REPORT

Surface Water and Groundwater Assessment - Technical Study Report

DURHAM YORK
RESIDUAL WASTE EA STUDY

REPORT NO. 1009497

EXECUTIVE SUMMARY

The Proposed Thermal Treatment Facility (the Facility) is being designed to meet the ongoing post-diversion waste disposal needs of the two Regions by thermally processing the waste and recovering both material and energy thereby minimizing the amount of material requiring landfill disposal. The evaluations were completed for two (2) design capacity scenarios for the Facility. These are the initial design capacity of 140,000 tonnes per year (tpy) and a maximum design capacity of 400,000 tpy (addressed in **Appendix D**). For the purposes of this technical study report, the initial design capacity for the Facility is based on 140,000 tpy waste processing capacity , including the footprint for the main treatment plant as well as ancillary structures at the Site. The maximum design capacity of 400,000 tpy is also assessed (see **Appendix D**). The Proposed Thermal Treatment Facility Site (the Site) is located on the west side of Osbourne Road and north of the CN Rail corridor in the Municipality of Clarington.

An Environmental Assessment (EA) Study was conducted to address the social, economic, and environmental concerns resulting from the planned construction and operation of the Facility. This Surface Water and Groundwater Technical Study Report is **Appendix C-2** of the EA, and describes the baseline surface and groundwater conditions in the study area, water demand, and wastewater servicing, stormwater management planning, and potential effects, mitigation, and net effects related to the Facility.

The Site is located within the Tooley Creek watershed which in its lower reaches supports cold water fisheries. Tooley Creek is a small meandering watercourse receiving a majority of its flow from agricultural and rural runoff and groundwater inputs in its northern reaches. Water courses in the area are characterized by a range of flow conditions dictated by the heterogeneity of the underlying materials. The high infiltration potential of the Oak Ridges Moraine (ORM) in the north represents an area of groundwater recharge and subsequently leads to continual baseflow additions to surrounding streams. Infiltration potential decreases with proximity to Lake Ontario (Tooley Creek outlet), representing a shift to more silt and clay dominant materials. The Site is situated above an east-west band of glaciolacustrine deposits generally contributing little to groundwater resources and subsequently baseflow contributions. However, a geotechnical investigation conducted on the Site suggested that subsurface materials were sandy-silts to silty sands which may facilitate more infiltration than Regionally suggested. In general, watercourse inputs in the lower, or southern, portion of the Tooley Creek watershed are runoff sourced.

The Site is located in an area with previously installed municipal watermain and sewermain infrastructure. The Facility is based on the initial design capacity of 140,000 tpy. The Facility is conceived as having a maximum design capacity of 400,000 tpy of waste material. Although this report focuses on the water demand, wastewater generation and stormwater management requirements of the initial design capacity of 140,000 tpy Facility, it also discusses how increasing the Facility to the maximum design capacity of 400,000 tpy may increase water demand, wastewater generation and stormwater management requirements (**Appendix D**). The Facility would require a maximum of 42,000 m^3/yr or 115,068 L/day of water demand based on initial design capacity of 140,000 tpy. Preliminary assessments suggest that this demand can be met by connection to the 300 mm Osbourne Road watermain. If the Facility water demand cannot be met by this single connection, a secondary 300 mm watermain located approximately 3.5 km away would be accessed to fulfill the extra demand. A maximum of 3000 m^3/yr or 8219 L/day of wastewater discharge is anticipated from the Facility. An 1800 mm sewermain located on Osbourne Road and routing wastewater to the nearby Courtice Water Pollution Control Plant (WPCP) would be capable of receiving this volume of discharge.

This Technical Study Report documents the examination of the pre-development water balance and runoff flows arising from the Site. This data was used to develop a Stormwater Management (SWM) Plan to control stormwater quantity and quality for the Site during construction and operation phases. Stormwater runoff from the Site drains towards the southwest until reaching an east-west running swale located immediately north of the CN Rail corridor easement. Runoff is subsequently conveyed approximately 1000 m west to Tooley Creek. Stormwater management objectives are to maintain stormwater volumes, rates, and quality comparable to pre-development flow conditions, to the extent possible.

The Facility footprint is proposed to occupy the 12.4 ha property of which approximately 45% would comprise impervious surfaces. Without mitigation measures, increased runoff could adversely affect receiving surface and groundwater resources. Stormwater management design would reduce peak discharges, attenuate flows, and improve water quality through the introduction of infiltration, settling and storage features. Stormwater would receive the highest level of environmental protection to preserve water quality in receivers.

Erosion and sediment controls (ESC) would be implemented during the construction phase to reduce potential soil loss and runoff velocities. During the construction phase, stormwater would be routed via conveyance swales and/or stormsewers draining catchbasins to a SWM pond in the southwest corner of the Site. The pond would discharge to the CN Rail swale and stormwater would subsequently be conveyed to Tooley Creek. In addition to the pond, lot level, and conveyance controls such as surface stabilization measures, sediment traps, and swales enhanced with rock check dams would also be employed.

Grading plans would be designed to maintain existing drainage patterns which would ensure all captured stormwater would be routed through stormwater management features onsite.

Post-construction, stormwater conveyance would be accomplished through a combination of previously implemented swales and underground stormsewers. All stormwater from the developed Site would continue to be routed to the southwestern SWM pond for quality and quantity control purposes. Pond design would entirely capture the 100-year design storm event for flood control purposes and provide a minimum 24-hour draw down for the 25 mm design storm event to ensure adequate water quality improvement.

The considerations for infiltration, evapotranspiration and runoff water quality enhancements would protect receiving water resources from the potential negative impacts of the Facility. As a result of the suggested mitigation measures, the assessment concluded that no significant negative net or cumulative environmental effects are likely to occur.

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GLOSSARY AND ABBREVIATIONS

** An asterisk (*) beside a defined term indicates that the term is defined in the Environmental Assessment Act.*

Abstraction:	Removal of water from a river, lake, or aquifer.
Air Emissions:	For stationary sources, the release or discharge of a pollutant from a facility or operation into the ambient air either by means of a stack or as a fugitive dust, mist or vapour.
Alternative Methods:	Alternative methods of carrying out the proposed undertaking are different ways of doing the same activity. Alternative methods could include consideration of one or more of the following: alternative technologies; alternative methods of applying specific technologies; alternative sites for a proposed undertaking; alternative design methods; and, alternative methods of operating any facilities associated with a proposed undertaking.
Alternatives:	Both “alternative methods” and “alternatives to” a proposed undertaking.
Alternatives To:	Alternatives to the proposed undertaking are functionally different ways of approaching and dealing with a problem or opportunity.
Antecedent Moisture Condition:	The prevailing soil moisture content present before the event or time period in question.
Application:	An application for approval to proceed with an undertaking under subsection 5(1) of the *Environmental Assessment Act.*
Aquifer:	An underground layer of water-bearing porous stone, earth, or gravel.
Aquitard:	A geological formation that restricts groundwater movement.
Artesian:	Water held under pressure in porous rock or soil confined by impermeable geologic formations.

Baseflow:	The component of the total stream flow due predominantly to groundwater discharge into a stream.
Baseflow Augmentation:	The act of decreasing the peak and lengthening the duration of a pond discharge for the purposes of water quality and quantity improvement.
Buffer Area:	That part of a disposal site or facility that is not a waste fill area (in the case of a landfill) or is not occupied by a building. (i.e., area between actual facility and the property boundary).
Calcareous:	Composed of or containing lime or limestone.
Candidate Site:	Property identified as suitable for consideration as a potential site for a waste management facility.
Certificate of Approval:	A license or permit issued by the Ministry of the Environment for the operation of a waste management site/facility.
Class Environmental Assessment (EA):	A planning and approvals process for a group of projects which are routine, similar in nature, limited in scale, and possess predictable environmental effects.
Climate Normals:	The average climatic conditions for an area derived from accumulated yearly records.
Contingency Plan:	A plan developed to be implemented should some aspect of the project need to be altered or some aspect of the operation fail (i.e. “Plan B”).
Conveyance:	The transport of ground or surface water from location to another.
Design and Operation (D&O) Plan/Report:	A document (plan/report), required for obtaining a Certificate of Approval, which describes in detail the function, elements or features of a landfill site/facility or waste management facility, and how a landfill site/facility or waste management facility would function including its monitoring, and control/management systems.
Disposal Facilities:	Facilities for disposing of solid waste, including landfills and incinerators, intended for permanent containment or destruction of waste materials.

Drawdown:	The peak to trough decline during the discharge of a stormwater management pond.
Durham:	The Regional Municipality of Durham or its geographic area, as the context requires.
Durham/York Residual Waste Study:	The Durham/York Residual Waste Study is a joint initiative between the Region of Durham and York Region to work together to find a way to manage solid waste remaining after at-source diversion.
Emissions:	Technically, all solid, liquid, or gaseous discharges from a processing facility, but normally referring to Air Emissions (with solids referred to as residue and liquids as effluent).
Energy-from-Waste (EFW):	The recovery of energy in the form of heat and/or power from the thermal treatment of waste. Generally applied to incineration, pyrolysis, gasification but can also include the combustion of landfill gas and gas produced from anaerobic digestion of organic materials.
Environment*:	The environment is broadly defined under the *Environmental Assessment Act* as follows: (a) Air, land or water; (b) Plant and animal life, including human life; (c) The social, economic and cultural conditions that influence the life of humans or a community; (d) Any building, structure, machine or other device or thing made by humans; (e) Any solid, liquid, gas, odour, heat, sound, vibration or radiation resulting directly or indirectly from human activities; or, (f) Any part or combination of the foregoing and the interrelationships between any two or more of them.

Environmental Assessment:	Environmental assessment is a study, which assesses the potential environmental effects (positive or negative) of a proposal. Key components of an environmental assessment include consultation with government agencies and the public; consideration and evaluation of alternatives; and, the management of potential environmental effects. Conducting an environmental assessment promotes good environmental planning before decisions are made about proceeding with a proposal.
Environmental Assessment Act:	The *Environmental Assessment Act* (and amendments and regulations thereto) is a provincial statute that sets out a planning and decision-making process to evaluate the potential environmental effects of a proposed undertaking. Proponents wishing to proceed with an undertaking must document their planning and decision-making process and submit the results from their environmental assessment to the Minister for approval.
Environmental Effect:	The effect that a proposed undertaking or its alternatives has or could potentially have on the environment, either positive or negative, direct or indirect, short- or long-term.
Environmental Protection Act (EPA):	An Ontario Act to provide for the protection and conservation of the natural environment.
Evapotranspiration:	The combination of water transpired from the plant and evaporated from the soil and plant surfaces.
Filter strip:	Strip or area of vegetation used for removing sediment, organic matter, and other pollutants from runoff.
Forebay:	An extra storage space provided near the inlet of a stormwater management pond to trap incoming sediments before they accumulate in the pond.
Freeboard:	Height of the crest of a structure above the anticipated still water level.
Fugitive Emissions:	Emissions not caught by a capture system.
Geomorphology:	The study of landforms, their classification, origin, development, and history.

Geotextile:	Permeable fabrics which, when used in association with soil, have the ability to separate, filter, reinforce, protect, or drain.
Government Review Team:	Staff from government ministries and agencies (federal; provincial, including local Conservation Authorities; and, municipal, including local Boards of Health) who contribute to the review of environmental assessment documentation (terms of reference and environmental assessment) by providing comments from their mandated areas of responsibility.
Grubbing:	The removal of all trees, stumps, plants and rocks.
GTA:	Greater Toronto Area.
Hedgerows:	A group or row of trees and shrubs separating two grassy areas.
Hickenbottom Riser	A standpipe inlet or outlet with perforations for water passage.
Hummocky:	Containing low mounds or ridges of earth.
Hydraulics:	Of or relating to water or other liquid in motion.
Hydraulic Conductivity:	Ability of water to flow through soil.
Hydrograph:	Graph of variation of stream flow over time.
Hyetograph:	Graph of variation of precipitation over time.
Impact Management Measures:	Measures which can lessen potential negative environmental effects or enhance positive environmental effects. These measures could include mitigation, compensation, or community enhancement.
Impact Studies:	Studies that predict negative consequences (if any) of a proposed undertaking. Air, visual, natural environmental, traffic, hydrogeological, Noise, Health Risk, Land Use and Hydrological Impact Studies are required under the *Environmental Protection Act*.

Incineration:	A thermal treatment technology involving destruction of waste by controlled burning at high temperatures with the overall aim of reducing the volume of waste.
Incinerator:	A furnace for burning waste under controlled conditions.
Individual Environmental Assessment:	An Individual Environmental Assessment requires the following steps to fully address the requirements of the EAA: Preparation of the Proposed EA Terms of Reference; Submission of the EA Terms of Reference to the Minister of the Environment for Approval; Completion of the EA Study in accordance with approved EA Terms of Reference, and; Submission of the EA Study to the Minister of the Environment for Approval.
Infiltration:	The movement of water through the soil surface into the soil.
Interflow:	The lateral motion of water through the upper layers until it enters a stream channel.
Intermittent:	Flowing most of the time but seasonally or occasionally ceasing to flow in response to decreased water availability.
Köppen Climate Classification System:	The Köppen climate classification is one of the most widely used climate classification systems and is based on the concept that native vegetation is the best expression of climate; thus, climate zone boundaries have been selected with vegetation distribution in mind. It combines average annual and monthly temperatures and precipitation, and the seasonality of precipitation. Dfb is a Köppen climate zone extending across the southern portion of Canada from Newfoundland to British Columbia.
Lacustrine:	Deposits accumulated in lakes or of lake origin are lacustrine.
Lithology:	Mineralogy, grain size, texture, and other physical properties of granular soil, sediment or rock.
Meander:	Curves in the stream channel where the stream dissipates energy.

Ministry of the Environment (MOE) Ontario:	The MOE monitors pollution and restoration trends in Ontario and uses that information to develop environmental laws, regulations, standards, policies, programs, and guidelines. The MOE works to provide cleaner air, land, and water for Ontarians.
Mitigation:	Measures taken to reduce adverse impacts on the environment.
Monitoring:	Periodic or continuous surveillance or testing to determine the characteristics of a substance or the level of compliance with statutory requirements and/or pollutant levels in various media or in humans, plants, and animals.
Municipal Solid Waste (MSW):	Common garbage or trash generated by industries, businesses, institutions, and homes.
Ontario:	The Province of Ontario, or its geographic area, as the context requires.
Ontario Regulation 347 (O. Reg. 347):	A regulation under the *Environmental Protection Act* that specifies standards and approval requirements for waste management sites and systems in Ontario.
Outlet:	An opening through which water can be freely discharged from a reservoir.
Overburden:	Soil and other material that overlays the Regional bedrock.
Oxbow:	The area resulting from the meandering of a river or stream.
Particulate:	A particle of a solid or liquid that is suspended in air.
Peak Shaving:	The act of decreasing the peak discharge from a stormwater management facility for the purposes of decreasing downstream erosion.
Piezometer:	A device used to measure ground-water pressure head at a point in the subsurface.
Pollutant:	Generally, any substance introduced into the environment that can adversely affect the usefulness of a resource or the health of humans, animals, or ecosystems.

Pollution:	Generally, the presence of a substance in the environment that because of its chemical composition or quantity can prevent the functioning of natural processes and produce undesirable environmental and health effects.
Post-Closure:	The time period, following the shutdown of a landfill, waste management or manufacturing facility; established for monitoring purposes.
Potable Water:	Water that is safe for drinking and cooking.
Project:	Encompasses the design, construction (including construction financing) and operation of the EFW Facility, and includes the EA Study, the supply of municipal waste, and the sale of energy.
Proponent*:	A person, agency, group or organization that carries out or proposes to carry out an undertaking or is the owner or person having charge, management or control of an undertaking.
Rating Curve:	Relationship between depth and amount of flow in a channel.
Recharge:	Water added to an aquifer or the process of adding water to an aquifer.
Regions:	Durham and York collectively.
Residual:	Amount of a pollutant remaining in the environment after a natural or technological process has taken place; e.g., the sludge remaining after initial wastewater treatment, or particulates remaining in air after it passes through a scrubbing or other process.
Runoff:	Water, including rain and melted snow, which is not absorbed into the ground but instead flows across the land and eventually runs into streams and rivers.
Salmonids:	Trout or salmon. Many species of each belong to this family.
Scarification:	Loosening top soil or breaking up the forest floor to improve conditions for seed germination, depression storage or tree planting.
Scour:	Removal of sediment from the streambed by flowing water.

Silt Fencing: A semi-permeable membrane used to slow the flow of runoff and induce sedimentation of suspended particulate.

Sinuosity: The ratio of the channel length between two points on a channel to the straight-line distance between the same two points.

Siting: The process of choosing a location for a facility.

Stack: A chimney, smokestack, or vertical pipe that discharges flue gas or used air.

Stratigraphy: The order of rock or soil layers in a geological formation.

Stomata: A pore in the epidermis of vascular plants used for gaseous exchange and transpiration.

Stormwater: That portion of rainfall that does not infiltrate into the soil.

Swale: A wide, shallow depression in the ground to form a channel for water drainage.

Terms of Reference: A document prepared by the proponent and submitted to the Ministry of the Environment for approval. The terms of reference sets out the framework for the planning and decision-making process to be followed by the proponent during the preparation of an environmental assessment. In other words, it is the proponent's work plan for what is going to be studied. If approved, the environmental assessment must be prepared according to the terms of reference.

Thermal Treatment: Use of elevated temperatures to treat wastes (e.g., combustion or gasification).

Treatment Train: A series of Best Management Practices and/or natural features, each planned to treat a different aspect of potential contamination.

Undertaking*: An enterprise, activity or a proposal, plan, or program that a proponent initiates or proposes to initiate.

Waste Management System:	A set of facilities or equipment used in, and any operations carried out for, the management of waste including the collection, handling, transportation, storage, processing or disposal of waste, and may include diversion programs and facilities and one or more waste disposal sites.
Waste Stream:	The total flow of solid waste from homes, businesses, institutions, and manufacturing plants that is recycled, burned, or disposed of in landfills, or segments thereof such as the "residential waste stream" or the "recyclable waste stream."
Wastewater:	Water that has been used in homes, industries, and businesses that is not for reuse unless it is treated.
Waste:	1. Refuse from places of human or animal habitation. 2. Unwanted materials left over from a manufacturing process.
Waste-to-Energy (WTE) Facility/Municipal-Waste Combustor:	Facility where recovered municipal solid waste is converted into a usable form of energy, usually via combustion.
Water Balance:	A measure of the amount of water entering and the amount of water leaving a system.
Well-Head Protection Area:	Areas of land where human activities are regulated to protect the quality of ground water that supplies public drinking water wells.
York:	The Regional Municipality of York or its geographic area, as context requires.

List of Abbreviations

AMC	Antecedent Moisture Condition
ANSI	Area of Natural and Scientific Interest
BG	Below Ground
C of A	Certificate of Approval
CCME	Canadian Council of Ministers of the Environment
CEAA	*Canadian Environmental Assessment Act*

CLOCA	Central Lake Ontario Conservation Authority
CO	Carbon Monoxide
CO_2	Carbon Dioxide
COPC	Chemicals of Potential Concern
CWQG	Canadian Water Quality Guidelines
DFO	Department of Fisheries and Oceans
DO	Dissolved Oxygen
EA	Environmental Assessment
EA ToR	Environmental Assessment Terms of Reference:
EAA	*Environmental Assessment Act*
EAAB	Ministry of Environment Environmental Assessment and Approvals Branch
EC	Environment Canada
EFW	Energy-from-Waste
EPA	*Environmental Protection Act*
ERA	Ecological/Environmental Risk Assessment
ESA	Environmentally Significant Area
ESC	Erosion and Sediment Control
GGHACA	Greater Golden Horseshoe Area Conservation Authorities
GIS	Geographic Information System
GTA	Greater Toronto Area
ha	Hectares
HEC-HMS	Hydrologic Engineering Center – Hydrologic Modeling System
Hr	hour
IDF	Intensity Duration Frequency Curve
km	kilometre
L/day	Litre per day
masl	metres above sea level

Max	Maximum
mbg	metres below grade
mg/L	milligram per litre
Min	Minimum
MNR	Ministry of Natural Resources
MOE	Ontario Ministry of the Environment
MSC	Meteorological Service of Canada
MTO	Ontario Ministry of Transportation
m^3/day	metres cubed per day
m^3/s	metres cubed per second
mm/yr	millimetre per year
NHIC	Natural Heritage Information Centre
NRCS	Natural Resource Conservation Service
OEAA	Ontario *Environmental Assessment Act*
O.Reg	Ontario Regulation
ORM	Oak Ridges Moraine
OWRA	*Ontario Water Resources Act*
PM	Particulate Matter
PPS	Provincial Policy Statement
PTTW	Permit to Take Water
PWQMN	Provincial Water Quality Monitoring Network
PWQO	Provincial Water Quality Objectives
QA	Quality Assurance
QA/QC	Quality assurance/quality control
QC	Quality Control
SCS	Soil Conservation Service
SWM	Stormwater Management

t/yr Tonnes/year

TOC Time of Concentration

ToR Terms of Reference

Tpy Tonnes per year

USLE Universal Soil Loss Equation

U.S.A. United States

WPCP Water Pollution Control Plant

WSC Water Survey of Canada

WWIS Water Well Information System

YPDT-CAMC York-Peel-Durham-Toronto Conservation Authorities Moraine Coalition

UNITS OF MEASUREMENT

Area

m^3 cubic metre

Mass/Weight

Re. Orders of Magnitude: x 10^2 = x 100, x10^3 = x 1000, etc.

g gram

mg milligrams 1 x 10^{-3} grams

kg kilogram 1 x 10^3 g

t metric tonne 1 x 10^3 kg

Volume

L litre

mL millilitre 1 L = 1 x 10^3 mL

m^3 cubic metre 1 m3 = 1 x 10^3 L

Time

s second

min minute

hr hour

yr year

Equation Symbols

P Precipitation

ET Evapotranspiration

RO Runoff

I Infiltration and Storage

R Rainfall and Runoff Factor

K Soil Erodibility Factor

LS Slope Length-Gradient Factor

C Crop/Vegetation and Management Factor

P Support Practice factor

REPORT

1.0 INTRODUCTION

Durham and York Regions (the Regions) have partnered to undertake a joint Residual Waste Planning Study. Both municipalities are in need of a solution to manage the residual solid waste that remains after diversion. The Regions are working together to address the social, economic, and environmental concerns through an Environmental Assessment (EA) Study process to examine potential long-term residual waste management alternatives.

1.1 The Environmental Assessment Process

The purpose of the undertaking (i.e., what the outcome of this EA Study is intended to do) as described in the Approved EA Terms of Reference is:

> *"To process - physically, biologically and/or thermally - the waste that remains after the application of both Regions' at-source waste diversion programs in order to recover resources - both material and energy - and to minimize the amount of material requiring landfill disposal. In proceeding with this undertaking only those approaches that will meet or exceed all regulatory requirements will be considered."*

The EA Study follows a planning approach where environmental constraints or opportunities are considered in the context of the broadly defined environment under the *Environmental Assessment Act* (EAA) (i.e., the natural environment as well as the social, economic, and heritage and other "environments" relevant to the undertaking) and potential effects are understood and addressed before development occurs. In accordance with the Approved EA Terms of Reference and EAA, the EA process evaluates: alternatives considering potential effects on the environment; the availability of mitigation measures that address, in whole or in part, the potential effects; and, the comparison of the advantages and disadvantages of the remaining or "net" effects. The result of this process provides the planning rationale and support for a preferred approach and method to implement the undertaking.

The EA document has been prepared and conducted in accordance with EAA, including in accordance with the Terms of Reference approved by Ontario's Minister of the Environment on March 31, 2006. There are currently no federal EA process triggers identified and, therefore, this project does not require approval under the *Canadian Environmental Assessment Act* (CEAA).

It is understood and contemplated that environmental management measures recommended as part of the EA process and this Technical Study Report will in many cases be refined, updated, modified and/or superceded as a result of subsequent approval processes.

This EA process essentially consists of three parts taking place in stages including:

- the Development and Approval of an EA Terms of Reference,

- the evaluation of "Alternatives to" the undertaking, and;
- the evaluation of "Alternative methods" of implementing the undertaking.

Refer to the EA for a detailed description of the process undertaken as part of the Durham/York Residual Waste EA.

1.2 Purpose of this Report

This Report entitled *Surface Water and Groundwater Assessment - Technical Study Report* has been prepared to confirm the potential water resource related effects associated with the development of the Proposed Thermal Treatment Facility (the Facility) at the preferred site, Clarington Site 01, mitigation measures, and, potential net effects. This Report will form part of the supporting documentation and materials for the "Description of the Undertaking", completed as part of the EA Study.

1.3 Overview of Report Contents

This Report describes the existing surface water and groundwater conditions within, and in the vicinity of, the Proposed Thermal Treatment Facility Site (the Site), followed by an analysis of potential effects, mitigation measures and net effects of the Facility on the subject aspect(s) of the environment as well as a summary of the required monitoring. The key components of this Report are as follows:

- Summary of methodologies, assumptions, information sources and regulatory requirements used when conducting this assessment (Section 2).
- Description of the prevailing climate, soil, geology, topography and vegetation conditions present at the Site (Section 3.1).
- Existing condition water balance (Section 3.2).
- Assessment of the existing groundwater quantity and quality conditions (Section 3.3).
- Assessment of the existing surface water quantity and quality conditions (Section 3.4).
- An assessment of the existing conditions soil loss potential (Section 4.5).
- Results of post-development analysis including development features and impacts, water balance discussion, stormwater runoff modelling, offsite stormwater conveyance, construction phase soil loss, water demand and wastewater discharge requirements and an assessment of airborne contaminants on groundwater resources (Section 4.0).
- Construction phase erosion and sediment control planning (Section 5.1).
- Operational phase stormwater management planning (Section 5.2).
- Regulatory approvals required (Section 5.3).
- Groundwater management (Section 5.4).
- Discussion of potential accidents and malfunctions and management procedures (Section 5.5).
- Potential climate change effects and recommended mitigation measures (Section 6.1).

- Summary of potential effects to evapotranspiration, infiltration and runoff and the mitigation measures recommended to offset these effects (Section 6.2).
- A discussion of potential water quality effects resulting from stormwater discharge and the mitigation measures designed to minimize impacts (Section 6.3).
- Summary of potential cumulative effects resulting from the Facility (Section 6.4).
- Summary of major findings and recommended monitoring activities (section 7.0).

The information contained in this Report has been used to complete the EA Study.

2.0 STUDY METHODOLOGY

The evaluations, documented in this Technical Study Report, were completed for two (2) design capacity scenarios for the Facility. These are: an initial design capacity of 140,000 tonnes per year (tpy); and a maximum design capacity of 400,000 tpy for the Facility.

This hydrological and hydrogeological investigation was used to characterize the existing ground and surface water quality and quantity conditions present, identify potential effects caused by the Facility and identify mitigation measures to minimize the potential effects at, and in the vicinity of, the Site. This assessment considered the following factors:

- Site location.
- Regional and local lithological conditions.
- Meteorological influences.
- Groundwater levels.
- Spatial distribution of surface water features.
- Existing ground and surface water quality.
- Stormwater management design criteria.
- Facility water demand.
- Facility wastewater discharge.
- Facility infrastructure design.

The standards, methods, and approaches used in this Report are sourced primarily in major government and industry technical guidance documentation and regulations. The following are resources used in the analysis of groundwater, surface water quantity and quality, local fluvial geomorphology and soil loss and erosion conditions.

Groundwater

- Hydrogeological Technical Information requirements for Land Development Applications (MOE, 1995).

- Permit To Take Water Manual (MOE, 2005).
- Best Practices for Assessing Water Takings Proposals (GLL, 2002)

Water quantity

- Water Measurement Manual. A Water Resources Technical publication. U.S. Bureau of Reclamation (USBR, 2001).
- Measurement and Computation of Streamflow. Vol. 1: Measurement of Stage and Discharge, (Rantz, 1982); Vol. 2: Computation of Discharge. U.S. Geological Survey Water Supply Paper 2175 (Rantz, 1982a).
- Flow Measurement. Performance Testing Code 19.5, American Society of Mechanical Engineers (ASME, 2004).
- Hydrometric Technicians Training Program. Water Survey of Canada - Environment Canada (EC, 1999).
- River and Streams Systems: Flooding Hazard Limit Technical Guide. Ontario Ministry of Natural Resources, Peterborough, Ontario (MNR, 2002b).
- Lakes and Rivers Improvement Act Draft Technical Guidelines – Criteria and Standards for Approval. Ministry of Natural Resources, Peterborough, Ontario (MNR, 2004).
- Stormwater Management Planning and Design Manual, Ministry of the Environment (MOE, 2003).
- Drainage Management Manual, Ministry of Transportation (MTO, 1997).
- Ontario Water Resource Act Section 34, Taking of Water, Ministry of the Environment (OWRA, 1990).
- O. Reg. 387/04, Water Taking and Transfer Regulation, Ministry of the Environment (MOE, 2004a).
- Permit to Take Water Manual, Ministry of the Environment (MOE, 2005).
- Ontario Drainage Act (RRO, 1990).
- O. Reg. 42/06, Central Lake Ontario Conservation Authority: Regulation of Development, Interference with Wetlands and Alterations to Shorelines and Watercourses. (CLOCA, 2006)
- ISO 748:1997. Measurement of liquid flow in open channels – Velocity – Area method (ISO, 1997).
- ISO 1100-1:1996. Measurement of liquid flow in open channels - Part 1: Establishment and operation of a gauging station (ISO, 1996).
- ISO 1100-2:1998. Measurement of liquid flow in open channels - Part 2: Determination of the stage-discharge relation (ISO, 1998b).
- ISO/TR 8363:1997. Measurement of liquid flow in open channels - General guidelines for selection of method (ISO/TR, 1997).
- ISO 1088:2007. Hydrometry - Velocity-area methods using current-meters - Collection and processing of data for determination of uncertainties in flow measurement (ISO, 2007).

World Meteorological Organization – Hydrological Operational Multipurpose System guidance documents:

- E70.1.02 – Manual on procedures in operational hydrology (WMO, 1998).
- K10.1.04 – Methods in hydrological basin comparison (WMO, 1999a).
- K10.2.05 – Regionalization of flow-duration curves (WMO, 1999b).
- K70.1.01 – Storm Drainage (WMO, 2000a).
- C79 – Water velocity, current meters and floats (WMO, 2000b).
- C71.3.09 – Pressure type water level gauge (WMO, 2000c).
- K22.2.02 – Hydrologic Modelling System (HEC-HMS) (WMO, 1999d).
- K35.3.06 – River Analysis System (HEC-RAS) (WMO, 1999e).
- J15.3.01 – Manual Calibration Program (WMO, 1999f).

Water quality

- A Canada-Wide Framework for Water Quality Monitoring. PN 1369. Canadian Council of Ministers of the Environment (CCME, 2006a).
- Water Quality Guidelines for the Protection of Aquatic Life – Freshwater, update 6.0, Canadian Council of Ministers of the Environment, (CCME, 2006b).
- Guidance on Sampling and Analytical Methods for Use at Contaminated Sites in Ontario, Ministry of Environment and Energy (MOE 1996).
- Water Management: Policies, Guidelines and Provincial Water Quality Objectives or the Ministry of Environment and Energy, Queens Printer for Ontario (MOEE, 1994b).
- Protocol for sampling and analysis of industrial/municipal wastewater. Ontario Ministry of the Environment (MOE, 1994c).
- Ontario Water Resource Act Section 53, 1990 (OWRA, 1990).
- Lakes and Rivers Improvement Act Draft Technical Guidelines – Criteria and Standards for Approval. Ministry of Natural Resources, Peterborough, Ontario (MNR, 2004).
- ISO 5667-1:2006, Water quality - Sampling - Part 1: Guidance on the design of sampling programmes and sampling techniques (ISO, 2006).
- ISO 5667-3:2003, Water quality - Sampling - Part 3: Guidance on the preservation and handling of water samples (ISO, 2003).
- ISO 5667-6:2005, Water quality - Sampling - Part 6: Guidance on sampling of rivers and streams (ISO, 2005).
- ISO 5667-14:1998, Water quality - Sampling - Part 14: Guidance on quality assurance of environmental water sampling and handling (ISO,1998a).

Erosion and fluvial geomorphology

- River and Streams Systems: Erosion Hazard Limit Technical Guide. Ontario Ministry of Natural Resources, Peterborough, Ontario (MNR, 2002a).
- Erosion and Sediment Control Guideline for Urban Construction, Greater Golden Horseshoe Area Conservation Authorities, December 2006 (GGHACA, 2006).
- Guidelines for Evaluating Construction Activities Impacting on Water Resources, Guidelines B-6 (MOEE, 1995)

2.1 Assumptions

Three timeframes were assumed for the analysis of potential environmental effects. These are:

- ***The Construction Period:*** The time during which the Facility would be constructed and commissioned (a 30-month period starting in June 2010).
- ***The Operational Period:*** The time during which the Facility would be operated (about 30 years).
- ***The Post-closure Period:*** The time after the Facility would be closed (after operations cease). Activities are normally limited to de-commissioning, post-closure monitoring and property maintenance.

The timeframes for the construction, operation and post-closure periods are commensurate with an undertaking of this type and scale.

Without detailed design information for the Facility, some assumptions regarding development function and processes were necessary. In addition, further assumptions regarding the physical environment were needed. The assumptions used for this assessment were as follows:

1) During the construction phase all incident precipitation to the Site would be controlled via erosion and sediment control features and contained within onsite stormwater management facilities.
2) During the operation phase, all incident precipitation would be controlled, conveyed and contained using adequately sized stormwater management features.
3) The Facility would not discharge any wastewater effluent to the surrounding surface water features.
4) The Facility infrastructure would reach approximately 7.6 mbg.
5) Regional bedrock geology adequately describes the conditions present onsite.
6) The hydrological soil group present onsite is a B.
7) The Facility would occupy all 12.4 ha of the subject property.
8) Approximately 45% of the post-construction Site would be comprised of impervious cover.
9) Approximately 2% of the existing Site can be considered impervious.
10) One stormwater end-of-pipe facility would be located in the southwest corner of the property.

11) The Facility's water supply requirements would be 115,068 L/day (42,000 m^3/yr) and would be facilitated by municipal water supply system. This water supply requirement is based on the initial design capacity of 140,000 tpy (140,000 tpy scenario) of waste material.

12) Wastewater, not including stormwater, from the Facility would be 8,219 L/day (3000 m^3/yr) and would be conveyed via municipal sewage infrastructure to the Courtice Water Pollution Control Plant located due south of the subject lands. This wastewater generation level is also based on 140,000 typ scenario of waste material.

13) Proximal water well records are a reliable method of describing onsite groundwater levels.

14) Development and operation of the Facility would not influence any federal level triggers and would therefore not involve the *Canadian Environmental Assessment Act*.

2.2 Regulatory Requirements

Construction, operation, and decommissioning of the Facility would require approval for a number of its components arising from differing levels of regulatory authority. Below is a summary of keystone components requiring regulatory approval together with the respective agency.

2.2.1 Groundwater

MOE (1995) provides the hydrogeological technical requirements necessary for land development projects involving subsurface infrastructure. Conservation of local and Regional water quality and quantity while safely designing proposed developments involves a complex mixture of stratigraphic knowledge, appropriate engineering criteria and seasonal hydrologic cycle patterns.

Under Ontario Regulation (O.Reg.) 387/04, Water Taking and Transfer Regulation (MOE, 2004a) the extraction of groundwater resources are defined and guidance criteria explained. The Permit To Take Water (PTTW) Manual (MOE, 2005) describes the application process and necessary details required to obtain a PTTW in Ontario (See Section 2.2.5). A Permit To Take Water may be required for construction dewatering.

2.2.2 Surface Water

The Provincial Policy Statement (PPS) provides guidance to planning authorities pertaining to the protection, conservation, and enhancement of the province's water quantity and quality. The PPS states planners should ensure that:

- The watershed is used as the ecologically meaningful scale of planning;
- Negative impacts including cross-jurisdictional and cross-watershed cases are minimized;
- All areas which are necessary for the ecological and hydrological integrity of a watershed are identified;
- Necessary restrictions on development and Site alterations are implemented;

- Groundwater and surface water linkage are maintained;
- Efficient and sustainable use of water resources are promoted, and;
- Stormwater management practices minimize stormwater volumes and contaminant loads.

The PPS also states that development within all sensitive surface water and groundwater features shall be avoided if possible and that mitigation measures and alternative development approaches should be utilized to protect, improve and restore the hydrologic function of these features where necessary.

Under O.Reg 42/06, the Central Lake Ontario Conservation Authority developed a *Policy for the Administration of the Development, Interference with Wetlands and Alterations to Shorelines and Watercourses Regulation.* This regulation governs any development taking place within a given distance (usually 120 m) from a watercourse, wetland or shoreline. Developments planned for this regulated area would require an EA and approval from the Conservation Authority.

