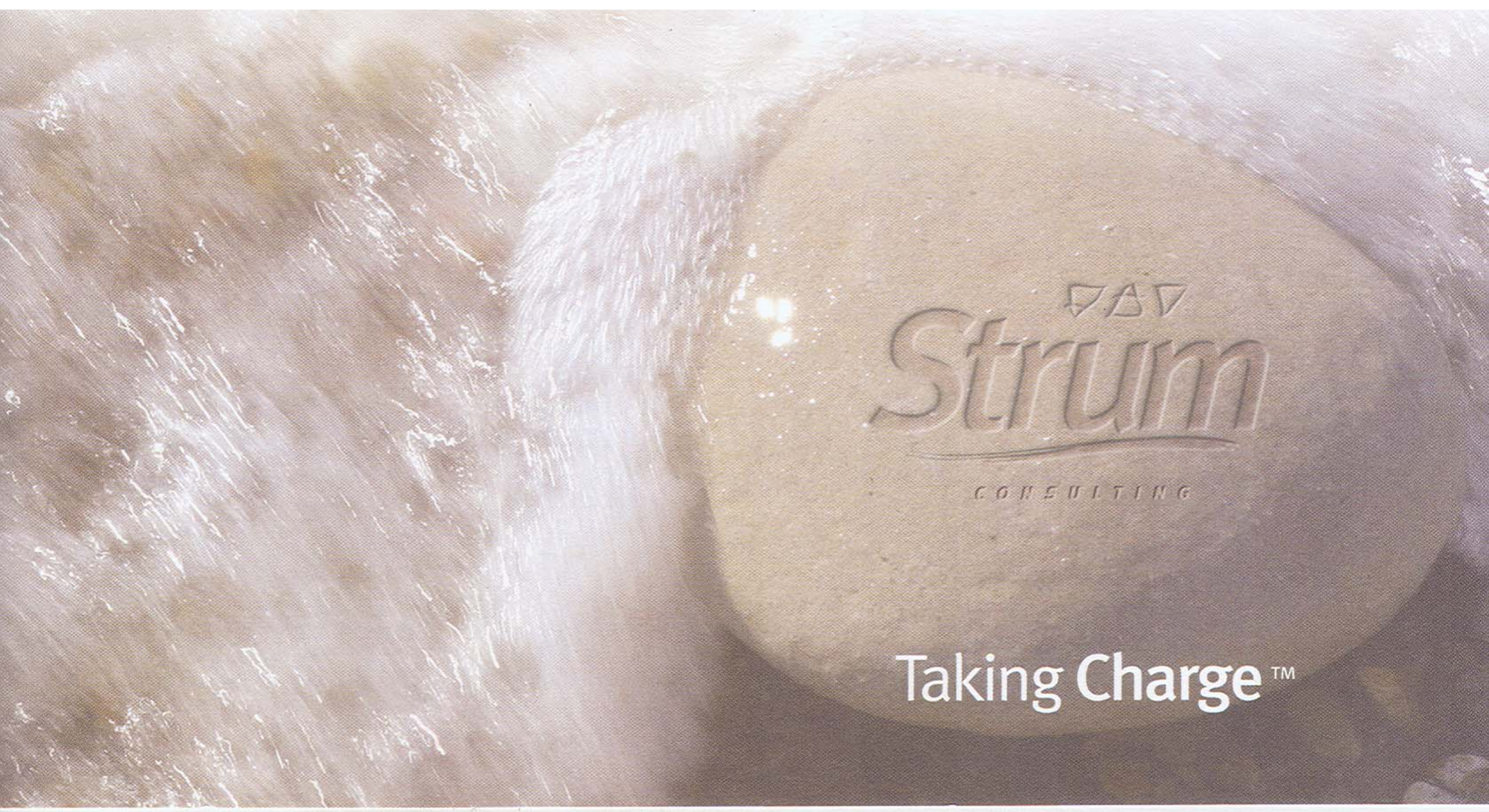
Strum

CONSULTING

PHOSPHORUS NET LOADING ASSESSMENT
Townhomes of Lake Thomas

June 12, 2020



June 12, 2020

Mr. John MacDonald
3293309 N.S.U.L.C
3136 Highway 2
Fall River, NS   B2T 1J5

Dear Mr. MacDonald,

Re: Phosphorus Net Loading Assessment (PNLA)
Townhomes of Lake Thomas

Attached is the Phosphorus Net Loading Assessment prepared for the Townhomes of Lake Thomas Development.

This report documents our observations, findings, and recommendations.

We trust this to be satisfactory at this time. Once you have had an opportunity to review this correspondence, please contact us to address any questions you may have.

Thank you,

[redacted]

Chris Boudreau, P.Eng.
Manager, Civil Engineering
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APPENDICES

1.0 INTRODUCTION

The proposed Townhomes of Lake Thomas Subdivision is located on two existing parcels, both with frontage on Highway No. 2 in Fall River, Nova Scotia. The existing lands consist of two parcels (Civic 3124 on PID 00504415 and civic 3134/3136 on PID 40103202) zoned as Village Main Street (VMS) and are owned by 3293309 Nova Scotia ULC. The project consists of removing an office building from civic 3124 and replacing it with 18 townhomes (six dwellings, each with three units). The existing residential unit located on civic 3134-3136 will remain. Refer to Appendix A, CSK-1 for the Concept Site Plan, completed by Servant, Dunbrack, McKenzie, and MacDonald Ltd. (SDMM). The project lands are within the River-Lakes Secondary Planning Strategy boundary of Halifax Regional Municipality's Municipal Planning Strategy for Planning Districts 14 and 17 (Shubenacadie Lakes), which requires a Phosphorus Net Loading Assessment (PNLA) to be submitted along with the Development Agreement application in accordance with policy RL-22 of the River-Lakes Secondary Planning Strategy defined below:

> *The River-Lakes Secondary Planning Strategy shall establish a no net increase in phosphorus as the performance standard for all large scale developments […] A study prepared by a qualified person shall be required for any proposed development pursuant to these policies to determine if the proposed development will export any greater amount of phosphorus from the subject land area during or after the construction of the proposed development than the amount of phosphorus determined to be leaving the site prior to the development taking place. If the study reveals that the phosphorus levels predicted to be exported from the proposed development exceed the phosphorus levels currently exported from the site, then the proposed development will not be permitted to take place unless there are reductions in density or other methods that reduce phosphorus export levels to those current before the proposed development. […] Any stormwater management devices designed to treat phosphorus must be located on the privately-owned land included in the proposed development agreement.*

It is expected that through the development of this site we will see the increase of total phosphorus (TP) loadings due to fertilizers, soil erosion, stormwater surface runoff, and additional septic flow generation, which are all large contributors to the production of TP. This increase in TP can be mitigated through the use of stormwater treatment best management practices (BMPs) and on-site sewage disposal systems (OSSDS) with dedicated TP treatment.

This document is intended to satisfy the requirements of RL-22 listed above and confirm that the post-development scenario will not export any greater amount of phosphorus from the subject land than the pre-development scenario. Several BMPs and OSSDSs were investigated in the post-development scenario in order to satisfy the policy provision of no net increase in TP values during or after construction.

This report presents the findings of the water quality analysis conducted in April and May 2020.

1.1 Design Criteria

With the removal of the existing office building on civic 3124 and construction of six new dwellings (each with three units), a portion of the land use will shift from existing commercial with forest and grass covered areas to developed medium density residential with asphalt driveways. This change in land use will require specific stormwater management features to adequately maintain pre-development TP levels. In addition to stormwater management features, special consideration will be required when designing the OSSDSs such that the percentage of TP released through these systems is minimized. This water quality study was completed with a focus on low impact development (LID) BMPs designed to balance pre and post development TP, as well as utilizing advanced OSSDS technology to reduce the TP discharged from the OSSDSs. Pre-post TP balancing was completed per the guidelines put forth within Halifax's Municipal Planning Strategy for Planning Districts 14/17 (Shubenacadie Lakes) and the Halifax Regional Municipality Stormwater Management Guidelines published by Dillon Consulting in March 2006.

2.0 SCOPE AND METHODOLOGY

2.1 Scope

The purpose of this water quality study is to analyze the proposed Townhomes of Lake Thomas' pre-development TP loadings, estimate uncontrolled post-development TP loadings, and propose stormwater BMPs and OSSDS design features to provide a balanced site (i.e. pre/post TP export balancing). Stormwater peak-flow management design is outside the scope of this report and is to be covered by others.

2.2 Methodology

The methodology undertaken for this analysis consisted of four primary elements listed below. More detailed information on each is contained in Section 3.0.

2.2.1 Historical Data Review

Historical records relating to the site and its surrounding climatic data were reviewed as part of this study. The primary sources of information included aerial photographs, the development agreement application compiled by KWR Approvals Inc., registered survey plans, and Environment Canada's 1981-2010 Canadian Climate Normals for Halifax Stanfield International Airport, NS (8202250).

2.2.2 Hydrological Model

The project site was modeled as a single watershed, using Nova Scotia 1:10,000 topographical mapping. It was assumed that areas within the delineated watershed that were not to be altered throughout the development process would be ignored while modeling water quality (i.e. a large portion of the existing residential lot will remain unaltered including undisturbed areas on the property's east side). This assumption meant only the developed portion of the site would be considered throughout the analysis. Existing and developed surface characteristics were classified and are discussed further in Section 3.1.3.

2.2.3 Water Quality Analysis

Through the use of desktop modeling processes and empirical data presented in the HRM Stormwater Management Guidelines a simulation of TP production for the proposed development was completed in both the pre-development and post-development conditions. Considerations for accurate calculation included:

- Accurately identifying ground surface characteristics
- Assigning TP pollutant washoff values
- Removal rates for a range of different stormwater BMPs

2.2.4 On-Site Sewage Disposal System Analysis

Pre-development conditions were analyzed to understand the current TP output of the project site from the existing OSSDSs. This formed a baseline to work with when considering particular OSSDS TP treatment units and what values needed to be attained in the post-development scenario.

2.3 Existing Conditions

The surrounding lands in the area are largely residential and commercial, with homes having on-site sewage and well water. Highway No. 2 in this area appears to have curb and gutter street construction, with municipal stormwater collection. The closest lake (Lake Thomas) is almost completely surrounded by un-serviced development and appears to have existing OSSDSs within approximately 50-70 m of the edge of water. These existing OSSDSs in such close proximity to the surrounding lake systems have been negatively impacting the lakes nutrient levels. In a 1993 study by Vaughan Engineering Associates Limited titled Shubenacadie Lakes Planning/Pollution Control Study it is noted that the Shubenacadie Headwater lakes of Charles, William, Thomas, Fletcher's, and Grand are already under significant development related stress, which has contributed to existing excess phosphorus loading.

Within the study area there are overhead power lines, a utility pole, existing wells, and existing OSSDSs. Provided topographical information did not indicate the location of existing OSSDSs but it has been assumed there are two existing systems, one for the existing office, and one for the existing residential dwelling. It has also been assumed that the OSSDS for the existing residential building will remain in place and the OSSDS for the existing office building will be removed to facilitate the construction of the new residential units.

3.0 MODEL CONFIGURATION

The project site consists of two existing driveways, a residential dwelling, an office, garage, two sheds, a portion of tree cover, and grass landscaping. Due to the absence of a completed stormwater design at this time it has been assumed that in the fully developed condition, there will be a single stormwater outlet proposed for the project area that directs water towards Lake Thomas either via an existing culvert under Highway No. 2. A model was created that simulated a full year of precipitation and calculated the anticipated TP, in kilograms, transported from the site through stormwater runoff. Additionally, it has been assumed the all 18 of the proposed units will share a single OSSDS.

3.1 Hydrology

3.1.1 Rainfall

Average annual precipitation data was collected from Environment Canada's 1981-2010 Canadian Climate Normals for Halifax Stanfield International Airport, NS (8202250). To represent the winter months adequately, both average annual rainfall and average annual snowfall were used as contributors to the production of TP throughout a full year. Table 3.1 below outlines the precipitation values used during the analysis.

Table 3.1: 1981-2010 Canadian Climate Normals, Halifax Stanfield Int'l Airport

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Rainfall (mm)	83.5	65.0	86.9	98.2	109.8	96.2	95.5	93.5	102.0	124.6	139.1	101.8	1196.1
Snowfall (cm)	58.5	45.4	37.1	15.9	2.0	0.0	0.0	0.0	0.0	0.4	16.6	45.4	221.2
Precipitation (mm)	134.3	105.8	120.1	114.5	111.9	96.2	95.5	93.5	102.0	124.9	154.2	143.3	1396.2

Due to the relatively small catchment area on the site, we do not anticipate significant localized evaporation to occur and therefore evapotranspiration was not considered during the analysis.

3.1.2 Catchment Delineation

Catchment delineation was completed using the Nova Scotia 1:10,000 topographical mapping, and SDMM concept site plan in AutoCAD Civil3D. As discussed in Section 2.2.2, the water quality model catchment consists only of areas that will experience a change in land-use, surface type, or construction. This means that areas within the catchment area but outside of the proposed development will not be considered in TP calculations as the surface cover and use will not change throughout the life of the development. The overall stormwater catchment area was calculated to be ±42,470 m^2 and the development area was calculated to be ±8,500 m^2. Refer to Drawing 1 in Appendix A for the area considered as development area.

3.1.3 Land Use and Surface Cover

The following land use scenarios were used during analysis:

- Scenario 1: Pre-development conditions
- Scenario 2: Post-development conditions, no BMPs (uncontrolled)
- Scenario 3: Post-development conditions, with BMPs

As discussed in Section 3.1.2, only the areas within the delineated watershed that will be altered during the development's construction process have been considered in the water quality model.

The existing properties currently contain a residential dwelling, an office, a garage, and two sheds. The existing office, garage, and sheds will be removed to make way for the proposed 18 unit development. The two parcels have a total land area of ±2.44 hectares, which consists of trees, grass, building structures, and gravel/asphalt driveways.

Pre and post-development land use and corresponding phosphorus loading concentrations were assigned using the information presented in Table 5-5 of the HRM Stormwater Management Guidelines, see Appendix B for portions of the HRM document. Pre-development conditions were estimated using a combination of aerial photography as well data provided in the SDMM Concept Site Plan. Table 3.2 below summarizes the land uses and corresponding phosphorus loading values utilized throughout the modelling process.

Table 3.2: Summary of Pre and Post-Development Land Uses

Development Condition	Land Use	Area (ha)	TP (mg/L)	Notes
Pre-Development	Commercial	0.85	0.2	*Buildings, Gravel and asphalt driveways/ parking areas, trees, and grass*
Post-Development	Medium-Density Residential	0.85	0.2	*Townhome units, asphalt driveways, and grass landscaping*

Refer to Drawing 2 in Appendix A for a breakdown of the post-development land uses included in the water quality model.

Based on topographical field data collected by SDMM the natural terrain generally rises from west to east, with slopes ranging between 1 and 15%. The site also contains existing ditching on the east side of the property as well as a wet area that will remain undisturbed throughout the project.

3.1.4 Geology and Groundwater

Using the Nova Scotia Groundwater Atlas and reviewing the well log from civic 3134/3136, it is approximated that bedrock is situated at an average depth of 11.57 m below surface. The well logs for the project properties did not contain any information on groundwater elevations, but reviewing surrounding well logs indicates that groundwater is approximately 3.8 m below surface. The low groundwater table will help to encourage infiltration, groundwater recharge, and be more conducive to the LID approach and associated use of stormwater BMPs.

Published Nova Scotia surficial geology data indicates ground morraine and streamlined drift on a stony till plain for the project site. Additionally, a review of the geological map of Nova Scotia indicates the bedrock in the area is Goldenville Formation of the Meguma Group.

3.1.5 Runoff Coefficients

Runoff coefficients were used in determining the annual volume of rainfall that runs off of the site. These runoff coefficients are commonly used in rational stormwater models and are also known as rational C values. The runoff coefficient is essentially a ratio of runoff to rainfall and varies based on land use, soil type, and land slope. Runoff coefficients are a value between 0 and 1 that can be taken directly from published tables or used aggregately as a weighted value to represent an area which incorporates multiple land uses. The closer the value is to 1, the more runoff is expected to occur, so for an area covered in asphalt, which would see large quantities of runoff and little infiltration, a runoff coefficient of 0.7-0.95 would be expected.

