Bay of Quinte Regional Master Drainage Planning Project

AUGUST 2011

HOSPITAL CREEK

MASTER DRAINAGE PLAN

WaterPlan Associates

XCG

Quinte CONSERVATION

Bay of Quinte Remedial Action Plan

Acknowledgements

The Hospital Creek Master Drainage Plan was undertaken as part of the 3-year *Stormwater Management/ Pollution Prevention and Control Plan for Bay of Quinte Municipalities*.

This project was a large team effort. Key agencies and individuals are acknowledged below.

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- Federal – the Great Lakes Sustainability Fund administered by Environment Canada
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Substantial in-kind work effort was provided by the staff of:

- Quinte Conservation:
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  - Tim Trustham, Plan Review Ecologist;
  - Brad McNevin, Fisheries Biologist;
  - Mark Boone, P.Geo., Hydrogeologist.
- Prince Edward County:
  - Jo-Anne Egan, Manager of Planning
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- WaterPlan Associates – Project Management and report coordination
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Executive Summary

The Hospital Creek Master Drainage Plan (MDP) was prepared for the entire Hospital Creek watershed. As Hospital Creek is tributary to the Bay of Quinte, these lands are of specific interest for stormwater management (SWM) in the Bay of Quinte Remedial Action Plan.

The study was completed as a co-operative effort through the financial inputs and work efforts of the federal and provincial governments, Quinte Conservation, staff of Prince Edward County and consultant support.

The Master Drainage Plan serves as a SWM strategy document to assist practitioners in developing SWM plans that take a watershed-based approach (as opposed to a site-specific approach) for proposed developments within the study area. Stormwater management guidance is provided at a sub-basin level. The following is a list of the major findings/conclusions of the report:

- **Water quantity controls** are required for proposed developments north and south of Johnson Street (County Road 5) based on existing hydraulic constraints along the reach of the creek between Johnson Street and Main Street.
    - A centralized on-line stormwater management (SWM) pond is proposed to provide water quantity control for the residential developments upstream and within Sub-basin 1B with an overall active storage volume of between 13,000 m^3 and 16, 500 m^3 under Scenario A and B, respectively. Scenario A is where approximately 3,000 m^3 of storage is available for peak flow attenuation just upstream of Johnson Street (backwater from the existing culvert) while Scenario B is where this storage is not available due to future developments and/or site constraints. An alternative location for the pond is just upstream of Johnson Street; however, the total storage required increases to 18,400 m^3.
    - The proposed water quantity control system accommodates the Anderson Subdivision, which is a registered plan of subdivision and located in Sub-basin 1C, that was approved with no water quantity controls proposed.
    - Future residential developments in Sub-basin 1C do not require water quantity controls.
    - A centralized SWM pond is approved (designed by others) to provide water quantity control for runoff generated from future developments within the Picton Industrial Park (except the Hydro One development) with a total active storage volume of 2,440 m^3. The Hydro One development is implementing on-site water quantity control under separate site plan approval.
- **Water Quality Control**
    - Water quality control to a Level 1 water protection criteria will need to be provided for all future developments.
    - The proposed on-line centralized SWM facility located within Sub-basin 1B will not be designed to provide water quality control. Instead, water quality controls for developments upstream of Johnson Street will be provided on-site. The County discourages the use of oil-grit separators. Therefore, on-site water quality

SWM ponds or a centralized SWM pond that serves multiple developments are the recommended solution to achieve Level 1 protection criteria.
- Water quality controls for proposed developments within the Picton Industrial Park will be achieved via the proposed centralized SWM pond downstream of Johnson Street.

- **Infiltration**
  - Infiltration of stormwater is recommended where feasible. Due mainly to the variation in water table elevation, site-specific geotechnical data would be required to confirm the feasibility of infiltration.

This report is intended to serve as a companion document to future planning documents.

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1. INTRODUCTION

This Master Drainage Plan (MDP) has been prepared as part of a cooperative effort between Prince Edward County and Quinte Conservation. Funding for this project was provided by the Great Lakes Sustainability Fund and the Ontario Ministry of the Environment. It is part of a larger initiative to improve water quality around the Bay of Quinte by establishing Pollution Prevention and Control Plans in the communities encircling the bay as well as preparing selected Master Drainage Plans in developing areas of Belleville, Trenton, Picton and Napanee.

Approximately half of the 2.1 km^2 (210 ha) Hospital Creek watershed is located in the Picton Urban Centre, with the balance of the area located along the west side of abandoned CNR rail line, which is commonly referred to as the Millenium Trail (refer to Map 1.1). Being a tributary of the Bay of Quinte, Hospital Creek is of specific interest in the Bay of Quinte Remedial Action Plan (BQRAP) as it falls within the implementation area for stormwater management guidelines developed for the BQRAP. Quinte Conservation implements the stormwater management guidelines for the Bay of Quinte Remedial Action Plan.

Hospital Creek flow originates west of Talbot Street. Total elevation drop is about 35m, from ~110 m (geodetic) to ~75m at the mouth of Hospital Creek. The lower part of Hospital Creek is urbanized, where it flows through the Picton Bay Industrial Park via an altered trapezoidal channel. The Town is growing in this area and the presently undeveloped lands located between Talbot Street and County Road 5 are experiencing development pressure.

Three reports provide useful background information pertaining to stormwater and floodplain management within the watershed. These include the 1986 Hospital Creek Study, the 1988 Channel Design and Preliminary Stormwater Management Assessment – Hospital Creek, Industrial Park and the December 1989 Hospital Creek Stormwater Management Review , which were prepared by J.D. Paine Engineering Inc. These reports are discussed further in Section 7.

1.1 Cultural History

The Bay of Quinte was an important waterway to both Native Peoples and the early French explorers who traversed the area as early as 1615 a.d. The first Loyalists began to settle the region as early as 1770, which lead to clear cutting of the forests in order to develop the land for agriculture (Soil Survey of Prince Edward County, 1948). Prince Edward County has been essentially agricultural since settlement, and until recently, the towns and villages have been oriented toward servicing the agrarian economy. In addition to a small specialized field crop and tender fruit sector; the commercial use of Lake Ontario waters led to a (now diminishing) commercial fishing industry within the County (PEC Conservation Report, 1968). More recently, the area is becoming known for locally produced wines and waterfront cottage/retirement land uses.

1.2 Existing Land Uses

The majority of the watershed west of County Road 5 is used for agricultural purposes save and except a small portion of land immediately west of County Rd 5 which contains both residential and park land uses. Approximately half of the watershed that is located east of County Rd 5 is comprised of industrial and institutional land uses while the balance is used for agricultural purposes.

1.3 Surface Water

The main branch of Hospital Creek is approximately 3.1 km long, flowing northeasterly from west of Talbot Street to the Bay of Quinte. Hospital Creek enters the Bay of Quinte as a perched outlet after negotiating the steep bank along the bay. The stream gradient in the developed reach north of County Road 5 and west of Highway 49 is approximately 0.3 %. The stream reach east of Highway 49 to the Bay is approximately 3.0% (not including the drop associated with the perched outlet while the reach south of Johnston Street and north of Talbot Street has a stream gradient of about 1.7%.

Within the developed portion of the watershed, there is a stormwater outfall located at the Highway 49 culvert. This discharge lacks water quality management controls. Furthermore, runoff generated from the industrial developments that drain directly to the creek does not undergo water quality treatment.

The stream is intermittent and temperatures suggest it is a cool water system based on the Ontario Stream Assessment Program protocol. Field observations were taken in one growing season via a data logger. Further discussion on field observations can be found in Section 10.

As described in Section 2, a separate Pollution Prevention and Control Planning (PPCP) process was undertaken in parallel with this Master Drainage Plan.

2. SCOPE AND OBJECTIVES

The development of Master Drainage Plans (MDPs) for urban growth centers around the Bay of Quinte is part of a 3-year cooperative, multi-partner, *Bay of Quinte Regional Master Drainage Planning Project* initiative that includes: the federal government (Environment Canada through the Great Lakes Sustainability Fund), the Ontario provincial government (Ministry of Environment), Quinte Conservation (Project Coordinator), Lower Trent Conservation and local municipalities.

The Hospital Creek MDP is the last of four similar Master Drainage Plan projects around the Bay of Quinte. Mayhew Creek MDP was completed in March 2009 and thus serves as a template for this MDP. Other MDPs for Selby Creek in the Town of Greater Napanee and Norbelle Creek in the City of Belleville were completed in March 2010 and July 2010, respectively, under separate covers. The three aforementioned MDPs are considered drafts because the public consultation and adoption by council has not yet occurred.

Each of the four MDP processes for Belleville and the other urban growth areas around the Bay of Quinte is being conducted separately but in parallel with a Pollution Prevention and Control Planning (PPCP) analysis. Separate MDP and PPCP reports will be produced within the three-year program timeframe.

The ultimate goal of undertaking select MDPs is to identify and implement stormwater management strategies and treatments that will improve stormwater quality outflows entering the Bay. Collectively, the stormwater management improvements achieved through master drainage planning will provide overall ecological, health, recreation, tourism and aesthetic enhancement benefits to the Bay of Quinte.

For the Hospital Creek MDP process, both Prince Edward County and Quinte Conservation are directly involved. Project input contribution from all partners includes both funding and commitment of in-house staff time. In addition, consultant support is included to undertake project management and specialized technical water quality and hydrology support functions.

Stormwater management associated with urban growth is the primary force driving water management planning for the defined MDP area.

SWM options are identified and evaluated from a technical, location, growth sequencing, and environmental standpoint. Detailed design is not part of this study. The County will integrate the recommended agreed SWM treatments and associated policies into the Official Plan.

Beyond the strict analysis of determining SWM requirements to control storm runoff from future growth, other related water management and planning issues have been integrated into the report. These include discussion of flooding and flood control, hydraulic capacity of channels and structures, channel erosion, on-stream ponds, fish, aquatic and terrestrial wildlife, vegetation, water quality sampling, soils and hydrogeology and planning analysis. Recommendations are made regarding areas that require additional technical and policy analysis. Thus, the resultant plan expands beyond the traditional limited engineering concept of master drainage planning and establishes the framework for the development of a broader multi-disciplinary plan for the study area.

In consideration of the above-mentioned stormwater planning issues, the Master Drainage Plan is restrictive in terms of meeting SWM quality and quantity control objectives without being prescriptive as to specific treatments and locations – the development community is responsible for this level of detail to support their approval applications.

3. METHODOLOGY AND APPROACH

An issue common to Hospital Creek and other candidate MDP project watersheds around the Bay of Quinte is that existing topographic mapping is generally limited to Ontario Base Maps with contour intervals of 5 metres and a vertical accuracy of only $^{+}/_{-}$ 2.5m. This is not sufficiently precise to assess the stormwater management implications of proposed land use changes, especially in flatter areas. Therefore, an integral part of the MDP planning process was to obtain laser-based high resolution LiDAR (Light Detection And Ranging) contour mapping and photography. This process enables the integration of lasers, global positioning and inertial navigation systems with fixed-wing (or helicopter) flights to achieve a high degree of vertical (~15cm) and horizontal (~30cm) map resolution. LiDAR mapping per Map 3.1 was generated for the entire watershed area.

A heavy emphasis was placed on digital GIS (Geographic Information Systems) mapping to illustrate and integrate various plan elements – the maps included are labeled according to the report section in which they are referenced.

The Hospital Creek watershed is discretized into a numbered of sub-basins, as are shown on all maps, to enable the hydrologic modeling process to determine cumulative pre-development (ie existing) flows at specified system locations as the basis for assessing impacts of and necessary SWM treatments for controlling future development.

The tasks undertaken to complete the Hospital Creek Master Drainage Plan include:

a. Characterize the hydrology, hydraulics, land use, hydrogeology, aquatic and land-based biology of the watershed from existing information and mapping;
b. Define the urban growth areas;
c. Disaggregate the watershed into relevant sub-basins;
d. Using detailed LiDAR contour mapping of the growth area, undertake detailed hydrologic and hydraulic analysis of the regulatory flood event (100-year) in the stream at various points of interest such as road crossings;
e. Undertake hydrogeological soils and water table analysis in the growth area to determine which areas exhibit good infiltration potential versus areas normally above the water table where infiltration would not be appropriate;
f. Preliminary investigation of the important terrestrial, aquatic and fisheries biology resources along Hospital Creek;
g. Discuss existing impediments and enhancement opportunities and define areas both suitable and unsuitable for use as SWM facilities;
h. Define the characteristics of soils in the growth area and the potential for use of infiltration techniques to recharge the water table and sustain and enhance base flows of Hospital Creek;
i. Define possible SWM options at both a conceptual and a preliminary engineering level and evaluate the required size, locations, technical effectiveness, cost, sequencing, land ownership/ assembly and funding implications of each;
j. Assist the municipality in discussing options with the stakeholders and the public;
k. Prepare an implementation plan as a companion document to the Town Official Plan.

Sections 11 & 12 deal at the sub-basin level to describe the various SWM options and factors related to the identified growth areas in each individual sub-basin and to develop SWM management policies and strategies required to be undertaken by developers at the future site plan application phase.

4. CONSULTATION

The Hospital Creek watershed is located within Prince Edward County and the county is an integral team member and funding partner. County staff are directly involved along with staff from Quinte Conservation, including those involved and experienced in the Bay of Quinte Remedial Action Plan plus consultant support in the development of the initial planning and technical elements of the Master Drainage Plan for Hospital Creek.

Landowners, stakeholders and the general public will have an opportunity to review and comment on the draft plan prior to Town adoption of an SWM implementation strategy.

5. BACKGROUND INFORMATION

Each project team member accessed and utilized data and information sources from their own component perspective, including field work as needed. Mapping was referenced both at the general watershed and detailed LiDAR levels of detail.

The LiDAR mapping described in Section 3 provided a detailed topographic base for Master Drainage Planning. All maps provided comprise of digital data in layers that can be combined both to illustrate features of interest in various combinations and to do area, length, slope and other measurements required for SWM calculations.

6. PRESENT AND FUTURE LAND USE

6.1 Introduction and Planning Context

The area is approximately 210 hectares located in the central-east portion of the Municipality of Prince Edward County, a single tier municipality formed from the amalgamation of 10 former townships in 1998. The Picton Urban Centre with a population of approximately 4,000 is the largest urban centre in the County.

The lands within the Study Area are located partly within and to the north of the urban boundary of the Picton-Hallowell Secondary Plan, which was approved by the Town of Picton and the Township of Hallowell in 1979 and approved by the Minister of Housing as Amendment No. 19 to the County of Prince Edward Official Plan in 1980.

Generally, the Millennium Trail (former CN Rail line) forms the western boundary line of the Secondary Plan area. Lands located outside of the Secondary Plan area are designated Prime Agricultural. These agricultural lands have strong Provincial protection from further urban expansion through the policies of the Provincial Policy Statement.

The County Growth and Settlement Strategy adopted by Council in 2003 reviewed the boundaries of the various communities across the County. As a result the urban boundary of the

Picton Urban Centre was expanded to include the Macaulay Village area, the Loch Sloy Industrial Park, the Picton Golf course and lands to the south and southeast.