2.2.3 Water Balance

A water balance is an invaluable method of characterizing the local hydrologic cycle and for predicting the potential changes that may result from the Facility. The *Stormwater Management Planning and Design Manual* (MOE, 2003) provides guidelines, methods, examples and further references pertaining to the appropriate development and interpretation of both pre- and post-development water balances.

Further information regarding the infiltration components of a water balance can be found in *Hydrogeological Technical Information Requirements for Land Development Applications* (MOE, 1995).

2.2.4 Wastewater

The following sections discuss the applicable regulatory requirements associated with wastewater management, with particular relevance to a waste management facility.

2.2.4.1 Provincial

At the provincial level, wastewater discharges are permitted through the Certificate of Approval (C of A) process by the Ministry of the Environment (MOE).

A C of A (Industrial Sewage Works) is required to establish, alter, extend or replace new or existing sewage works used for the collection, transmission, treatment or disposal of wastewater to the environment. As required under Section 53 of the *Ontario Water Resources Act* (OWRA), an application for a C of A (Industrial Sewage Works) must be submitted to the Environmental Assessment and Approvals Branch (EAAB) for the Facility in the event that it would be discharging industrial wastewater and stormwater to a receiving waterbody. It is anticipated that a C of A for stormwater would be required for the Facility (Refer to Section 2.2.6).

Generally, effluent water quality criteria are referenced to the Provincial Water Quality Objectives (PWQOs) in light of the water quality of the receiving waterbody.

Any waste material that cannot be treated onsite or sent to a wastewater treatment plant would be transferred to an appropriate waste storage or treatment facility in accordance with O.Reg. 347.

Trucking the Facility's wastewater offsite would not be subject to either of the above mentioned provincial regulations yet would be governed by MOE waste management protocol. A waste transfer permit to dispose of wastewater at a treatment facility would be required.

2.2.4.2 Municipal

Wastewater discharge from the Facility would likely be routed to the municipal sewage infrastructure. The effluent would be required to meet the Regional Municipality of Durham's guidelines in Part 2 of the Sewer Use By-law 43-2004. In the event that the wastewater was unable to meet the guidance criteria required for direct discharge, an onsite water treatment facility may be required. An amendment may also be filed to discharge effluent outside of the range indicated by the Sewer Use By-law; however, a special discharge agreement with the municipality would be required.

2.2.5 Water Supply and Water Taking

Water demand for a Thermal Treatment Facility can vary depending on the specific configuration of the facility. Preliminary design estimations indicate that water demand would be 115,068 L/day. Water takings are regulated by different federal, provincial and municipal agencies, each with its own area(s) of jurisdictional interest. There are no federal level interests within this project.

2.2.5.1 Provincial

The MOE is involved with monitoring an industry's interactions with the environment, as well as enforcing its laws and regulations. The MOE exerts authority over water supply through the issuance of PTTW when the extraction of more than 50,000 L/day from ground or surface water sources is proposed. Water takings in Ontario are governed by the OWRA and the Water Taking and Transfer Regulation (O. Reg. 387/04), a regulation under the OWRA.

The Regulation may restrict water takings in high use watersheds and the Great Lakes Basin. There is the potential for the development foundations to interfere with the water table, and dewatering may be required during construction activities. In this scenario, a Category 2 Permit to Take Water may need to be obtained. According to the PTTW Manual (MOE, 2005), water takings for the purpose of fire fighting does not require a PTTW.

2.2.5.2 Municipal

The local Municipality would govern the use of, and connection to, the pre-existing watermains. Approval to utilize such resources would depend upon supply line capacity, existing supply line proximity to the Site and any prohibitive costs associated with the proposed connection.

2.2.5.3 Other Requirements

Under the *Planning Act*, 1990 – Part 1: Provincial Administration: Section 2, the Minister of Municipal Affairs and Housing, the council of a municipality, a local board, a planning board and the Municipal Board, in carrying out their responsibilities under the *Planning Act*, may intervene in matters of provincial interest such as:

- The protection of ecological systems, including natural areas, features and functions;
- The supply, efficient use and conservation of energy and water;
- The adequate provision and efficient use of communication, transportation, sewage and water services and waste management systems;
- The minimization of waste; and,
- The protection of public health and safety.

2.2.6 Stormwater

Stormwater is considered wastewater, and therefore stormwater management facilities are deemed sewage works under Section 53 of the OWRA. Section 30 of the OWRA prohibits the discharge of polluting materials into any water. As a result, a C of A (Industrial Sewage Works) is required for stormwater management facilities (See Section 2.2.4).

MOE's Stormwater Management Planning and Design Manual (MOE, 2003) provides guidance for stormwater management planning, design and implementation for construction, operational and closure phases. The Provincial Policy Statement defers to the MOE's stormwater manual for stormwater guidance.

The following Ministry of Natural Resources (MNR) documents provide additional details on the requirements for assessing flooding, flood proofing, erosion and slope stability impacts and performing hydrologic and hydraulic analysis:

- Understanding Natural Hazards, 2001 (MNR 2001);
- Technical Guide - River and Stream Systems: Flood Hazard Limit, 2002 (MNR, 2002);
- Technical Guide - River and Stream Systems: Erosion Hazard Limit, 2002 (MNR 2002); and,
- Great Lakes - St. Lawrence System and Large Inland Lakes. Technical Guides for Flooding, Erosion and Dynamic Beaches in Support of Natural Hazards Policies 3.1 of the Provincial Policy Statement (MNR 2001).

The Conservation Authority has an interest in stormwater management as part of their fiduciary responsibilities under O. Reg. 42/06 Development, Interference with Wetlands & Alteration to Shorelines & Watercourses Regulation, as outlined in Section 2.2.2.

2.3 Information Sources

A wide array of information sources, hydrological modeling and field investigations were utilized in the completion of this report. The review of background information included the following:

- Durham Groundwater Management Study (GLL, 2003);
- York-Peel-Durham-Toronto - Conservation Authorities Moraine Coalition (YPDT-CAMC) Groundwater Management Study (2006);
- Soil Survey of Canada #9 (1947);
- Watershed Resource Management Plans (CLOCA, 2002);
- Aquatic Resource Management Plans (Lynde Creek (CLOCA, 2006), Soper/Bowmanville Creek (DFO *et al*, 2000);
- Meteorological Service of Canada's climate data;
- Geotechnical Investigation Report (Jacques Whitford, 2008);
- Natural Heritage Information Centre (NHIC);
- Water Survey of Canada's HYDAT CD;
- Provincial Water Quality Monitoring Network (PWQMN);
- Courtice Water Pollution Control Plant Environmental Screening Document;
- Courtice Water Pollution Control Plant Geotechnical Investigation Report (Geo-Canada Ltd., 2004);
- Draft Clarington Energy Business Park Master Drainage Plan (Aecom, 2009); and,
- Ministry of the Environment's Water Well Information System (WWIS).

The modeling performed for the study included:

- Existing water balance;
- Storm class post-development runoff assessments;
- Stormwater quantity and quality control;
- Channel conveyance capacity assessments;
- Preliminary stormwater pond capacity and discharge considerations; and,
- Existing and post-development soil loss.

Field investigations completed for this study included:

- Site reconnaissance;
- Discharge swale survey;
- Receiving water flow characterization;
- Receiving water quality sampling; and,
- Geomorphological assessment of receiving waters.

3.0 DESCRIPTION OF EXISTING CONDITIONS

3.1 Site Description

Located south of Highway 401, the Site is situated on the west side of Osbourne Road immediately north of a CN Rail corridor. Within the "Application of Short-lList Criteria", documents (Genivar and JW, 2007) this Site was called Short-list Site: Clarington 01. The Site occupies approximately 12.4 ha of rural land and is located in the Clarington Energy Business Park.

The lands to the east and west of the Site are currently undeveloped and used for agricultural land uses. Directly south of the Site (between Lake Ontario and the CN Rail corridor) is the Courtice WPCP (See Figure 3-1) which includes a paved access route built along the eastern edge of the Site. The Darlington Nuclear Generating Station is situated approximately 1.8 km to the east. The Site is within the designated Clarington Energy Business Park.

3.1.1 Climate

The subject property is located within the Dfb climatic Region of Ontario as described by the Köppen climate classification system. (Dfb extends across the southern portion of Canada from Newfoundland to British Columbia.) This climate has an average temperature of above 10^oC in the warmest months and a coldest month average below -3^oC. Typically located in the interiors of continents, these climates have warm, humid summers and cold winters with an average annual precipitation of between 750 and 1000 mm that is fairly evenly distributed throughout the year with a slight summer peak (McKnight & Hess, 2008).

Climate data was obtained from Environment Canada (EC) online (2008). Climate normals, calculated by averaging 30 years of climatic data (1971 to 2000), were used to represent the average conditions present at the Site. Climate normals were derived from precipitation and temperature data from the Oshawa, Ontario weather station ID# 6155878 which is approximately 6.5 km away from the Site.

The total annual precipitation for the Oshawa station is approximately 877.9 mm with the wettest month being September (87.9mm) and the driest being February (52.7 mm). Average temperatures are below freezing from December to February, and are highest in July at 20.3^oC (Environment Canada, 2008). See Table 3-1 for a summary of temperature and precipitation climate normals.

The Site is within 1 km of the Lake Ontario shoreline. This proximity would have significant influence over the average temperature, wind speed, wind direction and humidity experienced onsite. In general the climatic effect of the lake moderates temperature by 1 to 2 degrees (DFO et al., 2000)

A closer review of the most recent 37 years of meteorological trends suggests that when compared to the climate normals for the period, the average annual temperature is increasing and the average annual precipitation shows a slight decreasing trend. See Figure 3-2 for average annual temperature deviation from Environment Canada's Climate Normals for the Oshawa WPCP station. See Figure 3-3 for average annual precipitation deviation from Environment Canada's Climate Normals for the Oshawa WPCP station.

YORK REGION AND DURHAM REGION
RESIDUAL WASTE STUDY

Clarington 01 Site Location

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- Courtice Water Pollution Control Plant
- Railway
- Collector
- Highway
- Clarington 1 Site Area
- Clarington Energy Park
- Municipal Lower tier Boundaries
- Wooded Area

Hydrology

- Watercourse
- Waterbody
- Wetland Area

0 100 200 300

Metres - 1:10,000

1009497-015

Jacques Whitford

FIGURE NO.

3-1

Last Modified: Feb 20, 2009 By: S.Allen

Table 3-1 - Climate Normals Data for Oshawa, Ontario Weather Station ID# 6155878

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Temperature:													
Daily Average (°C)	-5.3	-4.4	0.1	6.3	12.3	17.2	20.3	19.6	15.5	9.2	4	-2	7.7
Daily Maximum (°C)	-1.4	-0.6	4.1	10.5	17	21.9	25	24	19.7	13.1	7.2	1.5	11.8
Daily Minimum (°C)	-9.2	-8.2	-3.8	2	7.6	12.4	15.5	15.2	11.2	5.2	0.7	-5.4	3.6
Precipitation:													
Rainfall (mm)	32.1	29.5	46.8	70.1	74.7	80.6	67.3	83.3	87.9	66.2	74.2	46.8	759.5
Snowfall (cm)	38.9	23.2	15.5	3.1	0	0	0	0	0	0.1	5.7	31.9	118.4
Total Precipitation (mm)	71	52.7	62.3	73.1	74.7	80.6	67.3	83.3	87.9	66.3	79.9	78.7	877.9
Snow Depth at Month-end (cm)	10	4											
Extreme Daily Rainfall (mm)	42.6	42.8	32.8	47.6	41.6	144.8	39.8	75.4	80.8	45.6	59	35.6	
Date (yyyy/dd)	1979/24	1985/23	1991/27	1984/04	2000/12	1971/27	1985/15+	1986/26	1986/10	1995/05+	1985/03	1969/10	
Extreme Daily Snowfall (cm)	27.9	27	18.4	20.3	0	0	0	0	0	6.6	17.8	29	
Date (yyyy/dd)	1977/08	1988/11	2001/05	1975/03	1970/01+	1970/01+	1970/01+	1969/21+	1969/01+	1969/21	1972/19	1992/10	
Extreme Daily Precipitation (mm)	42.6	42.8	32.8	47.6	41.6	144.8	39.8	75.4	80.8	45.6	59	39.1	
Date (yyyy/dd)	1979/24	1985/23	1991/27	1984/04	2000/12	1971/27	1985/15+	1986/26	1986/10	1995/05+	1985/03	1972/12	

Figure 3-2 Average Annual Temperature Deviation From Environment Canada's Climate Normals (1971-2000) (YPDT-CAMC, 2008)

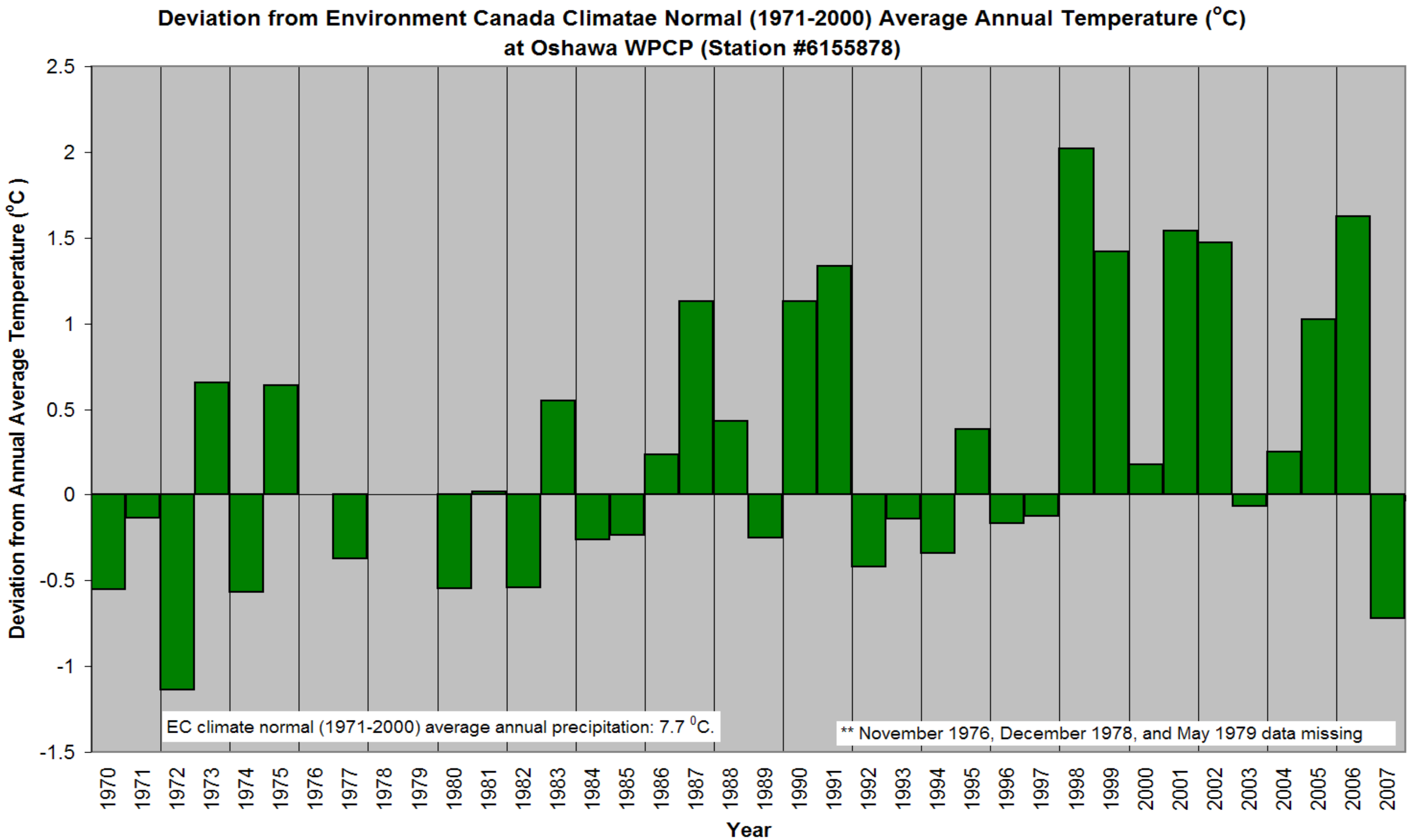
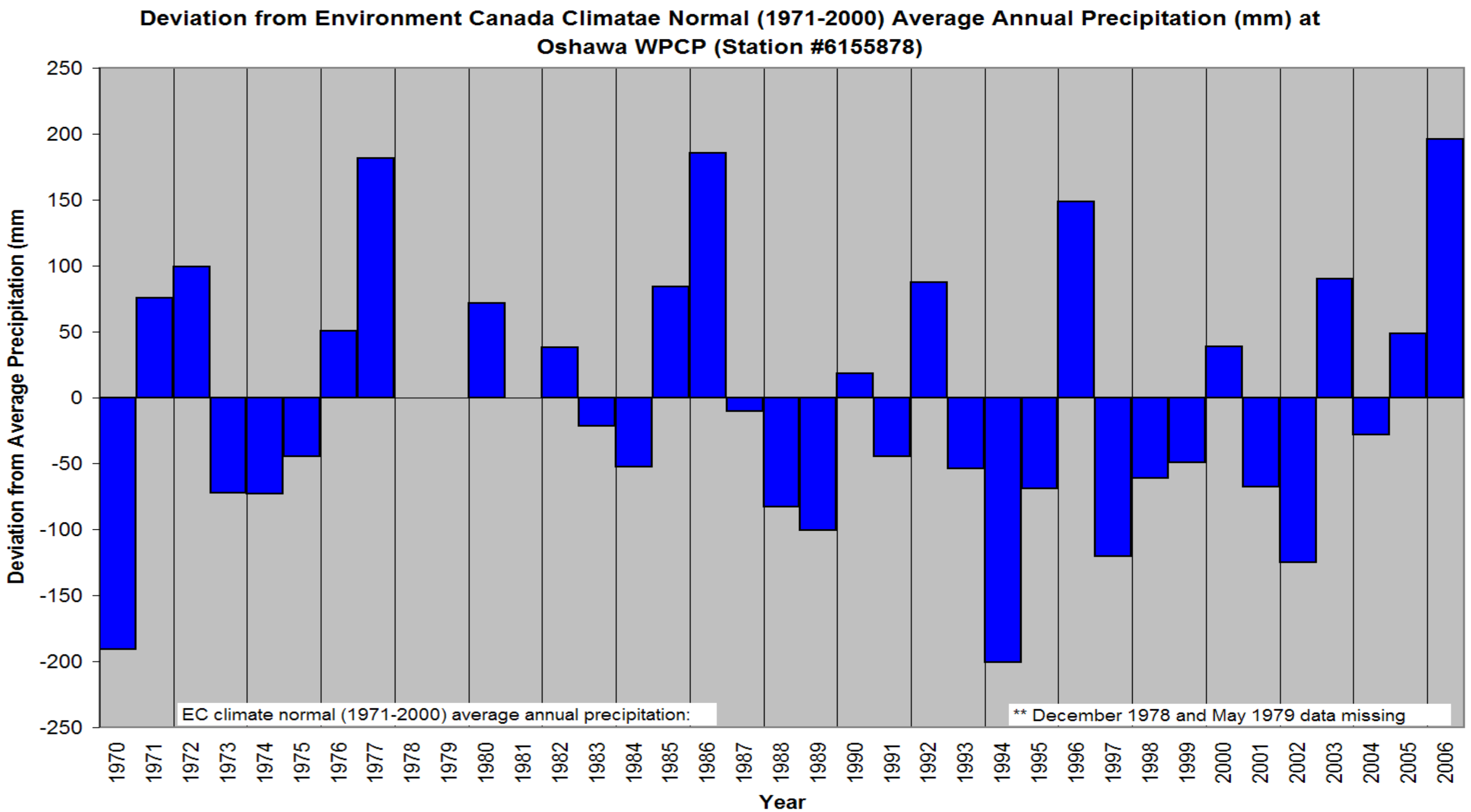


Figure 3-3 Average Annual Precipitation Deviation From Environment Canada's Climate Normals (1971-2000) (YPDT-CAMC, 2008)



3.1.1.1 Climate Change Effects on Water Resources

Climate change is a subject of intense investigation by EC, Natural Resources Canada, as well as many other scientific organizations. These research efforts focus on the prediction of climate change over the coming century, on the interpretation of impacts and on generating recommendations regarding adaptation to climate change impacts. The following represents a summary of climate change impact findings related to surface water resources in southern Ontario predicted by EC (1997), the Canadian Environmental Assessment Agency (CEAA, 2004), the Union of Concerned Scientists and the Ecological Society of America (UCS-ESA, 2003), and Natural Resources Canada (NRC, 2004). These include:

- Increases in water temperature in the Great Lakes could reduce the frequency of lake turnover, which could affect aquatic ecosystems;
- The warming of Great Lakes waters in the summer could cause fish species to shift northward;
- Increasing water temperature and changes in summer stratification in the Great Lakes and inland lakes and streams of the Region would affect fundamental physical, chemical, and biological processes in lakes. Higher temperatures would result in lower oxygen levels. Phosphorus release would be enhanced and mercury release and uptake by biota would also be likely to increase. Some heavy metals would be likely to respond in a similar fashion;
- Increasing nutrient release can lead to eutrophication, causing increased algal growth, including noxious algal blooms and degraded water quality and ultimately reduce fish production in lakes;
- Climatic and hydrologic modelling predictions for the Region also suggest that over the next 100 years precipitation (rainfall) would increase during winter and spring. This could increase the magnitude of spring floods, especially if the floods coincide with snowmelt when soils are still frozen;
- Changes in the frequency or intensity of extreme events would have consequences for the increased flooding and erosion risk;
- Average water levels in the Great Lakes may decline to record low levels;
- Dramatically lower lake levels would reduce the maximum capacity of vessels and could increase operating costs for ports and shipping channels. Changes in shipping conditions on the Great Lakes could affect demand for bulk shipment by rail;
- River flows are expected to become more variable in the future with more flash floods and lower minimum flows. Intense rainfalls increase the risk of flooding and the release of contaminants into receiving waters;
- While water demand could increase during the summer months, water supply capacity from surface and groundwater sources is expected to decrease;
- Anticipated changes in the hydrologic cycle could result in more variability in water supply for hydroelectric power production and further restriction of large water takings;
- Changes in wetlands and littoral areas could alter their efficacy as spawning and nursery areas;

- Some wetlands could retract or disappear, some new wetlands could be created while others may expand, or migrate;
- Extrapolations from 80 to 150 years of record suggest that ice cover will decline in the Great Lakes in the future;
- Reduced ice on the Great Lakes is expected to increase the length of the shipping season;
- Decreased snow load may result in reduced cost of buildings and infrastructure maintenance (i.e. snow clearing);
- The need for snow removal could be reduced in Southern Ontario; and,
- More frequent freeze-thaw cycles could increase weathering.

3.1.2 Soils

The Site is located on the Iroquois Plain (Chapman & Putnam, 1984) approximately 600 m north of the sand bluffs that make up the current Lake Ontario shoreline. Within a Regional context soils are known to be fertile, rich loams of the Darlington and Newcastle series. These soils are calcareous in nature and considered to be Class 1 agricultural lands (AE Environmental Associates, 1998). According to the Soil Survey of Durham County (Soil Survey of Ontario No. 9, 1946), the Site is mostly underlain by a Darlington Loam with the northeast corner of the property underlain by a Newcastle Clay Loam.

Darlington Loams are characterized by dark grey brown heavy loams and greyish loams over weak reddish brown clay loam which are then underlain by grey stoney compact loams. The Newcastle Clay Loam is characterized by dark grey brown clay loams and light brown loams over weak reddish brown clay which is then underlain by silt and clay.

According to The *Physiography of the South Central Portion of Southern Ontario*, map #2226, (Department of Mines and Northern Affairs, 1972) the Site is located in the middle of a large clay plain (See Section 3.1.3 for explanation of subsurface conditions).

A review of water well records within a 1 km radius of the Site supports the above mentioned stratigraphy with a majority of well logs indicating high clay percentages within the upper 10 m. Fourteen of the 39 well records (35%) with stratigraphy information show exclusively clay textures within the upper 9 m. Another 20 wells describe subsurface horizons comprising clay mixtures with sand, silt, stones and gravel. Below the 9 m elevation many well records describe a heterogeneous composition with clay still being the major constituent.

In contrast to the regionally-based information, a geotechnical investigation of the Site conducted by Jacques Whitford in early 2008 suggested a sand and silt dominant subsurface. The investigation included the advancing of 17 boreholes to depths of 5.1 to 12.2 mbg. A particle size distribution conducted on soils at varying depths within each borehole indicated that the general Site’s underlying lithology is as follows:

- Topsoil (average depth 395 mm ranging from 300 to 620 mm thick);
- Periodic thin layer of sandy-silt; and,

- Silty-sand to the extent of the borehole depth.

The topsoil horizon was described as a black to dark brown sandy silt and/or silty sand. See **Appendix A** for a complete record of borehole logs from the geotechnical investigation.

The geotechnical investigation conducted on the immediately adjacent Courtice WPCP (Geo-Canada, 2004) described the underlying lithology as follows:

- Topsoil;
- Sandy Silt, Silty Sand to Sand and Silt (Glacial Till);
- Sandy Clayey Silt to Clayey Silt (Glacial Till);
- Silty Clay to Clayey Silt; and,
- Gravel and Sand.

These geotechnical investigations describe a considerably different lithological regime than the general Regional scale studies for the area suggest. It appears that the Site may be situated upon a localized bluff or ridge representing a remnant of former glacial activities. This inference is supported by geological material described in Section 3.1.3 below.

3.1.3 Geology

The physiography and surficial geology of the Region play important roles in varying components of the hydrologic system. The major geological feature in the area is the Oak Ridges Moraine (ORM) to the north of the Site. The ORM is comprised of sand and gravel to a depth of approximately 100 m. Beneath this lies a dense till material on top of bedrock.

South of the ORM lies the wide east-west running, Iroquois Shoreline and Plain which have a complex geological history. During the Pleistocene time, receding continental ice sheets left behind thick glacial debris in the form of ground moraines and drumlins. This area was then inundated by post-glacial Lake Iroquois (the precursor to Lake Ontario) which deposited fine grained soils, predominantly clays and silts, which did not completely cover the moraines and drumlins previously dominating the landscape. As a result, the area is characterized by rapidly changing subsurface conditions with drumlin ridges separating lacustrine deposits (Geo-Canada, 2004). The Facility is located within the Iroquois Plain several kilometres south of the Iroquois Shoreline.

The Iroquois Plain is primarily a dense Newmarket till, comprised of clay silts and sand. The Newmarket Till can be traced across much of the Greater Toronto Area (GTA) (Sharpe et al., 1999).This layer is estimated to be between 25 and 30 m in depth (Genivar and Jacques Whitford Ltd. 2007, CLOCA 2008a, DFO et al. 2000).

This layer is underlain by a thin (approximately 5 m) layer of intertill sediments (DFO *et al*., 2000). This underlying layer includes both Thorncliffe and Scarborough formations. Directly beneath this thin layer is Whitby shale bedrock.

Boreholes drilled as part of the geotechnical investigation conducted on the Site (Jacques Whitford, 2008) were not advanced deep enough to reach bedrock. However, boreholes drilled as part of the Environmental Assessment of the Courtice WPCP, located approximately 150 m south of the Site, identified bedrock at a depth of 16 m (DFO, 2005). This same report made note of several rock outcroppings within close proximity of the Site.

Since overburden thickness increases with distance from the Lake Ontario shoreline (DFO *et al.,* 2000), it is assumed that overburden depth on the Site may be several metres deeper than the more southern location.

3.1.4 Topography

The general slope of Durham Region is from the northeast to the southwest, originating with the ORM in the north and sloping to the Lake Ontario shoreline in the south. The ORM has an elevation of between 275 to 375 m above mean sea level and the Lake Ontario shoreline is approximately 90 m above mean sea level which corresponds to an average slope of 1.5%.

Although the ORM has a moderate slope and a very hummocky terrain, the Iroquois Plain is much flatter and has a more uniform grade. The average slope of the Iroquois Plain is 1% (DFO *et al*, 2005).

Drumlins and eskers dominate the Durham Region landscape as evidence of the most recent glacial retreat. The Site contains undulating to rolling hills and a general southwestern slope of 1.9% based on a detailed Site topographic survey (*Region of Durham Residual Waste Disposal Study*, 2005). Figure 3-4 presents the existing Site topography at and surrounding the Site.

YORK REGION AND DURHAM REGION RESIDUAL WASTE STUDY

Existing Site Topography

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Collector
Highway
1m Contour
Railway
Clarington 1 Site Area
Clarington Energy Park
Municipal Lower tier Boundaries

Hydrology
Watercourse
Waterbody
Wetland Area

0 50 100 150

Metres - 1:5,000

1009497-014

Jacques Whitford

FIGURE NO.

3-4

Last Modified: April 21, 2009 By: S.Allen

3.1.5 Vegetation

During a Site visit in December of 2008, the subject lands were comprised of either ploughed fields or grassed meadows. A review of aerial photographs suggests that in the recent past the entire development Site and surrounding area has been tilled for agriculture. The dominant crop harvested from this property is hay (Jacques Whitford and Genivar, 2007b).

From a natural heritage perspective, all of the onsite fields, as well as the surrounding lands, have been highly altered and represent an area of extreme and continuous disturbance. Vegetation species in the area are comprised of feral forage crops and invasive European species (DFO, 2005).

The Site is composed of four fields with a periphery of hedgerows. The hedgerows consist of a variety of common shrub and tree species representative of the area. The area surrounding the Site consists of fallow and cultivated agricultural fields, which contain hedgerows with similar shrub and tree species. The loss of sensitive vegetation as a result of the development of this Site would be minimal (Jacques Whitford and Genivar, 2007b).

A small quantity of the property has already been grubbed to construct an access road, which runs westward from Courtice Road to the centre of the property where it turns south and continues until it approaches the CN Rail tracks. A small grassed drainage swale has been constructed alongside the south running access road. It is presumed that its major function is to drain the northern fields past the east-west access road.

3.2 Water Balance

A water balance for the pre-development conditions was conducted to characterize the Site's natural drainage and storage and infiltration of incident precipitation. A discussion of post-development water balance implications from the Facility is provided in Section 4.2. The Site falls entirely within the Tooley Creek watershed (Figure 3-5).


YORK REGION AND DURHAM REGION
RESIDUAL WASTE STUDY
Tooley Creek Watershed
Produced by Jacques Whitford under Licence with the Ontario Ministry of Natural Resources © Queen's Printer for Ontario, 2004-2008
Railway
Collector; Local / Street
Highway
Clarington 1 Site Area
Clarington Energy Park
Tooley Creek Watershed Boundary
Municipal Lower tier Boundaries
Wooded Area
Hydrology
Watercourse
Waterbody
Wetland Area
0
250
500
750
Metres - 1:25,000
1009497-016
MB
QC
ON
USA
Area of Interest
Jacques Whitford
FIGURE NO.
3-5
Last Modified: April 15, 2009 By: S.Allen
N
NASH ROAD
COURTICE ROAD
SOLINA ROAD
RUNDLE ROAD
GREEN ROAD
2 HIGHWAY
HANCOCK ROAD
TRULLS ROAD
BLOOR STREET
CLARINGTON
MAPLE GROVE ROAD
HOLT ROAD
Tooley
BASELINE ROAD
BLOOR STREET EAST
Tooley Creek
TOWNLINE ROAD SOUTH
PRESTONVALE ROAD
CIGAS ROAD
SOUTH SERVICE ROAD
OSHAWA
401 HIGHWAY
DARLINGTON PARK ROAD
DOWN ROAD
OSBOURNE ROAD
Lake Ontario

Water balance calculations were carried out with the program THORNPRO. This software is designed to calculate the Thornthwaite Water Budget (Thornthwaite and Mather, 1957). The program provides rapid determination of potential evapotranspiration and other hydrologic environment components (Black, 1996).

The general equation that describes the water balance estimation is:

$$P = ET + RO + I \qquad \text{(equation 3.1)}$$

Where:

P = precipitation
ET = evapotranspiration
RO = surface runoff
I = infiltration and storage

Thornthwaite and Mathers method relies on the amount of energy available to evaporate water from free water surfaces such as streams, wetlands, ponds, lakes, oceans, and the intercepting surfaces on which it falls as precipitation. Water loss can also take place in vegetation at the openings of stomates normally on the lower surface of leaves. Energy also vaporizes water drops present in the atmosphere.

In this model, the change of state of water is a function of the amount of water and energy available. That, in turn, is governed by the temperature, latitude, length of day and season which combine to control the amount of energy received at the earth's surface. Infiltration factors and vegetation type then control the fraction of excess water that infiltrates into the ground to recharge groundwater versus the fraction that runs off to nearby streams as baseflow.

To adequately describe the amount of both energy and water within a given system, the Thornthwaite and Mather method requires the input of monthly or daily temperature and precipitation, Site hemisphere, latitude and elevation, vegetation type, land use, soil storage characteristics and study area size, slope, and relative location within the governing watershed.

3.2.1 Input Parameters

Water balance calculations also require the input of local land use, geographical, and environmental characteristics to further identify site specific conditions. Using aerial photography, GIS applications, and Regional soil data, parameters best representing the Site were chosen. See Table 3.2 for the list of input parameters used in the development of the water balance model.

Table 3-2 - Summary of Water Balance Input Parameters

	Latitude	Longitude	Elevation (masl)
Climate station	43.86 N	78.83 W	102
Proposed site	43.87 N	78.75 W	98
Parameter	**Value**	**Note**	
mm of soil storage	150	Sandy silt to silty sand	
Drainage Area (ha)	12.4	Proposed Site	
Land Use	Rural	Mixture of agric. and rural	
Watershed Location	Lower 1/5	Near L. Ontario discharge point	
Slope (m/m)	0.019	Average for site	
Other	Well drained	Sandy upper horizon	

3.2.2 Evapotranspiration

Evapotranspiration (ET) estimations were obtained using the Thornthwaite and Mather Method. This method calculates ET amounts based on average monthly temperatures, precipitation, soil storage and vegetation cover type. To facilitate calculations the ET values were calculated with the THORNPRO software package (Black, 1996).

Since the method uses monthly temperature averages and estimates of the transpiration of vegetation, it was assumed that in months with average temperatures below 0°C (Jan, Feb, Mar, Dec) there was no ET. Following the same assumption, ET was assumed to reach its peak value in July in agreement with the peak in temperature according to the climate data.

A review of hydrometeorological data for Ontario suggests that the average ET rate for the St. Lawrence and Great Lakes basin is 472.2 mm per year (Fernandes, 2007). The same study has suggested that in Ontario ET values have risen almost 10% (or 0.698 mm/yr) between 1961 and 2000 while precipitation had remained generally constant.

The annual loss to ET was estimated to be 514 mm or 59% of the total incident precipitation to the Site. This is slightly higher than the 472.2 mm Regional average suggested in Fernandes (2007). Topographic relief generally decreases within the lower reaches of a watershed which could act to induce precipitation ponding and subsequent increases in evaporation.

3.2.3 Runoff

Runoff estimations were obtained using the Thornthwaite and Mather method within the THORNPRO software package (Black, 1996). Runoff is calculated based on the antecedent moisture conditions, soil type, slope and vegetative cover of the Site in question. A steep slope with little vegetation and tight soils will contribute significant quantities of precipitation to surface water features via direct runoff. In addition, if the soil moisture storage capacity is low or heavy rains have saturated the local soil medium, newly introduced precipitation will be more likely to runoff then infiltrate into the soil.

Runoff values within an Ontario context can vary significantly depending upon the Site's proximity to or location within an urbanized watershed. Tooley Creek watershed comprises a mixture of agricultural and rural land uses with very little in the way of urban development. The only areas within the watershed that contain hardened surfaces (imperviousness) are a few small hamlets and the network of municipal and arterial roadways.

The YPDT-CAMC groundwater study (Earthfx, 2006) estimated an average water surplus (precipitation-actual ET) for the rural areas within the study area of 290 mm/yr.

The geotechnical investigation conducted on the Site (Jacques Whitford, 2008) indicated that the Site consists of a shallow layer of topsoil underlain by several metres of sandy till. These results coupled with a relatively low slope on the Site (0.019 m/m), suggests that infiltration of incident precipitation is far more probable then was expected given the Regional soil characteristics.

The water balance model estimated the annual runoff for the Site to be 331 mm or 38% (runoff coefficient) of total annual precipitation. Currently, the precipitation incident upon the Site drains southwest via overland flow towards Tooley Creek approximately 1000 m away. The Facility would be designed to capture this component of the water balance in its entirety.