Table 3.3 below summarizes the runoff coefficients used for each land use outlined in Section 3.1.3.

Table 3.3: Site Runoff Coefficients

Development Condition	Land Use	Runoff Coefficient
Pre-Development	Commercial	0.32*
Post-Development	Medium-Density Residential	0.45*

*Weighted runoff coefficient based on multiple land uses within the catchment

3.2 Water Quality Models

Two separate water quality models were completed; one for TP contributions from stormwater and one for TP contributions from OSSDSs. These two models estimated the proposed development's annual generation of TP in kilograms.

TP loading from stormwater runoff is dependant on the land use of a particular area. Land use and corresponding TP concentrations are outlined in Section 3.1.3 and were selected from the HRM Stormwater Management Guidelines. Additionally, sewage generation numbers from the proposed dwellings were estimated to determine TP loading from OSSDSs and are discussed in Section 3.2.2.

Using the provided TP concentrations, an annual mass of phosphorus in kilograms was calculated using the estimated annual rainfall for the area. Additionally, the existing on-site OSSDS TP generation values were estimated. These TP values were investigated and compared in pre and post-development separately as the stormwater component is transported over ground and the OSSDS portion is transported below ground not to be combined until they reach their ultimate discharge points.

<u>3.2.1 Stormwater Phosphorus Treatment - Best Management Practices (BMPs)</u>

BMPs are devices or features included in a stormwater system with the goal of improving water quality. Typically, BMPs are introduced in areas that experience a change in land use and have an increased percentage of impervious area, causing more direct runoff and pollutant transfer to occur. The performance of various BMPs has been monitored in studies across North America and published values for removal efficiency are widely available. Removal efficiency values quantify the BMPs ability to remove pollutants, one of which being TP. BMP removal efficiencies used during analysis were retrieved from the following source:

- Halifax Regional Municipality Stormwater Management Guidelines prepared by Dillon Consulting in March 2006

Refer to Appendix B, for portions of the report stated above.

Table 3.4 below outlines some examples of BMPs and their TP removal efficiencies that are often introduced to a development. The values presented below have been compiled from the HRM resource listed above.

Table 3.4: BMPs and Related TP Removal Efficiency Ranges

Best Management Practice (BMP)	HRM TP Removal Efficiency (%)
Wet Pond	50
Grass Swale	40
Permeable Pavement	80
Constructed Stormwater Wetland	50
Sand Filter	60
Infiltration Trench	70

The BMPs listed above can be incorporated into the design topography of most developments but some need special consideration for placement due to size requirements (i.e. a wet pond may require a minimum plan area for effective removal).

BMPs can act as stand-alone features that work to remove a defined percentage of waterborne pollutants but they can also be arranged in-line in a series configuration, known as a train, to increase the overall removal efficiency.

Equation 3-1 below is used to determine the removal efficiency of BMPs in series:

BMPs in Series

$$R = A + B - \frac{AB}{100} \qquad \textbf{Equation 3-1}$$

Where,

R = Total aggregate removal rate
A = Removal rate of the upstream BMP (%)
B = Removal rate of the downstream BMP (%)

<u>3.2.2 Sewage Phosphorus Treatment - On-site Sewage Disposal System (OSSDS)</u>
Through the use of desktop modeling processes an OSSDS TP loading and removal model for the proposed development was completed. Considerations for our calculations included:

- Assigning TP loading values from the existing and proposed residential
- Nutrient removal rates for a biological wastewater treatment unit

For this proposed development, it has been assumed that a single OSSDS that serves all 18 units will be designed and installed by others in accordance with Nova Scotia Environment (NSE) regulations. Conventional OSSDSs (i.e. septic tank followed by disposal field of sand) were initially analyzed as an option for wastewater treatment for this development, however, without additional phosphorus removal technology, the removal of phosphorus was inadequate due to increase of expected sewage production from the proposed additional townhome units. As a result, the use of just a conventional OSSDS alone is not adequate, and phosphorous targeted treatment was explored.

For this development, the shared OSSDS will consist of a standard OSSDS with the addition of a Waterloo EC-P unit. The EC-P unit is installed within the septic tank and contains iron electrodes which have a small current applied to them. The iron is then dissolved in the sewage stream where it reacts with phosphorus to form highly stable iron phosphate minerals. The effluent from the septic tank is dispersed evenly over a bed of sand where the iron-phosphate minerals precipitate out, preventing the phosphorus from entering the natural environment. Waterloo EC-P testing results indicate that when coupled with a 0.9 m thick sand filter (mixture of 60% C33 sand and 40% silty loam, with the percolation rate estimated at about 10 min/cm) they have the capability of removing approximately 99.5% of TP that is generated in domestic wastewater based on data collected over a three year period. The detailed design for the OSSDS shall meet all NSE regulations and guidelines and will be completed by a Qualified Person. Testing data and results completed by the Massachusetts Alternative Septic System Test Center (MASSTC) for the Waterloo EC-P unit can be found in Appendix C.

The anticipated daily TP generated from both the existing office building and proposed townhomes was calculated using Equation 3-2. This value represents what they both produce in TP prior to entering the OSSDSs. It was assumed that the existing office building does not have any dedicated TP removal processes in place so the TP generation numbers calculated are equivalent to TP that is discharged to the environment.

$$OSSDS\ P\ Generation = \frac{C_{SEPTIC} * Q_{SEPTIC}}{10^6}$$ **Equation 3-2**

Where,
OSSDS P Generation = phosphorus load generated as influent, prior to any form of treatment (kg)
C_{SEPTIC} = concentration of phosphorus in wastewater influent (mg/L)
Q_{SEPTIC} = daily flow rate of wastewater influent (L/day)
10^6 = mg to kg conversion

The values for C_{SEPTIC} and Q_{SEPTIC} were approximated using industry standards as well as NSE guidelines and regulations.

Appendix F of the NSE On-Site Sewage Disposal Systems: Technical Guidelines document outlines the recommended sewage generation flows to be used in design. The following was used for this development:

- 1,000 L/day for the existing commercial building (assuming 20 employees)
- 1,500 L/day for each 4-bedroom townhome unit

The 4-bedroom townhome generation values were applied to each townhome unit to calculate the Q_{SEPTIC} for each all of the anticipated townhomes. Table 3.5 presents the input parameters provided by NSE, which were used in Equation 3-2 to determine the OSSDS P Generation loads in the pre-development scenario as well as prior to entering the OSSDS in the post-development scenario.

Table 3.5: OSSDS P Generation Input Parameters

Structure	C_{SEPTIC} (mg/L)	Q_{SEPTIC} (L/day)
Existing Commercial Building	14.4	1,000
Proposed 18 Townhome Units		27,000

For the next computational component of the model, the TP being discharged from the septic tank with Waterloo EC-P treatment unit in the post-development scenario is calculated using the following equation:

$$OSSDS\ P\ Load = \frac{(1 - RED_{OSSDS,P}) * C_{SEPTIC} * Q_{SEPTIC}}{10^6} \quad \textbf{Equation 3-3}$$

Where,
OSSDS P Load = remaining phosphorus load after EC-P treatment unit (kg)
$RED_{OSSDS,P}$ = removal rate of phosphorus expected by the OSSDS
C_{SEPTIC} = concentration of phosphorus in wastewater influent (mg/L)
Q_{SEPTIC} = daily flow rate of wastewater influent (L/day)
10^6 = mg to kg conversion

$RED_{OSSDS,\ P}$ is assumed to be constant for the design life of the EC-P treatment unit (99.5%) as long as the unit is maintained and serviced under a service contract and the filter media in the disposal field has adequate sorption capacity.

Typically, the next component of the OSSDS TP loading model would be calculated as the amount of TP removal that is completed by the surrounding soils into which the OSSDS effluent is discharged. It has been decided that for this particular project that portion of the analysis will not be investigated as specific site soil parameters are not known or accessible at this time. Instead, the biological wastewater treatment unit has been selected such that its TP removal rate is high enough that it can produce pre-development target numbers without having to rely on the surrounding soils ability to provide TP sorption.

3.5 Construction Period

Construction should proceed with care to ensure that the prescribed erosion and sediment control measures are adhered to and enforced properly. Limits of disturbance should be clearly marked on site in an effort to prevent disturbance beyond the intended impact area. During construction of this development, the Site Engineer should monitor how and where material stockpiles are stored. If topsoil and grubbings are stored on site during construction, there is potential that increased phosphorus and sediment concentrations could be generated in surface water that contacts those materials.

To mitigate this potential concern, topsoil and grubbings piles on the site should be covered with tarps prior to rainfall events to limit exposure to precipitation and surface water. In order to deal with exposed soils that cannot be easily covered or removed from the site, other erosion and sedimentation controls (e.g. sediment fence) should be installed and maintained on the site during construction. This will limit the transport and loss of sediment from topsoil or grubbings that may contain elevated phosphorus concentrations.

Short-term erosion sediment control measures are designed to help minimize the amount of surface water that flows across the construction site and limit the exposure time to free sediment. Short-term measures that are proposed for this site should include silt fencing, grass swales with temporary check dams, and strawbale berms around catchbasins. These short-term measures are to be removed or cleaned once suitable vegetation is established near project completion. Long-term erosion sediment control measures to be used on the site include permanent check dams, placed within the grass swales, and vegetated filter strips.

The locations of all proposed BMPs should be clearly marked on the site to avoid any unnecessary disturbance during construction. No vehicular traffic will be allowed within the BMP areas aside from those required to complete the construction of the BMPs. Final grading and final planting will not occur until the adjacent areas draining into the BMPs are stabilized. Construction runoff will be directed away from any BMPs by means of spill-off ditches that are designed to dissipate channel flow to sheet flow overland into established vegetated areas for sediment transport reduction. If BMPs are used during construction, temporary check dams will be added to limit downstream transport. When sediment gathers within the BMPs during construction, it is important that they be regraded and revegetated after construction has been completed to establish the design cross section and ensure proposed nutrient removal characteristics. Where possible, final vegetation planting, with native planting, will be completed in the spring when vegetation can become established with minimal irrigation.

Other than topsoil and grubbings, the main sources of increased phosphorus loading during, and in the period shortly after, construction are through the introduction of fertilizers, biosolids, or other concentrated organics, and industrial wastes. The contractors constructing this project should not be permitted to utilize these items. With proper care and inspection by the Site Engineer during construction of the above noted erosion and sediment control measures, including modifications based on site constraints, it is expected that no net increase of phosphorus will occur during construction. Since no increase in phosphorus is anticipated during the construction phase, it was not included in site modeling.

4.0 MODEL RESULTS

Both the stormwater and OSSDS water quality models were initially run in the pre-development scenario to determine the base-line values. Then, models were created that ran uncontrolled in the post-development scenario and did not include any pollutant loading attenuation features (i.e. stormwater BMPs or OSSDS phosphorus treatment). This provided an understanding of how the expected pollutant loading would be affected by a developed site and quantified how much excess TP is being generated that needs to be treated to reach a no-net increase of TP. The pre and post-development (uncontrolled) TP values for stormwater and OSSDS outputs are presented below in Tables 4.1 and 4.2 respectively.

Table 4.1: Stormwater Annual TP Loadings for Pre and Post-Development (Uncontrolled)

Development Scenario	Annual TP Loading (kg)
Pre-Development	0.76
Post-Development (Uncontrolled)	1.06

Table 4.2: OSSDS TP Annual Loadings for Pre and Post-Development (Uncontrolled)

Development Scenario	Annual TP Loading (kg)
Pre-Development	5.26
Post-Development (Uncontrolled)	141.91

Based on the values stated above it was determined that stormwater BMPs and additional OSSDS phosphorus treatment are required in order to achieve a balanced site for TP. Comparing the pre-development and the uncontrolled post-development values illustrates that the sites require the implementation of stormwater measures with a 28.3% removal efficiency of TP and OSSDS treatment with a removal efficiency of 96.3% in order to achieve Halifax's River-Lakes Secondary Planning Strategy requirement of no net increase in phosphorus during or after construction. To satisfy these removal efficiencies, several BMPs and OSSDS treatment systems were investigated to help produce a post-development site that would meet this requirement.

Several iterations of the stormwater water quality model were run in the controlled post-development condition to find the best pollutant loading attenuation methods. Table 3.4 in Section 3.2.1 summarizes the BMPs investigated to create a balanced post-development site.

Section RL-22 of Halifax's Municipal Planning Strategy for Planning Districts 14/17 states that "*Any stormwater management devices designed to treat phosphorus must be located on the privately-owned land included in the proposed development agreement*", therefore all BMPs must be contained within the project's property boundary. Because of the space constraints of the site it was determined that BMPs such as wet ponds and stormwater wetlands were not feasible. Also due to space constraints, BMP treatment trains were also not feasible. An efficient combination of BMPs to achieve the necessary minimum 28.3% stormwater TP removal rate was determined to be three separate grass swales strategically placed on the site to receive stormwater runoff from the townhomes roofs as well as the parking are driveway areas. Table 4.3 below outlines some special considerations required when selecting a grass swale as a site BMP.

Table 4.3: BMP Design Requirements and Considerations

Best Management Practice (BMP)	Design Considerations *
Grass Swale	• Contributing drainage <2 ha • Maximum 2.5:1 interior side slopes • Minimum depth of 750 mm • Minimum bottom width of 750 mm • Use of natural and native vegetation • Effective for stormwater treatment if length is at least 60 m • Requires permanent check dams at 60 m spacing • Longitudinal sloping should range between 0.5-5% • Requires regular inspection and maintenance of vegetation

*Based on data provided in HRM Stormwater Management Guidelines – 2006

Industry standard for BMP design suggests that for enhanced grass swales to achieve the optimal published TP removal efficiency the swale should be 60 m long for tributary areas up to 2 ha. Therefore, it was determined that every 60 m of grass swale would act as a single grass swale BMP. Three separate 60 m grass swales were positioned on the site to maximize the amount of stormwater each of them would receive from the developed area. It has been assumed that each of the townhome's roof leaders will be discharged overland at the rear of the buildings so they can be intercepted by the grass swale BMPs. Refer to Drawing 3 in Appendix A for preliminary BMP layout, BMP tributary areas, typical BMP detailing, and anticipated stormwater surface flow directions. Final positioning, grading, and design of the hydraulic connection to the stormwater system outlet is to be completed by the project Civil engineer during detailed design.

As discussed in Section 3.2.2, an OSSDS with an added EC-P treatment unit is proposed to aid in the removal of sewage generated TP. The 18 proposed townhome units will all share an OSSDS disposal bed and EC-P treatment units. Based on published independent testing completed by Massachusetts Alternative Septic System Test Center (MASSTC), the EC-P unit, coupled with a 0.9 m thick sand filter will remove approximately 99.5% of all TP found in the system effluent. OSSDS positioning and detailed design will be completed by the Qualified Person completing the septic design. Refer to Appendix C for detailed information and testing results for the EC-P unit.

4.1 Model Outputs

Pre and post-development stormwater TP loadings with and without the use of BMPs are summarized for the proposed site in Table 4.4, with detailed calculations and model results presented in Appendix D.

Table 4.4: Stormwater Pollutant Loading Summary

Development Scenario	BMPs Used	TP Removal Efficiency (%)	Annual TP Loading (kg)
Pre-Development	N/A	N/A	0.76
Post-Development	Uncontrolled	0	1.06
Post-Development	Grass Swales	32.1	0.72

Pre and post-development OSSDS TP loadings with and without the use of a TP treatment unit is summarized in Table 4.5, with detailed calculations and model results presented in Appendix E.

Table 4.5: OSSDS Pollutant Loading Summary

Development Scenario	TP Treatment Used	TP Removal Efficiency (%)	Annual TP Loading (kg)
Pre-Development	N/A	N/A	5.26
Post-Development	Uncontrolled	0	141.91
Post-Development	OSSDS with EC-P unit	99.5	0.71

4.2 Maintenance

4.2.1 Maintenance of Stormwater BMPs

In order to provide BMPs that maintain their TP removal potential throughout their lifespans it is important that regular maintenance be completed. For natural BMPs such as grass swales, making sure they are free of debris and excess sediment will help them operate at their full potential. Ultimately, maintenance schedules are the responsibility of the owner but it is imperative that regular maintenance be performed to ensure peak operational efficiency of any BMP implemented.