There is a mix of land uses throughout the study area including single detached residential, agricultural/rural, plans of subdivision, open space, institutional and industrial. Municipal sewage and water services are available in a portion of the study area.

6.2 Land Use Strategy

The County has recently engaged the services of consultants to prepare a new secondary plan for the Picton-Hallowell area to replace the existing 1979 secondary plan. The consultants will be utilizing the existing Picton-Hallowell urban boundary. Planning staff will ensure that the results of this master drainage plan be incorporated into the new secondary plan to guide development in the study area.

6.3 Approved Residential Development

Within the study area there currently exist a number of fully approved and registered plans of subdivision as well as a number of draft approved plans of subdivision:

1. *Frank Subdivision* – 31 single detached residential lots and 1 block for an apartment complex (number of units unknown). Registered plan of subdivision; currently vacant.

2. *Wellbanks Subdivision* Phase 1 – 25 single detached residential lots. Registered plan of subdivision; under construction.

3. *Welbanks Subdivision* Block – 98 residential units. Draft approved plan of subdivision.

4. *Anderson Subdivision* – 64 single detached residential lots. Registered plan of subdivision; currently vacant.

5. *Talbot Ridge Condos* – 40 condominium townhouse units. Zoning amendment approved. Condominium plan currently under review by staff; not yet approved.

6. *Port of Picton Industrial Park* – Municipally owned and created Industrial subdivision. 18 vacant lots remain.

6.4 Transportation & Service Corridors

The area is adjacent to two major transportation corridors with the County: the Loyalist Parkway and Highway No. 49. In addition County Roads No. 4 and 5, classified as Arterial Connectors, run north-west to south-east through the study area. The trunk municipal water main extends along Highway No. 49 to Folkard Lane whereas the sanitary sewer terminates at the McFarland seniors residence.

The County Industrial Park is fully serviced along MacSteven Drive and MacDonald Drive connected to the Highway No. 49 municipal system along Mc Farland Drive. Portions of Paul Street and Century Drive within the study area are also fully serviced.

7. HYDROLOGY AND HYDRAULICS

Hospital Creek, a small (< 5 km2) watershed located in Prince Edward County, with headwaters in an agricultural dominated area, eventually passes thru the Picton Urban Area and finally discharges into Picton Bay (see Map 7.1).

7.1 Previous Hydrologic Modelling Studies

The "Hospital Creek Study", J.D. Paine Engineering Inc. (1986), developed floodplain mapping for Hospital Creek, recommended channel details for the re-routed portions of the creek, summarized pre-development flows and identified capacities of structures that existed within the creek at the time of the report.

In the "Hospital Creek Stormwater Management Review", J.D. Paine Engineering Inc. (1989) furthered the work completed in 1986 by applying the HYMO hydrologic event model to simulate existing conditions and post-development peak flows for six sub-basins and three channel locations throughout the watershed. Three different hydrologic events were simulated:

1. a three hour duration Chicago distribution for summer conditions based on Picton IDF data;
2. a three hour Chicago distribution for frozen ground conditions; and
3. an AES 12-h 30th percentile storm using Picton IDF data.

The report highlighted that stormwater controls would be required to prevent flooding. Flood lines were generated upstream of Johnson St. based on a peak flow of 2.7 m3/s. It was further noted that the downstream channel had a peak capacity of 5.0 m3/s; this capacity was expected to be reduced to approximately 3.5 m3/s based on proposed culvert construction.

7.2 Stormwater Management Objectives

Typically, in stormwater management, the practice is to control future condition flows to pre-development levels. In the case of Hospital Creek, this may or may not be feasible due to the substantially larger flows resulting from the lack of stormwater management measures implemented for developments constructed in the period 1989 – 2010. For the purposes of stormwater management, it is more appropriate in Hospital Creek to design to the hydraulic constraints of the system; namely the Johnson St. culvert (overly high peak flows may cause flooding upstream of this location) and the capacity of the channel between Main and Johnson streets.

To ensure the 1989 Hospital Creek floodline is not increased with future developments, the stormwater management objectives are to control peak flows to 2.7 m3/s at the Johnston Street road culvert to control peak flows to 3.5 m3/s along the channel reach between Main Street and Johnston Street. Hydraulic constraints will be met via stormwater storage facility(or facilities).

7.3 Hydrologic Model Overview

Determination of the storage required to meet the stormwater management objectives under specified rainfall inputs requires the use of a hydrologic simulation model of the event type. There are numerous candidate models of this type. XCG selected the model HEC-HMS (which was developed and is maintained by the U.S. Army Corps of Engineers) for the following reasons.

1. It is in the public domain.
2. It is used widely in Canada and the United States.
3. It is the successor to HEC-1, the first version of which was published in 1968 and subsequently extensively revised in 1973, 1981 and 1990, and as such has subjected to extensive testing by the hydrologic community.
4. It incorporates algorithms that have been published and peer reviewed in technical literature.

7.3.1 Data Requirements

Data required for modeling can be classified as meteorological data, watershed & channel data and reservoir data, as defined below.

1. Meteorological data are essentially rainfall data, which are presented in the form of "Design Storms".
2. Watershed & channel data include physiographic data (drainage area, length and slope), soils data and land use data, all on a sub-basin basis. Sub-basins are delineated in a process known as "basin discretization" - in the case of Hospital Creek, Quinte Conservation staff used GIS procedures to discretize the overall basin and determine sub-basin data. The GIS developed areas were then reviewed and modified accordingly to best represent the existing physical features.
3. Reservoir data include the locations of all reservoirs in the network and characteristics for each reservoir.

7.3.2 Three Conditions Modeled

Watershed data must be determined for three watershed conditions:

1. those existing at the time the 1989 report was prepared;
2. those that would exist under planed future development - future conditions; and
3. future conditions with stormwater management measures in place.

7.4 Design Storm

7.4.1 Depth of Rainfall

A review of Atmospheric Environmental Service"s (AES) updated "Rainfall Intensity – Duration – Frequency Values" for Picton, Ontario, station number 6156533 revealed that the 100-year intensity fit line had been identified as unreliable by AES. To ensure that the IDF results were acceptable, the Picton 2, 6 and 12 h depths for the 100-year event were compared to those at

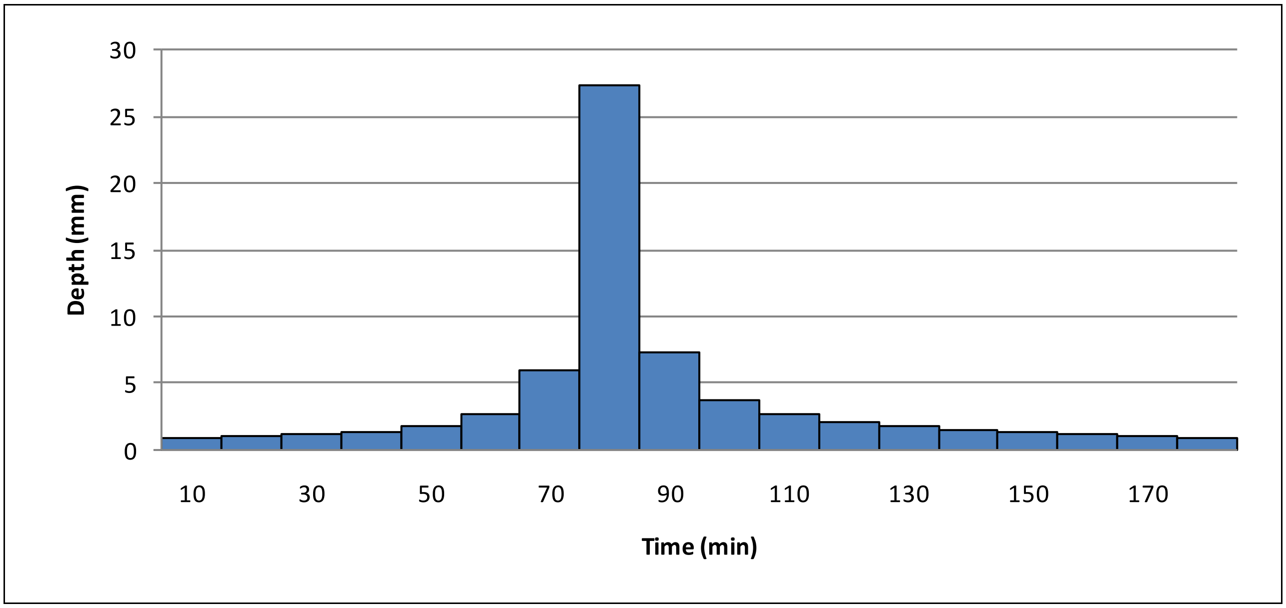
Belleville, Trenton and Kingston. Table 7.1 shows that Picton rainfall depths are larger than in Trenton and Belleville and are similar in magnitude to Kingston. This brief comparison suggests that the Picton IDF values are suitable for design in Picton. It should be further noted that the current quantiles at Picton are slightly smaller than those applied for the work completed in 1989.

Table 7.1 Design Storm Rainfall Depths

Climate Station	Period of Record	2 h depth (mm)	6 h depth (mm)	12 h depth (mm)
Picton (6156533)	1966 – 1994	66.5	83.7	97.7
Belleville (6150689)	1960 - 2003	54.9	70.5	81.5
Trenton (6158875)	1965 – 1997	52.1	68.2	74.9
Kingston (6104175)	1914 – 2003	58.0	77.9	92.3

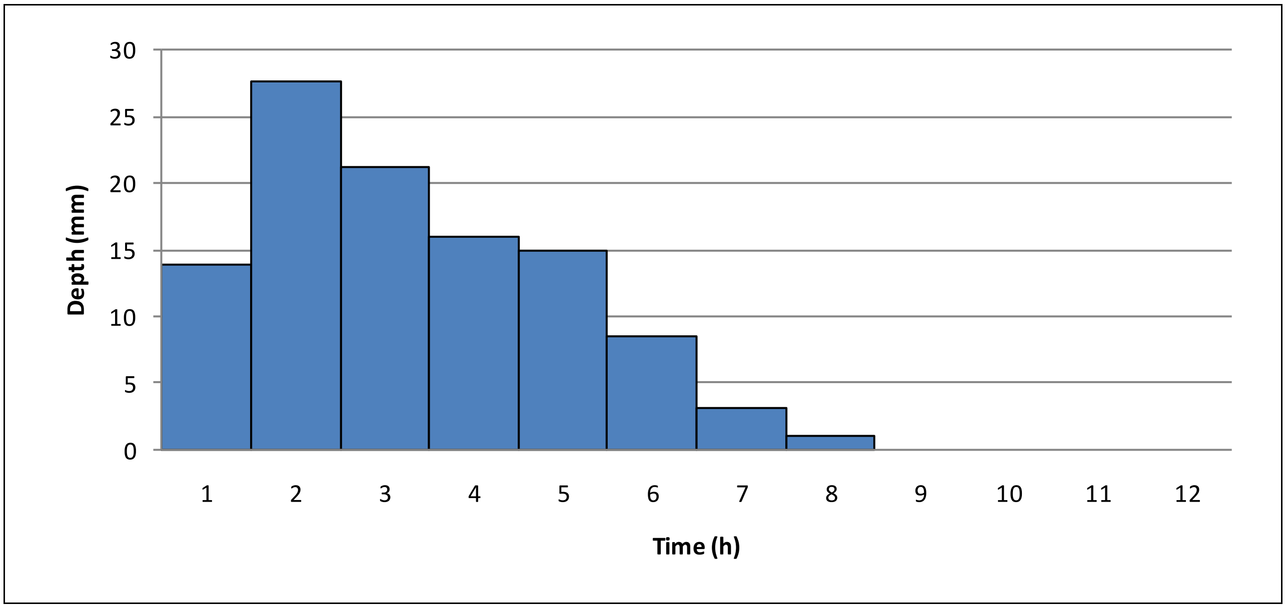
* Depths estimated by AES

FIGURE 7.1 CHICAGO 3-HOUR DESIGN STORM FOR PICTON



For the AES 12-h storm, the updated 100-year rainfall depth of 97.7 mm (see Table 7.1) was used. The AES 12-h design storm hyetograph was applied in the modelling and is shown in Figure 7.2.

FIGURE 7.2 AES 12-HOUR 30TH PERCENTILE DESIGN STORM FOR PICTON



7.5 Basin Discretization

7.5.1 General Principles

The following general principles guided the configuration of the basin into sub-basin, reservoir, channel, diversion and junction elements.

i. Sub-basin elements were provided to represent the drainage basin routing process for all sub-basins obvious on the LiDAR generated topographic map.

ii. Sub-basin elements were added when a drastic change in land use was anticipated.

iii. Reservoir elements were added when a reservoir existed or was anticipated.

iv. Channel elements were added to represent the delays in channels.

v. Junction elements were added to link sub-basin hydrographs or tributary hydrographs with each other or with the main branch.

7.5.2 Types of Elements and Parameters

Sub-basin elements: An element refers to a component of the hydrologic event model. Elements incorporate algorithms that attempt to represent flow generation and routing processes. Sub-basin elements (of area A, in km2) represent two processes:

a. abstractions (or losses) from rainfall, and
b. routing of net water input through the sub-basin

In this study, abstractions are modelled using the SCS curve number algorithm. This algorithm has one primary parameter, the curve number, CN. The values used for CN are those corresponding to antecedent moisture condition II (AMC-II), the average condition preceding annual floods. Values for CN were selected from tables developed by the US Soil Conservation Service, where CN depends on soil type and land use. The algorithm has one additional parameter, the percent of the total sub-basin area that is impervious.

Watershed routing is modelled using the SCS dimensionless unit hydrograph algorithm. This algorithm has one parameter, sub-basin lag time. An equation developed by Watt and Chow (1986) was used to estimate lag time using values for sub-basin length and slope.

Reservoir elements: Reservoir elements, which represent constructed storage or detention ponds, are modelled using the modified Muskingum method. This method requires two relations: reservoir storage-elevation relation, and outflow structure hydraulic description, the two relations are typically combined to from storage-discharge relation.

Channel elements: Channel elements, which represent the process of channel routing, are modelled using a simple lag routine. The routine does not adjust the discharge it only affects the timing. Timing was adjusted based on the length of channel and an assumed velocity of 0.8 m/s. Channel routing remained constant for all simulations.

Junction elements: The outflow from a junction is equal to the sum of all inflows to the junction.

7.6 Pre-Development (1989 Conditions) Case

The pre-development case was described in the 1989 Study by J.D. Paine Engineering Ltd. Since that time, there has been a significant amount of changes to the tail waters of Hospital Creek, specifically, the relocation of the drainage channel and the installation of numerous culvert crossings within the industrial park.

The pre to post peak flow matching methodology is typically completed to ensure that no additional flooding occurs at downstream locations. In the case of Hospital Creek, the design rainfall depth applied in the past has decreased and as a result so will the peak flows. Rather than try to re-simulate pre-development values with new rainfall inputs, it is suggested that hydraulic constraints be imposed. These constraints are 2.7 m3/s at Johnson St. and 3.5 m3/s at Main St, based on the 1989 Study.