3.2.4 Infiltration

Infiltration estimations are calculated through the equation (adaptation of equation 3.1):

$$\mathbf{I = P - RO - ET} \qquad \textbf{(equation 3.1a)}$$

Where:

I = Infiltration
P = Precipitation
RO = Surface Runoff
ET = Evapotranspiration

Infiltration values are dependent upon antecedent moisture conditions, soil permeability and slope. In effect, incident precipitation that is not infiltrated is lost to ET and runoff.

Infiltration can be broken down further into two sub-components; baseflow, and recharge. Baseflow describes the infiltrated precipitation that travels horizontally through the unsaturated upper soil horizons as interflow until its eventual discharge into surface water features. In contrast, the recharge component of infiltration is explained as all water that migrates vertically downward eventually recharging the groundwater aquifer.

Baseflow and recharge components are estimated using the infiltration factor described in the Ministry of the Environment's, *Hydrogeological Technical Information Requirements for Land Development Applications* (MOE, 1995). The sum of factors for topography, soil, and vegetation, the infiltration factor, is used to compute the proportion of total infiltration that is contributed to groundwater recharge. Reciprocally, "1 - infiltration factor" will compute the baseflow discharged to watercourses.

The infiltration factor for the Site was calculated to be 0.66, representing values of 0.28, 0.2 and 0.18 for topography, soil and vegetation respectively. This implies that 34% of infiltrated precipitation would be discharged to surface water via baseflow.

The *Durham Region Groundwater Use Assessment* (GLL, 2003) used GIS layers for topography, vegetative cover and soil types to interpolate an infiltration factor for the Region. This study suggested a soil value of 0.2, a cover value of 0.1 and a topographic value of 0.2, equating to an overall infiltration factor of 0.5 for the Site. This is slightly lower than that calculated using site-specific information noted above.

Although no watershed study could be found specifically relating to Tooley Creek, the Soper/Bowmanville Creek watershed, one with similar slopes, landuses, landforms and soil type, has been examined as part of an Aquatic Resource Management Plan report in a joint effort by the Department of Fisheries and Oceans (DFO), the Central Lake Ontario Conservation Authority (CLOCA), and the MNR (DFO *et al,* 2000). Both the Tooley and Soper/Bowmanville Creek watersheds drain approximately north to south from their origins near the ORM to their discharge point in Lake Ontario. Within this north-south orientation a considerable range of infiltration rates are present as a result of considerable subsurface heterogeneity.

The ORM (northern extent) represents an area of high infiltration and lateral subsurface transmission due to the sandy gravel textures. The Site is located in the lower 1/5th of the Tooley Creek watershed which lies in an area underlain primarily of lacustrine deposits which are not conducive to high infiltration rates or lateral flow. The Soper/Bowmanville Creek study (DFO *et al*, 2000) indicated infiltration rates for longitudinal sections of watershed and not specifically for soil type. The Iroquois Plain through the Site is said to have infiltrations rates <150 mm per year.

Although the geotechnical investigation conducted onsite (Jacques Whitford, 2008) suggested that the subsurface comprised a glacial till, the general Regional lithology includes thick layers of lacustrine deposits. The YDPT-CAMC groundwater study (Earth*fx*, 2006) indicated that the Newmarket Till in the area would facilitate approximately 30 mm of recharge per year.

The total infiltration calculated for the Site was 32.9 mm per year. Of that value, approximately 21.6 mm (0.66*32.9) is routed into the overburden aquifer as recharge and the other 11.2 mm ((1-0.66)*32.9) migrates laterally towards surface water features as baseflow. See Table 3-3 for a summary of water balance estimates.

3.2.5 Pre-Development Water Balance Results

Below in Table 3-3 is a summary of the pre-development water balance results.

Table 3-3 - Pre-development Water Balance Results

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Precipitation (mm)	71	52.7	62.3	73.1	74.7	80.6	67.3	83.3	87.9	66.3	79.9	78.7	877.9
Evapotranspiration (mm)	0	0	0	30	72	104	86	88	80	40	14	0	514
Runoff (mm)	0	0	50	128	66	36	18	12	8	3	10	0	331
Infiltration and Storage (mm)	71	52.7	12.3	-84.9	-63.3	-59.4	-36.7	-16.7	-0.1	23.3	55.9	78.7	32.9

3.3 Groundwater

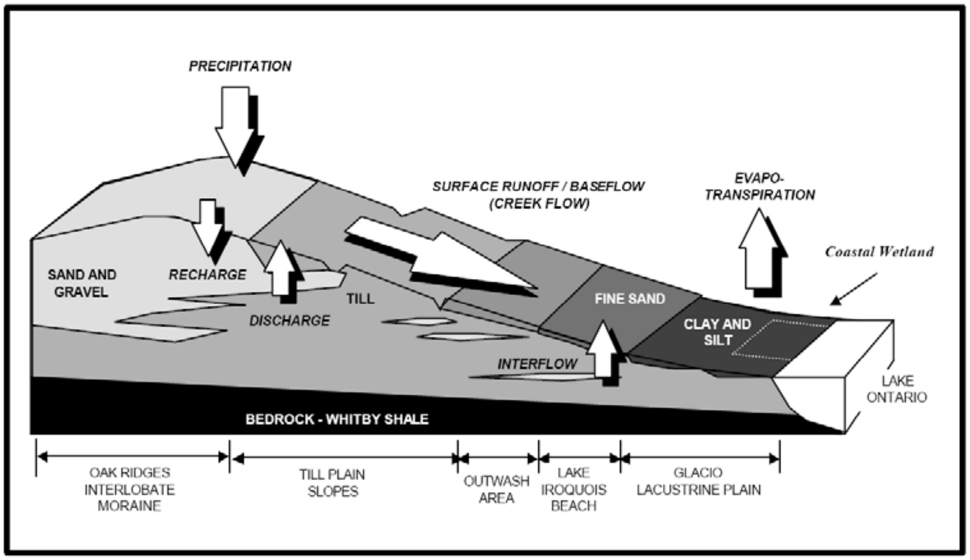
The Site is planned to have an imperviousness of approximately 45% and an infrastructure excavation depth of approximately 7.6 mbg, both of which may impact the groundwater elevation and/or migration patterns. The release of stormwater and airborne particulate (fugitive emissions), if dealt with incorrectly, may negatively affect groundwater quality. This hydrogeological assessment would characterize the currently existing conditions to provide a baseline for future comparative analysis.

3.3.1 Geological Influences

Groundwater flow in this Region is generally a subdued version of the surrounding topography that slopes from northeast to southwest towards Lake Ontario (DFO et al., 2000, Geo-Canada, 2004). The northern watershed divide, the ORM, comprises a mixture of sand and gravel deposits which allows considerable infiltration and southward baseflow. This baseflow travels laterally and discharges to local surface water features along the way, with a substantial Region of recharge/discharge noted within the Iroquois Beach/Shoreline area (DFO et al., 2000). See Figure 3-6 for general cross-sectional view of the Regional hydrological/hydrogeological interactions.

The Iroquois Plain Region is generally underlain by a dense Newmarket Till with low permeability and limited infiltration potential. This layer may act as an aquitard and create confined aquifer conditions in the thin layer of permeable till beneath (approximately 25-30 mbg) (DFO et al., 2000). Baseflow/interflow which is not discharged along the north-south route migrates beneath this aquitard layer where it is unable to move vertically upwards due to the low permeability of the layer above. This silty clay plain may have artesian conditions that are held under pressure by the aquitard (DFO et al., 2000).

Figure 3-6 Cross-Sectional View of Regional Hydrologic Cycle



Source: DFO *et al.,* 2000.

According to the Regional scale lithological and geological models, the Site would be underlain by glaciolacustrine deposits and subsequently the dense Newmarket till suggesting low infiltration potential. However, isolated drumlin ridges higher in elevation than the lacustrine deposits have been identified in the Region. Studies conducted as part of this EA have suggested that the Site may be situated on an above mentioned feature.

The geotechnical investigation conducted onsite (Jacques Whitford, 2008) and at the adjacent Courtice WPCP (Geo-Canada, 2004) suggested a predominantly sandy silt and silty sand to approximately 10 m which would lead to favourable conditions for infiltration onsite. The water balance developed for the Site (See Section 3.2) suggests a low to moderate infiltration rate for the area. The boreholes advanced on the Courtice WPCP were terminated at bedrock in most cases and never identified a higher permeability layer beneath the Newmarket Till that was capable of serving as a confined aquifer unit.

The Site appears to be on an isolated drumlin ridge, presumably a remnant of the past glacial recession. A watershed study was conducted on Tooley Creek (CLOCA, 2008c) which suggested a hydrological soil group of C for the area encompassing the Site. Given the results of the geotechnical investigation (Jacques Whitford, 2008) a hydrological soil group of B was used for modeling in this report.

The Site would not act as an area of significant discharge (GLL, 2003) nor is it likely to act as an area of significant recharge given the presence of the Regional Newmarket till in the area.

3.3.2 Groundwater Levels

Within the broader Regional context, groundwater flow reflects the general topographical gradient running from north to south. In general, groundwater travels from the ORM to the shores of Lake Ontario. Within the Site, varying water levels make determining the exact direction of groundwater flow difficult. As discussed below, groundwater migrates generally from east to west.

The geotechnical investigation conducted on the Site, recorded groundwater levels as encountered within each borehole. These observations were done concurrently with drilling and therefore do not represent the long-term static water level in the area. Water levels recorded during drilling varied considerably between 0.9 and 7.2 mbg. The average water level on the Site was 3.71 mbg. This variation of water level over such a small geographical Region suggests subsurface heterogeneity.

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The geotechnical investigation conducted at the Courtice WPCP (Geo-Canada, 2004) included the installation of 23 piezometers which were monitored twice: in March of 2000; and, February of 2004. This study showed the predominant direction of groundwater flow as from east to west (i.e., towards Tooley Creek) and to a lesser degree from northeast to southwest (i.e., towards Lake Ontario). It is presumed that, given the proposed site's proximity to the Courtice WPCP, that groundwater flow direction on the proposed Site is comparable. This implies that the groundwater gradient onsite slopes from east to west towards Tooley Creek.

As part of the Courtice WPCP hydrogeological study, water levels observed in March of 2004 at two piezometers immediately north and south of the CN Rail corridor showed that groundwater was approximately 0.3 mbg. These observations suggest that the lower (southern) portion of the Site may experience elevated springtime water levels. It is also worth noting that during the drilling of the 37 onsite boreholes, holes remained dry until the time of backfilling which suggests that interflow in the area is not a predominant process. All piezometers on the Courtice WPCP site showed water levels with a general gradient from east to west suggesting that although recharge rates may differ, there is one consistent water table across the site.

A search of the MOE Water Well Information System (WWIS) indicated 49 wells within a 1 km radius of the Site. The average groundwater depth in the area is approximately 5.01 mbg with a range from 0.61 to 12.12 mbg. See **Appendix B** for a complete list of water well records.

The average depth of water well is 15 mbg which is not sufficient to breach the lower confined aquifer layer that is presumed to be below approximately 25 to 30 m of Newmarket Till in this area. This suggests that the static water level mentioned above is not a result of a penetration of the underlying confined aquifer but of an overburden aquifer within the overlying soil unit. This suggests that the Newmarket Till may not represent exclusively aquitard characteristics.

According to Earth*fx* (2006), the average horizontal hydraulic conductivity of the Newmarket till unit is 5.0 X10 $^{-8}$ and average vertical hydraulic conductivity is 1.0 x10 $^{-8}$. No hydraulic conductivity testing was completed during the geotechnical investigation on site, nor as part of the hydrogeological investigation at the adjacent Courtice WPCP property. Further hydrogeological investigation should be conducted during the detailed design stage of the project. An additional borehole program including the installation of monitoring wells is recommended to determine dewatering requirements, inform foundation and stormwater infrastructure design and fulfill permitting requirements.

3.3.3 Groundwater Quality

Many rural hamlets in Durham Region rely upon groundwater resources as their sole source of potable water. Interference with groundwater quantity or quality from construction and operational activities can have adverse effects. In the combined Tooley, Darlington, Soper and Bowmanville Creek watersheds approximately 6100 residents rely upon groundwater for consumption (GLL, 2003). These residents extract an estimated 1,080 m^3/day which comprises approximately 8% of the estimated renewable groundwater resources.

Wellhead protection areas describe the two-dimensional spatial extent to which disturbances could influence a community's drinking water supplies. According to the *Durham Region Groundwater Study* (GLL, 2003) the Site does not fall within any wellhead protection areas.

No groundwater quality monitoring has been conducted as part of the geotechnical investigation at the Site. Similarly, the geotechnical investigation conducted at the Courtice WPCP immediately south of the Site assessed groundwater levels but did not test for groundwater quality.

3.4 Surface Water

Watercourses receiving discharge from stormwater management facilities are subject to a range of flows and water qualities. Documenting the natural variation in seasonal flows, the hydrograph response to precipitation inputs and the compositional make up of the receiving water will characterize baseline conditions to ensure future monitoring and sampling results have a representative condition for comparative analysis.

3.4.1 Surface Water Flows

The Site lies within the Tooley Creek watershed which drains approximately 1050 ha of mainly agricultural and rural land (CLOCA, 2008b). Tooley Creek watershed has an approximately 5 km, north to south length between its headwaters at Hwy No. 2 and its discharge point into Lake Ontario. The average slope along the longitudinal axis of the watershed is approximately 0.019 m/m or 1.9%.

There are 6.5 kilometers of defined channel within the Tooley Creek watershed (CLOCA, 2008c) of which most are meandering in nature. Tooley Creek is a permanently flowing, warm water stream throughout its northern reaches. However, immediately north of Highway 401, the Conservation Authority has reported cold water spring inputs which may offer refuge areas for migratory salmonids (DFO, 2005). Tooley Creek is considered a cold water stream for the purposes of this assessment due to the fish species present within the watercourse and the proposed discharge point being south of Highway 401.

The entire Tooley Creek watershed has been characterized by the Conservation Authority in the report, *Hydrologic and Hydraulic Modeling for Tooley Creek* (CLOCA, 2008c). The assessment was based on preliminary field work, aerial photographs and Regional soils data. Since no stream gauging is conducted on Tooley Creek, the modeling developed within this report is approximate in nature and can only be used as an estimative tool.

Within the CLOCA watershed assessment (CLOCA, 2008c) 12 subwatersheds were identified. Each subwatershed was characterized by area, land use, hydrologic soil group, SCS curve number and transport parameters. There were also six road crossings of Tooley Creek identified within the study.

Results from the hydrologic and hydraulic modeling are presented on a subwatershed basis for the 2, 5, 10, 25, 50, and 100-yr storms. The subwatershed that the Site is located within does not account for discharge route and therefore peak discharges would not correspond with approximate discharges expected at the CN Rail culvert discharge point (proposed discharge point for the Site).

To provide an average annual flow estimate for Tooley Creek, the existing conditions water balance for the Site was adapted to simulate the broader Tooley Creek watershed conditions. The flow within Tooley Creek should equal approximately the combination of total runoff and total groundwater recharge values from the water balance. The resultant average annual flow in Tooley Creek was estimated to be 0.12 m^3/s. Since the Site is in close proximity to Lake Ontario, some of the groundwater discharge would be directly to Lake Ontario and would therefore reduce this flow estimate.

The Facility's runoff would be conveyed westward via a swale within the CN Rail corridor before discharging into a small tributary of Tooley Creek approximately 1000 m west of the Site. Adjacent to the Site, the swale is relatively small and lined with bulrushes and other marshy vegetation (See Figure 3-7). The CN Rail swale increases in size westward away from the Site. Flows in this watercourse are intermittent and likely seasonally based.

Figure 3-7 Swale within CN Rail Corridor Adjacent to the Site

The tributary of Tooley Creek, to which water would be discharged, is currently the conveyance channel for additional stormwater management facilities located to the north. The channel runs southwest until joining with the CN Rail swale and continuing on towards Tooley Creek running parallel to the CN Rail tracks (See Figure 3-8).

Figure 3-8 Tributary of Tooley Creek Before Joining Swale Alongside CN Rail Corridor

Stormwater from the Site would then enter the main channel of Tooley Creek and travel due south for approximately 850 m before discharging to Lake Ontario. No direct stream flow gauging was conducted within Tooley Creek, and therefore an empirically derived rating curve cannot be provided.

The Water Survey of Canada (WSC) conducts stream gauging on several of the watercourses surrounding Tooley Creek. This data is included in the WSC's HYDAT database. Since all of the gauged watercourses are considerably larger in drainage area and flow than Tooley Creek, values were not used as surrogate data.

3.4.2 Surface Water Quality

There has not been extensive surface water quality monitoring conducted on Tooley Creek or any of its tributaries. Since Tooley Creek drains mainly agricultural and rural lands, it is assumed that surface waters are of moderate to good quality. To provide the most representative surface water quality information possible, spot sampling was conducted on Tooley Creek, and a review of all applicable local and Regional water quality reports was completed and summarized here.

3.4.2.1 Literature Review

The major watersheds in proximity to the Site have their headwaters approximately 15 to 18 km north, on the southern slope of the ORM. The surficial sedimentary units of the ORM are a mixture of sands and gravels which encourage considerable amounts of infiltration. Groundwater flow in the Region is generally from north to south similar to surface drainage (DFO et al., 2000).

Within the north-south trajectory of these larger watersheds are several areas which demonstrate considerable groundwater discharge as a result of substantial quantities of baseflow derived from the ORM (DFO et al., 2000). Relatively large quantities of groundwater added to surface water features can have an impact on both surface water quality and temperature. Down gradient of the ORM, the generally cold water inputs from groundwater sources mixing with the warmer agricultural field-derived waters mix to create coolwater streams (DFO *et al*, 2000)

The Soper and Bowmanville Creek watersheds, which are located northeast of the Site, were the subject of an Aquatic Resource Management Plan study in 2000 (DFO et al., 2000). These creeks, although significantly larger in drainage area (245.2 km^2 compared to 10.5 km^2 for Tooley Creek), represent similar land uses, land forms and lithological makeups. An assessment of surface water quality conducted for this study reported that Soper and Bowmanville Creeks had excellent health, unimpaired water quality and displayed healthy functioning aquatic communities throughout. As a result of the considerable baseflow contributions to this and other large watercourses in the area, all major channels are considered coolwater streams (DFO et al., 2000; CLOCA, 2006; CLOCA 2008a).

3.4.2.2 Field Sampling

During the Site visit conducted by the Jacques Whitford Water Resources project team on December 17, 2008, water quality spot measurements were taken from Tooley Creek using a YSI™ multi-parameter sonde. Dissolved oxygen (DO), temperature, conductivity and pH values were acquired from immediately downstream of the Hwy 401 culvert, approximately 200 m north of the proposed stormwater discharge location. Considering the proximity to Hwy 401 during the winter de-icing season, it was presumed that samples would have high conductivities.

Two water samples were taken and the following averages calculated. The average pH was 8.15, average DO of 21 mg/L and average conductivity of1613.5 µs/cm. A DO value of 21 mg/L at an average water temperature of 0.54^{o}C is equivalent to 125.9% saturation. According to the Canadian Water Quality Guidelines (CWQG)(CCREM, 1987) and PWQO for the Protection of Aquatic Life in Freshwater, the lowest acceptable DO value for early life stages in a cold water ecosystem is 9.5 mg/L and 7 mg/L, respectively.

The PWQO states that the pH of surface water should be maintained within the range of 6.5 to 8.5 to protect aquatic life. Similarly, the CWQG stipulate a pH range of 6.5 to 9. Neither of these water quality guidelines specify a threshold value for conductivity.

3.4.3 Existing Condition Runoff Modeling

Existing condition runoff is currently controlled by the existing topography and vegetation present on the Site. Precipitation that does not evapotranspire or infiltrate into the upper soil horizons generally runs towards the southwest corner of the property. Modeling of stormwater runoff was used to characterize the peak discharge and total runoff volume arising from the Site during precipitation events. These results will be used as objectives for stormwater management planning.

Stormwater modeling was developed using HEC-HMS 3.2 software to best represent actual conditions. HEC-HMS 3.2 models use precipitation inputs and user defined watershed characteristics to simulate runoff. The model includes four main components: basin models, meteorological models, control specifications and time series data. This model is particularly suitable to watershed modelling and incorporates sufficient flexibility to subsequently alter hydrological parameters for use in determining post-development runoff.

Drainage areas in HEC-HMS are characterized by their catchment area as well as by selecting one of various methods to describe the loss, transformation, and baseflow of the system. For this model, the Site comprises one drainage watershed which was characterized through a review of soil records, aerial photography, topography, land uses, vegetative cover and geometrical shape. Table 3.4 shows the methods selected to model loss, transformation, and baseflow, all of which are explained in further detail below. Due to the preliminary nature of the model, baseflow was not simulated.

Table 3-4 - Methods Used for Site Characterization

Characteristic	Method
Loss	SCS Curve Number
Transform	Clark Unit Hydrograph
Baseflow	- None -

The Site has a total area of 12.4 ha of which currently approximately 50% is plowed agricultural land with the remaining 50% comprising fallow fields.

The Natural Resource Conservation Service (NRCS formerly and more commonly referred to as the Soil Conservation Service (SCS)) curve number was determined for the Site by analyzing aerial photography of the Region to determine land usage. Land type and surface vegetation by percentage was used to describe the Site as required by the SCS methodology. Soil drainage type was classified as ‘B’, for a sandy-silt to silty-sand (Jacques Whitford, 2008). Together, the soil classification and surface composition by percentage were used to calculate an aggregate SCS curve number for the site. Percent imperviousness for the site was based on land use and existing groundcover. An SCS curve number of 64 was chosen to represent the sandy-silt and silty-sand soil currently used as agricultural and rural land.

Initial abstraction amounts were set to three millimetres for all subbasins. Initial abstraction values for the subwatersheds were calculated based on 20% of the potential total maximum subwatershed retention as calculated by the respective curve number. This is in accordance with assumptions made in the SCS's TR-55 method (USSCS 1986).

The antecedent moisture condition (AMC) was considered in the development of the curve number and curve numbers were selected based on an AMC II condition (average soil moisture). Note, however, that AMC condition is not an input variable in the SCS Curve Number Loss Method used.

Time of Concentration (TOC) values used in the Clark Unit Hydrograph transform method represent the longest time it would take for water to runoff from the subbasin. The flow path representing the TOC for each subbasin was assumed to be comprised of two portions:

- a distance where water would travel via overland flow to the main subbasin channel; and
- a distance where water would travel in a grassed waterway or swale to the outlet point or the extent of the Site.

These distances and slopes were determined via Site reconnaissance and analysis of topographic maps. Aerial photography was then analyzed to determine the surface vegetation of the overland flow portion, and TOC for the entire flow path calculated using the NRCS lag method (Folmar *et al*., 2006).

Values for the storage coefficient were set at approximately two-thirds of the time of concentration for all subwatersheds. This value was chosen as an average representative estimate based on the calibrated storage coefficients determined for several other models. Lacking any defined discharge channels and stream gauging data for the site, calibration of TOC and storage coefficients was not possible. Values used in this model were deemed appropriate given site characteristics and past experience in similar environments.

Desired model outputs included storm runoff hydrographs, peak discharges and total storm runoff volumes. A summary of hydrologic parameters for the pre-development site is included in Table 3-5.

Table 3-5 - Summary of Existing Conditions Hydrologic Parameters

Parameter	Existing Site
Area (ha)	12.4
Hydraulic Length (m)	390
Average Slope %	1.9
Hydrologic Soil Group	B
Time of Concentration (hr)	0.73
Storage Coefficient (hr)	0.49
% Imperviousness	2
SCS Curve #	64
Initial Abstraction (mm)	3

A total of nine design storms were used in model simulation: 10 mm, 25 mm, 2-yr, 5-yr, 10-r, 25-yr, 50-yr, 100-yr and the Regional storm event derived using the Gumbel method of moments for 31 years of precipitation data from the Oshawa weather station (ID# 6155878) provided in Meteorological Service of Canada's (MSC) Intensity-Duration-Frequency (IDF) curves. The 10 mm, 25 mm, 2-yr and 5-yr rainfall events were designed according to a Chicago type distribution and the 10-yr through 100-yr events were based on a SCS type II distribution (See Table 3-6 for storm durations). The method for modeling of the area's Regional Storm (Hurricane Hazel) was based on the final 12 hours of the storm record as per the method described in MNR (2002).

Summary data for the nine modeled design storms is found below in Table 3-6. See Figure 3-9 for hyetographs of Chicago Distribution storms and Figure 3-10 for hyetographs of SCS Type II Distribution and Hurricane Hazel storms.

Table 3-6 - Summary Data for Modeled Design Storms

Design Storm	Distribution	Total Rainfall (mm)	Storm Duration (hrs)
10 mm Storm	Chicago Distribution	10	4
25 mm Storm	Chicago Distribution	25	4
2-Year Storm	Chicago Distribution	30	4
5-Year Storm	Chicago Distribution	40	4
10-Year Storm	SCS Type II	47	4
25-Year Storm	SCS Type II	63.7	12
50-Year Storm	SCS Type II	77.5	24
100-Year Storm	SCS Type II	84.7	24
Hurricane Hazel	Prescribed	210.8	12

Figure 3-9 Hyetographs for Chicago Distribution Design Storms

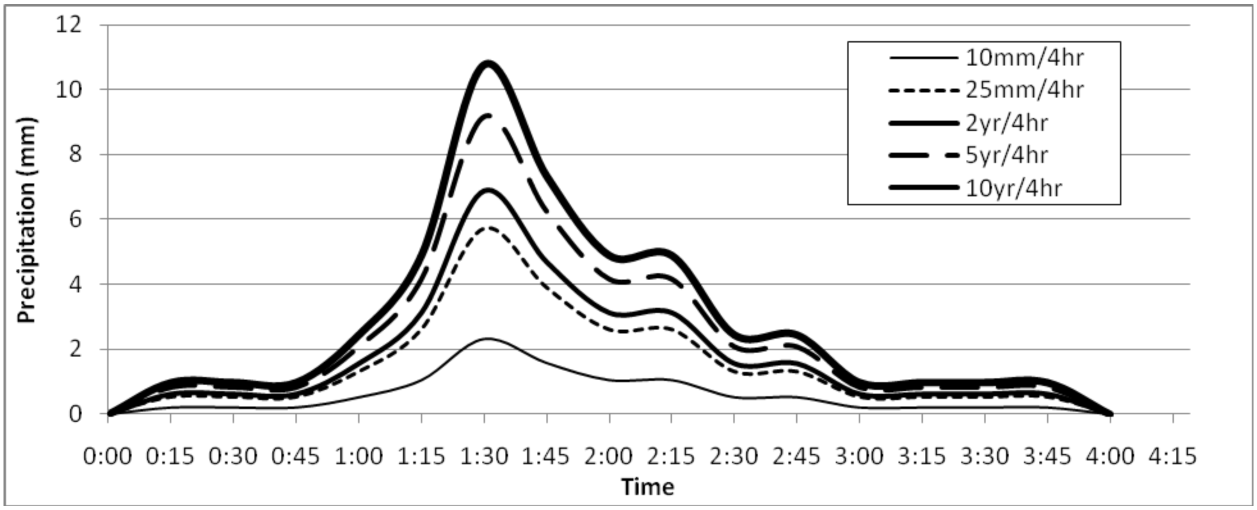
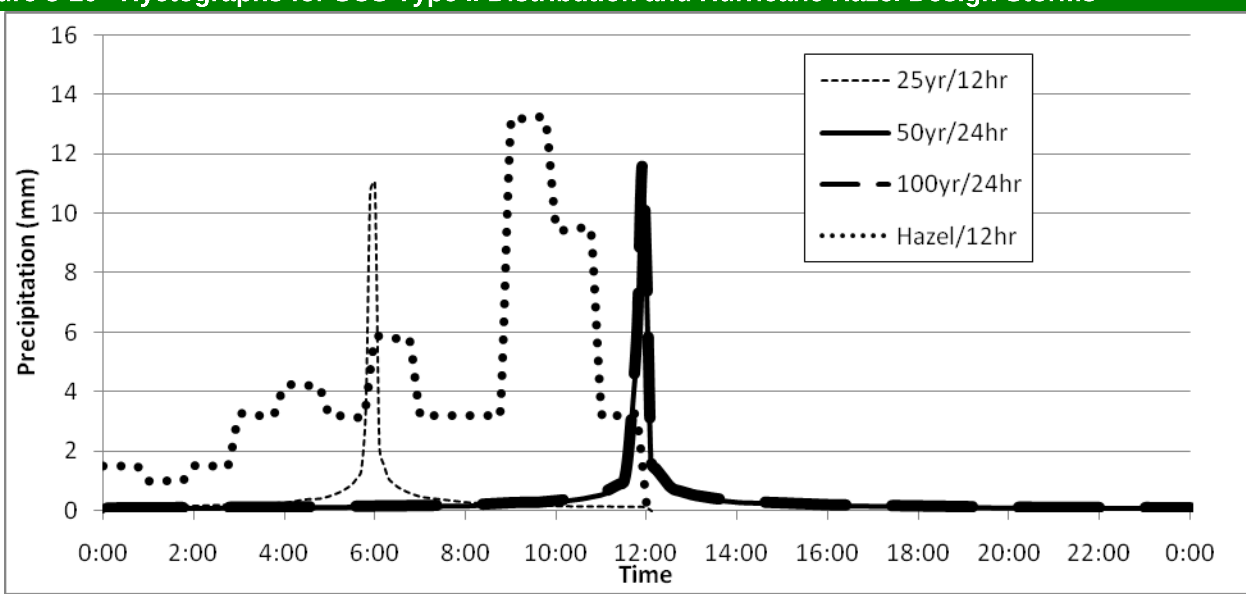


Figure 3-10 Hyetographs for SCS Type II Distribution and Hurricane Hazel Design Storms



As part of the requirements of CLOCA (2007), the 1-hr American Environmental Service (AES) storm distribution and 24-hr SCS Type II storm distribution events have been modelled for the existing conditions, post-development 140,000 tpy and post-development 400,000 tpy scenarios. Results from this hydrological modeling are included in **Appendix C**.

The site was subjected to all nine design storms to model the range of possible conditions present at the site. The peak discharge and total runoff volume for each design storm is included in Table 3-7.

Table 3-7 Existing Conditions Runoff Model Results

Watershed	Parameter	10mm/ 4hr	25mm/ 4hr	2yr/ 4hr	5yr/ 4hr	10yr/ 4hr	25yr/ 12hr	50yr/ 24hr	100yr/ 24hr	Hazel
Existing Conditions	Peak Discharge (m^3/s)	<0.01	0.06	0.08	0.15	0.2	0.36	0.43	0.50	1.17
	Runoff Volume (m^3)	65	419	596	1024	1375	2355	3295	3822	15486

Runoff hydrographs for all design storms are provided in **Appendix C**. An objective of mitigation measures suggested in the SWM design (Section 5.2) is to shave post-development peak discharges and attenuate flow to below pre-development conditions listed above.

The site currently slopes from northeast to southwest where the lowest point of the property is immediately adjacent to the CN Rail corridor. According to the existing conditions water balance infiltration on the Site is minimal and runoff can account for up to 37.7% of incident precipitation per year. In the event of a large precipitation event, stormwater would likely runoff to the lower southwest corner and pond once soil holding capacity has been reached. It is unlikely that any other locations on the Site would be subject to flooding given the continual slope to the southwest.

3.4.3.1 CN Rail Swale Capacity

The stormwater pond planned to occupy the southwest corner of the Site would discharge into an existing swale running parallel to the CN Rail corridor along the southern boundary of the Site. Adjacent to the Site, the swale is relatively small and densely vegetated (See Section 3.4.1. for full description of conveyance route) suggesting that a large flood event may overflow its banks. Increased flows from the site to the CN Rail swale would increase flooding and erosion potential.

During a Site visit on December 17, 2008, three swale cross-sections were obtained in order to conduct a conveyance capacity assessment. The first cross-section acquired (approximately 100 m west or downstream of the Site) was approximately 1 m wide and 0.5 m deep (See Figure 3-11). Using WinXSPro the cross section conveyance capacity was tested.

To simulate the densely vegetated channel a Manning's n value of 0.065 and an average slope of 0.015 was used. This assessment suggested that the conveyance capacity of transect #1 (Figure 3-11) of the existing channel at bankfull is 0.14 m^3/s which is consistent with the capacity to handle the peak discharge from the 5-year storm event. To prevent flooding in the CN Rail easement, peak discharges from the Site should either be managed to below the 0.14 m^3/s threshold or channel widening/upgrading to increase hydraulic capacity would be needed within the proximal channel reach.

Within 300 m of the Site the swale's conveyance capacity increases significantly to over 1 m in depth at bankfull. By this point the conveyance capacity is over 2.3 m^3/s which is more than adequate to contain the pre-development 100-yr peak discharge (See swale transect #2, Figure 3-12).

Figure 3-11 Swale Cross-Section #1 – 100 m West of Site

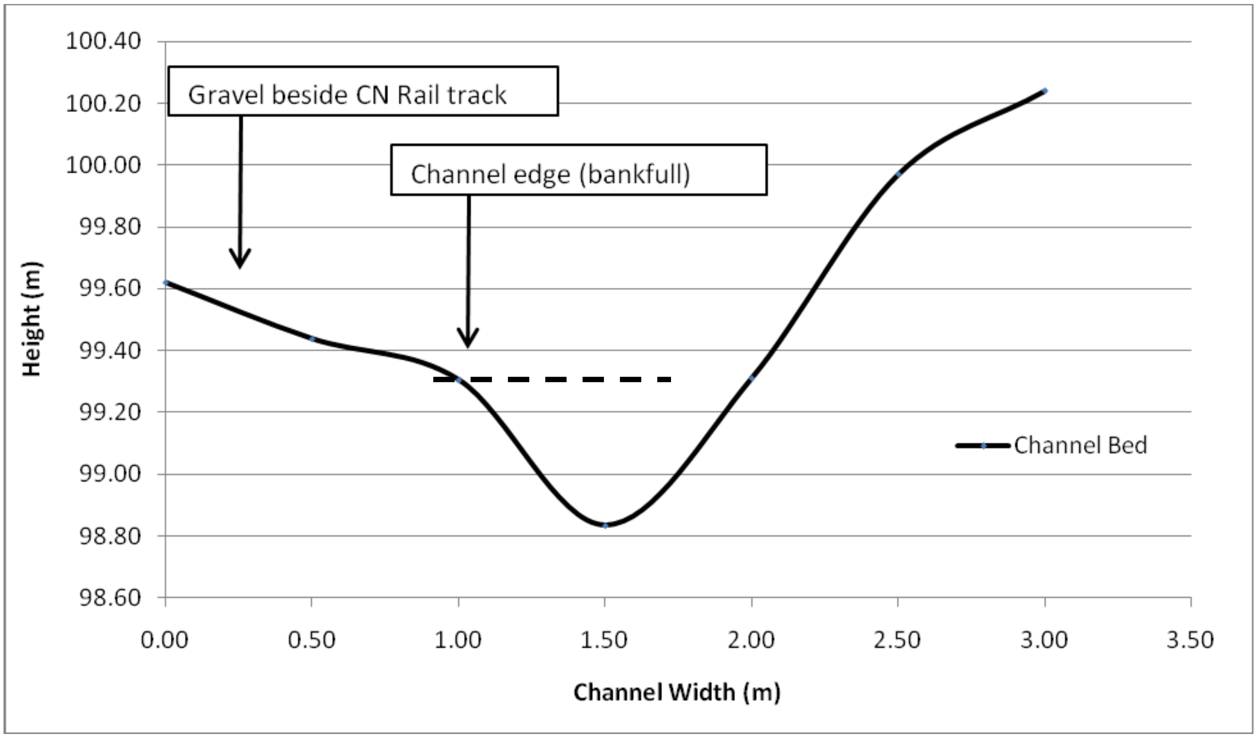
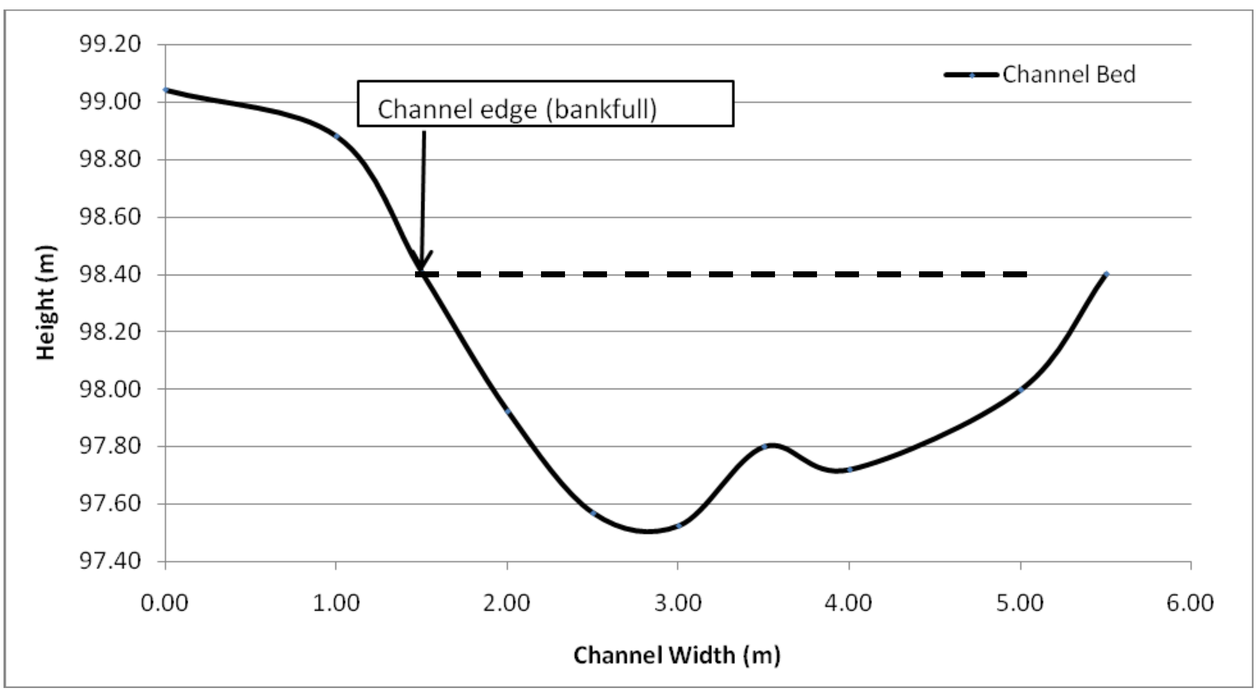


Figure 3-12 Swale Cross-Section #2 – 300 m West of Site



3.4.4 Geomorphological Conditions

During the Site visit conducted by the Jacques Whitford Water Resources project team, the CN Rail swale and main channel of Tooley Creek were investigated to characterize their current geomorphological conditions and identify potential areas or sources of concern if the watercourses were to act as the receiving water for the Facility.

