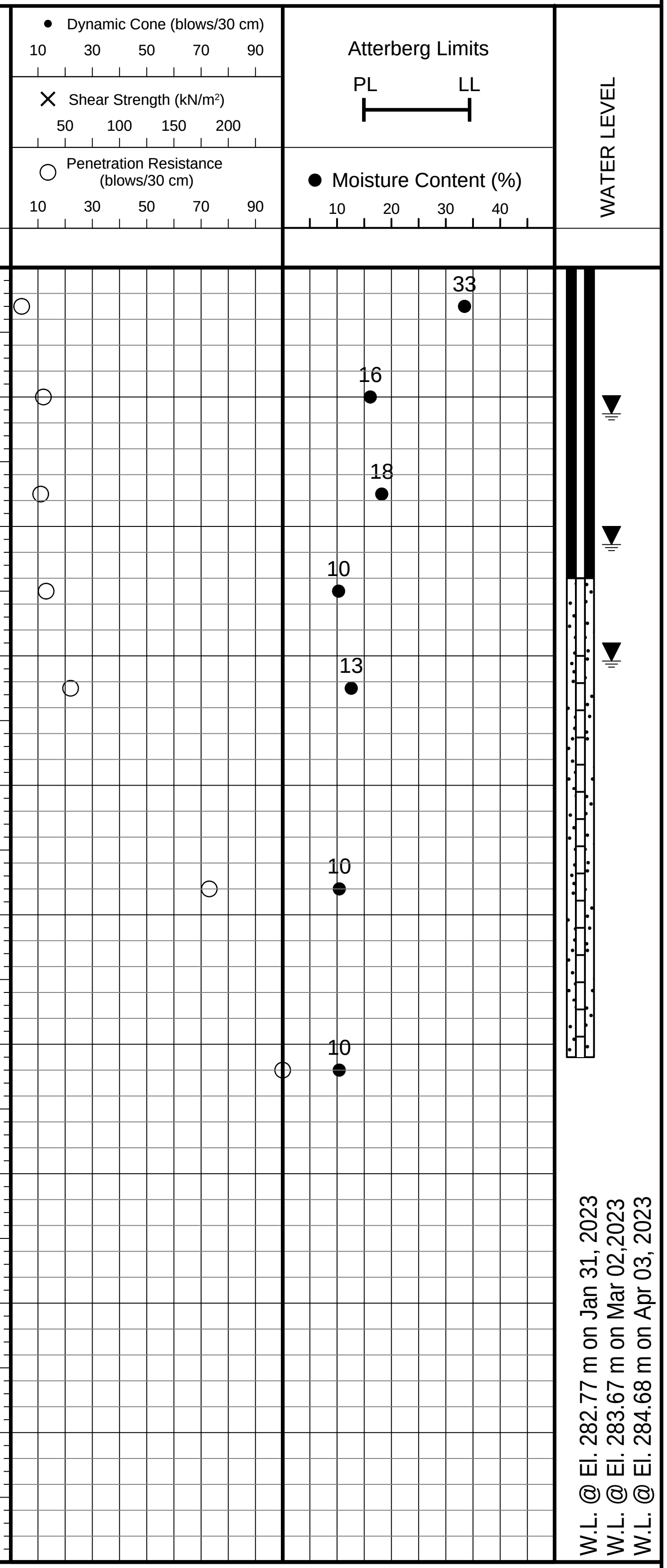
METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	Sample Number	Type	N-Value	Depth Scale (m)	Penetration Resistance (blows/30 cm)	Moisture Content (%)
285.8	Ground Surface						
0.0 / 285.5 / 0.3	**30 cm Topsoil**	1	DO	4	0		33
	Brown, loose to compact **SANDY SILT** a trace of clay occ. gravel — weathered	2	DO	12	1		16
		3	DO	11			18
283.6 / 2.2	Brown, compact to very dense **SANDY SILT TILL / SILTY SAND TILL** a trace of clay some gravel to gravelly	4	DO	13	2		10
		5	DO	22	3		13
		6	DO	73	4		10
279.5 / 6.3		7	DO	50/15	6		10
	END OF BOREHOLE						

Installed 50 mm Ø monitoring well to 6.1 m
completed with 3 m PVC slotted screen
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a monument casing



W.L. @ El. 282.77 m on Jan 31, 2023
W.L. @ El. 283.67 m on Mar 02,2023
W.L. @ El. 284.68 m on Apr 03, 2023

Soil Engineers Ltd.

JOB NO.: 2301-W042

LOG OF BOREHOLE: BH/MW 2

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Moisture Content (%)
281.7	Ground Surface					
0.0	**25 cm Topsoil**				0	
281.4 / 0.3	Brown, very loose to compact **SANDY SILT TILL** traces of clay and gravel (weathered)	1	DO	3		36
		2	DO	15	1	11
280.1 / 1.7	Brown, compact to very dense **SILT** very moist to wet a trace to some sand	3	DO	18	2	17
		4	DO	15		22
		5	DO	21	3	20
		6	DO	28	4–5	17
	occ. cobbles a trace of gravel	7	DO	50/15	6	17
275.4 / 6.4	**END OF BOREHOLE**					

Installed 50 mm Ø monitoring well to 6.1 m
completed with 3 m PVC slotted screen
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a monument casing

Dynamic Cone (blows/30 cm): 10 30 50 70 90
Shear Strength (kN/m²): 50 100 150 200
Penetration Resistance (blows/30 cm): 10 30 50 70 90
Atterberg Limits: PL LL
Moisture Content (%): 10 20 30 40

WATER LEVEL

W.L. @ El. 278.23 m on Jan 31, 2023
W.L. @ El. 279.55 m on Mar 02,2023
W.L. @ El. 281.09 m on Apr 03, 2023

Soil Engineers Ltd.

JOB NO.: 2301-W042

LOG OF BOREHOLE: BH/MW 3

FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	Number	Type	N-Value	Penetration Resistance (blows/30 cm)	Moisture Content (%)
282.8	Ground Surface					
282.6 / 0.2	**20 cm Topsoil**					
281.3 / 1.5	Dark brown **EARTH FILL** sand, some silt occ. topsoil inclusion occ. organics and rootlets	1	DO	3		15
		2	DO	5		3
280.5 / 2.3	Brown, compact **SILTY SAND** occ. silty clay layers	3	DO	11		11
278.8 / 4.0	Brown, compact, wet **SILT** traces of clay and gravel	4	DO	19		25
		5	DO	13		25
277.2 / 5.6	Brown, compact **SANDY SILT** a trace of clay occ. gravel	6	DO	15		18
276.6 / 6.2	Brown, hard **SILTY CLAY TILL** occ. shale fragments	7	DO	50/3		10
	END OF BOREHOLE					

Installed 50 mm Ø monitoring well to 6.2 m
completed with 3 m PVC slotted screen
Sand backfill from 2.6 to 6.2 m
Bentonite seal from 0.0 m to 2.6 m
Provided with a monument casing

Legend: ● Dynamic Cone (blows/30 cm) 10 30 50 70 90; × Shear Strength (kN/m²) 50 100 150 200; ○ Penetration Resistance (blows/30 cm) 10 30 50 70 90; Atterberg Limits PL LL; ● Moisture Content (%) 10 20 30 40; Depth Scale (m) 0–10; WATER LEVEL

W.L. @ El. 279.27 m on Jan 31, 2023
W.L. @ El. 280.05 m on Mar 02,2023
W.L. @ El. 280.72 m on Apr 03, 2023

Soil Engineers Ltd.

JOB NO.: 2301-W042

LOG OF BOREHOLE: BH/MW 4

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed Residential Development

METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

Legend: ● Dynamic Cone (blows/30 cm) 10 30 50 70 90; × Shear Strength (kN/m²) 50 100 150 200; ○ Penetration Resistance (blows/30 cm) 10 30 50 70 90; Atterberg Limits PL LL; ● Moisture Content (%) 10 20 30 40; WATER LEVEL

El. (m) / Depth (m)	SOIL DESCRIPTION
277.2	Ground Surface
277.1 / 0.2	**20 cm Topsoil**
275.1 / 2.2	Dark brown **EARTH FILL** mixture of sand, silt and clay, a trace of gravel, occ. topsoil inclusion, occ. organics and rootlets
272.4 / 4.8	Brown, dense **SAND AND GRAVEL** a trace to some silt (– – – silty)
270.9	Brown, dense **SANDY SILT TILL** traces of clay and gravel
270.6 / 6.6	Brown, hard **SILTY CLAY TILL**

Sample Number	Type	N-Value	Moisture Content (%)
1	DO	5	17
2	DO	6	18
3	DO	3	24
4	DO	37	18
5	DO	35	3
6	DO	34	18
7A	DO	45	10
7B	DO	45	9

Depth Scale (m): 0 1 2 3 4 5 6 7 8 9 10

END OF BOREHOLE

Installed 50 mm Ø monitoring well to 6.1 m
completed with 3 m PVC slotted screen
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a monument casing

W.L. @ El. 272.32 m on Jan 31, 2023
W.L. @ El. 273.08 m on Mar 02,2023
W.L. @ El. 273.83 m on Apr 03, 2023

Soil Engineers Ltd.

JOB NO.: 2301-W042

LOG OF BOREHOLE: BH/MW 5

FIGURE NO.: 5

PROJECT DESCRIPTION: Proposed Residential Development

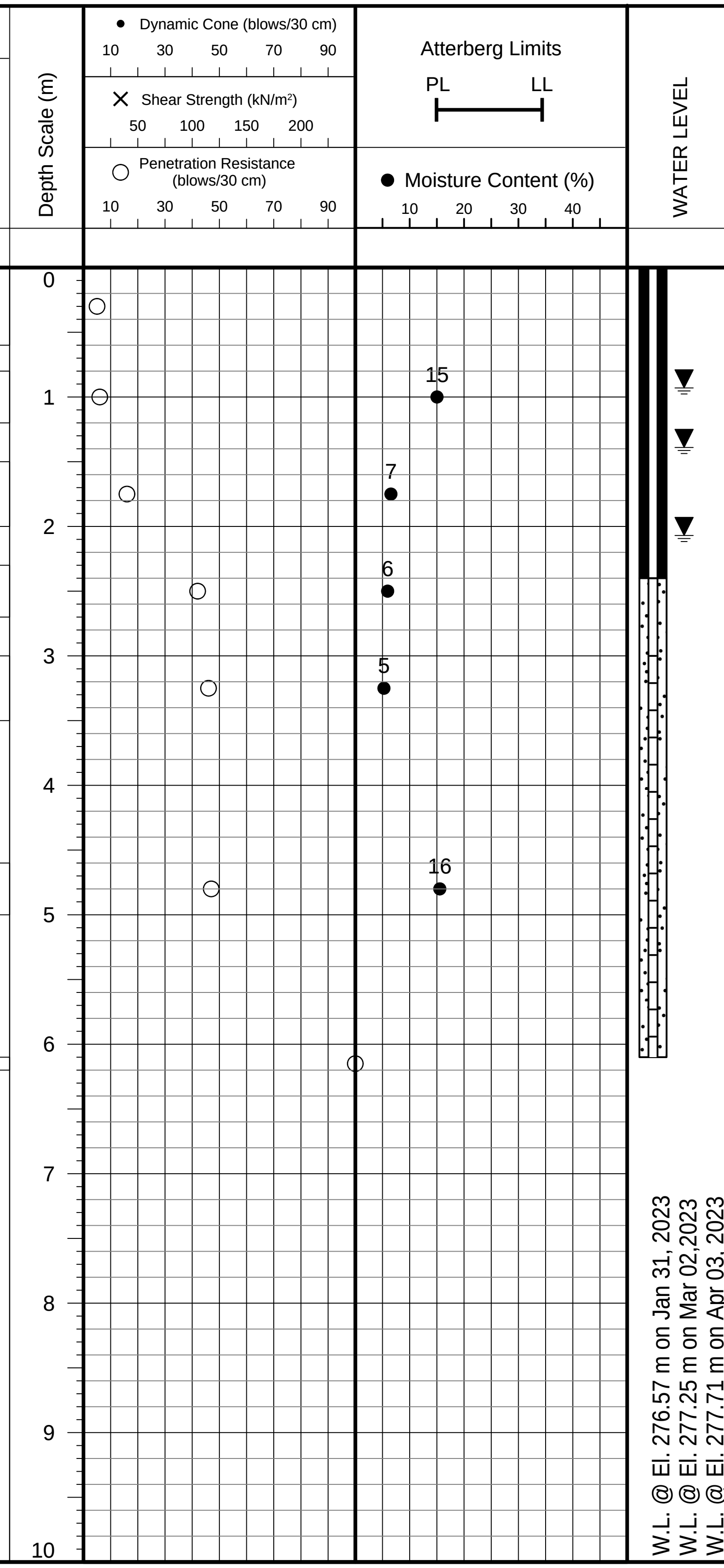
METHOD OF BORING: Flight Auger (Solid Stem)

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

DRILLING DATE: Janaury 24, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
278.6	Ground Surface			
278.4 / 0.2	**15 cm Topsoil**			
	Dark brown **EARTH FILL** sand, some silt to silty occ. topsoil inclusion occ. organics and rootlets	1	DO	5
		2	DO	6
277.1 / 1.5	Brown, compact **SILTY SAND** a trace of clay	3	DO	16
276.3 / 2.3	Brown, dense **SAND AND GRAVEL** a trace to some silt	4	DO	42
		5	DO	46
274.6 / 4.0	Brown, hard **SILTY CLAY TILL** a trace of gravel	6	DO	47
272.9 / 5.7	Grey, weathered **SHALE** water seepage	7	DO	50/3
272.4 / 6.2	**END OF BOREHOLE**			

Installed 50 mm Ø monitoring well to 6.1 m
completed with 3 m PVC slotted screen
Sand backfill from 2.4 to 6.1 m
Bentonite seal from 0.0 m to 2.4 m
Provided with a monument casing



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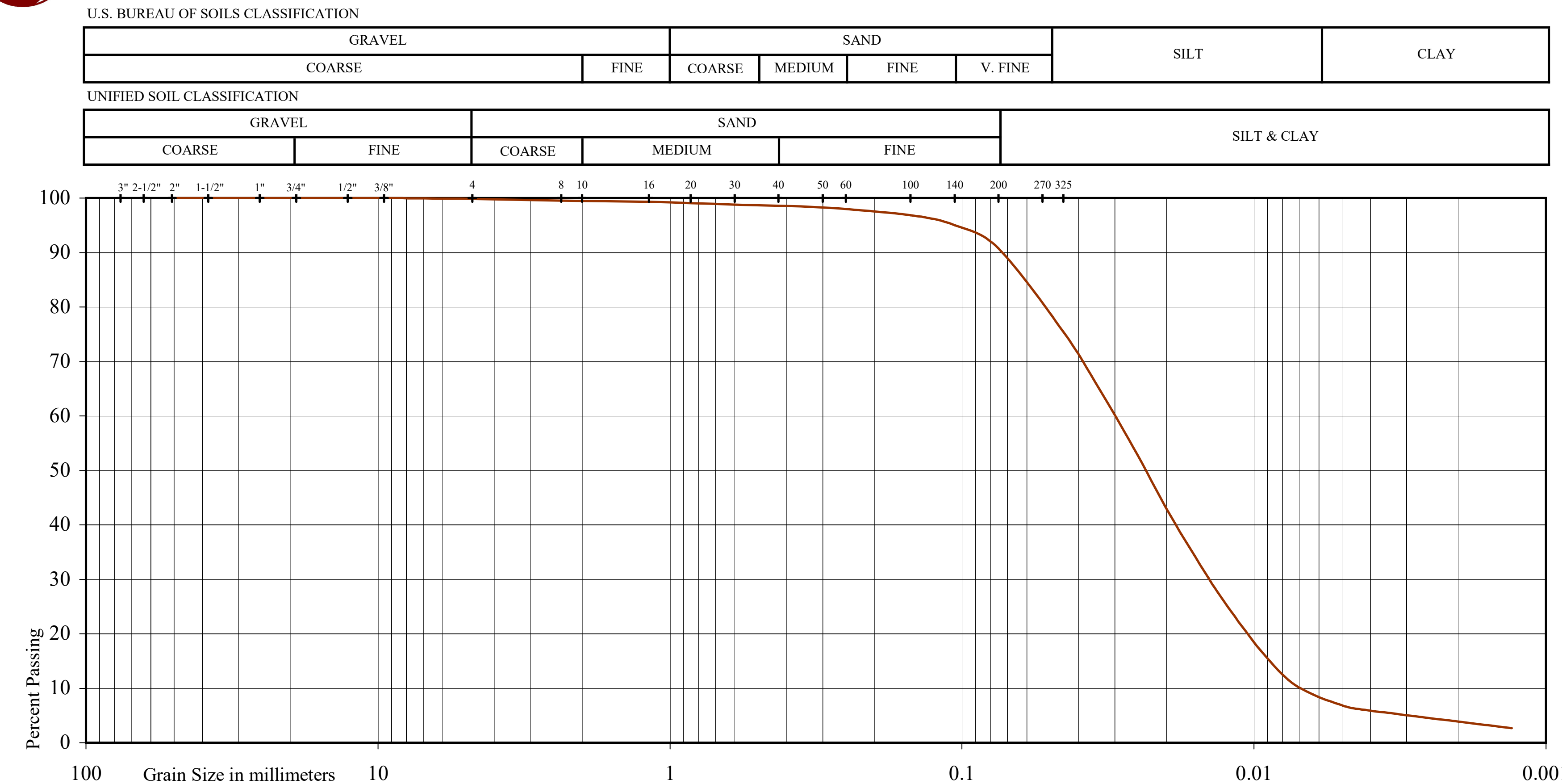
GRAIN SIZE DISTRIBUTION

Reference No: 2301-W042



Project: Proposed Residential Development

Location: 15544 McLaughlin Road, Town of Caledon

Liquid Limit (%) = -

Plastic Limit (%) = -

Borehole No: 2

Plasticity Index (%) = -

Sample No: 5

Moisture Content (%) = 20

Depth (m): 3.3

Estimated Permeability

Elevation (m): 278.5

(cm./sec.) = 10^{-4}

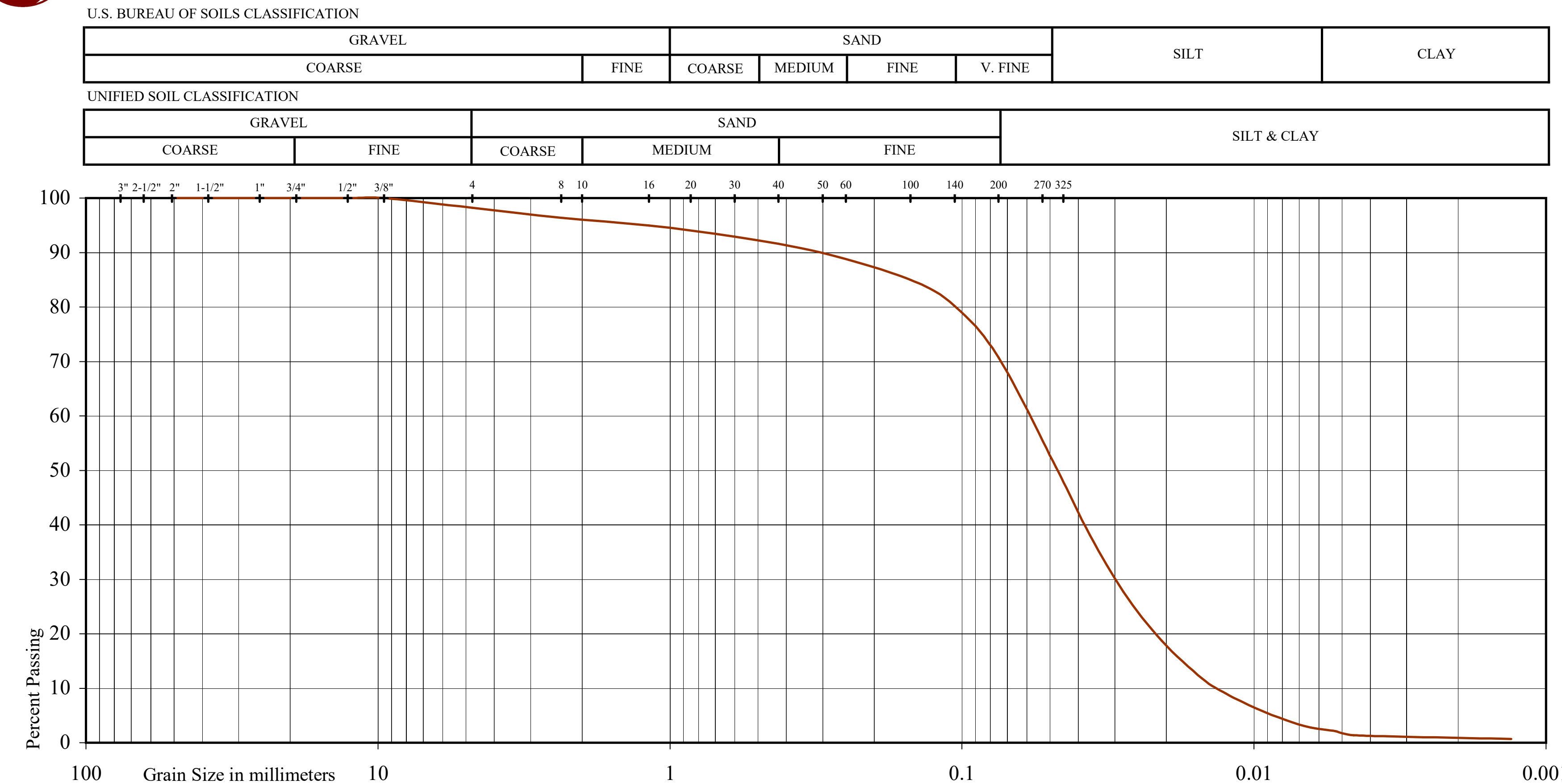
Classification of Sample [& Group Symbol]:	SILT traces of clay and sand

Figure: 6

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 2301-W042



Project: Proposed Residential Development

Location: 15544 McLaughlin Road, Town of Caledon

Borehole No: 3

Sample No: 6

Depth (m): 4.8

Elevation (m): 278.0

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 18

Estimated Permeability (cm./sec.) = 10^{-4}

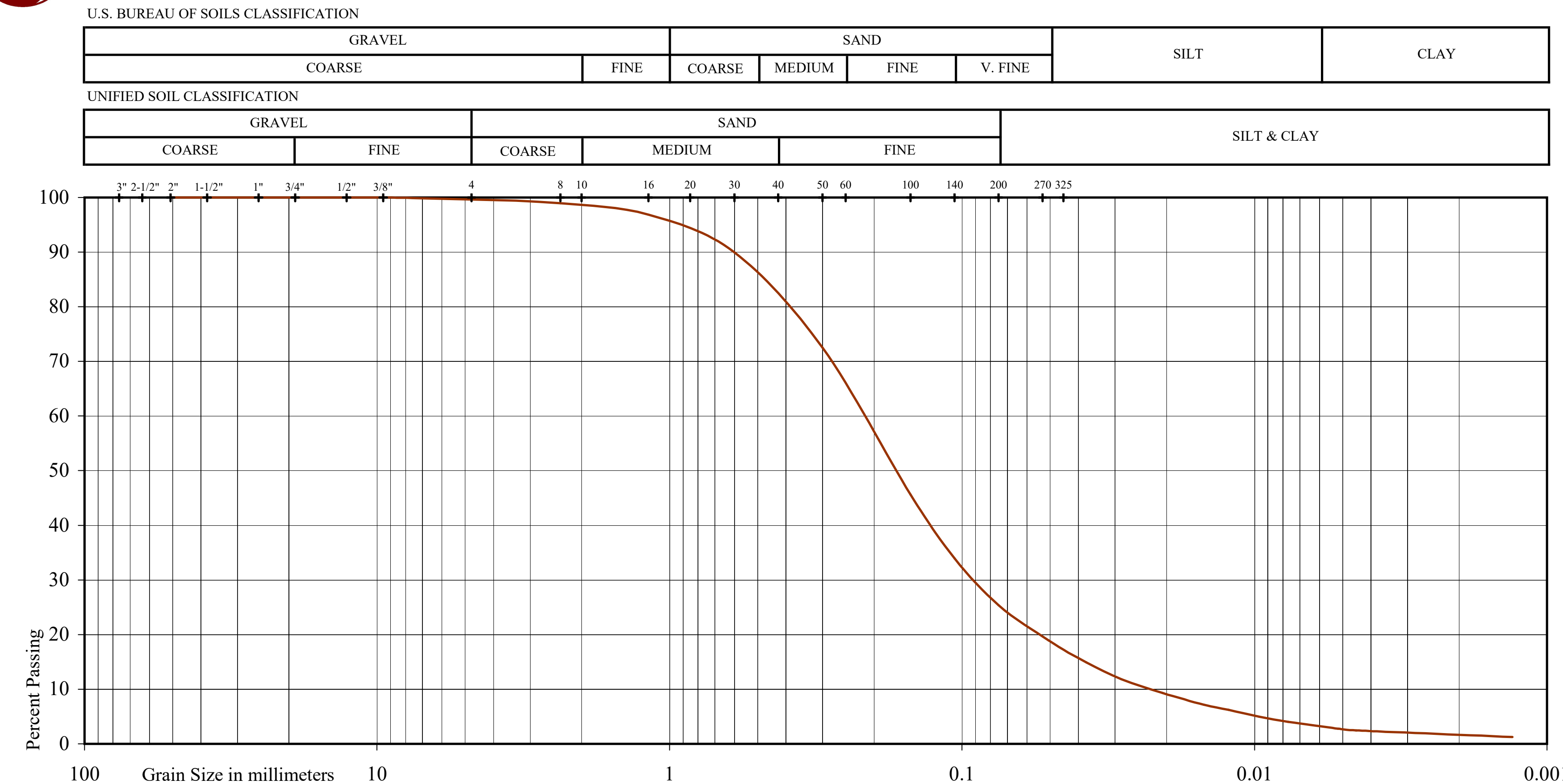
Classification of Sample [& Group Symbol]:	SANDY SILT a trace of clay and occ. gravel

Figure: 7

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 2301-W042



Project: Proposed Residential Development

Location: 15544 McLaughlin Road, Town of Caledon

Borehole No: 5

Sample No: 3

Depth (m): 1.8

Elevation (m): 276.9

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 7

Estimated Permeability (cm./sec.) = 10^{-3}

Classification of Sample [& Group Symbol]:	SILTY SAND a trace of clay

Figure: 8

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

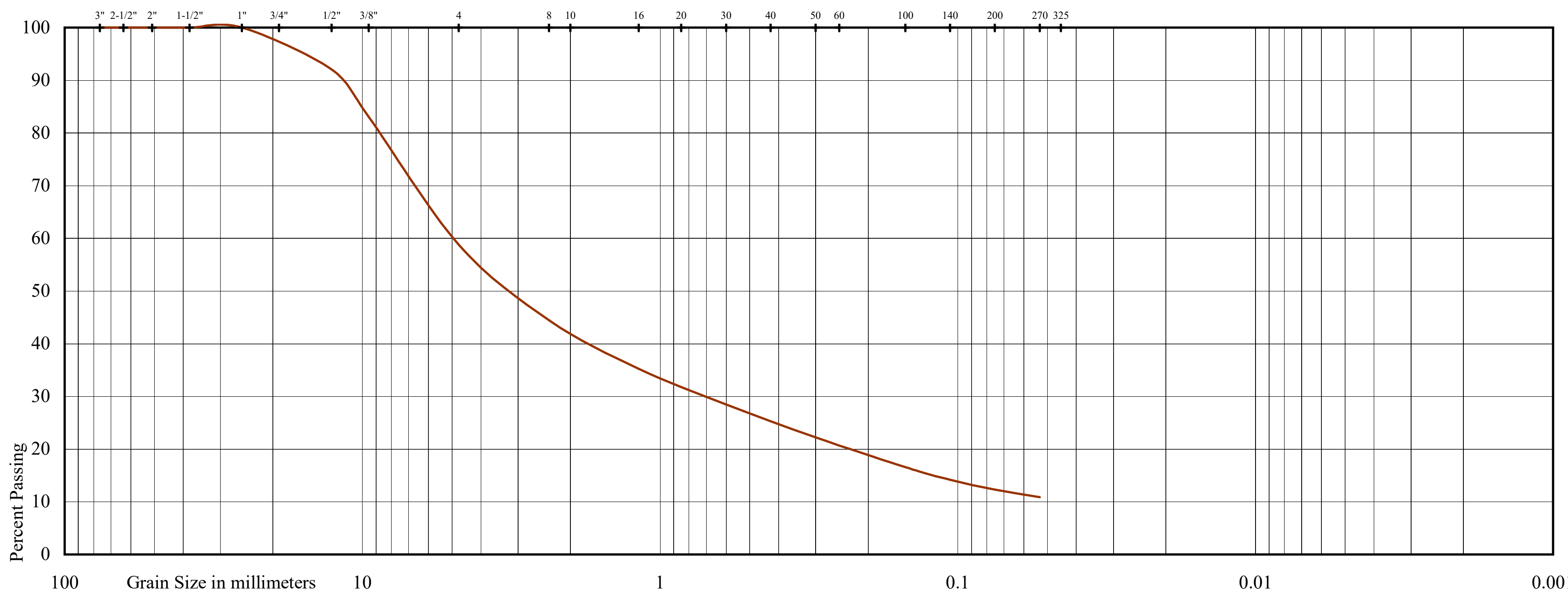
Reference No: 2301-W042

U.S. BUREAU OF SOILS CLASSIFICATION

GRAVEL		SAND				SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	V. FINE		

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND			SILT & CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Project: Proposed Residential Development

Location: 15544 McLaughlin Road, Town of Caledon

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 3

Estimated Permeability (cm./sec.) = 10^{-3}

Borehole No: 4

Sample No: 5

Depth (m): 3.3

Elevation (m): 274.0

Classification of Sample [& Group Symbol]: SAND AND GRAVEL some silt

Figure: 9

JOB NO.: 2301-W042 **LOG OF BOREHOLE: Test Pit 1** **FIGURE NO.:** 10

PROJECT DESCRIPTION: Proposed Residential Development **METHOD** Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon **TEST PIT DATE:** May 30, 2023

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90; Shear Strength (kN/m²) 50 100 150 200; Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL; Moisture Content (%) 10 20 30 40	WATER LEVEL
285.2	Ground Surface							
0.0 284.9	**30 cm Topsoil**				0			
0.3	Brown, loose to compact **SANDY SILT** a trace of clay occ. gravel				1			cave-in occured elevation @ 284.40 masl
283.6 1.6	Brown, compact to very dense **SANDY SILT TILL / SILTY SAND TILL** a trace of clay some gravel to gravelly				2, 3, 4			water seepage elevation @ 282.50 masl
280.2 5.0	**END OF TEST PIT**				5, 6, 7, 8, 9, 10			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
Water seepage occured @ 2.7 mbgs

Minor seepage rate

Cave-In
Cave-In occured @ 0.8 mbgs

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 6.0 hours

Time	Water Level (from bottom of test pit)
10:00 am	1 cm
10:10 am	2 cm
10:30 am	8 cm
11:45 am	15 cm
12:15 pm	18 cm
01:15 pm	19 cm
02:30 pm	21 cm
03:30 pm	23 cm
04:00 pm	24 cm

JOB NO.: 2301-W042 **LOG OF BOREHOLE: Test Pit 2** **FIGURE NO.:** 11

PROJECT DESCRIPTION: Proposed Residential Development **METHOD** Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon **TEST PIT DATE:** May 30, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES: Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90; Shear Strength (kN/m²) 50 100 150 200; Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL; Moisture Content (%) 10 20 30 40	WATER LEVEL
281.7	Ground Surface							
0.0 / 281.4	**30 cm Topsoil**				0			cave-in occured elevation @ 281.4 masl
0.3	Brown, very loose to compact **SANDY SILT TILL** traces of clay and gravel				1			
280.1 / 1.6	Brown, compact to very dense **SILT** a trace to some sand				2, 3, 4			
276.7 / 5.0	**END OF TEST PIT**				5			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
No water seepage occured during the time interval

Cave-In
Cave-In occured @ 0.3 mbgs

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 4.0 hours

Time	Water Level (from bottom of test pit)
10:45 am	dry
11:15 am	dry
12:00 pm	dry
12:45 pm	dry
01:15 pm	dry
02:15 pm	dry
02:45 pm	dry

Depth Scale (m): 0 1 2 3 4 5 6 7 8 9 10

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JOB NO.: 2301-W042 **LOG OF BOREHOLE: Test Pit 3** **FIGURE NO.:** 12

PROJECT DESCRIPTION: Proposed Residential Development **METHOD** Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon **TEST PIT DATE:** May 30, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES: Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90; Shear Strength (kN/m²) 50 100 150 200; Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL; Moisture Content (%) 10 20 30 40	WATER LEVEL
283.0	Ground Surface							
0.0 / 282.7	**30 cm Topsoil**				0			
0.2	Dark brown **EARTH FILL** sand, some silt occ. topsoil inclusion occ. organics and rootlets				1			
281.3 / 1.7	Brown, compact **SILTY SAND** occ. silty clay layers				2			▼ water seepage elevation @ 281.40 masl
280.5 / 2.5	Brown, compact **SILT** traces of clay and gravel				3, 4			
278.8 / 4.2	Brown, compact **SANDY SILT** a trace of clay occ. gravel							
278.0 / 5.0	**END OF TEST PIT**				5–10			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
Water seepage occured @ 1.6 mbgs

Minimal Seepage rate

Cave-In
No cave-in occured during the time interval

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 6.0 hours

Time	Water Level (from bottom of test pit)
11:20 am	1 cm
11:45 am	3 cm
12:05 pm	7 cm
01:15 pm	9 cm
02:00 pm	11 cm
03:00 pm	13 cm
04:15 pm	15 cm
05:20 pm	18 cm

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JOB NO.: 2301-W042 **LOG OF BOREHOLE: Test Pit 4** **FIGURE NO.:** 13

PROJECT DESCRIPTION: Proposed Residential Development **METHOD** Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon **TEST PIT DATE:** May 30, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90; Shear Strength (kN/m²) 50 100 150 200; Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL; Moisture Content (%) 10 20 30 40	WATER LEVEL
277.3	Ground Surface							
0.0	**20 cm Topsoil**				0			
0.2	Dark brown **EARTH FILL** mixture of sand, silt and clay a trace of gravel occ. topsoil inclusion occ. organics and rootlets				1			
275.1 2.2	Brown, dense **SAND AND GRAVEL** a trace to some silt				2, 3, 4			water seepage elevation @ 273.80 masl
272.3 5.0	**END OF TEST PIT**				5			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
Water seepage occured @ 3.5 mbgs

Medium to Fast seepage rate

Cave-In
No cave-In occured during the time interval

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 4.0 hours

Time	Water Level (from bottom of test pit)
12:00 pm	50 cm
12:20 pm	70 cm
01:15 pm	85 cm
02:00 pm	95 cm
03:10 pm	110 cm
04:00 pm	120 cm

Depth Scale (m): 0 1 2 3 4 5 6 7 8 9 10

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JOB NO.: 2301-W042 **LOG OF BOREHOLE: Test Pit 5** **FIGURE NO.:** 14

PROJECT DESCRIPTION: Proposed Residential Development **METHOD** Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon **TEST PIT DATE:** May 30, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90; Shear Strength (kN/m²) 50 100 150 200; Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL; Moisture Content (%) 10 20 30 40	WATER LEVEL
278.4	Ground Surface							
0.0	**20 cm Topsoil**				0			
0.2	Dark brown **EARTH FILL** sand, some silt to silty occ. topsoil inclusion occ. organics and rootlets				1			
277.1 1.3	Brown, compact **SILTY SAND** a trace of clay				2			
276.3 2.1	Brown, dense **SAND AND GRAVEL** a trace to some silt				3			
274.6 3.8	Brown, hard **SILTY CLAY TILL** a trace of gravel				4			water seepage elevation @ 273.65 masl
273.4 5.0	**END OF TEST PIT**				5–10			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
Water seepage occured @ 4.75 mbgs

Minor seepage rate

Cave-In
No cave-In occured during the time interval

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 4.0 hours

Time	Water Level (from bottom of test pit)
12:30 pm	3 cm
01:30 pm	9 cm
02:15 pm	12 cm
03:30 pm	14 cm
04:30 pm	16 cm

Soil Engineers Ltd.

Soil Engineers Ltd.
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GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE 100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

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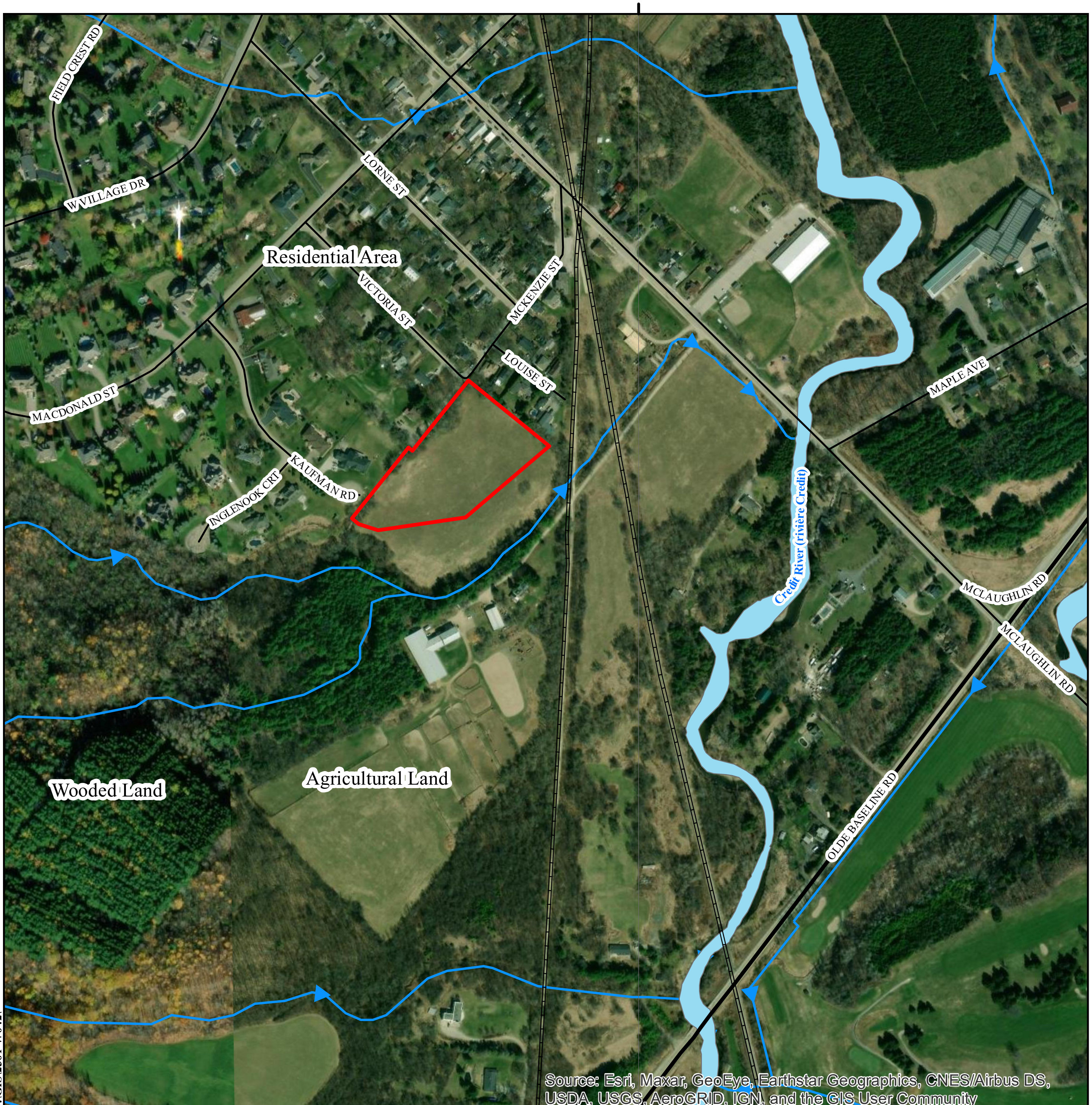
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 542-2769

DRAWINGS 1 to 9

REFERENCE NO. 2110-W007

586000



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N

Legend

- Approximate Boundary of Subject Site
- Waterbody
- Watercourse
- Road
- Railway

Soil Engineers Ltd.

Title: Site Location Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
Address: 15544 McLaughlin Road
Town of Caledon

Reference No. 2301-W042

Date: January 20, 2023

Scale:
0 25 50 100 150 200 250
Metres

Drawing No. 1

Source: Ontario Ministry of Natural Resources and Forestry

4849500

4849500

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N

Legend

- Approximate Boundary of Subject Site
- Borehole with Monitoring Well
- Test Pit
- Watercourse
- Road

Soil Engineers Ltd.

Title: Borehole and Monitoring Well Location Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
Address: 15544 McLaughlin Road
Town of Caledon

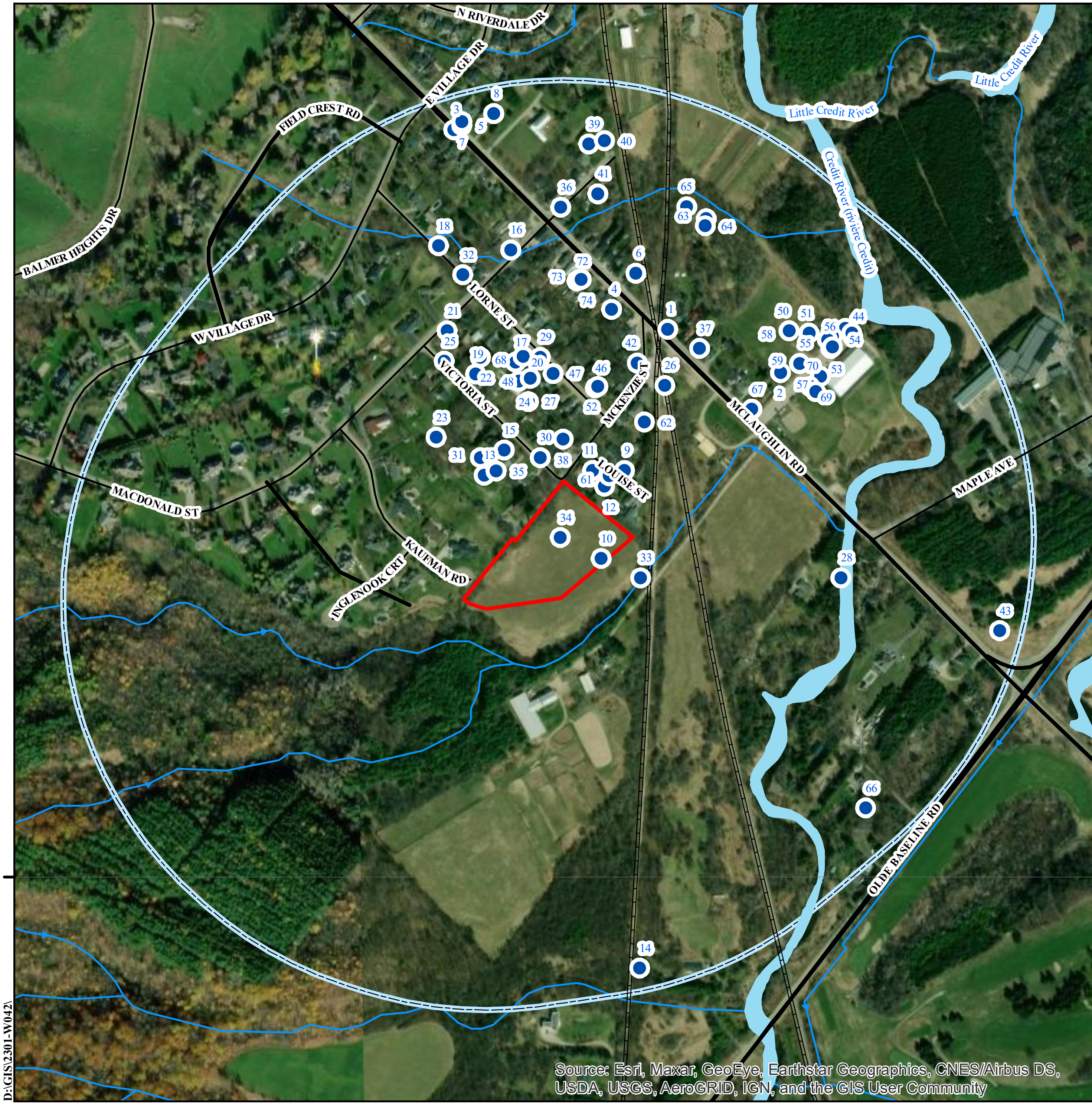
Reference No. 2301-W042

Date: July 04, 2023

Scale:
0 5 10 20 30 40 50
Metres

Drawing No. 2 (e)

Source: Ontario Ministry of Natural Resources and Forestry



N

Legend

Approximate Boundary of Subject Site

500 metres from Subject Site Boundary

Well Location from MECP Well Records (see Appendix 'A')

Major Road

Local Road

Waterbody

Watercourse

Soil Engineers Ltd.

Title: MECP Well Location Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
Address: 15544 McLaughlin Road
Town of Caledon

Reference No. 2301-W042

Date: January 20, 2023

Scale:
0 30 60 120 180 240 300
Metres

Drawing No. 3

Source: Ontario Ministry of Natural Resources and Forestry

4849000

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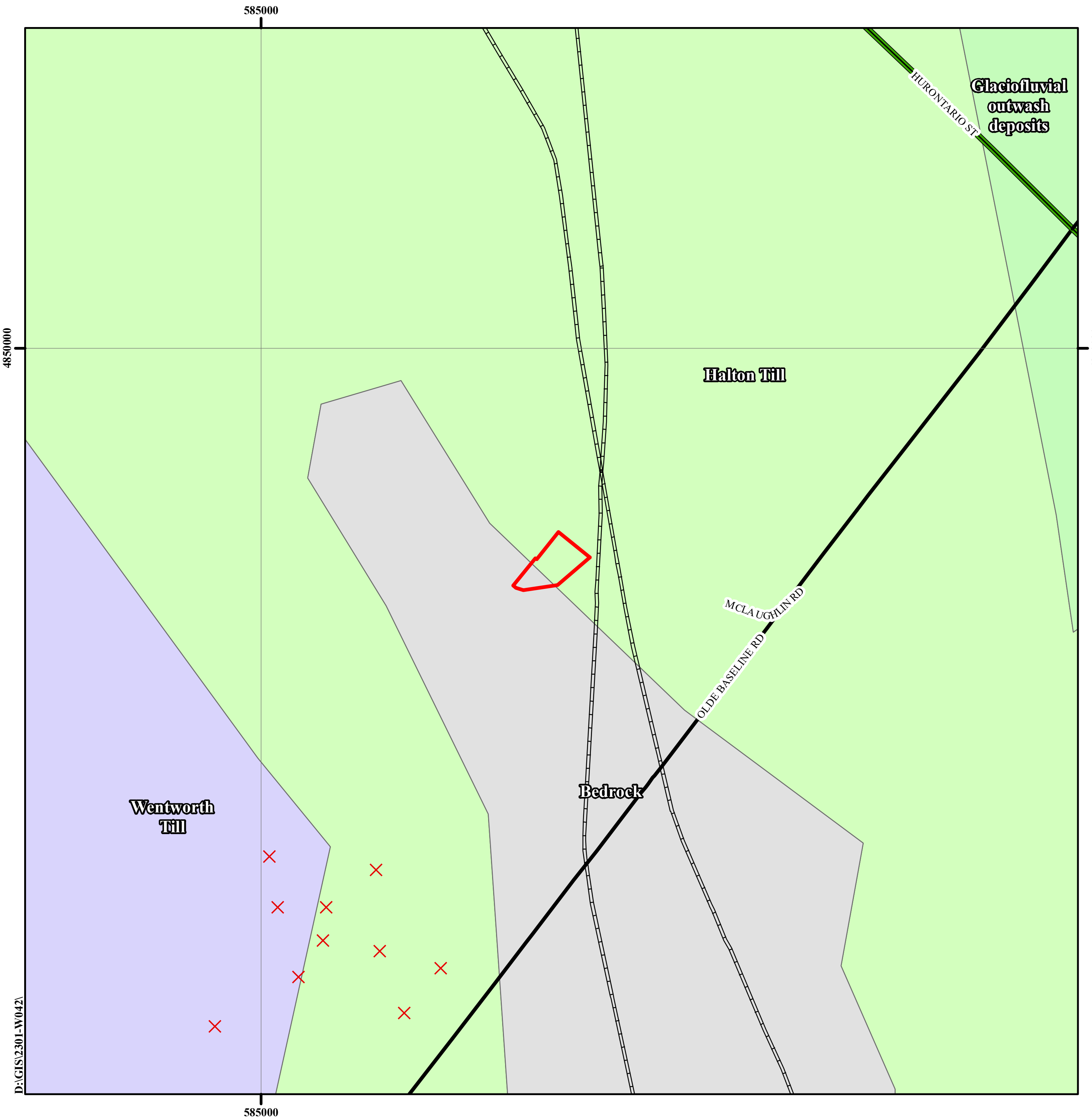


N

Legend

- Approximate Boundary of Subject Site
- Halton Till
  Material: predominantly silt to silty clay matrix, high in matrix carbonate content and clast poor
- Wentworth Till
  material: sandy silt to silt matrix, becoming finer grained to silty clay near Lake Erie, highly calcareous, clast content moderate to low decreasing southward
- Bedrock, undifferentiated igneous and metamorphic rock, or carbonate and clastic sedimentary rock, exposed at surface or covered by a discontinuous, thin layer of drift
- Bedrock Outcrops
- Expressway/Freeway
- Major Road
- Railway

Soil Engineers Ltd.

Title: Quarternary and Surface Geology Map

Project:

Hydrogeological Assessment
Proposed Residential Development
Address: 15544 McLaughlin Road
Town of Caledon

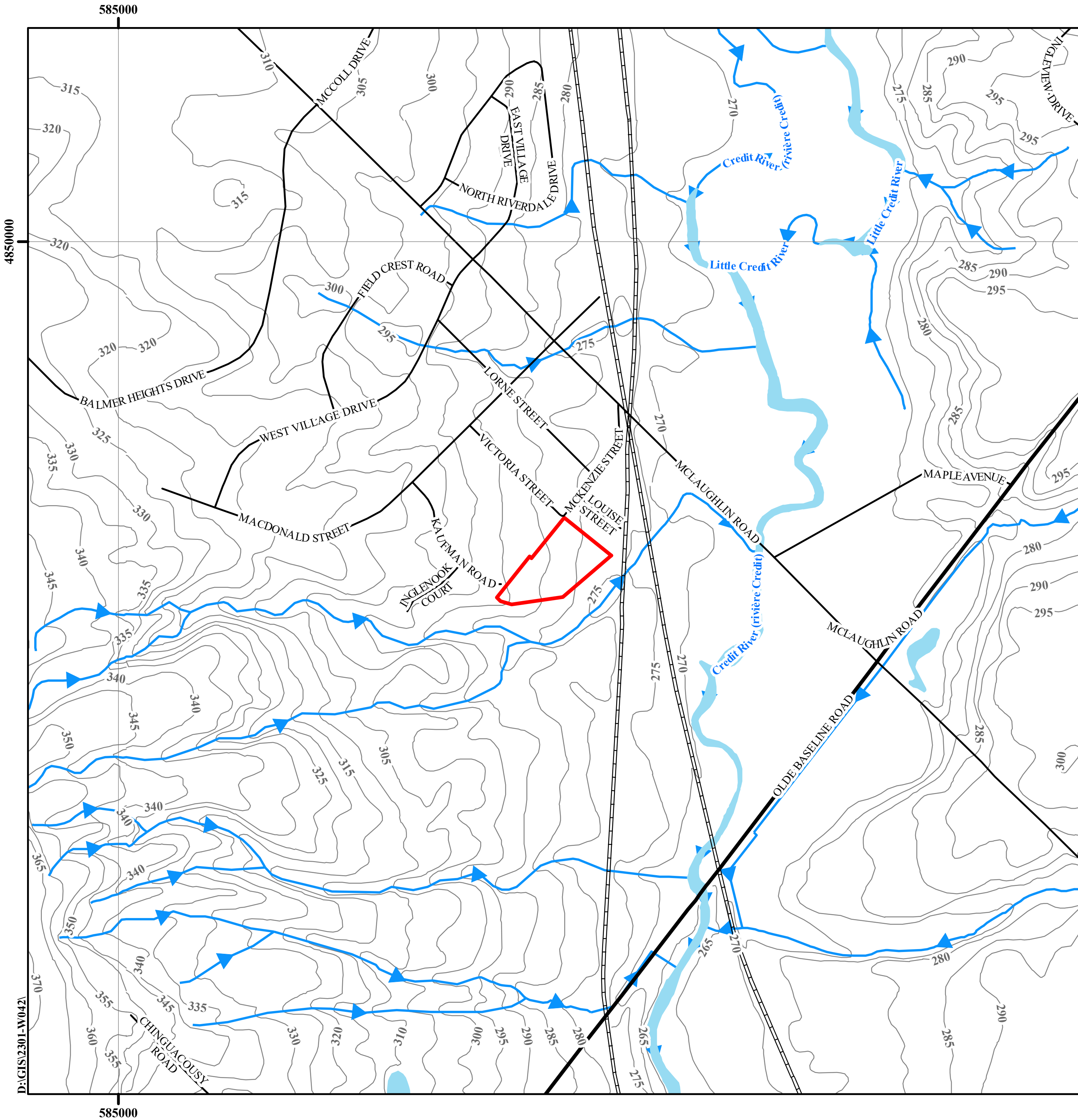
Reference No. 2301-W042

Date: January 20, 2023

Scale:
0 100 200 300 400 500
Metres

Drawing No. 4

Source: Ontario Geological Survey, 1997, Miscellaneous Release---Data 0014



N

Legend

Approximate Boundary of Subject Site

Waterbody

Watercourse

Road

Railway

100 Topographic Contour (masl)

Soil Engineers Ltd.

Title: Topographic Map

Project:

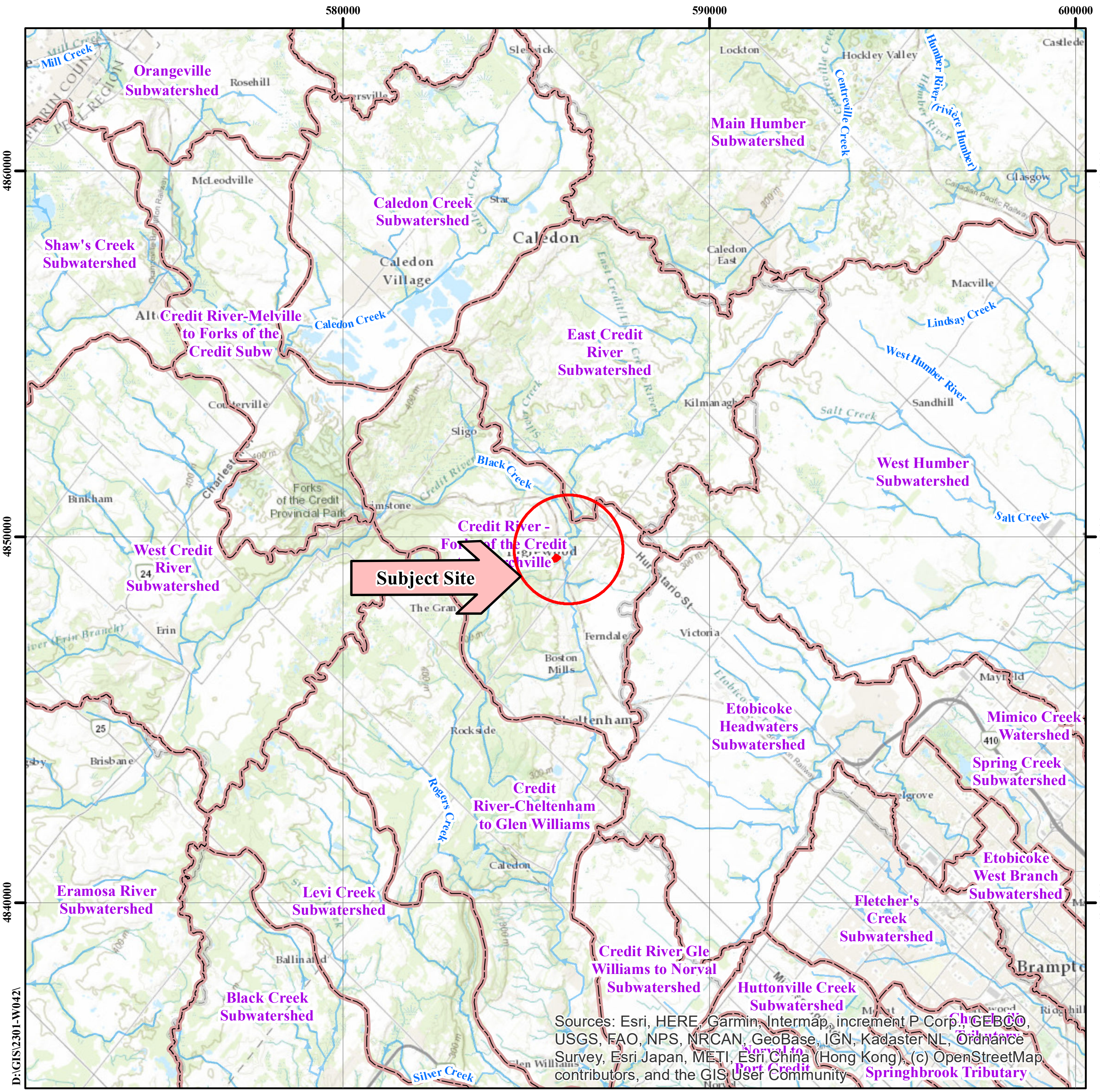
Hydrogeological Assessment
Proposed Residential Development
Address: 15544 McLaughlin Road
Town of Caledon

Reference No. 2301-W042

Date: January 20, 2023

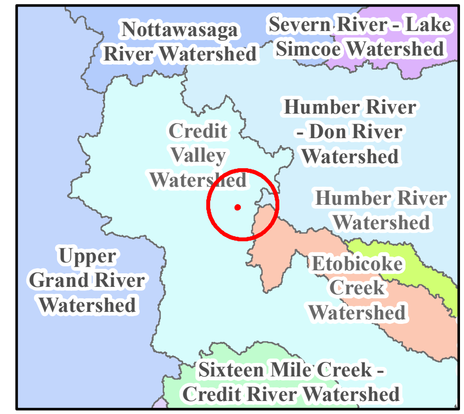
Scale:
0 50 100 200 300 400 500
Metres

Drawing No. 5



N

Watershed:



Legend

- Approximate Boundary of Subject Site
- Watershed Boundaries
- Waterbody
- Watercourse
- Expressway/Major Road

Soil Engineers Ltd.

Title: Watershed and Subwatershed Map

Project:

Hydrogeological Assessment
Proposed Residential Development
Address: 15544 McLaughlin Road
Town of Caledon

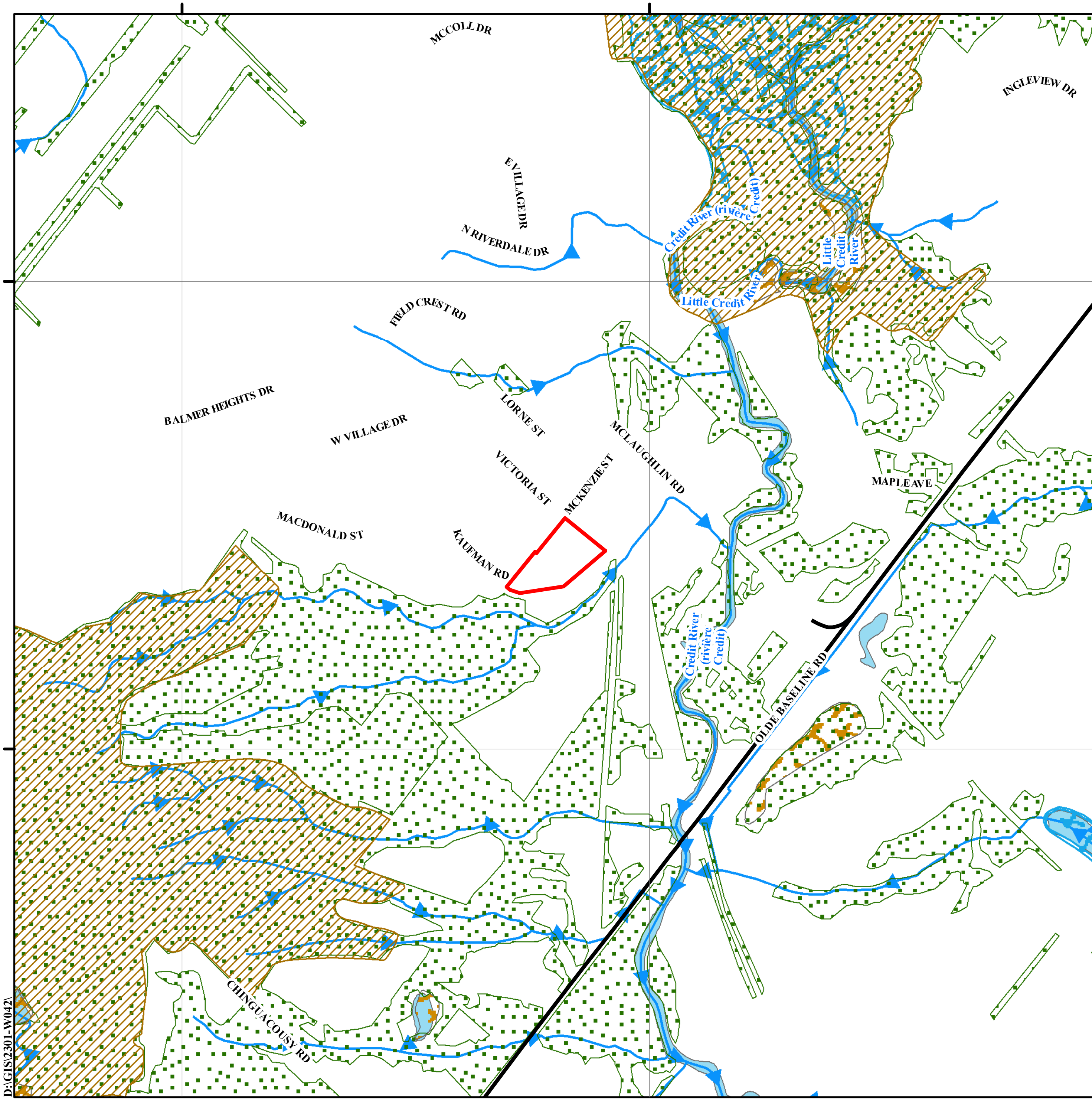
Reference No. 2301-W042

Date: January 20, 2023

Scale:
0 1,600 3,200 4,800 6,400 8,000
Metres

Drawing No. 6

This mapping was produced by SEL and should be used for information purposes only.
Data sources used in its production are of varying quality and accuracy and all boundaries should be considered approximate.



Legend

- Approximate Boundary of Subject Site
- Area of Natural and Scientific Interest (ANSI)
- Wetland (classified as Provincial)
- Wetland (Not evaluated per OWES)
- Wooded Area
- Waterbody
- Watercourse
- Major Road

Soil Engineers Ltd.

Title: Natural Features and Protection Area Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
Address: 15544 McLaughlin Road
Town of Caledon

Reference No. 2301-W042

Date: January 20, 2023

Scale:
0 80 160 240 320 400
Metres

Drawing No. 7

Includes information: Provincial Park, Conservation Reserve, Area of Natural and Scientific Interest, Wetland, Niagara Escarpment Protection Area, Oak Ridges Moraine Conservation and Wilderness Areas

Source: Ontario Ministry of Natural Resources and Forestry

OWES: Ontario Wetland Evaluation System

N

Legend

Approximate Boundary of Subject Site

Borehole with Monitoring Well

Watercourse

A A' Cross-Section Direction

—100— Topographic Contour (masl)

Soil Engineers Ltd.

Title: Cross-Section Key Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
Address: 15544 McLaughlin Road
Town of Caledon

Reference No. 2301-W042

Date: January 20, 2023

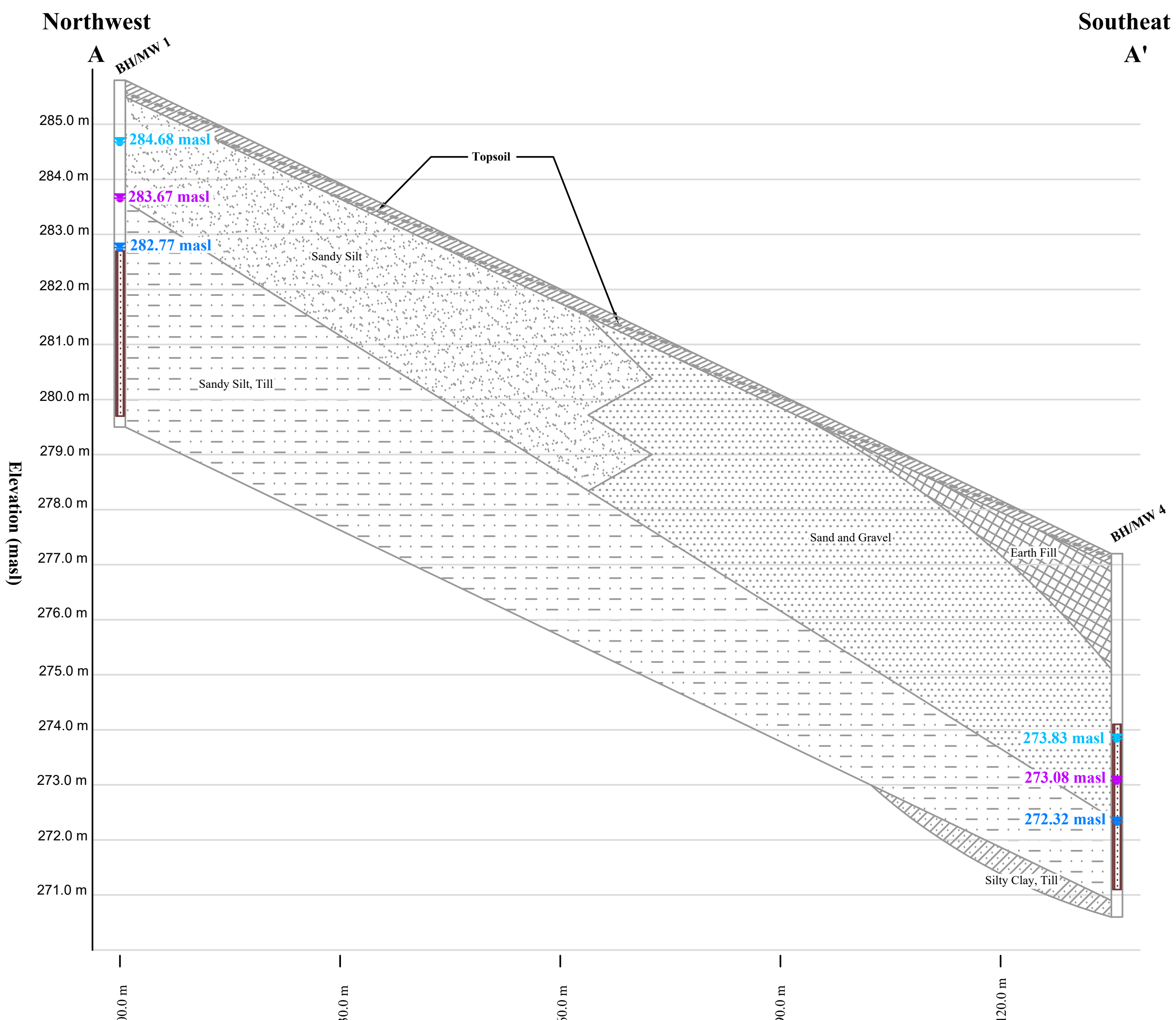
Scale:

0 5 10 20 30 40 50

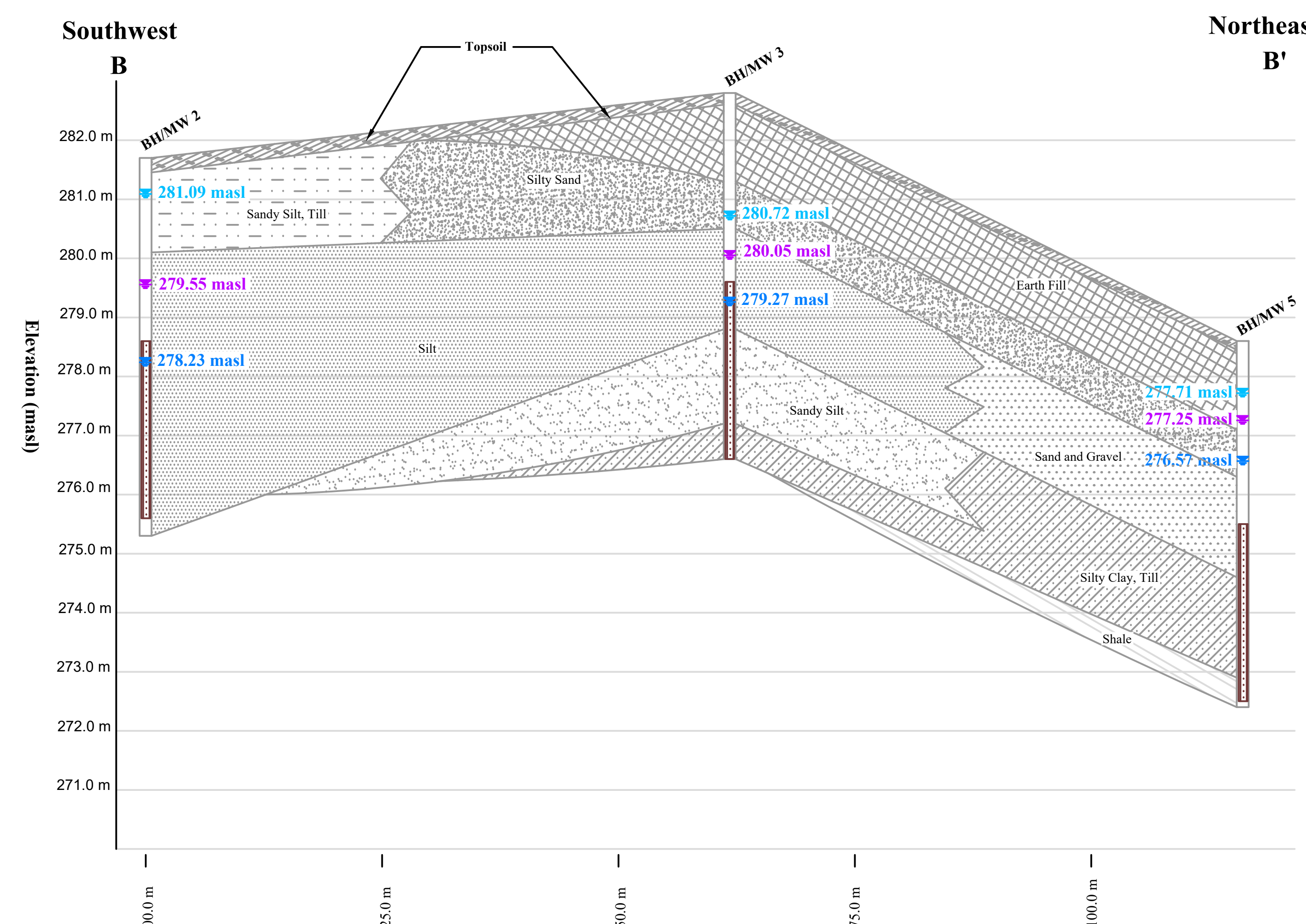
Metres

Drawing No. 8-1

Source: Ontario Ministry of Natural Resources and Forestry



Section A-A'



Section B-B'

Topsoil	Silty Clay	Silty Clay, Till
Earth, Fill	Silty Sand	Sandy Silt, Till
Silt	Sand and Gravel	Shale

3.0 m Screen

Water Table on March 16, 2023

Water Table on April 17, 2023

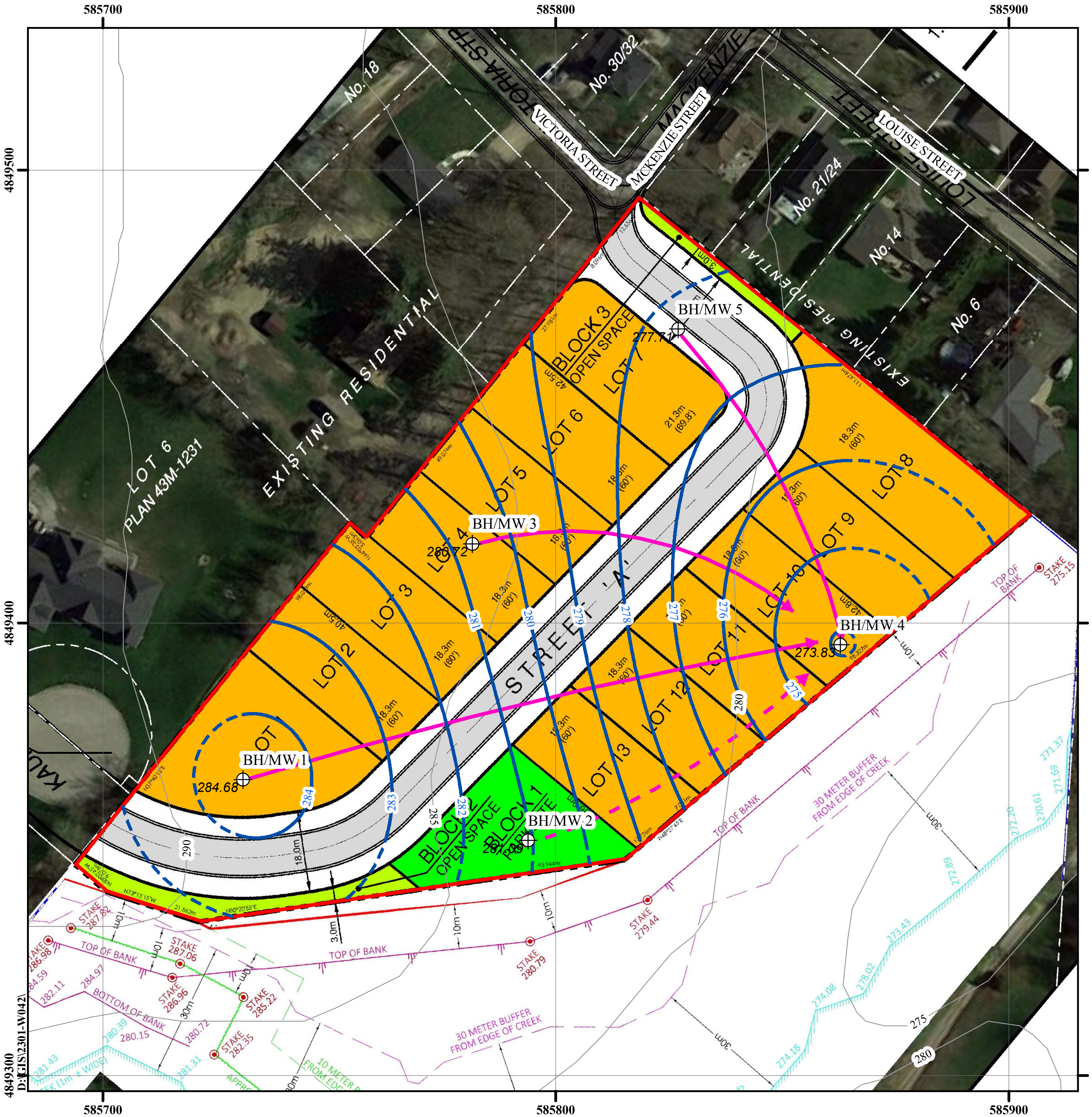
Water Table on May 18, 2023

Soil Engineers Ltd.
CONSULTING SOIL, FOUNDATION & ENVIRONMENTAL ENGINEERS

Title: Geological Cross-Section (A-A' and B-B')

Project: Hydrogeological Assessment and Baseline Well Canvassing and Interference Assessment: Proposed Synagogue and Residential Development
3825 Coronation Road, Town of Whitby, Ontario

Reference No:	Date:	Scale: V	Scale: H	Drawing No.
2303-W005	June, 2023	1:100	1:2500	8-2



N

Legend

Approximate Boundary of Subject Site

Borehole with Monitoring Well

Interpreted Shallow Groundwater Flow Direction

Inferred Shallow Groundwater Flow Direction

Interpreted Shallow Groundwater Level Elevation (masl)

Inferred Shallow Groundwater Level Elevation (masl)

Local Road

(285.7) Highest Shallow Groundwater Level Elevation (masl)

Soil Engineers Ltd.

Title: Shallow Groundwater Flow Pattern Plan

Project:

Hydrogeological Assessment
Proposed Residential Development
Address: 15544 McLaughlin Road
Town of Caledon

Reference No. 2301-W042

Date: January 20, 2023

Scale:

0 5 10 20 30 40 50

Metres

Drawing No. 9

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90 WEST BEAVER CREEK ROAD, SUITE 100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	MUSKOKA	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 542-2769

APPENDIX ‘A’

MECP WATER WELL RECORDS SUMMARY

REFERENCE NO. 2301-W042

Ontario Water Well Records

WELL ID	MECP WWR ID	Construction Method	Well Depth (m)	Well Usage		Water Found (m)	Static Water Level (m)	Top of Screen Depth (m)	Bottom of Screen Depth (m)
				Final Status	First Use				
1	4900713	Boring	3.70	Water Supply	Domestic	3.66	1.80	-	-
2	4900718	Boring	3.00	Water Supply	Industrial	1.22	0.90	-	-
3	4900719	Cable Tool	13.70	Water Supply	Domestic	11.89	3.00	-	-
4	4900720	Cable Tool	16.80	Water Supply	Public	13.72	3.40	-	-
5	4900721	Cable Tool	18.30	Water Supply	Domestic	12.19	4.30	-	-
6	4900722	Boring	6.10	Water Supply	Domestic	2.13	2.10	-	-
7	4900723	Cable Tool	18.30	Water Supply	Livestock	14.33	4.90	-	-
8	4900724	Cable Tool	59.70	Water Supply	Livestock	18.29	5.50	-	-
9	4900813	Cable Tool	18.30	Water Supply	Domestic	18.29	6.10	-	-
10	4900816	Cable Tool	20.40	Water Supply	Domestic	18.29	3.00	-	-
11	4900819	Cable Tool	17.40	Water Supply	Domestic	15.24	4.60	-	-
12	4900820	Cable Tool	18.00	Water Supply	Domestic	15.24	6.10	-	-
13	4900821	Boring	4.30	Water Supply	Domestic	3.35	2.40	-	-
14	4900822	Cable Tool	22.90	Water Supply	Domestic	12.19	9.10	-	-
15	4900823	Cable Tool	18.30	Water Supply	Domestic	18.29	6.10	-	-
16	4900824	Cable Tool	20.70	Water Supply	Domestic	16.76	4.90	-	-
17	4900825	Cable Tool	23.20	Water Supply	Domestic	23.17	8.50	-	-
18	4900826	Cable Tool	24.40	Water Supply	Domestic	24.38	9.80	-	-
19	4900827	Cable Tool	16.50	Water Supply	Domestic	13.72	7.60	-	-
20	4900828	Boring	8.80	Water Supply	Domestic	6.10	6.10	-	-
21	4900829	Cable Tool	15.20	Water Supply	Domestic	12.19	4.60	-	-
22	4900830	Cable Tool	25.90	Water Supply	Domestic	18.29	6.70	-	-
23	4900831	Cable Tool	17.70	Water Supply	Domestic	15.24	4.60	-	-
24	4900832	Cable Tool	20.40	Water Supply	Domestic	19.51	5.50	-	-
25	4900833	Boring	5.50	Water Supply	Domestic	4.27	2.10	-	-
26	4903526	Cable Tool	13.70	Water Supply	Domestic	12.19	0.60	12.50	13.72
27	4903646	Cable Tool	18.30	Water Supply	Domestic	15.24	6.70	-	-

Ontario Water Well Records

WELL ID	MECP WWR ID	Construction Method	Well Depth (m)	Well Usage		Water Found (m)	Static Water Level (m)	Top of Screen Depth (m)	Bottom of Screen Depth (m)
				Final Status	First Use				
28	4903787	Cable Tool	30.80	Water Supply	Domestic	29.26	-0.30	-	-
29	4903965	Cable Tool	17.10	Water Supply	Domestic	16.76	6.40	-	-
30	4903968	Cable Tool	15.80	Water Supply	Domestic	15.85	6.70	-	-
31	4903969	Cable Tool	15.20	Water Supply	Domestic	-	6.40	-	-
32	4904565	Cable Tool	22.90	Water Supply	Domestic	12.19	5.20	-	-
33	4906030	Cable Tool	29.60	Water Supply	Domestic	21.03	6.70	-	-
34	4906031	Rotary (Convent.)	61.60	Abandoned-Supply	Not Used	24.38	9.10	-	-
35	4906257	Rotary (Convent.)	19.80	Water Supply	Domestic	14.33	6.70	-	-
36	4908788	Not Known	-	Abandoned-Other	-	-	-	-	-
37	4908789	Not Known	-	Abandoned-Other	-	-	-	-	-
38	4908790	Not Known	-	Abandoned-Other	-	-	-	-	-
39	4908791	Not Known	-	Abandoned-Other	-	-	-	-	-
40	4908792	Not Known	-	Abandoned-Other	-	-	-	-	-
41	4908793	Not Known	-	Abandoned-Other	-	-	-	-	-
42	4908794	Not Known	-	Abandoned-Other	-	0.00	-	-	-
43	4907595	Rotary (Convent.)	35.70	Test Hole	Municipal	34.75	-	34.75	39.32
44	4907719	Rotary (Air)	14.00	Observation Wells	Not Used	-	2.40	-	-
45	4907720	Rotary (Air)	25.30	Observation Wells	Not Used	-	2.10	-	-
46	4910264	-	-	Abandoned-Other	-	-	2.20	-	-
47	4910275	-	-	-	-	-	7.10	-	-
48	4910276	-	-	Abandoned-Other	-	-	1.40	-	-
49	7112183	Rotary (Convent.)	11.60	Observation Wells	Monitoring	-	-	5.49	8.53
50	7112184	Rotary (Convent.)	11.60	Observation Wells	Monitoring	-	-	5.18	8.23
51	7112185	Rotary (Convent.)	11.60	Test Hole	Test Hole	1.22	1.30	5.49	8.53
52	7118560	-	-	Abandoned-Other	-	-	3.50	-	-
53	7145157	H.S.A.	-	Abandoned-Other	Dewatering	1.30	-	5.00	8.00
54	7145218	H.S.A.	-	Abandoned-Other	Dewatering	1.30	-	6.80	9.80

Ontario Water Well Records

WELL ID	MECP WWR ID	Construction Method	Well Depth (m)	Well Usage		Water Found (m)	Static Water Level (m)	Top of Screen Depth (m)	Bottom of Screen Depth (m)
				Final Status	First Use				
55	7145219	H.S.A.	-	Abandoned-Other	Dewatering	1.30	-	6.80	9.80
56	7145220	H.S.A.	-	Abandoned-Other	Dewatering	1.30	-	5.00	8.00
57	7150899	-	-	Abandoned-Other	-	1.00	-	-	-
58	7156441	-	-	Abandoned-Other	-	3.00	-	-	-
59	7160561	Jetting	7.00	Dewatering	Dewatering	1.00	-	6.00	7.00
60	7161740	-	-	Abandoned-Other	-	3.50	-	-	-
61	7168991	-	-	Abandoned-Other	Other	-	5.10	-	-
62	7180804	-	-	Abandoned-Other	-	-	-	-	-
63	7241495	Boring	6.10	Observation Wells	Monitoring	-	-	4.57	6.10
64	7241496	Boring	4.60	Observation Wells	Monitoring	3.05	-	3.05	4.57
65	7241497	Boring	6.10	Observation Wells	Monitoring	3.05	-	4.57	6.10
66	7255785	Other Method	48.20	-	Domestic	-	-0.30	5.49	8.53
67	7273717	-	-	Abandoned-Other	-	2.40	-	-	-
68	7315045	-	-	Abandoned-Other	-	4.60	-	-	-
69	7340775	-	-	Abandoned-Other	-	1.30	-	0.50	2.00
70	7340776	-	-	Abandoned-Other	-	1.30	-	0.50	2.00
71	7340777	-	-	Abandoned-Other	-	1.30	-	0.50	2.00
72	7381290	-	-	-	-	-	-	-	-
73	7381354	-	-	-	-	-	-	-	-
74	7382661	-	-	-	-	-	-	-	-

Notes:

*MECP WWID: Ministry of the Environment, Conservation and Parks Water Well Records Identification

**metres below ground surface

Soil Engineers Ltd.
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GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
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FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 542-2769

APPENDIX ‘B’

RESULTS OF SINGLE WELL RESPONSE TESTS

REFERENCE NO. 2301-W042

Falling Head Test (Slug Test)

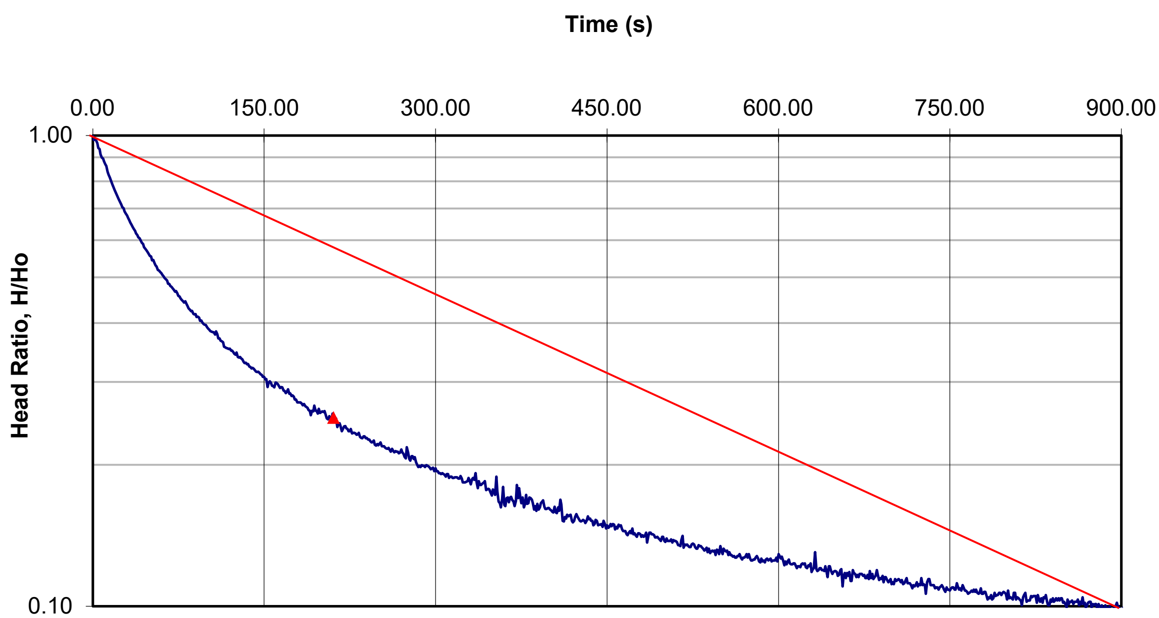
Test Date:		02-Mar-23	
Piezometer/Well No.:		BH/MW 1	
Ground level:		285.81	m
Screen top level:		282.71	m
Screen bottom level:		279.71	m
Test El. (at midpoint of screen):		281.21	m
Test depth (at midpoint of screen):		4.6	m
Screen length	L=	3.0	m
Diameter of undisturbed portion (	2R=	0.22	m
Standpipe diameter	2r=	0.05	m
Initial unbalanced head	Ho=	-0.175	m
Initial water depth		2.14	m
Aquifer material:		**Sandy Silt Till/Silty Sand Till**	

Shape factor F= $\dfrac{2 \times 3.14 \times L}{\ln(L/R)}$ = 5.701815 m

Permeability K= $\dfrac{3.14 \times r2}{F \times (t2 - t1)} \times \ln(H1/H2)$ (Bouwer and Rice Method)

$\dfrac{\ln(H1/H2)}{(t2 - t1)}$ = 0.01176031

K= **4.0E-04** cm/s
4.0E-06 m/s



Falling Head Test (Slug Test)

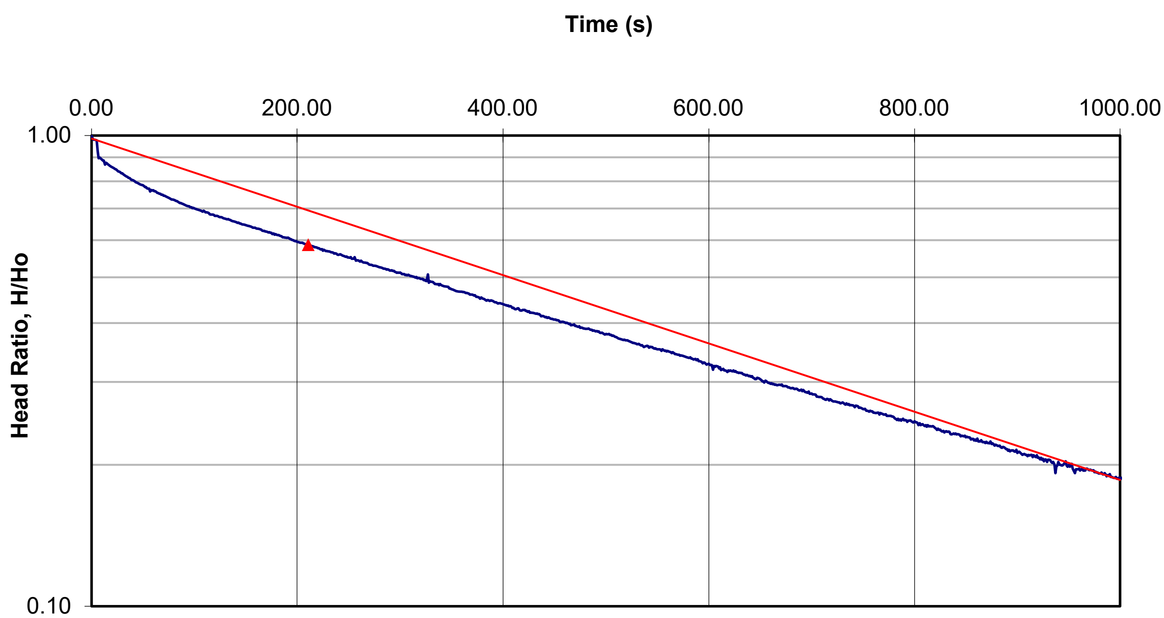
Test Date:		02-Mar-23	
Piezometer/Well No.:		BH/MW 2	
Ground level:		281.75	m
Screen top level:		278.65	m
Screen bottom level:		275.65	m
Test El. (at midpoint of screen):		277.15	m
Test depth (at midpoint of screen):		4.6	m
Screen length	L=	3.0	m
Diameter of undisturbed portion (	2R=	0.22	m
Standpipe diameter	2r=	0.05	m
Initial unbalanced head	Ho=	-0.2177	m
Initial water depth		2.2	m
Aquifer material:		**Silt**	

Shape factor F= $\dfrac{2 \times 3.14 \times L}{\ln(L/R)}$ = 5.701815 m

Permeability K= $\dfrac{3.14 \times r2}{F \times (t2 - t1)} \times \ln(H1/H2)$ (Bouwer and Rice Method)

$\dfrac{\ln(H1/H2)}{(t2 - t1)}$ = 0.00484079

K= **1.7E-04** cm/s
1.7E-06 m/s



Falling Head Test (Slug Test)

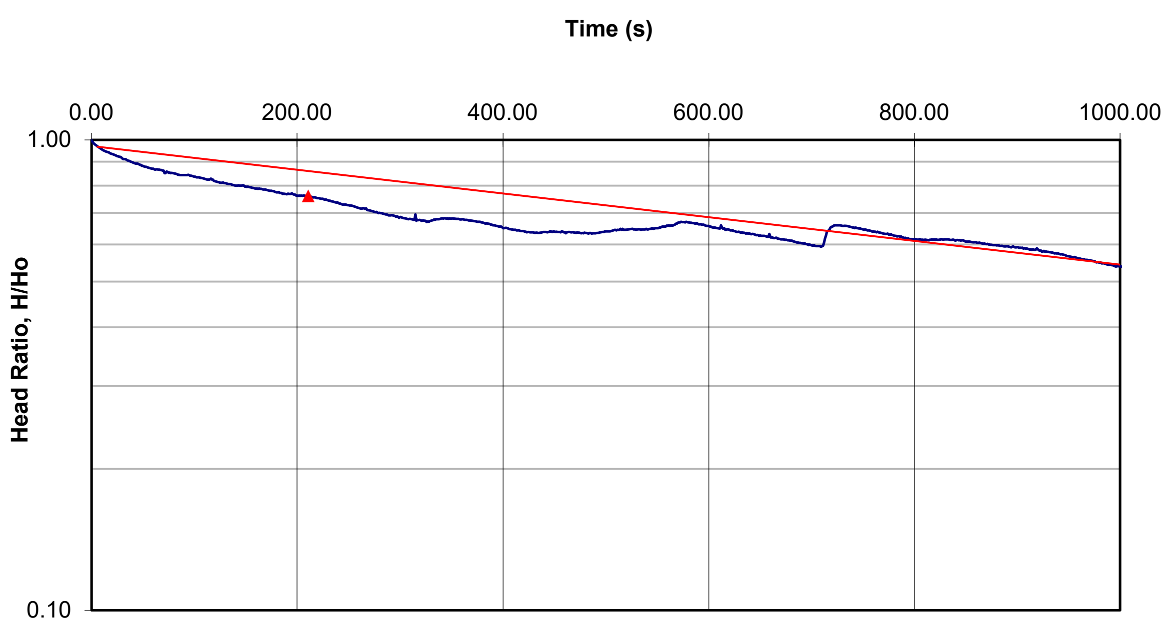
Test Date:		02-Mar-23	
Piezometer/Well No.:		BH/MW 3	
Ground level:		282.83	m
Screen top level:		279.63	m
Screen bottom level:		276.63	m
Test El. (at midpoint of screen):		278.13	m
Test depth (at midpoint of screen):		4.7	m
Screen length	L=	3.0	m
Diameter of undisturbed portion (	2R=	0.22	m
Standpipe diameter	2r=	0.05	m
Initial unbalanced head	Ho=	-0.2378	m
Initial water depth		2.78	m
Aquifer material:		**Sandy Silt/Silty Clay Till**	

Shape factor F= $\dfrac{2 \times 3.14 \times L}{\ln(L/R)}$ = 5.701815 m

Permeability K= $\dfrac{3.14 \times r2}{F \times (t2 - t1)} \times \ln(H1/H2)$ (Bouwer and Rice Method)

$\dfrac{\ln(H1/H2)}{(t2 - t1)}$ = 0.00314848

K= **1.1E-04** cm/s
1.1E-06 m/s



Falling Head Test (Slug Test)

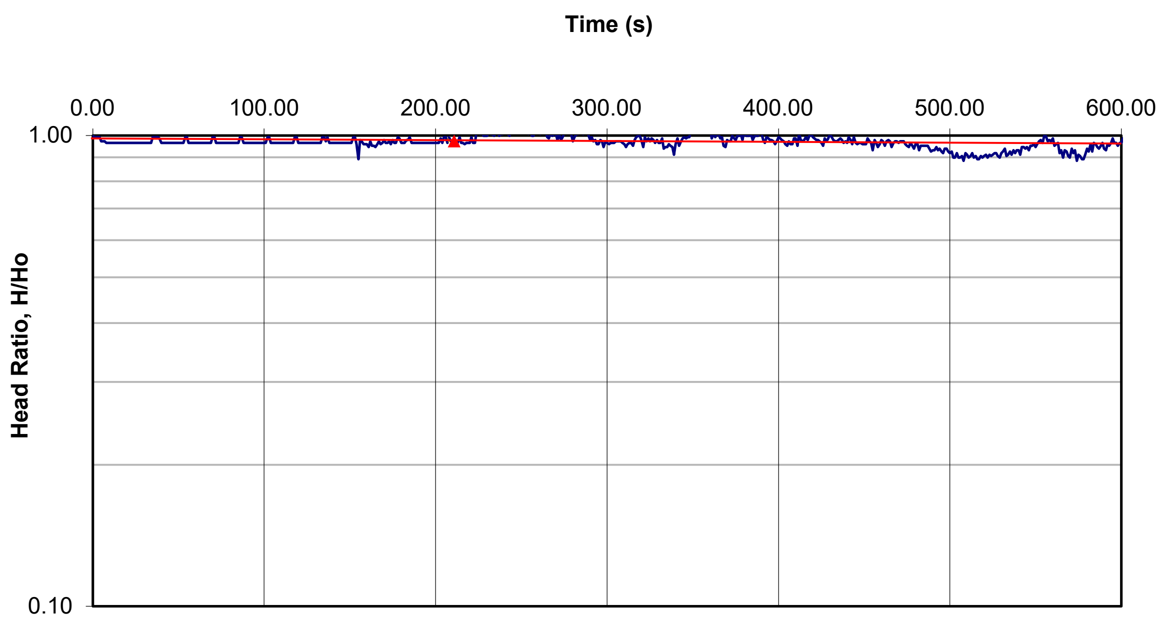
Test Date:		03-Apr-23	
Piezometer/Well No.:		BH/MW 4	
Ground level:		277.25	m
Screen top level:		274.15	m
Screen bottom level:		271.15	m
Test El. (at midpoint of screen):		272.65	m
Test depth (at midpoint of screen):		4.6	m
Screen length	L=	3.0	m
Diameter of undisturbed portion (	2R=	0.22	m
Standpipe diameter	2r=	0.05	m
Initial unbalanced head	Ho=	-0.0147	m
Initial water depth		3.42	m
Aquifer material:		**Sandy Silt Till / Silty Clay Till**	

Shape factor F= $\dfrac{2 \times 3.14 \times L}{\ln(L/R)}$ = 5.701815 m

Permeability K= $\dfrac{3.14 \times r2}{F \times (t2 - t1)}$ x ln (H1/H2) (Bouwer and Rice Method)

$\dfrac{\ln(H1/H2)}{(t2 - t1)}$ = 0.00173028

K= **6.0E-05** cm/s
6.0E-07 m/s



Falling Head Test (Slug Test)

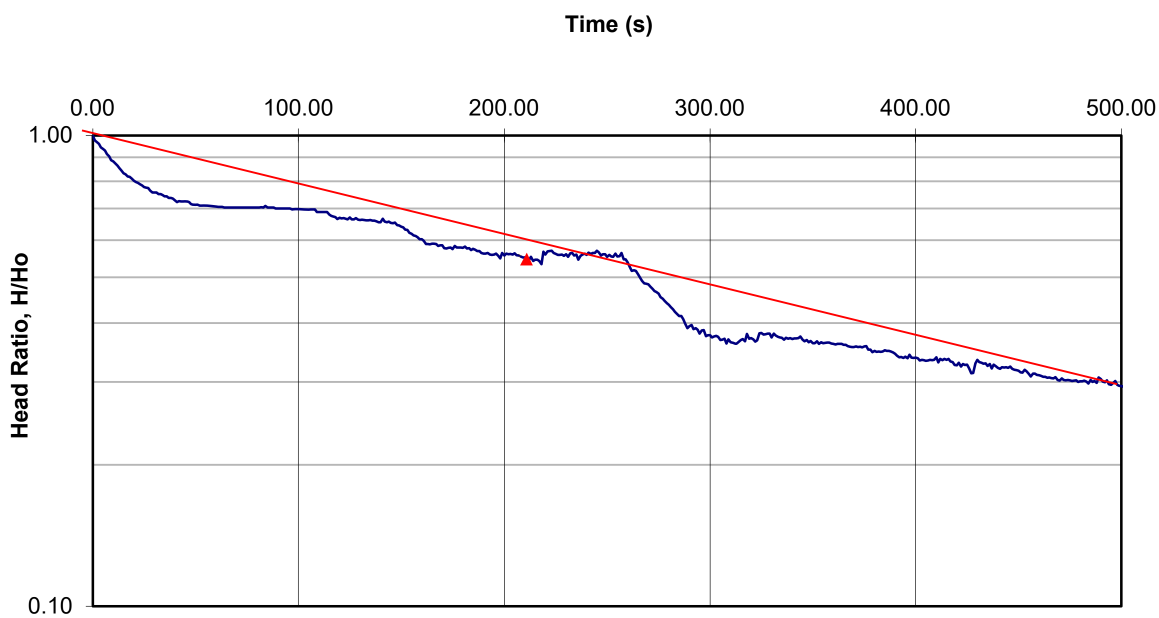
Test Date:		03-Apr-23	
Piezometer/Well No.:		BH/MW 5	
Ground level:		278.64	m
Screen top level:		275.54	m
Screen bottom level:		272.54	m
Test El. (at midpoint of screen):		274.04	m
Test depth (at midpoint of screen):		4.6	m
Screen length	L=	3.0	m
Diameter of undisturbed portion (	2R=	0.22	m
Standpipe diameter	2r=	0.05	m
Initial unbalanced head	Ho=	-0.1909	m
Initial water depth		0.93	m
Aquifer material:		**Silty Clay Till**	

Shape factor F= $\frac{2 \times 3.14 \times L}{\ln(L/R)}$ = 5.701815 m

Permeability K= $\frac{3.14 \times r2}{F \times (t2 - t1)} \times \ln(H1/H2)$ (Bouwer and Rice Method)

$\frac{\ln(H1/H2)}{(t2 - t1)}$ = 0.01007354

K= **3.5E-04** cm/s
3.5E-06 m/s



APPENDIX 'C'

WATER QUALITY TEST RESULTS

REFERENCE NO. 2301-W042

FINAL REPORT

CA40001-APR23 R1

2301-WO42, 15544 McLaughlin Rd, C.aledon

Prepared for

Soil Engineers Ltd.

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Soil Engineers Ltd.	Project Specialist	Maarit Wolfe, Hon.B.Sc
		Laboratory	SGS Canada Inc.
Address	90 West Beaver Creek Rd Richmond, ON M1S 3A7, Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Gurkaranbir Singh	Telephone	705-652-2000
Telephone	519-731-6442	Facsimile	705-652-6365
Facsimile		Email	Maarit.Wolfe@sgs.com
Email	gurkaranbir.singh@soilengineersltd.com	SGS Reference	CA40001-APR23
Project	2301-WO42, 15544 McLaughlin Rd, C.aledon	Received	04/03/2023
Order Number		Approved	04/11/2023
Samples	Ground Water (2)	Report Number	CA40001-APR23 R1
		Date Reported	04/11/2023

COMMENTS

RL - SGS Reporting Limit

Temperature of Sample upon Receipt: 6 degrees C
Cooling Agent Present: Yes
Custody Seal Present: Yes

Chain of Custody Number: 029455

F-ewl Spike Rep high, all other QC acceptable

SIGNATORIES

Maarit Wolfe, Hon.B.Sc

TABLE OF CONTENTS

FINAL REPORT

Client: Soil Engineers Ltd.
Project: 2301-WO42, 15544 McLaughlin Rd, C.aledon
Project Manager: Gurkaranbir Singh
Samplers: Gurkaranbir Singh

MATRIX: WATER

L1 = SANSEW / WATER / - - Peel Sewer Use ByLaw - Sanitary Sewer Discharge - BL_53_2010

L2 = SANSEW / WATER / - - Peel Sewer Use ByLaw - Storm Sewer Discharge - BL_53_2010

				Sample Number	8	9
				Sample Name	BH/MW1	BH/MW1 Dissolved
				Sample Matrix	Ground Water	Ground Water
				Sample Date	03/04/2023	03/04/2023
Parameter	**Units**	**RL**	**L1**	**L2**	**Result**	**Result**
General Chemistry						
Biochemical Oxygen Demand (BOD5)	mg/L	2	300	15	< 4↑	---
Total Suspended Solids	mg/L	2	350	15	12	---
Total Kjeldahl Nitrogen	as N mg/L	0.5	100	1	< 0.5	---
Metals and Inorganics						
Fluoride	mg/L	0.06	10		0.06	---
Cyanide (total)	mg/L	0.01	2	0.02	< 0.01	---
Sulphate	mg/L	2	1500		14	---
Aluminum (total)	mg/L	0.001	50		0.152	0.004
Antimony (total)	mg/L	0.0009	5		< 0.0009	< 0.0009
Arsenic (total)	mg/L	0.0002	1	0.02	< 0.0002	< 0.0002
Cadmium (total)	mg/L	0.000003	0.7	0.008	0.000144	0.000082
Chromium (total)	mg/L	0.00008	5	0.08	0.00133	0.00196
Copper (total)	mg/L	0.0002	3	0.05	0.0035	0.0019
Cobalt (total)	mg/L	0.000004	5		0.000245	0.000143
Lead (total)	mg/L	0.00009	3	0.12	0.00035	< 0.00009
Manganese (total)	mg/L	0.00001	5	0.05	0.0167	0.0139
Molybdenum (total)	mg/L	0.00004	5		0.00033	0.00026
Nickel (total)	mg/L	0.0001	3	0.08	0.0038	0.0068
Phosphorus (total)	mg/L	0.003	10	0.4	< 0.003	0.879
Selenium (total)	mg/L	0.00004	1	0.02	0.0128	0.00493
Silver (total)	mg/L	0.00005	5	0.12	< 0.00005	< 0.00005

FINAL REPORT

Client: Soil Engineers Ltd.

Project: 2301-WO42, 15544 McLaughlin Rd, C.aledon

Project Manager: Gurkaranbir Singh

Samplers: Gurkaranbir Singh

MATRIX: WATER

L1 = SANSEW / WATER / - - Peel Sewer Use ByLaw - Sanitary Sewer Discharge - BL_53_2010

L2 = SANSEW / WATER / - - Peel Sewer Use ByLaw - Storm Sewer Discharge - BL_53_2010

				Sample Number	8	9
				Sample Name	BH/MW1	BH/MW1 Dissolved
				Sample Matrix	Ground Water	Ground Water
				Sample Date	03/04/2023	03/04/2023
Parameter	**Units**	**RL**	**L1**	**L2**	**Result**	**Result**
Metals and Inorganics (continued)						
Tin (total)	mg/L	0.00006	5		0.00191	0.00047
Titanium (total)	mg/L	0.00005	5		0.00058	0.00110
Zinc (total)	mg/L	0.002	3	0.04	0.012	< 0.002
Microbiology						
E. Coli	cfu/100mL	0		200	< 2↑	---
Nonylphenol and Ethoxylates						
Nonylphenol	mg/L	0.001	0.02		< 0.001	---
Nonylphenol Ethoxylates	mg/L	0.01	0.2		< 0.01	---
Nonylphenol diethoxylate	mg/L	0.01			< 0.01	---
Nonylphenol monoethoxylate	mg/L	0.01			< 0.01	---
Oil and Grease						
Oil & Grease (total)	mg/L	2			< 2	---
Oil & Grease (animal/vegetable)	mg/L	4	150		< 4	---
Oil & Grease (mineral/synthetic)	mg/L	4	15		< 4	---

SGS

FINAL REPORT

CA40001-APR23 R1

Client: Soil Engineers Ltd.

Project: 2301-WO42, 15544 McLaughlin Rd, C.aledon

Project Manager: Gurkaranbir Singh

Samplers: Gurkaranbir Singh

MATRIX: WATER

L1 = SANSEW / WATER / - - Peel Sewer Use ByLaw - Sanitary Sewer Discharge - BL_53_2010

L2 = SANSEW / WATER / - - Peel Sewer Use ByLaw - Storm Sewer Discharge - BL_53_2010

				Sample Number	8	9
				Sample Name	BH/MW1	BH/MW1 Dissolved
				Sample Matrix	Ground Water	Ground Water
				Sample Date	03/04/2023	03/04/2023
Parameter	**Units**	**RL**	**L1**	**L2**	**Result**	**Result**
Other (ORP)						
pH	No unit	0.05	10	9	7.53	---
Mercury (total)	mg/L	0.00001	0.01	0.0004	< 0.00001	---
PCBs						
Polychlorinated Biphenyls (PCBs) - Total	mg/L	0.0001	0.001	0.0004	< 0.0001	---
Phenols						
4AAP-Phenolics	mg/L	0.002	1	0.008	< 0.002	---
SVOCs						
di-n-Butyl Phthalate	mg/L	0.002	0.08	0.015	< 0.002	---
Bis(2-ethylhexyl)phthalate	mg/L	0.002	0.012	0.0088	< 0.002	---
VOCs						
Chloroform	mg/L	0.0005	0.04	0.002	< 0.0005	---
1,2-Dichlorobenzene	mg/L	0.0005	0.05	0.0056	< 0.0005	---
1,4-Dichlorobenzene	mg/L	0.0005	0.08	0.0068	< 0.0005	---
cis-1,2-Dichloroethene	mg/L	0.0005	4	0.0056	< 0.0005	---
trans-1,3-Dichloropropene	mg/L	0.0005	0.14	0.0056	< 0.0005	---
Methylene Chloride	mg/L	0.0005	2	0.0052	< 0.0005	---
1,1,2,2-Tetrachloroethane	mg/L	0.0005	1.4	0.017	< 0.0005	---
Methyl ethyl ketone	mg/L	0.02	8		< 0.02	---
Styrene	mg/L	0.0005	0.2		< 0.0005	---
Tetrachloroethylene (perchloroethylene)	mg/L	0.0005	1	0.0044	< 0.0005	---
Trichloroethylene	mg/L	0.0005	0.4	0.008	< 0.0005	---

Client: Soil Engineers Ltd.

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MATRIX: WATER

L1 = SANSEW / WATER / - - Peel Sewer Use ByLaw - Sanitary Sewer Discharge - BL_53_2010

L2 = SANSEW / WATER / - - Peel Sewer Use ByLaw - Storm Sewer Discharge - BL_53_2010

				Sample Number	8	9
				Sample Name	BH/MW1	BH/MW1 Dissolved
				Sample Matrix	Ground Water	Ground Water
				Sample Date	03/04/2023	03/04/2023
Parameter	**Units**	**RL**	**L1**	**L2**	**Result**	**Result**
VOCs (continued)						
VOCs - BTEX						
Benzene	mg/L	0.0005	0.01	0.002	< 0.0005	---
Ethylbenzene	mg/L	0.0005	0.16	0.002	< 0.0005	---
Toluene	mg/L	0.0005	0.27	0.002	< 0.0005	---
Xylene (total)	mg/L	0.0005	1.4	0.0044	< 0.0005	---
m-p-xylene	mg/L	0.0005			< 0.0005	---
o-xylene	mg/L	0.0005			< 0.0005	---

EXCEEDANCE SUMMARY

Parameter	Method	Units	Result	SANSEW / WATER / - - Peel Sewer Use ByLaw - Sanitary Sewer Discharge - BL_53_2010 L1	SANSEW / WATER / - - Peel Sewer Use ByLaw - Storm Sewer Discharge - BL_53_2010 L2
BH/MW1 Dissolved					
Phosphorus	SM 3030/EPA 200.8	mg/L	0.879		0.4

QC SUMMARY

Anions by discrete analyzer

Method: US EPA 375.4 I Internal ref.: ME-CA-[ENV]EWL-LAK-AN-026

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate RPD	Duplicate AC (%)	LCS/Spike Blank Spike Recovery (%)	LCS/Spike Blank Recovery Limits (%) Low	LCS/Spike Blank Recovery Limits (%) High	Matrix Spike / Ref. Spike Recovery (%)	Matrix Spike / Ref. Recovery Limits (%) Low	Matrix Spike / Ref. Recovery Limits (%) High
Sulphate	DIO5011-APR23	mg/L	2	<2	1	20	110	80	120	112	75	125

Biochemical Oxygen Demand

Method: SM 5210 I Internal ref.: ME-CA-[ENV]EWL-LAK-AN-007

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate RPD	Duplicate AC (%)	LCS/Spike Blank Spike Recovery (%)	LCS/Spike Blank Recovery Limits (%) Low	LCS/Spike Blank Recovery Limits (%) High	Matrix Spike / Ref. Spike Recovery (%)	Matrix Spike / Ref. Recovery Limits (%) Low	Matrix Spike / Ref. Recovery Limits (%) High
Biochemical Oxygen Demand (BOD5)	BOD0001-APR23	mg/L	2	< 2	4	30	106	70	130	95	70	130

Cyanide by SFA

Method: SM 4500 I Internal ref.: ME-CA-[ENV]SFA-LAK-AN-005

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate RPD	Duplicate AC (%)	LCS/Spike Blank Spike Recovery (%)	LCS/Spike Blank Recovery Limits (%) Low	LCS/Spike Blank Recovery Limits (%) High	Matrix Spike / Ref. Spike Recovery (%)	Matrix Spike / Ref. Recovery Limits (%) Low	Matrix Spike / Ref. Recovery Limits (%) High
Cyanide (total)	SKA0039-APR23	mg/L	0.01	<0.01	ND	10	100	90	110	96	75	125

QC SUMMARY

Fluoride by Specific Ion Electrode

Method: SM 4500 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-014

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Fluoride	EWL0029-APR23	mg/L	0.06	<0.06	ND	10	103	90	110	58	75	125

Mercury by CVAAS

Method: EPA 7471A/SM 3112B | Internal ref.: ME-CA-[ENV]SPE-LAK-AN-004

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Mercury (total)	EHG0004-APR23	mg/L	0.00001	< 0.00001	ND	20	105	80	120	117	70	130

QC SUMMARY

Metals in aqueous samples - ICP-MS

Method: SM 3030/EPA 200.8 | Internal ref.: ME-CA-[ENV]SPE-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Silver (total)	EMS0010-APR23	mg/L	0.00005	<0.00005	ND	20	102	90	110	85	70	130
Aluminum (total)	EMS0010-APR23	mg/L	0.001	<0.001	2	20	95	90	110	108	70	130
Arsenic (total)	EMS0010-APR23	mg/L	0.0002	<0.0002	ND	20	99	90	110	102	70	130
Cadmium (total)	EMS0010-APR23	mg/L	0.000003	<0.000003	6	20	105	90	110	95	70	130
Cobalt (total)	EMS0010-APR23	mg/L	0.000004	<0.000004	2	20	100	90	110	94	70	130
Chromium (total)	EMS0010-APR23	mg/L	0.00008	<0.00008	ND	20	101	90	110	100	70	130
Copper (total)	EMS0010-APR23	mg/L	0.0002	<0.0002	1	20	102	90	110	85	70	130
Manganese (total)	EMS0010-APR23	mg/L	0.00001	<0.00001	4	20	100	90	110	113	70	130
Molybdenum (total)	EMS0010-APR23	mg/L	0.00004	<0.00004	7	20	103	90	110	102	70	130
Nickel (total)	EMS0010-APR23	mg/L	0.0001	<0.0001	20	20	103	90	110	84	70	130
Lead (total)	EMS0010-APR23	mg/L	0.00009	<0.00009	6	20	106	90	110	91	70	130
Antimony (total)	EMS0010-APR23	mg/L	0.0009	<0.0009	ND	20	107	90	110	111	70	130
Selenium (total)	EMS0010-APR23	mg/L	0.00004	<0.00004	ND	20	94	90	110	NV	70	130
Tin (total)	EMS0010-APR23	mg/L	0.00006	<0.00006	ND	20	102	90	110	NV	70	130
Zinc (total)	EMS0010-APR23	mg/L	0.002	<0.002	3	20	99	90	110	129	70	130
Phosphorus (total)	EMS0034-APR23	mg/L	0.003	0.008	1	20	100	90	110	NV	70	130
Titanium (total)	EMS0034-APR23	mg/L	0.00005	<0.00005	7	20	110	90	110	NV	70	130

QC SUMMARY

Microbiology

Method: SM 9222D | Internal ref.: ME-CA-[ENV]MIC-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
E. Coli	BAC9005-APR23	cfu/100mL	-	ACCEPTED	ACCEPTED							

Nonylphenol and Ethoxylates

Method: ASTM D7065-06 | Internal ref.: ME-CA-[ENV]GC-LAK-AN-015

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Nonylphenol diethoxylate	GCM0034-APR23	mg/L	0.01	<0.01			86	55	120			
Nonylphenol Ethoxylates	GCM0034-APR23	mg/L	0.01	0								
Nonylphenol monoethoxylate	GCM0034-APR23	mg/L	0.01	<0.01			87	55	120			
Nonylphenol	GCM0034-APR23	mg/L	0.001	<0.001			87	55	120			

FINAL REPORT

QC SUMMARY

Oil & Grease

Method: MOE E3401 | Internal ref.: ME-CA-[ENV]GC-LAK-AN-019

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Oil & Grease (total)	GCM0064-APR23	mg/L	2	<2	NSS	20	107	75	125			

Oil & Grease-AV/MS

Method: MOE E3401/SM 5520F | Internal ref.: ME-CA-[ENV]GC-LAK-AN-019

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Oil & Grease (animal/vegetable)	GCM0064-APR23	mg/L	4	< 4	NSS	20	NA	70	130			
Oil & Grease (mineral/synthetic)	GCM0064-APR23	mg/L	4	< 4	NSS	20	NA	70	130			

pH

Method: SM 4500 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0022-APR23	No unit	0.05	NA	0		100			NA		

QC SUMMARY

Phenols by SFA

Method: SM 5530B-D | Internal ref.: ME-CA-[ENV]SFA-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
4AAP-Phenolics	SKA0015-APR23	mg/L	0.002	<0.002	ND	10	96	80	120	102	75	125

Polychlorinated Biphenyls

Method: MOE E3400/EPA 8082A | Internal ref.: ME-CA-[ENV]GC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Polychlorinated Biphenyls (PCBs) - Total	GCM0050-APR23	mg/L	0.0001	<0.0001	NSS	30	89	60	140	NSS	60	140

QC SUMMARY

Semi-Volatile Organics

Method: EPA 3510C/8270D | Internal ref.: ME-CA-[ENV]GC-LAK-AN-005

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Bis(2-ethylhexyl)phthalate	GCM0078-APR23	mg/L	0.002	< 0.002	NSS	30	105	50	140	NSS	50	140
di-n-Butyl Phthalate	GCM0078-APR23	mg/L	0.002	< 0.002	NSS	30	110	50	140	NSS	50	140

Suspended Solids

Method: SM 2540D | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-004

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Total Suspended Solids	EWL0028-APR23	mg/L	2	< 2	0	10	100	90	110	NA		

Total Nitrogen

Method: SM 4500-N C/4500-NO3- F | Internal ref.: ME-CA-[ENV]SFA-LAK-AN-002

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Total Kjeldahl Nitrogen	SKA0028-APR23	as N mg/L	0.5	<0.5	1	10	99	90	110	98	75	125

QC SUMMARY

Volatile Organics

Method: EPA 5030B/8260C | Internal ref.: ME-CA-[ENV]GC-LAK-AN-004

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
1,1,2,2-Tetrachloroethane	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	101	60	130	106	50	140
1,2-Dichlorobenzene	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	103	60	130	105	50	140
1,4-Dichlorobenzene	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	101	60	130	103	50	140
Benzene	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	105	60	130	107	50	140
Chloroform	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	102	60	130	106	50	140
cis-1,2-Dichloroethene	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	103	60	130	106	50	140
Ethylbenzene	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	105	60	130	108	50	140
m-p-xylene	GCM0046-APR23	mg/L	0.0005	<0.0005	9	30	104	60	130	108	50	140
Methyl ethyl ketone	GCM0046-APR23	mg/L	0.02	<0.02	ND	30	103	50	140	111	50	140
Methylene Chloride	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	102	60	130	102	50	140
o-xylene	GCM0046-APR23	mg/L	0.0005	<0.0005	13	30	105	60	130	108	50	140
Styrene	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	105	60	130	108	50	140
Tetrachloroethylene (perchloroethylene)	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	103	60	130	106	50	140
Toluene	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	104	60	130	107	50	140
trans-1,3-Dichloropropene	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	103	60	130	105	50	140
Trichloroethylene	GCM0046-APR23	mg/L	0.0005	<0.0005	ND	30	103	60	130	104	50	140

QC SUMMARY

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS Insufficient sample for analysis.
RL Reporting Limit.
↑ Reporting limit raised.
↓ Reporting limit lowered.
NA The sample was not analysed for this analyte
ND Non Detect

Results relate only to the sample tested.
Data reported represent the sample as submitted to SGS. Solid samples expressed on a dry weight basis.
"Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.
Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the "Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act and Excess Soil Quality" published by the Ministry and dated March 9, 2004 as amended.

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated.

SGS Canada Inc. statement of conformity decision rule does not consider uncertainty when analytical results are compared to a specified standard or regulation.

This report supersedes all previous versions.

-- End of Analytical Report --

SGS

Request for Laboratory Services and CHAIN OF CUSTODY

No: 029455

Industries & Environment - Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Fax: 705-652-6365 Web: www.sgs.com/environment
- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361

Page ____ of ____

Laboratory Information Section - Lab use only

Received By: Scot Ran
Received By (signature):
Received Date: 04/03/2023 (mm/dd/yy)
Received Time: 11:45 (hr : min)
Custody Seal Present: Yes ☑ No ☐
Custody Seal Intact: Yes ☑ No ☐
Cooling Agent Present: Yes ☑ No ☐ Type: ice
Temperature Upon Receipt (°C) 6,7,3
LAB LIMS #: CA40001-APR23

REPORT INFORMATION

Company: SOIL ENGINEERS LTD
Contact: Gurkaran
Address: 90 West Beaver Creek Rd, Richmond Hill.
Phone: 519 7316442
Fax:
Email: gurkaranbir.singh@soilengineersltd.com

INVOICE INFORMATION

☐ (same as Report Information)
Company:
Contact:
Address:
Phone:
Email:

Quotation #:
P.O. #:
Project #: 2301-W042
Site Location/ID: 15544 Mclaughlin Rd Caledon

TURNAROUND TIME (TAT) REQUIRED

☑ Regular TAT (5-7days)

TAT's are quoted in business days (exclude statutory holidays & weekends). Samples received after 6pm or on weekends: TAT begins next business day

RUSH TAT (Additional Charges May Apply): ☐ 1 Day ☐ 2 Days ☐ 3 Days ☐ 4 Days

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date:

*NOTE: DRINKING (POTABLE) WATER SAMPLES FOR HUMAN CONSUMPTION MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY

REGULATIONS

☐ O.Reg 153/04 ☐ O.Reg 406/19
☐ Table 1 ☐ Res/Park Soil Texture:
☐ Table 2 ☐ Ind/Com ☐ Coarse
☐ Table 3 ☐ Agri/Other ☐ Medium/Fine
☐ Table ____ Appx. ____
Soil Volume ☐ <350m3 ☐ >350m3

Other Regulations:
☐ Reg 347/558 (3 Day min TAT)
☐ PWQO ☐ MMER
☐ CCME ☐ Other:
☐ MISA
☐ ODWS Not Reportable *See note

Sewer By-Law:
☑ Sanitary
☑ Storm
Municipality: Region of Peel

RECORD OF SITE CONDITION (RSC) ☐ YES ☐ NO

ANALYSIS REQUESTED

Analysis groups: M & I — Field Filtered (Y/N); Metals & Inorganics (incl CrVI, CN, Hg pH, (B(HWS), EC, SAR-soil) (Cl, Na-water)); Full Metals Suite (ICP metals plus B(HWS-soil only) Hg, CrVI); ICP Metals only (Sb, As, Ba, Be, B, Cd, Cr, Co, Cu, Pb, Mo, Ni, Se, Ag, Tl, U, V, Zn). SVOC — PAHs only; SVOCs (all incl PAHs, ABNs, CPs). PCB — PCBs (☐ Total ☐ Aroclor). PHC — F1-F4 + BTEX; F1-F4 only (no BTEX). VOC — VOCs (all incl BTEX); BTEX only. Pest — Pesticides (Organochlorine or specify other). Other (please specify) — Metal Dissolved; Sewer Use: Specify pkg: Sanitary/Storm, Region of Peel; Water Characterization Pkg (☐ General ☐ Extended). SPLP — Specify tests: ☐ Metals ☐ VOC ☐ 1,4-Dioxane ☐ OCP ☐ ABN. TCLP — Specify tests: ☐ M&I ☐ VOC ☐ PCB ☐ B(a)P ☐ ABN ☐ Ignit.

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	# OF BOTTLES	MATRIX	Field Filtered (Y/N)	Metal Dissolved	Sewer Use	COMMENTS
1	BH/MW1	03/04/23	10:00AM	17	GW	N		✓	
2	BH/MW1	03/04/23	10:00AM	1	GW	Y	✓		
3									
4									
5									
6									
7									
8									
9									
10									
11									
12									

Observations/Comments/Special Instructions

Sampled By (NAME): GURKARANBIR SINGH Signature: Date: 03/04/23 (mm/dd/yy) Pink Copy - Client
Relinquished by (NAME): Gurkaranbir Singh Signature: Date: 03/04/23 (mm/dd/yy) Yellow & White Copy - SGS

Revision #: 1.6
Date of Issue: 02 May 2022

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE 100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	MUSKOKA	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 542-2769

APPENDIX ‘D’

TEST PIT INVESTIGATION

REFERENCE NO. 2301-W042

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
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July 11, 2023

Reference No. 2301-W042

2868577 Ontario Inc.
4510 Eastgate Parkway
Mississauga, Ontario
L4W 3W6

Attention: Mr. Graziano Stefani

**Re: Follow-Up Test Pit Investigation - Groundwater Conditions Verification
Proposed Residential Development
15544 Mclaughlin Road
Town of Caledon**

Dear Sir:

On May 30, 2023, a Soil Engineers Ltd. representative performed a site visit to witness a test pit investigation program. Test pit excavations were completed at the subject subdivision, located about 200 m west of Mclaughlin Road, and approximately 470 m north of Old Base Line Road, at the Terminus of Kaufman Road, with a municipality address of 15544 McLaughlin Road, in the Town of Caledon, at the location shown on Drawing No. 1. An excavator was used to complete the test pit excavations to the target depth at the indicated test pit locations that were provided in advance by Candevcon Limited.

In total five (5) test pits were excavated on May 30, 2023, to depths, of about ±5.0 m respectively below the existing grade, or to the depth elevations, ranging from 272.3 to 280.2 masl, respectively. The test pit locations are shown on Drawing No. 2. The depths for the test pits were selected based on the anticipated depths for the proposed housing basement structures, and for the proposed underground services. Groundwater conditions were recorded at each of the open test pits, during the field investigation, along with the visual examination of the contacted subsoil strata, to confirm for the presence of ant groundwater seepage, or any caving and unstable subsoil conditions within the open test pits. The test pits were left open and were examined for a period of ±4.0 to 6.0 hours to allow for any groundwater seepage, if present, to accumulate and stabilize within the open excavations.

The ground surface elevations and horizontal coordinates at the test pit locations were determined at the time of the investigation, using a handheld Global Navigation Satellite System survey equipment (Trimble Geoexplorer unit TSC3) which has an accuracy of ±0.05 m. The UTM coordinates and ground surface elevations at the test pit locations, along with the field observations recorded from the test pit investigation are summarized in Table 1, below.

Table 1 - Summary of Test Pit Investigation Findings

Test Pit No.	Existing Ground El. (masl)	Depth of Test Pit Excavation (mbgs/masl)	UTM Coordinates		Sub-Soil Type	Groundwater Seepage Depth (mbgs/masl)	Test Pit Observations
			East (m)	North (m)			
1	±285.2	5.0/280.2	585737	4849365	Topsoil 0 to 0.30 mbgs Brown, loose to compact Sandy Silt, a trace of Clay and occ. Gravel 0.3 to 1.6 mbgs Brown, compact to very dense, Sandy Silt Till/Silty Sand Till, having a trace of clay and some gravel to gravelly 1.6 to 5.0 mbgs	2.7/282.50	Minimal groundwater seepage at depth of 2.7 mbgs (282.50 masl) Minimal accumulation of groundwater within the test pit after leaving the test pit remained open for ±5.0 hours Cave-In occurred at a depth of 0.8 mbgs (El. 284.4 masl)
2	±281.7	5.0/276.7	585794	4849357	Topsoil 0 to 0.30 mbgs Brown, very loose to compact, Sandy Silt Till and traces of clay and gravel 0.3 to 1.6 mbgs Brown, compact to very dense, Silt, and a trace to some Sand 1.6 to 5.0 mbgs	No Groundwater Seepage	No groundwater seepage Test pit left open for ±4.0 hours Cave-In occurred at 0.3 mbgs (El 281.4 masl)

Table 1 - Summary of Test Pit Investigation Findings (Cont'd-1)

Test Pit No.	Existing Ground El. (masl)	Depth of Test Pit Excavation (mbgs/masl)	UTM Coordinates		Sub-Soil Type	Groundwater Seepage Depth (mbgs/masl)	Test Pit Observations
			East (m)	North (m)			
3	±283.0	5.0/278.0	585780	4849412	Topsoil 0 to 0.30 mbgs Dark Brown, Earth Fill, Sand, some Silt, occ. Organics and Rootlets 0.3 to 1.7 mbgs Brown, compact Silty Sand, occ. Silty Clay Layers 1.7 to 2.5 mbgs Brown, compact Silt and traces of Clay and Gravel 2.5 to 4.2 mbgs Brown, compact Sandy Silt and traces of Clay and occ. Gravel 4.2 to 5.0 mbgs	1.6 / 281.4	Minimal water seepage at depth of 1.6 mbgs (281.4 masl) Minimal accumulation of groundwater within the test pit after leaving the test pit remained open for ±6.0 hours
4	±277.3	5.0/272.3	585857	4849398	Topsoil 0 to 0.20 mbgs Dark Brown, Earth Fill, Sand, Silt, Clay, a trace of Gravel, occ. Organics and Rootlets 0.2 to 2.2 mbgs Brown, dense Sand and, Gravel and a trace to some Silt 2.2 to 5.0 mbgs	3.5 / 273.8	Medium to minor ground water seepage at depth of 3.5 mbgs (El. 273.80 masl) Minimal to medium accumulation of groundwater seepage within the test pit after leaving the test pit remained open for ±4.0 hours

Table 1 - Summary of Test Pit Investigation Findings (Cont'd-2)

Test Pit No.	Existing Ground El. (masl)	Depth of Test Pit Excavation (mbgs/masl)	UTM Coordinates		Sub-Soil Type	Groundwater Seepage Depth (mbgs/masl)	Test Pit Observations
			East (m)	North (m)			
5	±278.4	5.0/273.4	585829	4849469	Topsoil 0 to 0.20 mbgs Dark Brown, Earth Fill, Sand, some Silt to Silty, occ. Organics and Rootlets 0.2 to 1.2 mbgs Brown, compact Silty Sand and a trace of Clay 1.3 to 2.1 mbgs Brown, dense Sand, and Gravel and trace to some Silt 2.1 to 3.8 mbgs Brown, hard Silty Clay Till and traces of Gravel 3.8 to 5.0 mbgs	4.75/273.65	Minor groundwater seepage at depth of 4.7 mbgs (El. 273.65 masl) Minimal accumulation of groundwater seepage within the test pit after leaving the test pit left open for ±4.0 hrs

The subsoil at all of the test pits is comprised, primarily of silty sand, sand and gravel and silty clay till, silt and sandy silt, having trace to some gravel. Detailed descriptions are shown on Figures 1 and 5, inclusive.

Comparison of Groundwater Elevations and Observed Groundwater Levels within the Test Pits

Test Pits 1, 2, 3, 4 and 5 are located, adjacent to the BH/MWs 1, 2, 3, 4 and 5 locations. The records for the groundwater level measurements and the comparison between the levels within the monitoring wells and the TPs are summarized in the following Table 6-4 below.

Table 6-4 - Comparison of Previous Groundwater Level Measurements and Groundwater at Test Pit locations

Well ID	Depth Units	Groundwater Level (May 30, 2023)	Test Pit (TP)	Depth Units	Groundwater Seepage Elevations in Test Pits
BH/MW 1	mbgs	1.94	TP 1	mbgs	2.7
	masl	283.87		masl	282.5
BH/MW 2	mbgs	1.61	TP 2	mbgs	<5.0
	masl	280.1		masl	<276.7
BH/MW 3	mbgs	2.82	TP 3	mbgs	1.6
	masl	280.0		masl	281.4
BH/MW 4	mbgs	3.94	TP 4	mbgs	3.5
	masl	273.3		masl	273.8
BH/MW 5	mbgs	2.2	TP 5	mbgs	4.75
	masl	276.4		masl	273.65

Review of the groundwater level elevations recorded from within the test pits when compared to the concurrent groundwater level elevations within the monitoring wells, indicates that that the water levels are higher within the BH/MWs than those observed within the adjacent test pit locations. The groundwater level at the BH/MW1 location is 0.8 m higher than the water level elevation for the groundwater seepage observed at the TP 1. The groundwater level at the BH/MW 2 location is 3.4 m higher than the elevation for the groundwater seepage observed at the TP 2. The groundwater level at the BH/MW 3 location, is about 1.2 m lower than the elevation for the groundwater seepage observed at the TP 3 location. The groundwater level at the BH/MW 4 location, is about 0.4 m lower than the elevation for the groundwater seepage observed at the TP 4 location. The groundwater level at the BH/MW 5 location, is about 2.5 m higher than the elevation for the groundwater seepage observed at the TP 5 location. Based on the overall current observations, only minor groundwater seepage was observed within the test pit excavations, and minor accumulation of groundwater seepage within all the open test pits, with the exception of TP 4 where a more moderate to medium accumulation of water seepage was observed after the pits were left open for four hours following excavation. Based on these findings, it is concluded that there will be only limited, un-sustained groundwater seepage at the anticipated depths for the proposed housing basement structures and associated underground services installation depths. As such only minor, un-sustained occasional groundwater seepage might occur at the depths for conventional foundations drainage networks for the completed housing basements.

We trust that this correspondence addresses your current requirements and ask that you contact us should you have any questions or require additional information.

Yours truly,
SOIL ENGINEERS LTD.

Bhawandeep Singh. Brar, B.Sc.

Gavin O'Brien, M.Sc. P.Geo.
BB/GO

ENCLOSURES

Test Pit Logs………………………………………..………………....	Figures 1 to 5
Site Location Plan ………………………………………………………..	Drawing No.1
Test Pit Location Plan ………………………………………………	Drawing No. 2

LOG OF BOREHOLE: Test Pit 1

JOB NO.: 2301-W042 **FIGURE NO.:** 10

PROJECT DESCRIPTION: Proposed Residential Development **METHOD** Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon **TEST PIT DATE:** May 30, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
285.2	Ground Surface							
0.0 / 284.9	**30 cm Topsoil**				0			
0.3 / 283.6	Brown, loose to compact **SANDY SILT** a trace of clay occ. gravel				1			cave-in occured elevation @ 284.40 masl
1.6 / 280.2	Brown, compact to very dense **SANDY SILT TILL / SILTY SAND TILL** a trace of clay some gravel to gravelly				2, 3, 4			water seepage elevation @ 282.50 masl
5.0	**END OF TEST PIT**				5			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
Water seepage occured @ 2.7 mbgs

Minor seepage rate

Cave-In
Cave-In occured @ 0.8 mbgs

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 6.0 hours

Time	Water Level (from bottom of test pit)
10:00 am	1 cm
10:10 am	2 cm
10:30 am	8 cm
11:45 am	15 cm
12:15 pm	18 cm
01:15 pm	19 cm
02:30 pm	21 cm
03:30 pm	23 cm
04:00 pm	24 cm

Depth scale: 6, 7, 8, 9, 10

Soil Engineers Ltd.

JOB NO.: 2301-W042 **LOG OF BOREHOLE: Test Pit 2** **FIGURE NO.:** 11

PROJECT DESCRIPTION: Proposed Residential Development **METHOD** Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon **TEST PIT DATE:** May 30, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES: Number	SAMPLES: Type	SAMPLES: N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
281.7	Ground Surface							
0.0 / 281.4	**30 cm Topsoil**				0			cave-in occured elevation @ 281.4 masl
0.3	Brown, very loose to compact **SANDY SILT TILL** traces of clay and gravel				1			
280.1 / 1.6	Brown, compact to very dense **SILT** a trace to some sand				2, 3, 4			
276.7 / 5.0	**END OF TEST PIT**				5, 6, 7, 8, 9, 10			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
No water seepage occured during the time interval

Cave-In
Cave-In occured @ 0.3 mbgs

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 4.0 hours

Time	Water Level (from bottom of test pit)
10:45 am	dry
11:15 am	dry
12:00 pm	dry
12:45 pm	dry
01:15 pm	dry
02:15 pm	dry
02:45 pm	dry

Soil Engineers Ltd.

JOB NO.: 2301-W042 **LOG OF BOREHOLE: Test Pit 3** **FIGURE NO.:** 12

PROJECT DESCRIPTION: Proposed Residential Development **METHOD** Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon **TEST PIT DATE:** May 30, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
283.0	Ground Surface							
0.0 / 282.7	**30 cm Topsoil**				0			
0.2	Dark brown **EARTH FILL** sand, some silt occ. topsoil inclusion occ. organics and rootlets				1			water seepage elevation @ 281.40 masl
281.3 / 1.7	Brown, compact **SILTY SAND** occ. silty clay layers				2			
280.5 / 2.5	Brown, compact **SILT** traces of clay and gravel				3, 4			
278.8 / 4.2	Brown, compact **SANDY SILT** a trace of clay occ. gravel							
278.0 / 5.0	**END OF TEST PIT**				5			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
Water seepage occured @ 1.6 mbgs

Minimal Seepage rate

Cave-In
No cave-in occured during the time interval

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 6.0 hours

Time	Water Level (from bottom of test pit)
11:20 am	1 cm
11:45 am	3 cm
12:05 pm	7 cm
01:15 pm	9 cm
02:00 pm	11 cm
03:00 pm	13 cm
04:15 pm	15 cm
05:20 pm	18 cm

Depth scale: 0 1 2 3 4 5 6 7 8 9 10

Soil Engineers Ltd.

JOB NO.: 2301-W042 **LOG OF BOREHOLE: Test Pit 4** **FIGURE NO.:** 13

PROJECT DESCRIPTION: Proposed Residential Development **METHOD** Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon **TEST PIT DATE:** May 30, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES: Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90; Shear Strength (kN/m²) 50 100 150 200; Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL; Moisture Content (%) 10 20 30 40	WATER LEVEL
277.3	Ground Surface							
0.0 / 0.2	**20 cm Topsoil**				0			
275.1 / 2.2	Dark brown **EARTH FILL** mixture of sand, silt and clay a trace of gravel occ. topsoil inclusion occ. organics and rootlets				1, 2			
272.3 / 5.0	Brown, dense **SAND AND GRAVEL** a trace to some silt				3, 4, 5			water seepage elevation @ 273.80 masl
	END OF TEST PIT				6, 7, 8, 9, 10			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
Water seepage occured @ 3.5 mbgs

Medium to Fast seepage rate

Cave-In
No cave-In occured during the time interval

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 4.0 hours

Time	Water Level (from bottom of test pit)
12:00 pm	50 cm
12:20 pm	70 cm
01:15 pm	85 cm
02:00 pm	95 cm
03:10 pm	110 cm
04:00 pm	120 cm

Soil Engineers Ltd.

JOB NO.: 2301-W042

LOG OF BOREHOLE: Test Pit 5

FIGURE NO.: 14

PROJECT DESCRIPTION: Proposed Residential Development

METHOD: Backhoe

PROJECT LOCATION: 15544 McLaughlin Road, Town of Caledon

TEST PIT DATE: May 30, 2023

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES: Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90; Shear Strength (kN/m²) 50 100 150 200; Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL; Moisture Content (%) 10 20 30 40	WATER LEVEL
278.4	Ground Surface							
0.0	**20 cm Topsoil**				0			
0.2 / 277.1	Dark brown **EARTH FILL** sand, some silt to silty occ. topsoil inclusion occ. organics and rootlets				1			
1.3 / 276.3	Brown, compact **SILTY SAND** a trace of clay				2			
2.1 / 274.6	Brown, dense **SAND AND GRAVEL** a trace to some silt				3			
3.8 / 273.4	Brown, hard **SILTY CLAY TILL** a trace of gravel				4			water seepage elevation @ 273.65 masl
5.0	**END OF TEST PIT**				5			

DETAILED INFORMATION

All the measurements are from existing grade

WATER SEEPAGE
Water seepage occured @ 4.75 mbgs

Minor seepage rate

Cave-In
No cave-In occured during the time interval

Test Pit Monitoring
Water levels were measured at various time intervals after leaving the test pit open for 4.0 hours

Time	Water Level (from bottom of test pit)
12:30 pm	3 cm
01:30 pm	9 cm
02:15 pm	12 cm
03:30 pm	14 cm
04:30 pm	16 cm

Depth scale: 0 1 2 3 4 5 6 7 8 9 10

Soil Engineers Ltd.

APPENDIX "F"
SUPPORTING REFERENCE DOCUMENTS

Draft

Tributary Study

Village of Inglewood

Prepared for

Town of Caledon

By

Marshall Macklin Monaghan Limited

May 1999

Tribulary Study - Village of Inglewood

Table of Contents

1.0 INTRODUCTION

The Town of Caledon, the Region of Peel and Credit Valley Conservation jointly initiated the Inglewood Village Study in 1998. The purpose of the study is to develop a Community Plan, a Servicing Plan and an Environmental Management Plan, which will collectively guide the future evolution of Inglewood to the year 2021. These plans will not only address the form and structure of new development on the currently vacant lands within the settlement boundary, but will also identify proposed improvements to the existing developed portions of the Village.

The Study is being conducted in four phases. Phase I involved a comprehensive review and analysis of existing planning, servicing and environmental conditions within the study area boundary. This phase was completed on November 2, 1998 with Council's receipt of the Phase 1 Background Issues Report. Phase II consisted of the development and evaluation of future planning and servicing alternatives for the Village, and the selection of preferred planning and servicing alternatives. Phase II was completed in March of 1999.

The Study has now moved to Phase III, in which a draft Community Plan, a draft Servicing Plan and a draft Environmental Management Plan will be formulated. A necessary part of Phase III is the preparation of a Tributary Study, in order to provide direction for stormwater servicing and control.

1.1 Purpose and Context

Marshall Macklin Monaghan Limited has been contracted to prepare a Tributary Study for the drainage courses within Inglewood. The approach will be to prepare a study, addressing the preferred methods of providing stormwater servicing and control. Modern Tributary Studies, while focussing on stormwater drainage, are formulated based on information in many disciplines (eg. natural environment, hydrogeology, and geomorphology). With the exception of geomorphology, which will be addressed in finer detail in this study, the majority of background information (defining constraints and opportunities) will be drawn from the Phase I and II reports completed as part of the Inglewood Village Study.

The Tributary Study will concentrate on the stormwater and hydraulic components. It will concentrate on lands that are west of the Credit River and are either developed or are proposed for development.