To ensure that the new model and previous models were returning consistent results, XCG constructed a model for the 1989 conditions case. The set-up of this model is described below.

Rainfall Event: The 100-year 3-h Chicago Storm event with a total rainfall depth of 73.9 mm was applied to maintain consistency with the 1989 hydrologic modeling.

Basin discretization: The total drainage area of Hospital Creek encompasses approximately 1.8 km2. Sub-basins were defined in collaboration with Quinte Conservation staff. Contributing drainage areas to each point were calculated using GIS interpretation of LiDAR and OBM mapping and field checking and then confirmed/modified by XCG to represent actual conditions.

Drainage network: The Hospital Creek drainage basin was delineated in such a way that the peak flows for the 1989 conditions could easily be compared to the previous work completed by J.D. Paine Engineering Ltd. (1989). As such, the following groupings of sub-basins were identified:

c. sub-basins upstream of Johnson St.;
d. sub-basins between Johnson St. and Main St.; and
e. the sub-basin located downstream of Main St.

The relation between the J.D. Paine and the current discretization is shown in Table 7.2.

Table 7.2 Design Storm Rainfall Depths

Study Area	XCG (2010)	J.D. Paine (1989)
Upstream of Johnson St.	1a, 1b, 1c	1
Between Johns St. & Main St.	2345	2, 3, 4, 5
Downstream of Main St.	6	6

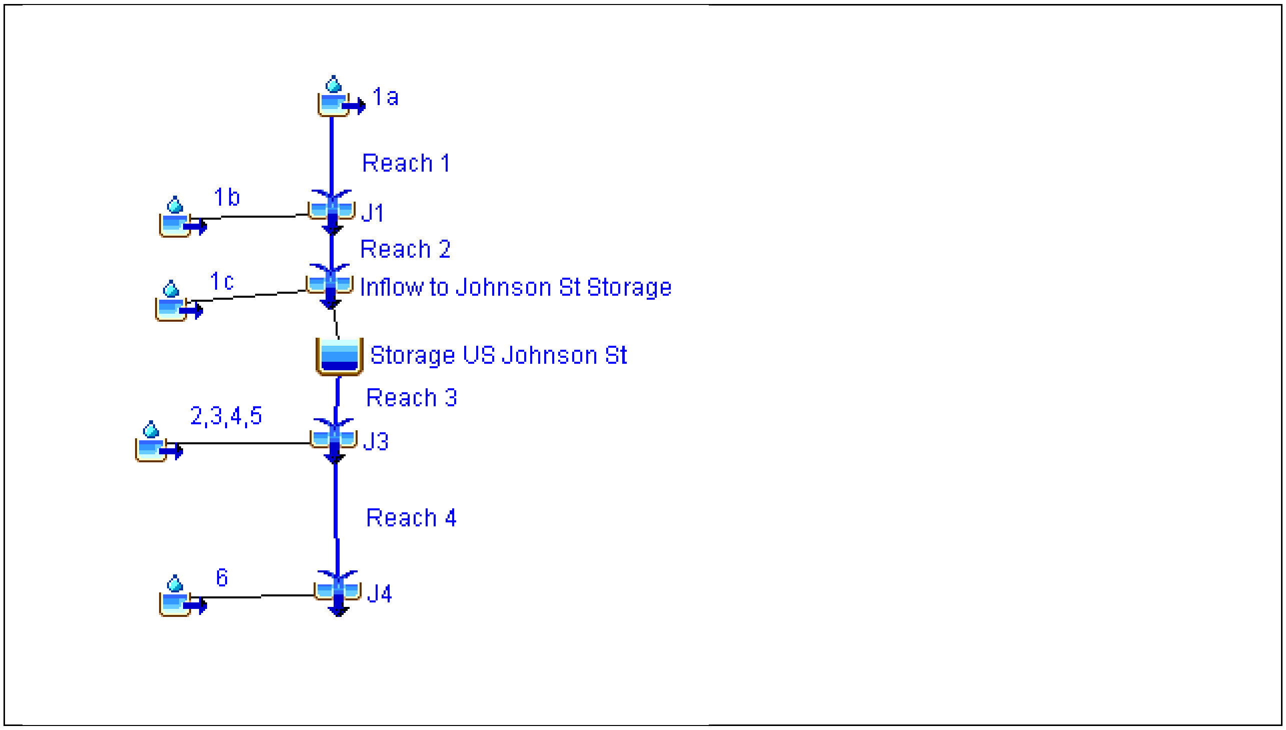
Soils: The distribution of soil types is shown in Map 7.2. Data for this figure were taken from the Soil Survey of Prince Edward County (Richards and Morwick, 1963).

Developed areas: Developed areas under pre-development (1989) and existing conditions (2010) are generally located northeast of Johnson St. towards Picton Bay with a mix of residential and industrial uses. Review of aerial photography suggests that little to no stormwater controls have been implemented in the existing developed areas.

Reservoir Parameters: The J.D. Paine (1989) report identified a natural storage area upstream of Johnston Street. A reservoir was simulated in the model using the stage-discharge relationship of the existing 1200 mm diameter corrugated steel pipe. The stage-storage relationship was developed using the LIDAR generated contours.

The model schematic for pre-development conditions is shown in Figure 7.3.

Figure 7.3 HEC-HMS Model Schematic for Pre-Development Conditions



Sub-basin Parameters: Estimates of the sub-basin parameters for pre-development conditions (area, time to peak and curve number), were determined using topographical and soils data provided by GIS procedures (see Table 7.3).

Table 7.3 Sub-basin Parameters – Pre-Development Conditions

Sub-basin	Area (sq. km)	Time to Peak (min)	SCS CN
1a	0.242	18	63
1b	0.739	21	63
1c	0.297	9	63
2345	0.791	33	69
6	0.042	18	76

Channel Parameters: The channel elements are summarized in Table 7.4.

Table 7.4 Channel Parameters

Channel	Approximate Length (m)	Lag (min)
Reach 1	900	20
Reach 2	550	12
Reach 3	1200	27
Reach 4	180	4

Peak Flows – Pre-development (1989) Conditions:

Johnson St. and Main St. are the critical locations for peak flow comparison between the 1989 report and the current study. The peak flows generated by each study under the 3-h Chicago design storm at the critical locations are summarized in Table 7.5. The table shows that there are no significant differences in the peak flows generated by the two studies.

Table 7.5 Comparison of Peak Flow Estimates for 1989 Conditions

Location	1989 Study (m^3/s)	2010 Study (m^3/s)
Johnson St.	2.6	2.6
Main St.	3.4	3.4

Therefore, the HEC-HMS model can be used to generate pre-development flows (1989 conditions). Accordingly, the HEC-HMS was applied using the updated 100-year Picton 12-h rainfall depth, distributed in time according to the AES 30th percentile distribution. Peak flows, at the outlet of each sub-basin and at selected junctions generated by the AES 12-h, 100-year rainfall input are given in Table 7.6.

Table 7.6 Sub-basin and Junction Flows – Pre-Development (1989) Conditions

Sub-basin	Peak Flow (m^3/s)	Junction Node On Hospital Creek	Peak Outflow (m^3/s)
1a	0.38	J1	1.45
1b	1.13		
1c	0.50	Inflow to Johnson St.	1.84
2345	1.63	J3	3.23
6	0.13	J4	3.32

7.7 Future Conditions Case

Considerable development has taken place in the interval between 1989 and 2010. Planned and existing development was defined by the „Picton-Hallowell – Option 3 – Growth‟ scenario excerpted from the Growth and Settlement/Servicing Strategy (Bousfield, et.al, 2003). Changes in land use are expected in all sub-basins. The degree of development varies widely from sub-basin to sub-basin.

In the case of sub-basin 2345, a substantial amount of development has already taken place since the 1989 study. However, further development is proposed within the proposed Picton Industrial Park subdivision. There are two proposed pond facilities; an on-site pond to service the Picton Hydro-One development site (776 m^3 active storage) and a centralized pond facility (2440 m^3 active storage)to service the remaining future development sites within the park (refer to Section 11 for further details on the proposed ponds). Therefore, the pre-development discretization of Sub-basin 2345 was further discretized into small sub-basins in the post-development model to reflect the location of the proposed ponds.

The boundaries for sub-basins located upstream of Johnson Street remain consistent under the post-development conditions case as in the existing conditions case. A proposed subdivision, located just upstream of Johnston Street (within Sub-basin 1c), commonly referred to as the Anderson Subdivision, is a 64-unit detached residential lot and is a registered plan of subdivision. Stormwater water quantity controls were not provided in the approved design. Furthermore, Hospital Creek was proposed to be conveyed through a 20-metre wide block. Therefore, for the future conditions case, no water quantity controls are provided for drainage generated from this site.

To meet the hydraulic constraints of the Hospital Creek system, the proposed stormwater ponds located within the Picton Industrial Park will play a critical role in meeting the hydraulic constrains of the Hospital Creek system under post-development conditions. However, there is considerable residential development proposed upstream of Johnson Street. To control runoff from these developments and to compensate for the uncontrolled runoff generated from the Anderson Subdivision, additional storage is required. As mentioned previously, there also exists natural storage upstream of the Johnson Street crossing. However, future development is proposed to be located within this natural storage area. Consequently, a loss of this natural storage in the future due to development is a possibility. Therefore, a centralized on-line pond facility is proposed at the outlet of Sub-basin 1b (refer to Section 11 for further details).

7.7.1 Sub-basin Elements, Parameters and Flows

Channel elements in the pre-development conditions model were retained with no change in parameter values.

Sub-basin parameters: The impact of changes in land use was modelled by changes in sub-basin parameter values for relevant sub-basins. The parameters were adjusted based on the following conditions.

- **i.** Site specific stormwater management plans were incorporated at face value (located in 2345). As mentioned previously, Sub-basin 2345 was further disrcretized based on the location of proposed ponds.
- **ii.** For areas (1a, 1b, 1c and 6) not covered by existing or proposed stormwater management facilities, the abstraction sub model was revised to include a value for impervious area. Time to peak for these areas was not adjusted as the times were already sufficiently short.

Sub-basin parameter values are given in Table 7.7.

Table 7.7 Sub-basin Parameters – Future Conditions

Sub-basin	Area (sq. km)	Tributary to Pond(s)	Time to Peak[1] (min)	% Impervious (%)	SCS CN
1a	0.242	On-Line Pond	18	1.6	63
1b	0.739		21	19	63
1c	0.297	Johnson Street Natural Storage	9	37	63
Hydro-One	0.027	Hydro One Pond/Industrial Park Pond	n/a	50	78(7)
Picton Industrial Park (Controlled Runoff)	0.247	Industrial Park Pond	n/a	55	70
2345	0.519	no	33	26	62
6	0.042	no	18	51	76

1. Represents time to peak value of inflow hydrograph to pond (where applicable)
( ) Initial abstraction when not equal to 0.2S

The percentage impervious for each Sub-basin noted in Table 7.7 is a weighted-average value based on the area. For example, the future and proposed development areas of Sub-basin 1B have an assumed impervious level of 50% but since a large portion of the sub-basin (the area west of Millennium Trail) does not have planned (at this time) future development, the overall impervious level of the sub-basin is much lower than 50%.

Reservoir parameters: For the site specific stormwater management plans, the storage-discharge information was incorporated into the model.

7.8 Future Conditions with Stormwater Management Case

As mentioned previously, stormwater management ponds are proposed to meet the hydraulic constraints under full-build out conditions. In addition to the two proposed ponds within the Picton Industrial park, further details are provided for the proposed storage systems upstream of Johnson Street.

Because of the uncertainty in the longevity of the Johnson Street natural storage, three scenarios were examined. The first scenario incorporates the attenuation of both the Johnson Street natural storage and the proposed Sub-basin 1b pond. The second scenario only incorporates the Sub-basin 1b pond in the hydrologic modeling. The proposed centralized pond is located just upstream of Johnson Street in the third scenario.

Under the first scenario, approximately 3,000 m^3 of storage is required at the Johnson Street culvert and 13,000 m^3 of storage is required at the proposed centralized pond at the outlet of Sub-basin 1b. The storage required of the Sub-basin 1b facility increases to 16,500 m^3 under the second scenario and 18,400 m^3 under the third scenario. In all three scenarios, the centralized pond facility is designed to over-control the peak flow generated from the upstream lands.

7.8.1 Critical Junction Flows

Flows at two critical junctions, upstream of Johnson St. and upstream of Main St., are provided in Table 7.8. It is noted that the two afore-mentioned storage scenarios result in the post-development peak flows listed in Table 7.8.

Table 7.8 Comparison of Critical Junction Flows

Location	Pre-development Peak Flow (m^3/s)	Post-development Peak Flow (m^3/s)	Hydraulic Capacity of Location (m^3/s)
Johnson St.	1.8	1.8	2.7
Main St.	3.2	3.4	3.5*

* Estimated by J.D. Paine (1989) based on planned developments (post 1989)

8. SOILS AND GROUNDWATER

The urban development of a rural watershed typically results in an increase in impervious area and a corresponding decrease in recharge to the underlying aquifers. This decrease in recharge can result in impact to users of groundwater such as properties serviced by private wells and nearby creeks which rely on discharge of groundwater to maintain baseflow during periods of low runoff. Urbanization can also result in the introduction of contaminants to an area, potentially leading to contamination of the underlying groundwater resource. Such contaminants could include runoff from roads, deicing agents, fertilizers, pesticides etc. To protect vulnerable groundwater resources, proper development of a watershed requires careful consideration of the hydrogeologic conditions. Efforts are required to maintain the natural hydrologic balance of a developing area through proper watershed planning and implementation of effective storm water management solutions. Such measures are developed in consideration of the physical and ecological characteristics of the watershed to consider sensitive areas and possibly implement effective storm water management measures to promote groundwater recharge and manage sources of potential contamination.

To assist with the development of a suitable master drainage plan for the Hospital Creek Study Area, a summary of hydrogeologic conditions has been completed. This summary is provided through an overview of the geology (bedrock and overburden) together with the hydrogeology of the area as determined through a review of a variety of sources of information such as the Quinte Regional Groundwater Study (Dillon Consulting, October, 2004) and Ontario Water Well Records.

8.1 Geology

The landscape of the Hospital Creek watershed has been formed in direct relation to the geology of this area. This geology serves as the foundation for the area and provides control of the various hydrogeologic processes which occur in a watershed. For this summary, the geology will be divided into two distinct classifications; first the bedrock geology and the second the overburden geology or soils.

8.1.1 Bedrock Geology

The Hospital Creek watershed is located within the physiographic region referred to as the Prince Edward Peninsula which is comprised of thin soil over limestone bedrock. Within the study area, the bedrock found directly beneath the soil is comprised of limestone of the Lindsay Formation (upper and lower members) as well as a small area of the Verulam formation along Picton Bay. The distribution of these two rock types is illustrated by Map 8.1, with the younger Lindsay formation covering the majority of the study area. This formation is described as crystalline limestone interbedded with shale and the Verulam formation is described as finely crystalline limestone of pale to medium brown color. The presence of the two rock types is largely attributed to a fault zone which vertically displaces the bedrock. This fault is observed by the bedrock escarpment following the west shoreline of Picton Bay with elevation difference of approximately 15 metres above the Bay. Other then this vertical displacement, the bedrock is predominantly flat lying with slope towards Picton Bay at the east as illustrated by Map 8.2 with bedrock surface elevations ranging from 93 to 85 metres.