Adjacent to the development Site, the CN Rail swale is small (approximately 0.5 m deep and 1.0 m wide at bank full) and densely vegetated. There were no signs of active erosion or slope instability. Further downstream the channel increases in size considerably to approximately 2 m in depth and was filled with large woody debris and intermittent marshy vegetation. There were no signs of erosion or instability throughout the length of the CN Rail swale. Since this swale runs alongside the CN Rail corridor, it maintains a straight path or a sinuosity of approximately 1.

The CN Rail swale conveys intermittent flows at low velocities due to the dense vegetation present in most parts. In addition, the swale is bordered by a compact, gravel covered embankment (CN Rail track feature) on the south side and a steep, well-vegetated bank on the north side. Given slow intermittent flows bordered by relatively stable embankments, the CN Rail swale is not likely to experience any planimetric evolution given typical pre-development flows.

The lower sections of most watersheds display relatively less in topographic relief and as a result tend to contain relatively wide, deep channel geometries with slower moving water when compared to the headwaters of the watershed. Receiving discharge from the entire watershed, these lower sections can experience flows with high erosion and sediment transport capacities. In contrast, sediment laden flows from upstream reaches can deposit suspended sediment in the slower moving waters of the downstream reaches.

The main Tooley Creek channel, at the confluence point with the CN Rail swale, is approximately 3 m wide with a gravel/cobble substrate. Before converging with the CN Rail swale, the channel traverses an open floodplain where a number of oxbow features are visible (See Figure 3-13). This planimetric evolution is typical of meandering streams in their lower reaches.

Directly downstream of the confluence of the CN Rail swale with Tooley Creek is a 12 m wide plunge pool directly south of the CN Rail culvert. The pool is approximately 0.3 m deep and has a sandy/silt bottom except for the narrow pool outlet, where a small area of gravel is exposed. Both upstream and downstream of the pool outlet large areas of exposed bank indicate active erosion and suggest the flashiness of the watercourse (DFO, 2005). Vegetation in the area consists of old field species including goldenrod, milkweed and long grasses.

Figure 3-13 Tooley Creek Immediately Upstream of CN Rail Culvert

3.5 Soil Loss

Various environmental phenomena can act to erode topsoil from one location and deposit it elsewhere, typically downgradient. Determination of the existing condition soil loss for the Site would create a baseline to which further mitigation measures should be referenced, if necessary.

A geotechnical investigation of the Site (Jacques Whitford, 2008) has suggested that a majority of the Site is underlain by a thick layer of sandy silt and silty sand with traces of clay and gravel. According to MNR et al., (1987) this soil type corresponds with a soil erodibility rating of medium (See Table 3-8) and a soil erosion potential of moderate (See Table 3-9).

Table 3-8 - Soil Erodibility Classification Chart

Soil Type	Erodibility Classification	Soil Erodibility Rating
Silt	Most	High
Silt Loam		High
Loam		High
Silty Loam		High
Sandy Loam		Medium
Silty Clay Loam		Medium
Sandy Clay Loam		Medium
Silty Clay		Medium
Sandy Clay		Low
Clay		Low
Heavy Clay		Low
Loamy Sand		Low
Sand		Low
Poorly Graded Gravel		Low
Well Graded Gravel	Least	Low

Source: Adapted from Guideline on Erosion and Sediment Control for Urban Construction Sites (MNR et al. 1987)

Table 3-9 - Soil Erosion Potential Classification

Slope Gradient	Soil Erodibility	Slope Length	
		<30 m	>30 m
<2% Gentle Slope	Low	Low	Moderate
	Medium	Moderate	Moderate
	High	Moderate	High
2-10% Moderate Slope	Low	Low	Moderate
	Medium	Moderate	High
	High	High	High
>10% Steep Slope	Low	Low	Moderate
	Medium	High	High
	High	High	High

Source Adapted from Guideline on Erosion and Sediment Control for Urban Construction Sites (MNR et al. 1987)

Preliminary assessment of soil loss from the Facility lands currently in rural land use indicates that there is potential for erosion warranting further analysis regarding soil losses with current and future land uses.

The Universal Soil Loss Equation (USLE) was used to estimate existing soil erosion from the Site:

$$A = R \times K \times LS \times C \times P \qquad \text{(equation 3.2)}$$

Where R = Rainfall and Runoff Factor (90)

K = Soil Erodibility Factor (0.16)

LS = Slope Length-Gradient Factor (0.37)

C = Crop/Vegetation and Management Factor (1)

P = Support Practice factor (0.005)

Based on low topographic relief, sandy-silt to silty-sand soils, a 240 m average slope length, pasture land use, and no-till practice, the estimated soil loss is 0.03 tonnes/ha/year. Soil loss rates of less than 3.05 tonnes/ha/yr are considered very low and tolerable. All USLE values were obtained from OMAFRA (2000).

3.6 Water Resource Goals

The above review of existing conditions has identified the key hydrological, hydrogeological and geomorphological characteristics of the Site and would form the basis for the water resource-related goals to be carried forward to the development and mitigation sections. The pre-development water balance has suggested that maintaining the current levels of ET and runoff would be important strategies in post-development scenarios. Since infiltration comprises only a small proportion (approximately 3.7%) of the yearly water balance, infiltration measures would be recommended but would be of lower importance than those components mentioned above.

Currently, the CN Rail swale proposed for downstream conveyance of the Site's stormwater discharge, is capable of conveying the peak discharge from the 5-year precipitation event at the receiving point. To maintain the geomorphological stability and the safe passage of CN Rail vehicles, flow in this swale must not exceed this threshold during post-development conditions unless conveyance swale improvements are undertaken. In addition, the water quality of the receiving waters is described as moderate to good and must be maintained at this level to ensure the continued health of aquatic ecosystems within the waterbody.

Although groundwater transmission rates are low in this area, water levels have been shown to be within 0.3 m of the surface at the southern end of the Site. Appropriate spill prevention, stormwater management and Facility waste management measures would be necessary to prevent the contamination of groundwater resources.

4.0 POTENTIAL EFFECTS OF FACILITY

This section provides the results from the assessment of anticipated post-development impacts and forms the basis for the design and selection of mitigation measures. A general description of the Facility is provided and the potential impacts suggested. Post-development water balance impacts are then discussed with an emphasis on important parameters requiring mitigation. Subsequently, post-development stormwater runoff, and soil loss modeling is conducted and discussed. This section concludes with preliminary assessments of Facility water demand and wastewater discharges and the ability to provide such services given existing municipal infrastructure.

4.1 Proposed Facility and Potential Impacts

The Facility would occupy 12.4 ha of land of which it is estimated that 45% would be covered with an impervious layer (See Figure 4.1 – submitted as part of the preferred vendor's proposal package). All vegetation would be removed and replaced with appropriate surfaces. The Site would include paved vehicle access routes, parking areas and building infrastructure to house the waste processing components. The infrastructure foundation is planned to reach approximately 7.6 mbg, which would likely intercept the local groundwater resources.

It is anticipated that the entire Site would be grubbed in order to facilitate the construction of the requisite buildings, storage areas and impervious surface coverings. It is expected that stormwater runoff from impervious areas would be routed to management features via underground stormsewers. These and other subsurface components (i.e., waste storage and incineration chambers) would require excavations which may penetrate the surficial overburden aquifer present in the area. It is anticipated that stormwater conveyance and storage features would be installed during the initial stages of construction.

The main facility buildings would be located centrally between the east and west boundaries of the Site in the northern half of the property. According to the preliminary Site plan (See Figure 4-1 – submitted as part of the preferred vendors proposal package) all building infrastructure would be clustered around this point. Access to the Site would be provided via one main plant access point in the southeast corner of the property (adjacent to the CN Rail corridor). Paved roads would loop around the main Facility to provide access and parking for all components of the development.

The Site can be serviced with municipal water supply infrastructure and therefore no extraction of groundwater resources would be necessary for the operation of this Facility. Sanitary sewage from the Facility would be conveyed to the WCPC located approximately 200 m south of the Site. No onsite septic system would be required.

All precipitation incident upon the Site would be captured by lot level stormwater management features and directed toward an end-of-pipe stormwater management facility. Stormwater would be retained and detained according to requisite water quality improvement criteria. The facility is designed to ensure that no solid waste would be released to the surrounding environment and would therefore not contaminate stormwater or subsequently the receiving waters.

Total infiltration on the Site may be reduced as a result of the introduction of imperviousness. In addition, the decrease in vegetation coverage and soil moisture may decrease the Site's ET and cause an increase in runoff volumes. Mitigation measures would be necessary to offset any potential impacts caused by the Facility.

4.2 Post-Development Water Balance

Based on climate normals, the existing condition water balance model estimated that ET, runoff and infiltration represented 58.5%, 37.7%, and 3.7% of water balance outputs, respectively. In many cases, the existing condition water balance is used specifically to set targets for stormwater management of infiltration capacity as well as maintaining ET opportunities.

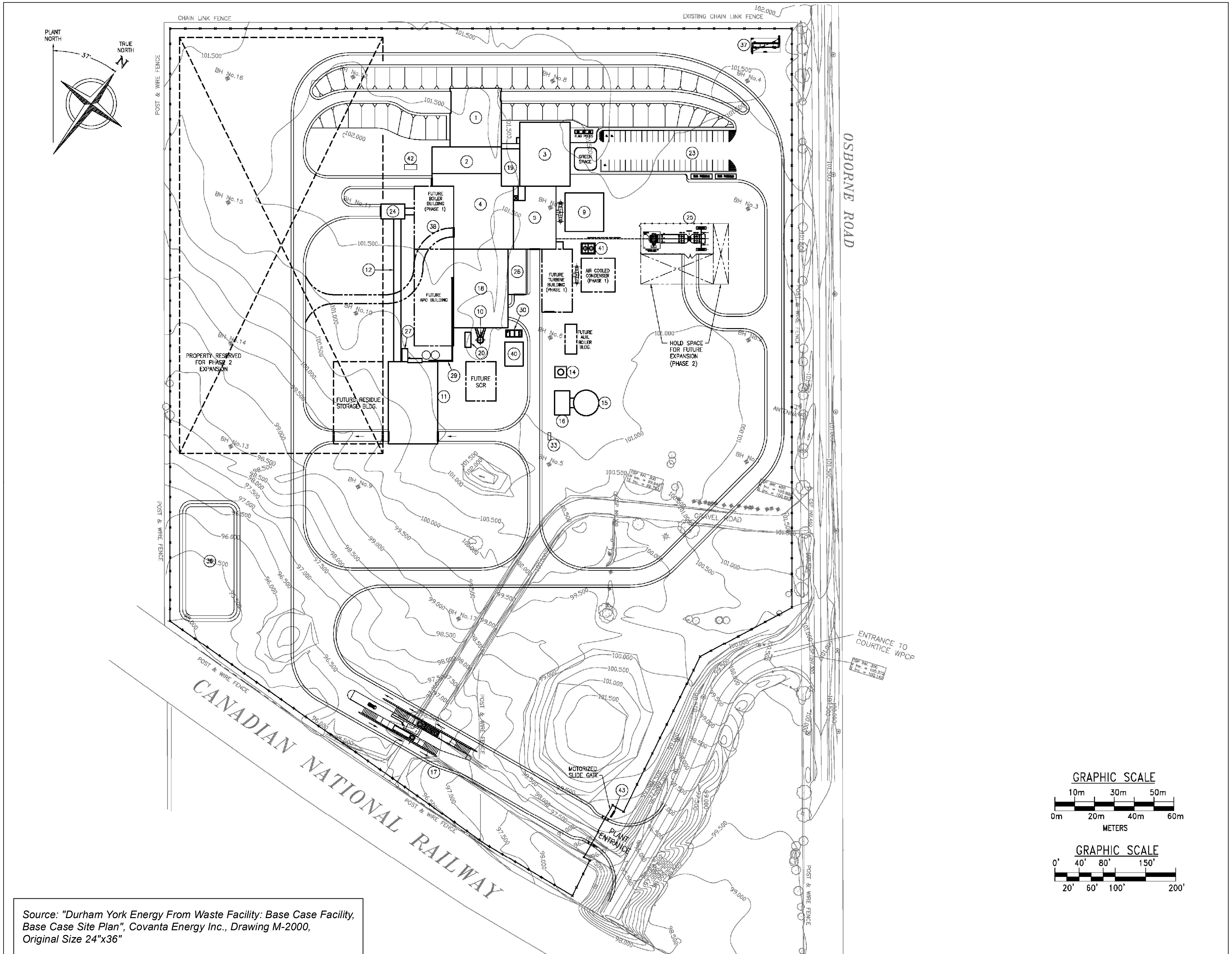
Infiltration can be encouraged through lot level controls such as rooftop storage, disconnection of roof leaders from the stormsewer system as well as appropriate site grading, and the use of infiltration galleries, swales and trenches. Infiltration would be promoted in open, landscaped areas of the Site. However, due to low existing infiltration conditions and the industrial nature of the Facility, infiltration from active industrial areas of the Site is not encouraged.

ET would be encouraged through the preservation of some open, landscaped areas of the Site, as well as through Site grading at the lot level to slow runoff rates. Finally, the proposed stormwater pond would introduce a permanent open water feature to the Site with expected pond fringe vegetation, which would facilitate ET.

Total runoff volumes are expected to increase in the post-development case, however the discharge rates from the Site would be managed to reduce erosion and flooding potential as well as extend baseflow contributions to the receiving CN Rail swale.

YORK REGION AND DURHAM REGION
RESIDUAL WASTE STUDY

Durham/York Thermal Treatment Facility Site Layout (140,000 tpy Scenario)



Source: "Durham York Energy From Waste Facility: Base Case Facility, Base Case Site Plan", Covanta Energy Inc., Drawing M-2000, Original Size 24"x36"

LEGEND

1. TIPPING FLOOR
2. REFUSE BUILDING
3. ADMINISTRATION BUILDING
4. BOILER BUILDING
5. TURBINE BLDG.
6. NOT USED
7. NOT USED
8. NOT USED
9. AIR COOLED CONDENSER
10. STACK
11. RESIDUE STORAGE BUILDING
12. INCLINED BELT CONVEYOR GALLERY ENCLOSURE
13. NOT USED
14. AMMONIA STORAGE TANK & CONTAINMENT
15. FIRE WATER STORAGE TANK
16. FIRE WATER PUMP HOUSE
17. TRUCK SCALE AREA
18. FDG/APC BUILDING/BAGHOUSE BLDG.
19. CONTROL/ELECTRICAL ROOMS
20. CEMS BUILDING
21. NOT USED
22. NOT USED
23. PARKING LOT
24. GRIZZLY BUILDING
25. SWITCHYARD
26. MAINTENANCE AND STORAGE BUILDING
27. RESIDUE PROCESSING ELECTRICAL BUILDING
28. NOT USED
29. FLY ASH TRANSPORT CONVEYORS
30. SETTLING BASIN
31. NOT USED
32. NOT USED
33. MAINT. TRUCK DIESEL OIL STORAGE TANK
34. NOT USED
35. NOT USED
36. RETENTION POND
37. GAS METERING STATION
38. GRAVEL ACCESS ROAD
39. NOT USED
40. ID FAN VFD BUILDING
41. CLOSED COOLING WATER HEAT EXCHANGER
42. EMERGENCY DIESEL GENERATOR ENCLOSURE
43. PLANT ENTRANCE SIGN

JW-1009497-NS2-4

Jacques Whitford

FIGURE NO.
4-1

Last Modified: June 3, 2009 By: L. Kendell

4.3 Construction Dewatering

Due to the maximum proposed depth of the Thermal Treatment Facility infrastructure (7.6 mbg) it is likely that groundwater would be encountered during excavation. In order to construct the required infrastructure foundation below grade in the requisite dry conditions, dewatering would be necessary.

In general, the east-west Iroquois Plain on which the Site is located is thought to be underlain by a clayey-silt layer with low hydraulic conductivity. This soil unit is not known to produce significant quantities of groundwater and would therefore represent an aquitard layer confining the underlying sand layer aquifer (DFO *et al*. 2000). However, small areas of exposed drumlin ridges have been identified in the area and geotechnical investigations conducted in the direct vicinity of the Thermal Treatment Facility suggest the Site may be underlain by the above mentioned glacial till.

The geotechnical investigation conducted by Jacques Whitford (Jacques Whitford, 2008) suggested that the Site was underlain by a thick layer of sandy silt and silty sand and that groundwater could be found at varying depths (0.9 to 7.2 mbg) throughout the boreholes. The geotechnical investigation conducted at the adjacent Courtice WPCP (Geo-Canada, 2004) reported similar subsurface conditions with static water levels of up to 0.3 mbg. Groundwater did not demonstrate artesian conditions during either geotechnical assessment and therefore aquifer depressurization would not be necessary within the top 7.6 m of excavation.

Construction and operational phase dewatering and permitting requirements would be determined during the detailed design phase.

4.4 Stormwater Management

The introduction of imperviousness, Site grubbing and Facility infrastructure could have adverse effects on the quantity and quality of stormwater leaving the Site if not properly mitigated. Hydrological models were used to characterize the existing and anticipated post-development conditions, quantify the magnitude of post-development influence and suggest mitigation measures capable of offsetting such effects.

4.4.1 Post-Development Stormwater Runoff Modelling

The construction of the proposed Facility along with the associated surface hardening, Site grading and removal of native vegetation would decrease the overall Site ET and infiltration potential which in turn is expected to result in an increase in runoff rates and volumes. In order to design appropriate mitigation features to offset these effects, the post-development runoff events must be estimated.

A post-development hydrologic model was developed using similar methodologies as outlined in Section 3.4.3 for the existing conditions model. Post-development conditions were simulated by increasing the SCS curve number and percent imperviousness for the Site. These changes subsequently influence the average Time of Concentration, storage coefficient and ultimately runoff coefficients.

The same suite of storm events was used to simulate the range of conditions possible on the Site. The 25-mm storm is important for SWM planning as it forms the reference basis of water quality design. Similarly, the 100-year event is used to define the water quantity design objectives. Modeling the Regional Storm event, Hurricane Hazel, is only necessary when potential floodplain encroachment issues affect the Facility. The Regional storm event was still modeled here to quantify the maximum size event possible in the area. All other storm events are used for comparison with existing condition results to form mitigation targets for SWM.

The Facility would include the paving of access roads, an aggregate top dressing over storage areas and the introduction of permanent building structures. It is assumed that the development would best be represented by a SCS curve number of 82 for a hydrological soil group B with an industrial setting introducing impervious conditions over 45% of a 12.4 ha developed area. Although modeling this unmitigated post-development scenario may lead to an over-estimation, considering no proposed mitigation measures have been taken into account, this is seen as a worst-case scenario.

According to the preferred vendor's preliminary Site drawing, the entire 12.4 ha would contain some type of development and therefore would require stormwater management. It is assumed that the slope would remain similar to that of existing conditions. A similar slope incorporating increased imperviousness would cause a decrease in Time of Concentration values used in the hydrological model. However, the storage coefficient would not be affected by above grade activities.

A summary of post-development hydrologic input parameters are included in Table 4-1.

Table 4-1 - Post-Development Model Input Parameters

Parameter	Post-Development
Area (ha)	12.4
Hydraulic Length (m)	390
Average Slope %	1.9
Hydrologic Soil Group	B
Time of Concentration (hr)	0.49
Storage Coefficient (hr)	0.49
% Imperviousness	45
SCS Curve #	82
Initial Abstraction (mm)	3

As a result, of the Facility, runoff volumes onsite would increase, which would also cause an increase in peak discharges when compared to pre-development conditions. Table 4-2 provides a summary of post-development runoff volumes and peak discharges for all of the design events. The values in Table 4-2 represent unmitigated post-development conditions in order to provide a worst case modeling scenario. Subsequent stormwater mitigation options would be designed to reduce these peak discharges to pre-development levels and attenuate flows.

Table 4-2 - Post-Development Runoff Model Results

Watershed	Parameter	10mm/ 4hr	25mm/ 4hr	2yr/ 4hr	5yr/ 4hr	10yr/ 4hr	25yr/1 2hr	50yr/2 4hr	100yr/2 4hr	Hazel
Post-Development	Peak Discharge (m^3/s)	0.09	0.26	0.33	0.47	0.57	0.96	1.02	1.14	1.52
	Runoff Volume (m^3)	611	1820	2275	3239	3946	5709	7231	8038	22936

The results presented in Table 4-2 show an increase in runoff volume which infers a decrease in ET and infiltration volumes. When compared to the existing condition runoff modeling, these results suggest an increase of 1401 m^3 for the 25 mm event and a 546 m^3 increase for the 10 mm event.

The objective of SWM design will be to offset to the extent feasible the effects this increase in impervious area is expected to have on ET and runoff. Increases in runoff volume and flow rate would require appropriate SWM at the lot, conveyance and end-of-pipe levels.

4.4.2 Offsite Stormwater Conveyance

According to Section 3.4.3.1, the CN Rail swale adjacent to the Site is capable of conveying up to $0.14 m^3/s$ within its banks. Post-development runoff modeling suggests that this bank full discharge represents the flow from between the 10 mm and 25 mm storm event. Stormwater management facilities would have to be designed to maintain discharge flows from all precipitation events below the 0.14 m^3/s threshold or channel widening would be necessary.

4.5 Soil Loss

The grubbing and excavation of previously vegetated lands can result in increased soil loss through erosion and bank instability. An assessment of the existing potential for soil loss was conducted in Section 3.5 and indicated an annual pre-development soil loss of 0.03 tonne/yr. A secondary assessment was performed to determine the effect the Facility would have on soil loss. The predicted construction phase soil loss rates can be used to estimate the degree of influence the Facility may have on the local soil stability and the type and magnitude of mitigation measures necessary to offset effects.

Using equation 3.2 from Section 3.5, soil loss during the construction phase was estimated at 0.1 tonnes/ha/yr. The estimated soil loss amount was based on the following values for equation 3.2 parameters R, K, LS, C, and P being 90, 0.16, 0.37, 0.02, and 0, respectively and were derived from OMAFRA (2000). Soil loss rates of less than 3.05 tonnes/ha/year are considered very low and tolerable.

Although construction phase soil loss rates are well within the tolerable range, the Facility would result in an increase to annual soil loss. If unmitigated, this increase in suspended solids discharged into local watercourses could have negative effects. Onsite construction phase erosion and sediment control features would be required to ensure the receiving waters are not negatively affected by development.

4.6 Facility Water Demand and Wastewater Discharge Requirements

The maximum annual water demand is estimated to be 42,000 m^3/yr or, assuming a continuous 365 day operation, 115,068 L/day or 1.33 L/s. This water demand threshold is based on the Facility receiving 140,000 tpy of waste material. Currently a 300 mm municipal water supply main runs along the western side of Osbourne Road adjacent to the Site. Preliminary assessments (Jacques Whitford and Genivar, 2007a) assumed that the exclusive use of a 300 mm main could provide approximately 100L/s which would be adequate to supply a 250,000 tpy Facility's needs. A full hydraulic assessment should be carried out during detailed design to ensure the firewater and Facility demands can be met.

The Facility is conceived as having a maximum design capacity of 400,000 tpy of waste material (The water supply requirements for the maximum design capacity of 400,000 tpy Facility are addressed in **Appendix D**). Based on prorated water efficiency from the 140,000 to 400,000 tpy scenarios, the future expanded Facility would have an estimated water demand of 3.8 L/s. While it is anticipated that the existing watermain along Osbourne Road could meet future water demand estimates, this will be determined through detailed design and pressure testing at the time. If Facility water demands cannot be met through connection to the primary 300 mm watermain on Osbourne Road, a secondary 300 mm watermain would be required. The closest available watermain capable of providing the additional water is located on the South Service Road east of Maple Grove Road approximately 3.5 km away.

The proposed Facility would be located within the Clarington Energy Business Park which has developed a master drainage plan (Aecom, 2009) which outlines the recommended design criteria for respective developments and suggests water conservation measures to be implemented on each development Site. This new document promotes the reusing of collected stormwater, the limiting or elimination of the use of potable water for landscape irrigation and the minimization of Facility footprint. These and other recommendations would be considered during the detailed design phase.

The firewater storage tank for the Facility is proposed to hold 1,135,000 L. Online firewater demand would be determined during the detailed design phase. In addition, a demineralized and condensate water tank with a capacity of 30,000 L is proposed for the Site.

The maximum annual wastewater discharge is proposed to be 3,000 m^3/yr or, assuming a continuous 365 day operation, 8219 L/day (0.1 L/s). This wastewater generation threshold is based on 140,000 scenario. To the extent possible, wastewater generated onsite would be reused within Facility operations. Preliminary Facility design suggests that onsite wastewater treatment would be minimal (solids removal) and that wastewater discharge would be to the sanitary sewer. There is an existing 1800 mm diameter sanitary sewer stub located north of the CN Rail tracks on Osbourne Road adjacent to the Site. The Facility is conceived as having a maximum design capacity of 400,000 tpy of waste material. The higher waste handling rate is not expected to increase wastewater generation considerably above the current expectations of 8219 L/d based on 140,000 tpy scenario (See **Appendix D** for description of wastewater requirements of the 400,000 tpy scenario). However, a preliminary assessment (Jacques Whitford and Genivar, 2007a) concluded that connecting a 450 mm gravity drain to the existing 1800 mm municipal sewer stub would be capable of conveying 63 L/s which would more than meet the Facility's wastewater discharge requirements.

The Courtice WPCP, located immediately south of the Site, would treat the Facility's wastewater. Wastewater quality from the Site is expected to comply with the Regional Municipality of Durham's Sewer Use By-law # 43-2004.

5.0 IMPACT MANAGEMENT

This section begins with a detailed discussion of the recommended construction phase erosion and sediment control features (lot level, conveyance and end-of-pipe classes) that would be necessary for a development of this type. Subsequently, the operational phase stormwater management features are summarized with minimum regulatory sizing guidelines provided. This section concludes with a discussion on the mitigation measures recommended for the subject development.

5.1 Erosion and Sediment Control Plan

The grubbing, topsoil stripping and grading of lands slated for development increases the potential for erosion, can increase the quantity and decrease the quality of runoff and can adversely affect water quality in nearby watercourses. To minimize these deleterious effects, erosion and sediment control measures could be implemented and surface preparation scheduled accordingly.

An Erosion and Sediment Control (ESC) plan would be developed to ensure that all appropriate measures are taken and that surface preparation is scheduled to minimize soil loss. An effective ESC plan would outline Site properties, areas of concern, mitigation measures planned, project scheduling, maintenance requirements and emergency response information. In addition, drawings detailing the current Site topography, location of existing watercourses and drainage, proposed contours and location of erosion and sediment control measures must be provided. A properly detailed and planned ESC plan is an important component of any land development project and is required to meet municipal and provincial permitting and approval requirements. A sample ESC plan checklist is provided in **Appendix E**.

Grubbing and stripping of land should be scheduled so as to minimize the amount of time the land is left bare and exposed. Parcels not immediately required for construction should be left in their natural state to avoid unnecessary erosion and soil loss. Subsequently, the timing of surface alteration should be planned to avoid periods of heavy rain and snow melt if possible.

Parcels requiring clearing should be designed to minimize the size of disturbed area, minimize the grade and slope length and attempts should be made to retain as much native vegetation as possible. In addition, stockpiles of soil should be kept a minimum of 15 m away from any watercourses, drainage features or slope toes. In general, it is required that on both banks a minimum buffer of 15 m for warm water streams and 30 m for cold water streams be left undisturbed (MNR/MOEE, 1991). The swale traversing the southern boundary of the Site has not been officially designated and is intermittent in nature and therefore a 15 m buffer is recommended.

Once a parcel of land is cleared, runoff originating from this area is typically larger in volume, has a higher average velocity and transports larger quantities of suspended sediment compared to pre-development conditions. Properly designed and implemented erosion and sediment control measures would act to minimize these effects. Erosion and sediment control can be accomplished through measures at the lot level (erosion control), during conveyance (sediment control) and at end-of-pipe facilities. For the most effective results, runoff waters should be subjected to a number of erosion and sediment control measures in series, referred to as a treatment train.

During construction, rough grading should be designed to convey all stormwater runoff to the southwestern stormwater pond. Lot level and conveyance controls could be implemented to reduce runoff velocities, prevent erosion and trap mobile sediment. The following mitigation measures are recommended at the Site.

For ESC recommendations pertaining to the construction activities required to upgrade the Facility to the 400,000 tpy scenario, refer to **Appendix D**.

5.1.1 Construction Phase Lot Level Control

The use of surface stabilization measures to reduce the amount of soil detached and transported during precipitation events is a pivotal aspect of the treatment train. Prevention of soil detachment and transport could be accomplished through the use of vegetation, geotextiles and surface manipulation.

If native vegetation must be removed, then vegetated filter strips should be planted on slopes <10% and parallel to existing watercourses, if possible. Vegetated buffer strips running perpendicular to flows act to decrease runoff velocities, increase infiltration and filter sediment-laden flows. For cold water streams, filter strips should be at least 30 m wide and for warm water streams they should be at least 15 m wide (GGHACA, 2006). Filter strips should have no vehicle access, be properly maintained and receive weekly inspections. The 15 m watercourse setback zones recommended in Section 5.1 should act as filter strips preferably by retaining existing vegetation, or by vegetation planting and establishment. Filter strips should not be mowed or clipped.

All other areas not required for equipment storage, building construction or vehicle access should be seeded as soon as possible to avoid excess soil loss. Seeding should be performed for shallow slopes (<2H:1V) containing a minimum of 150 mm of disturbed topsoil at final grade and left for 30 days. Seed application should take place between April 15 – May 30 or August 15 – September 30 (GGHACA, 2006).

Slopes >5% may require a stabilizing lining. Erosion control blankets or growth media should be installed on slopes, in swales and on the sides of berms when these areas are being left for prolonged periods of time (>6 months) (GGHACA, 2006). Erosion control/growth media can be synthetic or derived from composted materials. Both erosion control blankets and growth media should be applied in the direction of flow with a minimum overlap of 10 cm and 1 to 3 m of material being staked down over the shoulder of slopes. Whereas erosion control blankets are applied as surficial measures, growth media should be buried from 25 to 100 mm below grade to ensure seed survival (GGHACA, 2006).

Sediment traps should also be installed within flow paths, slope toes and surrounding drains to minimize the amount of sediment deposited in conveyance networks and detention ponds.

To ensure no negative effects to nearby watercourses, it is recommended that silt fencing be installed around the perimeter of all laydown areas, disturbed working areas and the boundary of the Site. A synthetic, geotextile material with a minimum weave density of 270 R stretched between posts should be used to filter any runoff not routed to the detention pond. Fencing should be buried to a depth of 150 mm. Silt fencing should be inspected after every heavy rainfall event and accumulated sediment should be removed once it reaches half the structure height (MOE, 2003 and GGHACA, 2006).

All laydown areas, storage areas and access roads should receive a top dressing of gravel as soon as possible after project initiation. This would act to reduce runoff velocities, and retain underlying soil materials.

Surface roughening (scarification) is an effective method of increasing infiltration, decreasing runoff velocity and limiting the amount of soil loss through erosion. It is recommended that all areas not required for equipment storage or infrastructure foundation receive adequate scarification in the form of 50 to 100 mm depressions organized 100 to 150 mm apart. This surface manipulation applied to final grade would enhance seed bed preparation.

It is anticipated that during the construction phase, ESC lot level controls including vegetated filter strips, seed application, stabilizing liners, sediment traps, silt fencing, gravel top dressing and scarification would be implemented. These measures would act to retain sediment, reduce runoff velocities and encourage infiltration of stormwater.

5.1.2 Construction Phase Conveyance Controls

Appropriate measures should be implemented to intercept sediment laden runoff waters before they are transported offsite. The reduction of runoff velocities and the filtering of runoff flows would help detain mobile sediment and enhance the quality of stormwater leaving a development parcel. It is anticipated that onsite construction phase stormwater conveyance would be accomplished through the construction of swales and/or the installation of stormsewers routing to the stormwater pond.

If conveyance swales are to be used, they should run parallel to road surfaces and around the perimeter of the Site and construction laydown areas. Swales should be constructed during initial grading or at the earliest possible date. Perimeter swales should span the lowest end of construction laydowns, parking lots and storage areas. These perimeter swales should be located down-slope of silt fences and discharge to the main conveyance swales to be routed to the stormwater pond.

Swale channel gradient should not exceed 1% (MTO, 1997) and side walls should be compacted with a slope not exceeding 2H:1V. To convey adequate quantities of runoff, swales should have minimum dimensions of 300 mm deep and 600 mm wide. Swales in use for more than 30 days should be vegetated if possible and riprap installed at inlet/outlet locations to avoid further erosion (GGHACA, 2006). Swales remaining for periods of 6 to 12 months should have erosion control blankets installed (GGHACA, 2006).

To encourage infiltration of stormwater runoff during conveyance, swales should be lined with a crushed stone lining underlain by native sands. This design would limit erosion within the swale but still allow infiltration throughout. All impervious or hardened areas should be drained to these pervious swales in an attempt to offset the expected decrease in infiltration potential.