The maintenance for the grass swales requires a low- level attention once mature vegetation is present. It is important to provide routine inspections to confirm dense mature vegetation is maintained and to confirm that no concentrated channels are created that allow surface runoff to bypass the vegetated side slopes intended for treatment. Vehicles should not be driven or parked on grass swales. Also, the grass swales should not be scraped or re-graded and any routine mowing should be completed using the lightest possible equipment to avoid unwanted soil compaction.

Credit Valley Conservation of Ontario, Canada has published literature on typical maintenance and inspection activities for grass swales. Table 4.6 presents their recommendations below.

Table 4.6: Typical Maintenance Activities for Grass Swales

Activity	Schedule
• Inspect for vegetation density (at least 80% coverage), damaged by foot or vehicular traffic, channelization, accumulation of debris, trash and sediment, and structural damage to pre-treatment devices.	After every major storm event (>25 mm), quarterly for the first two years, and twice annually thereafter.
• Regular watering may be required during the first two years while vegetation is becoming established; • Mow grass to maintain height between 75 to 150 mm; • Remove trash and debris from pre-treatment devices, the swale surface and inlet and outlets.	At least twice annually. More frequently if desired for aesthetic reasons.
• Remove accumulated sediment from pre-treatment devices, inlets and outlets; • Replace dead vegetation, remove invasive growth, dethatch, remove thatching and aerate (PDEP, 2006); • Repair eroded or sparsely vegetated areas; • Replace mulch in spring; • Trim trees and shrubs; • Remove accumulated sediment on the swale surface when dry and exceeds 25 mm depth (PDEP, 2006); • If gullies or pools of standing water are observed along the swale, regrading and revegetating may be required.	Annually or as needed

4.2.2 Maintenance of OSSDSs

It is important to have regular maintenance on the EC-P treatment units throughout their usage. This will be achieved through service contracts between the owners and the supplier to mitigate degradation of performance of the system. It should also be noted that these systems are typically designed for life cycles of 25 year. At or prior to the 25-year milestone, it is understood that the systems will require a significant maintenance event to continue adequate treatment of TP.

5.0 CONCLUSIONS AND RECOMMENDATIONS

All aspects of the proposed development and existing site constraints and features have been considered in the management of stormwater quality as well as OSSDS effluent quality.

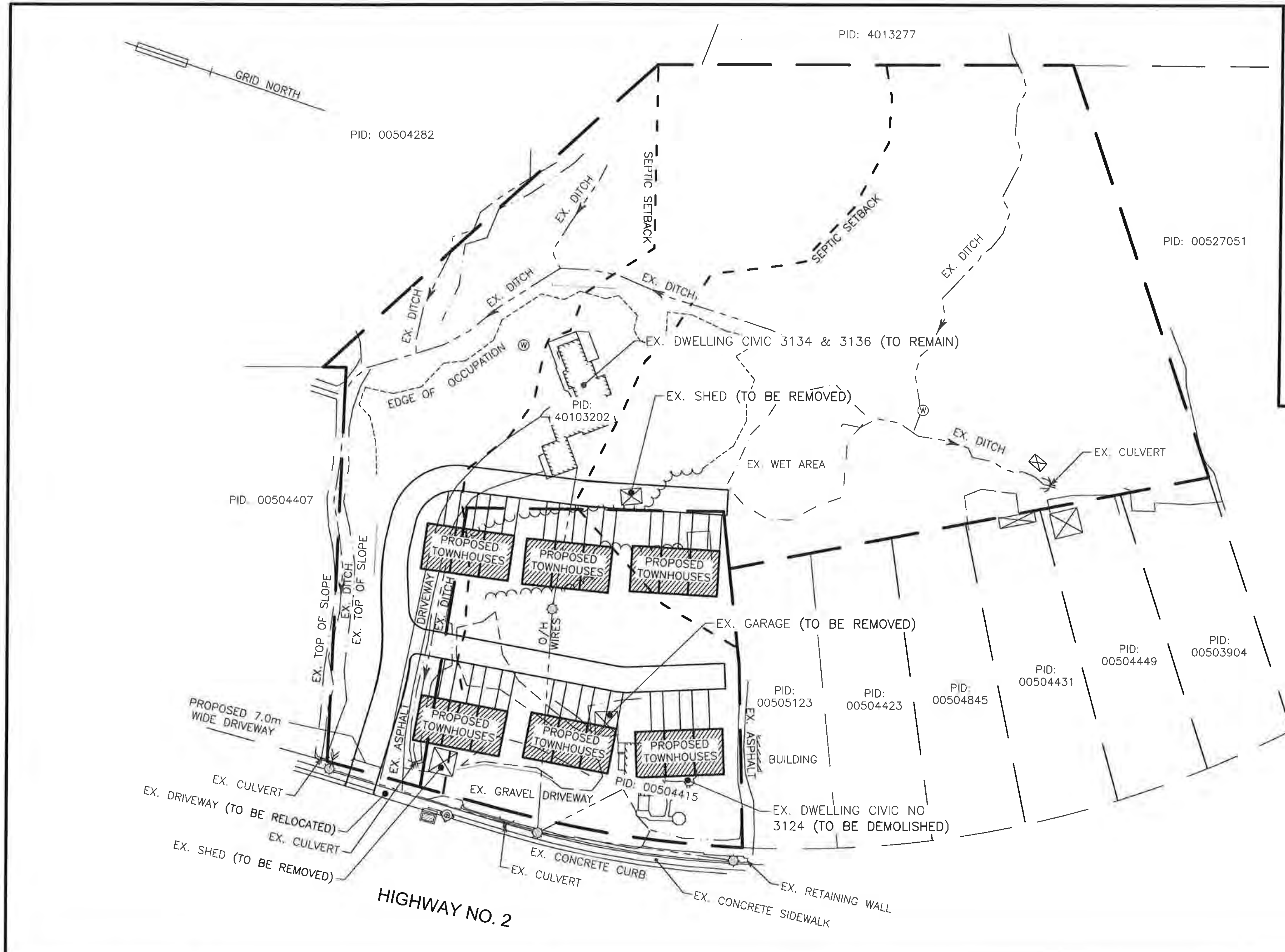
Based on the data presented in this report, it is required that BMPs and OSSDS treatment be introduced into the site design to treat site runoff and nutrients in order to achieve a balanced water quality site as required by the PNLA and RL-22. Using grass swale stormwater BMPS an overall site TP removal efficiency of 32.1% can be achieved, reducing the post-development TP loadings to a value less than that experienced in the pre-development scenario. Refer to Drawing 3 in Appendix A for typical preliminary BMP layout. Final layout of BMPs to be determined by others during detailed site design.

Additionally, the data presented in this report noted that conventional OSSDSs will not adequately remove TP as required by the PNLA and RL-22. To address this, specialized biological wastewater treatment units (Waterloo EC-P units or approved equivalent) are required in order to provide additional TP removal efficiencies. With regular maintenance and monitoring of the EC-P unit and the sewage treatment disposal field, it is anticipated that a TP removal efficiency of 99.5% will be achieved over the design life cycle of the OSSDS. There are also potential additional TP removal avenues that exist in the surrounding area within the soils and from plant uptake that would further reduce the OSSDS TP loads into downstream Thomas Lake. These may include removal through surface water features (i.e. wetlands), evapotranspiration, aquatic plant uptake, etc. The actual amount of TP removal expected from these additional features is difficult to quantify due to the numerous natural variables that exist. Given the margin of error that exists in determining removal efficiency, it is anticipated that these removal rates will only provide further TP removal to continue to help achieve a no-net increase of TP from the OSSDSs to the surrounding surface water features.

A thorough investigation into the development's design, phasing intentions, and finished product has been completed to provide erosion and sediment control measures that will mitigate sediment transport during and after construction. During construction, a Site Engineer should be present to monitor all construction activities and ensure the suggested erosion and sediment control measures are performing adequately.

Stormwater quality balancing and OSSDS TP effluent mitigation have been jointly achieved through the measures outlined in this document. TP loadings have been comprehensively considered and modeled during and after construction to ensure all requirements of section RL-22 of Halifax Regional Municipality's Municipal Planning Strategy for Planning Districts 14 and 17 (Shubenacadie Lakes) are satisfied. If alternate site layouts or OSSDS treatment solutions or products are considered for use the data presented in this report should be re-visited and the data revised to confirm the ability to remove and treat site TP.

APPENDIX A
DRAWINGS



NOTES:

1. THIS PLAN IS METRIC.
2. SITE FEATURES COLLECTED BY SDMM FIELD SURVEY COMPLETED ON JANUARY 21, 2019.
3. PROPERTY LINES ARE APPROXIMATE.
4. SITE & BUILDING STATISTICS
   - # UNITS – 18 TOWNHOUSES + 1 HOUSE
   - TH UNIT DIMENSIONS – 7.0mX11.0m
     HOUSE FOOTPRINT–254 sq.m.
   - CONDO DEVELOPMENT
   - 1 CENTRAL SEWER SYSTEM (PRIVATE)
   - 1 CENTRAL WATER SUPPLY
   - 1 DRIVEWAY ACCESS
   - LAND AREA ± 25,745sq.m.(6.35+/–Ac.)
   - LOT COVERAGE –> ± 6.37% (BUILDINGS)

SDMM
Servant, Dunbrack, McKenzie & MacDonald Ltd.

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BAYERS LAKE BUSINESS PARK
HALIFAX, NOVA SCOTIA
OFFICE: (902) 455–1537
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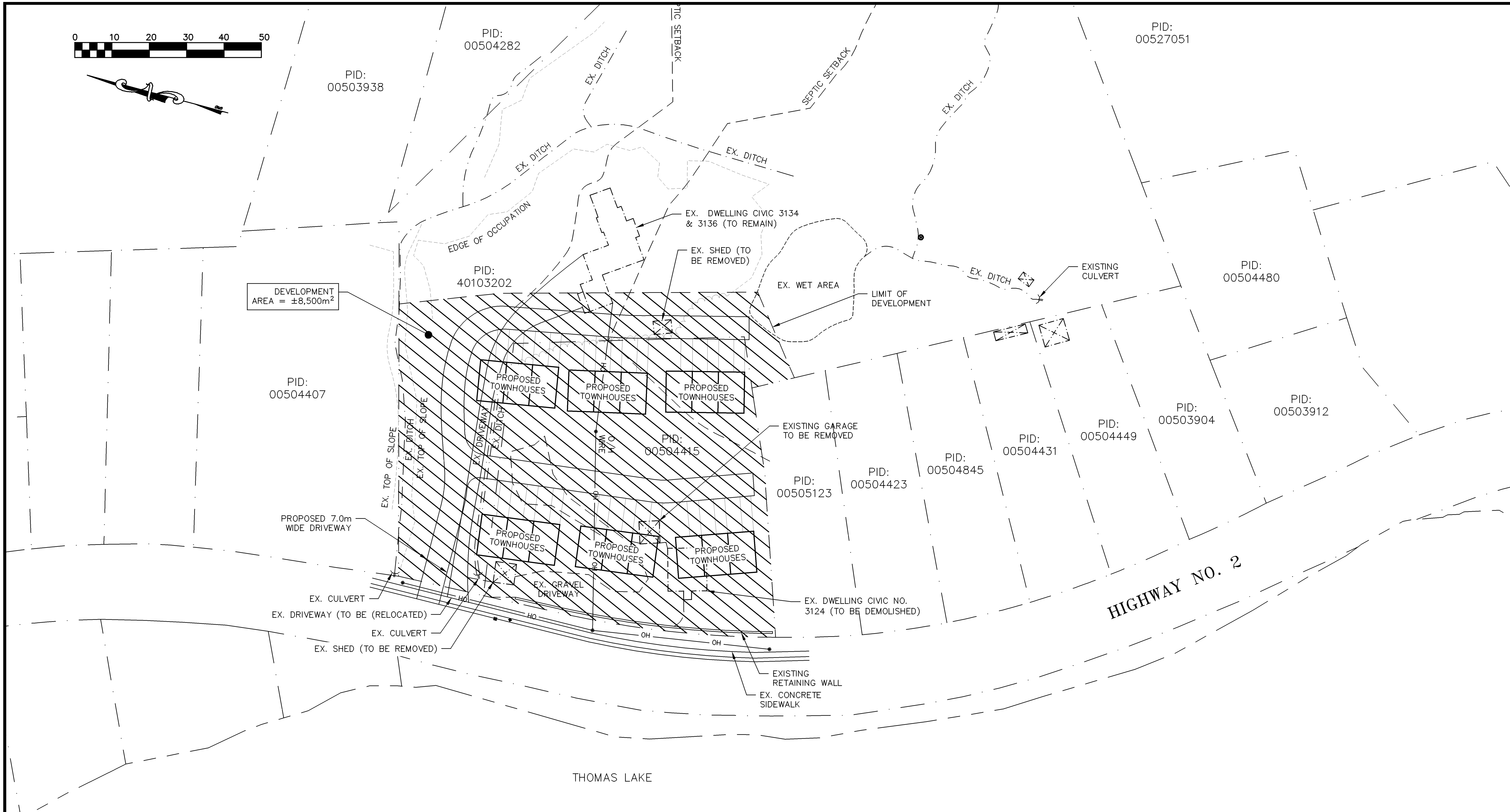
WEB: www.sdmm.ca

Project: TOWNHOUSE DEVELOPMENT 3124, 3134, & 3136 HIGHWAY NO 2, FALL RIVER, NOVA SCOTIA	Date: MARCH 8, 2019	Scale: 1:1,000	Project No.: 1–14–21 (34048)
Title: CONCEPT SITE PLAN	Prepared by: G. Maclean		CSK-1



NOTES:

1. EXISTING AND PROPOSED LINEWORK BASED OFF OF "CONCEPT SITE PLAN, CSK-1" COMPLETED BY SDMM, DATED MARCH 8, 2019.

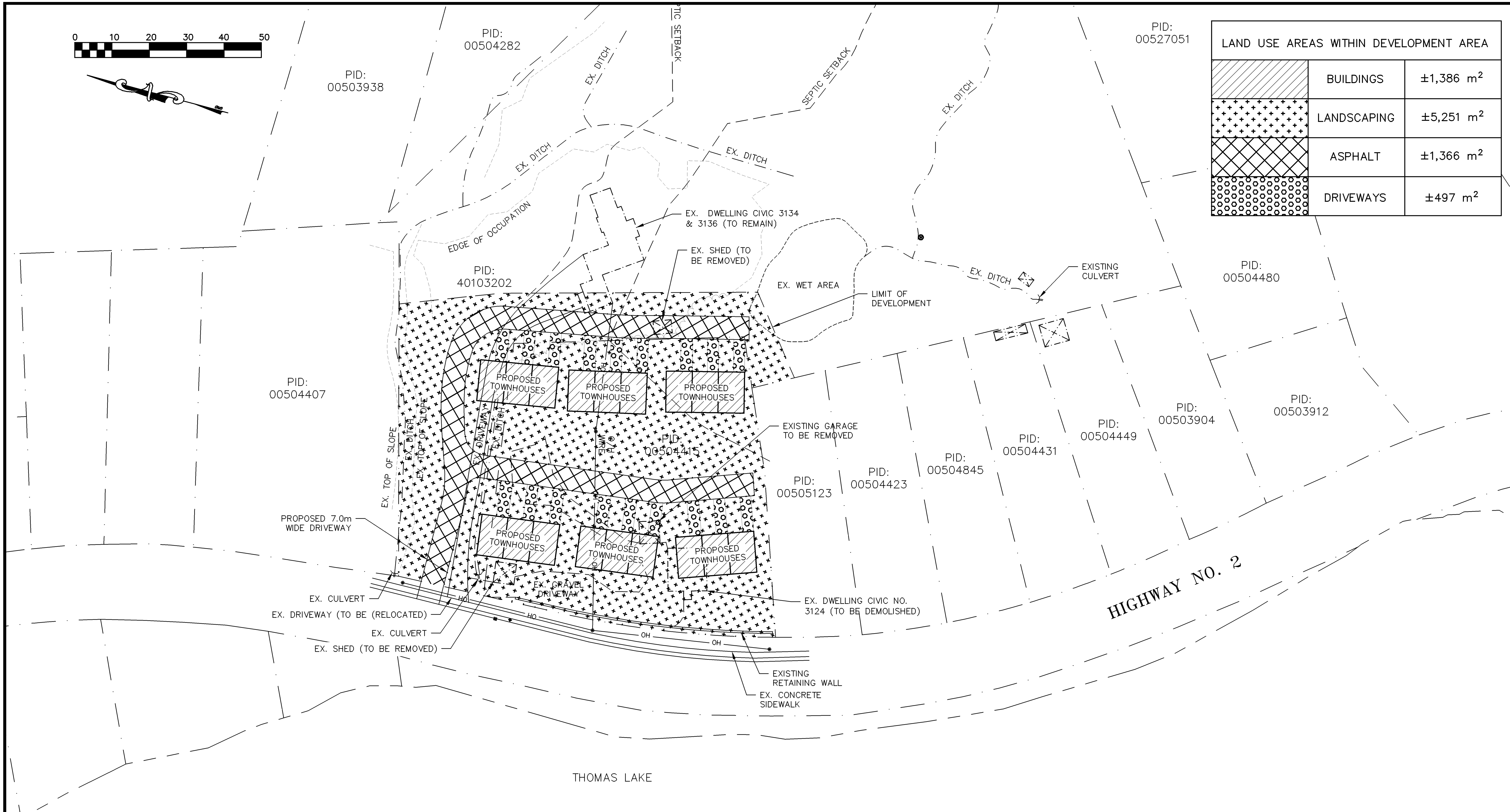
Strum CONSULTING

THE COURTYARD TOWNHOMES OF LAKE THOMAS

DEVELOPMENT AREA

Revision			
No.	Comments	Date	By
0.	Issued for Report	Jun 10/2020	RAW

Date:	June 10, 2020
Scale:	1:1000
Project No.:	20-7270
Drawing No.:	Drawing 1



LAND USE AREAS WITHIN DEVELOPMENT AREA		
	BUILDINGS	±1,386 m^2
	LANDSCAPING	±5,251 m^2
	ASPHALT	±1,366 m^2
	DRIVEWAYS	±497 m^2