The limestone found in this area was formed as a result of the deposition of sediment on the floor of the Ocean which covered this area approximately 400 to 500 million years ago. Directly beneath the limestone is the billion year old Precambrian bedrock which forms the core of the North American continent. The thickness of the limestone above the Precambrian basement is variable but estimates are that it ranges up to 90 metres.

8.1.2 Overburden Geology

The soils of the study area that comprise the deeper sub-surface layer (say 0.30 to 0.45 m below the ground surface) are as mapped by Map 8.3 showing the majority of the area to be underlain by Sandy Silt and Till soils with a small area of shallow soil adjacent to Picton Bay. Some of the soils of this area (western portion) are associated with a drumlin formation oriented in an east west direction with the top forming a topographic divide and western boundary of the study area. Soil depth as illustrated by Map 8.4, is variable and extends from a minimum of approximately 1.5 metres near the shore of Picton Bay up to a maximum of 40 metres at the crest of the drumlin formation along the western boundary. Surface soils, which represent the surface or near surface soils (refer to Map 7.2), are mapped as predominantly comprising loam and sandy loam of good natural drainage. The records for 7 water wells drilled within the area, at the locations illustrated by Map 8.5, indicate soils as comprising clay and gravel at depths ranging from 2.4 to 19.5 metres. Although there are only records for 7 wells within this area, the well record data generally confirms mapping of soil depth as thickening from east to west. The topsoil layer, above the various subsoil groups, is classified as loam with low to high permeability.

8.2 Groundwater

The Rural residences within the watershed rely on private wells as a source of domestic water supply. The records for these wells provide valuable information about the underlying aquifers and have been reviewed as a part of this study. To obtain this information, a review was completed of the records for 7 wells located within the watershed boundary (see Map 8.5 for location of wells). From these records, it would appear that residents obtain supply from a limestone bedrock aquifer. Water is typically found at depths of 13 to 20 metres in the limestone aquifer. The yields of the wells are good and more than suitable for domestic needs with flow rates ranging from 1 to 30 gallons per minute. Additional information taken from the well records indicated the depth to the water table as ranging from 3.6 to 10 metres below ground with water table elevation in the order of 90 to 104 metres. Maps of water table depth and elevation are illustrated by Maps 8.6 and 8.7, respectively. The depth to water map indicates the depth to the water table as ranging up to 20 metres with greatest depths along the watershed boundary and least along the Hospital Creek water course. This information suggests potential for areas of groundwater recharge along the watershed boundaries and discharge along the Creek and at Picton Bay. The map of water table elevation, as taken from the Quinte Regional Groundwater Study (Dillon Consulting, 2004) shows the water table at highest elevation along the west boundary and decreasing to the east towards Picton Bay indicating groundwater flow similar to surface topography.

8.3 Overview

From the review of hydrogeological information about the Hospital Creek Study Area, the following conclusions can be made:

- The area is underlain by sandy silt and till soils of good drainage,
- Soils range in thickness from a minimum of 1.5 metres at the east along Picton Bay to a maximum of 40 metres along the western boundary of the watershed,
- Water wells obtain supply from a limestone aquifer with water bearing zones at depths of 13 to 20 metres below grade,
- The water table generally follows the topography with areas of recharge in topographically high areas at the west and discharge along the Hospital Creek and at the east in the vicinity of Picton Bay,
- Well records indicate the depth to the water table ranges from approximately 3.6 to 10 metres below ground.

From this overview of the hydrogeologic setting, the area can be described as being underlain by moderately permeable soils of variable depth above the water table and bedrock. Some areas may present good potential for the artificial infiltration of storm water, given the soil conditions and clearance from the water table. The most suitable area for infiltration would be along the western boundary; however, development is not proposed in this area. The soil and water table conditions may enable the use of infiltration of stormwater generated from the proposed developments (refer to Section 11 for more detail). It is also noted that based on the interpretation of groundwater movement, potential exists for the migration of contaminants with the groundwater to areas of discharge along the surface water courses.

8.4 Recommendations

To mitigate the potential impacts of urban development on the study area, various methods of control exist to promote maintenance of the hydrologic cycle through promotion of ground water recharge. Lot level controls and infiltration end-of-pipe facilities are encouraged where feasible. Refer to Section 11 for further discussion.

9. ECOLOGY

The land cover within the study area was characterized by roadside field reconnaissance (July, 2009), and the use of printed digital infrared air photo images (provided by Ontario Ministry of Natural Resources, collected in 1999). Lines defining the polygons were drawn on the printed images in the field, and were later „heads up‟ digitized on the computer screen using the „Lidar – First Surface‟ images (collected April, 2008). Digital base layers provided by the OMNR, and „ArcGIS v. 9x‟ geographic information system software were also utilized to depict the various landuses within the study area.

As per Map 9.1, land covers were divided into five major categories. „Agricultural‟ lands included those lands being actively worked for agriculture (dominated by soy beans, corn, and hay crops). The „Developed‟ lands included those previously or actively being developed for housing, commercial, and industrial uses. The remaining plant communities were described using the community series descriptions from the Ecological Land Classification System for Southern Ontario (1998).The wetland boundaries identified through this field work are illustrated on all maps, where wetlands are shown.

9.1 Natural Areas

No significant natural areas or species at risk have been noted within the Hospital Creek watershed. (NHIC, 2010)

9.2 Vegetation Communities

The original forest cover of the watershed would be broadly described as a shade tolerant hardwood type (Prince Edward Region Conservation Report, 1968). Given the moderating affects of winter temperatures from the close proximity of the Great Lakes and local soil types, it is expected that the coniferous components of the original forest were minimal. The forest cover of the watershed may have been cleared in earnest during the settlement of the Loyalists (circa 1770). The vegetation within the study area is dominated by abandoned fields and small patches of secondary growth, immature forests – primarily dominated by Red Cedar, White Pine, Poplar, Bur Oak, and Green Ash.

9.3 Fisheries

The Hospital Creek system can be considered as a „warm water‟ fishery, with Fat Head Minnows being the only species caught during the field work conducted as part of this study (during the field season of 2009). These fish species are tolerant of warm water temperatures (Coker, et. al.; 2001). Based on mapping provided by Fisheries and Oceans Canada, no species at risk are present within the watershed (Doolittle, et. al., 2007).

9.4 Wildlife

According to the Canada Land Inventory – Land Capability for Waterfowl, the watershed has such severe limitations to the production of waterfowl that almost no waterfowl are produced. In addition, the Canada Land Inventory – Land Capability for Ungulates indicates that the watershed has moderate limitations for the production of ungulate species. Given the present agricultural and urban land uses, no species of concern are expected within the area.

10. WATER QUALITY CONDITIONS

10.1 Historic Water Quality Data

There are no historic water quality data available for the watershed.

10.2 Benthic Investigation

Hospital Creek is part of the Ontario Benthos Biomonitoring Network with benthic macroinvertebrate communities collected at one monitoring site in 2008 and 2009 once per year in spring. Samples from each sample date were assessed for community health using a weighed average with tolerance values called the Hilsenhoff Biotic Index, %EPT representing sensitive organisms, and community Richness. The location was considered possibly impaired do to a moderate Hilsenhoff Biotic Index with low %EPT in 2009 but good community Richness in both samples. Continued monitoring over a five year period is required to make a definitive assessment.

10.3 Provincial Water Quality Monitoring Network

Water samples were collected at one location in Hospital Creek between 2004 and 2008 as part of the Provincial Water Quality Monitoring Network. Twelve parameters for nutrients and general chemistry were reviewed showing good water quality condition with some elevated levels. Total Phosphorous had 28% of 18 samples that were greater than the Provincial Water Quality Objective (PWQO) for rivers. E.coli had 29% of 17 samples that were greater than the PWQO. Turbidity had 6% of 16 samples that were greater than the Aesthetic Objective of the Ontario Drinking Water Standards used by municipal water treatment plants. Both E.coli and Chloride have increasing trends over time (2004 to 2007) which may indicate a decline in water quality condition.

10.4 Water Temperature/Base Flow Measurements

For the purposes of this study, one water temperature monitoring site was established to collect water temperatures throughout one growing season. The average temperature was recorded at 21.93 C for the site. These average temperatures would be indicative of a „cool water" system which is influenced by groundwater discharge.

11. STORMWATER MANAGEMENT

As development proceeds in the study area, it must do so in concert with management of stormwater runoff to mitigate water quantity and quality impacts.

11.1 Policy

- All development shall be accompanied by measures intended to minimize the degradation of water quality that are consistent with the Ontario Stormwater Management Planning and Design Manual (MOE, 2003).

11.1.1 Water Quality Control

- Lot level controls and measures are encouraged.
- The level of water quality control for facilities shall be the ***Enhanced Protection***, which corresponds to the long term removal of 80% of suspended solids.

11.1.2 Water Quantity Control

- Stormwater generated from proposed developments must be controlled such that the 100-year peak flow rate of Hospital Creek is at or below 2.7 m^3/s and 3.5 m^3/s at Johnson Street and Main Street, respectively.

11.2 Selection of Stormwater Management Options

The onus is on the developer to use and extend the information provided at draft plan submission or formal approval stage to:

- analyze and confirm appropriate SWM treatments, location, land requirements and ownership, pond configuration and geometry, and
- be compatible with guidelines in the Hospital Creek Master Drainage Plan, relevant provincial and local polices and planning requirements.

Of the three general categories of SWM treatments – lot level control, conveyance facilities and end-of-pipe facilities – it is essential that all levels of government, environmental agencies plus the development community and the public maximize source controls as an important first-line and cost-effective upstream measure in the chain of more expensive downstream SWM conveyance and end-of-pipe treatment facilities.

Many options are available to reduce the volume of storm runoff at the source and promote groundwater recharge for protection of the natural environment. These measures are generally suitable at the lot level where they may be relatively contaminant-free runoff. Such potential measures are listed below. Roads and parking lots are excluded due to high suspended solids content which could plug such systems and the presence of other potential contaminants which could impair ground water quality.

- Reduced lot grading,
- Directing roof leaders to ponding areas,
- Directing roof leaders to soak away pits,
- Use of rain barrels
- Infiltration trenches,
- Pervious pipes,
- Grassed swales and vegetated filter strips, and
- Pervious catch basins.

The use of lot level controls for infiltration of storm water may be considered at the planning stage for the development of individual properties as they occur. Confirmation on the potential of a property to incorporate such measures would require both geotechnical and engineering assessment prior to development. It is recommended that such measures be made a condition of all draft plan approvals for future development such that they may be evaluated and implemented where feasible. A program to educate the public and homeowners about the need, use and maintenance of lot level controls would be of benefit and compliment such a program. General education about storm water and contaminants in it would also be of benefit to the local watershed

11.2.1 Developments South of Johnson Street

Residential development is proposed south of Johnson Street within the Hospital Creek watershed. Due to the nature of the development, a centralized on-line pond is proposed, which will provide water quantity control for runoff generated from any proposed developments upstream of its location. As mentioned previously, the pond could be located in Sub-basin 1B or just upstream of Johnson Street. Since the pond will be on-line, the function of the pond will be limited to water quantity control only. This is to avoid contaminated stormwater generated from proposed developments entering the creek, adversely impacting the water quality of the creek.

On-site infiltration basins may be feasible for developments as a means of reducing the amount of runoff tributary to the centralized facility and increasing the amount of water recharging the groundwater system.

For the developments south of Johnson Street, the overburden thickness (Map 8.4) is between 4 and 10 meters, the depth of the water table varies between near surface (adjacent to Hospital Creek) to approximately 15 meters (Map 8.6), and the sub-surface soil is nearshore (sand and till), which has a moderate permeability. Due to the variation in the depth of the water table and to confirm the permeability of the sub-surface soils, site specific data is required to determine if infiltration as an end-of-pipe stormwater management facility is feasible. The feasibility of infiltration can be based on the three noted constraints below.

a. According to Schueler (1987), “infiltration BMPs cannot be applied on sites with soils that have infiltration rates (f_c) less than 0.27 inches/hour as defined by the least permeable layer in the soil profile. This excludes most “C” and “D” soils which cannot exfiltrate enough water through the subsoil.”
b. Schueler (1987), also points out that “a close bedrock layer prevents an infiltration basin from draining properly. Therefore, if the bedrock layer extends to within 2 to 4 feet of the bottom of an infiltration BMP, the site is not feasible”.
c. The seasonally high water table should be greater than 1 metre below the bottom of the infiltration basin per MOE, 2003.

It is recommended that for developments that employ an infiltration basin(s), pre-treatment of the stormwater occur prior to outletting to the infiltration basin to avoid potential groundwater contamination and/or clogging of the basin substrate.

Water quality controls to a Level 1 protection criteria will need to be provided for each development. The municipality discourages the use of oil-grit separators as a water quality control device. Therefore, end-of-pipe SWM Facilities such as wetlands, hybrid pond/wetlands, and wet ponds can be used to provide Level 1 water quality treatment, according to Table 3.2 of MOE (2003).

There are two alternative pond configurations to enable developments to meet the water quality control requirements.

1. Ponds can be constructed for each development. Each pond would then outlet to Hospital Creek (and the centralized water quantity pond for developments upstream or adjacent to the pond).
2. To reduce the number of ponds and land footprint, a centralized water quality control pond could be constructed. However, since untreated stormwater cannot be outletted to the Creek and the on-line pond is intended to only provide quantity control, this alternative would require the construction of a conveyance system running parallel with the Creek that would outlet to the water quality pond, which would then outlet to the on-line water quantity control pond.

The J.D Paine (1989) report indicated that the soils along Hospital Creek are relatively non-cohesive. With the absence of vegetation or other protection, channel banks will be prone to erosion with flow velocities as low as 0.5 m/s. Therefore, the release of uncontrolled flows from proposed developments upstream of the pond has the potential to cause excessive erosion along the channel. It is recommended that the municipality undertake erosion control studies to determine the type of revetment required to mitigate erosive action along the channel.

11.2.2 Developments North of Johnson Street

The majority of the proposed development north of Johnson Street is within the Picton Industrial Park. As previously mentioned, the undeveloped lots will drain to a centralized wetpond providing both water quantity and quality control save and except the Hydro-One development, which has on-site stormwater management controls proposed. Therefore, no further stormwater management facilities are recommended under this MDP.

Stormwater controls for small in-fill developments outside the Picton Industrial Park area will need to be addressed on a site-by-site basis. Water quality controls will need to be provided for these developments. Water quantity controls are not required from the perspective of controlling the peak flow of Hospital Creek but may need to be provided depending on the conveyance capacity of the receiving drainage system.