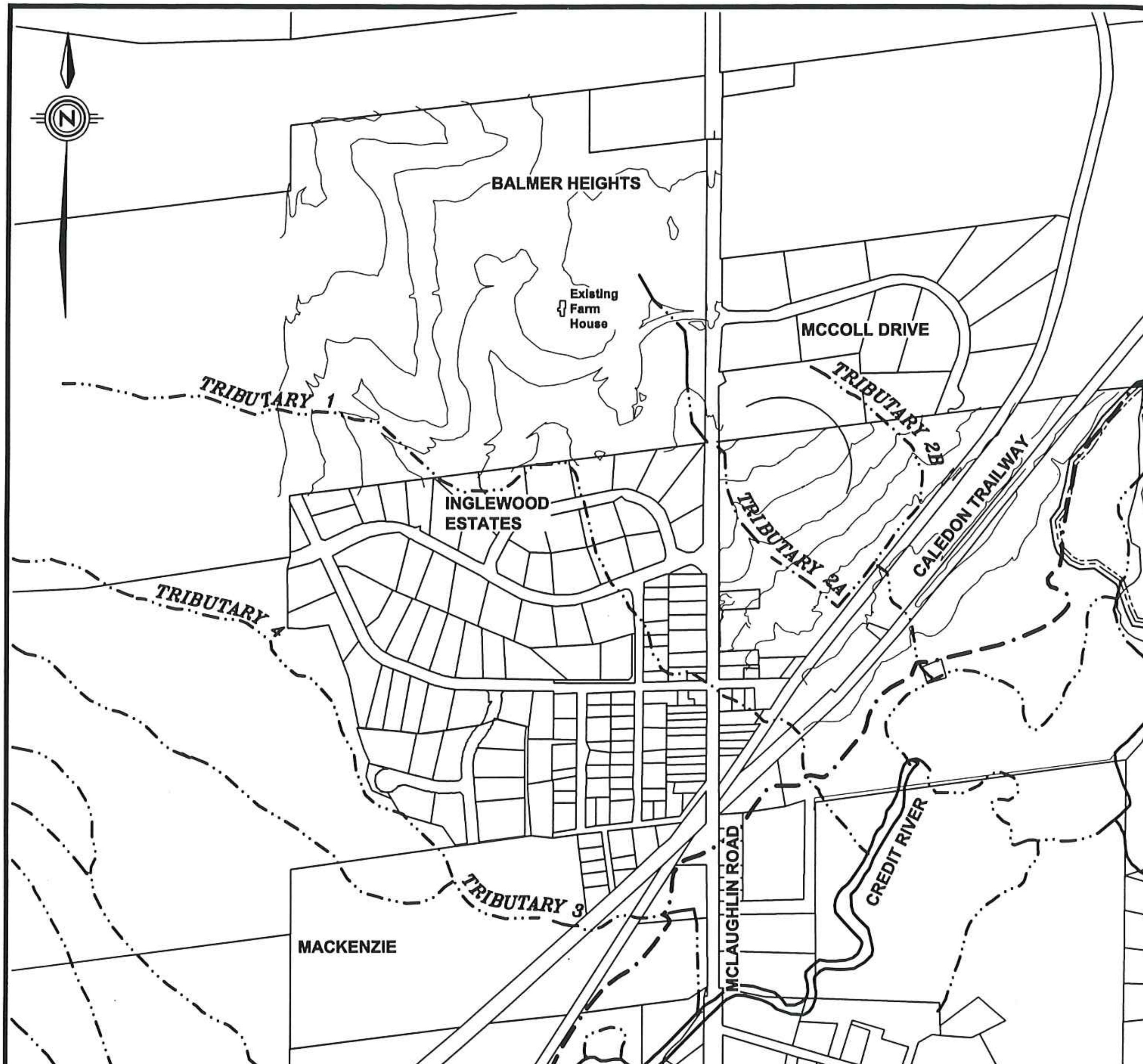
The study will provide detailed modelling of the study area with respect to hydrology and will develop a strategy that addresses water quality, erosion and flood control with respect to the post developed condition. Additional assessment will also include hydraulic analyses of culverts as well as a detailed geomorphic assessment of those tributaries within the study area. Enhanced phosphorus removal will be considered. The study will include a functional design of stormwater management facilities and an implementation strategy, which includes construction controls, maintenance and monitoring and a proposed schedule/cost sharing approach.

1.2 Background

Three independent tributary systems provide drainage through Inglewood, discharging ultimately to the Credit River. For the purposes of this report, the tributary systems will be referred to as 1 (middle tributary), 2 (north tributary – two branches, a and b), 3 (south tributary – south branch) and 4 (south tributary – north branch). The tributaries are illustrated in Figure 1.1.

Existing urban development in Inglewood has taken place primarily on Tributary 1 and Tributary 3. Both flooding and water quality problems are known to exist on Tributary 1.

The majority of the proposed developments are located on Tributaries 1 and 2. The Balmer Heights development would be located on both tributaries, while the South Slope development would be located on Tributary 2. Substantial background work has been completed by the proponents of each of these developments. This has been reviewed and considered as part of the development of the proposed Tributary Plan, as have the results of the first two phases of the Inglewood Village Study.



LEGEND

APPROXIMATE REGIONAL STORM FLOODLINE

0 0.1 0.2 0.3 0.4 0.5km

CLIENT TOWN OF CALEDON	Marshall Macklin Monaghan Consulting Engineers . Surveyors . Planners 80 Commerce Valley Dr. E, Thornhill, Ont. L3T 7N4 Tele (905)882-1100 Fax (905)882-0055
TITLE CREDIT RIVER SITE TRIBUTARIES	Checked D.J.A. / Drawn AutoCAD/B.K.B. Date APRIL 1999 / Proj. No. 1499012-01-WR1 Scale AS SHOWN / Figure No. 1.1 / Gr.No. 01

2.0 EXISTING CONDITIONS

The existing conditions within the Inglewood study area have been described in the Phase I and II reports of the Inglewood Village Study. The following sections highlight the conditions that have significance in terms of the Tributary Study.

2.1 Land Use

The land use in the northern section of the study area (Tributary 2) is dominated by intensive and non-intensive agricultural uses, with an area of large residential estate development (on about 1.0 hectare (2.5 acre) lots) along McColl Drive. The majority of the agricultural lands are proposed for development under either the Balmer Heights or the South Slope proposals.

The primary land use in the downstream sections of the central portion of the study area (Tributary 1) is urban, with residential development predominating. The historic portion of Inglewood, along McLaughlin Road, is generally developed on smaller sized lots, about 0.1 hectares (0.2 to 0.33 acres). Moving west, up the tributary, a more recent development, Inglewood Estates, exists on 0.4 hectare (1 acre) lots. Beyond Inglewood Estates the land use is intensive and non-intensive agricultural, with about half of the land proposed for development (Balmer Heights).

The southern section of the study area (Tributary 3 /4) is comprised of limited agricultural uses and wooded areas (associated with a more defined valley section).

2.2 Drainage

Drainage within the study area is based primarily on the system of overland flow routes provided by the tributary system. Tributary 2 (branches 2a and 2b) and the upper portions of Tributary 1 are intermittent. Branch 2a is a defined ditch east of McLaughlin Road. The lower portion of Tributary 1 and all of Tributary 3 /4 are more defined watercourses.

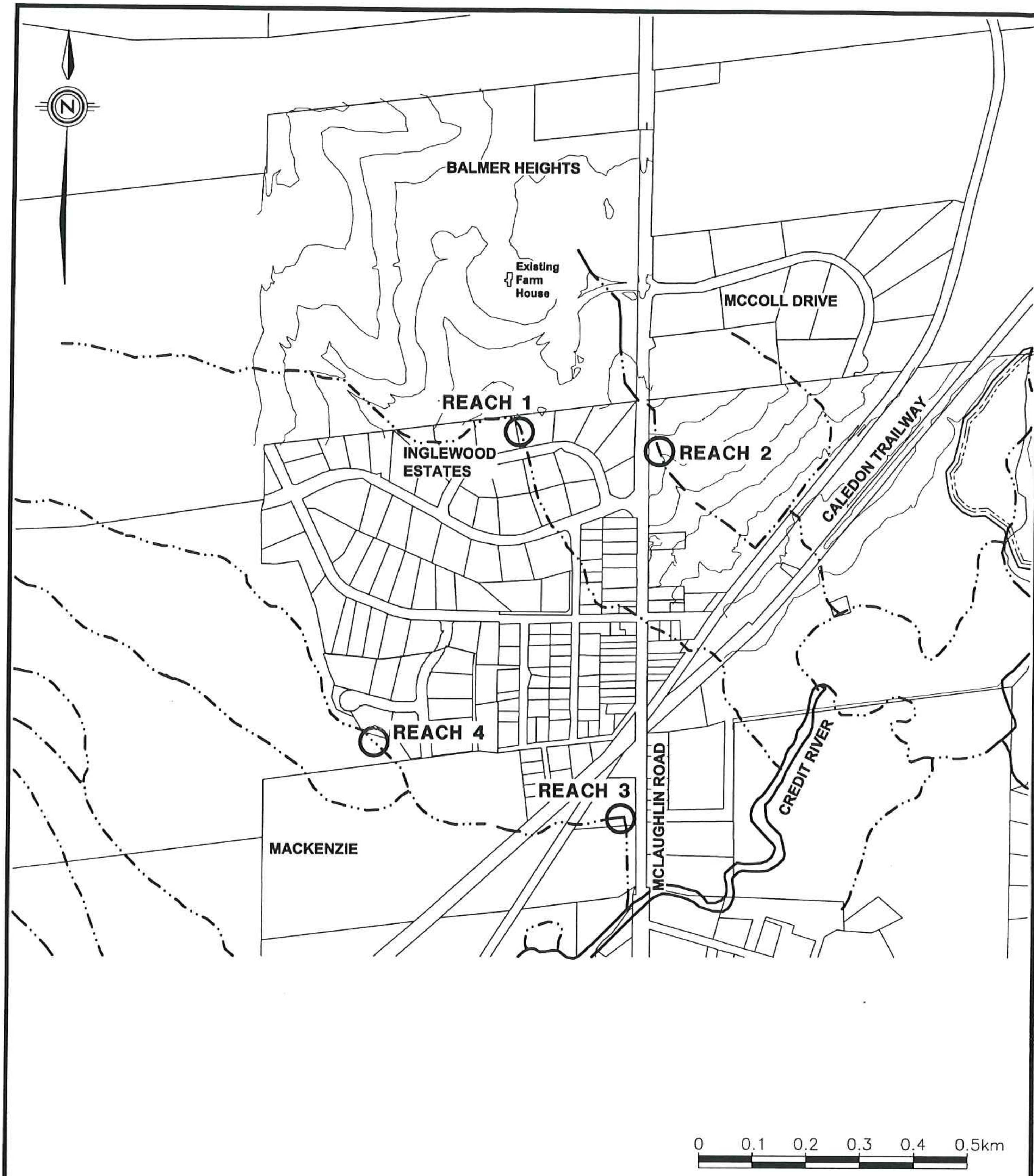
As in most urban areas that use overland flow routes, a significant aspect of the drainage network is the many culverts, which exist throughout the area. A number of the culverts are known to create flooding problems (Culverts 1-7, 1-8, 1-9, 1-10, 2-3, 3-3 and 3-4 – shown in Figure 4.2) on a relatively frequent basis, under existing conditions. The

In order to assess the potential impacts of proposed development on the individual tributaries within Inglewood, detailed fieldwork was undertaken as part of this study, in four reaches. All the tributaries are classified as low-order (1^{st} – 3^{rd}) streams. The locations the sites assessed are shown in Figure 2.1.

Reach 1 is characterized by a poorly defined channel. Although the channel is better defined in the downstream end, dense macrophytes (eg. cattails) were observed within the channel, increasing roughness and flow resistance; further downstream the channel is situated in an urban setting. Reach 2 is situated downstream of McLaughlin Rd. Reach 3, a highly sensitive channel, is situated immediately upstream of McLaughlin Rd.; much of this channel has been altered (ditched). Reach 4 joins with another tributary to form Tributary 3.

As part of the detailed field investigation, geomorphic data pertaining to channel cross-sectional shape, substrate material, and channel-bed morphology were gathered. The fieldwork was conducted on March 16 and 24, 1999 (necessitated by the study schedule) and so snow occupied some reaches and channel banks were frozen. Such conditions interfere with the qualitative and quantitative observations that can be made. Once the data were collected and compiled, various analyses were completed to quantify both critical discharge and critical velocity, and to assess the general sediment transport conditions of the channel. Because the reaches that were subject to the detailed geomorphic work are representative of the tributary in which they are situated, the results of any analyses reflect processes operative within the entire tributary.

Several general characteristics of each of the reaches are presented in Table 2.1, in addition to results of hydraulic analyses. From this table it is evident that the average gradient of each of the reaches is steep. The average substrate materials tend to be fine-grained (very fine sand) in reaches 1 – 3 but are coarser (gravels) in reach 4. Roughness characteristics of the channel bed reflect the physical characteristics of the substrate (i.e. lower in reaches 1-3 and higher in reach 4).



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TOWN OF CALEDON

TITLE

GEOMORPHIC ASSESSMENT SITES

Marshall Macklin Monaghan
Consulting Engineers . Surveyors . Planners
80 Commerce Valley Dr. E, Thornhill, Ont. L3T 7N4
Tele (905)882-1100 Fax (905)882-0055

Checked D.J.A.	Drawn AutoCAD/B.K.B.	
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Table 2.1. Summary of significant geomorphic variables and of critical threshold values for storm water management planning.

	Reach			
	1	**2**	**3 (ditch)**	**4**
Bankfull Discharge (cms)	N/A	0.27	1.38	1.52
Channel Gradient	~ 4.0 %	5.55 %	1.40 %	4.60 %
Bankfull Width (m)	1.00	1.19	2.70	1.95
Bankfull Depth (m)	0.05	0.11	0.25	0.29
Sediment (cm): D50**	Clay, silt, very fine	0.0036	0.0030	0.83
D84	sand	1.19	1.15	4.80
Manning's n	0.020	0.025	0.023	0.034
Critical Velocity (m/s) D50	0.038	0.02		
D84		1.28	0.93	1.39
Critical Depth (m) D50	Not *	0.02		0.02
D84	Available	0.05	0.08	0.10
Critical Shear Stress (N/m^2) D50	0.02	0.03	0.03	8.05
D84		11.54	11.16	46.56
Erosion Potential (N/ms)	N/A	66.91	46.47	226
Dominant Process	deposition	transport	planform adjustment	erosion
Critical Discharge (cms)	0.07	0.18	0.88	1.01

* Reach is poorly defined and very shallow
** D50 and D84 are terms relating to grain size distribution; For example, 84% of the particles are smaller than the D84 size

Through an examination of all the data that were collected in each reach, it is evident that Tributary 4 is dominated by erosion, that Tributary 1 is dominated by deposition in its upstream end (especially as a consequence of in-channel macrophytes), and that Tributary 2 is dominated by sediment transport. Currently, Tributary 3 is in the process of working towards an equilibrium channel as a consequence of changes to its natural planform caused by watercourse crossings and by channel straightening. The ditch that is situated downstream of the culvert is especially sensitive to changes in flow regime and therefore data from this segment of the tributary were used in the hydraulic analyses. In general, reaches 3 and 4 will be most sensitive to changes in flow regime.

Critical discharge and critical velocity were determined by drawing upon a range of hydraulic techniques so that the most representative value could be ascertained (Table 2.1). Variance in the results of the different techniques is a consequence of substrate characteristics, floodplain surficial geology, and surrounding land use. The critical discharge values deemed to reflect the conditions within each of the reaches were derived through a consideration of channel roughness (substrate material), the critical depth and velocity required to entrain substrate materials, and the average cross-sectional shape of the channel.

In Table 2.1 it is evident that although Tributary 4 flows into Tributary 3, the discharge of the former is larger than that of the latter. Such an observation is counter-intuitive. A closer examination of the field data and of the physical setting, however, reveals that it is warranted. The gradient and substrate materials in reach 4 are greater than in reach 3. Further, some water is diverted from Tributary 3 at its upstream end. Similarly, although the cross-sectional dimensions of reaches 2 and 4 are very similar, any quantitative measure or estimate of discharge at these sites differs substantially. Again, the observed trend may be accounted for by properties of the substrate and channel gradient.

2.7 Water Quality

Water quality data examined in Phases I and II of the Inglewood Village Study indicate that for most parameters, the water quality of the Credit River in the vicinity of Inglewood is quite good. The river however is classified as Policy 2 (Provincial Water Quality Objectives are currently exceeded and no further degradation will be permitted) on the basis of Total Phosphorus and bacteria. The bacteria objective is based upon body-contact recreation. The phosphorus objective is actually a guideline and should be assessed using other data sources where possible (eg. dissolved oxygen variation, fish and benthic communities). Diurnal oxygen studies, completed as part of the Inglewood Village Study, indicate no significant dissolved oxygen depression during the night. This, together with the presence of spawning redds in the area suggest that phosphorus is probably not having a major impact on the Credit River locally. Never the less, frequency of phosphorus exceedences should be taken as a warning and all practical efforts should be made to limit phosphorus inputs.

3.0 OPPORTUNITIES AND CONSTRAINTS

The existing conditions in the study area produce a number of issues that should be considered in completing the Tributary Study. These issues lead to a number of constraints and opportunities. The next section describes the constraints and opportunities (relevant to stormwater management and drainage) as documented in the Village of Inglewood Study – Phase II Report. Section 3.2 indicates the specific implications to the drainage strategy.

3.1 Village of Inglewood Study Phase II Findings

The Village of Inglewood Phase II report indicated constraints and opportunities under a number of categories:

Hydrogeology

"The majority of the proposed development within the study area is situated primarily in areas of moderate recharge potential. The concept plan should be designed so that infiltration conditions for proposed developments maintains or enhances pre development conditions."

Hydrology

"Any Planning Concept should be flexible enough to incorporate the following needs:

- Accommodate the need for stormwater management facilities;
- Restricts all form of development in the floodplain;
- Not allow erosion to be aggravated along all swales, tributaries and the Credit River;
- Allow for "at-source" controls to be implemented throughout."

Terrestrial

The Phase II report provides direction specific to each area of proposed development:

Balmer Heights

"With regard to constraints on the Balmer Heights development area, the tributary watercourse on this property has been classified as a moderately sensitive corridor. This watercourse corridor will require restoration in order to maintain water quality and quantity, and enhance its capacity to function as a corridor for wildlife if development proceeds in the area. This restoration would require the establishment of a 100 metre buffer along the length of the watercourse and an extension of the vegetation to link the corridor to the Niagara Escarpment Core Natural Area. In addition, the swales are considered to be contributing to fish habitat, and therefore consideration for the protection of their function will be encouraged. Such an approach to the protection and enhancement of the corridor will also provide an opportunity for a future open space link to a trail system."

South Slopes

"Due to the slope and proximity of this property to the Credit River valley, its important to note that there are wetlands and floodplain within approximately 160 metres of the proposed development. The wetlands in this portion of the Credit River valley have been classified as 'Alder Mineral Thicket Swamp' and a 'Forb Mineral Meadow Marsh', while the floodplain vegetation has been classified as 'Fresh-Moist Manitoba Maple Lowland Deciduous Forest', Fresh-Moist Willow Lowland Deciduous Forest', and 'Fresh Moist White Cedar Coniferous Forest'. Old oxbows on the floodplain provide clear evidence of the sensitive and dynamic nature of the Credit River throughout the Inglewood Village Study area. These old oxbows were noted to contain water and fish during the field investigations conducted by the CVC in late July 1998.

Limitations to the development of the South Slope area include the maintenance of water quality and quantity, and the protection of fish habitat associated with the existing drainage features and wetlands on the land adjacent to the proposed development. A portion of this objective can be accomplished through the establishment of naturally vegetated buffers around these swales and restoring a functional riparian system. The re-establishment of a naturally vegetated swale system will also provide opportunities for a open space linkage."

McKenzie (Tributary 3 /4)

"The presence of the two tributaries and the steep and potentially hazardous slopes (i.e. greater than 3:1) associated with these watercourses creates substantial limitations on the extent of developable land and provision of road access. The watercourse and valley land corridors on this property have been classified as highly sensitive. An integral part of the quality and functionality of these corridors on the property is attributed to the aforementioned coniferous plantation and cultural woodland, therefore every effort should be made to preserve their continuing function."

Fluvial Geomorphology

"Any planning concept should take the following considerations into account:

- Surface channels should be preserved, especially the function (storage of water and sediment) that they provide;
- Application of the meander belt width as a setback/buffer for the tributary;
- Minimize the number of crossings over the tributaries and avoiding any crossing of the mainstream of the Credit River.
- Avoid any realignment of the tributaries."

Fisheries

"The following points are concerns that should be accounted for:

- Alternatives vary in density and population such that the greater population would correspondingly have a greater potential for impact to fisheries related to decreases in infiltration potential and increases in the amount sewage effluent requiring disposal into the Credit River.

- The tributaries within the proposed areas for development likely do not contain fish habitat but do contribute to downstream fisheries. It is necessary to preserve or duplicate these functions including flow regimes, water temperature and water quality. Buffers along these tributaries and swales should be maintained."

3.2 Implications to the Drainage Strategy

Based on the direction provided above, there are three combinations of opportunities and constraints that will influence the alternatives considered in the Tributary Study.

3.2.1 Maintaining Infiltration

The entire urban settlement area is considered moderately sensitive in terms of its infiltration and recharge potential. The presence of sensitive habitat and a defined discharge area in the Credit River valley at Inglewood leads to a requirement that infiltration be maintained to the extent feasible. Further, the fairly sensitive geomorphological thresholds indicate the need to limit both the peak and the duration of runoff events to the greatest extent possible.

Unfortunately the relatively tight till soils and steep stream gradients make infiltration difficult. In order to maximize the potential for infiltration, runoff must be slowed down and dispersed as much as possible. Lot level controls will be mandatory and conveyance controls (eg. swales and open drainage ways) will be desirable. The requirement to maintain some of the tributarie's as corridors and to maintain the function of the existing swales, lends itself to the use of conveyance controls.

3.2.2 Existing Problems within the Tributaries

Based on the geomorphological data collected, it is expected that flow control for erosion protection will be required on each of the tributaries in order to support even the lowest concentration of development being considered. Further, it is expected that existing problems with localized flooding will be aggravated even if pre to post peak flow control is practiced (eg. the duration and frequency of flooding will be increased).

These conditions point to the need for quantity and erosion control in the upper reaches of the tributaries, within the proposed development areas.

3.2.3 Stormwater Quality Control

The presence of the coldwater fishery in the Credit River points to the need for level 1 protection for stormwater. However, the river is also a Policy 2 receiver for phosphorus and bacteria, and as such, no increase in the loading of these parameters will generally be permitted without a policy deviation. In the case of Inglewood, a high level of sewage treatment is proposed, but despite this, there will still be a small increase in phosphorus loading (although bacteria can be eliminated). As a result, the study team for the Inglewood Village Study has agreed to investigate and utilize every opportunity to achieve a reduction in the phosphorus loading to the Credit River. With respect to stormwater, this will entail reviewing new technologies and examining the potential to provide facilities that treat runoff from the Village Core.

In most cases stormwater is not considered in the "no increase in loading" requirement for a Policy 2 receiver (ie. the focus is on the sewage treatment effluent). However, the treatment of stormwater can be used to "offset" expected load increases from the sewage treatment system and hence avoid the need for a policy deviation. In the case of Inglewood, the potential for this "offset" is high, especially if water quality ponds can be sited at the lower end of tributaries so that the entire upstream area (including existing uncontrolled development areas) can be effectively controlled.

The existing development in Inglewood is currently serviced by private septic systems. While there is limited data to indicate that there is a problem, it would be unusual (in the soil types that exist in the area) if there were not relatively high phosphorus loads as a result of malfunctioning septic systems. This condition suggests that there would be a benefit to having an on-line water quality facility near the bottom of the tributaries. These facilities would provide the opportunity to reduce the phosphorus loading generated by older septic systems <u>and</u> by stormwater runoff.

4.0 PRE-DEVELOPMENT HYDROLOGY

An initial step in developing a Tributary plan for the proposed development of the Village of Inglewood, involves developing a hydrologic model to determine the pre-development flow rates through the drainage system under existing conditions. These flow rates allow an assessment of how well the existing drainage system is functioning and indicate a minimum set of flow targets which need to be achieved under post-development conditions.

4.1 Hydrologic Model

The three main tributaries of the Credit River through the settlement area were modelled. As discussed previously, Tributary 1 drains the majority of the proposed Balmer Heights development area, and flows through the historic Village through a combination of swales, culverts and sewers. The tributary empties into the Credit River just north of the Inglewood Arena. Tributary 2 drains the eastern portion of the proposed Balmer Heights development area, and discharges to the Credit River via a swale through the proposed South Slopes development area. Tributary 3 is located south of the Historic Village, discharging to the Credit River where it crosses McLaughlin Road. Upstream of the CNR trail the drainage course splits, and the north branch is referred to as Tributary 4.

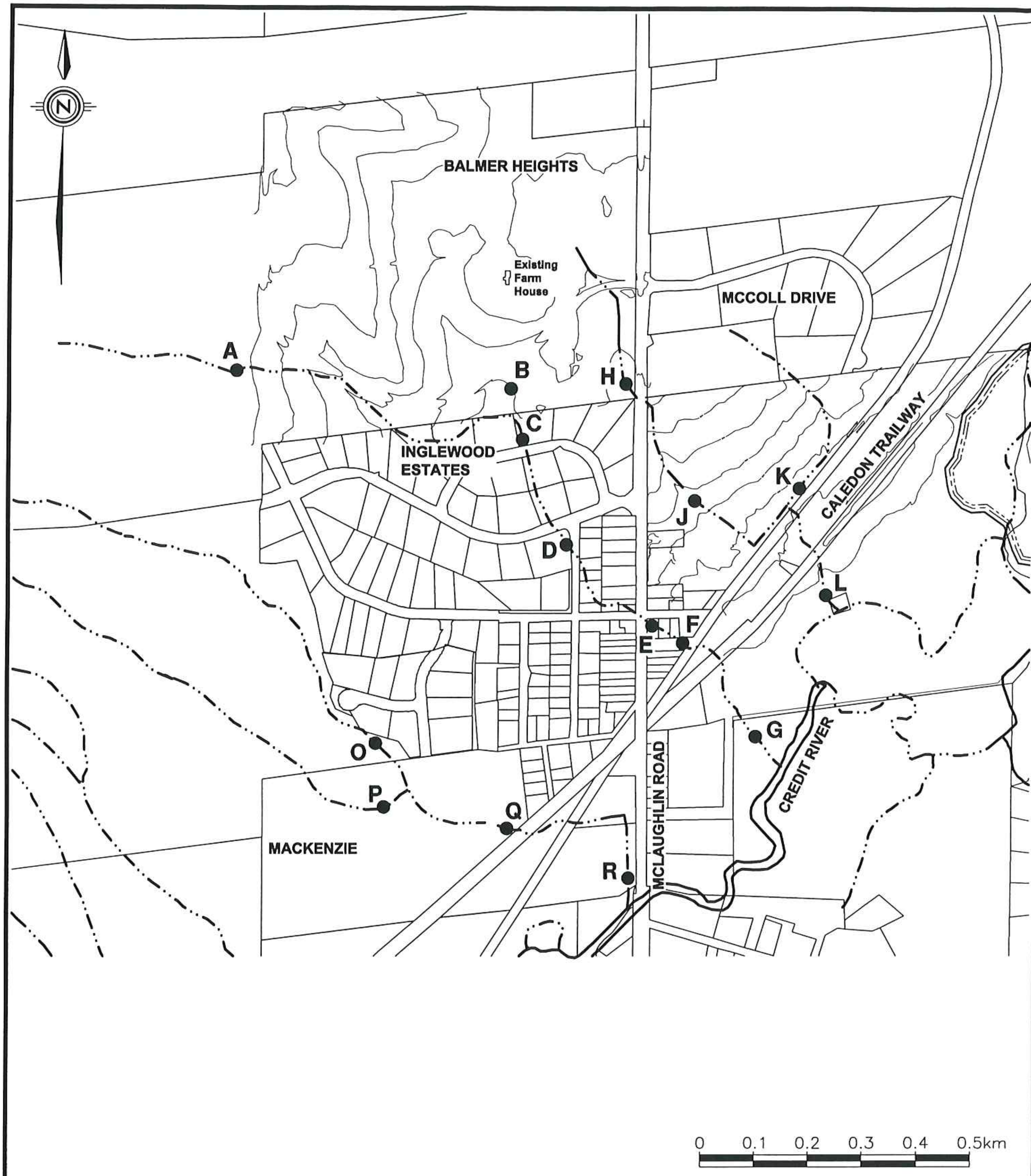
The OTTHYMO computer model was used in the study. This is a single event model that predicts runoff from a discreet rainfall event. It is the most widely accepted model for stormwater management studies in Ontario.

The various tributaries were discretized into a number of smaller sub-catchments and flow points were established at selected locations, as illustrated in Drawing 1 (Map Pocket) and Figure 4.1. Subcatchments have been grouped by Tributary (Tributary 2: Series 100; Tributary 1: Series 200; Tributary 3: Series 300). The parameters used to model each sub-catchment are presented in Table 4.1. The curve number (CN) was based on existing soil conditions and land use, and the initial abstractions (Ia) was calculated as 0.1*S, the soil storage volume. For the NASHYD command, the time to peak (Tp) was calculated as 1.5 times the original Williams equation. The values of XIMP and TIMP for the STANDHYD command were taken from available mapping and contract drawings for the existing developments.

TABLE 4.1 PRE-DEVELOPMENT OTTHYMO PARAMETERS

Sub Catchment	Drainage Area (ha)	CN	Ia (mm)	Tp (hr)	*XIMP*	TIMP
101	9.51	80	6.5	.209	-	-
102	1.54	82	6.0	.092	-	-
103	3.11	81	6.5	.090	-	-
104	0.61	80	6.5	.048	-	-
105	6.93	83	5.5	.146	-	-
106	6.90	85	5.5	.117	-	-
107	4.64	85	5.5	.104	-	-
108	4.54	80	6.5	.134	-	-
109	0.48	85	5.0	.042	-	-
201	16.72	78	7.0	.158	-	-
202	4.87	80	6.5	.120	-	-
203	16.16	78	7.0	.168	-	-
204	17.96	80	6.5	.198	-	-
205	2.00	80	6.5	.086	-	-
206	7.57	84	5.5	.155	-	-
207	3.14	-	-	-	.20	.25
208	1.18	82	6.0	.050	-	-
209	2.42	86	4.5	.068	-	-
210	2.50	-	-	-	.18	.24
211	0.90	-	-	-	.35	.40
212	2.91	84	5.0	.086	-	-
213	1.21	-	-	-	.30	.40
214	0.79	82	6.0	.071	-	-
215	2.39	78	7.0	.110	-	-
301	2.96	80	6.5	.080	-	-
302	6.31	-	-	-	.15	.20
303	2.08	-	-	-	.30	.35
304	4.44	-	-	-	.25	.30
305	26.45	78	7.0	.210	-	-
306	2.04	-	-	-	.15	.20
307	3.71	82	6.0	.089	-	-
308	13.87	78	7.0	.180	-	-
309	1.87	80	6.5	.074	-	-
310	8.15	85	4.5	.123	-	-
311	1.23	85	5.0	.081	-	-
312	2.80	-	-	-	0.20	0.25

Two different design storm distributions were selected for the study. For the purposes of stormwater quantity, 24 SCS storm distributions were used. The 24 hour SCS distribution is typically used for stormwater management studies for developments in the Credit River watershed. Although they generally produce lower peak flow rates than 4 hour Chicago



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TITLE

FLOW POINTS

Marshall Macklin Monaghan
Consulting Engineers . Surveyors . Planners
80 Commerce Valley Dr. E. Thornhill, Ont. L3T 7N4
Tele (905)882-1100 Fax (905)882-0055

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storms for the same return period, 24 hour SCS storms typically result in larger volumetric storage requirements and are considered conservative from a quantity control perspective. The second distribution used was the 4 hour Chicago storm distribution. The Chicago distribution is more typical of a summer thunderstorm and generally produces higher peak flow rates. The Chicago storms were selected for evaluating the hydraulic capacities of existing culverts, sewers and channels and for the assessment of potential erosion impacts.

The Atmospheric Environment Service (AES) rainfall data based on the Toronto Pearson International Airport station was used in the construction of both the 24 hour SCS and 4 hour Chicago storms. For the regional storm model, only the last 12 hours of the Hurricane Hazel storm event were simulated, with the CN values in the model adjusted to represent saturated conditions (AMC III).

4.1.1 Results of Modelling

The peak flow rates at different points along the tributaries, as defined in Figure 4.1, are presented in Table 4.2 (24 hour SCS storms) and Table 4.3 (4 hour Chicago storms).

TABLE 4.2 PRE-DEVELOPMENT FLOW RATES – 24 HOUR SCS STORMS

Flow Point	Peak Flow Rates (m^3/s)						
	2 year	5 year	10 year	25 year	50 year	100 year	Regional *
A	0.52	0.98	1.31	1.75	2.08	2.42	3.30
B	0.40	0.74	0.98	1.30	1.55	1.79	2.38
C	1.25	2.30	3.06	4.06	4.82	5.59	7.45
D	1.34	2.46	3.27	4.34	5.14	5.96	7.91
E	1.43	2.64	3.50	4.64	5.50	6.37	8.43
F	1.50	2.76	3.66	4.83	5.73	6.62	8.73
G	1.54	2.82	3.73	4.94	5.85	6.77	8.93
H	0.15	0.29	0.53	0.77	0.91	1.06	1.46
J	0.31	0.56	0.75	0.99	1.17	1.36	1.78
K	0.59	1.04	1.37	1.79	2.12	2.44	3.10
L	0.54	0.99	1.41	2.00	2.45	2.80	3.55
M	0.14	0.22	0.54	0.60	0.84	0.86	1.04
N	0.30	0.53	0.96	1.09	1.41	1.53	1.95
O	0.60	1.13	1.51	2.03	2.42	2.81	3.89
P	0.27	0.50	0.66	0.89	1.06	1.23	1.70
Q	1.03	1.92	2.56	3.40	4.05	4.70	6.37
R	1.43	2.61	3.73	4.78	5.57	6.53	8.77

* Last 12 hours of Hurricane Hazel

TABLE 4.3 PRE-DEVELOPMENT FLOW RATES – 4 HOUR CHICAGO STORMS

Flow Point	Peak Flow Rates (m^3/s)						
	25 mm	2 year	5 year	10 year	25 year	50 year	100 year
A	0.25	0.61	1.23	1.70	2.38	2.91	3.47
B	0.19	0.44	0.86	1.18	1.63	1.98	2.36
C	0.63	1.46	2.89	3.95	5.46	6.65	7.90
D	0.72	1.64	3.18	4.32	5.95	7.23	8.57
E	0.83	1.80	3.45	4.65	6.38	7.73	9.30
F	0.92	1.94	3.68	4.94	6.75	8.16	9.80
G	0.94	1.98	3.76	5.06	6.91	8.36.	10.03
H	0.06	0.12	0.23	0.35	0.60	0.88	1.18
J	0.07	0.14	0.28	0.41	0.70	0.92	1.14
K	0.25	0.53	0.98	1.30	1.75	2.10	2.46
L	0.30	0.63	1.18	1.57	2.13	2.55	3.00
M	0.07	0.15	0.21	0.48	0.99	1.15	1.22
N	0.27	0.43	0.68	0.83	1.43	1.70	1.95
O	0.28	0.62	1.23	1.68	2.35	2.87	3.55
P	0.12	0.29	0.58	0.80	1.12	1.37	1.64
Q	0.53	1.19	2.30	3.12	4.31	5.24	6.35
R	0.90	1.76	3.28	4.34	5.85	7.18	9.00

The flow rates presented in Tables 4.2 and 4.3 generally appear to be relatively large, given the critical discharge rates estimated in the geomorphological assessment (Section 2.5). In particular, at flow point B, the critical discharge rate is 0.07 m3/s, which is considerably less than the 2 year peak flow rate of 0.44 m3/s, and less than the peak flow of 0.19 m3/s resulting from the 25 mm rainfall event. At points J and O, the critical discharge rate falls between the 2 year and 5 year peak flow rates. The critical discharge rate at flow point R, in the ditch along McLaughlin Road, is also relatively small, corresponding to the peak flow from a 25 mm rainfall event.

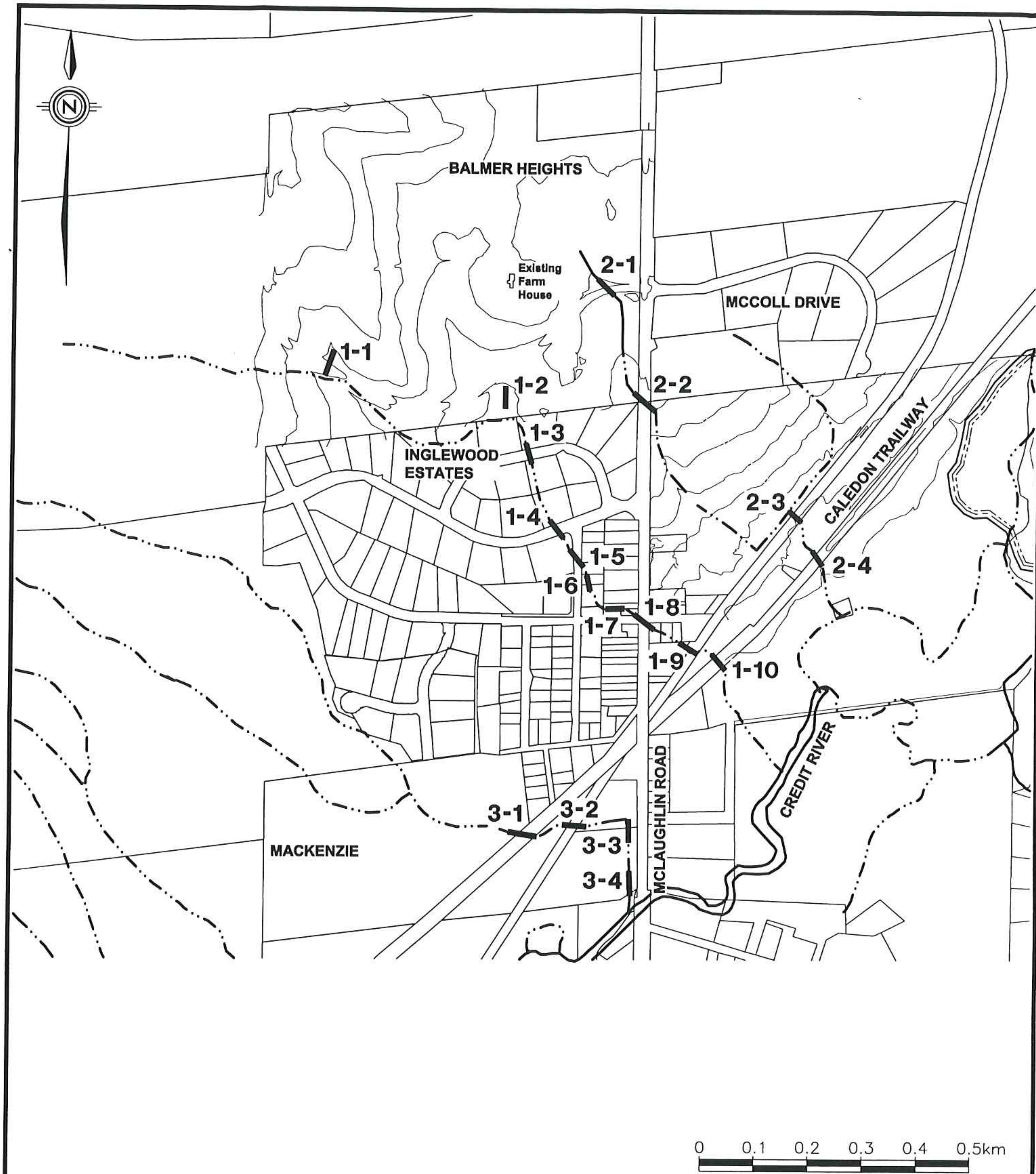
Based on the above, channel stability is a concern even under existing conditions. In the Credit River watershed (unless a Subwatershed Plan is available that provides more specific criteria) post-development peak flow rates are typically controlled to pre-development levels and extended release the runoff from a 25 mm rainfall event over 24 hours is used to define erosion control storage. However, for the proposed developments in the Village of Inglewood, adopting the above criteria will not alleviate erosion potential in the subject tributaries. It may be necessary to adopt more stringent controls, such as increasing the erosion control storage volumes and/or releasing it over a longer duration, in order to prevent further degradation of the drainage channels.

4.2 Hydraulic Assessment

The existing drainage system in Inglewood is based on the overland flow routes provided by the tributaries. As a result, there are numerous culverts on the subject tributaries throughout the study area. As part of the development of a drainage strategy, an analysis of these culverts was conducted to determine their capacities. This purpose was to determine the location of critical flow restrictions in the systems, and to recommend appropriate methods of improving performance.

The HEC-2 model was selected for the analysis. Although the capacities of individual culverts can be determined relatively easily with design charts, several culverts in the study area are sufficiently close in distance and elevation that the headwater elevation at the downstream culvert may impact the flow in the upstream culvert. These culverts were examined in series, which was best accomplished with the HEC-2 model. Furthermore, several culverts in the study area have irregular sections and cannot be evaluated with available design sheets. Thus, to maintain consistency, the HEC-2 model was used to determine the capacities of all culverts and sewers along the subject tributaries of the Credit River.

Two different hydraulic capacities were calculated for each drainage structure. The first is the flow rate that can be accommodated before the headwater elevation reaches the culvert obvert. The second is the flow rate that causes the roadway or rail bed serviced by the culvert to be overtopped. The calculated flow rates for each culvert are presented in Tables 4.4 through 4.6, along with the approximate return period corresponding to each flow. The return periods are based on the peak flow rates resulting from 4 hour Chicago design storms under existing conditions. The culverts listed in Tables 4.4 through 4.6 are labelled on Figure 4.2.



CLIENT TOWN OF CALEDON	**Marshall Macklin Monaghan** Consulting Engineers . Surveyors . Planners 80 Commerce Valley Dr. E, Thornhill, Ont. L3T 7N4 Tele (905)882-1100 Fax (905)882-0055
TITLE CULVERT LOCATIONS	Checked: D.J.A. Drawn: AutoCAD/B.K.B. Date: APRIL 1999 Proj. No.: 1499012-01-WR1 Scale: AS SHOWN Figure No. 4.2 Gr.No. 01

TABLE 4.4 CULVERT CAPACITIES IN TRIBUTARY 1

Culvert	Size	Full Flow		Road Overtop	
		Capacity (m^3/s)	Return Period (yr)	Capacity (m^3/s)	Return Period (yr)
1-1	1050 mm CSP	1.40	< 25	3.50	> 100
1-2	900 mm CSP	0.95	< 10	2.25	> 100
1-3	1200 mm CSP	1.95	< 5	4.40	< 25
1-4	1200 mm CSP	1.95	< 5	5.00	< 25
1-5	1200 mm CSP	2.05	< 5	2.00	< 25
1-6	1200 mm Conc.	2.00	< 5	3.80	< 25
1-7	900 mm CSP	1.00	< 2	1.05	< 2
1-8	1050 mm CSP	1.45	< 2	1.75	< 2
1-9	1200 mm X 1200 mm 'flat base' Conc. & 600 mm CSP	1.45	< 2	3.00	< 5
1-10	Twin 600 mm CSPs	0.70	< 2	2.25	< 5

TABLE 4.5 CULVERT CAPACITIES IN TRIBUTARY 2

Culvert	Size	Full Flow		Road Overtop	
		Capacity (m^3/s)	Return Period (yr)	Capacity (m^3/s)	Return Period (yr)
2-1	750 mm CSP	0.65	< 25	1.30	> 100
2-2	600 mm Conc.	0.40	< 25	0.80	< 50
2-3	680 mm X 750 mm 'flat base' Conc.	0.35	< 2	0.85	< 5
2-4	1050 mm CSP	1.50	< 10	3.70	> 100

TABLE 4.6 CULVERT CAPACITIES IN TRIBUTARY 3

Culvert	Size	Full Flow		Road Overtop	
		Capacity (m^3/s)	Return Period (yr)	Capacity (m^3/s)	Return Period (yr)
3-1	2000 mm Conc.	7.25	> 100	17.50	> 100
3-2	2.3 m X 2.1 m open footing Conc.	11.50	> 100	18.35	> 100
3-3	900 mm CSP	0.95	< 2	1.85**	< 2
3-4	900 mm CSP	0.95	< 2	1.70	< 2

** There is a 600 mm relief CSP which has an additional capacity of ~0.45 m^3/s

In Tributary 1, the culverts under the access road in the proposed Balmer Heights development area are all adequately sized, and the culverts through the existing Inglewood

Estates development have satisfactory capacity. However, the 900 mm CSP along MacDonald Street and the 1050 mm CSP under McLaughlin Road (which it ties into) appear to be significantly undersized, with runoff overtopping MacDonald Street on average more than once a year. This is consistent with the reported flooding problems along MacDonald Street. The remaining culverts in Tributary 1 also appear to be undersized. However, the above mentioned culverts at MacDonald Street and McLaughlin Road remain the most significant restrictions in the system.

Along Tributary 2, there is adequate hydraulic capacity in the system from the top of the subwatershed to the CPR tracks. The HEC-2 analysis indicated that the CPR rail bed would be overtopped on average more than once every 5 years. It is anticipated that significant ponding would occur on the upstream side of the CPR tracks. Although this is not a problem under existing conditions, it must be taken into account in the stormwater management plan for the proposed South Slopes development.

In Tributary 3, the culverts under the CNR trail and CPR tracks provide sufficient capacity for the 100 year event. However, the two 900 mm CSPs under the private entrances along McLaughlin Road are unable to accommodate the 2 year peak flow rates. Although these entranceways are overtopped relatively frequently, it is expected that McLaughlin Road acts as a dike, and is overtopped only during extreme events. Runoff overtopping the entranceways would flow in the channel and overland west of the channel into the Credit River, immediately to the south. Furthermore, these culverts are located within the existing 100 year floodline, and are likely to be inundated during rare events.

4.2.1 Discussion and Recommendations

Tributary 1

A significant drainage problem exists on Tributary 1 as a result of the capacities of the 900 mm and 1050 mm CSPs along MacDonald Street and under McLaughlin Road (Culverts 1-7 and 1-8 on Figure 4.2). This problem has existed for some time. EMC Group prepared a design in 1995 for the reconstruction of Lorne Street from West Village Drive to MacDonald Street and MacDonald Street from Lorne Street to the CPR tracks, including storm sewer improvements. The proposed system would have intercepted Tributary 1 at Lorne Street and piped it south under Lorne Street and east under MacDonald Street to a stormwater pond which was proposed between the CPR tracks and the CNR trail. The system has not been constructed, but a 600 mm sewer along

Macdonald Street, between Victoria and Lorne Street has been. This latter sewer is currently not operational (it is plugged at Lorne Street) but would intercept flow from subcatchments 301, 302 and 303 if an outlet were to be provided by the proposed sewer. At present, the flow from these catchments enters a 300 mm csp and is routed through backyard swales to the corner of McKenzie and McLaughlin Road.

A possible alternative to the proposed sewer construction would involve over-controlling flows from the proposed Balmer Heights subdivision (as well as the upstream external lands which could be controlled by facilities at Balmer Heights). Examination of the flows draining to Culvert 1-7 indicates that about 63 (of 75) hectares could be subject to over-control. If the flows from this area were to be entirely eliminated, the existing culvert capacities could be raised to about a 10 year return period storm. All flows of course cannot be eliminated and hence the best that could be done through over-control would be about a 5 year level of service. Over-control would be relatively inefficient since about half of the potentially controllable land is not proposed for development. It is clear that over-control cannot accomplish an adequate solution on its own. Therefore, a storm sewer under Lorne and MacDonald Streets, similar to that designed by EMC Group is recommended. It is noteworthy that the provision of some detention storage at Balmer Heights (eg. if required to prevent erosion and local flooding) would reduce peak flow rates in the system and therefore potentially reduce the size and cost of a storm sewer system to convey runoff through the historic Village.

Tributary 2

The hydraulic structures along Tributary 2 are generally satisfactory. However, due to the restriction at the CPR tracks, improvements are required should the South Slopes area be developed. A stormwater management pond could be constructed west of the CPR tracks to control post-development peak flows to the capacity of the existing 680 mm x 750 mm flat base pipe, such that runoff does not back up into the development area during extreme events. If it is desired to place the stormwater management pond east of the CPR tracks, the existing culvert must be replaced with one which can adequately convey the uncontrolled post development peak flow from the development area and external drainage areas.

Tributary 3

The major culverts upstream of the railway and the Caledon Trailway on Tributary 3 have adequate capacity and are not considered to be a problem. Adjacent to McLaughlin Road, the two culverts passing under the private entrances are inadequately sized. This area is within the floodplain of the Credit River and is likely to experience some flooding, regardless of the flow from Tributary 3. As a result no improvements to the hydraulic structures along Tributary 3 are recommended.

5.0 POST-DEVELOPMENT HYDROLOGY

As part of Phase II of the Village of Inglewood study, a Structural Concept plan, indicating proposed future land uses, was prepared. The concept plan is reproduced in Figure 5.1.

There are three different areas in the Village of Inglewood for which residential development is proposed. The proposed Balmer Heights development area is approximately 41 ha, and is located west of McLaughlin Road, north of the existing Inglewood Estates residential development. The proposed South Slopes development area, east of McLaughlin Road and south of McColl Drive, is approximately 12 ha. There have been development plans prepared for both of the above proposed developments. Finally, it has been proposed that McKenzie Street be extended to access a proposed development area of approximately 7 ha. The area is referred to as McKenzie, and is bisected by Tributary 3. In addition to the above residential development, the Inglewood Village Plan includes the re-development of McLaughlin Road north of the CNR crossing as mixed use.

Open space corridors are indicated on both the Balmer Heights and the South slope lands. The corridors generally follow natural drainage ways. The corridors are consistent with the need to disperse runoff to the greatest possible extent, in order to foster continued infiltration in the post development condition.

5.1 Proposed Land Use

Phase II of the Inglewood Village Study examined three planning alternatives with different approaches to lot sizes and densities. Each of the alternatives would result in different potential population ranges. The alternatives included:

Alternative 1 Historic Village Densities

This alternative assumed that the remaining development lands within the Village would be developed with lot sizes similar to the historic core area (e.g. generally ranging from 0.2 to 0.33 acres – about 0.1 ha). This alternative leads to a potential population range of 1,700 to 2,300 persons by the year 2021.

Alternative 2 Village Estate Densities

This alternative assumed that the remaining development lands within the Village would be developed with larger "estate-type" lots (e.g. generally ranging from 0.5 to 1.0 acres in size – about 0.2 – 0.4 ha). This leads to a potential population range of 1,100 to 1,400 persons by the year 2021.

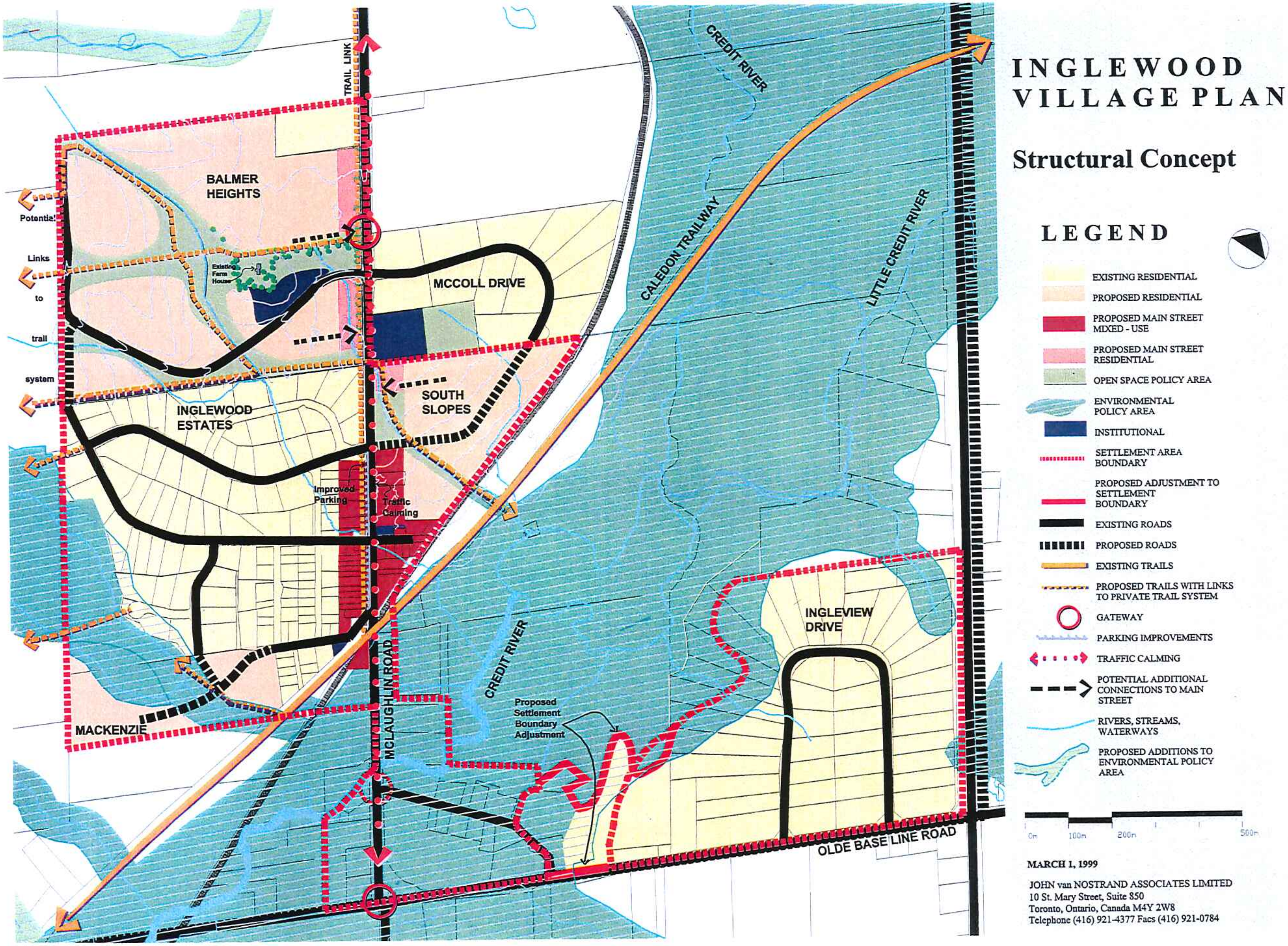
Alternative 3 Mixed Village Densities

This alternative was based on a mix of lot sizes (e.g. generally ranging from 0.2 to 1.0 acres 0.08 – 0.4 ha). This alternative leads to a potential population range of approximately 1,200 to 1,700 persons by the year 2021.

The Phase II report identified Alternative 3 as the preferred planning scenario based on existing development, environmental and community impacts, community input and Provincial and Local policies and guidelines. A mix of lot sizes was proposed. Using an average lot size of 0.20 ha to 0.35 ha., an imperviousness ranging from 22 – 28% has been estimated.

5.2 Post Development Model – Uncontrolled Conditions

Two sets of models were constructed for post-development conditions: one representing the lower limit of the development densities (22% imperviousness) described in Section 5.1, and the other representing the upper development density limit (28% imperviousness). The parameters used in the post-development OTTHYMO models are presented in Table 5.1, based on the sub-catchments illustrated in Drawing 2 (Map Pocket). The model parameters for the catchments outside of the development areas remain unchanged from the pre-development model, although the drainage boundaries were altered for a number of sub-catchments bordering the development areas. Sub-catchments within the development area were further discretized for the analysis. As in the pre-development modelling subcatchments have been grouped by Tributary (Tributary 2: Series 1000; Tributary 1: Series 2000; Tributary 3: Series 3000).


INGLEWOOD VILLAGE PLAN
Structural Concept
LEGEND
EXISTING RESIDENTIAL
PROPOSED RESIDENTIAL
PROPOSED MAIN STREET MIXED - USE
PROPOSED MAIN STREET RESIDENTIAL
OPEN SPACE POLICY AREA
ENVIRONMENTAL POLICY AREA
INSTITUTIONAL
SETTLEMENT AREA BOUNDARY
PROPOSED ADJUSTMENT TO SETTLEMENT BOUNDARY
EXISTING ROADS
PROPOSED ROADS
EXISTING TRAILS
PROPOSED TRAILS WITH LINKS TO PRIVATE TRAIL SYSTEM
GATEWAY
PARKING IMPROVEMENTS
TRAFFIC CALMING
POTENTIAL ADDITIONAL CONNECTIONS TO MAIN STREET
RIVERS, STREAMS, WATERWAYS
PROPOSED ADDITIONS TO ENVIRONMENTAL POLICY AREA
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100m
200m
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MACKENZIE
CREDIT RIVER
CALEDON TRAILWAY
LITTLE CREDIT RIVER
INGLEVIEW DRIVE
Proposed Settlement Boundary Adjustment
OLDE BASE LINE ROAD

TABLE 5.1 POST DEVELOPMENT OTTHYMO PARAMETERS

Sub Catchment	Drainage Area (ha)	CN	Ia (mm)	Tp (hr)	Lower Density Limit		Upper Density Limit	
					XIMP	TIMP	XIMP	TIMP
1010	8.86	-	-	-	.16	.22	.19	.28
1020	1.96	-	-	-	.20	.25	.25	.30
1030	3.11	-	-	-	.16	.22	.19	.28
1040	0.61	80	6.5	.048	-	-	-	-
1050	6.82	83	5.5	.146	-	-	-	-
1060	2.67	-	-	-	.25	.30	.25	.30
1070	9.73	-	-	-	.16	.22	.19	.28
1080	4.54	80	6.5	.134	-	-	-	-
1090	0.48	85	5.0	.042	-	-	-	-
2010	16.95	78	7.0	.158	-	-	-	-
2020	6.01	-	-	-	.16	.22	.19	.28
2030	12.83	78	7.0	.152	-	-	-	-
2031	3.47	-	-	-	.16	.22	.19	.28
2040	10.95	-	-	-	.16	.22	.19	.28
2041	7.25	-	-	-	.16	.22	.19	.28
2050	2.17	-	-	-	.16	.22	.19	.28
2060	2.93	84	5.5	0.155	-	-	-	-
2061	4.24	-	-	-	.15	.22	.15	.22
2070	3.26	-	-	-	.20	.25	.20	.25
2080	1.14	82	6.0	.050	-	-	-	-
2090	2.37	86	4.5	.068	-	-	-	-
2100	2.45	-	-	-	.18	.24	.18	.24
2110	0.80	-	-	-	.35	.40	.35	.40
2120	2.58	-	-	-	.16	.22	.19	.28
2130	1.34	-	-	-	.30	.40	.30	.40
2140	0.76	82	6.0	.071	-	-	-	-
2150	2.40	78	7.0	.110	-	-	-	-
3010	3.04	80	6.5	.080	-	-	-	-
3020	6.46	-	-	-	.15	.20	.15	.20
3030	2.02	-	-	-	.30	.35	.30	.35
3040	4.76	-	-	-	.25	.30	.25	.30
3050	26.45	78	7.0	.210	-	-	-	-
3060	2.27	-	-	-	.15	.20	.15	.20
3070	3.77	82	6.0	.089	-	-	-	-
3080	13.69	78	7.0	.180	-	-	-	-
3081	3.25	-	-	-	.16	.22	.19	.28
3090	1.04	80	6.5	.065	-	-	-	-
3100	4.68	85	4.5	.089	-	-	-	-
3101	3.07	-	-	-	.16	.22	.19	.28
3102	1.30	-	-	-	.18	.24	.18	.24
3110	1.21	85	5.0	.081	-	-	-	-
3120	2.96	-	-	-	.20	.25	0.20	0.25

The post-development uncontrolled models were run with the 25 mm, 4 hour rainfall event as well as the 2 through 100 year 24 hour SCS design storms. Table 5.2 presents the model output for selected storm events, comparing the uncontrolled peak flow rates for both the low and high density development scenarios to the corresponding pre-development peak flow rates. The modelling output can be found in Appendix A.

TABLE 5.2 POST-DEVELOPMENT FLOW RATES - UNCONTROLLED

Flow Point	Peak Flow Rates (m^3/s)											
	25 mm			5 year			25 year			100 year		
	Existing	Light	Dense	Existing	Light	Dense	Existing	Light	Dense	Existing	Light	Dense
A	0.25	0.25	0.25	0.98	0.98	0.98	1.75	1.76	1.76	2.42	2.44	2.44
B	0.19	0.45	0.53	0.74	0.89	0.94	1.30	1.62	1.67	1.79	2.22	2.28
C	0.63	1.04	1.16	2.30	2.60	2.68	4.06	4.68	4.76	5.59	6.47	6.55
D	0.72	0.90	1.00	2.46	2.78	2.85	4.34	5.02	5.11	5.96	6.99	7.09
E	0.83	1.00	1.10	2.64	2.94	3.02	4.64	5.31	5.40	6.37	7.38	7.48
F	0.92	1.12	1.23	2.76	3.15	3.23	4.83	5.66	5.75	6.62	7.85	7.95
G	0.94	1.13	1.25	2.82	3.21	3.29	4.94	5.76	5.86	6.77	7.99	8.10
H	0.06	0.32	0.39	0.29	0.64	0.67	0.77	1.13	1.17	1.06	1.59	1.63
J	0.07	0.42	0.51	0.37	1.12	1.17	0.94	1.95	2.01	1.38	2.72	2.78
K	0.25	0.61	0.69	0.84	1.50	1.55	1.75	2.60	2.66	2.46	3.60	3.66
L	0.30	0.65	0.74	0.99	1.64	1.69	2.00	2.85	2.91	2.80	3.94	4.00
M	0.07	0.08	0.08	0.22	0.24	0.24	0.60	0.64	0.64	0.86	0.86	0.86
N	0.27	0.28	0.28	0.53	0.54	0.54	1.09	1.14	1.14	1.53	1.55	1.55
O	0.28	0.28	0.28	1.13	1.14	1.14	2.03	2.04	2.04	2.81	2.84	2.84
P	0.12	0.19	0.21	0.50	0.65	0.66	0.89	1.16	1.16	1.23	1.59	1.60
Q	0.53	0.62	0.64	1.92	2.08	2.10	3.40	3.71	3.73	4.70	5.13	5.14
R	0.90	0.99	1.02	2.61	2.80	2.81	4.78	5.12	5.14	6.53	7.04	7.05

As may be expected, Table 5.2 indicates that the proposed developments, if uncontrolled, will increase peak flow rates in all downstream channels. Increases are not as great as may be expected because of the soil conditions and the relatively low imperviousness of the proposed developments. Under major flood-type storms Tributary 1 would experience increases in the 15 to 20 % range, while the lower development potential on Tributary 3 would produce only about a 10% increase in flow. Tributary 2 would experience the largest flow increases, in the order of 50 to 100% (on both sub-branches).

Despite the relatively small increases in flows on Tributary 1 it is anticipated that flow control for flood protection will be required because of the already limited capacity of downstream culverts. Further, as indicated in Section 4.1.1, flow rates on Tributary 1 are at or above critical discharge rates for erosion during the 25 mm and 2 year storm events, even under existing conditions. As a result, fairly restrictive erosion controls will be required with even the relatively small increase in flows predicted, unless the tributary is improved substantially from a geomorphological perspective.

Similar erosion concerns exist on both Tributary 2 and Tributary 3. On Tributary 2 the substantial flow increases under the smaller storms indicates the potential for problems (due to an increase in frequency) even though peak flows for these events are less than the critical flow threshold. On Tributary 3, the concern is simply that the lower reaches of the tributary have been noted as being highly susceptible to erosion.

Based on the above, it is anticipated that flow control will be required in the upstream areas of Tributaries 1 and 2 unless the downstream channels are improved. It is noteworthy that this conclusion is valid for both levels of development density considered. At most flow points, especially those further downstream in the tributaries, the difference in peak flow rates between the light and dense development scenarios is not significant. Although they represent the lower and upper limits of development density in the preferred planning scenario, the directly connected impervious areas for the proposed developments are similar for both scenarios.

In Tributary 3, the development area is less than 10 % of the total area. As the model output indicates, the increase from the lower to upper limit of development density has a negligible impact on the total flow discharged from Tributary 3 to the Credit River. Concerns with erosion in the downstream reaches are best dealt with through geomorphological improvements.

5.3 Stormwater Management Alternatives

Based on the results of preceding sections, two alternative stormwater management strategies were formulated. Each alternative has two variations that could be implemented. The alternatives and their variations are illustrated in Figure 5.2. Each of the alternatives (and variations) must meet the following criteria, as a minimum:

i. Maintain infiltration conditions in order to continue the moderate recharge capacity of lands proposed for development.

ii. Control post development peak flow rates to pre-development levels for the 2 through 100 year return period storm events.

iii. Provide erosion control storage in order prevent exceedence of erosion thresholds.

iv. Provide water quality control for Level 1 habitat protection, as set out in the Stormwater Management Practices Planning and Design Manual (MOE, 1994).

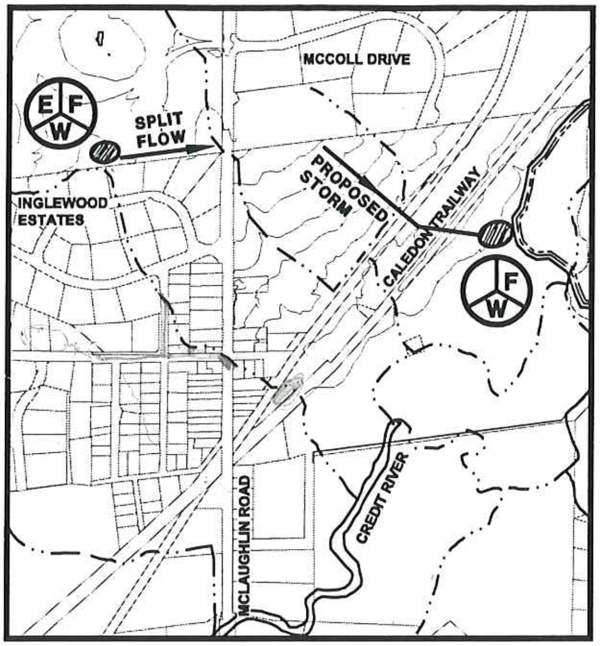
v. Seek to provide enhanced treatment for phosphorus.

As noted in Section 3.2.1, structural infiltration measures are not well suited to the till soils present throughout the area. In order meet criteria i) it will therefore be necessary to disperse runoff to the greatest extent possible. This can be done by a combination of lot level and conveyance controls. Roof discharge to lawns (and perhaps shallow ponding areas) should be employed in all areas. Further, the use of swale drainage, connected into open space drainage ways is recommended. While the Town of Caledon has a municipal standard which requires curbed roadways, swales may be set back inside the curb. The routing of overland flows to the open space drainage ways is consistent with the Town's standards and policies.

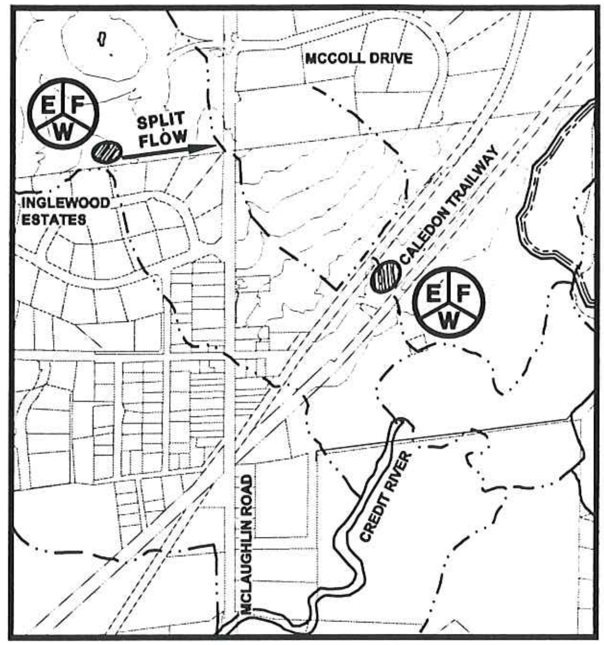
The implementation of measures to maintain infiltration will assist in achieving each of the remaining four criteria. They will be particularly useful in addressing the last two. However, end-of-pipe facilities will also be needed to fully meet these requirements. The combinations of the facilities that have been considered are described below.

Alternative 1

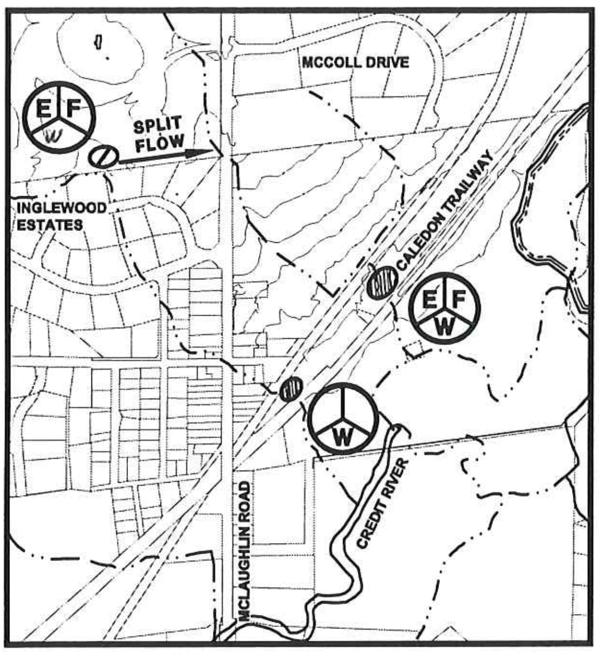
The first alternative consists of two stormwater facilities as illustrated in Figure 5.2. The alternative is based on utilizing the stormwater management concepts prepared by the proponents of the proposed South Slopes and Balmer Heights developments. However, the sizing of the proposed stormwater ponds has been based on projected imperviousness levels, appropriate to the range of development densities contained in the preferred planning scenario (Inglewood Village Study – Phase II). In addition, some modifications were made to the concepts to reflect problems and concerns identified in the present study. The original stormwater management concepts are documented in "Comprehensive Servicing Study for the Balmer Heights Subdivision" (Winter Associates Ltd, April 1998) and the "Servicing Options Study for the South Slopes of Riverdale Farms" (R. J. Burnside & Associates Ltd, June 1996).



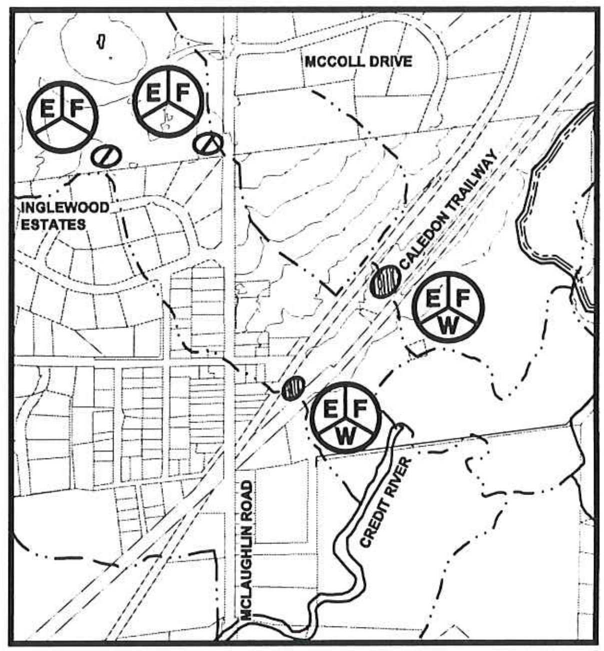
ALTERNATIVE 1A



ALTERNATIVE 1B



ALTERNATIVE 2A



ALTERNATIVE 2B

LEGEND

EROSION CONTROL — E
FLOOD CONTROL — F
WATER QUALITY CONTROL — W

WET POND/ WETLAND

DRY POND

0.1 0 0.1 0.3 0.5km

CLIENT TOWN OF CALEDON	Marshall Macklin Monaghan Consulting Engineers . Surveyors . Planners 80 Commerce Valley Dr. E, Thornhill, Ont. L3T 7N4 Tele (905)882-1100 Fax (905)882-0055
TITLE PREFERRED ALTERNATIVE SWM POND LOCATIONS	Checked D.J.A. / Drawn AutoCAD/B.K.B. Date MAY 1999 / Proj. No. 1499012-01-WR1 Scale AS SHOWN / Figure No. 5.2 / Gr.No. 01

In the original Balmer Heights concept, two connected ponds at a central location were proposed. The west pond element received flow primarily from external areas, while the east pond element received flow primarily from the proposed development area. Smaller storms (eg. design water quality and erosion control events) which entered the west pond were directed to the east pond for treatment. Less frequent (flood level) flows entering the west pond were by-passed. The east pond received flow from a portion of Balmer Heights that currently drains to Tributary 2. The stormwater concept split the east pond discharge between Tributaries 1 and 2 at pre-development levels.

Due to the existing problems with downstream flooding and erosion sensitivity, the original concept was modified to eliminate the bypassing of the west pond flows. A single pond is proposed on the south edge of the property to provide quantity, quality and erosion control for the entire drainage area (development land and external areas). As in the original concept, the discharge is split to return flow to Tributaries 1 and 2 in proportion to their pre-development levels.

For the South Slopes development, an off-line pond is proposed, located east of the CNR trail (Caledon Trailway). This facility would provide quality, quantity and erosion control for the development area, and would discharge to the Credit River directly, rather than returning flow to Tributary 2. The stormwater pond would take flow from only the internal South Slope area. External area flows (from the Balmer Heights and McColl Drive areas) would be routed through the development (without treatment) and discharge to the Credit River through existing culverts. As a result, an additional storm sewer crossing would be required under both the CPR tracks and the CNR trail in order to convey the internal area flows to the proposed stormwater facility.

No development plans have been prepared for the proposed McKenzie development area. The area consists of two parcels of about 3.5 hectares, bisected by Tributaries 3 and 4 (see Figure 5.1). The western-most parcel currently has only limited development potential because of it location, size and difficulty of servicing. In order to gain access to these lands, a roadway and infrastructure would have to be extended across the sensitive valley section. For the purposes of this report, it has therefore been assumed that a comprehensive study (including stormwater management) would be required in order to support any development application in this area.

The second parcel in the McKenzie area is viable as a potential development area, but is small and steeply sloped. In general, the area is not well suited for either lot level or end-of-pipe controls. Stormwater controls for this area will need to be implemented as part of the lot layout and grading strategy. For this reason, the proposed stormwater management for this area should be determined when specific development plans are formulated.

In addition to the undeveloped lands in the McKenzie area, a relatively large portion of the existing development area drains to Tributary 3 / 4. Unfortunately, the majority of existing development (subcatchments 3010, 3020, 3030, 3040, 3060, 3100, and 3102 on Drawing 2 (map pocket)) comprising about 24-25 hectares, drains to a point east of Louise and south of McKenzie and then under the Trailway and the railway tracks to a ditch paralleling McLaughlin Road. There is little scope for providing a water quality facility to service these lands. An existing tractor dealership is located west of McLaughlin Road and north of the farm lane which bisects subcatchment 3120. The Credit River's fifty year flood line extends up through the ditch which crosses under the farm lane. The 20 year floodline extends to a point about 75 m south of the farm lane (about midway between the lane and the Credit River).

Subcatchments 3020 and 3030 (about 8.5 hectares) would be diverted to Tributary 1 if storm sewer improvements discussed in Section 4.2.1 were to be implemented. These lands would then receive water quality treatment if Alternatives 2A or 2B were to be implemented. Subcatchments 3020 and 3030 are relatively recent developments and are not part of the older core area.

The above scenario is referred to as Alternative 1A. In alternative 1B (Figure 5.2) the stormwater management plan for the proposed South Slopes development area is altered slightly. In the original development plan, the area south of Tributary 2 was not proposed for development. However, the preferred planning scenario for the Village of Inglewood includes residential development both north and south of the drainage course (See Figure 5.1). These additional lands cannot be easily drained to a pond located as in Alternative 1A. In the 1B variation, all flows (internal and external) are directed through existing culverts to an on-line stormwater management pond, located on Tributary 2 between the CPR railway and the CNR trail. This option permits treatment of flows from some areas of existing development.

Alternative 2

The basis for alternative 2 is to provide quality control storage at the lower end of the subject tributaries. This would allow the provision of stormwater quality treatment for some of existing development areas as well as the proposed development areas within Inglewood. In addition, an on-line facility at the lower end of Tributary 1 would intercept dry weather flow on the tributary. It is suspected that some private septic systems in the older section of the Village may not be functioning properly. By placing quality control storage at the downstream end of the tributaries, it is possible to provide Level 1 control for the proposed developments while providing an opportunity for the uptake of phosphorus and the removal of other pollutants from surface and subsurface flows from a portion of the Historic Village.

Due to existing restrictions in the system caused by undersized culverts and erosion sensitivity, it is not possible to discharge flows uncontrolled from the development areas, to receive quantity and quality treatment at a downstream facility. However, it is possible to provide quantity and erosion control storage on-site, and to provide Level 1 quality control at a downstream, on-line facility. The upstream facility could be a dry pond, since water quality control would not be required.

As with Alternative 1, two variations were included in Alternative 2, with respect to stormwater management in the proposed Balmer Heights development area. For Alternative 2A, a single stormwater management pond is proposed to control runoff from the entire development area. This would be similar to the pond proposed in Alternative 1, constructed on-line at the south edge of the property. The pond discharge would be split to return flow to Tributaries 1 and 2 at pre-development levels. However, the pond would a dry facility and would only provide quantity and erosion control for the proposed development.

In Alternative 2B (Figure 5.2) two separate facilities are proposed for the Balmer Heights development. The first of these, located on-line on the south edge of the property, would provide quantity and erosion control for the development area in Tributary 1. The second pond would be constructed in the south-east corner of the Balmer Heights property, in approximately the same location as the existing pond, to provide quantity and erosion control storage for the development area that is in Tributary 2.

For both options A and B in Alternative 2, quality control for the proposed Balmer Heights development area in Tributary 1 would be provided in an on-line facility between the CPR railway and the CNR trail on Tributary 1. A pond on Tributary 2 (also between the CPR tracks and the Trailway) as proposed in Alternative 1B would provide quantity and erosion control for the South Slopes development, and Level 1 quality control for both the South Slopes development and the Balmer Heights development within the Tributary 2 watershed.

For the proposed McKenzie development area, the comments provided under 1A again apply.

5.3.1 Modelling Approach

The stormwater management alternatives incorporate elements that address flooding, erosion and water quality. A different basis was used in modelling or assessing each element. Where modelling was required the OTTHYMO model was used.

Quantity (Flood) Control

A common criteria used for quantity control is to limit post development peak flow rates to pre-development levels for the 2 through 100 year return period storm events. However, under existing conditions, culverts in the lower end of Tributary 1 are unable to adequately convey moderate storm events. Therefore consideration was given to controlling post-development peak flow rates to below pre-development levels (eg. over-control).

It was determined that the flow rate from just those areas discharging to Tributary 1 downstream of the proposed Balmer Heights development area is sufficient to overtop MacDonald street at the 900 mm CSP more than once every 10 years, on average. Thus, although controlling post-development flows (in the Balmer Heights development) to significantly below pre-development levels may reduce the frequency of flooding in the

MacDonald Street area, the proposed storm sewer described in Section 4.2.1 would still be required. Incidental over-control of post-development flows (eg. as a result of meeting erosion control targets) may reduce the size of the storm sewers required. However, it is concluded that explicit over-control for flooding is not warranted and that flood control should be based on the pre-development flow rates from the Balmer Heights development area. In determining the quantity control storage requirements, 24 hour SCS storm distributions were used in the OTTHYMO model.

A dry detention pond exists in the southeast corner of the proposed Balmer Heights development area. This was constructed on Tributary 2 for a proposed 48 lot estate residential subdivision that was never initiated. The pond does reduce peak flow rates under existing conditions during the more frequent design storm events, but has little impact on the peak flow rates during the 50 and 100 year return period storms. In sizing the ponds (for the above alternatives) to control post-development peak flow rates to pre-development levels, the effect of the existing pond on pre-development peak flow rates was ignored. Instead, the ponds were sized to control peak flows from the development areas to historic pre-development levels.

Erosion

It is common practice to provide extended detention for erosion control purposes. A common criterion is to capture and release the runoff from a short duration, 25 mm rainfall event over a 24 hour duration. This will act to reduce the peak flow rates and velocities resulting from frequent storm events, reducing potential erosion impacts. The amount of storage required varies depending upon the sensitivity and geomorphology of the stream.

As documented in Section 2.6, a geomorphological assessment was undertaken as part of this study. Critical discharge rates were calculated at particular points in the subject tributaries. Sufficient storage was provided in the stormwater facilities so that the critical discharge rate in the receiving tributary was exceeded no more than once in every 1.5 years, on average. Four hour Chicago distributions were used in the sizing of the ponds for the critical discharge rate.

Water Quality

The basis for water quality control was Level 1 habitat protection, as set out in the Stormwater Management Practices Planning and Design Manual (MOE, 1994). Water quality is not typically modelled because continuous modelling using precipitation records for a number of years is required if the modelling is to be meaningful. Continuous modelling was done in formulating the volumetric requirements specified in the Manual and therefore does not need to be repeated in each study.

For the light development density scenario, with a total imperviousness of approximately 22 % in the proposed development areas, a permanent pool volume of 25 m^3/ha is required for a wetland, and 70 m^3/ha for a wet pond. For the denser development scenario, with a total imperviousness of approximately 28 % in the development areas, the permanent pool volumes required for a wetland and a wet pond are 35 m^3/ha and 85 m^3/ha, respectively. In addition to the permanent pool volumes, an active storage volume of 40 m^3/ha is required, to be released over a 24 hour period. The active storage volume may be omitted in facilities which also provide extended detention (24 hour) erosion control storage, if the latter exceeds 40 m^3/ha.

5.3.2 Modelling Results

Alternative 1A

Alternative 1A represents the stormwater management plans submitted by the developers of the proposed Balmer Heights and South Slopes development areas. Two stormwater management facilities are required for this alternative.

Balmer Heights

A single combined facility will provide water quality, erosion and quantity control. The pond for the proposed Balmer Heights development receives runoff from the entire development area and from approximately 30 hectares west of the development. The outflow from the pond is split to direct the outflow to Tributaries 1 and 2 at predevelopment levels. For water quality sizing purposes, the drainage area of 76.5 ha has an average imperviousness of approximately 16 % and 19.5 % for the lower and higher development density scenarios, respectively. In determining the average imperviousness a value of 8 % was used for undeveloped rural areas. Although these areas contain few impervious surfaces, they may generate significant runoff volumes during relatively frequent events.

To achieve Level 1 control for a drainage area at 16 % impervious, a wetland requires a permanent pool volume of 16.3 m^3 per hectare of drainage area, for a total volume of 1250 m^3. A wet detention pond requires 53 m^3/ha, or 4050 m^3. With an average imperviousness of 19.5 %, 20.6 m^3/ha is required for the permanent pool of a wetland, which corresponds to a volume of 1275 m^3. For a wet pond, 61 m^3/ha, or 4670 m^3, is required for the permanent pool.

For erosion control purposes, it was found that capturing the runoff from a 25 mm rainfall event and releasing it over a 24 hour period was not sufficient to reduce the peak outflow from the pond to Tributary 1 below the critical discharge rate of 0.07 m^3/s. Thus, the pond was sized to release the runoff volume resulting from a 4 hour, 30 mm rainfall event over 24 hours. At the lower development density, this resulted in an extended detention volume of 6150 m^3, comprised of 4650 m^3 from Tributary 1 and 1500 m^3 from Tributary 2. At the higher development density, a 30 mm rainfall event produced a runoff volume of 4950 m^3 in Tributary 1 and 1650 m^3 in Tributary 2, for a total runoff volume of 6600 m^3 to the proposed pond. For both scenarios, this resulted in a peak discharge rate to Tributary 1 of 0.04 m^3/s during the 25 mm event, which closely corresponds to a 1.5 year return period. As the peak outflow of 0.04 m^3/s is less than the critical discharge rate, the pond design is acceptable. The peak flow of 0.01 m^3/s discharged to Tributary 2 during the 25 mm event is significantly lower than the critical discharge rate of 0.18 m^3/s. Aggradation however is not considered to be a major concern because of the intermittent nature of the watercourse.

The volumes indicated above are greater than the standard 40 m^3/ha required for quality control, and therefore govern the extended detention storage volume.

The control of post-development peak flow rates to pre-development levels for up to the 100 year event requires a total active storage volume of 13,000 m^3 for the lower density scenario, and 14,020 m^3 for the higher density scenario. Table 5.3 presents the uncontrolled peak flow rates and the storage required to reduce them to pre-development levels for the 25 mm and 2 through 100 year return period storm events. Storage values shown are cumulative (ie. total storage volume listed for the 100 year storm includes erosion storage and storage for lesser storms).

TABLE 5.3 SUMMARY OF POST DEVELOPMENT FLOWS – ALTERNATIVE 1A

Storm	Pre-development (m^3/s)	Post-Development Flows (m^3/s)					
		Light Development			Dense Development		
		Uncontrolled (m^3/s)	Storage (m^3)	Controlled (m^3/s)	Uncontrolled (m^3/s)	Storage (m^3)	Controlled (m^3/s)
Balmer Heights Pond							
25 mm	-	1.27	3910	0.05	1.46	4240	0.05
2	1.33	1.64	6480	0.66	1.73	6970	0.66
5	2.45	3.05	7500	2.45	3.15	8100	2.45
10	3.27	4.05	8540	3.19	4.18	9260	3.21
25	4.35	5.45	10150	4.35	5.57	11000	4.35
50	5.17	6.51	11530	5.14	6.66	12530	5.17
100	5.99	7.57	13000	5.99	7.69	14020	5.98
South Slopes Pond							
25 mm	-	0.29	860	0.01	0.34	950	0.01
2	0.11	0.33	1400	0.09	0.36	1530	0.09
5	0.20	0.63	2070	0.20	0.65	2230	0.20
10	0.25	0.82	2650	0.25	0.85	2830	0.26
25	0.33	1.06	3430	0.33	1.09	3640	0.33
50	0.38	1.25	4050	0.38	1.28	4280	0.38
100	0.44	1.43	4680	0.44	1.46	4920	0.44

<u>South Slopes</u>

In alternative 1A, an off-line facility is proposed to provide quality, quantity and erosion control storage for the proposed South Slopes development area. As the pond is off-line, it receives runoff from only the 12 ha development area. The average imperviousness of the development area is 22 % for the lower density development scenario, and 28 % in the higher density model. At 22 % impervious, a wetland requires a permanent pool volume of 25 m^3/ha and a wet pond requires 70 m^3/ha, for total pool volumes of 300 m^3 and 850 m^3, respectively. With the drainage area at 28% imperviousness, a wetland requires a permanent pool volume of 35 m^3/ha, or 380 m^3, and a wet detention pond requires 85 m^3/ha, or 1020 m^3.

Again, the erosion control requirement of storing the runoff volume from a 25 mm rainfall event governs over the 40 m^3/ha extended detention storage requirement for quality control. As the pond is to discharge directly to the Credit River, rather than returning flow to Tributary 2, there are no critical discharge criteria. With development densities at the lower limit, an extended detention storage volume of 960 m^3 is required, with a total active storage volume of 4680 m^3 needed to control post-development peak flows to pre-development levels. At the upper limit of development density, a total active storage volume of 4920 m^3 is required, of which 1060 m^3 is extended detention storage.

Alternative 1B

Alternative 1B is very similar to Alternative 1A. The configuration of the pond for the proposed Balmer Heights area remains unchanged. However, the pond for the proposed South Slopes development area is moved on-line in Alternative 1B, and is to be located between the CPR railway and the CNR trail on Tributary 2. The proposed pond drains a total area of 41 hectares. However, the single pond for the proposed Balmer Heights development provides quantity, quality and erosion control for 14 ha in Tributary 2. Thus, the pond is to provide quality, quantity and erosion control for the remaining 27 ha, which has an average imperviousness of 16.6 % and 19.3 % for the lower and higher density development scenarios, respectively. At 16.6 % impervious, a wetland requires a permanent pool volume of 460 m^3 (17 m^3/ha), and a wet detention pond requires 1450 m^3 (54 m^3/ha). With the higher development densities, a permanent pool volume of 550 m^3 (20 m^3/ha) is required for a wetland, and 1650 m^3 (61 m^3/ha) for a wet pond.

The runoff volume from a 25 mm rainfall event is greater than the 40 m^3/ha required for quality control, and was used to size the extended detention storage volume of the pond. For the lower development density scenario, an extended detention storage of 1650 m^3 results, and a total active storage volume of 5570 m^3 is required for peak flow attenuation. For the higher development density situation, a total active storage volume of 5830 m^3 is required, of which 1750 m^3 is for extended detention.

TABLE 5.4 SUMMARY OF POST DEVELOPMENT FLOWS – ALTERNATIVE 1B

Storm	Pre-development (m^3/s)	Post Development Flows (m^3/s)					
		Light Development			Dense Development		
		Uncontrolled (m^3/s)	Storage (m^3)	Controlled (m^3/s)	Uncontrolled (m^3/s)	Storage (m^3)	Controlled (m^3/s)
Balmer Heights Pond							
25 mm	-	1.27	3910	0.05	1.46	4240	0.05
2	1.33	1.64	6480	0.66	1.73	6970	0.66
5	2.45	3.05	7500	2.45	3.15	8100	2.45
10	3.27	4.05	8540	3.19	4.18	9260	3.21
25	4.35	5.45	10150	4.35	5.57	11000	4.35
50	5.17	6.51	11530	5.14	6.66	12530	5.17
100	5.99	7.57	13000	5.99	7.69	14020	5.98
South Slopes Pond							
25 mm	-	0.52	1530	0.02	0.57	1630	0.02
2	0.66	0.63	1920	0.36	0.66	2050	0.36
5	1.19	1.43	2530	1.19	1.46	2700	1.19
10	1.56	2.03	3140	1.53	2.06	3340	1.53
25	2.04	2.70	4070	2.04	2.73	4290	2.04
50	2.41	3.20	4810	2.40	3.24	5070	2.41
100	2.78	3.70	5570	2.78	3.73	5830	2.77

Alternative 2A

In Alternative 2A, the pond locations in Balmer Heights and South Slopes are unchanged from Alternative 1B. However, upstream pond (at Balmer Heights) provides quantity and erosion control only. A water quality (only) pond is added to the bottom of Tributary 1 in this alternative. Tributary 2 continues to be controlled by the on-line pond described in Alternative 1B (water quality, erosion and quantity) but its permanent pool is increased to provide water quality control for the flows directed to Tributary 2 from the Balmer Heights pond. The active storage and discharge rates for each of the ponds remains as shown in Table 5.4. Permanent pool volumes (for water quality) are discussed below.

<u>Balmer Heights</u>

The pond in Balmer Heights is to provide quantity and erosion control, but not quality control. Thus, it is a dry detention pond with the same storage volumes and discharge rates as the active storage zone of the pond for Balmer Heights in Alternative 1A, presented in Table 5.3. Quality control for the proposed Balmer Heights development area will be provided in a downstream facility on Tributary 1, located between the CPR railway and CNR trail.

The downstream facility would drain 79 hectares, with an average imperviousness of 16 % with the lower development densities, and 18.5 % with the higher densities. With the densities in the proposed development areas at the lower limit, a permanent pool volume of 1300 m^3 (16.5 m^3/ha) is required for a wetland, and 4200 m^3 (53 m^3/ha) for a wet detention pond. At the upper limit of development density, 1530 m^3 (19.3 m^3/ha) is required for the permanent pool of a wetland, and 4650 m^3 (59 m^3/ha) for a wet pond. Because the wetland or wet detention pond is separate from the quantity and erosion control facility, 40 m^3/ha of extended detention storage is also required. With 79 ha draining to the quality control facility, an extended detention storage volume of 3200 m^3 is required for the wetland or wet pond in both the lower and higher development density scenarios.

<u>South Slopes</u>

The pond for the proposed South Slopes development area remains at the same location as in Alternative 1B, between the CPR tracks and the CNR trail on Tributary 2. Although the pond for the Balmer Heights development is providing quantity and erosion control for 14 ha in Tributary 2, no quality control is provided. Thus, the South Slopes pond is to provide quantity and erosion control for the uncontrolled 27 ha upstream of the pond, and quality control for the entire 41 ha drainage area. Including the Balmer Heights development area in Tributary 1 increases the average imperviousness of the area the pond is to control, to 18.6 % and 22.4 % for the lower and upper limits of development density, respectively.

Modelling indicated that the runoff volume from a 25 mm rainfall event over the uncontrolled 27 ha area was slightly greater than the 40 m^3/ha over the entire 41 ha required for quality control, and therefore determined the extended detention storage volume of the pond. Thus, the active storage zone of the proposed pond, including the extended detention volume, is the same as that for alternative 1B, presented in Table 5.4. However, the permanent pool was sized to provide quality control for an additional 14 ha. In the lower density development scenario, a permanent pool volume of 800 m^3 (19.5

m^3/ha) is required for a wetland, and 2400 m^3 (59 m^3/ha) for a wet detention pond. For the higher development density, a wetland requires a permanent pool volume of 1000 m^3 (24.3 m^3/ha), and a wet pond requires 3000 m^3 (73 m^3/ha).

Alternative 2B

Alternative 2B is similar to Alternative 2A, with quality control provided downstream on Tributaries 1 and 2. However, rather than a single pond in the proposed Balmer Heights development area, two ponds provide quantity and erosion control on each of Tributaries 1 and 2 in the development area.

Balmer Heights on Tributary 1

For the pond on Tributary 1, it is again required to release the runoff from a 30 mm rainfall event over a 24 hour duration. This is equivalent to a volume of 4650 m^3 for the lower development density, and 4950 m^3 for the more dense development scenario. To control post-development peak flows to pre-development levels for up to the 100 year event, a total pond volume of 9600 m^3 is required for the lower development density, and 10350 m^3 for the higher density scenario. Table 5.5 presents the uncontrolled peak flow rates resulting from the design storm events and the storage required to control them to pre-development levels. The downstream quality control facility, located between the CPR railway and CNR trail on Tributary 1, remains unchanged from Alternative 2A.

Balmer Heights on Tributary 2

The pond on Tributary 2 drains 14 ha on the eastern side of the development area, with an average imperviousness of 22 % for the lower density development scenario, and 28 % for the higher development densities. Capturing the runoff from a 25 mm rainfall event and releasing it over 24 hours is sufficient to reduce the peak flow rate in Tributary 2 below the critical discharge rate. For the lower density scenario, an extended detention

TABLE 5.5 SUMMARY OF POST DEVELOPMENT FLOWS – ALTERNATIVE 2B

Storm	Pre-development (m^3/s)	Post Development Flows (m^3/s)					
		Light Development			Dense Development		
		Uncontrolled (m^3/s)	Storage (m^3)	Controlled (m^3/s)	Uncontrolled (m^3/s)	Storage (m^3)	Controlled (m^3/s)
Balmer Heights Pond on Tributary 1							
25 mm	-	0.95	2930	0.03	1.07	3150	0.04
2	1.08	1.29	4890	0.52	1.35	5230	0.52
5	1.99	2.41	5650	1.99	2.48	6100	1.98
10	2.66	3.21	6410	2.60	3.30	6940	2.61
25	3.54	4.32	7590	3.54	4.40	8200	3.54
50	4.21	5.13	8530	4.17	5.24	9260	4.20
100	4.88	5.98	9600	4.88	6.07	10340	4.87
Balmer Heights Pond on Tributary 2							
25 mm	-	0.32	980	0.01	0.39	1090	0.01
2	0.25	0.35	1300	0.17	0.38	1430	0.18
5	0.46	0.64	1660	0.46	0.67	1830	0.46
10	0.61	0.84	1950	0.60	0.88	2160	0.60
25	0.81	1.13	2390	0.81	1.17	2640	0.81
50	0.96	1.38	2850	0.97	1.42	3130	0.97
100	1.11	1.59	3240	1.11	1.63	3540	1.11
South Slopes Pond							
25 mm	-	0.52	1540	0.02	0.57	1640	0.02
2	0.66	0.63	1970	0.41	0.66	2110	0.42
5	1.19	1.47	2620	1.19	1.50	2810	1.19
10	1.56	2.01	3220	1.53	2.04	3430	1.53
25	2.04	2.68	4110	2.04	2.70	4350	2.04
50	2.41	3.21	4910	2.43	3.24	5160	2.43
100	2.78	3.69	5640	2.78	3.71	5900	2.78

storage volume of 1100 m^3 is required, and a total pond volume of 3240 m^3 is needed to control peak flows to pre-development levels. With development at the upper limit of density, the 25 mm rainfall event generates a runoff volume of 1200 m^3, and an additional 2340 m^3 is required for quantity control, for a total pond volume of 3540 m^3.

South Slopes

With a single pond providing stormwater management for the entire Balmer Heights development area and external lands to the west, it was required to capture the runoff from a 30 mm rainfall event over the entire drainage area and release it over a 24 hour period. However, with two separate ponds, it is possible to reduce the erosion control criteria from capturing the runoff from a 30 mm storm to capturing the runoff from a 25 mm storm for the eastern portion of the development area in Tributary 2. Although this decreases the required storage volume for the pond in Balmer Heights somewhat, at also increases the inflow rates to the pond for the South Slopes development area slightly. As the south slopes pond is controlling the same areas, the quality and erosion control requirements are unchanged from Alternative 2A. However, to control post development peak flow rates to pre-development levels, the active storage requirements increase to 5640 m^3 for the lower density scenario, and 5900 m^3 for development densities at the upper limit.

5.3.3 Evaluation

The alternatives presented in Section 5.2 and modelled in Section 5.3.2 were evaluated based on the following criteria:

- Quality control (basic stormwater quality requirements)
- Quality control (enhanced phosphorus reduction stormwater loading)
- Erosion protection
- Technical Feasibility
- Preservation and enhancement of natural environment
- Estimated costs

Each of the stormwater management alternatives have been sized to provide a minimum of Level 1 water quality control, to control post-development peak flows to pre-development levels, and to provide sufficient extended detention storage for water quality and erosion control, for at least the areas of proposed new development. For erosion control purposes, the larger of a 25 mm runoff volume (discharged over 24 hours) and the volume needed to ensure that the critical discharge rates for the specific Tributaries are exceeded less than once every 1.5 years (on average) have been employed. In addition, alternatives 1B, 2A and 2B allow water quality control to be provided for some existing portions of the Village. These alternatives result in varying degrees of enhanced phophorus removal.

In addition, a preliminary feasibility/design assessment indicates that sufficient space exists for the ponds at all of the specified locations. It can therefore be stated that each of the alternatives could conceivably be implemented as part of a workable strategy. Deciding between alternatives therefore can be based on an assessment of the benefits and difficulties associated with each, together with their expected costs.

Detailed design of the ponds was not undertaken for the current phase of the study. However, gross preliminary cost estimates have been prepared for pond construction required in each alternative. The estimates are based on average construction costs of stormwater management facilities in the GTA ("Financial Contribution Toward Stormwater Controls – Draft", Toronto and Region Conservation Authority, January 1999). Average unit costs of $\$42.18/m^3$ (of total permanent pool and dry storage volume) for water quality control facilities, and $\$35.67/m^3$ for combined facilities (providing quality, erosion and peak flow control) have been used. These unit costs are base costs only and exclude land value, design and review costs and GST. The costs are applicable for planning purposes only and it should be recognized that there was a fair bit of scatter in the data used to derive these averages.

In determining the total storage volumes required, the ponds have been assumed to be constructed as wet detention ponds. Due to the limited areas available for stormwater management facilities and the steep slopes throughout much of the development area, wet detention ponds were selected over wetlands. The option of providing water quality control with wetland facilities will be re-examined when the detailed design of the ponds is carried out.

Alternative 1A

Two stormwater management facilities are required for this alternative, as indicated in Figure 5.2. Each would be a combined facility (eg. quality, erosion, quantity control). The Balmer Heights facility would be located upstream of existing development. The South Slopes facility would be at the downstream end, but would be off-line and would by-pass all external flows. Stormwater for the McKenzie lands would be addressed in the future, once specific development plans for the area are formulated. Due to the small development area and the steep topography, any stormwater solutions would have to be an integral part of the development concept.

Although sufficient space exists for the ponds, as mentioned in Section 5.3.2, it will be difficult to direct runoff to the South Slopes pond from the entire potential development area. Since development is proposed on both sides of the tributary, minor and major system flows will have to be directed north across the tributary to the off-line facility east of the CNR trail. This will require an oversized storm sewer and an additional culvert under the CPR Tracks. The feasibility of the storm sewer alignment would need to be confirmed.

Although directing runoff from the proposed South Slopes development area to the off-line facility would require additional culverts under the CPR railway and CNR tracks, the approach would result in the flow through the existing culvert on Tributary 2 under the CPR railway being reduced somewhat. In Section 4.2, it was determined that this culvert is currently undersized. The reduced flow results in the CPR tracks being overtopped once on average every 7 to 8 years, rather than every 4 to 5 years under existing conditions. In addition, the area flooded upstream of the culvert during events which do not overtop the railway will be reduced. However, it still may be necessary to enlarge the culvert under the CPR tracks to prevent flooding impacts in the development area during rare events. Alternatively, lands subject to infrequent flooding (upstream of the culvert) could be used as open space or parkland.

The east branch of Tributary 2 (draining external areas from the McColl Drive lands) will have no erosion control storage to protect it. Care must be taken in the design/redesign of this branch to ensure that it remains geomorphologically stable and capable of conveying the anticipated flows from internal and external areas.

The estimated pond construction cost for Alternative 1A, with the combined facilities providing a total volume of 22,580 m^3 in the low density scenario is $805,400. With development density at the upper limit, the required pond volumes total 24,630 m^3, for an estimated cost of $878,600. Table 5.6 presents a summary of the costs for each alternative.

Alternative 1B

Alternative 1B is similar to Alternative 1A. The pond for the proposed Balmer Heights is unchanged and continues to provide the desired quantity, quality and erosion control. However, the stormwater management facility for the proposed South Slopes development area is modelled as an on-line facility in this alternative. The pond is to be located between the CPR railway and the CNR trail on Tributary 2. Unlike Alternative 1A, in Alternative 1B, external flows from the east and west branches of Tributary 2 are conveyed to the downstream stormwater management facility. This eliminates the need to separate flows from the South Slopes area from the tributary flow, making the alternative more technically feasible than Alternative 1A. It also allows the provision of treatment for the external lands draining from McColl Drive (lands draining through the west branch of Tributary 2, from Balmer Heights, receive water quality treatment upstream and therefore do not contribute to the sizing of the water quality pool).

Unlike the previous alternative, using the tributary to convey runoff from the development area to the stormwater management facility will increase the flow through the culvert under the CPR. Rather than being overtopped every 4 to 5 years on average, with the existing culvert the increased flows result in the CPR railway being overtopped approximately every 3 years. Thus, to convey flows to the proposed pond, the culvert under the CPR tracks must be enlarged or a second culvert constructed, increasing the cost of the alternative. This also applies to Alternatives 2A and 2B, in which the South Slopes pond is to be constructed on-line.

Furthermore, with the pond constructed on-line, it would be difficult, if not impossible, to provide fish passage beyond the facility. However, as flow in the tributary is intermittent, it is likely that the Tributary provides little fish habitat and performs only a supporting function in this regard. The pond design presented in Table 5.4 is capable of providing the required quality, quantity and erosion control storage. It has been confirmed that sufficient area exists between the railway and the trail for the pond, which could be incorporated into the open space corridor.

However, with the pond on-line, erosion control, peak flow attenuation and quality control storage requirements are increased. With development at the lower limit of density, an additional 1490 m^3 is required, bringing the total cost of the alternative to $856,600. An additional 1540 m^3 is required for the higher density development scenario, bringing the total cost to $933,500. For both scenarios, the total costs are approximately $55,000 than those estimated for Alternative 1A.

Alternative 2A

In Alternative 2A, three stormwater ponds are required (see Figure 5.2). Quality control for the proposed Balmer Heights development is provided in downstream facilities. The active storage volumes of the ponds are unchanged from Alternative 1A, and are capable of providing the required levels of quantity and erosion control. The pond for the proposed Balmer Heights development would be a dry detention pond, with no permanent pool. However, as there is a significant extended detention storage volume that will remain wet for a considerable amount of time after a rainfall event, it is not expected to be possible to use the detention facility as a recreational area.

The permanent pool of the proposed South Slopes stormwater management pond would be increased to provide quality control for the eastern portion of the proposed Balmer Heights development. Preliminary assessment/design indicates that there is sufficient room between the CPR railway and the CNR tracks for the larger pond.

To provide quality control for the remainder of the Balmer Heights development, a quality control pond is proposed downstream on Tributary 1, between the CPR tracks and the CNR trail. This pond would provide quality control for the proposed development, as well as some of the older sections of the Village. The alternative has the potential to reduce pollutant loadings to the Credit River to a greater degree than Alternatives 1A and B. There is sufficient space to construct the quality control pond at this location, although it will consume most of the area between the CPR tracks and CNR trail from McLaughlin road up to or beyond the level of MacDonald Street. The permanent pool requirements for this pond are not significantly greater than that for the Balmer Heights pond in the previous alternatives. However, an extended detention storage volume of 3200 m^3 is required for the quality control pond, in addition to the erosion control storage in the pond in the proposed Balmer Heights development area. As an additional pond is required and the total required pond volumes are increased, Alternative 2A is more expensive than Alternatives 1A and 1B. For the lower development density, a total volume of 27810 m^3 is required for the ponds, of which the quality control pond consumes 7400 m^3. This brings the total construction cost to $1,040,200. With the larger development densities, the water quality control facility is required to be 7850 m^3, and the remaining Balmer Heights dry detention pond and the combined facilities for South Slopes and McKenzie require an additional 22850 m^3, for a total cost of $1,146,200. Compared to Alternative 1A, the stormwater management facilities for both the lower and higher development density scenarios are about $267,000 more expensive to construct.

Alternative 2B

Alternative 2B is similar to Alternative 2A, in that quality control for the proposed Balmer Heights development area is provided downstream on Tributaries 1 and 2. However, rather than a single pond, two dry detention ponds are proposed for Balmer Heights. Rather than capturing the runoff from a 30 mm event over the entire development area and releasing it over 24 hours, this criteria can be relaxed from a 30 mm event to a 25 mm event for the pond on Tributary 2. Both ponds control post-development peak flow rates to pre-development levels and with this configuration, the critical discharge rates in the tributaries continue to be exceeded less than once on average every 1.5 years.

Although the total active storage requirements are reduced for the Balmer Heights development area, the storage for peak flow attenuation in the South Slopes pond is increased somewhat, as the erosion control storage requirement is relaxed somewhat upstream. For both the light and dense development scenarios, the total pond volume requirements are reduced by less than 100 m^3. There may be additional cost savings with this scenario, as runoff from Tributary 2 in the proposed Balmer Heights development would not be conveyed to the single facility to the west, and then back to Tributary 2 below the flow splitter at the pond outlet. However, the preliminary design of the ponds indicates that the surface area of the two ponds is approximately 15 % greater than that of the single pond in the previous alternatives.

The quality control pond on Tributary 1 remains unchanged from Alternative 2A, as does the permanent pool required for the South Slopes pond.

For the light development density scenario, the total pond volume is reduced to 28030 m^3, and the construction cost to $1,037,100. With development density at the upper limit, the pond volumes total to 30640 m^3, for a cost of $1,144,000. The cost difference between Alternatives 2A and 2B is insignificant and so variation 2A would be preferred due to the fewer number of ponds.

Summary of Costs

Table 5.6 provides a summary of the estimated construction costs for each alternative, broken down by area. In addition, the total area provided with water quality treatment is indicated for each alternative, together with a unit area cost for this water quality control.

TABLE 5.6 SUMMARY OF CONSTRUCTION COSTS

Alternative	Total Construction Costs – High Density Development			Water Quality	
	Balmer Heights	South Slopes	Total	Area Served	Unit Cost
1A	$667,000	$212,000	$879,000	88.5ha	$9927/ha
1B	667,000	267,000	$934,000	103.5ha	$9019/ha
2A	831,000	315,000	$1,146,000	120ha	$9550/ha
2B	827,000	317,000	$1,144,000	120ha	$9533/ha

These costs do not include costs for sewer infrastructure in conjunction with the Balmer Heights pond (Alternatives 1A, 1B and 2A), modifications to the sewer systems within the Village (to alleviate existing flooding) or costs for stormwater management for the McKenzie parcel. Each would be common to all alternatives.

5.4 Preferred Alternative

As noted previously, each of the alternatives could be implemented as a feasible surface water management strategy. Any of proposed alternatives is capable of providing the required levels of quality and erosion control and peak flow attenuation <u>for future development</u>. Sufficient area exists for the ponds at the specified locations. Critical velocities in the subject tributaries can be controlled such that they will be exceeded less than once every 1.5 years on average, which is acceptable. The exception to this statement is the east branch or Tributary 2 (draining the McColl Drive area). Here a sound geomorphic design will be required to ensure that the rebuilt tributary remains stable.

Selection of the preferred alternative therefore becomes a question of the overall water quality benefit (eg. the ability to offset the increased phosphorus load which will be generated by the communal sewage treatment systems) and the overall cost. On this basis Alternative 1B is clearly preferable to Alternative 1A and Alternative 2A is preferable to 2B. The decision therefore lies between Alternative 1B and 2A.

Alternative 2A has a projected cost which exceeds that of 1B by about $267,000. However it provides treatment for about 16.5 more hectares of existing development. The unit costs of providing water quality treatment are comparable between the alternatives ($9550/ha vs $9019/ha). Based on this similarity in cost and the agreement by the study team for the Inglewood Village Study to utilize every opportunity to achieve a reduction in the phosphorus loading to the Credit River, Alternative 2A is selected as the preferred alternative. The location of ponds for this alternative is shown in Figure 5.3.

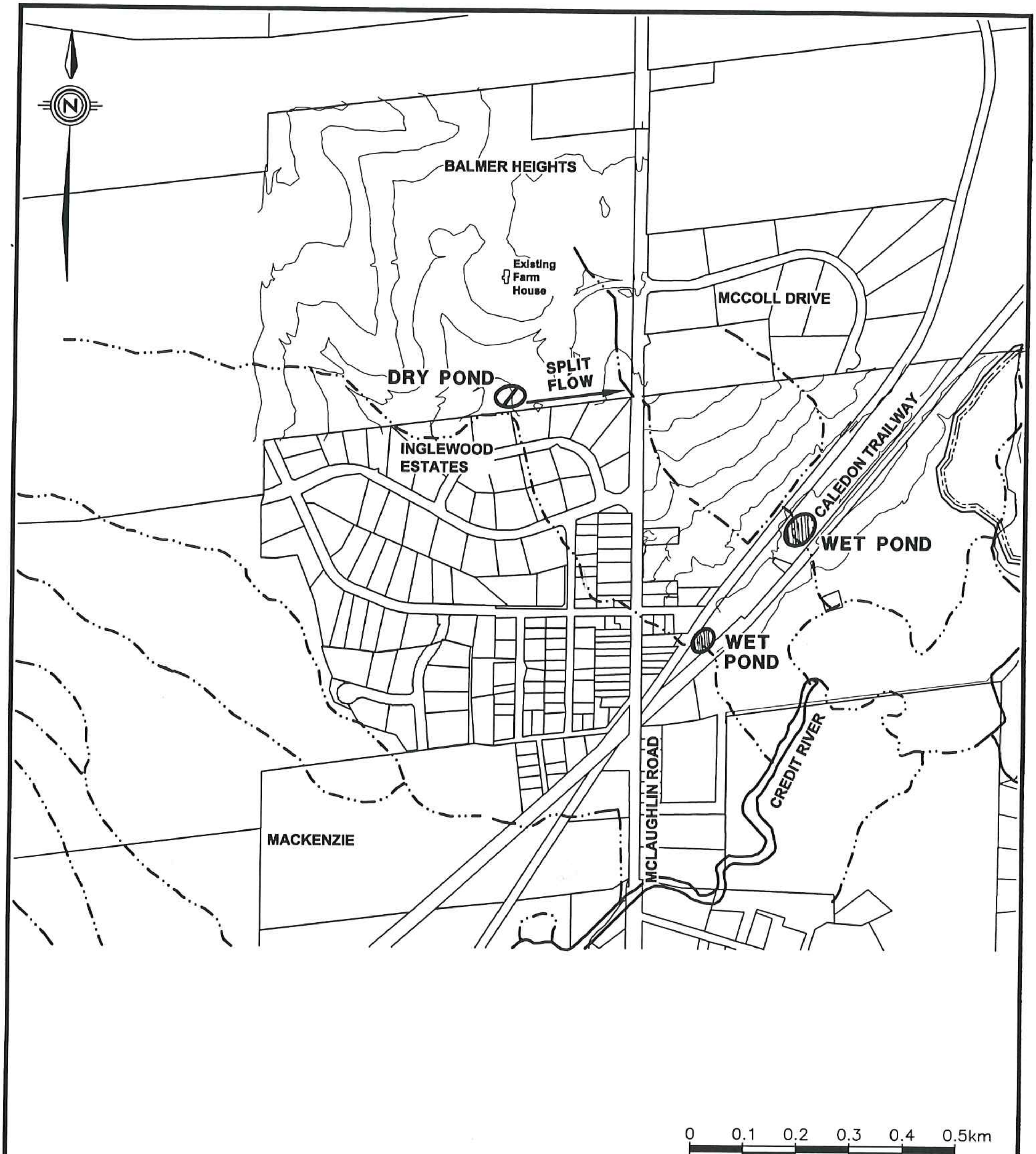
In Alternative 2A, a single dry pond is proposed in the Balmer Heights development area, to provide quantity and erosion control for the development area and external areas to the west. A flow splitter at the pond outlet would return flow to Tributaries 1 and 2 at or below pre-development levels. Downstream on Tributaries 1 and 2, two wet ponds are proposed between the CPR railway and the CNR trail. Each pond would be on-line and would provide quality, quantity and erosion control for the proposed development areas and areas of existing development which drain to these Tributaries. The proposed ponds would provide water quality treatment for a total of 120 ha.

It is recommended that stormwater controls for the Mackenzie area be implemented as part of the lot layout and grading strategy, when this area develops. For this reason, the proposed stormwater management for this area should be determined when specific development plans are formulated.

TABLE 5.7 SUMMARY OF POND VOLUMES FOR PREFERRED ALTERNATIVE

Pond	Permanent Pool (Wet Pond) (m^3)	Extended Detention (m^3)	Peak Flow Attenuation (m^3)	Total (m^3)
Balmer Heights - Low Density	0	6150	6850	13000
Balmer Heights - High Density	0	6600	7420	14020
Tributary 1 WQ Pond – Low Density	4200	3200	0	7400
Tributary 1 WQ Pond – High Density	4650	3200	0	7850
South Slopes - Low Density	2400	1750	4080	8230
South Slopes - High Density	3000	1750	4080	8830

Alternative 2A allows quality control to be provided for both the proposed Balmer Heights and about 16.5 ha of the historic Village. If this alternative is combined with an overall upgrade to the Village conveyance system, an additional 8.4 hectares could conceivably provided with water quality treatment. As was mentioned previously, sewer improvements proposed along Lorne and MacDonald Streets would divert lands along MacDonald (west of Lorne) from Tributary 3 to Tributary 1.



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TOWN OF CALEDON

TITLE

PREFERRED ALTERNATIVE
SWM POND LOCATIONS

Marshall Macklin Monaghan
Consulting Engineers . Surveyors . Planners
80 Commerce Valley Dr. E, Thornhill, Ont. L3T 7N4
Tele (905)882-1100 Fax (905)882-0055

Checked D.J.A.	Drawn AutoCAD/B.K.B.
Date MAY 1999	Proj. No. 1499012-01-WR1
Scale AS SHOWN	Figure No. 5.3 — Gr.No. 01

6.0 CONCLUSIONS AND RECOMMENDATIONS

The urban settlement area of Inglewood Village is drained by three separate tributaries, which discharge to the Credit River. There are existing problems on the tributaries and proposed future development, if uncontrolled, will aggravate and increase these problems. A tributary study has therefore been undertaken to provide a strategy for surface water management that will permit development in a sustainable manner.

The study has drawn heavily upon the ongoing work being completed as part of the Inglewood Village Study. Conclusions and direction provided by the Phase I and II reports (prepared for that study) have been combined with more detailed work in hydrology, geomorphology, hydraulics and stormwater management to arrive at the recommended strategy.

6.1 PRINCIPLE CONCLUSIONS

The following conclusions have been drawn from the Village of Inglewood Study and detailed work completed in the current study. The conclusions define the requirements of the proposed Tributary Study.

6.1.1 Soils and Hydrogeology

The study area is considered to be a moderate recharge zone. The Credit River in the vicinity of Inglewood is a discharge zone. The maintenance or enhancement of infiltration conditions in developing areas is therefore critical for down-gradient quality and quantity conditions.

The soils in Inglewood have low hydraulic conductivities and hence their infiltration capability, on a unit area basis, is low. As a result, structural or "point" infiltration techniques will have limited scope. The primary method of infiltration in the post development condition will have to be based on dispersion of runoff over as much land as possible so that the greatest possible natural infiltration can occur.

The dispersion of runoff is best accomplished through a combination of lot level and conveyance controls. Roof discharge to lawns (and perhaps shallow ponding areas) should be employed in all areas. Further, the use of swale drainage, connected into open space drainage ways should be considered (especially since open space corridors are recommended for other reasons). While the Town of Caledon has a municipal standard which requires curbed roadways, swales may be set back inside the curb. The routing of overland flows to the open space drainage ways is consistent with the Town's standards and policies.

6.1.2 Natural Areas

The majority of significant natural areas in the vicinity of Inglewood are located outside of the urban settlement area and are associated primarily with the Niagara Escarpment or the valley of the Credit River. The most important natural elements within the study area are the watercourses and their associated function (or potential function) as valley corridors and linkages between the Escarpment and the Credit River.

The tributaries within the urban area provide limited fish habitat because of their low to intermittent flow in the summer and the presence of culverts that impede fish passage during periods of higher flow. Each of the tributaries is considered to contribute to fish habitat (primarily due to their hydrologic and geomorphological functions or as a food source).

Based on their corridor function and contribution to downstream fisheries, the tributary system should be preserved to the extent feasible.

From a stormwater perspective, the preservation or incorporation of open space corridors in the development form will provide opportunities to disperse runoff and hence maximize natural infiltration.

6.1.3 Geomorphology

Geomorphological assessments, combined with hydrological assessments, indicate that each of the tributaries is relatively sensitive to increases on flow. Tributary 1 in particular has a low threshold before erosion problems may be expected. Each of the tributaries will be susceptible to any increase in flow magnitude or frequency of occurrence. Extended detention storage for erosion control will be required in upstream areas in order to avoid problems in the post development condition.

6.1.4 Flooding

Flooding concerns exist on Tributary 1 due to the existing culvert capacities. While the problems could be reduced through over-control of flows from upstream areas, the problems would continue to exist. Flows generated by just the areas of existing development (excluding existing upstream flows) are large enough to cause flooding during a 10 year return period storm. As a result, a combination of flow control for upstream areas and upgrading of downstream storm sewer systems should be pursued.

6.1.5 Water Quality

The water quality of the Credit River in the vicinity of Inglewood is classified as Policy 2 (Provincial Water Quality Objectives are currently exceeded and no further degradation will be permitted) on the basis of Total Phosphorus and bacteria. Bacteria is primarily a concern where body-contact recreation occurs. Phosphorus is a concern because it can accelerate eutrophication and lead to excessive plant and algae growth with consequent reductions in dissolved oxygen, which can impair fish and benthic communities. Studies completed as part of the Inglewood Village Study however, indicate no significant dissolved oxygen problems. However, because of the fact that the communal sewage treatment systems will contribute some additional phosphorus, all reasonable actions (in controlling stormwater inputs) that will limit the discharge of phosphorus to the Credit River should be encouraged.

6.1.6 Development Densities

An acceptable tributary management strategy can be implemented for either of the development densities considered (eg. a population range of approximately 1,200 to 1,700 persons by the year 2021). While lower densities will make the use of source and conveyance stormwater practices easier, the levels of imperviousness associated with either end of the density range will permit their implementation. There is sufficient space for the implementation of end-of-pipe facilities designed to accommodate either density level.

6.2 Recommendations

The recommendations that follow can be applied to either of the development densities contemplated. At this time, volumes consistent with the higher end of the density range are provided. This <u>does not</u> imply an endorsement of the higher density. The selection of the final density will be based on considerations that are outside of the areas considered in this study. The Tributary Study can be formulated to accommodate the density selected.

It is recommended that a Tributary Study be adopted that will implement a surface water management strategy consisting of a combination of source, conveyance and end-of-pipe stormwater controls. Specifically that:

a) Lot level and conveyance controls be employed to the greatest extent feasible in all proposed urban developments. Impervious surfaces such as roofs and driveways should discharge to lawns and then to swales. Roads, storm sewers (if employed) and swales should discharge to broad, shallow drainage corridors where possible. The objective should be to spread runoff out over as great an area as possible, in order to take advantage of the limited natural infiltration capacity of the local soils.

b) End-of-pipe controls be employed in the proposed Balmer Heights development to provide erosion and peak flow control. The Balmer Heights facility should be a dry pond total active storage volume of 14,020 m^3. The extended detention portion of active storage should comprise 6600 m^3. Since the facility will drain lands in both Tributaries 1 and 2, its discharge should be split between these tributaries in proportion to their pre-development flows.

c) Water quality control be provided for Tributary 1 (including both new and existing development) by implementing a pond between the CPR tracks and the Caledon Trailway. The facility should be a wet pond, with a permanent pool of 4650 m^3 and extended detention storage of 3200 m^3.

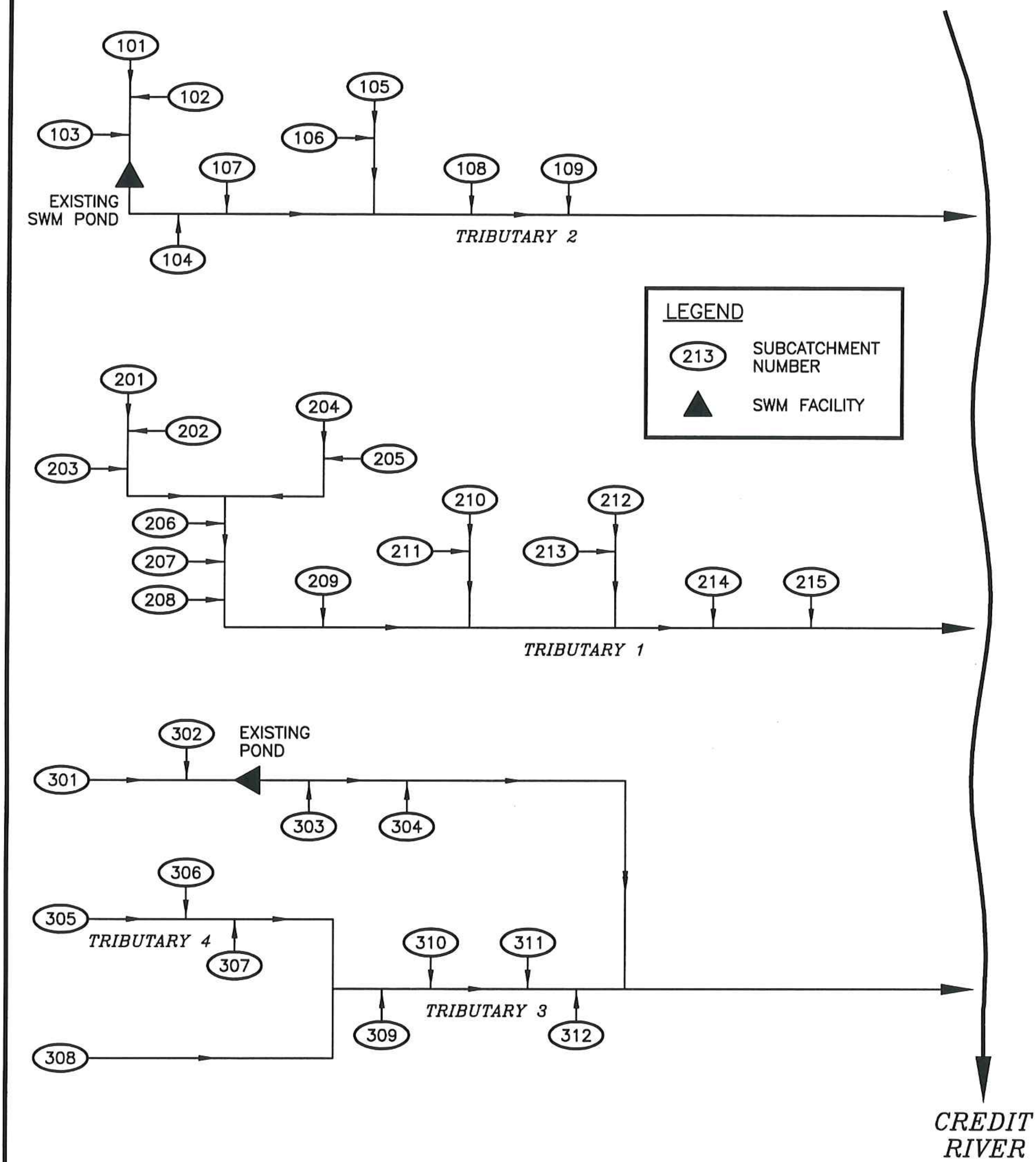
d) The proposed South Slopes development be served by a single end-of-pipe facility located on-line, between the CPR tracks and the Caledon Trailway. The facility should be a combined wet pond with a permanent pool of 3000 m^3 and total active storage of 5830 m^3 (1750 m^3 of which would be extended detention storage). Care must be taken in the design/redesign of the branches of Tributary 2 through the South Slopes property to ensure that they are geomorphologically stable and capable of conveying the anticipated flows from internal and external areas.

e) Due to the location, size, topography and difficulty of servicing anticipated for the McKenzie lands stormwater controls for this area will need to be implemented as part of the lot layout and grading strategy. For this reason, the proposed stormwater management for this area should be determined when specific development plans have been formulated.

6.3 Future Work

This draft of the Tributary Study has been prepared within the context of decisions being made under the broader umbrella of the Inglewood Village Study. The Tributary Study requires additional decisions and approvals related to the larger study before it can be completed. Once the required decisions have been made, appropriate modifications will be made to the Tributary Study and it will be finalized. The final report will include a preliminary functional design of stormwater management facilities and an implementation strategy, which includes construction controls, maintenance and monitoring and a proposed schedule/cost sharing approach.