NOTES:

1. EXISTING AND PROPOSED LINEWORK BASED OFF OF "CONCEPT SITE PLAN, CSK-1" COMPLETED BY SDMM, DATED MARCH 8, 2019.

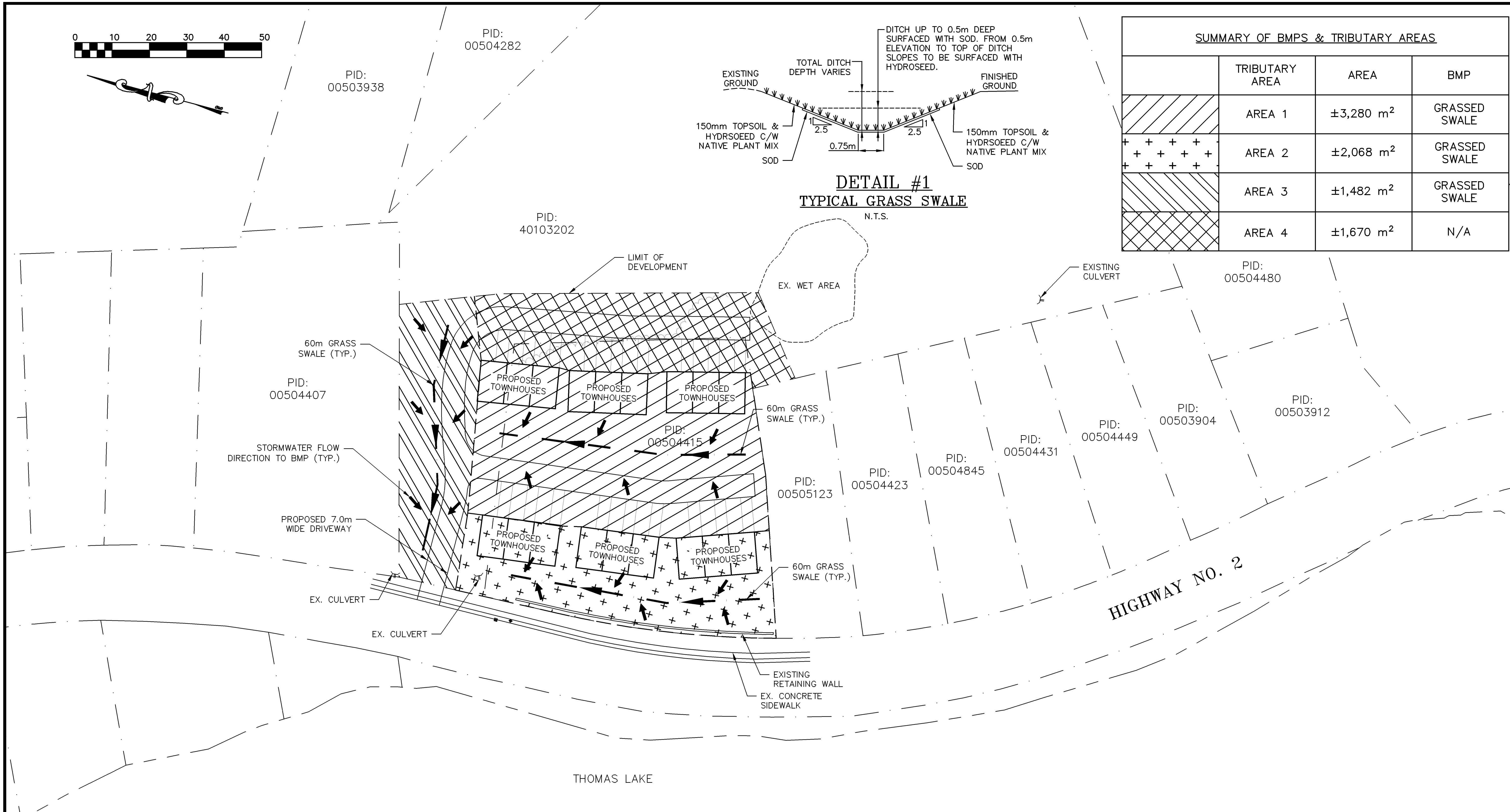
Strum CONSULTING

THE COURTYARD TOWNHOMES OF LAKE THOMAS

LAND USE AREAS

Revision			
No.	Comments	Date	By
0.	Issued for Report	Jun 10/2020	RAW

Date:	June 10, 2020
Scale:	1:1000
Project No.:	20-7270
Drawing No.:	Drawing 2



SUMMARY OF BMPS & TRIBUTARY AREAS

	TRIBUTARY AREA	AREA	BMP
	AREA 1	±3,280 m^2	GRASSED SWALE
	AREA 2	±2,068 m^2	GRASSED SWALE
	AREA 3	±1,482 m^2	GRASSED SWALE
	AREA 4	±1,670 m^2	N/A

NOTES:
1. EXISTING AND PROPOSED LINEWORK BASED OFF OF "CONCEPT SITE PLAN, CSK-1" COMPLETED BY SDMM, DATED MARCH 8, 2019.
2. TOWNHOME DOWNSPOUTS SHALL BE DIRECTED OVERLAND TO CONNECT DISCHARGE TO REAR LOT GRASS SWALE BMPs.

Strum CONSULTING

THE COURTYARD TOWNHOMES OF LAKE THOMAS

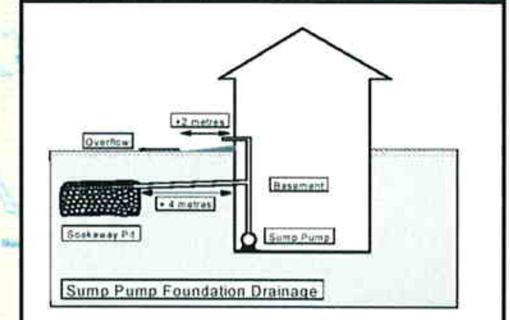
PRELIMINARY BMP LAYOUT

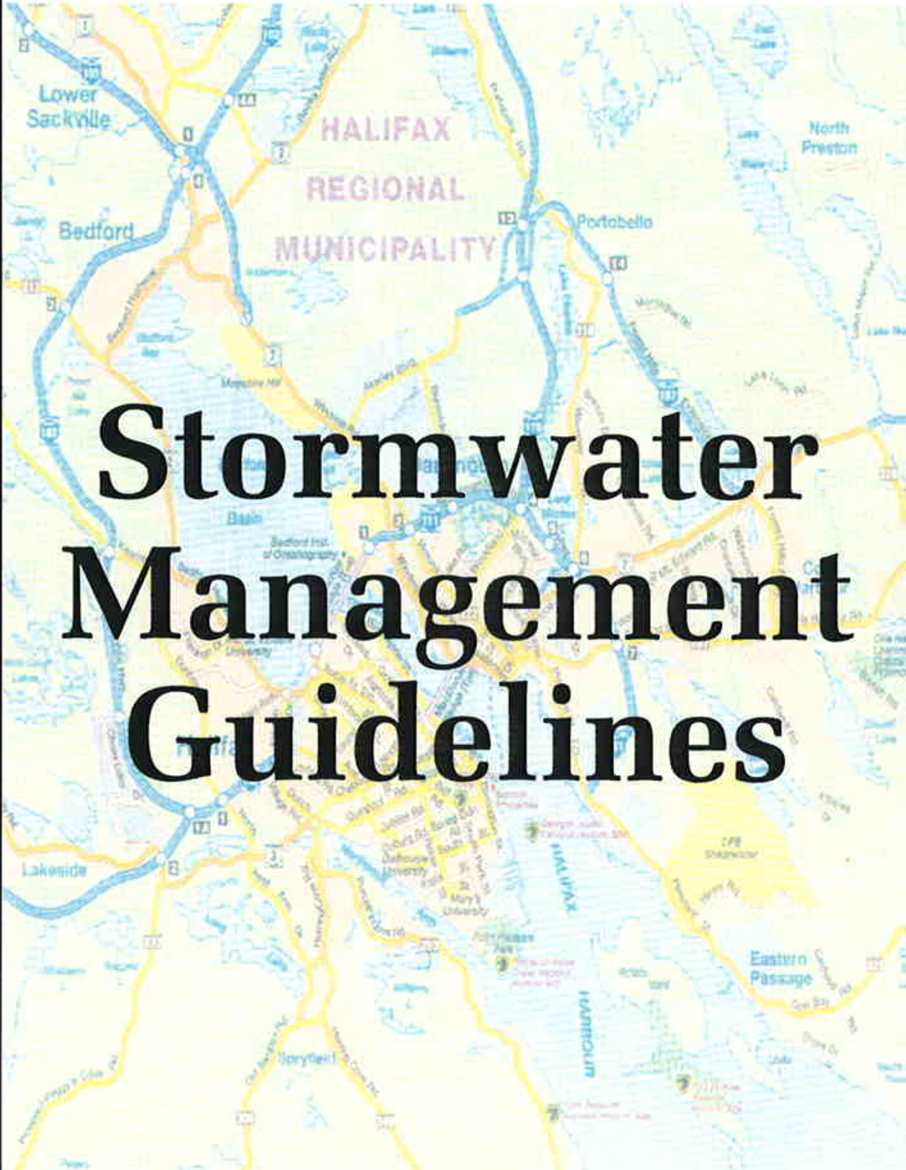
Revision			
No.	Comments	Date	By
0.	Issued for Report	Jun 10/2020	RAW

Date:	June 10, 2020
Scale:	1:1000
Project No.:	20-7270
Drawing No.:	Drawing 3

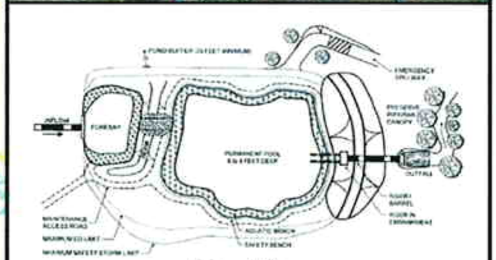
APPENDIX B
PORTIONS OF HALIFAX REGIONAL MUNICIPALITY STORMWATER MANAGEMENT GUIDELINES – MARCH 2006

HALIFAX
REGIONAL MUNICIPALITY





Stormwater Management Guidelines



Plan View

Profile

March 2006

DILLON
CONSULTING

Stormwater Management Guidelines

March 2006

Halifax Regional Municipality

05-4680-0400

Submitted by:

Dillon Consulting Limited

Executive Summary

The purpose of the Stormwater Management Guidelines is to describe a set of criteria for the design of stormwater management practices to protect the environment of the Halifax Regional Municipality from adverse impacts of urban storm water runoff. The Guidelines describe Best Management Practices (BMPs), techniques and methods of managing stormwater drainage for adequate control and pollutant reduction by using the most effective and practical means that are economically acceptable to the community.

The ultimate selection of recommended stormwater BMPs is dependent on the tributary-specific and in some instances, the reach-specific characteristics, sensitivities and functionalities present within the watershed. Ideally, all BMP design criteria should be based on recommendations developed as part of a comprehensive watershed or subwatershed plan prepared for the subject location's basin. These plans are produced through the study of the environmental and land use features of a watershed. The purpose of the plan is to identify those areas that should be protected and preserved as part of the land use planning process, to evaluate the impact of future land use changes and to develop criteria to mitigate potential cumulative impacts in the watershed.

In the absence of watershed/subwatershed study recommendations, the Guidelines provide general design criteria that should be used in HRM for quantity, quality, erosion, and base flow control. The use of this unified approach should result in a design of stormwater management practices that would meet the flood, water quality, erosion control and groundwater recharge criteria adopted until the completion of the watershed and subwatershed studies.

The overall objectives of introducing BMPs are to minimize the adverse effects on and off the development site. An important part of the selection of BMPs is to preserve the sensitive, natural features and to develop a new stormwater system that can reproduce, as closely as possible, the natural conditions of the undeveloped state. This approach stresses the importance of preserving natural storage, infiltration and pollutant filtering functions where feasible, thus reducing the lifecycle cost for stormwater management and minimizing the need for costly capital improvements to the existing system.

There is no single BMP that suits every development, and a single BMP cannot satisfy all stormwater control objectives. Therefore, cost-effective combinations of BMPs may be required that will achieve the objectives.

These Guidelines are intended to be a tool to be used by HRM to guide developers and their designers toward the selection and design of appropriate stormwater management facilities. It will also be used by HRM staff for the review and design of facilities. It is intended that it will be used in combination with the Regional Plan and other planning and design tools already in place to achieve HRM's long-term goals and objectives.

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Acronyms

BMP	Best Management Practice
HRM	Halifax Regional Municipality
MGA	Municipal Government Act
MSS	Municipal Services System
NP	Not practical
NSEL	Nova Scotia Environment and Labour
OP	Operating Procedure
SWM	Stormwater Management
SUDS	Sustainable Urban Drainage Systems
TN	Total Nitrogen
TP	Total Phosphorus
TSS	Total Suspended Solids
US	United States
USEPA	United States Environmental Protection Agency

Ideally, watershed or subwatershed studies should evaluate requirements for post-development water quantity controls based on the potential cumulative impacts of development and potential flood hazards. Where such studies do not exist, requirements for water quantity control should be based on potential downstream flooding hazard. Generally, the criteria are to control post-development peak flows for the 2, 5, 25, 50 and 100–year storms to pre-development levels. If a proposed development is located in the lower reaches of a watershed or subwatershed discharging to coastal waters or large lakes with no downstream developments, quantity control may not be required.

For sizing wet ponds and constructed wetlands, a 24-hour duration event should be selected, as shorter rainfall durations may under-estimate design runoff volumes and associated storage volume requirements. Hydrographs for the individual return period events should be generated by hydrologic models using the Shearwater gauge Intensity-Duration-Frequency data. A more detailed discussion on design storms is presented in ***Appendix E.***

5.3 Design Criteria for Water Quality Control

Maintenance of healthy aquatic ecosystems requires that pre-development water quality be maintained and enhanced where feasible. The goal is to restore, protect and enhance water quality and associated aquatic resources and water supplies of the receiving watercourse. This goal mandates the prevention of contamination of streams and lakes from urban runoff containing nutrients, pathogenic organisms, organic substances, heavy metals and toxic substances.