11.3 Water Quantity Control Impervious Level Limits

In general, water quantity control requirements are heavily influenced by the impervious level. The sizes of the proposed ponds upstream and downstream of Johnson Street are based on the assigned full build-out impervious levels per sub-basin (as per Table 7.7). The impervious levels were based on the future growth land use designations per the Picton-Hallowell – Option 3 – Growth Scenario (Bousfield, et. al, 2003) and existing development. Future Residential development, which is proposed in Sub-basin 1B and 1C, has an assumed impervious level of 50%. Future development within the Picton Industrial Park has an assumed impervious level of 55% while future development in Sub-basin 2345 has an assumed impervious level of 80%. The future development within the Picton Industrial Park and Sub-basin 2345 were classified as Employment per the Growth Scenario study.

It is noted that a proposed development that exceeds the assumed impervious level could impact the effectiveness of the proposed water quantity control system. For example, a proposed residential development with an impervious level of 60% within Sub-basin 1B may cause the proposed Sub-basin centralized facility to not provide sufficient water quantity control to prevent downstream flooding. Therefore, a pre-consultation meeting with Quinte Conservation is recommended to determine if additional on-site water quantity control is required.

12. MASTER DRAINAGE PLAN IMPLEMENTATION

12.1 Introduction

Hospital Creek has hydraulic constraints between Johnston Street and Main Street that impose water quantity control requirements on future developments. To enable future development to take place without adverse flooding impacts, this Master Drainage Plan has recommended that an on-line pond be constructed at the outlet of sub-basin 1b. The storage volume required varies between 13,000 m^3 and 18, 400 m^3 depending on the extent of natural storage intact just upstream of Johnson Street or exact pond location under the full build-out condition. The pond storage volume can be reduced if the municipality undertakes:

- A detailed hydraulic assessment for the reach of Hospital Creek between Main and Johnson Streets that demonstrates that the actual capacity of the creek through the aforementioned reach is greater than 3.5 m^3/s and/or
- Channel improvements are carried out to increase the channel capacity.

The proposed centralized wet pond within the Picton Industrial Park is also a critical component of the storage system proposed by this MDP. The MDP has incorporated the design of the wet pond design into the entire water quantity control system proposed.

12.2 Centralized Pond Considerations

12.2.1 Construction Timing

It is recommended that the Sub-basin 1B centralized pond be constructed during the initial development of sites upstream of Johnson Street that are tributary to Hospital Creek.

12.2.2 Cost Considerations

A preliminary conservative cost estimate for the centralized pond would be approximately $715,000 plus land acquisition, engineering and contingency costs. This estimate is based on findings indicated in the 2007 Potter Creek Master Drainage Plan (XCG Consultants) pond construction costs in Toronto, which range from $50 to $60 per m^3 of design storage volume for ponds with total design storage volume in the range of 6,000 to 10,000 m^3 (unit costs for larger ponds are expected to be somewhat lower). This price range incorporates the costs pertaining to all construction items including excavation, erosion control, outlet control structure, final grading and landscaping but does not include any land acquisition costs, engineering and contingency costs.

Since there are multiple future proposed development sites tributary to the centralized pond, there will be a need for a system to apportion costs. The following cost sharing strategy is based on the premise that the County will fund and construct the centralized SWM facility up front and then re-coup the expense as development proceeds. This is the methodology Quinte Conservation recommends based on our experiences with other proposed and/or constructed centralized SWM facilities in other municipalities.

The premise of the cost-sharing strategy is that each developer who is proposing a development located within Sub-basin 1B or 1C (between Talbot Street and Johnson Street) will contribute a fee to the County. This fee will be based on the formula provided below, which is a function of the land size of the development in question as it relates to the total anticipated development area. Please note that this cost-sharing strategy is only a recommendation and can be modified in the future if required.

Developer"s Cost:

$$=\left(\frac{Site\ Development\ Area\ (ha)}{Total\ Development\ Area\ (ha)(1)} X Total\ Cost\right) X \left(\frac{Storage\ A\ Volume\ (ha.m)(2)}{Total\ Volume\ (ha.m)(3)} + \frac{Storage\ B\ Volume\ (ha.m)(4)}{Total\ Volume\ (ha.m)}\right)$$

$$=\left(\frac{Site\ Development\ Area\ (ha)}{Total\ Development\ Area\ (ha)(1)} X Total\ Cost\right)$$

Notes: (1) Total Development Area= 36.3 ha
(2) Storage A Volume= 0.38 ha.m
(3) Total Volume= 1.63 ha.m
(4) Storage B Volume=1.25 ha.m

To meet the hydraulic constraints along the Johnson Street to Main Street reach of Hospital Creek, runoff generated from the portion of the watershed upstream of Johnson Street needs to be over-controlled to compensate for the uncontrolled runoff from existing development downstream of Johnson Street. The storage „A" volume represents the volume required to meet the hydraulic constraints and is independent of any development upstream of Johnson Street. The storage „A" volume was determined by running the hydrologic model under a scenario whereby the sub-basins located downstream of Johnson Street are at full build-out conditions (with water quantity controls in place) while the sub-basins upstream of Johnson Street are in an un-developed condition.

The storage „B" volume is the portion of the total pond volume that is related directly to the proposed developments upstream of Johnson Street. The total future development area was determined to be 36.3 ha within Sub-basin 1B and 1C. The impervious level for each development was assumed to be 50%.

A potential issue with the funding formula provided is that if the impervious level of the proposed developments is below the assumed impervious level of 50%, the pond may have been over-sized. The following table demonstrates the relationship between the impervious level for developments located within Sub-basin 1B and 1C and total storage required (A+B). Given that the impervious level for typical residential developments would be between 35% and 50%, the decrease in storage required is considered negligible. The freeboard provided by the pond would just increase as the impervious level decreased.

Table 12.1 Impervious Level and Pond Total Storage

Impervious Level (%)	Total Storage Required (ha.m)	% Change
50	1.63	n/a
40	1.48	-9.2
30	1.14	-30.0%

12.2.3 Maintenance Strategies

- Contributing factors
  - Pond Design: Maintenance costs can be reduced with a properly designed pond (i.e. sediment forebay inlet, length to width ratios, vegetative buffers, etc.)
  - Rate of Development: Complete development of the contributing drainage area to the pond presents a challenge in terms of maintenance. There may be a need for a system to apportion costs (for the first clean-out) for the case where different parts of a sub-basin may be developed at different times or where different companies develop at the same time.
  - Sediment Control: High level of erosion and sediment control during construction is essential for reducing the time before the first cleanout is required.
  - Municipal budget: Experience has shown that access for the municipal capital budget is particularly challenging and hence an ongoing operation budget is generally preferable.
- Suggested maintenance strategy
  - Operating budget: Because of the difficulty in securing funds through the capital budget, it is recommended that pond maintenance be added to the annual maintenance budget.
  - Linkages to road maintenance: Because of the somewhat mutually exclusive goals of maximizing winter road safety and minimizing pond maintenance costs, it is recommended one department (the department responsible for roads and streets) be given responsibility of determining the optimum allocation and cost.

12.3 Prince Edward County Implementation Requirements

12.3.1 General Recommendations

Recommendation 1 - The County will adopt this Master Drainage Plan report as a companion document to the Official Plan, to provide SWM and other water resources engineering and planning guidance to ongoing Picton Urban Area planning and development review for Hospital Creek.

Recommendation 2 - Floodplains should be developed for the entire Hospital Creek development area and appropriate zoning bylaws applied.

Recommendation 3 - The County, in conjunction with Quinte Conservation, should explore incentives and support for developers and private landowners to increase the use of lot level controls. For example, roof runoff rain barrel subsidies, assistance with infiltration education and treatment design.

Recommendation 4 - The County should draft, adopt and implement a site alteration bylaw. Such a measure would insure that grade alterations are not conducted prior to the approval of a grading/site plan.

Recommendation 5 - Geotechnical analysis and evaluation of infiltration potential by the developer is a required condition of all draft plan approvals for future development, with infiltration measures maximized and implemented as part of the SWM plan insofar as feasible.

12.3.2 Cost Recommendations

Recommendation 6 – For developments where the drainage generated from the site is tributary to the proposed Sub-basin 1B centralized SWM facility, the County will work with Quinte Conservation to develop an appropriate funding formula - a cost allocation method is preferred over a cash-in-lieu approach. A recommended cost-sharing strategy has been provided in Section 12.

Recommendation 7 - The County will work with Quinte Conservation to estimate SWM capital costs and long-term maintenance costs and to establish a cost apportionment schedule.

12.4 Quinte Conservation Implementation Requirements

Recommendation 8 - Quinte Conservation will maintain and update the HEC-HMS model in the future.

12.5 Developer Implementation Requirements

12.5.1 Submission Requirements

Recommendation 9 - Developers are responsible to follow the SWM guidelines in this MDP report as approved in accordance with accepted MOE stormwater management planning and design in preparing development application submissions to the County and Quinte Conservation. The developer must:

1. satisfy water quality control requirements (Level 1 water quality control protection criteria),
2. meet the MOE design guidelines, and,
3. follow the water quantity control guidelines (refer to Section 7.8).

Recommendation 10 –Proposed developments located within the Hospital Creek watershed that exceed the stated impervious level limits (refer to Table 11.2) will require additional study to determine the appropriate on-site water quantity control volumes to be used. It is foreseen that the proponent"s consultant would use the HEC-HMS model to determine the appropriate level of water quantity control. Quinte Conservation staff would review the consultant"s recommendations.

Recommendation 11 –Developers must demonstrate to the County and Quinte Conservation that the proposed development will not cause adverse flooding impacts to adjacent property owners along Hospital Creek.

Recommendation 12 - Developers are required to discuss SWM options in general with the County and Quinte Conservation prior to undertaking detailed analysis, modeling (if necessary) and design.

12.5.2 SWM Pond Requirements

Recommendation 13 - Developers must ultimately be charged for procuring SWM sites, be responsible for designing and developing the approved treatment facilities and for ongoing maintenance and performance monitoring cost until facilities are assumed by the County.

Recommendation 14 – Stormwater pond design should consider both immediate and long-term planned development storage requirements.

Recommendation 15 - The developer(s) should be required to pay for water quality sampling for a two-year period after the facility is constructed to ensure the facility is functioning within design parameters and if not, undertake appropriate re-design and reconstruction and pay for all associated costs.

Recommendation 16 - The developer must plant the periphery of the SWM ponds with 50 mm caliper (ball & burlap) native tree stock to promote shading.

13. References

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Map 1.1
Study Area

Legend

- Study Area
- Major Roads
- Minor Roads
- Railways
- Waterbodies
- Wetlands
- Watercourses

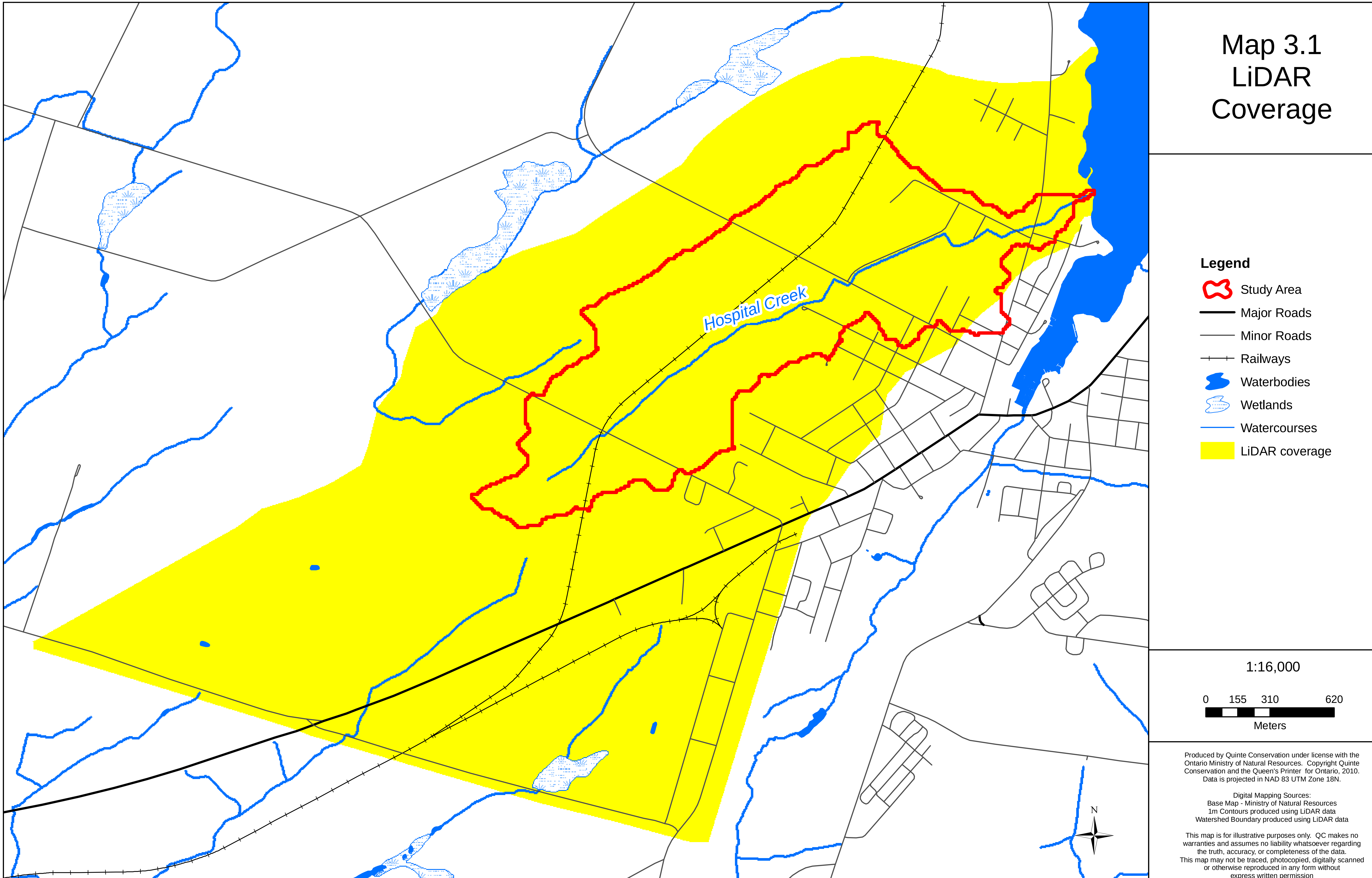
1:10,000

0 100 200 400

Meters

Data is projected in NAD 83 UTM Zone 18N.