Predevelopment 24 Hour SCS Storms



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TOWN OF CALEDON

TITLE

PRE-DEVELOPMENT DRAINAGE SCHEMATIC

Marshall Macklin Monaghan
Consulting Engineers . Surveyors . Planners
80 Commerce Valley Dr. E, Thornhill, Ont. L3T 7N4
Tele (905)882-1100 Fax (905)882-0055

Checked S.J.H.	Drawn AutoCAD/B.K.B.
Date MAY 1999	Proj. No. 1499012-01-WR1
Scale N.T.S.	Figure No. A1 — Gr.No. 01

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      [ N= 3.0:Tp=  .09]
      ------------------
      ADD [0305 + 0306]  0704  4 15.0   28.49     .56 12.00  13.86  n/a     .000
      ------------------
      ADD [0704 + 0307]  0705  5 15.0   32.20     .60 12.00  12.93  n/a     .000
      ------------------
      CALIB NASHYD       0308  8 15.0   13.87     .26 12.00  12.50  .26     .000
      [CN=78.0         ]
      [ N= 3.0:Tp=  .18]
      ------------------
      ADD [0705 + 0308]  0706  6 15.0   46.07     .86 12.00  12.80  n/a     .000
      ------------------
      CALIB NASHYD       0309  5 15.0    1.87     .01 12.00   2.89  .06     .000
      [CN=80.0         ]
      [ N= 3.0:Tp=  .07]
      ------------------
      ADD [0706 + 0309]  0707  7 15.0   47.94     .87 12.00  12.41  n/a     .000
      ------------------
      CALIB NASHYD       0310 10 15.0    8.15     .16 12.00  12.96  .27     .000
      [CN=85.0         ]
      [ N= 3.0:Tp=  .12]
      ------------------
      ADD [0707 + 0310]  0708  8 15.0   56.09    1.03 12.00  12.49  n/a     .000
      ------------------
      CALIB NASHYD       0311  1 15.0    1.23     .01 12.00   5.10  .11     .000
      [CN=85.0         ]
      [ N= 3.0:Tp=  .08]
      ------------------
      ADD [0708 + 0311]  0709  7 15.0   57.32    1.04 12.00  12.33  n/a     .000
      ------------------
   *  CALIB STANDHYD     0312  2 15.0    2.80     .09 12.00  25.95  .55     .000
      [I%=20.0:S%= 2.00]
      ------------------
      ADD [0709 + 0312]  0710 10 15.0   60.12    1.13 12.00  12.97  n/a     .000
      ------------------
      ADD [0703 + 0710]  0711  1 15.0   75.91    1.43 12.00  14.34  n/a     .000
      ------------------
      ADD [0999 + 0711]  0999  2 15.0  196.89    3.51 12.00  13.08  n/a     .000
      ------------------
   ** END OF SIMULATION :   1

*******************************************************************************


   ****************************
   ** SIMULATION NUMBER:   2 **
   ****************************

   W/E COMMAND            HYD ID   DT     AREA   Qpeak Tpeak    R.V. R.C.    Qbase
                                   min     ha     cms   hrs      mm           cms

      START @    .00 hrs
      ------------------
      READ STORM                  15.0
      [ Ptot= 67.20 mm ]
      fname :PRSN-SCS.005
      remark:5 YR 24 HR SCS II - PEARSON AIRPORT
      ------------------
      CALIB NASHYD       0101  1 15.0    9.51     .38 12.00  27.01  .40     .000
      [CN=80.0         ]
      [ N= 3.0:Tp=  .21]
      ------------------
      CALIB NASHYD       0102  2 15.0    1.54     .03 12.00  11.41  .17     .000
      [CN=82.0         ]
      [ N= 3.0:Tp=  .09]
      ------------------
      CALIB NASHYD       0103  3 15.0    3.11     .05 12.00  10.31  .15     .000
```

```
   [CN=81.0          ]
   [ N= 3.0:Tp=   .09]
   ------------------
   ADD [0101 + 0103]  0500  4 15.0   12.62     .43 12.00  22.90  n/a    .000
   ------------------
   ADD [0102 + 0500]  0501  5 15.0   14.16     .46 12.00  21.65  n/a    .000
   ------------------
   RESRVR [ 5 : 0501] 0001  1 15.0   14.16     .29 12.25  21.65  n/a    .000
   {ST=   .07 ha.m }
   ------------------
   CALIB NASHYD       0104  4 15.0     .61     .00 12.00    .32  .00    .000
   [CN=80.0          ]
   [ N= 3.0:Tp=   .05]
   ------------------
   ADD [0104 + 0001]  0502  2 15.0   14.77     .29 12.25  20.77  n/a    .000
   ------------------
   CHANNEL[ 2 : 0502] 0000  1 15.0   14.77     .29 12.25  20.76  n/a    .000
   ------------------
   CALIB NASHYD       0105  5 15.0    6.93     .26 12.00  24.96  .37    .000
   [CN=83.0          ]
   [ N= 3.0:Tp=   .15]
   ------------------
   CALIB NASHYD       0106  6 15.0    6.90     .22 12.00  20.55  .31    .000
   [CN=85.0          ]
   [ N= 3.0:Tp=   .12]
   ------------------
   ADD [0105 + 0106]  0503  3 15.0   13.83     .48 12.00  22.76  n/a    .000
   ------------------
   CALIB NASHYD       0107  7 15.0    4.64     .12 12.00  16.76  .25    .000
   [CN=85.0          ]
   [ N= 3.0:Tp=   .10]
   ------------------
   ADD [0000 + 0107]  0504  4 15.0   19.41     .37 12.00  19.81  n/a    .000
   ------------------
   ADD [0503 + 0504]  0505  5 15.0   33.24     .84 12.00  21.03  n/a    .000
   ------------------
   CALIB NASHYD       0108  8 15.0    4.54     .14 12.00  20.33  .30    .000
   [CN=80.0          ]
   [ N= 3.0:Tp=   .13]
   ------------------
   ADD [0505 + 0108]  0506  6 15.0   37.78     .99 12.00  20.95  n/a    .000
   ------------------
   CALIB NASHYD       0109  9 15.0     .48     .00 12.00    .11  .00    .000
   [CN=85.0          ]
   [ N= 3.0:Tp=   .04]
   ------------------
   ADD [0506 + 0109]  0507  7 15.0   38.26     .99 12.00  20.69  n/a    .000
   ------------------
   CALIB NASHYD       0201  1 15.0   16.72     .56 12.00  21.82  .32    .000
   [CN=78.0          ]
   [ N= 3.0:Tp=   .16]
   ------------------
   CALIB NASHYD       0202  2 15.0    4.87     .13 12.00  17.70  .26    .000
   [CN=80.0          ]
   [ N= 3.0:Tp=   .12]
   ------------------
   ADD [0201 + 0202]  0601  4 15.0   21.59     .69 12.00  20.89  n/a    .000
   ------------------
   CALIB NASHYD       0203  3 15.0   16.16     .56 12.00  22.72  .34    .000
   [CN=78.0          ]
   [ N= 3.0:Tp=   .17]
   ------------------
   ADD [0203 + 0601]  0602  1 15.0   37.75    1.25 12.00  21.67  n/a    .000
   ------------------
   CALIB NASHYD       0204  4 15.0   17.96     .71 12.00  26.52  .39    .000
   [CN=80.0          ]
   [ N= 3.0:Tp=   .20]
   ------------------
   CALIB NASHYD       0205  5 15.0    2.00     .03 12.00   8.81  .13    .000
   [CN=80.0          ]
   [ N= 3.0:Tp=   .09]
   ------------------
   ADD [0204 + 0205]  0603  3 15.0   19.96     .74 12.00  24.74  n/a    .000
   ------------------
   ADD [0602 + 0603]  0604  4 15.0   57.71    1.99 12.00  22.74  n/a    .000
   ------------------
   CALIB NASHYD       0206  6 15.0    7.57     .31 12.00  27.07  .40    .000
   [CN=84.0          ]
   [ N= 3.0:Tp=   .16]
   ------------------
   ADD [0604 + 0206]  0605  5 15.0   65.28    2.30 12.00  23.24  n/a    .000
   ------------------
   CHANNEL[ 5 : 0605] 0000  1 15.0   65.28    2.31 12.00  23.24  n/a    .000
   ------------------
 * CALIB STANDHYD     0207  9 15.0    3.14     .16 12.00  38.10  .57    .000
   [I%=20.0:S%= 6.00]
   ------------------
   ADD [0605 + 0207]  0606  6 15.0   68.42    2.46 12.00  23.92  n/a    .000
   ------------------
   CALIB NASHYD       0208  8 15.0    1.18     .00 12.00    .56  .01    .000
   [CN=82.0          ]
   [ N= 3.0:Tp=   .05]
   ------------------
   ADD [0606 + 0208]  0607  4 15.0   69.60    2.46 12.00  23.52  n/a    .000
   ------------------
   CALIB NASHYD       0209  9 15.0    2.42     .02 12.00   4.82  .07    .000
```

	[CN=86.0] [N= 3.0:Tp= .07]									
	ADD [0607 + 0209]	0608	8	15.0	72.02	2.48	12.00	22.90	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	0210	10	15.0	2.50	.11	12.00	37.57	.56	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	0211	1	15.0	.90	.05	12.00	43.66	.65	.000
	ADD [0210 + 0211]	0609	9	15.0	3.40	.16	12.00	39.18	n/a	.000
	ADD [0608 + 0609]	0610	10	15.0	75.42	2.64	12.00	23.63	n/a	.000
	CALIB NASHYD [CN=84.0] [N= 3.0:Tp= .09]	0212	2	15.0	2.91	.05	12.00	10.39	.15	.000
*	CALIB STANDHYD [I%=40.0:S%= 6.00]	0213	3	15.0	1.21	.07	12.00	46.64	.69	.000
	ADD [0212 + 0213]	0611	1	15.0	4.12	.12	12.00	21.03	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	79.54	2.76	12.00	23.50	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	0214	4	15.0	.79	.01	12.00	4.91	.07	.000
	ADD [0612 + 0214]	0613	3	15.0	80.33	2.76	12.00	23.31	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	0215	5	15.0	2.39	.05	12.00	14.29	.21	.000
	ADD [0613 + 0215]	0614	4	15.0	82.72	2.82	12.00	23.05	n/a	.000
	ADD [0614 + 0507]	0999	9	15.0	120.98	3.80	12.00	22.30	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	0301	1	15.0	2.96	.03	12.00	7.04	.10	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	0302	2	15.0	6.31	.32	12.00	36.25	.54	.000
	ADD [0301 + 0302]	0701	4	15.0	9.27	.35	12.00	26.92	n/a	.000
	RESRVR [4 : 0701] {ST= .05 ha.m }	0002	1	15.0	9.27	.22	12.25	26.92	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	0303	3	15.0	2.08	.12	12.00	41.80	.62	.000
	ADD [0002 + 0303]	0702	2	15.0	11.35	.33	12.00	29.65	n/a	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	0304	4	15.0	4.44	.20	12.00	39.95	.59	.000
	ADD [0702 + 0304]	0703	3	15.0	15.79	.53	12.00	32.55	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .21]	0305	5	15.0	26.45	.98	12.00	25.07	.37	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	0306	6	15.0	2.04	.08	12.00	36.24	.54	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .09]	0307	7	15.0	3.71	.06	12.00	10.46	.16	.000
	ADD [0305 + 0306]	0704	4	15.0	28.49	1.07	12.00	25.87	n/a	.000
	ADD [0704 + 0307]	0705	5	15.0	32.20	1.13	12.00	24.09	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .18]	0308	8	15.0	13.87	.49	12.00	23.59	.35	.000
	ADD [0705 + 0308]	0706	6	15.0	46.07	1.62	12.00	23.94	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .07]	0309	5	15.0	1.87	.02	12.00	5.35	.08	.000
	ADD [0706 + 0309]	0707	7	15.0	47.94	1.64	12.00	23.22	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .12]	0310	10	15.0	8.15	.28	12.00	22.58	.34	.000
	ADD [0707 + 0310]	0708	8	15.0	56.09	1.92	12.00	23.12	n/a	.000
	CALIB NASHYD	0311	1	15.0	1.23	.02	12.00	8.94	.13	.000

```
      [CN=85.0          ]
      [ N= 3.0:Tp=  .08]
      -----------------
      ADD [0708 + 0311]  0709  7 15.0   57.32    1.93 12.00  22.82  n/a    .000
      -----------------
  *   CALIB STANDHYD     0312  2 15.0    2.80     .15 12.00  42.32  .63    .000
      [I%=20.0:S%= 2.00]
      -----------------
      ADD [0709 + 0312]  0710 10 15.0   60.12    2.08 12.00  23.73  n/a    .000
      -----------------
      ADD [0703 + 0710]  0711  1 15.0   75.91    2.61 12.00  25.56  n/a    .000
      -----------------
      ADD [0999 + 0711]  0999  2 15.0  196.89    6.41 12.00  23.56  n/a    .000
      -----------------
 ** END OF SIMULATION :   2

*******************************************************************************


 ****************************
 ** SIMULATION NUMBER:   3 **
 ****************************

 W/E COMMAND              HYD ID   DT    AREA   Qpeak Tpeak   R.V.  R.C.   Qbase
                                   min    ha     cms   hrs     mm           cms

     START @    .00 hrs
     -----------------
     READ STORM                   15.0
     [ Ptot= 80.40 mm ]
     fname :PRSN-SCS.010
     remark:10 YR 24 HR SCS II - PEARSON AIRPORT
     -----------------
     CALIB NASHYD        0101  1 15.0    9.51     .51 12.00  36.20  .45    .000
     [CN=80.0          ]
     [ N= 3.0:Tp=  .21]
     -----------------
     CALIB NASHYD        0102  2 15.0    1.54     .04 12.00  15.15  .19    .000
     [CN=82.0          ]
     [ N= 3.0:Tp=  .09]
     -----------------
     CALIB NASHYD        0103  3 15.0    3.11     .07 12.00  13.77  .17    .000
     [CN=81.0          ]
     [ N= 3.0:Tp=  .09]
     -----------------
     ADD [0101 + 0103]   0500  4 15.0   12.62     .57 12.00  30.67  n/a    .000
     -----------------
     ADD [0102 + 0500]   0501  5 15.0   14.16     .61 12.00  28.98  n/a    .000
     -----------------
     RESRVR [ 5 : 0501]  0001  1 15.0   14.16     .53 12.00  28.98  n/a    .000
     {ST=   .08 ha.m }
     -----------------
     CALIB NASHYD        0104  4 15.0     .61     .00 12.00    .44  .01    .000
     [CN=80.0          ]
     [ N= 3.0:Tp=  .05]
     -----------------
     ADD [0104 + 0001]   0502  2 15.0   14.77     .53 12.00  27.80  n/a    .000
     -----------------
     CHANNEL[ 2 : 0502]  0000  1 15.0   14.77     .52 12.25  27.80  n/a    .000
     -----------------
     CALIB NASHYD        0105  5 15.0    6.93     .34 12.00  32.96  .41    .000
     [CN=83.0          ]
     [ N= 3.0:Tp=  .15]
     -----------------
     CALIB NASHYD        0106  6 15.0    6.90     .28 12.00  26.94  .34    .000
     [CN=85.0          ]
     [ N= 3.0:Tp=  .12]
     -----------------
     ADD [0105 + 0106]   0503  3 15.0   13.83     .62 12.00  29.96  n/a    .000
     -----------------
     CALIB NASHYD        0107  7 15.0    4.64     .15 12.00  21.97  .27    .000
     [CN=85.0          ]
     [ N= 3.0:Tp=  .10]
     -----------------
     ADD [0000 + 0107]   0504  4 15.0   19.41     .59 12.00  26.41  n/a    .000
     -----------------
     ADD [0503 + 0504]   0505  5 15.0   33.24    1.22 12.00  27.88  n/a    .000
     -----------------
     CALIB NASHYD        0108  8 15.0    4.54     .19 12.00  27.24  .34    .000
     [CN=80.0          ]
     [ N= 3.0:Tp=  .13]
     -----------------
     ADD [0505 + 0108]   0506  6 15.0   37.78    1.41 12.00  27.81  n/a    .000
     -----------------
     CALIB NASHYD        0109  9 15.0     .48     .00 12.00    .14  .00    .000
     [CN=85.0          ]
     [ N= 3.0:Tp=  .04]
     -----------------
     ADD [0506 + 0109]   0507  7 15.0   38.26    1.41 12.00  27.46  n/a    .000
     -----------------
     CALIB NASHYD        0201  1 15.0   16.72     .75 12.00  29.49  .37    .000
     [CN=78.0          ]
     [ N= 3.0:Tp=  .16]
```

	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .12]	0202	2	15.0	4.87	.18	12.00	23.72	.30	.000
	ADD [0201 + 0202]	0601	4	15.0	21.59	.93	12.00	28.19	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .17]	0203	3	15.0	16.16	.75	12.00	30.70	.38	.000
	ADD [0203 + 0601]	0602	1	15.0	37.75	1.67	12.00	29.26	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .20]	0204	4	15.0	17.96	.95	12.00	35.53	.44	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .09]	0205	5	15.0	2.00	.04	12.00	11.80	.15	.000
	ADD [0204 + 0205]	0603	3	15.0	19.96	.98	12.00	33.15	n/a	.000
	ADD [0602 + 0603]	0604	4	15.0	57.71	2.66	12.00	30.61	n/a	.000
	CALIB NASHYD [CN=84.0] [N= 3.0:Tp= .16]	0206	6	15.0	7.57	.40	12.00	35.62	.44	.000
	ADD [0604 + 0206]	0605	5	15.0	65.28	3.06	12.00	31.19	n/a	.000
	CHANNEL[5 : 0605]	0000	1	15.0	65.28	3.09	12.00	31.19	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	0207	9	15.0	3.14	.21	12.00	48.98	.61	.000
	ADD [0605 + 0207]	0606	6	15.0	68.42	3.27	12.00	32.01	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	0208	8	15.0	1.18	.00	12.00	.82	.01	.000
	ADD [0606 + 0208]	0607	4	15.0	69.60	3.27	12.00	31.48	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	0209	9	15.0	2.42	.02	12.00	6.27	.08	.000
	ADD [0607 + 0209]	0608	8	15.0	72.02	3.30	12.00	30.63	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	0210	10	15.0	2.50	.14	12.00	48.42	.60	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	0211	1	15.0	.90	.07	12.00	55.01	.68	.000
	ADD [0210 + 0211]	0609	9	15.0	3.40	.21	12.00	50.17	n/a	.000
	ADD [0608 + 0609]	0610	10	15.0	75.42	3.50	12.00	31.51	n/a	.000
	CALIB NASHYD [CN=84.0] [N= 3.0:Tp= .09]	0212	2	15.0	2.91	.06	12.00	13.64	.17	.000
*	CALIB STANDHYD [I%=40.0:S%= 6.00]	0213	3	15.0	1.21	.09	12.00	58.33	.73	.000
	ADD [0212 + 0213]	0611	1	15.0	4.12	.15	12.00	26.76	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	79.54	3.66	12.00	31.26	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	0214	4	15.0	.79	.01	12.00	6.51	.08	.000
	ADD [0612 + 0214]	0613	3	15.0	80.33	3.66	12.00	31.02	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	0215	5	15.0	2.39	.07	12.00	19.32	.24	.000
	ADD [0613 + 0215]	0614	4	15.0	82.72	3.73	12.00	30.68	n/a	.000
	ADD [0614 + 0507]	0999	9	15.0	120.98	5.14	12.00	29.66	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	0301	1	15.0	2.96	.04	12.00	9.44	.12	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	0302	2	15.0	6.31	.41	12.00	46.97	.58	.000
	ADD [0301 + 0302]	0701	4	15.0	9.27	.46	12.00	34.99	n/a	.000
*	RESRVR [4 : 0701] {ST= .06 ha.m }	0002	1	15.0	9.27	.54	12.00	34.98	n/a	.000

```
      ------------------
  *   CALIB STANDHYD      0303  3 15.0     2.08     .15 12.00  53.00  .66     .000
      [I%=30.0:S%= 6.00]
      ------------------
      ADD [0002 + 0303]   0702  2 15.0    11.35     .69 12.00  38.29  n/a     .000
      ------------------
  *   CALIB STANDHYD      0304  4 15.0     4.44     .27 12.00  50.99  .63     .000
      [I%=25.0:S%= 5.00]
      ------------------
      ADD [0702 + 0304]   0703  3 15.0    15.79     .96 12.00  41.86  n/a     .000
      ------------------
      CALIB NASHYD        0305  5 15.0    26.45    1.32 12.00  33.88  .42     .000
      [CN=78.0         ]
      [ N= 3.0:Tp=  .21]
      ------------------
  *   CALIB STANDHYD      0306  6 15.0     2.04     .11 12.00  46.97  .58     .000
      [I%=15.0:S%= 6.00]
      ------------------
      CALIB NASHYD        0307  7 15.0     3.71     .08 12.00  13.89  .17     .000
      [CN=82.0         ]
      [ N= 3.0:Tp=  .09]
      ------------------
      ADD [0305 + 0306]   0704  4 15.0    28.49    1.44 12.00  34.81  n/a     .000
      ------------------
      ADD [0704 + 0307]   0705  5 15.0    32.20    1.51 12.00  32.40  n/a     .000
      ------------------
      CALIB NASHYD        0308  8 15.0    13.87     .66 12.00  31.87  .40     .000
      [CN=78.0         ]
      [ N= 3.0:Tp=  .18]
      ------------------
      ADD [0705 + 0308]   0706  6 15.0    46.07    2.18 12.00  32.24  n/a     .000
      ------------------
      CALIB NASHYD        0309  5 15.0     1.87     .02 12.00   7.16  .09     .000
      [CN=80.0         ]
      [ N= 3.0:Tp=  .07]
      ------------------
      ADD [0706 + 0309]   0707  7 15.0    47.94    2.20 12.00  31.27  n/a     .000
      ------------------
      CALIB NASHYD        0310 10 15.0     8.15     .36 12.00  29.47  .37     .000
      [CN=85.0         ]
      [ N= 3.0:Tp=  .12]
      ------------------
      ADD [0707 + 0310]   0708  8 15.0    56.09    2.56 12.00  31.00  n/a     .000
      ------------------
      CALIB NASHYD        0311  1 15.0     1.23     .02 12.00  11.69  .15     .000
      [CN=85.0         ]
      [ N= 3.0:Tp=  .08]
      ------------------
      ADD [0708 + 0311]   0709  7 15.0    57.32    2.58 12.00  30.59  n/a     .000
      ------------------
  *   CALIB STANDHYD      0312  2 15.0     2.80     .19 12.00  53.85  .67     .000
      [I%=20.0:S%= 2.00]
      ------------------
      ADD [0709 + 0312]   0710 10 15.0    60.12    2.77 12.00  31.67  n/a     .000
      ------------------
      ADD [0703 + 0710]   0711  1 15.0    75.91    3.73 12.00  33.79  n/a     .000
      ------------------
      ADD [0999 + 0711]   0999  2 15.0   196.89    8.87 12.00  31.26  n/a     .000
      ------------------
 ** END OF SIMULATION :    3

****************************************************************************


 ***************************
 ** SIMULATION NUMBER:   4 **
 ***************************

 W/E COMMAND             HYD ID   DT     AREA   Qpeak Tpeak   R.V. R.C.    Qbase
                                 min      ha     cms   hrs     mm           cms

      START @     .00 hrs
      ------------------
      READ STORM                 15.0
      [ Ptot= 97.00 mm ]
      fname :PRSN-SCS.025
      remark:10 YR 24 HR SCS II - PEARSON AIRPORT
      ------------------
      CALIB NASHYD        0101  1 15.0     9.51     .67 12.00  48.43  .50     .000
      [CN=80.0         ]
      [ N= 3.0:Tp=  .21]
      ------------------
      CALIB NASHYD        0102  2 15.0     1.54     .05 12.00  20.10  .21     .000
      [CN=82.0         ]
      [ N= 3.0:Tp=  .09]
      ------------------
      CALIB NASHYD        0103  3 15.0     3.11     .09 12.00  18.37  .19     .000
      [CN=81.0         ]
      [ N= 3.0:Tp=  .09]
      ------------------
      ADD [0101 + 0103]   0500  4 15.0    12.62     .76 12.00  41.02  n/a     .000
      ------------------
      ADD [0102 + 0500]   0501  5 15.0    14.16     .81 12.00  38.75  n/a     .000
```

```
  ------------------
  RESRVR [ 5 : 0501] 0001  1 15.0   14.16     .77 12.00  38.74  n/a    .000
  {ST=   .09 ha.m }
  ------------------
  CALIB NASHYD       0104  4 15.0     .61     .00 12.00    .66  .01    .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .05]
  ------------------
  ADD [0104 + 0001]  0502  2 15.0   14.77     .77 12.00  37.17  n/a    .000
  ------------------
  CHANNEL[ 2 : 0502] 0000  1 15.0   14.77     .75 12.00  37.17  n/a    .000
  ------------------
  CALIB NASHYD       0105  5 15.0    6.93     .45 12.00  43.50  .45    .000
  [CN=83.0         ]
  [ N= 3.0:Tp=  .15]
  ------------------
  CALIB NASHYD       0106  6 15.0    6.90     .36 12.00  35.31  .36    .000
  [CN=85.0         ]
  [ N= 3.0:Tp=  .12]
  ------------------
  ADD [0105 + 0106]  0503  3 15.0   13.83     .81 12.00  39.41  n/a    .000
  ------------------
  CALIB NASHYD       0107  7 15.0    4.64     .20 12.00  28.79  .30    .000
  [CN=85.0         ]
  [ N= 3.0:Tp=  .10]
  ------------------
  ADD [0000 + 0107]  0504  4 15.0   19.41     .94 12.00  35.17  n/a    .000
  ------------------
  ADD [0503 + 0504]  0505  5 15.0   33.24    1.75 12.00  36.93  n/a    .000
  ------------------
  CALIB NASHYD       0108  8 15.0    4.54     .25 12.00  36.45  .38    .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .13]
  ------------------
  ADD [0505 + 0108]  0506  6 15.0   37.78    2.00 12.00  36.88  n/a    .000
  ------------------
  CALIB NASHYD       0109  9 15.0     .48     .00 12.00    .19  .00    .000
  [CN=85.0         ]
  [ N= 3.0:Tp=  .04]
  ------------------
  ADD [0506 + 0109]  0507  7 15.0   38.26    2.00 12.00  36.42  n/a    .000
  ------------------
  CALIB NASHYD       0201  1 15.0   16.72    1.00 12.00  39.78  .41    .000
  [CN=78.0         ]
  [ N= 3.0:Tp=  .16]
  ------------------
  CALIB NASHYD       0202  2 15.0    4.87     .23 12.00  31.74  .33    .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .12]
  ------------------
  ADD [0201 + 0202]  0601  4 15.0   21.59    1.23 12.00  37.97  n/a    .000
  ------------------
  CALIB NASHYD       0203  3 15.0   16.16    1.00 12.00  41.41  .43    .000
  [CN=78.0         ]
  [ N= 3.0:Tp=  .17]
  ------------------
  ADD [0203 + 0601]  0602  1 15.0   37.75    2.23 12.00  39.44  n/a    .000
  ------------------
  CALIB NASHYD       0204  4 15.0   17.96    1.26 12.00  47.54  .49    .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .20]
  ------------------
  CALIB NASHYD       0205  5 15.0    2.00     .05 12.00  15.79  .16    .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .09]
  ------------------
  ADD [0204 + 0205]  0603  3 15.0   19.96    1.30 12.00  44.36  n/a    .000
  ------------------
  ADD [0602 + 0603]  0604  4 15.0   57.71    3.54 12.00  41.14  n/a    .000
  ------------------
  CALIB NASHYD       0206  6 15.0    7.57     .52 12.00  46.85  .48    .000
  [CN=84.0         ]
  [ N= 3.0:Tp=  .16]
  ------------------
  ADD [0604 + 0206]  0605  5 15.0   65.28    4.06 12.00  41.81  n/a    .000
  ------------------
  CHANNEL[ 5 : 0605] 0000  1 15.0   65.28    4.12 12.00  41.81  n/a    .000
  ------------------
* CALIB STANDHYD     0207  9 15.0    3.14     .27 12.00  63.24  .65    .000
  [I%=20.0:S%= 6.00]
  ------------------
  ADD [0605 + 0207]  0606  6 15.0   68.42    4.33 12.00  42.79  n/a    .000
  ------------------
  CALIB NASHYD       0208  8 15.0    1.18     .00 12.00   1.14  .01    .000
  [CN=82.0         ]
  [ N= 3.0:Tp=  .05]
  ------------------
  ADD [0606 + 0208]  0607  4 15.0   69.60    4.34 12.00  42.08  n/a    .000
  ------------------
  CALIB NASHYD       0209  9 15.0    2.42     .03 12.00   8.16  .08    .000
  [CN=86.0         ]
  [ N= 3.0:Tp=  .07]
  ------------------
  ADD [0607 + 0209]  0608  8 15.0   72.02    4.37 12.00  40.94  n/a    .000
  ------------------
* CALIB STANDHYD     0210 10 15.0    2.50     .19 12.00  62.66  .65    .000
```

```
     [I%=18.0:S%= 5.00]
     ------------------
  *  CALIB STANDHYD       0211  1 15.0     .90      .08 12.00  69.75  .72    .000
     [I%=35.0:S%= 4.00]
     ------------------
     ADD [0210 + 0211]    0609  9 15.0    3.40      .27 12.00  64.54  n/a    .000
     ------------------
     ADD [0608 + 0609]    0610 10 15.0   75.42     4.64 12.00  42.01  n/a    .000
     ------------------
     CALIB NASHYD         0212  2 15.0    2.91      .08 12.00  17.90  .18    .000
     [CN=84.0          ]
     [ N= 3.0:Tp=  .09]
     ------------------
  *  CALIB STANDHYD       0213  3 15.0    1.21      .12 12.00  73.42  .76    .000
     [I%=40.0:S%= 6.00]
     ------------------
     ADD [0212 + 0213]    0611  1 15.0    4.12      .19 12.00  34.21  n/a    .000
     ------------------
     ADD [0611 + 0610]    0612  2 15.0   79.54     4.83 12.00  41.60  n/a    .000
     ------------------
     CALIB NASHYD         0214  4 15.0     .79      .01 12.00   8.64  .09    .000
     [CN=82.0          ]
     [ N= 3.0:Tp=  .07]
     ------------------
     ADD [0612 + 0214]    0613  3 15.0   80.33     4.84 12.00  41.28  n/a    .000
     ------------------
     CALIB NASHYD         0215  5 15.0    2.39      .09 12.00  26.07  .27    .000
     [CN=78.0          ]
     [ N= 3.0:Tp=  .11]
     ------------------
     ADD [0613 + 0215]    0614  4 15.0   82.72     4.94 12.00  40.84  n/a    .000
     ------------------
     ADD [0614 + 0507]    0999  9 15.0  120.98     6.94 12.00  39.44  n/a    .000
     ------------------
     CALIB NASHYD         0301  1 15.0    2.96      .06 12.00  12.63  .13    .000
     [CN=80.0          ]
     [ N= 3.0:Tp=  .08]
     ------------------
  *  CALIB STANDHYD       0302  2 15.0    6.31      .54 12.00  61.08  .63    .000
     [I%=15.0:S%= 6.00]
     ------------------
     ADD [0301 + 0302]    0701  4 15.0    9.27      .59 12.00  45.61  n/a    .000
     ------------------
  *  RESRVR [ 4 : 0701]   0002  1 15.0    9.27      .60 11.75  45.60  n/a    .000
     {ST=   .05 ha.m }
     ------------------
  *  CALIB STANDHYD       0303  3 15.0    2.08      .19 12.00  67.58  .70    .000
     [I%=30.0:S%= 6.00]
     ------------------
     ADD [0002 + 0303]    0702  2 15.0   11.35      .77 11.75  49.63  n/a    .000
     ------------------
  *  CALIB STANDHYD       0304  4 15.0    4.44      .35 12.00  65.41  .67    .000
     [I%=25.0:S%= 5.00]
     ------------------
     ADD [0702 + 0304]    0703  3 15.0   15.79     1.09 12.00  54.07  n/a    .000
     ------------------
     CALIB NASHYD         0305  5 15.0   26.45     1.77 12.00  45.70  .47    .000
     [CN=78.0          ]
     [ N= 3.0:Tp=  .21]
     ------------------
  *  CALIB STANDHYD       0306  6 15.0    2.04      .15 12.00  61.07  .63    .000
     [I%=15.0:S%= 6.00]
     ------------------
     CALIB NASHYD         0307  7 15.0    3.71      .10 12.00  18.43  .19    .000
     [CN=82.0          ]
     [ N= 3.0:Tp=  .09]
     ------------------
     ADD [0305 + 0306]    0704  4 15.0   28.49     1.92 12.00  46.80  n/a    .000
     ------------------
     ADD [0704 + 0307]    0705  5 15.0   32.20     2.03 12.00  43.53  n/a    .000
     ------------------
     CALIB NASHYD         0308  8 15.0   13.87      .89 12.00  43.00  .44    .000
     [CN=78.0          ]
     [ N= 3.0:Tp=  .18]
     ------------------
     ADD [0705 + 0308]    0706  6 15.0   46.07     2.91 12.00  43.37  n/a    .000
     ------------------
     CALIB NASHYD         0309  5 15.0    1.87      .03 12.00   9.58  .10    .000
     [CN=80.0          ]
     [ N= 3.0:Tp=  .07]
     ------------------
     ADD [0706 + 0309]    0707  7 15.0   47.94     2.94 12.00  42.05  n/a    .000
     ------------------
     CALIB NASHYD         0310 10 15.0    8.15      .46 12.00  38.48  .40    .000
     [CN=85.0          ]
     [ N= 3.0:Tp=  .12]
     ------------------
     ADD [0707 + 0310]    0708  8 15.0   56.09     3.40 12.00  41.53  n/a    .000
     ------------------
     CALIB NASHYD         0311  1 15.0    1.23      .03 12.00  15.30  .16    .000
     [CN=85.0          ]
     [ N= 3.0:Tp=  .08]
     ------------------
     ADD [0708 + 0311]    0709  7 15.0   57.32     3.43 12.00  40.97  n/a    .000
     ------------------
  *  CALIB STANDHYD       0312  2 15.0    2.80      .26 12.00  68.81  .71    .000
```

```
     [I%=20.0:S%= 2.00]
     ------------------
     ADD [0709 + 0312]  0710 10 15.0   60.12    3.69 12.00  42.27  n/a    .000
     ------------------
     ADD [0703 + 0710]  0711  1 15.0   75.91    4.78 12.00  44.72  n/a    .000
     ------------------
     ADD [0999 + 0711]  0999  2 15.0  196.89   11.72 12.00  41.48  n/a    .000
     ------------------
 ** END OF SIMULATION :   4

******************************************************************************

 *****************************
 ** SIMULATION NUMBER:   5 **
 *****************************

 W/E COMMAND            HYD ID   DT    AREA   Qpeak Tpeak   R.V. R.C.   Qbase
                                 min    ha     cms   hrs     mm          cms

     START @    .00 hrs
     ------------------
     READ STORM                 15.0
     [ Ptot=109.30 mm ]
     fname :PRSN-SCS.050
     remark:10 YR 24 HR SCS II - PEARSON AIRPORT
     ------------------
     CALIB NASHYD       0101  1 15.0    9.51     .80 12.00  57.87  .53    .000
     [CN=80.0         ]
     [ N= 3.0:Tp=  .21]
     ------------------
     CALIB NASHYD       0102  2 15.0    1.54     .05 12.00  23.90  .22    .000
     [CN=82.0         ]
     [ N= 3.0:Tp=  .09]
     ------------------
     CALIB NASHYD       0103  3 15.0    3.11     .10 12.00  21.91  .20    .000
     [CN=81.0         ]
     [ N= 3.0:Tp=  .09]
     ------------------
     ADD [0101 + 0103]  0500  4 15.0   12.62     .90 12.00  49.01  n/a    .000
     ------------------
     ADD [0102 + 0500]  0501  5 15.0   14.16     .96 12.00  46.28  n/a    .000
     ------------------
     RESRVR [ 5 : 0501] 0001  1 15.0   14.16     .91 12.00  46.27  n/a    .000
     {ST=   .10 ha.m }
     ------------------
     CALIB NASHYD       0104  4 15.0     .61     .00 12.00    .80  .01    .000
     [CN=80.0         ]
     [ N= 3.0:Tp=  .05]
     ------------------
     ADD [0104 + 0001]  0502  2 15.0   14.77     .92 12.00  44.40  n/a    .000
     ------------------
     CHANNEL[ 2 : 0502] 0000  1 15.0   14.77     .98 12.00  44.39  n/a    .000
     ------------------
     CALIB NASHYD       0105  5 15.0    6.93     .52 12.00  51.56  .47    .000
     [CN=83.0         ]
     [ N= 3.0:Tp=  .15]
     ------------------
     CALIB NASHYD       0106  6 15.0    6.90     .42 12.00  41.68  .38    .000
     [CN=85.0         ]
     [ N= 3.0:Tp=  .12]
     ------------------
     ADD [0105 + 0106]  0503  3 15.0   13.83     .95 12.00  46.63  n/a    .000
     ------------------
     CALIB NASHYD       0107  7 15.0    4.64     .23 12.00  33.99  .31    .000
     [CN=85.0         ]
     [ N= 3.0:Tp=  .10]
     ------------------
     ADD [0000 + 0107]  0504  4 15.0   19.41    1.21 12.00  41.91  n/a    .000
     ------------------
     ADD [0503 + 0504]  0505  5 15.0   33.24    2.15 12.00  43.87  n/a    .000
     ------------------
     CALIB NASHYD       0108  8 15.0    4.54     .30 12.00  43.55  .40    .000
     [CN=80.0         ]
     [ N= 3.0:Tp=  .13]
     ------------------
     ADD [0505 + 0108]  0506  6 15.0   37.78    2.45 12.00  43.83  n/a    .000
     ------------------
     CALIB NASHYD       0109  9 15.0     .48     .00 12.00    .23  .00    .000
     [CN=85.0         ]
     [ N= 3.0:Tp=  .04]
     ------------------
     ADD [0506 + 0109]  0507  7 15.0   38.26    2.45 12.00  43.29  n/a    .000
     ------------------
     CALIB NASHYD       0201  1 15.0   16.72    1.19 12.00  47.76  .44    .000
     [CN=78.0         ]
     [ N= 3.0:Tp=  .16]
     ------------------
     CALIB NASHYD       0202  2 15.0    4.87     .28 12.00  37.92  .35    .000
     [CN=80.0         ]
     [ N= 3.0:Tp=  .12]
     ------------------
     ADD [0201 + 0202]  0601  4 15.0   21.59    1.47 12.00  45.54  n/a    .000
```

	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .17]	0203	3	15.0	16.16	1.19	12.00	49.72	.45	.000
	ADD [0203 + 0601]	0602	1	15.0	37.75	2.66	12.00	47.33	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .20]	0204	4	15.0	17.96	1.49	12.00	56.81	.52	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .09]	0205	5	15.0	2.00	.06	12.00	18.87	.17	.000
	ADD [0204 + 0205]	0603	3	15.0	19.96	1.55	12.00	53.01	n/a	.000
	ADD [0602 + 0603]	0604	4	15.0	57.71	4.21	12.00	49.29	n/a	.000
	CALIB NASHYD [CN=84.0] [N= 3.0:Tp= .16]	0206	6	15.0	7.57	.61	12.00	55.42	.51	.000
	ADD [0604 + 0206]	0605	5	15.0	65.28	4.82	12.00	50.00	n/a	.000
	CHANNEL[5 : 0605]	0000	1	15.0	65.28	4.91	12.00	50.00	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	0207	9	15.0	3.14	.32	12.00	74.12	.68	.000
	ADD [0605 + 0207]	0606	6	15.0	68.42	5.14	12.00	51.11	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	0208	8	15.0	1.18	.00	12.00	1.39	.01	.000
	ADD [0606 + 0208]	0607	4	15.0	69.60	5.14	12.00	50.27	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	0209	9	15.0	2.42	.03	12.00	9.60	.09	.000
	ADD [0607 + 0209]	0608	8	15.0	72.02	5.18	12.00	48.90	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	0210	10	15.0	2.50	.23	12.00	73.53	.67	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	0211	1	15.0	.90	.10	12.00	80.91	.74	.000
	ADD [0210 + 0211]	0609	9	15.0	3.40	.32	12.00	75.48	n/a	.000
	ADD [0608 + 0609]	0610	10	15.0	75.42	5.50	12.00	50.10	n/a	.000
	CALIB NASHYD [CN=84.0] [N= 3.0:Tp= .09]	0212	2	15.0	2.91	.09	12.00	21.15	.19	.000
*	CALIB STANDHYD [I%=40.0:S%= 6.00]	0213	3	15.0	1.21	.13	12.00	84.81	.78	.000
	ADD [0212 + 0213]	0611	1	15.0	4.12	.22	12.00	39.85	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	79.54	5.73	12.00	49.57	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	0214	4	15.0	.79	.01	12.00	10.28	.09	.000
	ADD [0612 + 0214]	0613	3	15.0	80.33	5.74	12.00	49.18	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	0215	5	15.0	2.39	.11	12.00	31.29	.29	.000
	ADD [0613 + 0215]	0614	4	15.0	82.72	5.85	12.00	48.67	n/a	.000
	ADD [0614 + 0507]	0999	9	15.0	120.98	8.30	12.00	46.96	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	0301	1	15.0	2.96	.07	12.00	15.09	.14	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	0302	2	15.0	6.31	.63	12.00	71.86	.66	.000
	ADD [0301 + 0302]	0701	4	15.0	9.27	.70	12.00	53.73	n/a	.000
*	RESRVR [4 : 0701] {ST= .06 ha.m }	0002	1	15.0	9.27	.84	11.75	53.73	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	0303	3	15.0	2.08	.22	12.00	78.65	.72	.000
	ADD [0002 + 0303]	0702	2	15.0	11.35	1.05	11.75	58.30	n/a	.000

```
 *    CALIB STANDHYD      0304  4 15.0     4.44      .41 12.00  76.38  .70     .000
      [I%=25.0:S%= 5.00]
      ------------------
      ADD [0702 + 0304]   0703  3 15.0    15.79     1.41 11.75  63.38  n/a     .000
      ------------------
      CALIB NASHYD        0305  5 15.0    26.45     2.12 12.00  54.87  .50     .000
      [CN=78.0         ]
      [ N= 3.0:Tp=  .21]
      ------------------
 *    CALIB STANDHYD      0306  6 15.0     2.04      .18 12.00  71.85  .66     .000
      [I%=15.0:S%= 6.00]
      ------------------
      CALIB NASHYD        0307  7 15.0     3.71      .12 12.00  21.92  .20     .000
      [CN=82.0         ]
      [ N= 3.0:Tp=  .09]
      ------------------
      ADD [0305 + 0306]   0704  4 15.0    28.49     2.30 12.00  56.09  n/a     .000
      ------------------
      ADD [0704 + 0307]   0705  5 15.0    32.20     2.42 12.00  52.15  n/a     .000
      ------------------
      CALIB NASHYD        0308  8 15.0    13.87     1.06 12.00  51.63  .47     .000
      [CN=78.0         ]
      [ N= 3.0:Tp=  .18]
      ------------------
      ADD [0705 + 0308]   0706  6 15.0    46.07     3.48 12.00  51.99  n/a     .000
      ------------------
      CALIB NASHYD        0309  5 15.0     1.87      .03 12.00  11.45  .10     .000
      [CN=80.0         ]
      [ N= 3.0:Tp=  .07]
      ------------------
      ADD [0706 + 0309]   0707  7 15.0    47.94     3.51 12.00  50.41  n/a     .000
      ------------------
      CALIB NASHYD        0310 10 15.0     8.15      .54 12.00  45.33  .41     .000
      [CN=85.0         ]
      [ N= 3.0:Tp=  .12]
      ------------------
      ADD [0707 + 0310]   0708  8 15.0    56.09     4.05 12.00  49.67  n/a     .000
      ------------------
      CALIB NASHYD        0311  1 15.0     1.23      .03 12.00  18.04  .17     .000
      [CN=85.0         ]
      [ N= 3.0:Tp=  .08]
      ------------------
      ADD [0708 + 0311]   0709  7 15.0    57.32     4.08 12.00  48.99  n/a     .000
      ------------------
 *    CALIB STANDHYD      0312  2 15.0     2.80      .30 12.00  80.13  .73     .000
      [I%=20.0:S%= 2.00]
      ------------------
      ADD [0709 + 0312]   0710 10 15.0    60.12     4.38 12.00  50.44  n/a     .000
      ------------------
      ADD [0703 + 0710]   0711  1 15.0    75.91     5.57 12.00  53.14  n/a     .000
      ------------------
      ADD [0999 + 0711]   0999  2 15.0   196.89    13.87 12.00  49.34  n/a     .000
      ------------------
 ** END OF SIMULATION :    5

***********************************************************************************


 ****************************
 ** SIMULATION NUMBER:   6 **
 ****************************

 W/E COMMAND               HYD ID   DT     AREA    Qpeak Tpeak   R.V. R.C.    Qbase
                                   min      ha      cms   hrs     mm           cms

      START @    .00 hrs
      ------------------
      READ STORM                  15.0
      [ Ptot=121.50 mm ]
      fname :PRSN-SCS.100
      remark:100 YR 24 HR SCS II - PEARSON AIRPORT
      ------------------
      CALIB NASHYD        0101  1 15.0     9.51      .93 12.00  67.47  .56     .000
      [CN=80.0         ]
      [ N= 3.0:Tp=  .21]
      ------------------
      CALIB NASHYD        0102  2 15.0     1.54      .06 12.00  27.75  .23     .000
      [CN=82.0         ]
      [ N= 3.0:Tp=  .09]
      ------------------
      CALIB NASHYD        0103  3 15.0     3.11      .12 12.00  25.50  .21     .000
      [CN=81.0         ]
      [ N= 3.0:Tp=  .09]
      ------------------
      ADD [0101 + 0103]   0500  4 15.0    12.62     1.04 12.00  57.13  n/a     .000
      ------------------
      ADD [0102 + 0500]   0501  5 15.0    14.16     1.11 12.00  53.93  n/a     .000
      ------------------
      RESRVR [ 5 : 0501]  0001  1 15.0    14.16     1.06 12.00  53.93  n/a     .000
      {ST=   .10 ha.m }
      ------------------
      CALIB NASHYD        0104  4 15.0      .61      .00 12.00   1.00  .01     .000
      [CN=80.0         ]
```

```
  [ N= 3.0:Tp=  .05]
  -----------------
  ADD [0104 + 0001]  0502  2 15.0   14.77    1.06 12.00  51.74  n/a    .000
  -----------------
  CHANNEL[ 2 : 0502] 0000  1 15.0   14.77    1.11 12.00  51.74  n/a    .000
  -----------------
  CALIB NASHYD       0105  5 15.0    6.93     .60 12.00  59.72  .49    .000
  [CN=83.0         ]
  [ N= 3.0:Tp=  .15]
  -----------------
  CALIB NASHYD       0106  6 15.0    6.90     .48 12.00  48.11  .40    .000
  [CN=85.0         ]
  [ N= 3.0:Tp=  .12]
  -----------------
  ADD [0105 + 0106]  0503  3 15.0   13.83    1.08 12.00  53.93  n/a    .000
  -----------------
  CALIB NASHYD       0107  7 15.0    4.64     .26 12.00  39.23  .32    .000
  [CN=85.0         ]
  [ N= 3.0:Tp=  .10]
  -----------------
  ADD [0000 + 0107]  0504  4 15.0   19.41    1.38 12.00  48.75  n/a    .000
  -----------------
  ADD [0503 + 0504]  0505  5 15.0   33.24    2.46 12.00  50.90  n/a    .000
  -----------------
  CALIB NASHYD       0108  8 15.0    4.54     .34 12.00  50.78  .42    .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .13]
  -----------------
  ADD [0505 + 0108]  0506  6 15.0   37.78    2.80 12.00  50.89  n/a    .000
  -----------------
  CALIB NASHYD       0109  9 15.0     .48     .00 12.00    .27  .00    .000
  [CN=85.0         ]
  [ N= 3.0:Tp=  .04]
  -----------------
  ADD [0506 + 0109]  0507  7 15.0   38.26    2.80 12.00  50.25  n/a    .000
  -----------------
  CALIB NASHYD       0201  1 15.0   16.72    1.38 12.00  55.91  .46    .000
  [CN=78.0         ]
  [ N= 3.0:Tp=  .16]
  -----------------
  CALIB NASHYD       0202  2 15.0    4.87     .32 12.00  44.21  .36    .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .12]
  -----------------
  ADD [0201 + 0202]  0601  4 15.0   21.59    1.70 12.00  53.27  n/a    .000
  -----------------
  CALIB NASHYD       0203  3 15.0   16.16    1.39 12.00  58.20  .48    .000
  [CN=78.0         ]
  [ N= 3.0:Tp=  .17]
  -----------------
  ADD [0203 + 0601]  0602  1 15.0   37.75    3.09 12.00  55.38  n/a    .000
  -----------------
  CALIB NASHYD       0204  4 15.0   17.96    1.73 12.00  66.23  .55    .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .20]
  -----------------
  CALIB NASHYD       0205  5 15.0    2.00     .07 12.00  22.00  .18    .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .09]
  -----------------
  ADD [0204 + 0205]  0603  3 15.0   19.96    1.79 12.00  61.80  n/a    .000
  -----------------
  ADD [0602 + 0603]  0604  4 15.0   57.71    4.88 12.00  57.60  n/a    .000
  -----------------
  CALIB NASHYD       0206  6 15.0    7.57     .70 12.00  64.08  .53    .000
  [CN=84.0         ]
  [ N= 3.0:Tp=  .16]
  -----------------
  ADD [0604 + 0206]  0605  5 15.0   65.28    5.59 12.00  58.35  n/a    .000
  -----------------
  CHANNEL[ 5 : 0605] 0000  1 15.0   65.28    5.71 12.00  58.35  n/a    .000
  -----------------
* CALIB STANDHYD     0207  9 15.0    3.14     .37 12.00  85.10  .70    .000
  [I%=20.0:S%= 6.00]
  -----------------
  ADD [0605 + 0207]  0606  6 15.0   68.42    5.96 12.00  59.58  n/a    .000
  -----------------
  CALIB NASHYD       0208  8 15.0    1.18     .00 12.00   1.68  .01    .000
  [CN=82.0         ]
  [ N= 3.0:Tp=  .05]
  -----------------
  ADD [0606 + 0208]  0607  4 15.0   69.60    5.96 12.00  58.60  n/a    .000
  -----------------
  CALIB NASHYD       0209  9 15.0    2.42     .04 12.00  11.04  .09    .000
  [CN=86.0         ]
  [ N= 3.0:Tp=  .07]
  -----------------
  ADD [0607 + 0209]  0608  8 15.0   72.02    6.00 12.00  57.00  n/a    .000
  -----------------
* CALIB STANDHYD     0210 10 15.0    2.50     .26 12.00  84.50  .70    .000
  [I%=18.0:S%= 5.00]
  -----------------
* CALIB STANDHYD     0211  1 15.0     .90     .11 12.00  92.15  .76    .000
  [I%=35.0:S%= 4.00]
  -----------------
  ADD [0210 + 0211]  0609  9 15.0    3.40     .37 12.00  86.52  n/a    .000
```

	ADD [0608 + 0609]	0610	10	15.0	75.42	6.37	12.00	58.33	n/a	.000
	CALIB NASHYD [CN=84.0] [N= 3.0:Tp= .09]	0212	2	15.0	2.91	.10	12.00	24.44	.20	.000
*	CALIB STANDHYD [I%=40.0:S%= 6.00]	0213	3	15.0	1.21	.15	12.00	96.24	.79	.000
	ADD [0212 + 0213]	0611	1	15.0	4.12	.26	12.00	45.52	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	79.54	6.62	12.00	57.67	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	0214	4	15.0	.79	.01	12.00	11.93	.10	.000
	ADD [0612 + 0214]	0613	3	15.0	80.33	6.64	12.00	57.22	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	0215	5	15.0	2.39	.13	12.00	36.63	.30	.000
	ADD [0613 + 0215]	0614	4	15.0	82.72	6.77	12.00	56.62	n/a	.000
	ADD [0614 + 0507]	0999	9	15.0	120.98	9.57	12.00	54.61	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	0301	1	15.0	2.96	.08	12.00	17.59	.14	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	0302	2	15.0	6.31	.73	12.00	82.76	.68	.000
	ADD [0301 + 0302]	0701	4	15.0	9.27	.81	12.00	61.95	n/a	.000
*	RESRVR [4 : 0701] {ST= .06 ha.m }	0002	1	15.0	9.27	.86	11.75	61.95	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	0303	3	15.0	2.08	.25	12.00	89.80	.74	.000
	ADD [0002 + 0303]	0702	2	15.0	11.35	1.10	11.75	67.05	n/a	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	0304	4	15.0	4.44	.48	12.00	87.45	.72	.000
	ADD [0702 + 0304]	0703	3	15.0	15.79	1.53	11.75	72.79	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .21]	0305	5	15.0	26.45	2.47	12.00	64.23	.53	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	0306	6	15.0	2.04	.21	12.00	82.75	.68	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .09]	0307	7	15.0	3.71	.14	12.00	25.45	.21	.000
	ADD [0305 + 0306]	0704	4	15.0	28.49	2.68	12.00	65.56	n/a	.000
	ADD [0704 + 0307]	0705	5	15.0	32.20	2.81	12.00	60.94	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .18]	0308	8	15.0	13.87	1.23	12.00	60.44	.50	.000
	ADD [0705 + 0308]	0706	6	15.0	46.07	4.05	12.00	60.79	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .07]	0309	5	15.0	1.87	.04	12.00	13.35	.11	.000
	ADD [0706 + 0309]	0707	7	15.0	47.94	4.08	12.00	58.94	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .12]	0310	10	15.0	8.15	.61	12.00	52.24	.43	.000
	ADD [0707 + 0310]	0708	8	15.0	56.09	4.70	12.00	57.96	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .08]	0311	1	15.0	1.23	.04	12.00	20.80	.17	.000
	ADD [0708 + 0311]	0709	7	15.0	57.32	4.73	12.00	57.17	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 2.00]	0312	2	15.0	2.80	.34	12.00	91.51	.75	.000
	ADD [0709 + 0312]	0710	10	15.0	60.12	5.08	12.00	58.77	n/a	.000
	ADD [0703 + 0710]	0711	1	15.0	75.91	6.53	12.00	61.68	n/a	.000

ADD [0999 + 0711] 0999 2 15.0 196.89 16.10 12.00 57.34 n/a .000

FINISH

===

Post Development Uncontrolled - 24 Hour SCS Storms
High Density Development

```
==============================================================================

      OOO    TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    I N T E R H Y M O
     O   O     T      T    H   H   Y Y   MM MM  O   O  * * * 1989b * * *
     O   O     T      T    HHHHH    Y    M M M  O   O
     O   O     T      T    H   H    Y    M   M  O   O
      OOO      T      T    H   H    Y    M   M   OOO        162   00014

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                  *****  S U M M A R Y   O U T P U T  *****

  Input   filename: INGLE-HI.DAT
  Output  filename: INGLE-HI.OUT
  Summary filename: INGLE-HI.SUM

DATE: 04-12-1999                     TIME: 13:53:58

USER: ________________________________________

COMMENTS: ____________________________________________________________

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

  W/E COMMAND              HYD ID   DT     AREA    Qpeak Tpeak    R.V. R.C.    Qbase
                                    min     ha      cms   hrs      mm           cms

      START @    .00 hrs
      ------------------
      READ STORM                   5.0
      [ Ptot= 25.00 mm ]
      fname :PEARSN4H.25M
      remark:TORONTO PEARSON 25 MM - 4 HR CHICAGO
      ------------------
      CALIB NASHYD        2010  1 15.0    16.95      .15  1.50    2.87  .11     .000
      [CN=78.0         ]
      [ N= 3.0:Tp=  .16]
      ------------------
      CALIB NASHYD        2030  3 15.0    12.83      .11  1.50    2.79  .11     .000
      [CN=78.0         ]
      [ N= 3.0:Tp=  .15]
      ------------------
      CALIB NASHYD        2060  6 15.0     2.93      .05  1.50    4.38  .18     .000
      [CN=84.0         ]
      [ N= 3.0:Tp=  .16]
      ------------------
      ADD [2010 + 2030]   0601  9 15.0    29.78      .25  1.50    2.83  n/a     .000
      ------------------
      ADD [0601 + 2060]   0602  1 15.0    32.71      .30  1.50    2.97  n/a     .000
      ------------------
   *  CALIB STANDHYD      2020  2 15.0     6.01      .15  1.50    8.61  .34     .000
      [I%=19.0:S%= 5.00]
      ------------------
   *  CALIB STANDHYD      2031  3 15.0     3.47      .09  1.50    8.60  .34     .000
      [I%=19.0:S%= 7.00]
      ------------------
      ADD [2020 + 2031]   0603  9 15.0     9.48      .24  1.50    8.61  n/a     .000
      ------------------
   *  CALIB STANDHYD      2040  4 15.0    10.95      .28  1.50    8.61  .34     .000
      [I%=19.0:S%= 6.00]
      ------------------
   *  CALIB STANDHYD      2041  5 15.0     7.25      .19  1.50    8.61  .34     .000
      [I%=19.0:S%= 8.00]
      ------------------
      ADD [2040 + 2041]   0604  8 15.0    18.20      .47  1.50    8.61  n/a     .000
      ------------------
   *  CALIB STANDHYD      2050  5 15.0     2.17      .06  1.50    8.60  .34     .000
      [I%=19.0:S%= 6.00]
      ------------------
      ADD [0604 + 2050]   0605  7 15.0    20.37      .53  1.50    8.61  n/a     .000
      ------------------
      ADD [0605 + 0603]   0606  8 15.0    29.85      .77  1.50    8.61  n/a     .000
      ------------------
   *  CALIB STANDHYD      1010 10 15.0     8.86      .24  1.50    8.61  .34     .000
      [I%=19.0:S%= 6.00]
      ------------------
   *  CALIB STANDHYD      1020  2 15.0     1.96      .07  1.50    9.56  .38     .000
      [I%=25.0:S%= 4.00]
      ------------------
      ADD [1010 + 1020]   0901  7 15.0    10.82      .30  1.50    8.78  n/a     .000
      ------------------
   *  CALIB STANDHYD      1030  3 15.0     3.11      .08  1.50    8.61  .34     .000
      [I%=19.0:S%= 5.00]
      ------------------
      ADD [0901 + 1030]   0902  2 15.0    13.93      .39  1.50    8.74  n/a     .000
      ------------------
      ADD [0602 + 0606]   0999  3 15.0    62.56     1.07  1.50    5.66  n/a     .000
      ------------------
```

	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .05]	1040	4	15.0	.61	.00	1.50	.02	.00	.000
	ADD [1040 + 0902]	0999	5	15.0	14.54	.39	1.50	8.38	n/a	.000
	CHANNEL[5 : 0999]	0000	1	15.0	14.54	.32	1.75	8.37	n/a	.000
	CALIB NASHYD [CN=83.0] [N= 3.0:Tp= .15]	1050	5	15.0	6.82	.10	1.50	3.96	.16	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	1060	6	15.0	2.67	.09	1.50	9.55	.38	.000
	ADD [1050 + 1060]	0999	2	15.0	9.49	.18	1.50	5.54	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	1070	7	15.0	9.73	.27	1.50	8.61	.34	.000
	ADD [0000 + 1070]	0999	4	15.0	24.27	.51	1.50	8.47	n/a	.000
	ADD [0000 + 0999]	0999	4	15.0	24.03	.43	1.50	7.25	n/a	.000
	ADD [1070 + 0999]	0999	5	15.0	33.76	.69	1.50	7.64	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .13]	1080	8	15.0	4.54	.04	1.50	2.86	.11	.000
	ADD [0999 + 1080]	0506	6	15.0	38.30	.74	1.50	7.08	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .04]	1090	9	15.0	.48	.00	1.50	.00	.00	.000
	ADD [0506 + 1090]	0507	1	15.0	38.78	.74	1.50	6.99	n/a	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	2061	6	15.0	4.24	.09	1.50	7.65	.31	.000
	ADD [2061 + 0999]	0999	2	15.0	66.80	1.16	1.50	5.79	n/a	.000
	CHANNEL[2 : 0999]	0000	3	15.0	66.80	.91	1.50	5.79	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	2070	7	15.0	3.26	.09	1.50	8.54	.34	.000
	ADD [0000 + 2070]	0606	6	15.0	70.06	1.00	1.50	5.92	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	2080	8	15.0	1.14	.00	1.50	.08	.00	.000
	ADD [0606 + 2080]	0607	7	15.0	71.20	1.00	1.50	5.82	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	2090	9	15.0	2.37	.01	1.50	.87	.03	.000
	ADD [0607 + 2090]	0608	8	15.0	73.57	1.01	1.50	5.66	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.06	1.50	8.19	.33	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.04	1.50	11.59	.46	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.10	1.50	9.03	n/a	.000
	ADD [0608 + 0609]	0610	10	15.0	76.82	1.10	1.50	5.81	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2120	2	15.0	2.58	.07	1.50	8.61	.34	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.05	1.50	10.92	.44	.000
	ADD [2120 + 2130]	0611	9	15.0	3.92	.13	1.50	9.40	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	80.74	1.23	1.50	5.98	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.00	1.50	.74	.03	.000
	ADD [0612 + 2140]	0613	3	15.0	81.50	1.23	1.50	5.93	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	2150	5	15.0	2.40	.02	1.50	1.88	.08	.000
	ADD [0613 + 2150]	0614	4	15.0	83.90	1.25	1.50	5.81	n/a	.000
	ADD [0507 + 0614]	0614	10	15.0	122.68	1.98	1.50	6.19	n/a	.000

```
     CALIB NASHYD         3010  1 15.0      3.04      .01  1.50     .99  .04     .000
     [CN=80.0        ]
     [ N= 3.0:Tp=  .08]
     ------------------
 *   CALIB STANDHYD       3020  2 15.0      6.46      .14  1.50    7.53  .30     .000
     [I%=15.0:S%= 6.00]
     ------------------
     ADD [3010 + 3020]    0701  4 15.0      9.50      .15  1.50    5.44  n/a     .000
     ------------------
     RESRVR [ 4 : 0701]   0002  1 15.0      9.50      .08  1.75    5.43  n/a     .000
     {ST=    .01 ha.m }
     ------------------
 *   CALIB STANDHYD       3030  3 15.0      2.02      .08  1.50   10.57  .42     .000
     [I%=30.0:S%= 6.00]
     ------------------
     ADD [0002 + 3030]    0702  2 15.0     11.52      .13  1.50    6.33  n/a     .000
     ------------------
 *   CALIB STANDHYD       3040  4 15.0      4.76      .15  1.50    9.56  .38     .000
     [I%=25.0:S%= 5.00]
     ------------------
     ADD [0702 + 3040]    0703  3 15.0     16.28      .28  1.50    7.28  n/a     .000
     ------------------
     CALIB NASHYD         3050  5 15.0     26.45      .21  1.50    3.30  .13     .000
     [CN=78.0        ]
     [ N= 3.0:Tp=  .21]
     ------------------
 *   CALIB STANDHYD       3060  6 15.0      2.27      .05  1.50    7.52  .30     .000
     [I%=15.0:S%= 6.00]
     ------------------
     CALIB NASHYD         3070  7 15.0      3.77      .02  1.50    1.58  .06     .000
     [CN=82.0        ]
     [ N= 3.0:Tp=  .09]
     ------------------
     ADD [3050 + 3060]    0704  4 15.0     28.72      .26  1.50    3.63  n/a     .000
     ------------------
     ADD [0704 + 3070]    0705  5 15.0     32.49      .28  1.50    3.39  n/a     .000
     ------------------
     CALIB NASHYD         3080  8 15.0     13.69      .12  1.50    3.10  .12     .000
     [CN=78.0        ]
     [ N= 3.0:Tp=  .18]
     ------------------
 *   CALIB STANDHYD       3081  1 15.0      3.25      .09  1.50    8.61  .34     .000
     [I%=19.0:S%= 6.00]
     ------------------
     ADD [3080 + 3081]    0706  6 15.0     16.94      .21  1.50    4.16  n/a     .000
     ------------------
     ADD [0705 + 0706]    0707  7 15.0     49.43      .49  1.50    3.65  n/a     .000
     ------------------
     CALIB NASHYD         3090  9 15.0      1.04      .00  1.50     .43  .02     .000
     [CN=80.0        ]
     [ N= 3.0:Tp=  .07]
     ------------------
     CALIB NASHYD         3100  6 15.0      4.68      .04  1.50    2.10  .08     .000
     [CN=85.0        ]
     [ N= 3.0:Tp=  .09]
     ------------------
 *   CALIB STANDHYD       3101  1 15.0      3.07      .08  1.50    8.61  .34     .000
     [I%=19.0:S%= 7.00]
     ------------------
 *   CALIB STANDHYD       3102  2 15.0      1.30      .03  1.50    8.18  .33     .000
     [I%=18.0:S%= 4.00]
     ------------------
     ADD [3100 + 3101]    0708  8 15.0      7.75      .13  1.50    4.68  n/a     .000
     ------------------
     ADD [0708 + 3090]    0709  1 15.0      8.79      .13  1.50    4.18  n/a     .000
     ------------------
     ADD [0709 + 3102]    0710  6 15.0     10.09      .16  1.50    4.69  n/a     .000
     ------------------
     ADD [0707 + 0710]    0711  8 15.0     59.52      .64  1.50    3.83  n/a     .000
     ------------------
     CALIB NASHYD         3110  1 15.0      1.21      .01  1.50    1.53  .06     .000
     [CN=85.0        ]
     [ N= 3.0:Tp=  .08]
     ------------------
     ADD [0711 + 3110]    0712  9 15.0     60.73      .65  1.50    3.78  n/a     .000
     ------------------
 *   CALIB STANDHYD       3120  2 15.0      2.96      .09  1.50    9.92  .40     .000
     [I%=20.0:S%= 2.00]
     ------------------
     ADD [0712 + 3120]    0713  1 15.0     63.69      .74  1.50    4.07  n/a     .000
     ------------------
     ADD [0703 + 0713]    0714  2 15.0     79.97     1.02  1.50    4.72  n/a     .000
     ------------------
     ADD [0614 + 0714]    0714  3 15.0    202.65     3.00  1.50    5.61  n/a     .000
     ------------------
 ** END OF SIMULATION :    1

****************************************************************************************


 ****************************
 ** SIMULATION NUMBER:    2 **
 ****************************
```

W/E	COMMAND	HYD	ID	DT min	AREA ha	Qpeak cms	Tpeak hrs	R.V. mm	R.C.	Qbase cms
	START @ .00 hrs									
	READ STORM [Ptot= 47.40 mm] fname :PRSN-SCS.002 remark:2 YR 24 HR SCS II - PEARSON AIRPORT			15.0						
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .16]	2010	1	15.0	16.95	.30	12.00	11.56	.24	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .15]	2030	3	15.0	12.83	.22	12.00	11.23	.24	.000
	CALIB NASHYD [CN=84.0] [N= 3.0:Tp= .16]	2060	6	15.0	2.93	.07	12.00	15.22	.32	.000
	ADD [2010 + 2030]	0601	9	15.0	29.78	.53	12.00	11.42	n/a	.000
	ADD [0601 + 2060]	0602	1	15.0	32.71	.60	12.00	11.76	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 5.00]	2020	2	15.0	6.01	.16	12.00	23.22	.49	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2031	3	15.0	3.47	.09	12.00	23.21	.49	.000
	ADD [2020 + 2031]	0603	9	15.0	9.48	.25	12.00	23.22	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	2040	4	15.0	10.95	.25	12.00	23.22	.49	.000
*	CALIB STANDHYD [I%=19.0:S%= 8.00]	2041	5	15.0	7.25	.20	12.00	23.22	.49	.000
	ADD [2040 + 2041]	0604	8	15.0	18.20	.44	12.00	23.22	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	2050	5	15.0	2.17	.06	12.00	23.21	.49	.000
	ADD [0604 + 2050]	0605	7	15.0	20.37	.50	12.00	23.22	n/a	.000
	ADD [0605 + 0603]	0606	8	15.0	29.85	.75	12.00	23.22	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	1010	10	15.0	8.86	.24	12.00	23.22	.49	.000
*	CALIB STANDHYD [I%=25.0:S%= 4.00]	1020	2	15.0	1.96	.06	12.00	24.46	.52	.000
	ADD [1010 + 1020]	0901	7	15.0	10.82	.29	12.00	23.44	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 5.00]	1030	3	15.0	3.11	.08	12.00	23.22	.49	.000
	ADD [0901 + 1030]	0902	2	15.0	13.93	.38	12.00	23.39	n/a	.000
	ADD [0602 + 0606]	0999	3	15.0	62.56	1.35	12.00	17.23	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .05]	1040	4	15.0	.61	.00	12.00	.12	.00	.000
	ADD [1040 + 0902]	0999	5	15.0	14.54	.38	12.00	22.42	n/a	.000
	CHANNEL[5 : 0999]	0000	1	15.0	14.54	.36	12.00	22.41	n/a	.000
	CALIB NASHYD [CN=83.0] [N= 3.0:Tp= .15]	1050	5	15.0	6.82	.15	12.00	13.94	.29	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	1060	6	15.0	2.67	.06	12.00	24.46	.52	.000
	ADD [1050 + 1060]	0999	2	15.0	9.49	.21	12.00	16.90	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	1070	7	15.0	9.73	.28	12.00	23.22	.49	.000
	ADD [0000 + 1070]	0999	4	15.0	24.27	.64	12.00	22.74	n/a	.000
	ADD [0000 + 0999]	0999	4	15.0	24.03	.58	12.00	20.24	n/a	.000
	ADD [1070 + 0999]	0999	5	15.0	33.76	.86	12.00	21.10	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .13]	1080	8	15.0	4.54	.08	12.00	10.98	.23	.000
	ADD [0999 + 1080]	0506	6	15.0	38.30	.93	12.00	19.90	n/a	.000

	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .04]	1090	9	15.0	.48	.00	12.00	.00	.00	.000
	ADD [0506 + 1090]	0507	1	15.0	38.78	.93	12.00	19.65	n/a	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	2061	6	15.0	4.24	.11	12.00	21.67	.46	.000
	ADD [2061 + 0999]	0999	2	15.0	66.80	1.46	12.00	17.51	n/a	.000
	CHANNEL[2 : 0999]	0000	3	15.0	66.80	1.45	12.00	17.51	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	2070	7	15.0	3.26	.09	12.00	22.91	.48	.000
	ADD [0000 + 2070]	0606	6	15.0	70.06	1.54	12.00	17.76	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	2080	8	15.0	1.14	.00	12.00	.25	.01	.000
	ADD [0606 + 2080]	0607	7	15.0	71.20	1.54	12.00	17.48	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	2090	9	15.0	2.37	.01	12.00	2.81	.06	.000
	ADD [0607 + 2090]	0608	8	15.0	73.57	1.55	12.00	17.01	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.05	12.00	22.44	.47	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.03	12.00	27.54	.58	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.08	12.00	23.70	n/a	.000
	ADD [0608 + 0609]	0610	10	15.0	76.82	1.63	12.00	17.29	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2120	2	15.0	2.58	.07	12.00	23.22	.49	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.04	12.00	26.78	.57	.000
	ADD [2120 + 2130]	0611	9	15.0	3.92	.12	12.00	24.44	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	80.74	1.75	12.00	17.64	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.00	12.00	2.70	.06	.000
	ADD [0612 + 2140]	0613	3	15.0	81.50	1.75	12.00	17.50	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	2150	5	15.0	2.40	.03	12.00	7.58	.16	.000
	ADD [0613 + 2150]	0614	4	15.0	83.90	1.78	12.00	17.21	n/a	.000
	ADD [0507 + 0614]	0614	10	15.0	122.68	2.71	12.00	17.98	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	3010	1	15.0	3.04	.02	12.00	3.80	.08	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3020	2	15.0	6.46	.17	12.00	21.37	.45	.000
	ADD [3010 + 3020]	0701	4	15.0	9.50	.19	12.00	15.75	n/a	.000
	RESRVR [4 : 0701] {ST= .02 ha.m }	0002	1	15.0	9.50	.14	12.25	15.75	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	3030	3	15.0	2.02	.06	12.00	26.00	.55	.000
	ADD [0002 + 3030]	0702	2	15.0	11.52	.19	12.00	17.55	n/a	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	3040	4	15.0	4.76	.11	12.00	24.46	.52	.000
	ADD [0702 + 3040]	0703	3	15.0	16.28	.31	12.00	19.57	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .21]	3050	5	15.0	26.45	.52	12.00	13.29	.28	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3060	6	15.0	2.27	.05	12.00	21.36	.45	.000
	CALIB NASHYD [CN=82.0]	3070	7	15.0	3.77	.03	12.00	5.76	.12	.000

W/E	COMMAND	HYD	ID	DT min	AREA ha	Qpeak cms	Tpeak hrs	R.V. mm	R.C.	Qbase cms
	[N= 3.0:Tp= .09]									
	ADD [3050 + 3060]	0704	4	15.0	28.72	.57	12.00	13.92	n/a	.000
	ADD [0704 + 3070]	0705	5	15.0	32.49	.60	12.00	12.98	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .18]	3080	8	15.0	13.69	.26	12.00	12.50	.26	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	3081	1	15.0	3.25	.09	12.00	23.22	.49	.000
	ADD [3080 + 3081]	0706	6	15.0	16.94	.35	12.00	14.56	n/a	.000
	ADD [0705 + 0706]	0707	7	15.0	49.43	.96	12.00	13.52	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .07]	3090	9	15.0	1.04	.00	12.00	1.63	.03	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .09]	3100	6	15.0	4.68	.05	12.00	6.85	.14	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	3101	1	15.0	3.07	.09	12.00	23.22	.49	.000
*	CALIB STANDHYD [I%=18.0:S%= 4.00]	3102	2	15.0	1.30	.03	12.00	22.43	.47	.000
	ADD [3100 + 3101]	0708	8	15.0	7.75	.14	12.00	13.34	n/a	.000
	ADD [0708 + 3090]	0709	1	15.0	8.79	.14	12.00	11.95	n/a	.000
	ADD [0709 + 3102]	0710	6	15.0	10.09	.17	12.00	13.30	n/a	.000
	ADD [0707 + 0710]	0711	8	15.0	59.52	1.13	12.00	13.48	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .08]	3110	1	15.0	1.21	.01	12.00	5.10	.11	.000
	ADD [0711 + 3110]	0712	9	15.0	60.73	1.14	12.00	13.31	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 2.00]	3120	2	15.0	2.96	.09	12.00	25.95	.55	.000
	ADD [0712 + 3120]	0713	1	15.0	63.69	1.23	12.00	13.90	n/a	.000
	ADD [0703 + 0713]	0714	2	15.0	79.97	1.53	12.00	15.05	n/a	.000
	ADD [0614 + 0714]	0714	3	15.0	202.65	4.25	12.00	16.83	n/a	.000

** END OF SIMULATION : 2

****

****
** SIMULATION NUMBER: 3 **
****

W/E	COMMAND	HYD	ID	DT min	AREA ha	Qpeak cms	Tpeak hrs	R.V. mm	R.C.	Qbase cms
	START @ .00 hrs									
	READ STORM [Ptot= 67.20 mm] fname :PRSN-SCS.005 remark:5 YR 24 HR SCS II - PEARSON AIRPORT			15.0						
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .16]	2010	1	15.0	16.95	.57	12.00	21.82	.32	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .15]	2030	3	15.0	12.83	.42	12.00	21.20	.32	.000
	CALIB NASHYD [CN=84.0] [N= 3.0:Tp= .16]	2060	6	15.0	2.93	.12	12.00	27.07	.40	.000
	ADD [2010 + 2030]	0601	9	15.0	29.78	.98	12.00	21.55	n/a	.000
	ADD [0601 + 2060]	0602	1	15.0	32.71	1.10	12.00	22.05	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 5.00]	2020	2	15.0	6.01	.28	12.00	38.60	.57	.000
*	CALIB STANDHYD	2031	3	15.0	3.47	.16	12.00	38.60	.57	.000

```
     [I%=19.0:S%= 7.00]
     ------------------
     ADD [2020 + 2031]  0603  9 15.0   9.48    .44 12.00  38.60  n/a   .000
     ------------------
  *  CALIB STANDHYD     2040  4 15.0  10.95    .49 12.00  38.60  .57   .000
     [I%=19.0:S%= 6.00]
     ------------------
  *  CALIB STANDHYD     2041  5 15.0   7.25    .34 12.00  38.60  .57   .000
     [I%=19.0:S%= 8.00]
     ------------------
     ADD [2040 + 2041]  0604  8 15.0  18.20    .83 12.00  38.60  n/a   .000
     ------------------
  *  CALIB STANDHYD     2050  5 15.0   2.17    .10 12.00  38.60  .57   .000
     [I%=19.0:S%= 6.00]
     ------------------
     ADD [0604 + 2050]  0605  7 15.0  20.37    .94 12.00  38.60  n/a   .000
     ------------------
     ADD [0605 + 0603]  0606  8 15.0  29.85   1.38 12.00  38.60  n/a   .000
     ------------------
  *  CALIB STANDHYD     1010 10 15.0   8.86    .42 12.00  38.60  .57   .000
     [I%=19.0:S%= 6.00]
     ------------------
  *  CALIB STANDHYD     1020  2 15.0   1.96    .10 12.00  39.95  .59   .000
     [I%=25.0:S%= 4.00]
     ------------------
     ADD [1010 + 1020]  0901  7 15.0  10.82    .52 12.00  38.84  n/a   .000
     ------------------
  *  CALIB STANDHYD     1030  3 15.0   3.11    .15 12.00  38.60  .57   .000
     [I%=19.0:S%= 5.00]
     ------------------
     ADD [0901 + 1030]  0902  2 15.0  13.93    .67 12.00  38.79  n/a   .000
     ------------------
     ADD [0602 + 0606]  0999  3 15.0  62.56   2.48 12.00  29.94  n/a   .000
     ------------------
     CALIB NASHYD       1040  4 15.0    .61    .00 12.00    .32  .00   .000
     [CN=80.0         ]
     [ N= 3.0:Tp=  .05]
     ------------------
     ADD [1040 + 0902]  0999  5 15.0  14.54    .67 12.00  37.17  n/a   .000
     ------------------
     CHANNEL[ 5 : 0999] 0000  1 15.0  14.54    .65 12.00  37.17  n/a   .000
     ------------------
     CALIB NASHYD       1050  5 15.0   6.82    .26 12.00  24.96  .37   .000
     [CN=83.0         ]
     [ N= 3.0:Tp=  .15]
     ------------------
  *  CALIB STANDHYD     1060  6 15.0   2.67    .12 12.00  39.95  .59   .000
     [I%=25.0:S%= 5.00]
     ------------------
     ADD [1050 + 1060]  0999  2 15.0   9.49    .38 12.00  29.18  n/a   .000
     ------------------
  *  CALIB STANDHYD     1070  7 15.0   9.73    .52 12.00  38.60  .57   .000
     [I%=19.0:S%= 7.00]
     ------------------
     ADD [0000 + 1070]  0999  4 15.0  24.27   1.17 12.00  37.74  n/a   .000
     ------------------
     ADD [0000 + 0999]  0999  4 15.0  24.03   1.03 12.00  34.01  n/a   .000
     ------------------
     ADD [1070 + 0999]  0999  5 15.0  33.76   1.55 12.00  35.34  n/a   .000
     ------------------
     CALIB NASHYD       1080  8 15.0   4.54    .14 12.00  20.33  .30   .000
     [CN=80.0         ]
     [ N= 3.0:Tp=  .13]
     ------------------
     ADD [0999 + 1080]  0506  6 15.0  38.30   1.69 12.00  33.56  n/a   .000
     ------------------
     CALIB NASHYD       1090  9 15.0    .48    .00 12.00    .11  .00   .000
     [CN=85.0         ]
     [ N= 3.0:Tp=  .04]
     ------------------
     ADD [0506 + 1090]  0507  1 15.0  38.78   1.69 12.00  33.14  n/a   .000
     ------------------
  *  CALIB STANDHYD     2061  6 15.0   4.24    .19 12.00  36.67  .55   .000
     [I%=15.0:S%= 6.00]
     ------------------
     ADD [2061 + 0999]  0999  2 15.0  66.80   2.68 12.00  30.37  n/a   .000
     ------------------
     CHANNEL[ 2 : 0999] 0000  3 15.0  66.80   2.68 12.00  30.37  n/a   .000
     ------------------
  *  CALIB STANDHYD     2070  7 15.0   3.26    .17 12.00  38.10  .57   .000
     [I%=20.0:S%= 6.00]
     ------------------
     ADD [0000 + 2070]  0606  6 15.0  70.06   2.85 12.00  30.73  n/a   .000
     ------------------
     CALIB NASHYD       2080  8 15.0   1.14    .00 12.00    .56  .01   .000
     [CN=82.0         ]
     [ N= 3.0:Tp=  .05]
     ------------------
     ADD [0606 + 2080]  0607  7 15.0  71.20   2.85 12.00  30.25  n/a   .000
     ------------------
     CALIB NASHYD       2090  9 15.0   2.37    .02 12.00   4.82  .07   .000
     [CN=86.0         ]
     [ N= 3.0:Tp=  .07]
     ------------------
     ADD [0607 + 2090]  0608  8 15.0  73.57   2.87 12.00  29.43  n/a   .000
     ------------------
```

*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.10	12.00	37.57	.56	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.05	12.00	43.66	.65	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.15	12.00	39.07	n/a	.000
	ADD [0608 + 0609]	0610	10	15.0	76.82	3.02	12.00	29.84	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2120	2	15.0	2.58	.14	12.00	38.60	.57	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.07	12.00	42.91	.64	.000
	ADD [2120 + 2130]	0611	9	15.0	3.92	.21	12.00	40.07	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	80.74	3.23	12.00	30.33	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.01	12.00	4.91	.07	.000
	ADD [0612 + 2140]	0613	3	15.0	81.50	3.24	12.00	30.10	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	2150	5	15.0	2.40	.05	12.00	14.29	.21	.000
	ADD [0613 + 2150]	0614	4	15.0	83.90	3.29	12.00	29.64	n/a	.000
	ADD [0507 + 0614]	0614	10	15.0	122.68	4.98	12.00	30.75	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	3010	1	15.0	3.04	.03	12.00	7.04	.10	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3020	2	15.0	6.46	.32	12.00	36.25	.54	.000
	ADD [3010 + 3020]	0701	4	15.0	9.50	.36	12.00	26.90	n/a	.000
	RESRVR [4 : 0701] {ST= .05 ha.m }	0002	1	15.0	9.50	.24	12.25	26.90	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	3030	3	15.0	2.02	.11	12.00	41.80	.62	.000
	ADD [0002 + 3030]	0702	2	15.0	11.52	.33	12.00	29.51	n/a	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	3040	4	15.0	4.76	.22	12.00	39.95	.59	.000
	ADD [0702 + 3040]	0703	3	15.0	16.28	.54	12:00	32.57	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .21]	3050	5	15.0	26.45	.98	12.00	25.07	.37	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3060	6	15.0	2.27	.09	12.00	36.24	.54	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .09]	3070	7	15.0	3.77	.06	12.00	10.46	.16	.000
	ADD [3050 + 3060]	0704	4	15.0	28.72	1.08	12.00	25.95	n/a	.000
	ADD [0704 + 3070]	0705	5	15.0	32.49	1.14	12.00	24.15	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .18]	3080	8	15.0	13.69	.49	12.00	23.59	.35	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	3081	1	15.0	3.25	.17	12.00	38.60	.57	.000
	ADD [3080 + 3081]	0706	6	15.0	16.94	.66	12.00	26.47	n/a	.000
	ADD [0705 + 0706]	0707	7	15.0	49.43	1.80	12.00	24.95	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .07]	3090	9	15.0	1.04	.01	12.00	3.09	.05	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .09]	3100	6	15.0	4.68	.08	12.00	11.94	.18	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	3101	1	15.0	3.07	.15	12.00	38.60	.57	.000
*	CALIB STANDHYD [I%=18.0:S%= 4.00]	3102	2	15.0	1.30	.06	12.00	37.56	.56	.000

```
     ADD [3100 + 3101]  0708  8 15.0    7.75     .23 12.00  22.50  n/a    .000
     ------------------
     ADD [0708 + 3090]  0709  1 15.0    8.79     .24 12.00  20.21  n/a    .000
     ------------------
     ADD [0709 + 3102]  0710  6 15.0   10.09     .30 12.00  22.44  n/a    .000
     ------------------
     ADD [0707 + 0710]  0711  8 15.0   59.52    2.10 12.00  24.52  n/a    .000
     ------------------
     CALIB NASHYD       3110  1 15.0    1.21     .02 12.00   8.94  .13    .000
     [CN=85.0        ]
     [ N= 3.0:Tp=  .08]
     ------------------
     ADD [0711 + 3110]  0712  9 15.0   60.73    2.11 12.00  24.21  n/a    .000
     ------------------
  *  CALIB STANDHYD     3120  2 15.0    2.96     .16 12.00  42.32  .63    .000
     [I%=20.0:S%= 2.00]
     ------------------
     ADD [0712 + 3120]  0713  1 15.0   63.69    2.27 12.00  25.05  n/a    .000
     ------------------
     ADD [0703 + 0713]  0714  2 15.0   79.97    2.81 12.00  26.58  n/a    .000
     ------------------
     ADD [0614 + 0714]  0714  3 15.0  202.65    7.79 12.00  29.11  n/a    .000
     ------------------
  ** END OF SIMULATION :   3

*******************************************************************************


  ****************************
  ** SIMULATION NUMBER:   4 **
  ****************************

  W/E COMMAND             HYD ID   DT     AREA   Qpeak Tpeak   R.V. R.C.   Qbase
                                   min     ha     cms   hrs     mm          cms

     START @    .00 hrs
     ------------------
     READ STORM                   15.0
     [ Ptot= 80.40 mm ]
     fname :PRSN-SCS.010
     remark:10 YR 24 HR SCS II - PEARSON AIRPORT
     ------------------
     CALIB NASHYD       2010  1 15.0   16.95     .76 12.00  29.49  .37    .000
     [CN=78.0        ]
     [ N= 3.0:Tp=  .16]
     ------------------
     CALIB NASHYD       2030  3 15.0   12.83     .56 12.00  28.64  .36    .000
     [CN=78.0        ]
     [ N= 3.0:Tp=  .15]
     ------------------
     CALIB NASHYD       2060  6 15.0    2.93     .16 12.00  35.62  .44    .000
     [CN=84.0        ]
     [ N= 3.0:Tp=  .16]
     ------------------
     ADD [2010 + 2030]  0601  9 15.0   29.78    1.32 12.00  29.12  n/a    .000
     ------------------
     ADD [0601 + 2060]  0602  1 15.0   32.71    1.47 12.00  29.71  n/a    .000
     ------------------
  *  CALIB STANDHYD     2020  2 15.0    6.01     .37 12.00  49.59  .62    .000
     [I%=19.0:S%= 5.00]
     ------------------
  *  CALIB STANDHYD     2031  3 15.0    3.47     .21 12.00  49.59  .62    .000
     [I%=19.0:S%= 7.00]
     ------------------
     ADD [2020 + 2031]  0603  9 15.0    9.48     .58 12.00  49.59  n/a    .000
     ------------------
  *  CALIB STANDHYD     2040  4 15.0   10.95     .65 12.00  49.59  .62    .000
     [I%=19.0:S%= 6.00]
     ------------------
  *  CALIB STANDHYD     2041  5 15.0    7.25     .45 12.00  49.59  .62    .000
     [I%=19.0:S%= 8.00]
     ------------------
     ADD [2040 + 2041]  0604  8 15.0   18.20    1.10 12.00  49.59  n/a    .000
     ------------------
  *  CALIB STANDHYD     2050  5 15.0    2.17     .15 12.00  49.59  .62    .000
     [I%=19.0:S%= 6.00]
     ------------------
     ADD [0604 + 2050]  0605  7 15.0   20.37    1.25 12.00  49.59  n/a    .000
     ------------------
     ADD [0605 + 0603]  0606  8 15.0   29.85    1.83 12.00  49.59  n/a    .000
     ------------------
  *  CALIB STANDHYD     1010 10 15.0    8.86     .55 12.00  49.59  .62    .000
     [I%=19.0:S%= 6.00]
     ------------------
  *  CALIB STANDHYD     1020  2 15.0    1.96     .13 12.00  50.99  .63    .000
     [I%=25.0:S%= 4.00]
     ------------------
     ADD [1010 + 1020]  0901  7 15.0   10.82     .68 12.00  49.85  n/a    .000
     ------------------
  *  CALIB STANDHYD     1030  3 15.0    3.11     .20 12.00  49.59  .62    .000
     [I%=19.0:S%= 5.00]
     ------------------
     ADD [0901 + 1030]  0902  2 15.0   13.93     .88 12.00  49.79  n/a    .000
```

	ADD [0602 + 0606]	0999	3	15.0	62.56	3.30	12.00	39.19	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .05]	1040	4	15.0	.61	.00	12.00	.44	.01	.000
	ADD [1040 + 0902]	0999	5	15.0	14.54	.88	12.00	47.72	n/a	.000
	CHANNEL[5 : 0999]	0000	1	15.0	14.54	.86	12.00	47.72	n/a	.000
	CALIB NASHYD [CN=83.0] [N= 3.0:Tp= .15]	1050	5	15.0	6.82	.34	12.00	32.96	.41	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	1060	6	15.0	2.67	.16	12.00	50.99	.63	.000
	ADD [1050 + 1060]	0999	2	15.0	9.49	.50	12.00	38.03	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	1070	7	15.0	9.73	.67	12.00	49.59	.62	.000
	ADD [0000 + 1070]	0999	4	15.0	24.27	1.52	12.00	48.47	n/a	.000
	ADD [0000 + 0999]	0999	4	15.0	24.03	1.35	12.00	43.89	n/a	.000
	ADD [1070 + 0999]	0999	5	15.0	33.76	2.02	12.00	45.53	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .13]	1080	8	15.0	4.54	.19	12.00	27.24	.34	.000
	ADD [0999 + 1080]	0506	6	15.0	38.30	2.21	12.00	43.37	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .04]	1090	9	15.0	.48	.00	12.00	.14	.00	.000
	ADD [0506 + 1090]	0507	1	15.0	38.78	2.21	12.00	42.83	n/a	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	2061	6	15.0	4.24	.26	12.00	47.47	.59	.000
	ADD [2061 + 0999]	0999	2	15.0	66.80	3.56	12.00	39.72	n/a	.000
	CHANNEL[2 : 0999]	0000	3	15.0	66.80	3.58	12.00	39.72	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	2070	7	15.0	3.26	.22	12.00	48.98	.61	.000
	ADD [0000 + 2070]	0606	6	15.0	70.06	3.80	12.00	40.15	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	2080	8	15.0	1.14	.00	12.00	.82	.01	.000
	ADD [0606 + 2080]	0607	7	15.0	71.20	3.80	12.00	39.52	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	2090	9	15.0	2.37	.02	12.00	6.27	.08	.000
	ADD [0607 + 2090]	0608	8	15.0	73.57	3.82	12.00	38.45	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.14	12.00	48.42	.60	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.06	12.00	55.01	.68	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.20	12.00	50.04	n/a	.000
	ADD [0608 + 0609]	0610	10	15.0	76.82	4.02	12.00	38.94	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2120	2	15.0	2.58	.18	12.00	49.59	.62	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.10	12.00	54.28	.68	.000
	ADD [2120 + 2130]	0611	9	15.0	3.92	.27	12.00	51.19	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	80.74	4.29	12.00	39.54	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.01	12.00	6.51	.08	.000
	ADD [0612 + 2140]	0613	3	15.0	81.50	4.30	12.00	39.23	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	2150	5	15.0	2.40	.07	12.00	19.32	.24	.000
	ADD [0613 + 2150]	0614	4	15.0	83.90	4.37	12.00	38.66	n/a	.000

```
      ------------------
      ADD [0507 + 0614]   0614 10 15.0  122.68    6.59 12.00  39.98  n/a    .000
      ------------------
      CALIB NASHYD        3010  1 15.0    3.04     .04 12.00   9.44  .12    .000
      [CN=80.0        ]
      [ N= 3.0:Tp=  .08]
      ------------------
   *  CALIB STANDHYD      3020  2 15.0    6.46     .42 12.00  46.97  .58    .000
      [I%=15.0:S%= 6.00]
      ------------------
      ADD [3010 + 3020]   0701  4 15.0    9.50     .47 12.00  34.96  n/a    .000
      ------------------
   *  RESRVR [ 4 : 0701]  0002  1 15.0    9.50     .58 12.00  34.96  n/a    .000
      {ST=   .06 ha.m }
      ------------------
   *  CALIB STANDHYD      3030  3 15.0    2.02     .14 12.00  53.00  .66    .000
      [I%=30.0:S%= 6.00]
      ------------------
      ADD [0002 + 3030]   0702  2 15.0   11.52     .73 12.00  38.12  n/a    .000
      ------------------
   *  CALIB STANDHYD      3040  4 15.0    4.76     .28 12.00  50.99  .63    .000
      [I%=25.0:S%= 5.00]
      ------------------
      ADD [0702 + 3040]   0703  3 15.0   16.28    1.01 12.00  41.88  n/a    .000
      ------------------
      CALIB NASHYD        3050  5 15.0   26.45    1.32 12.00  33.88  .42    .000
      [CN=78.0        ]
      [ N= 3.0:Tp=  .21]
      ------------------
   *  CALIB STANDHYD      3060  6 15.0    2.27     .13 12.00  46.97  .58    .000
      [I%=15.0:S%= 6.00]
      ------------------
      CALIB NASHYD        3070  7 15.0    3.77     .08 12.00  13.89  .17    .000
      [CN=82.0        ]
      [ N= 3.0:Tp=  .09]
      ------------------
      ADD [3050 + 3060]   0704  4 15.0   28.72    1.45 12.00  34.91  n/a    .000
      ------------------
      ADD [0704 + 3070]   0705  5 15.0   32.49    1.53 12.00  32.47  n/a    .000
      ------------------
      CALIB NASHYD        3080  8 15.0   13.69     .66 12.00  31.87  .40    .000
      [CN=78.0        ]
      [ N= 3.0:Tp=  .18]
      ------------------
   *  CALIB STANDHYD      3081  1 15.0    3.25     .22 12.00  49.59  .62    .000
      [I%=19.0:S%= 6.00]
      ------------------
      ADD [3080 + 3081]   0706  6 15.0   16.94     .88 12.00  35.27  n/a    .000
      ------------------
      ADD [0705 + 0706]   0707  7 15.0   49.43    2.41 12.00  33.43  n/a    .000
      ------------------
      CALIB NASHYD        3090  9 15.0    1.04     .01 12.00   4.14  .05    .000
      [CN=80.0        ]
      [ N= 3.0:Tp=  .07]
      ------------------
      CALIB NASHYD        3100  6 15.0    4.68     .11 12.00  15.59  .19    .000
      [CN=85.0        ]
      [ N= 3.0:Tp=  .09]
      ------------------
   *  CALIB STANDHYD      3101  1 15.0    3.07     .21 12.00  49.59  .62    .000
      [I%=19.0:S%= 7.00]
      ------------------
   *  CALIB STANDHYD      3102  2 15.0    1.30     .08 12.00  48.42  .60    .000
      [I%=18.0:S%= 4.00]
      ------------------
      ADD [3100 + 3101]   0708  8 15.0    7.75     .32 12.00  29.06  n/a    .000
      ------------------
      ADD [0708 + 3090]   0709  1 15.0    8.79     .32 12.00  26.11  n/a    .000
      ------------------
      ADD [0709 + 3102]   0710  6 15.0   10.09     .40 12.00  28.98  n/a    .000
      ------------------
      ADD [0707 + 0710]   0711  8 15.0   59.52    2.81 12.00  32.68  n/a    .000
      ------------------
      CALIB NASHYD        3110  1 15.0    1.21     .02 12.00  11.69  .15    .000
      [CN=85.0        ]
      [ N= 3.0:Tp=  .08]
      ------------------
      ADD [0711 + 3110]   0712  9 15.0   60.73    2.83 12.00  32.26  n/a    .000
      ------------------
   *  CALIB STANDHYD      3120  2 15.0    2.96     .20 12.00  53.85  .67    .000
      [I%=20.0:S%= 2.00]
      ------------------
      ADD [0712 + 3120]   0713  1 15.0   63.69    3.03 12.00  33.26  n/a    .000
      ------------------
      ADD [0703 + 0713]   0714  2 15.0   79.97    4.04 12.00  35.02  n/a    .000
      ------------------
      ADD [0614 + 0714]   0714  3 15.0  202.65   10.63 12.00  38.02  n/a    .000
      ------------------
 ** END OF SIMULATION :   4

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

```
 ****************************
 ** SIMULATION NUMBER:   5 **
 ****************************

 W/E COMMAND              HYD ID   DT     AREA    Qpeak Tpeak   R.V.  R.C.    Qbase
                                   min     ha      cms   hrs     mm            cms
     START @    .00 hrs
     ------------------
     READ STORM                   15.0
     [ Ptot= 97.00 mm ]
     fname :PRSN-SCS.025
     remark:10 YR 24 HR SCS II - PEARSON AIRPORT
     ------------------
     CALIB NASHYD         2010  1 15.0    16.95    1.01 12.00  39.78  .41     .000
     [CN=78.0         ]
     [ N= 3.0:Tp=  .16]
     ------------------
     CALIB NASHYD         2030  3 15.0    12.83     .75 12.00  38.64  .40     .000
     [CN=78.0         ]
     [ N= 3.0:Tp=  .15]
     ------------------
     CALIB NASHYD         2060  6 15.0     2.93     .20 12.00  46.85  .48     .000
     [CN=84.0         ]
     [ N= 3.0:Tp=  .16]
     ------------------
     ADD [2010 + 2030]    0601  9 15.0    29.78    1.76 12.00  39.29  n/a     .000
     ------------------
     ADD [0601 + 2060]    0602  1 15.0    32.71    1.96 12.00  39.97  n/a     .000
     ------------------
 *   CALIB STANDHYD       2020  2 15.0     6.01     .49 12.00  63.98  .66     .000
     [I%=19.0:S%= 5.00]
     ------------------
 *   CALIB STANDHYD       2031  3 15.0     3.47     .28 12.00  63.98  .66     .000
     [I%=19.0:S%= 7.00]
     ------------------
     ADD [2020 + 2031]    0603  9 15.0     9.48     .76 12.00  63.98  n/a     .000
     ------------------
 *   CALIB STANDHYD       2040  4 15.0    10.95     .86 12.00  63.98  .66     .000
     [I%=19.0:S%= 6.00]
     ------------------
 *   CALIB STANDHYD       2041  5 15.0     7.25     .63 12.00  63.98  .66     .000
     [I%=19.0:S%= 8.00]
     ------------------
     ADD [2040 + 2041]    0604  8 15.0    18.20    1.49 12.00  63.98  n/a     .000
     ------------------
 *   CALIB STANDHYD       2050  5 15.0     2.17     .19 12.00  63.98  .66     .000
     [I%=19.0:S%= 6.00]
     ------------------
     ADD [0604 + 2050]    0605  7 15.0    20.37    1.67 12.00  63.98  n/a     .000
     ------------------
     ADD [0605 + 0603]    0606  8 15.0    29.85    2.44 12.00  63.98  n/a     .000
     ------------------
 *   CALIB STANDHYD       1010 10 15.0     8.86     .72 12.00  63.98  .66     .000
     [I%=19.0:S%= 6.00]
     ------------------
 *   CALIB STANDHYD       1020  2 15.0     1.96     .17 12.00  65.41  .67     .000
     [I%=25.0:S%= 4.00]
     ------------------
     ADD [1010 + 1020]    0901  7 15.0    10.82     .89 12.00  64.24  n/a     .000
     ------------------
 *   CALIB STANDHYD       1030  3 15.0     3.11     .27 12.00  63.98  .66     .000
     [I%=19.0:S%= 5.00]
     ------------------
     ADD [0901 + 1030]    0902  2 15.0    13.93    1.17 12.00  64.18  n/a     .000
     ------------------
     ADD [0602 + 0606]    0999  3 15.0    62.56    4.40 12.00  51.42  n/a     .000
     ------------------
     CALIB NASHYD         1040  4 15.0      .61     .00 12.00    .66  .01     .000
     [CN=80.0         ]
     [ N= 3.0:Tp=  .05]
     ------------------
     ADD [1040 + 0902]    0999  5 15.0    14.54    1.17 12.00  61.52  n/a     .000
     ------------------
     CHANNEL[ 5 : 0999]   0000  1 15.0    14.54    1.15 12.00  61.51  n/a     .000
     ------------------
     CALIB NASHYD         1050  5 15.0     6.82     .44 12.00  43.50  .45     .000
     [CN=83.0         ]
     [ N= 3.0:Tp=  .15]
     ------------------
 *   CALIB STANDHYD       1060  6 15.0     2.67     .21 12.00  65.41  .67     .000
     [I%=25.0:S%= 5.00]
     ------------------
     ADD [1050 + 1060]    0999  2 15.0     9.49     .65 12.00  49.66  n/a     .000
     ------------------
 *   CALIB STANDHYD       1070  7 15.0     9.73     .86 12.00  63.98  .66     .000
     [I%=19.0:S%= 7.00]
     ------------------
     ADD [0000 + 1070]    0999  4 15.0    24.27    2.01 12.00  62.50  n/a     .000
     ------------------
     ADD [0000 + 0999]    0999  4 15.0    24.03    1.80 12.00  56.83  n/a     .000
     ------------------
     ADD [1070 + 0999]    0999  5 15.0    33.76    2.66 12.00  58.89  n/a     .000
     ------------------
     CALIB NASHYD         1080  8 15.0     4.54     .25 12.00  36.45  .38     .000
     [CN=80.0         ]
```

```
      [ N= 3.0:Tp=  .13]
      ------------------
      ADD [0999 + 1080]  0506  6 15.0   38.30    2.91 12.00  56.23  n/a    .000
      ------------------
      CALIB NASHYD       1090  9 15.0     .48     .00 12.00    .19  .00    .000
      [CN=85.0         ]
      [ N= 3.0:Tp=  .04]
      ------------------
      ADD [0506 + 1090]  0507  1 15.0   38.78    2.91 12.00  55.54  n/a    .000
      ------------------
    * CALIB STANDHYD     2061  6 15.0    4.24     .36 12.00  61.65  .64    .000
      [I%=15.0:S%= 6.00]
      ------------------
      ADD [2061 + 0999]  0999  2 15.0   66.80    4.76 12.00  52.07  n/a    .000
      ------------------
      CHANNEL[ 2 : 0999] 0000  3 15.0   66.80    4.83 12.00  52.07  n/a    .000
      ------------------
    * CALIB STANDHYD     2070  7 15.0    3.26     .28 12.00  63.24  .65    .000
      [I%=20.0:S%= 6.00]
      ------------------
      ADD [0000 + 2070]  0606  6 15.0   70.06    5.11 12.00  52.59  n/a    .000
      ------------------
      CALIB NASHYD       2080  8 15.0    1.14     .00 12.00   1.14  .01    .000
      [CN=82.0         ]
      [ N= 3.0:Tp=  .05]
      ------------------
      ADD [0606 + 2080]  0607  7 15.0   71.20    5.11 12.00  51.77  n/a    .000
      ------------------
      CALIB NASHYD       2090  9 15.0    2.37     .03 12.00   8.16  .08    .000
      [CN=86.0         ]
      [ N= 3.0:Tp=  .07]
      ------------------
      ADD [0607 + 2090]  0608  8 15.0   73.57    5.14 12.00  50.36  n/a    .000
      ------------------
    * CALIB STANDHYD     2100 10 15.0    2.45     .19 12.00  62.66  .65    .000
      [I%=18.0:S%= 5.00]
      ------------------
    * CALIB STANDHYD     2110  7 15.0     .80     .07 12.00  69.75  .72    .000
      [I%=35.0:S%= 4.00]
      ------------------
      ADD [2100 + 2110]  0609  9 15.0    3.25     .26 12.00  64.41  n/a    .000
      ------------------
      ADD [0608 + 0609]  0610 10 15.0   76.82    5.40 12.00  50.96  n/a    .000
      ------------------
    * CALIB STANDHYD     2120  2 15.0    2.58     .23 12.00  63.98  .66    .000
      [I%=19.0:S%= 7.00]
      ------------------
    * CALIB STANDHYD     2130  3 15.0    1.34     .12 12.00  69.06  .71    .000
      [I%=30.0:S%= 6.00]
      ------------------
      ADD [2120 + 2130]  0611  9 15.0    3.92     .35 12.00  65.71  n/a    .000
      ------------------
      ADD [0611 + 0610]  0612  2 15.0   80.74    5.75 12.00  51.68  n/a    .000
      ------------------
      CALIB NASHYD       2140  4 15.0     .76     .01 12.00   8.64  .09    .000
      [CN=82.0         ]
      [ N= 3.0:Tp=  .07]
      ------------------
      ADD [0612 + 2140]  0613  3 15.0   81.50    5.76 12.00  51.27  n/a    .000
      ------------------
      CALIB NASHYD       2150  5 15.0    2.40     .10 12.00  26.07  .27    .000
      [CN=78.0         ]
      [ N= 3.0:Tp=  .11]
      ------------------
      ADD [0613 + 2150]  0614  4 15.0   83.90    5.86 12.00  50.55  n/a    .000
      ------------------
      ADD [0507 + 0614]  0614 10 15.0  122.68    8.77 12.00  52.13  n/a    .000
      ------------------
      CALIB NASHYD       3010  1 15.0    3.04     .06 12.00  12.63  .13    .000
      [CN=80.0         ]
      [ N= 3.0:Tp=  .08]
      ------------------
    * CALIB STANDHYD     3020  2 15.0    6.46     .55 12.00  61.08  .63    .000
      [I%=15.0:S%= 6.00]
      ------------------
      ADD [3010 + 3020]  0701  4 15.0    9.50     .61 12.00  45.57  n/a    .000
      ------------------
    * RESRVR [ 4 : 0701] 0002  1 15.0    9.50     .64 11.75  45.57  n/a    .000
      {ST=    .05 ha.m }
      ------------------
    * CALIB STANDHYD     3030  3 15.0    2.02     .18 12.00  67.58  .70    .000
      [I%=30.0:S%= 6.00]
      ------------------
      ADD [0002 + 3030]  0702  2 15.0   11.52     .81 11.75  49.43  n/a    .000
      ------------------
    * CALIB STANDHYD     3040  4 15.0    4.76     .37 12.00  65.41  .67    .000
      [I%=25.0:S%= 5.00]
      ------------------
      ADD [0702 + 3040]  0703  3 15.0   16.28    1.14 11.75  54.10  n/a    .000
      ------------------
      CALIB NASHYD       3050  5 15.0   26.45    1.77 12.00  45.70  .47    .000
      [CN=78.0         ]
      [ N= 3.0:Tp=  .21]
      ------------------
    * CALIB STANDHYD     3060  6 15.0    2.27     .17 12.00  61.07  .63    .000
      [I%=15.0:S%= 6.00]
```

```
      ------------------
      CALIB NASHYD         3070  7 15.0     3.77      .10 12.00  18.43  .19     .000
      [CN=82.0          ]
      [ N= 3.0:Tp=   .09]
      ------------------
      ADD [3050 + 3060]    0704  4 15.0    28.72     1.94 12.00  46.92  n/a     .000
      ------------------
      ADD [0704 + 3070]    0705  5 15.0    32.49     2.04 12.00  43.61  n/a     .000
      ------------------
      CALIB NASHYD         3080  8 15.0    13.69      .88 12.00  43.00  .44     .000
      [CN=78.0          ]
      [ N= 3.0:Tp=   .18]
      ------------------
  *   CALIB STANDHYD       3081  1 15.0     3.25      .29 12.00  63.98  .66     .000
      [I%=19.0:S%= 6.00]
      ------------------
      ADD [3080 + 3081]    0706  6 15.0    16.94     1.16 12.00  47.02  n/a     .000
      ------------------
      ADD [0705 + 0706]    0707  7 15.0    49.43     3.21 12.00  44.78  n/a     .000
      ------------------
      CALIB NASHYD         3090  9 15.0     1.04      .01 12.00   5.53  .06     .000
      [CN=80.0          ]
      [ N= 3.0:Tp=   .07]
      ------------------
      CALIB NASHYD         3100  6 15.0     4.68      .14 12.00  20.36  .21     .000
      [CN=85.0          ]
      [ N= 3.0:Tp=   .09]
      ------------------
  *   CALIB STANDHYD       3101  1 15.0     3.07      .27 12.00  63.98  .66     .000
      [I%=19.0:S%= 7.00]
      ------------------
  *   CALIB STANDHYD       3102  2 15.0     1.30      .10 12.00  62.66  .65     .000
      [I%=18.0:S%= 4.00]
      ------------------
      ADD [3100 + 3101]    0708  8 15.0     7.75      .41 12.00  37.64  n/a     .000
      ------------------
      ADD [0708 + 3090]    0709  1 15.0     8.79      .42 12.00  33.84  n/a     .000
      ------------------
      ADD [0709 + 3102]    0710  6 15.0    10.09      .52 12.00  37.55  n/a     .000
      ------------------
      ADD [0707 + 0710]    0711  8 15.0    59.52     3.73 12.00  43.56  n/a     .000
      ------------------
      CALIB NASHYD         3110  1 15.0     1.21      .03 12.00  15.30  .16     .000
      [CN=85.0          ]
      [ N= 3.0:Tp=   .08]
      ------------------
      ADD [0711 + 3110]    0712  9 15.0    60.73     3.76 12.00  42.99  n/a     .000
      ------------------
  *   CALIB STANDHYD       3120  2 15.0     2.96      .27 12.00  68.82  .71     .000
      [I%=20.0:S%= 2.00]
      ------------------
      ADD [0712 + 3120]    0713  1 15.0    63.69     4.03 12.00  44.19  n/a     .000
      ------------------
      ADD [0703 + 0713]    0714  2 15.0    79.97     5.14 12.00  46.21  n/a     .000
      ------------------
      ADD [0614 + 0714]    0714  3 15.0   202.65    13.91 12.00  49.79  n/a     .000
      ------------------
 ** END OF SIMULATION :    5

*****************************************************************************


 ****************************
 ** SIMULATION NUMBER:   6 **
 ****************************

 W/E COMMAND              HYD ID   DT     AREA    Qpeak Tpeak   R.V.  R.C.   Qbase
                                  min      ha      cms   hrs     mm           cms

      START @    .00 hrs
      ------------------
      READ STORM                  15.0
      [ Ptot=109.30 mm ]
      fname :PRSN-SCS.050
      remark:10 YR 24 HR SCS II - PEARSON AIRPORT
      ------------------
      CALIB NASHYD         2010  1 15.0    16.95     1.21 12.00  47.76  .44     .000
      [CN=78.0          ]
      [ N= 3.0:Tp=   .16]
      ------------------
      CALIB NASHYD         2030  3 15.0    12.83      .89 12.00  46.40  .42     .000
      [CN=78.0          ]
      [ N= 3.0:Tp=   .15]
      ------------------
      CALIB NASHYD         2060  6 15.0     2.93      .24 12.00  55.42  .51     .000
      [CN=84.0          ]
      [ N= 3.0:Tp=   .16]
      ------------------
      ADD [2010 + 2030]    0601  9 15.0    29.78     2.10 12.00  47.17  n/a     .000
      ------------------
      ADD [0601 + 2060]    0602  1 15.0    32.71     2.33 12.00  47.91  n/a     .000
      ------------------
  *   CALIB STANDHYD       2020  2 15.0     6.01      .61 12.00  74.93  .69     .000
```

	[I%=19.0:S%= 5.00]									
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2031	3	15.0	3.47	.33	12.00	74.93	.69	.000
	ADD [2020 + 2031]	0603	9	15.0	9.48	.93	12.00	74.93	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	2040	4	15.0	10.95	1.02	12.00	74.94	.69	.000
*	CALIB STANDHYD [I%=19.0:S%= 8.00]	2041	5	15.0	7.25	.74	12.00	74.94	.69	.000
	ADD [2040 + 2041]	0604	8	15.0	18.20	1.76	12.00	74.94	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	2050	5	15.0	2.17	.22	12.00	74.93	.69	.000
	ADD [0604 + 2050]	0605	7	15.0	20.37	1.98	12.00	74.94	n/a	.000
	ADD [0605 + 0603]	0606	8	15.0	29.85	2.91	12.00	74.94	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	1010	10	15.0	8.86	.90	12.00	74.94	.69	.000
*	CALIB STANDHYD [I%=25.0:S%= 4.00]	1020	2	15.0	1.96	.20	12.00	76.38	.70	.000
	ADD [1010 + 1020]	0901	7	15.0	10.82	1.10	12.00	75.20	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 5.00]	1030	3	15.0	3.11	.32	12.00	74.93	.69	.000
	ADD [0901 + 1030]	0902	2	15.0	13.93	1.42	12.00	75.14	n/a	.000
	ADD [0602 + 0606]	0999	3	15.0	62.56	5.24	12.00	60.81	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .05]	1040	4	15.0	.61	.00	12.00	.80	.01	.000
	ADD [1040 + 0902]	0999	5	15.0	14.54	1.42	12.00	72.02	n/a	.000
	CHANNEL[5 : 0999]	0000	1	15.0	14.54	1.41	12.00	72.02	n/a	.000
	CALIB NASHYD [CN=83.0] [N= 3.0:Tp= .15]	1050	5	15.0	6.82	.52	12.00	51.56	.47	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	1060	6	15.0	2.67	.25	12.00	76.38	.70	.000
	ADD [1050 + 1060]	0999	2	15.0	9.49	.76	12.00	58.54	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	1070	7	15.0	9.73	1.01	12.00	74.94	.69	.000
	ADD [0000 + 1070]	0999	4	15.0	24.27	2.42	12.00	73.19	n/a	.000
	ADD [0000 + 0999]	0999	4	15.0	24.03	2.17	12.00	66.70	n/a	.000
	ADD [1070 + 0999]	0999	5	15.0	33.76	3.18	12.00	69.07	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .13]	1080	8	15.0	4.54	.30	12.00	43.55	.40	.000
	ADD [0999 + 1080]	0506	6	15.0	38.30	3.48	12.00	66.05	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .04]	1090	9	15.0	.48	.00	12.00	.23	.00	.000
	ADD [0506 + 1090]	0507	1	15.0	38.78	3.48	12.00	65.23	n/a	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	2061	6	15.0	4.24	.42	12.00	72.48	.66	.000
	ADD [2061 + 0999]	0999	2	15.0	66.80	5.67	12.00	61.55	n/a	.000
	CHANNEL[2 : 0999]	0000	3	15.0	66.80	5.78	12.00	61.55	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	2070	7	15.0	3.26	.33	12.00	74.12	.68	.000
	ADD [0000 + 2070]	0606	6	15.0	70.06	6.11	12.00	62.13	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	2080	8	15.0	1.14	.00	12.00	1.39	.01	.000
	ADD [0606 + 2080]	0607	7	15.0	71.20	6.12	12.00	61.16	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	2090	9	15.0	2.37	.03	12.00	9.60	.09	.000

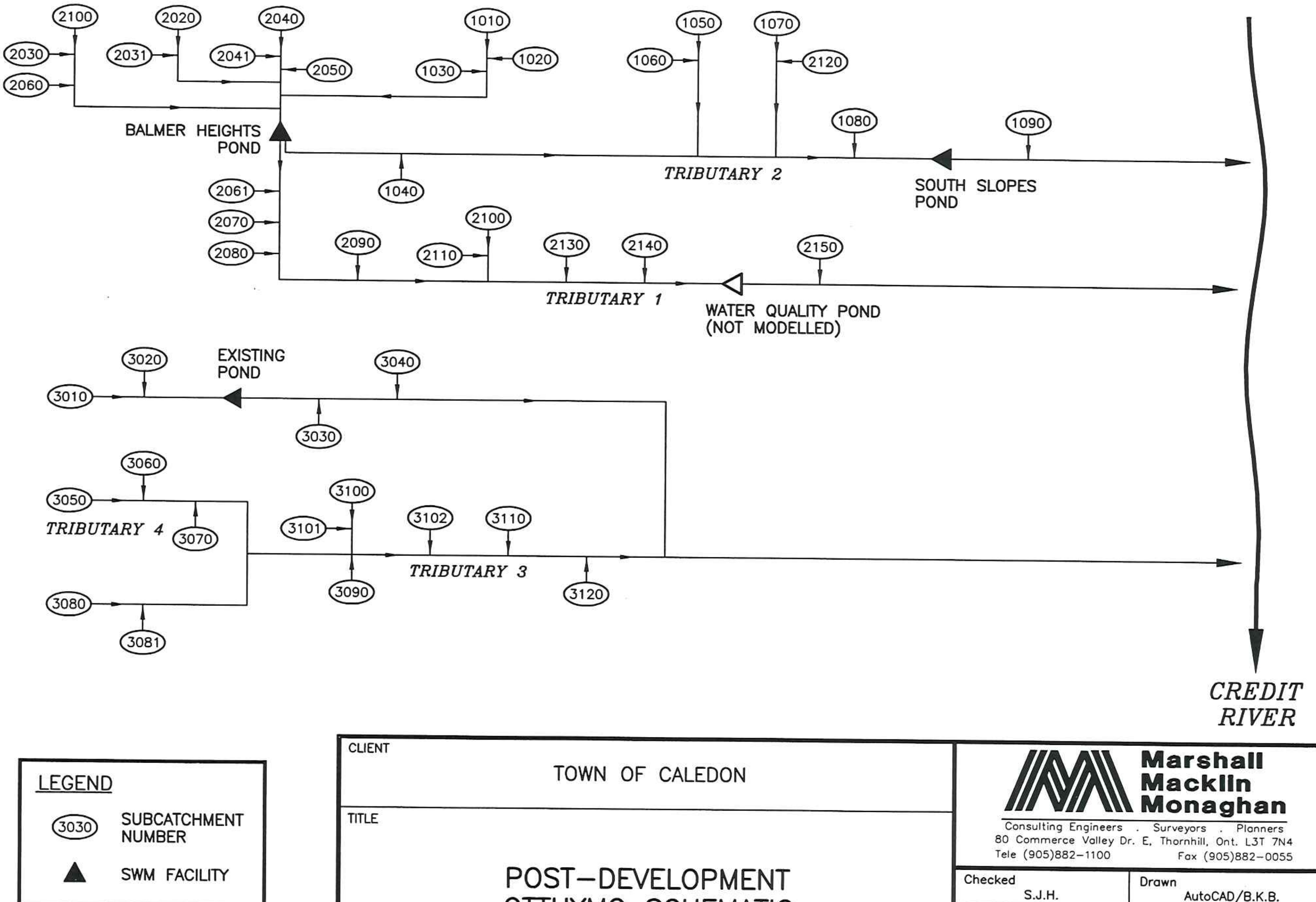
	ADD [0607 + 2090]	0608	8	15.0	73.57	6.15	12.00	59.50	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.22	12.00	73.53	.67	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.09	12.00	80.91	.74	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.31	12.00	75.34	n/a	.000
	ADD [0608 + 0609]	0610	10	15.0	76.82	6.46	12.00	60.17	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2120	2	15.0	2.58	.27	12.00	74.93	.69	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.14	12.00	80.25	.73	.000
	ADD [2120 + 2130]	0611	9	15.0	3.92	.41	12.00	76.75	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	80.74	6.87	12.00	60.97	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.01	12.00	10.28	.09	.000
	ADD [0612 + 2140]	0613	3	15.0	81.50	6.88	12.00	60.50	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	2150	5	15.0	2.40	.11	12.00	31.29	.29	.000
	ADD [0613 + 2150]	0614	4	15.0	83.90	6.99	12.00	59.67	n/a	.000
	ADD [0507 + 0614]	0614	10	15.0	122.68	10.47	12.00	61.43	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	3010	1	15.0	3.04	.07	12.00	15.09	.14	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3020	2	15.0	6.46	.65	12.00	71.86	.66	.000
	ADD [3010 + 3020]	0701	4	15.0	9.50	.72	12.00	53.69	n/a	.000
*	RESRVR [4 : 0701] {ST= .06 ha.m }	0002	1	15.0	9.50	.84	11.75	53.69	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	3030	3	15.0	2.02	.21	12.00	78.65	.72	.000
	ADD [0002 + 3030]	0702	2	15.0	11.52	1.05	11.75	58.07	n/a	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	3040	4	15.0	4.76	.44	12.00	76.38	.70	.000
	ADD [0702 + 3040]	0703	3	15.0	16.28	1.44	11.75	63.42	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .21]	3050	5	15.0	26.45	2.12	12.00	54.87	.50	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3060	6	15.0	2.27	.20	12.00	71.85	.66	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .09]	3070	7	15.0	3.77	.12	12.00	21.92	.20	.000
	ADD [3050 + 3060]	0704	4	15.0	28.72	2.32	12.00	56.21	n/a	.000
	ADD [0704 + 3070]	0705	5	15.0	32.49	2.44	12.00	52.23	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .18]	3080	8	15.0	13.69	1.05	12.00	51.63	.47	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	3081	1	15.0	3.25	.34	12.00	74.93	.69	.000
	ADD [3080 + 3081]	0706	6	15.0	16.94	1.38	12.00	56.10	n/a	.000
	ADD [0705 + 0706]	0707	7	15.0	49.43	3.82	12.00	53.56	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .07]	3090	9	15.0	1.04	.01	12.00	6.61	.06	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .09]	3100	6	15.0	4.68	.16	12.00	23.98	.22	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	3101	1	15.0	3.07	.32	12.00	74.94	.69	.000

```
 *   CALIB STANDHYD      3102  2 15.0     1.30      .12 12.00  73.52  .67     .000
     [I%=18.0:S%= 4.00]
     ------------------
     ADD [3100 + 3101]   0708  8 15.0     7.75      .48 12.00  44.17  n/a     .000
     ------------------
     ADD [0708 + 3090]   0709  1 15.0     8.79      .49 12.00  39.72  n/a     .000
     ------------------
     ADD [0709 + 3102]   0710  6 15.0    10.09      .61 12.00  44.08  n/a     .000
     ------------------
     ADD [0707 + 0710]   0711  8 15.0    59.52     4.43 12.00  51.95  n/a     .000
     ------------------
     CALIB NASHYD        3110  1 15.0     1.21      .03 12.00  18.04  .17     .000
     [CN=85.0         ]
     [ N= 3.0:Tp=  .08]
     ------------------
     ADD [0711 + 3110]   0712  9 15.0    60.73     4.46 12.00  51.28  n/a     .000
     ------------------
 *   CALIB STANDHYD      3120  2 15.0     2.96      .32 12.00  80.13  .73     .000
     [I%=20.0:S%= 2.00]
     ------------------
     ADD [0712 + 3120]   0713  1 15.0    63.69     4.78 12.00  52.62  n/a     .000
     ------------------
     ADD [0703 + 0713]   0714  2 15.0    79.97     6.02 12.00  54.82  n/a     .000
     ------------------
     ADD [0614 + 0714]   0714  3 15.0   202.65    16.50 12.00  58.82  n/a     .000
     ------------------
 ** END OF SIMULATION :    6

*******************************************************************************


 ****************************
 ** SIMULATION NUMBER:    7 **
 ****************************

 W/E COMMAND               HYD ID   DT     AREA    Qpeak Tpeak   R.V. R.C.    Qbase
                                   min      ha      cms   hrs     mm           cms

     START @    .00 hrs
     ------------------
     READ STORM                    15.0
     [ Ptot=121.50 mm ]
     fname :PRSN-SCS.100
     remark:100 YR 24 HR SCS II - PEARSON AIRPORT
     ------------------
     CALIB NASHYD        2010  1 15.0    16.95     1.40 12.00  55.91  .46     .000
     [CN=78.0         ]
     [ N= 3.0:Tp=  .16]
     ------------------
     CALIB NASHYD        2030  3 15.0    12.83     1.03 12.00  54.31  .45     .000
     [CN=78.0         ]
     [ N= 3.0:Tp=  .15]
     ------------------
     CALIB NASHYD        2060  6 15.0     2.93      .27 12.00  64.08  .53     .000
     [CN=84.0         ]
     [ N= 3.0:Tp=  .16]
     ------------------
     ADD [2010 + 2030]   0601  9 15.0    29.78     2.44 12.00  55.22  n/a     .000
     ------------------
     ADD [0601 + 2060]   0602  1 15.0    32.71     2.71 12.00  56.02  n/a     .000
     ------------------
 *   CALIB STANDHYD      2020  2 15.0     6.01      .70 12.00  85.99  .71     .000
     [I%=19.0:S%= 5.00]
     ------------------
 *   CALIB STANDHYD      2031  3 15.0     3.47      .38 12.00  85.99  .71     .000
     [I%=19.0:S%= 7.00]
     ------------------
     ADD [2020 + 2031]   0603  9 15.0     9.48     1.08 12.00  85.99  n/a     .000
     ------------------
 *   CALIB STANDHYD      2040  4 15.0    10.95     1.18 12.00  85.99  .71     .000
     [I%=19.0:S%= 6.00]
     ------------------
 *   CALIB STANDHYD      2041  5 15.0     7.25      .85 12.00  85.99  .71     .000
     [I%=19.0:S%= 8.00]
     ------------------
     ADD [2040 + 2041]   0604  8 15.0    18.20     2.03 12.00  85.99  n/a     .000
     ------------------
 *   CALIB STANDHYD      2050  5 15.0     2.17      .25 12.00  85.99  .71     .000
     [I%=19.0:S%= 6.00]
     ------------------
     ADD [0604 + 2050]   0605  7 15.0    20.37     2.28 12.00  85.99  n/a     .000
     ------------------
     ADD [0605 + 0603]   0606  8 15.0    29.85     3.36 12.00  85.99  n/a     .000
     ------------------
 *   CALIB STANDHYD      1010 10 15.0     8.86     1.03 12.00  85.99  .71     .000
     [I%=19.0:S%= 6.00]
     ------------------
 *   CALIB STANDHYD      1020  2 15.0     1.96      .23 12.00  87.45  .72     .000
     [I%=25.0:S%= 4.00]
     ------------------
     ADD [1010 + 1020]   0901  7 15.0    10.82     1.26 12.00  86.26  n/a     .000
     ------------------
 *   CALIB STANDHYD      1030  3 15.0     3.11      .36 12.00  85.99  .71     .000
```

	[I%=19.0:S%= 5.00]										
	ADD [0901 + 1030]	0902	2	15.0	13.93	1.63	12.00	86.20	n/a	.000
	ADD [0602 + 0606]	0999	3	15.0	62.56	6.07	12.00	70.32	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .05]	1040	4	15.0	.61	.00	12.00	1.00	.01	.000
	ADD [1040 + 0902]	0999	5	15.0	14.54	1.63	12.00	82.62	n/a	.000
	CHANNEL[5 : 0999]	0000	1	15.0	14.54	1.62	12.00	82.62	n/a	.000
	CALIB NASHYD [CN=83.0] [N= 3.0:Tp= .15]	1050	5	15.0	6.82	.59	12.00	59.72	.49	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	1060	6	15.0	2.67	.29	12.00	87.45	.72	.000
	ADD [1050 + 1060]	0999	2	15.0	9.49	.88	12.00	67.52	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	1070	7	15.0	9.73	1.16	12.00	85.99	.71	.000
	ADD [0000 + 1070]	0999	4	15.0	24.27	2.78	12.00	83.97	n/a	.000
	ADD [0000 + 0999]	0999	4	15.0	24.03	2.50	12.00	76.66	n/a	.000
	ADD [1070 + 0999]	0999	5	15.0	33.76	3.66	12.00	79.35	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .13]	1080	8	15.0	4.54	.34	12.00	50.78	.42	.000
	ADD [0999 + 1080]	0506	6	15.0	38.30	4.00	12.00	75.96	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .04]	1090	9	15.0	.48	.00	12.00	.27	.00	.000
	ADD [0506 + 1090]	0507	1	15.0	38.78	4.00	12.00	75.03	n/a	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	2061	6	15.0	4.24	.49	12.00	83.43	.69	.000
	ADD [2061 + 0999]	0999	2	15.0	66.80	6.55	12.00	71.15	n/a	.000
	CHANNEL[2 : 0999]	0000	3	15.0	66.80	6.70	12.00	71.15	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	2070	7	15.0	3.26	.38	12.00	85.10	.70	.000
	ADD [0000 + 2070]	0606	6	15.0	70.06	7.08	12.00	71.80	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	2080	8	15.0	1.14	.00	12.00	1.68	.01	.000
	ADD [0606 + 2080]	0607	7	15.0	71.20	7.09	12.00	70.68	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	2090	9	15.0	2.37	.04	12.00	11.04	.09	.000
	ADD [0607 + 2090]	0608	8	15.0	73.57	7.13	12.00	68.76	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.26	12.00	84.50	.70	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.10	12.00	92.15	.76	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.35	12.00	86.38	n/a	.000
	ADD [0608 + 0609]	0610	10	15.0	76.82	7.48	12.00	69.50	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2120	2	15.0	2.58	.31	12.00	85.99	.71	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.16	12.00	91.51	.75	.000
	ADD [2120 + 2130]	0611	9	15.0	3.92	.47	12.00	87.88	n/a	.000
	ADD [0611 + 0610]	0612	2	15.0	80.74	7.95	12.00	70.39	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.01	12.00	11.93	.10	.000
	ADD [0612 + 2140]	0613	3	15.0	81.50	7.96	12.00	69.85	n/a	.000
	CALIB NASHYD [CN=78.0]	2150	5	15.0	2.40	.13	12.00	36.63	.30	.000

```
      [ N= 3.0:Tp=  .11]
      ------------------
      ADD [0613 + 2150]  0614  4 15.0   83.90    8.10 12.00  68.90  n/a   .000
      ------------------
      ADD [0507 + 0614]  0614 10 15.0  122.68   12.10 12.00  70.84  n/a   .000
      ------------------
      CALIB NASHYD       3010  1 15.0    3.04     .08 12.00  17.59  .14   .000
      [CN=80.0        ]
      [ N= 3.0:Tp=  .08]
      ------------------
   *  CALIB STANDHYD     3020  2 15.0    6.46     .75 12.00  82.76  .68   .000
      [I%=15.0:S%= 6.00]
      ------------------
      ADD [3010 + 3020]  0701  4 15.0    9.50     .82 12.00  61.91  n/a   .000
      ------------------
   *  RESRVR [ 4 : 0701] 0002  1 15.0    9.50     .86 11.75  61.90  n/a   .000
      {ST=   .06 ha.m }
      ------------------
   *  CALIB STANDHYD     3030  3 15.0    2.02     .24 12.00  89.80  .74   .000
      [I%=30.0:S%= 6.00]
      ------------------
      ADD [0002 + 3030]  0702  2 15.0   11.52    1.10 11.75  66.80  n/a   .000
      ------------------
   *  CALIB STANDHYD     3040  4 15.0    4.76     .51 12.00  87.45  .72   .000
      [I%=25.0:S%= 5.00]
      ------------------
      ADD [0702 + 3040]  0703  3 15.0   16.28    1.55 11.75  72.83  n/a   .000
      ------------------
      CALIB NASHYD       3050  5 15.0   26.45    2.47 12.00  64.23  .53   .000
      [CN=78.0        ]
      [ N= 3.0:Tp=  .21]
      ------------------
   *  CALIB STANDHYD     3060  6 15.0    2.27     .23 12.00  82.75  .68   .000
      [I%=15.0:S%= 6.00]
      ------------------
      CALIB NASHYD       3070  7 15.0    3.77     .14 12.00  25.45  .21   .000
      [CN=82.0        ]
      [ N= 3.0:Tp=  .09]
      ------------------
      ADD [3050 + 3060]  0704  4 15.0   28.72    2.70 12.00  65.70  n/a   .000
      ------------------
      ADD [0704 + 3070]  0705  5 15.0   32.49    2.84 12.00  61.03  n/a   .000
      ------------------
      CALIB NASHYD       3080  8 15.0   13.69    1.22 12.00  60.44  .50   .000
      [CN=78.0        ]
      [ N= 3.0:Tp=  .18]
      ------------------
   *  CALIB STANDHYD     3081  1 15.0    3.25     .39 12.00  85.99  .71   .000
      [I%=19.0:S%= 6.00]
      ------------------
      ADD [3080 + 3081]  0706  6 15.0   16.94    1.60 12.00  65.34  n/a   .000
      ------------------
      ADD [0705 + 0706]  0707  7 15.0   49.43    4.44 12.00  62.51  n/a   .000
      ------------------
      CALIB NASHYD       3090  9 15.0    1.04     .01 12.00   7.71  .06   .000
      [CN=80.0        ]
      [ N= 3.0:Tp=  .07]
      ------------------
      CALIB NASHYD       3100  6 15.0    4.68     .19 12.00  27.64  .23   .000
      [CN=85.0        ]
      [ N= 3.0:Tp=  .09]
      ------------------
   *  CALIB STANDHYD     3101  1 15.0    3.07     .36 12.00  85.99  .71   .000
      [I%=19.0:S%= 7.00]
      ------------------
   *  CALIB STANDHYD     3102  2 15.0    1.30     .14 12.00  84.50  .70   .000
      [I%=18.0:S%= 4.00]
      ------------------
      ADD [3100 + 3101]  0708  8 15.0    7.75     .55 12.00  50.75  n/a   .000
      ------------------
      ADD [0708 + 3090]  0709  1 15.0    8.79     .56 12.00  45.66  n/a   .000
      ------------------
      ADD [0709 + 3102]  0710  6 15.0   10.09     .70 12.00  50.66  n/a   .000
      ------------------
      ADD [0707 + 0710]  0711  8 15.0   59.52    5.14 12.00  60.50  n/a   .000
      ------------------
      CALIB NASHYD       3110  1 15.0    1.21     .04 12.00  20.80  .17   .000
      [CN=85.0        ]
      [ N= 3.0:Tp=  .08]
      ------------------
      ADD [0711 + 3110]  0712  9 15.0   60.73    5.18 12.00  59.71  n/a   .000
      ------------------
   *  CALIB STANDHYD     3120  2 15.0    2.96     .36 12.00  91.51  .75   .000
      [I%=20.0:S%= 2.00]
      ------------------
      ADD [0712 + 3120]  0713  1 15.0   63.69    5.54 12.00  61.18  n/a   .000
      ------------------
      ADD [0703 + 0713]  0714  2 15.0   79.97    7.05 12.00  63.56  n/a   .000
      ------------------
      ADD [0614 + 0714]  0714  3 15.0  202.65   19.15 12.00  67.96  n/a   .000
      ------------------
  FINISH
=========================================================================
```

Preferred Alternative - 24 Hour SCS Storms
High Density Development



CLIENT

TOWN OF CALEDON

TITLE

POST–DEVELOPMENT
OTTHYMO SCHEMATIC

Marshall
Macklin
Monaghan
Consulting Engineers . Surveyors . Planners
80 Commerce Valley Dr. E, Thornhill, Ont. L3T 7N4
Tele (905)882–1100 Fax (905)882–0055

Checked	Drawn	
S.J.H.	AutoCAD/B.K.B.	
Date MAY 1999	Proj. No. 1499012–01–WR1	
Scale N.T.S.	Figure No. A2	Gr.No. 01

```
===============================================================================

      OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO     I N T E R H Y M O
     O   O    T      T    H   H   Y Y   MM MM  O   O    * * * 1989b * * *
     O   O    T      T    HHHHH    Y    M M M  O   O
     O   O    T      T    H   H    Y    M   M  O   O
      OOO     T      T    H   H    Y    M   M   OOO          162   00014
```


***** S U M M A R Y   O U T P U T *****

Input filename: opt1b-hi.dat
Output filename: opt1b-hi.out
Summary filename: opt1b-hi.sum

DATE: 04-12-1999 TIME: 13:53:58

USER: ____

COMMENTS: ____

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

W/E	COMMAND	HYD	ID	DT min	AREA ha	Qpeak cms	Tpeak hrs	R.V. mm	R.C.	Qbase cms
	START @ .00 hrs									
	READ STORM			5.0						
	[Ptot= 25.00 mm]									
	fname :PEARSN4H.25M									
	remark:TORONTO PEARSON 25 MM - 4 HR CHICAGO									
	CALIB NASHYD	2010	1	15.0	16.95	.15	1.50	2.87	.11	.000
	[CN=78.0]									
	[N= 3.0:Tp= .16]									
	CALIB NASHYD	2030	3	15.0	12.83	.11	1.50	2.79	.11	.000
	[CN=78.0]									
	[N= 3.0:Tp= .15]									
	CALIB NASHYD	2060	6	15.0	2.93	.05	1.50	4.38	.18	.000
	[CN=84.0]									
	[N= 3.0:Tp= .16]									
	ADD [2010 + 2030]	0601	9	15.0	29.78	.25	1.50	2.83	n/a	.000
	ADD [0601 + 2060]	0602	1	15.0	32.71	.30	1.50	2.97	n/a	.000
*	CALIB STANDHYD	2020	2	15.0	6.01	.15	1.50	8.61	.34	.000
	[I%=19.0:S%= 5.00]									
*	CALIB STANDHYD	2031	3	15.0	3.47	.09	1.50	8.60	.34	.000
	[I%=19.0:S%= 7.00]									
	ADD [2020 + 2031]	0603	9	15.0	9.48	.24	1.50	8.61	n/a	.000
*	CALIB STANDHYD	2040	4	15.0	10.95	.28	1.50	8.61	.34	.000
	[I%=19.0:S%= 6.00]									
*	CALIB STANDHYD	2041	5	15.0	7.25	.19	1.50	8.61	.34	.000
	[I%=19.0:S%= 8.00]									
	ADD [2040 + 2041]	0604	8	15.0	18.20	.47	1.50	8.61	n/a	.000
*	CALIB STANDHYD	2050	5	15.0	2.17	.06	1.50	8.60	.34	.000
	[I%=19.0:S%= 6.00]									
	ADD [0604 + 2050]	0605	7	15.0	20.37	.53	1.50	8.61	n/a	.000
	ADD [0605 + 0603]	0606	8	15.0	29.85	.77	1.50	8.61	n/a	.000
*	CALIB STANDHYD	1010	10	15.0	8.86	.24	1.50	8.61	.34	.000
	[I%=19.0:S%= 6.00]									
*	CALIB STANDHYD	1020	2	15.0	1.96	.07	1.50	9.56	.38	.000
	[I%=25.0:S%= 4.00]									
	ADD [1010 + 1020]	0901	7	15.0	10.82	.30	1.50	8.78	n/a	.000
*	CALIB STANDHYD	1030	3	15.0	3.11	.08	1.50	8.61	.34	.000
	[I%=19.0:S%= 5.00]									
	ADD [0901 + 1030]	0902	2	15.0	13.93	.39	1.50	8.74	n/a	.000
	ADD [0602 + 0606]	0999	3	15.0	62.56	1.07	1.50	5.66	n/a	.000

	ADD [0902 + 0999]	0999	1	15.0	76.49	1.46	1.50	6.22	n/a	.000
	RESRVR [1 : 0999] {ST= .42 ha.m }	0000	8	1.9	76.49	.05	4.47	4.35	n/a	.000
	DIVERT HYD	0000	8	1.9	76.49	.05	4.47	4.35	n/a	.000
	Outflow	0002	2	1.9	19.12	.01	4.47	4.35	n/a	.000
	Outflow	0003	3	1.9	57.37	.04	4.47	4.35	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .05]	1040	4	15.0	.61	.00	1.50	.02	.00	.000
	ADD [1040 + 0002]	0999	5	1.9	19.73	.01	4.47	4.22	n/a	.000
	CHANNEL[5 : 0999]	0000	1	1.9	19.73	.01	4.72	4.20	n/a	.000
	CALIB NASHYD [CN=83.0] [N= 3.0:Tp= .15]	1050	5	15.0	6.82	.10	1.50	3.96	.16	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	1060	6	15.0	2.67	.09	1.50	9.55	.38	.000
	ADD [1050 + 1060]	0999	2	15.0	9.49	.18	1.50	5.54	n/a	.000
	ADD [0000 + 0999]	0999	4	1.9	29.22	.19	1.50	4.64	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	1070	7	15.0	9.73	.27	1.50	8.61	.34	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2120	2	15.0	2.58	.07	1.50	8.61	.34	.000
	ADD [1070 + 2120]	0999	1	15.0	12.31	.34	1.50	8.61	n/a	.000
	ADD [0999 + 0999]	0999	2	1.9	41.53	.52	1.50	5.81	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .13]	1080	8	15.0	4.54	.04	1.50	2.86	.11	.000
	ADD [0999 + 1080]	0506	5	1.9	46.07	.57	1.50	5.52	n/a	.000
	RESRVR [5 : 0506] {ST= .16 ha.m }	0000	6	1.9	46.07	.02	4.69	3.46	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .04]	1090	9	15.0	.48	.00	1.50	.00	.00	.000
	ADD [0000 + 1090]	0507	1	1.9	46.55	.02	4.69	3.43	n/a	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	2061	6	15.0	4.24	.09	1.50	7.65	.31	.000
	ADD [2061 + 0003]	0999	2	1.9	61.61	.10	1.50	4.58	n/a	.000
	CHANNEL[2 : 0999]	0000	3	1.9	61.61	.09	1.56	4.57	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	2070	7	15.0	3.26	.09	1.50	8.54	.34	.000
	ADD [0000 + 2070]	0606	6	1.9	64.87	.17	1.53	4.77	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	2080	8	15.0	1.14	.00	1.50	.08	.00	.000
	ADD [0606 + 2080]	0607	7	1.9	66.01	.17	1.53	4.69	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	2090	9	15.0	2.37	.01	1.50	.87	.03	.000
	ADD [0607 + 2090]	0608	8	1.9	68.38	.18	1.53	4.56	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.06	1.50	8.19	.33	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.04	1.50	11.59	.46	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.10	1.50	9.03	n/a	.000
	ADD [0608 + 0609]	0610	10	1.9	71.63	.28	1.50	4.76	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.05	1.50	10.92	.44	.000
	ADD [2130 + 0610]	0612	2	1.9	72.97	.33	1.50	4.87	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.00	1.50	.74	.03	.000

	ADD [0612 + 2140]	0613	3	1.9	73.73	.33	1.50	4.83	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	2150	5	15.0	2.40	.02	1.50	1.88	.08	.000
	ADD [0613 + 2150]	0614	4	1.9	76.13	.35	1.50	4.74	n/a	.000
	ADD [0507 + 0614]	0614	10	1.9	122.68	.35	1.50	4.24	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	3010	1	15.0	3.04	.01	1.50	.99	.04	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3020	2	15.0	6.46	.14	1.50	7.53	.30	.000
	ADD [3010 + 3020]	0701	4	15.0	9.50	.15	1.50	5.44	n/a	.000
	RESRVR [4 : 0701] {ST= .01 ha.m }	0002	1	15.0	9.50	.08	1.75	5.43	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	3030	3	15.0	2.02	.08	1.50	10.57	.42	.000
	ADD [0002 + 3030]	0702	2	15.0	11.52	.13	1.50	6.33	n/a	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	3040	4	15.0	4.76	.15	1.50	9.56	.38	.000
	ADD [0702 + 3040]	0703	3	15.0	16.28	.28	1.50	7.28	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .21]	3050	5	15.0	26.45	.21	1.50	3.30	.13	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3060	6	15.0	2.27	.05	1.50	7.52	.30	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .09]	3070	7	15.0	3.77	.02	1.50	1.58	.06	.000
	ADD [3050 + 3060]	0704	4	15.0	28.72	.26	1.50	3.63	n/a	.000
	ADD [0704 + 3070]	0705	5	15.0	32.49	.28	1.50	3.39	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .18]	3080	8	15.0	13.69	.12	1.50	3.10	.12	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	3081	1	15.0	3.25	.09	1.50	8.61	.34	.000
	ADD [3080 + 3081]	0706	6	15.0	16.94	.21	1.50	4.16	n/a	.000
	ADD [0705 + 0706]	0707	7	15.0	49.43	.49	1.50	3.65	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .07]	3090	9	15.0	1.04	.00	1.50	.43	.02	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .09]	3100	6	15.0	4.68	.04	1.50	2.10	.08	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	3101	1	15.0	3.07	.08	1.50	8.61	.34	.000
*	CALIB STANDHYD [I%=18.0:S%= 4.00]	3102	2	15.0	1.30	.03	1.50	8.18	.33	.000
	ADD [3100 + 3101]	0708	8	15.0	7.75	.13	1.50	4.68	n/a	.000
	ADD [0708 + 3090]	0709	1	15.0	8.79	.13	1.50	4.18	n/a	.000
	ADD [0709 + 3102]	0710	6	15.0	10.09	.16	1.50	4.69	n/a	.000
	ADD [0707 + 0710]	0711	8	15.0	59.52	.64	1.50	3.83	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .08]	3110	1	15.0	1.21	.01	1.50	1.53	.06	.000
	ADD [0711 + 3110]	0712	6	15.0	60.73	.65	1.50	3.78	n/a	.000
	RESRVR [6 : 0712] {ST= .21 ha.m }	0000	9	1.9	60.73	.02	4.31	2.64	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 2.00]	3120	2	15.0	2.96	.09	1.50	9.92	.40	.000
	ADD [0000 + 3120]	0713	1	1.9	63.69	.09	1.50	2.98	n/a	.000
	ADD [0703 + 0713]	0714	2	1.9	79.97	.37	1.50	3.85	n/a	.000

```
   ADD [0614 + 0714]  0714  3  1.9  202.65     .72  1.50   4.09  n/a    .000
   ------------------
** END OF SIMULATION :   1

***************************************************************************


**************************
** SIMULATION NUMBER:   2 **
**************************

W/E COMMAND              HYD ID   DT     AREA   Qpeak Tpeak   R.V.  R.C.   Qbase
                                 min      ha     cms   hrs     mm           cms
   START @    .00 hrs
   ------------------
   READ STORM                   15.0
   [ Ptot= 47.40 mm ]
   fname :PRSN-SCS.002
   remark:2 YR 24 HR SCS II - PEARSON AIRPORT
   ------------------
   CALIB NASHYD       2010  1 15.0   16.95     .30 12.00  11.56  .24    .000
   [CN=78.0          ]
   [ N= 3.0:Tp=   .16]
   ------------------
   CALIB NASHYD       2030  3 15.0   12.83     .22 12.00  11.23  .24    .000
   [CN=78.0          ]
   [ N= 3.0:Tp=   .15]
   ------------------
   CALIB NASHYD       2060  6 15.0    2.93     .07 12.00  15.22  .32    .000
   [CN=84.0          ]
   [ N= 3.0:Tp=   .16]
   ------------------
   ADD [2010 + 2030]  0601  9 15.0   29.78     .53 12.00  11.42  n/a    .000
   ------------------
   ADD [0601 + 2060]  0602  1 15.0   32.71     .60 12.00  11.76  n/a    .000
   ------------------
*  CALIB STANDHYD     2020  2 15.0    6.01     .16 12.00  23.22  .49    .000
   [I%=19.0:S%= 5.00]
   ------------------
*  CALIB STANDHYD     2031  3 15.0    3.47     .09 12.00  23.21  .49    .000
   [I%=19.0:S%= 7.00]
   ------------------
   ADD [2020 + 2031]  0603  9 15.0    9.48     .25 12.00  23.22  n/a    .000
   ------------------
*  CALIB STANDHYD     2040  4 15.0   10.95     .25 12.00  23.22  .49    .000
   [I%=19.0:S%= 6.00]
   ------------------
*  CALIB STANDHYD     2041  5 15.0    7.25     .20 12.00  23.22  .49    .000
   [I%=19.0:S%= 8.00]
   ------------------
   ADD [2040 + 2041]  0604  8 15.0   18.20     .44 12.00  23.22  n/a    .000
   ------------------
*  CALIB STANDHYD     2050  5 15.0    2.17     .06 12.00  23.21  .49    .000
   [I%=19.0:S%= 6.00]
   ------------------
   ADD [0604 + 2050]  0605  7 15.0   20.37     .50 12.00  23.22  n/a    .000
   ------------------
   ADD [0605 + 0603]  0606  8 15.0   29.85     .75 12.00  23.22  n/a    .000
   ------------------
*  CALIB STANDHYD     1010 10 15.0    8.86     .24 12.00  23.22  .49    .000
   [I%=19.0:S%= 6.00]
   ------------------
*  CALIB STANDHYD     1020  2 15.0    1.96     .06 12.00  24.46  .52    .000
   [I%=25.0:S%= 4.00]
   ------------------
   ADD [1010 + 1020]  0901  7 15.0   10.82     .29 12.00  23.44  n/a    .000
   ------------------
*  CALIB STANDHYD     1030  3 15.0    3.11     .08 12.00  23.22  .49    .000
   [I%=19.0:S%= 5.00]
   ------------------
   ADD [0901 + 1030]  0902  2 15.0   13.93     .38 12.00  23.39  n/a    .000
   ------------------
   ADD [0602 + 0606]  0999  3 15.0   62.56    1.35 12.00  17.23  n/a    .000
   ------------------
   ADD [0902 + 0999]  0999  1 15.0   76.49    1.73 12.00  18.35  n/a    .000
   ------------------
   RESRVR [ 1 : 0999] 0000  8  1.9   76.49     .66 13.00  11.89  n/a    .000
   {ST=    .70 ha.m }
   ------------------
   DIVERT HYD         0000  8  1.9   76.49     .66 13.00  11.89  n/a    .000
      Outflow         0002  2  1.9   16.97     .13 13.00  11.89  n/a    .000
      Outflow         0003  3  1.9   59.52     .54 13.00  11.89  n/a    .000
   ------------------
   CALIB NASHYD       1040  4 15.0     .61     .00 12.00    .12  .00    .000
   [CN=80.0          ]
   [ N= 3.0:Tp=   .05]
   ------------------
   ADD [1040 + 0002]  0999  5  1.9   17.58     .13 13.00  11.48  n/a    .000
   ------------------
   CHANNEL[ 5 : 0999] 0000  1  1.9   17.58     .13 13.03  11.42  n/a    .000
   ------------------
   CALIB NASHYD       1050  5 15.0    6.82     .15 12.00  13.94  .29    .000
```

```
        [CN=83.0          ]
        [ N= 3.0:Tp=   .15]
        ------------------
    *   CALIB STANDHYD     1060  6 15.0    2.67     .06 12.00  24.46  .52     .000
        [I%=25.0:S%= 5.00]
        ------------------
        ADD [1050 + 1060]  0999  2 15.0    9.49     .21 12.00  16.90  n/a     .000
        ------------------
        ADD [0000 + 0999]  0999  4  1.9   27.07     .22 12.00  13.34  n/a     .000
        ------------------
    *   CALIB STANDHYD     1070  7 15.0    9.73     .28 12.00  23.22  .49     .000
        [I%=19.0:S%= 7.00]
        ------------------
    *   CALIB STANDHYD     2120  2 15.0    2.58     .07 12.00  23.22  .49     .000
        [I%=19.0:S%= 7.00]
        ------------------
        ADD [1070 + 2120]  0999  1 15.0   12.31     .36 12.00  23.22  n/a     .000
        ------------------
        ADD [0999 + 0999]  0999  2  1.9   39.38     .58 12.00  16.43  n/a     .000
        ------------------
        CALIB NASHYD       1080  8 15.0    4.54     .08 12.00  10.98  .23     .000
        [CN=80.0          ]
        [ N= 3.0:Tp=   .13]
        ------------------
        ADD [0999 + 1080]  0506  5  1.9   43.92     .66 12.00  15.86  n/a     .000
        ------------------
        RESRVR [ 5 : 0506] 0000  6  1.9   43.92     .36 12.28  12.04  n/a     .000
        {ST=    .20 ha.m }
        ------------------
        CALIB NASHYD       1090  9 15.0     .48     .00 12.00    .00  .00     .000
        [CN=85.0          ]
        [ N= 3.0:Tp=   .04]
        ------------------
        ADD [0000 + 1090]  0507  1  1.9   44.40     .36 12.28  11.94  n/a     .000
        ------------------
    *   CALIB STANDHYD     2061  6 15.0    4.24     .11 12.00  21.67  .46     .000
        [I%=15.0:S%= 6.00]
        ------------------
        ADD [2061 + 0003]  0999  2  1.9   63.76     .58 13.00  12.54  n/a     .000
        ------------------
        CHANNEL[ 2 : 0999] 0000  3  1.9   63.76     .58 13.00  12.51  n/a     .000
        ------------------
    *   CALIB STANDHYD     2070  7 15.0    3.26     .09 12.00  22.91  .48     .000
        [I%=20.0:S%= 6.00]
        ------------------
        ADD [0000 + 2070]  0606  6  1.9   67.02     .61 13.00  13.02  n/a     .000
        ------------------
        CALIB NASHYD       2080  8 15.0    1.14     .00 12.00    .25  .01     .000
        [CN=82.0          ]
        [ N= 3.0:Tp=   .05]
        ------------------
        ADD [0606 + 2080]  0607  7  1.9   68.16     .61 13.00  12.80  n/a     .000
        ------------------
        CALIB NASHYD       2090  9 15.0    2.37     .01 12.00   2.81  .06     .000
        [CN=86.0          ]
        [ N= 3.0:Tp=   .07]
        ------------------
        ADD [0607 + 2090]  0608  8  1.9   70.53     .61 13.00  12.47  n/a     .000
        ------------------
    *   CALIB STANDHYD     2100 10 15.0    2.45     .05 12.00  22.44  .47     .000
        [I%=18.0:S%= 5.00]
        ------------------
    *   CALIB STANDHYD     2110  7 15.0     .80     .03 12.00  27.54  .58     .000
        [I%=35.0:S%= 4.00]
        ------------------
        ADD [2100 + 2110]  0609  9 15.0    3.25     .08 12.00  23.70  n/a     .000
        ------------------
        ADD [0608 + 0609]  0610 10  1.9   73.78     .65 13.00  12.96  n/a     .000
        ------------------
    *   CALIB STANDHYD     2130  3 15.0    1.34     .04 12.00  26.78  .57     .000
        [I%=30.0:S%= 6.00]
        ------------------
        ADD [2130 + 0610]  0612  2  1.9   75.12     .66 13.00  13.21  n/a     .000
        ------------------
        CALIB NASHYD       2140  4 15.0     .76     .00 12.00   2.70  .06     .000
        [CN=82.0          ]
        [ N= 3.0:Tp=   .07]
        ------------------
        ADD [0612 + 2140]  0613  3  1.9   75.88     .67 13.00  13.10  n/a     .000
        ------------------
        CALIB NASHYD       2150  5 15.0    2.40     .03 12.00   7.58  .16     .000
        [CN=78.0          ]
        [ N= 3.0:Tp=   .11]
        ------------------
        ADD [0613 + 2150]  0614  4  1.9   78.28     .67 13.00  12.93  n/a     .000
        ------------------
        ADD [0507 + 0614]  0614 10  1.9  122.68    1.00 13.00  12.56  n/a     .000
        ------------------
        CALIB NASHYD       3010  1 15.0    3.04     .02 12.00   3.80  .08     .000
        [CN=80.0          ]
        [ N= 3.0:Tp=   .08]
        ------------------
    *   CALIB STANDHYD     3020  2 15.0    6.46     .17 12.00  21.37  .45     .000
        [I%=15.0:S%= 6.00]
        ------------------
        ADD [3010 + 3020]  0701  4 15.0    9.50     .19 12.00  15.75  n/a     .000
```

```
  ------------------
  RESRVR [ 4 : 0701] 0002  1 15.0    9.50    .14 12.25  15.75  n/a   .000
  {ST=   .02 ha.m }
  ------------------
* CALIB STANDHYD     3030  3 15.0    2.02    .06 12.00  26.00  .55   .000
  [I%=30.0:S%= 6.00]
  ------------------
  ADD [0002 + 3030]  0702  2 15.0   11.52    .19 12.00  17.55  n/a   .000
  ------------------
* CALIB STANDHYD     3040  4 15.0    4.76    .11 12.00  24.46  .52   .000
  [I%=25.0:S%= 5.00]
  ------------------
  ADD [0702 + 3040]  0703  3 15.0   16.28    .31 12.00  19.57  n/a   .000
  ------------------
  CALIB NASHYD       3050  5 15.0   26.45    .52 12.00  13.29  .28   .000
  [CN=78.0         ]
  [ N= 3.0:Tp=  .21]
  ------------------
* CALIB STANDHYD     3060  6 15.0    2.27    .05 12.00  21.36  .45   .000
  [I%=15.0:S%= 6.00]
  ------------------
  CALIB NASHYD       3070  7 15.0    3.77    .03 12.00   5.76  .12   .000
  [CN=82.0         ]
  [ N= 3.0:Tp=  .09]
  ------------------
  ADD [3050 + 3060]  0704  4 15.0   28.72    .57 12.00  13.92  n/a   .000
  ------------------
  ADD [0704 + 3070]  0705  5 15.0   32.49    .60 12.00  12.98  n/a   .000
  ------------------
  CALIB NASHYD       3080  8 15.0   13.69    .26 12.00  12.50  .26   .000
  [CN=78.0         ]
  [ N= 3.0:Tp=  .18]
  ------------------
* CALIB STANDHYD     3081  1 15.0    3.25    .09 12.00  23.22  .49   .000
  [I%=19.0:S%= 6.00]
  ------------------
  ADD [3080 + 3081]  0706  6 15.0   16.94    .35 12.00  14.56  n/a   .000
  ------------------
  ADD [0705 + 0706]  0707  7 15.0   49.43    .96 12.00  13.52  n/a   .000
  ------------------
  CALIB NASHYD       3090  9 15.0    1.04    .00 12.00   1.63  .03   .000
  [CN=80.0         ]
  [ N= 3.0:Tp=  .07]
  ------------------
  CALIB NASHYD       3100  6 15.0    4.68    .05 12.00   6.85  .14   .000
  [CN=85.0         ]
  [ N= 3.0:Tp=  .09]
  ------------------
* CALIB STANDHYD     3101  1 15.0    3.07    .09 12.00  23.22  .49   .000
  [I%=19.0:S%= 7.00]
  ------------------
* CALIB STANDHYD     3102  2 15.0    1.30    .03 12.00  22.43  .47   .000
  [I%=18.0:S%= 4.00]
  ------------------
  ADD [3100 + 3101]  0708  8 15.0    7.75    .14 12.00  13.34  n/a   .000
  ------------------
  ADD [0708 + 3090]  0709  1 15.0    8.79    .14 12.00  11.95  n/a   .000
  ------------------
  ADD [0709 + 3102]  0710  6 15.0   10.09    .17 12.00  13.30  n/a   .000
  ------------------
  ADD [0707 + 0710]  0711  8 15.0   59.52   1.13 12.00  13.48  n/a   .000
  ------------------
  CALIB NASHYD       3110  1 15.0    1.21    .01 12.00   5.10  .11   .000
  [CN=85.0         ]
  [ N= 3.0:Tp=  .08]
  ------------------
  ADD [0711 + 3110]  0712  6 15.0   60.73   1.14 12.00  13.31  n/a   .000
  ------------------
  RESRVR [ 6 : 0712] 0000  9  1.9   60.73    .73 12.19  10.27  n/a   .000
  {ST=   .28 ha.m }
  ------------------
* CALIB STANDHYD     3120  2 15.0    2.96    .09 12.00  25.95  .55   .000
  [I%=20.0:S%= 2.00]
  ------------------
  ADD [0000 + 3120]  0713  1  1.9   63.69    .80 12.19  11.00  n/a   .000
  ------------------
  ADD [0703 + 0713]  0714  2  1.9   79.97   1.07 12.16  12.74  n/a   .000
  ------------------
  ADD [0614 + 0714]  0714  3  1.9  202.65   1.73 12.19  12.63  n/a   .000
  ------------------
** END OF SIMULATION :   2

*****************************************************************************


****************************
** SIMULATION NUMBER:   3 **
****************************

W/E COMMAND            HYD ID   DT    AREA   Qpeak Tpeak   R.V. R.C.   Qbase
                                min    ha     cms   hrs     mm          cms

    START @    .00 hrs
```

```
------------------------------------------------------------------------------------------------------------------------

    ------------------
    READ STORM                  15.0
    [ Ptot= 67.20 mm ]
    fname :PRSN-SCS.005
    remark:5 YR 24 HR SCS II - PEARSON AIRPORT
    ------------------
    CALIB NASHYD        2010  1 15.0    16.95     .57 12.00  21.82  .32    .000
    [CN=78.0         ]
    [ N= 3.0:Tp=  .16]
    ------------------
    CALIB NASHYD        2030  3 15.0    12.83     .42 12.00  21.20  .32    .000
    [CN=78.0         ]
    [ N= 3.0:Tp=  .15]
    ------------------
    CALIB NASHYD        2060  6 15.0     2.93     .12 12.00  27.07  .40    .000
    [CN=84.0         ]
    [ N= 3.0:Tp=  .16]
    ------------------
    ADD [2010 + 2030]   0601  9 15.0    29.78     .98 12.00  21.55  n/a    .000
    ------------------
    ADD [0601 + 2060]   0602  1 15.0    32.71    1.10 12.00  22.05  n/a    .000
    ------------------
  * CALIB STANDHYD      2020  2 15.0     6.01     .28 12.00  38.60  .57    .000
    [I%=19.0:S%= 5.00]
    ------------------
  * CALIB STANDHYD      2031  3 15.0     3.47     .16 12.00  38.60  .57    .000
    [I%=19.0:S%= 7.00]
    ------------------
    ADD [2020 + 2031]   0603  9 15.0     9.48     .44 12.00  38.60  n/a    .000
    ------------------
  * CALIB STANDHYD      2040  4 15.0    10.95     .49 12.00  38.60  .57    .000
    [I%=19.0:S%= 6.00]
    ------------------
  * CALIB STANDHYD      2041  5 15.0     7.25     .34 12.00  38.60  .57    .000
    [I%=19.0:S%= 8.00]
    ------------------
    ADD [2040 + 2041]   0604  8 15.0    18.20     .83 12.00  38.60  n/a    .000
    ------------------
  * CALIB STANDHYD      2050  5 15.0     2.17     .10 12.00  38.60  .57    .000
    [I%=19.0:S%= 6.00]
    ------------------
    ADD [0604 + 2050]   0605  7 15.0    20.37     .94 12.00  38.60  n/a    .000
    ------------------
    ADD [0605 + 0603]   0606  8 15.0    29.85    1.38 12.00  38.60  n/a    .000
    ------------------
  * CALIB STANDHYD      1010 10 15.0     8.86     .42 12.00  38.60  .57    .000
    [I%=19.0:S%= 6.00]
    ------------------
  * CALIB STANDHYD      1020  2 15.0     1.96     .10 12.00  39.95  .59    .000
    [I%=25.0:S%= 4.00]
    ------------------
    ADD [1010 + 1020]   0901  7 15.0    10.82     .52 12.00  38.84  n/a    .000
    ------------------
  * CALIB STANDHYD      1030  3 15.0     3.11     .15 12.00  38.60  .57    .000
    [I%=19.0:S%= 5.00]
    ------------------
    ADD [0901 + 1030]   0902  2 15.0    13.93     .67 12.00  38.79  n/a    .000
    ------------------
    ADD [0602 + 0606]   0999  3 15.0    62.56    2.48 12.00  29.94  n/a    .000
    ------------------
    ADD [0902 + 0999]   0999  1 15.0    76.49    3.15 12.00  31.55  n/a    .000
    ------------------
    RESRVR [ 1 : 0999]  0000  8  1.9    76.49    2.45 12.13  25.03  n/a    .000
    {ST=    .81 ha.m }
    ------------------
    DIVERT HYD          0000  8  1.9    76.49    2.45 12.13  25.03  n/a    .000
       Outflow          0002  2  1.9    15.52     .45 12.13  25.03  n/a    .000
       Outflow          0003  3  1.9    60.97    1.97 12.13  25.03  n/a    .000
    ------------------
    CALIB NASHYD        1040  4 15.0      .61     .00 12.00    .32  .00    .000
    [CN=80.0         ]
    [ N= 3.0:Tp=  .05]
    ------------------
    ADD [1040 + 0002]   0999  5  1.9    16.13     .46 12.13  24.10  n/a    .000
    ------------------
    CHANNEL[ 5 : 0999]  0000  1  1.9    16.13     .44 12.22  24.03  n/a    .000
    ------------------
    CALIB NASHYD        1050  5 15.0     6.82     .26 12.00  24.96  .37    .000
    [CN=83.0         ]
    [ N= 3.0:Tp=  .15]
    ------------------
  * CALIB STANDHYD      1060  6 15.0     2.67     .12 12.00  39.95  .59    .000
    [I%=25.0:S%= 5.00]
    ------------------
    ADD [1050 + 1060]   0999  2 15.0     9.49     .38 12.00  29.18  n/a    .000
    ------------------
    ADD [0000 + 0999]   0999  4  1.9    25.62     .70 12.09  25.94  n/a    .000
    ------------------
  * CALIB STANDHYD      1070  7 15.0     9.73     .52 12.00  38.60  .57    .000
    [I%=19.0:S%= 7.00]
    ------------------
  * CALIB STANDHYD      2120  2 15.0     2.58     .14 12.00  38.60  .57    .000
    [I%=19.0:S%= 7.00]
    ------------------
    ADD [1070 + 2120]   0999  1 15.0    12.31     .65 12.00  38.60  n/a    .000
    ------------------
```