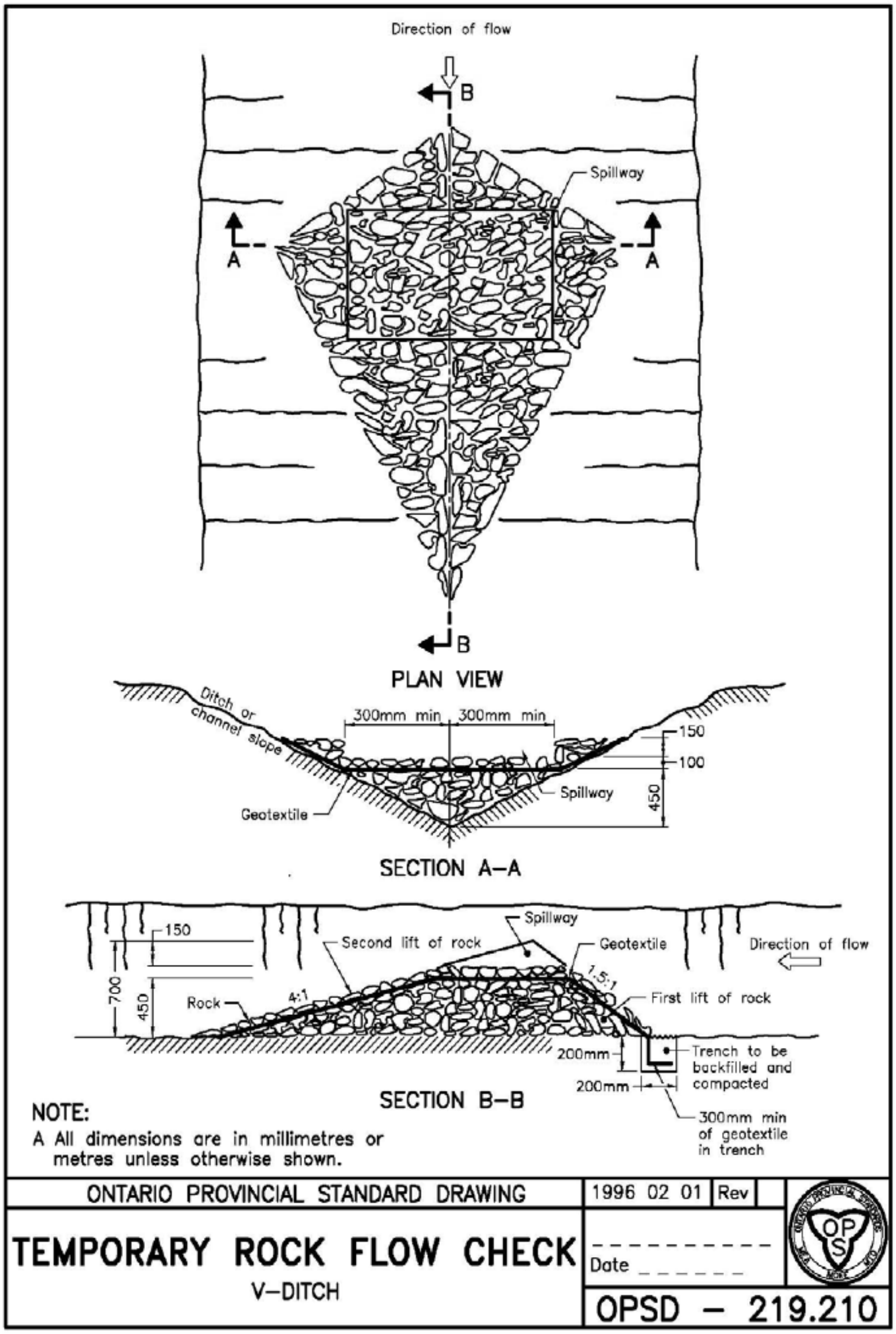
To control the quantity, and in turn quality, of stormwater runoff in the perimeter swales, in-channel devices to slow the propagation of runoff water may be necessary. Creating physical obstructions or diversions to the flow of stormwater runoff acts to decrease velocity and allow for deposition of suspended sediment. Straw bales and filter fences are routinely used but rock check dams have proven to be the most structurally sound and long lasting velocity dissipation alternatives.

Rock check dams are comprised of a layer of smaller stone (approximately 0.45 m of 50 mm stone) under a layer of larger stone (150 mm stone to the top of the conveyance invert) separated by a non-woven geotextile. Check dams should have a maximum upstream slope of 1.5H:1V and a maximum downstream slope of 4H:1V. Flow is directed toward the top-centre of the check dam by including a 0.15 m V- or U-notch spillway. A diagram outlining the important features and specifications of rock check dams is provided in Figure 5-1. Dams such as this should not be used for drainage areas over 3 ha (MTO, 1997 and GGHACA, 2006).

If stormsewers are to be installed during the preliminary stages of construction, swales may be unnecessary. Stormsewers installed during construction phases would remain as operational stormwater conveyance routes and would therefore be sized according to SWM criteria.

If stormwater from the Site is to be routed to the swale along the CN Rail corridor during the construction phase, the conveyance capacity of the swale must not be exceeded or the channel would require widening prior to the initiation of construction.

Figure 5-1 General Design Specifications of a Rock Check Dam



5.1.2.1 Swale and/or Stormsewer Sizing

There is no specific regulatory guideline for temporary ESC swale design. The recommended precipitation event sizing criteria range from the 2 to the 10-year event depending upon Site conditions, geographic location, and duration of construction. The planned construction period for the Facility is approximately 30 months and therefore the 5-year storm has been recommended as a reasonable sizing event. The runoff component of this storm event would form the volumetric sizing criteria for swale design. Perimeter swales should be constructed during rough grading activities to assist with the effective management of stormwater during the construction phase.

Swales to remain onsite as permanent SWM conveyance features would need to be sized according to SWM criteria. SWM swale sizing regulations are described in Section 5.2.1. The location and route of temporary ESC swales and permanent SWM swales would be determined during detailed design.

If stormsewers are to be installed as construction phase stormwater conveyance mechanisms they would be sized according to SWM criteria as described in Section 5.2.1.

Conveyance swales represent the minor SWM system onsite (MOE, 2003 and MTO, 2008). The major system is considered overland flow for the events which overflow the minor system capacity. Site grading should be designed to provide the requisite major system overland flow route for larger events.

5.1.2.2 Vehicle Access

Since vehicle access would be required for the development of the Site, vehicular grade mud mats would be required. Where soil loss via vehicle transport is deemed extreme, tire washing stations may be necessary. Any site greater than 1 ha requires a mud mat to reduce the amount of sediment leaving disturbed sites via vehicle tires. Mud mats are comprised of a stone pad that must be a minimum of 20 m long, occupy the entire width of the entrance/exit and extend 300-450 mm below grade. The first 10 m from the access road should be 50 mm diameter clear stone whereas the rest of the pad can be 150 mm clear stone. The entire mud mat must be underlain by an appropriate geotextile material (GGHACA, 2006). Where access points also perform culvert crossings, sediment retention fencing must be in place to prevent materials deposited from vehicle tires from reaching local watercourses or drains.

At any point that a vehicle access route crosses a swale within the Site (i.e., construction laydowns) a construction phase culvert would be required. Ontario Ministry of Transportation (MTO) guidance recommends that the conveyance design storm for temporary culverts with a span of <6 m is the 5-year event (MTO, 2008). Although there would not be any Ministry of Transportation (MTO) governed roads onsite, this criteria was deemed acceptable for temporary culverts. For design guidelines for permanent culverts and culverts located along Osbourne Road see Section 5.2.1.2.

Culvert crossings should include a 0.3 m freeboard between high water level and the edge of the traveled lane. In order to account for this, a 0.1 m void should be assumed at obvert of the culvert assuming the remaining 0.2 m of freeboard could be accounted for by road surface elevation. This 0.1 m void would also help to prevent debris jams and ice effects. Since onsite swales would be intermittent conveyers, the 15 to 30% submersion recommended in GGHACA (2006) is not applicable.

Vehicle access culverts within permanent conveyance swales would be sized according to SWM criteria. Permanent culvert design is described in Section 5.2.1.2. Details regarding the number, size and temporal requirement of vehicle access culverts would be determined in the detailed design phase.

5.1.3 Erosion and Sediment Control End-of-Pipe Facility

Typically during civil construction, the rough grading and stormwater management end-of-pipe facilities are completed during the initial phases. Since the erosion and sediment control pond necessary during construction is also expected to remain to serve as the operational phase stormwater management pond, more rigorous SWM sizing criteria would be used. Section 5.2.2.1 describes the SWM pond design criteria. The ESC and SWM sizing criteria would be reviewed and compared in this later section.

5.2 Operational Phase Stormwater Management

The Facility would be designed to ensure that there would be no contamination of surface water from solid waste processing activities. Stormwater management would be designed to convey all onsite stormwater to one retention pond located within the lower southwest corner of the property. Discharge from this Facility is to be conveyed approximately 1000 m northwest via a small swale and discharged into Tooley Creek approximately 850 m north of its Lake Ontario outlet. To maintain appropriate water quality and quantity within Tooley Creek, care must be taken to ensure discharged stormwater from this development is of adequate quality and appropriate quantity to preserve geomorphological conditions and maintains the aquatic integrity of the watercourse.

To achieve water quality and quantity goals, SWM features must be designed to meet pre-development runoff conditions described in Section 3.4.3 while accommodating post-development runoff volumes described in Section 4.4.1, addressing potential changes in the water balance and following applicable stormwater quality improvement and flood control criteria provided in MOE (2003).

The two main objectives of stormwater quantity and quality control are: post-development peak discharges are similar or less than those arising from the existing conditions, and a 24 hour drawdown must be accommodated for the discharge of the extended detention volume. These objectives are met through the design of lot, conveyance level controls and end-of-pipe SWM features.

Since infiltration only accounted for approximately 3.7% of the pre-development water balance, the reduction in infiltration arising from the development would be minimal and would be mitigated by encouraging infiltration in open green spaces. The anticipated decrease in ET would be mitigated by maintaining green space and vegetation onsite to the extent possible. The main focus of post-development mitigation would be the control of runoff quality and quantity.

The following mitigation measures are recommended for the 140,000 tpy scenario Site for permanent stormwater management purposes. For SWM recommendations pertaining to the 400,000 tpy scenario, refer to **Appendix D**.

5.2.1 Lot Level and Conveyance Controls

In general, an increase in impervious area within the Site would cause an increase in runoff and a decrease in ET and infiltration potential. To offset these impacts lot level and conveyance level SWM features are recommended accordingly to detain the volume and reduce the flow rate of runoff at the lot level before stormwater enters and as it routes through the conveyance system. Detention of runoff at the lot level through depression storage and reduced runoff flow rates would act to encourage ET and infiltration.

Maintaining the existing condition water balance for the Site would help ensure no local water resources are negatively affected. Lot and conveyance measures recommended for this Site would encourage depression storage, the slowing of runoff waters and the maximization of ET opportunities. Since a decrease in infiltration is not a significant concern on this Site, open landscaped areas would suffice to allow adequate infiltration to occur. The measures recommended in this section are specifically designed to reduce runoff quantities and speeds, and encourage ET where possible.

Permanent, post-development lot level controls planned for the Facility would include the following:

- All area not used for parking, vehicle access, waste storage or facility infrastructure would be re-vegetated and returned to drainage and infiltration conditions similar to existing conditions;
- Impervious surfacing would be minimized to asphalt road and parking surfaces, concrete pads and building roofs; and,
- To the extent possible, developed lands not requiring asphalt would receive a layer of crushed stone and maintain minimal slopes which would act to increase lot level depression storage, and slow the overland runoff rate of stormwater.

5.2.1.1 Onsite Conveyance

Permanent stormwater conveyance is anticipated to be a combination of both swales and underground piping draining catchbasins within impervious areas. It is important that these structures are designed properly to minimize runoff velocity and therefore reduce sediment transport to end-of-pipe facilities. Conveyance measures should be implemented as soon as possible during construction in an attempt to limit sediment loss. The swales and underground stormsewers would represent the minor stormwater system onsite however, the major system, necessary for the conveyance of large runoff events, would be accomplished through grading design. The major system would also direct overland flow towards the permanent SWM pond.

Generally, minor system stormwater conveyance is sized to convey between the 2-year and the 10-year precipitation event (MOE, 2003). MTO (2008) suggests for local scale roads, the minor conveyance system should convey the 5-year event; however MTO drainage Directive B-100 (MTC, 1980) recommends culverts on local road classes to be sized to convey the 10-year storm event.

As this Site is located within the Clarington Energy Business Park, it is ultimately subject to the stormwater management criteria developed as part of the Park's Master Drainage Plan (Aecom, 2009) and that of the Municipality of Clarington. Onsite stormwater conveyance systems in Clarington are to be designed to convey the 5-year pre-development precipitation event.

The minimum acceptable diameter for underground storm sewers is 250 mm. Stormsewer infrastructure would be specified and designed in accordance with the Municipality of Clarington infrastructure design and construction standards (Aecom, 2009). The proportion of swales and underground storm sewers utilized, location and route of conveyance network would be determined as part of the detailed design phase.

It may be prudent to consider the use of Oil and Grit Separators (OGS) within the subsurface stormwater conveyance network. Upstream of the end-of-pipe facility, OGS can provide spill control and pre-treatment for stormwater discharged to the SWM pond (CLOCA, 2007b). The design specifications and total suspended solids removal efficiencies can vary significantly between OGS features. The potential for installation, number of OGS needed and type used will be considered during detailed design.

5.2.1.2 Vehicle Access

Vehicle access points crossing permanent swales or those providing access from Osbourne Road would require appropriately sized culverts. The minimum culvert diameter for a crossing of <10 m is 400 mm. The recommended sizing event for culverts <6 m in length is the 10-year storm and for culverts >6 m in length is the 25-year storm (MTO, 2008). The Municipality of Clarington and the Clarington Energy Business Park defer to the above-mentioned design guidelines. The number, location and size of required culverts would be determined during the detailed design phase.

5.2.2 End-of-Pipe Facilities

The end-of-pipe facility design recommended for this project takes into account water quality and erosion control considerations according to MOE (2003). There are three levels of receiving water protection established for stormwater management features: enhanced, normal and basic. End-of-pipe storage volumes are based upon this level of protection. The Central Lake Ontario Conservation Authority has suggested that discharge from this Site be subject to Enhanced protection levels due to Tooley Creek's (the Site's eventual receiving water) cold water fisheries (CLOCA, 2007a). Therefore, SWM features would be designed to the Enhanced protection level.

Erosion control considerations stipulated by applicable regulatory agencies state that peak discharges arising from end-of-pipe facilities must be equal to, or lower than, the peak discharges generated from the Site, given existing conditions. The water quality consideration is served by ensuring a minimum 24 hour drawdown for the extended detention volume of the SWM pond. These objectives would be controlled by pond storage volume and outlet size, shape and configuration.

During detailed design, additional hydrogeological assessment should be carried out in the location of the SWM pond. This investigation will be used to determine groundwater levels and soil conditions and avoid groundwater : surface water interactions with the stormwater pond.

5.2.2.1 Pond Design

Although many pond designs are possible, this section provides recommended specifications given development land conditions and constraints. The most commonly utilized end-of-pipe facility is the wet pond. The wet pond is a good alternative due to its superior suspended sediment removal capabilities. Although adequate water quality and quantity control can be obtained through the exclusive use of a wet pond, the inclusion of a wetland component (described as a “hybrid” pond system) can improve water quality, aesthetics and ecological objectives.

Both wet pond and hybrid pond system specifications are described separately below.

5.2.2.2 Wet Pond

To achieve appropriate internal water velocities within the pond, pond length should be designed to ensure that waters taking a direct path from inlet to outfall (effectively maintaining the highest average velocity) are reduced to the target suspended solid settling velocity. In general, the overall length to width ratio (including inlet forebay – discussed below) should be greater than 3:1. The maximum pond depth should be between 3 to 5 m for both permanent and active storage, yet still provide the required 0.3 m of freeboard between the high water mark and the crest of the pond embankment. The overall bed slope from inlet to outlet should be approximately 1% (MOE, 2003).

The side walls of sedimentation ponds need to resist erosion and remain accessible for maintenance purposes. Side walls should have a maximum slope of 5:1 for 3 m on either side of the permanent pool and approximately 3:1 everywhere else (MOE, 2003).

To reduce velocities, lengthen flow paths and enhance rapid suspended solid settling, SWM ponds should contain a forebay area. Forebays are the first section that runoff waters enter and are responsible for the majority of velocity reductions and sedimentation of larger grain-sized particles. Forebays should be a minimum depth of 1 m and comprise a maximum of 33% of the total permanent pool area. Marking the end of the forebay area and the beginning of the remaining pond Region requires an underwater berm sitting approximately 0.15-0.30 m below the permanent pool level (MTO, 2006). The forebay should have a maximum length to width ratio of 2:1 (MOE, 2003).

Sediment accumulation in ponds should be measured a minimum of every six months and removal/dredging activities performed when 50% of the forebay capacity has been filled (GGHACA, 2006). Inlets, outlets, embankments and spillways should be inspected weekly and after every heavy rainfall or snow melt event (GGHACA, 2006).

Each SWM pond should be equipped with a permanent pool clean-out pipe which would remain closed until needed. This bottom draw drain would inlet at the bottom of the permanent pond pool and outlet beyond the pond outlet structure. This mechanism is used to completely drain the SWM pond for maintenance purposes.

5.2.2.3 Hybrid Pond System

The hybrid wet pond/wetland system is simply the combination of wet pond and wetland elements in series. This hybrid system provides the deep water element (wet pond) necessary for continued functionality during winter months as well as enhanced biological removal and aesthetic properties (wetland) during the summer months (MOE, 2003).

A wet pond element of the hybrid pond system should be designed according to all wet pond specific specifications listed above. The wetland element should be designed according to the following specifications.

The constructed wetland element of an end-of-pipe stormwater management facility is more land intensive compared to simple wet ponds due to their shallower depth in both permanent pool and extended storage. The benefits of constructed wetlands are as follows:

- Performance is not dependent upon soil characteristics;
- Vegetation minimizes the re-suspension of particulates;
- Enhanced biological removal of pollutants; and,
- Lower velocities encourage extended settling.

To maximize the overall flow path and minimize short-circuiting potential the wetland element should have a minimum length–to-width ratio of 3:1. The average permanent pool depth should range from 150 mm to 300 mm depending upon vegetation requirements. In order to sustain vegetation within the wetland element the active storage depth should be a maximum of 1 m for storms <10-year event (MOE, 2003).

For safety purposes, the wetland's side wall ratio should be 5:1 for 3 m above and below the permanent pool and the maximum slope should be 3:1 elsewhere. The inlet/outlet for the wetland element should have a minimum slope of 1% and a minimum diameter of 450 mm to avoid clogging and freezing (MOE, 2003).

During the winter months the permanent pool volume would freeze and the wetland element would effectively behave similar to a dry pond with its active storage component providing the only benefit.

The combination of the wet pond and wetland elements require the following caveats to ensure appropriate design and functionality.

- a forebay is not necessary in the wetland element as the wet pond component serves this purpose;
- the detention time for entire system should be 24 hours;
- the length-to-width ratio for wet pond element can be reduced to 2 to 1; and,
- the active storage depth restrictions for wetlands apply to the entire system, unless a terraced overflow configuration is adopted.

5.2.2.4 Minimum Pond Storage Volumes

To adequately control the water quality discharging from the end-of-pipe SWM facilities, minimum retention and detention volumes are required (MOE, 2003). The stormwater pond consists of both a permanent pool retention level and an extended storage detention level. The permanent pool level is the lowest level to which the pond can be drawn down during normal operation. The extended storage volume is storage above the permanent pool which accommodates both the water quality event and the flood control volume. During a typical precipitation event stormwater conveyed to the SWM pond mixes with the existing permanent pool water.

The extended storage volume is detained and released slowly in order to obtain the maximum reduction in suspended solids load specified by the enhanced level of protection. The pond and outlet structure should be designed to allow a minimum average detention time of 24 hrs for the pond's extended detention volume. The volume above the extended detention is called the flood control volume, which should accommodate the 100-year event. Discharge from the flood control volume (and all those of lesser magnitude) must not exceed the peak discharge rate of the pre-development conditions for a similar sized storm event.

Wet ponds and hybrid pond systems have different minimum storage requirements and are therefore described separately below. The following end-of-pipe features discussion is based on the current Facility footprint and layout, which is in turn, based on Facility initial design capacity of 140,000 tpy. The Facility is conceived as having a maximum design capacity of 400,000 tpy of waste material. The higher waste handling rate may require an increase in the impervious footprint of the Facility, which would subsequently require expansion of end-of-pipe stormwater storage capacity (See **Appendix D**). Such SWM system alterations would require MOE approval and an amendment to the stormwater C of A.

5.2.2.5 Wet pond

For a site with approximately 45% imperviousness, the minimum permanent pool volume for an enhanced level of protection is 125 m^3/ha of drainage area. Similarly, the minimum extended storage volume for enhanced level of protection is 40 m^3/ha (MOE, 2003). These values were obtained by interpolating between the respective values for sites with 35% and 55% imperviousness.

For the 12.4 ha Site, the minimum required extended detention storage 496 m^3 which is considerably less than the 1820 m^3 of post-development runoff generated during the 25 mm event. For this reason, the extended detention component of the SWM pond is expected to be sized to approximately 1820 m^3 or 147 m^3/ha.

A conservative approach to designing the flood control volume for the stormwater pond is to assume full volumetric containment of the 100-year storm runoff event. Some water above the permanent pool storage capacity would be discharged during the inflow of the flood control volume but to employ the above conservative approach, the total 100-year precipitation event volume was used. Based on the HEC-HMS model developed for this assessment, the post-development total runoff volume for the 100-

yr event was 8038 m^3. A summary of required stormwater pond storage volumes is provided in Table 5-1.

Table 5-1 - Required Pond Volumes for an Enhanced Level of Protection in a Wet Pond

Pond Volumes	Enhanced Level of Protection	
	Required Pond Volumes (m3/ha)	Development Site Pond Volume (m3)
Quality Control Criteria	80% SS removal	na
Permanent Pool	125	1550
Extended Storage*	25mm event runoff	1820
Flood Control Volume	100 yr event runoff	6218 (8038-1820)
Total Stormwater Pond Volume	na	9588

*40 m^3/ha is the minimum required extended detention storage volume (MOE, 2003). The extended detention volume should ensure a minimum 24 hours of drawdown to the 25 mm precipitation event.

The SWM pond would first act as an ESC pond during construction. ESC pond sizing regulations require at least 125 m^3/ha for permanent pool storage and an additional 125 m^3/ha for extended detention storage (GGHACA, 2006 and MTO, 2008). The flood control volume for ESC ponds is dependent upon the planned duration of construction. In this case the 5-yr precipitation event is considered reasonable. Given these design guidelines, the total required ESC pond storage volume is 6339 m^3 which is less than the SWM regulations calculated above. Designing the ESC pond with SWM sizing guidelines would enable the pond to provide appropriate water quantity and quality control during both construction and operational phases.

5.2.2.6 Hybrid Pond System

For a site with approximately 45% imperviousness, the minimum permanent pool volume for an enhanced level of protection is 90 m^3/ha of drainage area. Similarly, the minimum extended storage volume for enhanced level of protection is 40 m^3/ha (MOE, 2003). These values were obtained by interpolating between the respective values for sites with 35% and 55% imperviousness. Similar to the wet pond assessment, the post-development runoff from the 25mm design storm would be used to further assess the extended storage volume.

Assumptions used to determine the volume of the flood control event (100-year event) are applicable to this assessment as well. A summary of required stormwater pond storage volume is provided in Table 5-2.

Table 5-2 - Required Pond Volumes for and Enhanced Level of Protection in a Hybrid Pond System

	Enhanced Level of Protection	
Pond Volumes	Required Pond Volumes (m3/ha)	Development Site Pond Volume (m3)
Quality Control Criteria	80% SS removal	na
Permanent Pool	90	1116
Extended Storage	25mm event runoff	1820
Flood Control Volume	100 yr event runoff	6218 (8038-1820)
Total Stormwater Pond Volume	na	9154

Regardless of the type chosen, the stormwater end-of-pipe facility should be constructed during the rough grading phase in order to minimize the water quality degradation caused by sediment loss from a disturbed site.

The stormwater quantity criteria requires that the peak discharge rates from the stormwater pond be equivalent to, or less than, the peak discharge arising from the same event given pre-development conditions. In addition, the 25 mm design storm must receive a minimum of 24-hours of drawdown to ensure an enhanced level of water quality protection. These stormwater objectives would be accomplished through the design of the SWM pond and outlet size and configuration.

5.2.2.7 SWM Facility Outlet

The primary pond outlet structure is presented conceptually as a bottom-draw hickenbottom riser to discharge the extended detention volume estimated at 1820 m^3. The orifice opening configuration of the riser would ensure that stormwater receives at least a 24-hour drawdown period. The riser would inlet below the elevation of the permanent pool to reduce thermal impacts associated with discharging water from pond surfaces. The riser would also connect to a reversed slope pipe which would have an outlet invert at the elevation of the permanent pool. As such the reversed slope pipe outlet would control discharge and ensure that the permanent pool elevation is maintained. The hickenbottom riser should be located 150 mm below the maximum expected ice depth to ensure continued functionality during winter conditions and 30 cm below the permanent pool elevation is the recommended riser height.

Subsequently, storage volumes exceeding the extended detention volume up to the flood control volume may be discharged additionally by a weir structure. The total flood control volume includes the 100-year event (8038 m^3). The combination of a primary hickenbottom riser and weir discharges must never exceed 0.5 m^3/s which equates to the pre-development peak discharge for the 100-year event. The outlet configuration would be finalized during detailed design to ensure that post-development peak flows are less than existing conditions.

The SWM pond outlet structures should be designed with a gate or valve mechanism capable of restricting pond discharge if an accidental spill has migrated into the SWM system. See Section 5.5 for further discussion on potential spill containment.

Although peak shaving can be accomplished through outlet design, the volume of discharge arising from a precipitation event in the post-development scenario would inevitably be larger than that arising from a similar sized event during pre-development conditions. The other important component to stormwater management is the accommodation of the addition event runoff volumes. This is mitigated through baseflow augmentation. By ensuring pre-development peak discharge rates are not exceeded while releasing larger volumes of stormwater, the temporal duration of event discharge is extended. This baseflow augmentation can be beneficial to downstream aquatic species by providing more sustained streamflow during otherwise intermittent periods.

Detention time is an important component of stormwater quality management and the pond is assumed to behave hydraulically as a plug flow system. Since the extended detention volume is only slightly larger than the permanent pool volume, it is assumed that during a 25 mm event most of the permanent pool volume is discharged. The average inter-event rainfall period in the Oshawa area is 3.08 days, meaning that the permanent pool volume is retained typically for a minimum of 3.08 days between runoff events. Therefore, the runoff volume of the 25 mm event would receive a minimum of 3.08 days of detention in the SWM pond in addition to the additional average detention afforded by the 24 hour drawdown required by stormwater management guidelines.

In summary, the SWM pond and outlet structures would be designed to ensure that post-development peak discharges would not exceed pre-development peak discharges for similar sized precipitation events. In addition, the SWM facility would provide at least 24-hours of drawdown to the 25 mm precipitation event. The final SWM pond and outlet configuration would be provided during the detailed design.

5.2.3 Outfall Channel

Pond outlet channels require a design that resists erosion and scour and effectively conveys the peak discharge from the design storm away from the outlet. Although energy dissipaters are recommended at all outlets, they should not be the sole method of erosion control. Outlet channel geometry must be able to limit the peak discharge to below erosional thresholds.

Immediately downstream of the pond outlet, the conveyance channel would act as a splash pool for higher velocity outlet discharge and therefore requires adequate armouring to minimize erosion and scour. An energy dissipating riprap or cobble is recommended to slow outflow velocities before deposition into the CN Rail swale.

To avoid tail water effects at or behind the outlet, an outlet discharge channel slope of 2 to 3% is recommended.

5.2.3.1 Outfall Protection

To reduce downstream flow velocity and dissipate energy, riprap, gravel, sand bags or concrete can be used as dissipating media dependent upon flow magnitude. As a general rule, dissipater length L=4.5*d, where d is the diameter of the outfall pipe or channel. Similarly, dissipater width W= 4*d. For plan and cross section views of a general energy dissipater design refer to Figure 5-2.

Energy dissipaters should not be relied upon as the sole source of flow attenuation, instead, channel geometry and bed slope should be designed to minimize outfall velocities to below 0.15 m/s which is the rate at which natural channels begin to erode (MOE, 2003). Alternatively, higher water velocities may be possible if channel vegetation and stone armouring are considered. If rock or gravel is being used for energy dissipation, a D_{50} of between 150 mm to 300 mm is advisable and a total pad thickness of 1.5 times the maximum rock thickness is required. Stone dissipaters should be underlain by a filtration bedding of 0.15 m of well graded sand or appropriate geotextile fabric (OPSS 577, 2006). Table 5.3 provides stone sizing criteria for use in slowing runoff waters of varying velocities (MTO, 2008).

Table 5-3 - Recommended Stone Sizing for Erosion Control (MTO, 2008)

Stone Sizes For Scour And Erosion Protection - Low Volume Roads							
Velocity (m/s)	< 2.0	< 2.6	< 3.0	< 3.5	< 4.0	< 4.7	< 5.2
Nominal Stone Size (1)	100	200	300	400	500	800	1000
Notes 1) maximum stone size to be 1.5 times the nominal stone size. 80% of stones (by mass) must have a diameter of at least 60% of nominal stone size.							

5.2.4 Offsite Conveyance Channel Upgrading

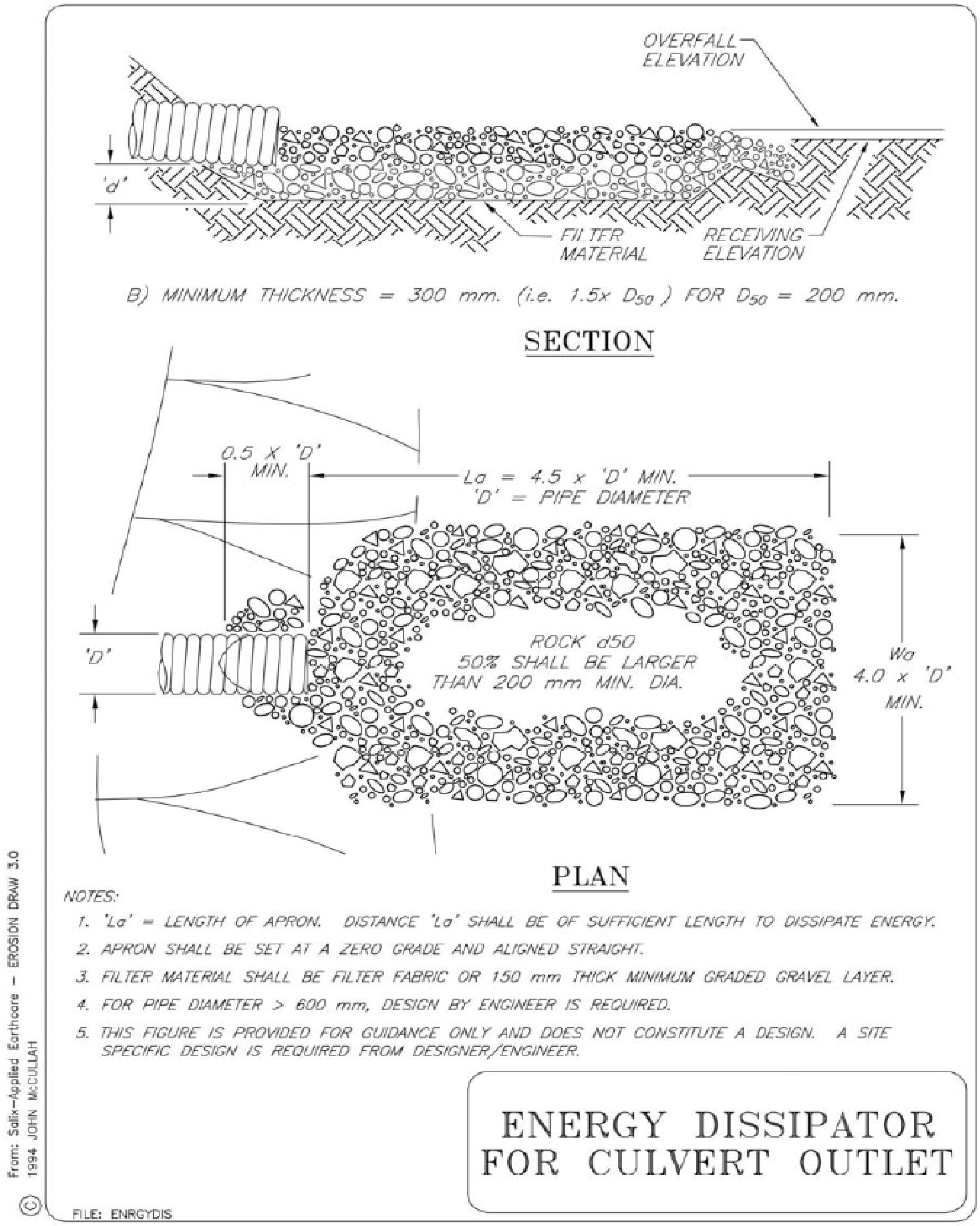
Currently the CN Rail swale adjacent to the Site can convey an estimated maximum discharge of $0.14 m^3/s$ (Section 3.4.3.1). This flow rate corresponds approximately to the peak discharge for the pre-development 5-year precipitation event. Since post-development stormwater discharges would not exceed pre-development values, the probability of the CN Rail swale overflowing its banks would not change with the development of this Site.

If there is concern regarding the conveyance capacity of the CN Rail swale, two options are available: the SWM pond and outlet structures can be designed to maintain peak discharges below the $0.14 m^3/s$ threshold; or, the swale can be widened/upgraded. Within 200 to 300 m of the Site's discharge point, the CN Rail swale increases in size and is capable of conveying flows in excess of the peak discharge from the 100-year event. Expansion of CN Rail swale capacity would require approval from and completion by CN Rail.

Future development of the Clarington Energy Business Park is expected to include centralization of SWM ponds and increased conveyance requirements for existing stormwater routing channels (Aecom, 2009). As development proceeds, upgrades would be conducted as necessary.

Conveyance capacity details would be determined during the detailed design phase.

Figure 5-2 Profile and Plan View of a Cobble Energy Dissipater (OPSS 577, 2006).



5.3 Regulatory Approvals Required

The proposed project would require several approvals in order to finalize the design stage and proceed into the beginning stages of construction. The Site is located within the jurisdiction of the Central Lake Ontario Conservation Authority and may need to complete an Application for Development, Interference with Wetlands and Alterations to Shorelines and Watercourses permit according to the Conservation Authorities Act-Regulation 179/06.

In addition, according to Section 53 of the *Ontario Water Resources Act*, the Facility would require a Certificate of Approval for a sewage works from the Ministry of the Environment for the stormwater management system. Since the Facility infrastructure is planned to reach approximately 7.6 mbg and would most likely penetrate the surficial aquifer, a Permit To Take Water is expected to be required from the MOE for construction phase dewatering.

5.4 Groundwater Management

Preliminary designs for the solid waste containment infrastructure suggest the foundation footing would be approximately 7.6 mbg which may penetrate the surficial aquifer. However, the geotechnical investigation conducted onsite (Jacques Whitford, 2007) advanced boreholes to below 7.6 mbg and did not encounter any artesian conditions and therefore it is not anticipated that the Facility infrastructure would penetrate the confined aquifer which is suggested to occur in the area (See hydrogeological discussion in Section 3.3). Dewatering and excavation pumping is expected in order to establish a sufficiently dry environment to construct the Facility foundations. Once the foundation is in place, lateral groundwater flow would once again saturate the area and pressure would be placed on the concrete infrastructure.

The concrete foundation and floor slabs must be designed to withstand the pore pressure that would be exerted by the surrounding groundwater table. To relieve some of the groundwater pressure, perimeter drains designed to encompass the foundation and convey groundwater towards the lower southwest corner of the property may be installed.

It is recommended that a series of groundwater monitoring wells be installed within the Site to assess the Facility's long term effects on both groundwater quantity and quality.

5.5 Accidents and Malfunctions

The Facility would have a fully developed environmental control plan that examines all potential accidents and malfunctions. Specifically, this Report considers accidents and malfunctions that may result in a spill release of material with the potential to contaminate soil or water. The environmental control plan would focus on spill prevention through reduction of chemical use, the use of biodegradable chemical materials, work practice training and accidents and malfunctions monitoring and surveillance. In the unlikely event of a spill or malfunction, Facility design, contingency plans and backup systems would be in place to ensure no deleterious substances are released to the surrounding environment. The potential for contaminant spills within the subject lands of the Facility are represented by three categories: Spills within facility buildings; Spills outside facility buildings, and; firewater release. Each would be described separately and contingency systems outlined.

Potential spills within facility buildings:

It is anticipated that the main Facility building would be situated on a concrete pad with appropriately sized spill containment systems equipped with a shutoff valve to ensure any spills are contained within the Facility. It is anticipated that all main Facility containment reservoirs would be exclusively within the building structure and connected to the sanitary sewer system therefore ensuring contaminants would not be released to the Facility's surroundings. Collected wastewater would meet or exceed the Regional Municipality of Durham's Sewer Use By-law #43-2004 water quality criteria prior to be discharged to the municipal sewermain.

It is anticipated that included in the wastewater and spill containment systems would be adequately sized oil and water separators and that separated oil would be removed by vacuum truck and shipped offsite to an approved treatment facility. Onsite spill containment systems would be completed during the detailed design phase.

Spills outside facility buildings:

Several major process zones would be located outside Facility buildings or within infrastructure possessing large access openings. These include the tipping floor and the refuse building. It is anticipated that these components would be situated on asphalt surfacing or concrete pads equipped with drains and catchbasins. The initial response in case of a spill outside of the main Facility buildings would be to deploy absorbent material to the spill area to capture as much of the released substance as possible. If the spilled material migrates away from the spill containment measures, it would be routed to the stormsewer network which would convey the substance to the SWM pond. The pond is expected to be equipped with an outlet gate capable of retaining all stormwater within the feature. Subsequently, retained contaminated stormwater can be trucked offsite and treated appropriately. Since this stormwater pond would be designed to capture and attenuate the flood control event, accidental spills migrating to the pond are not at risk of overflowing the gated outlet in instances of heavy rainfall events.

The probability of spills occurring outside of Facility buildings and asphalt or concrete surfaced process areas is minimal. Landscaped and open areas surrounding the Facility would not contain operational machinery or chemical storage and would therefore be the source of "clean" stormwater. Therefore landscaped and non-operational open areas within the 12.4 ha development area would be at very low risk of receiving spill contaminated stormwater.