Digital Mapping Sources:
Base Map - Ministry of Natural Resources
1m Contours produced using LiDAR data
Watershed Boundary produced using LiDAR data


Map 3.1
LiDAR
Coverage
Hospital Creek
Legend
Study Area
Major Roads
Minor Roads
Railways
Waterbodies
Wetlands
Watercourses
LiDAR coverage
1:16,000
0 155 310 620
Meters
N
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Base Map - Ministry of Natural Resources
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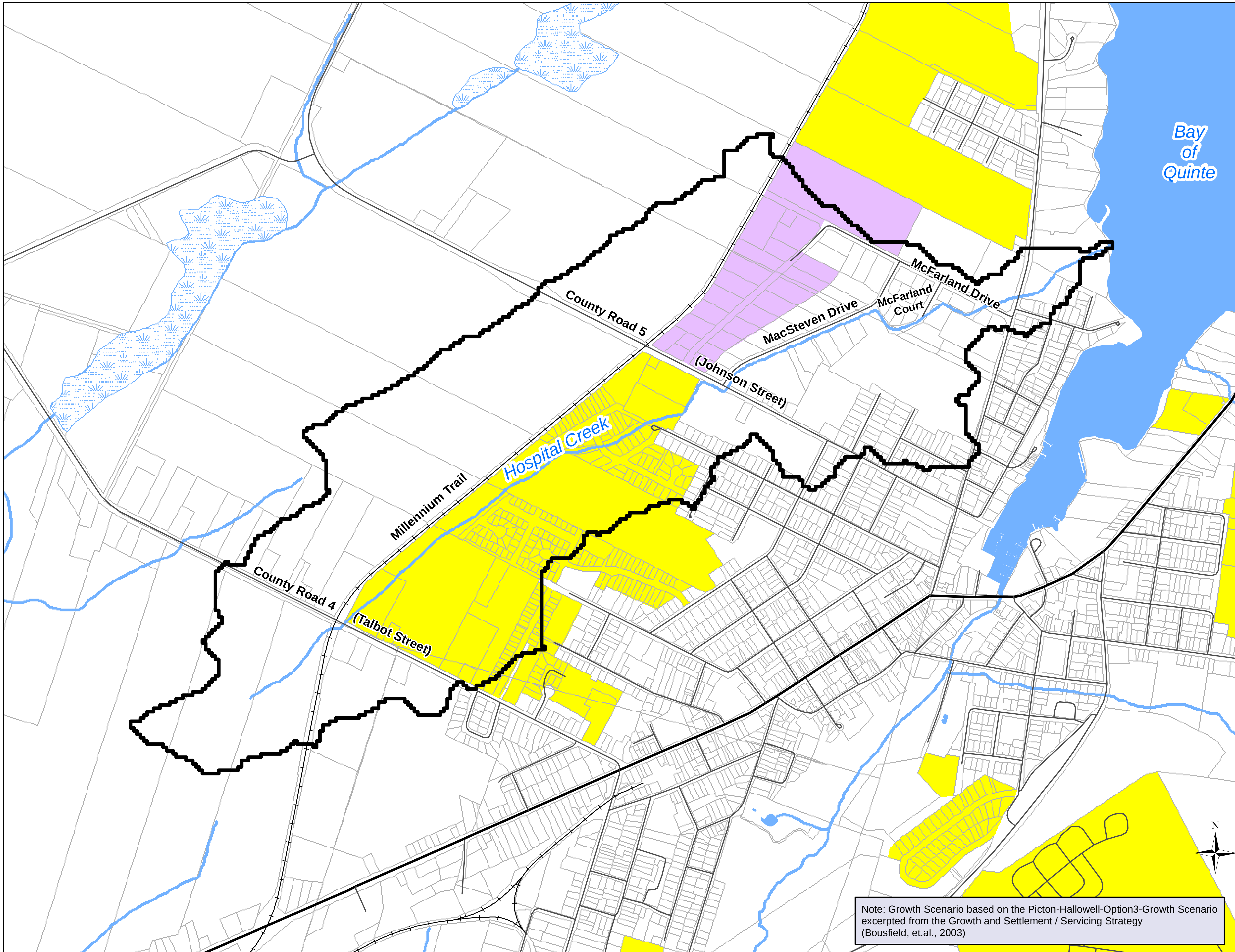
Map 6.1 Future Land Use: Full Build-Out Growth Scenerio



Legend

- Study Area
- Major Roads
- Minor Roads
- Railways
- Waterbodies
- Wetlands
- Watercourses
- Residential
- Employment

1:11,000

0 125 250 500

Meters

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Digital Mapping Sources:
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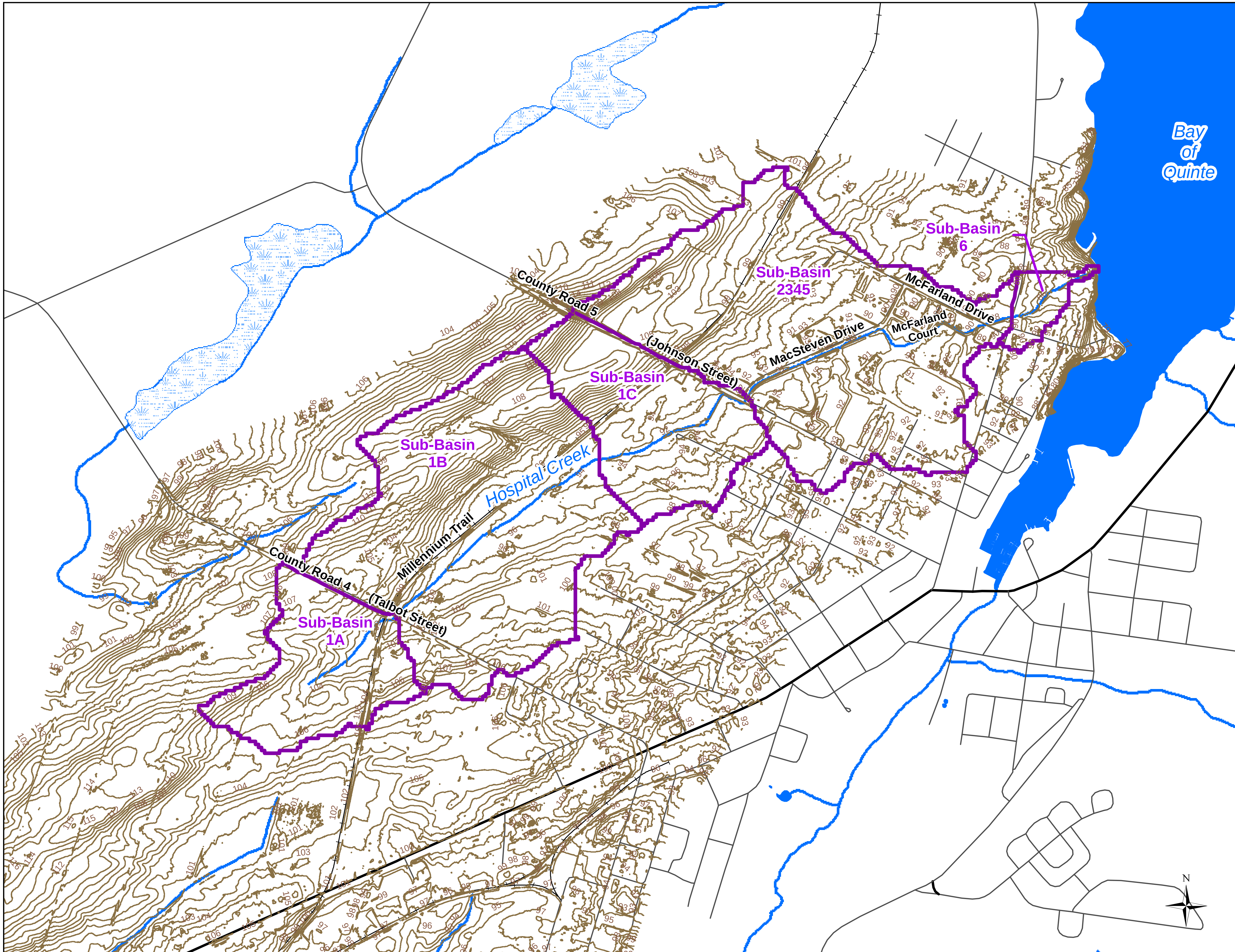
1m Contours produced using LiDAR data
Watershed Boundary produced using LiDAR data

Map 7.1 Pre-Development Sub-Basin Delineation



Legend

- Catchments
- 1m Contours
- Major Roads
- Minor Roads
- Railways
- Waterbodies
- Wetlands
- Watercourses

1:12,000

0 100 200 400

Meters

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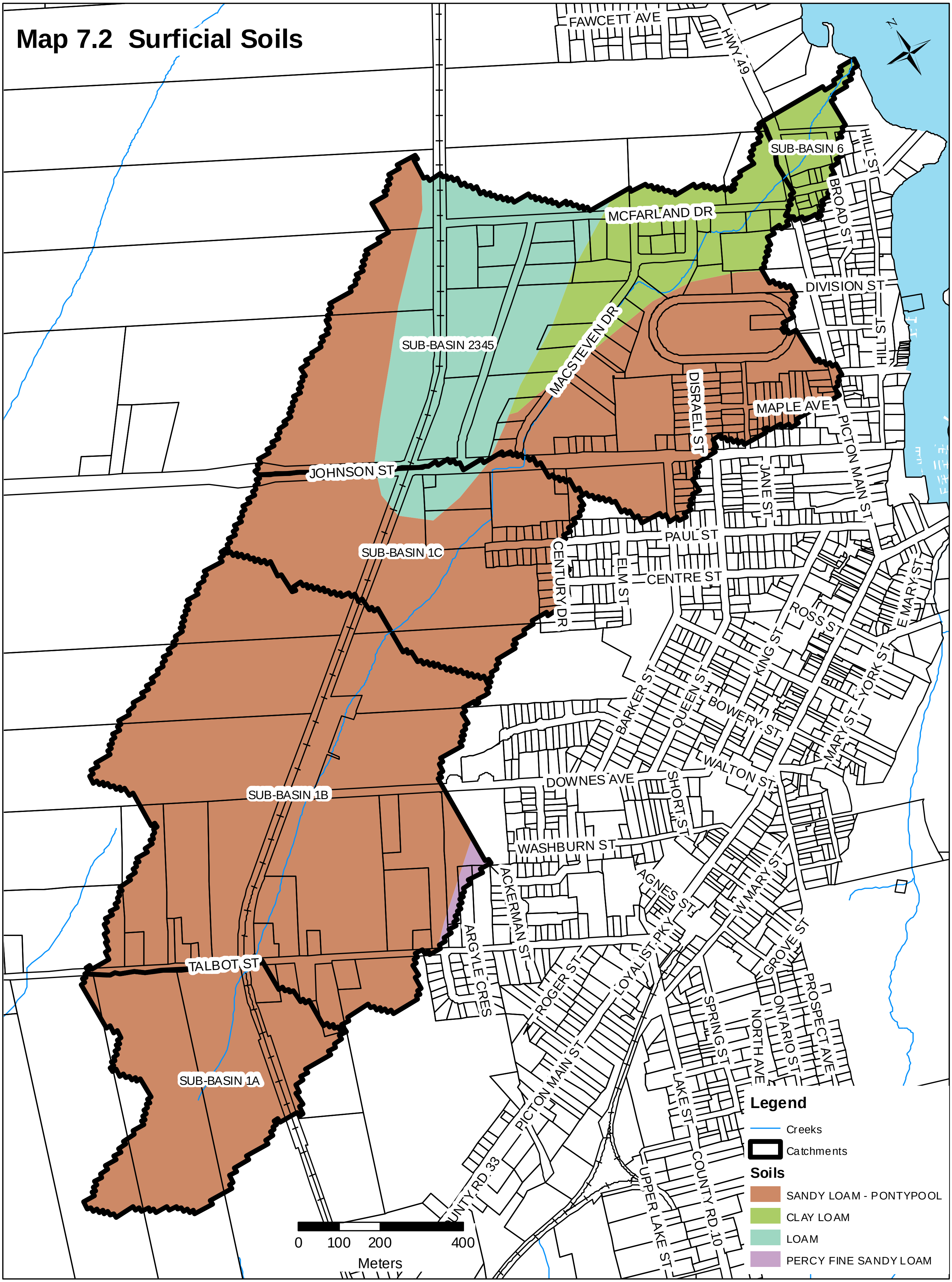
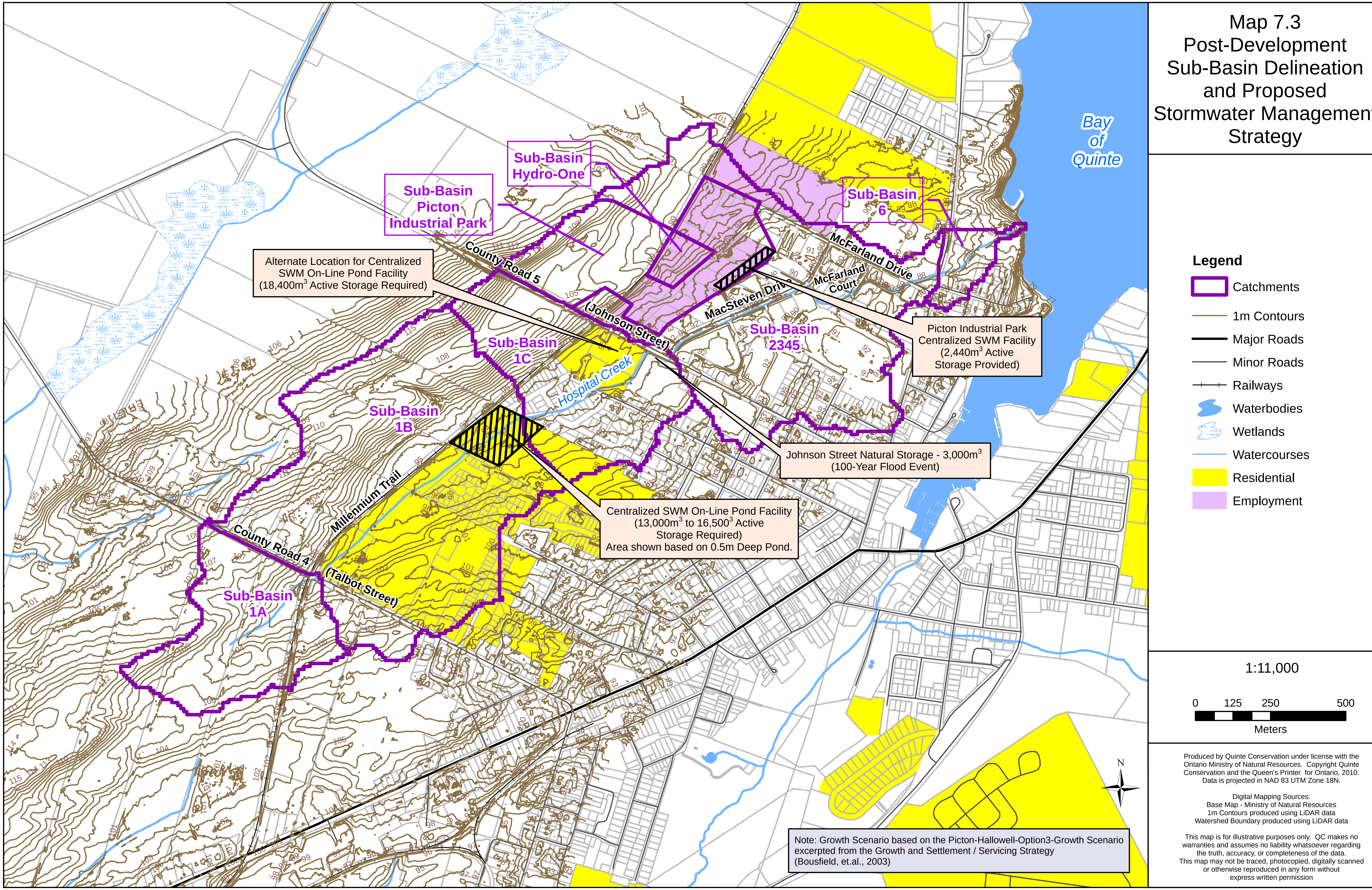
Digital Mapping Sources:
Base Map - Ministry of Natural Resources
1m Contours produced using LiDAR data
Watershed Boundary produced using LiDAR data