Similar to the quantity criteria, water quality criteria should be based on the premise that where feasible the post-development water quality should be similar to the pre-development water quality.

The selection of water quality criteria is influenced to a great extent by the receiving system environment. Protection of receiving waters from impacts of sediments generated by urban development construction and post construction periods have been recommended by most provincial and municipal agencies across the North American continent. In Canada the Federal Government prepared guidelines on the potential impacts of sediment on aquatic organisms and their habitat.

In controlling the pollutant efficiency of a BMP, it is recommended that Total Suspended Solids (TSS) be adopted as a primary indicator. As a rule of thumb, when rural land use becomes urbanized, the resulting runoff volume could double. At the same time the TSS loads from urban land uses are twice as high as from rural land uses. Therefore, the combined effect could be a fourfold increase in the TSS loads caused by urbanization. To match the pre-urbanized TSS loading, the selected BMP should reduce the post-development load by approximately 75%. Wet ponds and constructed wetlands are capable of removing 80% of TSS or higher.

The design criteria selection should start by assessing the state of the environment in the downstream receiving water bodies. There are two alternative indicators of the downstream water quality that could be considered in the selection of design criteria: 1) fish habitat, and/or 2) the nutrient concentration in the receiving system.

For the first alternative indicator, consideration should be given to the selection of design criteria based on the potential effects of urban runoff on the aquatic habitats of the receiving system streams and lakes. A simple classification is presented in ***Table 5-1*** to describe the downstream habitat:

Table 5-1
Classification of Downstream Habitat

Category	Fishery	Type of species	Suggested TSS control
I	Cold water fishery	Salmonids, lobster fishery, aquaculture	80%
II	Warm water fishery	Perch, minnows, suckers and urbanized lakes	70%
III	No existing or prospect of future habitat	Habitat in ditches, intermittent streams, stream with blockage	60%

The TSS indicator could also be used to assess receiving system impacts of the health on existing or potential future fish habitat. Impacts on this health can be measured by the relative changes in in-stream fish population or by the severity of impacts due to sediment concentration and duration of exposure.

The following table compares the suspended solids concentration guidelines prepared by the European Inland Fisheries Advisory Commission and the Government of Canada, in the Yukon Placer Authorization 1993, document, based on suspended solids increases.

Table 5-2
Risk to Fish Habitat by Increase in TSS

European Commission		Canada	
TSS – mg/L	**Risk Level**	**TSS – mg/L**	**Risk Level**
<25	Not harmful	<25	Very low risk
25-80	Somewhat diminished yield	25-100	Low risk
80-400	Unlikely to support fisheries	100-200	Moderate risk
>400	Only poor fisheries	200-400	High risk

Researchers on fish and exposure to increases in sediment concentration identified that most species of fish can withstand higher exposure of elevated levels of TSS, but impairment will occur when sediment exposure increases beyond threshold values which are a function of both the sediment concentration and its duration. According to Ward (1992) sediment concentration in the receiving stream below 25 mg/L would result in few ill effects regardless of the duration. For typical runoff events lasting less than 4 hours, moderate impacts would occur at about 200 mg/L. For duration of more than 10 hours, a concentration of 1,000 mg/L could result in major impacts.

Where body contact recreation, aesthetic or other uses require the control of nutrients entering the receiving system, it is recommended that Total Phosphorus (TP) removal be adopted as an alternative or as an additional primary design criterion. The following general relationship exists between TSS and TP removal rates:

TSS %	**TP %**
80	50
70	45
60	35

Based on estimated 50% higher TP concentration and 100% increase in runoff caused by urbanization, there could be an associated 150% increase in the TP loads. To match the pre-urbanized TP loads, the selected BMP should reduce the post-development load by approximately 67%. Wet ponds and constructed wetlands TP removal capability is limited to approximately 45% to 50%. Therefore, where the TP design criteria requires a reduction in excess of that range, additional BMPs would be required to meet the desired level of control. There is extensive background information available on the water quality of local lakes and rivers in the HRM area (http://lakes.chebucto.org), assembled by the Soil and Water Conservation Society of Metro Halifax.

Just as comprehensive watershed studies may include flood control requirements based on cumulative effects of multiple developments, nutrient loading and trophic status modelling may be required to determine TP removal requirements. These studies may even identify linkages between nutrient levels and fish habitat as excessive algae and plant growth can result in the depletion of dissolved oxygen as plant material decomposes.

The water quality criterion for sizing stormwater management facilities has two components: 1) for sizing storage facilities a volume criterion; and 2) for flow-through BMPs a peak flow criterion is recommended. Water quality control BMPs use primarily sedimentation processes to remove pollutants, through settling and/or filtering. Particulate pollutants such as sediment and metals are relatively easy to remove, while soluble pollutants such as nitrates and phosphates are more difficult to remove. A volume generated by a relatively low rainfall and runoff design event generally defines the detention volume requirement for water quality control with a storage facility. Design criteria for BMPs that permit runoff to a flow-through filtration or settling system are related to flow rates and velocities.

When managing runoff for water quality impacts, the control of more frequent and smaller rainfall events are selected. This approach is based on the fact that the percentage of annual precipitation for very large events is relatively small, and the construction cost of storage facilities based on extreme rainfall events would be prohibitive. This approach can still provide partial benefit for larger storms as the BMP can continue to control pollutants from the first portion of the larger storm's runoff.

The water quality volume criteria for sizing BMPs for the HRM area was determined from an analytical model as described in ***Appendix F***. Long-term local rainfall data was analyzed to determine storage requirements for different impervious conditions and TSS removal efficiencies. The total storage volume in a wet pond or in a constructed wetland consisting of a permanent pool and an extended detention should generally be equivalent to the runoff volume generated by 90% of the long-term rainfall events observed in HRM. (For rainfall information see ***Appendix E***)

An example of the relationship between permanent pool storage and TSS removal efficiency as described in ***Appendix F*** is reproduced on ***Figure 5-1***. Increasing the active storage over 40 m^3/ha would only marginally increase the TSS removal.

The peak flow water quality criterion is based on a statistical analysis of local precipitation data. It is recommended that a 25 mm winter rain event should be used to estimate the peak flow generated by the proposed land use.

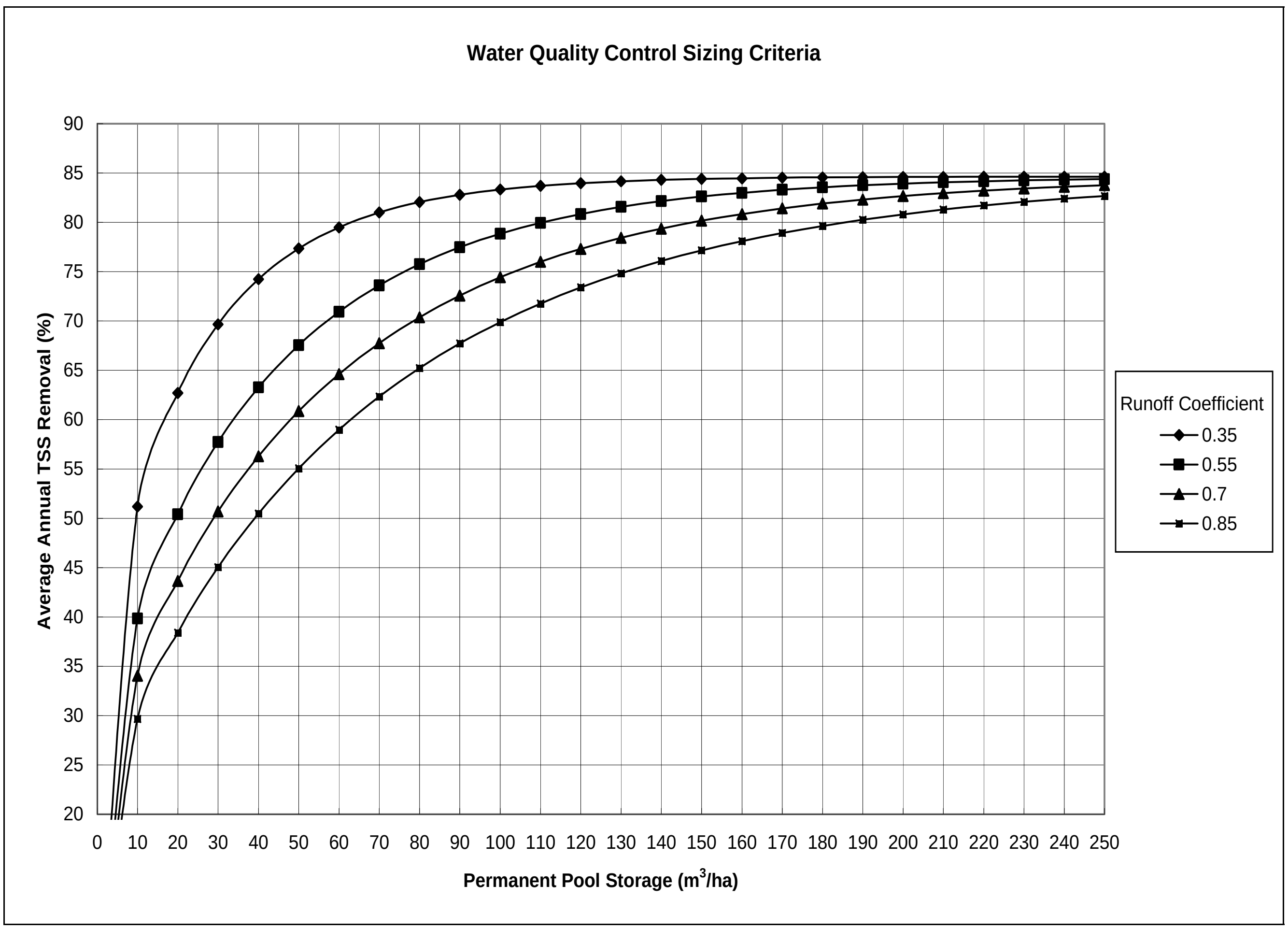

Figure 5-1 **Example of Sizing Permanent Pool Storage for Water Quality Control**

5.4 Design Criteria for Erosion Control

The preferred approach for addressing erosion concerns is at the watershed/subwatershed planning level. During watershed/subwatershed planning, pre and post-development exceedance erosive index values are computed for a watercourse to determine the need for and the magnitude of erosion control measures.

To select the erosion criterion when no such information is available, it is recommended to undertake an analysis of downstream channel conditions to assess the potential effects of post-development flows, water levels, and velocities on erosion. Such an analysis of erosion potential should extend downstream to a point where the runoff from the upstream drainage area controlled by the pond represents only 10% of the total drainage area.

In the absence of information on downstream channel conditions, a 25 mm winter storm is recommended for the erosion control design event. This storm should be based on a 6 hour Chicago distribution event and should be routed through a storage facility assuming a gradual release rate with a drawdown time of 24-48 hours. For sensitive streams, the longer drawdown time should be used. The required storage is then compared to the extended quality control storage, and the greater of the two is used for design.

For BMPs other than wetpond/wetland, the analysis of downstream channel conditions should determine the need for flow control or erosion protection requirements based on velocities and erosive forces generated by a 25 mm winter rain.

5.5 Recharge and Base Flow Maintenance

The need for providing groundwater recharge at a particular site will depend on the use of local aquifers. Where there is a potential risk of adversely affecting groundwater supply (quantity or quality) in the area, or the risk of reduction in base flow, the recharge from a proposed development should attempt to match the pre-development recharge. The pre- and post-development recharge can be estimated by a simple computation of the hydrologic cycle components.

The local average annual precipitation and evaporation components of the hydrological cycle in the HRM area are:

Precipitation	1421 mm
Evapotranspiration	552 mm
Surplus	869 mm (made up of recharge/base flow and surface runoff)

The recharge and base flow components of the surplus can be estimated by an infiltration factor determined by summing the following factors for topography, soils and cover (Ontario Ministry of the Environment, Stormwater Management Planning and Design Manual (2003)):

Topography	**Factor**
Flat Land, average slope <0.6 m/km	0.3
Rolling Land, average slope 2.8 m to 3.8 m/km	0.2
Hilly Land, average slope 28 m to 47 m/km	0.1
Soils	
Tight impervious clay	0.1
Medium combinations of clay and loam	0.2
Open sandy loam	0.3
Cover	
Cultivated Land	0.1
Woodland	0.2

The range of infiltration factor to be applied is 0.3 to 0.8, therefore the minimum recharge and base flow component of the hydrological cycle could be 260 mm (= 0.3 x 869 mm). For post-development conditions when an area is paved and becomes impermeable, the infiltration/base flow and evapotranspiration components are removed from the hydrologic cycle.

Infiltration through BMPs can provide groundwater recharge by diverting runoff from small and moderate storms into an infiltration facility. An additional benefit is achieved by providing opportunities for a number of physical, chemical and biological processes that remove pollutants from the recharge water. A general guideline for recharge and base flow maintenance is to capture where feasible the first 5 mm of rainfall.

A summary of the recommended design criteria for BMPs is listed in ***Table 5-3.***

Table 5-3
Summary of Design Criteria

Control	Criteria	Comments
Flood and water quantity control	Control peak discharges from the 2, 5, 25, 50 and 100-year storms to pre-development rates	• Downstream system analysis may reveal that flood control criterion may not be required. • Should consider the cumulative effects of development and controls.
Water quality	Volume control for storage facilities, or control of peak flow from a 25 mm winter rainfall	• Compute storage from design graphs, or generate hydrographs for the single event design storm
Stream channel erosion	Control of peak flows	• 24 hour-48 hour extended detention of post-development 25 mm winter storm event. • Should consider the cumulative effects of development and controls.

Control	Criteria	Comments
Baseflow	Infiltrating the first 5 mm rainfall	• Where feasible, the pre-development hydrologic cycle components should be maintained.

5.6 Municipal Infrastructure Criteria

A set of storm drainage guidelines was released by HRM in 2005 as part of the Municipal Services Systems Design Guidelines. This municipal document describes the guidelines to be used in the design of municipal storm sewer pipes, ditches and other appurtenances. In particular, the document deals with the design of the major-minor drainage components of urban drainage systems, such as sewers, catch basins, and foundations drains. The stormwater sections of the Guideline document, reproduced in ***Appendix G,*** contains information on:

- Design parameters for the Minor Drainage system;
- Storm sewer system design: pipes, catchbasins, street drainage, ditches, culverts;
- Minor drainage system connections, roof leaders, foundation drains; and
- Erosion and sediment control.

Table 5-4 summarizes the various guidelines listed in the Municipal document. It also details design requirements in addition to those outlined in the Municipal Services System Guidelines.