The detailed environmental control plan would be in place prior to the initiation of Facility construction. This plan should include information on the following:

- pollution prevention and source control by best management land use practices and best management stormwater practices;
- construction refuelling precautions;

- monitoring the Facility for leaks;
- stockpiling of materials or devices for spill control;
- spill containment;
- buffers and setbacks; and,
- fast accurate reporting of spills.

Fire water release:

In the unlikely event of a fire at the Proposed Thermal Treatment Facility, water used to suppress the fire may be contaminated with any of the substances used or stored onsite, fire-suppressing materials as well as with substances generated by the fire itself. Given the volume of water necessary to fight a plant fire, there is a potential for runoff contamination. It is anticipated that contaminated firewater would be contained within Facility floor drains and sump pits and potentially routed to the SWM pond which would have its outlet valve closed prior to reception of the contaminants. Potentially contaminated firewater runoff would be collected in the SWM pond and evacuated by pump truck for appropriate treatment.

It is anticipated that spill containment infrastructure and protocols both within the Facility and in-place outside of the Facility would be designed to minimize accidental contamination of groundwater. Floor drains and underground tanks are expected to collect and contain all potential spills. All spilled material would be evacuated from underground tanks and pits prior to discharge to sanitary sewers.

6.0 NET ENVIRONMENTAL EFFECTS

The following provides a discussion of net effects related to the Proposed Thermal Treatment Facility specifically related to the onsite and surrounding water resources. The discussion focuses on how the Facility and SWM design accommodates climate change considerations and discusses potential effects to the receiving water environment, including groundwater and surface water features, and potential effects on water balance, water quality, erosion, and fluvial geomorphology and aquatic habitat.

6.1 Climate Change

Potential climate change impacts on stormwater management include the potential for increasing frequency and magnitude of more large precipitation events, potential effects of warmer temperatures on stormwater discharge and the potential for the general decrease in snowpack depth and thus snowmelt. Reductions in snowpack depth would appear to reduce the flooding potential associated with large rainfall and snowmelt events in winter, however, it is expected that more wet precipitation would fall during winter. This may increase rain-induced snowmelt and result in increased winter flooding.

The concern arising from the increased frequency of extreme precipitation events is that the current, statistically-derived 100-year storm event, may be considerably smaller than the 100-year storm event that may be applicable nearer to the end of the proposed Facility's expected life cycle. Thus, the challenge in planning for climate change is that the future size of the 100-year design storm is unclear, though it is expected to be larger than it is currently.

It has been recommended that the SWM pond be sized to fully contain the current 100-year event. In fact, the pond would be discharging during the inflow of the 100-year event and therefore oversized for capture of the actual required volume. This overestimate can be used to accommodate for climate changes mentioned above. In addition, the additional 30 cm freeboard above the flood control volume can be used for additional storage if the size of the 100-year event were to increase.

As a contingency consideration, the pond is to be located in the southwest corner of the property which represents the lowest elevation onsite and also an area free from development. In the event of a storm larger than the predicted 100-year, the immediate surroundings may be able to act as flood storage without incurring property or infrastructure damage.

Currently, SWM pond discharges represent a warm water contribution to the cold water Tooley Creek. This influence may worsen with an increased temperature caused by climate change. The use of a submerged inlet hickenbottom riser as the primary outlet structure would discharge water from the pond from deeper and cooler pond depths.

6.2 Evapotranspiration, Infiltration and Runoff

Development of previously undisturbed lands can alter the existing water balance by decreasing infiltration and evapotranspiration potential and in turn increasing runoff. A loss of infiltration and ET through the introduction of imperviousness reduces opportunities for interception and depression storage and directly adding volume to runoff. Potential influences like these can have negative environmental effects unless mitigation measures are adequately designed and implemented.

Prudent stormwater management proposed for the Facility was designed to offset the effects of the development on ET and runoff, and to a lesser degree, infiltration. Recommendations and project components which would offset the loss of ET include:

- Re-vegetation of lands not needed for Facility operation;
- Open water evaporation from stormwater management end-of-pipe facility; and,
- Encourage use of green roof technology, rooftop storage and other lot level controls that increase depression storage and runoff detention before it enters the conveyance system.

During construction, low slope grading, the use of gravel top dressing in laydown areas, silt fencing, sediment traps, rock check dams, vegetated filter strips and ESC pond would be employed to reduce runoff velocities and encourage evaporation, sedimentation and infiltration.

The net reduction in ET onsite caused by grubbing and the increase in imperviousness would act to increase the overall runoff volumes generated onsite. Although this effect cannot be mitigated fully, steps would be taken to control runoff water quantity and quality including the use of low slope Facility grading, maximizing facility greenspacing and end-of–pipe controls. These mechanisms act to slow the overland release of stormwater and thereby increase evaporation and transpiration potential. The SWM pond and outlet have been designed specifically for the purpose of peak shaving, attenuation of flow and water quality improvement. Attenuated stormwater discharge to the CN Rail swale would act to augment baseflow which would enhance downstream aquatic habitat and moderate intermittent flows within the watercourse.

6.3 Water Quality

Potential water quality impacts stemming from the Facility include the discharge of degraded quality runoff and the accidental release of contaminants. Mitigation and emergency spill response measures discussed in Section 5.5 have specifically targeted the topic of accidental contaminant release.

The Site's SWM pond has been designed according to specific project based topography, climatic regime and receiving water classification. An enhanced level of protection has been recommended given aquatic habitat conditions in Tooley Creek. Lot level and conveyance controls have also been recommended to reduce runoff velocities and trap/deposit mobile sediment. Based on the treatment train approach utilized in the SWM plans, suspended sediment levels in runoff discharging offsite would be minimal.

Detention time is an important component of stormwater quality management. Since the extended detention volume is only slightly larger than the permanent pool volume, it is assumed that the 25mm event discharge would behave hydraulically as a plug flow system. In other words the SWM pond discharge resulting from the inflow of the 25 mm event would comprise stormwater previously stored within the permanent pool volume. Since the average inter-event rainfall period in the Oshawa area is 3.08 days, the permanent pool volume is retained typically for a minimum of 3.08 days between runoff events. Therefore, the runoff volume of the 25 mm event would receive a minimum of 3.08 days of detention in the SWM pond in addition to the additional average detention afforded by the 24-hour drawdown required by stormwater management guidelines.

In the unlikely event of a spill, emergency response and spill containment plans proposed for the Facility would ensure that the surrounding water resources would not be impacted. Facility wastewater containment pits, enclosed chemical storage areas, outdoor spill containment protocols and a controllable SWM pond outlet have all been proposed for the Proposed Thermal Treatment Facility Site.

6.4 Cumulative Effects

The previous sections describe how the current proposed design as well as mitigation recommendations combine to yield no or minimal environmental effects or net effects. However, while no or minimal net effects are anticipated individually, it is important to consider all net effects of the Facility in addition with effects of other current and planned and disclosed future projects in the area to determine whether any cumulative effects may result.

The proposed Facility would be located within the Clarington Energy Business Park which has been identified as an appropriate location for prestige employment and light industrial uses that can benefit from close proximity to the Darlington Nuclear Generating Station. Since a majority of the park remains undeveloped, it is likely that the properties surrounding the Facility would become developed at some point in the future. Currently, there is only one other development proposed for the Clarington Energy Business Park. The prospective developer is still currently reviewing site applicability and therefore no facility specifications are available.

Based on the foregoing discussion of combined effects associated with water resource management for the Facility, and that there is only one other known future project proposed for the area, no negative cumulative effects are anticipated.

7.0 SUMMARY AND CONCLUSION

Summary of major findings includes:

- The Facility would discharge to a coldwater stream;
- The Site is underlain by approximately 395mm of topsoil followed by a silty-sand layer to the extent of the boreholes developed onsite;
- Due to low slopes and vegetation cover, soil erosion from overland flow is considered minimal;
- The existing condition water balance for the Site is reasonably subdivided into outputs as follows: 58.5% ET, 2.5% net recharge, 1.3% interflow, and 37.7% runoff;
- The swale located within the CN Rail corridor adjacent to the Site is estimated to have the capacity to convey the 5-year storm event runoff from the Site;
- Any capacity upgrades considered for the CN Rail swale to accommodate larger runoff events must be approved and completed by CN Rail;
- The Facility's water demand and wastewater discharge requirements can be accommodated through a connection to the municipal service systems;
- The Facility foundation would penetrate the local water table, however it is not anticipated that excavations would extend to a deeper underlying confined aquifer;
- The total required stormwater pond volume for permanent pool, extended detention, and flood control volumes is approximately 9588 m^3;
- Pond design criteria would meet or exceed design guidance criteria found in the MOE Stormwater Management Planning and Design Manual;
- Increase in runoff potential would be mitigated with peak flow attenuation, baseflow augmentation and stormwater management design that provides an enhanced level of receiving water protection;
- Further hydrogeological investigation is recommended during detailed design to fulfill permitting and dewatering requirements as well as inform foundation and stormwater infrastructure design;

- Accidents and malfunctions planning and spill management redundancy and stormwater control from source to discharge would ensure the protection of surface water and groundwater resources; and,
- No net negative cumulative effects are anticipated from the Facility.

Summary of recommended monitoring includes:

- Stormwater end-of-pipe facility discharge quality (required as part of C of A);
- Groundwater quality monitoring at and surrounding the Facility, and;
- Groundwater levels.

8.0 CLOSURE

This Report has been prepared by Jacques Whitford Stantec Limited. The assessment represents the conditions at the subject property only at the time of the assessment, and is based on the information referenced and contained in the Report. The conclusions presented herein respecting current conditions, and potential future conditions are at the subject property resulting from the Facility, represent the best judgment of the assessor based on current environmental standards. Jacques Whitford Stantec Limited attests that to the best of our knowledge, the information presented in this Report is accurate. The use of this Report for other projects without written permission of Durham Region, York Region and Jacques Whitford Stantec Limited is solely at the user's own risk.

9.0 REFERENCES

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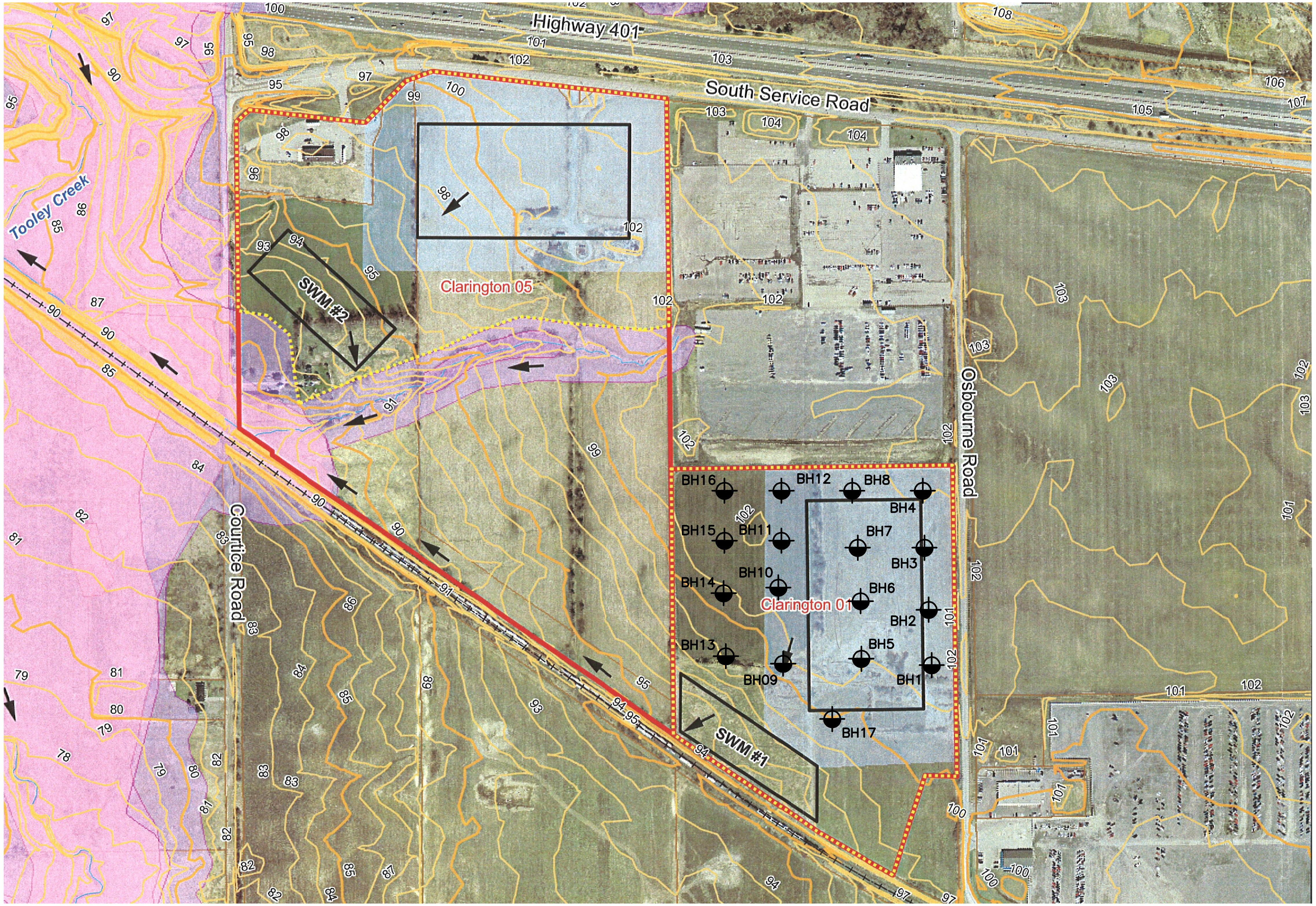
APPENDIX A

Bore Hole Logs and Location Site Plan



MAP FEATURES

- Alternative Site
- Usable Site Area
- Site Infrastructure
- Site Layout
- Parcel
- Drainage Flow Pattern
- Elevation Contour (1 m Interval)
- Elevation Contour (5 m Interval)
- Natural Heritage Feature
- Waterbody
- Watercourse
- Roadway CL
- Railway

LEGEND

- BOREHOLE LOCATION

NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A JACQUES WHITFORD LIMITED REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

Reference: DURHAM/YORK RESIDUAL WASTE STUDY. FIGURE 2. DATED: SEPTEMBER 2007	**Job No.:** 1009497.01 **Scale:** 1:5000 **Date:** 2008/04/10 **Dwn. By:** JTG **App'd By:**	**Client:** REGIONAL MUNICIPALITY OF DURHAM **Site Address** DURHAM AND YORK REGION, ONTARIO, CANADA	DURHAM/YORK RESIDUAL WASTE STUDY
BOREHOLE LOCATIONS	**Dwg. No.:** 1	Jacques Whitford	

T:\1009XXX\1009497.01 Clarington\1009497.01-1.dwg PRINTED: Apr 10, 2008

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 1

CLIENT: Region of Durham — PROJECT No.: 1009497.01

LOCATION: Osbourne Rd., Clarington, Ontario — DATUM: Local

DATES: BORING January 16, 2008 — WATER LEVEL — TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	99.2	TOPSOIL
	98.7	Compact to very dense brown silty SAND - with gravel - weathered and trace clay at upper layer
3		- grey below 3m

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) / TCR(%) / SCR(%)	N-VALUE OR RQD(%)
2	SS	1	360/460	10
4	SS	2	460/460	27
7	SS	3	430/460	32
9	SS	4	150/460	50/150mm
14	SS	5	460/460	24
19	SS	6	430/460	30
24	SS	7	360/460	44
29	SS	8	380/460	60/100mm

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

□ Field Vane Test (kPa)

▣ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 1

CLIENT Region of Durham

LOCATION Osbourne Rd., Clarington, Ontario

DATES: BORING January 16, 2008

WATER LEVEL

PROJECT No. 1009497.01

DATUM Local

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	STRATA PLOT	WATER LEVEL	DEPTH (ft)	SAMPLES TYPE	SAMPLES NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
10	89.2				33				
					34	SS	9	150/460	50/150mm
11					36				
12	87.0				39	SS	10	130/460	50/30mm
		- Water table at a depth of 0.9m at the end of Boring			40				

UNDRAINED SHEAR STRENGTH (kPa) 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 2

CLIENT Region of Durham

PROJECT No. 1009497.01

LOCATION Osbourne Rd., Clarington, Ontario

DATUM Local

DATES: BORING January 16, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	99.1	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL	0				
	98.6	Compact brown sandy SILT - trace gravel and clay - weathered	3	SS	1	460/460	14
	97.7	Dense brown silty SAND - with gravel	6	SS	2		37
			8	SS	3	410/460	42
3		- trace clay below 3m	11	SS	4	430/460	36
		- grey below 3.6m					
			16	SS	5	460/460	22
		- Very dense below 5m					
			21	SS	6	360/460	50/150mm
	92.5	- Water table at a depth of 3.0m at the end of Boring					

UNDRAINED SHEAR STRENGTH (kPa) 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 3

CLIENT: Region of Durham

LOCATION: Osbourne Rd., Clarington, Ontario

DATES: BORING January 16, 2008

WATER LEVEL

PROJECT No. 1009497.01

DATUM Local

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	99.3	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL	0–2				
0.6	98.7	Compact brown sandy SILT - trace gravel	3	SS	1	460/460	19
1.4	97.9	Compact to dense brown silty SAND - with gravel	6	SS	2	460/460	26
			8	SS	3	430/460	28
		- trace clay below 3.1m	11	SS	4	430/460	36
		- grey and compact below 3.6m	16	SS	5	460/460	22
			21	SS	6	460/460	22
6.6	92.7	- Borehole is dry at the end of Boring					

UNDRAINED SHEAR STRENGTH (kPa) 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 4

CLIENT: Region of Durham

LOCATION: Osbourne Rd., Clarington, Ontario

DATES: BORING: January 16, 2008

WATER LEVEL:

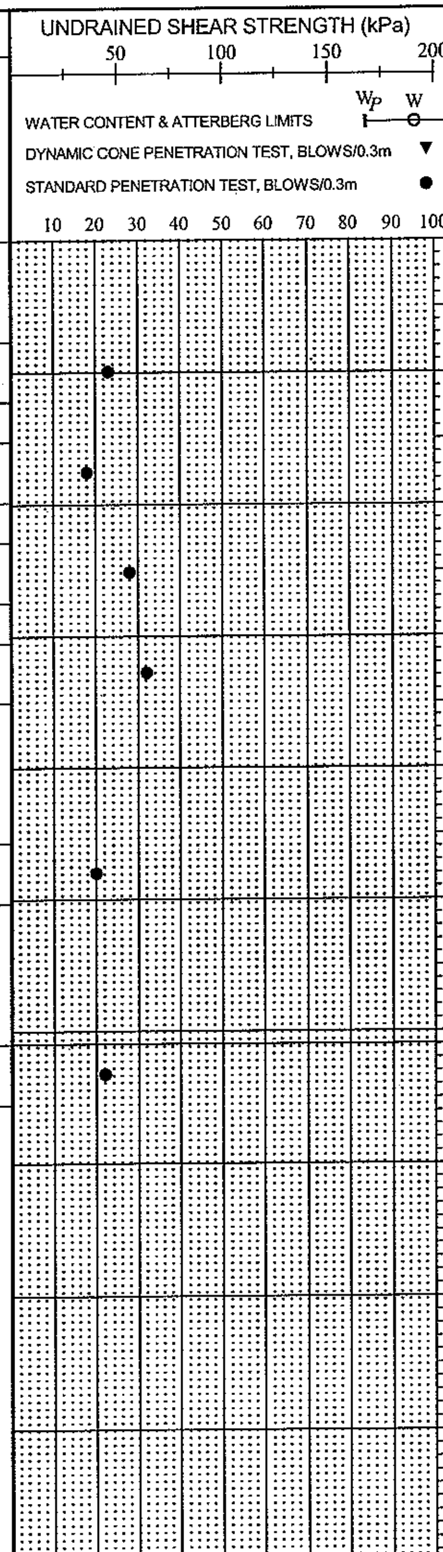
PROJECT No.: 1009497.01

DATUM: Local

TPC ELEV.:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	99.7	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL
	99.2	Compact brown sandy SILT - with clay trace gravel
1	98.5	Compact to dense brown silty SAND - with gravel - trace clay at upper layer - grey below 3.3m - compact below 4m
	93.1	- Water table at a depth of 1.0m at the end of Boring

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
3	SS	1	460/460	23
6	SS	2	460/460	18
8	SS	3	460/460	28
11	SS	4	410/460	32
16	SS	5	430/460	20
21	SS	6	460/460	22



REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 5

CLIENT Region of Durham

PROJECT No. 1009497.01

LOCATION Osbourne Rd., Clarington, Ontario

DATUM Local

DATES: BORING January 17, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	99.8	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL	0				
	99.3	Compact to very dense brown silty SAND - with gravel					
1			3	SS	1	460/460	30
2			6	SS	2	410/460	50/125mm
3			8	SS	3	230/460	50/75mm
			11	SS	4	100/460	50/100mm
4		- grey below 4m					
5			16	SS	5	430/460	68
6	93.7	- Borehole is dry at the end of Boring	21	SS	6	100/460	60/75mm
10			32				

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P, W, W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 6

CLIENT: Region of Durham

PROJECT No. 1009497.01

LOCATION: Osbourne Rd., Clarington, Ontario

DATUM: Local

DATES: BORING January 17, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	WATER LEVEL
0	99.2	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL	
0.3	98.9	Compact to dense brown clayey SILT - trace sand and gravel, weathered	
2.7	96.5	Compact to very dense silty SAND - with gravel - grey below 3.6m	▼ (2.7 m)
6.1	93.1	- Water table at a depth of 2.7m at the end of Boring	

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
3	SS	1	430/460	19
6	SS	2	460/460	43
8	SS	3	430/460	27
11	SS	4	430/460	28
16	SS	5	150/460	50/50mm
21	SS	6	150/460	50/50mm

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

□ Field Vane Test (kPa)

■ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 7

SHEET 1 of 2

CLIENT Region of Durham

PROJECT No. 1009497.01

LOCATION Osbourne Rd., Clarington, Ontario

DATUM Local

DATES: BORING January 17, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	99.4	
	99.1	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL
		Compact brown clayey SILT - with sand trace gravel, weathered
	98.0	Compact to very dense silty SAND - with gravel
		- grey below 4.0m - trace clay between depth 3m to 4m
10		

Water level symbol shown at approximately 18.5 ft depth.

SAMPLES

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
3	SS	1	460/460	18
6	SS	2	430/460	20
8	SS	3	460/460	33
11	SS	4	460/460	46
16	SS	5	430/460	60
21	SS	6	460/460	95
26	SS	7	410/460	50/125mm
31	SS	8	410/460	89

UNDRAINED SHEAR STRENGTH (kPa)

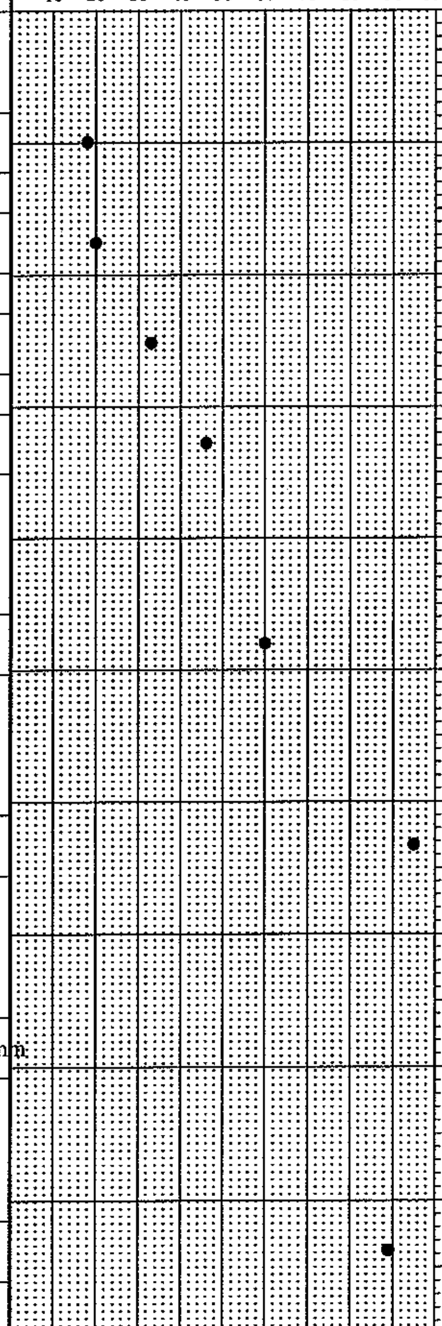
50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100



REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 7

CLIENT Region of Durham

PROJECT No. 1009497.01

LOCATION Osbourne Rd., Clarington, Ontario

DATUM Local

DATES: BORING January 17, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	STRATA PLOT	WATER LEVEL	DEPTH (ft)	SAMPLES TYPE	SAMPLES NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
10	89.4				33					
					34					
					35					
11					36	SS	9	100/460	50/50mm	
					37					
					38					
12	87.2				39	SS	10	100/460	50/50mm	
		- Water table at a depth of 5.7m at the end of Boring			40–65					
13–20										

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

□ Field Vane Test (kPa)

■ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

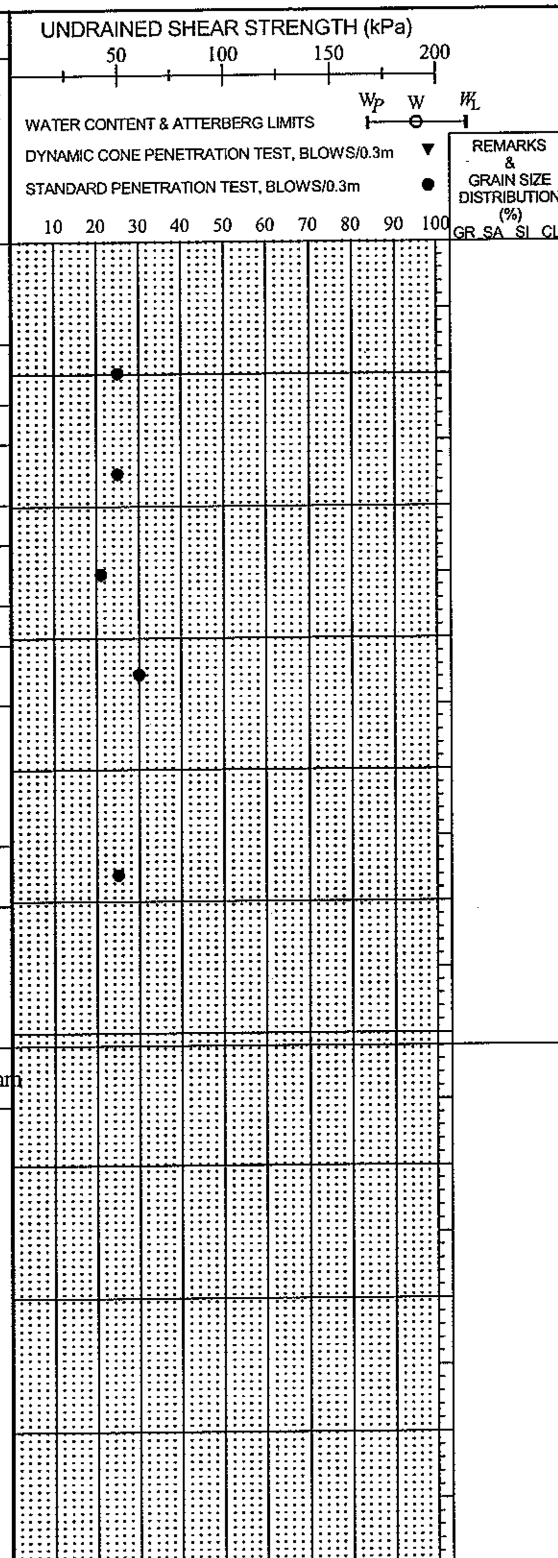
BH 8

CLIENT: Region of Durham — PROJECT No. 1009497.01

LOCATION: Osbourne Rd., Clarington, Ontario — DATUM: Local

DATES: BORING January 17, 2008 — WATER LEVEL — TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	99.5	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL	0				
	99.0	Compact brown clayey SILT - with sand trace gravel, weathered	3	SS	1	460/460	25
	98.1	Compact to very dense silty SAND - with gravel - trace clay at upper layer	6	SS	2	460/460	25
			8	SS	3	460/460	21
			11	SS	4	230/460	30
		- grey below 3.6m					
			16	SS	5	460/460	25
6	93.4	- Borehole is dry at the end of Boring	21	SS	6	100/460	50/30mm



□ Field Vane Test (kPa)
■ Remoulded Vane Test (kPa)
△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 9

CLIENT Region of Durham
LOCATION Osbourne Rd., Clarington, Ontario
DATES: BORING January 17, 2008
WATER LEVEL
PROJECT No. 1009497.01
DATUM Local
TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	WATER LEVEL	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	97.7	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL		0				
	97.2	Compact to very brown dense silty SAND - with gravel	▼	3	SS	1	430/460	23
	96.2	- Very dense SAND and GRAVEL		6	SS	2	230/460	55/75mm
3		- grey below 3m		11	SS	3	51/460	60/75mm
				16	SS	4	150/460	60/125mm
				19	SS	5	180/460	50/50mm
6	91.6	- Water table at a depth of 0.9m at the end of Boring		20				

UNDRAINED SHEAR STRENGTH (kPa) 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)
◘ Remoulded Vane Test (kPa)
△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 10

CLIENT Region of Durham

PROJECT No. 1009497.01

LOCATION Osbourne Rd., Clarington, Ontario

DATUM Local

DATES: BORING January 18, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	99.1	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL	0				
	98.6	Very dense brown silty SAND - with gravel					
1			3	SS	1	410/460	90
2			6	SS	2	410/460	95
3			11	SS	3	100/460	50/75mm
5		- grey below 4.8m	16	SS	4	380/460	88
6	93.0	- Borehole is dry at the end of Boring	21	SS	5	410/460	85

UNDRAINED SHEAR STRENGTH (kPa) 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)

■ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 11

CLIENT Region of Durham

PROJECT No. 1009497.01

LOCATION Osbourne Rd., Clarington, Ontario

DATUM Local

DATES: BORING January 18, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	SAMPLES TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	99.8	Dark brown clayey silty sand, organic matter and rootlets: TOPSOIL	0–2				
	99.3	Compact brown sandy SILT - with gravel trace clay	3	SS	1	410/460	19
	98.4	Very dense brown silty SAND - with gravel	6	SS	2	410/460	50/100mm
			11	SS	3	150/460	60/125mm
		- grey below 4.7m	16	SS	4	230/460	50/75mm
6	93.7	- Borehole is dry at the end of Boring	21	SS	5	150/460	60/125mm

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 12

CLIENT: Region of Durham

PROJECT No.: 1009497.01

LOCATION: Osbourne Rd., Clarington, Ontario

DATUM: Local

DATES: BORING: January 18, 2008

WATER LEVEL:

TPC ELEV.:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	99.6	TOPSOIL	0				
	99.3	Compact to very dense brown silty SAND - with gravel	1–2				
1			3–4	SS	1	430/460	24
			5–6	SS	2	230/460	20
2		- trace silt below 2.1m	7–9				
3			10–11	SS	3	100/460	60/75mm
4			12–15				
5			16	SS	4	150/460	60/100mm
		- grey below 5.1m	17–18				
			19	SS	5	360/460	60/100mm
6	93.5	- Water table at a depth of 5.4m at the end of Boring	20–32				
10							

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P, W, W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 13

CLIENT: Region of Durham

PROJECT No.: 1009497.01

LOCATION: Osbourne Rd., Clarington, Ontario

DATUM: Local

DATES: BORING January 17, 2008

WATER LEVEL:

TPC ELEV.:

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0	96.5		0					
	96.2	TOPSOIL						
		Compact to very dense brown silty SAND - with gravel	3	SS	1	460/460	16	
			6	SS	2	460/460	58	
			11	SS	3	460/460	87	
		- grey below 4m	16	SS	4	100/460	60/75mm	
			19	SS	5	100/460	57/75mm	
6	90.4	- Borehole is dry at the end of Boring	20					

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 14

CLIENT: Region of Durham

PROJECT No.: 1009497.01

LOCATION: Osbourne Rd., Clarington, Ontario

DATUM: Local

DATES: BORING January 18, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	97.5						
	97.2	TOPSOIL					
		Very dense brown silty SAND - with gravel					
1			3	SS	1	430/460	56
		- dense between depth 1.3m to 2.4m	5–6	SS	2	410/460	28
		- very dense below 2.4m					
3			10–11	SS	3	230/460	50/50mm
4		- grey below 4m					
5			15–16	SS	4	150/460	60/100mm
6	91.4	- Water table at a depth of 5.1m at the end of Boring	20–21	SS	5	360/460	60/100mm

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 15

CLIENT Region of Durham

LOCATION Osbourne Rd., Clarington, Ontario

DATES: BORING January 18, 2008

WATER LEVEL

PROJECT No. 1009497.01

DATUM Local

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	99.4		0				
	99.1	TOPSOIL					
		Dense to very dense brown silty SAND - with gravel					
1			3	SS	1	430/460	28
			6	SS	2	410/460	62
3			11	SS	3	100/460	50/75mm
5		- grey below 4.8m	16	SS	4	100/460	60/75mm
			19	SS	5	250/460	60/75mm
6	93.3	- Water table at a depth of 5.4m at the end of Boring	20				
10							

UNDRAINED SHEAR STRENGTH (kPa) 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 16

SHEET 1 of 2

CLIENT Region of Durham

PROJECT No. 1009497.01

LOCATION Osbourne Rd., Clarington, Ontario

DATUM Local

DATES: BORING January 18, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	WATER LEVEL	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	99.3			0				
	99.0	TOPSOIL						
		Very dense brown silty SAND - with gravel		1–2				
1				3	SS	1	430/460	73
				5–6	SS	2	430/460	55/75mm
2		- trace silt below 2.1m		7–9				
3				10–11	SS	3	150/460	55/50mm
4				12–14				
				15–16	SS	4	230/460	74
5		- grey below 4.8m		17–19				
6				20–21	SS	5	430/460	60
7			▼	22–24				
		- trace clay below 7.5m		25–26	SS	6	460/460	31
8				27–29				
9				30–31	SS	7	380/460	50/125mm
10				32				

UNDRAINED SHEAR STRENGTH (kPa) 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 16

CLIENT Region of Durham

PROJECT No. 1009497.01

LOCATION Osbourne Rd., Clarington, Ontario

DATUM Local

DATES: BORING January 18, 2008

WATER LEVEL

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	STRATA PLOT	WATER LEVEL	DEPTH (ft)	SAMPLES TYPE	SAMPLES NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
10	89.3				33					
					35–36	SS	8	230/460	50/75mm	
11										
					39	SS	9	280/460	60/75mm	
12	87.1	- Water table at a depth of 7.2m at the end of Boring			40					
13–20					41–65					

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P, W, W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

JACQUES WHITFORD LIMITED

BOREHOLE RECORD

BH 17

CLIENT Region of Durham

LOCATION Osbourne Rd., Clarington, Ontario

DATES: BORING January 17, 2008

WATER LEVEL

PROJECT No. 1009497.01

DATUM Local

TPC ELEV.

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	97.1		0				
	96.8	TOPSOIL					
		Dense to very dense brown silty SAND - with gravel - trace clay at upper layer	1–2				
1			3	SS	1	460/460	42
2			6	SS	2	150/460	50/125mm
			8	SS	3	100/460	50/75mm
3		- grey below 3.3m	11	SS	4	230/460	50/75mm
			16	SS	5	100/460	60/50mm
5	92.1	- Borehole is dry at the end of Boring	17				

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

□ Field Vane Test (kPa)

◘ Remoulded Vane Test (kPa)

△ Pocket Penetrometer Test (kPa)

Jacques Whitford

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil	-	mixture of soil and humus capable of supporting good vegetative growth
Peat	-	fibrous fragments of visible and invisible decayed organic matter
Till	-	unstratified and unsorted glacial deposit which may include particle sizes from clay to boulders
Fill	-	materials not identified as deposited by natural geological processes

Terminology describing soil structure:

Desiccated	-	having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	-	material breaks along plane of fracture
Varved	-	composed of regular alternating layers of silt and clay
Stratified	-	alternating layers or beds greater than 6mm (1/4") thick
Laminated	-	alternating layers or beds less than 6mm (1/4") thick
Blocky	-	material can be broken into small and hard angular lumps
Lensed	-	irregular shaped pockets of soil with differing textures
Seam	-	a thin, confined layer of soil having different particle size, texture, or color from materials above and below
Well Graded	-	having wide range in grain sizes and substantial amounts of all intermediate particles sizes
Uniformly Graded	-	predominantly one grain size

Soil descriptions and classification are based on the Unified Soil Classification System (USCS) (ASTM D-2488), which classifies soils on the basis of engineering properties. The system divides soils into three major categories: (1) coarse grained, (2) fine-grained, and (3) highly organic. The soil is then subdivided based on either gradation or plasticity characteristics. This system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification. The classification excludes particles larger than 76 mm.