Map 7.2 Surficial Soils




Map 7.3
Post-Development
Sub-Basin Delineation
and Proposed
Stormwater Management
Strategy
Sub-Basin Hydro-One
Sub-Basin Picton Industrial Park
Sub-Basin 6
Sub-Basin 1C
Sub-Basin 1B
Sub-Basin 1A
Sub-Basin 2345
Bay of Quinte
Hospital Creek
County Road 5
(Johnson Street)
County Road 4
(Talbot Street)
Millennium Trail
McFarland Drive
McFarland Court
MacSteven Drive
Alternate Location for Centralized SWM On-Line Pond Facility (18,400m³ Active Storage Required)
Picton Industrial Park Centralized SWM Facility (2,440m³ Active Storage Provided)
Johnson Street Natural Storage - 3,000m³ (100-Year Flood Event)
Centralized SWM On-Line Pond Facility (13,000m³ to 16,500³ Active Storage Required) Area shown based on 0.5m Deep Pond.
Note: Growth Scenario based on the Picton-Hallowell-Option3-Growth Scenario excerpted from the Growth and Settlement / Servicing Strategy (Bousfield, et.al., 2003)
Legend
Catchments
1m Contours
Major Roads
Minor Roads
Railways
Waterbodies
Wetlands
Watercourses
Residential
Employment
1:11,000
0 125 250 500
Meters
N
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Digital Mapping Sources:
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1m Contours produced using LiDAR data
Watershed Boundary produced using LiDAR data
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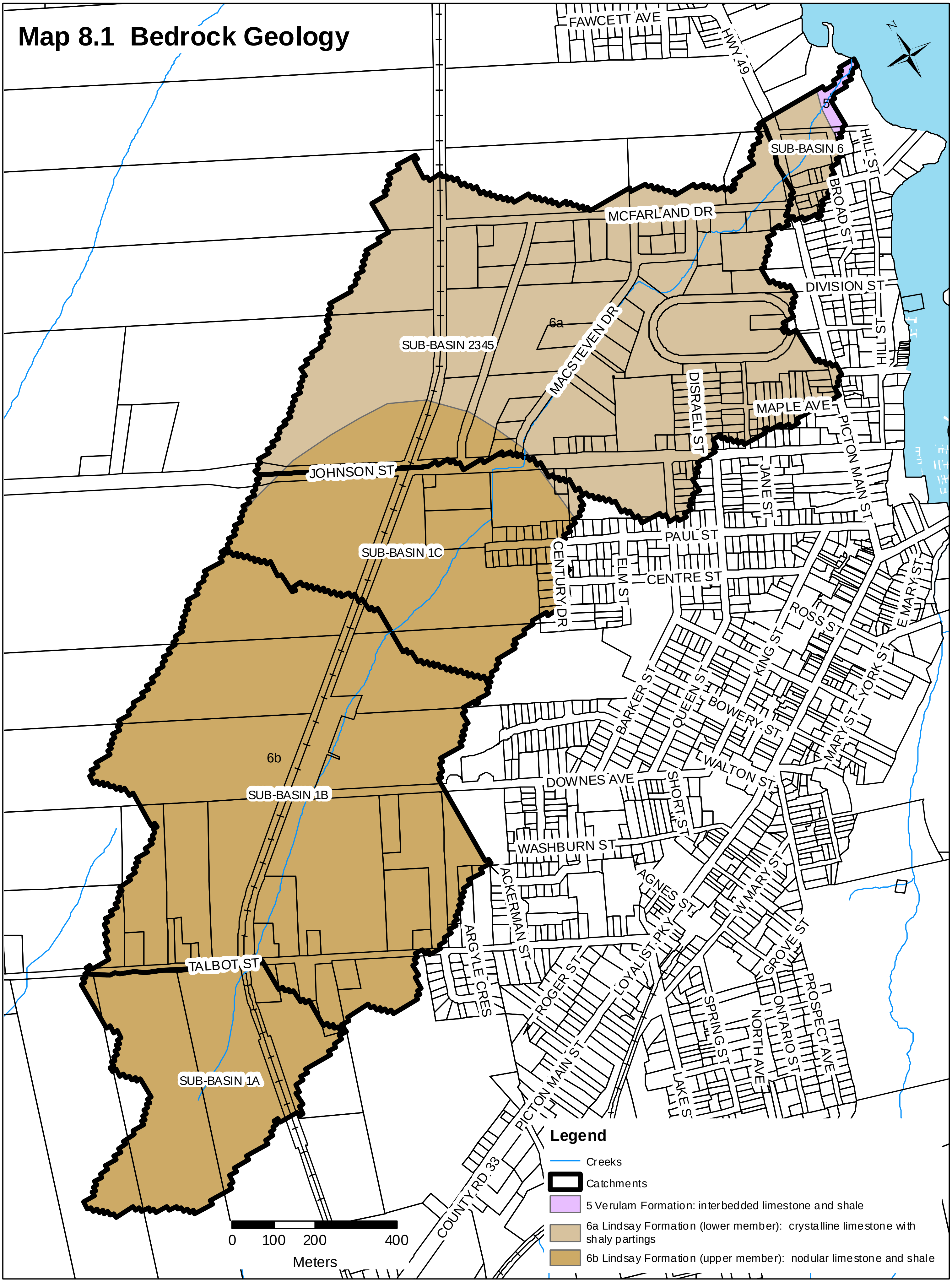
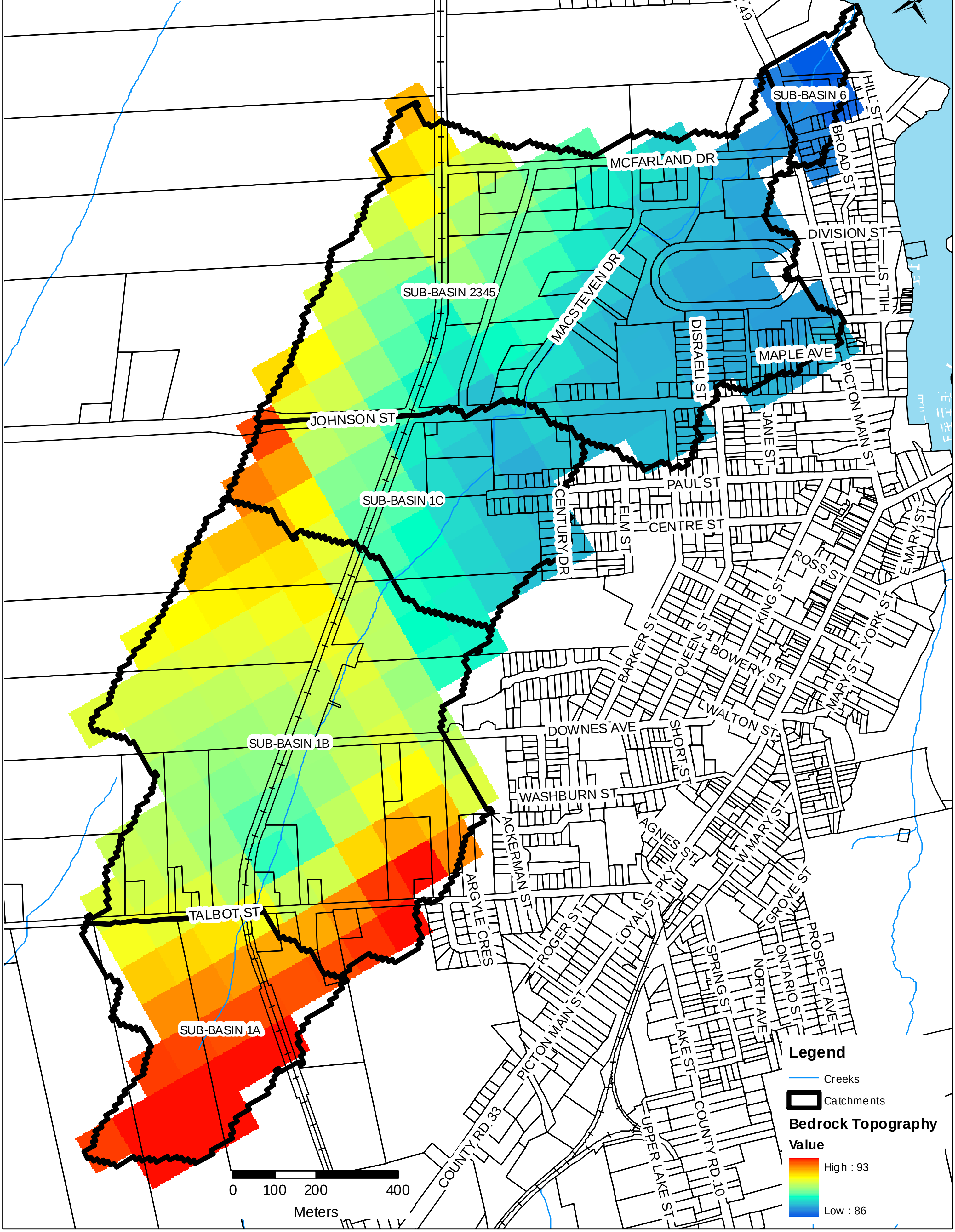
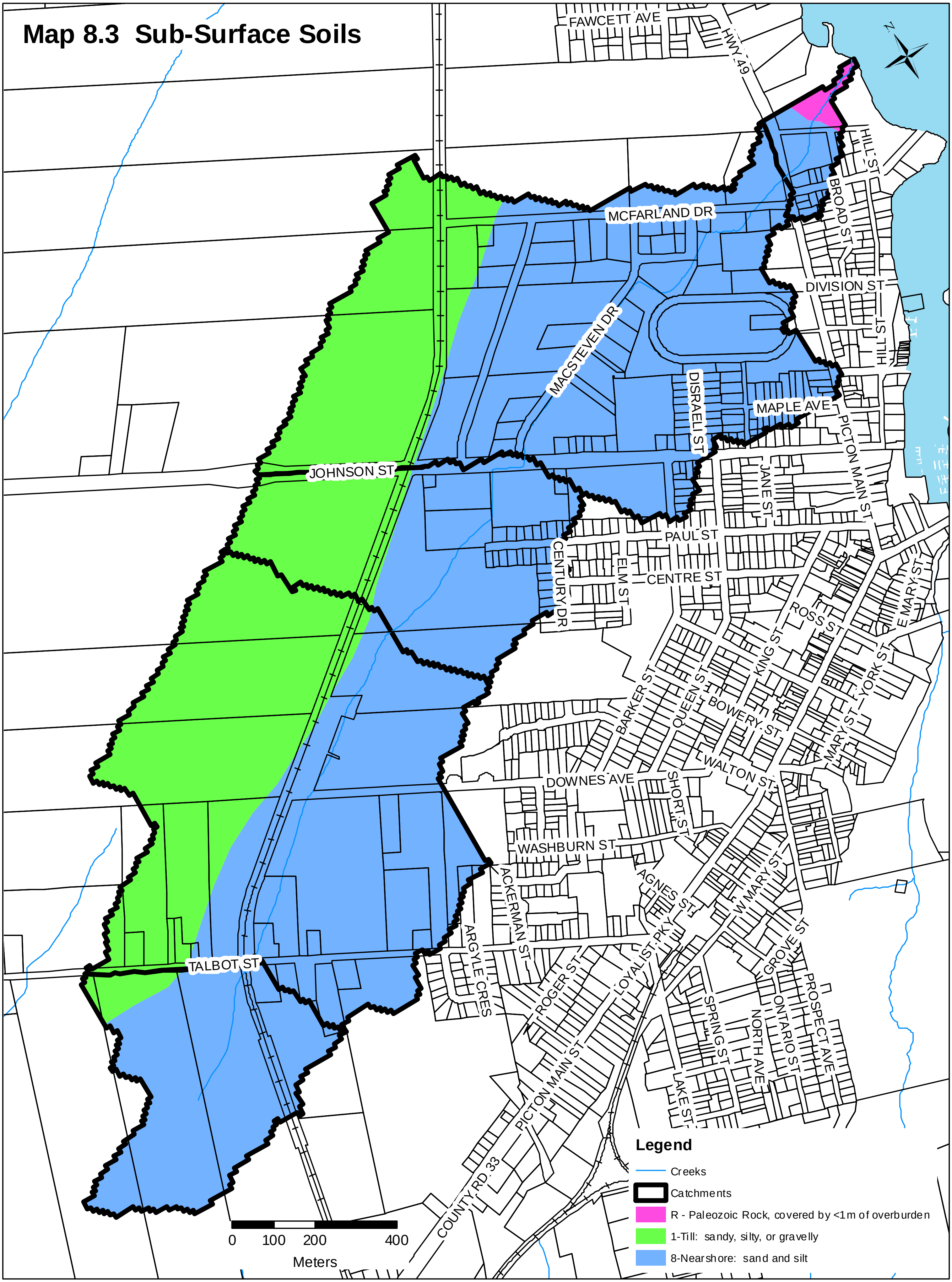
Map 8.1 Bedrock Geology