	ADD [0999 + 0999]	0999	2	1.9	37.93	1.31	12.00	30.05	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .13]	1080	8	15.0	4.54	.14	12.00	20.33	.30	.000
	ADD [0999 + 1080]	0506	5	1.9	42.47	1.46	12.00	29.01	n/a	.000
	RESRVR [5 : 0506] {ST= .27 ha.m }	0000	6	1.9	42.47	1.19	12.16	25.05	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .04]	1090	9	15.0	.48	.00	12.00	.11	.00	.000
	ADD [0000 + 1090]	0507	1	1.9	42.95	1.19	12.16	24.77	n/a	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	2061	6	15.0	4.24	.19	12.00	36.67	.55	.000
	ADD [2061 + 0003]	0999	2	1.9	65.21	2.16	12.13	25.79	n/a	.000
	CHANNEL[2 : 0999]	0000	3	1.9	65.21	2.15	12.16	25.76	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	2070	7	15.0	3.26	.17	12.00	38.10	.57	.000
	ADD [0000 + 2070]	0606	6	1.9	68.47	2.26	12.16	26.35	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	2080	8	15.0	1.14	.00	12.00	.56	.01	.000
	ADD [0606 + 2080]	0607	7	1.9	69.61	2.26	12.16	25.93	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	2090	9	15.0	2.37	.02	12.00	4.82	.07	.000
	ADD [0607 + 2090]	0608	8	1.9	71.98	2.27	12.16	25.23	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.10	12.00	37.57	.56	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.05	12.00	43.66	.65	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.15	12.00	39.07	n/a	.000
	ADD [0608 + 0609]	0610	10	1.9	75.23	2.40	12.13	25.83	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.07	12.00	42.91	.64	.000
	ADD [2130 + 0610]	0612	2	1.9	76.57	2.45	12.13	26.13	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.01	12.00	4.91	.07	.000
	ADD [0612 + 2140]	0613	3	1.9	77.33	2.46	12.13	25.92	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	2150	5	15.0	2.40	.05	12.00	14.29	.21	.000
	ADD [0613 + 2150]	0614	4	1.9	79.73	2.49	12.13	25.57	n/a	.000
	ADD [0507 + 0614]	0614	10	1.9	122.68	3.68	12.13	25.29	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	3010	1	15.0	3.04	.03	12.00	7.04	.10	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3020	2	15.0	6.46	.32	12.00	36.25	.54	.000
	ADD [3010 + 3020]	0701	4	15.0	9.50	.36	12.00	26.90	n/a	.000
	RESRVR [4 : 0701] {ST= .05 ha.m }	0002	1	15.0	9.50	.24	12.25	26.90	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	3030	3	15.0	2.02	.11	12.00	41.80	.62	.000
	ADD [0002 + 3030]	0702	2	15.0	11.52	.33	12.00	29.51	n/a	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	3040	4	15.0	4.76	.22	12.00	39.95	.59	.000
	ADD [0702 + 3040]	0703	3	15.0	16.28	.54	12.00	32.57	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .21]	3050	5	15.0	26.45	.98	12.00	25.07	.37	.000

```
 *   CALIB STANDHYD      3060  6 15.0     2.27      .09 12.00  36.24  .54     .000
     [I%=15.0:S%= 6.00]
     ------------------
     CALIB NASHYD        3070  7 15.0     3.77      .06 12.00  10.46  .16     .000
     [CN=82.0         ]
     [ N= 3.0:Tp=  .09]
     ------------------
     ADD [3050 + 3060]   0704  4 15.0    28.72     1.08 12.00  25.95  n/a     .000
     ------------------
     ADD [0704 + 3070]   0705  5 15.0    32.49     1.14 12.00  24.15  n/a     .000
     ------------------
     CALIB NASHYD        3080  8 15.0    13.69      .49 12.00  23.59  .35     .000
     [CN=78.0         ]
     [ N= 3.0:Tp=  .18]
     ------------------
 *   CALIB STANDHYD      3081  1 15.0     3.25      .17 12.00  38.60  .57     .000
     [I%=19.0:S%= 6.00]
     ------------------
     ADD [3080 + 3081]   0706  6 15.0    16.94      .66 12.00  26.47  n/a     .000
     ------------------
     ADD [0705 + 0706]   0707  7 15.0    49.43     1.80 12.00  24.95  n/a     .000
     ------------------
     CALIB NASHYD        3090  9 15.0     1.04      .01 12.00   3.09  .05     .000
     [CN=80.0         ]
     [ N= 3.0:Tp=  .07]
     ------------------
     CALIB NASHYD        3100  6 15.0     4.68      .08 12.00  11.94  .18     .000
     [CN=85.0         ]
     [ N= 3.0:Tp=  .09]
     ------------------
 *   CALIB STANDHYD      3101  1 15.0     3.07      .15 12.00  38.60  .57     .000
     [I%=19.0:S%= 7.00]
     ------------------
 *   CALIB STANDHYD      3102  2 15.0     1.30      .06 12.00  37.56  .56     .000
     [I%=18.0:S%= 4.00]
     ------------------
     ADD [3100 + 3101]   0708  8 15.0     7.75      .23 12.00  22.50  n/a     .000
     ------------------
     ADD [0708 + 3090]   0709  1 15.0     8.79      .24 12.00  20.21  n/a     .000
     ------------------
     ADD [0709 + 3102]   0710  6 15.0    10.09      .30 12.00  22.44  n/a     .000
     ------------------
     ADD [0707 + 0710]   0711  8 15.0    59.52     2.10 12.00  24.52  n/a     .000
     ------------------
     CALIB NASHYD        3110  1 15.0     1.21      .02 12.00   8.94  .13     .000
     [CN=85.0         ]
     [ N= 3.0:Tp=  .08]
     ------------------
     ADD [0711 + 3110]   0712  6 15.0    60.73     2.11 12.00  24.21  n/a     .000
     ------------------
     RESRVR [ 6 : 0712]  0000  9  1.9    60.73     1.92 12.03  21.16  n/a     .000
     {ST=    .34 ha.m }
     ------------------
 *   CALIB STANDHYD      3120  2 15.0     2.96      .16 12.00  42.32  .63     .000
     [I%=20.0:S%= 2.00]
     ------------------
     ADD [0000 + 3120]   0713  1  1.9    63.69     2.07 12.03  22.14  n/a     .000
     ------------------
     ADD [0703 + 0713]   0714  2  1.9    79.97     2.60 12.03  24.26  n/a     .000
     ------------------
     ADD [0614 + 0714]   0714  3  1.9   202.65     6.18 12.09  24.88  n/a     .000
     ------------------
 ** END OF SIMULATION :   3

*********************************************************************************


 ****************************
 ** SIMULATION NUMBER:   4 **
 ****************************

 W/E COMMAND              HYD ID   DT     AREA   Qpeak Tpeak    R.V. R.C.    Qbase
                                  min      ha     cms   hrs      mm           cms

     START @    .00 hrs
     ------------------
     READ STORM                  15.0
     [ Ptot= 80.40 mm ]
     fname :PRSN-SCS.010
     remark:10 YR 24 HR SCS II - PEARSON AIRPORT
     ------------------
     CALIB NASHYD        2010  1 15.0    16.95      .76 12.00  29.49  .37     .000
     [CN=78.0         ]
     [ N= 3.0:Tp=  .16]
     ------------------
     CALIB NASHYD        2030  3 15.0    12.83      .56 12.00  28.64  .36     .000
     [CN=78.0         ]
     [ N= 3.0:Tp=  .15]
     ------------------
     CALIB NASHYD        2060  6 15.0     2.93      .16 12.00  35.62  .44     .000
     [CN=84.0         ]
     [ N= 3.0:Tp=  .16]
     ------------------
```

	ADD [2010 + 2030]	0601	9	15.0	29.78	1.32	12.00	29.12	n/a	.000
	ADD [0601 + 2060]	0602	1	15.0	32.71	1.47	12.00	29.71	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 5.00]	2020	2	15.0	6.01	.37	12.00	49.59	.62	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2031	3	15.0	3.47	.21	12.00	49.59	.62	.000
	ADD [2020 + 2031]	0603	9	15.0	9.48	.58	12.00	49.59	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	2040	4	15.0	10.95	.65	12.00	49.59	.62	.000
*	CALIB STANDHYD [I%=19.0:S%= 8.00]	2041	5	15.0	7.25	.45	12.00	49.59	.62	.000
	ADD [2040 + 2041]	0604	8	15.0	18.20	1.10	12.00	49.59	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	2050	5	15.0	2.17	.15	12.00	49.59	.62	.000
	ADD [0604 + 2050]	0605	7	15.0	20.37	1.25	12.00	49.59	n/a	.000
	ADD [0605 + 0603]	0606	8	15.0	29.85	1.83	12.00	49.59	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	1010	10	15.0	8.86	.55	12.00	49.59	.62	.000
*	CALIB STANDHYD [I%=25.0:S%= 4.00]	1020	2	15.0	1.96	.13	12.00	50.99	.63	.000
	ADD [1010 + 1020]	0901	7	15.0	10.82	.68	12.00	49.85	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 5.00]	1030	3	15.0	3.11	.20	12.00	49.59	.62	.000
	ADD [0901 + 1030]	0902	2	15.0	13.93	.88	12.00	49.79	n/a	.000
	ADD [0602 + 0606]	0999	3	15.0	62.56	3.30	12.00	39.19	n/a	.000
	ADD [0902 + 0999]	0999	1	15.0	76.49	4.18	12.00	41.12	n/a	.000
	RESRVR [1 : 0999] {ST= .93 ha.m }	0000	8	1.9	76.49	3.21	12.16	34.59	n/a	.000
	DIVERT HYD	0000	8	1.9	76.49	3.21	12.16	34.59	n/a	.000
	Outflow	0002	2	1.9	15.16	.60	12.16	34.59	n/a	.000
	Outflow	0003	3	1.9	61.33	2.61	12.16	34.59	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .05]	1040	4	15.0	.61	.00	12.00	.44	.01	.000
	ADD [1040 + 0002]	0999	5	1.9	15.77	.60	12.13	33.27	n/a	.000
	CHANNEL[5 : 0999]	0000	1	1.9	15.77	.59	12.19	33.20	n/a	.000
	CALIB NASHYD [CN=83.0] [N= 3.0:Tp= .15]	1050	5	15.0	6.82	.34	12.00	32.96	.41	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	1060	6	15.0	2.67	.16	12.00	50.99	.63	.000
	ADD [1050 + 1060]	0999	2	15.0	9.49	.50	12.00	38.03	n/a	.000
	ADD [0000 + 0999]	0999	4	1.9	25.26	1.03	12.00	35.01	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	1070	7	15.0	9.73	.67	12.00	49.59	.62	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2120	2	15.0	2.58	.18	12.00	49.59	.62	.000
	ADD [1070 + 2120]	0999	1	15.0	12.31	.85	12.00	49.59	n/a	.000
	ADD [0999 + 0999]	0999	2	1.9	37.57	1.87	12.00	39.79	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .13]	1080	8	15.0	4.54	.19	12.00	27.24	.34	.000
	ADD [0999 + 1080]	0506	5	1.9	42.11	2.06	12.00	38.44	n/a	.000
	RESRVR [5 : 0506] {ST= .33 ha.m }	0000	6	1.9	42.11	1.53	12.16	34.44	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .04]	1090	9	15.0	.48	.00	12.00	.14	.00	.000
	ADD [0000 + 1090]	0507	1	1.9	42.59	1.53	12.16	34.06	n/a	.000
*	CALIB STANDHYD	2061	6	15.0	4.24	.26	12.00	47.47	.59	.000

	[I%=15.0:S%= 6.00]									
	ADD [2061 + 0003]	0999	2	1.9	65.57	2.83	12.13	35.42	n/a	.000
	CHANNEL[2 : 0999]	0000	3	1.9	65.57	2.83	12.16	35.40	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	2070	7	15.0	3.26	.22	12.00	48.98	.61	.000
	ADD [0000 + 2070]	0606	6	1.9	68.83	2.98	12.13	36.04	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	2080	8	15.0	1.14	.00	12.00	.82	.01	.000
	ADD [0606 + 2080]	0607	7	1.9	69.97	2.98	12.13	35.47	n/a	.000
	CALIB NASHYD [CN=86.0] [N= 3.0:Tp= .07]	2090	9	15.0	2.37	.02	12.00	6.27	.08	.000
	ADD [0607 + 2090]	0608	8	1.9	72.34	3.00	12.09	34.51	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.14	12.00	48.42	.60	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.06	12.00	55.01	.68	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.20	12.00	50.04	n/a	.000
	ADD [0608 + 0609]	0610	10	1.9	75.59	3.17	12.09	35.18	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.10	12.00	54.28	.68	.000
	ADD [2130 + 0610]	0612	2	1.9	76.93	3.24	12.06	35.51	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.01	12.00	6.51	.08	.000
	ADD [0612 + 2140]	0613	3	1.9	77.69	3.25	12.06	35.23	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	2150	5	15.0	2.40	.07	12.00	19.32	.24	.000
	ADD [0613 + 2150]	0614	4	1.9	80.09	3.31	12.06	34.75	n/a	.000
	ADD [0507 + 0614]	0614	10	1.9	122.68	4.82	12.09	34.51	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	3010	1	15.0	3.04	.04	12.00	9.44	.12	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3020	2	15.0	6.46	.42	12.00	46.97	.58	.000
	ADD [3010 + 3020]	0701	4	15.0	9.50	.47	12.00	34.96	n/a	.000
*	RESRVR [4 : 0701] {ST= .06 ha.m }	0002	1	15.0	9.50	.58	12.00	34.96	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	3030	3	15.0	2.02	.14	12.00	53.00	.66	.000
	ADD [0002 + 3030]	0702	2	15.0	11.52	.73	12.00	38.12	n/a	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	3040	4	15.0	4.76	.28	12.00	50.99	.63	.000
	ADD [0702 + 3040]	0703	3	15.0	16.28	1.01	12.00	41.88	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .21]	3050	5	15.0	26.45	1.32	12.00	33.88	.42	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3060	6	15.0	2.27	.13	12.00	46.97	.58	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .09]	3070	7	15.0	3.77	.08	12.00	13.89	.17	.000
	ADD [3050 + 3060]	0704	4	15.0	28.72	1.45	12.00	34.91	n/a	.000
	ADD [0704 + 3070]	0705	5	15.0	32.49	1.53	12.00	32.47	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .18]	3080	8	15.0	13.69	.66	12.00	31.87	.40	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	3081	1	15.0	3.25	.22	12.00	49.59	.62	.000

```
     ADD [3080 + 3081]  0706  6 15.0    16.94      .88 12.00  35.27  n/a     .000
     -----------------
     ADD [0705 + 0706]  0707  7 15.0    49.43     2.41 12.00  33.43  n/a     .000
     -----------------
     CALIB NASHYD       3090  9 15.0     1.04      .01 12.00   4.14  .05     .000
     [CN=80.0        ]
     [ N= 3.0:Tp=  .07]
     -----------------
     CALIB NASHYD       3100  6 15.0     4.68      .11 12.00  15.59  .19     .000
     [CN=85.0        ]
     [ N= 3.0:Tp=  .09]
     -----------------
   * CALIB STANDHYD     3101  1 15.0     3.07      .21 12.00  49.59  .62     .000
     [I%=19.0:S%= 7.00]
     -----------------
   * CALIB STANDHYD     3102  2 15.0     1.30      .08 12.00  48.42  .60     .000
     [I%=18.0:S%= 4.00]
     -----------------
     ADD [3100 + 3101]  0708  8 15.0     7.75      .32 12.00  29.06  n/a     .000
     -----------------
     ADD [0708 + 3090]  0709  1 15.0     8.79      .32 12.00  26.11  n/a     .000
     -----------------
     ADD [0709 + 3102]  0710  6 15.0    10.09      .40 12.00  28.98  n/a     .000
     -----------------
     ADD [0707 + 0710]  0711  8 15.0    59.52     2.81 12.00  32.68  n/a     .000
     -----------------
     CALIB NASHYD       3110  1 15.0     1.21      .02 12.00  11.69  .15     .000
     [CN=85.0        ]
     [ N= 3.0:Tp=  .08]
     -----------------
     ADD [0711 + 3110]  0712  6 15.0    60.73     2.83 12.00  32.26  n/a     .000
     -----------------
     RESRVR [ 6 : 0712] 0000  9  1.9    60.73     2.55 12.06  29.20  n/a     .000
     {ST=    .38 ha.m }
     -----------------
   * CALIB STANDHYD     3120  2 15.0     2.96      .20 12.00  53.85  .67     .000
     [I%=20.0:S%= 2.00]
     -----------------
     ADD [0000 + 3120]  0713  1  1.9    63.69     2.75 12.03  30.35  n/a     .000
     -----------------
     ADD [0703 + 0713]  0714  2  1.9    79.97     3.74 12.00  32.69  n/a     .000
     -----------------
     ADD [0614 + 0714]  0714  3  1.9   202.65     8.46 12.03  33.79  n/a     .000
     -----------------
 ** END OF SIMULATION :   4

*****************************************************************************



 ****************************
 ** SIMULATION NUMBER:   5 **
 ****************************

 W/E COMMAND             HYD ID   DT     AREA   Qpeak Tpeak   R.V. R.C.    Qbase
                                  min    ha      cms   hrs     mm           cms

     START @    .00 hrs
     -----------------
     READ STORM                  15.0
     [ Ptot= 97.00 mm ]
     fname :PRSN-SCS.025
     remark:25 YR 24 HR SCS II - PEARSON AIRPORT
     -----------------
     CALIB NASHYD       2010  1 15.0    16.95     1.01 12.00  39.78  .41     .000
     [CN=78.0        ]
     [ N= 3.0:Tp=  .16]
     -----------------
     CALIB NASHYD       2030  3 15.0    12.83      .75 12.00  38.64  .40     .000
     [CN=78.0        ]
     [ N= 3.0:Tp=  .15]
     -----------------
     CALIB NASHYD       2060  6 15.0     2.93      .20 12.00  46.85  .48     .000
     [CN=84.0        ]
     [ N= 3.0:Tp=  .16]
     -----------------
     ADD [2010 + 2030]  0601  9 15.0    29.78     1.76 12.00  39.29  n/a     .000
     -----------------
     ADD [0601 + 2060]  0602  1 15.0    32.71     1.96 12.00  39.97  n/a     .000
     -----------------
   * CALIB STANDHYD     2020  2 15.0     6.01      .49 12.00  63.98  .66     .000
     [I%=19.0:S%= 5.00]
     -----------------
   * CALIB STANDHYD     2031  3 15.0     3.47      .28 12.00  63.98  .66     .000
     [I%=19.0:S%= 7.00]
     -----------------
     ADD [2020 + 2031]  0603  9 15.0     9.48      .76 12.00  63.98  n/a     .000
     -----------------
   * CALIB STANDHYD     2040  4 15.0    10.95      .86 12.00  63.98  .66     .000
     [I%=19.0:S%= 6.00]
     -----------------
   * CALIB STANDHYD     2041  5 15.0     7.25      .63 12.00  63.98  .66     .000
     [I%=19.0:S%= 8.00]
     -----------------
```

	ADD [2040 + 2041]	0604	8	15.0	18.20	1.49	12.00	63.98	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	2050	5	15.0	2.17	.19	12.00	63.98	.66	.000
	ADD [0604 + 2050]	0605	7	15.0	20.37	1.67	12.00	63.98	n/a	.000
	ADD [0605 + 0603]	0606	8	15.0	29.85	2.44	12.00	63.98	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	1010	10	15.0	8.86	.72	12.00	63.98	.66	.000
*	CALIB STANDHYD [I%=25.0:S%= 4.00]	1020	2	15.0	1.96	.17	12.00	65.41	.67	.000
	ADD [1010 + 1020]	0901	7	15.0	10.82	.89	12.00	64.24	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 5.00]	1030	3	15.0	3.11	.27	12.00	63.98	.66	.000
	ADD [0901 + 1030]	0902	2	15.0	13.93	1.17	12.00	64.18	n/a	.000
	ADD [0602 + 0606]	0999	3	15.0	62.56	4.40	12.00	51.42	n/a	.000
	ADD [0902 + 0999]	0999	1	15.0	76.49	5.57	12.00	53.75	n/a	.000
	RESRVR [1 : 0999] {ST= 1.10 ha.m }	0000	8	1.9	76.49	4.35	12.13	47.21	n/a	.000
	DIVERT HYD	0000	8	1.9	76.49	4.35	12.13	47.21	n/a	.000
	Outflow	0002	2	1.9	14.90	.81	12.13	47.21	n/a	.000
	Outflow	0003	3	1.9	61.59	3.53	12.13	47.21	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .05]	1040	4	15.0	.61	.00	12.00	.66	.01	.000
	ADD [1040 + 0002]	0999	5	1.9	15.51	.81	12.13	45.38	n/a	.000
	CHANNEL[5 : 0999]	0000	1	1.9	15.51	.80	12.19	45.31	n/a	.000
	CALIB NASHYD [CN=83.0] [N= 3.0:Tp= .15]	1050	5	15.0	6.82	.44	12.00	43.50	.45	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	1060	6	15.0	2.67	.21	12.00	65.41	.67	.000
	ADD [1050 + 1060]	0999	2	15.0	9.49	.65	12.00	49.66	n/a	.000
	ADD [0000 + 0999]	0999	4	1.9	25.00	1.39	12.00	46.96	n/a	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	1070	7	15.0	9.73	.86	12.00	63.98	.66	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	2120	2	15.0	2.58	.23	12.00	63.98	.66	.000
	ADD [1070 + 2120]	0999	1	15.0	12.31	1.09	12.00	63.98	n/a	.000
	ADD [0999 + 0999]	0999	2	1.9	37.31	2.48	12.00	52.57	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .13]	1080	8	15.0	4.54	.25	12.00	36.45	.38	.000
	ADD [0999 + 1080]	0506	5	1.9	41.85	2.73	12.00	50.82	n/a	.000
	RESRVR [5 : 0506] {ST= .43 ha.m }	0000	6	1.9	41.85	2.04	12.16	46.81	n/a	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .04]	1090	9	15.0	.48	.00	12.00	.19	.00	.000
	ADD [0000 + 1090]	0507	1	1.9	42.33	2.04	12.16	46.28	n/a	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	2061	6	15.0	4.24	.36	12.00	61.65	.64	.000
	ADD [2061 + 0003]	0999	2	1.9	65.83	3.82	12.09	48.14	n/a	.000
	CHANNEL[2 : 0999]	0000	3	1.9	65.83	3.81	12.09	48.11	n/a	.000
*	CALIB STANDHYD [I%=20.0:S%= 6.00]	2070	7	15.0	3.26	.28	12.00	63.24	.65	.000
	ADD [0000 + 2070]	0606	6	1.9	69.09	4.04	12.06	48.83	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .05]	2080	8	15.0	1.14	.00	12.00	1.14	.01	.000
	ADD [0606 + 2080]	0607	7	1.9	70.23	4.04	12.06	48.05	n/a	.000
	CALIB NASHYD	2090	9	15.0	2.37	.03	12.00	8.16	.08	.000

	[CN=86.0] [N= 3.0:Tp= .07]									
	ADD [0607 + 2090]	0608	8	1.9	72.60	4.06	12.06	46.75	n/a	.000
*	CALIB STANDHYD [I%=18.0:S%= 5.00]	2100	10	15.0	2.45	.19	12.00	62.66	.65	.000
*	CALIB STANDHYD [I%=35.0:S%= 4.00]	2110	7	15.0	.80	.07	12.00	69.75	.72	.000
	ADD [2100 + 2110]	0609	9	15.0	3.25	.26	12.00	64.41	n/a	.000
	ADD [0608 + 0609]	0610	10	1.9	75.85	4.30	12.06	47.51	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	2130	3	15.0	1.34	.12	12.00	69.06	.71	.000
	ADD [2130 + 0610]	0612	2	1.9	77.19	4.40	12.06	47.88	n/a	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .07]	2140	4	15.0	.76	.01	12.00	8.64	.09	.000
	ADD [0612 + 2140]	0613	3	1.9	77.95	4.41	12.03	47.50	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .11]	2150	5	15.0	2.40	.10	12.00	26.07	.27	.000
	ADD [0613 + 2150]	0614	4	1.9	80.35	4.50	12.03	46.86	n/a	.000
	ADD [0507 + 0614]	0614	10	1.9	122.68	6.48	12.06	46.66	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .08]	3010	1	15.0	3.04	.06	12.00	12.63	.13	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3020	2	15.0	6.46	.55	12.00	61.08	.63	.000
	ADD [3010 + 3020]	0701	4	15.0	9.50	.61	12.00	45.57	n/a	.000
*	RESRVR [4 : 0701] {ST= .05 ha.m }	0002	1	15.0	9.50	.64	11.75	45.57	n/a	.000
*	CALIB STANDHYD [I%=30.0:S%= 6.00]	3030	3	15.0	2.02	.18	12.00	67.58	.70	.000
	ADD [0002 + 3030]	0702	2	15.0	11.52	.81	11.75	49.43	n/a	.000
*	CALIB STANDHYD [I%=25.0:S%= 5.00]	3040	4	15.0	4.76	.37	12.00	65.41	.67	.000
	ADD [0702 + 3040]	0703	3	15.0	16.28	1.14	11.75	54.10	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .21]	3050	5	15.0	26.45	1.77	12.00	45.70	.47	.000
*	CALIB STANDHYD [I%=15.0:S%= 6.00]	3060	6	15.0	2.27	.17	12.00	61.07	.63	.000
	CALIB NASHYD [CN=82.0] [N= 3.0:Tp= .09]	3070	7	15.0	3.77	.10	12.00	18.43	.19	.000
	ADD [3050 + 3060]	0704	4	15.0	28.72	1.94	12.00	46.92	n/a	.000
	ADD [0704 + 3070]	0705	5	15.0	32.49	2.04	12.00	43.61	n/a	.000
	CALIB NASHYD [CN=78.0] [N= 3.0:Tp= .18]	3080	8	15.0	13.69	.88	12.00	43.00	.44	.000
*	CALIB STANDHYD [I%=19.0:S%= 6.00]	3081	1	15.0	3.25	.29	12.00	63.98	.66	.000
	ADD [3080 + 3081]	0706	6	15.0	16.94	1.16	12.00	47.02	n/a	.000
	ADD [0705 + 0706]	0707	7	15.0	49.43	3.21	12.00	44.78	n/a	.000
	CALIB NASHYD [CN=80.0] [N= 3.0:Tp= .07]	3090	9	15.0	1.04	.01	12.00	5.53	.06	.000
	CALIB NASHYD [CN=85.0] [N= 3.0:Tp= .09]	3100	6	15.0	4.68	.14	12.00	20.36	.21	.000
*	CALIB STANDHYD [I%=19.0:S%= 7.00]	3101	1	15.0	3.07	.27	12.00	63.98	.66	.000
*	CALIB STANDHYD [I%=18.0:S%= 4.00]	3102	2	15.0	1.30	.10	12.00	62.66	.65	.000

```
      ADD [3100 + 3101]  0708  8 15.0    7.75     .41 12.00  37.64  n/a    .000
      -----------------
      ADD [0708 + 3090]  0709  1 15.0    8.79     .42 12.00  33.84  n/a    .000
      -----------------
      ADD [0709 + 3102]  0710  6 15.0   10.09     .52 12.00  37.55  n/a    .000
      -----------------
      ADD [0707 + 0710]  0711  8 15.0   59.52    3.73 12.00  43.56  n/a    .000
      -----------------
      CALIB NASHYD       3110  1 15.0    1.21     .03 12.00  15.30  .16    .000
      [CN=85.0        ]
      [ N= 3.0:Tp=  .08]
      -----------------
      ADD [0711 + 3110]  0712  6 15.0   60.73    3.76 12.00  42.99  n/a    .000
      -----------------
      RESRVR [ 6 : 0712] 0000  9  1.9   60.73    3.43 12.03  39.93  n/a    .000
      {ST=   .44 ha.m }
      -----------------
   *  CALIB STANDHYD     3120  2 15.0    2.96     .27 12.00  68.82  .71    .000
      [I%=20.0:S%= 2.00]
      -----------------
      ADD [0000 + 3120]  0713  1  1.9   63.69    3.68 12.03  41.27  n/a    .000
      -----------------
      ADD [0703 + 0713]  0714  2  1.9   79.97    4.77 12.00  43.88  n/a    .000
      -----------------
      ADD [0614 + 0714]  0714  3  1.9  202.65   11.19 12.03  45.56  n/a    .000
      -----------------
   ** END OF SIMULATION :   5

*****************************************************************************


   ****************************
   ** SIMULATION NUMBER:   6 **
   ****************************

   W/E COMMAND           HYD ID   DT    AREA   Qpeak Tpeak   R.V. R.C.    Qbase
                                  min    ha     cms   hrs     mm           cms

      START @    .00 hrs
      -----------------
      READ STORM                 15.0
      [ Ptot=109.30 mm ]
      fname :PRSN-SCS.050
      remark:10 YR 24 HR SCS II - PEARSON AIRPORT
      -----------------
      CALIB NASHYD       2010  1 15.0   16.95    1.21 12.00  47.76  .44    .000
      [CN=78.0        ]
      [ N= 3.0:Tp=  .16]
      -----------------
      CALIB NASHYD       2030  3 15.0   12.83     .89 12.00  46.40  .42    .000
      [CN=78.0        ]
      [ N= 3.0:Tp=  .15]
      -----------------
      CALIB NASHYD       2060  6 15.0    2.93     .24 12.00  55.42  .51    .000
      [CN=84.0        ]
      [ N= 3.0:Tp=  .16]
      -----------------
      ADD [2010 + 2030]  0601  9 15.0   29.78    2.10 12.00  47.17  n/a    .000
      -----------------
      ADD [0601 + 2060]  0602  1 15.0   32.71    2.33 12.00  47.91  n/a    .000
      -----------------
   *  CALIB STANDHYD     2020  2 15.0    6.01     .61 12.00  74.93  .69    .000
      [I%=19.0:S%= 5.00]
      -----------------
   *  CALIB STANDHYD     2031  3 15.0    3.47     .33 12.00  74.93  .69    .000
      [I%=19.0:S%= 7.00]
      -----------------
      ADD [2020 + 2031]  0603  9 15.0    9.48     .93 12.00  74.93  n/a    .000
      -----------------
   *  CALIB STANDHYD     2040  4 15.0   10.95    1.02 12.00  74.94  .69    .000
      [I%=19.0:S%= 6.00]
      -----------------
   *  CALIB STANDHYD     2041  5 15.0    7.25     .74 12.00  74.94  .69    .000
      [I%=19.0:S%= 8.00]
      -----------------
      ADD [2040 + 2041]  0604  8 15.0   18.20    1.76 12.00  74.94  n/a    .000
      -----------------
   *  CALIB STANDHYD     2050  5 15.0    2.17     .22 12.00  74.93  .69    .000
      [I%=19.0:S%= 6.00]
      -----------------
      ADD [0604 + 2050]  0605  7 15.0   20.37    1.98 12.00  74.94  n/a    .000
      -----------------
      ADD [0605 + 0603]  0606  8 15.0   29.85    2.91 12.00  74.94  n/a    .000
      -----------------
   *  CALIB STANDHYD     1010 10 15.0    8.86     .90 12.00  74.94  .69    .000
      [I%=19.0:S%= 6.00]
      -----------------
   *  CALIB STANDHYD     1020  2 15.0    1.96     .20 12.00  76.38  .70    .000
      [I%=25.0:S%= 4.00]
      -----------------
      ADD [1010 + 1020]  0901  7 15.0   10.82    1.10 12.00  75.20  n/a    .000
      -----------------
   *  CALIB STANDHYD     1030  3 15.0    3.11     .32 12.00  74.93  .69    .000
```

```
    [I%=19.0:S%= 5.00]
    ------------------
    ADD [0901 + 1030]  0902  2 15.0   13.93    1.42 12.00  75.14  n/a   .000
    ------------------
    ADD [0602 + 0606]  0999  3 15.0   62.56    5.24 12.00  60.81  n/a   .000
    ------------------
    ADD [0902 + 0999]  0999  1 15.0   76.49    6.66 12.00  63.42  n/a   .000
    ------------------
    RESRVR [ 1 : 0999] 0000  8  1.9   76.49    5.17 12.09  56.88  n/a   .000
    {ST=  1.25 ha.m }
    ------------------
    DIVERT HYD         0000  8  1.9   76.49    5.17 12.09  56.88  n/a   .000
       Outflow         0002  2  1.9   14.78     .96 12.09  56.88  n/a   .000
       Outflow         0003  3  1.9   61.71    4.19 12.09  56.88  n/a   .000
    ------------------
    CALIB NASHYD       1040  4 15.0     .61     .00 12.00    .80  .01   .000
    [CN=80.0         ]
    [ N= 3.0:Tp=  .05]
    ------------------
    ADD [1040 + 0002]  0999  5  1.9   15.39     .96 12.09  54.66  n/a   .000
    ------------------
    CHANNEL[ 5 : 0999] 0000  1  1.9   15.39     .95 12.16  54.59  n/a   .000
    ------------------
    CALIB NASHYD       1050  5 15.0    6.82     .52 12.00  51.56  .47   .000
    [CN=83.0         ]
    [ N= 3.0:Tp=  .15]
    ------------------
 *  CALIB STANDHYD     1060  6 15.0    2.67     .25 12.00  76.38  .70   .000
    [I%=25.0:S%= 5.00]
    ------------------
    ADD [1050 + 1060]  0999  2 15.0    9.49     .76 12.00  58.54  n/a   .000
    ------------------
    ADD [0000 + 0999]  0999  4  1.9   24.88    1.66 12.00  56.10  n/a   .000
    ------------------
 *  CALIB STANDHYD     1070  7 15.0    9.73    1.01 12.00  74.94  .69   .000
    [I%=19.0:S%= 7.00]
    ------------------
 *  CALIB STANDHYD     2120  2 15.0    2.58     .27 12.00  74.93  .69   .000
    [I%=19.0:S%= 7.00]
    ------------------
    ADD [1070 + 2120]  0999  1 15.0   12.31    1.28 12.00  74.94  n/a   .000
    ------------------
    ADD [0999 + 0999]  0999  2  1.9   37.19    2.94 12.00  62.33  n/a   .000
    ------------------
    CALIB NASHYD       1080  8 15.0    4.54     .30 12.00  43.55  .40   .000
    [CN=80.0         ]
    [ N= 3.0:Tp=  .13]
    ------------------
    ADD [0999 + 1080]  0506  5  1.9   41.73    3.24 12.00  60.29  n/a   .000
    ------------------
    RESRVR [ 5 : 0506] 0000  6  1.9   41.73    2.41 12.16  56.26  n/a   .000
    {ST=   .51 ha.m }
    ------------------
    CALIB NASHYD       1090  9 15.0     .48     .00 12.00    .23  .00   .000
    [CN=85.0         ]
    [ N= 3.0:Tp=  .04]
    ------------------
    ADD [0000 + 1090]  0507  1  1.9   42.21    2.41 12.16  55.62  n/a   .000
    ------------------
 *  CALIB STANDHYD     2061  6 15.0    4.24     .42 12.00  72.48  .66   .000
    [I%=15.0:S%= 6.00]
    ------------------
    ADD [2061 + 0003]  0999  2  1.9   65.95    4.56 12.06  57.88  n/a   .000
    ------------------
    CHANNEL[ 2 : 0999] 0000  3  1.9   65.95    4.55 12.09  57.86  n/a   .000
    ------------------
 *  CALIB STANDHYD     2070  7 15.0    3.26     .33 12.00  74.12  .68   .000
    [I%=20.0:S%= 6.00]
    ------------------
    ADD [0000 + 2070]  0606  6  1.9   69.21    4.82 12.06  58.62  n/a   .000
    ------------------
    CALIB NASHYD       2080  8 15.0    1.14     .00 12.00   1.39  .01   .000
    [CN=82.0         ]
    [ N= 3.0:Tp=  .05]
    ------------------
    ADD [0606 + 2080]  0607  7  1.9   70.35    4.83 12.06  57.70  n/a   .000
    ------------------
    CALIB NASHYD       2090  9 15.0    2.37     .03 12.00   9.60  .09   .000
    [CN=86.0         ]
    [ N= 3.0:Tp=  .07]
    ------------------
    ADD [0607 + 2090]  0608  8  1.9   72.72    4.85 12.06  56.13  n/a   .000
    ------------------
 *  CALIB STANDHYD     2100 10 15.0    2.45     .22 12.00  73.53  .67   .000
    [I%=18.0:S%= 5.00]
    ------------------
 *  CALIB STANDHYD     2110  7 15.0     .80     .09 12.00  80.91  .74   .000
    [I%=35.0:S%= 4.00]
    ------------------
    ADD [2100 + 2110]  0609  9 15.0    3.25     .31 12.00  75.34  n/a   .000
    ------------------
    ADD [0608 + 0609]  0610 10  1.9   75.97    5.14 12.03  56.95  n/a   .000
    ------------------
 *  CALIB STANDHYD     2130  3 15.0    1.34     .14 12.00  80.25  .73   .000
    [I%=30.0:S%= 6.00]
    ------------------
```

	ADD [2130 + 0610]	0612	2	1.9	77.31	5.27	12.03	57.35	n/a	.000
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     ------------------
     ADD [0709 + 3102]   0710  6 15.0    10.09      .70 12.00  50.66  n/a     .000
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     ADD [0707 + 0710]   0711  8 15.0    59.52     5.14 12.00  60.50  n/a     .000
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     [CN=85.0         ]
     [ N= 3.0:Tp=  .08]
     ------------------
     ADD [0711 + 3110]   0712  6 15.0    60.73     5.18 12.00  59.71  n/a     .000
     ------------------
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     ------------------
     ADD [0614 + 0714]   0714  3  1.9   202.65    15.40 12.00  63.73  n/a     .000
     ------------------
FINISH
==========================================================================
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Appendix A to Planning Report 99-37

Inglewood Village
Community Design Guidelines

Adopted by Town of Caledon Council
July 12, 1999

Prepared By:

John van Nostrand Associates

Prepared For:

Town of Caledon
Credit Valley Conservation
Region of Peel

CVC

TOWN OF CALEDON

Region of Peel

1. INTRODUCTION

Community Design has formed an important aspect of the overall planning process for Inglewood. It serves to bridge the gap between broad planning objectives and concrete design proposals for the development of new communities, and the upgrading of those that already exists.. At the same time, it allows existing residents and the Town to imbed their aspirations and objectives for Inglewood into community design guidelines that developers are encouraged to follow. Finally, it is clear that throughout the GTA, good community design adds value to existing and new development, and results in communities which residents - new and old - are apt to be more proud of.

The overriding principle that has guided the preparation of the guidelines has been residents' desires to ensure that the village character of Inglewood is maintained and strengthened throughout any development or redevelopment process. Consequently, the challenge of the accompanying Village Study has been to determine what the "character" of Inglewood is, and to identify how this character is defined from a community design point-of-view. In fact, Inglewood, as it currently exists, combines an historic village Core with a number of more contemporary rural estate developments - and all these communities contribute to the overall image of the Village.

At this point, the Guidelines are of a conceptual nature only, and are intended to guide developers, and their architects and planners, in the preparation of proposed new plans and residential designs. They embody local resident's views on what is most important about their Village, and what characteristics can be built on in future. As such, the Town will want to ensure that they are incorporated as comprehensively as possible into new development applications, and that detailed community designs associated with these applications builds on this conceptual base. Where gaps or omissions are identified, proponents should refer to existing village characteristics in order to resolve design issues.

2. COMMUNITY DESIGN PRINCIPLES

The following principles form the basis for the Conceptual Community Design Guidelines and will, in conjunction with the Guidelines, be used by the Town in its evaluation of development and/or redevelopment proposals submitted for parcels located within the Village boundaries.

Community Form - the form of all new development should be compatible with the existing forms of development found in Inglewood and all new development areas should be linked with the existing Village and its historic Core Area.

Land-Use - should encourage a mix of residential and employment land-use types, in order to encourage a range of housing types and to encourage local economic activities.

Population - population levels should be established for the Village which recognize its role within Caledon's "community of communities'" thus allowing for moderate growth while protecting the quality of life of existing residents.

Lot Size/Density - a variety of lot sizes is to be encouraged in all new developments within Inglewood, with provisions where possible for future infill and/or intensification. The Plan permits a range of densities that are based on those found within the existing Village, in order to allow for efficient use of land and cost-effective services and social facilities.

Lot Development - conceptual guidelines for setbacks, building heights, building coverage, accessory uses, and parking have been drawn up in order to ensure that new development or redevelopment is compatible with that which already exists.

Housing and Building Design - should encourage a variety of house forms that are compatible with the range of historic and contemporary house types found within the existing Village. These should also encourage a range of accommodation for a variety of households including families, singles, retirees, etc.

Streets and Roads - Should provide for a balance of vehicular and pedestrian movement, parking (where appropriate), and recreational modes of movement (e.g. hiking, cross country skiing, bicycling, horse-back riding, etc.). Existing streetscapes should be enhanced and new streetscapes should draw on the desirable characteristics of the historic Core Area (e.g., tree-lined; intimate; pedestrian friendly).

Natural Areas - should be incorporated into the community structure as open spaces and green corridors, with appropriate setbacks.

Community Facilities - existing facilities should be maintained and improved, as necessary, to meet the needs of planned growth, and should be accessible on foot, as well as by car.

Views and Panoramas - important views and/or panoramas of the Niagara Escarpment, the Credit River, and other significant natural and man-made features, should be identified and protected.

Recreation Corridors - the existing network of trails should be protected and enhanced to strengthen the role of the community as an important recreational node, and to recognize the potential economic and social benefits of this role.

3. CONCEPTUAL COMMUNITY DESIGN GUIDELINES

The following set forth conceptual community design guidelines to accompany the Village Plan. Conceptual means that they illustrate how the objectives of the Plan are proposed to be reflected in infrastructure, subdivision, and housing design in new development areas. It will be left to individual developers to interpret these concepts in greater detail in conjunction with the preparation of final detailed designs for each new development, prior to their submission to the Town for final approval.

3.1 PUBLIC REALM:

Main Street:
Figures A1 - A3 present conceptual cross sections of existing and proposed improvements to McLaughlin Road (i.e. Inglewood's Main Street). Final engineering design will be established through the conventional development approvals process. **Figure A2** includes the provision of selective new tree planting, plus the designation of dedicated parking lanes on both sides of Main Street (replacing the existing rolled curbs and asphalt boulevards), while retaining one traffic lane in each direction. The intent of these changes is to improve the pedestrian space while, at the same time, slowing traffic down through the village core. Further north, where new development abuts the Main Street, it is recommended that pedestrian space on the west side of the Street be widened even further to include a double row of new trees defining a proper walking trail (i.e. an important link in the Trans-Canada Trail) parallel to Main Street. This trail should be designed to accommodate hiking, cycling, and horseback riding, in addition to conventional walking. The total right-of-way in these locations would be widened from the existing 20 metres to 26 metres, in accordance with the Town's Official Plan. However, this widening is proposed to be used primarily for pedestrian, trail, and streetscape purposes, within the Village boundaries. Appropriate streetscape standards are also to be extended south of the Village core to strengthen pedestrian links to the Lloyd Wilson Arena and enhance the southerly approach to the core.

New Connector Road:
A new Connector Road is proposed to link certain existing connectors with those proposed in new development areas, to create a continuous circuit linking separate neighbourhoods with each other and with the core. **Figure B2** presents a conceptual cross-section for this road. Like the Main Street, this proposes a two traffic lanes, with parking on one side, and a tree-lined walking trail and sidewalk to either side of the

pavement. Where it is considered desirable not to allow for continuous vehicular traffic on this alignment, the walking trail would remain continuous as it provides pedestrian access to the wider regional trail system surrounding Inglewood.

Residential Streets:
New residential streets would range in size from 18 - 20 metres in width, be tree-lined, and be designed to accommodate normal residential traffic. While **Figure C2** assumes a curbed pavement, parking on one side, and finished sidewalks on both sides, this cross-section should be capable of being modified to include open swales and walking shoulders (i.e., as in **Figure C1** which illustrates one of Inglewood's historic residential streets), where local residents and Caledon's Engineering Department agree.

Street Lighting:
Street lights should be included in all new street and road designs. Consideration will need to be given to reducing night-time light pollution in the design and location of new fixtures, as well as to the fact that lighting will be required for both pedestrian and vehicular traffic.

Trails and Footpaths:
All existing and proposed trails and pedestrian footpaths throughout the Planning Area, including those located within designated street and road rights-of-way, should be finished in a similar material or materials to reinforce their continuity, and to differentiate them from conventional sidewalks. Detailed trail design is to be established during the approvals process for individual developments, subject to the approval of the Town.

Off-Street Parking (in Public Areas):
Existing public parking areas located at the Arena, the Library, and the Caledon Trailway (on the west side of Main Street) will likely need to be upgraded to accommodate increased parking. These should be designed to accommodate week-end visitors, a valuable source of economic development, and they should be developed in conjunction with adjacent facilities so as to reinforce these and make best use of available space. They will also require proper signage.

Public Open Space:
New public open space and parks should be designed using a lower-maintenance, naturalized, approach including the use of indigenous plant communities. New park design should also focus on environmental rehabilitation and restoration, wherever possible. A more formal approach to landscape design might be adopted on residential streets, and around community facilities, as appropriate. Detailed park and open space plans will be required as a condition of approval for new development.

Traffic Calming
Consideration will need to be given to reducing vehicular speeds, and encouraging a more effective balance of vehicular and pedestrian use along Main Street. Specific "traffic-calming" strategies may be required in order to establish methods for achieving overall goals. Strategies to be considered should be based on community design as much as transportation engineering. They would include creating "gateway" features at key entry points into the village; narrowing traffic pavements, widening sidewalks, and planting trees to reinforce the sense that drivers are within a residential setting; establishing major pedestrian cross-overs; encouraging increased residential frontage along Main Street, including more driveways accessed directly from Main Street; increasing the number of cross-streets intersecting with Main Street; improving on-street parking; considering the introduction of stop signs at key intersections and/or crossing points; etc.

3.2 PRIVATE REALM:

Subdivision Design:
New residential neighbourhoods should be subdivided to provide for a mix and variety of lot sizes, as discussed in the Inglewood Village Plan. However, while lots will vary in size, it is recommended that a modular frontage dimension be adopted in order to introduce consistent order and rhythm in terms of boundaries, fencing, landscaping, etc. For example, the surveyor's chain (i.e. 66 feet [20 metres] long) was used throughout the historic village to regulate street width (i.e., 33 or 66 feet), lot frontage (varying from 33 to 99 feet), and lot depth (i.e. typically 132 feet). The typical lot size in this area is 66 by 132 feet (i.e., 0.2 acres). These dimensions might also be used to define "wide, shallow" lots with frontages of, say, 99 or 132 feet (see **Figure E** - Lot Type 3). This approach also provides flexibility to adjust to changing market demands over the development period, without compromising the integrity of the overall community design. **Figures D** and **E** illustrate how similar dimensions should be used to regulate land division in the new development areas, while at the same time providing a variety of lot and block types. .

Lot Development Guidelines:
Figure F presents some preliminary regulations for the development of individual lots, of varying size and orientation. These introduce consistent front, side and rear yard setbacks that are based on the historic form of the village as well as Caledon's current rural building by-laws. They also illustrate recommended garage locations which seek to ensure that garage doors and on-lot, outdoor parking areas are recessed behind the front face of new housing (including Generic Lot C, which is illustrates a ''wide, shallow'' configuration). Garages with direct links to adjacent houses are permitted. **Figure G** illustrates complementary building envelope and height guidelines.

Building/Housing Design Guidelines:
It was considered premature, at this time, to draw up detailed housing design guidelines such as might be used to regulate building materials, detailed exterior design, front yard landscaping, etc. On the contrary, it is recommended that such designs, once prepared by the developer's architect, be presented to Inglewood residents and the Town for review and comment. In addition, the Town may elect to appoint peer reviewers to assist it and the community in its evaluation. These designs should be based on an analysis and evaluation of local, existing housing types. Clearly, every attempt should be made by the developer, to utilize materials, and detailed designs which, at least, refer to the character and materiality of existing housing and building design found in Inglewood. These are what gives the village its particular existing character and might include roof details, window sizes and openings, masonry patterns and/or colours, porch or veranda designs.

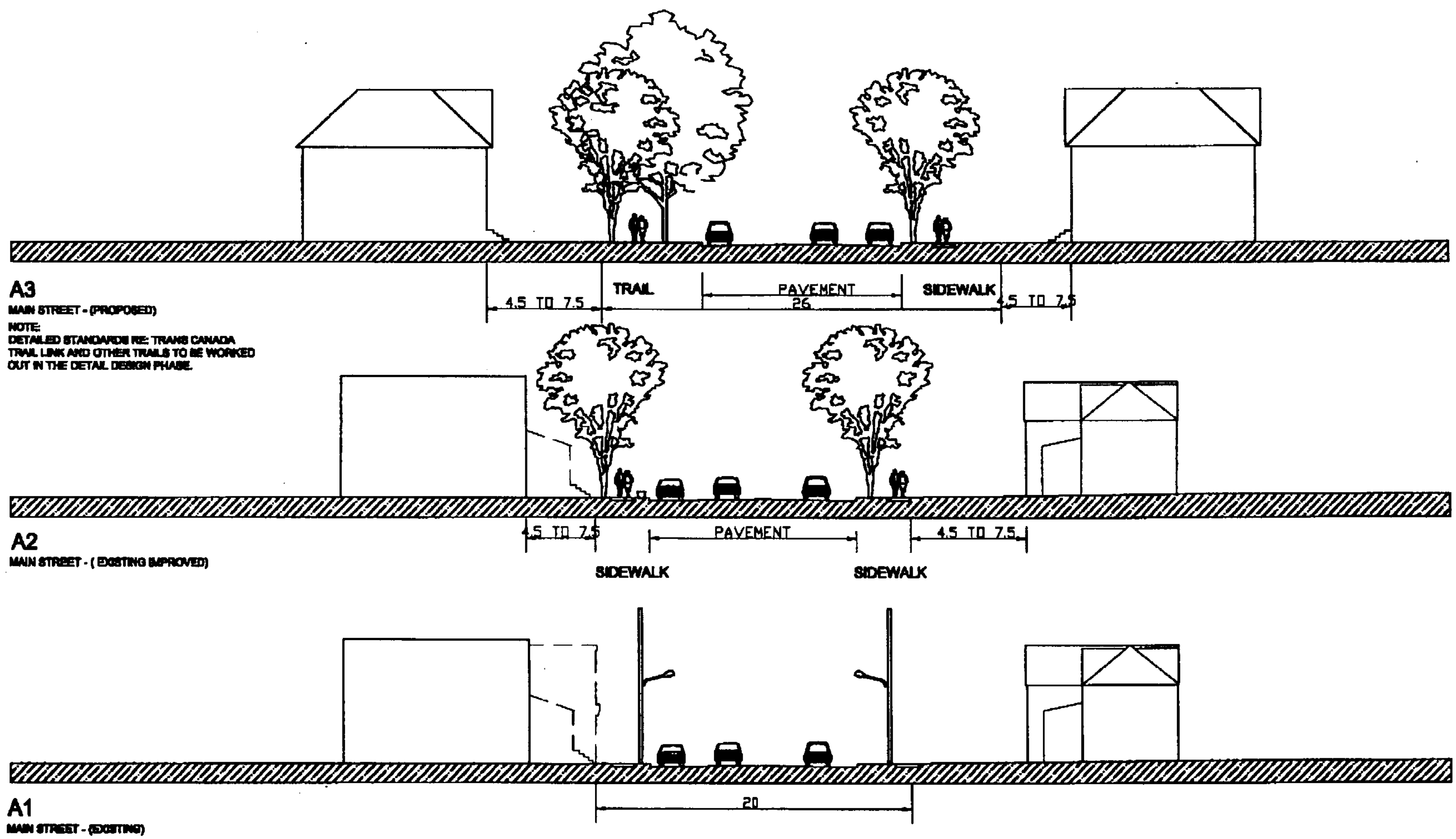
4 IMPLEMENTATION

The conceptual Community Design Guidelines are intended to be refined in detail by individual developers in the preparation of their submissions for development approvals. In turn, it is expected that these submissions will have been presented to the Inglewood Community prior to submission, and prior to the required formal Public Meeting.

Developers, working with their planners and architects, are expected to give full consideration to the Guidelines, and to incorporate, and elaborate on, them in their proposed detailed Community Plans. The development team should also refer to the "Landscape Standards and Urban Design Guidelines (25 May 1998)" which were approved by the Town for the West Bolton Secondary Plan Area and which will also be used in evaluating development submissions with respect to standard landscape details, specifications and approval and implementation procedures.

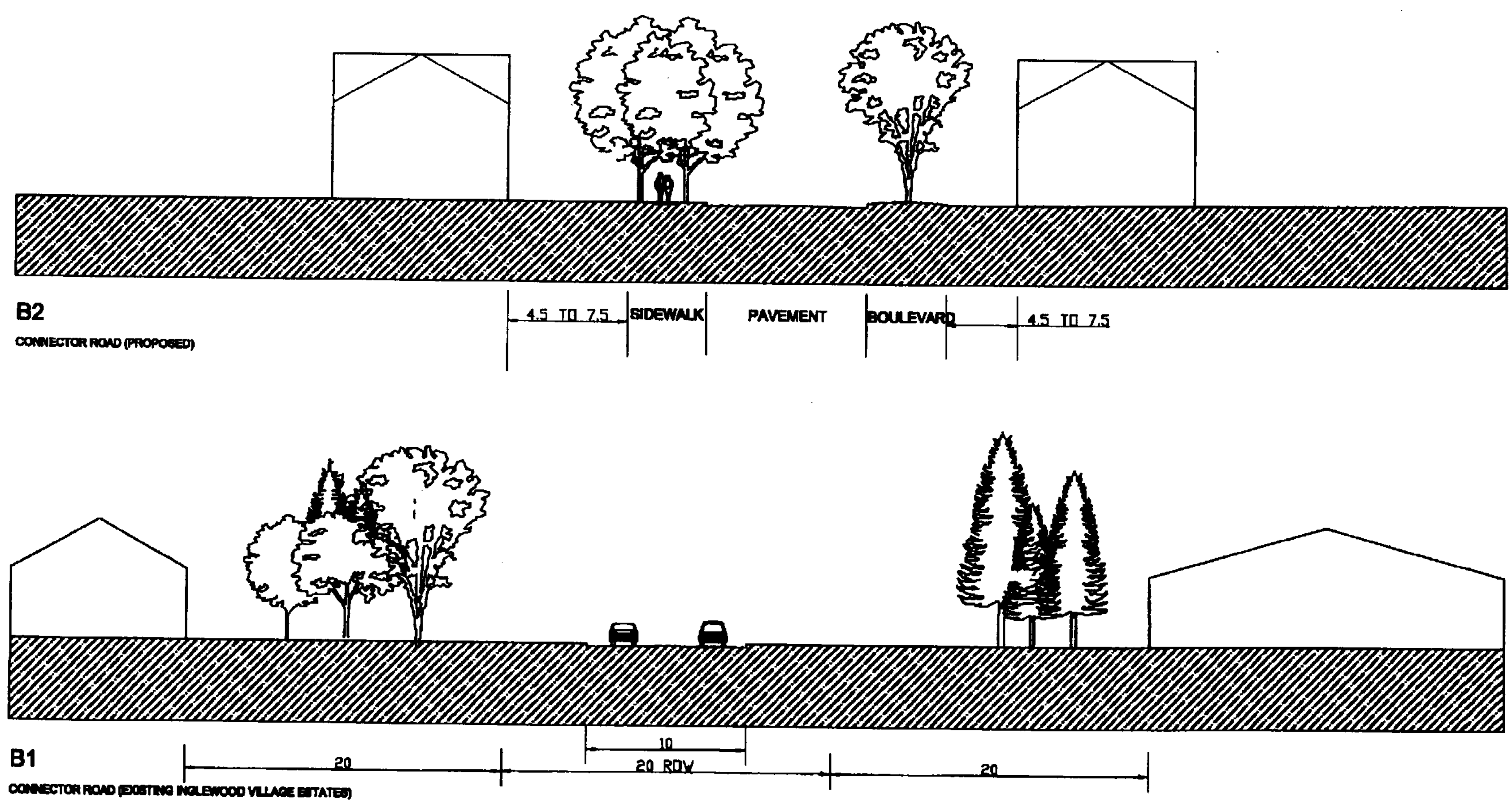
In turn, the Town will conduct its review of these plans and designs with both these Guidelines in hand, and will evaluate the extent to which submissions reflect the intentions of the Guidelines. It will make every effort to exercise flexibility in its evaluation, provided that the overall community design objectives raised in this document are addressed.

Similarly, the Town, the Region and other agencies will follow the Guidelines in designing and/or commissioning all future infrastructure and public works projects carried out in Inglewood - including, in particular those affecting the public realm within the Village.



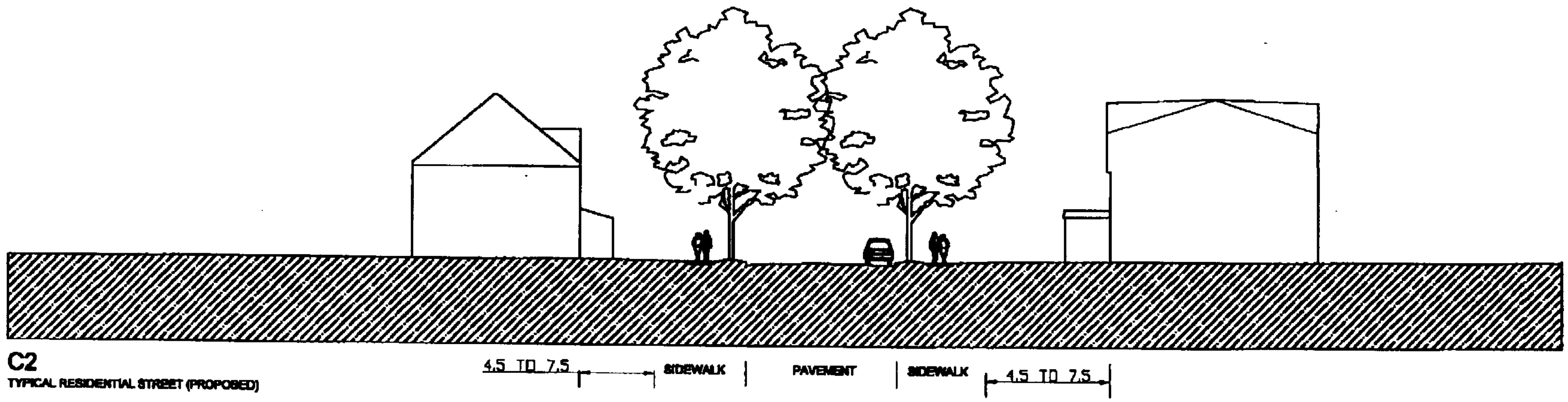
CONCEPTUAL DESIGN GUIDELINES

FIGURES A1-A3
MAIN STREET

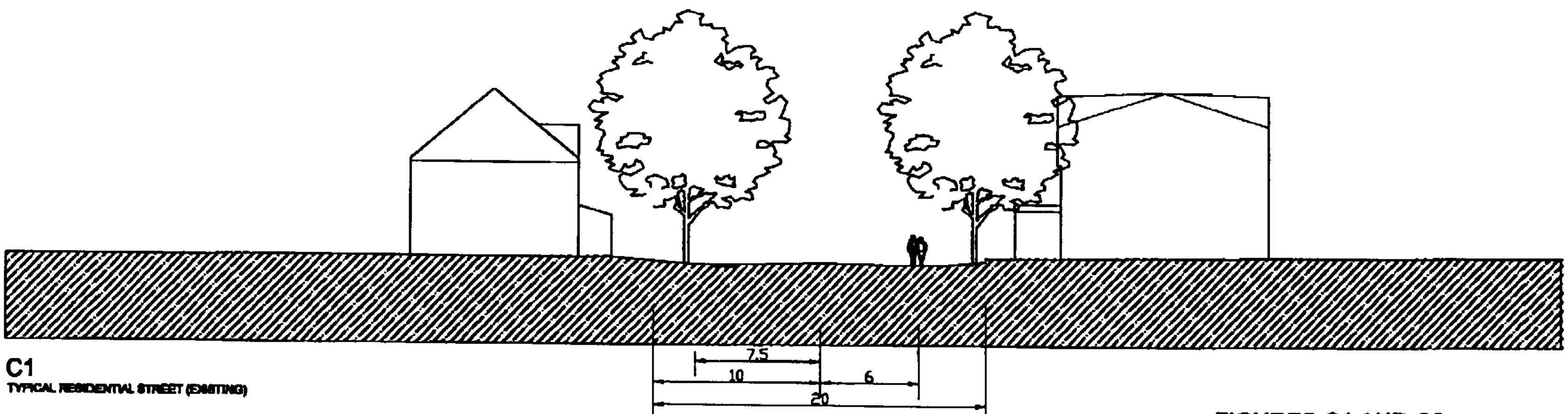


CONCEPTUAL DESIGN GUIDELINES

FIGURES B1 AND B2
CONNECTOR ROAD



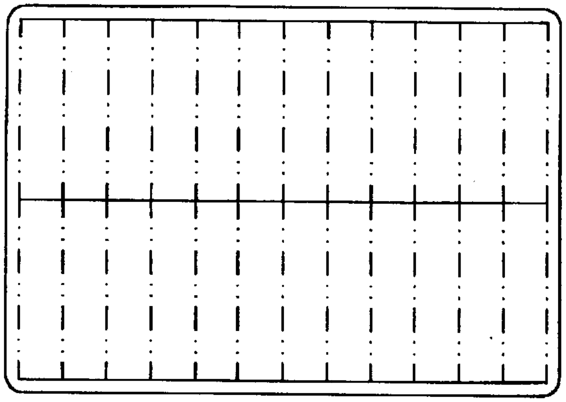
C2
TYPICAL RESIDENTIAL STREET (PROPOSED)



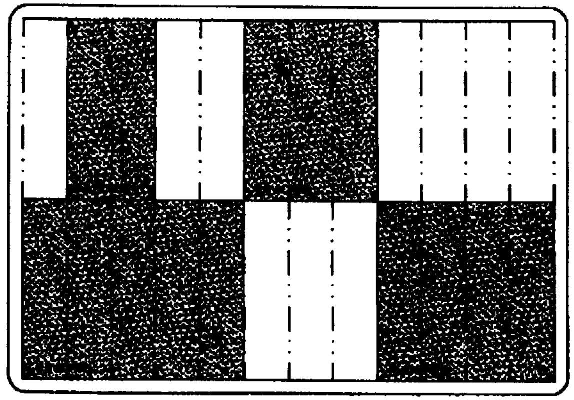
C1
TYPICAL RESIDENTIAL STREET (EXISTING)

CONCEPTUAL DESIGN GUIDELINES

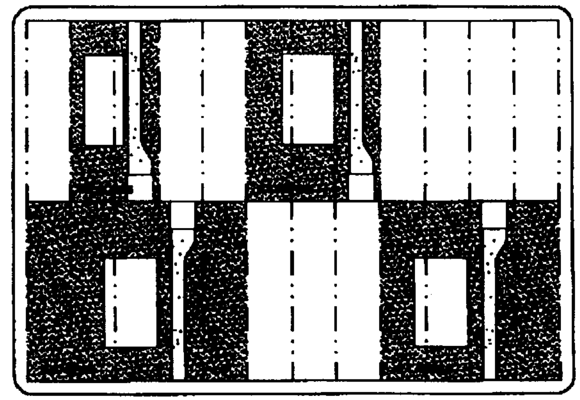
FIGURES C1 AND C2
TYPICAL RESIDENTIAL STREET



GENERIC (33'X132')



LOT TYPES



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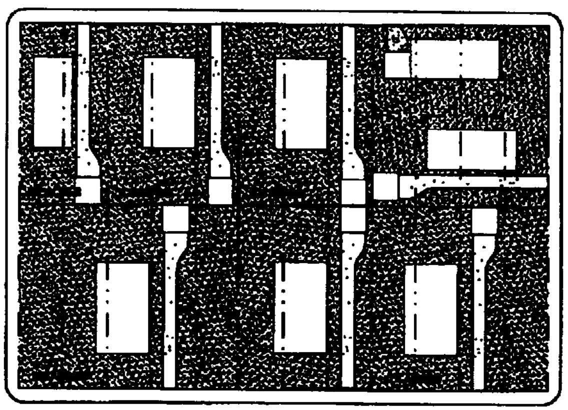
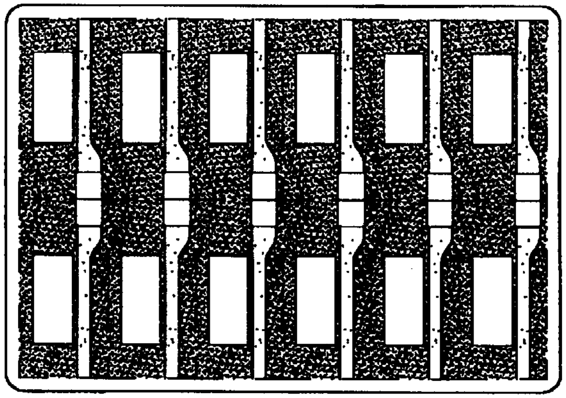


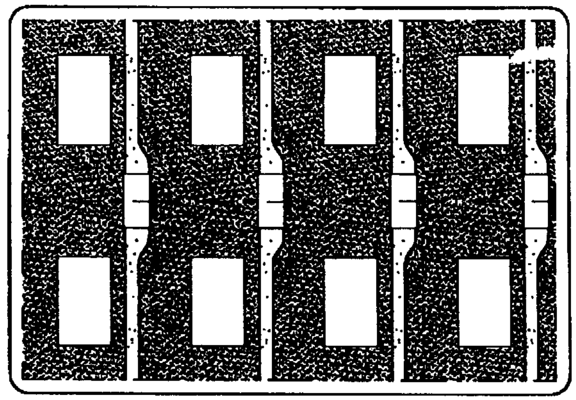
RANDOM LAYOUT OF LOT TYPES

CONCEPTUAL DESIGN GUIDELINES

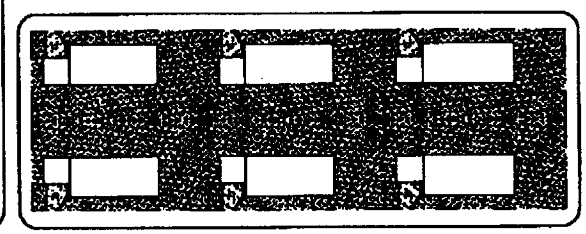
FIGURE D
BLOCK EVOLUTION



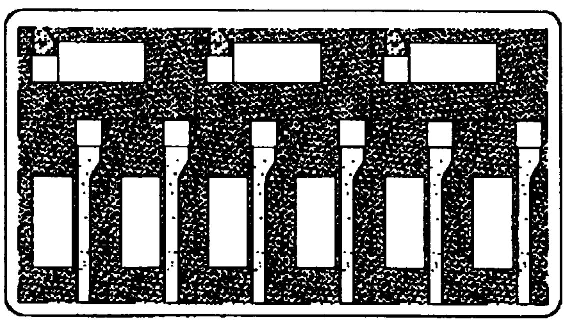
LOT A BLOCK



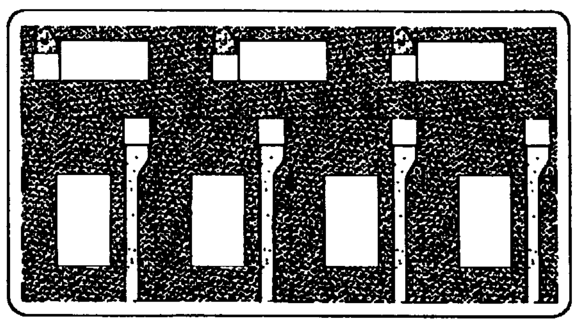
LOT B BLOCK



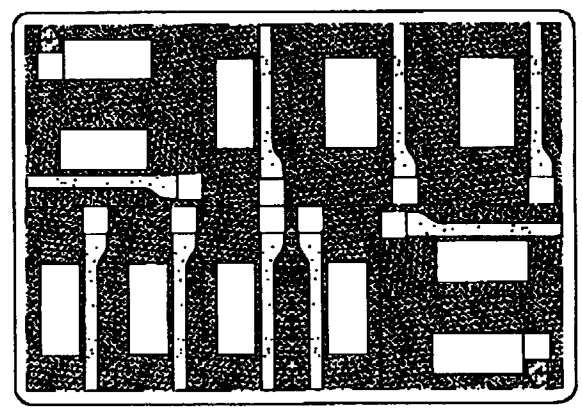
LOT C TYPE BLOCK



LOT A & C BLOCK



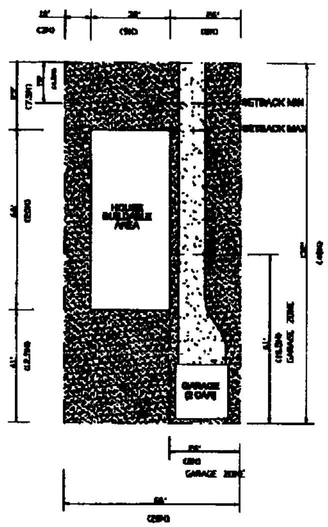
LOT B & C BLOCK

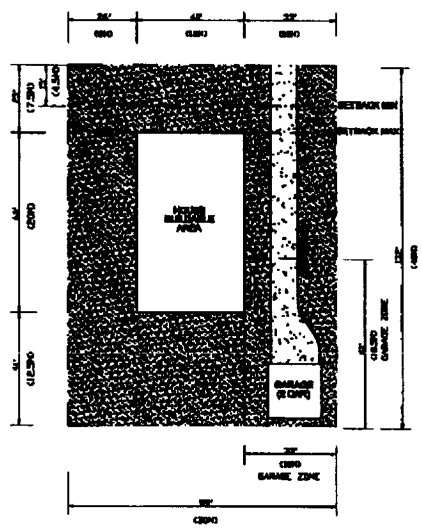


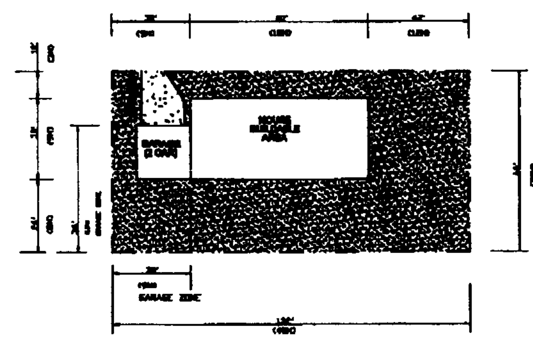
RANDOM LOT BLOCK

CONCEPTUAL DESIGN GUIDELINES

FIGURE E
BLOCK TYPES







LOT TYPE 1

LOT AREA
0.2 ACRES (800 SQ. M.)

BUILDABLE AREA:
1936 SQ FT (180SQ M) PER FLR
5812 SQ FT (540 SQ M) FOR THREE FLOORS

LOT TYPE 2

LOT AREA
0.3 ACRES (1200 SQ. M.)

BUILDABLE AREA:
2660 SQ FT (240 SQ M) PER FLR
7750 SQ FT (720 SQ M) FOR THREE FLOORS

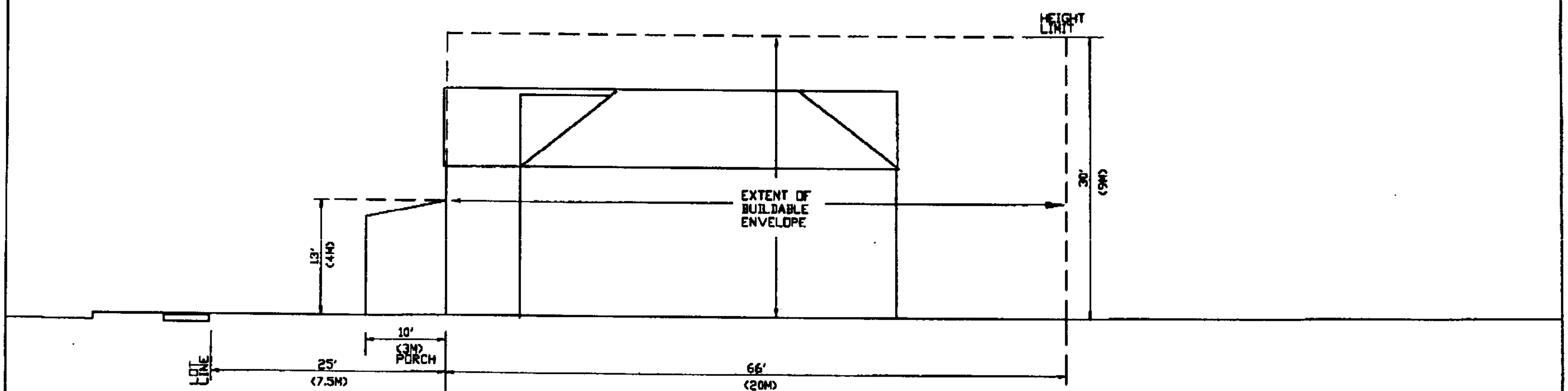
LOT TYPE 3

LOT AREA
0.2 ACRES (800 SQ. M.)

BUILDABLE AREA:
1936 SQ FT (180SQ M) PER FLR
5812 SQ FT (540 SQ M) FOR THREE FLOORS

FIGURE F
LOT DEVELOPMENT GUIDELINES

CONCEPTUAL DESIGN GUIDELINES



CONCEPTUAL DESIGN GUIDELINES

FIGURE G
BUILDING ENVELOPE GUIDELINES

DRAFT REPORT

Inglewood Village Environmental Management Plan

June 1999

Prepared By:

Credit Valley Conservation
Parish Geomorphic
Don Weatherbe and Associates

Prepared For:

Town of Caledon
Credit Valley Conservation
Region of Peel

CVC

TOWN OF CALEDON

Region of Peel

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APPENDICES

INTRODUCTION

1.1 Background

1.1.1 Inglewood Village Study

The Village of Inglewood is one of six medium sized communities located in the Town of Caledon. It is located on the main branch of the Credit River within Subwatershed 20. Figure 1.1 shows the location of the Village with respect to the Credit River Watershed. The Village is currently serviced by a municipal well and private septic systems. The 1996 Provincial Policy Statements (PPS) requires the protection/enhancement of the quality and quantity of groundwater and surface water resources and the function of sensitive recharge/discharge areas, headwaters and aquifers. The PPS promotes the use of communal services where full services are not available. This is supported by the Region of Peel's Official Plan. In addition, the Town of Caledon supports an ecosystem approach to planning with policies contained in OPA 124.

On March 27, 1997, the Region of Peel Council approved a "Water Protection Strategy and the Guidelines for the Provision of Communal Sewage Disposal Systems" (CSDS). This Water Protection Strategy sets out a community-based process known as a Servicing and Settlement Master Plan (SSMP) which is designed to address servicing, planning and environmental issues relating to all villages in a comprehensive manner. Given limited funds, SSMPs will be initiated for each village on a priority basis. It was then agreed by the Town and Region that Inglewood Village would be given the highest priority. The SSMP entitled the "Inglewood Village Study" was approved and detailed terms of reference were developed. The study was further divided into three components, Planning, Servicing and Environmental, and the Town of Caledon, Region of Peel and Credit Valley Conservation (CVC), respectively, have been designated to take the lead. The overall study is being managed by a Core Management Team (CMT) made up of representatives from the Town, Region and CVC. The Town of Caledon chairs the CMT. The overall management of the study, including coordination of the public consultation process and integration of the study components will be the responsibility of Town planning staff.

The issues related to Inglewood include: community form and character; rate of growth; compatibility and staging of new development; densities and population limits; assimilative capacity of the Credit River and surrounding Provincially Significant Wetlands; sustainable management of groundwater; viability of the commercial core; and, relevancy of work completed through the Inglewood Village Study. The overall study has been divided into four phases. Figure 1.2 shows the progression of the study and key points.

1.1.2 Environmental Component

The Environmental Component endeavors to develop and establish the key functions and linkages of the ecosystem within the study area. To do this, the approach taken was to break this component down into smaller sub-components, namely : hydrogeology, hydrology, fluvial geomorphology, terrestrial and fisheries. In addition, this component also includes an assimilative capacity analysis of the Credit River within the study area. The intent is to provide the Planning and Servicing Components of the

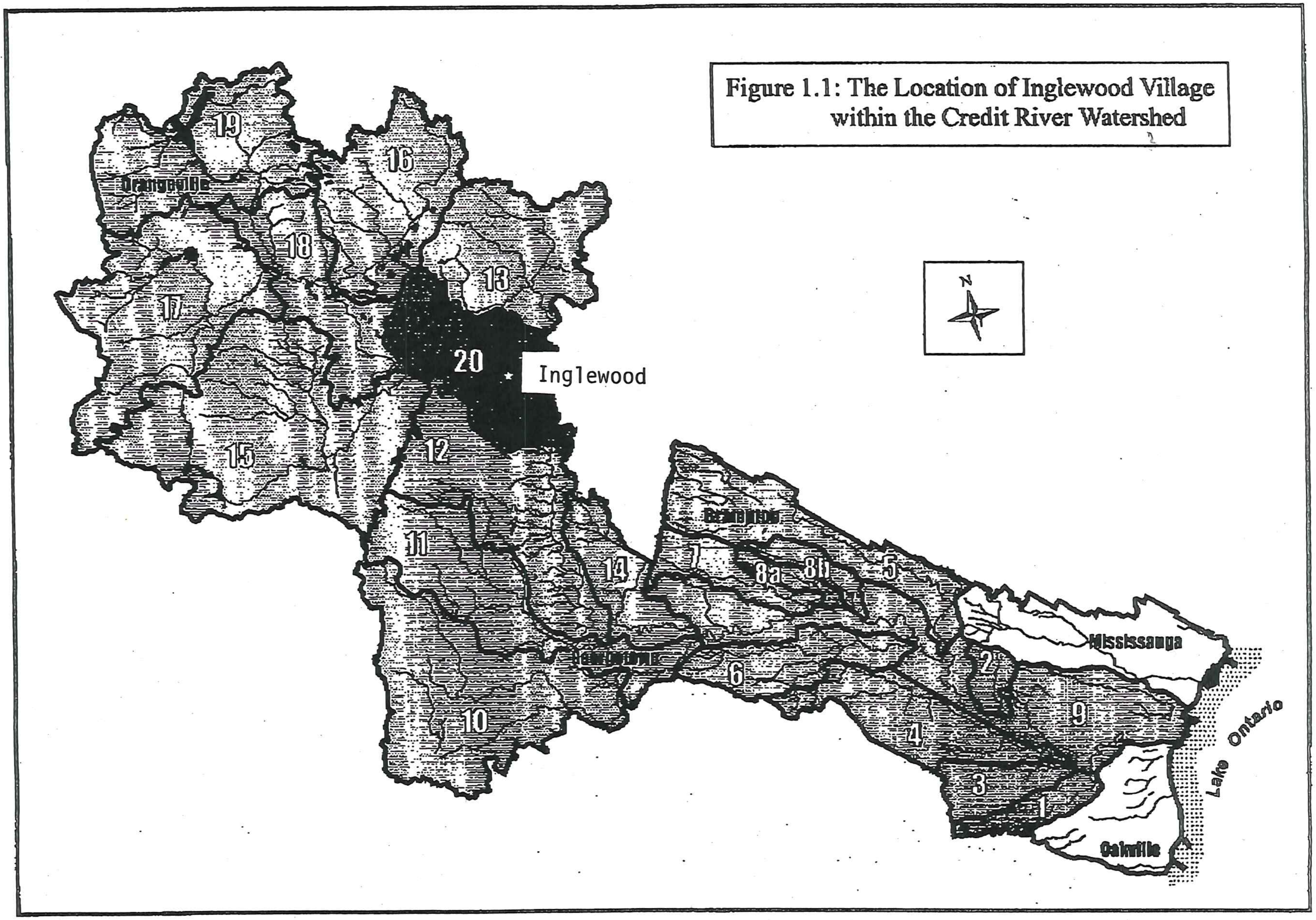

Figure 1.1: The Location of Inglewood Village within the Credit River Watershed

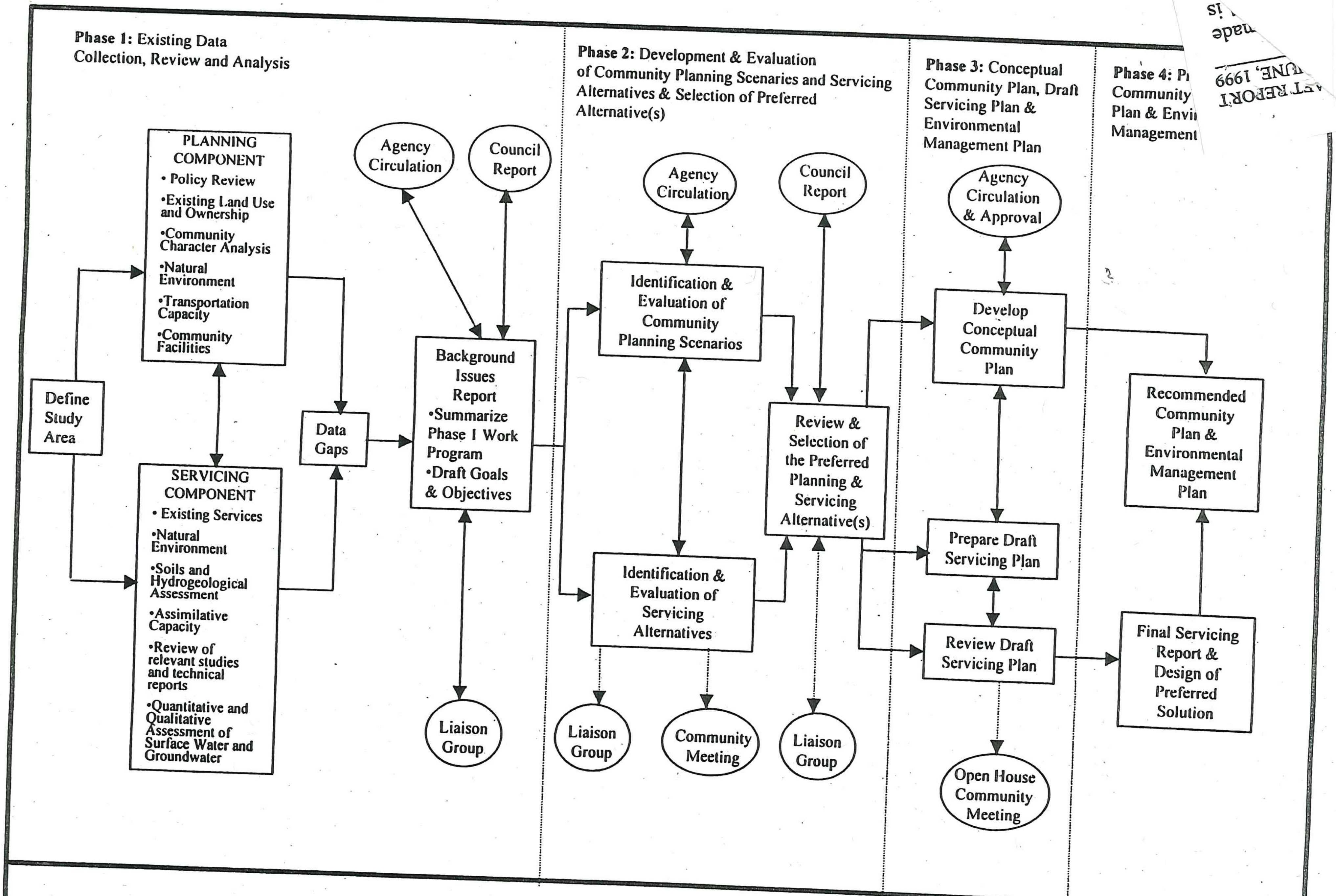

Figure 1.2
Inglewood Village Study Process

study with an understanding of the environmental constraints of the area so that decisions can be 1. while maintaining the integrity of the ecosystem. This work was carried out in Phases I and II ana available under separate cover.

The following objectives were defined for the Environmental Management Plan:

1. To adhere to the Ecosystem Planning and Management Objectives contained in the Caledon Official Plan.

2. To assess the assimilative capacity of the Credit River and ensure that development and servicing respect this assimilative capacity.

3. To ensure that the new water services do not negatively impact groundwater and surface water resources and related biological communities.

4. To identify measures to enhance the local environment.

5. To promote the awareness and stewardship of the local natural environment and to seek opportunities for residents to become involved in stewardship activities.

6. To manage the hydrologic cycle from the pre to post development perspective. This includes maintaining a water balance in terms of water takings and wastewater inputs.

7. To ensure that pre and post development infiltration conditions at the site plan level are maintained or enhanced based on the generally defined areas of high, moderate and low recharge potential.

8. To ensure that post development erosion rates do not exceed the erosive velocities that can cause the bed and banks of all watercourses to erode.

9. To maintain a healthy Dissolved Oxygen regime for fisheries and macroinvertebrates

10. To minimize the addition of other potentially deleterious substances including nitrates, ammonia, metals, organic and oxygen demanding matter

11. To allow no increase in bacteria (E. Coli) levels in the Credit River for human health concerns.

12. To not degrade surface water to produce objectionable colour, odour or turbidity in terms of aesthetics

Figure 1.3 shows the areas that are considered sensitive to change as a result of the analyses carried out in Phases I through III of the study process.

1.2 Intent of Report

This report sets out the components of the Environmental Management Plan (EMP). This Plan has been developed with the data and information collected and developed in the preceding phases of this study and is specific to the preferred planning and servicing (water supply and wastewater) scenarios

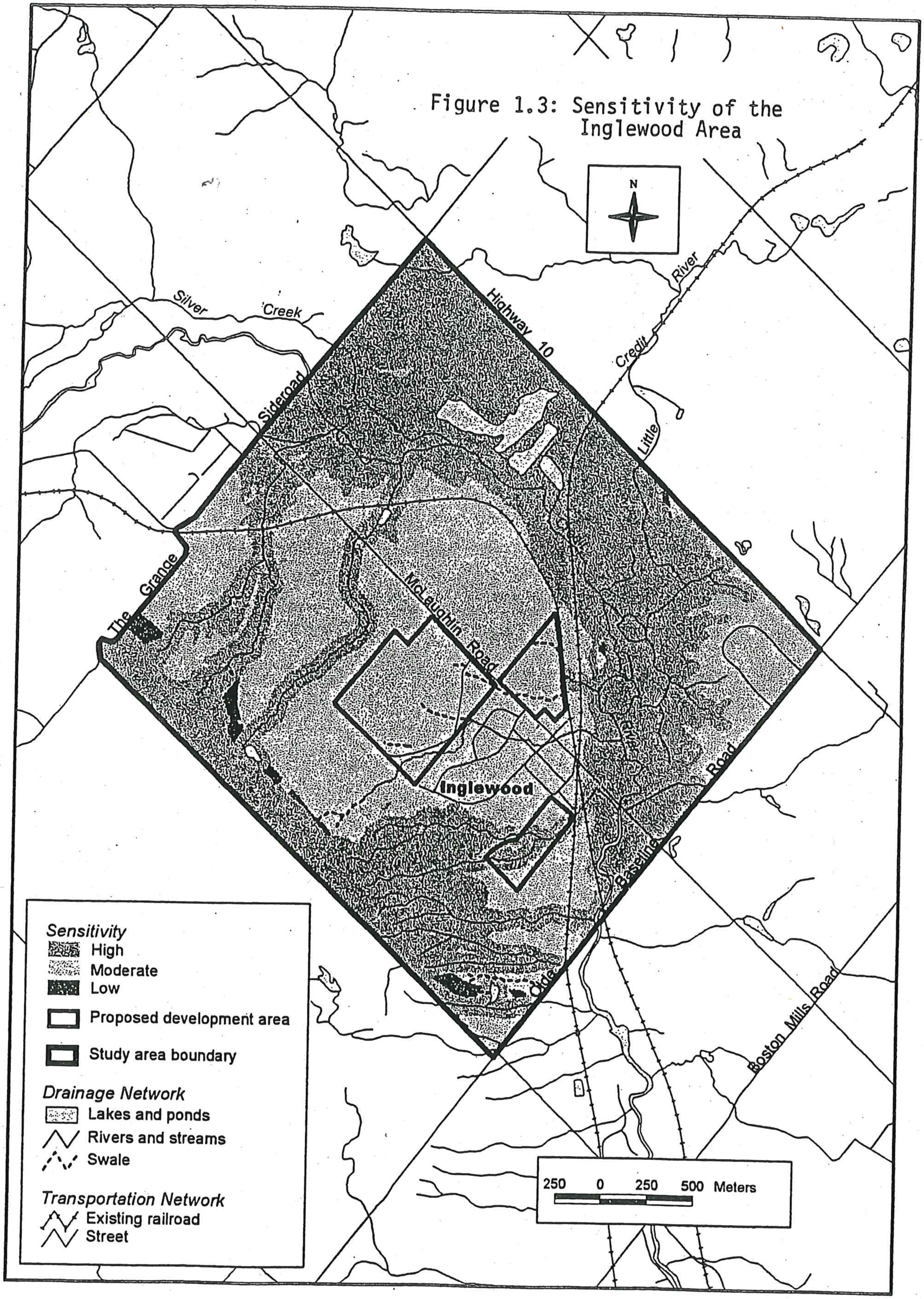

Figure 1.3: Sensitivity of the Inglewood Area
N
Silver Creek
Highway 10
Credit River
Little
Sideroad
The Grange
McLaughlin Road
Inglewood
Road
Baseline
Olde
Boston Mills Road
Sensitivity
High
Moderate
Low
Proposed development area
Study area boundary
Drainage Network
Lakes and ponds
Rivers and streams
Swale
Transportation Network
Existing railroad
Street
250 0 250 500 Meters

described in the Inglewood Village Study Phase II Report (Town of Caledon, 1999). The work carried out in Phase III of the study concluded that the preferred planning scenario was the Mixed Village Density. Water supply will be provided by increasing the capacity of Well #3. The preferred method of wastewater servicing will be a tertiary wastewater treatment plant with a continuous discharge to the Credit River.

This report is a companion document to the Inglewood Village Plan, the Inglewood Village Water and Wastewater Servicing Plan and the Inglewood Village Tributary Study.

1.3 Report Outline

The EMP is laid out in the following manner:

Chapter 2 - Environmental Resources and Constraints - identifies those aspects of the preferred scenarios that help to preserve and enhance key environmental features and functions. A synopsis of the stormwater management report is also included from a study entitled "Tributary Study – Village of Inglewood" which is being completed under separate cover for the Town of Caledon .

Chapter 3 - Future Studies - describes the need for site-specific and monitoring approaches prior to and after development occurs.

Chapter 4 – Implementation - is set out under the four major categories of planning and policy, rehabilitation and retrofit, stewardship and education, and monitoring and reporting.

Chapter 5 – Recommendations - sets out a number of recommendations for future consideration.

2.0 ENVIRONMENTAL RESOURCES AND CONSTRAINTS

The following, by environmental component, identifies those aspects of the companion documents that help to preserve and enhance key environmental features and functions.

2.1 Inglewood Village Plan

Hydrogeology – The land use designations shown on proposed Schedule M - Inglewood Village and Area Land Use Plan (included as Appendix A to this report) are appropriate in that the New Residential Neighbourhoods are situated within the areas identified as being low to moderate recharge potential. The main areas of high groundwater recharge are located outside of the New Residential Neighbourhoods. The Open Space Policy Area (OSPA) designation allows for the implementation of measures to ensure the maintenance of pre and post development infiltration conditions.

Hydrology - The Village Plan keeps all new development outside of the Regional Storm floodplain and this is acceptable from a flooding perspective. No further analysis is required for this component.

Terrestrial - All sensitive terrestrial features have been designated Environmental Policy Area (EPA) on Schedule M. The OSPA designation protects localized drainage features and landforms and allows for enhancement of the terrestrial system and compatible recreation, in keeping with the Town's Environmental Objectives. The Village Plan provides flexibility from an urban design perspective, which allows for a variety of design approaches that can maximize the protection and enhancement of the terrestrial systems within the Village.

Fluvial Geomorphology - Detailed field assessments and analysis were carried out on the tributaries that will be preserved and enhanced through implementation of the Village Plan. Details on these analyses can be found in the Section 2.2 as well as the Inglewood Village Tributary Study completed by Marshall Macklin Monaghan Limited. These analyses conclude that the tributaries can be maintained in a stable form with appropriate storm water management controls and revegetation schemes.

Fisheries - The Village Plan allows for the preservation of those tributaries which do contribute to downstream fisheries. The treatment of these tributaries should allow for stable, fully functional systems and have accounted for water and sediment transport, setbacks from development, and stormwater management techniques that allow for the treatment of stormwater runoff for quantity, quality and bank erosion.

2.2 Inglewood Village Water and Wastewater Plan

2.2.2 Wastewater

Hydrogeology - The following section outlines and describes potential locations available for the preferred surface discharge component of the treated sewage effluent to various watercourses within the study area and adjacent lands.

Surface Discharge to the Escarpment Tributaries - The Escarpment Tributaries originate off the Niagara Escarpment in the south and southwest portion of the study area. Surface runoff across the lower permeable till plains and exposed shale bedrock contribute to many first order streams which have been observed to flow during the spring snowmelt period and are generally dry during the summer. A surface discharge to these watercourses would raise concerns regarding erosion and sediment transport, and water quality, specifically nutrient and chemical concentrations.

Surface Discharge to main Credit River - The Little Credit River originating east of the study area, and Black and Silver Creeks to the north of the study area are significant areas of groundwater discharge which contribute to the maintenance of baseflow conditions within the study area. Streamflow within this section of the Credit River exhibits significant groundwater discharge contributions which provide an opportunity for enhanced assimilation of wastewater and is viewed as a preferred location for surface discharge. There is a need to consider the quality of the treated effluent prior to discharge.

Hydrology - Given the very small discharge rate proposed from the effluent pipe, no impact on the overall hydrology of the Credit River is anticipated.

Terrestrial - The preferred location for the treatment plant is outside of any sensitive terrestrial features. The discharge pipe location as proposed will result in minimal disturbance of natural communities. However further site specific investigations will be required. The surface discharge directly to the Credit River is preferred, as it will prevent potential broad scale impacts on the terrestrial system. In addition, through further detailed analysis, an exact outfall location can be chosen which minimizes disturbance and direct impacts on the wetlands and ESA, and specific mitigation and restoration requirements can be determined.

Geomorphology - The proposed location, near the Credit River, is situated within an area that is moderately sensitive to changes (increased volume) in flow. In addition, the river banks in this area are experiencing moderate erosion, which is largely due to a lack of vegetation. The proposed facility is situated beyond the meander belt width of the Credit River, however, immediately downstream of the proposed outfall, the river is currently close to 'cutting-off' a meander bend. This process is natural and part of the river's evolution. The meander bend has become exaggerated, due to the local geology and riparian vegetation, as well as the effects of the confluence with the western side channel. River flows have eroded the inside bank at both the upstream and downstream sections, thereby pinching off the bend. Eventually, the flows will cut through the inside banks, effectively abandoning the meander bend, resulting in an 'ox-bow lake'. The concern with this process with respect to the proposed wastewater facility is the uncertainty of the eventual channel form and configuration after the meander cut-off. Once the cut-off occurs, there will likely be a substantial sediment deposit near the outfall, as part of the ox-bow lake formation. The new channel form would likely have higher flow velocities along the western bank, resulting in greater erosion potential.

Water Quality - A background review of water quality in the Credit River near Inglewood showed no significant impairments in water quality other than elevated levels of total phosphorus (TP) and bacteria. Using current disinfection techniques, bacteria can be reduced to meet Provincial Water Quality Objectives (PWQO) from the discharge of the facility thereby leading to no increase in bacteria levels to the Credit River.

It was determined that the Credit River near Inglewood is a Policy 2 river with respect to total phosphorus, which indicates that TP levels in this reach exceed the Provincial Water Quality Objectives at least 25 percent of the time. The intent of a Policy 2 designation is to disallow any further loading of the elevated parameter (in this case, TP).

Based on the extensive analysis and modeling completed for Phase II of the Inglewood study, no negative water quality impacts to the Credit River from the proposed communal sewage system for the preferred scenario are expected. The proposed treatment system is designed to produce a high quality of effluent, especially in terms of phosphorus removal.

In fact, the communal system provides an opportunity for improvement in the water quality of the Credit River. Many of the septic systems in the existing village core are inadequately sized or maintained so that they are failing or have a high potential for failure. Hooking these systems up to the communal system and better stormwater management practices are expected to produce a net water quality improvement for the Credit River in the Inglewood area.

Fisheries - The option of using a surface water discharge pipe from the treatment facility is preferred as it avoids groundwater impacts in an area that is sensitive. The location of the outfall is supported given that it is not located near any spawning activity.

2.2.3 Water Supply

Groundwater - The Region will need to assess its current and projected water supply needs for the preferred population numbers. The Region will also need to consider the impact of additional piping and the expansion of the reservoir on areas identified as constraint areas. The Region's Inglewood Well #3 has been identified as the main production well and is permitted for a maximum amount of 1200 Litres per minute (Lpm). Inglewood Wells #1 and #2 will become secondary supplies at rates of 450 Lpm and 900 Lpm, respectively. The Permit was issued in 1996 with a special condition to submit an interpretive report based on five years of monitoring. The report is to evaluate the effect of the water taking on the groundwater and surface water resources in the area and it is to provide recommendations for changes to the monitoring program.

Prior to the amendment of the existing Permit To Take Water (PTTW) for an increased rate from Well #3, the Region will need to assess long-term impacts on the natural areas including the Little Credit River, main Credit River and adjacent provincially significant wetland areas. Based on significant groundwater discharge contribution to the Little Credit and down-gradient main Credit River, reductions in baseflow in the vicinity of Well #3 could contribute to reductions in the assimilative capacity of the Credit River downstream. The protocols for this program will be finalized in discussions between CVC, Region of Peel, MOE and the Department of Fisheries and Oceans (DFO).

Terrestrial - The preferred alternative (increased use of Well #3) would not appear to be a cause for concern related to the Credit River and the Provincially Significant Little Credit River Wetland Complex to the south, however a monitoring program is required (see below).

Fisheries - Under the Federal Fisheries Act, fisheries habitat protection is required. An appropriate monitoring program will be required to ascertain that an increase in water taking does not impact on the natural areas in the vicinity of Well #3 in the longterm. This will ensure that any adjustments in pumping can be made if needed.

2.3 Inglewood Village Tributary Study

The following excerpts were taken from a report entitled "Inglewood Village Tributary Study" dated May 1999, completed by Marshall Macklin Monaghan Limited. The purpose of the study is to address water quality, erosion and flood control with respect to the post developed condition and outline the preferred methods of providing stormwater servicing and control on the lands that are developed or proposed for development. This report is available under separate cover. Therefore only brief summaries of pertinent findings are incorporated here.

2.3.1 Hydrologic Modelling

A hydrologic model was developed to determine the pre-development flow rates through the drainage system under existing conditions. These flow rates allow an assessment of how well the existing drainage system is functioning and indicate a minimum set of flow targets which need to be achieved under post-development conditions.

The three main tributaries of the Credit River through the settlement area were modelled. Tributary 1 drains the majority of the proposed Balmer Heights development area, and flows through the historic Village through a combination of swales, culverts and sewers. The tributary empties into the Credit River just north of the Inglewood Arena. Tributary 2 drains the eastern portion of the proposed Balmer Heights development area, and discharges to the Credit River via a swale through the proposed South Slopes development area. Tributary 3 is located south of the Historic Village, discharging to the Credit River where it crosses McLaughlin Road. Upstream of the CNR trail the drainage course splits, and the north branch is referred to as Tributary 4 (figure 2.1).

An additional two models were constructed to test water flows in post-development conditions: one representing the lower limit of the development densities (22% imperviousness), and the other representing the upper development density limit (28% imperviousness). The post-development uncontrolled models were run with the 25 mm, 4 hour rainfall event as well as through 100 year 24 hour SCS design storms.

2.3.2 Findings

Results of Modelling - On the Balmer Heights property on Tributary 1 the critical discharge rate is 0.07 m^3/s, which is considerably less than the 2 year peak flow rate of 0.44 m^3/s, and less than the peak flow of 0.19 m^3/s resulting from the 25 mm rainfall event. On Tributary 2 east of McLaughlin Road, and on the Mackenzie development west of McLaughlin Road, the critical discharge rate falls between the 2 year and 5 year peak flow rates. The critical discharge rate on Tributary 3 upstream of the Credit River in the ditch along McLaughlin Road is also relatively small, corresponding to the peak flow from a 25 mm rainfall event.

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TITLE

Figure 2.1

CREDIT RIVER SITE TRIBUTARIES

Marshall Macklin Monaghan
Consulting Engineers . Surveyors . Planners
80 Commerce Valley Dr. E. Thornhill, Ont. L3T 7N4
Tele (905)882-1100 Fax (905)882-0055

Checked D.J.A.	Drawn AutoCAD/B.K.B.
Date APRIL 1999	Proj. No. 1499012-01-WR1
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Based on the above, channel stability is a concern even under existing conditions. In the Credit River watershed post-development peak flow rates are typically controlled to pre-development levels and extended release of the runoff from a 25 mm rainfall event over 24 hours is used to define erosion control storage, unless a Subwatershed Plan is available that provides more specific criteria. However, for the proposed developments in the Village of Inglewood, adopting the above criteria will not alleviate erosion potential in the subject tributaries. It may be necessary to adopt more stringent controls, such as increasing the erosion control storage volumes and/or releasing it over a longer duration, in order to prevent further degradation of the drainage channels.

As may be expected, the post development model of uncontrolled conditions indicates that the proposed developments, if uncontrolled, will increase peak flow rates in all downstream channels. Increases are not as great as may be expected because of the soil conditions and the relatively low imperviousness of the proposed developments. Under major flood-type storms Tributary 1 would experience increases in the 15 to 20 % range, while the lower development potential on Tributary 3 would produce only about a 10% increase in flow. Tributary 2 would experience the largest flow increases, in the order of 50 to 100% (on both sub-branches).

Similar erosion concerns exist on both Tributary 2 and Tributary 3. On Tributary 2 the substantial flow increases under the smaller storms indicates the potential for problems (due to an increase in frequency) even though peak flows for these events are less than the critical flow threshold. On Tributary 3, the concern is that the lower reaches of the tributary have been noted as being highly susceptible to erosion.

In Tributary 3, the development area is less than 10 % of the total area. Concerns with erosion in the downstream reaches are best dealt with through geomorphological improvements.

Hydrology and Geomorphology - Detailed studies of hydrology and geomorphology were undertaken in the Marshall Macklin Monaghan study. The general findings are as follows:

The most significant aspects of hydrology within the study are locally oriented (e.g. erosion or flooding on the tributaries themselves). The relatively small drainage areas associated with the tributaries limit their potential significance to the Credit River in terms of flow magnitude.

Excessive erosion was not found in the downstream reaches of the tributaries in the Inglewood urban growth area, the area the study focussed on, but several reaches were classified as highly or moderately sensitive. The lower portion of Tributary 3 (south of the Village and West of McLaughlin Road) and the Credit River immediately downstream of McLaughlin Road were classified as highly sensitive to changes in flow regime.

2.3.3 Discussion and Recommendations

Tributary 1 - A significant drainage problem exists on Tributary 1 as a result of the capacities of the 900 mm and 1050 mm Corrugated Steel Pipes (CSPs) along MacDonald Street and under McLaughlin Road. This problem has existed for some time. EMC Group prepared a design in 1995 for the reconstruction of Lorne Street from West Village Drive to MacDonald Street and MacDonald Street from Lorne Street to the CPR tracks, including storm sewer improvements. The system has not been constructed, but a 600 mm sewer along Macdonald Street, between Victoria and Lorne Street has been. At present, the flow from these catchments enters a 300 mm CSP and is routed through backyard swales to the corner of McKenzie and McLaughlin Road.

A possible alternative to the proposed sewer construction would involve over-controlling flows from the proposed Balmer Heights subdivision The best that could be done through over-control would be about a 5 year level of service. Over-control would be relatively inefficient since about half of the potentially controllable land is not proposed for development. It is clear that over-control cannot accomplish an adequate solution on its own. Therefore, a storm sewer under Lorne and MacDonald Streets, similar to that designed by EMC Group in 1995 is recommended. It is noteworthy that the provision of some detention storage at Balmer Heights would reduce peak flow rates in the system and therefore potentially reduce the size and cost of a storm sewer system to convey runoff through the historic Village.

Tributary 2 - The hydraulic structures along Tributary 2 are generally satisfactory. However, due to the restriction at the CPR tracks, improvements such as building a stormwater management pond are required should the South Slopes area be developed.

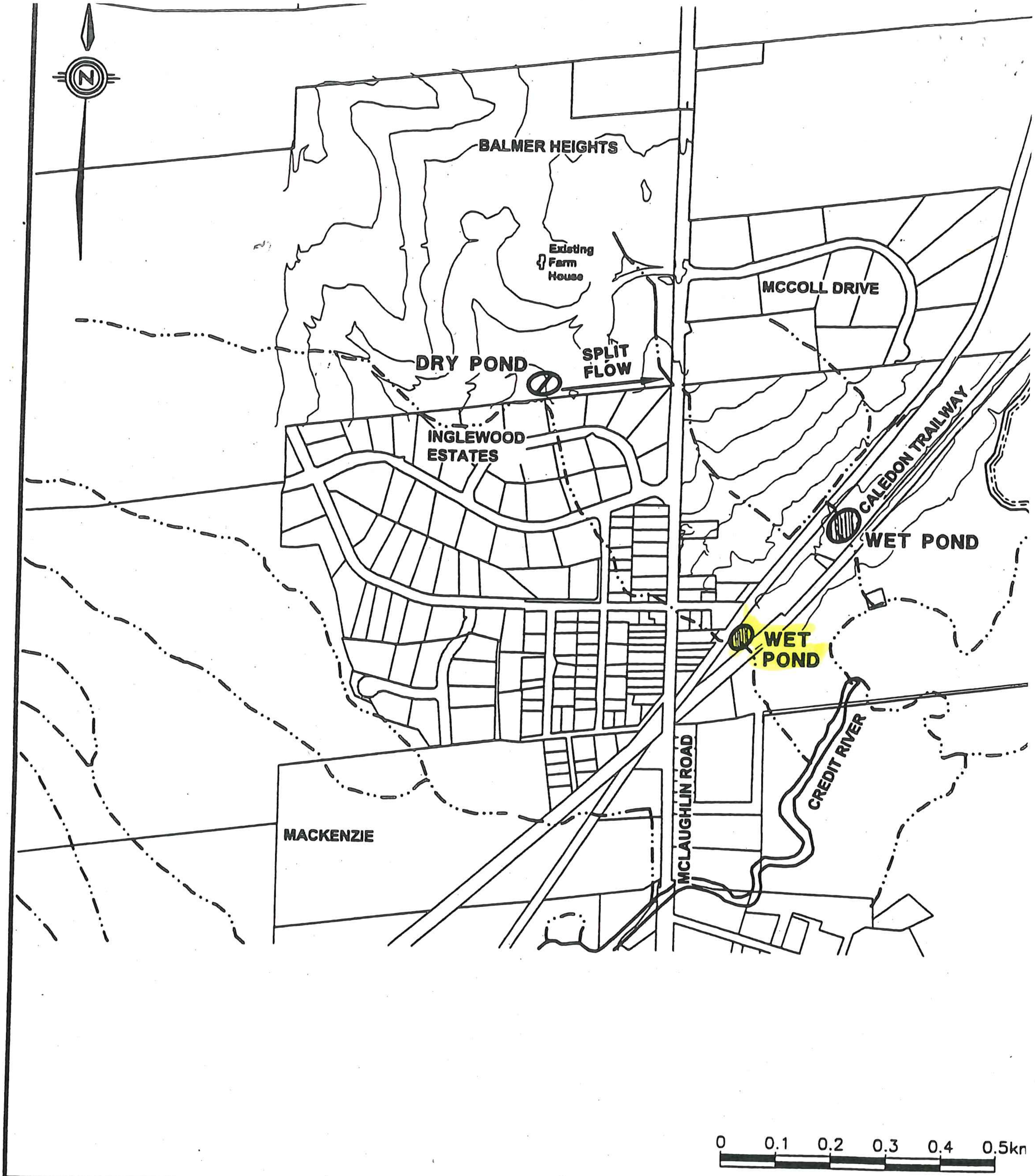
Tributary 3 - The major culverts upstream of the railway and the Caledon Trailway on Tributary 3 have adequate capacity and are not considered to be a problem. Adjacent to McLaughlin Road, the two culverts passing under the private entrances are inadequately sized. This area is within the floodplain of the Credit River and is likely to experience some flooding, regardless of the flow from Tributary 3. As a result no improvements to the hydraulic structures along Tributary 3 are recommended.

2.3.4 Stormwater Management Alternatives

Based on the results of preceding sections, two alternative stormwater management strategies were formulated. Each alternative has two variations that could be implemented. The alternatives and their variations are illustrated in Figure 2.2 Each of the alternatives (and variations) must meet the following criteria, as a minimum:

i. Maintain infiltration conditions in order to continue the moderate recharge capacity of lands proposed for development.
ii. Control post development peak flow rates to pre-development levels for the 2 through 100 year return period storm events.
iii. Provide erosion control storage in order prevent exceedence of erosion thresholds.
iv. Provide water quality control for Level 1 habitat protection, as set out in the Stormwater Management Practices Planning and Design Manual (MOE, 1994).
v. Seek to provide enhanced treatment for phosphorus.

Structural infiltration measures are not well suited to the till soils present throughout the area. In order meet criteria i) it will therefore be necessary to disperse runoff to the greatest extent possible. This can be done by a combination of lot level and conveyance controls. Roof discharge to lawns (and perhaps shallow ponding areas) should be employed in all areas. Further, the use of swale drainage, connected into open space drainage ways is recommended. In addition, end-of-pipe facilities will also be needed



CLIENT

TOWN OF CALEDON

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Figure 2.2

PREFERRED ALTERNATIVE SWM POND LOCATIONS

Marshall Macklin Monaghan
Consulting Engineers . Surveyors . Planners
80 Commerce Valley Dr. E. Thornhill, Ont. L3T 7N4
Tele (905)882-1100 Fax (905)882-0055

Checked	Drawn
D.J.A.	AutoCAD/B.K.B.
Date	Proj. No.
MAY 1999	1499012-01-WR1
Scale	
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to fully meet these requirements. The combinations of the facilities that have been considered are described below.

A detailed review of all the alternatives considered can be found in the Inglewood Village Tributary Study.

2.3.5 The Preferred Alternative

As noted previously, each of the alternatives could be implemented as a feasible surface water management strategy. Any of proposed alternatives is capable of providing the required levels of quality and erosion control and peak flow attenuation for future development. Sufficient area exists for the ponds at the specified locations. Critical velocities in the subject tributaries can be controlled such that they will be exceeded less than once every 1.5 years on average, which is acceptable. The exception to this statement is the east branch of Tributary 2 (draining the McColl Drive area). Here a sound geomorphic design will be required to ensure that the rebuilt tributary remains stable.

Selection of the preferred alternative therefore becomes a question of the overall water quality benefit and the overall cost. On this basis Alternative 1B is clearly preferable to Alternative 1A and Alternative 2A is preferable to 2B. The decision therefore lies between Alternative 1B and 2A.

Alternative 2A has a projected cost which exceeds that of 1B by about $267,000. However it provides treatment for about 16.5 more hectares of existing development. The unit costs of providing water quality treatment are comparable between the alternatives ($9550/ha vs $9019/ha). Based on this similarity in cost and the agreement by the study team for the Inglewood Village Study to utilize every opportunity to achieve a reduction in the phosphorus loading to the Credit River, Alternative 2A is selected as the preferred alternative.

In Alternative 2A, a single dry pond is proposed in the Balmer Heights development area, to provide quantity and erosion control for the development area and external areas to the west. A flow splitter at the pond outlet would return flow to Tributaries 1 and 2 at or below pre-development levels. Downstream on Tributaries 1 and 2, two wet ponds are proposed between the CPR railway and the CNR trail. Each pond would be on-line and would provide quality, quantity and erosion control for the proposed development areas and areas of existing development which drain to these Tributaries. The proposed ponds would provide water quality treatment for a total of 120 ha.

2.3.6 Study conclusions

The following conclusions have been drawn from the Village of Inglewood Tributary Study and detailed work completed in the current study.

1. The study area is considered to be a moderate recharge zone. The Credit River in the vicinity of Inglewood is a discharge zone. The maintenance or enhancement of infiltration conditions in developing areas is therefore critical for down-gradient quality and quantity conditions.

2. The soils in Inglewood have low hydraulic conductivities and hence their infiltration capability, on a unit area basis, is low. As a result, structural or "point" effectiveness of infiltration techniques will

be limited. The primary method of infiltration in the post development condition will have to be based on dispersion of runoff over as much land as possible so that the greatest possible natural infiltration can occur. Techniques recommended are roof discharge to lawns (perhaps shallow ponding areas) and use of swale drainage connected to open space drainage.

3. The majority of significant natural areas in the vicinity of Inglewood are located outside of the settlement area and are associated primarily with the Niagara Escarpment or the valley of the Credit River. The most important natural elements within the study area are the watercourses and their associated function (or potential function) as valley corridors and linkages between the Escarpment and the Credit River.

4. The tributaries within the settlement area provide limited fish habitat because of their low to intermittent flow in the summer and the presence of culverts that impede fish passage during periods of higher flow. However, each of the tributaries is considered to contribute to fish habitat through their hydrologic, ecologic or geomorphic function. Therefore the tributary system should be preserved to the greatest extent feasible.

5. From a stormwater perspective, the preservation or incorporation of open space corridors in the development form will provide opportunities to disperse runoff and hence maximize natural infiltration.

6. Each of the tributaries is relatively sensitive to increases in flow magnitude or frequency. Tributary 1 in particular has a low threshold before erosion problems may be expected. Extended detention storage for erosion control will be required in upstream areas in order to avoid problems in the post development condition.

7. Flooding concerns exist on Tributary 1 due to the existing culvert capacities. While the problems could be reduced through over-control of flows from upstream areas, the problems would continue to exist. Flows generated by the areas of existing development (excluding existing upstream flows) are large enough to cause flooding during a 10 year return period storm. As a result, a combination of flow control for upstream areas and upgrading of downstream storm sewer systems should be pursued.

8. The water quality of the Credit River in the vicinity of Inglewood is classified as Policy 2 (Provincial Water Quality Objectives are currently exceeded and no further degradation will be permitted) on the basis of Total Phosphorus and bacteria. Studies completed as part of the Inglewood Village Study however, indicate no significant dissolved oxygen problems. However, because of the fact that the communal sewage treatment systems will contribute some additional phosphorus, all reasonable actions that will limit the discharge of phosphorus to the Credit River should be encouraged.

9. An acceptable tributary management strategy can be implemented for the Village Plan.There is sufficient space for the implementation of at-source, conveyance and end-of-pipe facilities..

3.0 FURTHER WORK

This chapter is divided into two sub-sections. Section 3.1 outlines, by study component, site-specific analyses that will be required as part of the approvals process. Section 3.2 describes needed monitoring that will be required to ensure compliance of the EMP.

3.1 Site Specific Analyses

Groundwater - Based on proposed development, site specific consideration should be given to ensuring that pre and post development infiltration conditions are maintained or enhanced. To this end, a water balance should be conducted for each development proposal. It should be also demonstrated how various tools and techniques will be utilized to maintain infiltration rates prior to issuing Draft Plan Conditions.

Hydrology - Based on the findings of the Tributary Study and the needs outlined above in the Groundwater Section, each development must prepare a stormwater implementation report that meets these requirements.

Terrestrial - Fieldwork carried out during the summer of 1998 was focused on the potential areas of impact that were known at that time. As a result, field data was collected within some of the wetlands and ESAs within the main Credit River valley, east of the existing Village. The areas surveyed were determined by existing potential sanitary sewer outlets and authorization received from landowners for access.

As a result, several areas require additional detailed assessment of their terrestrial communities prior to developments being recommended for approval. In particular, the area referred to as Special Study Area in Schedule M has site specific environmental constraints (see Appendix A). Access from the east would require crossing two tributaries and their associated steep valley slopes which are part of an important and sensitive corridor. Additional study is necessary to determine if such a crossing is feasible, and if so what mitigating measures would be required.

An additional area requiring study will be the preferred location for the wastewater outfall. Based on a preliminary site walk with Town and Regional staff, the area east of Macdonald Street, between the two tributary watercourses and immediately downstream of the 'S' curve in the Credit River is the preferred location. An assessment of the natural communities to be effected will be required to finalize the alignment and prescribe any mitigating measures.

The tributary which is located within the southwest corner of the Balmer Heights development has been impacted by the historic agricultural uses on the adjacent lands (Figure 3.1). It does, however, present considerable opportunities for rehabilitation and enhancement as a habitat corridor, and a connecting open space link from the Village to the Niagara Escarpment. Prior to development proceeding in this area, an assessment of the specific constraints and opportunities for this corridor should be carried out by the proponent. The results of this study, when approved, will influence the final alignment and ultimate condition of the tributary corridor and the proposed residential development in this area.

Figure 3.1: Balmer Heights Tributary, looking upstream

Fisheries - Site assessments on how the tributaries draining the study area are treated with respect to the recommendations of the Environmental Management Plan will be required.

Other Initiatives - The Ministry of Natural Resources and Credit Valley Conservation should be supported to continue the Experimental Atlantic Salmon Reintroduction Program and Lake Ontario Fisheries Unit habitat and spawning assessments that provided much of the background information. Where feasible the new MNR protocol for habitat assessment should be adapted.

Habitat studies and rehabilitation techniques may be investigated along with Special Harvest Regulations for the recreational fishery. The Credit River Fisheries Management Plan recommends that brown trout over-wintering and migration areas be investigated.

Future research can utilize the monitoring data from all components and relate it to fish productivity and diversity as "environmental indicators" and recreational resources. Data gaps that remain relate to recreational users and other socioeconomic aspects of the fishery.

Credit Valley Conservation should be supported in developing and implementing a master plan for the Ken Whillans Resource Management Area, focusing on habitat enhancement, fishing opportunities in the river and ponds and as a regional/national rail trail head facility. Similar adjacent lands should likewise be part of an integrated management plan. The increase in the local population and others visiting the Conservation Area will soon make this evident. This will require the continued cooperation with the Town of Caledon park and trailway staff. Private landowner opportunities may include informal agreements such as that with Trout Unlimited who manages the river in "fish friendly" ways for private landowners in exchange for conditional public access. Much of the private land is already accessed by the public which if not monitored and managed could cause damage to the Provincially Significant Wetlands, Environmentally Sensitive Areas, the fishery and other recreational opportunities. There is a good opportunity for key private landowners to have a Conservation Plan completed through a cooperative program with Credit Valley Conservation and the University of Guelph.

It should also be noted that related terrestrial habitats (including wetlands and riparian corridor) should be studied and managed together.

3.2 Monitoring

Groundwater - Spot streamflow measurements during low flow conditions (summer) at selected locations upstream and downstream of the proposed outlet location(s) would help confirm long-term conditions as development and urbanization proceeds. In addition, a long term monitoring program for Well #3 would help ensure the maintenance of the natural environment.

Water quality sampling should be carried out on a seasonal basis during both dry and wet events, both upstream and downstream of proposed outlet location(s).

Terrestrial - Waste water should be monitored through direct sampling at the outfall and chemical analysis. See the water quality section for more detail.

Water Supply - Monitoring of wetland communities in proximity to Well #3 will be required. Protocols will be developed to monitor the condition of the wetland vegetation communities and their related wildlife. Vegetation plot sampling and amphibian monitoring is recommended. This monitoring should begin as soon as possible in order to establish base line conditions prior to development proceeding. Finalization of the monitoring program will be discussed with the Region of Peel, CVC, MOE and DFO.

Geomorphology - From a fluvial geomorphological perspective, monitoring of physical channel processes should be conducted along tributaries, as well the Credit River. As part of this work, controls (upstream on the Credit River and on tributaries where no development is proposed) should be implemented. Tributary monitoring should occur downstream of development areas, including stormwater management facilities and should include the measurement of several permanent cross-sections, substrate sizes and a long-profile survey. This should be conducted once a year for the first three years after construction. Monitoring on the Credit River should be downstream of the proposed wastewater facility and should consist of a series of erosion pins installed along the banks as well as the measurement of several cross-sections. Measurement of the erosion pins should occur at least twice a year (spring and fall), while the cross-section measurements only need to be taken once per year.

Water Quality - Although the CVC does not anticipate any negative impacts to water quality, monitoring is required to ensure our expectations are met. Furthermore, conditions may change in the watershed which could not have been taken into account in the impact assessment performed in Phase II. Two main types of monitoring will be considered: in-stream and effluent monitoring.

In-Stream (Credit River)

Presently at the Credit Valley Conservation (CVC), we are mapping out an integrated monitoring program which includes monitoring in terms of hydrology, hydrogeology, fluvial geomorphology, biology and water quality. The following objectives were developed for this monitoring strategy:

Objective 1: Protect and improve water quality and quantity in the watershed.
Objective 2: Protect and improve biological diversity and productivity of the watershed.
Objective 3: Promote the relationships between a healthy watershed and economic and social activities.

CVC has divided the monitoring effort into two tiers. Tier 1 includes mainly physical and biological monitoring for water quality while Tier 2 includes chemical and more detailed biological monitoring. The two tiered approach aims to collect data to assess baseline conditions and to detect changes in the watershed (Tier 1) while having the ability to initiate more detailed monitoring where it is deemed necessary (Tier 2).

As mentioned in the previous chapter, the Credit River has high enough levels in total phosphorus (TP), to achieve Policy 2 status from the MOE. The problems associated with high TP levels are excessive aquatic plant growth leading to nighttime depletion of dissolved oxygen.

The communal system will add an additional, albeit small, load of total phosphorus at Inglewood. However, the expected increase in TP in the Credit River cannot be measured with any statistical confidence. Instead, a monitoring program should attempt to measure any potential effects of the phosphorus from the communal system.

Some of the proposed Tier 1 monitoring efforts include benthic sampling and 24-hour dissolved oxygen and water temperature surveys, which could be applied to the Inglewood monitoring sites. Basic aquatic plant monitoring may also be included in Tier 1 monitoring as the CVC continues to adjust its watershed-wide monitoring program to adapt to the changing needs of the watershed. CVC proposes to adopt tier 1 type monitoring for the Inglewood area. A summary of instream water quality monitoring parameters follows. (Table 3.2.1).

Description of Monitoring Protocols and Reporting

Dissolved oxygen surveys should be completed over a 24-hour period, preferably on a sunny day to catch the maximum effects of aquatic plants. Approximately 5 measurements per site should be taken on a rotating basis over the 24 hours. Times of measurements, weather conditions (including air temperature), water pH and water temperature should be recorded. When reporting results from the DO survey, include percent saturation of dissolved oxygen.

On the same day of the DO survey, qualitative observations of aquatic plants should be reported. Aquatic plant monitoring should include at least a presence-absence description of aquatic plants. If present, the following observation should be made:

- type of aquatic plants (i.e. attached algae, macrophyte, or emergents)
- plant species (if possible)

- approximate size of patch
- whether the plant patch is sparse (a few plant per square ft), moderate (plants mostly cover a square ft), or dense (thick coverage over square ft)

Photographs could be taken to provide evidence of density of plant vegetation.

Benthic monitoring should include a site description, which includes substrate type, presence of aquatic plants, water depth and width, and presence of any odour. Some possible monitoring protocols include rapid bio-assessment, BioMap and various kick and sweep methods. Benthic data should be reported in terms of density (no./m²), number of taxa, total organism abundance, Shannon-Wiener Diversity, % Oligochaeta, and % Chironomidae.

Table 3.2.1: Instream Water Quality Monitoring Parameters

Monitoring Item	Parameter(s)	General Description	Frequency	Time of Year	Personnel Required	Costs Involved
1	-dissolved oxygen -pH (if possible) -water temperature -aquatic plants (both macrophytes and attached algae)	**DO, pH, H_2O Temp:** 24-hour survey using meters **Aquatic plants:** qualitative observation and possibly photographic evidence	monthly	summer	-access to a DO and pH (if possible) meter -experience with aquatic plants	-cost of meter -time -mileage -camera
2	macroinvertebrates	-method TBA (based on an ongoing study out of the University of Western looking at different sampling protocols)	annually	summer	-expert for collection and identification protocol	-cost of collection equipment -time -mileage

The following considerations were made in determining a monitoring strategy for the Credit River to pick-up impacts from the communal sewage system:

1. locations
2. parameters (24-hr DO and temperature surveys, bio-monitoring)
3. sampling technique and data analysis
4. frequency and time of year
5. monitoring expertise required
6. outline of potential costs (time, services and materials)
7. reporting (what form and how often)

Monitoring Locations

The following general locations are proposed for monitoring items 1 (DO, pH, etc) and 2 (benthics):

1. Upstream of discharge pipe
   - downstream of the Little Credit River (East Credit) since water quality parameters measured in the Little Credit were significantly different than those in the Credit River for all sampling days in the summer of 1998
   - far enough upstream of the discharge pipe to avoid backwater effects (migration of discharge upstream)

2. Just downstream of discharge pipe
   - far enough downstream to have the effluent completely mixed (or as close as practically possible)
   - upstream of any stormwater facility discharge to isolate changes from the communal sewage

3. Further downstream of discharge pipe
   - far enough downstream to have the majority of settling out of suspended solids
   - should coincides with the recovery zone

In addition, it is recommended the monitoring item 1 (DO, pH, etc.) be carried out further downstream of the locations proposed above. Three additional sites are recommended at 1-km intervals downstream of location #3, and preferably where existing aquatic plant growth can be observed. The Credit River at Boston Mills is proposed as a Tier 1 site for the watershed-wide monitoring program and could be used in conjunction with this study.

Effluent Monitoring

The proposed communal sewage system is anticipated to produce a very high quality effluent. The following discharge quality is expected:

Total Ammonia	:	2 mg/l (summer), 5 mg/l (winter)
Total Phosphorus	:	0.15 mg/l
Suspended Solids	:	10 mg/l
BOD	:	10 mg/l

To ensure that the communal sewage system is performing as predicted and to measure the removal efficiency of relevant parameters, raw and treated wastewater should be monitored for typical parameters found in municipal wastewaters. The Ministry of the Environment (MOE) must issue a Certificate of Approval (C of A's) for the wastewater treatment plant. Typically, these C of A's require monitoring of the treated wastewater and the CVC recommends the following monitoring program:

1. The relevant parameters listed below should be tested on a monthly basis after critical stages in the treatment:
   - nitrates (NO_{2+3})

- total ammonia, pH, water temperature (to calculate un-ionized ammonia)
- Total Kjeldahl Nitrogen (TKN)
- total phosphorus (TP)
- soluble phosphorus (Sol P)
- biochemical oxygen demand (BOD)
- dissolved oxygen (DO)
- *E. coli*
- suspended solids (SS)

2. All of the above parameters should be tested on a monthly basis at the end of the discharge pipe.

3. Wastewater flows should be monitored to generate average daily and monthly flows.

Fisheries - It is recommended that annual or biannual fish biomass samples be continued at Inglewood in a scientifically comparable manner using established protocols. Analysis will concentrate on changes to fish community diversity (number and type of species) and productivity (total weight of fish sampled) as indicators of environmental changes. Further analysis related to other associated components (e.g. water quality, geomorphology, groundwater) will then be required to identify causal relationships and take appropriate action. A similar sampling site is also recommended downstream (of the proposed sanitary outfall location) at Boston Mills where other cumulative impacts may occur or as a "control" in other investigations. Other "controls" should be located upstream as part of a CVC watershed wide monitoring program using similar protocols.

Although less reliable, spawning surveys should also be continued. The greatest value from this exercise is the ability to identify and monitor groundwater contributions in a manner that demonstrates a direct linkage to aquatic communities. Wetland communities might likewise be monitored. This could be important if increased water withdrawals or climate changes begin to affect the local water table.

Potential thermal impacts from both stormwater and wastewater facilities can easily be investigated with temperature loggers or spot checks with an accurate thermometer at different times and seasons.

4.0 IMPLEMENTATION

The following section has been developed to act as a guide to facilitate how agencies are organized by area of expertise, legal jurisdiction and function, and how this applies to study implementation. Four broad areas have been identified for implementation: planning and policy, rehabilitation and retrofit, stewardship and education, and monitoring and reporting.

4.1 Planning and Policy

Hydrogeology – All proposed development must ensure that water table elevations, baseflow and aquifer yield are maintained in the post developed condition.

Hydrology - Appropriate techniques should be employed as discussed in the Inglewood Village Study to ensure that the quantity, quality and erosive nature of stormwater runoff are managed and that steps be taken to include the existing Village core in this assessment.

Terrestrial - The sensitive terrestrial features are to be protected through the Environmental Policy Area and Open Space Policy Area designation on Schedule M and the related Village Plan policies (see Appendix A).

Water Quality - This reach of the Credit River should be recognized as a Policy 2 river for bacteria and phosphorus. As such, it is important that all efforts to achieve an offsetting of phosphorus inputs to the Credit River be required.

Fisheries - All planning decisions and those by private landowners will adhere to Federal Fisheries Act to protect fish habitat. Provincial and local policies should encourage proactive protection measures to avoid punitive action and to embrace an improvement in fish habitat as part of a healthier environment and community.

Site specific environmental impact statements and mitigation plans related to fish habitat will be required for planning applications that directly affect the streams and river. These may also include potential impacts related to recharge and groundwater upwellings, thermal impacts from urban runoff and other sources of deleterious water quality. The Ministry of the Environment also provides a supportive legislative role in protecting water resources through the Ontario Water Resources Act.

Planners and others should be consistent with and consult the Credit River Fisheries Management Plan (MNR / CVC 1999) for further guidelines.

4.2 Rehabilitation and Retrofit

Terrestrial - The current condition and function of the watercourse and valleyland corridors were assessed within Phase 2. The tributary which passes through the existing Village (Inglewood Village Estates and a portion of Balmer Heights) was determined to be highly impacted, but providing a moderate connectivity function. Its functional role could be improved considerably if it was enhanced along several sections, and extended through upland plantings to connect with the Inglewood Slope

Environmentally Significant Area to the west. This restoration would also provide enhanced open space or recreational opportunities by re-connecting with the Niagara Escarpment and Bruce Trail.

The corridor to the north of the existing Village and the railway crossing provides an important connection between the Credit River valley and the Escarpment. While it has been determined to be highly sensitive, it has suffered from some moderate level impacts primarily from existing and previous agricultural land uses (Figure 4.1). This corridor could benefit considerably from some streamside naturalization.

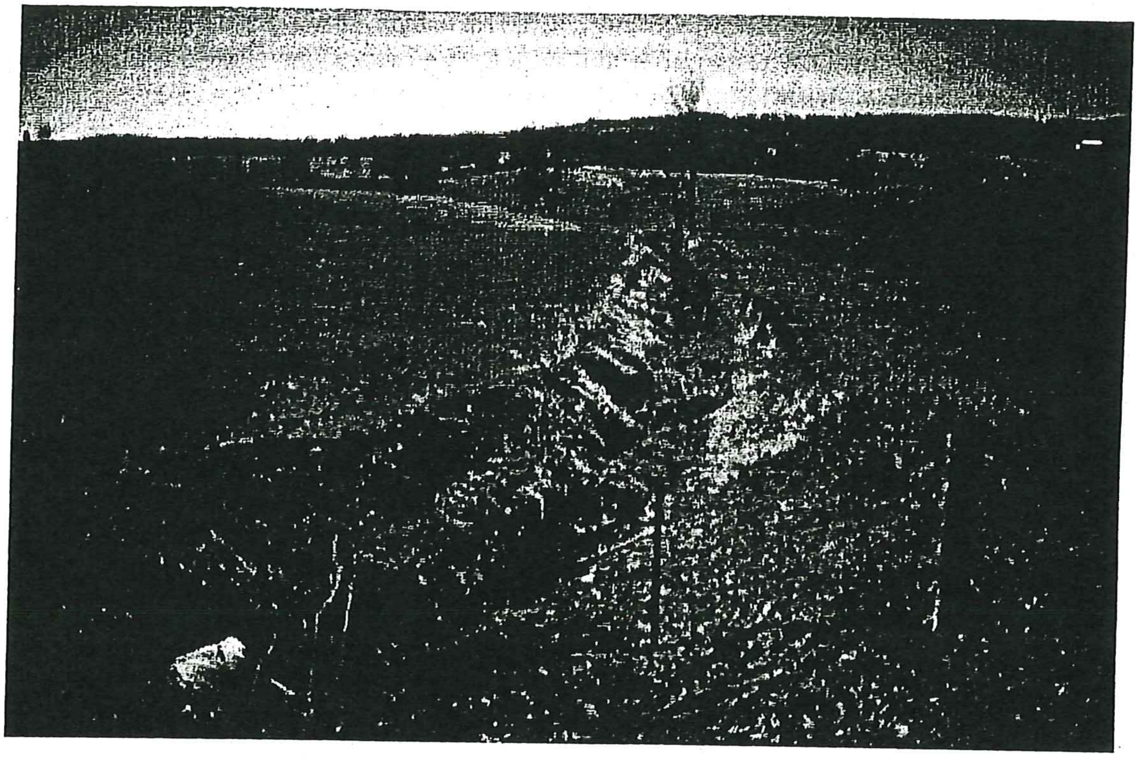

Figure 4.1: Tributary south of Grange Sideroad, West of McLaughlin Road

The corridors which extend through the proposed Mackenzie development to the Niagara Escarpment have been determined to be of high sensitivity. A northern tributary to this system has been impacted through the encroachment of intensive agriculture within the northern portion of its defined corridor boundary. This area is also recommended for re-naturalization along the corridor edge.

Geomorphology - There are several recommendations for rehabilitation of the channels within the study area. First, the highly to moderately sensitive channel reaches could be improved through increase plantings along the banks. Second, some erosion control work, including regrading of the bank and some aggressive plantings should be implemented around the outfall from the proposed wastewater facility. Finally, consideration should be given to assisting the river in 'cutting-off' the sharp meander bend. This permits the ability to control some the channel processes thereby avoiding the possibility of high erosive flows against the west bank, or the possible blockage of the western channel. This work is a significant undertaking and certainly requires some additional investigation and analysis. However, it would provide greater assurance regarding the channel and stability of the outfall location.

Fisheries - Much of the existing health of the fish community is based on the natural condition of the valley lands. This includes the woody material introduced into the channel that increases hydraulic roughness that reduces flood velocities and erosion. It also provides important nursery and adult cover for fish as well as other microhabitats for a full range of riverine life from insects to mink. The replacement of woody material where it has been removed at meander bends is strongly recommended to reduce aggravated erosion rates and replace fish cover. Volunteer labour from the community or fishing clubs may be utilized. Log jam removal should only be considered if it blocks the entire channel at low flow causing upstream sedimentation or side channel erosion. Technical advice from a geomorphologist or biologist should be sought in this case. Hard engineered constraints on the river, such as rip rap or gabion baskets should be avoided. The river and the floodplain should be respected as they provide vital riparian, fish and wetland habitat.

To reduce stream temperatures, bank erosion and provide fish and wildlife habitat, open riparian areas along the river and tributaries should be planted with suitable species. Note that beaver in the area will tend to leave certain trees such as cedar alone.

Decommissioning of the old migratory fish electric barrier should be done along with re-naturalization of the channel and eroding banks. A new barrier, if feasible, would have to be constructed downstream of the East/Little Credit confluence (Credit River Fisheries Management Plan, MNR/CVC, 1999). The next barrier downstream within the golf course lands also requires further investigation with a preference for removal due to siltation, thermal impacts and its impact on local fish migrations.

4.3 Stewardship and Education

Terrestrial - With the exception of the Ken Whillans Resource Management Area west of Highway 10, the Caledon Trailway, the Community Centre and Arena, the majority of the lands within the Inglewood Village study area are privately owned. A "Stewardship Brochure" should be produced for the residents of Inglewood describing the environment in and around the Village and how they can contribute its protection and enhancement.

A conservation planning service to be offered by CVC and the University of Guelph is currently under development. Under the guidance of CVC technical staff, with input from the landowner and others, a team of senior level university students would produce, for a fee, an inventory, assessment and series of stewardship recommendations in the form of a Conservation Plan. Conservation Plans should be considered for priority properties (i.e. those containing highly sensitive Core Natural Areas and Corridors).

Fisheries - A local program should be developed to better educate the growing community and visitors about the natural features and functions of the area with a focus on the river. Education and demonstration opportunities exist in the Ken Whillans Resource Management Area (Wild Trout Display Board), Arena Parkland and along the Caledon Trailway. Other opportunities include brochures, interpretive signage and special events such as a local riverfest. The Yellow Fish Road or Storm Drain Marking Program identifies storm drains in urban areas as being directly linked to the river. Literature provided door to door gives information on reducing local water pollution. Volunteers paint yellow fish "reminders" along the curb gutters.

It is important to conduct site visits with landowners to provide technical advice for stream, pond and wetland enhancement, and to resolve other issues regarding their private lands. A Conservation Plan (including terrestrial resource management options) should be done for the larger tracts of private land as recommended under Further Studies.

4.4 Monitoring and Reporting

Water Quality - In-stream Credit River monitoring reports should be generated every 2 years for the first 10 years for water quality and then every 5 years following. Reports should be circulated to the Town of Caledon and Credit Valley Conservation for review. Reviews may affect changes in monitoring protocols based on changes in the watershed, results requiring further investigation, or other suitable criteria.

All observations, measurements and other results should be reported as raw data, and in graphical format, where applicable. As a minimum, the following calculations are recommended for the reports:

- from the DO survey, include percent saturation of dissolved oxygen
- from the benthic surveys, density (no./m^2), include number of taxa, total organism abundance, Shannon-Wiener Diversity, % Oligochaeta, and % Chironomidae

Monitoring reports for the communal sewage system should be generated annually and submitted to Town of Caledon and the Credit Valley Conservation for review. The following calculations are recommended to be included in these reports:

- un-ionized ammonia
- removal efficiencies for relevant parameters (ammonia, nitrates, TP, BOD, *E. coli* and SS)
- average daily and monthly flows
- maximum flow per month

Both the in-stream and effluent monitoring reports will include a background review, presentation and analysis of the monitoring results, and discussion and conclusions from these results.

Terrestrial - Monitoring of effects from the proposed developments should occur during and after construction. A primary area of concern however will be potential draw down of the local ground water table from increased well use, and its effect on the terrestrial communities. Monitoring protocols should be established as soon as possible in order to determine base line conditions. Monitoring of vegetation should occur once a year after baseline establishment. Amphibian monitoring will be required during the critical spring period on an annual basis.

The potential effects from the construction of the wastewater treatment plant and discharge should be mitigated through immediate action (e.g. sediment discharge) or as part of the rehabilitation plan. Direct discharge to the Credit River will require water quality testing.

Reporting for the monitoring of the proposed developments should occur on a monthly basis beginning with the initiation of construction. If issues or concerns arise more frequent reporting may be required.

During the construction of the wastewater treatment facility and outfall, weekly reporting will be required. Post construction reporting of the water quality sampling should occur on a monthly basis.