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with the standard of the Ministry of Transportation of Ontario:

Trace or occasional	Less than 10%
Some	10-20%
With	20-30%

The standard terminology to describe cohesionless soils includes the compactness as determined by the Standard Penetration Test 'N'-value*.

Compactness	'N'-value
Very loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very dense	>50

Continued Next Page

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

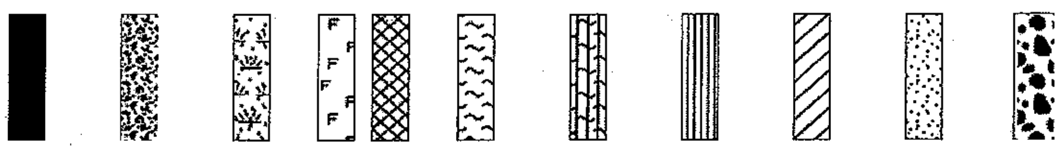
The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis. Standard Penetration Test 'N'-values* can also be used to provide an approximate indication of the consistency and shear strength of fine grained, cohesive soils.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12.5	<2
Soft	12.5-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

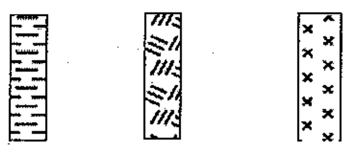
Note: *'N'-VALUE- The Standard Penetration Test records the number of blows of a 140 pound (64kg) hammer falling 30 inches (760mm), required to drive a 2 inch (50.8mm) O.D. split spoon sampler 1 foot (305mm). For split spoon samples where full penetration is not achieved, the number of blows is reported over the sampler penetration in millimeters (e.g. 50/75).

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



Asphalt *Concrete* *Topsoil* *Fill* *Peat* *Organic Silt* *Silt* *Clay* *Sand* *Gravel*



Sedimentary Rock *Metamorphic Rock* *Igneous Rock*

WATER LEVEL MEASUREMENT

Open Borehole or Test Pit

Monitoring Well, Piezometer or Standpipe

SAMPLE TYPE

SS	Split spoon sample (obtained from the Standard Penetration Test)
TW	Thin Wall Sample or Shelby Tube
PS	Piston sample
GS	Grab sample
AS	Auger sample
VT	Vane Test
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ,etc.	Rock core samples obtained with the use of standard size diamond drilling bits.

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

ROCK DESCRIPTION

Total Core Recovery (TCR): The percentage of drill core recovered, regardless of quality, or length measured relative to the length of the total core run.
Solid Core Recovery (SCR): The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.
Rock Quality Designation (RQD): The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run.

RQD	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very Poor, crushed, very severely fractured

Terminology describing rock mass:

Discontinuities		Bedding, Lamination, Bands
Spacing (mm)	Description	
2000-6000	Very Wide	Very Thick
600-2000	Wide	Thick
200-600	Moderate	Medium
60-200	Close	Thin
20-60	Very Close	Very Thin
6-20	Extremely Close	Laminated
<6		Thinly Laminated

Strength classification of rock:

Strength Classification	Field Identification Method	Range of Unconfined Compressive Strength (MPa)
Extremely weak	Indented by thumbnail	<1
Very weak	Crumbles under firm blows of geological hammer; can be peeled with a pocket knife	1-5
Weak rock	Can be peeled by a pocket knife with difficulty; shallow indentations made by a firm blow with point of geological hammer	5-25
Medium strong	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with a single firm blow of geological hammer	25-50
Strong	Specimen requires more than one blow of geological hammer to fracture	50-100
Very strong	Specimen requires many blows of geological hammer to fracture	100-250
Extremely strong	Specimen can only be chipped by geological hammer	>250

Weathering:
Unweathered: no signs of discoloration or oxidation of rock material
Slightly Weathered: discontinuities are stained or discolored; rock material partially discolored
Moderately Weathered: total discoloration; generally surface of core is intact and not friable; discontinuities may contain filling of altered material
Highly Weathered: total discoloration; surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water; discontinuities frequently contain filling of altered material
Completely Weathered: total discoloration; appearance of core is that of soil although internally the rock texture is usually partly preserved; discontinuities frequently contain filling of altered material

APPENDIX B

Water Well Records

TOWNSHIP CONCESSION (LOT)	UTM[1]	DATE[2] CNTR[3]	CASING DIA[4]	WATER[5,6] DETAIL	STAT LVL/PUMP LVL[7] RATE[8]/TIME HR:MIN	WATER USE[9]	SCREEN INFO[10]	WELL # (AUDIT#) WELL TAG # DEPTHS TO WHICH FORMATIONS EXTEND[5,11]
NEWCASTLE TOWN (DARL BF (025)	17 681539 4860251[W]	1967/08 2615	36	FR 0005	007 / 002 / :0	ST		1901119 () BLCK LOAM 0001 BRWN CLAY 0005 BRWN CLAY GRVL 0023 BLUE CLAY 0025
NEWCASTLE TOWN (DARL BF (026)	17 680840 4860616[L]	1997/11 2662	06 06	FR 0104	020 / 145 / 0:0	CO		1913543 (180689) BRWN CLAY 0006 GREY CLAY STNS 0028 BRWN SILT SAND 0035 GREY SILT CLAY 0051 BRWN SILT SAND 0060 GREY CLAY 0100 BRWN SILT SAND 0103 BLCK SHLE 0104 BRWN LMSN 0145
NEWCASTLE TOWN (DARL BF (026)	17 680840 4860616[L]	1999/01 2662	06	FR 0053	031 / 039 005 / 1:0	DO		1914269 (198422) BRWN SAND GRVL CLAY 0041 BRWN SAND GRVL WBRG 0053 GREY LMSN 0055
NEWCASTLE TOWN (DARL BF (027)	17 680695 4860303[W]	1958/04 2615	36	FR 0035	034 / / :0	DO		1901121 () LOAM GRVL 0002 CLAY STNS 0035 MSND 0039
NEWCASTLE TOWN (DARL BF (027)	17 680575 4860323[W]	1957/08 2113	06	FR 0106	020 / 100 002 / 2:0	ST DO		1901120 () FILL 0001 GREY CLAY MSND 0060 BLUE CLAY 0106 LMSN 0107
NEWCASTLE TOWN (DARL BF (027)	17 680600 4860103[W]	2004/06 1413	36					1917172 (Z06784)
NEWCASTLE TOWN (DARL BF (027)	17 680489 4860083[W]	2004/07 1413	00		005 / / :0	NU		1917203 (Z06804) 0015
NEWCASTLE TOWN (DARL BF (027)	17 680405 4860037[W]	2004/07 1413	00		004 / / :0	NU		1917202 (Z06805) 0020
NEWCASTLE TOWN (DARL BF (027)	17 680544 4860078[W]	2004/06 1413	36 06					1917171 (Z06785)
NEWCASTLE TOWN (DARL BF (027)	17 680446 4860521[L]	2004/04 6874	30	FR 0033	031 / / :0	DO		1917081 (Z02874) UNKN 0042
NEWCASTLE TOWN (DARL BF (027)	17 680449 4860521[L]	1997/11 1413	11 06	FR 0037	002 / 029 003 / 4:0	IN	0030 07	1913507 (178140) BRWN CLAY HARD 0020 BRWN SAND GRVL LOOS 0037
NEWCASTLE TOWN (DARL BF (027)	17 680449 4860521[L]	1997/03 1413	06	FR 0036	002 / 027 006 / 1:0	IN	0033 03	1913174 (178049) BRWN CLAY HARD 0010 GREY CLAY STNS HARD 0032 GREY GRVL CGRD CLN 0036
NEWCASTLE TOWN (DARL BF (027)	17 680449 4860521[L]	1997/03 1413	06			IN		1913173 (178048) BRWN CLAY HARD 0010 GREY CLAY GRVL LOOS 0035 GREY CLAY STNS HARD 0060 GREY CLAY DNSE 0075
NEWCASTLE TOWN (DARL BF (027)	17 680449 4860521[L]	1997/03 1413	06			IN		1913172 (178045) BRWN CLAY DNSE 0007 BRWN SILT SAND LOOS 0030 BRWN GRVL SILT CMTD 0037 GREY CLAY STNS LYRD 0070 GREY SILT STNS SOFT 0071 GREY SAND SILT SOFT 0073 GREY CLAY DNSE 0075
NEWCASTLE TOWN (DARL BF (027)	17 680449 4860521[L]	1997/02 1413	06 06			IN		1913170 (178043) BRWN CLAY SAND SOFT 0016 GREY CLAY STNS HPAN 0055 GREY CLAY DNSE 0108 BLCK SHLE HARD 0117

Well Computer Print Out Data as of November 27 2008

TOWNSHIP CONCESSION (LOT)	UTM[1]	DATE [2] CNTR [3]	CASING DIA [4]	WATER[5,6] DETAIL	STAT LVL/PUMP LVL[7] RATE[8]/TIME HR:MIN	WATER USE[9]	SCREEN INFO[10]	WELL # (AUDIT#) WELL TAG # DEPTHS TO WHICH FORMATIONS EXTEND[5,11]
NEWCASTLE TOWN (DARL BF (027)	17 680475 4860903[W]	1978/01 2214	30	FR 0031	020 / 025 006 / 0:30	CO		1905036 () BRWN CLAY STNS PCKD 0015 BLUE CLAY STNS CMTD 0031 GRVL WBRG LOOS 0033
NEWCASTLE TOWN (DARL BF (027)	17 680115 4861223[W]	1978/06 2104	06	FR 0043	012 / 030 015 / 5:30	DO	0043 04	1905057 () BRWN CLAY STNS 0025 GREY CLAY GRVL 0043 BRWN MSND GRVL 0046 GREY CLAY 0047
NEWCASTLE TOWN (DARL BF (027)	17 680055 4861203[W]	1978/10 2104	06	FR 0035	015 / 025 020 / 4:30	IN	0034 04	1905173 () BRWN CLAY STNS 0015 GREY CLAY GRVL HARD 0035 BRWN SAND GRVL LOOS 0037 GREY CLAY 0043
NEWCASTLE TOWN (DARL BF (027)	17 680195 4861203[W]	1982/04 1572	06	UK 0170	040 / 100 010 / 2:0	IN		1906356 () BRWN LOAM SOFT 0002 GREY CLAY BLDR HARD 0074 GREY CLAY HARD VERY 0110 GREY STNS HARD VERY 0180
NEWCASTLE TOWN (DARL BF (027)	17 680195 4861203[W]	1983/08 2214	30	FR 0020	015 / 018 007 / 0:30	IN		1906827 () BRWN CLAY PCKD 0010 BLUE CLAY STNS PCKD 0020 BLUE CLAY SAND LYRD 0031
NEWCASTLE TOWN (DARL BF (027)	17 680385 4861271[W]	1986/09 2104	06	UK 0109 UK 0107	035 / 090 008 / 4:0	CO		1907908 () BRWN CLAY STNS MGRD 0020 GREY CLAY GRVL MGRD 0107 BLCK SHLE LYRD 0109
NEWCASTLE TOWN (DARL BF (027)	17 680380 4861273[W]	1986/09 2104	06			NU		1907909 () GREY CLAY GRVL MGRD 0110 BLCK SHLE MGRD 0115 BRWN LMSN LYRD HARD 0143
NEWCASTLE TOWN (DARL BF (027)	17 680449 4860521[L]	1988/09 3129	30	FR 0032	015 / 023 008 / 1:0	DO		1909292 () LOAM 0001 BRWN CLAY 0026 BLUE CLAY 0043 GREY SAND GRVL 0045
NEWCASTLE TOWN (DARL BF (027)	17 680278 4861213[W]	1989/01 2104	06	UK 0110	040 / 119 001 / 21:0	CO		1909578 () BRWN CLAY MGRD 0030 GREY CLAY MGRD 0108 BLCK LMSN MGRD 0120 BLCK LMSN MGRD 0135
NEWCASTLE TOWN (DARL BF (027)	17 680033 4861102[W]	1989/01 2104	06	UK 0024	005 / 025 003 / 2:0	CO		1909579 () BRWN CLAY MGRD 0008 BRWN CLAY GRVL MGRD 0020 GREY CLAY GRVL MGRD 0024 BLCK SAND GRVL WBRG 0027
NEWCASTLE TOWN (DARL BF (027)	17 680449 4860521[L]	1988/11 2214	30	FR 0020 FR 0028	020 / 022 004 / 1:0	DO		1909658 () BLCK LOAM 0001 BRWN CLAY BLDR CMTD 0015 GREY CLAY PCKD 0020 BLUE CLAY SNDY WBRG 0025 BLUE CLAY STNS CMTD 0028 BRWN SAND GRVL WBRG 0035 BRWN SHST WBRG 0037
NEWCASTLE TOWN (DARL BF (027)	17 680267 4861212[W]	1989/05 1413				CO		1909887 () BRWN CLAY STNS SNDY 0011 BRWN CLAY GRVL LOOS 0019 GREY CLAY STNS SNDY 0026 GREY CLAY STNS PCKD 0035 BRWN SAND GRVL PCKD 0046 GREY CLAY PCKD 0109 BLCK SHLE HARD 0109
NEWCASTLE TOWN (DARL BF (027)	17 680285 4861204[W]	1989/05 1413				CO		1909889 () BRWN CLAY SAND LOOS 0005 BRWN CLAY GRVL LOOS 0019 BRWN SAND GRVL LOOS 0031 GREY CLAY STNS SNDY 0045 GREY CLAY STNS PCKD 0060

TOWNSHIP CONCESSION (LOT)	UTM[1]	DATE [2] CNTR [3]	CASING DIA [4]	WATER[5,6] DETAIL	STAT LVL/PUMP LVL[7] RATE[8]/TIME HR:MIN	WATER USE[9]	SCREEN INFO[10]	WELL # (AUDIT#) WELL TAG # DEPTHS TO WHICH FORMATIONS EXTEND[5,11]
NEWCASTLE TOWN (DARL BF (027)	17 680340 4861202[W]	1989/07 2214	18	FR 0026	005 / 028 014 / 1:40	CO		1910026 () BRWN CLAY STNS PCKD 0005 BRWN CLAY STNS CMTD 0015 GREY CLAY PCKD HARD 0026 GREY SAND WBRG PCKD 0034 GREY CLAY STNS CMTD 0036
NEWCASTLE TOWN (DARL BF (027)	17 680316 4861175[W]	1989/07 2214	30	FR 0039	006 / 029 014 / 1:10	CO		1910027 () SAND GRVL PCKD 0010 SAND GRVL CMTD 0018 GREY SAND SILT HARD 0023 GREY SILT SNDY PCKD 0028 GREY CLAY SILT PCKD 0042
NEWCASTLE TOWN (DARL BF (027)	17 680326 4861212[W]	1988/06 2214	18	FR 0022	007 / 027 / :0	DO CO		1910028 () CLAY STNS PCKD 0005 BRWN CLAY STNS CMTD 0015 GREY CLAY STNS PCKD 0022 GREY STNS WBRG PCKD 0034 BRWN CLAY STNS CMTD 0036
NEWCASTLE TOWN (DARL BF (027)	17 680336 4861196[W]	1989/07 2214	30	FR 0030 FR 0018	011 / 028 014 / :45	CO		1910029 () BRWN CLAY PCKD 0008 BRWN CLAY SNDS CMTD 0022 BRWN CLAY SNDS LYRD 0030 BRWN SAND WBRG 0035
NEWCASTLE TOWN (DARL BF (027)	17 680449 4860521[L]	1991/08 2214	24 18	FR 0028 FR 0043	015 / 036 005 / 1:0	DO		1911249 () BLCK LOAM 0002 BRWN CLAY STNS 0020 GREY CLAY STNS CMTD 0028 BLCK SAND WBRG 0029 GREY CLAY STNS CMTD 0042 BLCK SAND GRVL WBRG 0045
NEWCASTLE TOWN (DARL BF (027)	17 680492 4860935[W]	1993/08 3129	06 06	FR 0022 GS 0108	008 / 071 003 / 4:0	CO	0108 06	1911762 () BRWN LOAM 0005 GREY CLAY STNS 0022 GREY CLAY SNDS WBRG 0034 GREY CLAY STNS HARD 0068 GREY CLAY STNS SOFT 0106 GREY GRVL SNDY WBRG 0108 GREY STNS CLAY SNDY 0113 SHLE ROCK 0113
NEWCASTLE TOWN (DARL BF (027)	17 680449 4860521[L]	1997/02 1413	06			IN		1913169 (178042) BRWN CLAY DNSE HARD 0014 GREY CLAY SAND HARD 0057 GREY CLAY SAND LYRD 0062 GREY CLAY DNSE 0075
NEWCASTLE TOWN (DARL BF (028)	17 679775 4860683[W]	1963/05 2801	07	FR 0032	012 / 019 014 / 8:0	DO	0032 04	1901122 () LOAM 0001 CLAY GRVL 0014 GRVL CLAY 0025 CLAY SILT GRVL 0032 MSND GRVL CLAY 0044 CLAY GRVL 0053 GRVL SHLE CLAY 0059 CLAY GRVL 0081 BLUE CLAY GRVL 0092 SHLE 0096
NEWCASTLE TOWN (DARL BF (028)	17 680029 4860455[L]	1986/07 4814	06	FR 0100	005 / 094 004 / 10:0	CO		1908319 () BRWN CLAY STNS 0014 GREY CLAY STNS 0038 GRVL SNDY 0046 GREY CLAY STNS SNDY 0060 GREY CLAY STNS 0068 GREY SHLE CLAY GRVL 0096 GREY CGVL CLYY 0100 CGVL FGRD 0104
NEWCASTLE TOWN (DARL BF (029)	17 680055 4860693[W]	1955/09 5454	04	FR 0038	020 / 032 002 / 4:0	DO		1901123 () LOAM MSND 0001 CLAY 0011 BLUE CLAY 0022 BLUE CLAY GRVL 0038 CSND GRVL 0040
NEWCASTLE TOWN (DARL BF (029)	17 679615 4860673[W]	1963/05 2801				NU		1901125 () LOAM 0001 BRWN CLAY BLDR GRVL 0006 GREY CLAY GRVL 0030 CLAY GRVL 0037 GRVL CLAY 0038 CLAY GRVL 0059 SHLE 0067

TOWNSHIP CONCESSION (LOT)	UTM[1]	DATE[2] CNTR[3]	CASING DIA[4]	WATER[5,6] DETAIL	STAT LVL/PUMP LVL[7] RATE[8]/TIME HR:MIN	WATER USE[9]	SCREEN INFO[10]	WELL # (AUDIT#) WELL TAG # DEPTHS TO WHICH FORMATIONS EXTEND[5,11]
NEWCASTLE TOWN (DARL BF (029)	17 679595 4860963W	1956/10 4107	06	FR 0049	015 / 036 008 / 2:0	DO		1901124 () PRDG 0034 BLUE CLAY 0038 MSND 0046 GRVL 0049
NEWCASTLE TOWN (DARL CON 01(026)	17 679918 4861189W	1989/07 1413	06	FR 0052	014 / 004 / 5:30	DO	0049 03	1910033 () BRWN CLAY STNS SNDY 0018 BRWN GRVL CLAY CGRD 0020 GREY CLAY STNS SOFT 0028 BRWN GRVL CLAY CGRD 0034 GREY CLAY STNS SNDY 0040 BRWN GRVL SAND CLN 0052
NEWCASTLE TOWN (DARL CON 01(028)	17 680257 4861218W	1994/03 3136	06 06	FR 0109	032 / 150 003 / 1:0	IN		1911949 (24160) BRWN LOAM 0002 GREY CLAY SAND STNS 0025 GREY CLAY SAND SLTY 0109 BLCK ROCK LYRD 0157
NEWCASTLE TOWN (DARL CON 01(028)	17 679692 4861182W	1957/11 2113	06 06	FR 0039	036 / 049 005 / 6:0	DO ST	0039 10	1901183 () PRDG 0038 GRVL MSND 0048 GREY CLAY MSND STNS 0071 GREY CLAY STNS 0102 GREY CLAY 0112
NEWCASTLE TOWN (DARL 01(006)	17 680423 4860262W	2007/10 4102			/ / :0			7053112 (Z67907)
NEWCASTLE TOWN (DARL ()	17 680115 4861148W	2008/03 3030	36	FR 0022 0030 0045	017 / 004 / 1:0			7103759 (Z68288) A072506 BRWN CLAY FILL 0003 BRWN CLAY 0022 GREY SILT STNS HARD 0030 GREY SILT SAND LYRD 0050
BROCK TOWNSHIP (THOR CON 07(010)	17 680405 4860036W	2004/08 1413	36		010 / / :0			1917263 (Z06806)
BOWMANVILLE TOWN ()	17 681410 4860966W	2008/03 6809						7104714 (M01171) A071803
WHITBY TOWN 01(006)	17 680505 4860297W	2007/10 4102			/ / :0			7053102 (Z67906)
MANVERS TOWNSHIP CON 08(024)	17 680405 4860036W	2004/07 1413	06	FR 0062	010 / 049 016 / 1:0	DO	0060 04	6417757 (Z06802) A006672 BRWN SAND 0040 BRWN SAND 0064

Notes:

1. UTM in Zone, Easting, Northing and Datum is NAD83; L: UTM estimated from Centroid of Lot; W: UTM not from Lot Centroid
2. Date Work Completed
3. Well Contractor Licence Number
4. Casing diameter in inches
5. Unit of Depth in Feet
6. See Table 4 for Meaning of Code
7. STAT LVL: Static Water Level in Feet ; PUMP LVL: Water Level After Pumping in Feet
8. Pump Test Rate in GPM, Pump Test Duration in Hour : Minutes
9. See Table 3 for Meaning of Code
10. Screen Depth and Length in feet
11. See Table 1 and 2 for Meaning of Code

1. Core Material and Descriptive terms														
Code	Description	...	Code	Description	...	Code	Description	...	Code	Description	...	Code	Description
BLDR	BOULDERS		FCRD	FRACTURED		IRFM	IRON FORMATION		PORS	POROUS		SOFT	SOFT
BSLT	BASALT		FGRD	FINE-GRAINED		LIMY	LIMY		PRDG	PREVIOUSLY DUG		SPST	SOAPSTONE
CGRD	COARSE-GRAINED		FGVL	FINE GRAVEL		LMSN	LIMESTONE		PRDR	PREV. DRILLED		STKY	STICKY
CGVL	COARSE GRAVEL		FILL	FILL		LOAM	TOPSOIL		QRTZ	QUARTZITE		STNS	STONES
CHRT	CHERT		FLDS	FELDSPAR		LOOS	LOOSE		QSND	QUICKSAND		STNY	STONEY
CLAY	CLAY		FLNT	FLINT		LTCL	LIGHT-COLOURED		QTZ	QUARTZ		THIK	THICK
CLN	CLEAN		FOSS	FOSILIFEROUS		LYRD	LAYERED		ROCK	ROCK		THIN	THIN
CLYY	CLAYEY		FSND	FINE SAND		MARL	MARL		SAND	SAND		TILL	TILL
CMTD	CEMENTED		GNIS	GNEISS		MGRD	MEDIUM-GRAINED		SHLE	SHALE		UNKN	UNKNOWN TYPE
CONG	CONGLOMERATE		GRNT	GRANITE		MGVL	MEDIUM GRAVEL		SHLY	SHALY		VERY	VERY
CRYS	CRYSTALLINE		GRSN	GREENSTONE		MRBL	MARBLE		SHRP	SHARP		WBRG	WATER-BEARING
CSND	COARSE SAND		GRVL	GRAVEL		MSND	MEDIUM SAND		SHST	SCHIST		WDFR	WOOD FRAGMENTS
DKCL	DARK-COLOURED		GRWK	GREYWACKE		MUCK	MUCK		SILT	SILT		WTHD	WEATHERED
DLMT	DOLOMITE		GVLY	GRAVELLY		OBDN	OVERBURDEN		SLTE	SLATE			
DNSE	DENSE		GYPS	GYPSUM		PCKD	PACKED		SLTY	SILTY			
DRTY	DIRTY		HARD	HARD		PEAT	PEAT		SNDS	SANDSTONE			
DRY	DRY		HPAN	HARDPAN		PGVL	PEA GRAVEL		SNDY	SANDY			

2. Core Color	
Code	Description
WHIT	WHITE
GREY	GREY
BLUE	BLUE
GREN	GREEN
YLLW	YELLOW
BRWN	BROWN
RED	RED
BLCK	BLACK
BLGY	BLUE-GREY

3. Water Use			
Code	Description	Code	Description
DO	Domestic	OT	Other
ST	Livestock	TH	Test Hole
IR	Irrigation	DE	Dewatering
IN	Industrial	MO	Monitoring
CO	Commercial		
MN	Municipal		
PS	Public		
AC	Cooling And A/C		
NU	Not Used		

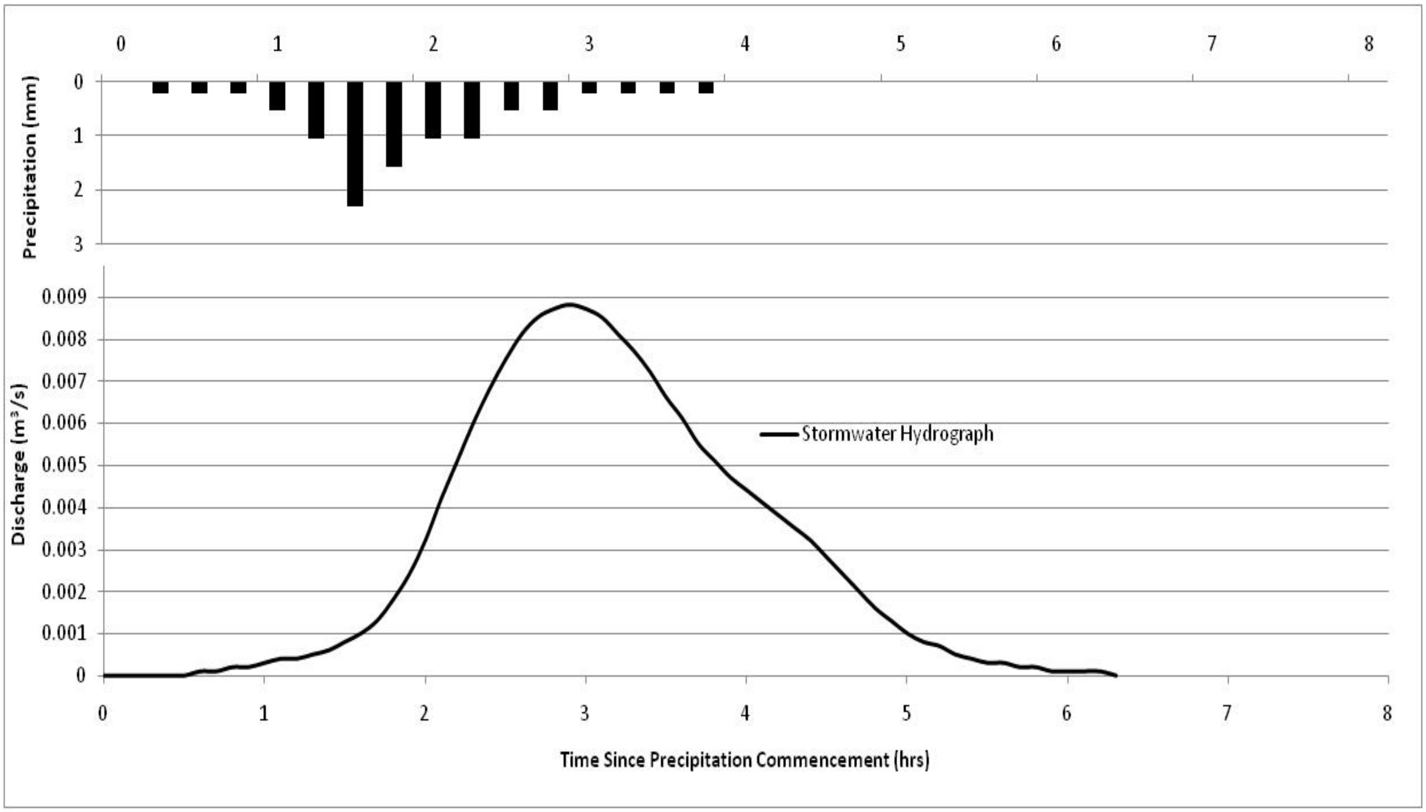
4. Water Detail			
Code	Description	Code	Description
FR	Fresh	GS	Gas
SA	Salty	IR	Iron
SU	Sulphur		
MN	Mineral		
UK	Unknown		