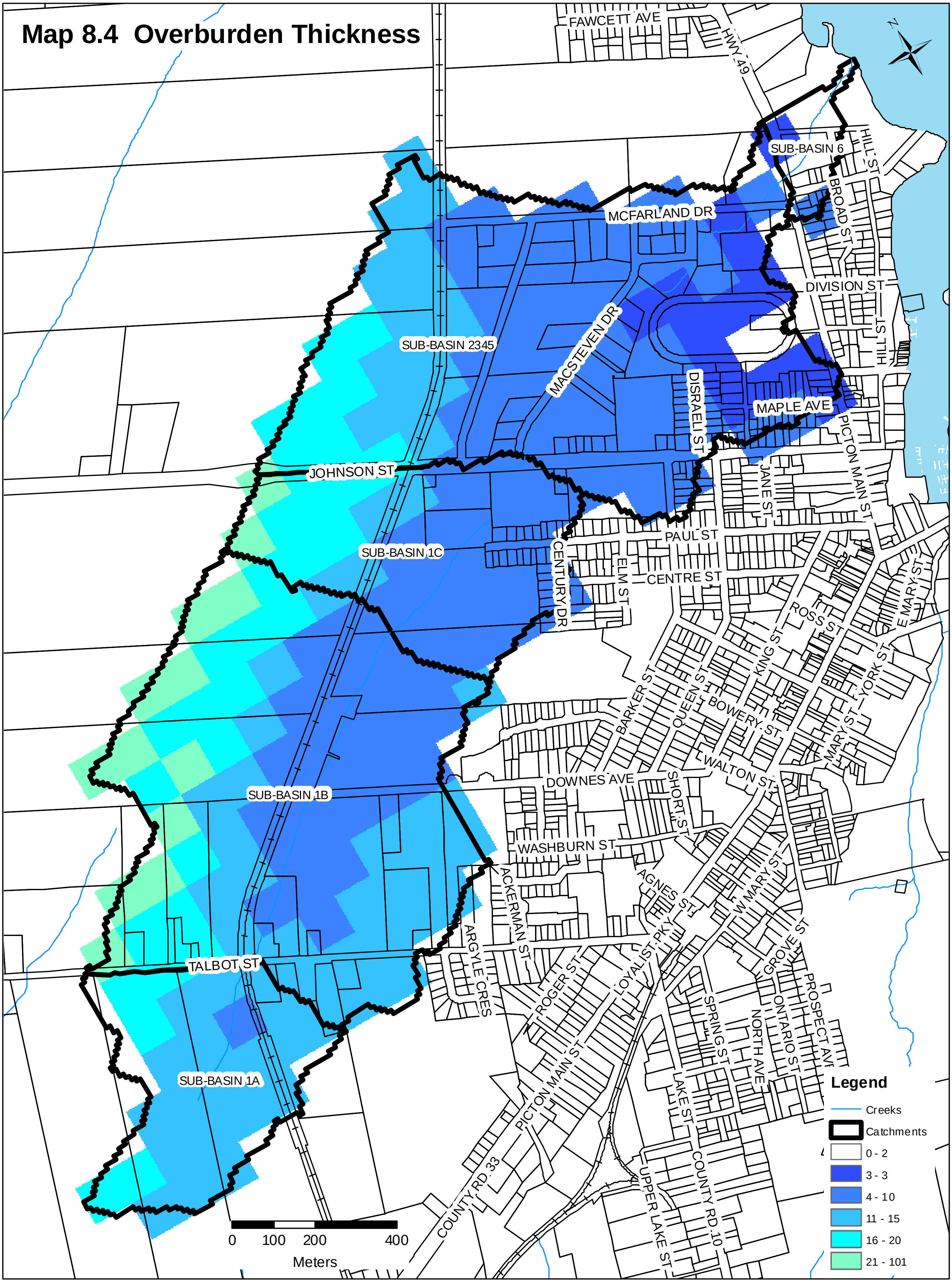
Map 8.2 Bedrock Topography



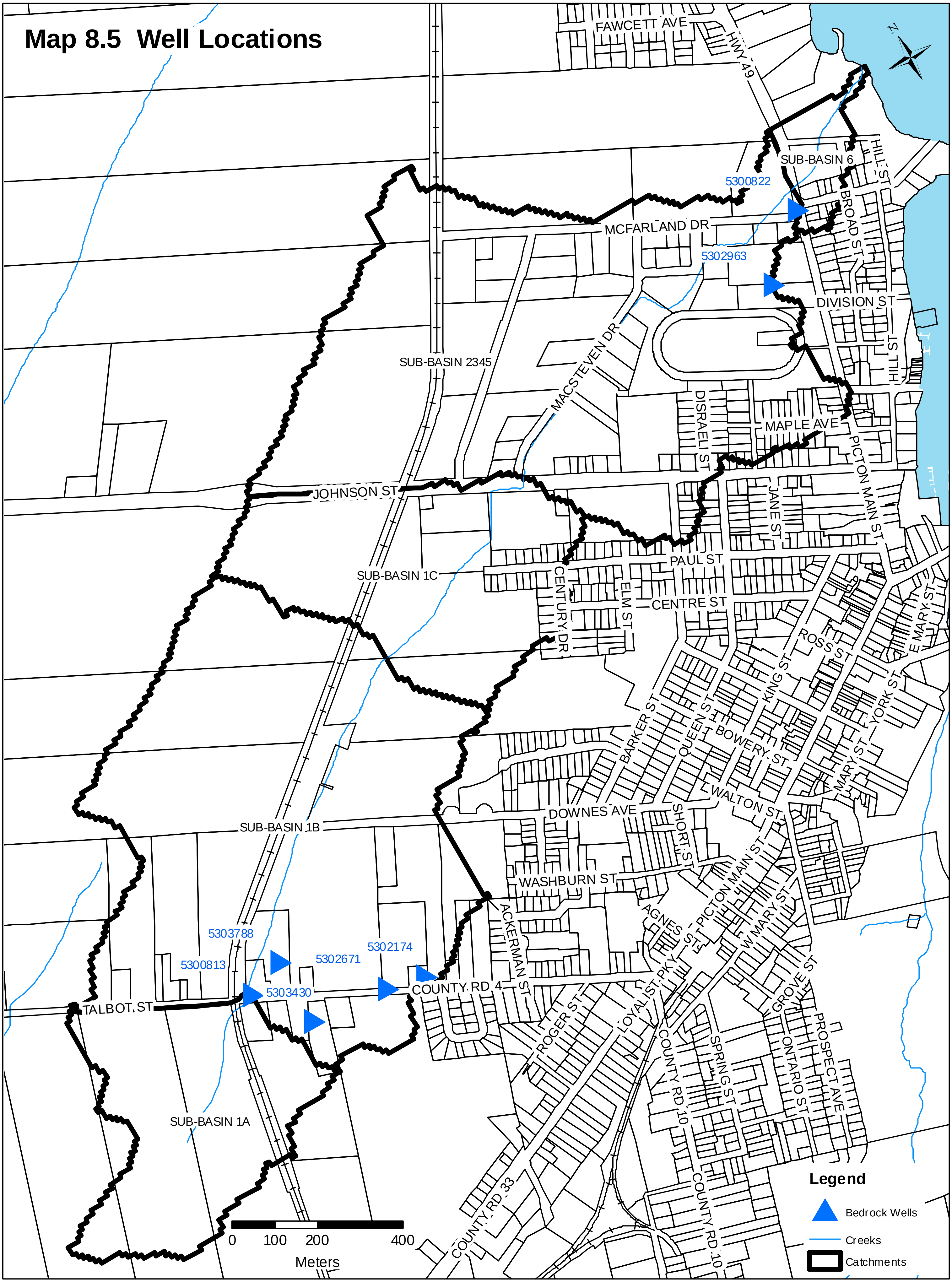
Map 8.3 Sub-Surface Soils



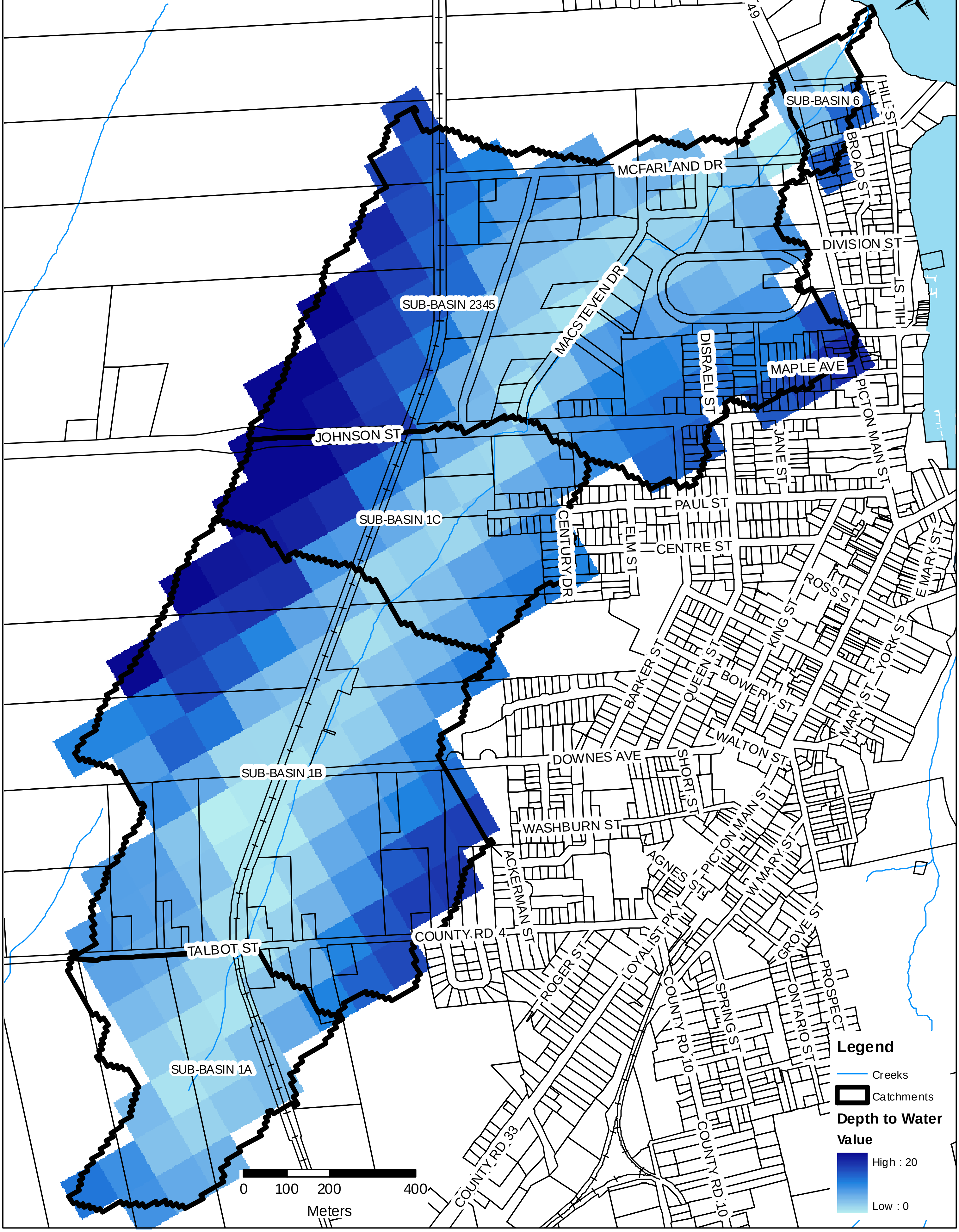
Map 8.4 Overburden Thickness

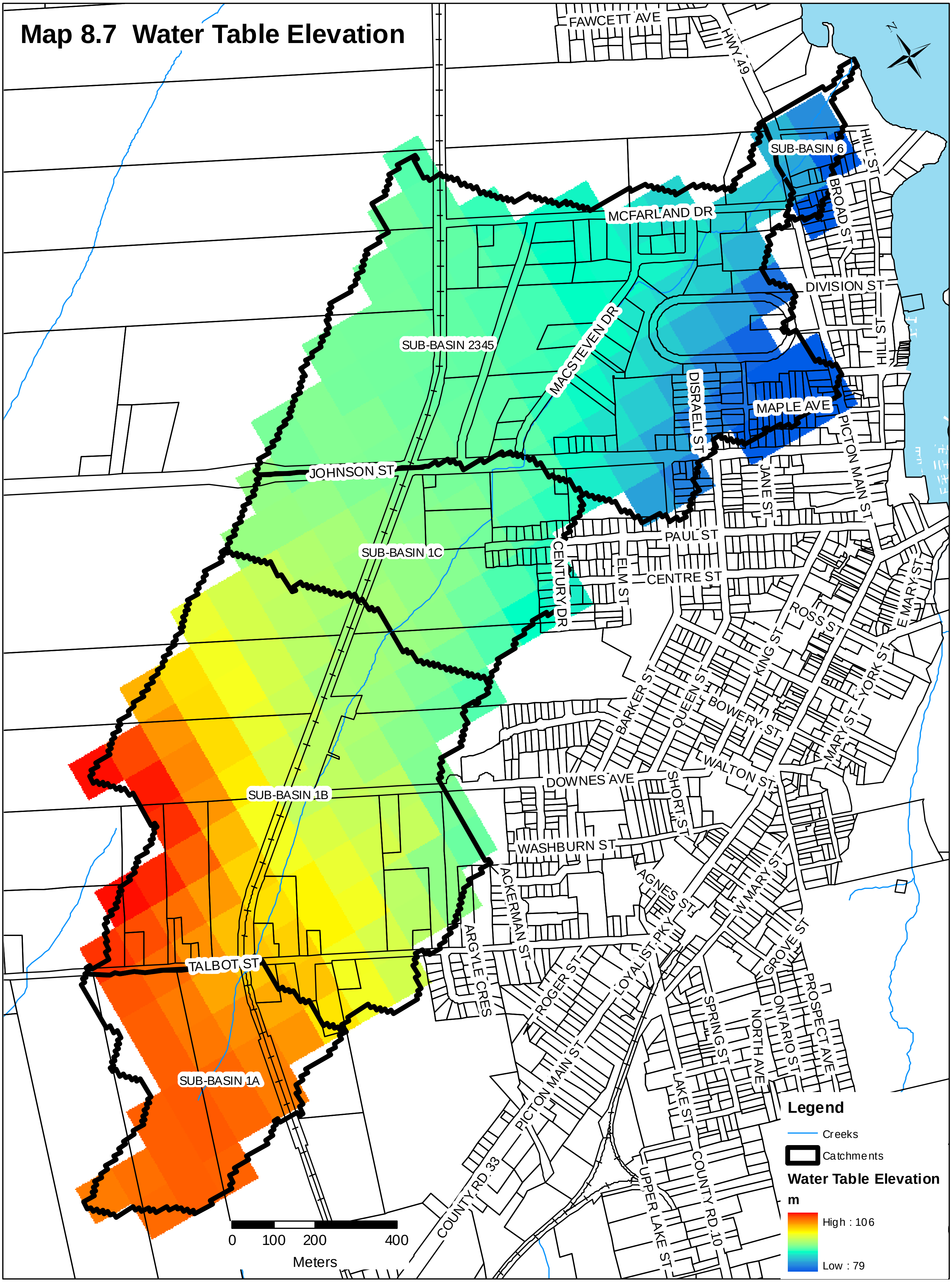


Map 8.5 Well Locations



Map 8.6 Depth to Water




Map 8.7 Water Table Elevation
FAWCETT AVE
HWY 49
SUB-BASIN 6
HILL ST
BROAD ST
MCFARLAND DR
DIVISION ST
SUB-BASIN 2345
MACSTEVEN DR
DISRAELI ST
MAPLE AVE
PICTON MAIN ST
JOHNSON ST
JANE ST
PAUL ST
SUB-BASIN 1C
CENTURY DR
ELM ST
CENTRE ST
ROSS ST
KING ST
E MARY ST
YORK ST
BARKER ST
QUEEN ST
BOWERY ST
MARY ST
WALTON ST
DOWNES AVE
SHORT ST
SUB-BASIN 1B
WASHBURN ST
ACKERMAN ST
AGNES ST
W MARY ST
ARGYLE CRES
LOYALIST PKY
GROVE ST
TALBOT ST
ROGER ST
SPRING ST
ONTARIO ST
PROSPECT AVE
NORTH AVE
SUB-BASIN 1A
LAKE ST
COUNTY RD 33
UPPER LAKE ST
COUNTY RD 10
0 100 200 400
Meters
Legend
Creeks
Catchments
Water Table Elevation
m
High : 106
Low : 79

Map 9.1
Ecological Features
DRAFT