Table 4.4.1: Implementation Summary

Plan Component	Action	Facilitator[1]	Contributor[2]	Related Implementation Mechanism
Planning and Policy				
	Detailed design of subdivision	Applicant	CVC, R. of Peel, T. of Caledon	Official Plan
	Environmental Impact Statements for new developments and waste water treatment facilities	T. of Caledon	CVC, Region of Peel	Official Plan
	Environmental Impact Statements for fish habitat that address impacts related to groundwater, recharge and water quality	CVC	MOE	The Fisheries Act, Ontario Water Resources Act, Official Plan
	Policies to protect sensitive environmental features and functions	T. of Caledon	Region of Peel, CVC	Official Plan
	Policies to support the need for rehabilitation and restoration	T. of Caledon	CVC, Region of Peel	Official Plan
Rehabilitation and Retrofit				
	Naturalization and preventative measures: Plant open riparian areas, naturalize around the defunct electric fish barrier, replace woody debris at meander bends	CVC, fishing clubs, local interest groups	Town of Caledon, Region of Peel	CVC stewardship and restoration services
	Plant the banks around the wastewater treatment facility outfall	Region of Peel	CVC	Construction of wastewater facility
	Construct a new fish barrier	Ministry of Natural Resources	CVC	
	Assist the river in cutting off the meander bend	Region of Peel	Region of Peel, CVC	Construction of wastewater treatment facility
	Restore important corridors	Landowners, CVC, Peel	T. of Caledon, Community Groups	Official Plan, Development Approvals, CVC Stewardship and Restoration

				Stewardship and Restoration Services
Stewardship and Education				
	Landowner Brochure and educational display board in local parks	CVC, Peel, Caledon Residents		CVC Stewardship and Restoration Services
	Storm drain marking program (yellow fish road)	CVC	Volunteers, local interest groups	Volunteer work
	Conservation Plans for priority properties	CVC, Landowners	Peel, Caledon, University of Guelph	CVC Stewardship and Restoration Services
Monitoring and Reporting				
	Monitor Developments During construction	Applicant	CVC, T. of Caledon, Peel	Planning Act Approvals
	Monitor Developments after construction	Applicant, Caledon, Peel	CVC, Naturalist Clubs	
	Monitor Waste Water Treatment Facilities during and after Construction	Peel	CVC	Environmental Assessment Approvals
	Amphibian and terrestrial monitoring	CVC	Local interest groups, Region of Peel, T. of Caledon	Volunteer work
	Monitor Effects from Well #3	Peel, CVC	Caledon, Naturalist Clubs, MOE	Ont. Water Resources Act

[1] The facilitator is responsible to carry out the action
[2] The contributor provides support to the facilitator

Note that if work is being completed in or adjacent to a river that hosts fish life or habitat, enforcement of Federal Fisheries Act for any harmful alteration, disruption or destruction of fish habitat with an emphasis on restoration orders is recommended.

If chemical or thermal impacts are detected, retrofitting of the stormwater facilities or communal septic system to meet predetermined targets should be done.

If baseflow or spawning areas are impacted by decreasing groundwater supplies, pumping rates or the redistribution of withdrawals from other wells should be investigated. Water conservation programs and rationing for some uses may also be considered.

5.0 RECOMMENDATIONS

Hydrogeology

1. All proposed development must ensure that water table elevations, baseflow and aquifer yield are maintained in the post developed condition.

Hydrology

2. All proposed development must ensure that stormwater management techniques are utilized to manage water quantity, quality and erosion. In order to achieve a net reduction in phosphorus to the Credit River, storm water runoff from the existing Village Core must also be treated for quality issues.

The following recommendations were made during the Inglewood Village Tributary Study:

It is recommended that a surface water management strategy consisting of a combination of source, conveyance and end-of-pipe stormwater controls. Specifically that:

3. Lot level and conveyance controls be employed to the greatest extent feasible in all proposed urban developments. Impervious surfaces such as roofs and driveways should discharge to lawns and then to swales. Roads, storm sewers (if employed) and swales should discharge to broad, shallow drainage corridors where possible. The objective should be to spread runoff out over as great an area as possible, in order to take advantage of the limited natural infiltration capacity of the local soils.

4. End-of-pipe controls be employed in the proposed Balmer Heights development to provide erosion and peak flow control. The Balmer Heights facility should be a dry pond total active storage volume of 14,020 m³. The extended detention portion of active storage should comprise 6600 m³. Since the facility will drain lands in both Tributaries 1 and 2, its discharge should be split between these tributaries in proportion to their pre-development flows.

5. Water quality control be provided for Tributary 1 (including both new and existing development) by implementing a pond between the CPR tracks and the Caledon Trailway. The facility should be a wet pond, with a permanent pool of 4650 m³ and extended detention storage of 3200 m³.

6. The proposed South Slopes development be served by a single end-of-pipe facility located on-line, between the CPR tracks and the Caledon Trailway. The facility should be a combined wet pond with a permanent pool of 3000 m³ and total active storage of 5830 m³ (1750 m³ of which would be extended detention storage). Care must be taken in the design/redesign of the branches of Tributary 2 through the South Slopes property to ensure that they are geomorphologically stable and capable of conveying the anticipated flows from internal and external areas.

7. Due to the location, size, topography and difficulty of servicing anticipated for the McKenzie lands stormwater controls for this area will need to be implemented as part of the lot layout and grading strategy. For this reason, the proposed stormwater management for this area should be determined when specific development plans have been formulated.

Terrestrial

8. Prior to development proceeding within the Special Study Area shown on Schedule M, a detailed Environmental Impact Statement will be required to investigate the proposed road extension through the sensitive corridors, and the feasibility of developing the triangular area south-west of the corridor (see Appendix A).

9. Prior to development proceeding within Balmer Heights south and west of McColl Drive, the tributary and its associated corridor should be investigated to determine its most appropriate treatment. The details on its restoration and/or realignment should be assessed in detail by the applicant.

10. A detailed assessment is required for the preferred location of the wastewater treatment facility and its outfall.

11. Stewardship and restoration opportunities should be discussed with the landowners of high sensitivity corridors.

Fluvial Geomorphology

13. Stable functional tributaries need to be implemented using the principles of natural channel design techniques. In particular consideration should be given to the assisting the Credit River in "cutting off" the sharp meander bend just downstream of the proposed outfall location.

Water Quality

14. All critical processes in the treatment, redundancy be built into the system to accommodate for emergencies and routine maintenance activities. Therefore, there will be no need for the wastewater to be by-passed through the system at any time. However, in any case where wastewater is by-passed through any of the processes, the event will be described and the effluent will be sampled. This information will be forwarded to the appropriate agencies, including the Credit Valley Conservation and the Town of Caledon, and included in the annual report.

15. This reach of the Credit River be recognized as a Policy 2 river for bacteria and phosphorus and as such offsetting strategies should be implemented with respect to these parameters.

Fisheries

16. Potential impacts from increased pumping at Well #3 should be assessed over time through a monitoring program. The details of the monitoring program will be discussed with the Region of Peel, CVC, MOE and DFO.

17. That wastewater effluent targets be set with standards to protect aquatic life.

18. That detailed stormwater plans demonstrate that thermal impacts are fully mitigated.

19. That fish habitat within the river and tributaries be improved through a stewardship and restoration program.

DRAFT REPORT

Inglewood Village
Water and Wastewater Servicing Plans

June 1999

Prepared By:

XCG Consultants Ltd.

Prepared For:

Town of Caledon
Credit Valley Conservation
Region of Peel

XCG

CVC

TOWN OF CALEDON

Region of Peel

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FIGURES

1. INTRODUCTION

1.1 Background

1.1.1 Inglewood Village Study

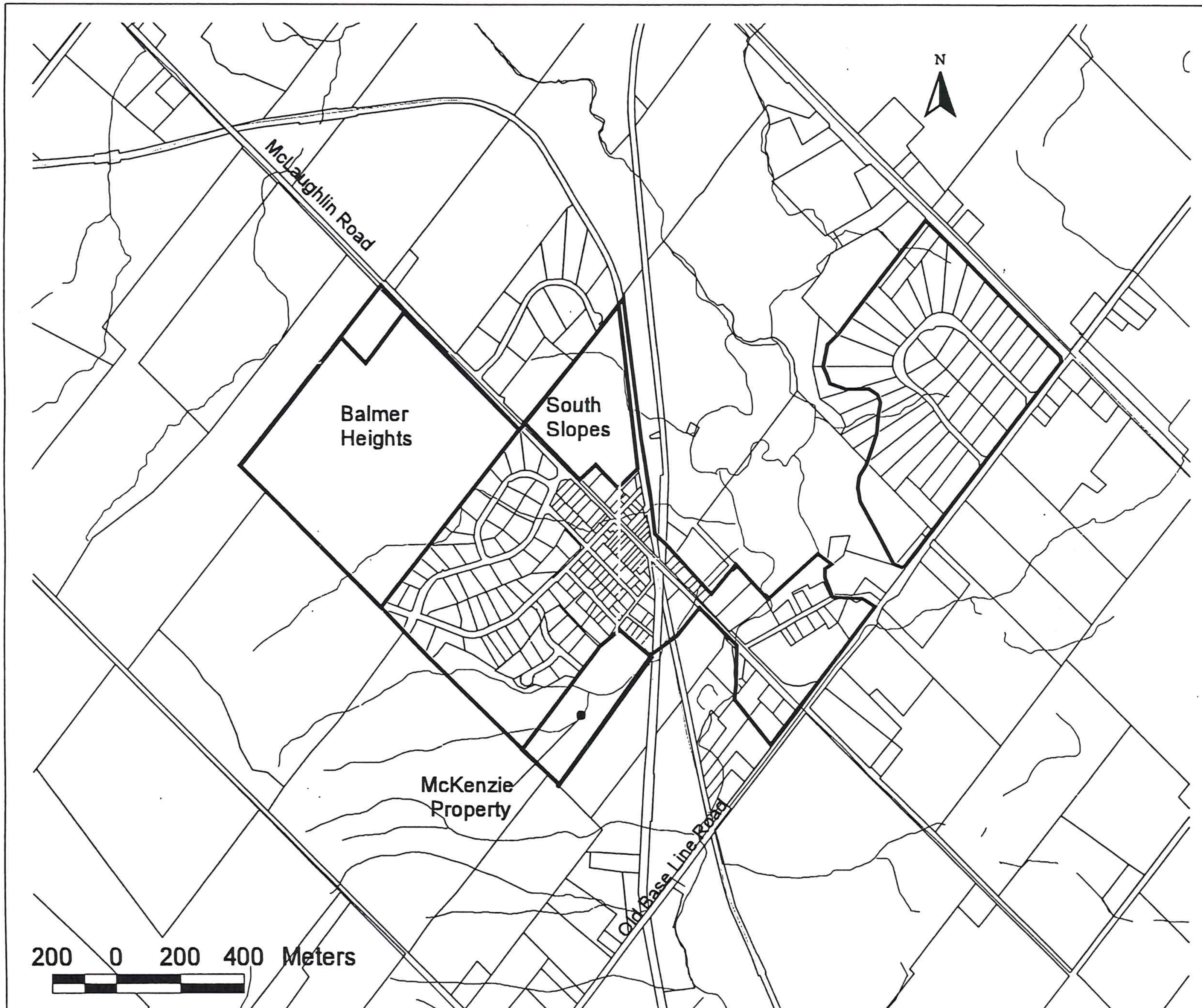
The Village of Inglewood in the Town of Caledon, Regional Municipality of Peel, is located west of the Credit River north of Olde Base Line Road in the Region of Peel. Figure 1.1 presents the village settlement boundary.

The Inglewood Village Study was initiated to address planning, environmental and servicing implications of growth to the year 2021 in a comprehensive and integrated manner. The study included the following three distinct but interrelated components:

- Community Planning Component: This component focussed on the identification of a preferred growth strategy for the community to the year 2021, including population, boundaries, land use patterns, densities and municipal financial impacts.
- Servicing Component: In this component, preferred water and wastewater servicing strategies for the recommended community plan were identified.
- Environmental Component: This component provides information on environmental conditions and resulted in the preparation of an environmental management plan.

The Inglewood Village Study was conducted in four distinct phases. Phase 1 consisted of the collection, review and analysis of existing data. Phase 2 involved of the development and evaluation of community planning scenarios and servicing alternatives and the selection of a preferred alternative(s). In Phase 3, draft community and servicing plans were developed, and in Phase 4, these plans were finalized. Figure 1.2 presents the study process. Planning are trying to Locate these. BE

The Inglewood Village Study management team includes representatives from the Town of Caledon, the Regional Municipality of Peel, and the Credit Valley Conservation (CVC). The Town led the Planning Component, the Region led the Servicing Component and the CVC led the Environmental Component.



Inglewood Village Servicing Study

Legend
Settlement Area Boundary
Undeveloped Land
Watercourses

Figure 1.1
Village of Inglewood
Settlement Boundary

XCG

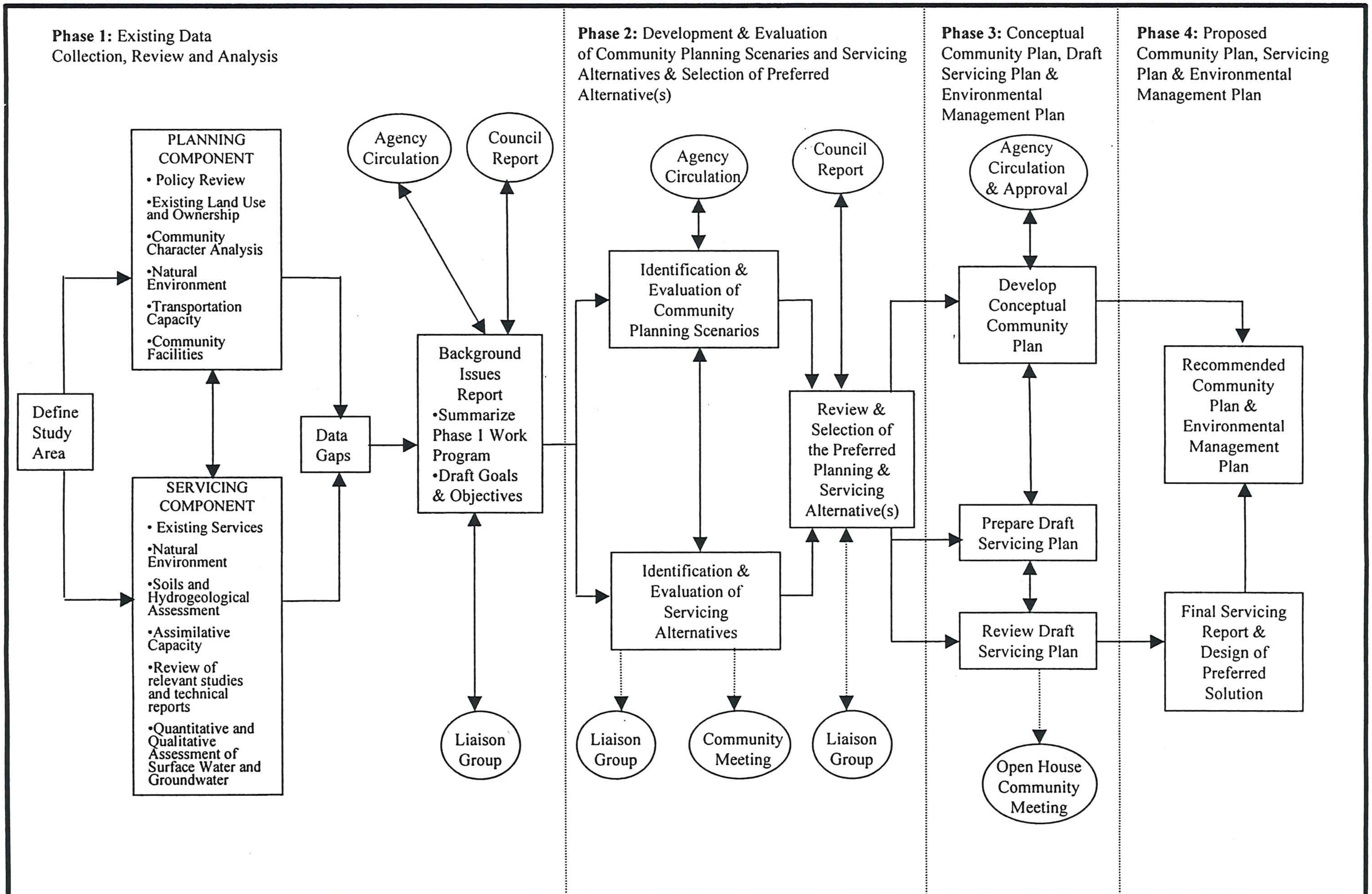


Figure 1.2
Inglewood Village Study Process

1.1.2 Servicing Component

The Servicing Component involved the examination of water and wastewater issues pertaining to the Community of Inglewood. It included an evaluation of existing water and wastewater servicing infrastructure, an assessment of alternatives for servicing the recommended growth scenarios, identification of the preferred servicing option, and development of water and wastewater servicing plans for the community to the year 2021.

The objectives of the water and wastewater servicing plans are as follows:

1. To provide water and wastewater servicing capacity to service planned to the year 2021.
2. To ensure that the development of the plan meets the requirements of the Class Environmental Assessment process
3. To provide wastewater servicing by municipal or communal systems, if feasible, rather than individual on-site systems.
4. To ensure adequate wastewater treatment to meet provincial regulations and policies for protection of the groundwater and surface water.
5. To provide adequate water treatment to ensure that water quality meets the Ontario Drinking Water Quality Objectives (ODWO) and disinfection requirements.
6. To size water and wastewater systems based on Regional Design Criteria, and in consideration of MOE design guidelines and standards.
7. To locate new water and wastewater facilities appropriately, in consideration of existing land use policies, and the recommended Community Plan and Environmental Management Plan.

The existing servicing in the Village of Inglewood, policy context for servicing, and a detailed evaluation of alternatives for water and wastewater servicing to the year 2021 were presented in the Phase 1 and Phase 2 Inglewood Village Study reports (November, 1998 and March, 1999). This report presents the results of Phases 3 and 4 of the Servicing Component. The objectives of Phases 3 and 4 were:

1. To evaluate alternative design concepts for the preferred water and wastewater servicing alternatives and to select the preferred design concepts.
2. To identify implementation considerations for water and wastewater servicing to the year 2021.
3. To finalize conceptual water and wastewater servicing plans for the Village of Inglewood to the year 2021.

1.2 Preferred Planning and Servicing Alternatives

The existing population of Inglewood village is 610. A municipal water supply system provides water servicing to the majority of the residents of Inglewood Village. The system includes three groundwater wells, a storage reservoir and a distribution system. Wastewater treatment in Inglewood Village is provided by private sewage disposal systems with subsurface discharge.

At the completion of Phase 2 of the Inglewood Village Study, the preferred planning and servicing scenarios for the Village to the year 2021 were identified. These include the following elements:

- Development of currently undeveloped areas to achieve mixed village densities, providing for a range of lot sizes, densities and household types, to achieve a maximum 2021 Village population of 1,500;
- Implementation of a communal wastewater system to service new development areas and also to provide for servicing of approximately 95 lots in the Village Core.
- Continued use of the local groundwater supply to provide water to existing and new development areas.

1.3 Class Environmental Assessment Process

Ontario's Environmental Assessment Act requires proponents to examine and document the environmental effects that might result from major projects or activities and their alternatives. Municipal undertakings became subject to the Act in 1981.

The process is based on five key principles:

1. Consultation with affected parties.
2. Consideration of a reasonable range of alternatives.
3. Identification and consideration of the effects of each alternative on all aspects of the environment.
4. Systematic evaluation of alternatives in terms of their advantages and disadvantages, to determine their net environmental impacts.
5. Provision of clean and complete documentation of the planning process followed, to allow "traceability" of decision- making with respect to the project.

The Class Environmental Assessment for Municipal Water and Wastewater Projects (June, 1994) was prepared by the Municipal Engineers Association (MEA) and outlines the procedures to be followed to satisfy EA requirements for water, wastewater and stormwater management facilities. The Class EA process involves a thorough, systematic and reproducible evaluation of alternatives for servicing and their impacts on the environment. Public and agency consultations are integral to the Class EA process.

The Class EA process requires that four phases of work be completed for Schedule C projects including problem definition, identification and evaluation of alternative solutions to determine a preferred solution, examination of alternative methods of implementing the preferred solution, and documentation of the planning, design and consultation process.

The development of the Servicing Component, led by the Region of Peel, followed the Class Environmental Assessment process for Schedule C projects. This report, in combination with Phase 1 and Phase 2 Village Study Report, meets the requirements of an Environmental Study Report, which is typically completed as part of Phase IV (documentation of the planning, design and consultation process) of the Class EA process.

1.4 Intent of Report

This report is intended to provide information on the development of Water and Wastewater Servicing Plans for the Village of Inglewood. Conceptual designs in this report were developed to estimate costs and to compare alternatives. This report is not intended to present preliminary design information.

All costs presented in this report are inclusive of engineering and contingency, but do not include applicable taxes or land purchase. Costs are considered accurate to ± 35% at this conceptual design level.

1.5 Report Outline

There are four sections in this document. Section 2 presents the development and implementation of a preferred alternative for water servicing. Section 3 provides details on the development and implementation of a preferred alternative for wastewater servicing. Section 4 presents the recommended water and wastewater servicing plans. References are contained in Section 5.

2. WATER SERVICING

2.1 Background

The existing water supply system includes three groundwater wells, a storage reservoir and a distribution system. Figure 2.1 presents the existing water supply system. Table 2.1 presents descriptions of the three groundwater wells.

Table 2.1 Description of Existing Groundwater Wells

Facility	Well Capacity (L/s)	Existing Well Permit (L/s) (MOE 96-P-3027)	Permitted Maximum Combined Water Taking
Well #1	7.5	Standby only, maximum 4.5h per day operation	1,295,000 L/d (15 L/s)
Well #2	15	15	
Well #3	60	20	

Currently, Well #3 is operated as the lead well and Wells #1 and #2 are used for standby capacity. Well #1 and Well #2 cannot be operated concurrently as defined by the current permit.

The existing reservoir provides a total storage volume of 730 m^3.

Regional design criteria were used to estimate the water servicing requirements to accommodate new growth to the year 2021. Table 2.2 presents criteria and capacity requirements for water supply and reservoir storage.

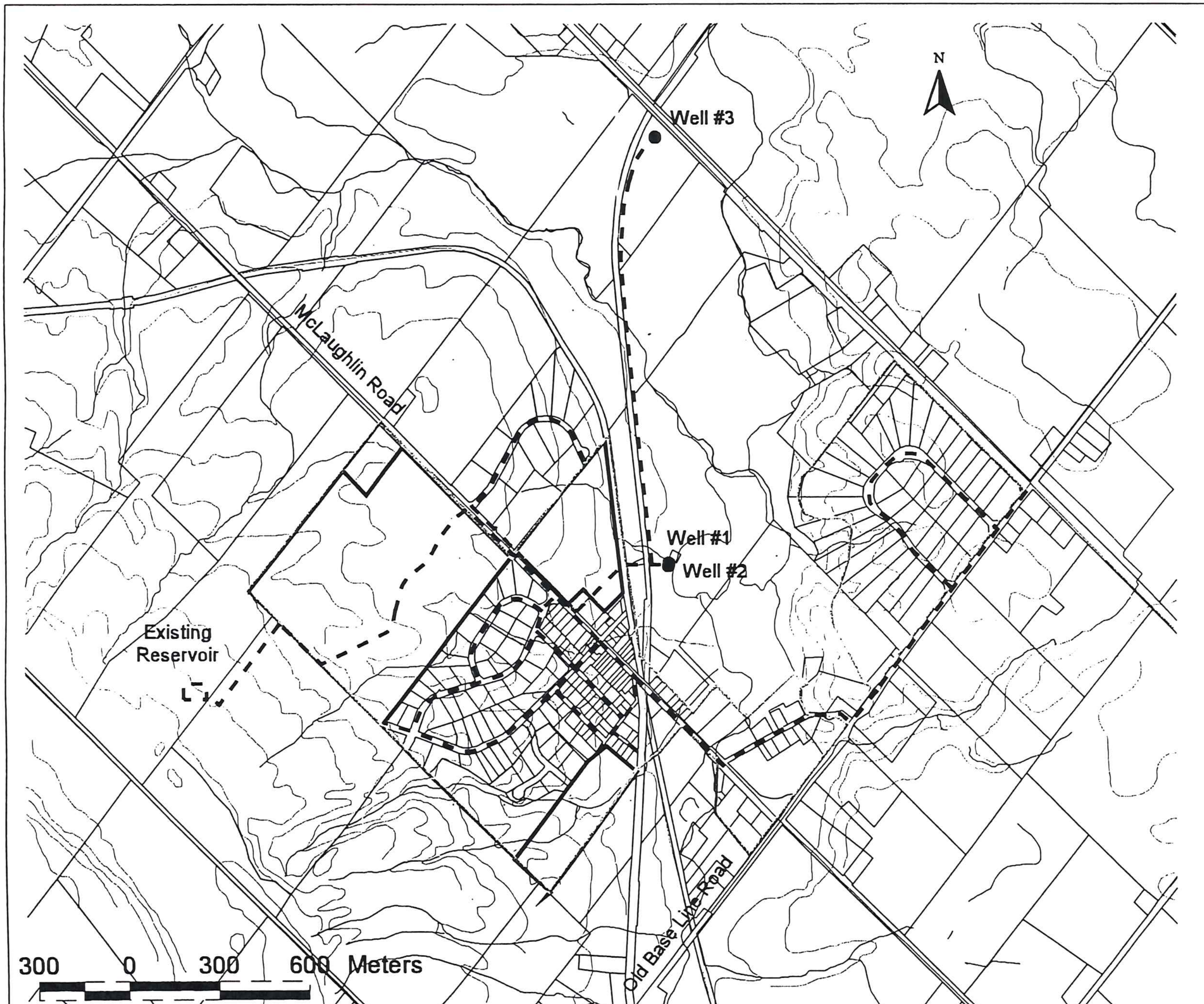
Table 2.2 Water Supply and Reservoir Capacity Required for 2021

Parameter	Regional Criteria	2021 Capacity
Average day flow	450 L/cap.d	680,000 L/d (7.9 L/s)
Peak day flow	2.5 times average	1,700,000 L/d (19.7 L/s)
Standby capacity	>10 L/s	>10 L/s
Reservoir volume	Average day flow plus 288 m^3	>968 m^3

The preferred servicing alternative to the year 2021 presented in Phase 2 of the Servicing Component included the following elements:

- Continued use of Well #3 as the operating well;
- An increase in the Permit to Take Water capacity for the combined water supply from the current capacity of 1,295,000 L/d to 1,700,000 L/d;

Inglewood Village Servicing Study



Legend

- Settlement Area Boundary
- Undeveloped Land
- Watercourses
- Existing Reservoir
- Existing Well
- Existing Watermain

Figure 2.1
Village of Inglewood
Existing Water
Distribution System

XCG

- Provision of a back-up or standby groundwater supply, providing at least 10 L/s capacity; and,
- Expansion of the reservoir from its existing volume of 730 m^3 to provide a minimum volume of 968 m^3.

2.2 Review of Design Concepts

Design concepts were review for providing water supply, standby capacity, storage and distribution for the preferred servicing alternative.

2.2.1 Water Supply

Well #3 has a rated capacity of 20 L/s, which is sufficient to meet the 2021 peak day demand of 19.7 L/s. An increase in the Permit to Take Water capacity to 1,700,000 L/d is required.

Three alternatives were considered for providing standby capacity, as follows:

- Use of Well #2, which has an existing capacity of 15 L/s;
- Interconnection of the existing distribution system with the Village of Cheltenham water supply system, about 4 km away; or,
- Installation of a new well.

Well #2 has historically provided water meeting the drinking water quality objectives. Furthermore, continued use of this well for standby capacity would have the lowest cost of the three alternatives. Therefore, the continued use of Well #2 was preferred for providing standby capacity.

2.2.2 Reservoir Storage

The existing reservoir has a total volume of 730 m^3 which is sufficient to service 990 persons. With an estimated maximum population of 1,500 persons in 2021, additional storage volume will be required at the reservoir. Expansion of the existing reservoir was identified as the preferred alternative for providing storage capacity.

The reservoir must be expanded to achieve a minimum volume of 968 m^3 to accommodate 1,500 persons. For the purposes of estimating capital costs, it was assumed that any reservoir expansion would involve a 50% increase in the capacity of the existing reservoir. The reservoir expansion would result in a total storage volume of 1,100 m^3. The total cost of expansion of the reservoir is estimated at $350,000.

2.2.3 Distribution System

The existing water distribution system consists of 150 mm, 200 mm and 300 mm diameter watermains which distribute the water pumped from the existing wells to the Village of Inglewood. There are two pressure zones within Inglewood with the upper pressure zone floating on the reservoir. Pressures in the lower pressure zone are controlled by a pressure reducing valve (PRV) located at McColl Street and McLaughlin Road. Figure 2.1 presents the existing water distribution system.

An analysis of the existing water distribution system was undertaken to determine if any improvements to the existing system would be required to accommodate development within the Balmer Heights, South Slopes or McKenzie Propertys. A previous study of water distribution systems in the Town of Caledon undertaken by KMK Consultants Limited (1997) involved the development of a Waterworks model of the water distribution system. This model was used to analyze the existing water distribution system for existing water demands and the 2021 water demand scenario.

The water distribution system was analyzed to ensure that residual pressures at all serviced areas would remain between 275 kPa and 690 kPa for both existing water demands and for the 2021 water demands. Analyses were conducted for the peak day demand and the peak day demand with fire flow requirements.

Appendix A contains the detailed results of the analyses conducted. The following sections describe the results of the analyses conducted.

Analysis of Existing Conditions

The existing water distribution system was analyzed for the existing service population for peak day demand and peak day demand with fire flow requirements. A fire flow demand of 40 L/s was simulated based on Regional design criteria.

The following points summarize the results of the analyses:

- Pressures were found to be above 690 kPa along the lower lengths of the feedermain from Well #3 as a result of the peak day demand. As no homes connected directly to the feedermain, pressures higher than 690 kPa in the feedermain have no negative impact on residents;
- Pressures were found to be marginally below 275 kPa at the intersection of Fieldcrest Road and West Village Drive due to a local high ground elevation as a result of the peak day demand; and,
- The results of the peak day demand and fire flow analysis indicated that a fire flow of 40 L/s was not possible at all locations within the service area.

In particular, a fire flow of 40 L/s could not be achieved within the Ingleview Drive subdivision and at the intersection of Fieldcrest Road and West Village Drive.

Analysis of 2021 Conditions

The water distribution system was also analyzed for the 2021 maximum service population of 1,500 persons. Analyses were conducted for the peak day demand and the peak day demand and fire flow requirements. A fire flow demand of 40 L/s was simulated based on Regional design criteria.

It was assumed that the development of the Balmer Heights area would result in the connection of the MacDonald Street watermain with a future watermain servicing Balmer Heights. A pressure reducing valve is also planned for installation at the intersection of West Village Drive and McLaughlin Road. Previous studies, commissioned by the Region, have shown that this PRV is required to increase water pressure to existing residences on West Village Drive.

The following points summarize the results of the analyses:

- Pressures were found to be above 690 kPa along the lower lengths of the feedermain from Well #3 as a result of the peak day demand. As no homes connected directly to the feedermain, pressures higher than 690 kPa in the feedermain have no negative impact on residents.
- Pressures throughout the remaining watermain in the village were found to be within the range of 275 kPa and 690 kPa as a result of the peak day demand.
- The results of the peak day and fire flow analysis indicated that a fire flow of 40 L/s was possible at all locations within the service area.

Based on this analysis, the water distribution system is adequately sized to continue to meet the water demand of the existing village and to meet the needs of future development. The servicing of the Balmer Heights area with the extension of the MacDonald Street watermain and the installation of a new pressure reducing valve at the intersection of West Village Drive and McLaughlin Road, would ensure that a fire flow of 40 L/s can be achieved within the entire service area.

2.3 *Implementation*

Implementation of the water servicing projects would be triggered by development within the new development areas. Table 2.3 presents the implementation considerations for the preferred alternative.

Table 2.3 Implementation Considerations for Water Servicing Alternative

Project	Implementation Considerations
Connection of MacDonald Street Watermain with Balmer Heights area	Connection and PRV will be required to service Balmer Heights.
Installation of a new PRV at the intersection of West Village Drive and McLaughlin Road	Installation will increase existing water pressures in the West Village Drive area. Installation can be completed following the completion of the Servicing Component and Class EA Study.
Permit to Take Water	New permit must be in place for more than 123 lots to be approved for development. Permit application can be completed following the completion of the Servicing Component and Class EA study.
Reservoir Expansion	Expansion will be triggered with new development. Project can be completed in a single phase.

The completion of the connection of a watermain in the Balmer Heights area and the existing MacDonald Street watermain will be required to service the Balmer Heights area. Installation of a new PRV at the intersection of West Village Drive and McLaughlin Road will improve existing water pressures and can be completed following the completion of the Servicing Component and Class EA Study.

A new Permit to Take Water can be applied for following the completion of this Class EA study. The new permit can also specify Well #1 and #2 as standby wells.

The reservoir expansion will be triggered by new development. The existing reservoir capacity is sufficient to service a population of 990 persons. Expansion of the reservoir can be completed in a single phase.

2.4 Capital Cost of Projects

The capital costs have been estimated for each project.

The connection of the MacDonald Street Watermain with the Balmer Heights area would be within the Balmer Heights lands. The cost of the new PRV located at the intersection of West Village Drive and McLaughlin Road is estimated at $75,000. The cost of the reservoir expansion is estimated at $350,000. Costs at this conceptual level are considered ±35%.

The capital cost of the Permit to Take Water will be negligible.

3. WASTEWATER SERVICING

3.1 Background

Wastewater treatment in Inglewood Village is currently provided by private sewage disposal systems with subsurface drainage. A 1999 survey of the septic systems in the Village (XCG) revealed that systems in the newer areas were designed according to current standards and are operating well. That survey also indicated that, primarily due to age and undersizing of the systems, systems in the older Village Core were deteriorating. There was evidence of environmental impacts from failing systems, which would be anticipated to worsen over the design period if a major overhaul or system replacement were not undertaken.

In light of the capacity requirements for servicing new development areas, and the condition of the existing septic systems, the recommended wastewater servicing alternative for the year 2021 includes:

- Construction of a new collection system to collect wastewater generated from new development areas and the Village Core area; and,
- Construction of a new tertiary wastewater treatment plant with a continuous discharge to the Credit River.

Regional design criteria were used to develop the wastewater collection system capacity requirements and conceptual designs to the year 2021. Table 3.1 presents the capacity requirement for wastewater treatment.

Table 3.1 Wastewater Treatment Plant Capacity Required for 2021

Parameter	Design Criteria	2021 Capacity
Average daily flow	393 L/cap.d	390 m^3/d[1]

Note:

1. Based on servicing a total population of 945, including 95 lots in the Village Core and 220 lots in new development areas (3.15 ppu).

3.2 Review of Design Concepts

Design concepts were reviewed for the collection system type, the treatment plant location and the treatment technology for the preferred wastewater servicing alternative.

3.3 Wastewater Treatment Plant Process

In light of the sensitivity of the Credit River to receive wastewater from the treatment plant, and regulations and policies governing direct discharges to Ontario surface waters, effluent limits were proposed for the new treatment facility in Phase 2 of the Inglewood Village study. These are presented in Table 3.2.

Table 3.2 Conceptual Effluent Requirements for Treatment Plant

Parameter	Value
Biochemical oxygen demand (BOD_5) (mg/L)	15
Total suspended solids (TSS) (mg/L)	15
Ammonia-nitrogen (mg/L)	2
Nitrate-nitrogen (mg/L)	No limit
Total phosphorus (mg/L)	0.15
Acute toxicity to Daphnia Magna and Rainbow Trout	Non-toxic

In order to achieve the proposed effluent limits, the conceptual design of the treatment plant includes the following processes:

- Headworks, including grit removal and screening;
- Secondary biological treatment with nitrification;
- Coagulant addition for phosphorus precipitation;
- Tertiary phosphorus removal;
- Ultraviolet disinfection; and,
- On-site sludge storage with off-site disposal.

The total estimated cost for the treatment plant is estimated at $1,100,000. At this conceptual level, costs are considered accurate to ±35%.

3.3.1 Collection System Technologies

Three collection system technologies were considered to collect wastewater from 95 lots within the existing Village Core and new development areas, and to convey it to the new wastewater treatment plant. The technologies considered included a gravity collection system, a vacuum sewer collection system, and a pumped collection system.

Gravity Collection System

A gravity collection system is the most commonly used type of system for collecting municipal wastewater and transferring it to centralized treatment facilities. For Inglewood, a new sewer system would involve the construction of a gravity sewer system along existing roadways in the Village Core, and along new roads within the new development areas. For existing homes, internal plumbing changes and new laterals from the building to the sewer at the road would be required. Existing septic systems would no longer be used. For new development areas, connections to the building would be constructed at the time of development.

Vacuum Sewers

For areas with existing septic tanks, a vacuum sewer collection system can be used to collect and transmit flow from the septic tanks to the treatment facilities. This type of system can be used where septic tanks are already in place, and has the primary advantages of lower collection and treatment costs (because partial treatment occurs in the septic tank) compared to conventional gravity sewers.

Vacuum sewers involve the construction of small diameter forcemain along existing roadways in the village core. The forcemain would be connected to the existing septic tanks. A pumping station would be required to create a negative pressure in the system to withdraw effluent from the septic tanks to a wastewater treatment plant.

Pumped Collection System

Similar to a vacuum system, a pumped collection system can also be used where septic tanks are already in place. For this type of system, each septic tank is equipped with a grinder pump, which operates automatically based on levels in the septic tank, to pump effluent from the septic tank to centralized treatment facilities. Like vacuum sewers, this type of system can be used where septic tanks are already in place, and has the primary advantages of lower collection and treatment costs (because partial treatment occurs in the septic tank).

Comparison of Collection System Alternatives

Table 3.3 presents a comparison of the advantages and disadvantages of three types of collection systems.

Table 3.3 Advantages and Disadvantages of Wastewater Collection System Technologies

System Type	Advantages	Disadvantages
Vacuum Sewer Collection System	• Existing septic systems would remain in place. • Vacuum sewers would be smaller diameter and could be installed at a shallower depth than gravity system. • Vacuum system would have a lower cost than gravity system. • Treatment plant requirements would be reduced because of partial treatment in septic tank.	• Residents would still be responsible for the maintenance of their septic tanks. • Higher operating costs associated with vacuum operation compared to gravity system. • Leaking septic tanks would have to be replaced by property owners. • Potential for septic tank overflow which would impact on local watercourses if septic tanks are not maintained.
Pumped Collection System	• Existing septic systems would remain in place. • Smaller diameter forcemain could be installed at a shallower depth than gravity sewers. • Pumped system would have a lower cost than gravity system. • Treatment plant requirements would be reduced because of partial treatment in septic tank.	• Residents would still be responsible for the maintenance of their septic tanks and pumps. • Operating costs associated with pump operation. • Leaking septic tanks would have to be replaced by property owners. • Potential for septic tank overflow which would impact on local watercourses if septic tanks are not maintained.
Gravity Sewer Collection System	• Operating costs associated with gravity system are less than other systems. • Residents would not be responsible for any aspect of operation or maintenance of wastewater system.	• Higher capital cost than vacuum or pumped system.

For new subdivision areas, a conventional gravity system is considered to be the preferred alternative. With other types of collection systems, the requirement for individual septic tanks for each new lot would significantly reduce the cost advantage of these systems.

For the Village Core area, the poor existing condition of the septic tanks suggest that systems which depend on these tanks would not be appropriate unless major repair or replacement of undersized and deteriorated tanks was undertaken. The costs to residents to complete this work would reduce the cost advantage of these types of systems. It should be noted that the septic system survey (XCG, 1999) was not comprehensive, and there may be certain areas in the Village Core where these types of systems could be considered at the detailed design stage.

The preferred design concept for collection of wastewater from the Village Core and new development areas is based on a gravity sewer system.

3.4 Wastewater Treatment Plant Site

Two alternative sites for location of the wastewater treatment plant were developed. To enable a comparison of alternatives, the conceptual collection system configurations and costs were developed for each alternative.

The following sections provide details on the development and evaluation of the alternatives and the selection of the preferred alternative.

3.4.1 Development of Alternatives

In consideration of the environmental sensitivities in the area as defined in the Environmental Component, and other practical constraints, criteria for wastewater treatment plant siting were identified.

Table 3.4 presents the criteria used to identify feasible wastewater treatment plant sites.

Table 3.4 Criteria for Identification of Alternative Wastewater Treatment Plant Sites

Criteria	Description
Floodline Criteria	The wastewater treatment plant must be located outside of the Regional floodline.
Outfall Criteria	Outfall selection: • Located within main Credit River, not discharged to tributaries; • Avoid discharging to sensitive wetland/Environmentally sensitive areas; • Avoid spawning areas (none mapped 700 m upstream of McLaughlin Road bridge)
Design Considerations	The treatment plant should be located at relatively low elevation, to minimize the costs associated with wastewater pumping. The treatment plant should be relatively close to the community and discharge point to minimize infrastructure costs.

Sites located within the Regional floodplain for the Credit River were eliminated from further consideration. Outfall siting criteria were also applied to identify alternative wastewater treatment plant sites. The Credit Valley Conservation Authority (CVC) indicated that any wastewater treatment plant outfall should not discharge into sensitive reaches of the Credit River. Reaches which have been indicated as sensitive include the bend located immediately downstream of the confluence of the Credit River and the Little Credit River.

Design considerations were also included as a criterion for the identification of wastewater treatment plant sites. Sites which were located on high ground were avoided to minimize pumping requirements. In addition, it was recognized that the wastewater treatment plant should be located in a reasonable proximity to the existing village to avoid the need to construct long pipelines.

The application of the criteria listed in Table 3.4 resulted in the identification of two alternative sites for the wastewater treatment plant. The first alternative involved the construction of the wastewater treatment plant at a site adjacent to the Caledon Trailway. The second alternative involved the construction of the wastewater treatment plant on a site located within the McKenzie Property. The following sections provide the details of these alternatives.

3.4.2 Description of Alternatives

Alternative 1 – Wastewater Treatment Plant Located Adjacent to Caledon Trailway

Figure 3.1 presents the location of the wastewater treatment plant and the associated collection system for this alternative. This alternative would involve the construction of the wastewater treatment plant on lands adjacent to the Caledon Trailway at the end of MacDonald Street. The outfall would be located downstream of the bend in the Credit River.

For the conceptual collection system design associated with this treatment plant site, wastewater from the Balmer Heights area would be directed either by forcemain or gravity sewer to a sewer on McLaughlin Road. Wastewater from the South Slopes area along with MacDonald Street and most of McLaughlin Road would be conveyed by gravity to the wastewater treatment plant. Wastewater from the remaining existing Village Core and the McKenzie Property would be conveyed to a new pump station on McLaughlin Road. This pump station would pump flow a short distance north into a new gravity sewer located on McLaughlin Road north of McKenzie Street.

Preliminary information on the collection system associated with this treatment plant location is provided in Appendix B.

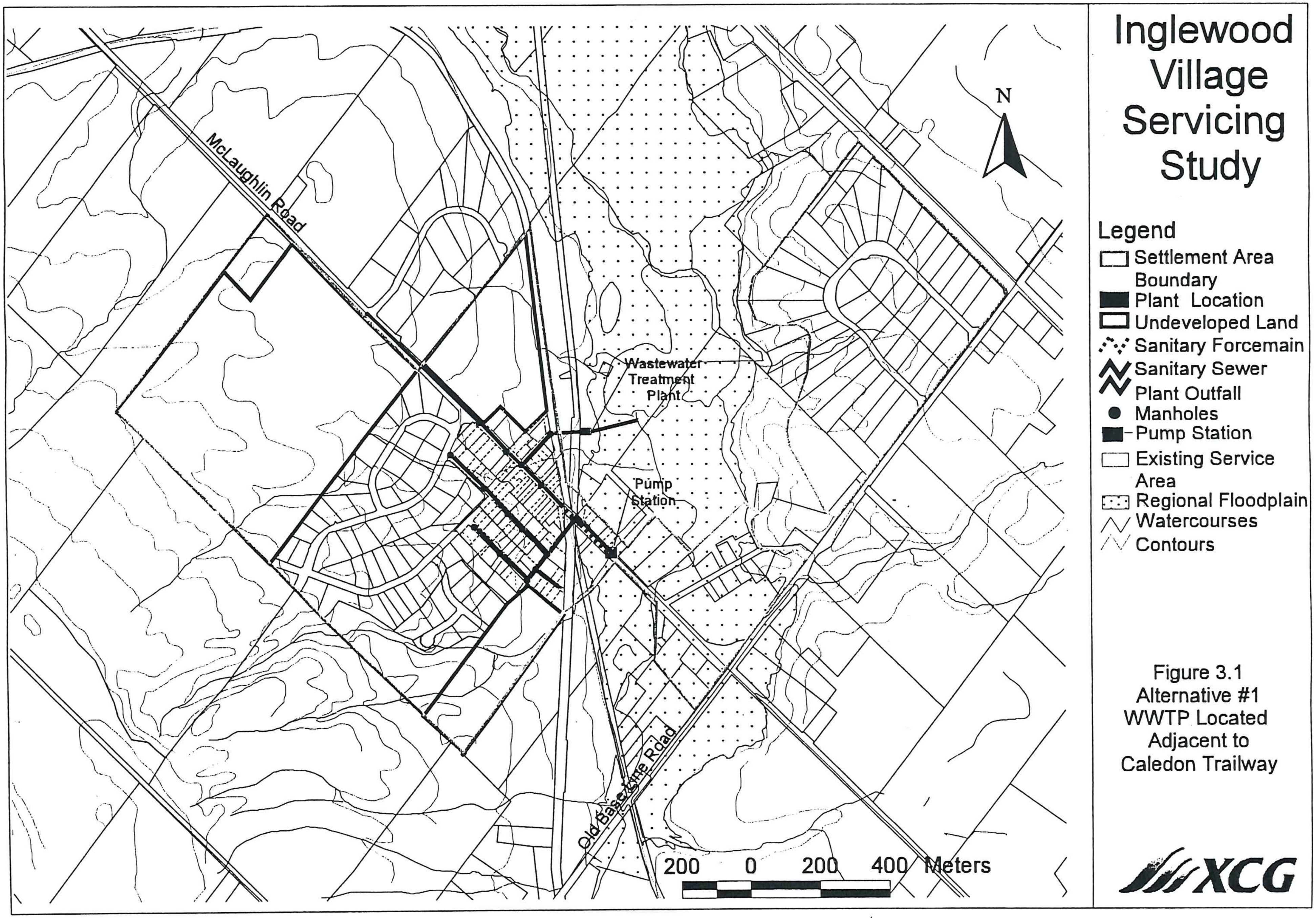


Figure 3.1
Alternative #1
WWTP Located
Adjacent to
Caledon Trailway

3.4.3 Alternative 2 – Wastewater Treatment Plant Located in McKenzie Property

Figure 3.2 presents the location of the wastewater treatment plant and associated collection system for this alternative. This alternative would involve the construction of the wastewater treatment plant on lands within the McKenzie Property adjacent to McKenzie Street. The outfall for this plant location would discharge into the Credit River at the McLaughlin Road bridge over the Credit River.

For the conceptual collection system design, wastewater from the Balmer Heights Area would be conveyed to a sewer on McLaughlin Road. Wastewater from the majority of the Village Core and the South Slopes area would be conveyed to a sewer located on McLaughlin Road to the wastewater treatment plant. Wastewater from McLaughlin Road south of McKenzie Street would be pumped to the treatment plant.

Preliminary information on the collection system associated with this treatment plant location is located within Appendix B.

3.4.4 Evaluation of Alternatives

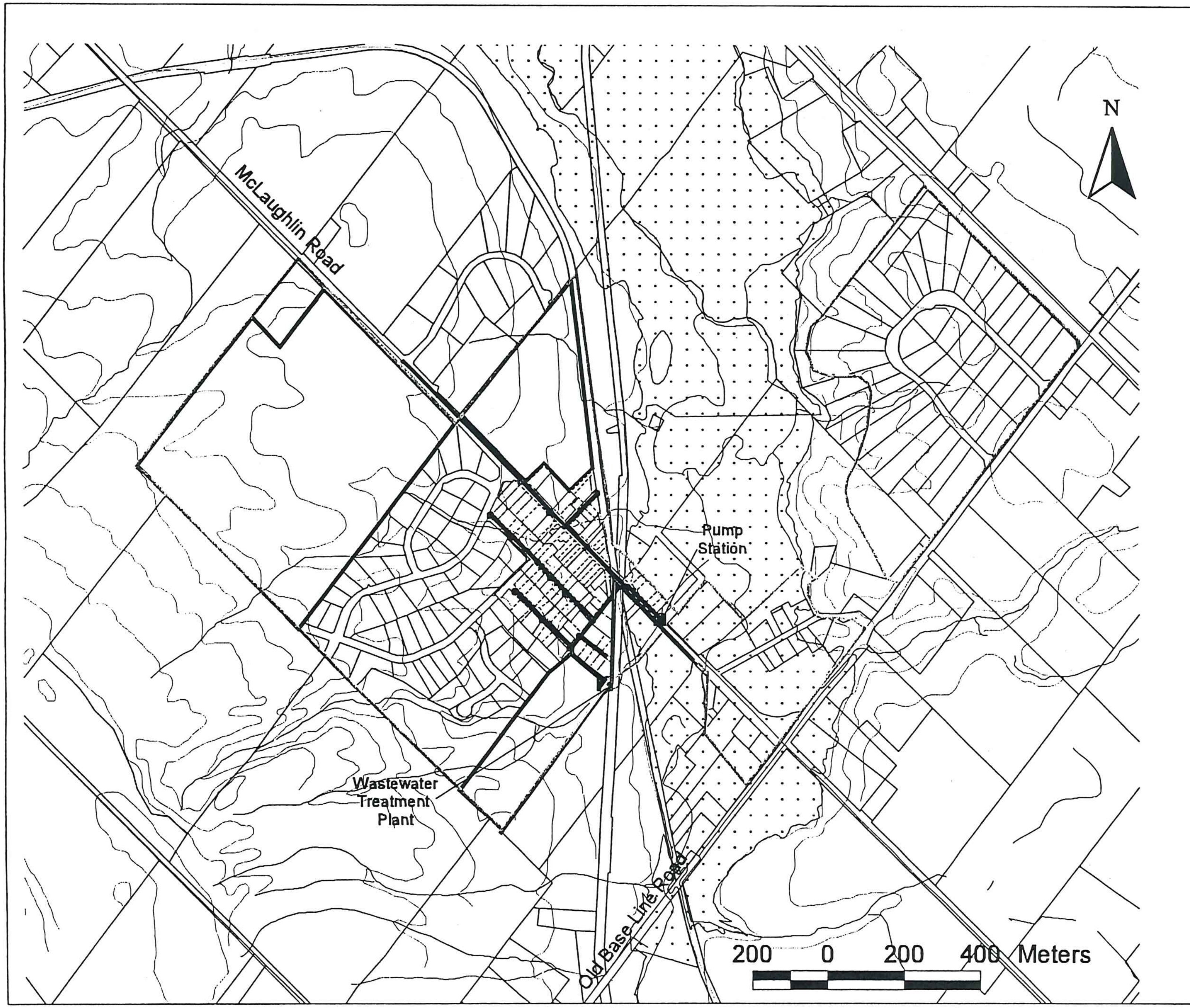
An evaluation of the two alternatives described in the previous section was undertaken to select a preferred wastewater treatment plant location. The following criteria were to evaluate the alternative design concepts.

- Environmental impacts;
- Community impacts;
- Impact on development;
- Expandability; and,
- Cost.

Table 3.5 presents descriptions of the evaluation criteria. The following sections present the evaluation of the alternatives.

Table 3.5 Description of Evaluation Criteria

Criteria	Description
Environmental Impacts	Impacts arising due to construction of wastewater treatment plant outfall to Credit River.
Community Impacts	Potential visual, noise, and odour impacts on existing and future residents and users of Village area.
Impact on Development	Impact on availability of developable lands.
Expandability	Ability to expand treatment plant in the future (after 2021).
Cost	Capital cost of alternative.



Inglewood Village Servicing Study

Legend

- Settlement Area Boundary
- Plant Location
- Undeveloped Land
- Sanitary Forcemain
- Sanitary Sewer
- Plant Outfall
- Manholes
- Pump Station
- Existing Service Area
- Regional Floodplain
- Watercourses
- Contours

Figure 3.2
Alternative #2
WWTP Located
in McKenzie
Property

XCG

Table 3.6 presents the information collected for the two alternatives. The following sections discuss the results of the evaluation.

Table 3.6 Information on Alternatives for Wastewater Treatment Plant Sites

Criteria	Alternative 1 – Wastewater Treatment Plant Located Adjacent to Caledon Trailway	Alternative 2 – Wastewater Treatment Plant Located Within McKenzie Property Area
Environmental Impacts	• Outfall sewer may be constructed at the location of an existing deciduous forest. Trees may need to be removed as part of construction. • Impact can be mitigated through good construction practice.	• Outfall sewer would be constructed along existing road allowances. • Outfall would be closer to fish spawning area than for Alternative 1.
Community Impacts	• Potential visual impact of treatment plant on users of the Caledon Trailway could be mitigated with fences and berms. • Trailway would provide a buffer between existing village and treatment plant. • Buffer of approximately 100 m from residents would ensure minimal noise impacts. • Minimal odour impacts as odour control would be incorporated into the design of the wastewater treatment plant.	• Potential visual impact of treatment plant on existing residents and future residents on Kaufman Road, Inglenook Court and in McKenzie Property as plant would be located at a lower elevation than many residences (i.e. if houses would look out over plant). • Buffer of 100 m from residents would ensure minimal noise impacts. • Minimal odour impacts as odour control would be incorporated into the design of the wastewater treatment plant.
Impact on Development	• No impact on developable lands as site is not located within a development area.	• Wastewater treatment plant and required buffer would reduce developable lands within McKenzie Property by 1 to 2 ha.
Expandability	• Adequate land area exists to expand the plant beyond the 2021 capacity.	• Adequate land area exists to expand the plant beyond the 2021 capacity.
Cost	• Estimated capital cost of $2.3 million for collection and treatment (excluding components within new development areas, including engineering and contingencies).	• Estimated capital cost of $2.6 million for collection and treatment (excluding components within new development areas, including engineering and contingencies).

Environmental Impacts

The outfall for Alternative 1 would be located downstream of the bend in the Credit River. A study of the area has determined that the outfall sewer may be located in close proximity to an area within an existing deciduous forest. As the method of construction would likely be open trench construction, trees in the vicinity of the outfall sewer may have to be removed. However, this impact could be mitigated as part of the detailed design of the outfall sewer by siting the outfall sewer to avoid the deciduous forest.

Alternative 2 would involve the construction of an outfall sewer along the existing road allowances of McKenzie Street and McLaughlin Road. The impact of the outfall on the Credit River would be similar for both alternatives as the river at both locations has a similar sensitivity to erosion.

Community Impacts

The wastewater treatment plant in Alternative 1 would be constructed on lands located east of the existing village adjacent to the Caledon Trailway. The wastewater treatment plant could have a visual impact on the users of the Caledon Trailway. However, fences or berms could be constructed to minimize this impact. Furthermore, the plant would not be visible to residents.

A wastewater treatment plant for Alternative 2 would be located on lands west of the existing village and would likely have a visual impact to residents on Kaufman Road, Inglenook Court, and future residents in the McKenzie Property. The plant would be located at a lower elevation and within the site line of residences located on Kaufman Road, Inglenook Court and in the McKenzie Property.

For both alternatives, noise impacts would be minimal. A buffer of 100 m, recommended by the Ministry of the Environment, would limit noise impacts to existing village core residents and future residents of the McKenzie Property.

Odour impacts on existing residents and future residents of the McKenzie Property and users of the Caledon Trailway would be minimal. Odour control would be incorporated into the design of the new wastewater treatment plant.

Impact on Development

Alternative 2 would involve the construction of a wastewater treatment plant on lands within the McKenzie Property. A buffer would also be required between future development and the wastewater treatment plant, resulting in a total land requirement of 1 to 2 ha. Therefore, Alternative 2 would reduce the amount of developable land within the McKenzie Property. Alternative 1 would not result in a reduction of developable lands.

Expandability

Adequate land area is available in both alternatives for future expansion of the treatment plant beyond the 2021 capacity. Alternative 1 would involve the construction of the wastewater treatment plant on lands located outside the village boundary while Alternative 2 would involve the construction of the wastewater treatment plant on land located inside the village boundary.

Capital Cost

The capital cost of each alternative was developed for both the wastewater treatment plant and the associated collection system. Costs estimates are for infrastructure outside of new development areas, do not include land purchase, and are inclusive of 30% engineering and contingencies. At this conceptual design level, capital costs are accurate to within ±35%.

Table 3.7 presents the capital costs developed for the two alternatives.

Table 3.7 Capital Costs of Wastewater Systems for Alternative Sites

Description	Alternative #1	Alternative #2
Collection Sewers	$940,000	$950,000
Outfall Sewer	$150,000	$300,000
Pump Stations	$200,000	$200,000
Wastewater Treatment Plant	$1,100,000	$1,100,000
Total	**$2,400,000**	**$2,600,000**

3.4.5 Rationale for Selection of Preferred Alternative

Alternative 1 was selected as the preferred alternative based on the evaluation presented above, for the following reasons:

- The potential visual impacts associated with the treatment plant would only affect Trail users, and could be mitigated. By comparison, for Alternative 2, the treatment plant would be visible to a number of residences.
- Construction outside of the Village boundaries means that developable lands are not used. By comparison, for Alternative 2, developable land would be reduced by 1 to 2 ha.
- Alternative 1 has the lower capital cost of the two alternatives.
- The Environmental Component identified this Alternative as preferred from an environmental standpoint.

3.5 Implementation

Table 3.8 presents the implementation considerations for the various components of the preferred alternative.

Table 3.8 Implementation Considerations for Wastewater Servicing Alternative

Project	Implementation Considerations
Wastewater treatment plant and outfall	Construction will be required for any new development to proceed, or to service existing Village Core.
Sewer on McLaughlin Road from McColl Drive to MacDonald Street	Construction will be required to service Balmer Heights, or to service Village Core lots on McLaughlin Road north of MacDonald Street.
Sewer on MacDonald Street to treatment plant	Construction will be required to service Balmer Heights, South Slopes, McKenzie Property or any Village Core lots.
Sewer on McKenzie Street, McLaughlin Road running south from MacDonald Street and McLaughlin Road pumping station and forcemain	Construction will be required to service Village Core lots on MacKenzie, Louise, Victoria and Lorne Streets, lower end of McLaughlin Road and the McKenzie Property.
Sewers on Louise, Victoria and Lorne Streets	Construction will be required to service Louise, Victoria and Lorne Streets in Village Core.

For development in either the Balmer Heights, South Slopes or the McKenzie Property, construction of the wastewater treatment plant and certain components of the collection system will be required.

The treatment plant could be constructed in one, two or three phases. However, most of the cost for the plant and all of the outfall cost would be required in the first phase. Construction of the first phase of the wastewater treatment plant would have to be completed before occupancy of the approved building lots.

Construction of sewers on Louise, Victoria and Lorne Streets and on McLaughlin Road south of McKenzie Street is required to service the Village Core. In addition, sewers on McKenzie Street and McLaughlin Road from McKenzie Street to MacDonald Street are also required to service the Village Core, as well as the McKenzie Property. For the Village Core, construction of connections to the property line is also required.

4. WATER AND WASTEWATER SERVICING PLANS

To provide for a maximum population of 1,500 in the year 2011, water and wastewater servicing plans were developed for the Village of Inglewood. The following sections present the conceptual water and wastewater servicing plans.

4.1 Water Servicing Plan

Table 4.1 presents the description, implementation considerations and cost estimates for each project that will be required to service the 2021 population in the Village of Inglewood. Cost estimates for each project includes engineering and contingencies.

Table 4.1 Water Servicing Plan

Project	Implementation Consideration	Project Cost
Connection of MacDonald Street Watermain with Balmer Heights area	Connection will be required to service Balmer Heights and therefore will be triggered by the development of these lands.	Within new development area
Installation of a new PRV at the intersection of West Village Drive and McLaughlin Road	Installation could be completed following completion of Servicing Component of Class EA as PRV will increase existing water pressures for residents on West Village Drive	$75,000
New Permit to Take Water	New permit must be in place for more than 123 lots to be approved for development. Permit application can be completed following the completion of the Servicing Component and Class EA study.	Permit application cost – negligible
Reservoir Expansion	Expansion will be triggered with new development. Project can be completed in a single phase.	$350,000

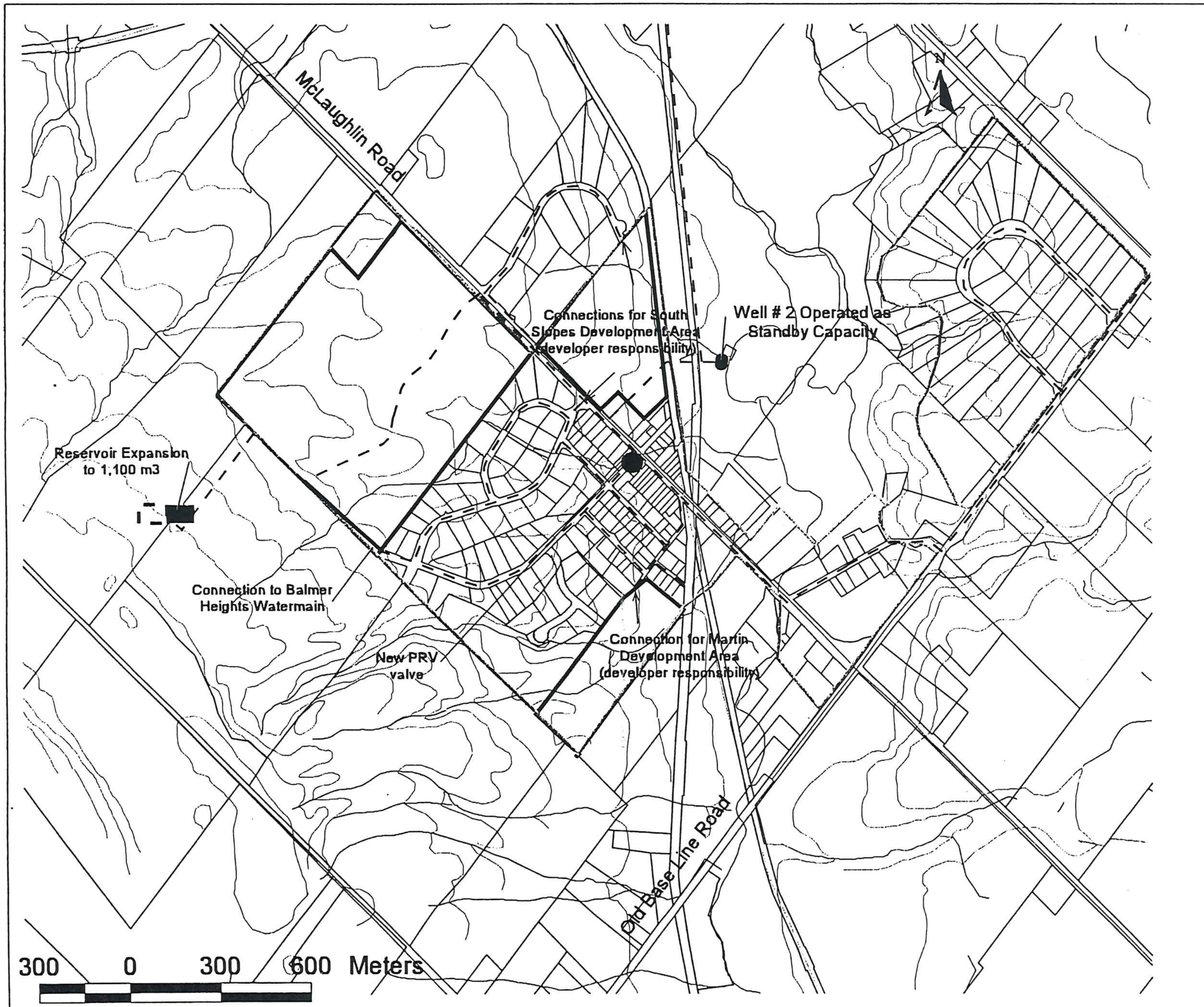
Figure 4.1 presents the servicing projects including in the water servicing plan. The following sections present detailed information for each project.

Connection of MacDonald Street Watermain to Balmer Heights Watermain

As part of the development of the Balmer Heights subdivision, a future watermain will be constructed to connect with the existing watermain on MacDonald Street. This project will be within the Balmer Heights subdivision.

Installation of a New Pressure Reducing Valve

To improve water pressures within the West Village Drive area, a new PRV valve should be installed at the intersection of West Village Drive and McLaughlin Road. This PRV will allow residents on West Village Drive to be within the upper



Inglewood Village Servicing Study

Legend

- Settlement Area Boundary
- Undeveloped Land
- Watercourses
- Existing Reservoir
- Existing Well
- Existing Watermain
- Reservoir Expansion

Figure 4.1
Village of Inglewood
Water Servicing Plan

XCG

pressure zone and will therefore ensure that fire flow requirements can be met in this area. This project can be completed following the completion of this study. The project cost for the new PRV has been estimated at $75,000.

New Permit Requirements

A new Permit to Take Water will be required for the existing wells to allow for a capacity expansion to 1,700,000 L/d to service growth to the year 2021. The existing wells have adequate capacity to allow for this expansion. The permit application can be completed following the completion of this study.

Reservoir Expansion

An expansion of the existing reservoir from 730 m^3 to 1,100 m^3 will be required to service future growth. The reservoir expansion will be required for new development to occur. This project can be completed in a single phase. The project cost for the reservoir expansion has been estimated at $350,000.

Monitoring

A monitoring program will need to be developed with input from the Region of Peel, CVC, Department of Fisheries (DFO) and the Ministry of the Environment (MOE).

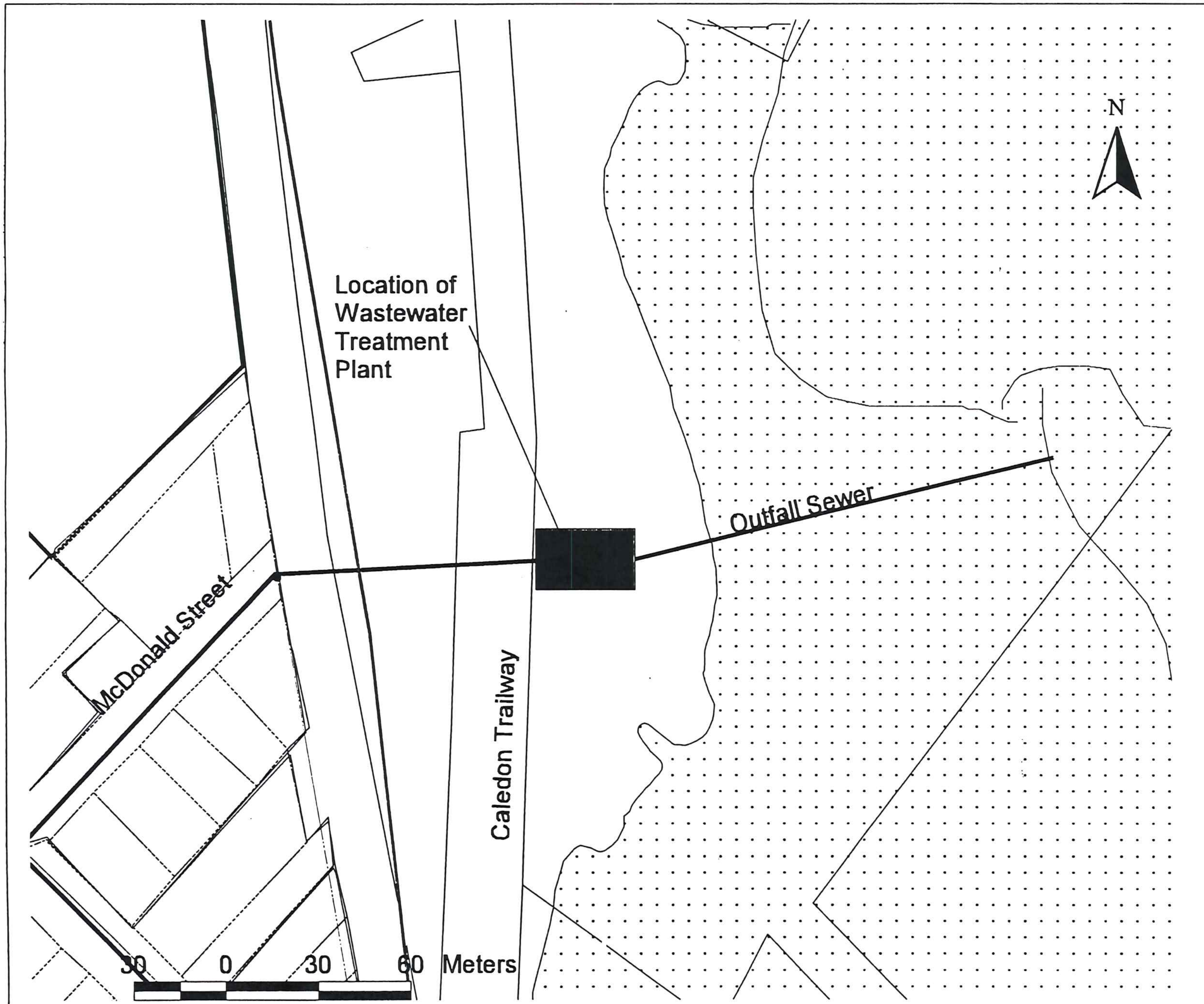
4.2 Wastewater Servicing Plan

Table 4.2 presents the description, implementation triggers and cost estimate for each component project for the Wastewater Servicing Plan. Cost estimates for each project include engineering and contingencies.

Table 4.2 Wastewater Servicing Plan

Project	Implementation Considerations	Project Cost
Wastewater treatment plant and outfall	Construction will be required for any new development to proceed, or to service existing Village Core.	$1,300,000
Sewer on McLaughlin Road from McColl Drive to MacDonald Street	Construction will be required to service Balmer Heights, Village Core lots on McLaughlin Road north of MacDonald Street.	$164,000
Sewer on MacDonald Street to treatment plant	Construction will be required to service Balmer Heights, McKenzie Property or any Village Core lots. A section of this sewer is required to service South Slopes.	$131,000
Sewer on McKenzie Street, McLaughlin Road running north from McKenzie Street to MacDonald Street and McLaughlin Road pumping station and forcemain	Construction will be required to service Village Core lots on MacKenzie, Louise, Victoria and Lorne Streets, lower end of McLaughlin Road and McKenzie Property.	$326,000
Sewers on Louise, Victoria and Lorne Streets	Construction will be required to service Louise, Victoria and Lorne Streets in Village Core.	$414,000
Total Cost		**$2,300,000**

Figure 4.2 presents the location of the wastewater treatment plant while Figure 4.3 presents the locations of the collection system projects. The following sections describe the projects.

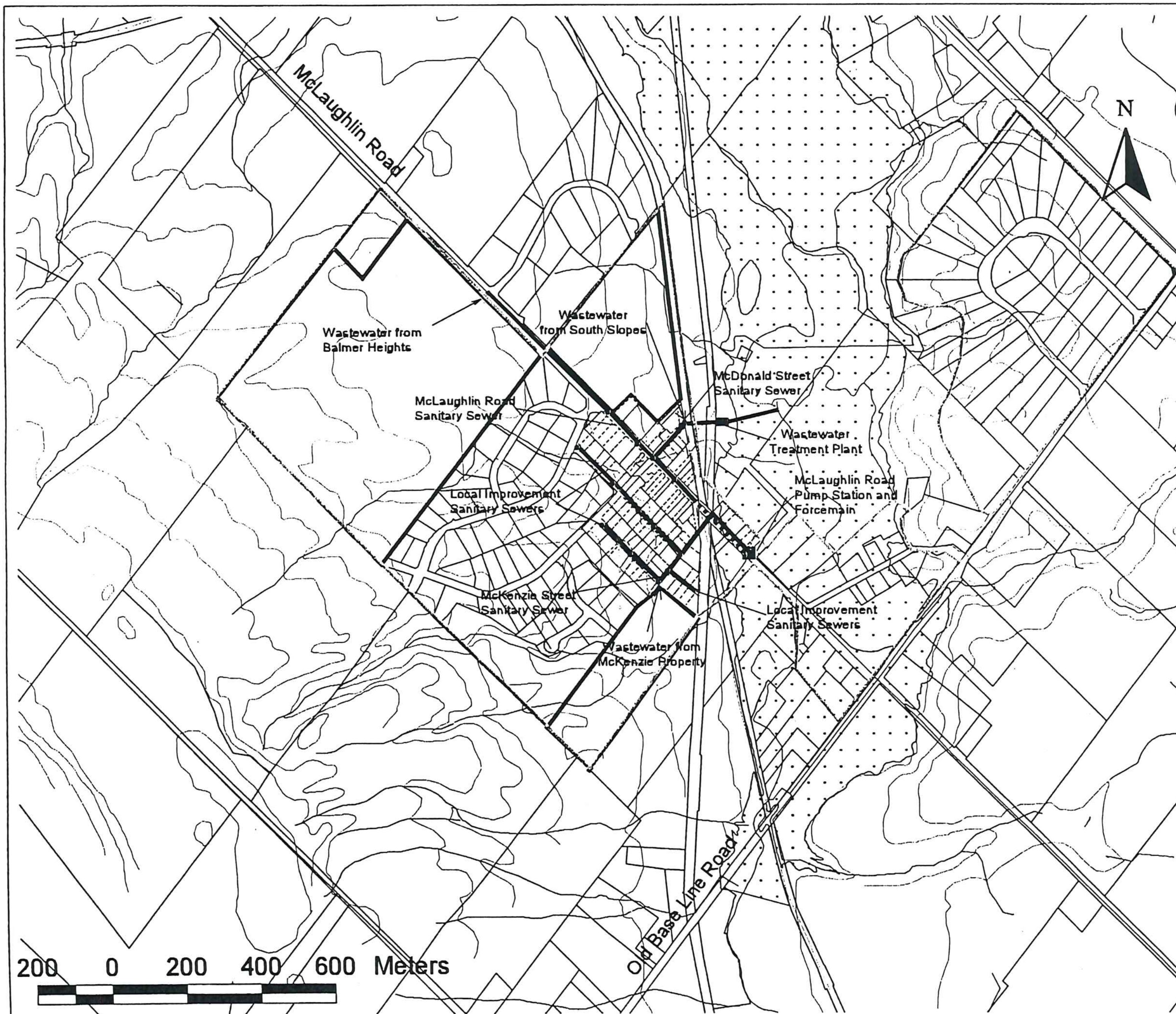


Inglewood Village Servicing Study

Legend
- Plant Location
- Undeveloped Land
- Sanitary Sewer
- Plant Outfall
- Manholes
- Existing Service Area
- Regional Floodplain
- Watercourses
- Contours

Figure 4.2
Wastewater Servicing Plan - WWTP Project

XCG



Inglewood Village Servicing Study

Legend

- Settlement Area Boundary
- Plant Location
- Undeveloped Land
- Sanitary Forcemain
- Sanitary Sewer
- Plant Outfall
- Manholes
- Pump Station
- Existing Service Area
- Regional Floodplain
- Watercourses
- Contours

Figure 4.3
Wastewater Servicing Plan - Collection System Projects

XCG

Wastewater Treatment Plant

A treatment plant, with a capacity of 390 m^3/d, will provide secondary biological treatment, nitrification, tertiary phosphorus removal, and ultraviolet disinfection. The facility can be constructed in one, two or three phases; however, the outfall and most of the facilities will be required in the first phase.

A location adjacent to the Caledon Trailway has been selected for the wastewater treatment plant.

The project also includes the construction of a plant outfall to the Credit River. The outfall will extend approximately 170 m to the Credit River.

The estimated capital cost for the treatment plant and outfall is $1.3 million.

McLaughlin Road and MacDonald Street Sewer

The McLaughlin Road Sewer will extend from McColl Drive to MacDonald Street. It will convey flows from the Balmer Heights area and from existing residences on McLaughlin Road. The MacDonald Street Sewer will extend from the corner of McLaughlin Road and MacDonald Street to the new wastewater treatment plant located adjacent to the Caledon Trailway. The sewer will convey flow from all three new development areas and the 95 lots within the existing Village Core.

This project will need to be completed to serve new development from Balmer Heights, South Slopes, and the McKenzie Property as well as to service the Village Core.

McKenzie Street and McLaughlin Road Sewers

The McKenzie Street Sewer will extend from Victoria Street to McLaughlin Road and will convey flow from the McKenzie Property and existing residences on Lorne, Victoria, Louise and McKenzie Streets.

There will be two sections of sanitary sewer on McLaughlin Road. One section will extend from McKenzie Street south to the McLaughlin Road Pump Station. The other section will extend from north of McKenzie Street to MacDonald Street. Both sections will convey flow from the McKenzie Property and existing residences on Lorne, Victoria, Louise, McKenzie Streets and McLaughlin Road.

Louise Street, Victoria Street, Lorne Street Sewers

These sewers will be required to service the existing Village Core. The Louise Street sewer will convey flows from 8 lots on Louise Street. The Victoria Street sewer will convey flows from 14 lots on Victoria Street. The Lorne Street sewer will convey flows from 23 lots on Lorne Street.

McLaughlin Road Pump Station and Forcemain

McLaughlin Street Pump Station and forcemain will pump flow from existing residents on McLaughlin Road, McKenzie Street, Lorne Street, Victoria Street and Louise Street as well as flows from new development within the McKenzie Property. This project will be required to service new development within the McKenzie Property, and most of the Village Core.

Monitoring

A monitoring program will need to be developed with input from The Region of Peel, CVC, Department of Fisheries (DFO) and the Ministry of the Environment (MOE).

4.3 Summary

Water and wastewater servicing plans were developed for the Village of Inglewood to provide for a maximum population of 1,500 persons in the year 2021. The Water Servicing Plan consists of four projects including the connection of the MacDonald Street Watermain with the Balmer Heights area, a new Permit to Take Water, the installation of a new pressure reducing valve at the intersection of West Village Drive and McLaughlin Road, and an expansion to the existing reservoir.

The Wastewater Servicing Plan consists of five projects including a new wastewater treatment plant, new sewers on McLaughlin Road, McDonald Street, McKenzie Street, Louise Street, Victoria Street and Lorne Street. These projects have been identified to provide wastewater servicing to a maximum population of 1,500 persons in the year 2021.

5. REFERENCES

1. CVC, Town of Caldeon, Region of peel, November 1998. Inglewood Village Study, Phase I Report.
2. CVC, Town of Caldeon, Region of Peel, March 1999. Inglewood Village Study Phase II Report, Planning and Servicing Alternatives.
3. KMK Consultants Limited, march 1997. Caledon Water Distribution Systems Interconnection Feasibility Study.
4. Municipal Engineers Association, June 1994. Class Environmental Assessments for Municipal Water and Wastewater Projects.
5. XCG Consultants Ltd., April 1999. Inglewood Village Septic Tank Survey.

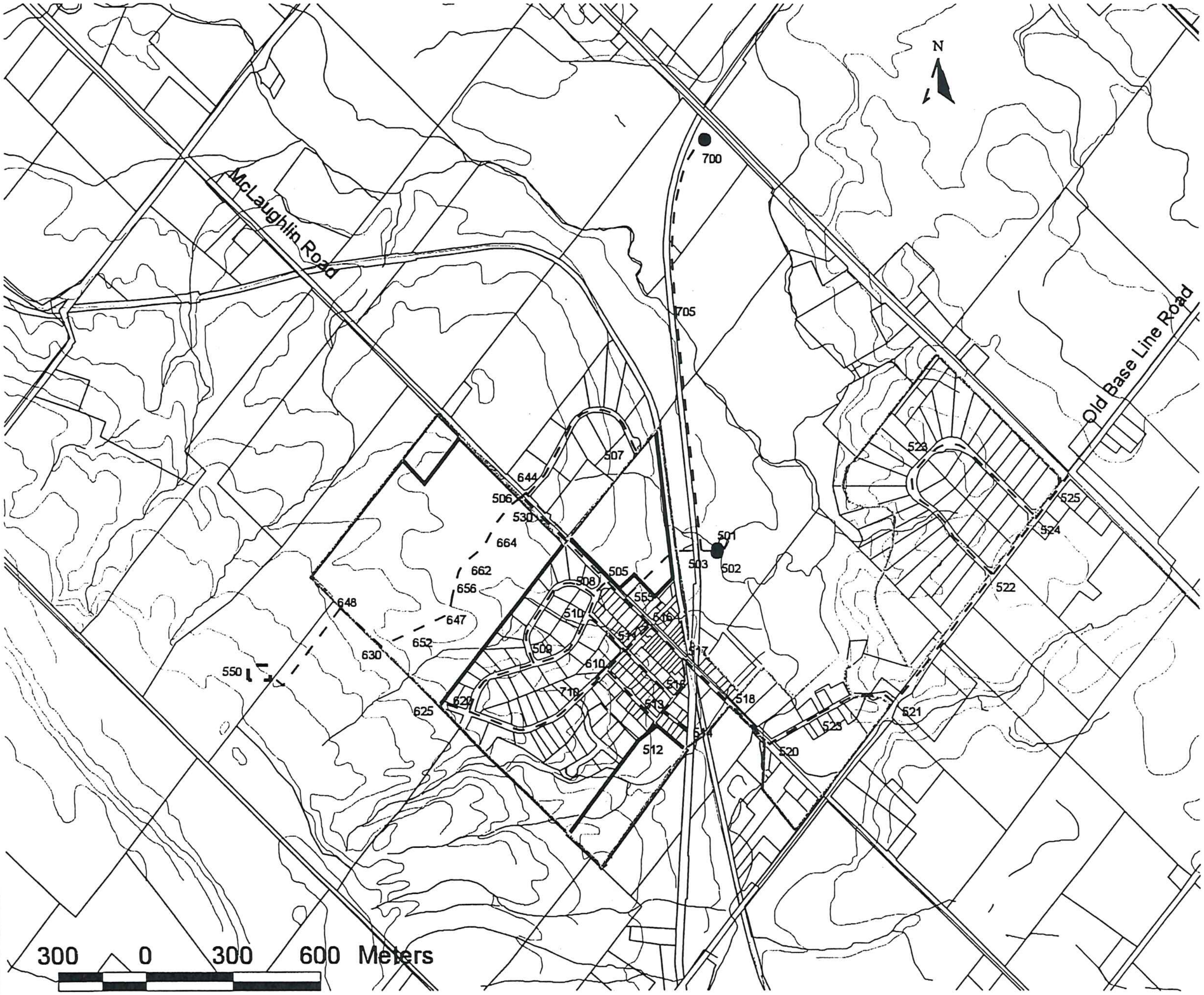
Appendix A
Water Distribution Modelling Results

Appendix A
Inglewood Village Servicing Study
Existing Water Distribution System Modelling Results

		Existing Maximum Day Results			Existing Maximum Day with new PRV			Existing Maximum Day with Fire Flow of 40L/s			Existing Maximum Day with new PRV and Fire Flow of 40L/s		
Node	Elevation m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m
700	273.00	0.00	829.12	357.65	0.00	829.12	357.65	0.00	579.84	332.20	0.00	579.84	332.20
705	273.92	0.00	820.11	357.65	0.00	820.11	357.65	0.00	570.84	332.20	0.00	570.84	332.20
501	273.92	0.00	820.11	357.65	0.00	820.11	357.65	0.00	570.84	332.20	0.00	570.84	332.20
503	275.00	0.00	809.53	357.65	0.00	809.53	357.65	0.00	560.26	332.20	0.00	560.26	332.20
555	287.00	0.00	692.02	357.65	0.00	692.02	357.65	0.00	442.75	332.20	0.00	442.75	332.20
504	287.00	0.50	493.55	337.38	0.50	493.40	337.37	0.50	97.55	296.95	0.50	97.55	296.95
516	280.00	0.50	561.08	337.28	0.50	560.73	337.24	0.50	139.50	294.23	0.50	139.50	294.23
517	277.00	1.08	589.95	337.23	1.08	589.56	337.19	1.08	154.96	292.81	1.08	154.96	292.81
518	270.00	0.27	658.41	337.22	0.27	658.02	337.18	0.27	223.43	292.80	0.27	223.43	292.80
512	282.50	0.50	536.16	337.23	0.50	535.76	337.19	40.50	72.39	289.88	40.50	72.39	289.88
513	280.00	0.00	560.65	337.23	0.00	560.25	337.19	0.00	107.17	290.93	0.00	107.17	290.93
514	276.00	0.36	599.81	337.23	0.36	599.41	337.19	0.36	146.34	290.93	0.36	146.34	290.93
515	278.00	0.28	580.24	337.24	0.28	579.84	337.19	0.28	137.21	292.00	0.28	137.21	292.00
511	288.00	0.77	482.63	337.27	0.77	482.22	337.23	0.77	56.69	293.78	0.77	56.69	293.78
610	289.50	1.85	467.66	337.24	1.85	467.25	337.20	1.85	29.57	292.51	1.85	29.57	292.51
510	295.00	0.59	414.85	337.35	0.59	414.28	337.29	0.59	14.37	296.45	0.59	14.37	296.45
508	294.50	0.63	420.21	337.39	0.63	419.56	337.33	0.63	29.67	297.52	0.63	29.67	297.52
505	292.00	0.00	445.50	337.48	0.00	445.50	337.48	0.00	68.50	298.98	0.00	68.50	298.98
509	311.00	0.68	258.22	337.35	0.68	257.63	337.29	0.68	-138.99	296.79	0.68	-138.99	296.79
530	306.00	0.00	324.70	339.14	0.00	324.70	339.14	0.00	249.52	331.46	0.00	249.52	331.46
506	306.00	0.05	505.96	357.65	0.05	505.96	357.65	0.05	256.68	332.20	0.05	256.68	332.20
630	322.80	0.00	348.26	358.35	0.00	348.26	358.35	0.00	214.73	344.71	0.00	214.73	344.71
644	306.00	0.00	505.95	357.65	0.00	505.95	357.65	0.00	256.68	332.20	0.00	256.68	332.20
507	287.00	0.54	691.90	357.64	0.54	691.90	357.64	0.54	442.63	332.18	0.54	442.63	332.18
550	357.30	0.00	18.29	359.15	0.00	18.29	359.15	0.00	18.29	359.15	0.00	18.29	359.15
520	269.00	0.41	668.08	337.21	0.41	667.70	337.17	0.41	233.10	292.79	0.41	233.10	292.79
521	282.00	0.00	540.62	337.19	0.00	540.24	337.15	0.00	105.64	292.77	0.00	105.64	292.77
522	302.00	0.14	344.60	337.17	0.14	344.22	337.13	0.14	-90.38	292.76	0.14	-90.38	292.76
523	306.00	0.10	305.34	337.16	0.10	304.96	337.12	0.10	-129.64	292.75	0.10	-129.64	292.75
524	305.00	0.95	315.08	337.16	0.95	314.69	337.12	0.95	-119.91	292.74	0.95	-119.91	292.74
525	305.00	0.00	315.05	337.16	0.00	314.67	337.12	0.00	-119.93	292.74	0.00	-119.93	292.74
526	305.00	0.45	315.04	337.15	0.45	314.66	337.12	0.45	-119.94	292.74	0.45	-119.94	292.74
664	305.84	0.00	507.53	357.65	0.00	507.53	357.65	0.00	258.42	332.21	0.00	258.42	332.21
662	307.00	0.00	497.13	357.75	0.00	497.13	357.75	0.00	264.26	333.97	0.00	264.26	333.97
656	301.70	0.00	550.31	357.88	0.00	550.31	357.88	0.00	339.11	336.31	0.00	339.11	336.31
652	315.90	0.00	414.66	358.23	0.00	414.66	358.23	0.00	261.19	342.56	0.00	261.19	342.56
648	331.00	0.00	271.21	358.68	0.00	271.21	358.68	0.00	192.63	350.65	0.00	192.63	350.65

Appendix A
Inglewood Village Servicing Study
Future Water Distribution System Modelling Results

		Future Maximum Day Results			Future Maximum Day Results with new PRV			Future Maximum Day Results with new PRV and Fire Flow of 40L/s		
Node	Elevation m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m
700	273.00	0.00	840.78	358.84	0.00	845.24	359.30	0.00	633.59	337.68
705	273.92	0.00	831.77	358.84	0.00	836.23	359.30	0.00	624.58	337.68
501	273.92	0.00	831.77	358.84	0.00	836.23	359.30	0.00	624.58	337.68
503	275.00	0.00	821.19	358.84	0.00	825.65	359.30	0.00	614.00	337.68
555	287.00	0.00	703.68	358.84	0.00	708.14	359.30	0.00	496.49	337.68
504	287.00	0.50	568.61	345.05	0.50	593.60	347.60	0.50	411.93	329.05
516	280.00	2.24	637.39	345.07	2.24	645.19	345.87	2.24	458.40	326.79
517	277.00	1.08	666.71	345.07	1.08	666.85	345.08	1.08	474.61	325.45
518	270.00	0.27	735.18	345.06	0.27	735.31	345.07	0.27	543.07	325.44
512	282.50	1.11	613.41	345.12	1.11	613.23	345.10	41.11	392.28	322.54
513	280.00	0.00	637.89	345.12	0.00	637.48	345.08	0.00	427.22	323.61
514	276.00	0.36	677.05	345.12	0.36	676.65	345.08	0.36	466.38	323.61
515	278.00	0.28	657.47	345.12	0.28	657.10	345.08	0.28	457.40	324.69
511	288.00	0.77	560.48	345.22	0.77	559.66	345.13	0.77	378.36	326.62
610	289.50	1.85	545.22	345.16	1.85	544.71	345.11	1.85	350.70	325.30
510	295.00	0.59	499.17	345.96	0.59	488.76	344.89	0.59	346.35	330.35
508	294.50	0.63	503.03	345.85	0.63	481.25	343.63	0.63	355.09	330.74
505	292.00	0.00	519.56	345.04	0.00	554.74	348.63	0.00	379.56	330.74
509	311.00	0.68	376.15	349.39	0.68	365.86	348.34	0.68	222.66	333.72
530	306.00	0.00	324.64	339.14	0.00	346.74	341.39	0.00	308.95	337.53
506	306.00	0.05	517.63	358.84	0.05	522.10	359.30	0.05	310.45	337.69
630	322.80	0.00	335.80	357.07	0.00	335.80	357.07	0.00	176.26	340.78
644	306.00	0.00	517.61	358.84	0.00	522.08	359.30	0.00	310.43	337.68
507	287.00	2.28	701.98	358.67	2.28	706.44	359.12	2.28	494.79	337.51
550	357.30	0.00	18.29	359.15	0.00	18.29	359.15	0.00	18.29	359.15
520	269.00	0.41	744.85	345.04	0.41	744.98	345.06	0.41	552.74	325.43
521	282.00	0.00	617.39	345.03	0.00	617.52	345.04	0.00	425.28	325.41
522	302.00	0.14	421.37	345.01	0.14	421.50	345.03	0.14	229.26	325.40
523	306.00	0.10	382.11	345.00	0.10	382.24	345.02	0.10	190.00	325.39
524	305.00	0.95	391.84	345.00	0.95	391.98	345.01	0.95	199.74	325.38
525	305.00	0.00	391.82	344.99	0.00	391.95	345.01	0.00	199.71	325.38
526	305.00	0.45	391.81	344.99	0.45	391.95	345.01	0.45	199.70	325.38
664	305.84	0.00	519.18	358.84	0.00	523.64	359.29	0.00	312.06	337.69
662	307.00	0.00	505.39	358.59	0.00	509.22	358.98	0.00	304.96	338.12
656	301.70	0.00	554.05	358.26	0.00	557.04	358.57	0.00	362.54	338.70
652	315.90	0.00	406.35	357.38	0.00	407.12	357.46	0.00	238.61	340.25
648	331.00	4.92	261.42	357.68	4.92	261.42	357.68	4.92	163.16	347.64
625	311	0	413.75	353.23	0	408.61	352.71	0	257.24	337.25



Inglewood Village Servicing Study

Legend

- Settlement Area Boundary
- Undeveloped Land
- Watercourses
- Existing Reservoir
- Existing Well
- Existing Watermain
- 525 Watermain Node

Figure A.1
Village of Inglewood
Existing Water
Distribution System

XCG

APPENDIX B
INFORMATION FOR WASTEWATER ALTERNATIVES

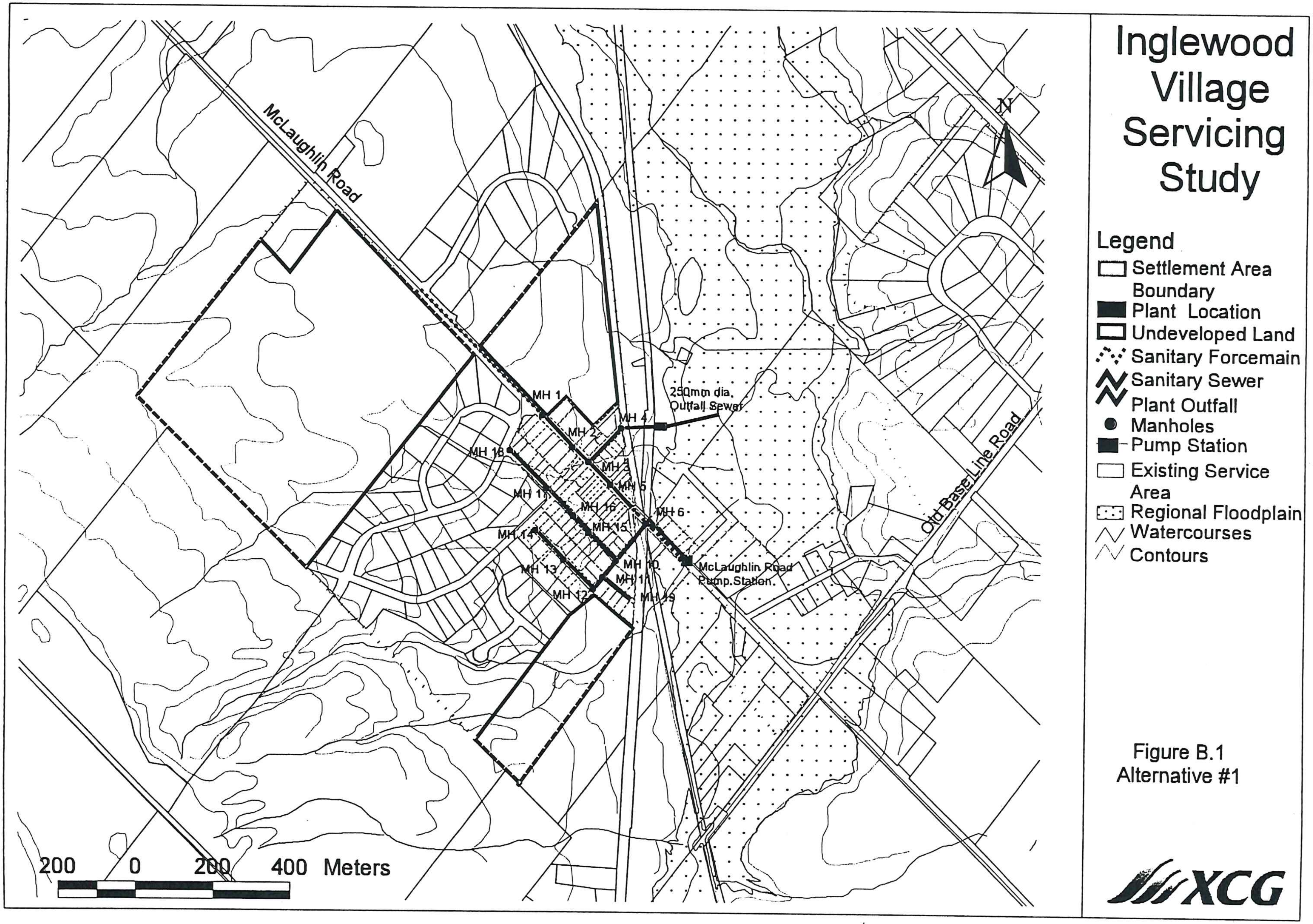


Figure B.1
Alternative #1

Alternative #1 - Plant located adjacent to Trailway, existing downtown conveyed by gravity sewer to McLaughlin Pump Station

Street	From Manhole	To Manhole	Population	umulativ Populatio	Area (ha)	umulativ Area	Peak Factor	Q avg (l/s)	Infiltration (l/s)	Qp (l/s)	Diameter (mm)	Upstream Ground Elevation	ownstream Ground Elevation	Upstrea Invert	Downstream Invert	Pipe Length	Slope (%)	Capacity (l/s)	Max Velocity (m/s)	Actual Velocity (m/s)
Louise	MH 19	MH 11	25	25	0.94	0.94	4.37	0.09	0.19	0.57	250	275.5	278	272.75	270.50	94	2.39	91.97	1.87	0.75
Victoria	MH 14	MH 13	22	22	1.35	1.35	4.37	0.08	0.27	0.61	250	287.9	285.4	285.15	282.65	86	2.91	101.36	2.07	0.83
	MH 13	MH 12	22	44	0.91	2.27	4.33	0.15	0.45	1.12	250	285.4	279	282.65	274.40	120	6.88	155.87	3.18	1.27
Lorne	MH 18	MH 17	19	19	0.88	0.88	4.38	0.07	0.18	0.47	250	290	284.2	287.25	281.45	120	4.83	130.70	2.66	1.07
	MH 17	MH 16	16	35	0.87	1.75	4.34	0.12	0.35	0.88	250	284.2	281.4	281.45	278.65	86	3.26	107.27	2.19	0.87
	MH 16	MH 15	19	54	0.55	2.30	4.31	0.19	0.46	1.27	250	281.4	280	278.65	276.50	86	2.50	94.00	1.92	0.77
	MH 15	MH 10	19	72	0.62	2.92	4.28	0.25	0.58	1.67	250	280	277	276.50	269.70	94	7.23	159.89	3.26	1.30
McKenzie	MH 12	MH 11	161	161	8.99	8.99	4.18	0.56	1.80	4.16	250	279	278	274.40	270.50	47	8.30	171.25	3.49	1.92
	MH 11	MH 10	32	193	1.28	10.27	4.15	0.67	2.05	4.86	250	278	277	270.50	269.70	60	1.33	68.65	1.40	0.77
	MH 10	MH 7	91	284	3.56	13.83	4.09	1.00	2.77	6.84	250	277	275.5	269.70	264.25	120	4.54	126.69	2.58	1.81
McLaughli	MH 7	PS	19	303	0.91	14.74	4.08	1.06	2.95	7.28	250	267	266.5	264.25	261.90	99	2.37	91.59	1.87	0.75
McLaughli	MH 6	MH 5	28	331	0.96	15.71	4.06	1.16	3.14	7.85	250	275.5	278.7	272.75	272.20	73	0.75	51.60	1.05	0.79
	MH 5	MH 3	25	356	0.94	16.65	4.05	1.25	3.33	8.38	250	278.7	278	272.20	271.68	86	0.60	46.23	0.94	0.75
	MH 1	MH 2	734	734	40.04	40.04	3.88	2.57	8.01	18.00	250	287	282.1	284.25	276.30	120	6.62	153.01	3.12	2.18
	MH 2	MH 3	16	750	0.68	40.72	3.88	2.63	8.14	18.33	250	282.1	278	276.30	271.68	56	8.25	170.75	3.48	2.44
McDonald	MH 3	MH 4	1128	1128	58.29	58.29	3.77	3.95	11.66	26.55	250	278	276	271.68	271.08	120	0.50	42.04	0.86	0.94
	MH 4	Plant	198	1326	9.24	67.53	3.72	4.65	13.51	30.78	250	276	272	271.08	270.45	120	0.52	43.07	0.88	0.79
	Total Length															1587				
	Plant Outfall		0	1326	0.00	67.53	3.72	4.65	13.51	30.78	250	272	266	268.23	266.00	65	3.43	110.11	2.24	2.02

Pump Stations	Qp (l/s)	Head Diff. (m)	Power (kW)
McLaughlin	7.3	10.85	0.77

Forcemain	Length (m)	Qp (l/s)	Dia. (mm)	vmax (m/s)
McLaughlin	200	6.3	100.00	0.80

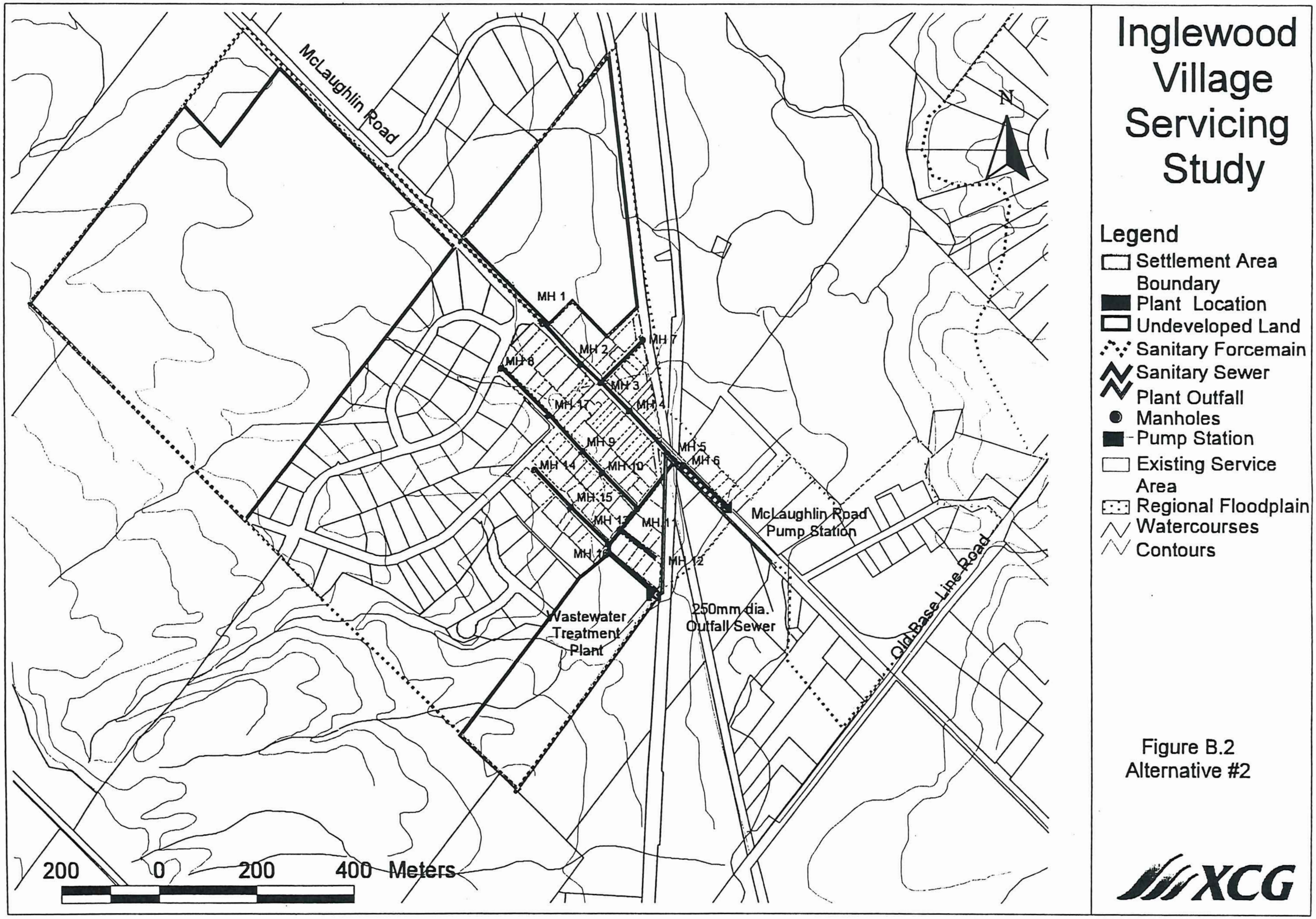


Figure B.2
Alternative #2

Alternative #2 - Plant located in Martin Development Area with pump station on McLaughlin

Street	From Manhole	To Manhole	Population	Cumulativ Population	Area (ha)	Cumulativ Area	Peak Factor	Q avg (l/s)	Infiltration (l/s)	Qp (l/s)	Diameter (mm)	Upstream Ground Elevation	Downstrea Ground Elevation	Proposed Design Upstream Invert	Downstream Invert	Pipe Length (m)	Slope (%)	Capacity (l/s)	Velocity (m/s)	Actual Velocity (m/s)
Victoria	MH 14	MH 15	25	25	1.49	1.49	4.37	0.09	0.30	0.68	250	286.7	283.0	283.9	278.0	120	4.92	131.82	2.69	1.07
	MH 15	MH 16	16	41	0.73	2.21	4.33	0.14	0.44	1.06	250	283.0	279.5	278.0	270.7	90	8.11	169.31	3.45	1.38
Louise	MH 12	MH 13	25	25	0.94	0.94	4.37	0.09	0.19	0.57	250	279.0	279.3	276.2	271.3	90	5.44	138.71	2.83	1.13
Lorne	MH 8	MH 17	13	13	0.54	0.54	4.40	0.04	0.11	0.30	250	291.1	288.8	288.3	284.0	81	5.31	136.97	2.79	1.12
	MH 17	MH 9	9	22	0.50	1.05	4.37	0.08	0.21	0.55	250	288.8	280.0	284.0	277.2	86	7.91	167.16	3.41	1.36
	MH 9	MH 10	25	47	1.10	2.15	4.32	0.17	0.43	1.14	250	280.0	279.6	277.2	275.9	107	1.26	66.78	1.36	0.75
	MH 10	MH 11	22	69	0.77	2.92	4.28	0.24	0.58	1.62	250	279.6	279.1	275.9	271.6	120	3.54	111.88	2.28	1.25
McDonald	MH 7	MH 3	22	22	0.81	0.81	4.37	0.08	0.16	0.50	250	277.0	279.4	274.2	272.8	112	1.25	66.47	1.35	0.75
McLaughlin	MH 1	MH 2	665	665	48.94	48.94	3.91	2.33	9.79	18.89	250	288.3	283.3	285.5	279.5	120	5.00	132.93	2.71	1.08
	MH 2	MH 3	19	684	1.08	50.02	3.90	2.40	10.00	19.35	250	283.3	279.4	279.9	272.8	86	8.26	170.81	3.48	1.39
	MH 3	MH 4	63	747	1.41	52.24	3.88	2.62	10.45	20.59	250	279.4	278.4	272.8	272.3	120	0.42	38.37	0.78	0.78
	MH 4	MH 5	19	765	0.81	53.05	3.87	2.68	10.61	21.00	250	278.4	277.6	272.3	271.9	94	0.38	36.79	0.75	0.75
	MH 6	Pump	19	19	0.88	0.88	4.38	0.07	0.18	0.47	250	277.6	276.3	274.8	271.9	120	2.42	92.42	1.88	0.75
McKenzie	MH 5	MH 11	16	800	0.63	54.56	3.86	2.80	10.91	21.74	250	277.6	279.1	271.9	271.6	43	0.70	49.66	1.01	1.01
	MH 11	MH 13	9	810	0.46	57.94	3.86	2.84	11.59	22.53	250	279.1	279.3	271.6	271.3	56	0.54	43.51	0.89	0.89
	MH 13	MH 16	25	835	0.00	58.88	3.85	2.93	11.78	23.04	250	279.3	279.5	271.3	270.7	120	0.50	42.04	0.86	0.86
	MH 16	plant	88	923	6.72	67.81	3.82	3.23	13.56	25.93	250	279.5	280.0	270.7	270.5	43	0.47	40.54	0.83	0.83
	Total Length (m) =															1608				
	Outfall		0	923	6.72	67.81	3.82	3.23	13.56	25.93	250	280.0	270.0	270.5	267.2	595	0.55	44.27	0.90	0.90

Pump Stations	Qp (l/s)	Head Diff. (m)	Power (kW)
McLaughlin	0	10.3	0.05

Assumptions: Water Level in plant is at 280.5 (.5m above ground elevation of 280).

Forcemain	Length (m)	Qp (l/s)	Dia. (mm)	vmax (m/s)
McLaughlin	355	0.47	100.00	0.06

DRAFT REPORT

Inglewood Village Environmental Management Plan

June 1999

Prepared By:

Credit Valley Conservation
Parish Geomorphic
Don Weatherbe and Associates

Prepared For:

Town of Caledon
Credit Valley Conservation
Region of Peel

CVC

TOWN OF CALEDON

Region of Peel

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INTRODUCTION

1.1 Background

1.1.1 Inglewood Village Study

The Village of Inglewood is one of six medium sized communities located in the Town of Caledon. It is located on the main branch of the Credit River within Subwatershed 20. Figure 1.1 shows the location of the Village with respect to the Credit River Watershed. The Village is currently serviced by a municipal well and private septic systems. The 1996 Provincial Policy Statements (PPS) requires the protection/enhancement of the quality and quantity of groundwater and surface water resources and the function of sensitive recharge/discharge areas, headwaters and aquifers. The PPS promotes the use of communal services where full services are not available. This is supported by the Region of Peel's Official Plan. In addition, the Town of Caledon supports an ecosystem approach to planning with policies contained in OPA 124.

On March 27, 1997, the Region of Peel Council approved a "Water Protection Strategy and the Guidelines for the Provision of Communal Sewage Disposal Systems" (CSDS). This Water Protection Strategy sets out a community-based process known as a Servicing and Settlement Master Plan (SSMP) which is designed to address servicing, planning and environmental issues relating to all villages in a comprehensive manner. Given limited funds, SSMPs will be initiated for each village on a priority basis. It was then agreed by the Town and Region that Inglewood Village would be given the highest priority. The SSMP entitled the "Inglewood Village Study" was approved and detailed terms of reference were developed. The study was further divided into three components, Planning, Servicing and Environmental, and the Town of Caledon, Region of Peel and Credit Valley Conservation (CVC), respectively, have been designated to take the lead. The overall study is being managed by a Core Management Team (CMT) made up of representatives from the Town, Region and CVC. The Town of Caledon chairs the CMT. The overall management of the study, including coordination of the public consultation process and integration of the study components will be the responsibility of Town planning staff.

The issues related to Inglewood include: community form and character; rate of growth; compatibility and staging of new development; densities and population limits; assimilative capacity of the Credit River and surrounding Provincially Significant Wetlands; sustainable management of groundwater; viability of the commercial core; and, relevancy of work completed through the Inglewood Village Study. The overall study has been divided into four phases. Figure 1.2 shows the progression of the study and key points.

1.1.2 Environmental Component

The Environmental Component endeavors to develop and establish the key functions and linkages of the ecosystem within the study area. To do this, the approach taken was to break this component down into smaller sub-components, namely : hydrogeology, hydrology, fluvial geomorphology, terrestrial and fisheries. In addition, this component also includes an assimilative capacity analysis of the Credit River within the study area. The intent is to provide the Planning and Servicing Components of the

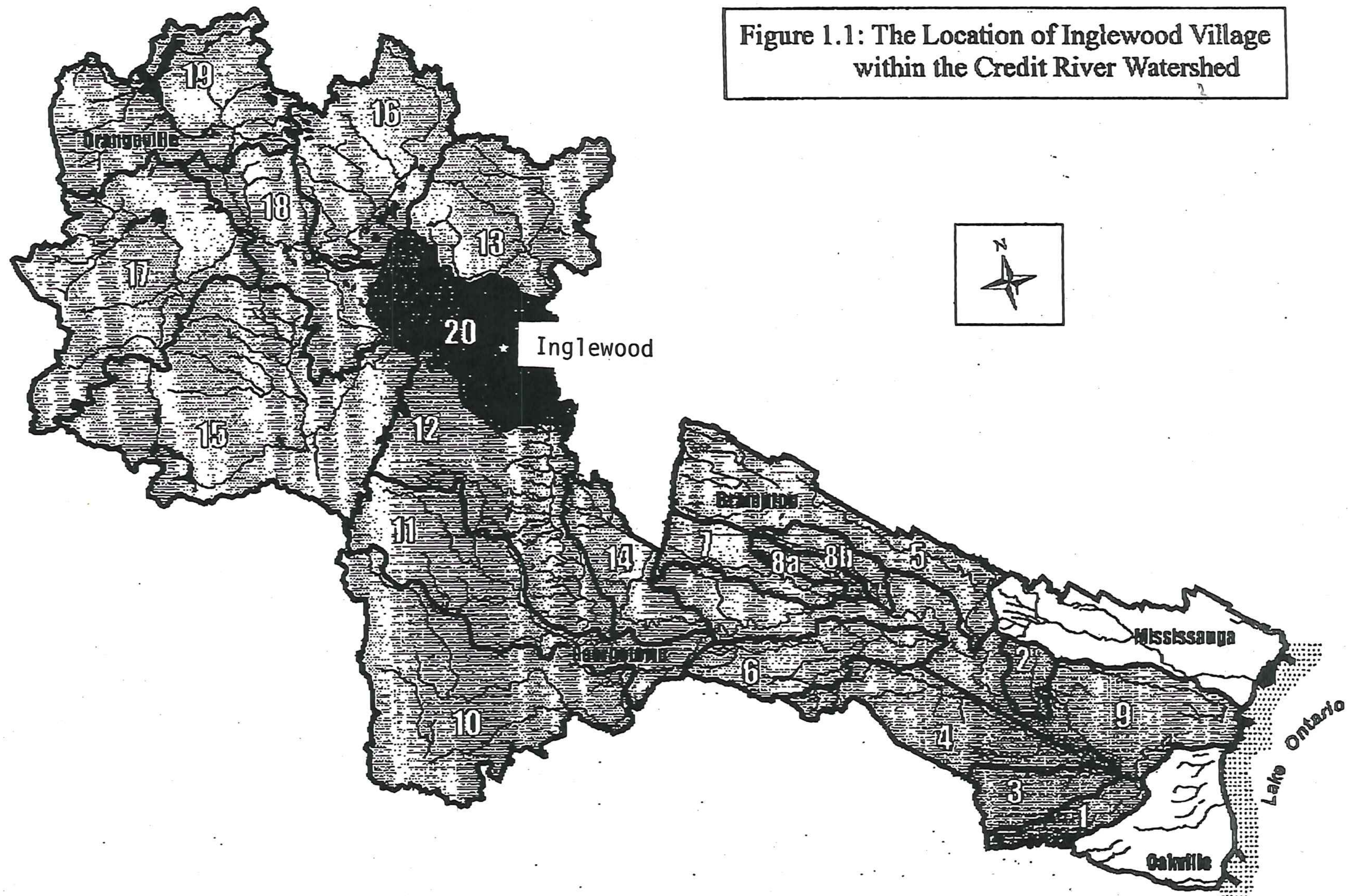


Figure 1.1: The Location of Inglewood Village within the Credit River Watershed

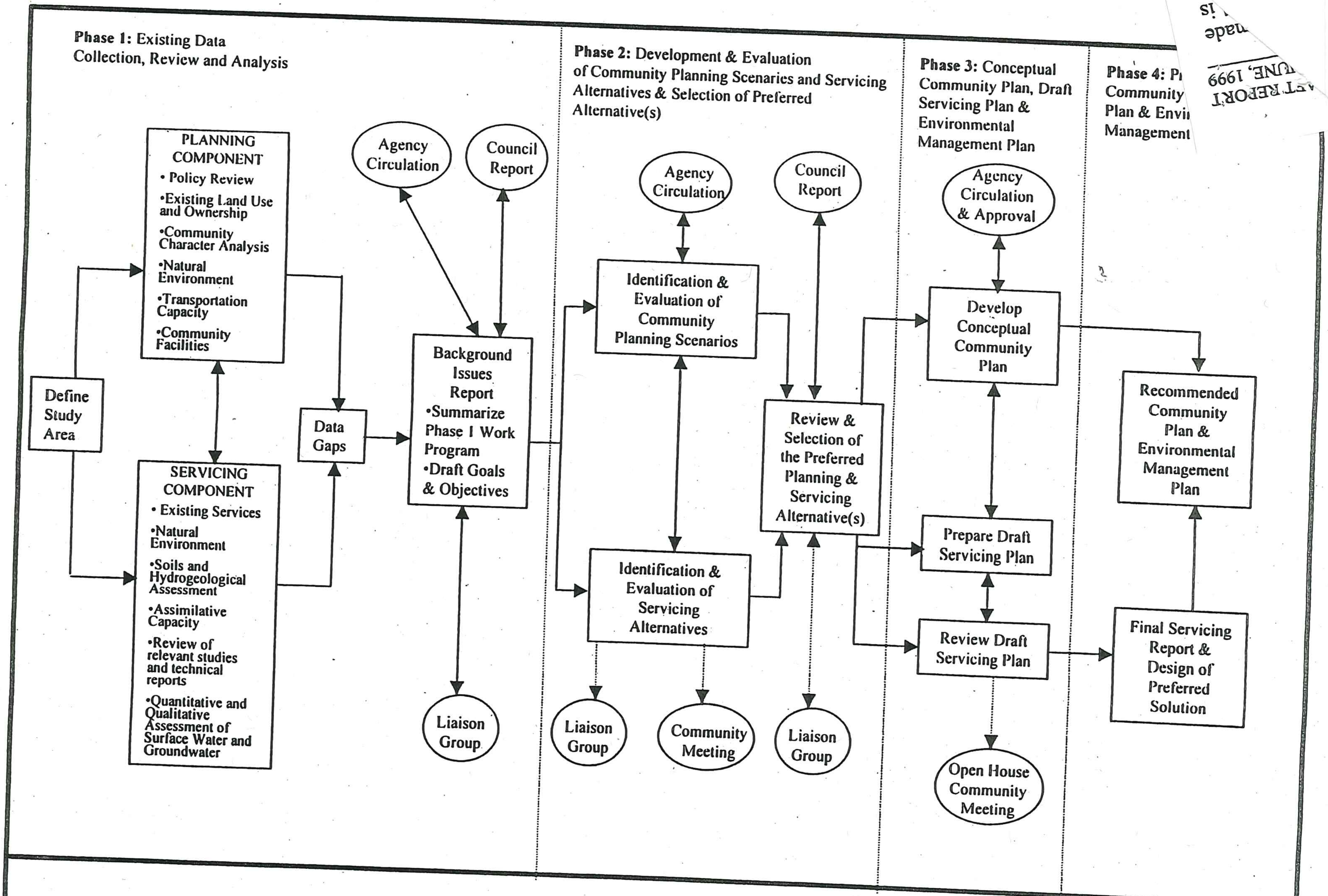


Figure 1.2
Inglewood Village Study Process

study with an understanding of the environmental constraints of the area so that decisions can be ı. while maintaining the integrity of the ecosystem. This work was carried out in Phases I and II ana available under separate cover.

The following objectives were defined for the Environmental Management Plan:

1. To adhere to the Ecosystem Planning and Management Objectives contained in the Caledon Official Plan.

2. To assess the assimilative capacity of the Credit River and ensure that development and servicing respect this assimilative capacity.

3. To ensure that the new water services do not negatively impact groundwater and surface water resources and related biological communities.

4. To identify measures to enhance the local environment.

5. To promote the awareness and stewardship of the local natural environment and to seek opportunities for residents to become involved in stewardship activities.

6. To manage the hydrologic cycle from the pre to post development perspective. This includes maintaining a water balance in terms of water takings and wastewater inputs.

7. To ensure that pre and post development infiltration conditions at the site plan level are maintained or enhanced based on the generally defined areas of high, moderate and low recharge potential.

8. To ensure that post development erosion rates do not exceed the erosive velocities that can cause the bed and banks of all watercourses to erode.

9. To maintain a healthy Dissolved Oxygen regime for fisheries and macroinvertebrates

10. To minimize the addition of other potentially deleterious substances including nitrates, ammonia, metals, organic and oxygen demanding matter

11. To allow no increase in bacteria (E. Coli) levels in the Credit River for human health concerns.

12. To not degrade surface water to produce objectionable colour, odour or turbidity in terms of aesthetics

Figure 1.3 shows the areas that are considered sensitive to change as a result of the analyses carried out in Phases I through III of the study process.

1.2 Intent of Report

This report sets out the components of the Environmental Management Plan (EMP). This Plan has been developed with the data and information collected and developed in the preceding phases of this study and is specific to the preferred planning and servicing (water supply and wastewater) scenarios

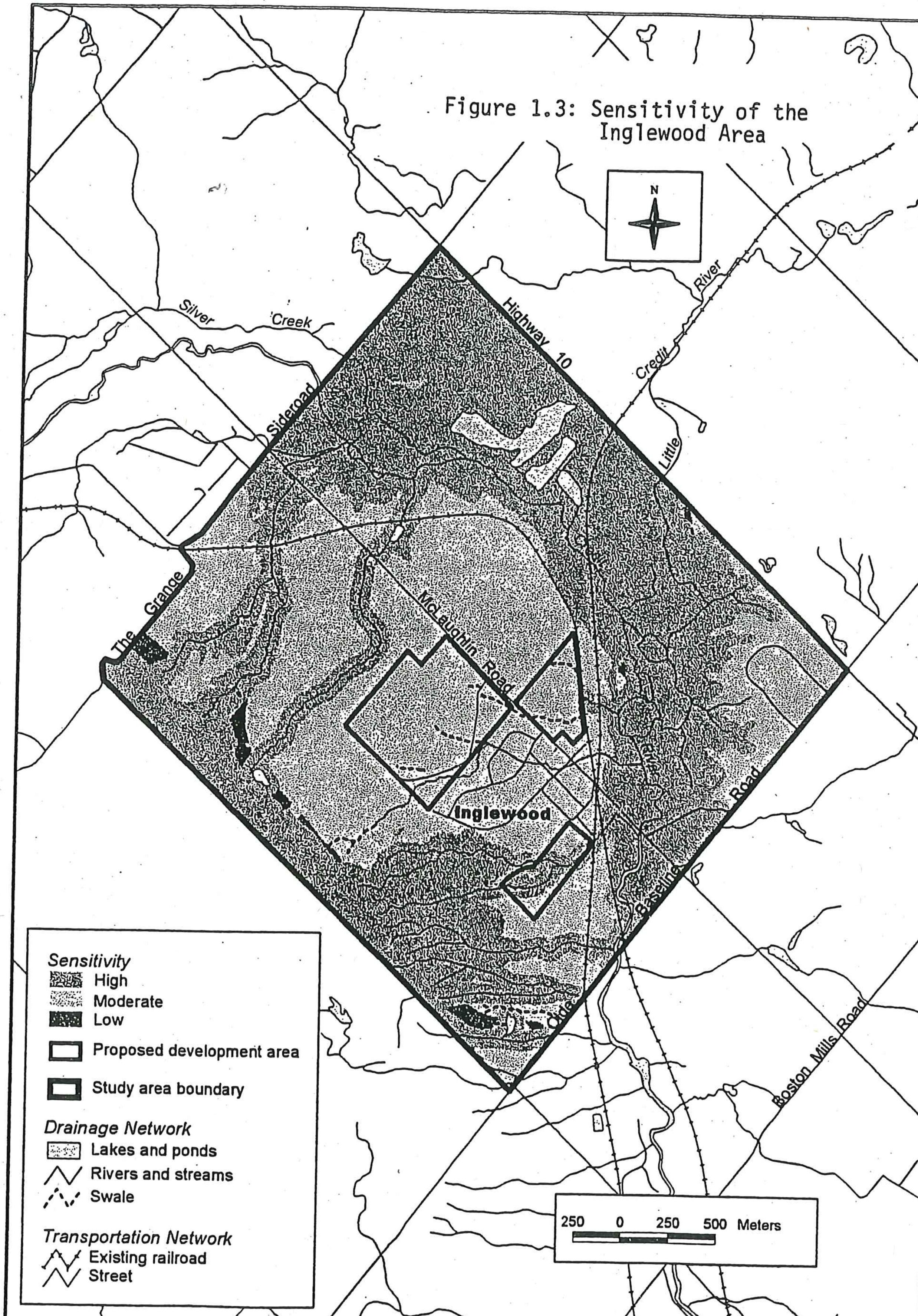


Figure 1.3: Sensitivity of the Inglewood Area
N
Silver Creek
Highway 10
Credit River
Little
Sideroad
The Grange
McLaughlin Road
Inglewood
Baseline Road
Olde
Boston Mills Road
Sensitivity
High
Moderate
Low
Proposed development area
Study area boundary
Drainage Network
Lakes and ponds
Rivers and streams
Swale
Transportation Network
Existing railroad
Street
250 0 250 500 Meters

described in the Inglewood Village Study Phase II Report (Town of Caledon, 1999). The work carried out in Phase III of the study concluded that the preferred planning scenario was the Mixed Village Density. Water supply will be provided by increasing the capacity of Well #3. The preferred method of wastewater servicing will be a tertiary wastewater treatment plant with a continuous discharge to the Credit River.

This report is a companion document to the Inglewood Village Plan, the Inglewood Village Water and Wastewater Servicing Plan and the Inglewood Village Tributary Study.

1.3 Report Outline

The EMP is laid out in the following manner:

Chapter 2 - Environmental Resources and Constraints - identifies those aspects of the preferred scenarios that help to preserve and enhance key environmental features and functions. A synopsis of the stormwater management report is also included from a study entitled "Tributary Study – Village of Inglewood" which is being completed under separate cover for the Town of Caledon .

Chapter 3 - Future Studies - describes the need for site-specific and monitoring approaches prior to and after development occurs.

Chapter 4 – Implementation - is set out under the four major categories of planning and policy, rehabilitation and retrofit, stewardship and education, and monitoring and reporting.

Chapter 5 – Recommendations - sets out a number of recommendations for future consideration.

2.0 ENVIRONMENTAL RESOURCES AND CONSTRAINTS

The following, by environmental component, identifies those aspects of the companion documents that help to preserve and enhance key environmental features and functions.

2.1 Inglewood Village Plan

Hydrogeology – The land use designations shown on proposed Schedule M - Inglewood Village and Area Land Use Plan (included as Appendix A to this report) are appropriate in that the New Residential Neighbourhoods are situated within the areas identified as being low to moderate recharge potential. The main areas of high groundwater recharge are located outside of the New Residential Neighbourhoods. The Open Space Policy Area (OSPA) designation allows for the implementation of measures to ensure the maintenance of pre and post development infiltration conditions.

Hydrology - The Village Plan keeps all new development outside of the Regional Storm floodplain and this is acceptable from a flooding perspective. No further analysis is required for this component.

Terrestrial - All sensitive terrestrial features have been designated Environmental Policy Area (EPA) on Schedule M. The OSPA designation protects localized drainage features and landforms and allows for enhancement of the terrestrial system and compatible recreation, in keeping with the Town's Environmental Objectives. The Village Plan provides flexibility from an urban design perspective, which allows for a variety of design approaches that can maximize the protection and enhancement of the terrestrial systems within the Village.

Fluvial Geomorphology - Detailed field assessments and analysis were carried out on the tributaries that will be preserved and enhanced through implementation of the Village Plan. Details on these analyses can be found in the Section 2.2 as well as the Inglewood Village Tributary Study completed by Marshall Macklin Monaghan Limited. These analyses conclude that the tributaries can be maintained in a stable form with appropriate storm water management controls and revegetation schemes.

Fisheries - The Village Plan allows for the preservation of those tributaries which do contribute to downstream fisheries. The treatment of these tributaries should allow for stable, fully functional systems and have accounted for water and sediment transport, setbacks from development, and stormwater management techniques that allow for the treatment of stormwater runoff for quantity, quality and bank erosion.

2.2 Inglewood Village Water and Wastewater Plan

2.2.2 Wastewater

Hydrogeology - The following section outlines and describes potential locations available for the preferred surface discharge component of the treated sewage effluent to various watercourses within the study area and adjacent lands.

Surface Discharge to the Escarpment Tributaries - The Escarpment Tributaries originate off the Niagara Escarpment in the south and southwest portion of the study area. Surface runoff across the lower permeable till plains and exposed shale bedrock contribute to many first order streams which have been observed to flow during the spring snowmelt period and are generally dry during the summer. A surface discharge to these watercourses would raise concerns regarding erosion and sediment transport, and water quality, specifically nutrient and chemical concentrations.

Surface Discharge to main Credit River - The Little Credit River originating east of the study area, and Black and Silver Creeks to the north of the study area are significant areas of groundwater discharge which contribute to the maintenance of baseflow conditions within the study area. Streamflow within this section of the Credit River exhibits significant groundwater discharge contributions which provide an opportunity for enhanced assimilation of wastewater and is viewed as a preferred location for surface discharge. There is a need to consider the quality of the treated effluent prior to discharge.

Hydrology - Given the very small discharge rate proposed from the effluent pipe, no impact on the overall hydrology of the Credit River is anticipated.

Terrestrial - The preferred location for the treatment plant is outside of any sensitive terrestrial features. The discharge pipe location as proposed will result in minimal disturbance of natural communities. However further site specific investigations will be required. The surface discharge directly to the Credit River is preferred, as it will prevent potential broad scale impacts on the terrestrial system. In addition, through further detailed analysis, an exact outfall location can be chosen which minimizes disturbance and direct impacts on the wetlands and ESA, and specific mitigation and restoration requirements can be determined.

Geomorphology - The proposed location, near the Credit River, is situated within an area that is moderately sensitive to changes (increased volume) in flow. In addition, the river banks in this area are experiencing moderate erosion, which is largely due to a lack of vegetation. The proposed facility is situated beyond the meander belt width of the Credit River, however, immediately downstream of the proposed outfall, the river is currently close to 'cutting-off' a meander bend. This process is natural and part of the river's evolution. The meander bend has become exaggerated, due to the local geology and riparian vegetation, as well as the effects of the confluence with the western side channel. River flows have eroded the inside bank at both the upstream and downstream sections, thereby pinching off the bend. Eventually, the flows will cut through the inside banks, effectively abandoning the meander bend, resulting in an 'ox-bow lake'. The concern with this process with respect to the proposed wastewater facility is the uncertainty of the eventual channel form and configuration after the meander cut-off. Once the cut-off occurs, there will likely be a substantial sediment deposit near the outfall, as part of the ox-bow lake formation. The new channel form would likely have higher flow velocities along the western bank, resulting in greater erosion potential.

Water Quality - A background review of water quality in the Credit River near Inglewood showed no significant impairments in water quality other than elevated levels of total phosphorus (TP) and bacteria. Using current disinfection techniques, bacteria can be reduced to meet Provincial Water Quality Objectives (PWQO) from the discharge of the facility thereby leading to no increase in bacteria levels to the Credit River.

It was determined that the Credit River near Inglewood is a Policy 2 river with respect to total phosphorus, which indicates that TP levels in this reach exceed the Provincial Water Quality Objectives at least 25 percent of the time. The intent of a Policy 2 designation is to disallow any further loading of the elevated parameter (in this case, TP).

Based on the extensive analysis and modeling completed for Phase II of the Inglewood study, no negative water quality impacts to the Credit River from the proposed communal sewage system for the preferred scenario are expected. The proposed treatment system is designed to produce a high quality of effluent, especially in terms of phosphorus removal.

In fact, the communal system provides an opportunity for improvement in the water quality of the Credit River. Many of the septic systems in the existing village core are inadequately sized or maintained so that they are failing or have a high potential for failure. Hooking these systems up to the communal system and better stormwater management practices are expected to produce a net water quality improvement for the Credit River in the Inglewood area.

Fisheries - The option of using a surface water discharge pipe from the treatment facility is preferred as it avoids groundwater impacts in an area that is sensitive. The location of the outfall is supported given that it is not located near any spawning activity.

2.2.3 Water Supply

Groundwater - The Region will need to assess its current and projected water supply needs for the preferred population numbers. The Region will also need to consider the impact of additional piping and the expansion of the reservoir on areas identified as constraint areas. The Region's Inglewood Well #3 has been identified as the main production well and is permitted for a maximum amount of 1200 Litres per minute (Lpm). Inglewood Wells #1 and #2 will become secondary supplies at rates of 450 Lpm and 900 Lpm, respectively. The Permit was issued in 1996 with a special condition to submit an interpretive report based on five years of monitoring. The report is to evaluate the effect of the water taking on the groundwater and surface water resources in the area and it is to provide recommendations for changes to the monitoring program.

Prior to the amendment of the existing Permit To Take Water (PTTW) for an increased rate from Well #3, the Region will need to assess long-term impacts on the natural areas including the Little Credit River, main Credit River and adjacent provincially significant wetland areas. Based on significant groundwater discharge contribution to the Little Credit and down-gradient main Credit River, reductions in baseflow in the vicinity of Well #3 could contribute to reductions in the assimilative capacity of the Credit River downstream. The protocols for this program will be finalized in discussions between CVC, Region of Peel, MOE and the Department of Fisheries and Oceans (DFO).

Terrestrial - The preferred alternative (increased use of Well #3) would not appear to be a cause for concern related to the Credit River and the Provincially Significant Little Credit River Wetland Complex to the south, however a monitoring program is required (see below).

Fisheries - Under the Federal Fisheries Act, fisheries habitat protection is required. An appropriate monitoring program will be required to ascertain that an increase in water taking does not impact on the natural areas in the vicinity of Well #3 in the longterm. This will ensure that any adjustments in pumping can be made if needed.

2.3 Inglewood Village Tributary Study

The following excerpts were taken from a report entitled "Inglewood Village Tributary Study" dated May 1999, completed by Marshall Macklin Monaghan Limited. The purpose of the study is to address water quality, erosion and flood control with respect to the post developed condition and outline the preferred methods of providing stormwater servicing and control on the lands that are developed or proposed for development. This report is available under separate cover. Therefore only brief summaries of pertinent findings are incorporated here.

2.3.1 Hydrologic Modelling

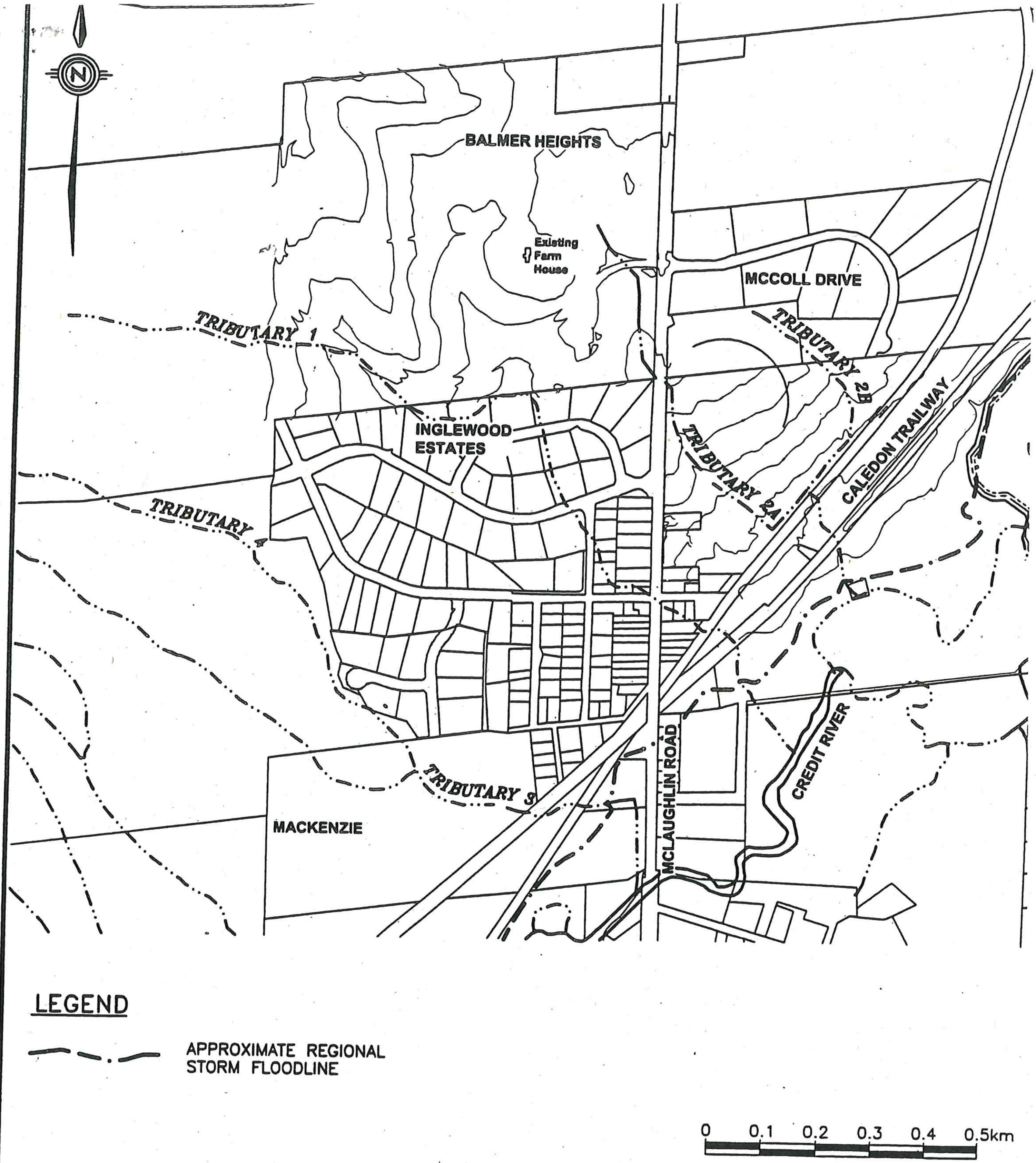
A hydrologic model was developed to determine the pre-development flow rates through the drainage system under existing conditions. These flow rates allow an assessment of how well the existing drainage system is functioning and indicate a minimum set of flow targets which need to be achieved under post-development conditions.

The three main tributaries of the Credit River through the settlement area were modelled. Tributary 1 drains the majority of the proposed Balmer Heights development area, and flows through the historic Village through a combination of swales, culverts and sewers. The tributary empties into the Credit River just north of the Inglewood Arena. Tributary 2 drains the eastern portion of the proposed Balmer Heights development area, and discharges to the Credit River via a swale through the proposed South Slopes development area. Tributary 3 is located south of the Historic Village, discharging to the Credit River where it crosses McLaughlin Road. Upstream of the CNR trail the drainage course splits, and the north branch is referred to as Tributary 4 (figure 2.1).

An additional two models were constructed to test water flows in post-development conditions: one representing the lower limit of the development densities (22% imperviousness), and the other representing the upper development density limit (28% imperviousness). The post-development uncontrolled models were run with the 25 mm, 4 hour rainfall event as well as through 100 year 24 hour SCS design storms.

2.3.2 Findings

Results of Modelling - On the Balmer Heights property on Tributary 1 the critical discharge rate is 0.07 m^3/s, which is considerably less than the 2 year peak flow rate of 0.44 m^3/s, and less than the peak flow of 0.19 m^3/s resulting from the 25 mm rainfall event. On Tributary 2 east of McLaughlin Road, and on the Mackenzie development west of McLaughlin Road, the critical discharge rate falls between the 2 year and 5 year peak flow rates. The critical discharge rate on Tributary 3 upstream of the Credit River in the ditch along McLaughlin Road is also relatively small, corresponding to the peak flow from a 25 mm rainfall event.



CLIENT

TOWN OF CALEDON

TITLE

Figure 2.1

CREDIT RIVER SITE TRIBUTARIES

Marshall Macklin Monaghan
Consulting Engineers . Surveyors . Planners
80 Commerce Valley Dr. E. Thornhill, Ont. L3T 7N4
Tele (905)882-1100 Fax (905)882-0055

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Based on the above, channel stability is a concern even under existing conditions. In the Credit River watershed post-development peak flow rates are typically controlled to pre-development levels and extended release of the runoff from a 25 mm rainfall event over 24 hours is used to define erosion control storage, unless a Subwatershed Plan is available that provides more specific criteria. However, for the proposed developments in the Village of Inglewood, adopting the above criteria will not alleviate erosion potential in the subject tributaries. It may be necessary to adopt more stringent controls, such as increasing the erosion control storage volumes and/or releasing it over a longer duration, in order to prevent further degradation of the drainage channels.

As may be expected, the post development model of uncontrolled conditions indicates that the proposed developments, if uncontrolled, will increase peak flow rates in all downstream channels. Increases are not as great as may be expected because of the soil conditions and the relatively low imperviousness of the proposed developments. Under major flood-type storms Tributary 1 would experience increases in the 15 to 20 % range, while the lower development potential on Tributary 3 would produce only about a 10% increase in flow. Tributary 2 would experience the largest flow increases, in the order of 50 to 100% (on both sub-branches).

Similar erosion concerns exist on both Tributary 2 and Tributary 3. On Tributary 2 the substantial flow increases under the smaller storms indicates the potential for problems (due to an increase in frequency) even though peak flows for these events are less than the critical flow threshold. On Tributary 3, the concern is that the lower reaches of the tributary have been noted as being highly susceptible to erosion.

In Tributary 3, the development area is less than 10 % of the total area. Concerns with erosion in the downstream reaches are best dealt with through geomorphological improvements.

Hydrology and Geomorphology - Detailed studies of hydrology and geomorphology were undertaken in the Marshall Macklin Monaghan study. The general findings are as follows:

The most significant aspects of hydrology within the study are locally oriented (e.g. erosion or flooding on the tributaries themselves). The relatively small drainage areas associated with the tributaries limit their potential significance to the Credit River in terms of flow magnitude.

Excessive erosion was not found in the downstream reaches of the tributaries in the Inglewood urban growth area, the area the study focussed on, but several reaches were classified as highly or moderately sensitive. The lower portion of Tributary 3 (south of the Village and West of McLaughlin Road) and the Credit River immediately downstream of McLaughlin Road were classified as highly sensitive to changes in flow regime.

2.3.3 Discussion and Recommendations

Tributary 1 - A significant drainage problem exists on Tributary 1 as a result of the capacities of the 900 mm and 1050 mm Corrugated Steel Pipes (CSPs) along MacDonald Street and under McLaughlin Road. This problem has existed for some time. EMC Group prepared a design in 1995 for the reconstruction of Lorne Street from West Village Drive to MacDonald Street and MacDonald Street from Lorne Street to the CPR tracks, including storm sewer improvements. The system has not been constructed, but a 600 mm sewer along Macdonald Street, between Victoria and Lorne Street has been. At present, the flow from these catchments enters a 300 mm CSP and is routed through backyard swales to the corner of McKenzie and McLaughlin Road.

A possible alternative to the proposed sewer construction would involve over-controlling flows from the proposed Balmer Heights subdivision The best that could be done through over-control would be about a 5 year level of service. Over-control would be relatively inefficient since about half of the potentially controllable land is not proposed for development. It is clear that over-control cannot accomplish an adequate solution on its own. Therefore, a storm sewer under Lorne and MacDonald Streets, similar to that designed by EMC Group in 1995 is recommended. It is noteworthy that the provision of some detention storage at Balmer Heights would reduce peak flow rates in the system and therefore potentially reduce the size and cost of a storm sewer system to convey runoff through the historic Village.

Tributary 2 - The hydraulic structures along Tributary 2 are generally satisfactory. However, due to the restriction at the CPR tracks, improvements such as building a stormwater management pond are required should the South Slopes area be developed.