Table 5-4
Summary of Existing HRM Storm Drainage Design Guidelines

System Component	Guideline	Additional Requirements
Minor System		
Design flow	• Larger of the winter or annual flow. • Where time of concentration >6 hours use winter precipitation and ice/snowmelt. • Where significant portion of area is underdeveloped use annual and winter data. • Piped systems and driveway culverts: minor storm. • Combined capacity of major and minor systems: major storm. • Watercourses, culverts, roadside ditches, in absence of minor system: major system. • Road culverts: 1:10 year storm.	• As recommended in watershed or subwatershed plans. • In absence of such plans the sewer sizing should be based on 1 in 5 year storm without surcharge.
Downstream effects	• Have capacity to convey discharge from fully developed watershed.	
Rainfall data	• Historical data IDF curves for nearby station. • Synthetic storms, Chicago distribution of 2 and 24 hours, r=0.5, discretization 5	• Storm discretization be selected considering basin size. Five minutes is less than the minimum Tc for

System Component	Guideline	Additional Requirements
	minutes and 1 hour for the two storms. • Historical storms used for verification of storage pond performance.	most rational method design – it can lead to very high peaks in small basins.
Runoff computation	• Model must be calibrated and verified. • Rational method for preliminary design for <20 ha, but not for storage.	
Hydraulic design of sewer pipe	• Manning formula, based on published roughness coefficients. • Minimum pipe size is 300 mm diameter. • No decrease in size in the downstream direction, except at intakes.	
Catch basins	• Located in the gutter line, should minimize ice accumulation and ponding. Double catch basins may be required at locations to prevent by-pass of storm flows. • Spacing not to exceed 120 m. • Interception capacity be compatible with the storm drainage capacity. • Where potential for contamination inverted siphons or separators may be required.	• For more details see Appendix G.
Catch basin leads	• Minimum size 200 mm. • Minimum cover 1 m at construction and 1.2 m at completion of construction. • Minimum slope 1%. • Incorporate flexible joint. • Generally, catch basin connection to another catch basin is not permitted.	• For more details see Appendix G
Storm sewer leads	• Connected from the building foundation should be PVC DR35, 150 mm diameter or less.	
Foundation drains	• Normally drained by gravity to storm sewers and located above the hydraulic grade of major storms, or above the major storm flood if connected to a watercourse.	• No connection permitted to sanitary sewers. Basement floor >1m above 100 year hydraulic grade line.
Roof drains	• May be connected to the storm sewer system if capacity available. • Discharge to a dry well normally not permitted. • Under the Lot Grading bylaw, roof drains are not permitted to be connected to the storm sewer except at discretion of HRM.	• Infiltration of roof runoff to be encouraged subject to soil conditions. Roof leaders should discharge to splash pads 4 m away from building.
Institutional, commercial and industrial connections	• Limit flow to 40% of uncontrolled fully developed flow.	
Major System		
Street and overland flow routes	• Minor storms, depth of flow in gutters <50 mm. • Major storms, depth of flows <50 mm at	• For major system use 100 year return storm event.

System Component	Guideline	Additional Requirements
	crown. • No overtopping of curbs and gutter enter driveways, except where a major system is provided. • Open ditches should not be overtopped and enter driveways.	
Ditches and open channels	• Minimum grade 1%. • For rural roads ditch capacity based on major storm. • Depth at bank full conditions <1.2 m, side slopes not steeper than 2H:1V. • Wetted perimeter stabilized above 4% grade. • Maximum velocity at unlined.	
Culverts	• Grade, obverts of outfalls <150 mm above minor storm level, above normal ice level, allowance for accumulation of debris at the outfall. Minimum grade 1%. • Hydraulic capacity to determined by inlet and outlet control computation. • Headwater depth <2 x diameter of pipe. No inundation of buildings. • Grates if structure >30 m long. • Inlet and outlet structure if piped diameter >375 mm extended >600 mm beyond toe of slope. • Minimum diameter for driveway culvert diameter 450 mm, or not smaller than upstream culvert. • Minimum diameter for roads 525 mm. • Culvert materials: reinforced concrete CSA 257.2 and STM C-76 or high-density polyethylene pipe CSA B182.6. ASTM F-667, and have a minimum stiffness of 320 kPa. • Watercourses with drainage area > 40 ha to be maintained as open.	Culvert design capacities: • Urban arterial road, 50-100 year return frequency. • Rural arterial road, 25 – 50 year return frequency. • Local road, 10-25 year return frequency.

5.7 Pollutant Loads

The goal in selecting the best BMP for a site is to minimize the adverse effects of the proposed development on the environment. The aim is to match predevelopment conditions in the receiving system. A list of pollutant loads generated by different land uses based on CH2M Hill is presented in ***Table 5-5*** to assist the designer in estimating pre and post development pollutant

Table 5-5
Mean Pollutant Concentration Generated by Different Land Uses

Land Use	Primary Indicators		Secondary Indicators					Metals						
	TSS (mg/L)	TP (mg/L)	BOD (mg/L)	COD (mg/L)	TKN (mg/L)	TDS (mg/L)	TN (mg/L)	Cd (ug/L)	Cr (ug/L)	Cu (ug/L)	Pb (ug/L)	Ni (ugL)	Zn (ug/L)
Forested wetland	19.0	0.2	4.1	29.4	0.6	52.0	1.1	0.5	2.8	5.3	3.0	4.7	22.9
Cropland and Pasture	19.2	0.2	4.2	29.7	0.6	52.0	1.1	0.5	2.9	5.4	3.1	4.7	23.5
Upland forest	19.7	0.2	4.3	30.4	0.7	52.0	1.1	0.5	2.9	5.6	3.2	4.7	24.8
Urban open	20.0	0.2	4.4	30.7	0.7	52.0	1.1	0.5	2.9	5.7	3.2	4.7	25.4
Communication and utilities	20.7	0.2	4.6	31.7	0.7	52.0	1.2	0.5	3.0	6.0	3.4	4.8	27.5
Low-density Residential	22.1	0.2	5.0	33.4	0.8	52.0	1.2	0.5	3.1	6.5	3.8	4.8	31.2
Medium-density residential	30.5	0.2	7.5	43.5	1.1	52.0	1.7	0.6	3.8	9.7	6.1	5.0	59.4
Institutional	41.9	0.3	11.3	56.7	1.5	52.0	2.4	0.6	4.5	14.7	9.9	5.3	112.9
High-density residential	47.7	0.3	13.3	63.1	1.7	52.0	2.7	0.7	4.9	17.3	12.0	5.4	145.9
Multifamily residential	47.7	0.3	13.3	63.1	1.7	52.0	2.7	0.7	4.9	17.3	12.0	5.4	145.9
Commercial	54.2		15.7	70.1	2.0		3.1	0.7	5.3	20.4	14.5	5.5	188.7
Highways	57.8		17.0	74.0	2.1	1.3	3.3	0.7	5.5	22.1	16.0	5.5	214.6
Industrial	57.8		17.0	74.0	2.1	1.3	3.3	0.7	5.5	22.1	16.0	5.5	214.6

loads for selected parameters. The data represents event mean concentrations monitored across North America. Generally, in the design of stormwater management facilities, only one or two key indicators, such as TSS and TP are considered. Runoff from impervious surfaces has a high potential for introducing pollutants to surface waters. Suspended solids, dissolved nutrients and oil/grease cause the most common water quality concerns. The existing and future pollutant loads could be estimated to provide an indication to the desired level of control. This early estimate will assist in the selection of the most appropriate alternative BMPs.

The portion of the HRM Waste Water Discharge by-law related to stormwater is presented in ***Appendix H***. This by-law describes limits for chemicals discharged to the municipal storm sewer system.

5.8 Exemptions From Runoff Control

Stormwater control would not normally be required for:

- Single lot development of one family dwelling should apply, as a minimum, basic source control measures, such as reduced lot grades and disconnection of roof leaders. Additional stormwater management measures may also be needed subject to local conditions;
- Addition to existing commercial buildings, provided the total impervious area is not increased, and the existing stormwater management facilities are adequate and are not altered; and
- Runoff from a development if it will be controlled by an external regional stormwater facility.

It is recommended that recognition should be given to any non-structural facility when selecting and sizing BMPs for a particular site. For example, appropriate reduction in the design volume or peak flow should be permitted for conservation of natural areas, disconnection of roof runoff if diverted to an infiltration facility, or use of vegetated swales with an infiltration function which will reduce the effective drainage area contributing to the BMP.

APPENDIX C
WATERLOO EC-P TESTING DATA – TESTING OF PHOSPHORUS REMOVAL TECHNOLOGY AT MASSTC

[redacted]

Field Test Results

1. ETI Recirculating Sand Filter (RSF)

General Configuration

The RSF system was pre-existing and consists of a 1500-gallon (5670 L) single-compartment concrete septic tank trickle feeding into a 1000-gallon (3800 L) recirculation tank where it mixes with sand filter effluent. Effluent from the sand filter is directed back to the recirculation tank over a floating ball valve where an estimated 80% of the effluent is mixed with septic tank effluent. The EC-P unit is installed under one of the risers in the recirculation tank.

Hydraulic Loading Rate

The ETI RSF received a continuous sewage flow of 330 gpd (1250 L/day) to conform to the design flow of a three-bedroom residence in Massachusetts. Sewage dosing conformed to Standard NSF-40 rates of 35% between 6:00 a.m. and 9:00 a.m., 25% between 11:00 a.m. and 2:00 p.m., and 40% between 5:00 p.m. and 8:00 p.m.

EC-P Installation & Operation

The Waterloo EC-P and control unit were provided by the manufacturer and installed under a riser of the recirculation tank on August 09 2011. The EC-P unit for Study 1 consisted of an electrode assembly of four (4) steel plates each 4" x 24" (10 cm x 60 cm) with the interior plates being 1/2" thick and the outer being 1/4" thick. The plates are wired to the control unit on surface and supplied. This electrode was changed to an array of 4 plates each 6" x 15" (15 cm x 38 cm) with the interior plates being 3/4" thick and the outer being 3/8" thick for Study 2, which had malfunctioning electrical wiring, and the 4" x 24" units were re-installed for Study 3.

Energy Consumption

In Study 1, without the manufacturer's optional flushing pump and not including the cost of the recirculation pump, the median average DC current through the EC-P was 13.3 V and 1.7 A from August to October 2011 to obtain an effluent value of 1.05 mg/L TP. The current was increased to a median of 25.8 V and 2.2 A to obtain 0.45 mg/L TP.

Using the arithmetic average of 13.5 V and 1.7 A for the initial settings, and 25.0 V and 2.2 A for the second settings, the current used for the equivalent of a 3-bedroom house peak flow was 0.55 and 1.32 kW-hours per day, respectively. Based on a cost of 15 cents per kW-hour for electricity, the running costs would be 8 cents a day for the 1.05 mg/L TP quality, or 20 cents a day to attain the 0.45 mg/L TP quality.

Sampling Procedure

Septic tank effluent could not be sampled due to the recirculation ball valve, but raw sewage values of DC West were taken using a 24-hour composite sampler. Composite samples of the RSF effluent were taken from the final overflow tank leading to disposal.

Sewage & RSF Phosphorus

RSF Study 1: Raw sewage TP values ranged from 4.0 mg/L in December 2011 to 7.5 mg/L in March 2012. The median average of the N = 47 sewage values between July 2011 and April 2012 was 6.3 mg/L TP and the arithmetic average was 6.8 mg/L. Total iron content of the N = 45 DC West sewage values ranged from 0.6 to 1.8 mg/L Fe with a median average of 0.80 mg/L and an arithmetic average of 0.88 mg/L.

The RSF effluent results of Study 1 after a 3-week start-up period are summarized in Table 1. The TP values decreased from 5.4 mg/L in August 2011 to 0.9 mg/L within 3 weeks, stabilizing at 0.9 – 1.2 mg/L TP until the current was increased from 1.7 A to 2.2 A in October 2011 (Figure 1).

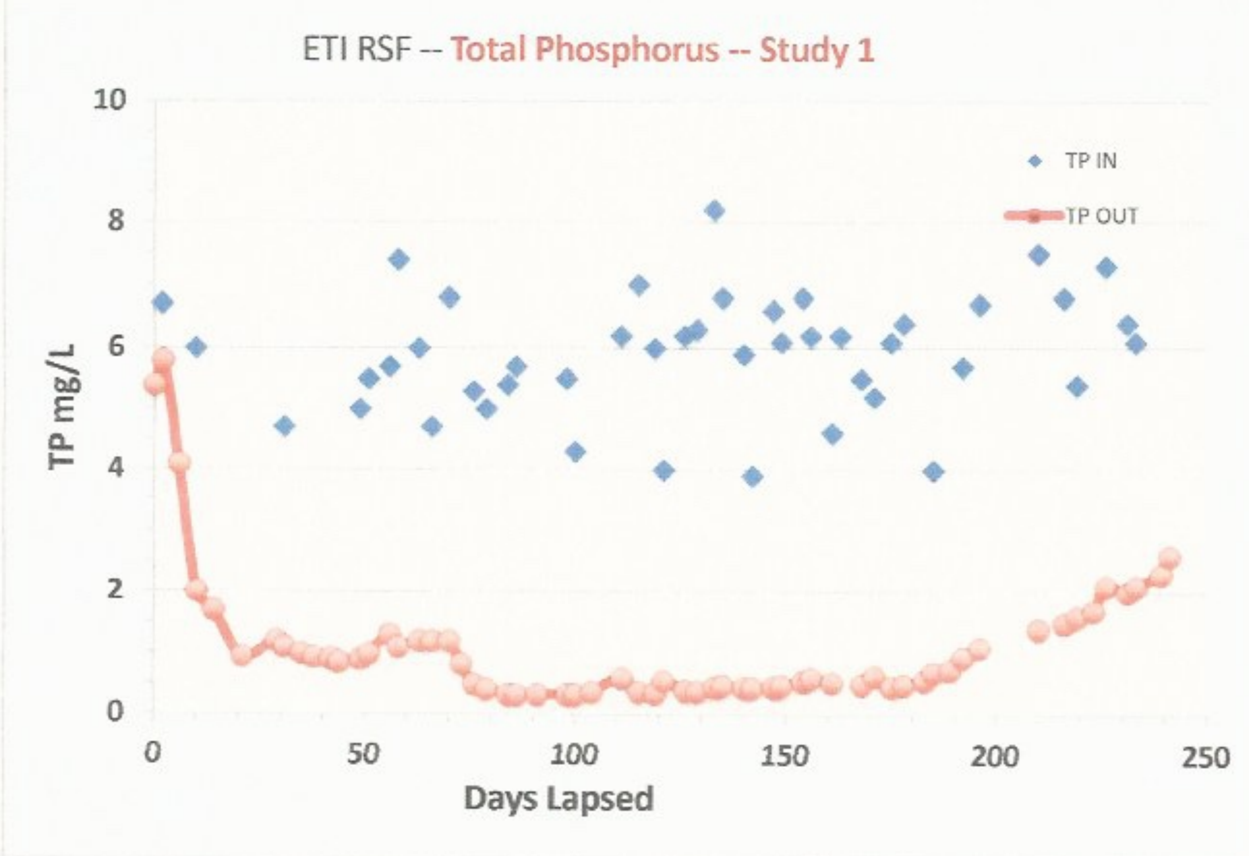

Figure 1. Study 1 of RSF testing at MASSTC; low amperage Study 1a changed to higher amperage Study 1b at Day 70. Influent numbers are raw sewage values.

The TP values stabilized at 0.3 – 0.6 mg/L until the unit began to short-circuit in February 2012. Total iron values in the RSF effluent decreased from 0.2 – 0.4 mg/L in the first 4 – 5 months to 0.1 – 0.2 mg/L in the last 1 – 2 months of operation, less than the raw sewage values. Iron oxide coating is visible on pipes and stone on the surface of the RSF (Figure 2.)

Figure 2. Iron oxide coating on pipes and pea gravel in the ETI RSF

In the sewage, total reactive phosphorus (PO_4-P) is 70 – 80% of the Total Phosphorus (e.g., median = 4.4 mg/L PO_4-P vs. 5.6 mg/L TP), likely due to phosphorus complexed in organic complexes. This ratio increases to 90 – 95% in the RSF effluent, where most of the organic solids have been degraded and the remaining phosphorus occurs as reactive ions.

The P-removal unit was thought to have started plugging as evidenced by high voltage in mid-February 2012 until it was removed in April 2012. TP values increased from 0.7 – 0.9 mg/L to 2.0 – 2.5 mg/L in this final period.

RSF Study 2: In Study 2, although DC current passing through the electrodes was 2.2 A then 3.3 A, the voltage across the electrodes was minimal, about 0.44 V and 0.65 V respectively. Little phosphorus was removed for the first month or so, then basically no removal after that. The electrodes were removed in July 2012 and poor wiring connections were discovered as the reason.

RSF Study 3: A new unit was installed in July 2012 and within the 3-week start-up period, was removing 85 – 90% of the TP and improved to 90 – 95% removal within 40 – 45 days after installation. Raw sewage TP values ranged from 5.0 mg/L in August 2012 to 8.0 mg/L in July 2012. The median average of the N = 20 TP sewage values between July 2012 and July 2013 was 6.5 mg/L TP and the arithmetic average was 6.6 mg/L.

Total iron content of the N = 26 *DC* West sewage values ranged from 0.64 to 2.4 mg/L Fe (with one anomaly at 6.0 mg/L) for a median average of 1.0 mg/L and an arithmetic average of 1.13 mg/L, not including the anomalous 6.0 mg/L value.