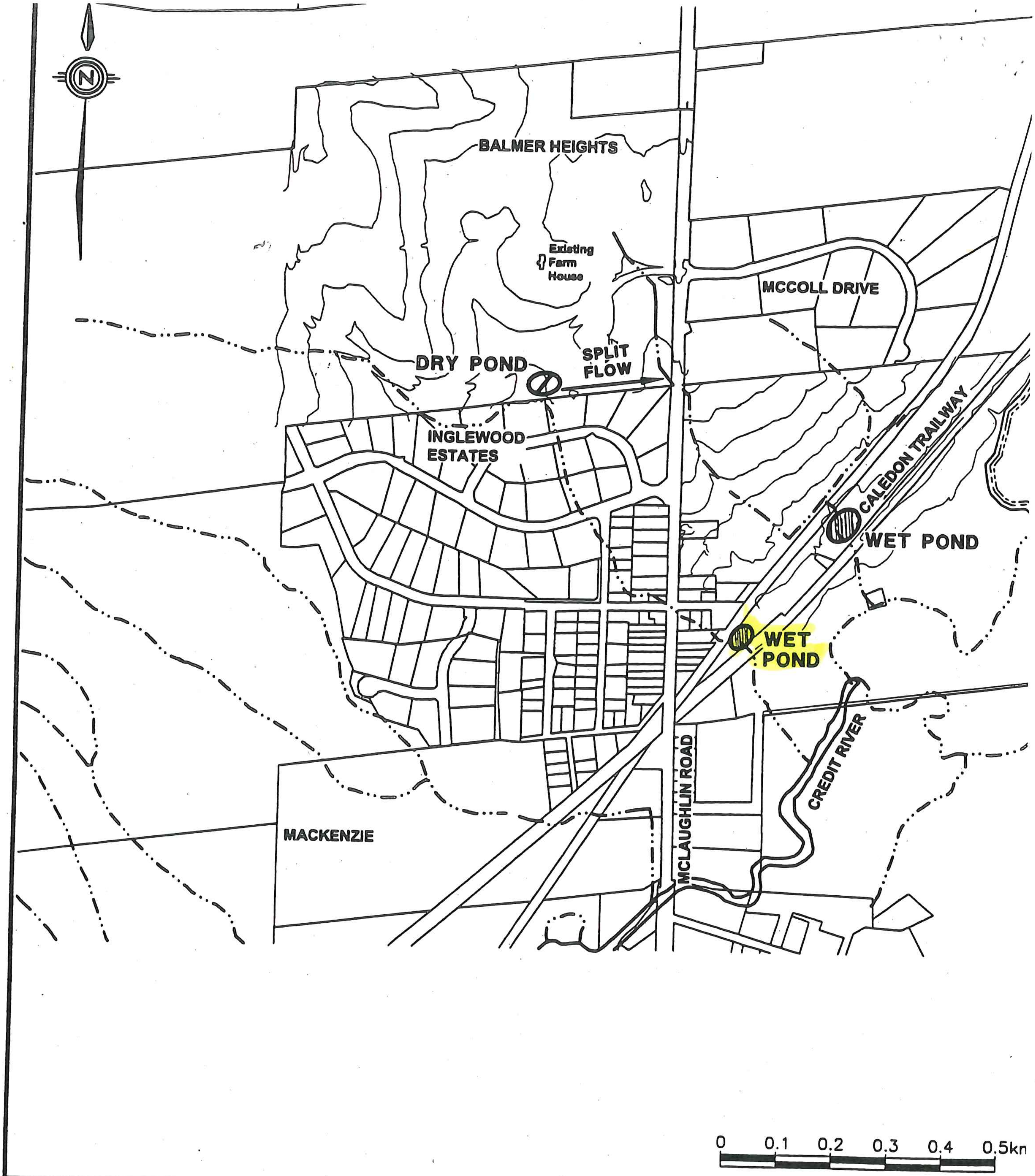
Tributary 3 - The major culverts upstream of the railway and the Caledon Trailway on Tributary 3 have adequate capacity and are not considered to be a problem. Adjacent to McLaughlin Road, the two culverts passing under the private entrances are inadequately sized. This area is within the floodplain of the Credit River and is likely to experience some flooding, regardless of the flow from Tributary 3. As a result no improvements to the hydraulic structures along Tributary 3 are recommended.

2.3.4 Stormwater Management Alternatives

Based on the results of preceding sections, two alternative stormwater management strategies were formulated. Each alternative has two variations that could be implemented. The alternatives and their variations are illustrated in Figure 2.2 Each of the alternatives (and variations) must meet the following criteria, as a minimum:

i. Maintain infiltration conditions in order to continue the moderate recharge capacity of lands proposed for development.
ii. Control post development peak flow rates to pre-development levels for the 2 through 100 year return period storm events.
iii. Provide erosion control storage in order prevent exceedence of erosion thresholds.
iv. Provide water quality control for Level 1 habitat protection, as set out in the Stormwater Management Practices Planning and Design Manual (MOE, 1994).
v. Seek to provide enhanced treatment for phosphorus.

Structural infiltration measures are not well suited to the till soils present throughout the area. In order meet criteria i) it will therefore be necessary to disperse runoff to the greatest extent possible. This can be done by a combination of lot level and conveyance controls. Roof discharge to lawns (and perhaps shallow ponding areas) should be employed in all areas. Further, the use of swale drainage, connected into open space drainage ways is recommended. In addition, end-of-pipe facilities will also be needed



CLIENT

TOWN OF CALEDON

TITLE

Figure 2.2

PREFERRED ALTERNATIVE
SWM POND LOCATIONS

Marshall Macklin Monaghan
Consulting Engineers . Surveyors . Planners
80 Commerce Valley Dr. E. Thornhill, Ont. L3T 7N4
Tele (905)882-1100 Fax (905)882-0055

Checked D.J.A.	Drawn AutoCAD/B.K.B.
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to fully meet these requirements. The combinations of the facilities that have been considered are described below.

A detailed review of all the alternatives considered can be found in the Inglewood Village Tributary Study.

2.3.5 The Preferred Alternative

As noted previously, each of the alternatives could be implemented as a feasible surface water management strategy. Any of proposed alternatives is capable of providing the required levels of quality and erosion control and peak flow attenuation <u>for future development</u>. Sufficient area exists for the ponds at the specified locations. Critical velocities in the subject tributaries can be controlled such that they will be exceeded less than once every 1.5 years on average, which is acceptable. The exception to this statement is the east branch of Tributary 2 (draining the McColl Drive area). Here a sound geomorphic design will be required to ensure that the rebuilt tributary remains stable.

Selection of the preferred alternative therefore becomes a question of the overall water quality benefit and the overall cost. On this basis Alternative 1B is clearly preferable to Alternative 1A and Alternative 2A is preferable to 2B. The decision therefore lies between Alternative 1B and 2A.

Alternative 2A has a projected cost which exceeds that of 1B by about $267,000. However it provides treatment for about 16.5 more hectares of existing development. The unit costs of providing water quality treatment are comparable between the alternatives ($9550/ha vs $9019/ha). Based on this similarity in cost and the agreement by the study team for the Inglewood Village Study to utilize every opportunity to achieve a reduction in the phosphorus loading to the Credit River, Alternative 2A is selected as the preferred alternative.

In Alternative 2A, a single dry pond is proposed in the Balmer Heights development area, to provide quantity and erosion control for the development area and external areas to the west. A flow splitter at the pond outlet would return flow to Tributaries 1 and 2 at or below pre-development levels. Downstream on Tributaries 1 and 2, two wet ponds are proposed between the CPR railway and the CNR trail. Each pond would be on-line and would provide quality, quantity and erosion control for the proposed development areas and areas of existing development which drain to these Tributaries. The proposed ponds would provide water quality treatment for a total of 120 ha.

2.3.6 Study conclusions

The following conclusions have been drawn from the Village of Inglewood Tributary Study and detailed work completed in the current study.

1. The study area is considered to be a moderate recharge zone. The Credit River in the vicinity of Inglewood is a discharge zone. The maintenance or enhancement of infiltration conditions in developing areas is therefore critical for down-gradient quality and quantity conditions.

2. The soils in Inglewood have low hydraulic conductivities and hence their infiltration capability, on a unit area basis, is low. As a result, structural or "point" effectiveness of infiltration techniques will

be limited. The primary method of infiltration in the post development condition will have to be based on dispersion of runoff over as much land as possible so that the greatest possible natural infiltration can occur. Techniques recommended are roof discharge to lawns (perhaps shallow ponding areas) and use of swale drainage connected to open space drainage.

3. The majority of significant natural areas in the vicinity of Inglewood are located outside of the settlement area and are associated primarily with the Niagara Escarpment or the valley of the Credit River. The most important natural elements within the study area are the watercourses and their associated function (or potential function) as valley corridors and linkages between the Escarpment and the Credit River.

4. The tributaries within the settlement area provide limited fish habitat because of their low to intermittent flow in the summer and the presence of culverts that impede fish passage during periods of higher flow. However, each of the tributaries is considered to contribute to fish habitat through their hydrologic, ecologic or geomorphic function. Therefore the tributary system should be preserved to the greatest extent feasible.

5. From a stormwater perspective, the preservation or incorporation of open space corridors in the development form will provide opportunities to disperse runoff and hence maximize natural infiltration.

6. Each of the tributaries is relatively sensitive to increases in flow magnitude or frequency. Tributary 1 in particular has a low threshold before erosion problems may be expected. Extended detention storage for erosion control will be required in upstream areas in order to avoid problems in the post development condition.

7. Flooding concerns exist on Tributary 1 due to the existing culvert capacities. While the problems could be reduced through over-control of flows from upstream areas, the problems would continue to exist. Flows generated by the areas of existing development (excluding existing upstream flows) are large enough to cause flooding during a 10 year return period storm. As a result, a combination of flow control for upstream areas and upgrading of downstream storm sewer systems should be pursued.

8. The water quality of the Credit River in the vicinity of Inglewood is classified as Policy 2 (Provincial Water Quality Objectives are currently exceeded and no further degradation will be permitted) on the basis of Total Phosphorus and bacteria. Studies completed as part of the Inglewood Village Study however, indicate no significant dissolved oxygen problems. However, because of the fact that the communal sewage treatment systems will contribute some additional phosphorus, all reasonable actions that will limit the discharge of phosphorus to the Credit River should be encouraged.

9. An acceptable tributary management strategy can be implemented for the Village Plan.There is sufficient space for the implementation of at-source, conveyance and end-of-pipe facilities..

3.0 FURTHER WORK

This chapter is divided into two sub-sections. Section 3.1 outlines, by study component, site-specific analyses that will be required as part of the approvals process. Section 3.2 describes needed monitoring that will be required to ensure compliance of the EMP.

3.1 Site Specific Analyses

Groundwater - Based on proposed development, site specific consideration should be given to ensuring that pre and post development infiltration conditions are maintained or enhanced. To this end, a water balance should be conducted for each development proposal. It should be also demonstrated how various tools and techniques will be utilized to maintain infiltration rates prior to issuing Draft Plan Conditions.

Hydrology - Based on the findings of the Tributary Study and the needs outlined above in the Groundwater Section, each development must prepare a stormwater implementation report that meets these requirements.

Terrestrial - Fieldwork carried out during the summer of 1998 was focused on the potential areas of impact that were known at that time. As a result, field data was collected within some of the wetlands and ESAs within the main Credit River valley, east of the existing Village. The areas surveyed were determined by existing potential sanitary sewer outlets and authorization received from landowners for access.

As a result, several areas require additional detailed assessment of their terrestrial communities prior to developments being recommended for approval. In particular, the area referred to as Special Study Area in Schedule M has site specific environmental constraints (see Appendix A). Access from the east would require crossing two tributaries and their associated steep valley slopes which are part of an important and sensitive corridor. Additional study is necessary to determine if such a crossing is feasible, and if so what mitigating measures would be required.

An additional area requiring study will be the preferred location for the wastewater outfall. Based on a preliminary site walk with Town and Regional staff, the area east of Macdonald Street, between the two tributary watercourses and immediately downstream of the 'S' curve in the Credit River is the preferred location. An assessment of the natural communities to be effected will be required to finalize the alignment and prescribe any mitigating measures.

The tributary which is located within the southwest corner of the Balmer Heights development has been impacted by the historic agricultural uses on the adjacent lands (Figure 3.1). It does, however, present considerable opportunities for rehabilitation and enhancement as a habitat corridor, and a connecting open space link from the Village to the Niagara Escarpment. Prior to development proceeding in this area, an assessment of the specific constraints and opportunities for this corridor should be carried out by the proponent. The results of this study, when approved, will influence the final alignment and ultimate condition of the tributary corridor and the proposed residential development in this area.

Figure 3.1: Balmer Heights Tributary, looking upstream

Fisheries - Site assessments on how the tributaries draining the study area are treated with respect to the recommendations of the Environmental Management Plan will be required.

Other Initiatives - The Ministry of Natural Resources and Credit Valley Conservation should be supported to continue the Experimental Atlantic Salmon Reintroduction Program and Lake Ontario Fisheries Unit habitat and spawning assessments that provided much of the background information. Where feasible the new MNR protocol for habitat assessment should be adapted.

Habitat studies and rehabilitation techniques may be investigated along with Special Harvest Regulations for the recreational fishery. The Credit River Fisheries Management Plan recommends that brown trout over-wintering and migration areas be investigated.

Future research can utilize the monitoring data from all components and relate it to fish productivity and diversity as "environmental indicators" and recreational resources. Data gaps that remain relate to recreational users and other socioeconomic aspects of the fishery.

Credit Valley Conservation should be supported in developing and implementing a master plan for the Ken Whillans Resource Management Area, focusing on habitat enhancement, fishing opportunities in the river and ponds and as a regional/national rail trail head facility. Similar adjacent lands should likewise be part of an integrated management plan. The increase in the local population and others visiting the Conservation Area will soon make this evident. This will require the continued cooperation with the Town of Caledon park and trailway staff. Private landowner opportunities may include informal agreements such as that with Trout Unlimited who manages the river in "fish friendly" ways for private landowners in exchange for conditional public access. Much of the private land is already accessed by the public which if not monitored and managed could cause damage to the Provincially Significant Wetlands, Environmentally Sensitive Areas, the fishery and other recreational opportunities. There is a good opportunity for key private landowners to have a Conservation Plan completed through a cooperative program with Credit Valley Conservation and the University of Guelph.

It should also be noted that related terrestrial habitats (including wetlands and riparian corridor) should be studied and managed together.

3.2 Monitoring

Groundwater - Spot streamflow measurements during low flow conditions (summer) at selected locations upstream and downstream of the proposed outlet location(s) would help confirm long-term conditions as development and urbanization proceeds. In addition, a long term monitoring program for Well #3 would help ensure the maintenance of the natural environment.

Water quality sampling should be carried out on a seasonal basis during both dry and wet events, both upstream and downstream of proposed outlet location(s).

Terrestrial - Waste water should be monitored through direct sampling at the outfall and chemical analysis. See the water quality section for more detail.

Water Supply - Monitoring of wetland communities in proximity to Well #3 will be required. Protocols will be developed to monitor the condition of the wetland vegetation communities and their related wildlife. Vegetation plot sampling and amphibian monitoring is recommended. This monitoring should begin as soon as possible in order to establish base line conditions prior to development proceeding. Finalization of the monitoring program will be discussed with the Region of Peel, CVC, MOE and DFO.

Geomorphology - From a fluvial geomorphological perspective, monitoring of physical channel processes should be conducted along tributaries, as well the Credit River. As part of this work, controls (upstream on the Credit River and on tributaries where no development is proposed) should be implemented. Tributary monitoring should occur downstream of development areas, including stormwater management facilities and should include the measurement of several permanent cross-sections, substrate sizes and a long-profile survey. This should be conducted once a year for the first three years after construction. Monitoring on the Credit River should be downstream of the proposed wastewater facility and should consist of a series of erosion pins installed along the banks as well as the measurement of several cross-sections. Measurement of the erosion pins should occur at least twice a year (spring and fall), while the cross-section measurements only need to be taken once per year.

Water Quality - Although the CVC does not anticipate any negative impacts to water quality, monitoring is required to ensure our expectations are met. Furthermore, conditions may change in the watershed which could not have been taken into account in the impact assessment performed in Phase II. Two main types of monitoring will be considered: in-stream and effluent monitoring.

In-Stream (Credit River)

Presently at the Credit Valley Conservation (CVC), we are mapping out an integrated monitoring program which includes monitoring in terms of hydrology, hydrogeology, fluvial geomorphology, biology and water quality. The following objectives were developed for this monitoring strategy:

Objective 1: Protect and improve water quality and quantity in the watershed.
Objective 2: Protect and improve biological diversity and productivity of the watershed.
Objective 3: Promote the relationships between a healthy watershed and economic and social activities.

CVC has divided the monitoring effort into two tiers. Tier 1 includes mainly physical and biological monitoring for water quality while Tier 2 includes chemical and more detailed biological monitoring. The two tiered approach aims to collect data to assess baseline conditions and to detect changes in the watershed (Tier 1) while having the ability to initiate more detailed monitoring where it is deemed necessary (Tier 2).

As mentioned in the previous chapter, the Credit River has high enough levels in total phosphorus (TP), to achieve Policy 2 status from the MOE. The problems associated with high TP levels are excessive aquatic plant growth leading to nighttime depletion of dissolved oxygen.

The communal system will add an additional, albeit small, load of total phosphorus at Inglewood. However, the expected increase in TP in the Credit River cannot be measured with any statistical confidence. Instead, a monitoring program should attempt to measure any potential effects of the phosphorus from the communal system.

Some of the proposed Tier 1 monitoring efforts include benthic sampling and 24-hour dissolved oxygen and water temperature surveys, which could be applied to the Inglewood monitoring sites. Basic aquatic plant monitoring may also be included in Tier 1 monitoring as the CVC continues to adjust its watershed-wide monitoring program to adapt to the changing needs of the watershed. CVC proposes to adopt tier 1 type monitoring for the Inglewood area. A summary of instream water quality monitoring parameters follows. (Table 3.2.1).

Description of Monitoring Protocols and Reporting

Dissolved oxygen surveys should be completed over a 24-hour period, preferably on a sunny day to catch the maximum effects of aquatic plants. Approximately 5 measurements per site should be taken on a rotating basis over the 24 hours. Times of measurements, weather conditions (including air temperature), water pH and water temperature should be recorded. When reporting results from the DO survey, include percent saturation of dissolved oxygen.

On the same day of the DO survey, qualitative observations of aquatic plants should be reported. Aquatic plant monitoring should include at least a presence-absence description of aquatic plants. If present, the following observation should be made:

- type of aquatic plants (i.e. attached algae, macrophyte, or emergents)
- plant species (if possible)

- approximate size of patch
- whether the plant patch is sparse (a few plant per square ft), moderate (plants mostly cover a square ft), or dense (thick coverage over square ft)

Photographs could be taken to provide evidence of density of plant vegetation.

Benthic monitoring should include a site description, which includes substrate type, presence of aquatic plants, water depth and width, and presence of any odour. Some possible monitoring protocols include rapid bio-assessment, BioMap and various kick and sweep methods. Benthic data should be reported in terms of density (no./m²), number of taxa, total organism abundance, Shannon-Wiener Diversity, % Oligochaeta, and % Chironomidae.

Table 3.2.1: Instream Water Quality Monitoring Parameters

Monitoring Item	Parameter(s)	General Description	Frequency	Time of Year	Personnel Required	Costs Involved
1	-dissolved oxygen -pH (if possible) -water temperature -aquatic plants (both macrophytes and attached algae)	**DO, pH, H_2O Temp:** 24-hour survey using meters **Aquatic plants:** qualitative observation and possibly photographic evidence	monthly	summer	-access to a DO and pH (if possible) meter -experience with aquatic plants	-cost of meter -time -mileage -camera
2	macroinvertebrates	-method TBA (based on an ongoing study out of the University of Western looking at different sampling protocols)	annually	summer	-expert for collection and identification protocol	-cost of collection equipment -time -mileage

The following considerations were made in determining a monitoring strategy for the Credit River to pick-up impacts from the communal sewage system:

1. locations
2. parameters (24-hr DO and temperature surveys, bio-monitoring)
3. sampling technique and data analysis
4. frequency and time of year
5. monitoring expertise required
6. outline of potential costs (time, services and materials)
7. reporting (what form and how often)

Monitoring Locations

The following general locations are proposed for monitoring items 1 (DO, pH, etc) and 2 (benthics):

1. Upstream of discharge pipe
   - downstream of the Little Credit River (East Credit) since water quality parameters measured in the Little Credit were significantly different than those in the Credit River for all sampling days in the summer of 1998
   - far enough upstream of the discharge pipe to avoid backwater effects (migration of discharge upstream)

2. Just downstream of discharge pipe
   - far enough downstream to have the effluent completely mixed (or as close as practically possible)
   - upstream of any stormwater facility discharge to isolate changes from the communal sewage

3. Further downstream of discharge pipe
   - far enough downstream to have the majority of settling out of suspended solids
   - should coincides with the recovery zone

In addition, it is recommended the monitoring item 1 (DO, pH, etc.) be carried out further downstream of the locations proposed above. Three additional sites are recommended at 1-km intervals downstream of location #3, and preferably where existing aquatic plant growth can be observed. The Credit River at Boston Mills is proposed as a Tier 1 site for the watershed-wide monitoring program and could be used in conjunction with this study.

Effluent Monitoring

The proposed communal sewage system is anticipated to produce a very high quality effluent. The following discharge quality is expected:

Total Ammonia	:	2 mg/l (summer), 5 mg/l (winter)
Total Phosphorus	:	0.15 mg/l
Suspended Solids	:	10 mg/l
BOD	:	10 mg/l

To ensure that the communal sewage system is performing as predicted and to measure the removal efficiency of relevant parameters, raw and treated wastewater should be monitored for typical parameters found in municipal wastewaters. The Ministry of the Environment (MOE) must issue a Certificate of Approval (C of A's) for the wastewater treatment plant. Typically, these C of A's require monitoring of the treated wastewater and the CVC recommends the following monitoring program:

1. The relevant parameters listed below should be tested on a monthly basis after critical stages in the treatment:
   - nitrates (NO_{2+3})

- total ammonia, pH, water temperature (to calculate un-ionized ammonia)
- Total Kjeldahl Nitrogen (TKN)
- total phosphorus (TP)
- soluble phosphorus (Sol P)
- biochemical oxygen demand (BOD)
- dissolved oxygen (DO)
- *E. coli*
- suspended solids (SS)

2. All of the above parameters should be tested on a monthly basis at the end of the discharge pipe.

3. Wastewater flows should be monitored to generate average daily and monthly flows.

Fisheries - It is recommended that annual or biannual fish biomass samples be continued at Inglewood in a scientifically comparable manner using established protocols. Analysis will concentrate on changes to fish community diversity (number and type of species) and productivity (total weight of fish sampled) as indicators of environmental changes. Further analysis related to other associated components (e.g. water quality, geomorphology, groundwater) will then be required to identify causal relationships and take appropriate action. A similar sampling site is also recommended downstream (of the proposed sanitary outfall location) at Boston Mills where other cumulative impacts may occur or as a "control" in other investigations. Other "controls" should be located upstream as part of a CVC watershed wide monitoring program using similar protocols.

Although less reliable, spawning surveys should also be continued. The greatest value from this exercise is the ability to identify and monitor groundwater contributions in a manner that demonstrates a direct linkage to aquatic communities. Wetland communities might likewise be monitored. This could be important if increased water withdrawals or climate changes begin to affect the local water table.

Potential thermal impacts from both stormwater and wastewater facilities can easily be investigated with temperature loggers or spot checks with an accurate thermometer at different times and seasons.

4.0 IMPLEMENTATION

The following section has been developed to act as a guide to facilitate how agencies are organized by area of expertise, legal jurisdiction and function, and how this applies to study implementation. Four broad areas have been identified for implementation: planning and policy, rehabilitation and retrofit, stewardship and education, and monitoring and reporting.

4.1 Planning and Policy

Hydrogeology – All proposed development must ensure that water table elevations, baseflow and aquifer yield are maintained in the post developed condition.

Hydrology - Appropriate techniques should be employed as discussed in the Inglewood Village Study to ensure that the quantity, quality and erosive nature of stormwater runoff are managed and that steps be taken to include the existing Village core in this assessment.

Terrestrial - The sensitive terrestrial features are to be protected through the Environmental Policy Area and Open Space Policy Area designation on Schedule M and the related Village Plan policies (see Appendix A).

Water Quality - This reach of the Credit River should be recognized as a Policy 2 river for bacteria and phosphorus. As such, it is important that all efforts to achieve an offsetting of phosphorus inputs to the Credit River be required.

Fisheries - All planning decisions and those by private landowners will adhere to Federal Fisheries Act to protect fish habitat. Provincial and local policies should encourage proactive protection measures to avoid punitive action and to embrace an improvement in fish habitat as part of a healthier environment and community.

Site specific environmental impact statements and mitigation plans related to fish habitat will be required for planning applications that directly affect the streams and river. These may also include potential impacts related to recharge and groundwater upwellings, thermal impacts from urban runoff and other sources of deleterious water quality. The Ministry of the Environment also provides a supportive legislative role in protecting water resources through the Ontario Water Resources Act.

Planners and others should be consistent with and consult the Credit River Fisheries Management Plan (MNR / CVC 1999) for further guidelines.

4.2 Rehabilitation and Retrofit

Terrestrial - The current condition and function of the watercourse and valleyland corridors were assessed within Phase 2. The tributary which passes through the existing Village (Inglewood Village Estates and a portion of Balmer Heights) was determined to be highly impacted, but providing a moderate connectivity function. Its functional role could be improved considerably if it was enhanced along several sections, and extended through upland plantings to connect with the Inglewood Slope

Environmentally Significant Area to the west. This restoration would also provide enhanced open space or recreational opportunities by re-connecting with the Niagara Escarpment and Bruce Trail.

The corridor to the north of the existing Village and the railway crossing provides an important connection between the Credit River valley and the Escarpment. While it has been determined to be highly sensitive, it has suffered from some moderate level impacts primarily from existing and previous agricultural land uses (Figure 4.1). This corridor could benefit considerably from some streamside naturalization.

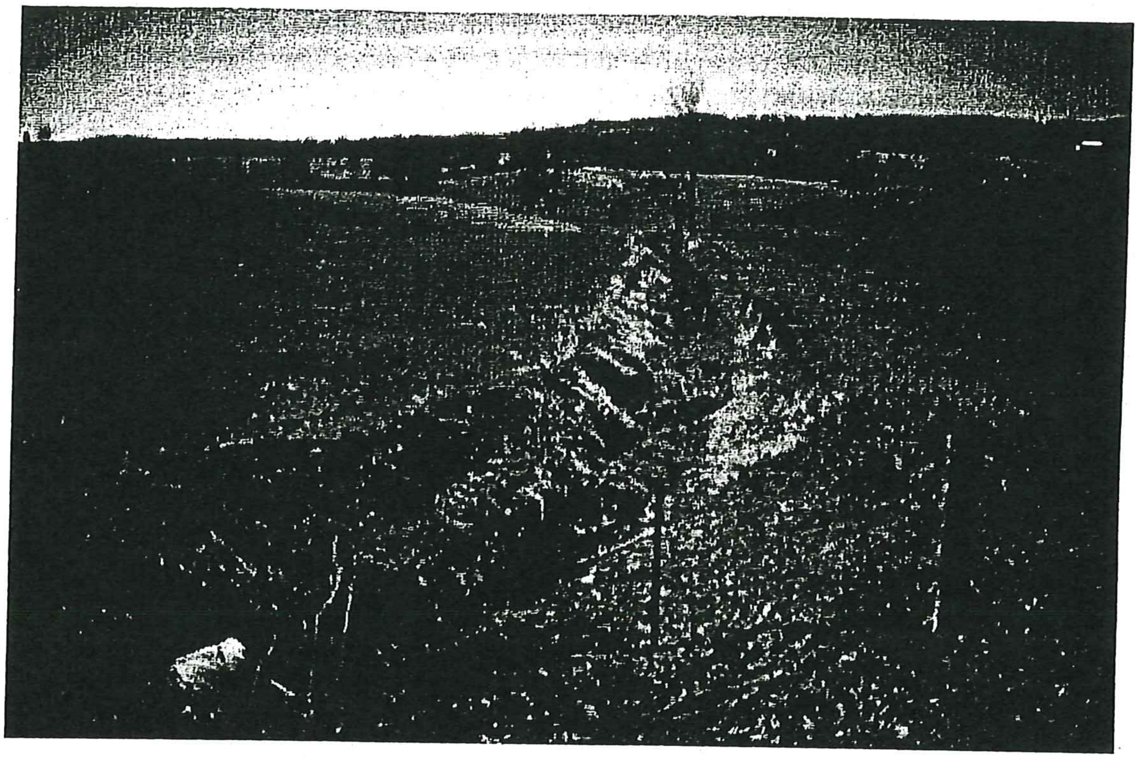

Figure 4.1: Tributary south of Grange Sideroad, West of McLaughlin Road

The corridors which extend through the proposed Mackenzie development to the Niagara Escarpment have been determined to be of high sensitivity. A northern tributary to this system has been impacted through the encroachment of intensive agriculture within the northern portion of its defined corridor boundary. This area is also recommended for re-naturalization along the corridor edge.

Geomorphology - There are several recommendations for rehabilitation of the channels within the study area. First, the highly to moderately sensitive channel reaches could be improved through increase plantings along the banks. Second, some erosion control work, including regrading of the bank and some aggressive plantings should be implemented around the outfall from the proposed wastewater facility. Finally, consideration should be given to assisting the river in 'cutting-off' the sharp meander bend. This permits the ability to control some the channel processes thereby avoiding the possibility of high erosive flows against the west bank, or the possible blockage of the western channel. This work is a significant undertaking and certainly requires some additional investigation and analysis. However, it would provide greater assurance regarding the channel and stability of the outfall location.

Fisheries - Much of the existing health of the fish community is based on the natural condition of the valley lands. This includes the woody material introduced into the channel that increases hydraulic roughness that reduces flood velocities and erosion. It also provides important nursery and adult cover for fish as well as other microhabitats for a full range of riverine life from insects to mink. The replacement of woody material where it has been removed at meander bends is strongly recommended to reduce aggravated erosion rates and replace fish cover. Volunteer labour from the community or fishing clubs may be utilized. Log jam removal should only be considered if it blocks the entire channel at low flow causing upstream sedimentation or side channel erosion. Technical advice from a geomorphologist or biologist should be sought in this case. Hard engineered constraints on the river, such as rip rap or gabion baskets should be avoided. The river and the floodplain should be respected as they provide vital riparian, fish and wetland habitat.

To reduce stream temperatures, bank erosion and provide fish and wildlife habitat, open riparian areas along the river and tributaries should be planted with suitable species. Note that beaver in the area will tend to leave certain trees such as cedar alone.

Decommissioning of the old migratory fish electric barrier should be done along with re-naturalization of the channel and eroding banks. A new barrier, if feasible, would have to be constructed downstream of the East/Little Credit confluence (Credit River Fisheries Management Plan, MNR/CVC, 1999). The next barrier downstream within the golf course lands also requires further investigation with a preference for removal due to siltation, thermal impacts and its impact on local fish migrations.

4.3 Stewardship and Education

Terrestrial - With the exception of the Ken Whillans Resource Management Area west of Highway 10, the Caledon Trailway, the Community Centre and Arena, the majority of the lands within the Inglewood Village study area are privately owned. A "Stewardship Brochure" should be produced for the residents of Inglewood describing the environment in and around the Village and how they can contribute its protection and enhancement.

A conservation planning service to be offered by CVC and the University of Guelph is currently under development. Under the guidance of CVC technical staff, with input from the landowner and others, a team of senior level university students would produce, for a fee, an inventory, assessment and series of stewardship recommendations in the form of a Conservation Plan. Conservation Plans should be considered for priority properties (i.e. those containing highly sensitive Core Natural Areas and Corridors).

Fisheries - A local program should be developed to better educate the growing community and visitors about the natural features and functions of the area with a focus on the river. Education and demonstration opportunities exist in the Ken Whillans Resource Management Area (Wild Trout Display Board), Arena Parkland and along the Caledon Trailway. Other opportunities include brochures, interpretive signage and special events such as a local riverfest. The Yellow Fish Road or Storm Drain Marking Program identifies storm drains in urban areas as being directly linked to the river. Literature provided door to door gives information on reducing local water pollution. Volunteers paint yellow fish "reminders" along the curb gutters.

It is important to conduct site visits with landowners to provide technical advice for stream, pond and wetland enhancement, and to resolve other issues regarding their private lands. A Conservation Plan (including terrestrial resource management options) should be done for the larger tracts of private land as recommended under Further Studies.

4.4 Monitoring and Reporting

Water Quality - In-stream Credit River monitoring reports should be generated every 2 years for the first 10 years for water quality and then every 5 years following. Reports should be circulated to the Town of Caledon and Credit Valley Conservation for review. Reviews may affect changes in monitoring protocols based on changes in the watershed, results requiring further investigation, or other suitable criteria.

All observations, measurements and other results should be reported as raw data, and in graphical format, where applicable. As a minimum, the following calculations are recommended for the reports:

- from the DO survey, include percent saturation of dissolved oxygen
- from the benthic surveys, density (no./m^2), include number of taxa, total organism abundance, Shannon-Wiener Diversity, % Oligochaeta, and % Chironomidae

Monitoring reports for the communal sewage system should be generated annually and submitted to Town of Caledon and the Credit Valley Conservation for review. The following calculations are recommended to be included in these reports:

- un-ionized ammonia
- removal efficiencies for relevant parameters (ammonia, nitrates, TP, BOD, *E. coli* and SS)
- average daily and monthly flows
- maximum flow per month

Both the in-stream and effluent monitoring reports will include a background review, presentation and analysis of the monitoring results, and discussion and conclusions from these results.

Terrestrial - Monitoring of effects from the proposed developments should occur during and after construction. A primary area of concern however will be potential draw down of the local ground water table from increased well use, and its effect on the terrestrial communities. Monitoring protocols should be established as soon as possible in order to determine base line conditions. Monitoring of vegetation should occur once a year after baseline establishment. Amphibian monitoring will be required during the critical spring period on an annual basis.

The potential effects from the construction of the wastewater treatment plant and discharge should be mitigated through immediate action (e.g. sediment discharge) or as part of the rehabilitation plan. Direct discharge to the Credit River will require water quality testing.

Reporting for the monitoring of the proposed developments should occur on a monthly basis beginning with the initiation of construction. If issues or concerns arise more frequent reporting may be required.

During the construction of the wastewater treatment facility and outfall, weekly reporting will be required. Post construction reporting of the water quality sampling should occur on a monthly basis.

Table 4.4.1: Implementation Summary

Plan Component	Action	Facilitator[1]	Contributor[2]	Related Implementation Mechanism
Planning and Policy				
	Detailed design of subdivision	Applicant	CVC, R. of Peel, T. of Caledon	Official Plan
	Environmental Impact Statements for new developments and waste water treatment facilities	T. of Caledon	CVC, Region of Peel	Official Plan
	Environmental Impact Statements for fish habitat that address impacts related to groundwater, recharge and water quality	CVC	MOE	The Fisheries Act, Ontario Water Resources Act, Official Plan
	Policies to protect sensitive environmental features and functions	T. of Caledon	Region of Peel, CVC	Official Plan
	Policies to support the need for rehabilitation and restoration	T. of Caledon	CVC, Region of Peel	Official Plan
Rehabilitation and Retrofit				
	Naturalization and preventative measures: Plant open riparian areas, naturalize around the defunct electric fish barrier, replace woody debris at meander bends	CVC, fishing clubs, local interest groups	Town of Caledon, Region of Peel	CVC stewardship and restoration services
	Plant the banks around the wastewater treatment facility outfall	Region of Peel	CVC	Construction of wastewater facility
	Construct a new fish barrier	Ministry of Natural Resources	CVC	
	Assist the river in cutting off the meander bend	Region of Peel	Region of Peel, CVC	Construction of wastewater treatment facility
	Restore important corridors	Landowners, CVC, Peel	T. of Caledon, Community Groups	Official Plan, Development Approvals, CVC Stewardship and Restoration

				Stewardship and Restoration Services
Stewardship and Education				
	Landowner Brochure and educational display board in local parks	CVC, Peel, Caledon Residents		CVC Stewardship and Restoration Services
	Storm drain marking program (yellow fish road)	CVC	Volunteers, local interest groups	Volunteer work
	Conservation Plans for priority properties	CVC, Landowners	Peel, Caledon, University of Guelph	CVC Stewardship and Restoration Services
Monitoring and Reporting				
	Monitor Developments During construction	Applicant	CVC, T. of Caledon, Peel	Planning Act Approvals
	Monitor Developments after construction	Applicant, Caledon, Peel	CVC, Naturalist Clubs	
	Monitor Waste Water Treatment Facilities during and after Construction	Peel	CVC	Environmental Assessment Approvals
	Amphibian and terrestrial monitoring	CVC	Local interest groups, Region of Peel, T. of Caledon	Volunteer work
	Monitor Effects from Well #3	Peel, CVC	Caledon, Naturalist Clubs, MOE	Ont. Water Resources Act

[1] The facilitator is responsible to carry out the action
[2] The contributor provides support to the facilitator

Note that if work is being completed in or adjacent to a river that hosts fish life or habitat, enforcement of Federal Fisheries Act for any harmful alteration, disruption or destruction of fish habitat with an emphasis on restoration orders is recommended.

If chemical or thermal impacts are detected, retrofitting of the stormwater facilities or communal septic system to meet predetermined targets should be done.

If baseflow or spawning areas are impacted by decreasing groundwater supplies, pumping rates or the redistribution of withdrawals from other wells should be investigated. Water conservation programs and rationing for some uses may also be considered.

5.0 RECOMMENDATIONS

Hydrogeology

1. All proposed development must ensure that water table elevations, baseflow and aquifer yield are maintained in the post developed condition.

Hydrology

2. All proposed development must ensure that stormwater management techniques are utilized to manage water quantity, quality and erosion. In order to achieve a net reduction in phosphorus to the Credit River, storm water runoff from the existing Village Core must also be treated for quality issues.

The following recommendations were made during the Inglewood Village Tributary Study:

It is recommended that a surface water management strategy consisting of a combination of source, conveyance and end-of-pipe stormwater controls. Specifically that:

3. Lot level and conveyance controls be employed to the greatest extent feasible in all proposed urban developments. Impervious surfaces such as roofs and driveways should discharge to lawns and then to swales. Roads, storm sewers (if employed) and swales should discharge to broad, shallow drainage corridors where possible. The objective should be to spread runoff out over as great an area as possible, in order to take advantage of the limited natural infiltration capacity of the local soils.

4. End-of-pipe controls be employed in the proposed Balmer Heights development to provide erosion and peak flow control. The Balmer Heights facility should be a dry pond total active storage volume of 14,020 m^3. The extended detention portion of active storage should comprise 6600 m^3. Since the facility will drain lands in both Tributaries 1 and 2, its discharge should be split between these tributaries in proportion to their pre-development flows.

5. Water quality control be provided for Tributary 1 (including both new and existing development) by implementing a pond between the CPR tracks and the Caledon Trailway. The facility should be a wet pond, with a permanent pool of 4650 m^3 and extended detention storage of 3200 m^3.

6. The proposed South Slopes development be served by a single end-of-pipe facility located on-line, between the CPR tracks and the Caledon Trailway. The facility should be a combined wet pond with a permanent pool of 3000 m^3 and total active storage of 5830 m^3 (1750 m^3 of which would be extended detention storage). Care must be taken in the design/redesign of the branches of Tributary 2 through the South Slopes property to ensure that they are geomorphologically stable and capable of conveying the anticipated flows from internal and external areas.

7. Due to the location, size, topography and difficulty of servicing anticipated for the McKenzie lands stormwater controls for this area will need to be implemented as part of the lot layout and grading strategy. For this reason, the proposed stormwater management for this area should be determined when specific development plans have been formulated.

Terrestrial

8. Prior to development proceeding within the Special Study Area shown on Schedule M, a detailed Environmental Impact Statement will be required to investigate the proposed road extension through the sensitive corridors, and the feasibility of developing the triangular area south-west of the corridor (see Appendix A).

9. Prior to development proceeding within Balmer Heights south and west of McColl Drive, the tributary and its associated corridor should be investigated to determine its most appropriate treatment. The details on its restoration and/or realignment should be assessed in detail by the applicant.

10. A detailed assessment is required for the preferred location of the wastewater treatment facility and its outfall.

11. Stewardship and restoration opportunities should be discussed with the landowners of high sensitivity corridors.

Fluvial Geomorphology

13. Stable functional tributaries need to be implemented using the principles of natural channel design techniques. In particular consideration should be given to the assisting the Credit River in "cutting off" the sharp meander bend just downstream of the proposed outfall location.

Water Quality

14. All critical processes in the treatment, redundancy be built into the system to accommodate for emergencies and routine maintenance activities. Therefore, there will be no need for the wastewater to be by-passed through the system at any time. However, in any case where wastewater is by-passed through any of the processes, the event will be described and the effluent will be sampled. This information will be forwarded to the appropriate agencies, including the Credit Valley Conservation and the Town of Caledon, and included in the annual report.

15. This reach of the Credit River be recognized as a Policy 2 river for bacteria and phosphorus and as such offsetting strategies should be implemented with respect to these parameters.

Fisheries

16. Potential impacts from increased pumping at Well #3 should be assessed over time through a monitoring program. The details of the monitoring program will be discussed with the Region of Peel, CVC, MOE and DFO.

17. That wastewater effluent targets be set with standards to protect aquatic life.

18. That detailed stormwater plans demonstrate that thermal impacts are fully mitigated.

19. That fish habitat within the river and tributaries be improved through a stewardship and restoration program.

DRAFT REPORT

Inglewood Village
Water and Wastewater Servicing Plans

June 1999

Prepared By:

XCG Consultants Ltd.

Prepared For:

Town of Caledon
Credit Valley Conservation
Region of Peel

XCG

CVC

TOWN OF CALEDON

Region of Peel

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1. INTRODUCTION

1.1 Background

1.1.1 Inglewood Village Study

The Village of Inglewood in the Town of Caledon, Regional Municipality of Peel, is located west of the Credit River north of Olde Base Line Road in the Region of Peel. Figure 1.1 presents the village settlement boundary.

The Inglewood Village Study was initiated to address planning, environmental and servicing implications of growth to the year 2021 in a comprehensive and integrated manner. The study included the following three distinct but interrelated components:

- Community Planning Component: This component focussed on the identification of a preferred growth strategy for the community to the year 2021, including population, boundaries, land use patterns, densities and municipal financial impacts.
- Servicing Component: In this component, preferred water and wastewater servicing strategies for the recommended community plan were identified.
- Environmental Component: This component provides information on environmental conditions and resulted in the preparation of an environmental management plan.

The Inglewood Village Study was conducted in four distinct phases. Phase 1 consisted of the collection, review and analysis of existing data. Phase 2 involved of the development and evaluation of community planning scenarios and servicing alternatives and the selection of a preferred alternative(s). In Phase 3, draft community and servicing plans were developed, and in Phase 4, these plans were finalized. Figure 1.2 presents the study process. Planning are trying to Locate these. BE

The Inglewood Village Study management team includes representatives from the Town of Caledon, the Regional Municipality of Peel, and the Credit Valley Conservation (CVC). The Town led the Planning Component, the Region led the Servicing Component and the CVC led the Environmental Component.

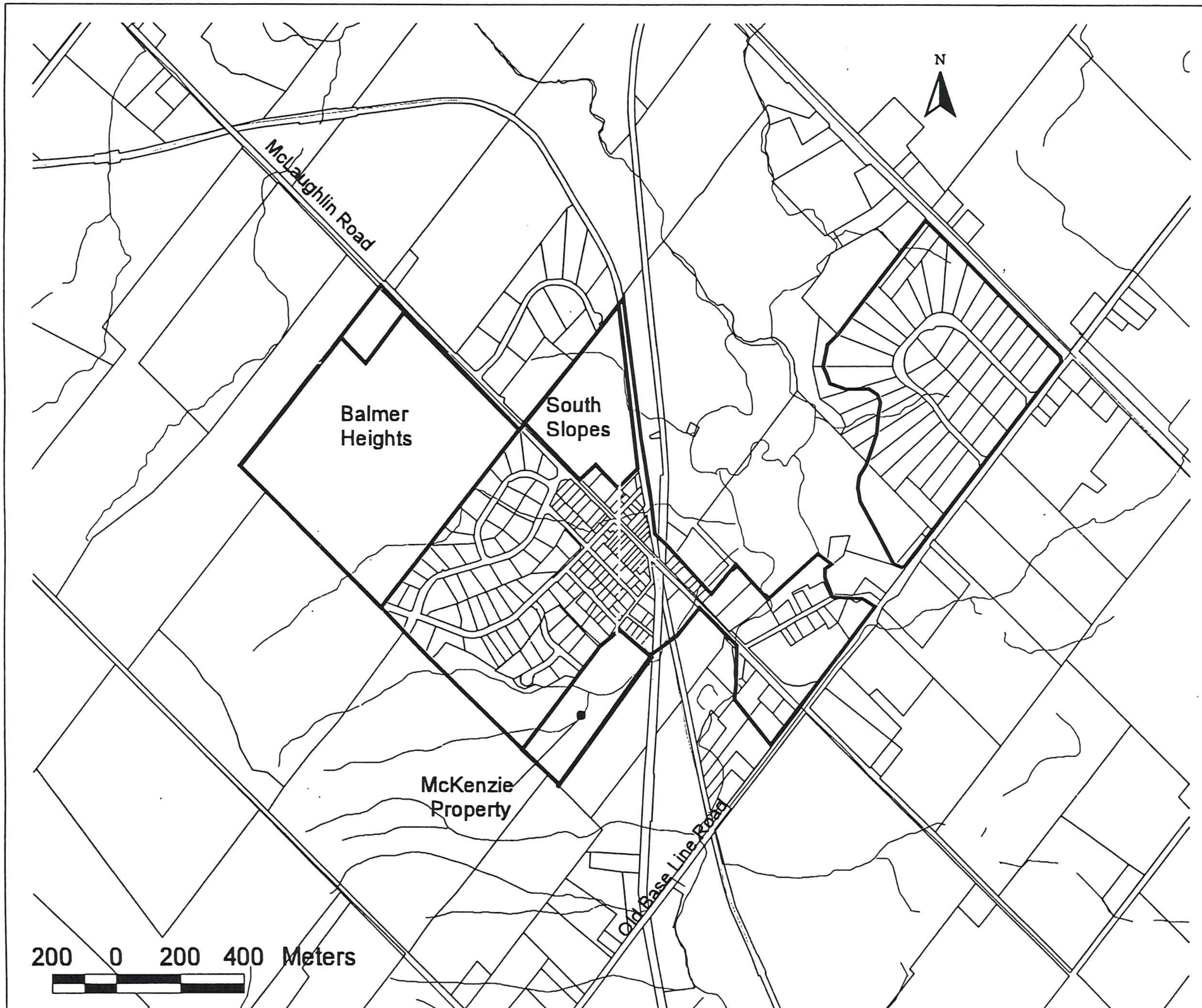

Figure 1.1
Village of Inglewood
Settlement Boundary

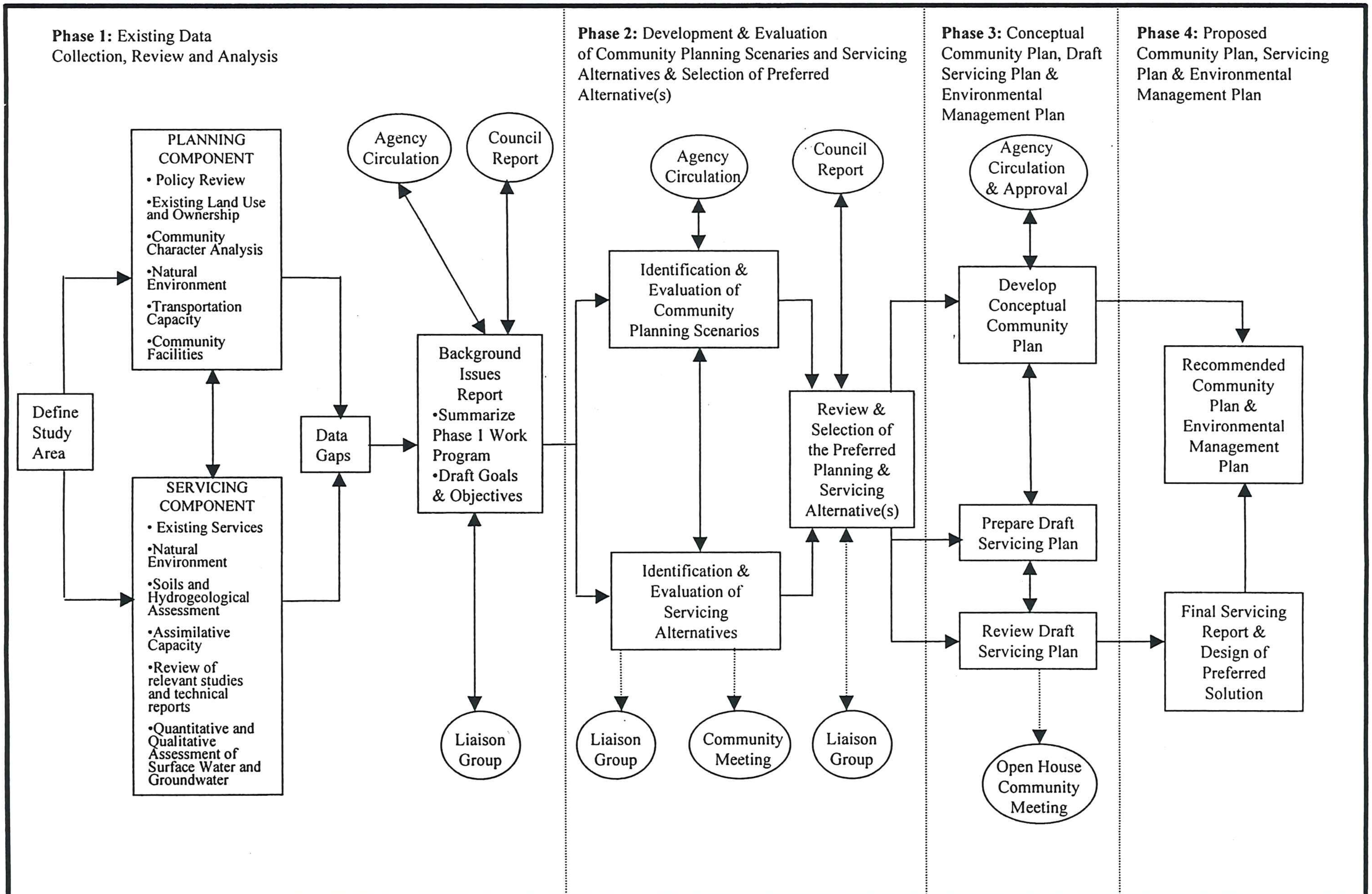

Figure 1.2
Inglewood Village Study Process

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1.1.2 Servicing Component

The Servicing Component involved the examination of water and wastewater issues pertaining to the Community of Inglewood. It included an evaluation of existing water and wastewater servicing infrastructure, an assessment of alternatives for servicing the recommended growth scenarios, identification of the preferred servicing option, and development of water and wastewater servicing plans for the community to the year 2021.

The objectives of the water and wastewater servicing plans are as follows:

1. To provide water and wastewater servicing capacity to service planned to the year 2021.
2. To ensure that the development of the plan meets the requirements of the Class Environmental Assessment process
3. To provide wastewater servicing by municipal or communal systems, if feasible, rather than individual on-site systems.
4. To ensure adequate wastewater treatment to meet provincial regulations and policies for protection of the groundwater and surface water.
5. To provide adequate water treatment to ensure that water quality meets the Ontario Drinking Water Quality Objectives (ODWO) and disinfection requirements.
6. To size water and wastewater systems based on Regional Design Criteria, and in consideration of MOE design guidelines and standards.
7. To locate new water and wastewater facilities appropriately, in consideration of existing land use policies, and the recommended Community Plan and Environmental Management Plan.

The existing servicing in the Village of Inglewood, policy context for servicing, and a detailed evaluation of alternatives for water and wastewater servicing to the year 2021 were presented in the Phase 1 and Phase 2 Inglewood Village Study reports (November, 1998 and March, 1999). This report presents the results of Phases 3 and 4 of the Servicing Component. The objectives of Phases 3 and 4 were:

1. To evaluate alternative design concepts for the preferred water and wastewater servicing alternatives and to select the preferred design concepts.
2. To identify implementation considerations for water and wastewater servicing to the year 2021.
3. To finalize conceptual water and wastewater servicing plans for the Village of Inglewood to the year 2021.

1.2 Preferred Planning and Servicing Alternatives

The existing population of Inglewood village is 610. A municipal water supply system provides water servicing to the majority of the residents of Inglewood Village. The system includes three groundwater wells, a storage reservoir and a distribution system. Wastewater treatment in Inglewood Village is provided by private sewage disposal systems with subsurface discharge.

At the completion of Phase 2 of the Inglewood Village Study, the preferred planning and servicing scenarios for the Village to the year 2021 were identified. These include the following elements:

- Development of currently undeveloped areas to achieve mixed village densities, providing for a range of lot sizes, densities and household types, to achieve a maximum 2021 Village population of 1,500;
- Implementation of a communal wastewater system to service new development areas and also to provide for servicing of approximately 95 lots in the Village Core.
- Continued use of the local groundwater supply to provide water to existing and new development areas.

1.3 Class Environmental Assessment Process

Ontario's Environmental Assessment Act requires proponents to examine and document the environmental effects that might result from major projects or activities and their alternatives. Municipal undertakings became subject to the Act in 1981.

The process is based on five key principles:

1. Consultation with affected parties.
2. Consideration of a reasonable range of alternatives.
3. Identification and consideration of the effects of each alternative on all aspects of the environment.
4. Systematic evaluation of alternatives in terms of their advantages and disadvantages, to determine their net environmental impacts.
5. Provision of clean and complete documentation of the planning process followed, to allow "traceability" of decision- making with respect to the project.

The Class Environmental Assessment for Municipal Water and Wastewater Projects (June, 1994) was prepared by the Municipal Engineers Association (MEA) and outlines the procedures to be followed to satisfy EA requirements for water, wastewater and stormwater management facilities. The Class EA process involves a thorough, systematic and reproducible evaluation of alternatives for servicing and their impacts on the environment. Public and agency consultations are integral to the Class EA process.

The Class EA process requires that four phases of work be completed for Schedule C projects including problem definition, identification and evaluation of alternative solutions to determine a preferred solution, examination of alternative methods of implementing the preferred solution, and documentation of the planning, design and consultation process.

The development of the Servicing Component, led by the Region of Peel, followed the Class Environmental Assessment process for Schedule C projects. This report, in combination with Phase 1 and Phase 2 Village Study Report, meets the requirements of an Environmental Study Report, which is typically completed as part of Phase IV (documentation of the planning, design and consultation process) of the Class EA process.

1.4 Intent of Report

This report is intended to provide information on the development of Water and Wastewater Servicing Plans for the Village of Inglewood. Conceptual designs in this report were developed to estimate costs and to compare alternatives. This report is not intended to present preliminary design information.

All costs presented in this report are inclusive of engineering and contingency, but do not include applicable taxes or land purchase. Costs are considered accurate to ± 35% at this conceptual design level.

1.5 Report Outline

There are four sections in this document. Section 2 presents the development and implementation of a preferred alternative for water servicing. Section 3 provides details on the development and implementation of a preferred alternative for wastewater servicing. Section 4 presents the recommended water and wastewater servicing plans. References are contained in Section 5.

2. WATER SERVICING

2.1 Background

The existing water supply system includes three groundwater wells, a storage reservoir and a distribution system. Figure 2.1 presents the existing water supply system. Table 2.1 presents descriptions of the three groundwater wells.

Table 2.1 Description of Existing Groundwater Wells

Facility	Well Capacity (L/s)	Existing Well Permit (L/s) (MOE 96-P-3027)	Permitted Maximum Combined Water Taking
Well #1	7.5	Standby only, maximum 4.5h per day operation	1,295,000 L/d (15 L/s)
Well #2	15	15	
Well #3	60	20	

Currently, Well #3 is operated as the lead well and Wells #1 and #2 are used for standby capacity. Well #1 and Well #2 cannot be operated concurrently as defined by the current permit.

The existing reservoir provides a total storage volume of 730 m^3.

Regional design criteria were used to estimate the water servicing requirements to accommodate new growth to the year 2021. Table 2.2 presents criteria and capacity requirements for water supply and reservoir storage.

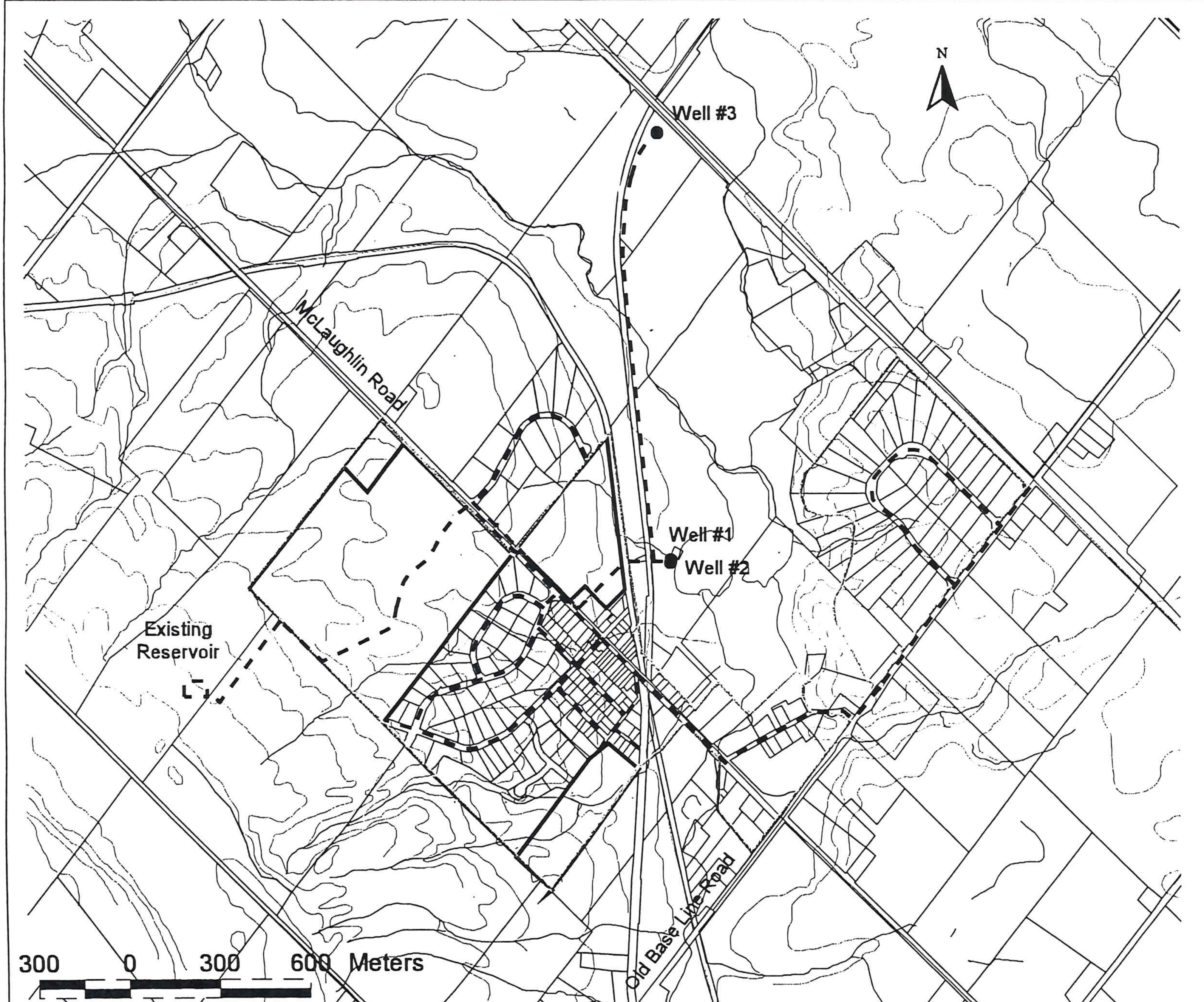
Table 2.2 Water Supply and Reservoir Capacity Required for 2021

Parameter	Regional Criteria	2021 Capacity
Average day flow	450 L/cap.d	680,000 L/d (7.9 L/s)
Peak day flow	2.5 times average	1,700,000 L/d (19.7 L/s)
Standby capacity	>10 L/s	>10 L/s
Reservoir volume	Average day flow plus 288 m^3	>968 m^3

The preferred servicing alternative to the year 2021 presented in Phase 2 of the Servicing Component included the following elements:

- Continued use of Well #3 as the operating well;
- An increase in the Permit to Take Water capacity for the combined water supply from the current capacity of 1,295,000 L/d to 1,700,000 L/d;

Inglewood Village Servicing Study



Legend

- Settlement Area Boundary
- Undeveloped Land
- Watercourses
- Existing Reservoir
- Existing Well
- Existing Watermain

Figure 2.1
Village of Inglewood
Existing Water
Distribution System

XCG

- Provision of a back-up or standby groundwater supply, providing at least 10 L/s capacity; and,
- Expansion of the reservoir from its existing volume of 730 m^3 to provide a minimum volume of 968 m^3.

2.2 Review of Design Concepts

Design concepts were review for providing water supply, standby capacity, storage and distribution for the preferred servicing alternative.

2.2.1 Water Supply

Well #3 has a rated capacity of 20 L/s, which is sufficient to meet the 2021 peak day demand of 19.7 L/s. An increase in the Permit to Take Water capacity to 1,700,000 L/d is required.

Three alternatives were considered for providing standby capacity, as follows:

- Use of Well #2, which has an existing capacity of 15 L/s;
- Interconnection of the existing distribution system with the Village of Cheltenham water supply system, about 4 km away; or,
- Installation of a new well.

Well #2 has historically provided water meeting the drinking water quality objectives. Furthermore, continued use of this well for standby capacity would have the lowest cost of the three alternatives. Therefore, the continued use of Well #2 was preferred for providing standby capacity.

2.2.2 Reservoir Storage

The existing reservoir has a total volume of 730 m^3 which is sufficient to service 990 persons. With an estimated maximum population of 1,500 persons in 2021, additional storage volume will be required at the reservoir. Expansion of the existing reservoir was identified as the preferred alternative for providing storage capacity.

The reservoir must be expanded to achieve a minimum volume of 968 m^3 to accommodate 1,500 persons. For the purposes of estimating capital costs, it was assumed that any reservoir expansion would involve a 50% increase in the capacity of the existing reservoir. The reservoir expansion would result in a total storage volume of 1,100 m^3. The total cost of expansion of the reservoir is estimated at $350,000.

2.2.3 Distribution System

The existing water distribution system consists of 150 mm, 200 mm and 300 mm diameter watermains which distribute the water pumped from the existing wells to the Village of Inglewood. There are two pressure zones within Inglewood with the upper pressure zone floating on the reservoir. Pressures in the lower pressure zone are controlled by a pressure reducing valve (PRV) located at McColl Street and McLaughlin Road. Figure 2.1 presents the existing water distribution system.

An analysis of the existing water distribution system was undertaken to determine if any improvements to the existing system would be required to accommodate development within the Balmer Heights, South Slopes or McKenzie Propertys. A previous study of water distribution systems in the Town of Caledon undertaken by KMK Consultants Limited (1997) involved the development of a Waterworks model of the water distribution system. This model was used to analyze the existing water distribution system for existing water demands and the 2021 water demand scenario.

The water distribution system was analyzed to ensure that residual pressures at all serviced areas would remain between 275 kPa and 690 kPa for both existing water demands and for the 2021 water demands. Analyses were conducted for the peak day demand and the peak day demand with fire flow requirements.

Appendix A contains the detailed results of the analyses conducted. The following sections describe the results of the analyses conducted.

Analysis of Existing Conditions

The existing water distribution system was analyzed for the existing service population for peak day demand and peak day demand with fire flow requirements. A fire flow demand of 40 L/s was simulated based on Regional design criteria.

The following points summarize the results of the analyses:

- Pressures were found to be above 690 kPa along the lower lengths of the feedermain from Well #3 as a result of the peak day demand. As no homes connected directly to the feedermain, pressures higher than 690 kPa in the feedermain have no negative impact on residents;
- Pressures were found to be marginally below 275 kPa at the intersection of Fieldcrest Road and West Village Drive due to a local high ground elevation as a result of the peak day demand; and,
- The results of the peak day demand and fire flow analysis indicated that a fire flow of 40 L/s was not possible at all locations within the service area.

In particular, a fire flow of 40 L/s could not be achieved within the Ingleview Drive subdivision and at the intersection of Fieldcrest Road and West Village Drive.

Analysis of 2021 Conditions

The water distribution system was also analyzed for the 2021 maximum service population of 1,500 persons. Analyses were conducted for the peak day demand and the peak day demand and fire flow requirements. A fire flow demand of 40 L/s was simulated based on Regional design criteria.

It was assumed that the development of the Balmer Heights area would result in the connection of the MacDonald Street watermain with a future watermain servicing Balmer Heights. A pressure reducing valve is also planned for installation at the intersection of West Village Drive and McLaughlin Road. Previous studies, commissioned by the Region, have shown that this PRV is required to increase water pressure to existing residences on West Village Drive.

The following points summarize the results of the analyses:

- Pressures were found to be above 690 kPa along the lower lengths of the feedermain from Well #3 as a result of the peak day demand. As no homes connected directly to the feedermain, pressures higher than 690 kPa in the feedermain have no negative impact on residents.
- Pressures throughout the remaining watermain in the village were found to be within the range of 275 kPa and 690 kPa as a result of the peak day demand.
- The results of the peak day and fire flow analysis indicated that a fire flow of 40 L/s was possible at all locations within the service area.

Based on this analysis, the water distribution system is adequately sized to continue to meet the water demand of the existing village and to meet the needs of future development. The servicing of the Balmer Heights area with the extension of the MacDonald Street watermain and the installation of a new pressure reducing valve at the intersection of West Village Drive and McLaughlin Road, would ensure that a fire flow of 40 L/s can be achieved within the entire service area.

2.3 Implementation

Implementation of the water servicing projects would be triggered by development within the new development areas. Table 2.3 presents the implementation considerations for the preferred alternative.

Table 2.3 Implementation Considerations for Water Servicing Alternative

Project	Implementation Considerations
Connection of MacDonald Street Watermain with Balmer Heights area	Connection and PRV will be required to service Balmer Heights.
Installation of a new PRV at the intersection of West Village Drive and McLaughlin Road	Installation will increase existing water pressures in the West Village Drive area. Installation can be completed following the completion of the Servicing Component and Class EA Study.
Permit to Take Water	New permit must be in place for more than 123 lots to be approved for development. Permit application can be completed following the completion of the Servicing Component and Class EA study.
Reservoir Expansion	Expansion will be triggered with new development. Project can be completed in a single phase.

The completion of the connection of a watermain in the Balmer Heights area and the existing MacDonald Street watermain will be required to service the Balmer Heights area. Installation of a new PRV at the intersection of West Village Drive and McLaughlin Road will improve existing water pressures and can be completed following the completion of the Servicing Component and Class EA Study.

A new Permit to Take Water can be applied for following the completion of this Class EA study. The new permit can also specify Well #1 and #2 as standby wells.

The reservoir expansion will be triggered by new development. The existing reservoir capacity is sufficient to service a population of 990 persons. Expansion of the reservoir can be completed in a single phase.

2.4 Capital Cost of Projects

The capital costs have been estimated for each project.

The connection of the MacDonald Street Watermain with the Balmer Heights area would be within the Balmer Heights lands. The cost of the new PRV located at the intersection of West Village Drive and McLaughlin Road is estimated at $75,000. The cost of the reservoir expansion is estimated at $350,000. Costs at this conceptual level are considered ±35%.

The capital cost of the Permit to Take Water will be negligible.

3. WASTEWATER SERVICING

3.1 Background

Wastewater treatment in Inglewood Village is currently provided by private sewage disposal systems with subsurface drainage. A 1999 survey of the septic systems in the Village (XCG) revealed that systems in the newer areas were designed according to current standards and are operating well. That survey also indicated that, primarily due to age and undersizing of the systems, systems in the older Village Core were deteriorating. There was evidence of environmental impacts from failing systems, which would be anticipated to worsen over the design period if a major overhaul or system replacement were not undertaken.

In light of the capacity requirements for servicing new development areas, and the condition of the existing septic systems, the recommended wastewater servicing alternative for the year 2021 includes:

- Construction of a new collection system to collect wastewater generated from new development areas and the Village Core area; and,
- Construction of a new tertiary wastewater treatment plant with a continuous discharge to the Credit River.

Regional design criteria were used to develop the wastewater collection system capacity requirements and conceptual designs to the year 2021. Table 3.1 presents the capacity requirement for wastewater treatment.

Table 3.1 Wastewater Treatment Plant Capacity Required for 2021

Parameter	Design Criteria	2021 Capacity
Average daily flow	393 L/cap.d	390 m^3/d^1

Note:

1. Based on servicing a total population of 945, including 95 lots in the Village Core and 220 lots in new development areas (3.15 ppu).

3.2 Review of Design Concepts

Design concepts were reviewed for the collection system type, the treatment plant location and the treatment technology for the preferred wastewater servicing alternative.

3.3 Wastewater Treatment Plant Process

In light of the sensitivity of the Credit River to receive wastewater from the treatment plant, and regulations and policies governing direct discharges to Ontario surface waters, effluent limits were proposed for the new treatment facility in Phase 2 of the Inglewood Village study. These are presented in Table 3.2.

Table 3.2 Conceptual Effluent Requirements for Treatment Plant

Parameter	Value
Biochemical oxygen demand (BOD_5) (mg/L)	15
Total suspended solids (TSS) (mg/L)	15
Ammonia-nitrogen (mg/L)	2
Nitrate-nitrogen (mg/L)	No limit
Total phosphorus (mg/L)	0.15
Acute toxicity to Daphnia Magna and Rainbow Trout	Non-toxic

In order to achieve the proposed effluent limits, the conceptual design of the treatment plant includes the following processes:

- Headworks, including grit removal and screening;
- Secondary biological treatment with nitrification;
- Coagulant addition for phosphorus precipitation;
- Tertiary phosphorus removal;
- Ultraviolet disinfection; and,
- On-site sludge storage with off-site disposal.

The total estimated cost for the treatment plant is estimated at $1,100,000. At this conceptual level, costs are considered accurate to ±35%.

3.3.1 Collection System Technologies

Three collection system technologies were considered to collect wastewater from 95 lots within the existing Village Core and new development areas, and to convey it to the new wastewater treatment plant. The technologies considered included a gravity collection system, a vacuum sewer collection system, and a pumped collection system.

Gravity Collection System

A gravity collection system is the most commonly used type of system for collecting municipal wastewater and transferring it to centralized treatment facilities. For Inglewood, a new sewer system would involve the construction of a gravity sewer system along existing roadways in the Village Core, and along new roads within the new development areas. For existing homes, internal plumbing changes and new laterals from the building to the sewer at the road would be required. Existing septic systems would no longer be used. For new development areas, connections to the building would be constructed at the time of development.

Vacuum Sewers

For areas with existing septic tanks, a vacuum sewer collection system can be used to collect and transmit flow from the septic tanks to the treatment facilities. This type of system can be used where septic tanks are already in place, and has the primary advantages of lower collection and treatment costs (because partial treatment occurs in the septic tank) compared to conventional gravity sewers.

Vacuum sewers involve the construction of small diameter forcemain along existing roadways in the village core. The forcemain would be connected to the existing septic tanks. A pumping station would be required to create a negative pressure in the system to withdraw effluent from the septic tanks to a wastewater treatment plant.

Pumped Collection System

Similar to a vacuum system, a pumped collection system can also be used where septic tanks are already in place. For this type of system, each septic tank is equipped with a grinder pump, which operates automatically based on levels in the septic tank, to pump effluent from the septic tank to centralized treatment facilities. Like vacuum sewers, this type of system can be used where septic tanks are already in place, and has the primary advantages of lower collection and treatment costs (because partial treatment occurs in the septic tank).

Comparison of Collection System Alternatives

Table 3.3 presents a comparison of the advantages and disadvantages of three types of collection systems.

Table 3.3 Advantages and Disadvantages of Wastewater Collection System Technologies

System Type	Advantages	Disadvantages
Vacuum Sewer Collection System	• Existing septic systems would remain in place. • Vacuum sewers would be smaller diameter and could be installed at a shallower depth than gravity system. • Vacuum system would have a lower cost than gravity system. • Treatment plant requirements would be reduced because of partial treatment in septic tank.	• Residents would still be responsible for the maintenance of their septic tanks. • Higher operating costs associated with vacuum operation compared to gravity system. • Leaking septic tanks would have to be replaced by property owners. • Potential for septic tank overflow which would impact on local watercourses if septic tanks are not maintained.
Pumped Collection System	• Existing septic systems would remain in place. • Smaller diameter forcemain could be installed at a shallower depth than gravity sewers. • Pumped system would have a lower cost than gravity system. • Treatment plant requirements would be reduced because of partial treatment in septic tank.	• Residents would still be responsible for the maintenance of their septic tanks and pumps. • Operating costs associated with pump operation. • Leaking septic tanks would have to be replaced by property owners. • Potential for septic tank overflow which would impact on local watercourses if septic tanks are not maintained.
Gravity Sewer Collection System	• Operating costs associated with gravity system are less than other systems. • Residents would not be responsible for any aspect of operation or maintenance of wastewater system.	• Higher capital cost than vacuum or pumped system.

For new subdivision areas, a conventional gravity system is considered to be the preferred alternative. With other types of collection systems, the requirement for individual septic tanks for each new lot would significantly reduce the cost advantage of these systems.

For the Village Core area, the poor existing condition of the septic tanks suggest that systems which depend on these tanks would not be appropriate unless major repair or replacement of undersized and deteriorated tanks was undertaken. The costs to residents to complete this work would reduce the cost advantage of these types of systems. It should be noted that the septic system survey (XCG, 1999) was not comprehensive, and there may be certain areas in the Village Core where these types of systems could be considered at the detailed design stage.

The preferred design concept for collection of wastewater from the Village Core and new development areas is based on a gravity sewer system.

3.4 Wastewater Treatment Plant Site

Two alternative sites for location of the wastewater treatment plant were developed. To enable a comparison of alternatives, the conceptual collection system configurations and costs were developed for each alternative.

The following sections provide details on the development and evaluation of the alternatives and the selection of the preferred alternative.

3.4.1 Development of Alternatives

In consideration of the environmental sensitivities in the area as defined in the Environmental Component, and other practical constraints, criteria for wastewater treatment plant siting were identified.

Table 3.4 presents the criteria used to identify feasible wastewater treatment plant sites.

Table 3.4 Criteria for Identification of Alternative Wastewater Treatment Plant Sites

Criteria	Description
Floodline Criteria	The wastewater treatment plant must be located outside of the Regional floodline.
Outfall Criteria	Outfall selection: • Located within main Credit River, not discharged to tributaries; • Avoid discharging to sensitive wetland/Environmentally sensitive areas; • Avoid spawning areas (none mapped 700 m upstream of McLaughlin Road bridge)
Design Considerations	The treatment plant should be located at relatively low elevation, to minimize the costs associated with wastewater pumping. The treatment plant should be relatively close to the community and discharge point to minimize infrastructure costs.

Sites located within the Regional floodplain for the Credit River were eliminated from further consideration. Outfall siting criteria were also applied to identify alternative wastewater treatment plant sites. The Credit Valley Conservation Authority (CVC) indicated that any wastewater treatment plant outfall should not discharge into sensitive reaches of the Credit River. Reaches which have been indicated as sensitive include the bend located immediately downstream of the confluence of the Credit River and the Little Credit River.

Design considerations were also included as a criterion for the identification of wastewater treatment plant sites. Sites which were located on high ground were avoided to minimize pumping requirements. In addition, it was recognized that the wastewater treatment plant should be located in a reasonable proximity to the existing village to avoid the need to construct long pipelines.

The application of the criteria listed in Table 3.4 resulted in the identification of two alternative sites for the wastewater treatment plant. The first alternative involved the construction of the wastewater treatment plant at a site adjacent to the Caledon Trailway. The second alternative involved the construction of the wastewater treatment plant on a site located within the McKenzie Property. The following sections provide the details of these alternatives.

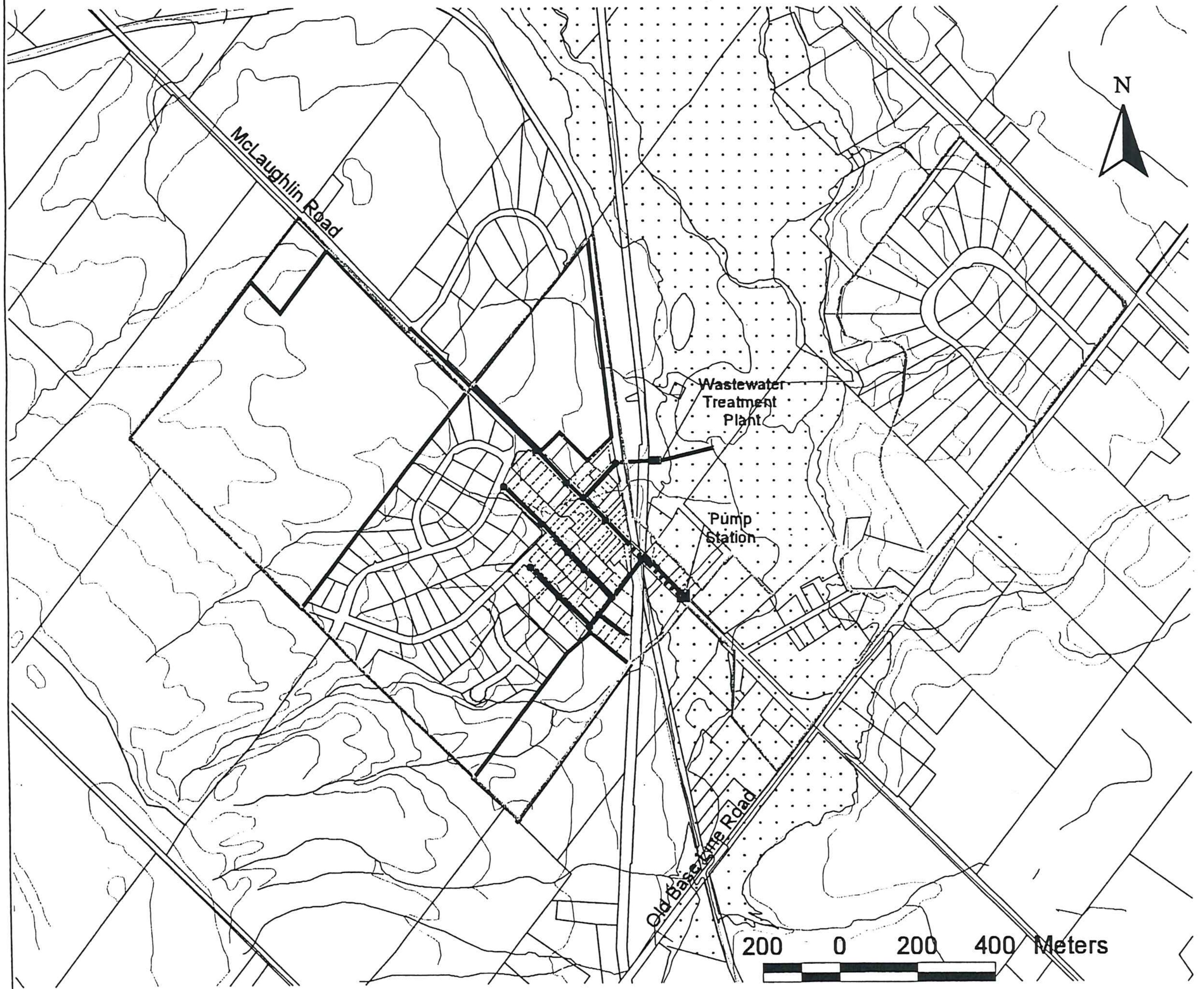
3.4.2 Description of Alternatives

Alternative 1 – Wastewater Treatment Plant Located Adjacent to Caledon Trailway

Figure 3.1 presents the location of the wastewater treatment plant and the associated collection system for this alternative. This alternative would involve the construction of the wastewater treatment plant on lands adjacent to the Caledon Trailway at the end of MacDonald Street. The outfall would be located downstream of the bend in the Credit River.

For the conceptual collection system design associated with this treatment plant site, wastewater from the Balmer Heights area would be directed either by forcemain or gravity sewer to a sewer on McLaughlin Road. Wastewater from the South Slopes area along with MacDonald Street and most of McLaughlin Road would be conveyed by gravity to the wastewater treatment plant. Wastewater from the remaining existing Village Core and the McKenzie Property would be conveyed to a new pump station on McLaughlin Road. This pump station would pump flow a short distance north into a new gravity sewer located on McLaughlin Road north of McKenzie Street.

Preliminary information on the collection system associated with this treatment plant location is provided in Appendix B.



Inglewood Village Servicing Study

Legend

- Settlement Area Boundary
- Plant Location
- Undeveloped Land
- Sanitary Forcemain
- Sanitary Sewer
- Plant Outfall
- Manholes
- Pump Station
- Existing Service Area
- Regional Floodplain
- Watercourses
- Contours

Figure 3.1
Alternative #1
WWTP Located
Adjacent to
Caledon Trailway

XCG

3.4.3 Alternative 2 – Wastewater Treatment Plant Located in McKenzie Property

Figure 3.2 presents the location of the wastewater treatment plant and associated collection system for this alternative. This alternative would involve the construction of the wastewater treatment plant on lands within the McKenzie Property adjacent to McKenzie Street. The outfall for this plant location would discharge into the Credit River at the McLaughlin Road bridge over the Credit River.

For the conceptual collection system design, wastewater from the Balmer Heights Area would be conveyed to a sewer on McLaughlin Road. Wastewater from the majority of the Village Core and the South Slopes area would be conveyed to a sewer located on McLaughlin Road to the wastewater treatment plant. Wastewater from McLaughlin Road south of McKenzie Street would be pumped to the treatment plant.

Preliminary information on the collection system associated with this treatment plant location is located within Appendix B.

3.4.4 Evaluation of Alternatives

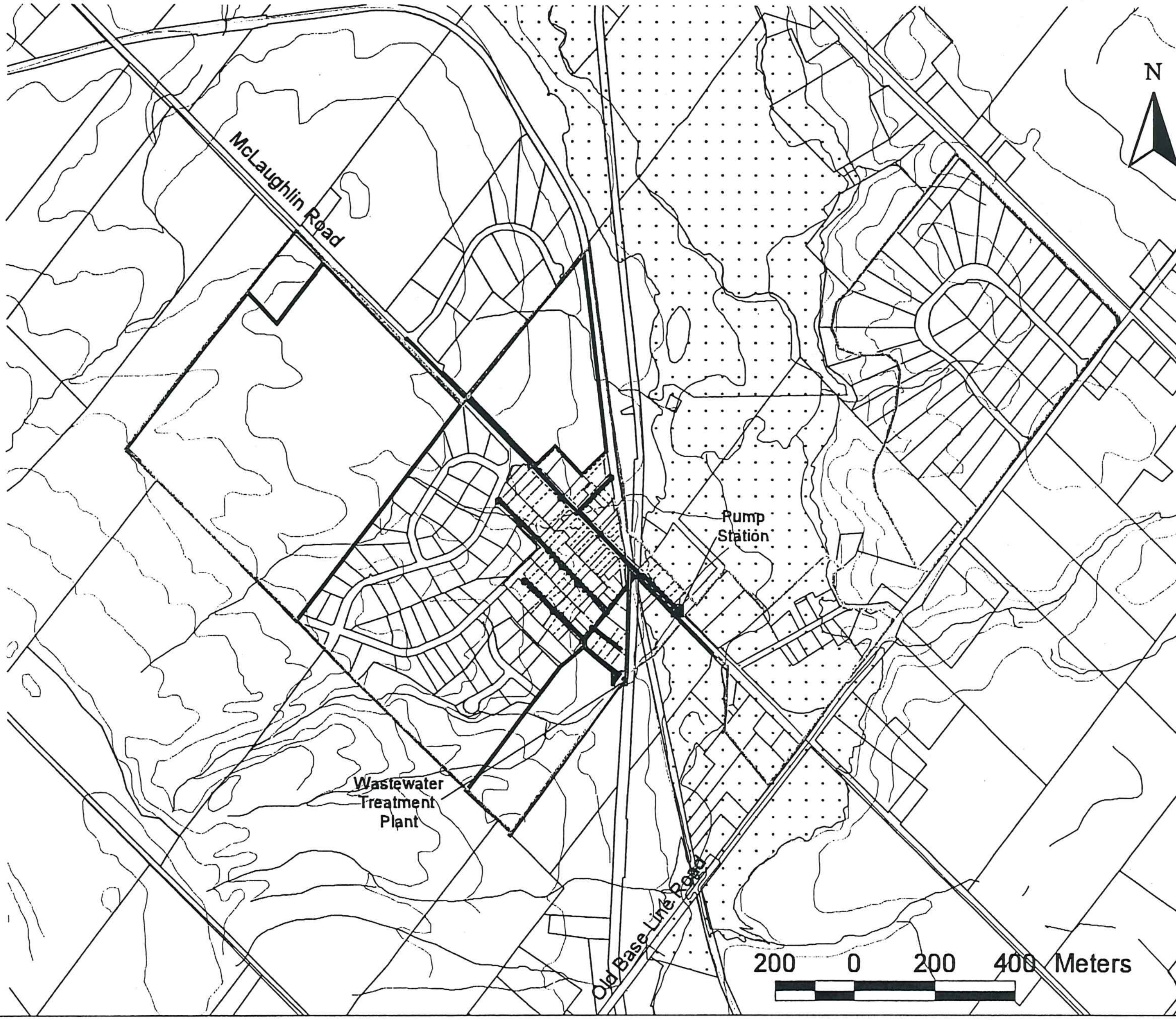
An evaluation of the two alternatives described in the previous section was undertaken to select a preferred wastewater treatment plant location. The following criteria were to evaluate the alternative design concepts.

- Environmental impacts;
- Community impacts;
- Impact on development;
- Expandability; and,
- Cost.

Table 3.5 presents descriptions of the evaluation criteria. The following sections present the evaluation of the alternatives.

Table 3.5 Description of Evaluation Criteria

Criteria	Description
Environmental Impacts	Impacts arising due to construction of wastewater treatment plant outfall to Credit River.
Community Impacts	Potential visual, noise, and odour impacts on existing and future residents and users of Village area.
Impact on Development	Impact on availability of developable lands.
Expandability	Ability to expand treatment plant in the future (after 2021).
Cost	Capital cost of alternative.



Inglewood Village Servicing Study

Legend

- Settlement Area Boundary
- Plant Location
- Undeveloped Land
- Sanitary Forcemain
- Sanitary Sewer
- Plant Outfall
- Manholes
- Pump Station
- Existing Service Area
- Regional Floodplain
- Watercourses
- Contours

Figure 3.2
Alternative #2
WWTP Located
in McKenzie
Property

XCG

Table 3.6 presents the information collected for the two alternatives. The following sections discuss the results of the evaluation.

Table 3.6 Information on Alternatives for Wastewater Treatment Plant Sites

Criteria	Alternative 1 – Wastewater Treatment Plant Located Adjacent to Caledon Trailway	Alternative 2 – Wastewater Treatment Plant Located Within McKenzie Property Area
Environmental Impacts	• Outfall sewer may be constructed at the location of an existing deciduous forest. Trees may need to be removed as part of construction. • Impact can be mitigated through good construction practice.	• Outfall sewer would be constructed along existing road allowances. • Outfall would be closer to fish spawning area than for Alternative 1.
Community Impacts	• Potential visual impact of treatment plant on users of the Caledon Trailway could be mitigated with fences and berms. • Trailway would provide a buffer between existing village and treatment plant. • Buffer of approximately 100 m from residents would ensure minimal noise impacts. • Minimal odour impacts as odour control would be incorporated into the design of the wastewater treatment plant.	• Potential visual impact of treatment plant on existing residents and future residents on Kaufman Road, Inglenook Court and in McKenzie Property as plant would be located at a lower elevation than many residences (i.e. if houses would look out over plant). • Buffer of 100 m from residents would ensure minimal noise impacts. • Minimal odour impacts as odour control would be incorporated into the design of the wastewater treatment plant.
Impact on Development	• No impact on developable lands as site is not located within a development area.	• Wastewater treatment plant and required buffer would reduce developable lands within McKenzie Property by 1 to 2 ha.
Expandability	• Adequate land area exists to expand the plant beyond the 2021 capacity.	• Adequate land area exists to expand the plant beyond the 2021 capacity.
Cost	• Estimated capital cost of $2.3 million for collection and treatment (excluding components within new development areas, including engineering and contingencies).	• Estimated capital cost of $2.6 million for collection and treatment (excluding components within new development areas, including engineering and contingencies).

Environmental Impacts

The outfall for Alternative 1 would be located downstream of the bend in the Credit River. A study of the area has determined that the outfall sewer may be located in close proximity to an area within an existing deciduous forest. As the method of construction would likely be open trench construction, trees in the vicinity of the outfall sewer may have to be removed. However, this impact could be mitigated as part of the detailed design of the outfall sewer by siting the outfall sewer to avoid the deciduous forest.

Alternative 2 would involve the construction of an outfall sewer along the existing road allowances of McKenzie Street and McLaughlin Road. The impact of the outfall on the Credit River would be similar for both alternatives as the river at both locations has a similar sensitivity to erosion.

Community Impacts

The wastewater treatment plant in Alternative 1 would be constructed on lands located east of the existing village adjacent to the Caledon Trailway. The wastewater treatment plant could have a visual impact on the users of the Caledon Trailway. However, fences or berms could be constructed to minimize this impact. Furthermore, the plant would not be visible to residents.

A wastewater treatment plant for Alternative 2 would be located on lands west of the existing village and would likely have a visual impact to residents on Kaufman Road, Inglenook Court, and future residents in the McKenzie Property. The plant would be located at a lower elevation and within the site line of residences located on Kaufman Road, Inglenook Court and in the McKenzie Property.

For both alternatives, noise impacts would be minimal. A buffer of 100 m, recommended by the Ministry of the Environment, would limit noise impacts to existing village core residents and future residents of the McKenzie Property.

Odour impacts on existing residents and future residents of the McKenzie Property and users of the Caledon Trailway would be minimal. Odour control would be incorporated into the design of the new wastewater treatment plant.

Impact on Development

Alternative 2 would involve the construction of a wastewater treatment plant on lands within the McKenzie Property. A buffer would also be required between future development and the wastewater treatment plant, resulting in a total land requirement of 1 to 2 ha. Therefore, Alternative 2 would reduce the amount of developable land within the McKenzie Property. Alternative 1 would not result in a reduction of developable lands.

Expandability

Adequate land area is available in both alternatives for future expansion of the treatment plant beyond the 2021 capacity. Alternative 1 would involve the construction of the wastewater treatment plant on lands located outside the village boundary while Alternative 2 would involve the construction of the wastewater treatment plant on land located inside the village boundary.

Capital Cost

The capital cost of each alternative was developed for both the wastewater treatment plant and the associated collection system. Costs estimates are for infrastructure outside of new development areas, do not include land purchase, and are inclusive of 30% engineering and contingencies. At this conceptual design level, capital costs are accurate to within ±35%.

Table 3.7 presents the capital costs developed for the two alternatives.

Table 3.7 Capital Costs of Wastewater Systems for Alternative Sites

Description	Alternative #1	Alternative #2
Collection Sewers	$940,000	$950,000
Outfall Sewer	$150,000	$300,000
Pump Stations	$200,000	$200,000
Wastewater Treatment Plant	$1,100,000	$1,100,000
Total	**$2,400,000**	**$2,600,000**

3.4.5 Rationale for Selection of Preferred Alternative

Alternative 1 was selected as the preferred alternative based on the evaluation presented above, for the following reasons:

- The potential visual impacts associated with the treatment plant would only affect Trail users, and could be mitigated. By comparison, for Alternative 2, the treatment plant would be visible to a number of residences.
- Construction outside of the Village boundaries means that developable lands are not used. By comparison, for Alternative 2, developable land would be reduced by 1 to 2 ha.
- Alternative 1 has the lower capital cost of the two alternatives.
- The Environmental Component identified this Alternative as preferred from an environmental standpoint.

3.5 Implementation

Table 3.8 presents the implementation considerations for the various components of the preferred alternative.

Table 3.8 Implementation Considerations for Wastewater Servicing Alternative

Project	Implementation Considerations
Wastewater treatment plant and outfall	Construction will be required for any new development to proceed, or to service existing Village Core.
Sewer on McLaughlin Road from McColl Drive to MacDonald Street	Construction will be required to service Balmer Heights, or to service Village Core lots on McLaughlin Road north of MacDonald Street.
Sewer on MacDonald Street to treatment plant	Construction will be required to service Balmer Heights, South Slopes, McKenzie Property or any Village Core lots.
Sewer on McKenzie Street, McLaughlin Road running south from MacDonald Street and McLaughlin Road pumping station and forcemain	Construction will be required to service Village Core lots on MacKenzie, Louise, Victoria and Lorne Streets, lower end of McLaughlin Road and the McKenzie Property.
Sewers on Louise, Victoria and Lorne Streets	Construction will be required to service Louise, Victoria and Lorne Streets in Village Core.

For development in either the Balmer Heights, South Slopes or the McKenzie Property, construction of the wastewater treatment plant and certain components of the collection system will be required.

The treatment plant could be constructed in one, two or three phases. However, most of the cost for the plant and all of the outfall cost would be required in the first phase. Construction of the first phase of the wastewater treatment plant would have to be completed before occupancy of the approved building lots.

Construction of sewers on Louise, Victoria and Lorne Streets and on McLaughlin Road south of McKenzie Street is required to service the Village Core. In addition, sewers on McKenzie Street and McLaughlin Road from McKenzie Street to MacDonald Street are also required to service the Village Core, as well as the McKenzie Property. For the Village Core, construction of connections to the property line is also required.

4. WATER AND WASTEWATER SERVICING PLANS

To provide for a maximum population of 1,500 in the year 2011, water and wastewater servicing plans were developed for the Village of Inglewood. The following sections present the conceptual water and wastewater servicing plans.

4.1 Water Servicing Plan

Table 4.1 presents the description, implementation considerations and cost estimates for each project that will be required to service the 2021 population in the Village of Inglewood. Cost estimates for each project includes engineering and contingencies.

Table 4.1 Water Servicing Plan

Project	Implementation Consideration	Project Cost
Connection of MacDonald Street Watermain with Balmer Heights area	Connection will be required to service Balmer Heights and therefore will be triggered by the development of these lands.	Within new development area
Installation of a new PRV at the intersection of West Village Drive and McLaughlin Road	Installation could be completed following completion of Servicing Component of Class EA as PRV will increase existing water pressures for residents on West Village Drive	$75,000
New Permit to Take Water	New permit must be in place for more than 123 lots to be approved for development. Permit application can be completed following the completion of the Servicing Component and Class EA study.	Permit application cost – negligible
Reservoir Expansion	Expansion will be triggered with new development. Project can be completed in a single phase.	$350,000

Figure 4.1 presents the servicing projects including in the water servicing plan. The following sections present detailed information for each project.

Connection of MacDonald Street Watermain to Balmer Heights Watermain

As part of the development of the Balmer Heights subdivision, a future watermain will be constructed to connect with the existing watermain on MacDonald Street. This project will be within the Balmer Heights subdivision.

Installation of a New Pressure Reducing Valve

To improve water pressures within the West Village Drive area, a new PRV valve should be installed at the intersection of West Village Drive and McLaughlin Road. This PRV will allow residents on West Village Drive to be within the upper

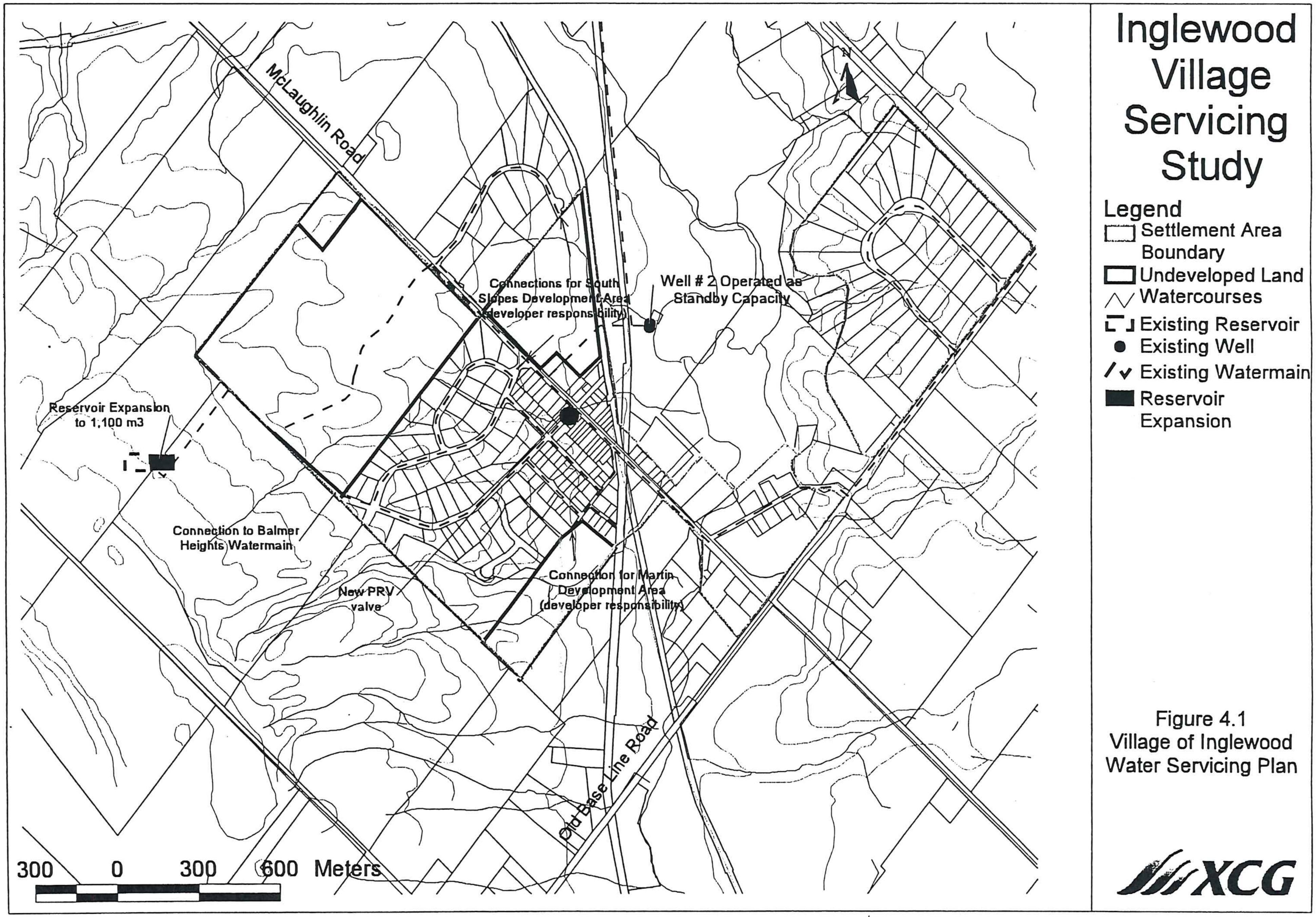


Figure 4.1
Village of Inglewood
Water Servicing Plan

pressure zone and will therefore ensure that fire flow requirements can be met in this area. This project can be completed following the completion of this study. The project cost for the new PRV has been estimated at $75,000.

New Permit Requirements

A new Permit to Take Water will be required for the existing wells to allow for a capacity expansion to 1,700,000 L/d to service growth to the year 2021. The existing wells have adequate capacity to allow for this expansion. The permit application can be completed following the completion of this study.

Reservoir Expansion

An expansion of the existing reservoir from 730 m^3 to 1,100 m^3 will be required to service future growth. The reservoir expansion will be required for new development to occur. This project can be completed in a single phase. The project cost for the reservoir expansion has been estimated at $350,000.

Monitoring

A monitoring program will need to be developed with input from the Region of Peel, CVC, Department of Fisheries (DFO) and the Ministry of the Environment (MOE).

4.2 Wastewater Servicing Plan

Table 4.2 presents the description, implementation triggers and cost estimate for each component project for the Wastewater Servicing Plan. Cost estimates for each project include engineering and contingencies.

Table 4.2 Wastewater Servicing Plan

Project	Implementation Considerations	Project Cost
Wastewater treatment plant and outfall	Construction will be required for any new development to proceed, or to service existing Village Core.	$1,300,000
Sewer on McLaughlin Road from McColl Drive to MacDonald Street	Construction will be required to service Balmer Heights, Village Core lots on McLaughlin Road north of MacDonald Street.	$164,000
Sewer on MacDonald Street to treatment plant	Construction will be required to service Balmer Heights, McKenzie Property or any Village Core lots. A section of this sewer is required to service South Slopes.	$131,000
Sewer on McKenzie Street, McLaughlin Road running north from McKenzie Street to MacDonald Street and McLaughlin Road pumping station and forcemain	Construction will be required to service Village Core lots on MacKenzie, Louise, Victoria and Lorne Streets, lower end of McLaughlin Road and McKenzie Property.	$326,000
Sewers on Louise, Victoria and Lorne Streets	Construction will be required to service Louise, Victoria and Lorne Streets in Village Core.	$414,000
Total Cost		**$2,300,000**

Figure 4.2 presents the location of the wastewater treatment plant while Figure 4.3 presents the locations of the collection system projects. The following sections describe the projects.

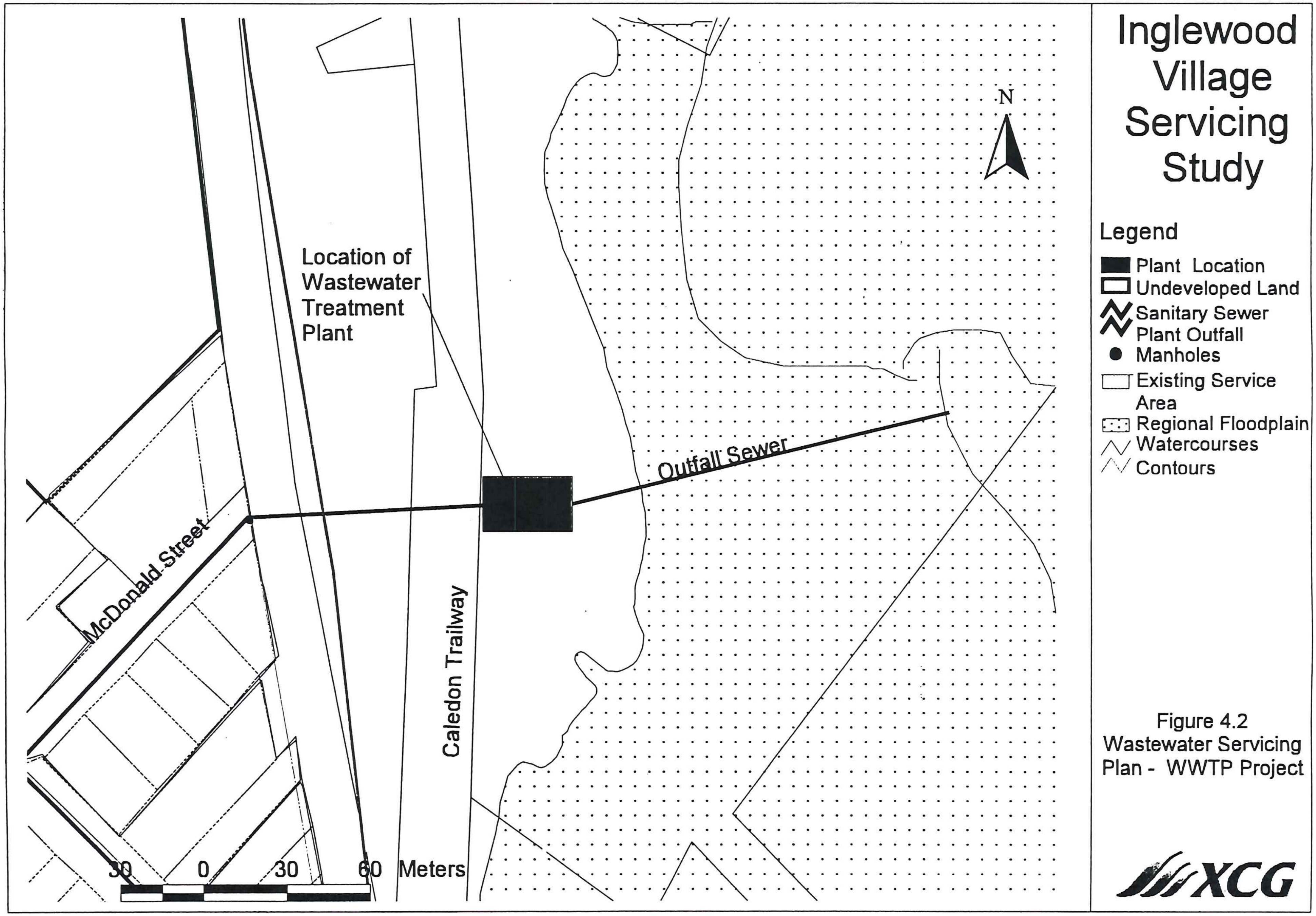
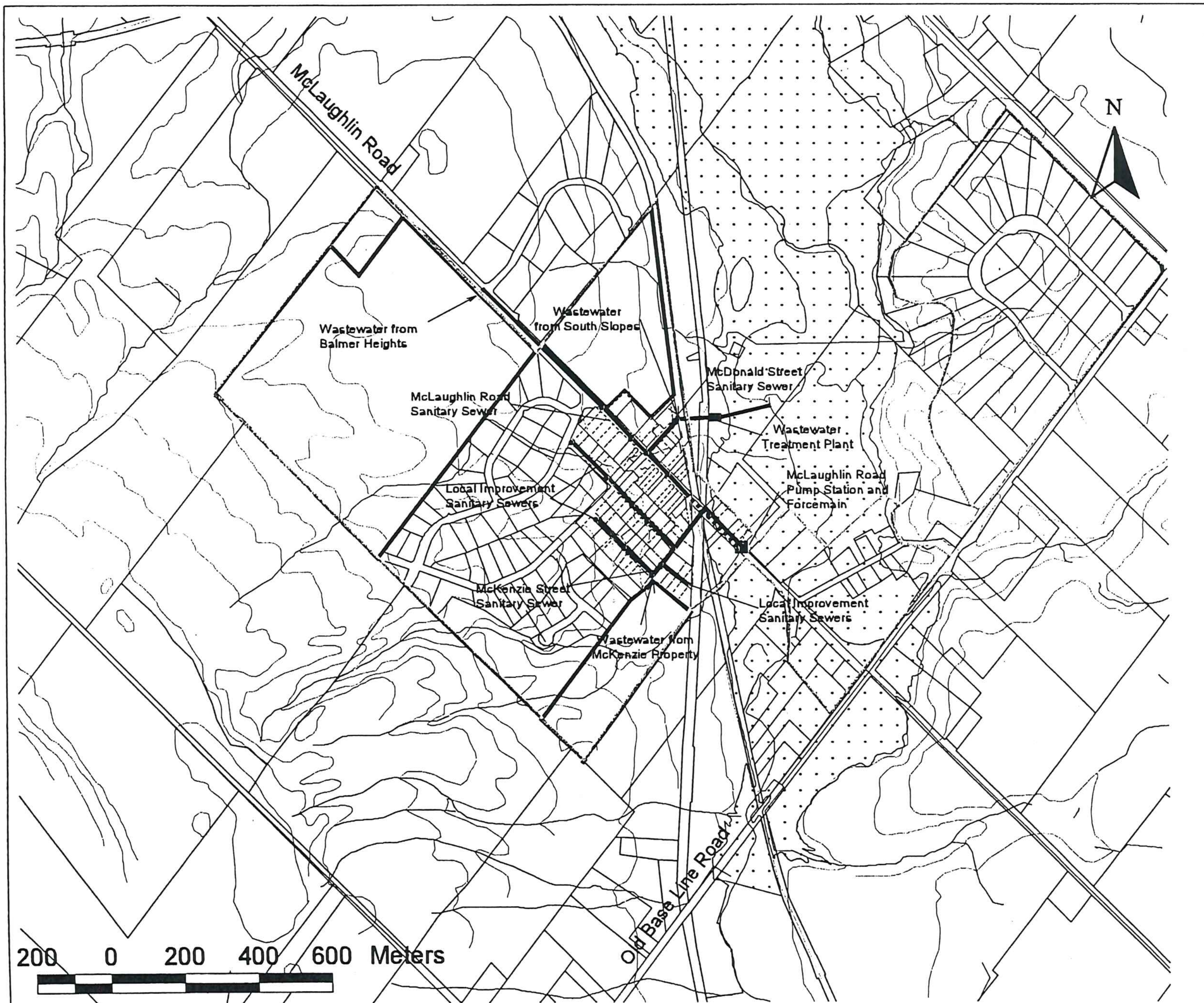


Figure 4.2
Wastewater Servicing Plan - WWTP Project



Inglewood Village Servicing Study

Legend

- Settlement Area Boundary
- Plant Location
- Undeveloped Land
- Sanitary Forcemain
- Sanitary Sewer
- Plant Outfall
- Manholes
- Pump Station
- Existing Service Area
- Regional Floodplain
- Watercourses
- Contours

Figure 4.3
Wastewater Servicing Plan - Collection System Projects

XCG

Wastewater Treatment Plant

A treatment plant, with a capacity of 390 m^3/d, will provide secondary biological treatment, nitrification, tertiary phosphorus removal, and ultraviolet disinfection. The facility can be constructed in one, two or three phases; however, the outfall and most of the facilities will be required in the first phase.

A location adjacent to the Caledon Trailway has been selected for the wastewater treatment plant.

The project also includes the construction of a plant outfall to the Credit River. The outfall will extend approximately 170 m to the Credit River.

The estimated capital cost for the treatment plant and outfall is $1.3 million.

McLaughlin Road and MacDonald Street Sewer

The McLaughlin Road Sewer will extend from McColl Drive to MacDonald Street. It will convey flows from the Balmer Heights area and from existing residences on McLaughlin Road. The MacDonald Street Sewer will extend from the corner of McLaughlin Road and MacDonald Street to the new wastewater treatment plant located adjacent to the Caledon Trailway. The sewer will convey flow from all three new development areas and the 95 lots within the existing Village Core.

This project will need to be completed to serve new development from Balmer Heights, South Slopes, and the McKenzie Property as well as to service the Village Core.

McKenzie Street and McLaughlin Road Sewers

The McKenzie Street Sewer will extend from Victoria Street to McLaughlin Road and will convey flow from the McKenzie Property and existing residences on Lorne, Victoria, Louise and McKenzie Streets.

There will be two sections of sanitary sewer on McLaughlin Road. One section will extend from McKenzie Street south to the McLaughlin Road Pump Station. The other section will extend from north of McKenzie Street to MacDonald Street. Both sections will convey flow from the McKenzie Property and existing residences on Lorne, Victoria, Louise, McKenzie Streets and McLaughlin Road.

Louise Street, Victoria Street, Lorne Street Sewers

These sewers will be required to service the existing Village Core. The Louise Street sewer will convey flows from 8 lots on Louise Street. The Victoria Street sewer will convey flows from 14 lots on Victoria Street. The Lorne Street sewer will convey flows from 23 lots on Lorne Street.

McLaughlin Road Pump Station and Forcemain

McLaughlin Street Pump Station and forcemain will pump flow from existing residents on McLaughlin Road, McKenzie Street, Lorne Street, Victoria Street and Louise Street as well as flows from new development within the McKenzie Property. This project will be required to service new development within the McKenzie Property, and most of the Village Core.

Monitoring

A monitoring program will need to be developed with input from The Region of Peel, CVC, Department of Fisheries (DFO) and the Ministry of the Environment (MOE).

4.3 Summary

Water and wastewater servicing plans were developed for the Village of Inglewood to provide for a maximum population of 1,500 persons in the year 2021. The Water Servicing Plan consists of four projects including the connection of the MacDonald Street Watermain with the Balmer Heights area, a new Permit to Take Water, the installation of a new pressure reducing valve at the intersection of West Village Drive and McLaughlin Road, and an expansion to the existing reservoir.

The Wastewater Servicing Plan consists of five projects including a new wastewater treatment plant, new sewers on McLaughlin Road, McDonald Street, McKenzie Street, Louise Street, Victoria Street and Lorne Street. These projects have been identified to provide wastewater servicing to a maximum population of 1,500 persons in the year 2021.

5. REFERENCES

1. CVC, Town of Caldeon, Region of peel, November 1998. Inglewood Village Study, Phase I Report.
2. CVC, Town of Caldeon, Region of Peel, March 1999. Inglewood Village Study Phase II Report, Planning and Servicing Alternatives.
3. KMK Consultants Limited, march 1997. Caledon Water Distribution Systems Interconnection Feasibility Study.
4. Municipal Engineers Association, June 1994. Class Environmental Assessments for Municipal Water and Wastewater Projects.
5. XCG Consultants Ltd., April 1999. Inglewood Village Septic Tank Survey.

APPENDIX A
WATER DISTRIBUTION MODELLING RESULTS

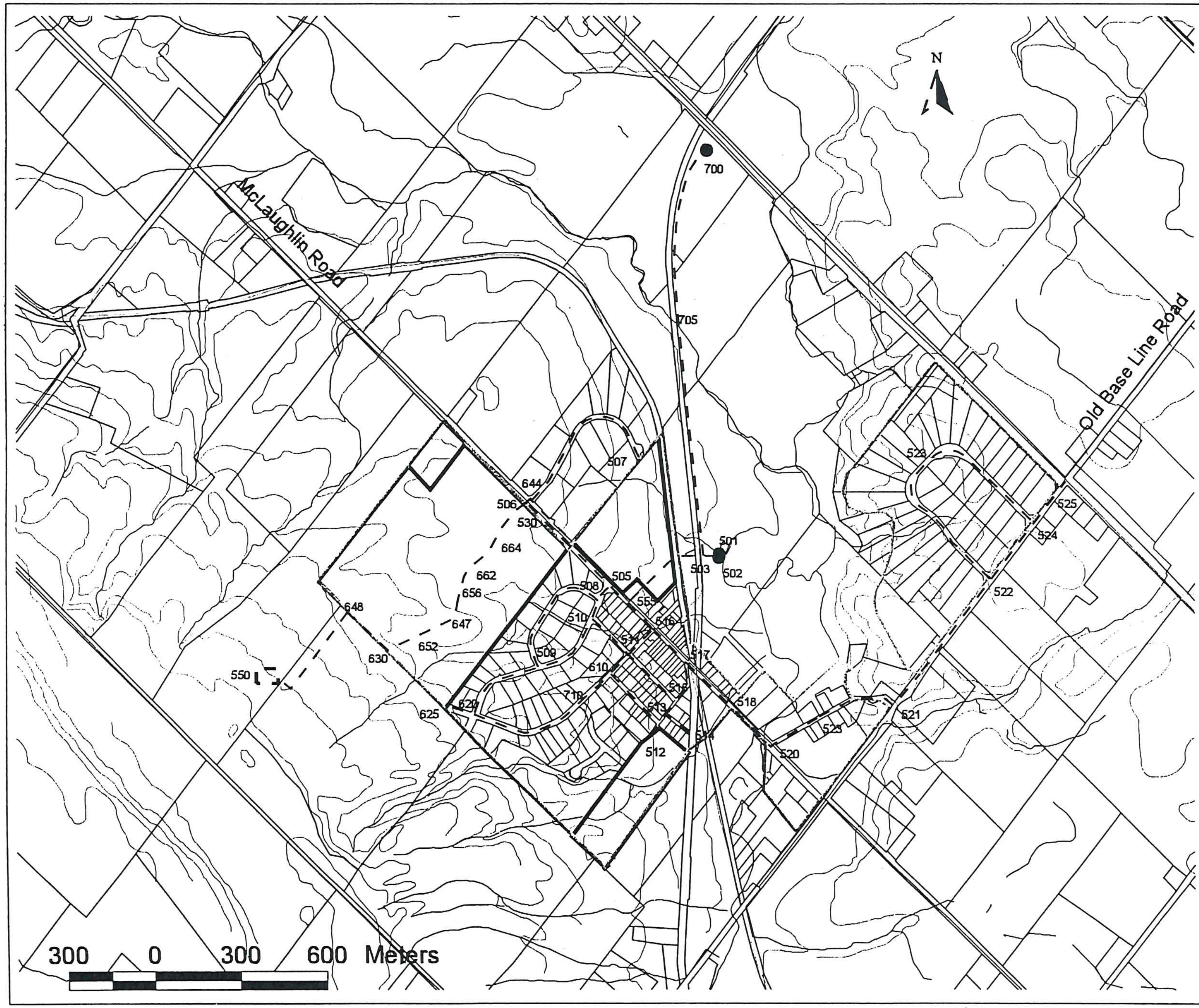
Appendix A
Inglewood Village Servicing Study
Existing Water Distribution System Modelling Results

		Existing Maximum Day Results			Existing Maximum Day with new PRV			Existing Maximum Day with Fire Flow of 40L/s			Existing Maximum Day with new PRV and Fire Flow of 40L/s		
Node	Elevation m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m
700	273.00	0.00	829.12	357.65	0.00	829.12	357.65	0.00	579.84	332.20	0.00	579.84	332.20
705	273.92	0.00	820.11	357.65	0.00	820.11	357.65	0.00	570.84	332.20	0.00	570.84	332.20
501	273.92	0.00	820.11	357.65	0.00	820.11	357.65	0.00	570.84	332.20	0.00	570.84	332.20
503	275.00	0.00	809.53	357.65	0.00	809.53	357.65	0.00	560.26	332.20	0.00	560.26	332.20
555	287.00	0.00	692.02	357.65	0.00	692.02	357.65	0.00	442.75	332.20	0.00	442.75	332.20
504	287.00	0.50	493.55	337.38	0.50	493.40	337.37	0.50	97.55	296.95	0.50	97.55	296.95
516	280.00	0.50	561.08	337.28	0.50	560.73	337.24	0.50	139.50	294.23	0.50	139.50	294.23
517	277.00	1.08	589.95	337.23	1.08	589.56	337.19	1.08	154.96	292.81	1.08	154.96	292.81
518	270.00	0.27	658.41	337.22	0.27	658.02	337.18	0.27	223.43	292.80	0.27	223.43	292.80
512	282.50	0.50	536.16	337.23	0.50	535.76	337.19	40.50	72.39	289.88	40.50	72.39	289.88
513	280.00	0.00	560.65	337.23	0.00	560.25	337.19	0.00	107.17	290.93	0.00	107.17	290.93
514	276.00	0.36	599.81	337.23	0.36	599.41	337.19	0.36	146.34	290.93	0.36	146.34	290.93
515	278.00	0.28	580.24	337.24	0.28	579.84	337.19	0.28	137.21	292.00	0.28	137.21	292.00
511	288.00	0.77	482.63	337.27	0.77	482.22	337.23	0.77	56.69	293.78	0.77	56.69	293.78
610	289.50	1.85	467.66	337.24	1.85	467.25	337.20	1.85	29.57	292.51	1.85	29.57	292.51
510	295.00	0.59	414.85	337.35	0.59	414.28	337.29	0.59	14.37	296.45	0.59	14.37	296.45
508	294.50	0.63	420.21	337.39	0.63	419.56	337.33	0.63	29.67	297.52	0.63	29.67	297.52
505	292.00	0.00	445.50	337.48	0.00	445.50	337.48	0.00	68.50	298.98	0.00	68.50	298.98
509	311.00	0.68	258.22	337.35	0.68	257.63	337.29	0.68	-138.99	296.79	0.68	-138.99	296.79
530	306.00	0.00	324.70	339.14	0.00	324.70	339.14	0.00	249.52	331.46	0.00	249.52	331.46
506	306.00	0.05	505.96	357.65	0.05	505.96	357.65	0.05	256.68	332.20	0.05	256.68	332.20
630	322.80	0.00	348.26	358.35	0.00	348.26	358.35	0.00	214.73	344.71	0.00	214.73	344.71
644	306.00	0.00	505.95	357.65	0.00	505.95	357.65	0.00	256.68	332.20	0.00	256.68	332.20
507	287.00	0.54	691.90	357.64	0.54	691.90	357.64	0.54	442.63	332.18	0.54	442.63	332.18
550	357.30	0.00	18.29	359.15	0.00	18.29	359.15	0.00	18.29	359.15	0.00	18.29	359.15
520	269.00	0.41	668.08	337.21	0.41	667.70	337.17	0.41	233.10	292.79	0.41	233.10	292.79
521	282.00	0.00	540.62	337.19	0.00	540.24	337.15	0.00	105.64	292.77	0.00	105.64	292.77
522	302.00	0.14	344.60	337.17	0.14	344.22	337.13	0.14	-90.38	292.76	0.14	-90.38	292.76
523	306.00	0.10	305.34	337.16	0.10	304.96	337.12	0.10	-129.64	292.75	0.10	-129.64	292.75
524	305.00	0.95	315.08	337.16	0.95	314.69	337.12	0.95	-119.91	292.74	0.95	-119.91	292.74
525	305.00	0.00	315.05	337.16	0.00	314.67	337.12	0.00	-119.93	292.74	0.00	-119.93	292.74
526	305.00	0.45	315.04	337.15	0.45	314.66	337.12	0.45	-119.94	292.74	0.45	-119.94	292.74
664	305.84	0.00	507.53	357.65	0.00	507.53	357.65	0.00	258.42	332.21	0.00	258.42	332.21
662	307.00	0.00	497.13	357.75	0.00	497.13	357.75	0.00	264.26	333.97	0.00	264.26	333.97
656	301.70	0.00	550.31	357.88	0.00	550.31	357.88	0.00	339.11	336.31	0.00	339.11	336.31
652	315.90	0.00	414.66	358.23	0.00	414.66	358.23	0.00	261.19	342.56	0.00	261.19	342.56
648	331.00	0.00	271.21	358.68	0.00	271.21	358.68	0.00	192.63	350.65	0.00	192.63	350.65

Appendix A
Inglewood Village Servicing Study
Future Water Distribution System Modelling Results

		Future Maximum Day Results			Future Maximum Day Results with new PRV			Future Maximum Day Results with new PRV and Fire Flow of 40L/s		
Node	Elevation m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m	Demand l/s	Pressure kPa	HGL m
700	273.00	0.00	840.78	358.84	0.00	845.24	359.30	0.00	633.59	337.68
705	273.92	0.00	831.77	358.84	0.00	836.23	359.30	0.00	624.58	337.68
501	273.92	0.00	831.77	358.84	0.00	836.23	359.30	0.00	624.58	337.68
503	275.00	0.00	821.19	358.84	0.00	825.65	359.30	0.00	614.00	337.68
555	287.00	0.00	703.68	358.84	0.00	708.14	359.30	0.00	496.49	337.68
504	287.00	0.50	568.61	345.05	0.50	593.60	347.60	0.50	411.93	329.05
516	280.00	2.24	637.39	345.07	2.24	645.19	345.87	2.24	458.40	326.79
517	277.00	1.08	666.71	345.07	1.08	666.85	345.08	1.08	474.61	325.45
518	270.00	0.27	735.18	345.06	0.27	735.31	345.07	0.27	543.07	325.44
512	282.50	1.11	613.41	345.12	1.11	613.23	345.10	41.11	392.28	322.54
513	280.00	0.00	637.89	345.12	0.00	637.48	345.08	0.00	427.22	323.61
514	276.00	0.36	677.05	345.12	0.36	676.65	345.08	0.36	466.38	323.61
515	278.00	0.28	657.47	345.12	0.28	657.10	345.08	0.28	457.40	324.69
511	288.00	0.77	560.48	345.22	0.77	559.66	345.13	0.77	378.36	326.62
610	289.50	1.85	545.22	345.16	1.85	544.71	345.11	1.85	350.70	325.30
510	295.00	0.59	499.17	345.96	0.59	488.76	344.89	0.59	346.35	330.35
508	294.50	0.63	503.03	345.85	0.63	481.25	343.63	0.63	355.09	330.74
505	292.00	0.00	519.56	345.04	0.00	554.74	348.63	0.00	379.56	330.74
509	311.00	0.68	376.15	349.39	0.68	365.86	348.34	0.68	222.66	333.72
530	306.00	0.00	324.64	339.14	0.00	346.74	341.39	0.00	308.95	337.53
506	306.00	0.05	517.63	358.84	0.05	522.10	359.30	0.05	310.45	337.69
630	322.80	0.00	335.80	357.07	0.00	335.80	357.07	0.00	176.26	340.78
644	306.00	0.00	517.61	358.84	0.00	522.08	359.30	0.00	310.43	337.68
507	287.00	2.28	701.98	358.67	2.28	706.44	359.12	2.28	494.79	337.51
550	357.30	0.00	18.29	359.15	0.00	18.29	359.15	0.00	18.29	359.15
520	269.00	0.41	744.85	345.04	0.41	744.98	345.06	0.41	552.74	325.43
521	282.00	0.00	617.39	345.03	0.00	617.52	345.04	0.00	425.28	325.41
522	302.00	0.14	421.37	345.01	0.14	421.50	345.03	0.14	229.26	325.40
523	306.00	0.10	382.11	345.00	0.10	382.24	345.02	0.10	190.00	325.39
524	305.00	0.95	391.84	345.00	0.95	391.98	345.01	0.95	199.74	325.38
525	305.00	0.00	391.82	344.99	0.00	391.95	345.01	0.00	199.71	325.38
526	305.00	0.45	391.81	344.99	0.45	391.95	345.01	0.45	199.70	325.38
664	305.84	0.00	519.18	358.84	0.00	523.64	359.29	0.00	312.06	337.69
662	307.00	0.00	505.39	358.59	0.00	509.22	358.98	0.00	304.96	338.12
656	301.70	0.00	554.05	358.26	0.00	557.04	358.57	0.00	362.54	338.70
652	315.90	0.00	406.35	357.38	0.00	407.12	357.46	0.00	238.61	340.25
648	331.00	4.92	261.42	357.68	4.92	261.42	357.68	4.92	163.16	347.64
625	311	0	413.75	353.23	0	408.61	352.71	0	257.24	337.25

Inglewood Village Servicing Study



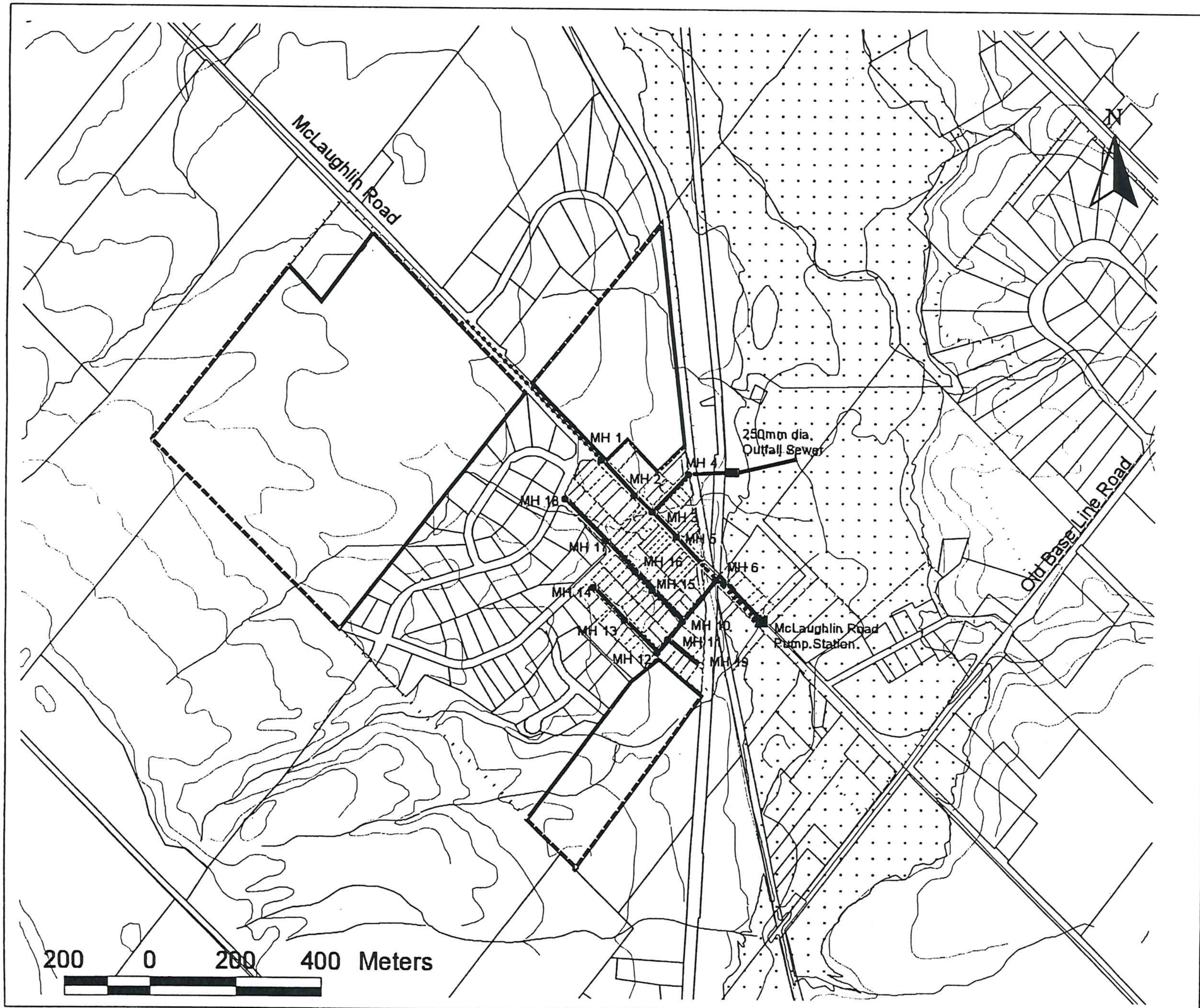
Legend

- Settlement Area Boundary
- Undeveloped Land
- Watercourses
- Existing Reservoir
- Existing Well
- Existing Watermain
- 525 Watermain Node

Figure A.1
Village of Inglewood
Existing Water
Distribution System

XCG

Appendix B
Information for Wastewater Alternatives



Inglewood Village Servicing Study

Legend

- Settlement Area Boundary
- Plant Location
- Undeveloped Land
- Sanitary Forcemain
- Sanitary Sewer
- Plant Outfall
- Manholes
- Pump Station
- Existing Service Area
- Regional Floodplain
- Watercourses
- Contours

Figure B.1
Alternative #1

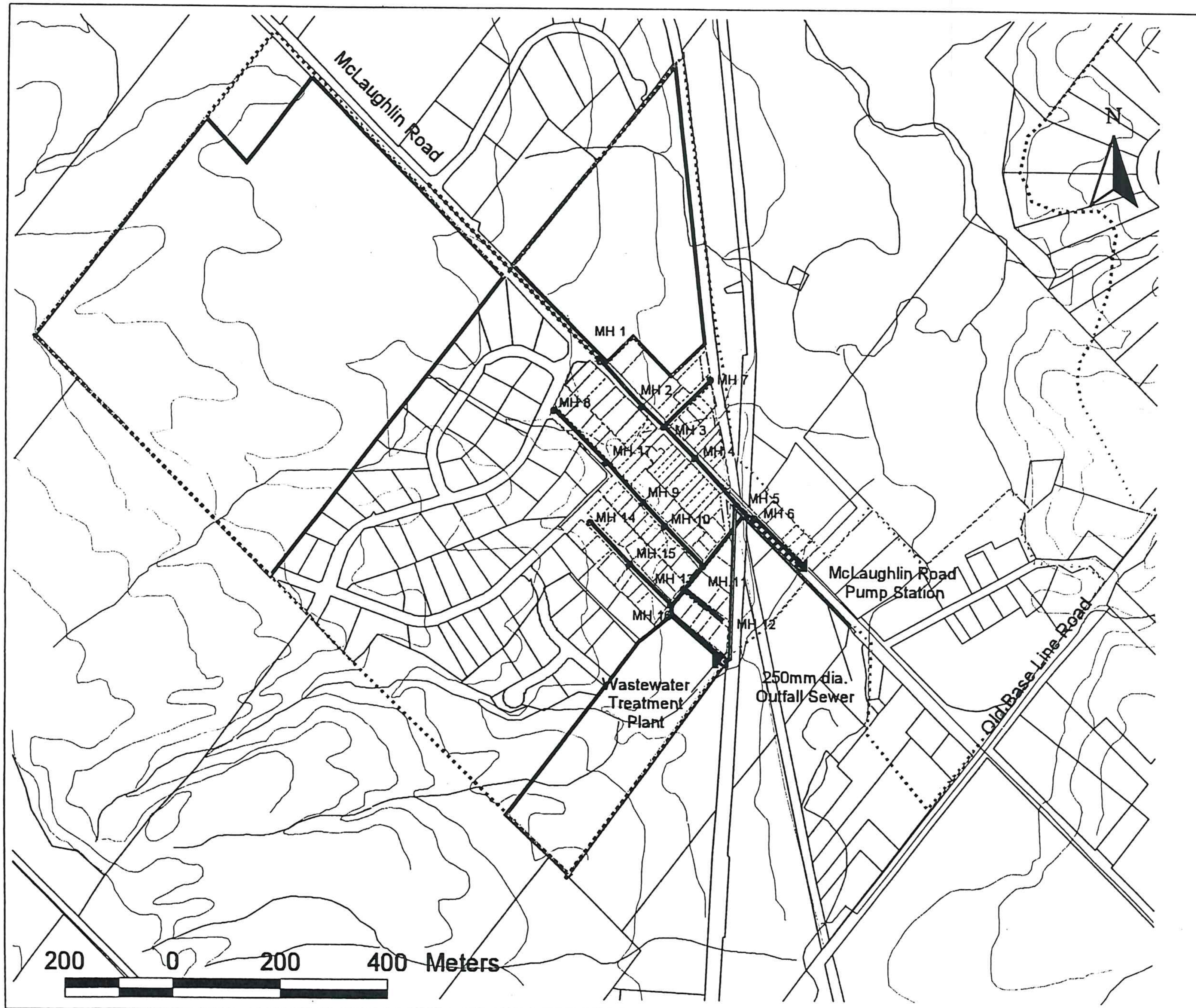
XCG

Alternative #1 - Plant located adjacent to Trailway, existing downtown conveyed by gravity sewer to McLaughlin Pump Station

Street	From Manhole	To Manhole	Population	umulativ Populatio	Area (ha)	umulativ Area	Peak Factor	Q avg (l/s)	Infiltration (l/s)	Qp (l/s)	Diameter (mm)	Upstream Ground Elevation	ownstream Ground Elevation	Upstrea Invert	Downstream Invert	Pipe Length	Slope (%)	Capacity (l/s)	Max Velocity (m/s)	Actual Velocity (m/s)
Louise	MH 19	MH 11	25	25	0.94	0.94	4.37	0.09	0.19	0.57	250	275.5	278	272.75	270.50	94	2.39	91.97	1.87	0.75
Victoria	MH 14	MH 13	22	22	1.35	1.35	4.37	0.08	0.27	0.61	250	287.9	285.4	285.15	282.65	86	2.91	101.36	2.07	0.83
	MH 13	MH 12	22	44	0.91	2.27	4.33	0.15	0.45	1.12	250	285.4	279	282.65	274.40	120	6.88	155.87	3.18	1.27
Lorne	MH 18	MH 17	19	19	0.88	0.88	4.38	0.07	0.18	0.47	250	290	284.2	287.25	281.45	120	4.83	130.70	2.66	1.07
	MH 17	MH 16	16	35	0.87	1.75	4.34	0.12	0.35	0.88	250	284.2	281.4	281.45	278.65	86	3.26	107.27	2.19	0.87
	MH 16	MH 15	19	54	0.55	2.30	4.31	0.19	0.46	1.27	250	281.4	280	278.65	276.50	86	2.50	94.00	1.92	0.77
	MH 15	MH 10	19	72	0.62	2.92	4.28	0.25	0.58	1.67	250	280	277	276.50	269.70	94	7.23	159.89	3.26	1.30
McKenzie	MH 12	MH 11	161	161	8.99	8.99	4.18	0.56	1.80	4.16	250	279	278	274.40	270.50	47	8.30	171.25	3.49	1.92
	MH 11	MH 10	32	193	1.28	10.27	4.15	0.67	2.05	4.86	250	278	277	270.50	269.70	60	1.33	68.65	1.40	0.77
	MH 10	MH 7	91	284	3.56	13.83	4.09	1.00	2.77	6.84	250	277	275.5	269.70	264.25	120	4.54	126.69	2.58	1.81
McLaughli	MH 7	PS	19	303	0.91	14.74	4.08	1.06	2.95	7.28	250	267	266.5	264.25	261.90	99	2.37	91.59	1.87	0.75
McLaughli	MH 6	MH 5	28	331	0.96	15.71	4.06	1.16	3.14	7.85	250	275.5	278.7	272.75	272.20	73	0.75	51.60	1.05	0.79
	MH 5	MH 3	25	356	0.94	16.65	4.05	1.25	3.33	8.38	250	278.7	278	272.20	271.68	86	0.60	46.23	0.94	0.75
	MH 1	MH 2	734	734	40.04	40.04	3.88	2.57	8.01	18.00	250	287	282.1	284.25	276.30	120	6.62	153.01	3.12	2.18
	MH 2	MH 3	16	750	0.68	40.72	3.88	2.33	8.14	18.33	250	282.1	278	276.30	271.68	56	8.25	170.75	3.48	2.44
McDonald	MH 3	MH 4	1128	1128	58.29	58.29	3.77	3.95	11.66	26.55	250	278	276	271.68	271.08	120	0.50	42.04	0.86	0.94
	MH 4	Plant	198	1326	9.24	67.53	3.72	4.65	13.51	30.78	250	276	272	271.08	270.45	120	0.52	43.07	0.88	0.79
	Total Length															1587				
	Plant Outfall		0	1326	0.00	67.53	3.72	4.65	13.51	30.78	250	272	266	268.23	266.00	65	3.43	110.11	2.24	2.02

Pump Stations	Qp (l/s)	Head Diff. (m)	Power (kW)
McLaughlin	7.3	10.85	0.77

Forcemain	Length (m)	Qp (l/s)	Dia. (mm)	vmax (m/s)
McLaughlin	200	6.3	100.00	0.80



Inglewood Village Servicing Study

Legend
- Settlement Area Boundary
- Plant Location
- Undeveloped Land
- Sanitary Forcemain
- Sanitary Sewer
- Plant Outfall
- Manholes
- Pump Station
- Existing Service Area
- Regional Floodplain
- Watercourses
- Contours

Figure B.2
Alternative #2

XCG

Alternative #2 - Plant located in Martin Development Area with pump station on McLaughlin

Street	From Manhole	To Manhole	Population	Cumulativ Population	Area (ha)	Cumulativ Area	Peak Factor	Q avg (l/s)	Infiltration (l/s)	Qp (l/s)	Diameter (mm)	Upstream Ground Elevation	Downstrea Ground Elevation	Proposed Design Upstream Invert	Downstream Invert	Pipe Length (m)	Slope (%)	Capacity (l/s)	Velocity (m/s)	Actual Velocity (m/s)
Victoria	MH 14	MH 15	25	25	1.49	1.49	4.37	0.09	0.30	0.68	250	286.7	283.0	283.9	278.0	120	4.92	131.82	2.69	1.07
	MH 15	MH 16	16	41	0.73	2.21	4.33	0.14	0.44	1.06	250	283.0	279.5	278.0	270.7	90	8.11	169.31	3.45	1.38
Louise	MH 12	MH 13	25	25	0.94	0.94	4.37	0.09	0.19	0.57	250	279.0	279.3	276.2	271.3	90	5.44	138.71	2.83	1.13
Lorne	MH 8	MH 17	13	13	0.54	0.54	4.40	0.04	0.11	0.30	250	291.1	288.8	288.3	284.0	81	5.31	136.97	2.79	1.12
	MH 17	MH 9	9	22	0.50	1.05	4.37	0.08	0.21	0.55	250	288.8	280.0	284.0	277.2	86	7.91	167.16	3.41	1.36
	MH 9	MH 10	25	47	1.10	2.15	4.32	0.17	0.43	1.14	250	280.0	279.6	277.2	275.9	107	1.26	66.78	1.36	0.75
	MH 10	MH 11	22	69	0.77	2.92	4.28	0.24	0.58	1.62	250	279.6	279.1	275.9	271.6	120	3.54	111.88	2.28	1.25
McDonald	MH 7	MH 3	22	22	0.81	0.81	4.37	0.08	0.16	0.50	250	277.0	279.4	274.2	272.8	112	1.25	66.47	1.35	0.75
McLaughlin	MH 1	MH 2	665	665	48.94	48.94	3.91	2.33	9.79	18.89	250	288.3	283.3	285.5	279.5	120	5.00	132.93	2.71	1.08
	MH 2	MH 3	19	684	1.08	50.02	3.90	2.40	10.00	19.35	250	283.3	279.4	279.9	272.8	86	8.26	170.81	3.48	1.39
	MH 3	MH 4	63	747	1.41	52.24	3.88	2.62	10.45	20.59	250	279.4	278.4	272.8	272.3	120	0.42	38.37	0.78	0.78
	MH 4	MH 5	19	765	0.81	53.05	3.87	2.68	10.61	21.00	250	278.4	277.6	272.3	271.9	94	0.38	36.79	0.75	0.75
	MH 6	Pump	19	19	0.88	0.88	4.38	0.07	0.18	0.47	250	277.6	276.3	274.8	271.9	120	2.42	92.42	1.88	0.75
McKenzie	MH 5	MH 11	16	800	0.63	54.56	3.86	2.80	10.91	21.74	250	277.6	279.1	271.9	271.6	43	0.70	49.66	1.01	1.01
	MH 11	MH 13	9	810	0.46	57.94	3.86	2.84	11.59	22.53	250	279.1	279.3	271.6	271.3	56	0.54	43.51	0.89	0.89
	MH 13	MH 16	25	835	0.00	58.88	3.85	2.93	11.78	23.04	250	279.3	279.5	271.3	270.7	120	0.50	42.04	0.86	0.86
	MH 16	plant	88	923	6.72	67.81	3.82	3.23	13.56	25.93	250	279.5	280.0	270.7	270.5	43	0.47	40.54	0.83	0.83
	Total Length (m) =															1608				
	Outfall		0	923	6.72	67.81	3.82	3.23	13.56	25.93	250	280.0	270.0	270.5	267.2	595	0.55	44.27	0.90	0.90

Pump Stations	Qp (l/s)	Head Diff. (m)	Power (kW)
McLaughlin	0	10.3	0.05

Assumptions: Water Level in plant is at 280.5 (.5m above ground elevation of 280).

Forcemain	Length (m)	Qp (l/s)	Dia. (mm)	vmax (m/s)
McLaughlin	355	0.47	100.00	0.06

MOE PHOSPHORUS REPORT

Hutchinson
Environmental Sciences Ltd.

Phosphorus Budget Tool in Support of Sustainable Development for the Lake Simcoe Watershed

Prepared By: **Hutchinson Environmental Sciences Ltd.**
Greenland International Consulting Ltd. and
Stoneleigh Associates Inc.

Prepared For: **Ontario Ministry of the Environment**

Date: March 30, 2012

Version 2 Report

3-1 Taylor Road, Bracebridge, ON P1L 1S6 ph: 705 645 0021

Executive Summary

Lake Simcoe is enriched by nutrients from land use activities in its watershed and has, for many years, been the focus of efforts to protect and restore its water quality. The *Lake Simcoe Protection Act* (LSPA) was passed by the Ontario legislature in 2008 and required establishment of the Lake Simcoe Protection Plan (LSPP). The LSPP was approved in 2009 and included a series of policies that were to be implemented to restore water quality and other ecological attributes of the lake. This document is prepared in response to Policy 4.8e of the LSPP, which states that:

> *"An application for major development shall be accompanied by a stormwater management plan that demonstrates...*
>
> > *e. through an evaluation of anticipated changes in phosphorus loadings between pre-development and post-development, how the loadings shall be minimized."*

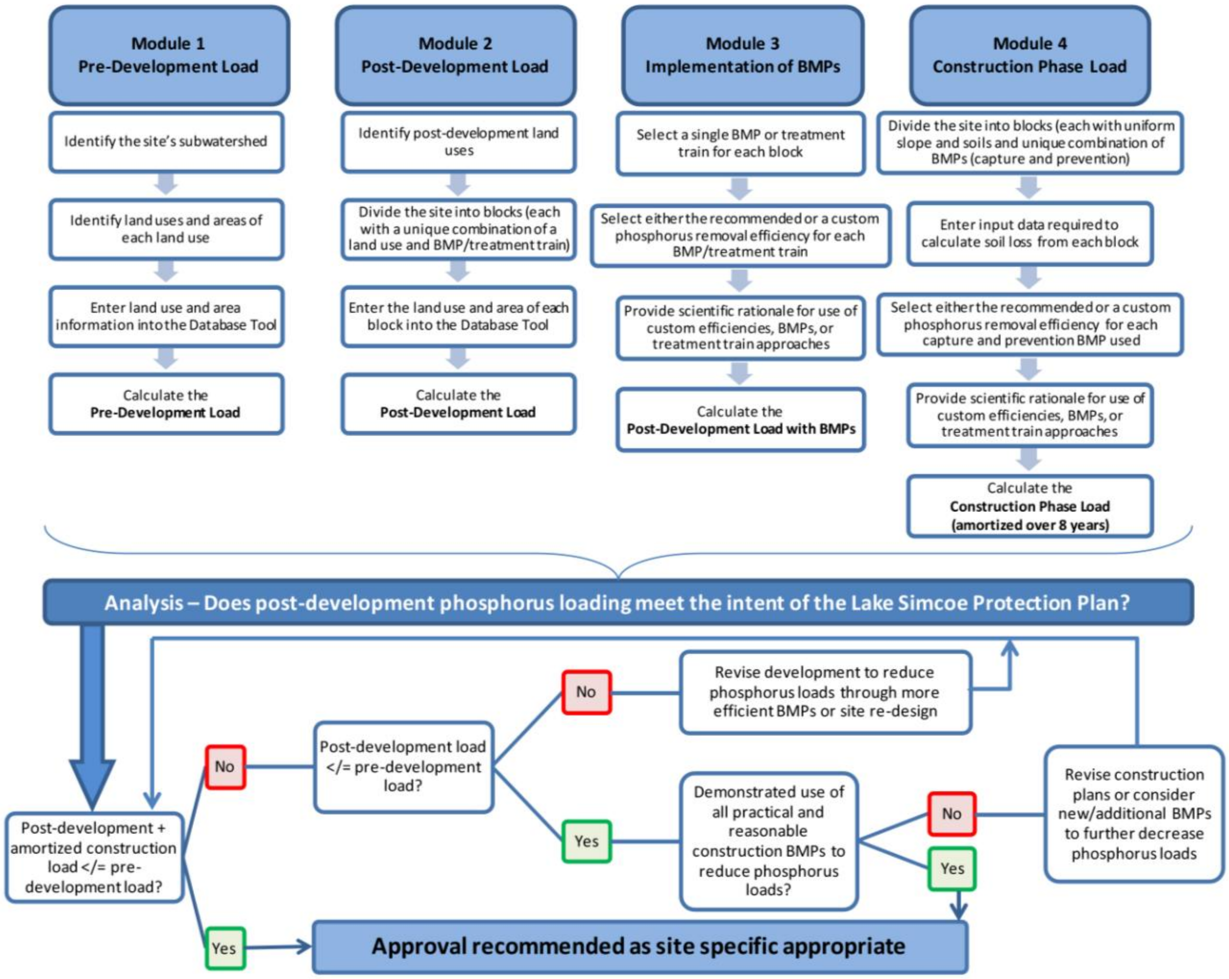
The intent of Policy 4.8e is that plans for new development in the Lake Simcoe watershed adopt Best Management Practices (BMPs), Low Impact Development (LID) techniques and innovative stormwater management techniques to achieve sustainable development practices that will reduce the phosphorus loading from new urban development. In practice, Policy 4.8e is interpreted as a requirement that post development loadings be reduced from pre-development loadings on any major development site, in order to achieve overall reductions in loadings to the lake. This interpretation is in line with Strategic Direction #3 in the Phosphorus Reduction Strategy, which requires a move to "no net increase" of phosphorus for new development in the Lake Simcoe watershed.

Policy 4.8e requires standardized methods to estimate and compare pre- and post-development phosphorus loadings with implementation of BMPs and LID techniques. In addition, the Ontario Ministry of the Environment (MOE) is recommending that municipalities require phosphorus loading from the construction phase of new development be minimized in support of other related designated policies in the LSPP, (i.e., 4.20 and 'have regard' for policy 4.21), with the objective that "post-development load + construction load" be less than "pre-development load".

The MOE retained Hutchinson Environmental Sciences Ltd. (HESL), Greenland International Consulting Ltd. and Stoneleigh Associates to develop the *Phosphorus Budget Guidance Tool to Guide New Development in the Lake Simcoe Watershed*. This "Tool" provides a transparent, technically sound approach to estimate phosphorus loading from stormwater runoff in the pre-, post- and construction phases of new development in the Lake Simcoe watershed. The Tool does not address atmospheric sources of phosphorus in dust generated from land use practices, as the science is not yet advanced to the point where estimates can be made. It does account for atmospheric deposition of phosphorus to open water and atmospheric deposition to land surfaces is included in the export coefficients for various land use practices.

The Tool couples an "Export Coefficient Modelling" approach with BMPs for stormwater management in the post-development and construction phases. It uses estimates of phosphorus export that were developed for specific land uses using the most recent and site specific estimates available. These are coupled to standard estimates of phosphorus reduction efficiencies for BMPs and LID techniques for stormwater management that were summarized from an extensive review of case studies and technical literature to estimate post-development phosphorus load after mitigation. Construction phase loadings are derived as a function of the area of land that is exposed during construction and soil loss, with adjustments for use of BMPs. These calculations and export coefficients are coded into four separate modules that consider sediment and nutrient loss, as summarized in Figure 1 of the report and reproduced below.

Figure 1. Schematic of modular approach to phosphorus guidance.



Module 1 Estimates pre-development phosphorus loads for standardized, subwatershed-specific land uses contained within the study site immediately prior to development. The guidance is, for the most part, specific to each subwatershed, in recognition that the Lake Simcoe watershed is made up of different subwatersheds and that export from each will vary in response to precipitation patterns, soils and slope. Land use categories are derived from those used in Berger (2010), as shown in Table 2 of the report and reproduced below. Subwatershed-specific export coefficients were developed for individual land uses using Berger (2010) as the basis, but were modified to address unexplained variance in export between land uses and subwatersheds in the Lake Simcoe basin.

Table 2. Land-Use Specific Phosphorus Export Coefficients (kg/ha/yr) for Lake Simcoe Subwatersheds

Subwatershed	Phosphorus Export (kg/ha/yr)											
	Cropland	Hay-Pasture	Sod Farm/Golf Course	High Intensity Development		Low Intensity Development	Quarry	Unpaved Road	Forest	Transition	Wetland	Open Water
				Commercial /Industrial	Residential							
Monitored Subwatersheds												
Beaver River	0.22	0.04	0.01	1.82	1.32	0.19	0.06	0.83	0.02	0.04	0.02	0.26
Black River	0.23	0.08	0.02	1.82	1.32	0.17	0.15	0.83	0.05	0.06	0.04	0.26
East Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hawkestone Creek	0.19	0.10	0.06	1.82	1.32	0.09	0.10	0.83	0.03	0.04	0.03	0.26
Lovers Creek	0.16	0.07	0.17	1.82	1.32	0.07	0.06	0.83	0.06	0.06	0.05	0.26
Pefferlaw/Uxbridge Brook	0.11	0.06	0.02	1.82	1.32	0.13	0.04	0.83	0.03	0.04	0.04	0.26
Whites Creek	0.23	0.10	0.42	1.82	1.32	0.15	0.08	0.83	0.10	0.11	0.09	0.26
Unmonitored Subwatersheds												
Barrie Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
GeorginaCreeks	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hewitts Creek	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Innisfil Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Maskinonge River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Oro Creeks North	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Oro Creeks South	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Ramara Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Talbot/Upper Talbot River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
West Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26

Module 2 – Estimates post-development phosphorus loads that are representative of the proposed changes in land use for the study site using the same data sources used in Module 1, but accounting for the change in land use that will occur with development.

Module 3 – Estimates efficiencies attributed to classes of BMPs that can be used to reduce stormwater phosphorus loads in the post-development scenario. These efficiencies are based on data that is sourced from relevant, regional studies. The Tool provides standardized phosphorus reduction efficiencies (with rationale) for specific BMPs, but also allows the user to enter their own efficiencies provided that the rationale is also documented and is acceptable to the MOE. The Tool also allows the user to use custom BMPs or to enter the net efficiency achieved using a Treatment Train approach, which would also require documentation in a rationale that is acceptable to the MOE. The BMP selection criteria and efficiencies are shown below as reproduced from Figure 5 and Table 3 of the report, as follows:

Figure 5. Decision tree for selecting appropriate phosphorus removal efficiencies for stormwater and construction BMPs.

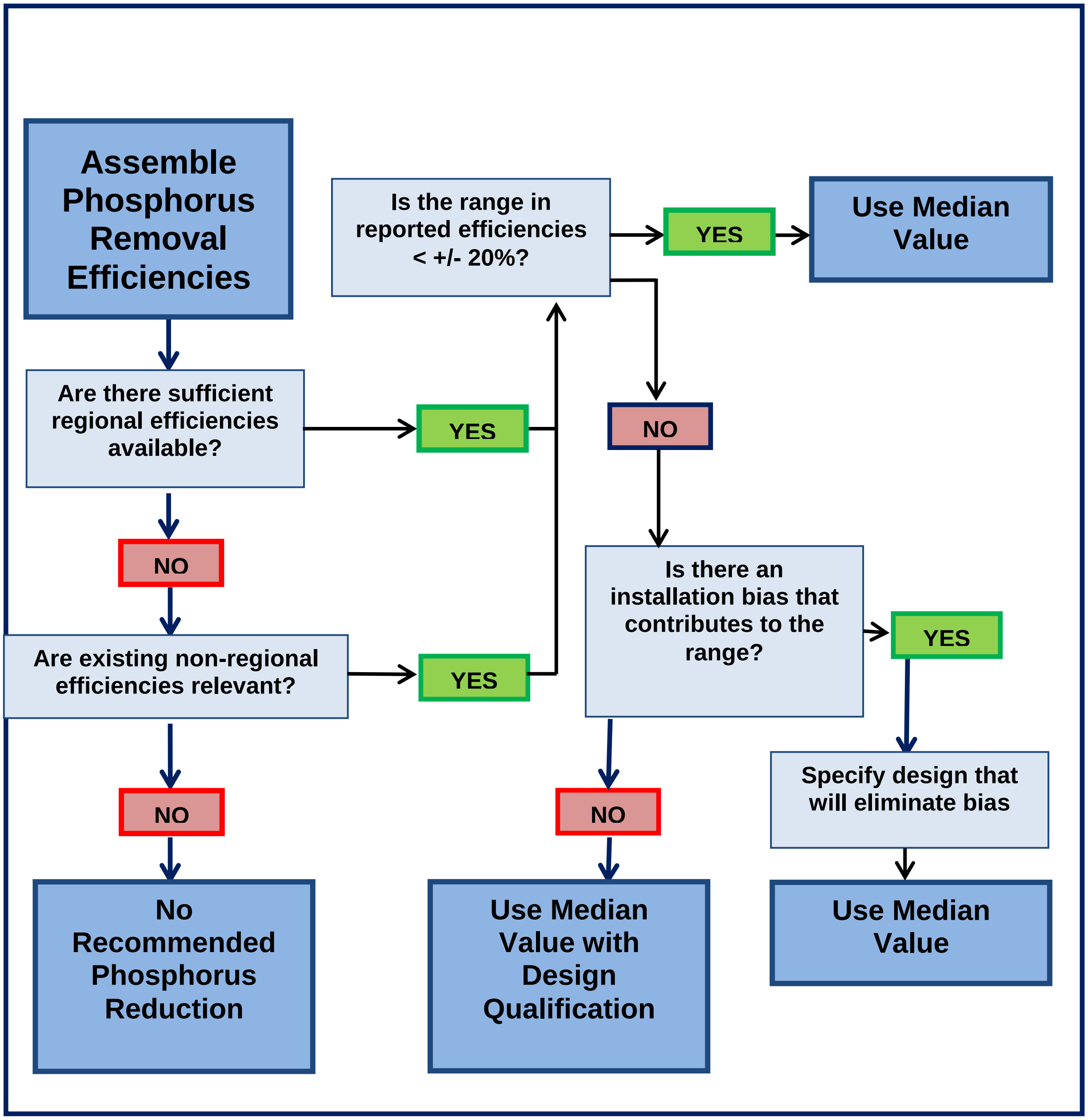


Table 3. Phosphorus Removal Efficiencies for Major Classes of BMPs Using the Decision Tree (Figure 5).

BMP Class	Reference IDs[1]	Reported Phosphorus Removal Efficiency (%)		Relevant to Ontario?	Range <40%?	Are Non-Ontario values acceptable?	Possible design criteria?	Median % Removal Efficiency
		Min	Max					
Post-development BMPs								
Bioretention Systems	8-10, 12,13, 34-38, 40	-1552	80	no	no	no	No	none
Constructed Wetlands	104, 106, 109	72	87	yes	yes			77
Dry Detention Ponds	104, 109	0	20	no	yes	yes		10
Dry Swales	24, 26-32	-216	94	no	no	no	possible	none
Enhanced Grass/Water Quality Swales	21, 104	34	55	no	yes	no	No	none
Flow Balancing Systems	106	77		no	?	yes	Min data	77
Green Roofs	2	-248		no	no	no	No	none
Hydrodynamic Devices	109	-8		no	?	yes		none
Perforated Pipe Infiltration/Exfiltration Systems	7, 4	81	93	yes	yes			87
Sand or Media Filters	104, 109	30	59	no	yes	yes		45
Soakaways - Infiltration Trenches	6, 104	50	70	no	yes	yes		60
Sorbtive Media Interceptors	111	78	80	no	yes	yes		79
Underground Storage	106	25		no	?	yes	Min data	25
Vegetated Filter Strips/Stream Buffers	6, 42, 104	60	70	no	yes	yes	Yes	65
Wet Detention Ponds	104-106, 109	42	85	yes	yes			63

Notes: [1]References associated with IDs are provided in Appendix 7.

Module 4 – Examines the potential for erosion and sediment loss during the construction phase on the basis of the Universal Soil Loss Equation and provides guidance to the user on appropriate BMPs that can be implemented during this phase to minimize sediment loss and resultant phosphorus export. The module calculates loads for the entire construction phase, but pro-rates this one-time load to annual loads to account for the eight-year hydraulic residence time in Lake Simcoe. The quantification of expected soil and phosphorus loss from a construction site is an uncertain process, even under ideal conditions. Determining expected loss reductions from the use of various on-site BMPs adds to the uncertainty. Even with inherent uncertainty, however, the Guidance proceeds from the principle that the process of quantifying soil and nutrient losses as part of the planning and approval process will have a beneficial impact on water quality regardless of whether the estimated loads are actually realized, as long as the appropriate BMPs are selected and properly implemented in a manner that minimizes soil and phosphorus losses from the site. The process of estimating construction phase loadings and the means to minimize them is one of awareness that can be translated into the site development process.

The guidance is based on information that is normally required of the proponent as part of the standard process of planning approvals. Pre- and post-development land uses are derived from the Environmental Impact Statement (EIS) prepared by the proponent and BMPs for stormwater management would be developed and described in the Stormwater Management Plan for the new development that is prepared in support of the application. The proponent uses these materials as input to the Database Tool to calculate loadings in a standard format by the approved process.

The Database Tool calculates resulting loads from each of the four modules and determines the net impact in terms of the phosphorus budget associated with the proposed development site. The analysis distinguishes permanent changes in phosphorus load resulting from changes in land use (i.e., pre- vs. post-development) from temporary loadings from construction.

To meet the intent of Policy 4.8e to minimize phosphorus loadings to Lake Simcoe from development, the MOE will recommend that municipalities approve development as site specific appropriate if:

a) Post-development load < or = pre-development load, and

b) (Post-development + amortized construction phase) load < or = pre-development loading,

OR

If (Post-development + amortized construction phase) load > pre-development loading,

THAT

All reasonable and feasible construction phase BMPs have been identified for implementation, documented and accounted for in the application.

The Tool consists of three elements:

1. A **Technical Guidance Manual** that provides the reference material used in developing the Tool, the rationale for the development of the Tool, and implementation guidance in line with Policy 4.8e of the LSPP,

2. A **Microsoft ACCESS© Database Tool** that facilitates the calculation of a phosphorus budget for new development in accordance with the technical guidance, and

3. A **Database User's Manual** explaining the operation of the database.

The "*Phosphorus Budget Guidance Tool to Guide New Development in the Lake Simcoe Watershed"* is intended for use by the development community, municipalities, the MOE and the Lake Simcoe Region Conservation Authority to facilitate review of new development applications for their compliance with Policy 4.8e of the Lake Simcoe Protection Plan. It includes a simplified checklist of required elements of any submissions made for the use of reviewers.

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1. Introduction

Lake Simcoe is enriched by nutrients from land use activities in its watershed and has, for many years, been the focus of efforts to protect and restore its water quality. These efforts began with the Lake Simcoe Environmental Strategy in the mid 1980s and led to passage of the *Lake Simcoe Protection Act* (LSPA) by the Ontario legislature in 2008. The *Act* required the establishment of the Lake Simcoe Protection Plan (LSPP) to regulate inputs of nutrients (specifically phosphorus) to Lake Simcoe. The LSPP was approved in 2009 and included a series of policies that were to be implemented to restore water quality and other ecological attributes of the lake.

This document addresses implementation of Policy 4.8e of the LSPP, which states that:

> *"An application for major development shall be accompanied by a stormwater management plan that demonstrates...*
>
> > *e. through an evaluation of anticipated changes in phosphorus loadings between pre-development and post-development, how the loadings shall be minimized."*

This direction by the MOE recognizes that, although the LSPP requires reductions in phosphorus loading, the Lake Simcoe watershed is the focus of substantial planned population growth in the next 20 years. Population growth brings the potential for additional phosphorus loading that can only be managed or reduced through: a) innovative wastewater treatment at advanced wastewater treatment plants (which is addressed through other elements of the LSPP), and b) innovations in stormwater management that would allow development to proceed without increasing phosphorus loads to the lake.

The intent of Policy 4.8e is that new development in the Lake Simcoe watershed adopts Best Management Practices (BMPs), Low Impact Development (LID) techniques and innovative stormwater management techniques to achieve sustainable development practices that will reduce the phosphorus loading from new urban development. In practice, Policy 4.8e is interpreted as a requirement that post-development loadings be reduced from pre-development loadings on any major development site, in order to achieve overall reductions in loadings to Lake Simcoe. This interpretation is in line with Strategic Direction #3 in the Phosphorus Reduction Strategy, which requires a move to "no net increase" of phosphorus for new development in the Lake Simcoe watershed.

Implementation of Policy 4.8e requires a method to quantify and compare pre- and post-development phosphorus loadings and an elaboration of BMP/LID methods that can minimize loading from new development. The guidance must be site-specific, in recognition that phosphorus export will vary in response to differing precipitation patterns, soils and slopes that occur across the Lake Simcoe watershed. In addition, the MOE recognizes that phosphorus loading during the construction phase of development needs to be considered, as construction is an ongoing process in the watershed that contributes non-point source phosphorus loads to the lake. The phasing of construction projects means that this loading can occur over an extended period of time. The loading itself, however, is temporary, and the construction load from each development will be assimilated within Lake Simcoe over time, with no long-term change to the phosphorus status of the lake.

The MOE retained Hutchinson Environmental Sciences Ltd. (HESL), Greenland International Consulting Ltd. and Stoneleigh Associates to develop the *"Phosphorus Budget Guidance Tool to Guide New Development in the Lake Simcoe Watershed"* (the "Tool"). The Tool provides a transparent, science-based and consistent approach to estimate phosphorus loadings from stormwater runoff[1] in the pre-, post- and construction phases of new development in the Lake Simcoe watershed, which can be utilized by the development community, municipalities, the MOE and the Lake Simcoe Region Conservation Authority (LSRCA). The Tool consists of three elements:

1. A **Technical Guidance Manual** that provides the reference material used in developing the Tool, the rationale for the development of the Tool, and implementation guidance in line with Policy 4.8e of the LSPP,

2. A **Microsoft ACCESS© Database Tool** that facilitates the calculation of a phosphorus budget for new development in accordance with the technical guidance, and

3. A **Database User's Manual** explaining the operation of the database.

2. Tool Development Considerations

Development of the Tool was guided by the MOE objective to:

> *"Provide the development community and municipalities with a consistent approach to estimating phosphorus loadings for pre- and post-development and the construction phase of development in the Lake Simcoe watershed that considers subwatershed characteristics."*

The intent of this objective is to support sustainable development while continuing to reduce the impact of phosphorus on Lake Simcoe by demonstrating through *"...an evaluation of anticipated changes in phosphorus loadings between pre-development and post-development, how the loadings shall be minimized"* in keeping with Policy 4.8e of the LSPP. Several key factors were considered in the development of the Tool to meet the objective.

The first is that the development of Low Impact Development (LID) techniques is a relatively new field and, as such, many techniques are innovative and new techniques will be developed over time. Although a BMP/LID technique may be worthwhile and effective, documented case studies that verify its performance with measured data may not be readily available. The Tool is based on proven techniques, as demonstrated through documented effectiveness, but must also accommodate innovation as it occurs. It cannot anticipate these innovations, but must be able to accommodate them by setting criteria and standards for their use.

The second is the complexity of monitoring storm water runoff to obtain the necessary data to estimate phosphorus load. The hydrologic response is highly variable and depends on

[1] *The Tool does not address atmospheric sources of phosphorus in dust generated from land use practices, as the science is not yet advanced to the point where estimates can be made. It does account for atmospheric deposition of phosphorus to open water and atmospheric deposition to land surfaces is included in the export coefficients for various land use practices.*

antecedent soil moisture, storm intensity and duration, site topography and soils and a host of factors that are site specific and therefore difficult to extrapolate to a variety of development sites. There is also variance in pollutant delivery to receiving water, which varies with the elapsed time since the previous storm and the stage of the hydrograph sampled (first flush vs. later storm stages). This complexity needs to be managed so that reasonable and reliable estimates can be used by all practitioners of the policy without the need for lengthy site-specific monitoring or detailed modelling. The intent is to develop a screening level tool.

Third, any Tool needs to find a balance of methods between site specific monitoring, modelling, or the use of reliable estimates from a database of previous studies. The ideal situation would be one in which phosphorus load was measured for a specific site in the pre-development stage and again in the post-development stage. This approach is impractical, however, because a) monitoring after development is too late to inform the decision of whether or not to develop the site, b) monitoring-based approaches do not allow assessment of a variety of BMPs, and c) many development sites are small and have no surface water drainage systems that would allow monitoring of runoff. A monitoring-based approach would require a long-term monitoring period that incorporated climatic variance and this is clearly not feasible for most applications. Model-based approaches, by contrast, have the advantage of allowing estimates of the current condition, future conditions, and the effectiveness of BMPs. Accurate estimates of these can be incorporated into models and usefully applied if the models incorporate the range of necessary factors and have been validated against good measured data.

Finally, the Tool must provide an approach that is:

- workable – allows practitioners and reviewers to complete or review the necessary phosphorus budgets without the need for undue additional expense or access to sophisticated software or modelling capabilities,
- timely - produces the required analysis within a reasonable time frame to allow for timely review and approvals.
- defensible – robust and providing reliable estimates that can stand up to review, and
- adaptable – such that new BMP/LID techniques or better estimates of phosphorus export can be used as they become available.

3. Technical Guidance Manual

3.1 Overview

The guidance is intended to complement and take advantage of the routine municipal planning process for new development, as it uses much of the same information on site conditions and proposed storm water management considerations. The guidance assists the user by providing adequate technical detail for inclusion in a submission to a municipality for development approval. It is, however, assumed that the user has some level of technical or engineering knowledge of soil erosion, nutrient loss processes and storm water management techniques. Some detailed ecological knowledge is valuable to assist with land use classifications.

The Technical Guidance Manual and Database Tool are divided into four modules that consider sediment and nutrient loss as follows:

- **Module 1** – Estimates pre-development phosphorus loads for representative, sub catchment level land uses contained within the study site,
- **Module 2** – Estimates post-development phosphorus loads that are representative of the proposed land uses for the study site without BMPs to reduce phosphorus loads,
- **Module 3** – Estimates effectiveness of proposed BMPs in reducing phosphorus loads in the post-development scenario, and
- **Module 4** – Examines the potential for erosion and sediment loss during the construction phase, provides guidance to the user on appropriate BMPs that can be implemented during this phase to minimize sediment loss and resultant phosphorus export and estimates sediment and phosphorus loss from the site for each phase of the construction process.

Once each of the four modules is completed by entering information into the Database Tool, the results are subjected to a comparative analysis between pre- and post-development phosphorus loads and with loads generated by construction activities. Decision rules are then applied to the comparative analysis to determine if phosphorus loads are reduced relative to existing conditions to meet the intentions of the LSPP Policy 4.8e and to support approval of the development application. The MOE will recommend that municipalities approve development as site specific appropriate if:

a) Post-development load < or = pre-development load, and

b) (Post-development + amortized construction phase) load < or = pre-development loading,

OR

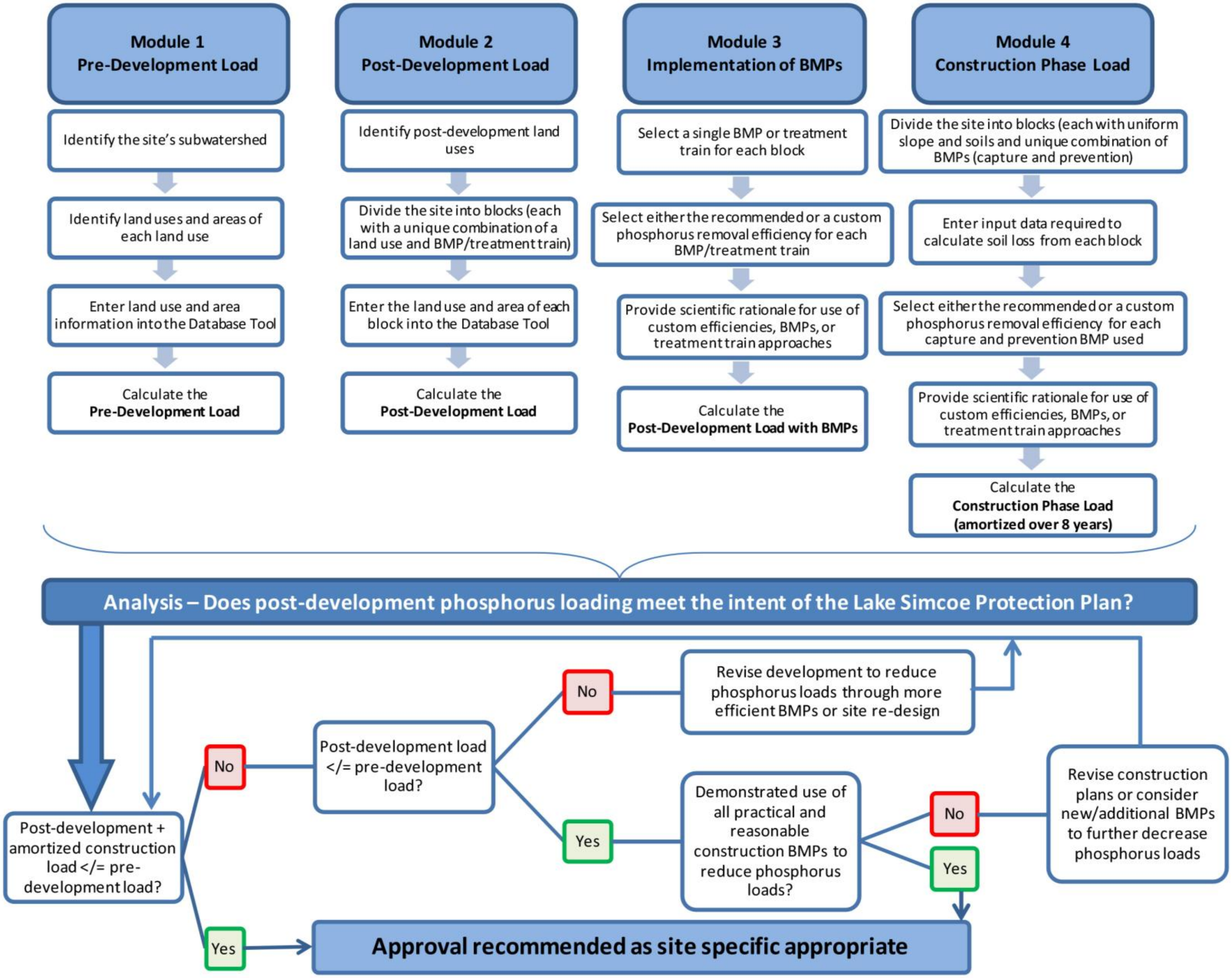
If (Post-development + amortized construction phase) load > pre-development loading,

THAT

All reasonable and feasible construction phase BMPs have been identified for implementation, documented and accounted for in the application.

The modular approach to completing a phosphorus budget using the Tool is illustrated in Figure 1. Technical guidance for each module including the approach, rationale for that approach and step-by-step instructions to complete the modules is provided in the following sections.

Figure 1. Schematic of modular approach to phosphorus guidance.



3.2 Modules 1 and 2: Pre- and Post-Development Phosphorus Load Estimation

3.2.1 Approach

An export coefficient approach is used to estimate non-point source phosphorus loadings for pre-development (Module 1) and post-development (Module 2) phases.

The export coefficient approach was developed in North America to predict nutrient inputs to lakes and streams (Dillon and Kirchner, 1975; Beaulac and Reckhow, 1982; Rast and Lee, 1983) and is now a well-established method of estimating phosphorus export when measured tributary flows and total phosphorus concentration data are lacking (e.g., Dillon *et al.* 1986, Johnes 1996, Winter and Duthie 2000, Paterson et al., 2006). The export coefficient approach is also used where it is desirable to forecast nutrient export from a land area prior to a change in land use or prior to implementing Best Management Practices, in which case it is used as a predictive tool.

The use of phosphorus export coefficients for estimating phosphorus loading is based on the knowledge that specific land forms and land uses yield or export known quantities of phosphorus over an annual cycle. Knowing the area of land in a watershed devoted to specific uses and the quantities of nutrients exported per unit area of these uses (nutrient export coefficients), annual phosphorus loading can be calculated as:

$$L = \Sigma\ EiAi,$$

where L is the total phosphorus load from a given area of land (e.g., development site), Ei is the export coefficient selected for a specific land use and Ai is the area of that land use.

A working group that included scientists from HESL, Greenland and the MOE was formed to select appropriate phosphorus export coefficients for different land uses that are applicable to the Lake Simcoe subwatersheds and that were developed and/or validated using recent measured data. The selected export coefficients were derived from 1) the results of CANWET™ modeling by The Louis Berger Group Inc. (Berger, 2010), 2) results of monitoring under the Stormwater Assessment Monitoring and Performance Program of MOE (SWAMP, 2005) and 3) analysis, review and refinement by the study team.

The Berger (2010) report used the CANWET™ model to estimate phosphorus load (in kg/yr) for land uses that are specific to each of the subwatersheds in the Lake Simcoe basin. The SWAMP studies provide recent measured total phosphorus export for urban land uses: commercial, industrial and residential development areas in southern Ontario, which were used by the MOE in the development of a phosphorus budget for Lake Simcoe under the Lake Simcoe Environmental Management Strategy (LSEMS; Scott et al., 2006, Winter et al., 2002 and 2007). A description of each of the land use classes is provided in Table 1. The final land-use specific export coefficients for the 19 Lake Simcoe subwatersheds (see Figure 2) are provided in Table 2. Details of the derivation of the export coefficients are provided in Section 3.2.1.1.

Figure 2. Lake Simcoe subwatersheds (from Berger (2010).

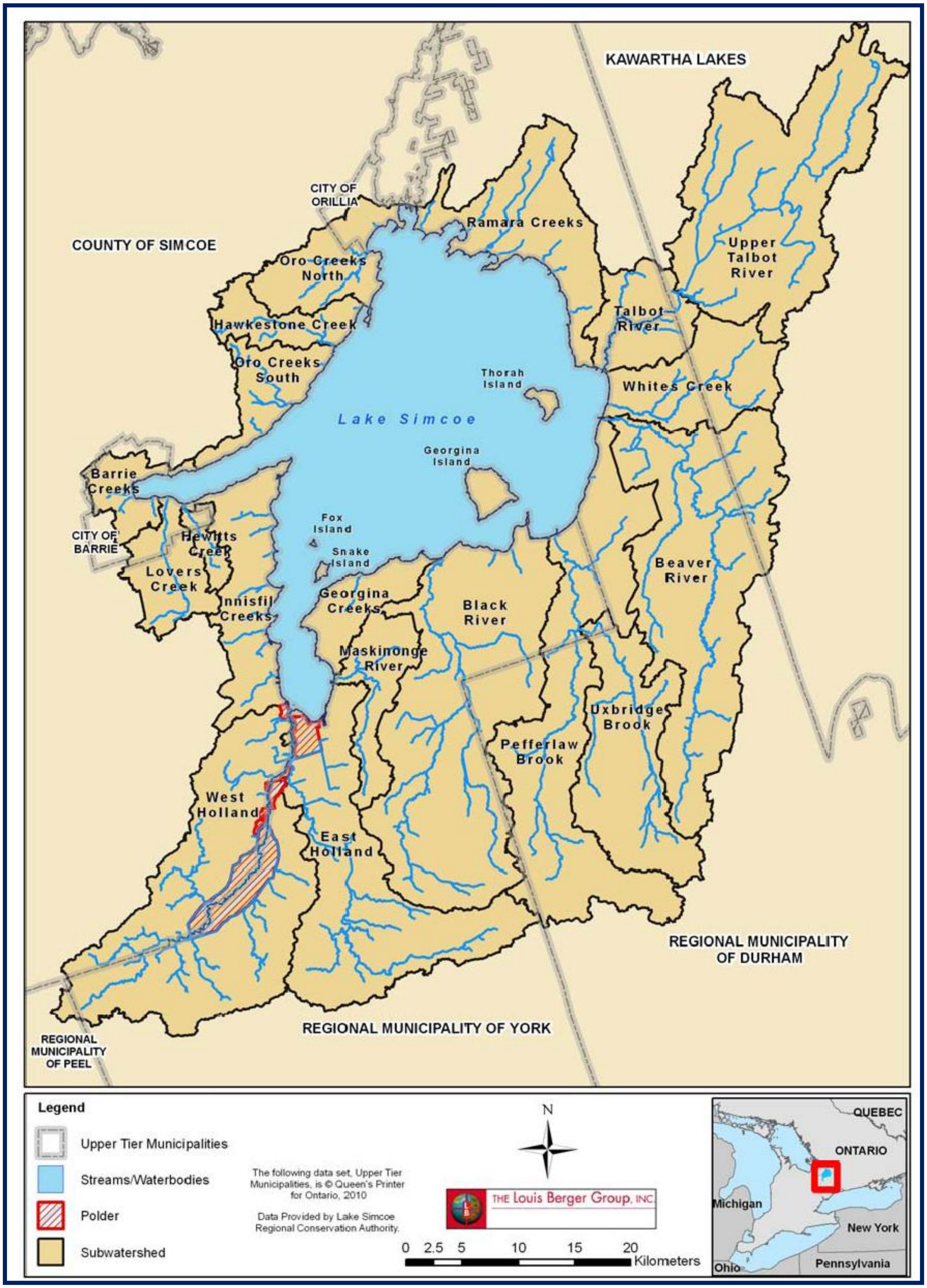


Table 1. Description of Berger (2010) Land Uses in the Lake Simcoe Watershed

Berger (2010) Land Use	Included LSRCA Land Use(s)	Land Use Description
Hay / Pasture	Non-intensive Agriculture	Hay and pasture fields, including the related agricultural buildings such as barns, silos and the farm residence. Fields are dominated with herbaceous vegetation and grasses with an understory of similar material in a state of decay. Weedy hay and/or pasture covers more than 50% of the area.
Crop Land	Intensive Agriculture	Cultivated row crops, including the related agricultural buildings (e.g., barns, silos and the farm residence), producing crops in varying degrees (e.g., corn and wheat) and includes specialty agriculture (i.e., orchards, market gardens, Christmas tree plantations and nurseries).
Sod Farm / Golf Course	Sod Farm	Sod farms.
	Golf Course	Golf courses, including lane ways, but not the isolated woodlots within, unless the area of the woodlots is < 0.5 ha.
Low Intensity Development	Estate Residential	A home including the manicured area around the home and driveway, within a natural heritage feature. The natural heritage feature is not included in the Estate Residential land use classification.
	Manicured Open Space	Cleared areas with a low density of trees, including lawns and landscaping. Land use is dominated by gardens, parkland and lawns, e.g., cemeteries, urban parks, ski hills and residential/industrial open space with a minimum size of 2 ha.
	Rail	Rail lines and the associated cleared adjacent areas.
	Rural Development	Properties not directly associated with an agricultural operation and that contain residential, commercial or other buildings, as well as a manicured open space, within a natural heritage or agricultural feature (e.g., estate residential or service station). On developed portions, these properties are under intensive use. Based on canopy cover, these areas will often appear as Cultural Savannah or Cultural Woodland in aerial photographs or satellite imagery. However, the presence of buildings and manicured lands identify the properties as Rural Development.
High Intensity Development[1] (Commercial /Industrial)	Commercial	Impervious properties that contain a building and an adjacent parking lot (e.g., shopping and strip malls, power centres, scrap yards). Excludes green land areas such as parks or river valleys.
	Industrial	Impervious properties that are not commercial and include industrial operations e.g., factories, manufacturing facilities, processing facilities, bulk fuel storage. Excludes green land areas such as parks or river valleys.
	Institutional	Schools, hospitals and other institutional structures. May include large storm water management ponds. Excludes green land areas such as parks or river valleys.
High Intensity Development[1] (Residential)	Urban	Urban related land uses including continuous ribbon development. Interpreted from aerial photographs or satellite imagery by many roof tops and/or groupings of 5 or more residential properties with a combined area of ≥ 2 ha. Residential properties include single and semi-detached dwellings, apartment buildings and associated out-buildings, driveways and parking lots. Excludes green land areas such as parks or river valleys.
Quarry	Active Aggregate	Areas that are currently being excavated or have recently been excavated. Identified by pits, extraction machinery, unvegetated landscape and/or piles of extracted materials. Active aggregate areas may contain open water.
	Inactive Aggregate	Former aggregate sites that have been recently revegetated; vegetation is established and growing. Depending on their characteristics, in aerial photographs or satellite imagery, these properties may appear to be comparable to an abandoned field or forming wetland.
Road	Road	Unpaved roads, including the shoulder. Does not include driveways.

Berger (2010) Land Use	Included Ecological Land Classifications (ELC(s))	Land Use Description
Transitional	Open Alvar	Cover varies from patchy shrub and tree cover to continuous meadow. Tree cover is ≤ 25%; shrub cover is ≤ 25%. Typically restricted to bare rock and patchy, shallow substrates.
	Cultural Meadow	Tree cover is ≤ 25% and shrub cover is ≤ 25%. The plant community is a result of, or maintained by, anthropogenic disturbances or culture.
	Cultural Thicket	Tree cover is ≤ 25% and shrub cover is > 25%. The plant community is a result of, or maintained by, anthropogenic disturbances or culture.
	Open Tallgrass Prairie	The ground layer of plants is dominated by prairie graminoids (grasses and grass-like plants, including sedges) such as Big and Little Bluestem, as well as Indian Grass. Tree cover is ≤ 25% and shrub cover is ≤ 25%. Soils are well drained with prolonged summer drought and frequent disturbance by fire.
Forest[2]	Cultural Plantation, Coniferous	Tree cover is > 60% of the area, with coniferous trees > 75% of the canopy area. The plant community is a result of, or maintained by, anthropogenic disturbances or culture.
	Cultural Woodland	Tree cover is between 35% and 60% of the area. There is often a large proportion of non-native plant species, and the plant community is a result of, or maintained by, anthropogenic disturbances or culture.
	Coniferous Forest	Tree cover is > 60% of the area, with coniferous trees > 75% of the canopy area.
	Cultural Plantation, Deciduous	Tree cover is greater than 60% of the area, with deciduous trees greater than 75% of the canopy area. The plant community is a result of, or maintained by, anthropogenic disturbances or culture.
	Deciduous Forest	Tree cover is > 60% of the area, with deciduous trees > 75% of the canopy area.
	Cultural Plantation	Tree cover > 60% of the area, with coniferous trees > 25% of the canopy area and deciduous trees > 25% of the canopy area. The plant community is a result of, or maintained by, anthropogenic disturbances or culture.
	Mixed Forest	Tree cover is > 60% of the area, with coniferous trees > 25% of the canopy area and deciduous trees > 25% of the canopy area.
Wetland[2]	Shrub Bog	Continuous *Sphagnum* spp. moss cover. Trees > 2 m tall cover ≤ 10% of the area and shrubs cover > 25% of the area. Land is rarely flooded but always saturated with water. Organic substrate > 40 cm deep consisting of *Sphagnum* peat.
	Treed Bog	Continuous *Sphagnum* spp. moss cover. Trees > 2 m tall cover 10% to 25% of the area. Land is rarely flooded but always saturated with water. Organic substrate > 40 cm deep consisting of *Sphagnum* peat.
	Open Fen	Sedges, grasses and low shrubs (< 2 m high) dominate; trees > 2 m high cover ≤ 10% of the area and shrubs cover ≤ 25% of the area. Land is rarely flooded but always saturated with water. Organic substrate > 40 cm deep consisting of moss or sedged peat.
	Shrub Fen	Sedges, grasses and low shrubs (< 2 m high) dominate; trees > 2 m high cover ≤ 10% of the area and shrubs cover > 25% of the area. Land is rarely flooded but always saturated with water. Organic substrate > 40 cm deep consisting of moss or sedged peat.
	Treed Fen	Sedges, grasses and low shrubs (< 2 m high) dominate; trees > 2 m high cover 10% to 25% of the area. Land is rarely flooded but always saturated with water with organic substrate and > 40 cm deep moss or sedged peat.

Berger (2010) Land Use	Included Ecological Land Classifications (ELC(s))	Land Use Description
	Meadow Marsh	Dominated by emergent hydrophytic aquatic plants (grow wholly or partially in water); tree and shrub cover ≤25%. Variable flooding regimes and water depth 2m.
	Shallow Marsh	Emergent hydrophytic aquatic plant cover ≥ 25%, tree and shrub cover ≤ 25% of the area. Water up to 2 m deep; standing or flowing water for much or all of the growing season.
Wetland[3]	Floating-Leaved Shallow Aquatic	Floating leaved aquatic vegetation covers > 25% of the area; no tree or shrub cover. Water up to 2 m deep; standing water is always present.
	Mixed Shallow Aquatic	A mixture of submerged and floating leaved aquatic vegetation covers > 25% of the area; no tree or shrub cover. Water up to 2 m deep; standing water is always present.
	Submerged Shallow Aquatic	Submerged aquatic vegetation covers > 25% of the area; no tree or shrub cover. Water up to 2 m deep; standing water is always present.
	Coniferous Swamp	Tree cover is > 25% of the area with trees > 5 m tall, and coniferous trees > 75% of the canopy area. Water depth is < 2 m with variable flooding regimes; standing water or spring (vernal) pooling covers > 20% of the ground.
	Deciduous Swamp	Tree cover is > 25% of the area with trees > 5 m tall, and deciduous trees > 75% of the canopy area. Water depth is < 2 m with variable flooding regimes; standing water or spring (vernal) pooling covers > 20% of the ground.
	Mixed Swamp	Tree cover is > 25% of the area with trees > 5 m tall; coniferous trees > 25% of the canopy area and deciduous trees > 25% of the canopy area. Water depth is < 2 m with variable flooding regimes; standing water or spring (vernal) pooling covers > 20% of the ground.
	Thicket Swamp	Tree or shrub cover > 25%, dominated by hydrophytic shrub and tree species (grow wholly or partially in water); tree cover ≤ 25%, hydrophytic shrub cover > 25%. Water depth is < 2 m with variable flooding regimes; standing water or spring (vernal) pooling covers > 20% of the ground.
Open Water[4]		Lakes, rivers and ponds including stormwater management ponds.

Notes: [1]High Intensity Development areas were further separated for the Tool into commercial/industrial and residential classes because the percentage of impervious area is typically much higher in commercial/industrial areas than in residential areas resulting in a greater amount of storm water runoff, [2]includes CANWET classes of Coniferous Woodland, Deciduous Woodland and Mixed Woodland, [3]includes CANWET classes of Emergent Wetland and Woody Wetland. [4]Not included in the Berger (2010) land classes but added for the purposes of the Tool recognizing that some development areas may have open water areas that should be included in calculations of phosphorus export.

3.2.1.1 Derivation of Export Coefficients

Export coefficients for all land classes were derived based on total phosphorus loading estimates reported by Berger (2010) for individual subwatersheds with the exception of High Intensity Development, which was derived from measured loads in MOE's Stormwater Management Monitoring and Performance Program (SWAMP, 2005; MOE, unpublished data) and Open Water which was derived from estimates of atmospheric loads to the surface of Lake Simcoe (Scott et al., 2006; LSRCA, 2009). The following describes the derivation and rationale for the selection of export coefficients from these sources for each land use in each subwatershed as provided in Table 1.

Berger (2010) provides total phosphorus loads (kg/yr) from the total areas devoted to specific lands uses in each of the 19 Lake Simcoe subwatersheds (Pefferlaw River and Uxbridge Brook subwatersheds were combined in the analysis as were the Talbot River and Upper Talbot River subwatersheds). Division of the total annual export (in kg) for each land use by the area (ha) devoted to that land use provides a standardized export coefficient in kg/ha/yr.

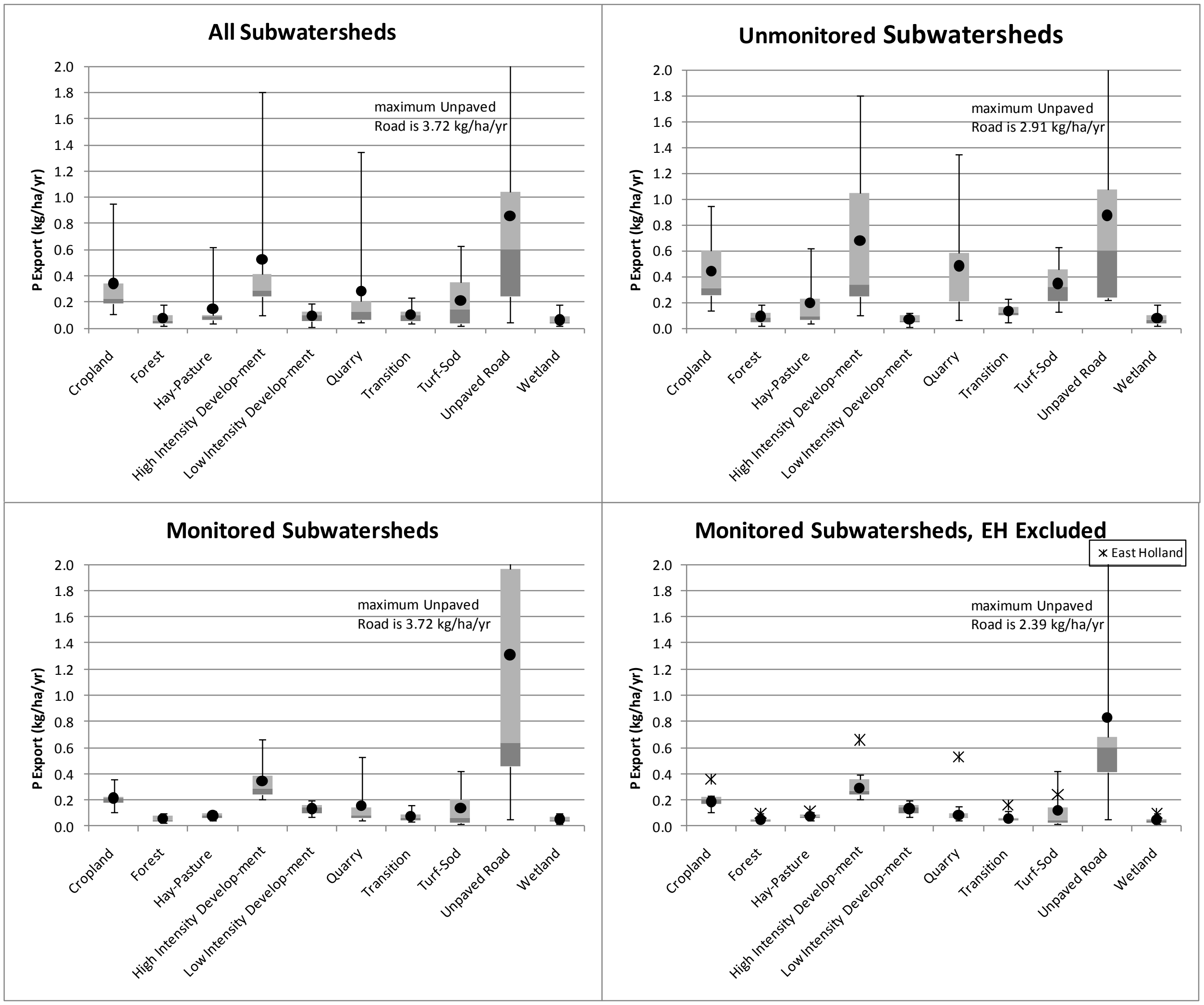
Phosphorus loads from groundwater, tile drainage and stream bank erosion were provided by Berger (2010) for the total subwatershed area only (and not for specific land uses) and so loads from these sources were allocated to the land use areas as follows:

1. **Groundwater** loads were added proportionally by area to all land use categories except High Intensity Development,
2. **Tile Drainage** loads were added to Cropland areas only, and
3. **Stream Bank Erosion** loads were added proportionally by area to Forest, Wetland and Transition areas

Groundwater loads were not allocated to High Intensity Development areas as these areas have a large amount of impermeable surfaces, thereby reducing groundwater infiltration and seepage. Tile drainage is used mostly for cropland agriculture. Stream Bank Erosion was only allocated to ‘natural’ land cover areas assuming that streams primarily occur in these land areas and are protected from development. Refined land use data would be required to determine the proportion of phosphorus loads from stream bank erosion in other land class areas (e.g., proportion of streams running through agricultural area or urban area). The resultant total phosphorus loads were used to calculate total phosphorus export (kg/ha/yr) for each land use in each subwatershed.

Considerable variance in phosphorus export coefficients derived from the Berger (2010) results occurred among subwatersheds, particularly among unmonitored subwatersheds (Table 2, Figure 3). Of the 19 subwatersheds, only 7 (with Pefferlaw River and Uxbridge Brooks subwatersheds combined) had measured data on flows and phosphorus loads for calibration of the CANWET model. Comparatively little variance occurred in export coefficients among these monitored subwatersheds, with the exception of higher export coefficients for most land classes in the East Holland River. Higher export coefficients in the East Holland River reflect the highly urbanized portions of that subwatershed as well as the amount of high intensity agriculture, which have both contributed to degraded water quality (LSRCA, 2010). The unmonitored subwatersheds were calibrated to estimated flows and total loads were estimated from the results of those monitored subwatersheds that were most similar in land cover (see Scott et al., 2006).

Figure 3. Boxplots showing variance in export coefficients derived from Berger (2010) for the Lake Simcoe Subwatersheds. Boxes represent 25th percentile, median and 75th percentile, whiskers are the minimum and maximum values, and the mean is denoted as the black dot.



Note: Excludes the export coefficient for Low Intensity Development (0.013 kg/ha/yr) for the East Holland River, which is suspected as being an error.

Some variation in phosphorus export between subwatersheds is expected for a given land cover type due to differences in environmental factors such as soil characteristics, physiography and runoff conditions. Principal Components Analysis (PCA, an analysis that displays patterns in multivariate data) was carried out to identify differences between subwatersheds based on the combination of key environmental factors affecting phosphorus export (see Appendix 4). Environmental factors included Soil K Factor (erosion coefficient), Slope Length, Base Runoff and Soil P (soil phosphorus concentration) as reported in Berger (2010) for each land use type in each subwatershed. Overall, results of the PCA did not reveal patterns in environmental characteristics that would explain the variance in export coefficients derived for the unmonitored subwatersheds (i.e., subwatersheds with similar environmental characteristics did not have similar phosphorus export coefficients). By contrast, for the monitored subwatersheds, the East Holland River was characterized by higher Soil K Factor, Base Runoff and Soil P values in comparison to the other monitored subwatersheds, explaining the higher phosphorus export coefficients for this subwatershed.

Given the high variance in export coefficients for the unmonitored subwatersheds that cannot be explained by major environmental characteristics, phosphorus export coefficients for the Tool were derived for the monitored subwatersheds only and these were also applied to the unmonitored subwatersheds. For the monitored subwatersheds, export coefficients for all land use types were those developed from Berger (2010) results with the following exceptions:

- Low Intensity Residential Development for the East Holland River subwatershed – The calculated export for this land use (0.013 kg/ha/yr) was an order of magnitude lower than for other land cover classes in the subwatershed, including forest (Table 1). This suggests that the calculated value may underestimate the export from Low Intensity Residential Development in this subwatershed. The mean phosphorus export coefficient of 0.13 kg/ha/yr for the other monitored subwatersheds was therefore selected for Low Intensity Residential Development in the East Holland subwatershed.
- Unpaved Road - The export coefficients among monitored subwatersheds for Unpaved Road ranged from 0.049 to 3.72 kg/ha/yr. Given the large range in export values, the working group selected the mean export from the monitored subwatersheds (excluding the East Holland River) of 0.83 kg/ha/yr for Unpaved Road to be applied for all Lake Simcoe subwatersheds.
- Quarry for Whites Creek subwatershed – No quarries were reported in the Whites Creek subwatershed, therefore the mean export of the monitored subwatersheds (0.08 kg/ha/yr) was selected for this land cover class.

At the request of the MOE, phosphorus export coefficients of 1.32 kg/ha/yr were selected for high intensity urban residential areas and 1.82 kg/ha/yr for commercial/industrial high intensity development. These were developed from measured data from the 2006 SWAMP studies (MOE, unpublished data). These values are higher than those derived using the Berger (2010) modeled phosphorus loads for High Intensity Development, which ranged from 0.21 to 0.67 kg/ha/yr for the monitored subwatersheds (mean = 0.35 kg/ha/yr). These higher export coefficient values were selected because they were derived from measured data, have been used in several Lake Simcoe studies by MOE and LSRCA (Winter *et al.*, 2002, 2007; Scott et al., 2006; LSRCA, 2007, LSRCA and MOE, 2009) and are comparable to other published export coefficients for urban development. For example, Reckhow et al. (1980) reports urban export coefficients ranging from 0.19 to 6.23 kg/ha/yr (mean 1.91 kg/ha/yr, standard deviation 1.70 kg/ha/yr) and the US Environmental Protection Agency's (1983) nationwide urban runoff report distinguishes between residential and commercial land use with export coefficients of 1.3 kg/ha/yr and 3.4 kg/ha/yr, respectively. More details for this rationale are provided by the MOE and included in Appendix 8

In the PCA of the environmental factors that was described previously, the characteristics of the Georgina Creeks, Oro Creeks North and West Holland River subwatersheds (all unmonitored) were most similar to the East Holland River subwatershed with generally higher soil K factors, Soil P and base runoff that would be consistent with higher phosphorus export. The export coefficients for the East Holland River were therefore applied to these unmonitored subwatersheds.

The mean phosphorus export coefficients for all monitored subwatersheds (excluding the East Holland River) were applied to the remaining unmonitored subwatersheds (i.e., Hewitts Creek, Innisfill Creeks, Maskinonge River, Oro Creeks South, Ramara Creeks and Talbot/Upper Talbot River) as these were characterized by lower soil K factors, soil P and base runoff relative to the East Holland River.

A phosphorus export coefficient of 0.26 kg/ha/yr was selected for Open Water, which represents the atmospheric deposition of phosphorus in the Lake Simcoe watershed. This export coefficient was calculated from the mean measured atmospheric load of 19 tonnes/yr averaged over 5 years from 2002 to 2007 to the surface of Lake Simcoe (surface area = 722 km^2) (Scott et al., 2006; LSRCA, 2009). Note that phosphorus loads from atmospheric deposition to land are incorporated into the export coefficients for the various land cover classes. The atmospheric/open water coefficient should not be interpreted as loading from dust generated by land use activities such as agriculture or construction. It represents a regional atmospheric contribution. The means to estimate dust generation and loading are the subject of current research initiatives being undertaken by the MOE, the LSRCA and various research partners.

The final export coefficients for all subwatersheds are provided in Table 2. These are coded into the database tool to derive subwatershed-specific estimates of phosphorus export from specific land uses for the pre- and post-development (with no BMPs) calculations.

Table 2. Land-Use Specific Phosphorus Export Coefficients (kg/ha/yr) for Lake Simcoe Subwatersheds

Subwatershed	Phosphorus Export (kg/ha/yr)											
	Cropland	Hay-Pasture	Sod Farm/Golf Course	High Intensity Development		Low Intensity Development	Quarry	Unpaved Road	Forest	Transition	Wetland	Open Water
				Commercial /Industrial	Residential							
Monitored Subwatersheds												
Beaver River	0.22	0.04	0.01	1.82	1.32	0.19	0.06	0.83	0.02	0.04	0.02	0.26
Black River	0.23	0.08	0.02	1.82	1.32	0.17	0.15	0.83	0.05	0.06	0.04	0.26
East Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hawkestone Creek	0.19	0.10	0.06	1.82	1.32	0.09	0.10	0.83	0.03	0.04	0.03	0.26
Lovers Creek	0.16	0.07	0.17	1.82	1.32	0.07	0.06	0.83	0.06	0.06	0.05	0.26
Pefferlaw/Uxbridge Brook	0.11	0.06	0.02	1.82	1.32	0.13	0.04	0.83	0.03	0.04	0.04	0.26
Whites Creek	0.23	0.10	0.42	1.82	1.32	0.15	0.08	0.83	0.10	0.11	0.09	0.26
Unmonitored Subwatersheds												
Barrie Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
GeorginaCreeks	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Hewitts Creek	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Innisfil Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Maskinonge River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Oro Creeks North	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26
Oro Creeks South	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Ramara Creeks	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
Talbot/Upper Talbot River	0.19	0.07	0.12	1.82	1.32	0.13	0.08	0.83	0.05	0.06	0.05	0.26
West Holland River	0.36	0.12	0.24	1.82	1.32	0.13	0.08	0.83	0.10	0.16	0.10	0.26

3.2.2 Methods - Calculating Pre-development Conditions

The pre-development or "existing conditions" phosphorus load is calculated through the following steps, by the user:

1. The user will rely on the information documented and detailed in the EIS for the development that will be used to support the planning application to the Municipality.
2. The user will choose the subwatershed or geographic area of the Lake Simcoe watershed in which the development is proposed from a drop down list provided by the database. If the development area spans two or more subwatersheds, the areas within each subwatershed should be modelled separately.
3. Specific land use classifications will be delineated and their boundaries overlain on an orthographic aerial photograph that shall be included in their submission.

a. The user will select the Table 1 land uses that most closely match those delineated in their mapping and will document the rationale for the choice in a comment field for the database report. (e.g.,"ELC classifications a, b and c are present – these correspond to "forest"", or "actively tilled corn fields are classified as "cropland"").

b. Land use classifications will be chosen by the user from a "drop down" list in the database, which will contain the land use classifications in Table 1.

c. The user will provide areas (in ha) of each identified land use on the development site.

d. The database will produce a table showing each land use, the area and export coefficient associated with each land use, the user comment or rationale for choice (as entered by the user in a text box) and the total area of the development.

4. The database links each land use classification to the respective phosphorus export coefficient for that land use for that subwatershed as shown in Table 2, calculates the total annual phosphorus load from each land use (as ha x kg/ha/yr) and sums the loads from each land use to produce the total annual pre-development load from the site.

5. The user may not adjust a particular export coefficient for site-specific characteristics in this version of the Tool, but user-defined export coefficients may be considered for future revisions of the Tool.

6. The database adds a final column of pre-development phosphorus loads for each land use to the table produced in Step 3d.

3.2.3 Methods - Calculating Post-Development Conditions

The post-development phosphorus load (without BMP implementation) will be calculated by the user, using the following steps:

1. The user will rely on the information on the proposed development that is documented and detailed in the planning application (EIS and SWM plans) to the Municipality.

2. The user will delineate the post-development land uses and overlay their boundaries on an orthographic aerial photograph that shall be included in their submission.

   a. Land uses will be defined using the same methods described for the pre-development conditions.

   b. The site will be divided into post-development blocks; each block with a unique combination of a land use and Best Management Practice or Treatment Train that will be applied to that land use in Module 3 (Figure 4).

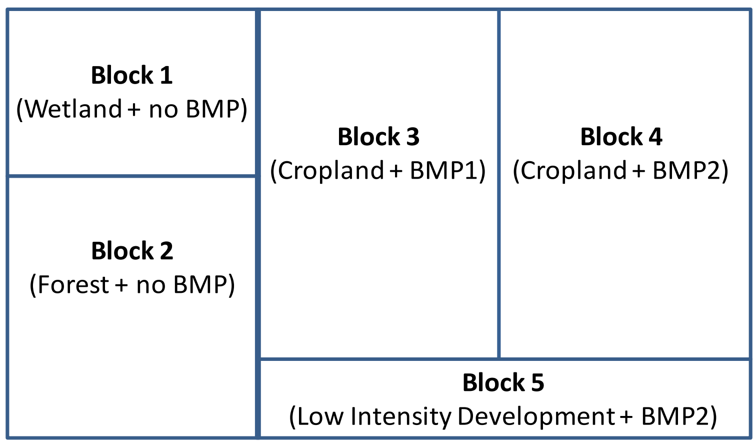
   c. Land use for each block will be chosen by the user from a "drop down" list in the database, which contains the land use classifications in Table 1.

   d. The user will provide areas (in ha) of each post-development block.

e. The database will produce a table showing land uses, areas and export coefficients associated with each land use for each post-development block, and will display the total area of the post-development site.

f. The database will provide a check to make sure that the sum of post-development blocks is the same as the sum of the pre-development land use areas.

Figure 4. Schematic of post-development blocks that comprise a unique land use and BMP (or Treatment Train approach).



3. The database links each land use to the respective phosphorus export coefficient for that land use in that subwatershed (from Table 2), calculates the total annual phosphorus load from each block (as ha x kg/ha/yr) and sums the loads from each block to produce the total post-development load from the development site without BMPs.

4. The user may not adjust a particular export coefficient for site-specific characteristics in this version of the Tool, but user-defined export coefficients may be considered for future revisions of the Tool.

5. The database adds a final column of phosphorus loads (in kg/yr) for each post-development block to the table produced in Step 2e.

6. The database produces a summary showing:

   a. Pre-development phosphorus load (in kg/yr) for the entire development site,

   b. Post-development phosphorus load (in kg/yr) for each block and for the entire development site, and the

   c. Difference between pre- and post-development phosphorus loads (in kg/yr and as a %).

3.3 Module 3: Post-Development Load Reduction with BMPs

3.3.1 Approach

Phosphorus removal efficiencies for a variety of Best Management Practices (BMPs) were compiled from a literature review (Appendix 1). These were evaluated for their applicability to the Lake Simcoe watershed and a representative % removal efficiency for each applicable BMP was derived where possible, according to the methods outlined in the following sections. The user is not limited to using the BMPs and % removal efficiencies recommended in the Tool, although these do represent "pre-approved" BMPs and efficiencies that are acceptable to MOE. If custom BMPs or % removal efficiencies are used, supporting scientific rationale for their use must be provided in the Stormwater Management (SWM) plan for the development. This rationale will be reviewed as part of the approval process.

3.3.1.1 Selection of Appropriate BMP Phosphorus Removal Efficiencies

For any given stormwater management BMP there are a range of reported values that describe the phosphorus reduction that can be expected. This is also true for stormwater mitigation strategies relating to the construction phase of development projects (see Module 4). In both cases, there may be a wide range in reported percent reductions of phosphorus and these numbers may be highly qualified by various elements of BMP design or setting. For this reason, it is difficult to derive a single removal efficiency value for even narrow categories of BMPs and almost all stormwater practice documents that were reviewed reported a range of removal efficiency values for a given BMP category.

There are, however, reasonable decisions that can be made to derive appropriate and applicable single numbers that represent average expected phosphorus removal efficiency of various BMPs. This involves an examination of the regional variation that is inherent in the range of observed values together with any specific design aspects that may be contributing to the reported range. If, for example, the focus is confined to only those reported values that are regionally significant and the range in those values that apply to well designed or appropriately installed measures, then the result should be a narrower range in reported values.

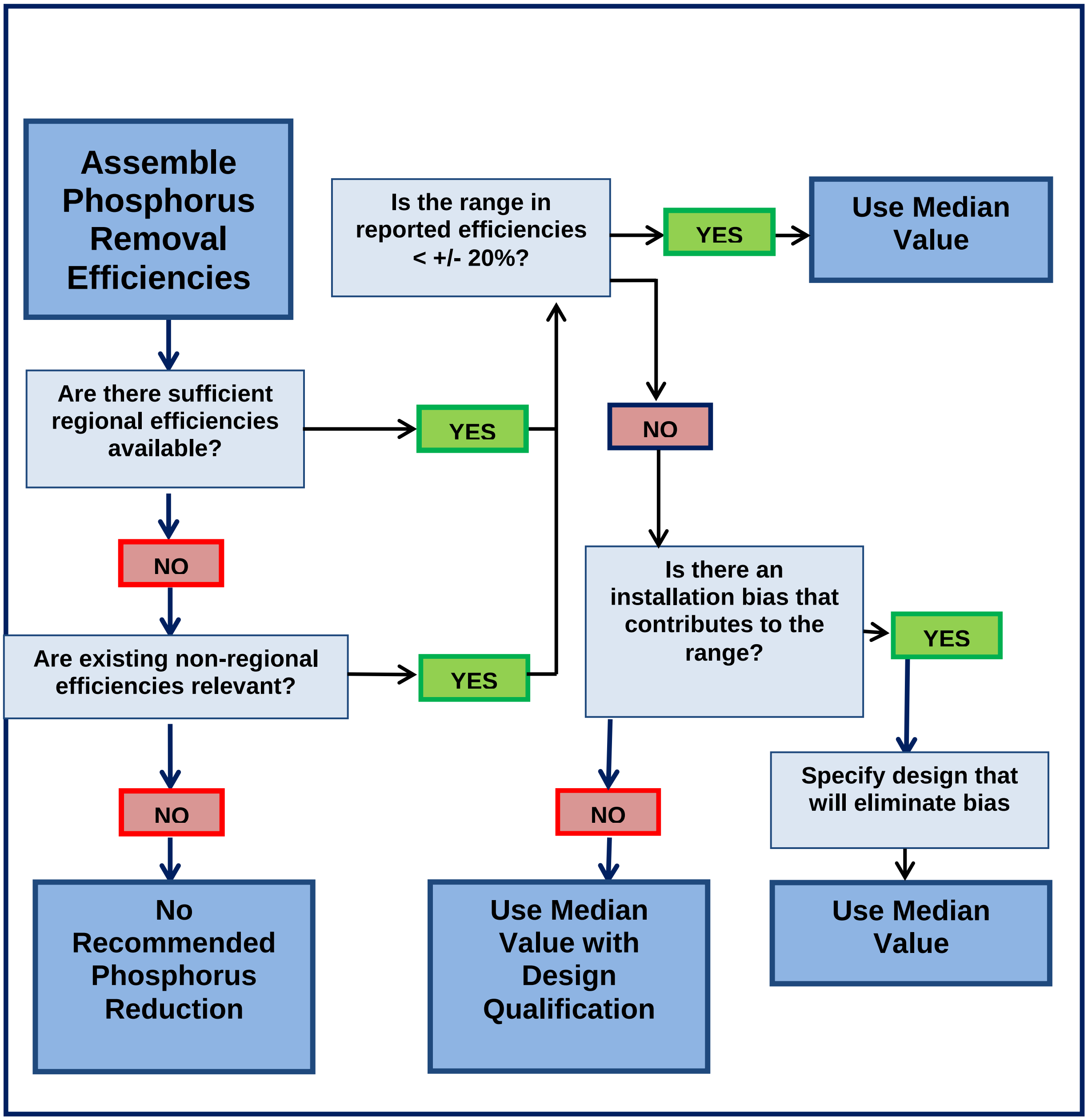
Much of the confidence in selecting a phosphorus removal efficiency for any given stormwater management technique will result from the collection of a large number of regionally significant values that fall within a narrow range. In most cases, however, our review of available information showed that the availability of these types of data was the exception rather than the rule.

The decision tree shown in Figure 5 allows the consistent, objective selection of phosphorus removal efficiencies for individual stormwater or construction runoff management techniques by considering the range of reported efficiencies, the applicability of the reported efficiencies to the Lake Simcoe watershed and design characteristics that may influence the reported efficiencies.

In the example below, a phosphorus removal efficiency range of +/-20% (40% total) is used to describe an acceptable range in values (this corresponds generally to the median range of values observed for the techniques described in the documents that have been reviewed). The median of these values is chosen as a conservative estimate of phosphorus reduction. In the most difficult cases where the ranges in reported values are >40%, the removal efficiency value may require a design qualification to be acceptable (see Figure 5).

The BMPs reviewed for the Tool (Table 3) are classes of BMPs and there may be unique features for any given BMP that make it more or less effective at phosphorus removal. Any BMP that is chosen should be assessed against the references given for the BMPs in Column 2 of Table 3 to determine whether or not the % phosphorus removal efficiency shown in Table 3 is applicable to the BMP of choice and for the specific characteristics of the development site. If not, the user should select the appropriate removal efficiency and provide details in support of that efficiency in the Stormwater Management (SWM) plan.

Figure 5. Decision tree for selecting appropriate phosphorus removal efficiencies for stormwater and construction BMPs.



3.3.1.2 Derivation of Single BMP Phosphorus Removal Efficiencies

Table 3 shows how the decision tree in Figure 5 is applied to the removal efficiencies that were assembled from the documents that were reviewed. The first step is to assess the efficiencies to identify those that are regionally significant. In this case, there is one BMP where the reported removal efficiencies are relevant to the Lake Simcoe watershed, namely perforated pipe infiltration/exfiltration system. The range of efficiencies for this BMP is less than 40% and so the median of the observed values is chosen as a single phosphorus removal efficiency for that class of BMP. In two cases, (sorbtive media interceptors and soakways/infiltration trenches), although there are no Ontario phosphorus removal efficiencies reported in the review materials, the techniques are not limited by geography. The reported ranges in efficiency for these BMP classes are narrow so the median efficiency is chosen as a representative phosphorus removal efficiency. In all other cases, there are unacceptable regional differences and wide ranges in efficiencies that would not support the derivation of single representative phosphorus removal efficiencies. In the case of dry swales, the non-Ontario removal efficiencies may be usable, but the range of reported values is large such that it will be necessary to identify design criteria that will limit the range in efficiencies for this class of BMPs before a value can be chosen.

Table 3. Phosphorus Removal Efficiencies for Major Classes of BMPs Using the Decision Tree (Figure 5)

BMP Class	Reference IDs[1]	Reported Phosphorus Removal Efficiency (%)		Relevant to Ontario?	Range <40%?	Are Non-Ontario values acceptable?	Possible design criteria?	Median % Removal Efficiency
		Min	Max					
Post-development BMPs								
Bioretention Systems	8-10, 12,13, 34-38, 40	-1552	80	no	no	no	No	none
Constructed Wetlands	104, 106, 109	72	87	yes	yes			77
Dry Detention Ponds	104, 109	0	20	no	yes	yes		10
Dry Swales	24, 26-32	-216	94	no	no	no	possible	none
Enhanced Grass/Water Quality Swales	21, 104	34	55	no	yes	no	No	none
Flow Balancing Systems	106	77		no	?	yes	Min data	77
Green Roofs	2	-248		no	no	no	No	none
Hydrodynamic Devices	109	-8		no	?	yes		none
Perforated Pipe Infiltration/Exfiltration Systems	7, 4	81	93	yes	yes			87
Sand or Media Filters	104, 109	30	59	no	yes	yes		45
Soakaways - Infiltration Trenches	6, 104	50	70	no	yes	yes		60
Sorbtive Media Interceptors	111	78	80	no	yes	yes		79
Underground Storage	106	25		no	?	yes	Min data	25
Vegetated Filter Strips/Stream Buffers	6, 42, 104	60	70	no	yes	yes	Yes	65
Wet Detention Ponds	104-106, 109	42	85	yes	yes			63

Notes: [1]References associated with IDs are provided in Appendix 7.

The Table 3 values are recommended as general, representative phosphorus reduction efficiencies for major classes of BMPs and have sufficient documentation to demonstrate their effectiveness in Ontario's climate according to the decision rules provided above. ***They are only representative, however, under the assumption that they are built to design specification and maintained to design standards, to assure their effectiveness.***

Where the user wishes to use innovative BMPs, or if they can provide documented information or engineering design characteristics that alter the values provided in Table 3, then they would document their rationale according to the guidance provided (Sections 3.3.1.1 and 3.3.1.2) and demonstrate the effectiveness of the BMP in a manner acceptable to MOE in the SWM plan submitted for the development. Choosing to provide a different BMP or efficiency value may better reflect site-specific knowledge or emerging technologies but will result in a thorough review of the development application by the approving agency (ies), which may require more time to assess.

A treatment train approach, where more than one BMP is used in a series to treat stormwater runoff from the same land use area, can be used in the Tool. In a treatment train approach, the total phosphorus removal efficiency of the train is not necessarily the sum of the efficiencies for the individual BMPs in the train. This occurs because the efficiencies of several BMPs are influenced by phosphorus input concentrations. Treatment of runoff by one BMP may reduce the phosphorus concentration in the runoff to a level that reduces the effectiveness of the next BMP in the train. In addition, the Tool cannot anticipate or accommodate the many combinations of techniques that can make up a treatment train. The Tool, therefore, does not provide suggested phosphorus removal efficiencies for a treatment train. The user must provide the total phosphorus removal efficiency of the proposed treatment train and document the scientific rationale for that efficiency in the SWM plan for the development.

3.3.2 Methods - BMP Implementation

BMP selection and calculation of phosphorus load reductions for the post-development scenario will be completed by the user as follows:

1. The user will rely on the information documented and detailed in the SWM plan for the site that will be used to support the planning application to the Municipality.
2. The user will select the type of BMP (or a Treatment Train approach) that will be used to capture or treat runoff from each post-development block using the drop-down menu in the database. The user can select "Other" from the drop-down list if they plan to use an innovative BMP that is not coded in the database.
3. The user can choose to use the phosphorus removal efficiencies for the BMPs that are coded in the database, or can enter a custom efficiency. The User must enter a custom efficiency if a Treatment Train is selected.
4. If "Other" or "Treatment Train" are selected as a BMP, or if a custom efficiency is used for any BMP, the user will enter a brief rationale in the 'rationale field' that refers the reviewer to the SWM Plan for the full technical justification.
5. The database links each combination of post-development phosphorus load and chosen BMP for each block to the phosphorus removal efficiency of the chosen BMP to provide the load reduction that will be applied to runoff from that area.

6. The database calculates the total annual phosphorus load from each block (i.e., each land use/BMP combination) with BMP implementation and sums the loads to produce the total post-development load with BMPs for the site.
7. The database produces a summary showing:
    a. Pre-development phosphorus load (in kg/yr) for the entire site,
    b. Post-development phosphorus load (in kg/yr) for the entire site, with and without BMPs, and
    c. Change in phosphorus load from pre-development conditions, with and without implementation of BMPs (in kg/yr and as a %).

3.4 Module 4: Construction Phase Phosphorus Loads

3.4.1 Approach

Quantification of phosphorus loads during construction is challenging given the variance in timing of construction processes, storm timing and frequency and site characteristics. In addition, phosphorus concentration in soil will vary across a site, and with depth. The Tool is therefore based on estimating soil loss during construction, and the effectiveness of various BMPs in preventing soil loss. A BMP that reduces soil loss from construction activity by 65% is assumed to reduce phosphorus loss by 65%, regardless of the actual concentration of phosphorus in the soil.

The approach used in this guidance is based on the Universal Soil Loss Equation (USLE) as described by Stone and Hilborn (2000). Users of the Guidance are required to divide potential development sites into blocks of continuous slope and relatively uniform soil characteristics and provide information needed to populate the USLE. From this it is possible to approximate soil loss during the construction phase. The construction phase assessment does not include losses of soil-bound phosphorus to the atmosphere by wind erosion, as the science is not well-enough advanced to guide estimates from this pathway. The Tool addresses losses through surface runoff only.

TRCA (2006) and MOE (2003, 2006) have developed Erosion and Sediment Control Guidelines for Urban Construction which are excellent resources for designing site controls to reduce sediment and nutrient loss during construction, but which provide no indication of the potential soil loss either with or without controls in place. Where available, the effectiveness of erosion and sediment control BMPs that apply during the construction phase to minimize soil, and hence phosphorus, runoff have been documented in this Guidance. These reductions are included as part of the calculation approach used in the database tool provided.

Using the USLE and documented construction phase BMPs, a reasonable estimate of construction phase sediment loading is produced.

3.4.2 Calculating Construction Phase Loading

The quantification of expected soil loss from a construction site is an uncertain process, even under the most well-defined conditions. Determining expected loss reductions from the use of various on-site BMPs adds to the uncertainty. Even with inherent uncertainty, however, this Guidance proceeds from the principle that the process of quantifying soil and nutrient losses as part of the planning and approval process will have a beneficial impact on water quality regardless of whether the estimated loads are actually realized, as long as the appropriate BMPs are selected and properly implemented in a manner that minimizes soil losses from the site. The process of estimating construction phase loadings and the means to minimize them is one of awareness that can be translated into the site development process.

This Guidance provides a means for users to estimate sediment and particulate phosphorus loading from the construction phase using the Universal Soil Loss Equation as described in Stone and Hilborn (2000) where average annual estimated soil loss (S_L) in kg/year from the construction site is calculated as:

$$S_L = \sum 2241.7 \times R \times K \times LS \times C \times P \times A_i$$

Where:

2241.7 is a unit conversion from tons / acre to kilograms per hectare;

R is the rainfall and runoff factor by geographic location with a value of 90 for the Lake Simcoe basin;

K is the soil erodibility factor based on soil textural class and organic matter content of exposed soil according Table 4;

LS is the slope length gradient factor which can be calculated as:

$$LS = [0.65 + 0.0456 \text{ (\% slope)}] + 0.006541 \text{ (\% slope)}^2 \times \text{(slope length in meters / constant)}^{NN}$$

Where:

The user would provide values for % slope and slope length.

Constant = 22.1, and

NN is determined according to slope via Table 5.

C is the C factor. The C factor in agricultural applications of the USLE is the product of a crop type factor and a tillage method factor which produces an estimate of the portion of the year during which there is exposed soil that is unprotected by vegetative cover. For a construction site application this could be calculated using input from the user as:

C = (months during construction phase that soil is exposed/12) / (duration of construction in months/12)

Table 4. K Factor Data (Organic Matter Content)

Textural Class	Average	Less than 2 %	More than 2 %
Clay	0.22	0.24	0.21
Clay Loam	0.30	0.33	0.28
Coarse Sandy Loam	0.07	--	0.07
Fine Sand	0.08	0.09	0.06
Fine Sandy Loam	0.18	0.22	0.17
Heavy Clay	0.17	0.19	0.15
Loam	0.30	0.34	0.26
Loamy Fine Sand	0.11	0.15	0.09
Loamy Sand	0.04	0.05	0.04
Loamy Very Fine Sand	0.39	0.44	0.25
Sand	0.02	0.03	0.01
Sandy Clay Loam	0.20	-	0.20
Sandy Loam	0.13	0.14	0.12
Silt Loam	0.38	0.41	0.37
Silty Clay	0.26	0.27	0.26
Silty Clay Loam	0.32	0.35	0.30
Very Fine Sand	0.43	0.46	0.37
Very Fine Sandy Loam	0.35	0.41	0.33

Table 5. NN Values

S	< 1	$1 \leq$ Slope < 3	$3 \leq$ Slope < 5	≥ 5
NN	0.2	0.3	0.4	0.5

P is the support practice factor and represents BMP practices that contribute to reducing soil erosion on the slope (“source reduction”) and practices that capture sediment at the bottom of the slope (“capture reduction”).

$$P = \{(1- BMP_{prev}) * a_1 + (1 - a_1)\} * \{(1 - BMP_{cap}) * a_2 + (1 - a_2)\}$$

Where:

BMP_{prev} is the efficiency of the erosion prevention BMP applied on the slope (i.e., source reduction)
a_1 is the portion of the slope the erosion prevention BMP is applied to
BMP_{cap} is the efficiency of the down gradient sediment capture BMP
a_2 is the portion of the slope runoff intercepted by the sediment capture BMP (i.e., capture reduction)

A_i is the area of slope i. Soil loss for the site is the sum of soil loss from each slope that comprises the site.

The phosphorus load (P_L) from the construction site area is the product of the soil loss (S_L), the subwatershed soil phosphorus concentration ($Soil_P$) and the duration of construction phase in years (D_{yrs}):

$$P_L = S_L * Soil_P * D_{yrs}$$

Soil phosphorus concentration was originally intended to be a subwatershed value derived from the CANWET model. However, due to the variability between subwatersheds it was decided that a single soil phosphorus value of 0.0004 kg-TP/kg soil would be provided for all subwatersheds. This value was derived from the mean of subwatershed aggregate values used in Berger (2010). The CANWET model applies an empirical enrichment factor to the initial estimate of soil phosphorus to account for the greater phosphorus adsorption surface of smaller particles that make up a greater portion of eroded material.

A summary of user supplied data requirements is presented in Table 6. This information is used as input to the included database tool to calculate an estimated base phosphorus loading from the construction phase.

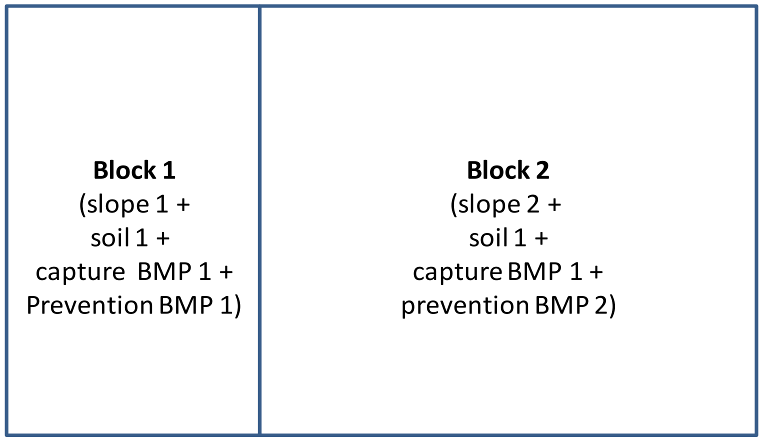
Table 6. Input Requirements for Calculating Construction Phase Soil Loss

Key Factors (to be input by guidance users for each continuous sloping portion of the construction site)
Area of slope being considered
Predominant soil texture class and organic matter content
Surface Slope Gradient (%)
Length of Slope
Aggregate efficiency of BMP(s) to be used on this sloped portion of the site
Duration of exposed soil on site
Duration of construction phase

In order for this approach to produce a defensible estimate of sediment and phosphorus loading from a construction site, the site must be divided into a series of sub-areas, or ‘blocks’, each with relatively uniform slope and soil characteristics to which a specific set of BMPs will be applied (Figure 6). The soil loss equation is applied to each block and the estimated site load is the summation of loads from each sub-area. Calculated loading from each block needs to consider the amount of time during the construction process that each area is undisturbed, exposed, stabilized and with or without sediment controls to capture runoff.

If a construction phasing approach is to be used for construction, the undisturbed portions of the site are assumed to contribute their pre-development loading rates of sediment and phosphorus until clearing and grading takes place after which the USLE estimate is applicable for the period of time until the ground reaches its post-development state.

Figure 6. Schematic of construction phase blocks that comprise relatively uniform slope and soil characteristics and a unique capture BMP and prevention BMP combination.



3.4.3 Construction Phase BMPs

In all cases there is a requirement that BMPs are maintained throughout the duration of the construction phase in order that they continually operate at their design efficiency. The literature reports a wide range of soil loss from uncontrolled construction sites. For example between 5 and 50 tonnes per hectare per year of sediment is reported by Dreher and Mertz-Erwin (1991). Properly installed and maintained controls and BMPs can significantly reduce losses of soil and phosphorus and these construction phase BMPs can be divided into general categories:

- Detention / retention systems – detain stormwater in some form of storage. This practice can produce a number of benefits including reduced flow velocity and hence reduced sheer stress on soil particles, reduced peak flows and increased sedimentation.
- Flow control structures – divert flow from off-site, less disturbed or stabilized areas and route it around areas with exposed soils thus preventing erosion in vulnerable areas. Structures may also be used to reduce sheer stress from runoff by reducing flow velocity through provision of storage.
- Construction practices – include strategic sequencing and phasing of site activities, strategic grading and minimizing soil loss from vehicle traffic leaving site.
- Filtration systems – include various methods of physical filtration of sediment from stormwater prior to release.
- Infiltration systems – capture and infiltrate stormwater.
- Soil erosion prevention – includes use of vegetative covers, mulches and fibre blankets to protect exposed soils from the erosive forces of incident rainfall and overland flow.

Schueler and Holland (2000, article 52) provide ten (10) key elements that are needed for an effective erosion and sediment control plan for construction sites. These are summarized below.

Minimize unnecessary clearing and grading

Clearing and grading must be carried out within a stream protection and sediment control strategy. These activities should be greatly restricted in sensitive areas including stream buffers, forest and wetland conservation areas, springs, seeps and infiltration areas, steep slopes, highly erodible soils and other environmentally sensitive features. These should be identified in both the site EIS and the SWM plan.

Only areas that need to be cleared and graded as part of the development foot print or in order to access the site should be disturbed. Features to be preserved need to be clearly marked on site plans and in the field. Contractors need to be made aware of and have a clear understanding of how the sediment control strategy and minimization of clearing is to take place.

Minimizing site disturbance is a critical factor in reducing the cost of other sediment and erosion control measures on a construction site.

Protect water courses and stabilize stream banks

Streams and watercourses are sensitive to construction activities. Where these features exist on a construction site, no clearing should be permitted within a prescribed setback in order to provide an adequate buffer. These should be identified in both the site EIS and the SWM plan.
Additional protection should be installed along the perimeter of the watercourse buffer in the form of a silt fence, swale or other form of filtration to intercept stormwater runoff carrying sediment from upland portion of the site to a watercourse.

Existing and future drainage ways traversing a construction site are a major conveyance of sediment from the site to watercourses as well as also being very prone to erosion from stormwater runoff. Ideally, drainage ways should be protected as a grass-lined channel or through the use of sod, erosion control blankets or jute netting. Check dams may be appropriate to slow stormwater passing through drainage ways and provide an opportunity for suspended sediment to settle. Check dams can also provide some storage to reduce peak flows that can impact receiving watercourses.

Use construction phasing to limit soil exposure

Large scale clearing and grading is a typical current practice for development sites, but such practices should be avoided because they produce the greatest loss by maximizing the duration that soils are exposed and the area of exposure. Construction phasing is an alternate approach whereby the site is divided into smaller sub-areas where clearing and grading take place only immediately before construction on a portion of the site. All other sub-areas of the site are either undisturbed or stabilized within 30 days of grading. This means that site grading cannot take place all in one step as is the current typical practice. Typical sediment load reductions compared with conventional non-phased approaches are estimated at around 40% for a subdivision development (Schueler and Holland (2000, article 54). Combining this reduced sediment loss with other practices that capture already suspended sediments can lead to a much reduced loss rate from a well managed site. Prevention of erosion is especially important on sites with fine soil particle sizes that can be very challenging to remove once they are suspended (Brown and Caraco, 1996)

The size of the project and the economics of grading in multiple phases are certain to be a consideration in the use of a phased approach. Schueler and Holland (2000 article 54) suggests a minimum 10 ha threshold. Because grading is an expensive process and involves the mobilization of large equipment it may be cost prohibitive to grade one phase, remove or idle equipment and then return it for grading a subsequent phase some time later.

If a phased approach is to be used, planning for it must begin in the early stages of the project as there is an added level of complexity inherent to the approach that will require additional coordination. The planning should set out “triggers” for initiating a subsequent phase and also for stabilization of the current phase. The sequence of construction for each phase and also for the overall project needs to be determined from the beginning.

Cut and fill must be balanced within each phase without dependence on undisturbed areas for storage of material or provision of additional material for the current phase. Therefore the existing and planned topography must be considered when delineating each phase of construction to ensure that a balance can be met.

Stormwater management, roads and other infrastructure need to be considered in each phase. Where stormwater management facilities are to exist within the final site plan, the phase(s) that contain these facilities should be initiated earlier in order that they can provide stormwater treatment for the disturbed site in advance of completion. Temporary facilities may need to be used to protect already completed phases or adjacent properties and watercourses that will receive runoff from the construction site.

Phasing planning also needs to consider the impact of on-going construction on completed phases both from disturbance from construction activities and traffic. This may involve the use of alternate access roads for each phase.

For each phase, erosion and sediment control practices need to be planned and installed prior to disturbance. Planning needs to define when and where stabilization techniques are to be used following grading. Maintenance and inspection schedules for sediment control elements must be specified and followed.

Although a phased approach will likely incur an added cost to the developer, this should be considered along with the reduction in cost of treating larger amounts of sediment laden stormwater through various capture techniques that require space, construction time, materials and subsequent maintenance.

Immediately stabilize exposed soils

The objective on every construction site should be to establish grass or mulch cover within two weeks after soils are exposed. Therefore fibre mulch is needed to stabilize soils during months when grass germination is slow or not possible. Compilation of data from four (4) studies of 17 erosion prevention techniques involving various types of ground cover including mulches, straw, compost, fibre and synthetic blankets suggests that establishing a soil cover immediately after soil exposure can reduce soil and/or TSS loss by 29% to 99% with an approximate median value of 90%. Slopes in these studies ranged from 9% to 34% with various soil textures and storm events (Schueler and Holland, 2000, article 55).

Lee and Skogergboe (1985) found a 99% reduction in suspended solids load after seeding exposed soil to increase biomass from zero to 2,762 kg/ha.

An effective erosion and sediment reduction plan for the site will need to consider contingency strategy for stabilizing soils when project schedules shift and climate conditions impact the establishment of vegetative cover.

Protect steep slopes and cuts

Steep slopes are the most highly erodible surface on a construction site. Land clearing, vegetation stripping, grading, cut-and-fill and other practices that disturb soil on a slope should not be conducted.

If soil disturbance on a slope cannot be avoided, upland flow should be prevented from flowing down over a slope. Severe gullies can form quickly from overland flow on a disturbed slope. Gully erosion results in large amounts of soil loss from a slope and can cause a slope to fail.

Upland flow should be diverted around the slope by installing an earthen berm, ditch or perforated drain along the top of the slope. Runoff will discharge from the end of the diversion and the designer should ensure a stabilized outlet with capacity for a 10 year storm event, and stabilized diversion channels.

A silt fence anchored securely into the ground at the top of the slope may be used in conjunction with a permanent diversion feature to capture sediment on slopes less than 15 m long. A silt fence is not effective at diverting overland flow as it is permeable. If mid- or base-of-slope sediment capture is required, and silt fence is installed to capture sediment, the silt fence must be installed to adequately handle high water velocities and sediment movement down the slope, otherwise water and sediment will overload or knock the silt fence down. If a traditional silt fence is not adequate for mid- or base-of-slope application, a scoop trap or super silt fence may be a suitable alternative. Schueler and Holland (2000, article 56) describes these structures.

Temporary seeding, mulch or other surface treatments may not be effective in preventing erosion on steep slopes. Additional stabilization measures such as erosion control blankets, geogrids/geotextiles and mulch binders are often required on steep slopes. In winter, steep slopes may be protected by a plastic sheet cover (like covering a soil stockpile). All stabilization measures must be appropriately tied-in to the ground at the top of the slope to prevent overland flow from flowing beneath them. Stabilization methods are not designed to prevent slope failure, only reduce erosion.

Install perimeter controls to filter sediments

Perimeter controls are installed at the edge of a construction site to retain or filter runoff before it leaves the site. Silt fences and earthen berms are two of the most common perimeter controls.

Silt fences are moderately effective in filtering sediment when installed, located and maintained properly, with reported sediment removal efficiencies ranging between 36% and 86% with a median of 70% reported in four (4) studies summarized in Schueler and Holland (2000, article

56). However, silt fences are commonly improperly installed and maintained, significantly reducing this efficiency.

Some basic guidance for proper installation of silt fencing includes:

Silt fencing must be aligned parallel with slope contours down gradient of the exposed area. Positioning should reflect the need for erosion and sediment control above property boundaries, but should consider construction traffic. The edges of the silt fence need to curve uphill to prevent flow from bypassing it. The length of the contributing slope should be no more than 30 m. Fabric must be deeply entrenched to prevent undercutting. Spacing between posts should be less than 2.5 m and portions of the fence receiving concentrated flow need to be reinforced.

If runoff does not infiltrate the ground faster than it accumulates behind berms or silt fences, it will flow to other areas of the construction site or will run off of the site. Runoff will discharge from the ends of berms and the designer should ensure a stabilized outlet with capacity for a 10 year storm event, stabilized diversion channels and berms (i.e., appropriate surface cover). There are typically fewer maintenance problems with earthen berms than silt fences, provided berms are designed to suit the site's conditions and climate. For small sites, a compacted 0.66 m high berm made of compacted soil and covered with an appropriate surface treatment is usually sufficient.

Straw bales should not be used as perimeter berms as they typically do not retain sediment well, can add to dissolved phosphorus loads in runoff and are commonly improperly installed and maintained.

Gravel or clear stone can be installed in conjunction with silt fences or earthen berms as a filtering outlet on small sites, provided that sediment will not flow through or plug the filter during construction or between maintenance cycles.

Even when erosion and sediment control BMPs are properly installed and maintained, construction sites will still discharge high concentrations of sediments during large storm events. Therefore, erosion and sediment control BMPs should include a trap or basin to settle sediments in runoff, before runoff leaves the site. For most soils, settling devices must operate at 95 – 99% efficiency to produce a non-turbid discharge. However, traditional settling basins have been shown to have variable efficiency because of the distribution of sediment grain size. Finer sediments take more time to settle out and can comprise the larger portion of the sediment load. The traditionally simple designs of settling basins may not be adequate to capture these fine materials.

To improve sediment settling efficiency, settling basins should include features to increase water retention time or decrease water energy/flow to promote more efficient sediment settling. These features could include: greater storage volume, internal geometry which reduces water flow rates, gentle side slopes, multiple cells, perforated riser pipes, and the use of baffles, skimmers and other outlet devices to reduce sediment discharge.

A detailed inspection and cleanout/maintenance plan should also be implemented with the use of settling basins/devices to increase efficiencies.

Use contractors trained in the use of sediment control techniques

The most important aspect of erosion and sediment control is having contractors on the construction site that are experienced in the installation and maintenance of erosion and sediment control BMPs that are appropriate to the site's conditions. This includes contractors who conduct earth works with minimal footprints and structure work to reduce erosion prone surfaces.

Erosion and sediment control courses are available from construction organizations and through some municipalities and conservation authorities (e.g., Toronto Regional Conservation Authority). Contractors with training from these courses may provide better erosion and sediment control services than those without. Hiring an environmental consultant or engineer with professional erosion and sediment control design is also advisable, especially on large or complex sites.

Adjust planning on-site to ensure appropriateness

Erosion and sediment control plans and best management practices are usually designed at the desk top. Site conditions may not be the same as those on site plans, and site conditions may change unexpectedly during construction. Therefore, erosion and sediment control plans and BMPs should be monitored and revised as necessary, to capture sediment before it migrates off of the construction site.

If sediment migrates off of the site, especially if the sediment contains contaminants, third party properties or the environment may be damaged, fines may be laid and the property owner may be mandated by the MOE or local conservation authority/municipality to remediate the impacts. Therefore, it is crucial to capture sediment before it leaves a site. If planned erosion and sediment control BMPs are not effectively capturing sediment, or it appears that the BMPs may fail, the erosion and sediment control plans should be amended.

Re-assess effectiveness of sediment management following large storms

Following the first storm on a site, the effectiveness of the erosion and sediment control BMPs should be assessed. This "first event" assessment will indicate if erosion and sediment control BMPs are appropriate or need to be amended, or if additional BMPs are required.

Include maintenance planning and implementation for sediment control practices

Sediment control features capture sediment and they become ineffective if accumulated sediment fills their basins or pore spaces. Therefore, maintenance (e.g., sediment removal) of sediment control features is required. The maintenance interval should be determined based on the type of erosion control feature installed, and intensity of erosion on the site (e.g., silt fences may need more frequent maintenance than large settling ponds).

Additionally, if construction activities continue longer than expected or unexpected site conditions arise (e.g., larger exposed areas or more precipitation than anticipated), maintenance may be required on 'one time' installation features that wouldn't normally require maintenance.

For the purpose of simplicity this Guidance will assume that soil and nutrient loss rates are uniform throughout the year and that the efficiency of BMPs also remains unchanged. These factors can be revised as information becomes available in the future.

3.4.4 Effectiveness of Construction Phase BMPs

The Database Tool uses a 2-tier reduction approach to calculating sediment reduction from construction phase BMPs. The first BMP reduction is applied to the base load as determined from the USLE equation that assumes no protection. This "source reduction" is applied to account for load reductions resulting from erosion prevention measures. These measures, and associated reductions and rationale are:

- Vegetative cover – 99% reduction after construction site areas are returned to vegetative cover (grass or open field vegetation) during the construction phase
- Mulch, fibre or geotextile blankets and mats – 90% reduction for a) areas where mulch coverage is maintained, mulch is applied thickly enough to prevent erosion from runoff and a second tier BMP is installed at the point of runoff, or b) areas that are completely covered with a fibre or geotextile blanket that is secured and maintained to prevent erosion from runoff and a second tier BMP is installed at the point of runoff.
- Check dams – Check dams do retain coarse particulate matter and associated phosphorus but the efficiency of these devices is not yet well enough known to provide an associated reduction.

The second tier BMP reduction is applied to the resulting load at the bottom of the slope or prior to the load leaving the site. This "capture reduction" is applied to account for load reductions resulting from sediment capture measures. These measures include practices such as:

- Dry Detention Ponds – 10% reduction as described in Section 3.3, Table 3
- Wet Detention Ponds - 63% reduction as described in Section 3.3, Table 3
- Vegetated Filter Strips/Stream Buffers – 65% reduction as described in Section 3.3, Table 3
- Silt fences – 70% reduction for areas where silt fences are properly installed, maintained and inspected to effectively to capture sediment.
- Sand or media filters (filter tubes and bags) - 45% reduction as described in Section 3.3, Table 3
- Soakaways - Infiltration Trenches – 60% reduction as described in Section 3.3, Table 3
- Anionic Polymer Runoff Treatment – 91% reduction for treatment of runoff from an area where TSS concentration in the runoff ranges from 171 to 706 mg/L.

The combined reduction in sediment load is represented as the "P" factor in the soil loss equation (Section 3.4.2) for each slope unit assessed. We assume that the same efficiency of these BMPs is applicable to runoff that has already been subject to Tier 1 BMPs, however, the effectiveness of the Tier 2 BMPs is likely reduced since larger particle sizes are already retained by Tier 1 BMPs leaving the more difficult to retain finer particles.

Additional techniques and details for construction phase reduction of sediment loss are presented in Appendix 2.

3.5 Analysis to Estimate Changes in Phosphorus Load

The intent of Policy 4.8e is to minimize phosphorus loadings to the lake from development and the test of meeting that intent has been interpreted as:

Post-Development Load < or = Pre-Development Load.

The MOE recommends that municipalities require that phosphorus loading from the construction phase be minimized in support of other related designated policies in the LSPP (i.e., Policy 4.20 and 'have regard' fro Policy 4.21), with the objective that:

Post-Development Load + Construction Load < or = Pre-Development Load.

In consideration of the above, the MOE recommends that municipalities approve development as site specific appropriate if:

a) Post-development load < or = pre-development load, and

b) (Post-development + amortized construction phase) load < or = pre-development loading,

   OR

   If (Post-development + amortized construction phase) load > pre-development loading,

   THAT

   All reasonable and feasible construction phase BMPs have been identified for implementation, documented and accounted for in the application.

In consideration of the above, the database tool calculates resulting loads from each of the four modules and determines the net impact in terms of the phosphorus budget associated with the proposed development site. The analysis needs to distinguish permanent changes in phosphorus load resulting from changes in land use (i.e., pre- vs. Post-development) from temporary loadings during each year of construction. The Database Tool calculates loadings on an annual time step for pre-, post and as a total load for the entire duration of the construction phase.

The impact of the construction phase load to the lake from any one year will be fully assimilated within eight years, as the average residence time of water in Lake Simcoe is 7.5 years (Scott et al., 2004). The annual contribution from the construction phase load is therefore calculated by dividing the total construction phase load by 8 (to "amortize" the loading from construction over the residence time of water in Lake Simcoe) and adding the result to the post-development condition. If the resulting load exceeds the pre-development load, the applicant would determine additional construction phase BMPs that will reduce the load to below pre-development levels or, alternatively, shorten the construction phase to meet the requirement that all reasonable and feasible construction phase BMPs have been considered for the development. This approach is illustrated for four hypothetical scenarios in Table 7.

Table 7. Sample Analysis to Achieve Reductions in Phosphorus Load. All figures are in kg/yr.

Pre Development Load		**600**
Post Development Load		480
Construction Phase Annual Load		120
Scenario 1 Two Year Build Out		
Construction Phase - 2 Year Total Load	240	
Construction Phase - Amortized annual load over 8 years		30
Post Development Load		480
Total Load : Post Development + Construction		**510**
Conclusion : Net Reduction in Load		
Scenario 2 Twelve Year Build Out		
Construction Phase - 12 Year Total Load	1440	
Construction Phase - Amortized annual load over 8 years		180
Post Development Load		480
Total Load : Post Development + Construction		**660**
Conclusion : No Net Reduction in Load		
Scenario 3 Reduce Build Out Time to Six Years		
Construction Phase - 6 Year Total Load	720	
Construction Phase - Amortized annual load over 8 years		90
Post Development Load		480
Total Load : Post Development + Construction		**570**
Conclusion : Net Reduction in Load		
Scenario 4 Twelve Year Build Out + Improve BMPs by 50%		
Construction Phase - 12 Year Total Load	720	
Construction Phase - Amortized annual load over 8 years		90
Post Development Load		480
Total Load : Post Development + Construction		**570**
Conclusion : Net Reduction in Load		

The final component of phosphorus management is verification that the development and its construction are carried out to achieve the development plan and BMPs that informed the phosphorus budget development. The Tool is developed with the purpose of demonstrating, through scientifically valid methods, the conditions under which “no net phosphorus load” can be achieved and verified at the planning stages of development. The need for verification that the development was implemented as proposed needs to be considered, but is beyond the scope of this document and must be addressed as part of the planning approval and implementation process.

4. Future Directions

The methodology for calculating a site level phosphorus budget presented in this Guidance needs to be considered a “living document” that is updated over time as new information and technology become available, or as the LSPP Phosphorus Reduction Strategy or other policies change. The following should be considered as part of the future direction of the evolving Guidance:

- Pphosphorus export coefficient values should be updated in response to new monitoring or modelling initiatives at the subwatershed, catchment, and potentially site level of resolution.

- The methodology provided in this Guidance uses the standard Universal Soil Loss Equation (USLE) as described by Stone and Hilborn (2000) rather than the more recent RUSLE2 which is more complex and involves the use of a more definitive database of parameter values. Future reviews might consider whether the RUSLE2 approach would produce a more reliable result and if data is available to support its use.

- This Guidance and the associated Database Tool could be made a web-based utility in the future in order to allow for easier updating of tables and parameters used in the calculations. We note, however, that proponents require stability in the planning and approval process and that this need must inform the decisions on timing of updates to the process or coefficients.

- Wind erosion from agricultural activities and construction sites has not been considered in the subwatershed modeling work completed to date and may contribute to the atmospheric deposition portion of loading to Lake Simcoe in both the pre-development (agricultural) and post-development (construction) phases. Many practices that reduce wind erosion potential may also reduce soil loss due to stormwater runoff. Therefore, future efforts should be made to quantify a) losses due to wind erosion from agricultural and construction activities, and b) the benefits of BMPs to reducing both types of soil loss.

- There is a need to account for changes in understanding of watershed processes or better estimates of phosphorus loads from specific land uses and to incorporate advances in storm water management, LID and BMPs as they are made available in the future. These could be accommodated by issuing addenda to the guidance document with updated phosphorus removal efficiencies for BMPs as they became available and were accepted by the MOE. These addenda would provide information requirements, rationale and criteria for adoption of new technologies and techniques. The modular approach of the Tool allows addenda to be issued for specific modules without the need to re-write the entire guidance document.

- There is a need to provide rationale and criteria to guide proponents who wish to consider alternative approaches and those who must review alternative approaches. This Guidance provides a generic methodology for quantifying phosphorus loading that is based on a set of assumptions used in an aggregated modeling approach. It makes generalizations about soil loss during construction phase and the efficiencies of a set group of BMPs. Proponents may wish to undertake more detailed site modeling and/or monitoring to justify a development application under special circumstances. Such alternative approaches need to be considered to determine the appropriateness of the assessment to the specific site conditions.

- The Ministry may consider reviewing existing guidance for LID, Construction Phase activities (i.e., erosion and sedimentation considerations) and updating the SWMPD Manual from time to time to reflect current and emerging practices in these sectors.

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Appendix 1
Annotated Bibliography of Development BMPs Literature

Ref. #	Citation	Reference	Comments
1	Berger Group 2010	*Estimation of Phosphorus Loadings to Lake Simcoe.*	reviewed to establish phosphorus loading coefficients for the land uses in each of the Lake Simcoe subwatersheds
2	Toronto Region Conservation Authority 2006	*Erosion and Sediment Control Guidelines for Urban Construction*	this document focuses on sediment runoff mitigation for construction sites. The document does not quantify either percents or concentrations
3	Credit Valley Conservation 2010	*Low Impact Development Stormwater Management Planning and Design Guide CVC Version 1, 2010*	uses the treatment train approach to Low Impact Development. Ten techniques are described and runoff reduction estimates or TP reduction estimates are given for each LID technique
4	Schueler, T.R., 2000a	*Comparative Pollutant Removal Capability of Stormwater Treatment Practices Technical Note #95 from Watershed Protection Techniques. 2(4): 515-520.*	compares median % pollutant removal efficiencies for several stormwater treatment practices from the Centre for Watershed Protection database including: wet and dry ponds, wetlands, filters, infiltration, water quality swales and ditches - insufficient monitoring data to confidently assess performance of several commonly used practices, i.e. infiltration, bioretention, filter strips and swales
5	Schueler, T.R., 2000b	*Pollutant Removal Dynamics of Three Wet ponds in Canada Technical Note #114 from Watershed Protection Techniques. 3(3): 721-728.*	removal efficiencies and design details reported in this document are also presented in Reference #6 along with those of other Ontario stormwater treatment practices monitored under the SWAMP program
6	MOE et al 2005, SWAMP	*Synthesis of Monitoring Studies Conducted Under the Stormwater Assessment Monitoring and Performance Program*	provides evaluation of four wet ponds (including the 3 ponds in Reference #5), one wetland, one flow-balancing system, one underground tank and two oil grit separators in Ontario. Provides an overview of stormwater management practices and guidelines in Ontario, maintenance considerations, monitoring designs, and operational costs. Performance evaluations in the report are more relevant to the Lake Simcoe watershed than those reported for similar systems in the US (References #4, 7 and 9). The report is therefore the recommended prime source of information for these stormwater treatment practices.
7	Winer, R., 2000	*National Pollutant Removal Performance Database for Stormwater Treatment Practices 2nd Edition March 2000. Report prepared for the EPA Office of Science and Technology*	performance results of stormwater treatment practices in the US from 135 studies contained in the database - as % removal efficiencies and effluent concentrations (no influent concentrations are reported). Specific site or design characteristics are not considered. Contains a bibliography for more detailed site and design information. This is the detailed report summarized in Reference #4. All primary findings from the report are noted in the review of Reference #4 above.
8	Ontario Ministry of the Environment, 2003	*Stormwater Management Planning and Design Manual, 2003 and Ministry Guideline: Erosion and Sediment Control Best Management Practices (December 2006)*	guidance for the selection and sizing of stormwater management infrastructure with information on cost and maintenance for each technology. Reference for describing those types of stormwater mitigation technologies that are known for use in Ontario climates. no performance details given. Some references to the fact that certain techniques under certain conditions will export no water from the watershed to the receiving water

Ref. #	Citation	*Reference*	Comments
9	*http://www.bmpdatabase.org*	*The International Stormwater Best Management Practices (BMP) Database Project website*	provides access to the downloadable MS Access database as well as summary reports. allows downloading information summaries for each practice study using specified criteria (facility type, state/province, water quality parameters) that include design details, site characteristics and monitoring results. useful to refine performance evaluations for specific practices.
10	Mary T. Nett1, Mark J. Carroll, Brian P. Horgan, A. Martin Petrovic, 2008 American Chemical Society Volume 997, September 12, 2008	*Fate of Pesticides and Nutrients in the Urban Environment.*	empirical dataset based on measurements taken in an urban watershed in Ithaca, NY. The study was limited to 3 types of urban land use, Forested urban, general urban and fertilized lawns. Outcomes were useful only in a descriptive manner because load differences were not significant between land use types unless precipitation and runoff characteristics met certain conditions. General export coefficients that are divided between dissolved and particulate fraction may have some use for comparison. these types of data are rare therefore tabulated
11	Dr. John Sansalone of the Dept. of Environmental Engineering Sciences at the Univ. of Florida. February 2009	*TARP Field Test Performance Evaluation of Sorbtive Filter using Sorbtive Media for Imbrium Systems Corporation*	very detailed and contains conclusive evidence with respect to both solids and P removal efficiencies for a single active sorbtive media stormwater treatment system The system monitored removed 78% of TP with 12% confidence limits
12	LSRCA	*Black River, East Holland River, West Holland River, Uxbridge Brook, Maskinonge River subwatershed Plans*	provide projected phosphorous loadings under subwatershed development scenarios. Berger 2010 provide projections of development phosphorous loading based on the 2010 modelling data - also provides details that characterize the land uses in each subwatershed pertinent to phosphorus loading - details provided in these reports are useful for assessing the conditions of development sites that could contribute to phosphorus loading in the subwatershed

Appendix 2
Table of Construction Phase BMPs, Descriptions and Efficiencies

Description	Beneficial Management Practice (BMP) Category	Addresses what Loading Source?	Applicable to what site features?	Known Limitations of BMP	Reported Efficiency	Efficiency References (see Appendix 7)	Efficiency to Use
Anionic Polymer Runoff Treatment - flocculation and or coagulation of fine particles using polymers for the clarification of construction runoff to enhance downstream detention practices.	Runoff capture	Surface Runoff	Interior site	Requires proper design and monitoring to ensure that floc or polymer-dosed water does not get released to the environment	TSS = 88 to 94% (mean = 91%) with TSS influent concentration of 171 to 706 mg/L	41	91%
Bioretention Systems - biologic activity to filter/clean stormwater (infiltration basins, rainwater gardens, surface sand filters)	Filtration Systems	Surface runoff	Interior site	Can't treat large drainage areas, susceptible to clogging, consume a large area, high cost	TSS = 95% (45cm) TP = -1552-80	8-10, 12, 13, 34-38, 40	Site and design specific
Check Dams- permanent or temporary barrier that present erosion and promote sedimentation by slowing flows and filtering	Soil erosion control	Surface runoff		Requires periodic repair and sediment removal, removal can be expensive and difficult			Not available
Construction Phasing - creating a specified work schedule that coordinate the time of land-disturbing activities and the installation of erosion and sedimentation control measures to minimize the area and duration of exposed soil	Construction practices		Interior site, Stream, Drainage Channels	Requires more complex planning; potentially more costly as grading in done in multiple steps	TSS= 40%	112, article 54	Site specific
Dry Detention Ponds - collects stormwater runoff and store temporarily until infiltration and evaporation can occur	Detention Systems	Surface runoff	Interior site	For drainage areas greater than 10 acres, clogging, marginal removal of pollutants, unattractive, collect trash and debris	TSS = 61% TP = 0-20% Soluble P = -11%	104, 109	10%
Flow Splitters - restricts stormwater flows and creates bypass around the exposed areas	Flow Control Structures	Surface runoff	Interior site	Can create flow reversal, only for small systems			Site specific

Description	Beneficial Management Practice (BMP) Category	Addresses what Loading Source?	Applicable to what site features?	Known Limitations of BMP	Reported Efficiency	Efficiency References (see Appendix 7)	Efficiency to Use
Inlet Protection- prevention methods around storm drains limiting the amount of sediment entering the unit (sediment filter, sand bag barrier, geotextile barrier, compost biofilters, etc)	Filtration Systems	Impervious areas	Interior site	Needs to be properly maintained, not as effective for find-grained sediments or large loads; compost biofilters increase in efficiency with increased number of rolls used	TSS = 69% (for 5 rolls each 45cm diameter compost biofilters	114	69%
Maintenance - maintaining the BMPs that you currently have in place	House-keeping techniques	House-keeping	Entire site	Expensive, needs to be done somewhat frequently			Site specific
Mulches and Fibre or Geotextile Blankets and Mats - the application of organic materials, blankets or mats to form a temporary protective soil cover	Soil erosion control	Exposed soil, surface runoff	Interior site, Stream, Drainage Channels	Must be installed properly to be effective, mulching may not be effective on slopes greater than 3:1	29% - 99% TSS reduction (median = 90%) for various natural mulches and fiber blankets on slopes between 9% and 34% with various soils	112	90%
Pavement Management - cleaning streets and construction areas (sweeping, minimizing sand and salt applications, etc)	Housekeeping techniques	Impervious areas	Interior site				Site specific
Silt Fences - temporary barrier to retain sediment along the perimeter and watercourses on a construction site	Filtration Systems	Stockpiling, watercourse and perimeter protection	Stream, Site perimeter, Stockpiles	Not always effective, proper installation is crucial, maintenance and inspection is required frequently, poor efficiency with fine particles	TSS = 70% (median)	112, article 56	70%

Description	Beneficial Management Practice (BMP) Category	Addresses what Loading Source?	Applicable to what site features?	Known Limitations of BMP	Reported Efficiency	Efficiency References (see Appendix 7)	Efficiency to Use
Soakaways-Infiltration Trenches - area to capture stormwater runoff, retain it, and then infiltrate it into the ground over a period of days	Infiltration Systems	Surface runoff	Interior site	Potential high failure if not designed properly, possible groundwater contamination, not for high sediment/polluted areas, cannot use in industrial areas, requires large flat area, maintenance, inspection	TSS = 95% TP = 50-70% Soluble P = 51%	6, 104	60%
Structural Methods - installation of inlet/outlet riprap, permanent diversion, temporary diversions	Soil erosion control	Stream and watercourse runoff	Stream, Drainage Channels	Removal of temporary diversion structures can be expensive and time consuming			Site and design specific
Vegetative Filter Strips/Stream Buffers - maintain densely vegetated, uniformly graded areas that treat sheet flow from adjacent impervious surfaces	Filtration Systems	Surface runoff	Interior site	Can't use in hilly areas, difficult to monitor effectiveness, can use in contaminate areas, large area required, ineffective if improperly graded	TSS=70% TP = 60-70%	6, 42, 104	65%
Vegetative Methods - vegetative stabilization on site to prevent erosion, e.g., temporary seeding, sod	Soil erosion control	Exposed soil, surface runoff	Interior site, Stream, Drainage Channels	Cannot be implemented during off-seasons. In the fall heavy mulches will be used instead of vegetation.	99% TSS reduction (biomass at 2464 lb/acre compared to zero.)	113	99%
Vehicle Tracking Pad - entrance pad at construction access locations reduces the amount of mud transported onto paved roads by vehicles or surface runoff	Construction practices	Surface runoff	Interior site	Some sites will require extensive maintenance, some pads can become quickly saturated and plugged reducing effectiveness	Not available		Site specific

Description	Beneficial Management Practice (BMP) Category	Addresses what Loading Source?	Applicable to what site features?	Known Limitations of BMP	Reported Efficiency	Efficiency References (see Appendix 7)	Efficiency to Use
Wet Detention Ponds - stormwater pond with permanent pool. Provides peak flow control and water quality treatment	Retention Systems	Surface runoff	Interior site	For drainage areas greater than 10 acres, high cost, large area required, engineered design required, warm water discharges. Less effective on fine soils	TSS = 80% TP = 42-85% Soluble P = 66%	104-106, 109	63%

Appendix 3
Database Tool Users Manual

Hutchinson
Environmental Sciences Ltd.

Phosphorus Budget Tool in Support of Sustainable Development for the Lake Simcoe Watershed

Database User's Manual

Prepared By: **Stoneleigh Associates Inc.**
Prepared For: **Ontario Ministry of the Environment**

Project #: J110008
Date: March 30, 2012

Using the Lake Simcoe Phosphorus Loading Database Tool

Introduction

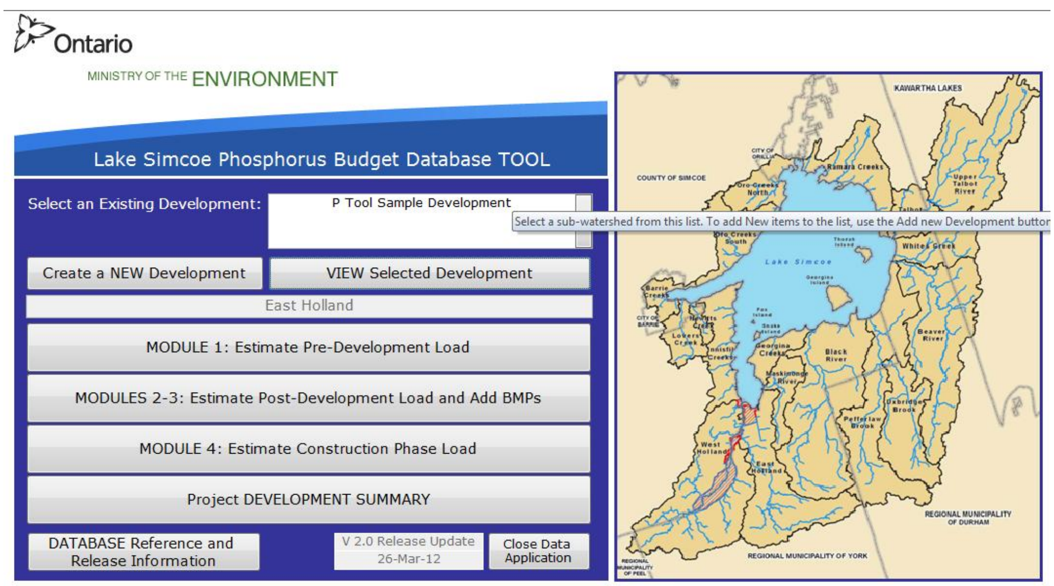
The "*Phosphorus Budget Guidance Tool to Guide New Development in the Lake Simcoe Watershed"* (the "Tool"; HESL, 2012) is intended for use by the development community, municipalities, the Ministry of Environment (MOE) and the Lake Simcoe Region Conservation Authority to facilitate review of major new development applications for their compliance with Policy 4.8e of the Lake Simcoe Protection Plan. The Tool provides a transparent and technically-sound approach to estimate phosphorus (P) loading from stormwater runoff in the pre-, post- and construction phases of development in the Lake Simcoe watershed. The Tool consists of three elements:

1. A **Technical Guidance Manual** that provides the reference material used in developing the Tool, the rationale for the development of the Tool, and implementation guidance in line with Policy 4.8e of the LSPP,
2. A **Microsoft ACCESS© Database Tool** that facilitates the calculation of a phosphorus budget for new development in accordance with the technical guidance, and
3. A **Database User's Manual** explaining the operation of the database.

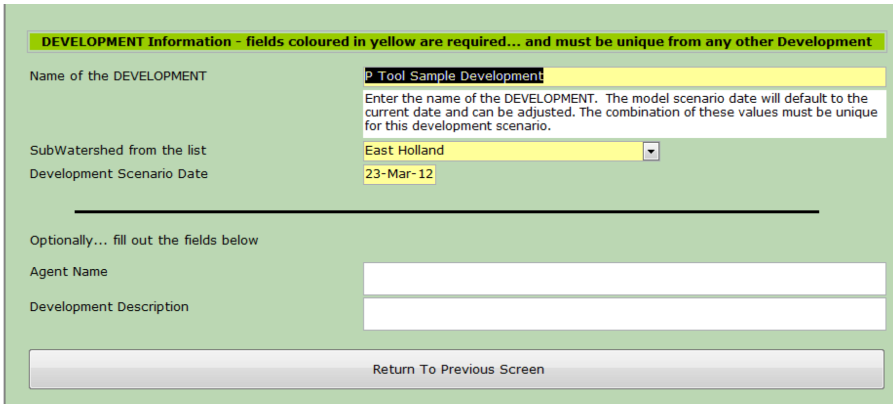
The following Database User's Manual provides step-by-step instructions to navigate the Database Tool. It is included as Appendix 3 of the Technical Guidance Manual of the Tool and is not intended as a "stand alone" description of the Tool or the estimation process, but rather as a set of instructions for operating the Microsoft ACCESS© Database Tool. ***The user must always rely on the Technical Guidance Manual as the primary technical source for instruction***.

Instructions

- Save the database file to any folder. All support reference data tables are warehoused within this single file.
- The database opens to a main screen. All features of the database are accessed from this opening view. The version code and date are displayed in the lower portion of this screen and cannot be adjusted by users.

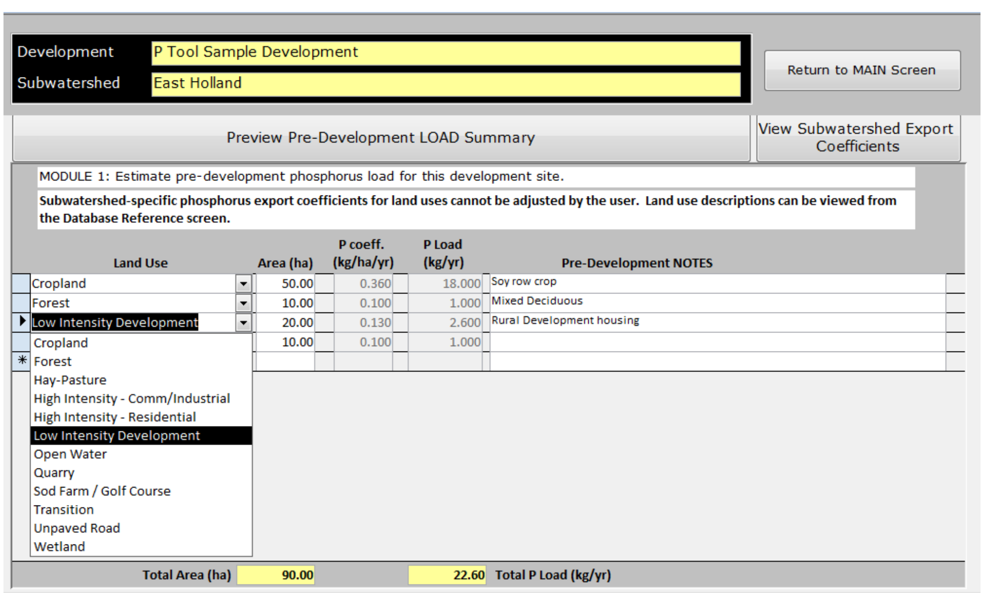


- To model a new development, the user will first need to enter information about the development. A unique development name, sub-watershed and date combination are required as input. Other optional information includes the developer or agent name and a description of the development.

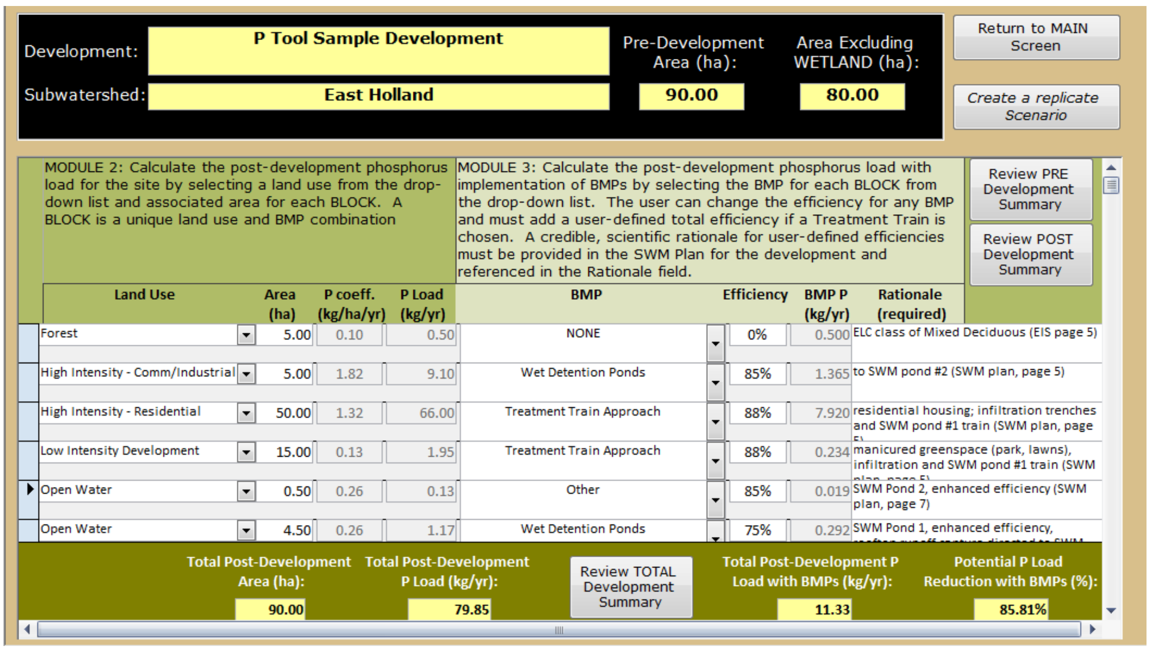


- The modules of the Tool are completed in sequence as information is entered for the pre-development, post-development and construction phase scenarios.

- MODULE 1: Pre-development conditions are entered by the user as displayed with the screen below. Users must have entered a new development or selected a previously entered development using the drop-down box on the main screen before they will be able to gain access to this screen. The landuse drop-down list options are contained in a reference table along with subwatershed-specific P export coefficients. The user must select a land use classification from among the options presented. The export coefficients are populated automatically by the tool and may NOT be adjusted by the user. A listing of all sub-watershed P export coefficients can be viewed from this screen by selecting the View Subwatershed Export Coefficients tab. After users enter the area values for each land use (to the nearest hundredth of a hectare) and press the tab key to advance to the notes field, the P load in kg/year is derived automatically. The total area of the development site is also derived automatically along with a total P load for the site and is displayed at the bottom of the screen. The user must verify that the total development area displayed is the same as in the development plan. A summary of the pre-development conditions can be viewed using the button provided. A sample summary report is shown in the Appendix of this document.



- MODULE 2: Post-development conditions can be added only after pre-development conditions have been entered in Module 1 (a blank screen will appear if this is not the case). The user must have also selected the development for which pre-development conditions were entered using the drop-down box on the main screen to display the information screen for Module 2. The name and total area of the development is displayed at the top of the screen. This information may not be adjusted, and displays and updates automatically. In the lower part of the screen, the user selects a land use from the drop-down menu and enters the area of that land use (to the nearest hundredth of a hectare) for each post-development block. The user must select a land use classification from among the options presented. The export coefficients are populated automatically by the tool and may NOT be adjusted by the user. A block is a unique combination of a land use and a specific Best Management Practice (BMP) that will be applied to that land use in Module 3. If the pre-development scenario in Module 1 contained wetland, it will automatically display on this screen and may not be altered by the user, under the assumption that development of wetlands is not approved in the Lake Simcoe watershed. The P export coefficient for each land use is a default value that is automatically entered from the lookup table and may not be adjusted.



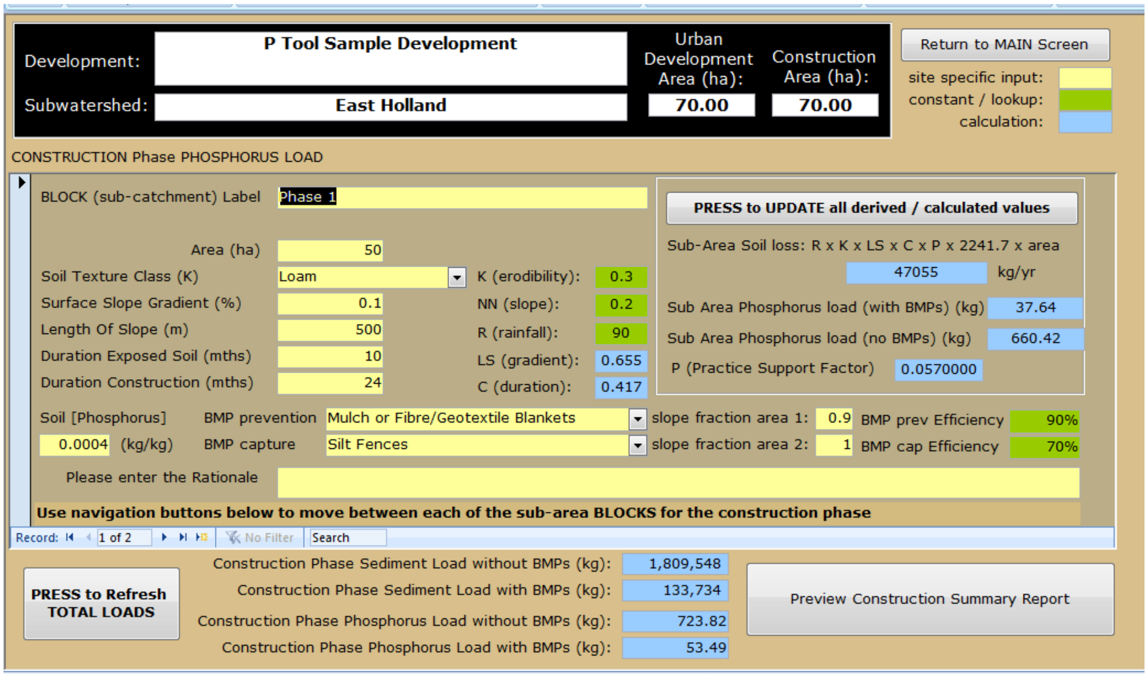
- MODULE 3: This step entails the selection of a BMP from the drop-down list for each post-development block. Some BMPs have defined P removal efficiencies whereas others do not. Percent efficiency values for the selected BMPs will be automatically displayed in the Efficiency field, but can be adjusted by the user. A BMP cannot be applied to wetlands. If a treatment train approach is selected from the drop-down menu, then the user must enter the total P removal efficiency for the approach. If different efficiency values for a specific BMP, a different BMP or a treatment train approach are provided by the user, credible scientific research and rationale in support of those value(s) or approach must be documented in the Stormwater Management (SWM) Plan for the development. The use of “custom” BMPs and efficiencies means that the application will be subject to a greater degree of review by the approving agency or agencies and so may require more time to assess. If users select “Other” or “Treatment Train” as a BMP, or adjust the pre-defined efficiency value, they will be prompted to enter a rationale. A brief rationale can be entered to the rationale field (up to 255 characters may be typed), but it should only provide a summary, and should refer the reviewer to the Stormwater Management (SWM) Plan for the full technical justification. Any change in the efficiency value from the base reference value provided will be reflected in the

Post-Development Summary report. Both the base reference efficiency and the user-adjusted value will be displayed along with an information note. A summary of the total development can be produced from this page using the button located at the middle of the bottom of the screen. Construction phase data will be displayed on the summary if it has previously been entered in Module 4. A sample summary report is shown in the Appendix of this document.

- Multiple scenarios of the same development area can be created to compare P loads with different combinations of post-development land uses and BMPs. A procedure to create a replicate scenario can be executed using the button marked 'Create a replicate scenario' at the top right of the screen (and shown below). A new Development will be created (and the message below will be displayed) when this button is pressed. The name of the replicated development will be the same as the one that the user has selected with a suffix added containing the words '-replicate scenario' followed by a data and time stamp. This enables users to create multiple replicates on the same day to assess different BMP scenarios. Users can adjust the name of a replicate scenario by returning to the main screen, selecting it from the drop-down list, then selecting 'VIEW Selected Development' tab. Adjustments to the post-development information will also be required to distinguish the replicate scenario from the original.

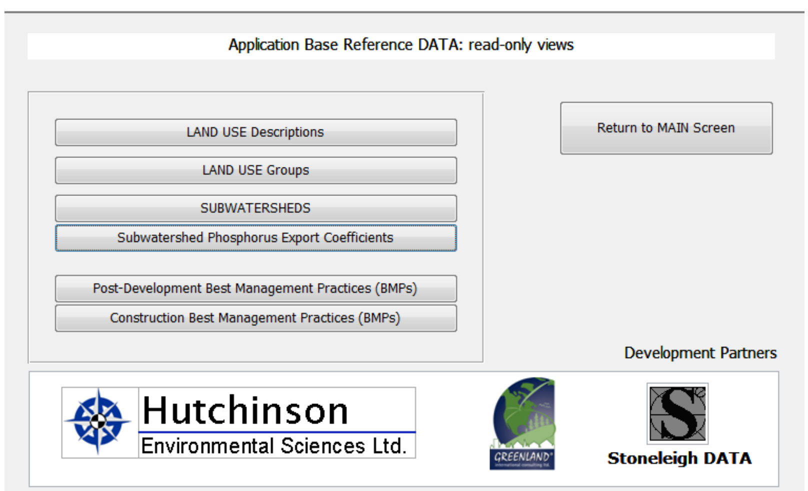
- MODULE 4: Construction phase information can be added only after pre-development conditions have been entered in Module 1(a blank screen will appear if this is not the case). The user must have also selected a development using the drop-down box on the main screen to display the information screen for Module 4. The following screen illustration shows both the total urban development and construction area at the top of the screen. These values may not be adjusted and are displayed automatically. There may be some delay in the update of the construction area value as users enter information for each construction block. A construction phase block is a subarea of the development site with relatively uniform slope and soil conditions where one prevention and one capture BMP, or one treatment train will be applied. There is no limit to the number of blocks in each construction development.

  - Using the lower part of the screen, enter the values shaded in yellow. Values in green will be entered as either constants or filled in automatically from reference lookup tables in the database. Fields shaded in blue are derived by the database using the formulae described in the Guidance Manual.
  - Enter the required input values for each construction phase block. Each block can be accessed using the record selectors at the bottom of the inner construction phase data field. The derived values will update automatically as new values or entered or changed.
  - The user can adjust the pre-defined BMP efficiency values, but must provide a credible scientific rationale for doing so in the SWM plan for the development. A brief rationale can be entered in the field provided.
  - A summary of the Construction Phase site sediment loss and P export can be viewed using the button labelled "Preview Construction Summary Report" provided at the lower right of the screen. A sample summary report is shown in the Appendix of this document.



- An overall summary of each Module can be displayed using the button on the main screen marked ‘Project Development Summary’. This summary includes each of the four module summary reports and a final conclusion about P load reduction or increase as a result of development and construction activities. A sample “Project Development Summary” report is provided in the Appendix of this document.

- Base reference data used in the model calculations can be viewed by clicking the button marked ‘Database Reference and Release Information’. All data from these views are read-only and may not be adjusted by the user.



APPENDIX A: Sample Summary Reports

A1 – Pre-Development Report
A2 – Post-Development Report
A3 – Construction Phase Report
A4 – Summary Report

A1 – Pre-Development Report

Ontario
MINISTRY OF THE ENVIRONMENT

Database Version: V 2.0 Release Update
Update Date: 28-Mar-12

PRE-DEVELOPMENT Phosphorus LOAD

DEVELOPMENT: **P Tool Sample Development**
Subwatershed: **East Holland**

Land Use	Area (ha)	P coeff. (kg/ha)	P Load (kg/yr)
Cropland	50	0.36	18.00
Forest	10	0.10	1.00
Low Intensity Development	20	0.13	2.60
Wetland	10	0.10	1.00
TOTALS:	**90**		**22.60**

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A2 – Post-Development Report

Database Version: V 2.0 Release Update
Update Date: 28-Mar-12

Ontario
MINISTRY OF THE ENVIRONMENT

POST-DEVELOPMENT Phosphorus LOAD - BMPs applied

Subwatershed: East Holland

Land Use	Area (ha)	P coeff. (kg/ha)	Best Management Practice applied with P Removal Efficiency		BMP P Load (kg/yr)
DEVELOPMENT:	**P Tool Sample Development**				
Forest	5	0.10	NONE	0%	0.50
			ELC class of Mixed Deciduous (EIS page 5)		
High Intensity - Comm/Industrial	5	1.82	Wet Detention Ponds	**85%**	1.36
			to SWM pond #2 (SWM plan, page 5)		
NOTE: BMP efficiency has been adjusted from the reference provided value by 22% (from 63% to 85%)					
High Intensity - Residential	50	1.32	Treatment Train Approach	88%	7.92
			residential housing; infiltration trenches and SWM pond #1 train (SWM plan, page 5)		
Low Intensity Development	15	0.13	Treatment Train Approach	88%	0.23
			manicured greenspace (park, lawns), infiltration and SWM pond #1 train (SWM plan, page 5)		
Open Water	4.5	0.26	Wet Detention Ponds	**75%**	0.29
			SWM Pond 1, enhanced efficiency, rooftop runoff capture directed to SWM (SWM Plan, page 6)		
NOTE: BMP efficiency has been adjusted from the reference provided value by 12% (from 63% to 75%)					
Open Water	0.5	0.26	Other	85%	0.02
			SWM Pond 2, enhanced efficiency (SWM plan, page 7)		
Wetland	10	0.10	NONE	0%	1.00
			cannot be altered		

Post-Development Area Altered:	**90.00**		**P Load (kg/yr)**
Total Pre-Development Area:	**90.00**	Pre-Development:	**22.60**
Unaffected Area:	**0**	Post-Development Load:	**79.85**
		Change (Pre- Post):	**-57.25**
			253% Net Increase in Load
		Post-Development (with BMP):	**11.33**
		Change (Pre-Post):	**11.27**
			49.86% Net Reduction in Load

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A3 – Construction Phase Report

Database Version: V 2.0 Release Update
Update Date: 28-Mar-12

Ontario
MINISTRY OF THE ENVIRONMENT

CONSTRUCTION Site Sediment and Phosphorus LOAD

DEVELOPMENT: **P Tool Sample Development**
Subwatershed: **East Holland**

Site Specific input: | constant / lookup: | *calculation:*

Sub Area: Phase 1

Duration of Construction (months):	24	R (Rainfall / Runoff for Lake Simcoe)	90
Duration of Exposed Soil (months):	10	K (Soil erodability factor):	0.3
Surface Slope Gradient (%):	0.1	NN (determined by slope):	0.2
Length Of Slope (m):	500	BMP prevention Efficiency:	90%
Slope Area (ha):	50	BMP capture Efficiency:	70%
% slope erosion prevention applied to:	0.9	LS (slope length gradient factor):	*0.65*
% slope runoff capture applied to:	1	C (portion of year of exposed soil):	*0.42*
subwatershed Soil [P] (kg/kg):	0.0004	P (prevention + capture):	*0.06*
		Soil Loss (kg/year):	*47054.96*
		Phosphorus load (kg):	*37.64*

Sub Area: Phase 2

Duration of Construction (months):	12	R (Rainfall / Runoff for Lake Simcoe)	90
Duration of Exposed Soil (months):	4	K (Soil erodability factor):	0.18
Surface Slope Gradient (%):	0.1	NN (determined by slope):	0.2
Length Of Slope (m):	250	BMP prevention Efficiency:	75%
Slope Area (ha):	20	BMP capture Efficiency:	60%
% slope erosion prevention applied to:	0.5	LS (slope length gradient factor):	*0.65*
% slope runoff capture applied to:	1	C (portion of year of exposed soil):	*0.33*
subwatershed Soil [P] (kg/kg):	0.0004	P (prevention + capture):	*0.25*
		Soil Loss (kg/year):	*39624.27*
		Phosphorus load (kg):	*15.85*

Developed AREA (ha): 70

	Total
Construction Phase Phosphorus Load with BMPs (kg):	53.49
Construction Phase Phosphorus Load no BMPs (kg):	723.82

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A4 – Summary Report

DEVELOPMENT: P Tool Sample Development
Subwatershed: East Holland

SUMMARY WITH IMPLEMENTATION OF BMPs	P Load (kg/yr)
Pre-Development:	**22.60**
Construction Phase Amortized Over 8 Years :	6.69
Post-Development:	**11.33**
Post Development + Amortized Construction:	**18.02**
Pre-Development Load - Post-Development Load:	**11.27**
Conclusion:	**50% Reduction in Load**
Pre-Development Load - (Post-Development + Amortized Construction Load):	**4.58**
Conclusion:	**20% Reduction in Load**

Based on a comparison of Pre-Development and Post-Development loads, and in consideration of Construction Phase loads, the Ministry would encourage the Municipality to:

Approve development as site specific appropriate.

March-28-12

Appendix 4
Analysis of Berger (2010) Export Coefficients

Corrected Export Coefficients Derived from Berger P Loads and Land Use Areas

Phosphorus loads (kg/yr) from land use areas were adjusted by adding loads from:

1) **Groundwater** proportionally by area to all land use categories except High Intensity Development,
2) **Tile Drainage** to Crop areas, and
3) **Stream Bank Erosion** proportionally by area to Forest, Wetland and Transition areas

Groundwater loads were not allocated to High Intensity Development areas considering that these areas have a large amount of impermeable surfaces, thereby reducing groundwater loads. Stream Bank Erosion was only allocated to 'natural' land cover areas assuming that streams primarily occur in these land areas. Refined land use data would be required to determine the proportion of P loads from stream bank erosion in other land class areas (e.g., proportion of streams running through agricultural area or urban area). The corrected loads were then used to calculate P export (kg/ha/yr) for each land use (Table 1).

As previously noted, there is considerable variance in P export coefficients among subwatersheds, but much of the variance occurs among unmonitored subwatersheds (Table 2, Figure 1). Export coefficients derived for the East Holland River (EH) subwatershed are higher on average than those for the other monitored subwatersheds (with the exception of LID, which is suspected as being an error and removed from Table 2 and Figure 1 results). Variance in export coefficients for the monitored subwatersheds is greatly reduced when EH coefficients are removed however, there is still considerable variance in export coefficients among monitored subwatersheds for Turf-Sod and Unpaved Road.

Variance in export coefficients was further assessed using Principal Component Analysis (PCA) (Figure 2). The first two axes of the PCA explain 94% of the variation in export coefficients for land cover classes between the subwatersheds. The first PCA axis is best described by variation in export coefficients for Unpaved Roads (UNPAV) and the second axis is best described by variation in export coefficients for High Intensity Development (HID) and Cropland (CROP). Hay/pasture (HAY_PAST) and Quarry (QU) contribute nearly equally to the variation along the first and second PCA axes. Contribution of the other land cover classes to the variation explained by the first and second PCA axes is negligeable. Overall, the results of the PCA indicate that the East Holland River River (EH), Oro Creeks North (ON), Hawkestone (HA), Barrie Creeks (BA) and Georgina Creeks (GE) differ from the other subwatersheds by having higher export coefficients for UNPAV, HID, CROP, HAY_PAST and QU.