The RSF effluent results of Study 3 after a 3-week start-up period are shown in Figure 3 and summarized in Table 1. The TP values decreased from 5.3 mg/L in July 2011 to 0.8 – 0.9 mg/L within 3 weeks, decreasing to 0.6 – 0.8 mg/L TP after 7 weeks and stabilizing at 0.3 – 0.5 mg/L TP until the unit was removed in January 2013 after 20 weeks of operation.

Total iron values in the RSF effluent decreased from 0.4 – 1.0 mg/L in the first month to 0.2 – 0.3 mg/L over the remainder of operation, less than the raw sewage values.

Total reactive phosphorus (PO_4-P) consists of ~88% of the Total Phosphorus (5.7 vs. 6.5 mg/L) during the 2.5 months that phosphate values were analyzed in Study 3. This ratio increases to >98% in the RSF effluent (0.73 vs. 0.74 mg/L) where residual P is very reactive and available for additional bonding in soils.

The P-removal unit was working properly but a new control panel was expected soon, and the unit was removed early to let the RSF background TP values rise to what they would be with no P-removal unit installed. The TP values gradually rose from 2.1 mg/L in January 2013, soon after the unit was removed, to 6.7 mg/L in July 2013, close to the background sewage value.

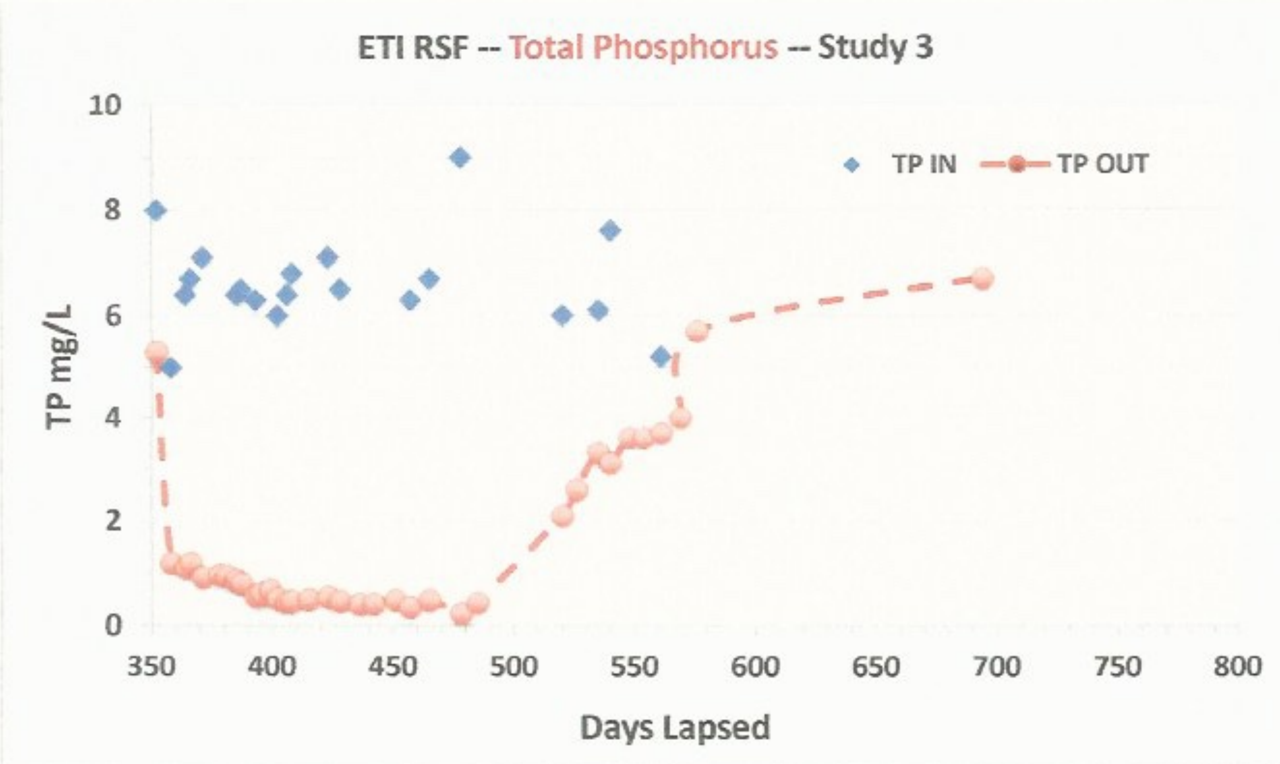

Figure 3. Study 3 of RSF testing at MASSTC; EC-P was disconnected on Day 491 and RSF effluent returned to background levels close to sewage levels.

Table 1. P-removal studies on MASSTC Recirculating Sand Filter, median averages between 3-week start-up and disconnection of unit. Study 2 had a malfunctioning electrode and is not included (see text).

	Sewage DC West			RSF Effluent			% Removal
	N	**TP**	**Fe**	**N**	**TP**	**Fe**	**TP**
Study 1a – Low Amp	5 - 8	5.6	0.9	14	1.05	0.2	81.3
Study 1b – High Amp	27 -29	6.0	0.8	31	0.46	0.2	92.3
Study 3	11 - 16	6.5	1.0	20	0.5	0.3	92.3

2. Soil Leach Field on MASSTC Site C3

System Configuration

The system consists of an existing 1500-gallon (5670 L) single-compartment concrete septic tank with the EC-P unit installed under the inlet riser. The effluent trickle feeds to a standard 3-outlet concrete distribution box, leading to a soil leach field consisting of three stone-pipe trenches. Each trench is 30 feet (9.1 m) in length or 90 feet (27.4 m) total length, 24 inches (0.6 m) wide, with pipes on 5 feet (1.5 m) centers. The crushed stone used was ¾" washed laid 12 inches (0.3 m) thick, surrounding the 4" perforated pipe situated in the center of the stone layer.

Native soil was excavated to about 5 feet (1.5 m) below grade and 36 inches (0.9 m) of sandy loam soil was placed in the excavation. The soil used was a mixture of 60% C33 sand and 40% silty loam both locally produced, with the percolation rate estimated at about 25 min/in (10 min/cm).

During backfilling, pan lysimeters were installed along the middle trench line to collect water from the soil bed at depths of 36 inches (0.9 m), 24 inches (0.6 m), and 12 inches (0.3 m). Sampling ports were brought to above grade and capped.

Hydraulic Loading Rate

As requested by the manufacturer, the C3 site received a continuous sewage flow of 145 gpd (550 L/day) to conform to peak Ontario loading rates for that type of soil receiving septic tank effluent. The basal loading rate on the trench floors was 0.81 gal/ft^2 (33.5 L/m^2), not including sidewall loading. Sewage dosing conformed to Standard NSF-40 rates of 35% between 6:00 a.m. and 9:00 a.m., 25% between 11:00 a.m. and 2:00 p.m., and 40% between 5:00 p.m. and 8:00 p.m.

EC-P Installation & Operation

The Waterloo EC-P and control unit were provided by the manufacturer and installed under the inlet riser of the septic tank on August 21 2012. The EC-P unit consists of an electrode assembly of four (4) steel plates each 6" x 15" (15 cm x 38 cm) with the interior plates being ¾" thick and the outer being 3/8" thick. The plates are wired to the control unit on surface and supplied.

The unit operated without issue over most of the ongoing study. A timer plug was accidentally unplugged for a couple of weeks in October 2012, disrupting the controls so no current passed through the EC-P. The readings on August 8 and 22 2013 were zero amperage indicating a short circuit, and the manufacturer cleaned the EC-P unit and reinstalled it on August 27 2013.

Energy Consumption

During normal operation (without the manufacturer's optional flushing pump), the median average DC current through the EC-P was 11.2 V and 0.9 A from August 2012 to October 2013, and 8.9 V and 0.8 A after the current settings were optimized to 0.7 – 0.9 A in January 2013. Using the arithmetic average of 14.1 V and 1.0 A, the current used was 14.1 W or 0.34 kW-hours per day. The current was also measured by MASSTC staff to be about 0.45 kW-hours per day for the continuous flow of 145 gpd (550 L/day).

For standard 3- and 4-bedroom houses with Massachusetts peak flows of 330 gpd (1250 L/d) and 440 gpd (1665 L/d), respectively, the power consumption would be 1.02 kW-hours/day and 1.37 kW-hours/day. Based on a cost of 15 cents per kW-hour for electricity, the running costs would be 15 cents a day for a 3-bedroom house and 20 cents per day for a 4-bedroom house at peak flow. A 120 V 9 A flushing pump operating 10 minutes a day would contribute about 0.2 kW-hours or 3 cents per day.

Trench Ponding

Three observation ports were sited on the stone – soil interface of each trench to inspect for ponding. No ponding was evident during the testing except for ¾" – 1" ponding in one trench in late June to mid-July 2013.

Sampling Procedure

Grab samples were taken of septic tank effluent at the inlet to the distribution box at the same sampling times as pan lysimeters. For the pan lysimeter sampling, the pan reservoirs were purged 2-3 days prior to the extraction of a grab sample from each reservoir.

Sewage & Septic Effluent Phosphorus

Septic tank effluent TP values ranged from 3.4 mg/L in November 2012 to 20 mg/L in January 2013. The median average of the N = 26 septic effluent values between October 2012 and October 2013 was 6.2 mg/L TP and the arithmetic average was 6.7 mg/L. These values are similar to the DC West raw sewage values, which had a median average of 6.5 mg/L and arithmetic average of 6.8 mg/L from July 2012 to July 2013, indicating that minimal TP was removed in the septic tank during this testing.

Total iron content of DC West sewage values ranged from 0.2 to 2.0 mg/L Fe with a median average of 0.93 mg/L and an arithmetic average of 1.0 mg/L. With the EC-P unit generating iron in the septic tank, the septic effluent throughout the C3 Soil Study has a median average of 7.1 mg/L and arithmetic average of 12.5 mg/L. After optimization in January 2013, the septic effluent had a median average of 5.6 mg/L total Fe and arithmetic average of 6.8 mg/L total Fe, considered by the manufacturer to be the desired concentration of total Fe in septic tank effluent for this concentration of phosphorus.

Soil Effluent Phosphorus

12" Soil Depth: The N = 25 soil effluent TP values ranged from 0.02 mg/L to 0.21 mg/L (with 0.4 mg/L at start-up) between October 2012 and October 2013, with a median average of 0.09 mg/L TP and an arithmetic average of 0.12 mg/L. These values represent removal rates of 98.5% and 98.2% respectively from septic tank effluent. The trends in TP attenuation for each of the pan lysimeters can be seen in Figure 4.

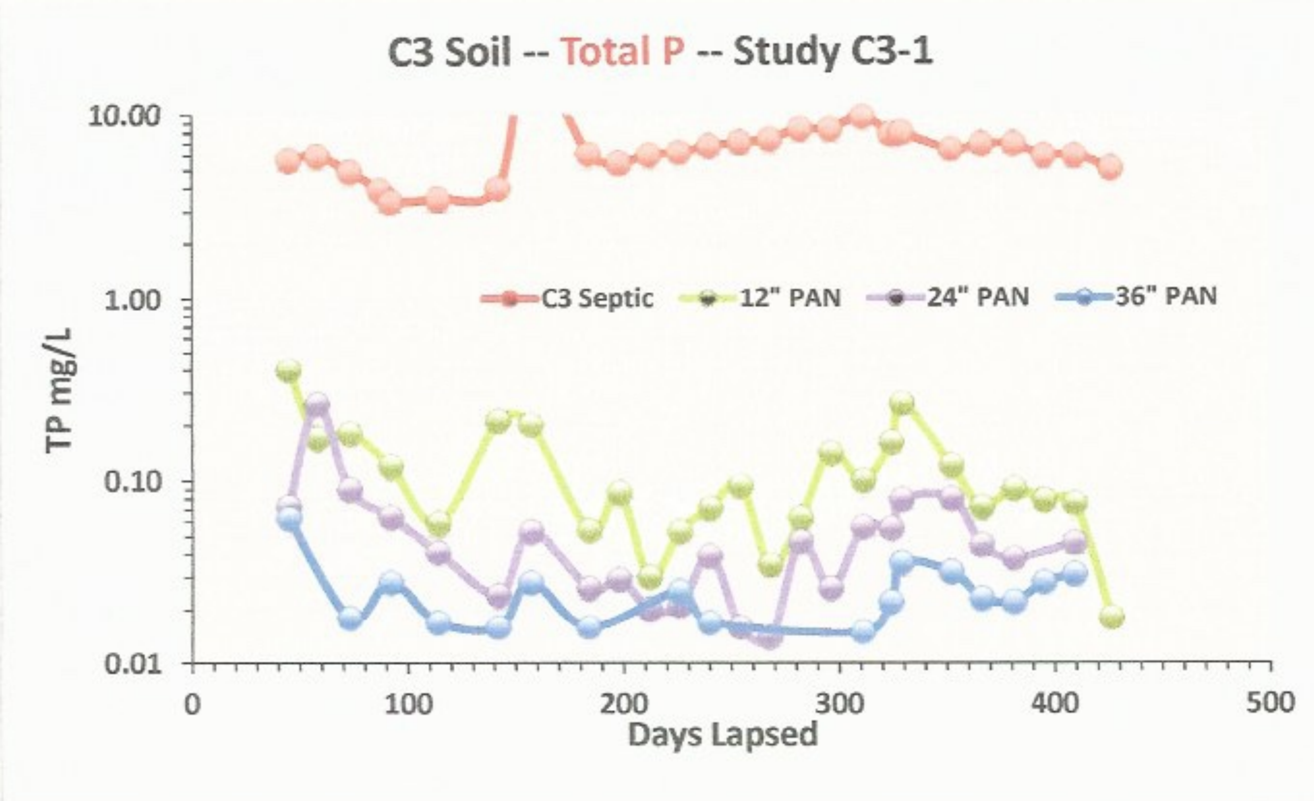

Figure 4. On-going EC-P Study C3-1 for P removal in septic tank + soil leach field. High percentages of P removal are attained and generally improve with time.

For total iron, the 12" pans had a median average of 0.76 mg/L and arithmetic average of 1.4 mg/L, similar to the 0.93 mg/L and 1.0 mg/L total iron in raw sewage. Although only 6 analyses of TSS were possible due to sample volumes, the highest TSS = 14 mg/L coincides with the highest value of 3.9 mg/L total iron, and the lowest TSS = 2.7 mg/L coincides with the lowest TP value of 0.02 mg/L. This relationship suggests that the higher iron values correlate with TSS and not TP values, and that the anomalous Fe values are not in the water phase but in the solid phase as *soil or sand particles*.

24" Soil Depth: The N = 23 soil effluent TP values ranged from 0.01 mg/L to 0.09 mg/L (with 0.26 mg/L at start-up) between October 2012 and October 2013, with a median average of 0.05 mg/L TP and an arithmetic average of 0.05 mg/L. These values represent removal rates of 99.2% and 99.2% respectively from septic tank effluent.

For total iron, the 24" pans had a median average of 0.97 mg/L and arithmetic average of 1.06 mg/L, similar to the total iron in raw sewage. Again, a high TSS value of 15 mg/L coincides with a high value of 12.0 mg/L total iron.

<u>*36" Soil Depth:*</u> The N =17 soil effluent TP values ranged from 0.02 mg/L to 0.04 mg/L (with 0.06 mg/L at start-up) between October 2012 and October 2013, with a median average of 0.02 mg/L TP and an arithmetic average of 0.03 mg/L. These values represent removal rates of 99.7% and 99.5% respectively from septic tank effluent.

For total iron, the 36" pans had a median average of 0.78 mg/L, similar to raw sewage, with an arithmetic average of 5.74 mg/L, caused by four anomalously high samples of 25 – 38 mg/L coinciding with high TSS values of 32 – 63 mg/L respectively. This correlation with suspended solids suggests that soil or other particles are carrying the iron whereas the water phase is low in both iron and phosphorus.

These high TSS values are seen only in the 36" pan lysimeter and not the 12" or 24" pans. Because the high iron does not correlate with phosphorus, but only TSS, it could represent iron oxide nodules or concretions forming in the one sampling port. Why it is seen only in one pan is not understood.