Table 1. Phosphorus Export Coefficients (kg/ha/yr) for Land Cover Types in the Lake Simcoe Subwatersheds

no value
GW inputs only

Subwatershed	Phosphorus Export (kg/ha/yr)									
	Cropland	Forest	Hay-Pasture	High Intensity Develop-ment	Low Intensity Develop-ment	Quarry	Transitio n	Turf-Sod	Unpaved Road	Wetland
East Holland	0.357	0.100	0.116	0.659	0.013	0.530	0.161	0.243	3.715	0.099
Beaver River	0.218	0.022	0.040	0.381	0.193	0.063	0.040	0.014	0.049	0.020
Black River	0.229	0.045	0.075	0.393	0.167	0.152	0.057	0.023	0.598	0.044
Hawkestone Creek	0.185	0.031	0.097	0.254	0.089	0.098	0.036	0.061	2.394	0.026
Lovers Creek	0.164	0.060	0.071	0.237	0.067	0.063	0.064	0.168	0.015	0.053
Pefferlaw-Uxbridge Brook	0.109	0.034	0.055	0.206	0.131	0.041	0.044	0.022	0.413	0.035
Whites Creek	0.226	0.096	0.103	0.286	0.149		0.113	0.424	0.682	0.094
Barrie Creeks	0.887	0.182	0.231	1.802	0.102	0.066	0.213		0.050	0.179
Georgina Creeks	0.598	0.018	0.498	1.048	0.013		0.122	0.633	1.152	0.016
Hewitts Creek	0.272	0.182	0.090	0.253	0.057		0.161		1.046	0.062
Innisfil Creeks	0.379	0.086	0.086	0.431	0.103	0.587	0.096	0.124	0.638	0.082
Maskinonge River	0.188	0.121	0.091	0.339	0.118	0.210	0.132		0.241	0.121
Oro Creeks North	0.953	0.049	0.619	1.696	0.060	1.348	0.231		2.911	0.040
Oro Creeks South	0.137	0.041	0.036	0.207	0.020		0.049	0.020	0.217	0.041
Ramara Creeks	0.309	0.052	0.048	0.103	0.043		0.056	0.237	0.217	0.048
West Holland	0.255	0.105	0.065	0.245	0.042	0.206	0.108	0.393	0.573	0.103

Table 2. Summary of Phosphorus Export Coefficients for the Lake Simcoe Watershed Derived from Berger (2010)

Phosphorus Export (kg/ha/yr)										
	Cropland	Forest	Hay-Pasture	High Intensity	Low Intensity	Quarry	Transition	Turf-Sod	Unpaved Road	Wetland
All Subwatersheds										
Mean	0.341	0.075	0.147	0.525	0.095	0.283	0.101	0.210	0.856	0.064
Maximum	0.953	0.182	0.619	1.802	0.193	1.348	0.231	0.633	2.911	0.179
75th Percentile	0.344	0.101	0.100	0.412	0.127	0.209	0.127	0.354	1.046	0.088
Median	0.229	0.052	0.086	0.286	0.095	0.125	0.096	0.146	0.598	0.048
25th Percentile	0.186	0.038	0.060	0.241	0.058	0.064	0.052	0.033	0.241	0.038
Minimum	0.109	0.018	0.036	0.103	0.013	0.041	0.036	0.014	0.049	0.016
Monitored Subwatersheds										
Mean	0.213	0.055	0.080	0.345	0.133	0.158	0.074	0.136	1.309	0.053
Maximum	0.357	0.100	0.116	0.659	0.193	0.530	0.161	0.424	3.715	0.099
75th Percentile	0.227	0.078	0.100	0.387	0.163	0.139	0.089	0.205	1.966	0.073
Median	0.218	0.045	0.075	0.286	0.140	0.081	0.057	0.061	0.640	0.044
25th Percentile	0.175	0.033	0.063	0.245	0.099	0.063	0.042	0.022	0.460	0.031
Minimum	0.109	0.022	0.040	0.206	0.067	0.041	0.036	0.014	0.049	0.020
Unmonitored Subwatersheds										
Mean	0.442	0.093	0.196	0.681	0.067	0.483	0.130	0.347	0.874	0.077
Maximum	0.953	0.182	0.619	1.802	0.118	1.348	0.231	0.633	2.911	0.179
75th Percentile	0.598	0.121	0.231	1.048	0.102	0.587	0.161	0.453	1.073	0.103
Median	0.309	0.086	0.090	0.339	0.058	0.210	0.122	0.315	0.606	0.062
25th Percentile	0.255	0.049	0.065	0.245	0.043	0.206	0.096	0.208	0.235	0.041
Minimum	0.137	0.018	0.036	0.103	0.013	0.066	0.049	0.124	0.217	0.016
Monitored Subwatersheds Excluding EH										
Mean	0.188	0.048	0.074	0.293	0.133	0.084	0.059	0.119	0.827	0.045
Maximum	0.229	0.096	0.103	0.393	0.193	0.152	0.113	0.424	2.394	0.094
75th Percentile	0.224	0.056	0.091	0.358	0.163	0.098	0.063	0.141	0.682	0.051
Median	0.202	0.040	0.073	0.270	0.140	0.063	0.050	0.042	0.598	0.040
25th Percentile	0.169	0.032	0.059	0.241	0.099	0.063	0.041	0.022	0.413	0.029
Minimum	0.109	0.022	0.040	0.206	0.067	0.041	0.036	0.014	0.049	0.020
East Holland										
P Export (kg/ha/yr)	0.357	0.100	0.116	0.659	-	0.530	0.161	0.243	3.715	0.099

Figure 1. Boxplots showing variance in export coefficients derived from Berger (2010) for the Lake Simcoe Subwatersheds. Boxes represent 25th percentile, median and 75th percentile, whiskers are the minimum and maximum values, and the mean is denoted as the black dot.

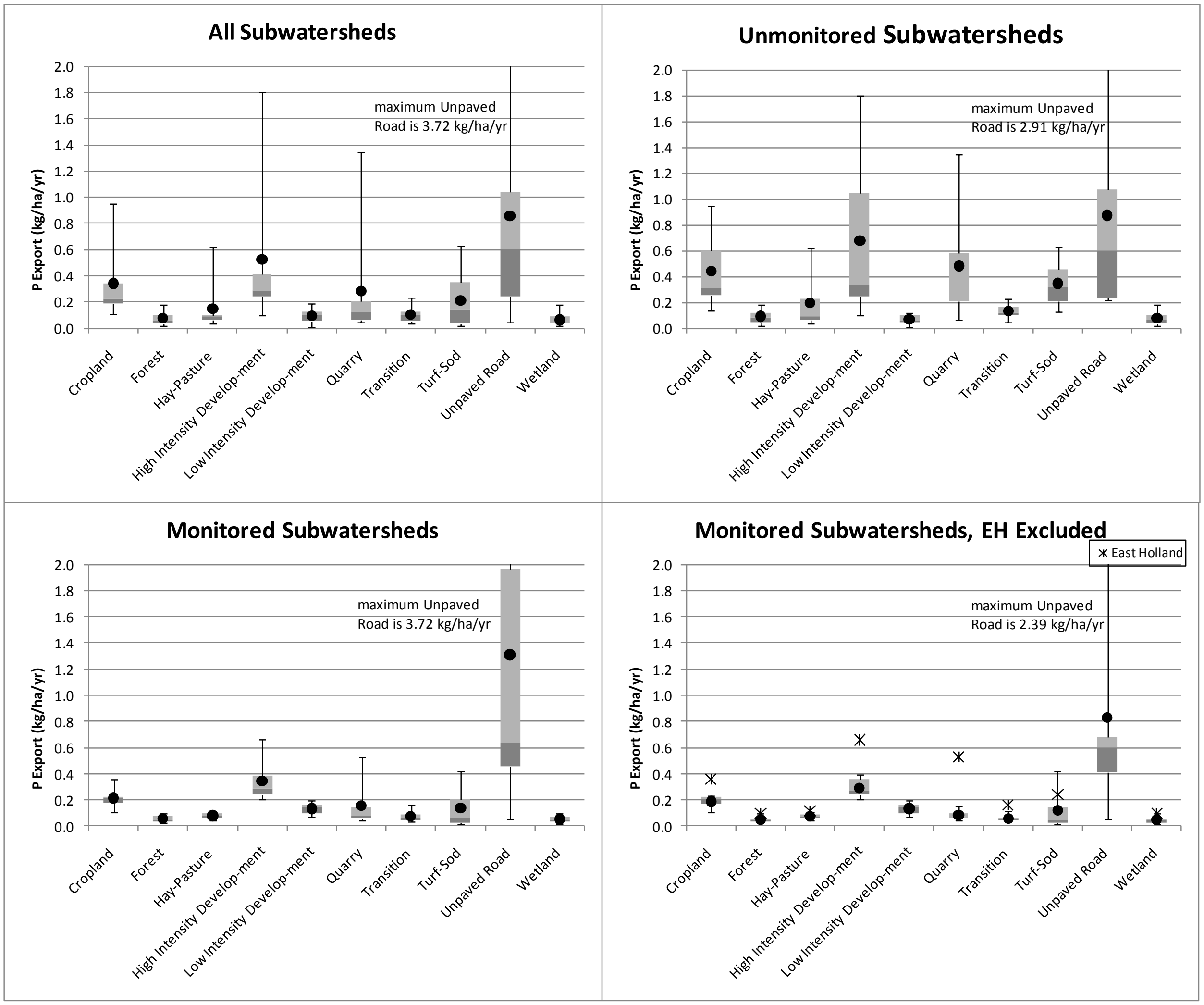
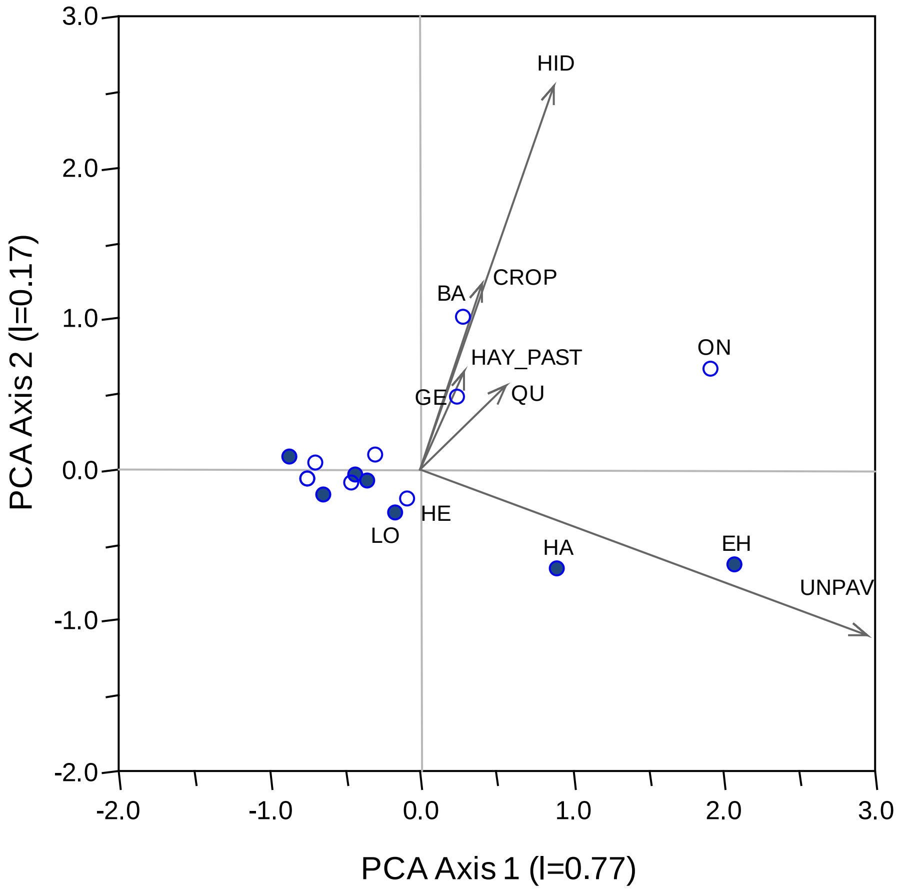


Figure 2. PCA of P export coefficients for land cover classes in Lake Simcoe subwatersheds. Solid circles indicate monitored subwatersheds.



Note: Factor arrows are not shown for Forest, Wetland, Transition, Turf-Sod or Low Impact Development as the contribution of these factors to the first two PCA axes is negligible.

Some variation in phosphorus export between subwatersheds is expected for a given land cover type due to differences in environmental factors such as soil characteristics and runoff conditions. The variation in P export coefficients for the Lake Simcoe subwatersheds was further investigated based on environmental factors used in the CANWET model. These included Soil K Factor (erosion coefficient), Slope Length, Base Runoff and Soil P as reported in Berger (2010) for each land cover type in each subwatershed.

In a PCA of the environmental factors, the first PCA axis describes 36% of the variation in the data set and is related to soil conditions (Soil P and Soil K Factor) (Figure 3). Slope length and base runoff best describe variation along the second axis, which describes 29% of the variation in the data set. It should be noted that the environmental factors for Quarry were eliminated from the PCA as these were strongly influenced by slope length and had a large influence on the ordination.

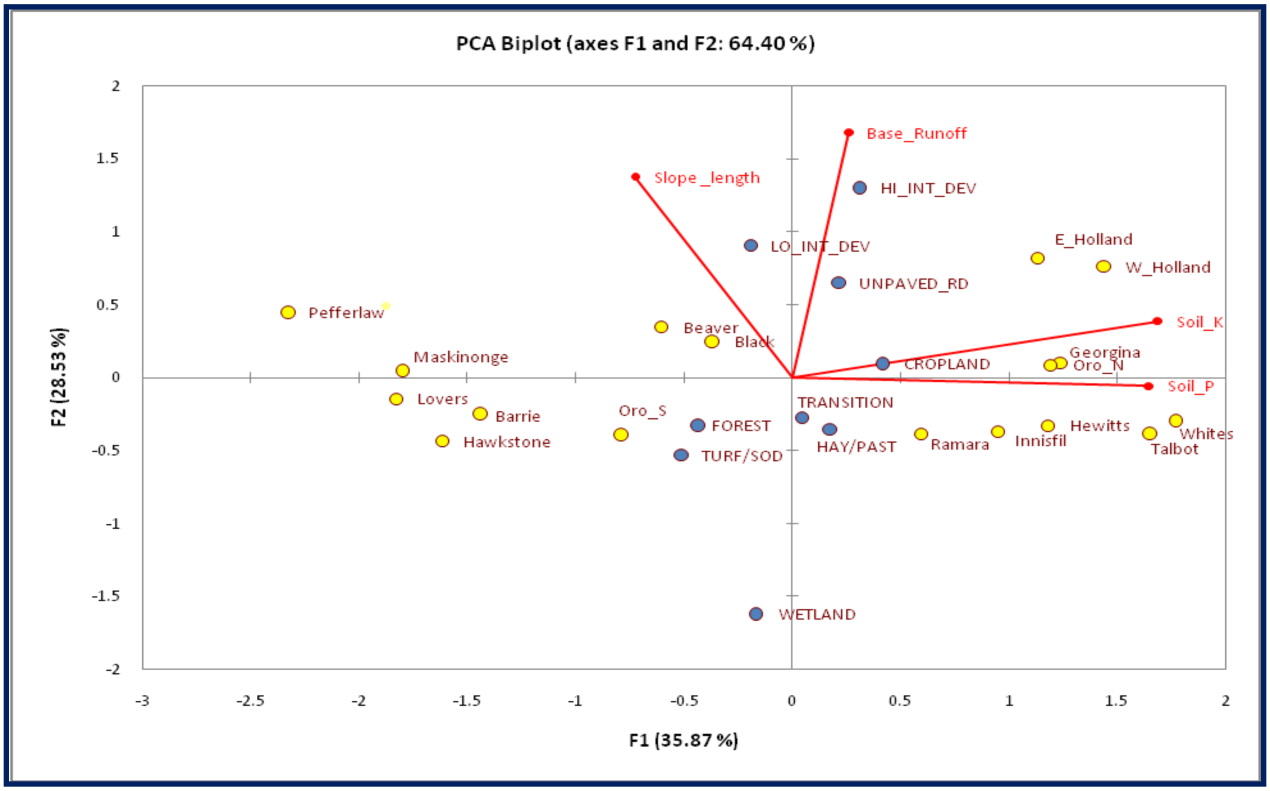
The centroids of the subwatersheds are separated primarily along the first PCA axis indicating that they differ along a gradient of Soil K and Soil P (increasing from left to right in the PCA biplot). The East Holland and West Holland subwatersheds are also characterized by higher base runoff in comparison to the other subwatersheds. Land cover types, by contrast, are separated primarily along the second PCA axis with High Intensity Development, Low Intensity Development and Unpaved Roads characterized by higher base runoff and greater slope length in comparison to the other land cover types.

As previously described, the East Holland River, Georgina Creeks and Oro North subwatersheds generally have higher export coefficients than the other subwatersheds. Centroids for these subwatersheds plot in the top right quadrat of the PCA indicating that they have generally higher soil K factors, Soil P and base runoff than the other subwatersheds, which would be consistent with higher P export. The West Holland River subwatershed plots in the same quadrat, however, export coefficients for this subwatershed are similar to the mean values.

Barrie Creeks subwatershed also had higher export coefficients, particularly for Cropland and High Intensity Development, but this subwatershed has environmental factors similar to other subwatersheds with comparatively lower export coefficients (i.e., Lovers, Maskinonge, Hawkestone).

Hawkestone subwatershed had high export coefficients for Unpaved Road and displayed relatively high slope lengths for this land cover class (not shown). Other subwatersheds had similarly high slope lengths for unpaved road areas, but did not have similarly high export (e.g., Pefferlaw, Lovers) for this land cover.

Figure 3. PCA biplot of environmental factors (n=4) for land cover classes in Lake Simcoe Subwatersheds (n=148). Yellow circles represent the centroids of the subwatershed sample scores while blue circles represent the centroids of the land cover type sample scores.



Summary and Recommendations

While patterns in the environmental factors appear to explain some variation in export coefficients, there is no clear, consistent relationship (e.g., weak correlations between environmental factors (actual values and PCA axis sample scores) and export coefficients) when considering both monitored and unmonitored subwatersheds. This may reflect complexities of data manipulation and calibration in CANWET or the use of other unknown coefficients or input parameters that influence phosphorus export in the model. In addition, there may be error in the allocation of phosphorus loads from groundwater, tile drainage and streambank erosion to the different land classes.

Despite the above uncertainties, the export coefficients derived for the monitored subwatersheds display little variability within land cover classes with few exceptions. The East Holland River has higher export coefficients relative to all other monitored subwatersheds, which is likely due to higher soil K Factors, soil P and base runoff of land cover areas in this subwatershed. For the remaining monitored subwatersheds, variation in export for unpaved roads is mainly due to high export from Hawkestone subwatershed which has very high slope length for this parameter in comparison to the other monitored subwatersheds. Similarly, the variability in turf/sod is mostly attributed to the high export coefficient for the Whites Creek subwatershed, which has higher soil phosphorus and a larger soil K factor than the other monitored subwatersheds (excluding the EH) for this land class.

Given the remaining uncertainty regarding variation in export coefficients within land classes among unmonitored Lake Simcoe subwatersheds, it is recommended that export coefficients from the monitored subwatersheds be used until additional information or data becomes available to better evaluate variation or to refine export estimates. One option is to apply the mean P export derived from the monitored subwatersheds excluding the East Holland River subwatershed to land cover areas of the unmonitored subwatersheds. P export coefficients from the EH subwatershed can be applied to unmonitored subwatersheds suspected of having higher export due to environmental conditions (i.e., West Holland, Georgina Creeks and Oro North).

Appendix 5
Responses to Comments from the Lake Simcoe Science Committee

Comment/Question	Report Reference	Disposition	Action
Add Winter et al., 2007 reference for urban land coefficients	pg. 6	Agreed	Citation and Reference added
Reword text describing Low Intensity Development and Unpaved Road coefficients	pg. 13	Agreed	Text reworded as recommended
Remove redundant text re. export coefficient uncertainties/error	Table 1, Appendix 4	Agreed	Redundant text removed from table
Recommendations for future directions related to policy amendments and the need to update the SWMPD Manual	general	The recommendations are noted, but are outside the scope of the project and may be considered in future updates of the P Budget Tool	Added " The Ministry may consider reviewing existing guidance for LID, Construction Phase activities (i.e., erosion and sedimentation considerations) and updating the SWMPD Manual from time to time to reflect current and emerging practices in these sectors." to Section 4 - Future Directions
Estimation of soil erosion through USLE is not intended for anything that may have significant channelized flow which is likely from a construction site; not certain how this is handled	pg. vi	We agree that USLE is appropriate for diffuse overland flow, not channelized flow. We stated that "The Tool addresses losses through surface runoff only." in Section 3.4.1 of the report.	None - concept is noted (Section 3.4.1) in the report
Do export coefficients vary between watersheds due to inherent differences in soils/landscapes/hydrology or because of location of the various land uses relative to flow paths to surface waters?	general	Export coefficients are expected to vary between watersheds for both reasons. Causes of expected variance between watersheds is discussed in Section 3.2.1.1	None - concept is noted (Section 3.2.2.1) in the report
K factor should be applied to exposed soil material and not necessarily what the top soil was/is.	general	Agreed	Changed definition to clarify application to exposed soils in Section 3.2.2 - "K is the soil erodibility factor based on soil textural class and organic matter content *of exposed soil..."*

Comment/Question	Report Reference	Disposition	Action
Studies have shown an enrichment factor in terms of the concentration of P in eroded sediments versus the bulk soil P content, i.e., the sediment/soil lost through erosion is enriched in P relative to the bulk soil. Is this considered in the tool?		This is considered in the CANWET model and therefore part of the pre- and post-development export coefficients. It was to be considered in the calculations for the construction phase through the use of subwatershed soil P values. However, due to the variability between subwatersheds it was decided that a single soil phosphorus value would be used globally in the watershed.	Added the following to Section 3.4.2 "Soil phosphorus concentration was originally intended to be a subwatershed value derived from the CANWET model. However, due to the variability between subwatersheds it was decided that a single soil phosphorus value would be used globally in the watershed. The CANWET model applies an empirical enrichment factor to the initial estimate of soil phosphorus to account for the greater phosphorus adsorption surface of smaller particles that make up a greater portion of eroded material."
Suggested taking a more conservative approach to derive export coefficients for undeveloped lands (i.e., lower export coefficient for undeveloped lands) using 30%ile rather than the mean to stimulate research	Section 3.2	We disagree with this approach as there is no scientific basis for using the 30th percentile or for prescribing a lower export from undeveloped lands only. While there is some error expected in the selected coefficients, these were calculated using a scientific approach with best available knowledge and are defensible. In recognition of possible site-specific differences in P export, we included an allowance to adjust the export coefficients as long as a detailed rationale is provided for consideration by the MOE.	None
What is the 20% adjustment allowance based on?	Section 3.2	The 20% adjustment allowance stems from previous drafts and discussions. This limited allowance has been eliminated and the User is able to adjust the coefficients with justification for site-specific characteristics. If so, a detailed rationale for the adjustment is required, including published references, for any adjustment (as entered by the user in a text box) for consideration by the MOE.	All references to 20% adjustment allowances have been removed from the text.

Comment/Question	Report Reference	Disposition	Action
Should specify the criteria required to satisfy the MOE for adjustments to the export coefficients	Section 3.2	Adjustments would be requested on a case by case basis and, in all cases, MOE would assess the request on its own merits.	None
Noted that efficiency of BMPs decline over time unless maintained and questions how the Tool deals with this.	Section 3.3	Agreed, BMPs are known to decline in P removal efficiency if not properly maintained. The document assumes that any BMP to be implemented will be maintained to ongoing standards. In Section 3.4.2, the manual states that: "In all cases there is a requirement that BMPs are maintained throughout the duration of the construction phase in order that they continually operate at their design efficiency."	Clarified assumption of BMP maintentance in Section 3.1.2.2 "...can be used, if built to design specification and maintained to design standards, with assurance of their effectiveness."
If a coefficient for a BMP reduction changes during design phase, what coefficient should apply to the Applicant	general	The applicant should ensure that the most recent version of the Tool is used for their application. It would be expected that the conditions of the MOE-approved application would apply for the duration of the development construction.	None

Appendix 6
BILD Comments and HESL Response to Technical Comments

HESL Responses to Technical Comments from BILD

BILD Comment: BILD noted the importance of atmospheric deposition to the total annual phosphorus load to Lake Simcoe and suggested that the manual "should acknowledge this issue and a commitment should be made by MOE to revise the model once the science of atmospheric deposition of phosphorous has been advanced".

HESL Response: We agree that changes in land use and BMP implementation to reduce phosphorus loading may reduce atmospheric loads to Lake Simcoe. As noted by BILD, however, the state of science is not presently sufficient to calculate the relative contribution of different land use practices to the atmospheric load. We have clarified this point in the report with the following statements:

- "The Tool does not address atmospheric sources of phosphorus in dust generated from land use practices, as the science is not yet advanced to the point where estimates can be made. It does account for atmospheric deposition of phosphorus to open water and atmospheric deposition to land surfaces is included in the export coefficients for various land use practices." (Executive Summary, page ii)

- "Note that phosphorus loads from atmospheric deposition to land are incorporated into the export coefficients for the various land cover classes. The atmospheric/open water coefficient should not be interpreted as loading from dust generated by land use activities such as agriculture or construction. It represents a regional atmospheric contribution. The means to estimate dust generation and loading are the subject of current research initiatives being undertaken by the MOE, the LSRCA and various research partners." (page 14)

Further, we have added a recommendation in Section 4 of the report that states:

- Wind erosion from agricultural activities and construction sites has not been considered in the subwatershed modeling work completed to date and may contribute to the atmospheric deposition portion of loading to Lake Simcoe in both the pre-development (agricultural) and post-development (construction) phases. Many practices that reduce wind erosion potential may also reduce soil loss due to stormwater runoff. Therefore, future efforts should be made to a) quantify losses from wind erosion from agricultural and construction activities and b) the benefits of BMPs to reducing both types of soil loss.

BILD Comment: BILD suggested a trial period for the Tool.

***HESL Response*:** We agree that a trial period would be useful to inform the process.

BILD Comment: Request for 'clear transition rules surrounding existing applications and approvals be outlined in the Budget Tool when it is posted to the Environmental Registry'.

HESL Response: This relates to MOE policy and should be addressed outside of the budget tool document.

BILD Comment: 'BILD members have identified concerns for the need to calculate phosphorus loading during the construction phase of a project.'

HESL Response: We included calculations to estimate construction phase phosphorus loads in the Manual using the best available information, as required by the project RfP. The Manual recognizes the difficulties in calculating construction phase loads, but is focussed on the use of BMPs to reduce these loads despite uncertainties in calculations.

BILD Comment: 'BILD members have expressed that the breakdown of the land use classifications is too detailed and requires the expertise of an ecologist to decipher. A number of sites do not require an environmental report which would make selecting the correct pre-development land use from the breakdown provided in the document difficult. BILD requests additional clarification and direction in this regard.

HESL Response: HESL notes that an Environmental Impact Study (EIS) should be done for all new major developments, which would include classification of land use areas. While an ecologist may be required to classify and delineate land uses, we suggest that this is good practice.

BILD Comment: 'BILD members are concerned that we may find ourselves in a situation where we have employed all of the practical Best Management Practices and Low Impact Development measures which support the reductions of phosphorous loading, but yet, we may still find ourselves in a shortfall when applying the Budget Tool. Since phosphorous trading is not yet available, our members request clarification as to whether or not the project would get rejected if this situation were to happen.'

HESL Response: This comment reflects policy decision that MOE will have to make. Our manual provides methods on how to make the calculations.

Appendix 7

References for BMP Phosphorus % Reduction Coefficients Shown in Table 3 and Appendix 2

Ref ID	Author	Year	Title	Publication
2	Van Seters et al.	2009	Referenced in: Low Impact Development Stormwater Management Planning and Design Guide - CVC Version1, 2010	Credit Valley Conservation
4	J.F. Sabourin & Ass.	2008		
6	ASCE	2000		
7	SWAMP	2002		
8	Dietz and Caausen	2005		
9	Hunt et al.	2006		
10	Davis	2007		
12	Hunt et al.	2008		
13	Roseen et al.	2009		
21	Deletic and Fletcher	2006		
23	U of Florida	2009	FDEP contract # WM 910	Dept. Env. Eng. Sciences, Gainesville FL
24	Wanielista et al.	1978	Shallow water roadside ditches for stormwater purification	www.stormwater.ucf.edu/FILES/wan1978paper.pdf
26	Harper, H.H.	1988	Effects of Stormwater Management Systems on Groundwater Quality	Florida Dept of Env Reg - project WM190
27	Dorman et al.	1989	Retention/Detention and overland flow for Pollutant removal from Highway stormwater runoff	Vol I research report. Federal Hwy Admin FJWA/RD-89/202pp
28	Yu, S.L. Et al.	1993	Testing of BMPs for controlling highway runoff	Virginia Transportation Research Council. FHWA/VA-93-R16.60pp
29	Goldberg, J.	1993	Dayton Ave Swale Biofiltration Study	Seattle Eng Dept - Seattle WA 67pp
30	Barrett et al.	1998	Performance of Vegetative Controls for Treating Highway Runoff	J. Environ Eng., 124(11) 1121-1128
31	Rushton et al.	2001	Florida Aquarium Parking Lot: A treatment train approach to SWM	SWFWMD, Brooksville, FL.
32	Lloyd, S.D. Et al.	2001	Assessment of Pollutant Removal in a Newly constructed Bio-retention system	2nd South Pacific Stormwater Conference, Auckland, New Zealand
34	Lombardo &Line	2004	Evaluating the effectiveness of LID NCSU Water Quality Group	NC State U - conf proc: http://lowimpactdevelopment.org
35	Sharkey & Hunt	2005	Case Studies on the performance of Bioretention Areas in NC	8th biennial Stormwater research & wshed man conf
36	Birch et al.	2005	Efficiency of an Infiltration Basin in Removing Contaminants from Urban	Env. Mon. and Ass. 101: 23-38

Ref ID	Author	Year	Title	Publication
			Stormwater	
37	Davis et al.	2006	WQ improvement through Bioretention Media:N amd P removal	Water Environment Research 78(3):284-293
38	Brown & Hunt	2008	Bioretention performance in the upper coastal plain of NC	ASCE/EWRI World Environmental and Water Resources Congress
40	Osborn & Packman	2008	A comparison of conventional and low impact dev stormwater BMPs	ASCE/EWRI World Environmental and Water Resources Congress
41	Toronto and Region Conservation Authority	2010	Performance Evaluation of an Anionic Polymer for treatment of Construction Runoff	TRCA
42	Woodard and Rock	1995	Control of Residential Stormwater by Natural Buffer Strips	Lake &Reservoir Management 11(1), 37-45
104	Schueler, T.R.	2000	Comparative Pollutant Removal Capability of Stormwater Treatment Practices	Technical Note #95 from Watershed Protection Techniques. 2(4): 515-520.
105	Schueler, T.R.	2000	Pollutant Removal Dynamics of Three Wet ponds in Canada	Technical Note #114 from Watershed Protection Techniques. 3(3): 721-728.
106	Ministry of the Environment	2005	Synthesis of Monitoring Studies Conducted Under the Stormwater Assessment Monitoring and Performance Program	Prepared by the SWAMP program for GLSF, TRCA, MEAO and MOE, published by Toronto and Region Conservation Authority
109	www.bmpdatabase.org		The International Stormwater Best Management Practices (BMP) Database Project website	
111	Sansalone, J	2009	TARP Field Test Performance Evaluation of Sorbtive Filter using Sorbtive Media for Imbrium Systems Corporation	Dept. of Environmental Engineering Sciences at the Univ. of Florida. February 2009
112	Schueler and Holland	2000	The Practice of Watershed Protection	Centre for Watershed Protection, Ellicott City, MD

Ref ID	Author	Year	Title	Publication
113	Lee, C.R. and Skogergboe, J.G.	1985	Quantification of Erosion Control by Vegetation on Problem Soils	Soil Conservation Society of America, Arkeny, IA. pp.437-444
114	Taleban, V., Finney, K., Gharabaghi, B., McBean, E., Rudra, R. and Van Seters, T.	2009	Effectiveness of Compost Biofilters in Removal of Sediments from Construction Site Runoff	Water Quality Research Journal of Canada Vol. 44, No.1, 71-80

Appendix 8

Phosphorus Budget Tool in Support of Sustainable Development for the Lake Simcoe Watershed: Background on the Recommended Export Coefficients (MOE, draft report)

Appendix 9

Checklist of Required Elements for Review of Submissions

Checklist of Required Elements for Review of Submission

The following provides a checklist of elements that are required for the review of a phosphorus budget submission for a new major development. The checklist will be used by reviewers to ensure that the submission is complete and that the results of the phosphorus budget meet the requirements necessary for the Ministry to recommend approval of the development to the Municipality.

The MOE will recommend that municipalities approve development as site specific appropriate if:

a) Post-development load < or = pre-development load, and

b) (Post-development + amortized construction phase) load < or = pre-development loading,

OR

If (Post-development + amortized construction phase) load > pre-development loading,

THAT

All reasonable and feasible construction phase BMPs have been identified for implementation, documented and accounted for in the application.

The phosphorus budget submission requires the inclusion of the four page Summary Report produced by the Database Tool. In support of the information contained in the Summary Report, the submission should include:

Module 1

1. An orthographic aerial photograph that shows the delineation of pre-development land uses as per the EIS for the development that will be used to support the planning application to the Municipality.
2. Rationale to support the selection of land uses as defined in the Guidance Manual (Table 1) to most closely match those defined in the EIS land use mapping.
3. The correct database entry for the Lake Simcoe watershed in which the development is proposed. If the development area spans two or more subwatersheds, the submission should include a separate phosphorus budget for each area within each subwatershed.
4. Correct areas for each land use on the development site; the sum of which are equal to the total development site area.
5. The use of the pre-defined subwatershed and landuse specific export coefficients (in Table 2 of the Guidance Manual and coded in the Database Tool) for calculation of phosphorus loading from the pre-development site.

Module 2

1. An orthographic aerial photograph that shows the delineation of the post-development land uses as defined in Table 1 of the Guidance Manual.

2. Appropriate division of the site into post-development blocks that contain a unique combination of a land use and Best Management Practice or Treatment Train that will be applied to that land use in Module 3.
3. Correct areas for each post-development block; the sum of which are equal to the total development site area. The "Post-Development Area Altered" should equal the "Total Pre-Development Area" on page 2 of the Summary Report.
4. The use of the pre-defined subwatershed and landuse specific export coefficients (in Table 2 of the Guidance Manual and coded in the Database Tool) for calculation of phosphorus loading from the pre-development site.

Module 3

1. Specific references to the Stormwater Management Plan, where detailed descriptions and scientific rationales are provided for:
    a. The type of BMP or a Treatment Train approach that was chosen for each post-development block;
    b. The use of any phosphorus removal efficiencies for BMPs that are not pre-defined in the Tool;
    c. Each treatment in a Treatment Train, their respective phosphorus removal efficiencies and the total efficiency of the Treatment Train, if this option is to treat stormwater.

    If a custom BMP, phosphorus removal efficiency or a Treatment Train is used, this will be noted in red text on page 2 of the Summary Report.

Module 4

1. Appropriate division of the site into construction phase blocks, each of which comprise relatively uniform slope and soil characteristics and a unique capture BMP and prevention BMP combination.
2. Demonstrated, accurate input data for the soil loss calculations including:
    a. Area of each block;
    b. Predominant soil texture class and organic matter content;
    c. Surface slope gradient and length of slope;
    d. Duration of exposed soil for each block;
    e. Total duration of the construction phase.
3. Detailed descriptions and rationale for:
    a. Capture and prevention BMPs selected for each block;
    b. Use of any phosphorus removal efficiencies for BMPs that are not pre-defined in the Tool;

c. Each treatment in a Treatment Train, their respective phosphorus removal efficiencies and the total efficiency of the Treatment Train, if this option is to treat stormwater.

Final Summary and Analysis

If all elements above are contained in the submission and data are correctly entered in the database, the final summary (page 4 of the Summary Report) can be used as the final assessment of whether or not the results of the phosphorus budget meet the requirements necessary for the Ministry to recommend approval of the development to the Municipality.

If:

a) Post-development load < or = pre-development load, and

b) (Post-development + amortized construction phase) load < or = pre-development loading,

OR

If (Post-development + amortized construction phase) load > pre-development loading,

THAT

All reasonable and feasible construction phase BMPs have been identified for implementation, documented and accounted for in the application.

The final statement of page 4 of the Summary Report will display:

Based on a comparison of Pre-Development and Post-Development loads, the Ministry would encourage the Municipality to:

Approve development as site specific appropriate.

The above conclusion, however, assumes that the Ministry is in agreement with all rationales provided in the submission and are satisfied that “All reasonable and feasible construction phase BMPs have been identified for implementation, documented and accounted for in the application.”

JELLYFISH ETV
CERTIFICATION REPORT

◆ Verification Statement ◆ Verification Statement ◆ Verification Statement ◆

VERIFICATION STATEMENT

GLOBE Performance Solutions

Verifies the performance of

Jellyfish® Filter JF4-2-1

Developed by Imbrium Systems, Inc.,
Whitby, Ontario, Canada

In accordance with

ISO 14034:2016

Environmental management —
Environmental technology verification (ETV)

John D. Wiebe, PhD
Executive Chairman
GLOBE Performance Solutions

August 3, 2017
Vancouver, BC, Canada

GLOBE PERFORMANCE SOLUTIONS

Verification Body
GLOBE Performance Solutions
404 – 999 Canada Place | Vancouver, B.C | Canada |V6C 3E2

Technology description and application

The Jellyfish® Filter is an engineered stormwater quality treatment technology designed to remove a variety of stormwater pollutants including floatable trash and debris, oil, coarse and fine suspended sediments, and particulate-bound pollutants such as nutrients, heavy metals, and hydrocarbons. The Jellyfish Filter combines gravitational pre-treatment (sedimentation and floatation) and membrane filtration in a single compact structure. The system utilizes membrane filtration cartridges comprised of multiple pleated filter elements ("filtration tentacles") that provide high filtration surface area with the associated advantages of high flow rate, high sediment capacity, and low filtration flux rate.

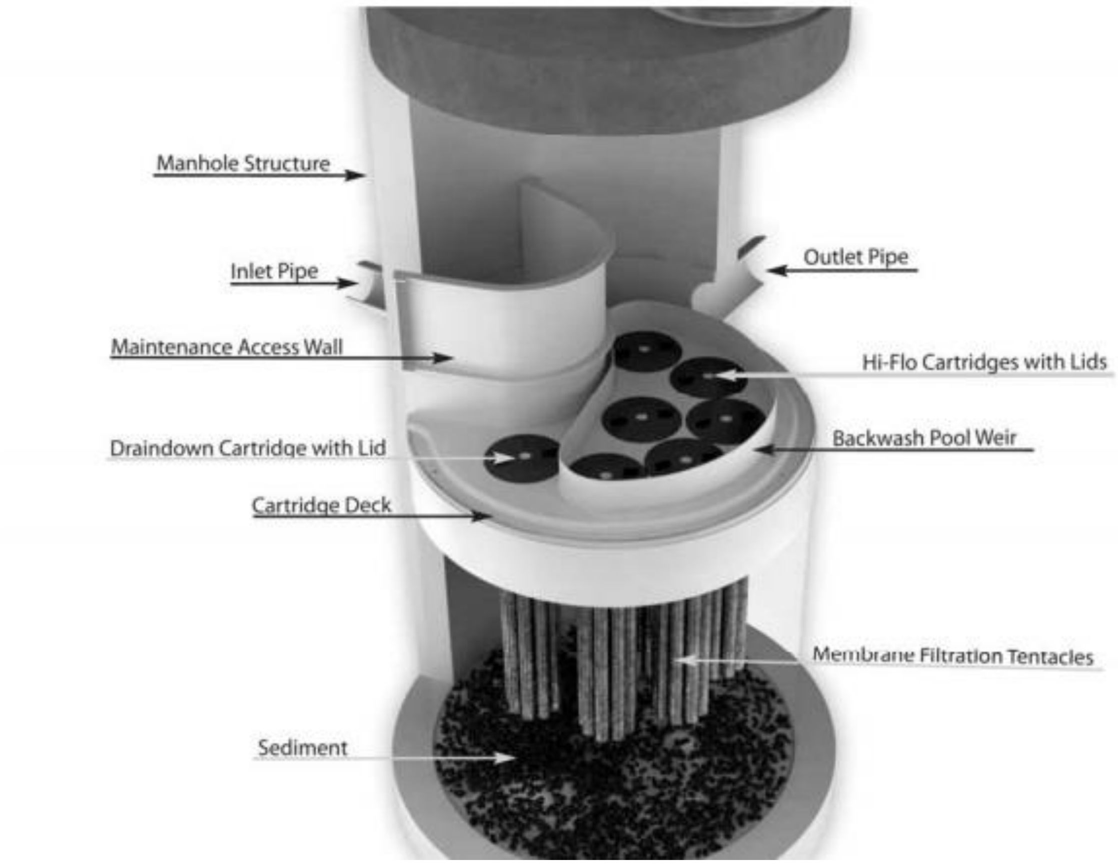


Figure 1. Cut-away graphic of a Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge

Figure 1 depicts a cut-away graphic of a typical 6-ft diameter Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge (JF6-6-1). Stormwater influent enters the system through the inlet pipe and builds a pond behind the maintenance access wall, with the pond elevation providing driving head. Flow is channeled downward into the lower chamber beneath the cartridge deck. A flexible separator skirt (not shown in the graphic) surrounds the filtration zone where the filtration tentacles of each cartridge are suspended, and the volume between the vessel wall and the outside surface of the separator skirt comprises a pretreatment channel. As flow spreads throughout the pretreatment channel, floatable pollutants accumulate at the surface of the pond behind the maintenance access wall and also beneath the cartridge deck in the pretreatment channel, while coarse sediments settle to the sump. Flow proceeds under the separator skirt and upward into the filtration zone, entering each filtration tentacle and depositing fine suspended sediment and associated particulate-bound pollutants on the outside surface of the membranes. Filtered water proceeds up the center tube of each tentacle, with the flow from each tentacle combining under the cartridge lid, and discharging to the top of the cartridge deck through the cartridge lid orifice. Filtered effluent from the hi-flo cartridges enters a pool enclosed by a 15-cm high weir, and if storm intensity and resultant driving head is sufficient, filtered water overflows the weir and proceeds across the cartridge deck to the outlet pipe. Filtered effluent discharging from the draindown cartridge(s) passes directly to the outlet pipe, and requires only a minimal amount of driving head (2.5 cm) to provide forward flow. As storm intensity subsides and driving head drops below 15 cm, filtered water within the backwash pool reverses direction and passes backward through the hi-flo cartridges, and thereby dislodges sediment from the membranes which subsequently settles to the sump below the filtration zone. During this passive backwashing process, water in the lower chamber is displaced only through the draindown cartridge(s). Additional self-cleaning processes include gravity, as well as vibrational pulses emitted when flow exits the orifice of each cartridge lid, and these combined processes significantly extend the cartridge service life and maintenance cleaning interval. Sediment removal from the sump by vacuum is required when sediment depths reach 30 cm, and cartridges are typically removed, externally rinsed, and recommissioned on an annual basis, or as site-specific maintenance conditions require. Filtration tentacle replacement is typically required every 3 – 5 years.

Performance conditions

The data and results published in this Technology Fact Sheet were obtained from a field monitoring program conducted on a Jellyfish® Filter JF4-2-1 (4-ft diameter manhole with 2 hi-flo cartridges and 1 draindown cartridge), in accordance with the provisions of the TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements—Amendments to TARP Tier II Protocol (NJDEP, 2009). Testing was completed by researchers led by Dr. John Sansalone at the University of Florida's Engineering School of Sustainable Infrastructure and Environment. The drainage area providing stormwater runoff to the test unit varied between 502 m² and 799 m² (5400 ft² to 8600 ft²) depending on storm intensity and wind direction. The unit was monitored for a total of 25 TARP qualifying storm events (i.e. $\geq$ 2.5 mm of rainfall) contributing cumulative rainfall of 381 mm (15 in) over the 13-month period between May 28, 2010 and June 27, 2011. Only TARP-qualified storms were routed through the unit, and maintenance was not required during the testing period based on sediment accumulation less than the depth indicated for maintenance, and also based on hydraulic testing performed on the system after the conclusion of monitoring.

Table 1 shows the specified and achieved amended TARP criteria for storm selection and sampling.
Table 2 shows the observed ranges of operational conditions that occurred over the testing period.

Table 1. Specified and achieved amended TARP criteria for storm selection and sampling

Description	Criteria value	Achieved value
Total rainfall	$\geq$ 2.5 mm (0.1 in)	> 2.5 mm (0.1 in)
Minimum inter-event period	6 hrs	10 hrs
Minimum flow-weighted composite sample storm coverage	70% including as much of the first 20% of the storm	100%
Minimum influent/effluent samples	10, but a minimum of 5 subsamples for composite samples	Minimum of 8 subsamples for composite samples
Total sampled rainfall	Minimum 381 mm (15 in)	384 mm (15.01 in)
Number of storms	Minimum 20	25

Table 2. Observed operational conditions for events monitored over the study period

Operational condition	Observed range
Storm durations	26 – 691 min
Previous dry hours	10 - 910 hrs
Rainfall depth	3 – 50 mm
Initial rainfall to runoff lag time	1 – 34 min
Runoff volume	206 – 13,229 L
Peak rainfall intensity	5 – 137 mm/hr
Peak runoff flow rate	0.5 – 14.3 L/s
Event median flow rate	0.01 – 5.5 L/s

The 4-ft diameter test unit has sedimentation surface area of 1.17 m^2 (12.56 ft^2). Each of the three filter cartridges employed in the test unit uses filtration tentacles of 137 cm (54 in) length, with filter surface area of 35.4 m^2 (381 ft^2) per cartridge, and total filter surface area of 106.2 m^2 (1143 ft^2) for the three cartridges combined. The design treatment flow rate is 5 L/s (80 gal/min) for each of the two hi-flo cartridges and 2.5 L/s (40 gal/min) for the single draindown cartridge, for a total design treatment flow rate of 12.6 L/s (200 gal/min) at design driving head of 457 mm (18 in). This translates to a filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m^2 (0.21 gal/min/ft^2) for each hi-flo cartridge and 0.07 L/s/m^2 (0.11 gal/min/ft^2) for the draindown cartridge. The design flow rate for each cartridge is controlled by the sizing of the orifice in the cartridge lid. The distance from the bottom of the filtration tentacles to the sump is 61 cm (24 in).

Performance claims

The Jellyfish® Filter demonstrated the removal efficiencies indicated in **Table 3** for respective constituents during field monitoring of 25 TARP qualified storm events with cumulative rainfall of 381 mm, conducted in accordance with the provisions of the TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements—Amendments to TARP Tier II Protocol (NJDEP, 2009), and using the following design parameters:

- System hydraulic loading rate (system treatment flow rate per unit of sedimentation surface area) of 10.8 L/s/m^2 (15.9 gal/min/ft^2) or lower
- Filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m^2 (0.21 gal/min/ft^2) or lower for each hi-flo cartridge and 0.07 L/s/m^2 (0.11 gal/min/ft^2) or lower for each draindown cartridge
- Distance from the bottom of the filtration tentacles to the sump of 61 cm (24 in) or greater
- Driving head of 457 mm (18 in) or greater

Table 3. Mean, median and 95% confidence interval (median) for removal efficiencies of selected stormwater constituents

Parameter	Mean	Median	Median - 95% Lower Limit	Median - 95% Upper Limit
TSS	84.7	85.6	82.8	89.8
SSC	97.5	98.3	97.1	98.7
Total phosphorus	48.8	49.1	43.3	60.1
Total nitrogen	37.9	39.3	31.2	54.6
Zinc	55.3	69	39	75
Copper	83.0	91.7	75.1	98.9
Oil and grease	60.1	60	42.7	100

N.B. As with any field test of stormwater treatment devices, removal efficiencies will vary based on pollutant influent concentrations and other site specific conditions.

Performance results

The frequency of rainfall depths monitored during the study is presented in **Figure 2**. The median and 90th percentile rainfall depths were 11 mm and 31.7 mm, respectively. These values represent the depth of rainfall that is not exceeded in 50 and 90 percent of the monitored rainfall events.

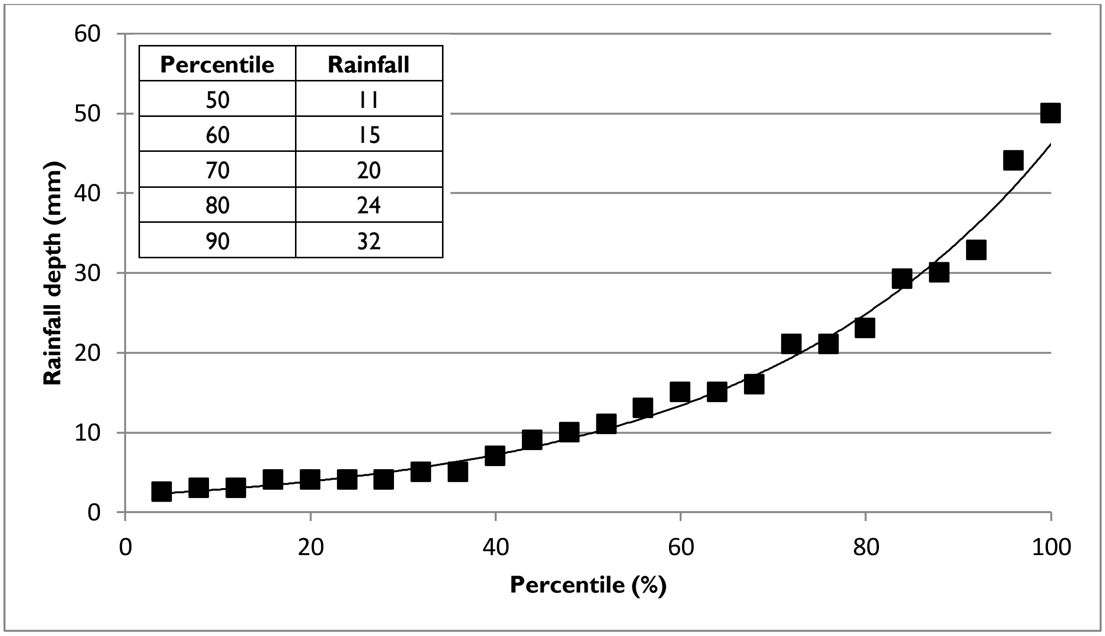


Figure 2. Rainfall depth frequency curve

Sediment removal performance was assessed by measuring the event mean concentration and mass of suspended sediment entering and leaving the unit during runoff events. This involved sampling the full cross-section of influent and effluent flows manually at 2 - 10 minute intervals for the full duration of each storm event and combining discrete samples into flow-weighted composites. Comparing the theoretical mass recovery from the sump calculated by the difference between the influent and effluent mass to the actual dry weight of the recovered sump mass showed an overall mass balance recovery of 94.5% over the study period.

The median d50 particle size (i.e. 50th percentile particle size) of the influent and effluent was 82 and 3 µm, respectively (**Figure 3**). The median influent particles sizes ranged between 22 and 263 µm, whereas median effluent particle sizes ranged between 1 and 11 µm.

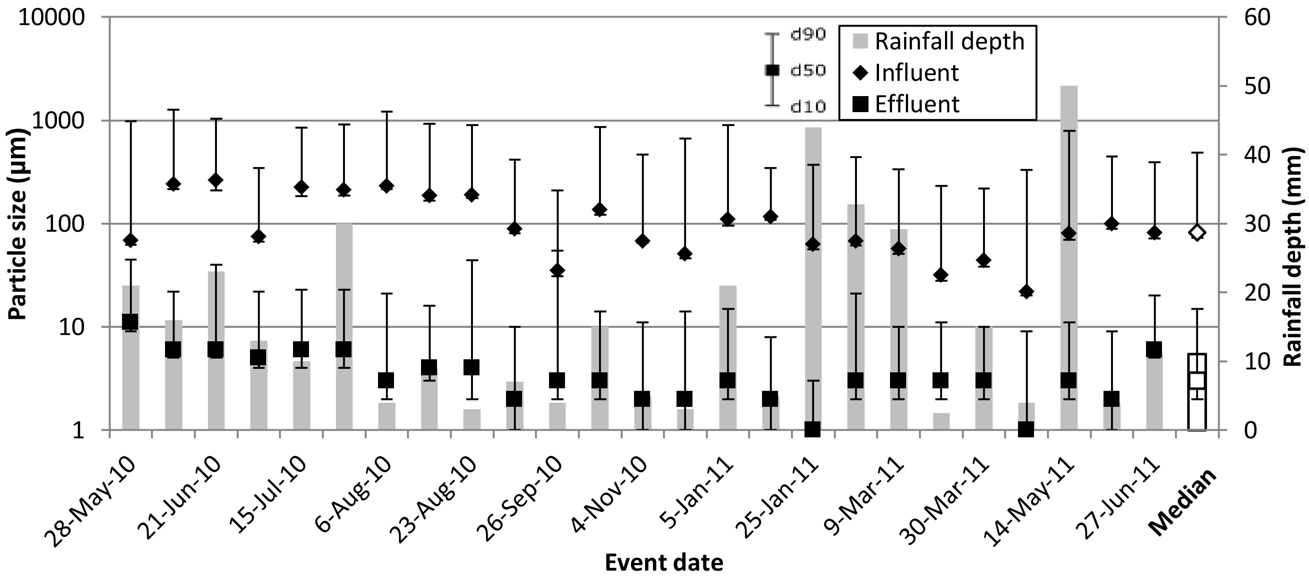


Figure 3. The rainfall depth and d10, d50, and d90 particle sizes of the influent and effluent composite samples for each monitored storm event over the 13-month testing period

Sampling of flows into and out of the Jellyfish Filter over the testing period showed statistically significant reductions ($p < 0.05$; Wilcoxon signed-rank test) in influent event mean concentrations for all selected stormwater constituents (**Table 4** and **Figure 4**). Effluent event mean Suspended Sediment Concentrations (SSC) were below 19 mg/L during all monitored events. Load-based removal rates were also calculated based on the sum of loads over the study period. These removal rages ranged from 46.3 for Total Nitrogen to 98.6 for SSC (**Table 4**).

Table 4. Summary statistics for influent and effluent event mean concentrations for selected constituents

Water Quality Variable	Sampling Location	Min	Max	Median	Range	Mean	SD	Load based removal efficiency (%)
TSS	Influent (mg/L)	16.30	261.00	79.30	244.70	86.26	51.37	87.2
	Effluent (mg/L)	3.20	21.70	11.80	18.50	10.99	4.79	
SSC	Influent (mg/L)	78.20	1401.70	444.50	1323.50	482.26	338.34	98.6
	Effluent (mg/L)	2.80	18.10	7.30	15.30	7.88	3.77	
TP	Influent (µg/L)	887.00	8793.00	3063.00	7906.00	3550.20	1914.50	64.2
	Effluent (µg/L)	472.00	4769.00	1480.00	4297.00	1688.08	1059.98	
TN	Influent (µg/L)	1170.00	10479.00	3110.00	9309.00	3519.32	2161.47	46.3
	Effluent (µg/L)	553.00	6579.00	1610.00	6026.00	2091.76	1613.61	
Zn	Influent (µg/L)	0.005	7600.00	1500.00	7600.00	1792.00	1852.91	76.1
	Effluent (µg/L)	0.005	2760.00	450.00	2760.00	561.64	594.70	
Cu	Influent (µg/L)	0.001	880.40	79.50	880.40	171.28	229.33	92.1
	Effluent (µg/L)	0.001	51.30	6.90	51.30	14.36	17.22	
Oil and Grease	Influent (mg/L)	0.20	4.06	0.93	3.86	1.07	0.82	46.4
	Effluent (mg/L)	0.00	2.32	0.35	2.32	0.50	0.60	

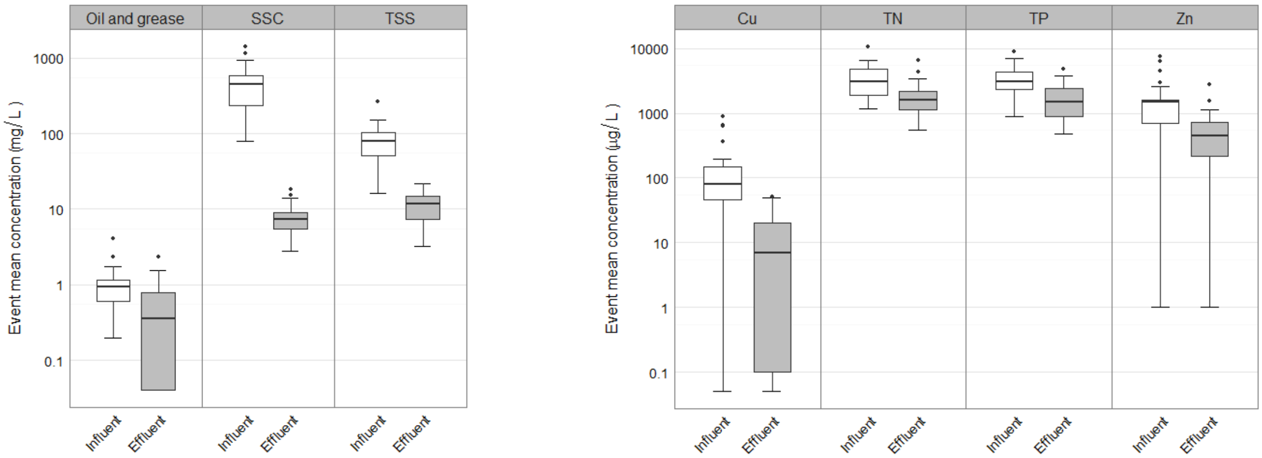


Figure 4. Boxplots showing the distribution of influent and effluent event mean concentrations (EMC) for selected stormwater constituents over the study period

Verification

The verification was completed by the Verification Expert, Toronto and Region Conservation Authority, contracted by GLOBE Performance Solutions, using the International Standard ***ISO 14034:2016 Environmental management -- Environmental technology verification (ETV)***. Data and information provided by Imbrium Systems to support the performance claim included the performance monitoring report prepared by University of Florida, Engineering School of Sustainable Infrastructure and Environment, and dated November 2011. This report is based on testing completed in accordance with the Technology Acceptance Reciprocity Partnership (TARP) Tier II Protocol (2003) and New Jersey Tier II Stormwater Test Requirements--Amendments to TARP Tier II Protocol (NJDEP, 2009).

What is ISO14034:2016 Environmental management – Environmental technology verification (ETV)?

ISO 14034:2016 specifies principles, procedures and requirements for environmental technology verification (ETV), and was developed and published by the *International Organization for Standardization (ISO)*. The objective of ETV is to provide credible, reliable and independent verification of the performance of environmental technologies. An environmental technology is a technology that either results in an environmental added value or measures parameters that indicate an environmental impact. Such technologies have an increasingly important role in addressing environmental challenges and achieving sustainable development.

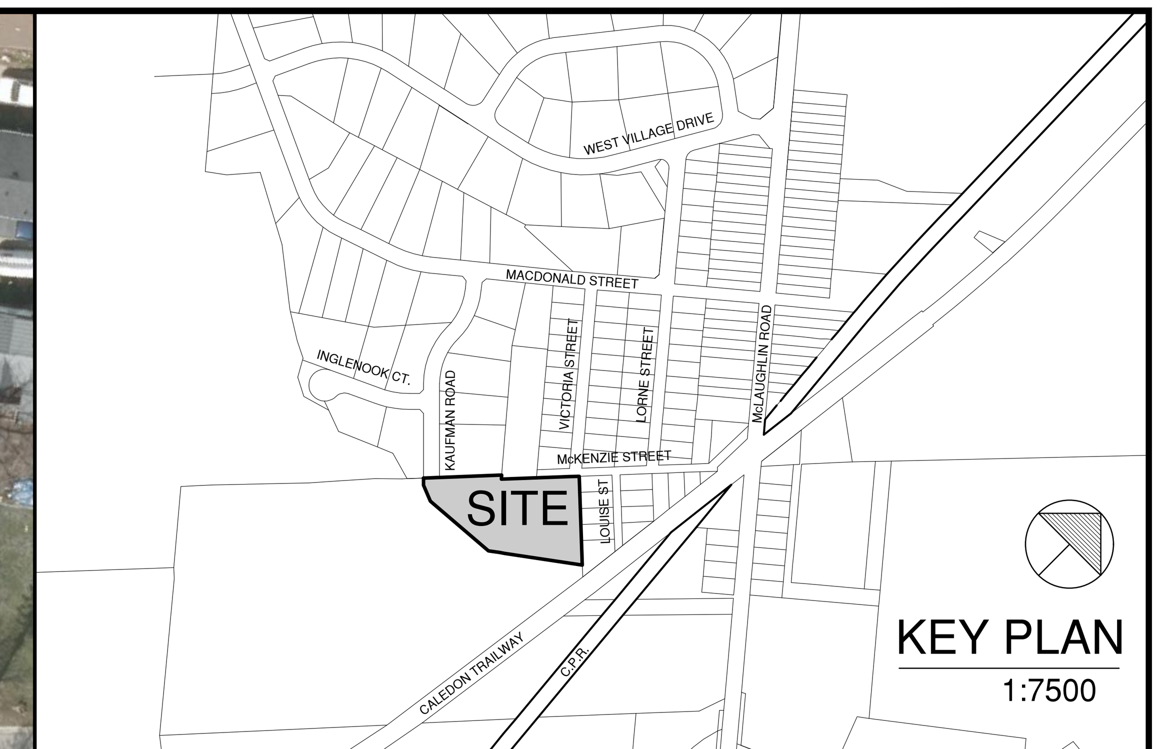
For more information on the Jellyfish® Filter please contact:

Imbrium Systems, Inc.
407 Fairview Drive
Whitby, ON
L1N 3A9, Canada
Tel: 416-960-9900
info@imbriumsystems.com

For more information on ISO 14034:2016 / ETV please contact:

GLOBE Performance Solutions
World Trade Centre
404 – 999 Canada Place
Vancouver, BC
V6C 3E2 Canada
Tel: 604-695-5018 / Toll Free: 1-855-695-5018
etv@globeperformance.com

DRAWINGS



LOTS / BLOCK AREA SCHEDULE

LAND USE	LOT / BLOCK	AREA	
RESIDENTIAL	LOTS 1-13	0.86ha.	2.13Ac.
OPEN SPACE	BLOCKS 1-1A	0.10ha.	0.25Ac.
SWM POND	BLOCK 2	0.21ha.	0.52Ac.
DRAINAGE BLOCK	BLOCK 3	0.03ha.	0.07Ac.
ROADS	STREET 'A'	0.43ha.	1.06Ac.
TOTAL		1.63ha.	4.03Ac.

LOT SCHEDULE

DESCRIPTION	TYPE	MINIMUM FRONTAGE	MINIMUM DEPTH	NUMBER OF LOTS	NUMBER OF UNITS
SINGLE DETACHED	X1	18.3m (60.0')	32.5m (106.6')	13	13
TOTAL				13	13

ADDITIONAL INFORMATION

AS REQUIRED UNDER SECTION 51(17) OF THE PLANNING ACT (R.S.O. 1990 C.P. 13)

A) AS SHOWN ON DRAFT PLAN.
B) AS SHOWN ON DRAFT AND KEY PLAN.
C) AS SHOWN ON KEY PLANS.
D) AS SHOWN IN LAND USE SCHEDULE.
E) AS SHOWN ON DRAFT PLAN.
F) AS SHOWN ON DRAFT PLAN.
G) AS SHOWN ON DRAFT AND KEY PLANS.
H) MUNICIPAL SERVICES TO BE PROVIDED.
I) SOIL IS CLAYEY SILT.
J) AS SHOWN ON DRAFT PLAN.
K) MUNICIPAL SERVICES TO BE PROVIDED.
L) NONE.

OWNER'S AUTHORIZATION:

THE UNDERSIGNED, BEING THE OWNER OF THE SUBJECT LANDS HEREBY AUTHORIZE CANDEVCON LIMITED TO ACT ON OUR BEHALF AS AGENTS AND TO PREPARE AND SUBMIT A DRAFT PLAN OF SUBDIVISION FOR APPROVAL.

DATE

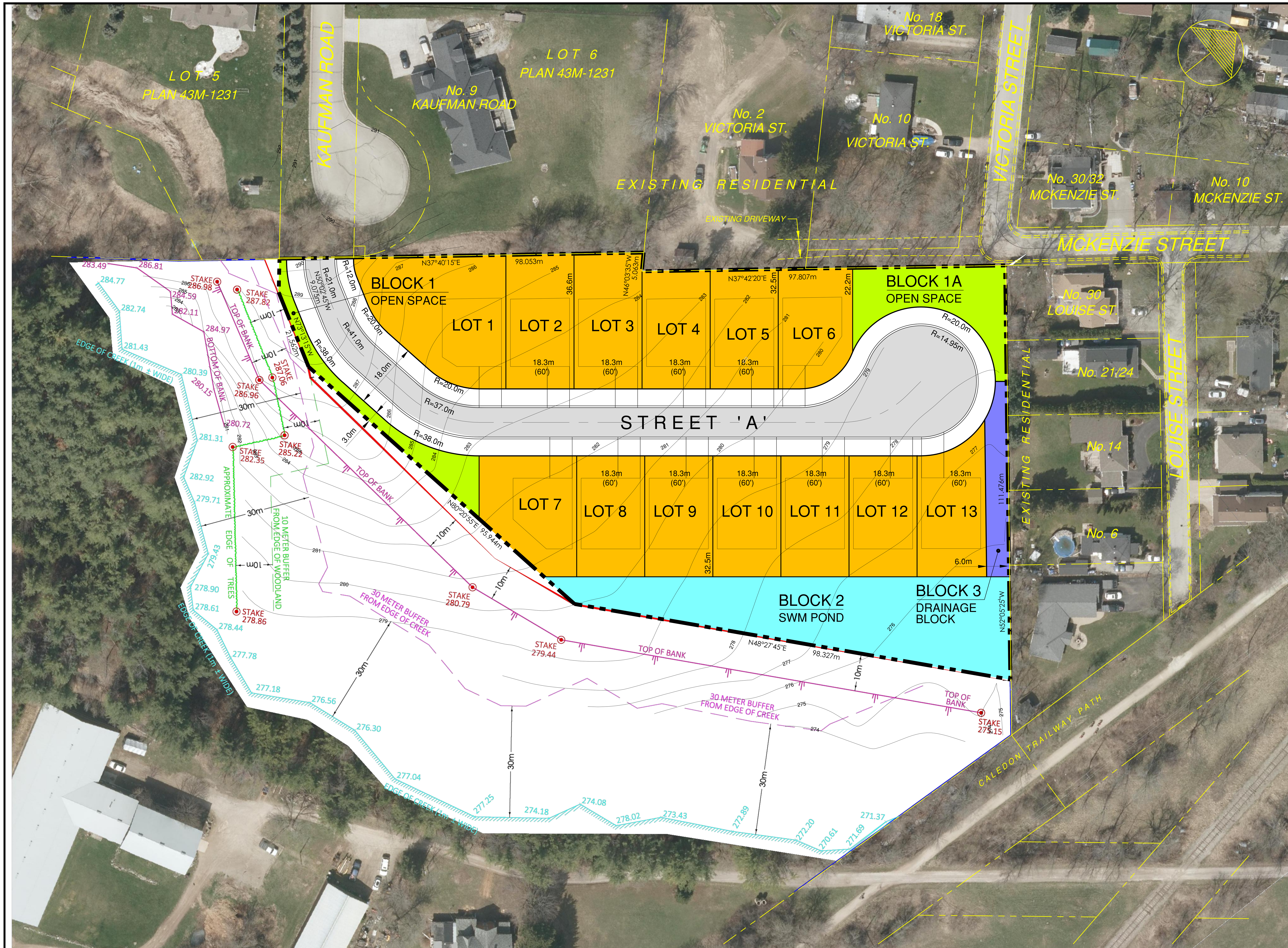
268577 ONTRIO INC.
(MANOJ SHARMA)

SURVEYOR'S CERTIFICATE:

I HEREBY CERTIFY THAT THE BOUNDARIES OF THE LANDS TO BE SUBDIVIDED ON THIS PLAN AND THEIR RELATIONSHIP TO THE ADJACENT LANDS ARE ACCURATELY AND CORRECTLY SHOWN.

DATE

MATT DE JAGER O.L.S.
VAN HARTEN SURVEYING INC.



INTERSECTION TREATMENT TABLE, CITY OF BRAMPTON STANDARD 245			
STREET TO STREET	CURB RADIUS	DAYLIGHT ROUNDING	DAYLIGHT TRIANGLE
MINOR LOCAL TO MINOR LOCAL	7.5m	5.0m	-
LOCAL TO LOCAL	7.5m	5.0m	-
MINOR LOCAL / LOCAL TO MINOR COLLECTOR	10.5m	-	7.5m
MINOR LOCAL / LOCAL TO MAJOR COLLECTOR	10.5m	-	15.0m
MINOR LOCAL / LOCAL TO ARTERIAL	15.0m	-	15.0m
MINOR LOCAL / MINOR COLLECTOR	15.0m	-	10.0m
MINOR / MAJOR COLLECTOR TO MAJOR COLLECTOR	15.0m	-	15.0m
INDUSTRIAL STREET TO INDUSTRIAL STREET	15.0m	-	10.0m
MINOR / MAJOR / INDUSTRIAL COLLECTOR TO ARTERIAL	18.0m	-	15.0m
ARTERIAL TO ARTERIAL	18.0m	-	15.0m

NO.	DESCRIPTION	DATE	BY

REVISIONS

DRAFT PLAN OF SUBDIVISION

PART OF LOT 1, CONCESSION 2, W.H.S.
TOWN OF CALEDON
REGIONAL MUNICIPALITY OF PEEL
PRE-2022-0187

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS

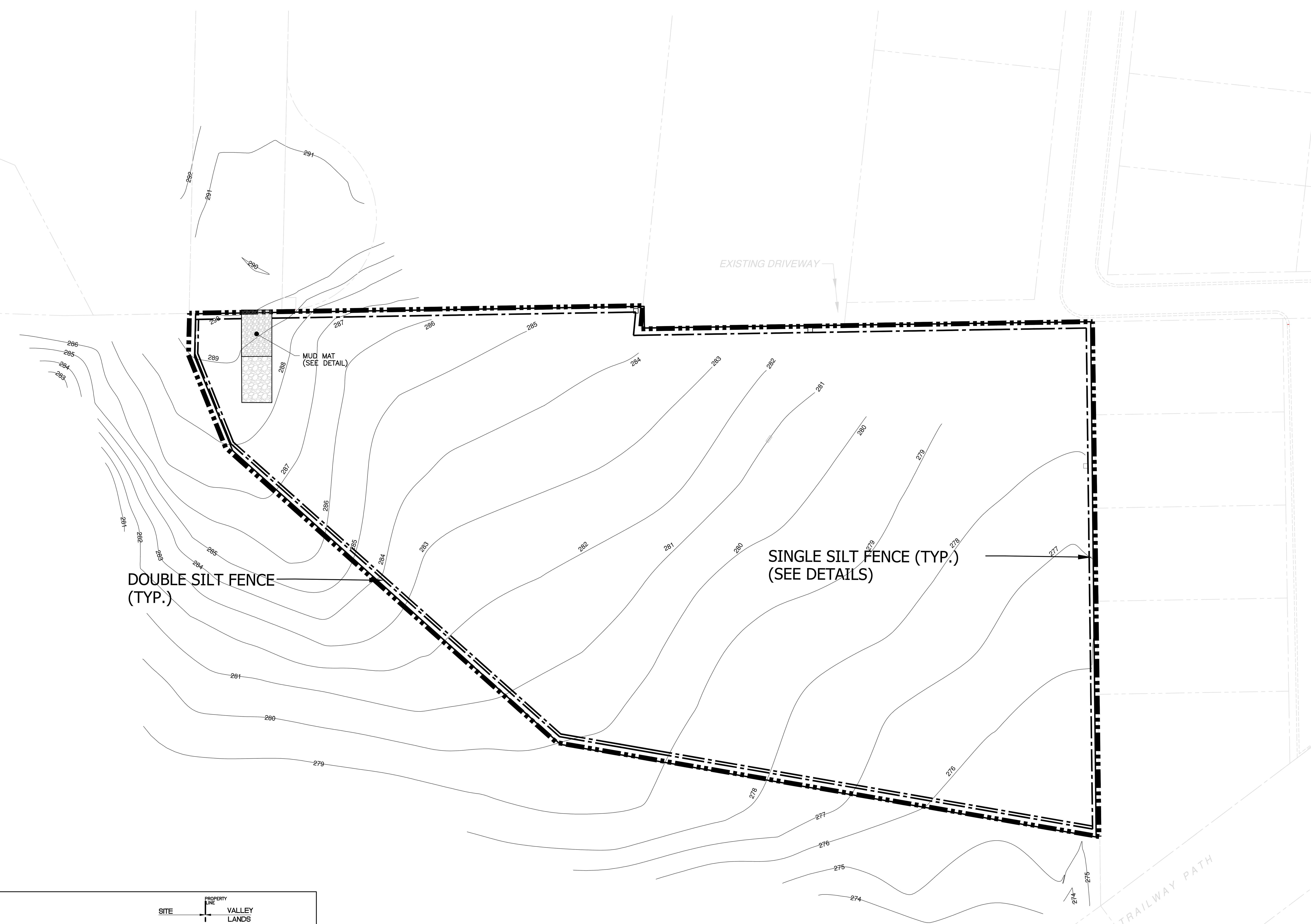
TEL. (905) 794-0600 FAX (905) 794-0611

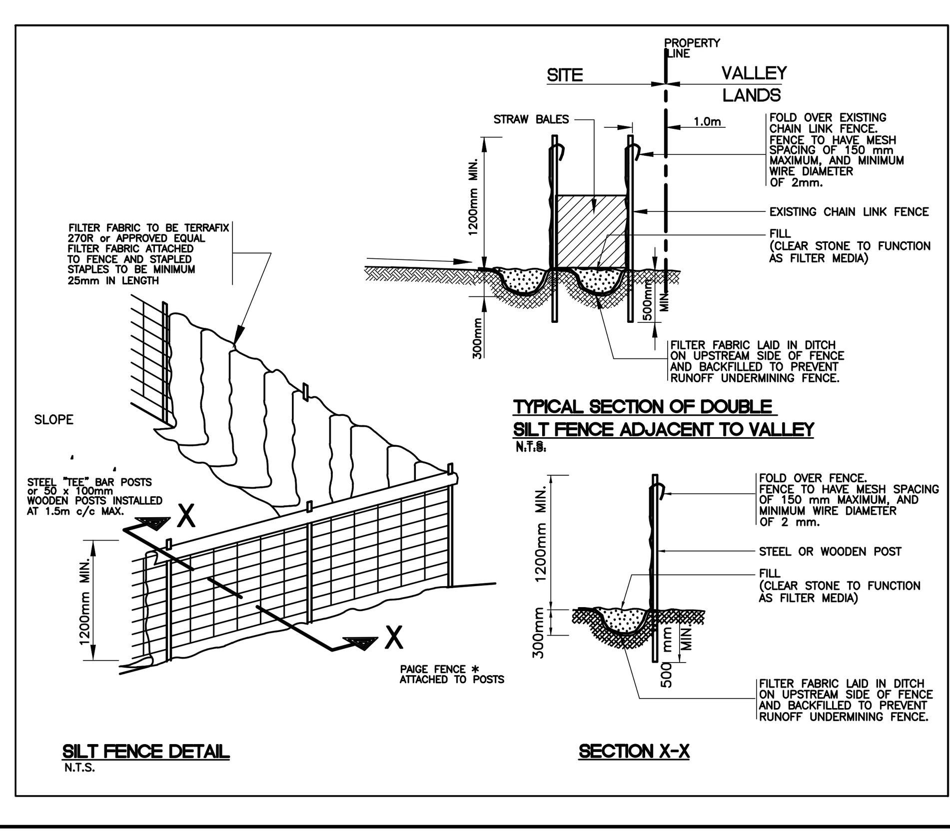
SCALE: 1:500 0m 5 10 15 20

DWG. No. PL-1

DATE: FEB., 7th 2024

PROJECT No. W22002





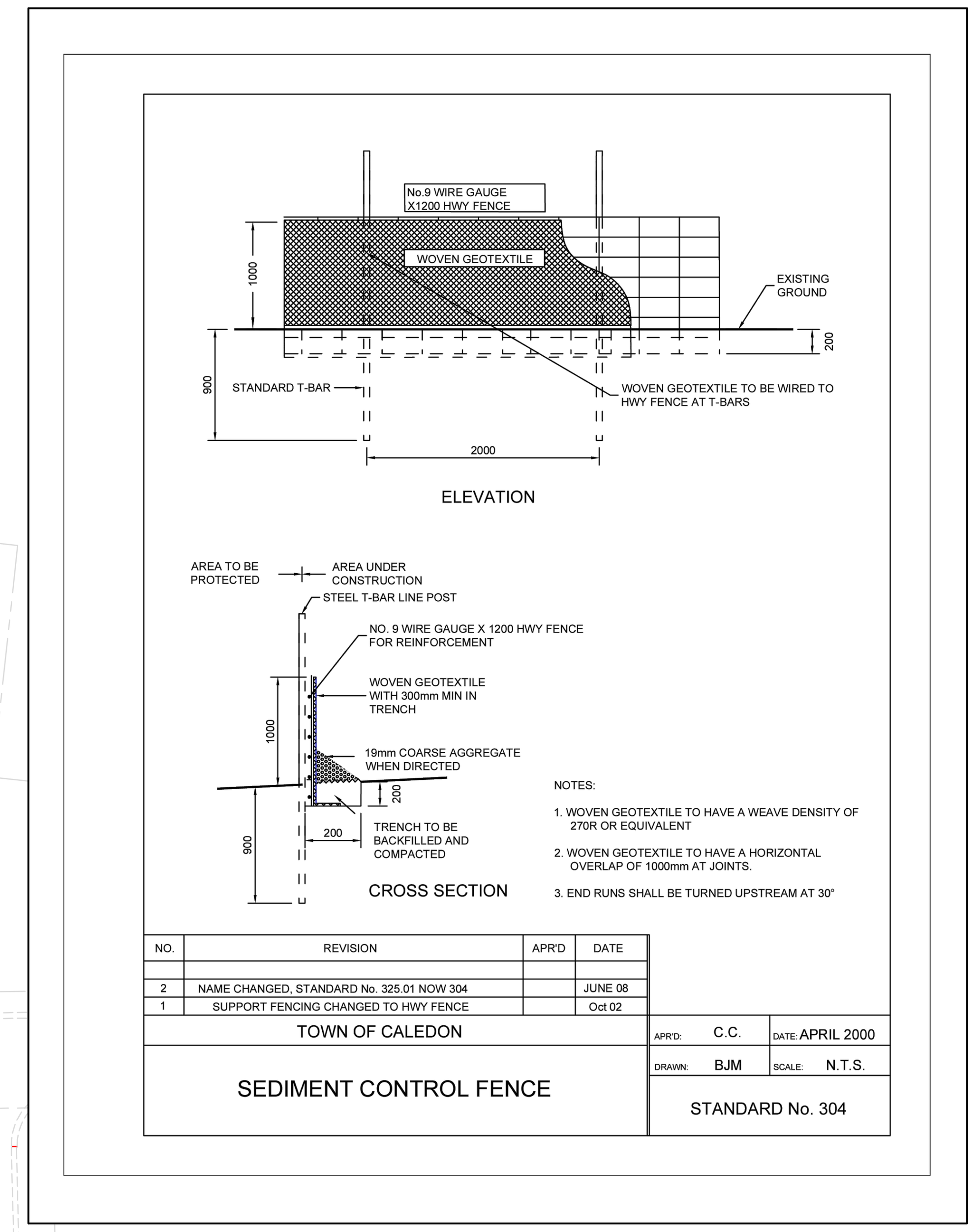
NOTES:

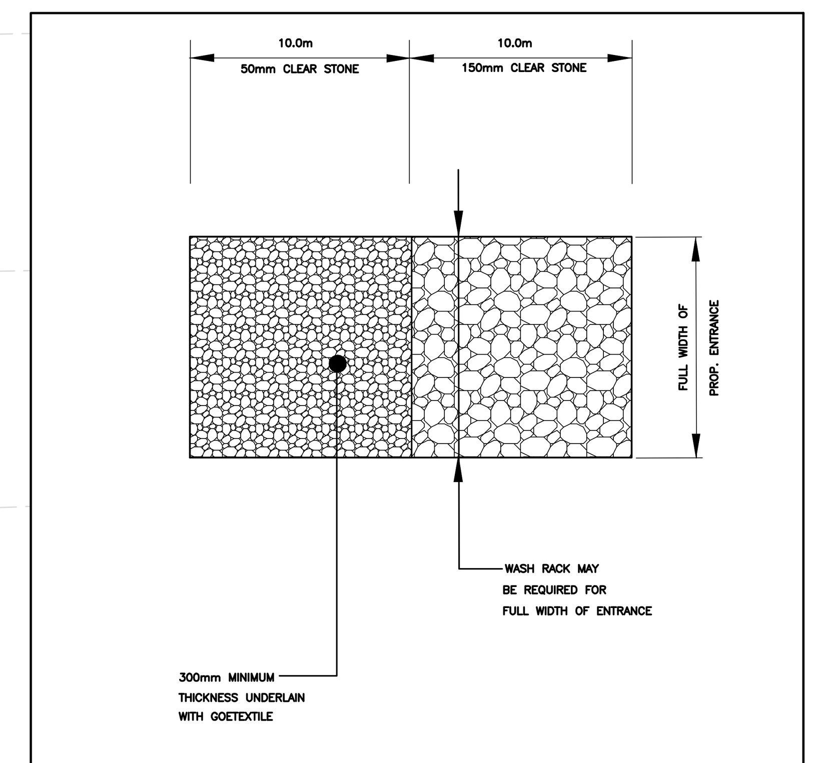
A. EROSION AND SEDIMENT CONTROL MEASURES:

1. ALL SEDIMENT AND EROSION CONTROL MEASURES SHALL BE INSTALLED AND IN PROPER WORKING ORDER PRIOR TO THE REMOVAL OF ANY TOPSOIL. THE EXACT LOCATION OF THE SEDIMENT CONTROL MEASURES SHALL BE CONFIRMED IN THE FIELD.
2. THE EROSION AND SEDIMENT CONTROL STRATEGIES OUTLINED ON THE PLANS ARE NOT STATIC AND MAY NEED TO BE UPGRADED/AMENDED AS SITE CONDITIONS CHANGE TO PREVENT SEDIMENT RELEASES TO THE NATURAL ENVIRONMENT.
3. ALL EROSION AND SEDIMENT CONTROL MEASURES SHALL BE INSPECTED WEEKLY OR IN THE EVENT OF A MAJOR STORM AND REPAIRS SHALL BE CONDUCTED WITHIN 48 HOURS.
4. ALL CONSTRUCTION VEHICLES SHALL EXIT THE SITE VIA THE TEMPORARY CONSTRUCTION ACCESS.
5. ALL TOPSOIL STOCKPILES SHALL BE SURROUNDED WITH SEDIMENT CONTROL FENCING. ALL PILES WHICH ARE STOCKPILED FOR MORE THAN 30 DAYS SHALL BE SEEDED.
6. SEDIMENT WHICH COLLECTS IN THE TEMPORARY SEDIMENT CONTROL FACILITIES SHALL BE REMOVED WHEN THE FACILITY IS HALF FULL.
7. CLEAN SCOTTSDALE DRIVE OF ANY TOPSOIL OR MUD ON A REGULAR BASIS.
8. DURING CONSTRUCTION AND UNTIL SUCH TIME AS THE SITE HAS BEEN PAVED AND SODDED, THE SEDIMENT CONTROL MEASURES SHALL BE MAINTAINED IN GOOD OPERATING CONDITION. THE CONTRACTOR SHALL PREVENT STORM WATER RUNOFF FROM DIRECTLY ENTERING THE MUNICIPAL STORM SEWER SYSTEM AND THE ADJOINING PROPERTY BY INSTALLING AND MAINTAINING SEDIMENT CONTROL FACILITIES AT ALL CATCHBASINS AND BY INSTALLING AND MAINTAINING THE SILT FENCE AS SHOWN ON THE DRAWING.

STORM WATER MANAGEMENT:

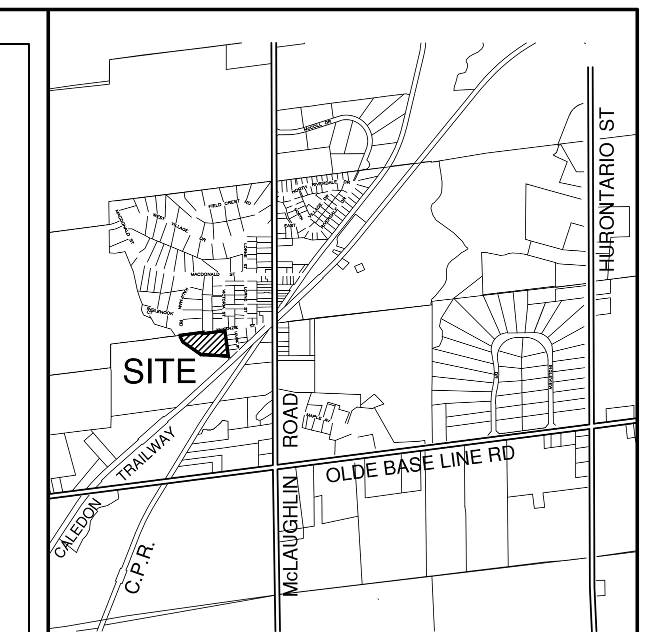
1. DURING CONSTRUCTION AND UNTIL SUCH TIME AS THE SITE HAS BEEN PAVED AND SODDED, THE CONTRACTOR SHALL PREVENT STORM WATER RUNOFF FROM DIRECTLY ENTERING THE MUNICIPAL STORM SEWER SYSTEM AND THE ADJOINING PROPERTIES BY INSTALLING AND MAINTAINING SILT CONTROL FACILITIES AT ALL CATCHBASINS AND BY INSTALLING AND MAINTAINING SILT FENCE AS PER TOWN OF CALEDON STANDARDS.
2. STORMWATER MANAGEMENT CONTROLS SHALL BE IN ACCORDANCE WITH THE STORM WATER MANAGEMENT REPORT PREPARED BY CANDEVCON GROUP INC. FOR THE PROJECT.





CONSTRUCTION ACCESS MUD MAT DETAILS
N.T.S

BAR SCALE
1:500



KEY PLAN (N.T.S.)

LEGEND:

- PROPERTY LINE
- SINGLE SILT FENCE
- DOUBLE SILT FENCE
- MUD MAT (SEE DETAIL)

NO.	DESCRIPTION	DATE	BY
5			
4			
3			
2			
1	FIRST SUBMISSION	24-05-07	SDL

REVISIONS

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS
9358 GOREWAY DRIVE
TEL. (905) 794-0600
BRAMPTON, ONTARIO L6P 0M7
FAX (905) 794-0611

REGISTERED PROFESSIONAL ENGINEER
S. D. LANG
MAY 28, 2024
PROVINCE OF ONTARIO

CONSTRUCTION NORTH

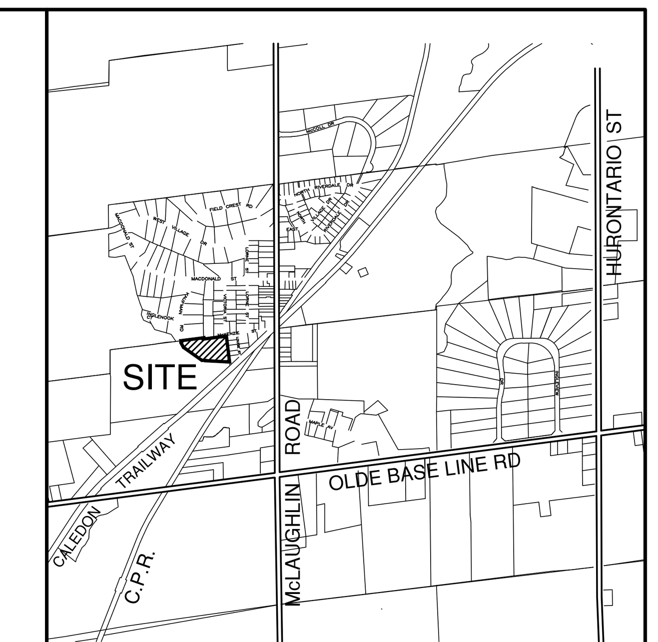
2868577 ONTARIO INC
PROPOSED RESIDENTIAL SUBDIVISION
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD
TOWN OF CALEDON

SHEET TITLE:

EROSION AND SEDIMENT CONTROL PLAN

TOWN FILE No. PRE-2029-0197
REGION FILE No.

DRAWN BY: Y.D.N./D.R.
PROJECT No. W22002
DESIGN/CHECK BY: S.D.L./D.K.H.
DRAWING No. ESC-1
SCALE: 1:500
DATE: AUGUST 15th. 2023



KEY PLAN (N.T.S.)

LEGEND:

	PROPERTY LINE
278.85	EXISTING ELEVATION
× 285.8	PROPOSED ELEVATION
150mm ø WM	PROPOSED WATERMAIN
EX H&V	EXISTING FIRE HYDRANT
	FLOW DIRECTION
	INFILTRATION GALLERY

NO.	DESCRIPTION	DATE	BY
5			
4			
3			
2			
1			

REVISIONS

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS
9358 GOREWAY DRIVE
TEL. (905) 794-0600
BRAMPTON, ONTARIO L6P 0M7
FAX (905) 794-0611

REGISTERED PROFESSIONAL ENGINEER
S. D. LANG
MAY 28, 2024
PROVINCE OF ONTARIO

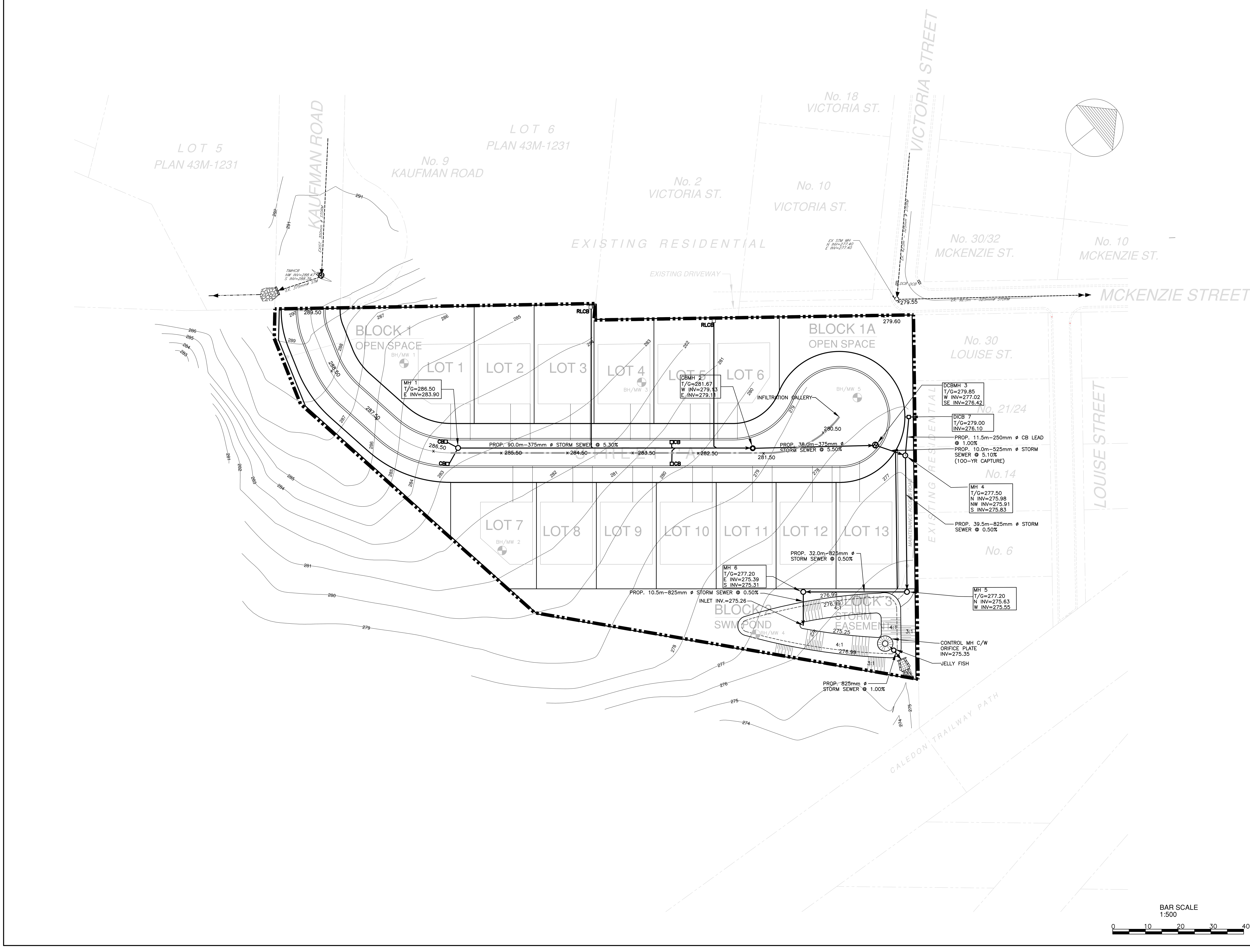
CONSTRUCTION NORTH
TRUE NORTH

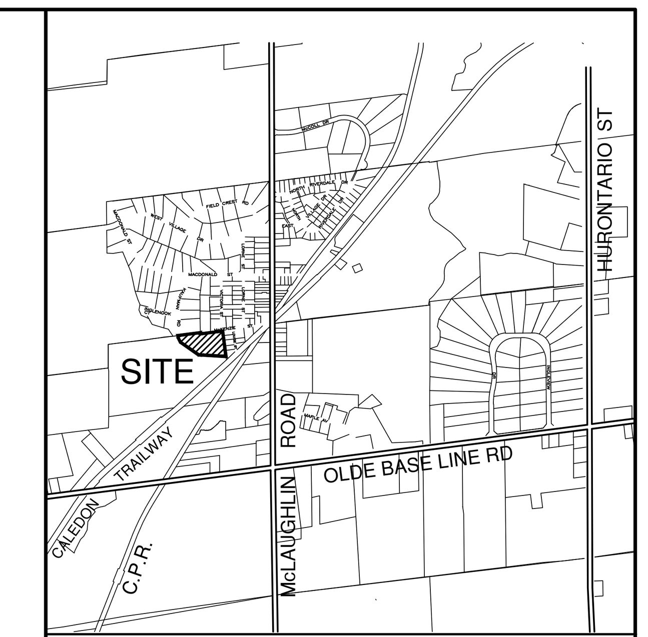
2868577 ONTARIO INC
PROPOSED RESIDENTIAL SUBDIVISION
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD
TOWN OF CALEDON

SHEET TITLE:
WATER DISTRIBUTION PLAN

TOWN FILE No. PRE-2029-0197
REGION FILE No.

DRAWN BY: D.R.	PROJECT No. W22002
DESIGN/CHECK BY: S.D.L./D.K.H.	DRAWING No.
SCALE: 1:500	WD-1
DATE: AUGUST 15th. 2023	1





KEY PLAN (N.T.S.)

LEGEND:

	PROPERTY LINE
	DRAINAGE AREA BOUNDARY
278.65	EXISTING ELEVATION
x285.8	PROPOSED ELEVATION
EX STM MH	EXISTING STORM MANHOLE
STM MH	PROPOSED STORM MANHOLE
	OVERLAND FLOW DIRECTION
CB	PROPOSED CATCH BASIN
CB	PROPOSED CATCH BASIN MANHOLE
RLCB	PROPOSED REAR LOT CATCH BASIN

NO.	DESCRIPTION	DATE	BY
5			
4			
3			
2			
1	FIRST SUBMISSION	24–05–07	SDL

REVISIONS

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS

9358 GOREWAY DRIVE
TEL. (905) 794–0600

BRAMPTON, ONTARIO L6P 0M7
FAX (905) 794–0611

REGISTERED PROFESSIONAL ENGINEER
S. D. LANG
MAY 28, 2024
PROVINCE OF ONTARIO

CONSTRUCTION NORTH
TRUE NORTH

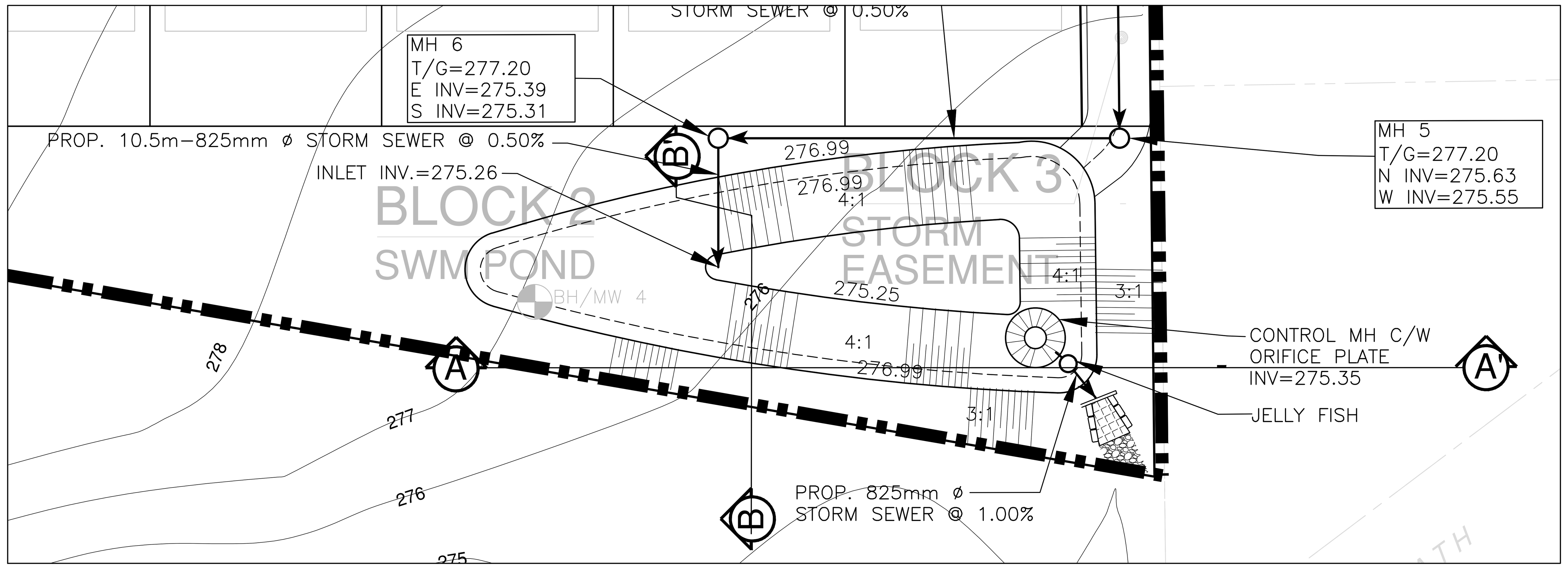
2868577 ONTARIO INC
PROPOSED RESIDENTIAL SUBDIVISION
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD
TOWN OF CALEDON

SHEET TITLE:

INFILTRATION TRENCH PLAN

TOWN FILE No. PRE–2029–0197
REGION FILE No.

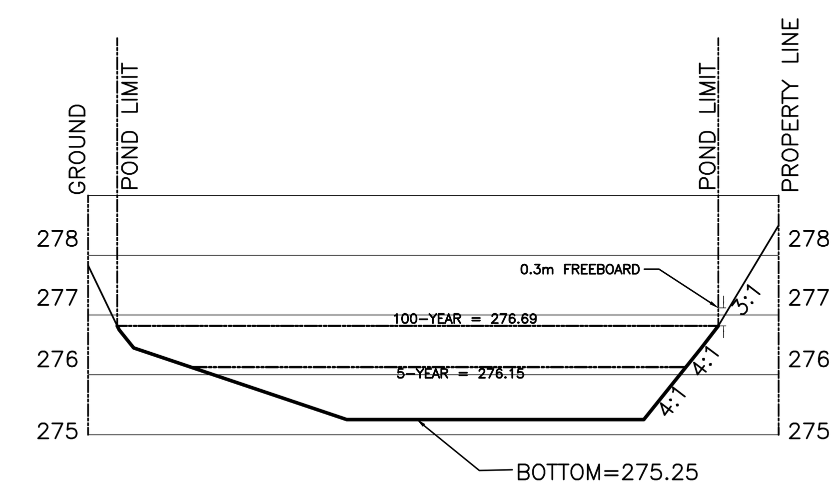
DRAWN BY: Y.D.N./D.R.
PROJECT No. W22002
DESIGN/CHECK BY: S.D.L./D.K.H.
DRAWING No. IT–1
SCALE: 1:500
DATE: AUGUST 15th, 2023



PLAN – SWM POND

1:500

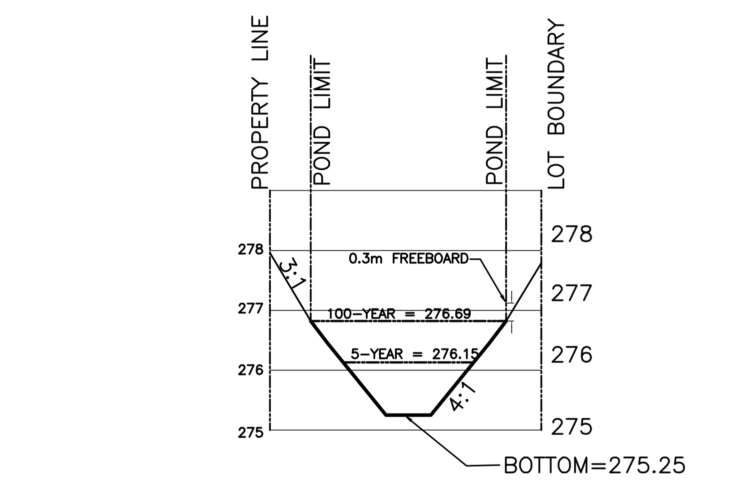
BLOCK 3
(STORMWWATER MANAGEMENT POND)



SECTION 'A' EAST-WEST

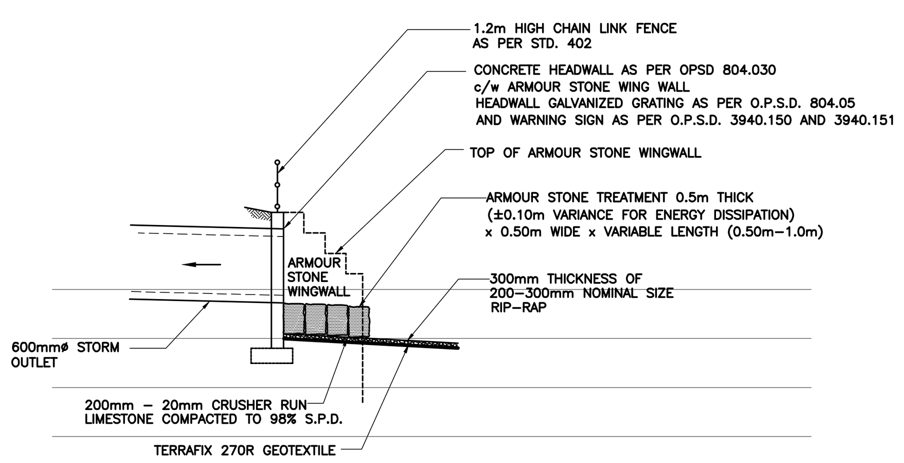
H-1:500 V-1:100

BLOCK 3
(STORMWWATER MANAGEMENT POND)



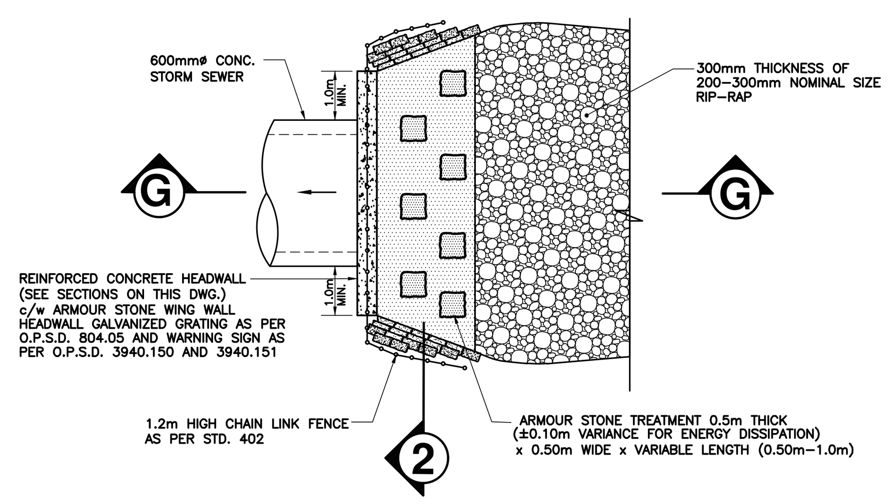
SECTION 'B' NORTH-SOUTH

H-1:500 V-1:100



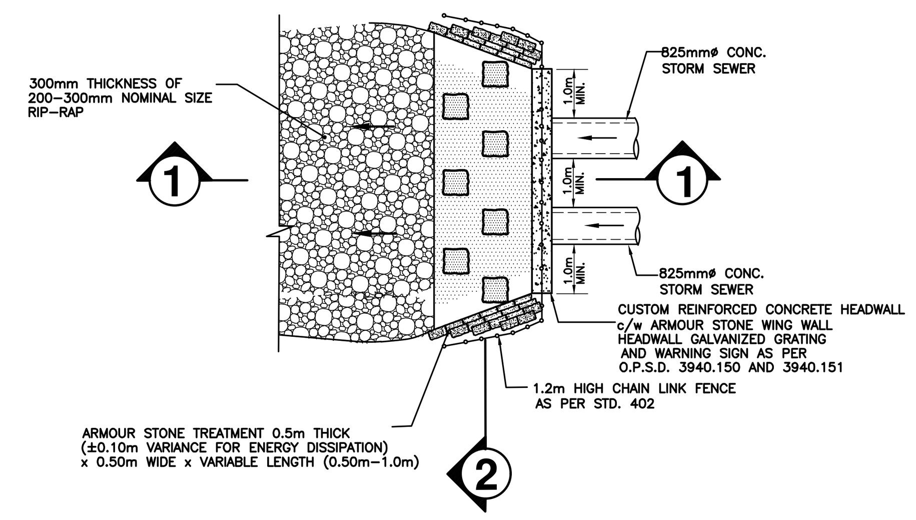
SECTION G-G AT POND OUTLET

H=1:500 V=1:100



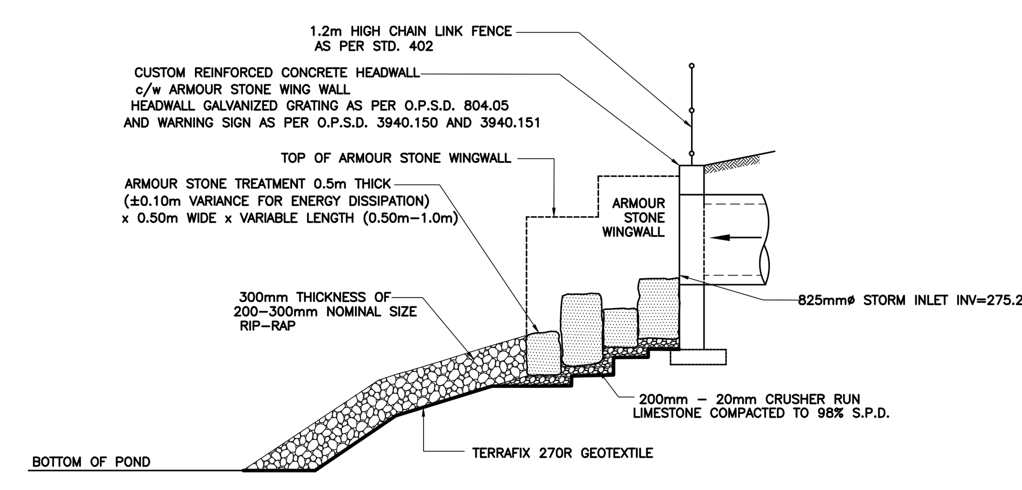
PLAN – HEADWALL DETAIL AT POND OUTLET

N.T.S.



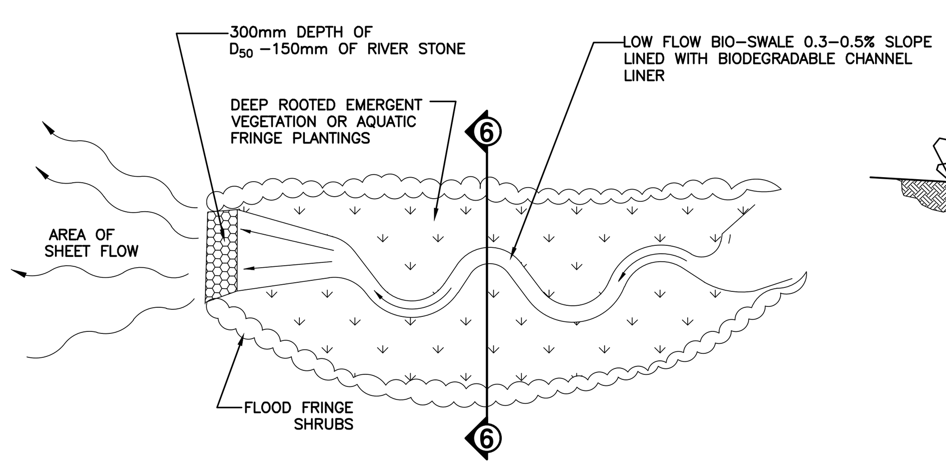
PLAN – HEADWALL DETAIL AT POND INLET

N.T.S.



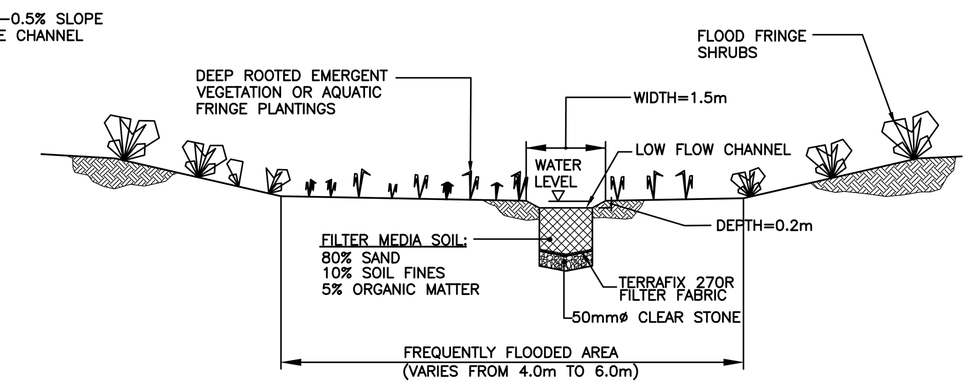
SECTION 1-1 AT POND INLET

H=1:100 V=1:50



PLAN (TYPICAL)

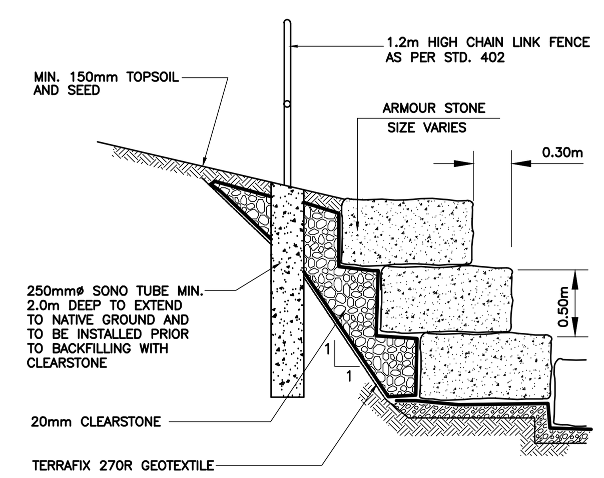
N.T.S.



SECTION '6-6'

DETAIL OF OUTLET CHANNEL

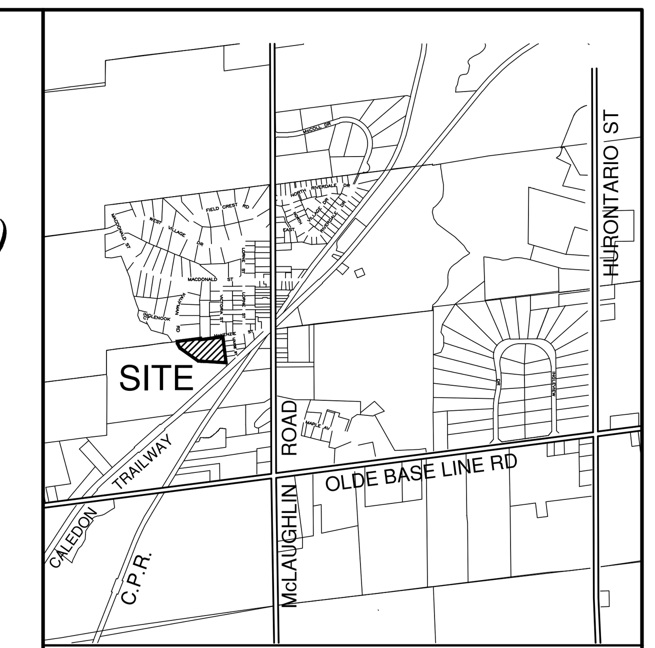
N.T.S.



SECTION 2 AT POND INLET/ OUTLET (TYP.)

N.T.S.

BAR SCALE
1:500
10 20 30 40



KEY PLAN (N.T.S.)

LEGEND:

LIMIT OF POND

NO.	DESCRIPTION	DATE	BY
5			
4			
3			
2			
1			

REVISIONS

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS
9358 GOREWAY DRIVE
TEL. (905) 794-0600
BRAMPTON, ONTARIO L6P 0M7
FAX (905) 794-0611

REGISTERED PROFESSIONAL ENGINEER
S. D. LANG
MAY 28, 2024
PROVINCE OF ONTARIO

CONSTRUCTION NORTH

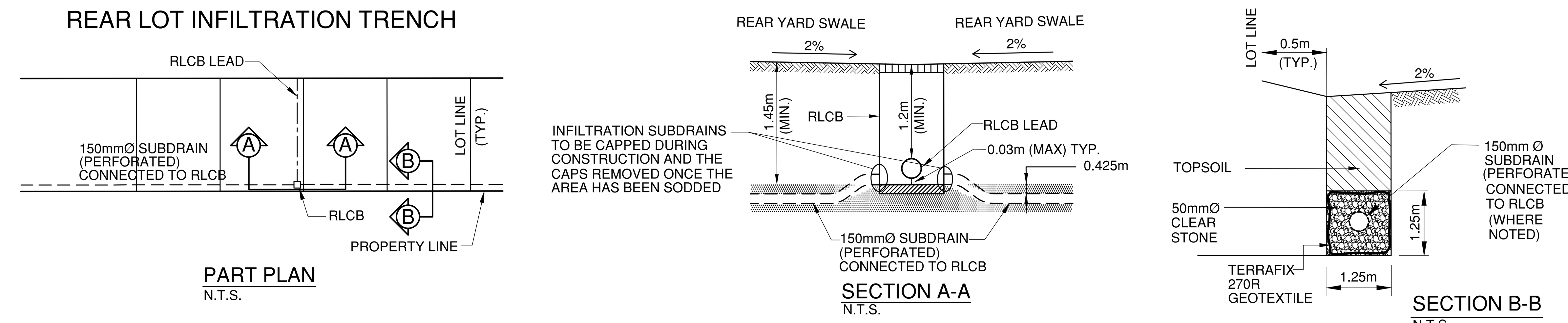
2868577 ONTARIO INC
PROPOSED RESIDENTIAL SUBDIVISION
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD
TOWN OF CALEDON

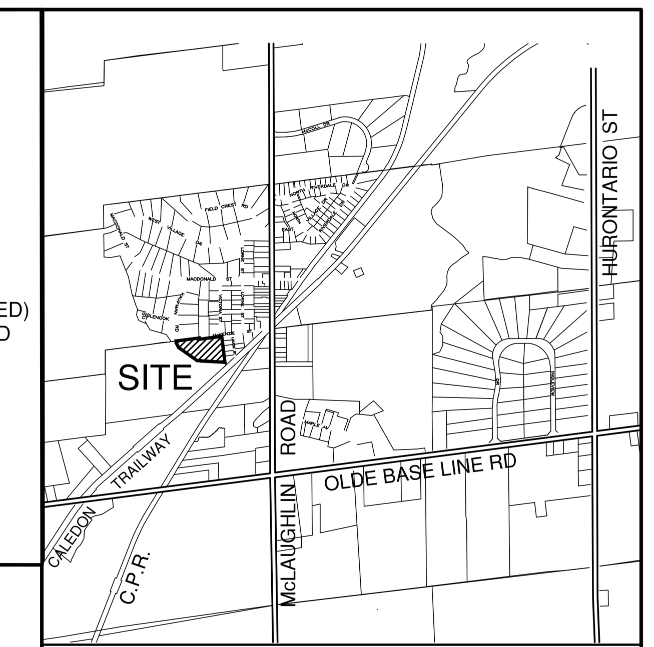
SHEET TITLE:
PRELIMINARY STORM WATER MANAGEMENT POND PLAN AND SECTIONS

TOWN FILE No. PRE-2029-0197
REGION FILE No.

DRAWN BY: Y.D.N./D.R.
PROJECT No. W22002
DESIGN/CHECK BY: S.D.L./D.K.H.
DRAWING No.
SCALE: SEE SPECIFICS
PSWM-1
DATE: AUGUST 15th, 2023

REAR LOT INFILTRATION TRENCH

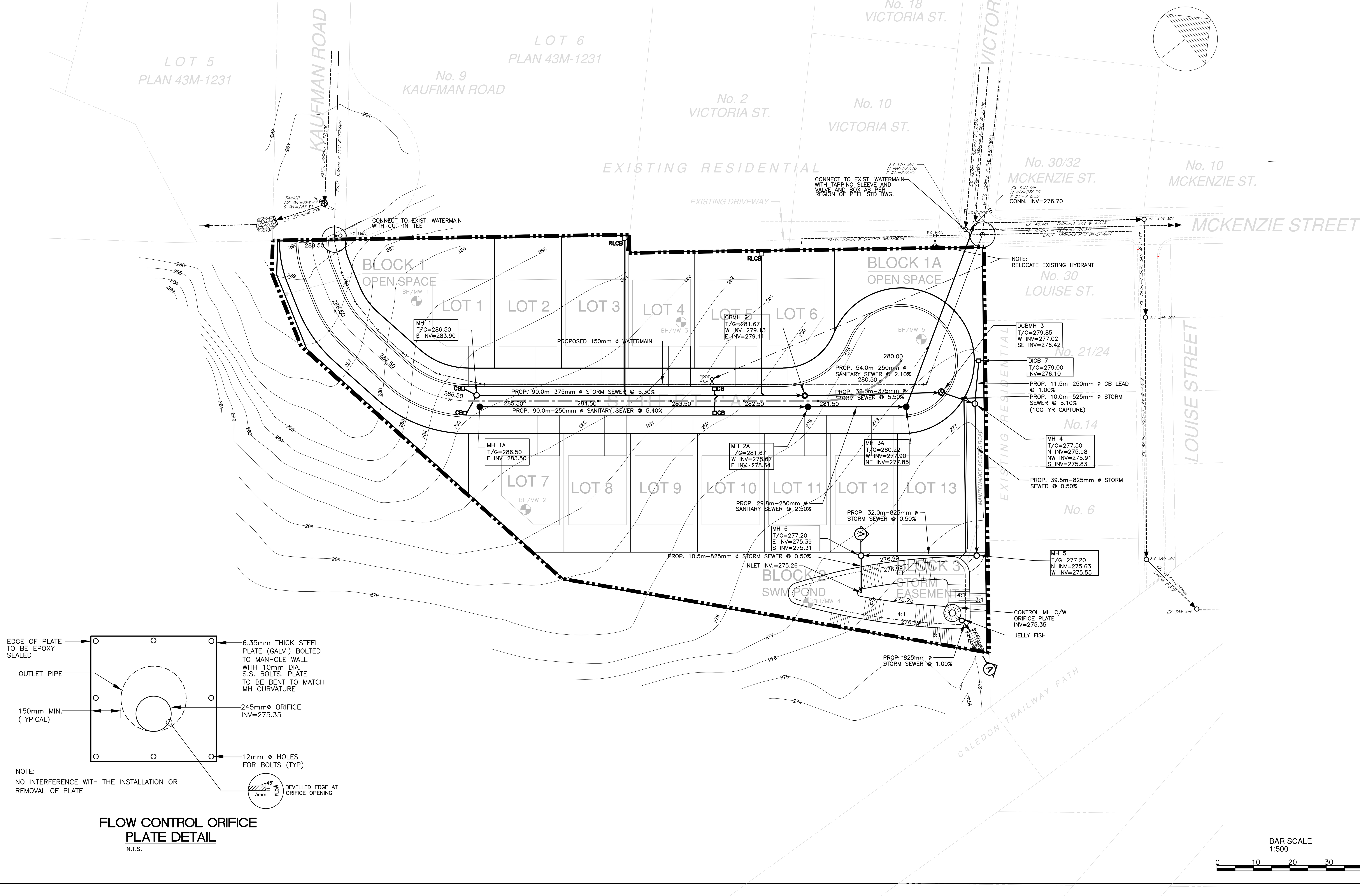




KEY PLAN (N.T.S.)

LEGEND:

	PROPERTY LINE
278.85	EXISTING ELEVATION
x 285.8	PROPOSED ELEVATION
EX STM MH	EXISTING STORM MANHOLE
EX SAN MH	EXISTING SANITARY MANHOLE
STM MH	PROPOSED STORM MANHOLE
SAN MH	PROPOSED SANITARY MANHOLE
150mm ø WM	PROPOSED WATERMAIN
EX. H&V	EXISTING FIRE HYDRANT
PROP. H&V	PROPOSED FIRE HYDRANT
CB	PROPOSED CATCH BASIN
RLCB	PROPOSED REAR LOT CATCH BASIN
	OVERLAND FLOW DIRECTION
	INFILTRATION GALLERY



NOTE:
NO INTERFERENCE WITH THE INSTALLATION OR REMOVAL OF PLATE

FLOW CONTROL ORIFICE PLATE DETAIL

N.T.S.

NO.	DESCRIPTION	DATE	BY
6			
5			
4			
3			
2			
1			

REVISIONS

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS

9358 GOREWAY DRIVE BRAMPTON, ONTARIO L6P 0M7
TEL. (905) 794-0600 FAX (905) 794-0611

REGISTERED PROFESSIONAL ENGINEER S. D. LANG MAY 28, 2024 PROVINCE OF ONTARIO

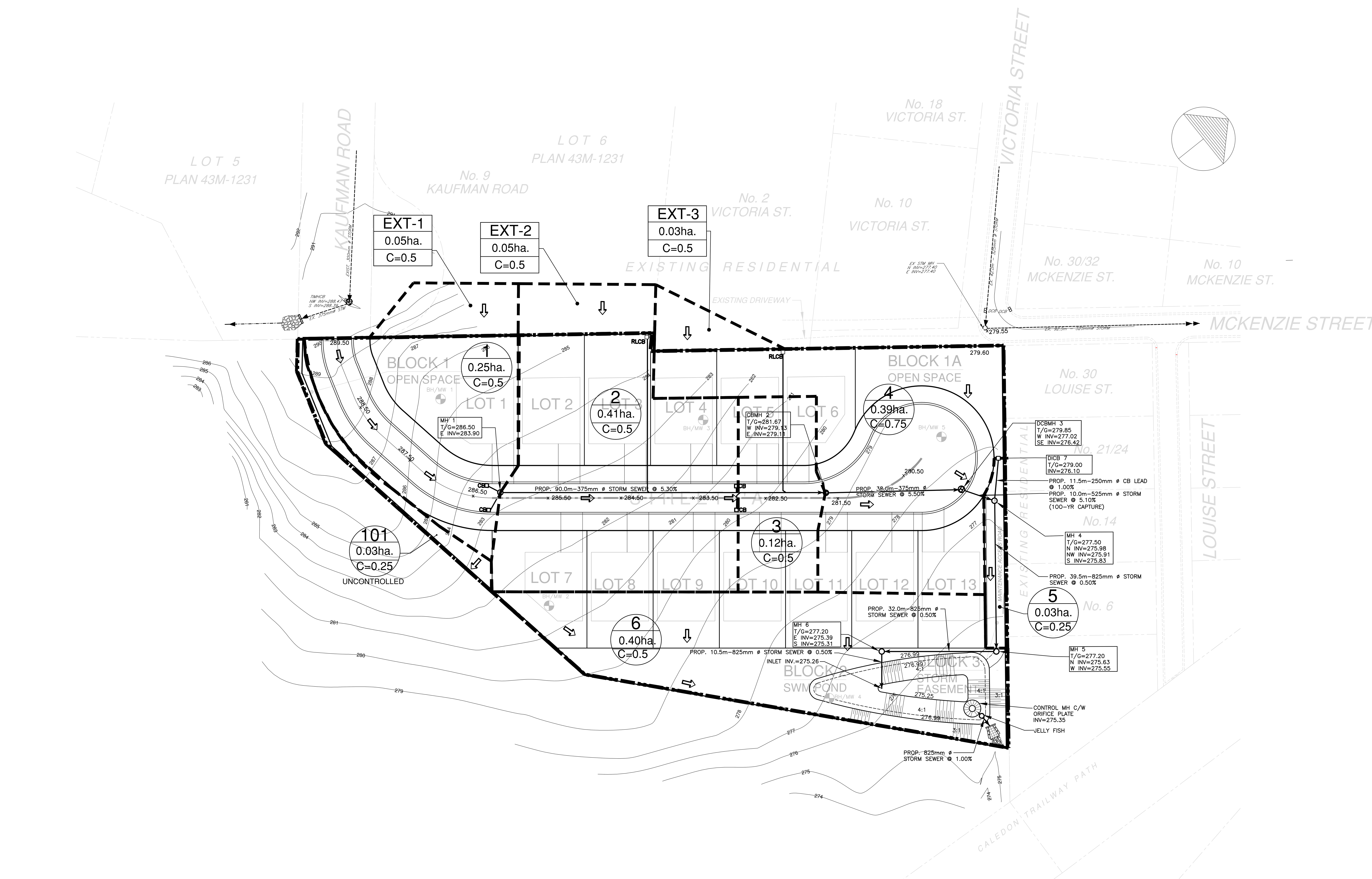
CONSTRUCTION NORTH

2868577 ONTARIO INC
PROPOSED RESIDENTIAL SUBDIVISION
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD
TOWN OF CALEDON

SHEET TITLE:
PRELIMINARY SERVICING PLAN

TOWN FILE No. PRE-2029-0197
REGION FILE No.

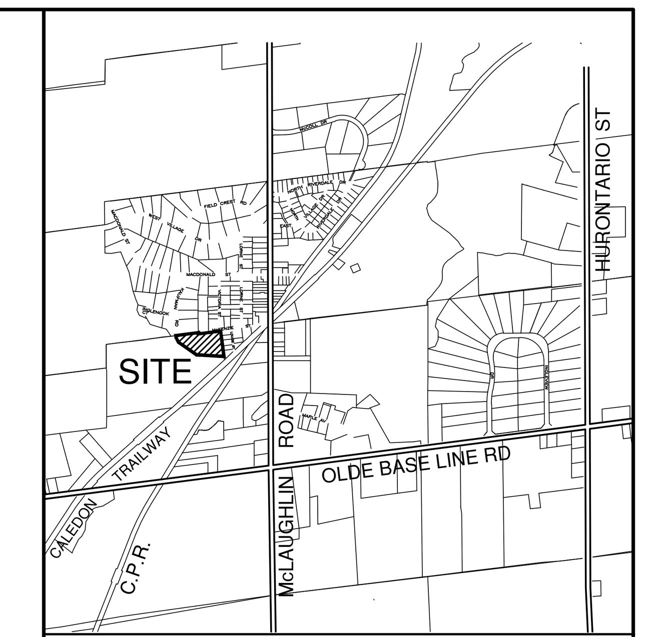
DRAWN BY: D.R.	PROJECT No. W22002
DESIGN/CHECK BY: S.D.L./D.K.H.	DRAWING No.
SCALE: 1:500	PS-1
DATE: AUGUST 15th. 2023	



BAR SCALE
1:500
0 10 20 30 40

GENERAL NOTES – STORM SEWERS

A. STORM SEWER TO BE PROVIDED ON ALL ROADS WITH CURB AND GUTTER.
B. PLACE ALL CATCH BASIN LATERALS AT 2% GRADE UNLESS OTHERWISE NOTED. PIPE SIZE MINIMUM 250mm DIA. SINGLE, 300mm DIA. DOUBLE.
C. STORM SEWERS SHALL BE CONSTRUCTED WITH BEDDING AS PER OPSD 802.030 FOR RIGID PIPE OR OPSD 802.010 WITH GRANULAR 'A' FOR FLEXIBLE PIPE UNLESS APPROVED OTHERWISE BY THE DIRECTOR OF ENGINEERING SERVICES.
D. MAINTENANCE HOLE TOPS (FRAMES) AND CATCHBASIN (FRAMES) ARE TO BE SET TO BASE COURSE ASPHALT AND THEN ADJUSTED TO FINAL GRADE WHEN THE TOP LIFT OF ASPHALT IS PLACED.
E. STORM SEWER TO BE LOCATED OFFSET 1.5m SOUTH OR WEST OF CENTERLINE UNLESS OTHERWISE SPECIFIED.
F. ALL CONNECTIONS TO THE STORM MAIN SHALL BE MADE WITH A STORM MANHOLE OR APPROVED FACTORY TEE CONNECTION AS PER OPSD 708.01 OR 708.03.
G. PIPE MATERIAL TO BE REINFORCED CONCRETE WITH A STRENGTH OF 50 N/m/mm CERTIFIED TO C.S.A. STANDARD A247.2–1982, CLASS 50–D (PREVIOUSLY C.S.A. STANDARD A257.2–1974 CLASS II) OR PVC CERTIFIED TO C.S.A. STANDARDS 182.2 AND 182.4 MAX. PVC PIPE DIA. IS 600mm BIG O BOSS 2000 POLYETHYLENE PIPE WITH GASKETED BELL AND SPIGOT JOINTS CERTIFIED C.S.A. B182.6 FOR STORM SEWERS UP TO 900mm DIA. WHERE ONLY CONNECTION STD CATCHBASINS ARE CONSIDERED.
H. STORM SEWER TO BE MINIMUM 300mm DIA. WITH JOINTS CONFORMING TO C.S.A. STANDARD A 257.3
I. ALL PIPE BEDDING MUST CONFORM TO OPSD MAXIMUM COVER TABLE. NO FLEXIBLE PIPE SEWERS WILL BE INSTALLED WITH A DEPTH COVER GREATER THAN 6m UNLESS SPECIFICALLY APPROVED BY THE DIRECTOR OF ENGINEERING SERVICES.
J. ALL PIPE HANDLING INSTRUCTIONS MUST BE IN STRICT COMPLIANCE WITH MANUFACTURERS INSTALLATION GUIDES AND THE OCPA OR UNIBELL GUIDELINES.



KEY PLAN (N.T.S.)

LEGEND:

	PROPERTY LINE
	DRAINAGE AREA BOUNDARY
278.85	EXISTING ELEVATION
x285.8	PROPOSED ELEVATION
EX STM MH	EXISTING STORM MANHOLE
STM MH	PROPOSED STORM MANHOLE
	OVERLAND FLOW DIRECTION
CB	PROPOSED CATCH BASIN
CB	PROPOSED CATCH BASIN MANHOLE
RLCB	PROPOSED REAR LOT CATCH BASIN

1 — AREA NUMBER
0.25ha. — DRAINAGE AREA
C=0.5 — COEFFICIENT

NO.	DESCRIPTION	DATE	BY
5			
4			
3			
2			
1	FIRST SUBMISSION	24–05–07	SDL

REVISIONS

CANDEVCON LIMITED
CONSULTING ENGINEERS AND PLANNERS
9358 GOREWAY DRIVE
TEL. (905) 794–0600
BRAMPTON, ONTARIO L6P 0M7
FAX (905) 794–0611

REGISTERED PROFESSIONAL ENGINEER
S. D. LANG
MAY 28, 2024
PROVINCE OF ONTARIO

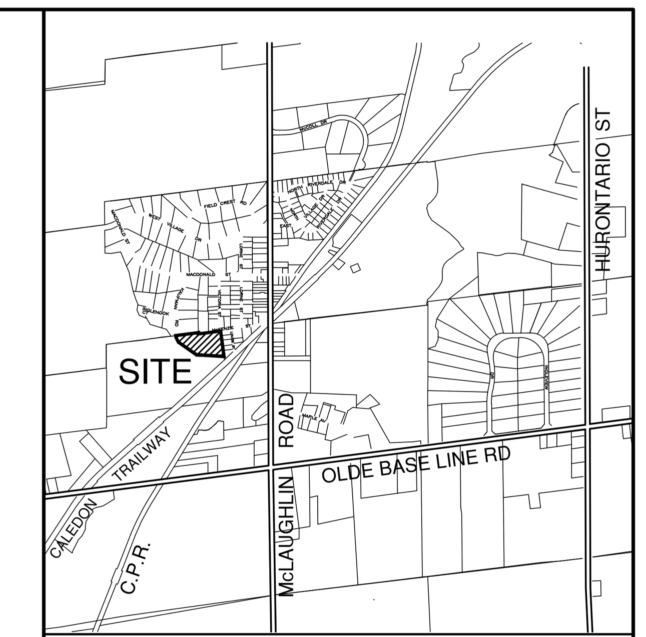
CONSTRUCTION NORTH
TRUE NORTH

2868577 ONTARIO INC
PROPOSED RESIDENTIAL SUBDIVISION
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD
TOWN OF CALEDON

SHEET TITLE:
POST-DEVELOPMENT DRAINAGE AREA PLAN

TOWN FILE No. PRE–2029–0197
REGION FILE No.

DRAWN BY: Y.D.N./D.R.	PROJECT No. W22002
DESIGN/CHECK BY: S.D.L./D.K.H.	DRAWING No.
SCALE: 1:500	STMDR–2
DATE: AUGUST 15th. 2023	1



KEY PLAN (N.T.S.)

LEGEND:

PROPERTY LINE
DRAINAGE AREA BOUNDARY
EXISTING ELEVATION
PROPOSED ELEVATION
EXISTING STORM MANHOLE
PROPOSED STORM MANHOLE
OVERLAND FLOW DIRECTION

AREA NUMBER
DRAINAGE AREA
COEFFICIENT

NO.	DESCRIPTION	DATE	BY
5			
4			
3			
2			
1	FIRST SUBMISSION	24-05-07	SDL

REVISIONS

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S. D. LANG
MAY 28, 2024
PROVINCE OF ONTARIO

CONSTRUCTION NORTH
TRUE NORTH

2868577 ONTARIO INC
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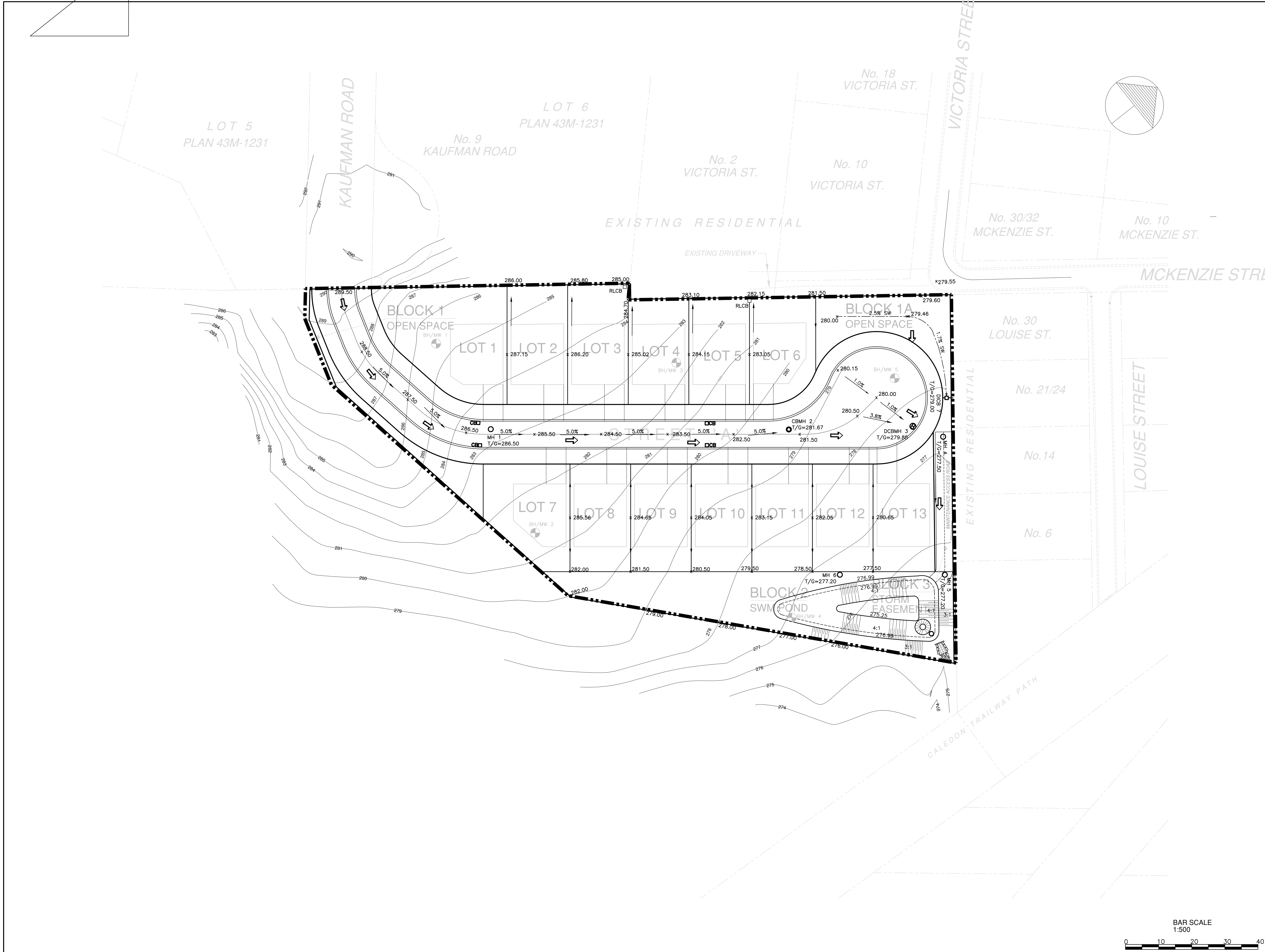
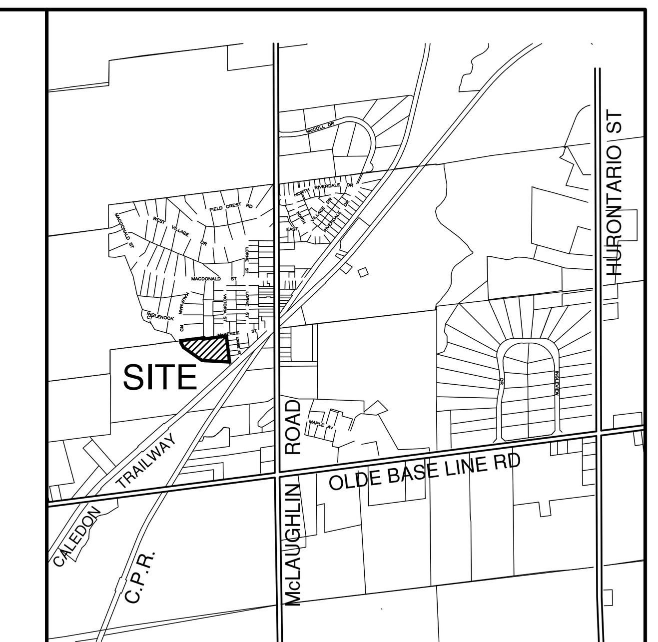
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD
TOWN OF CALEDON

SHEET TITLE:
PRE-DEVELOPMENT DRAINAGE AREA PLAN

TOWN FILE No. PRE-2029-0197
REGION FILE No.

DRAWN BY: Y.D.N./D.R.
PROJECT No. W22002
DESIGN/CHECK BY: S.D.L./D.K.H.
DRAWING No. STMDR-1
SCALE: 1:500
DATE: AUGUST 15th. 2023
1





KEY PLAN (N.T.S.)

LEGEND:

	PROPERTY LINE
278.85	EXISTING ELEVATION
x285.8	PROPOSED ELEVATION
	EXISTING STORM MANHOLE
	EXISTING SANITARY MANHOLE
STM MH	PROPOSED STORM MANHOLE
SAN MH	PROPOSED SANITARY MANHOLE
	EXISTING FIRE HYDRANT
	FLOW DIRECTION
RLCB	PROPOSED REAR LOT CATCH BASIN
	INFILTRATION GALLERY

NO.	DESCRIPTION	DATE	BY
5			
4			
3			
2			
1			

REVISIONS

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MAY 28, 2024
PROVINCE OF ONTARIO

CONSTRUCTION NORTH
TRUE NORTH

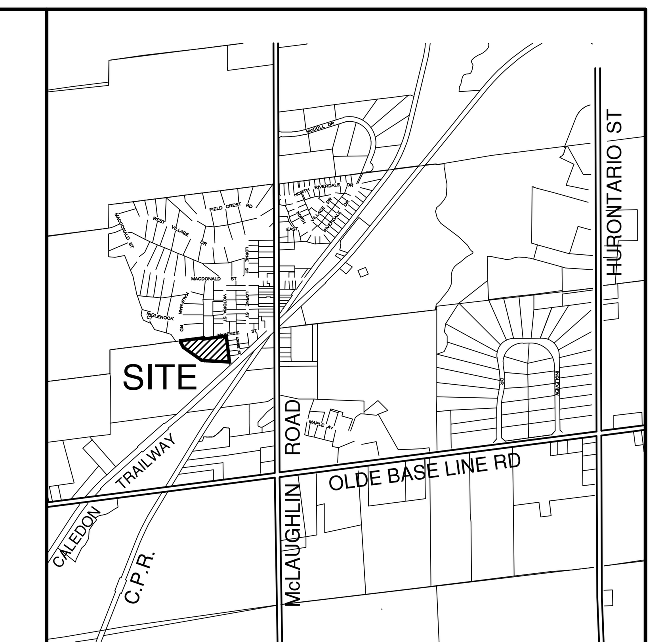
2868577 ONTARIO INC
PROPOSED RESIDENTIAL SUBDIVISION
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD
TOWN OF CALEDON

SHEET TITLE:

PRELIMINARY GRADING PLAN

TOWN FILE No. PRE-2029-0197
REGION FILE No.

DRAWN BY:	D.R./Y.D.N.	PROJECT No.	W22002
DESIGN/CHECK BY:	S.D.L./D.K.H.	DRAWING No.	PG-1
SCALE:	1:500		
DATE:	AUGUST 15th. 2023		1



KEY PLAN (N.T.S.)

LEGEND:

- PROPERTY LINE
- EXISTING ELEVATION
- 285.8 PROPOSED ELEVATION
- EXISTING SANITARY MANHOLE
- SAN MH PROPOSED SANITARY MANHOLE
- FLOW DIRECTION
- DRAINAGE AREA BOUNDARY
- 1 AREA NUMBER
- 0.017ha. DRAINAGE AREA
- P=46 POPULATION

NO.	DESCRIPTION	DATE	BY
5			
4			
3			
2			
1			

REVISIONS

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MAY 28, 2024
PROVINCE OF ONTARIO

CONSTRUCTION NORTH

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PROPOSED RESIDENTIAL SUBDIVISION
15544 MCLAUGHLIN ROAD
VILLAGE OF INGLEWOOD
TOWN OF CALEDON

SHEET TITLE:
SANITARY DRAINAGE AREA PLAN

TOWN FILE No. PRE-2029-0197
REGION FILE No.

DRAWN BY: Y.D.N./D.R.	PROJECT No. W22002
DESIGN/CHECK BY: S.D.L./D.K.H.	DRAWING No.
SCALE: 1:500	SANDR-1
DATE: AUGUST 15th. 2023	