Table 2. P-removal study on MASSTC Septic Tank + Soil Leach Field; median averages between start-up and present day.

	Septic Tank Effluent			**12" Soil Pan Lysimeter**		
	N	**TP**	**Fe**	**N**	**TP**	**Fe**
Study C3-1 **% Removal TP**	26	6.15	7.1	25	0.09 98.5	0.76
	24" Soil Pan Lysimeter			**36" Soil Pan Lysimeter**		
	N	**TP**	**Fe**	**N**	**TP**	**Fe**
Study C3-1 **% Removal TP**	23	0.05 99.2	0.97	17	0.02 99.7	0.78

3. Single Pass Sand Filter without EC-P

System Configuration

This system is not part of the Waterloo EC-P testing, but is included at the request of the manufacturer as a type of benchmark system for background TP removal. These sand filters were part of the 1999 – 2002 Environmental Technology Initiative Program at MASSTC on Sites F1, F2, and F3 and are the standard "Title 5" trenches by the standard in Massachusetts regulations[1]. They each consisted of a 1500-gallon (5670 L) single-compartment concrete septic tank followed by with gravity-fed trenches placed above 5' (1.5 m) of C33 sand.

Pan lysimeters were placed at 12", 24", and 36" depths below the stone – soil interface. (The manufacturer has chosen the 24" pan lysimeter to compare with the C3 Leach Field work in Section 2 of this report due to completeness of the data set and to 24" (0.6 m) being a standard vertical separation in many jurisdictions.)

Hydraulic Loading Rate

The F1, F2, F3 sites received a consistent dose of 330 gpd (1250 L/day) to conform to a 3-bedroom house in Massachusetts and septic tank effluent was split within a distribution box to supply 25% of the flow to the stone trench. The basal loading rate on the trench floors was 1.9 gal/ft^2 (78.6 L/m^2), not including sidewall loading which is normally allowed in MA. Sewage dosing conformed to Standard NSF-40 rates of 35% between 6:00 a.m. and 9:00 a.m., 25% between 11:00 a.m. and 2:00 p.m., and 40% between 5:00 p.m. and 8:00 p.m.

Sampling Procedure

Raw sewage from the influent source (DC West) was sampled using a 24-hour composite sampler. Composite samples of the septic tanks were sampled at the distribution box following the tanks. For the pan lysimeter sampling, the pan reservoirs were flushed out the week before retrieving a grab sample from each pan.

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[1] "Title 5" refers to Commonwealth of Massachusetts Regulations *310 CMR 15.000 THE STATE ENVIRONMENTAL CODE, TITLE 5.* STANDARD REQUIREMENTS FOR THE SITING, CONSTRUCTION, INSPECTION, UPGRADE AND EXPANSION OF ON-SITE SEWAGE TREATMENT AND DISPOSAL SYSTEMS AND FOR THE TRANSPORT AND DISPOSAL OF SEPTAGE.

Sewage & Septic Tank Effluent Phosphorus

Both reactive phosphorus or phosphate ion (PO_4-P) and total phosphorus (TP) were analyzed in the raw sewage, but only TP was analyzed in the septic tank effluents, and in the pan lysimeters only PO_4-P was analyzed. As a result, only an approximation of the removal percentage can be made between sewage and soil percolate. However, PO_4-P does approximate TP in highly treated effluent (as seen in the Section 1 RSF results), but is always somewhat less than TP. Therefore the percent removal of TP can be approximated and represents an upper limit removal rate of the C33 sand filter, i.e., if TP were analyzed in the soil percolate, the removal rate would be a few percent less.

Sewage TP values ranged from 3.4 mg/L in October 2000 to 7.9 mg/L in March 2002. The median average of the N = 65 sewage values between May 1999 and May 2002 was 5.25 mg/L TP and the arithmetic average was 5.22 mg/L. The median averages of the N = 64 septic tank effluent values for each tank was 4.65, 4.72 and 4.77 mg/L TP, with the arithmetic averages being 4.72, 4.48 and 4.80 mg/L TP. Overall, the septic tanks removed about 10% of TP from raw sewage.

Soil Effluent Phosphorus

24" Soil Depth: The N = 41 – 62 soil effluent values ranged from 1.78 mg/L PO_4-P in the F2 pan in July 2000 to 5.73 mg/L PO_4-P in the F3 pan in August 2001. The median averages of the N = 49 F1 pan was 3.62 mg/L PO_4-P, the N = 62 F2 pan was 3.69 mg/L PO_4-P, and the N = 41 F3 pan was 3.86 mg/L PO_4-P. These values represent a maximum removal rate from the sewage TP of 31.0%, 29.7%, and 26.5% for F1, F2, and F3 respectively, and from the septic tank effluent TP of 22.2%, 21.8%, and 19.1% respectively (see Figure 5 for F2 septic and pan results).

The removal rates of the C-3 Soil Leach Field Study were all >98% from the septic tank effluent, and the 24" pans were 99.2% removal compared to septic tank effluent. The three 24" pans for the ETI C-33 Title 5 sand filter averages 21.0% removal, as a maximum rate for TP removal from the septic tank effluent.

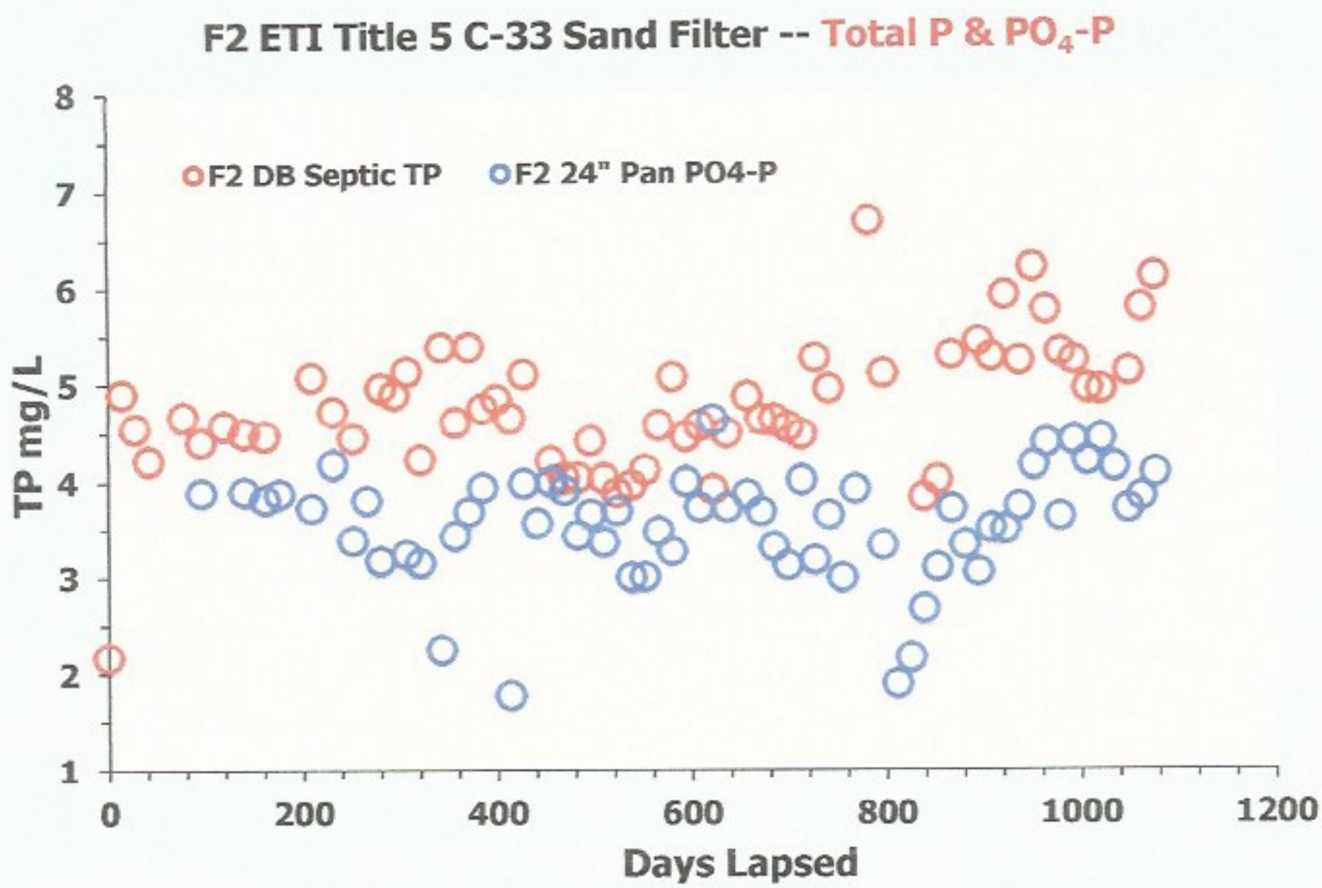

Figure 5. Results of MASSTC Site F2 for C-33 Sand Filter in 1999 – 2002. Removal of TP between septic tank effluent and soil percolate is 20 – 25%

APPENDIX D
DETAILED MODEL RESULTS – STORMWATER PHOSPHORUS TREATMENT

20-7270 - Townhomes of Lake Thomas Development - Water Quality Model (Pre-Development) June 10, 2020

Pre-Development Conditions - Total Disturbed Area								
Return Period	**Land Use**	**Area m^2**	**Area-ha**	**Precipitation (m)**	**Runoff C**	**Total Runoff (m^3)**	**TP-mg/L**	**TP-kg**
Annual	Commercial	8,500	0.85	1.396	0.32	3805.62	0.2	0.76

Post-Development Conditions With No BMPs - Total Disturbed Area								
Return Period	**Land Use**	**Area m^2**	**Area-ha**	**Precipitation (m)**	**Runoff C**	**Total Runoff (m^3)**	**TP-mg/L**	**TP-kg**
Annual	Medium-Density Residential	8,500	0.85	1.396	0.45	5322.10	0.2	1.06

Effect of urbanization with no control

	Existing Land Use	**Future Land Use**	**Net Change**
Annual TP Loading (kg)	0.76	1.06	Increase

Pre-development Runoff Coefficients

Land Type	% Land	Runoff C
Residential Impervious	19%	0.85
Residential Pervious	81%	0.2
Weighted Residential Runoff C	**0.32**	

Post-development Runoff Coefficients

Land Type	% Land	Runoff C
Residential Impervious	38%	0.85
Residential Pervious	62%	0.2
Weighted Residential Runoff C	**0.45**	

20-7270 - Townhomes of Lake Thomas Development - Water Quality Model (Post-Development)

June 10, 2020

Post-Development Conditions With BMPs

Return Period	Land Use	Area m^2	Area-ha	Runoff C	Precipitation (m)	Total Runoff (m^3)	TP-mg/L	TP-kg
Annual	Medium-Density Residential	8,500	0.85	0.45	1.396	5322.10	0.2	1.06

			Uncontrolled	End of Treatment Train
	Land Use	TP - mg/L	TP - kg	TP - kg
Annual	Medium-Density Residential	0.2	1.06	0.722

	BMP	Tribuary Area Size (m^2)	% of Total Area	Individual TP Removal Effeciency	Net Project Removal Effeciency
Area #1	Grass Swale	3,280	39%	40.0%	32.1%
Area #2	Grass Swale	2,068	24%	40.0%	
Area #3	Grass Swale	1,482	17%	40.0%	
Area #4	None	1,670	20%	0.0%	

Effect of urbanization with BMPs

	Existing Land Use	Future Land Use	Net Change
Annual TP Loading (kg)	0.761	0.722	Decrease

APPENDIX E
DETAILED MODEL RESULTS – SEWAGE PHOSPHORUS TREATMENT

Calculate P Loading in Exsiting OSSDS*

**assume existing system does not have TP removal infrastructure ($RED_{OSS,P}$ = 0)*

$$OSSDS\ P\ Load = \frac{(1 - RED_{OSS,P}) * C_{SEPTIC} * Q_{SEPTIC}}{10^6}$$

<u>Where:</u>

OSSDS P Load = OSS output P load (kg P)
$RED_{OSS,P}$ = OSS P reduction rate(-)

C_{SEPTIC} = P concentration of effluent (mg/L)

Q_{SEPTIC} = effluent daily flow rate (L/day)

Calculate P Loading from Exsiting Structures

Pre-Development	$RED_{OSS,P}$	C_{SEPTIC} (mg/L)	Q_{SEPTIC} (L/day)
4-Bedroom Residential Home	0	14.4	0
Commericial/Office Building	0	14.4	1,000

$RED_{OSS,P}$ = 0

Pre-Development	OSSDS P Load (kg/day)	OSSDS P Load (kg/year)
4-Bedroom Residential Home	0.00	0.00
Commericial/Office Building	0.01	5.26
Total Pre-Development Structures	0.01	5.26

Calculate cumulative P load from exsiting septic OSSDS

Pre-Development	OSSDS P Load (kg) (daily)	OSSDS P Load (kg) (10 Year)	OSSDS P Load (kg) (20 Year)	OSSDS P Load (kg) (30 Year)	OSSDS P Load (kg) (40 Year)	OSSDS P Load (kg) (50 Year)
Total Pre-Development Structures	0.014	52.56	105.12	157.68	210.24	262.80

Calculate P Loading in Proposed OSSDS

$$OSSDS\ P\ Load = \frac{(1 - RED_{OSS,P}) * C_{SEPTIC} * Q_{SEPTIC}}{10^6}$$

Where:

OSSDS P Load = OSS output P load (kg P)
$RED_{OSS,P}$ = OSS P reduction rate(-)

C_{SEPTIC} = P concentration of effluent (mg/L)

Q_{SEPTIC} = effluent daily flow rate (L/day)

Post-Development	$RED_{OSS,P}$	C_{SEPTIC} (mg/L)	Q_{SEPTIC} (L/day)
4-Bedroom Residential Home	0.995	14.4	0
Six, 3-unit (4-Bedroom) Townhomes	0.995	14.4	27,000

$RED_{OSS,P}$ = 0.995

Post-Development	OSSDS P Load (kg/day)	OSSDS P Load (kg/year)
4-Bedroom Residential Home	0.000	0.00
Six, 3-unit (4-Bedroom) Townhomes	0.002	0.71
Total Post-Development Structures	0.002	0.71

Calculate OSS P Loads

Calculate daily septic tank load

Post-Development	OSSDS P Generation (kg/day)	OSSDS P Generation (kg/year)
4-Bedroom Residential Home	0.00	0.00
Six, 3-unit (4-Bedroom) Townhomes	0.39	141.91
Total Post-Development Structures	0.39	141.91

$$OSSDS\ P\ Generation\ = \frac{C_{SEPTIC} * Q_{SEPTIC}}{10^6}$$

Where:

OSSDS P Genertaion = Phosphorus load generated as influent, prior to any form of treatment (kg)

Calculate cumulative P load from septic tank (untreated for TP)

Post-Development	OSSDS P Generation (kg) (daily)	OSSDS P Generation (kg) (10 Year)	OSSDS P Generation (kg) (20 Year)	OSSDS P Generation (kg) (30 Year)	OSSDS P Generation (kg) (40 Year)	OSSDS P Generation (kg) (50 Year)
Total Post-Development Structures	0.389	1419.12	2838.24	4257.36	5676.48	7095.60

Calculate cumulative P load from OSSDS (treated for TP)

Post-Development	OSSDS P Load (kg) (daily)	OSSDS P Load (kg) (10 Year)	OSSDS P Load (kg) (20 Year)	OSSDS P Load (kg) (30 Year)	OSSDS P Load (kg) (40 Year)	OSSDS P Load (kg) (50 Year)
Total Post-Development Structures	0.002	7.10	14.19	21.29	28.38	35.48

Summary of Reductions

Development Scenario	OSSDS P Load (kg) (annual)	OSSDS P Load (kg) (10 Year)	OSSDS P Load (kg) (20 Year)	OSSDS P Load (kg) (30 Year)	OSSDS P Load (kg) (40 Year)	OSSDS P Load (kg) (50 Year)
Total Pre-Development Structures	5.256	52.560	105.120	157.680	210.240	262.800
Total Post-Development Structures	0.710	7.096	14.191	21.287	28.382	35.478