APPENDIX C

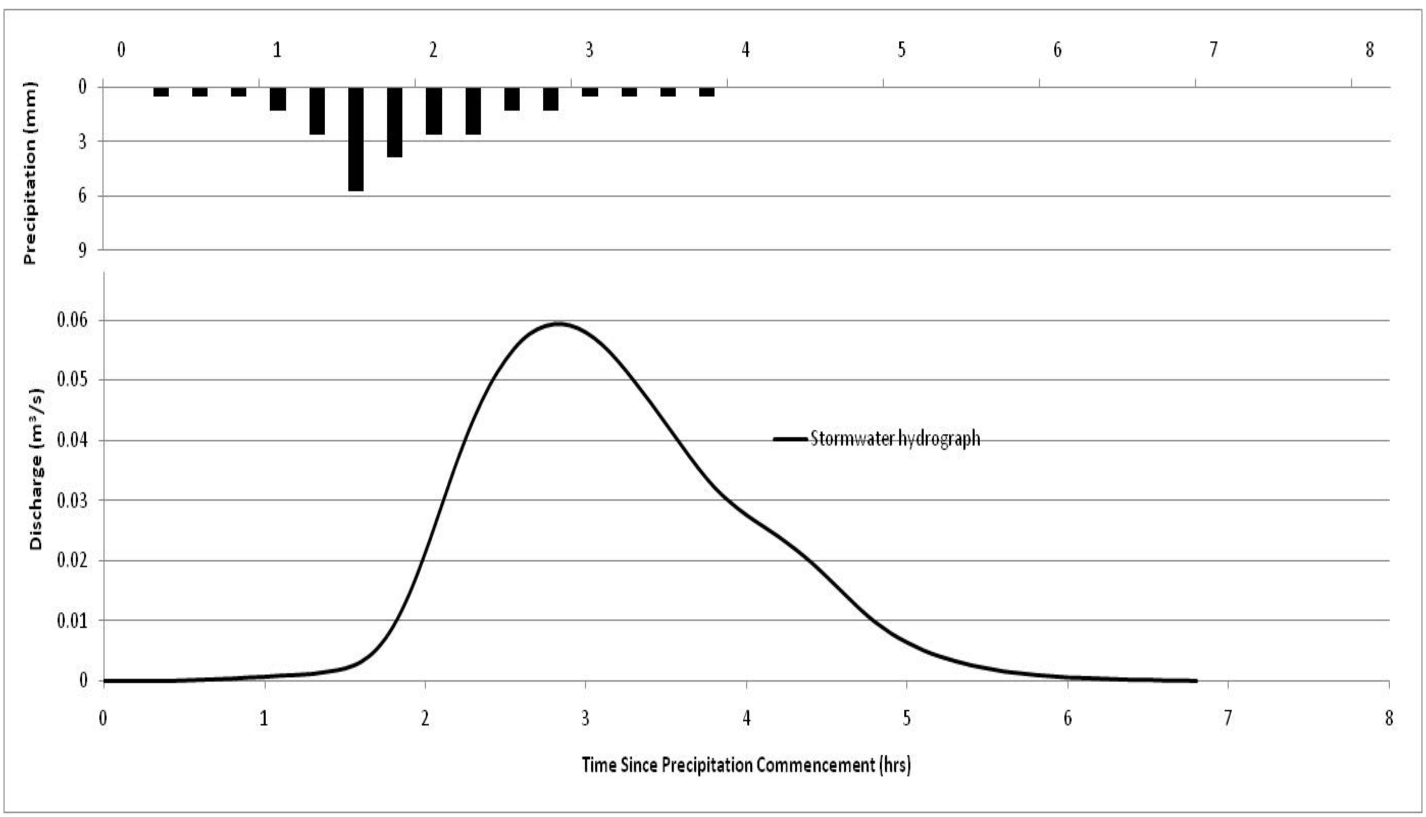
Hydrological Modelling Data

Appendix C

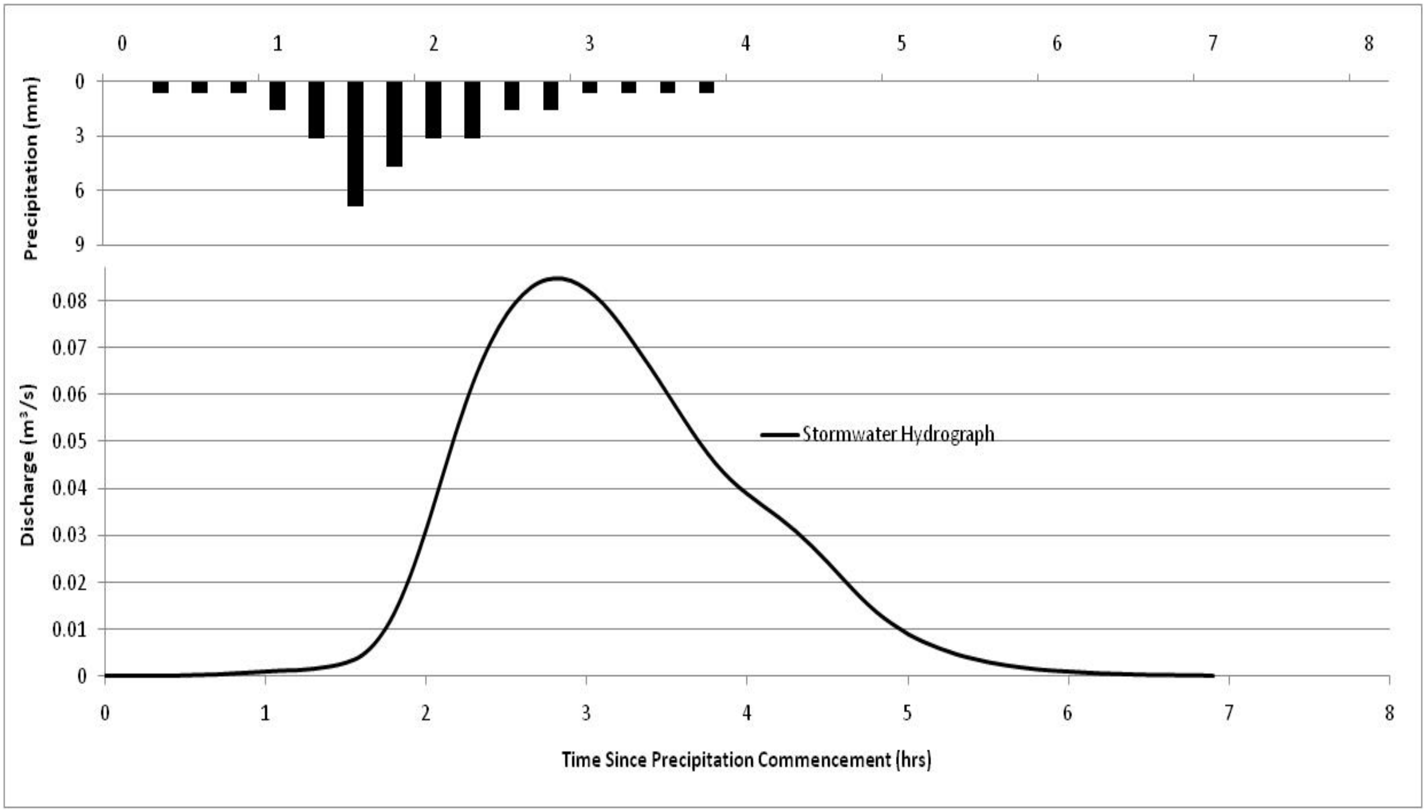
10mm event



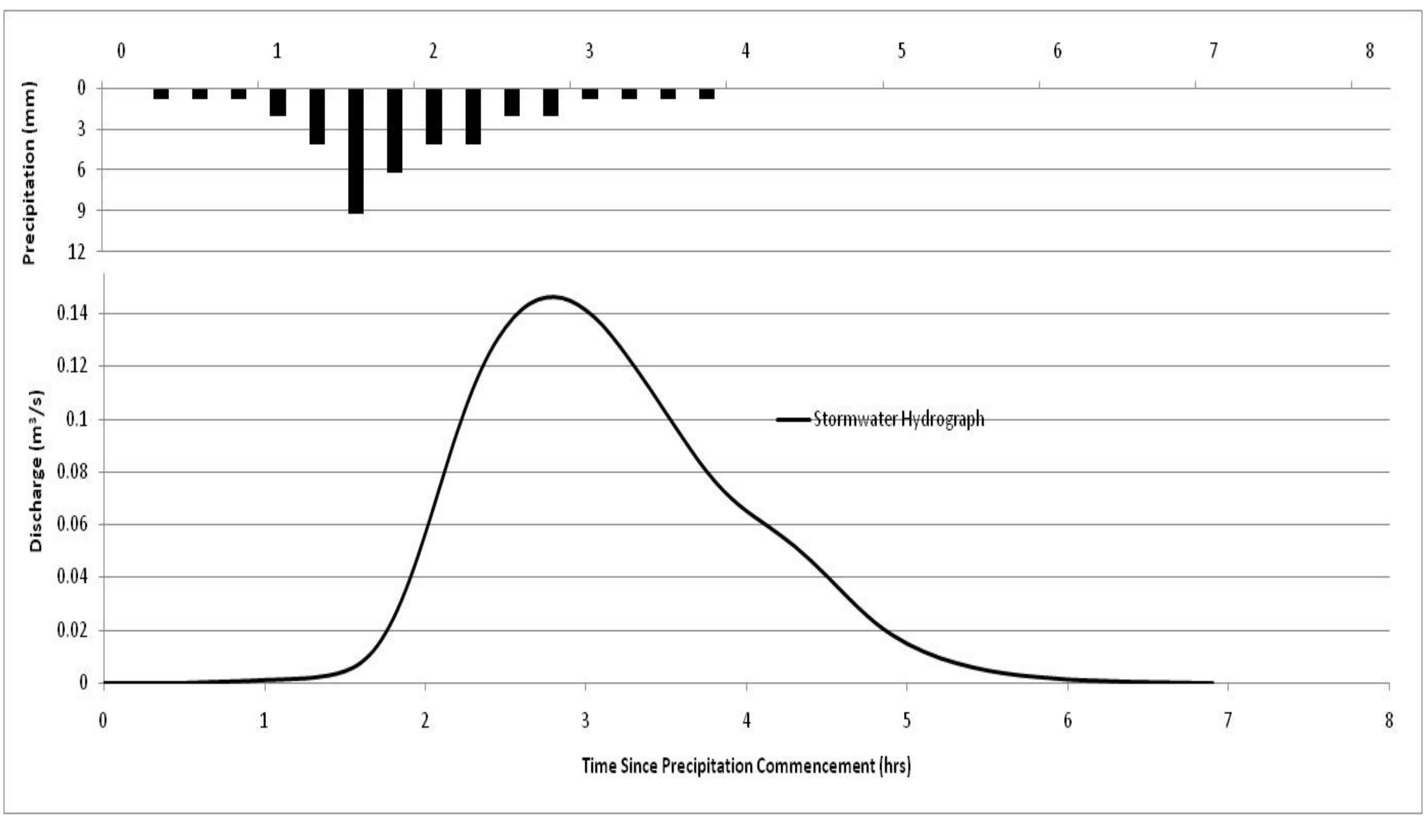
25mm event



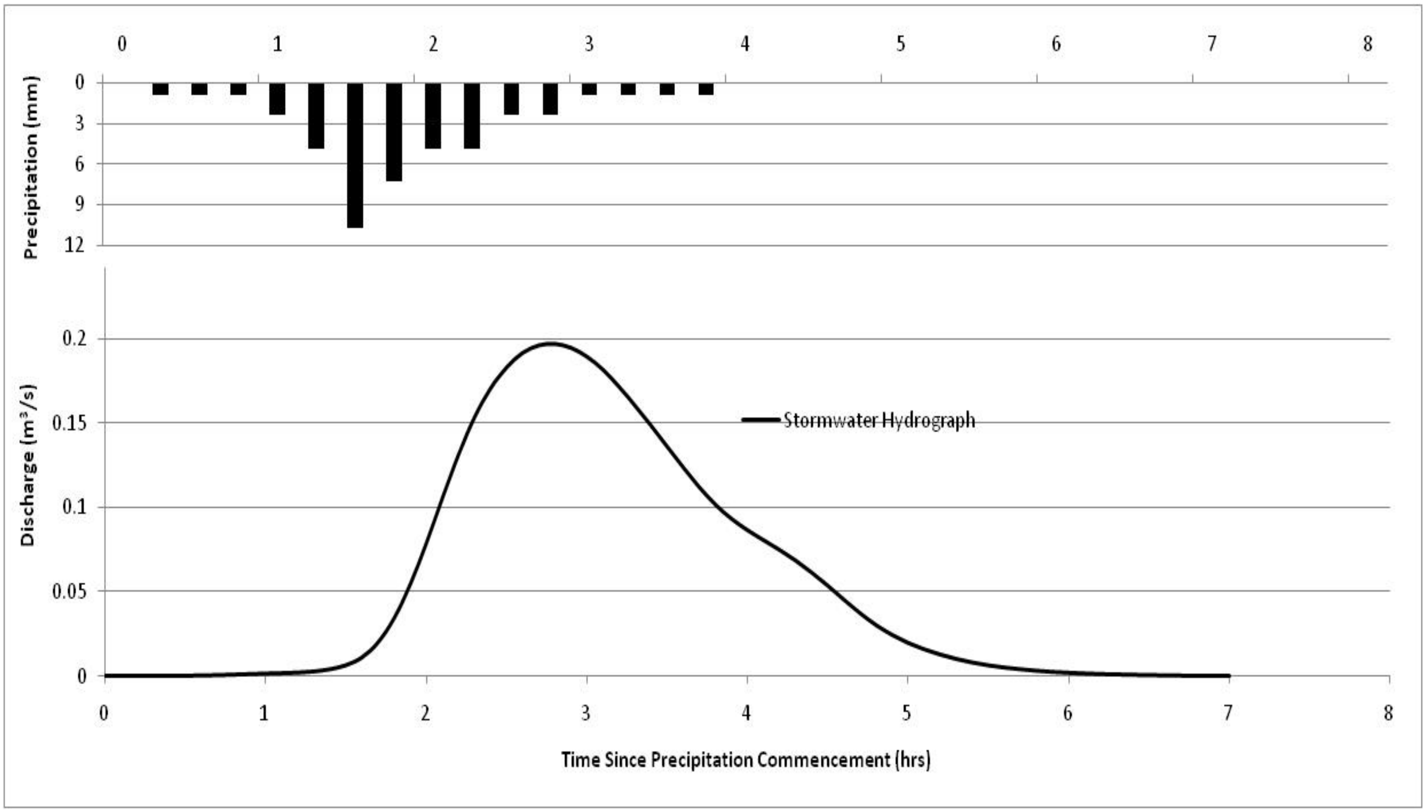
2-year event



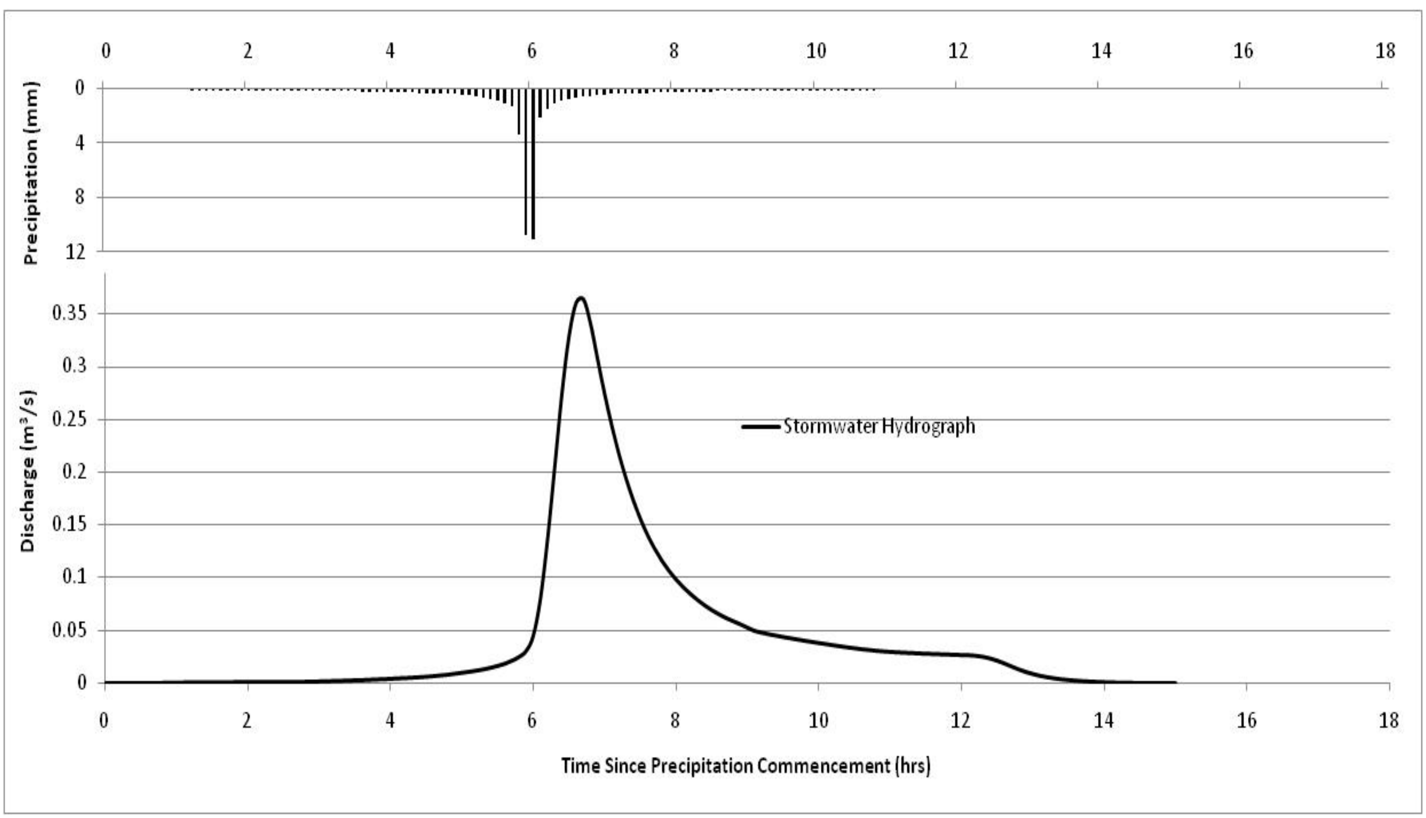
5-year event



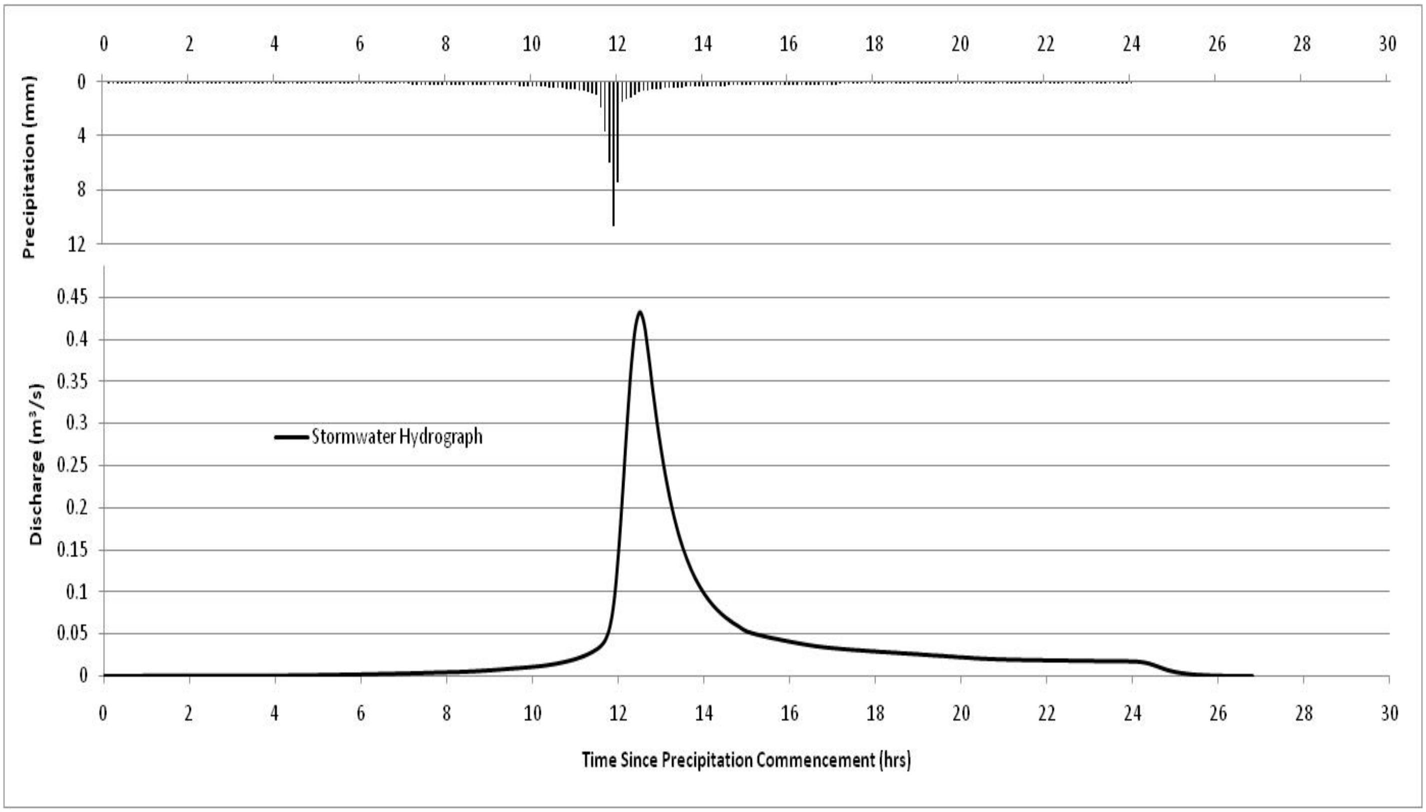
10-year event



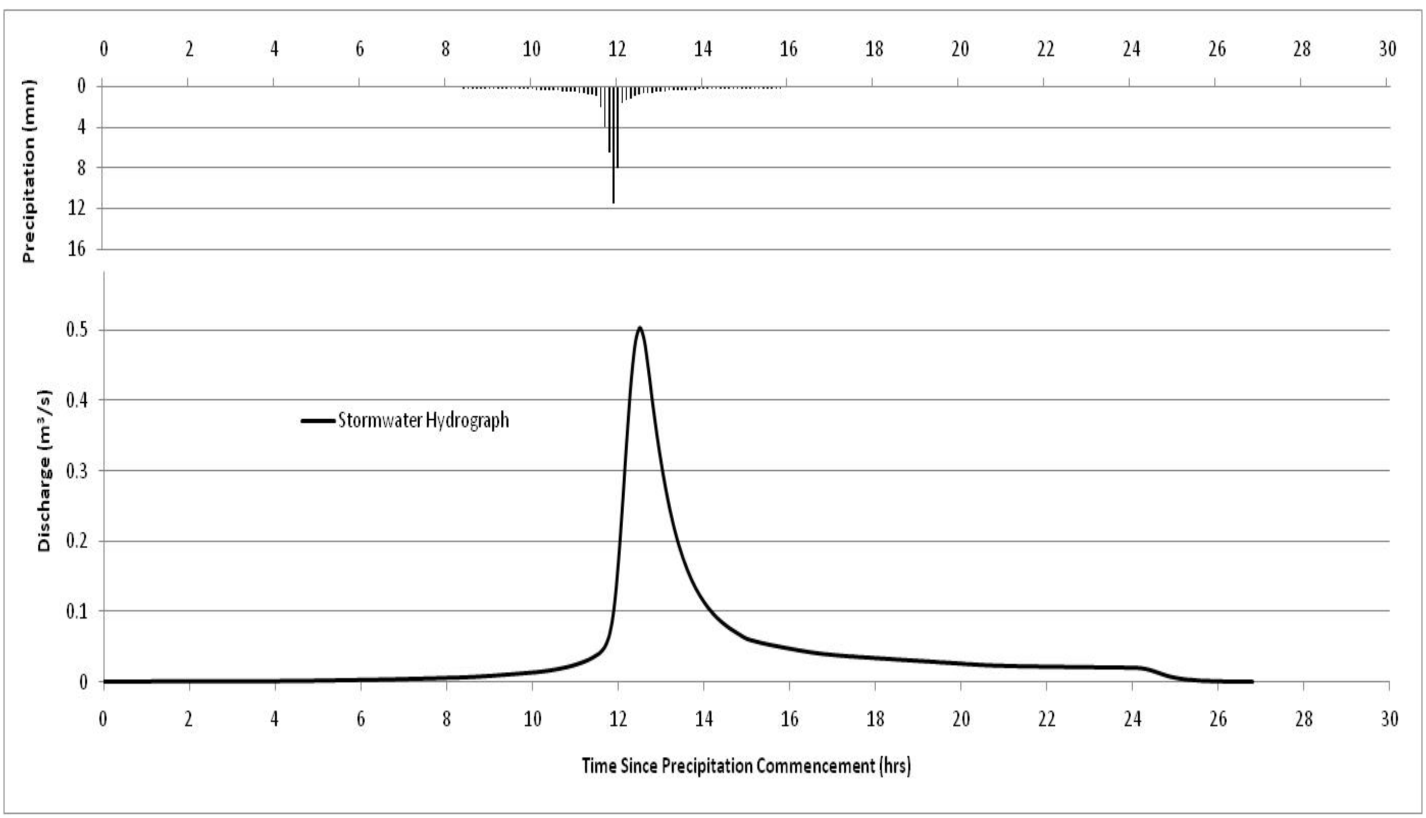
25-year event



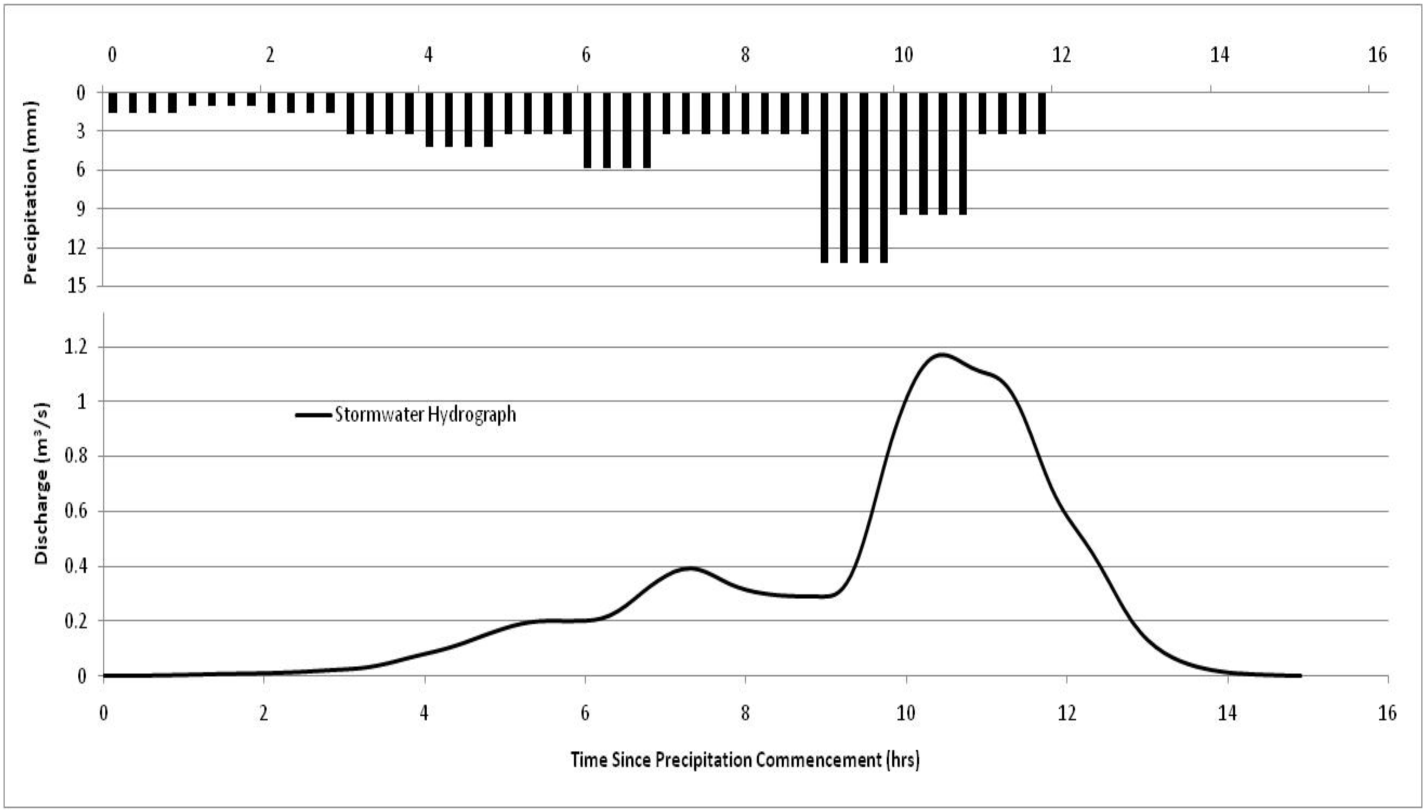
50-year event



100-year event



Hurricane Hazel



1-hr AES Distribution Stormwater Modelling Results

Watershed	Parameter	2yr/ 1hr	5yr/ 1hr	10yr/ 1hr	25yr/ 1hr	50yr/ 1hr	100yr/ 1hr
Existing Conditions	Peak Discharge (m^3/s)	0.07	0.14	0.19	0.27	0.33	0.39
	Runoff Volume (m^3)	275	497	674	931	1140	1365
Post-Development (140,000 tpy)	Peak Discharge (m^3/s)	0.43	0.63	0.76	0.94	1.08	1.22
	Runoff Volume (m^3)	1404	2026	2462	3041	3479	3926
Post-Development (400,000 tpy)	Peak Discharge (m^3/s)	0.53	0.76	0.92	1.12	1.27	1.43
	Runoff Volume (m^3)	1660	2364	2849	3487	3964	4449

24hr SCS Type II Distribution Stormwater Modelling Results

Watershed	Parameter	2yr/ 24hr	5yr/ 24hr	10yr/ 24hr	25yr/ 24hr	50yr/ 24hr	100yr/ 24hr
Existing Conditions	Peak Discharge (m^3/s)	0.14	0.22	0.28	0.37	0.43	0.50
	Runoff Volume (m^3)	1101	1712	2174	2800	3295	3822
Post-Development (140,000 tpy)	Peak Discharge (m^3/s)	0.47	0.64	0.76	0.91	1.02	1.14
	Runoff Volume (m^3)	3398	4508	5399	6444	7231	8038
Post-Development (400,000 tpy)	Peak Discharge (m^3/s)	0.55	0.74	0.87	1.03	1.15	1.28
	Runoff Volume (m^3)	3877	5153	6027	7132	7959	8803

APPENDIX D

400,000 tpy Scenario Upgrade Requirements

1.0 INTRODUCTION

The Proposed Thermal Treatment Facility (the Facility) in the Clarington Energy Business Park has an initial design capacity of 140,000 tonnes per year (tpy). If approved, construction of this Facility is scheduled to occur between 2010 and 2013. It is anticipated that waste production may continue to increase in the future and that this Facility may be expanded to a projected maximum design capacity of 400,000 tpy.

This technical appendix has been prepared to address the likely effects of the increased capacity Facility on water resources. The optional expansion/upgrade would affect the water supply requirements, wastewater discharge volumes and stormwater management features located both on- and offsite. As these upgrades may be constrained by future area restrictions and may include sub-surface activities, it may be prudent to consider integrating some of the recommended measures or incorporating contingency planning into the initial design capacity of 140,000 tpy (140,000 tpy scenario) Facility design.

2.0 STORMWATER MANAGEMENT

In order for the increased capacity Facility to accommodate a projected maximum design capacity of 400,000 tpy (400,000 tpy scenario), access roads, refuse storage areas and the number of waste processing buildings would need to be increased from the current 140,000 tpy scenario (See Figure 2-1 for the 400,000 tpy scenario Facility Site plan and Figure 4-1 in the Main Report for the 140,000 tpy scenario Facility Site plan). The addition of these Facility upgrades would lead to an increase in impervious area which in turn would necessitate an upgrade to the stormwater management features onsite. This section provides the results from the 400,000 tpy scenario stormwater runoff modeling and subsequently makes stormwater management recommendations necessary to mitigate the effects of upgrading the 140,000 tpy scenario Facility.

2.1 Post-Development Stormwater Runoff Modelling

A 400,000 tpy scenario hydrologic model was developed using similar methodologies as outlined in Section 3.4.3 of the Report. The 400,000 tpy scenario conditions were simulated by increasing the SCS curve number and percent imperviousness from the 140,000 tpy scenario. These changes subsequently influence the average Time of Concentration, storage coefficient and ultimately runoff coefficients. The same suite of storm events used for the pre-development and 140,000 tpy scenarios were used in this model. In addition, as part of the requirements of CLOCA (2007), the 1-hr AES and 24-hr SCS Type II storm distributions were modelled. Results for these addition storms are included in **Appendix C** of the Report.

The increased capacity Facility would include the paved access roads, an aggregate top dressing over storage areas and the introduction of additional permanent building structures. It is assumed that the development would best be represented by a SCS curve number of 86 for a hydrological soil group B with an industrial setting introducing impervious conditions over 55% of a 12.4 ha developed area. The percent impervious was determined through assessment of the 400,000 tpy scenario Facility Site plan (Figure 2-1) and represents an increase in site imperviousness of 10% from the 45% imperviousness of the 140,000 tpy scenario.

According to the preferred vendor's increased capacity Site drawing (Figure 2-1), the entire 12.4 ha would contain some type of development and therefore would require stormwater management. It is assumed that the slope would remain similar to that of existing conditions. A similar slope incorporating increased imperviousness would cause a decrease in Time of Concentration values used in the hydrological model. However, the storage coefficient would not be affected by above grade activities.

A summary of increased capacity post-development hydrologic input parameters are included in Table 2-1.

Table 2-1 - Post-Development Model Input Parameters

Parameter	Post-Development
Area (ha)	12.4
Hydraulic Length (m)	390
Average Slope %	1.9
Hydrologic Soil Group	B
Time of Concentration (hr)	0.38
Storage Coefficient (hr)	0.49
% Imperviousness	55
SCS Curve #	86
Initial Abstraction (mm)	3

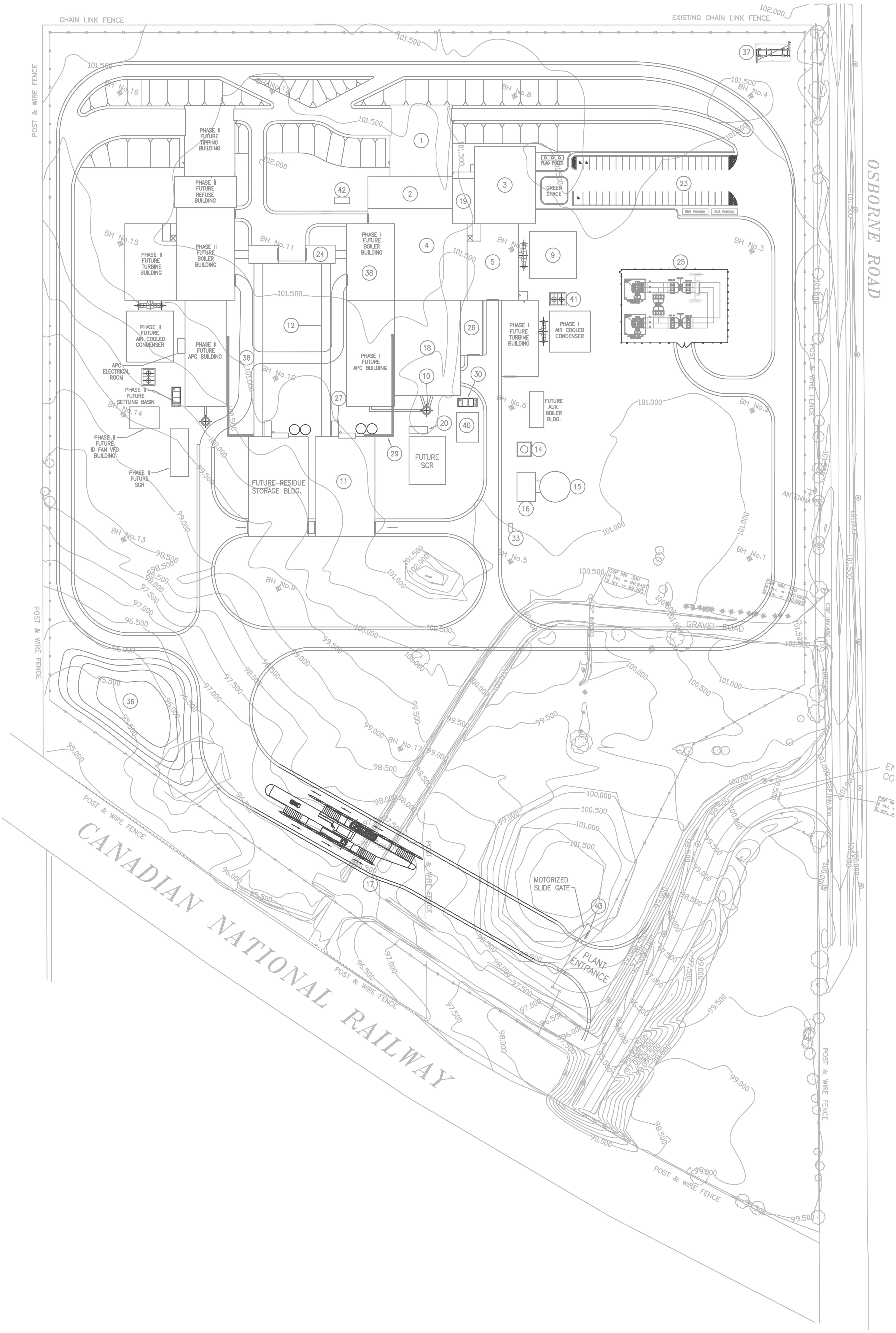
As a result of Facility upgrades, runoff volumes onsite would increase, which would also cause an increase in peak discharges when compared to the 140,000 tpy scenario. Table 2-2 provides a summary of the 400,000 tpy scenario post-development runoff volumes and peak discharges for all of the design events. Subsequent stormwater mitigation options would be designed to reduce these peak discharges to pre-development levels and attenuate flows.

Table 2-2 – 400,000 tpy Scenario Post-Development Runoff Model Results

Watershed	Parameter	10mm /4hr	25mm/ 4hr	2yr/4 hr	5yr/ 4hr	10yr/ 4hr	25yr/1 2hr	50yr/2 4hr	100yr/2 4hr	Hazel
400,000 tpy Scenario Post-Development	Peak Discharge (m^3/s)	0.11	0.31	0.39	0.54	0.66	1.18	1.15	1.28	1.56
	Runoff Volume (m^3)	739	2131	2641	3703	4471	6355	7960	8803	24047

The results presented in Table 2-2 show an increase in runoff volume which infers a decrease in ET and infiltration volumes. When compared to the 140,000 tpy post-development runoff modeling, these results suggest an increase of 128 m^3 for the 10 mm event and a 311 m^3 increase for the 25 mm event.

The objective of the following sections is to indicate the stormwater management features that will require upgrades in order to transition from the 140,000 to 400,000 tpy scenarios.


CHAIN LINK FENCE
EXISTING CHAIN LINK FENCE
POST & WIRE FENCE
OSBORNE ROAD
CANADIAN NATIONAL RAILWAY
PHASE II FUTURE TIPPING BUILDING
PHASE II FUTURE REFUSE BUILDING
PHASE II FUTURE BOILER BUILDING
PHASE II FUTURE TURBINE BUILDING
PHASE II FUTURE AIR COOLED CONDENSER
PHASE II FUTURE APC BUILDING
APC ELECTRICAL ROOM
PHASE II FUTURE SETTLING BASIN
PHASE II FUTURE ID FAN VFD BUILDING
PHASE II FUTURE SCR
PHASE I FUTURE BOILER BUILDING
PHASE I FUTURE APC BUILDING
PHASE I FUTURE TURBINE BUILDING
PHASE I AIR COOLED CONDENSER
FUTURE AUX. BOILER BLDG.
FUTURE SCR
FUTURE RESIDUE STORAGE BLDG.
FLAG POLES
GREEN SPACE
BUS PARKING
GRAVEL ROAD
ANTENNA
MOTORIZED SLIDE GATE
PLANT ENTRANCE

2.2 Construction Phase Stormwater Management

Since construction of the 400,000 tpy scenario upgrades would take place on previously developed lands it is not anticipated that site grubbing or subsurface disruption would be as extensive as the initial 140,000 tpy scenario construction phase.

Construction of the requisite buildings and access roads would occur mainly on the west side of the property. It is anticipated that the western side of the property required for infrastructure upgrades, presumably greenspace during 140,000 tpy scenario Facility operation, would be sequentially cleared, graded and developed in a similar fashion to the 140,000 tpy scenario construction. The main southern access route, central building complex and northern perimeter roads would remain relatively untouched during 400,000 tpy scenario Facility upgrades.

The following is a list of lot level mitigation measures recommended for the construction of 400,000 tpy scenario Facility upgrades.

- All cleared areas not required for equipment storage, building construction or vehicle access should be seeded as soon as possible to avoid excess soil loss;
- Sediment traps should be installed within flow paths, slope toes and surrounding drains to minimize the amount of sediment deposited in conveyance networks and detention ponds;
- Silt fencing should be installed around the perimeter of all laydown areas, disturbed working areas and the boundary of the construction Site; and,
- All laydown areas, storage areas and access roads should receive a top dressing of gravel as soon as possible after upgrade initiation.

The installation, design, material and maintenance recommendations for the above noted mitigation options are described in full within Section 5.1.1 of the Report.

It is anticipated that construction phase stormwater conveyance will be accommodated through a combination of swales and catchbasin/stormsewer infrastructure. If temporary conveyance swales are to be used, they should be located adjacent to access roads running north to south to effectively convey stormwater runoff to the SWM pond in the southwest corner of the property. For swale design guidelines refer to Section 5.1.2 of the Report. Swales running the western length of the development property should have rock check dams installed to control runoff velocities and encourage sediment retention prior to SWM pond discharge.

If additional catchbasins and stormsewers are to be installed, it is assumed that they would be constructed during the initial stages of the 400,000 tpy scenario construction. The number, location and route of sub-surface stormsewers would be determined during the detailed design of the upgrade components.

Similar to the 140,000 tpy scenario construction phase conveyance capacity, it is recommended that the five-year storm be used as the sizing criteria for any temporary conveyance infrastructure. Stormsewers or swales planned for permanent conveyance should be sized according to SWM sizing criteria described below. Conveyance infrastructure represents the minor SWM system and it is assumed that the major system (grading controlled) will have been designed and implemented during the construction of the 140,000 tpy scenario Facility.

The 140,000 tpy scenario SWM pond located in the southwest corner of the property is expected to serve as the ESC pond during upgrade construction. Since only a small area of the 12.4 ha property will be included in the upgrade construction, the existing SWM pond should provide adequate stormwater retention and drawdown requirements. However it is recommended that pond capacity expansion discussed below in Section 2.3 is undertaken in the early stages of the 400,000 tpy scenario expansion construction.

2.3 Operational Phase Stormwater Management

Once fully upgraded, the 400,000 tpy scenario Facility Site will comprise approximately 55% imperviousness. This is up 10% from the 140,000 tpy scenario and will therefore require conveyance and retention/detention upgrades to SWM features. Sections 2.3.1 through 2.3.4 of this appendix address the SWM upgrades necessary given the 400,000 tpy scenario.

2.3.1 Lot Level and Conveyance Controls

The 10% increase in imperviousness means that onsite infiltration will be further reduced and runoff volumes and rates will be further increased in addition to the influence the original 140,000 tpy scenario Facility had on the subject property. To offset these effects lot level and conveyance level SWM features are recommended accordingly to detain the volume and reduce the flow rate of runoff at the lot level before stormwater enters and as it routes through the conveyance system. Detention of runoff at the lot level through depression storage and reduced runoff flow rates would act to encourage ET and infiltration.

According to the existing conditions assessment in Section 2 of the Report, onsite infiltration is minimal and therefore promoting depression storage and running rooftop leaders to open greenspace should be adequate to ensure the 400,000 tpy scenario Facility maintains the Site’s infiltration balance. Slowing runoff waters is also encouraged as it will act to increase ET and infiltration.

Permanent stormwater conveyance necessary for the western side of the property (area of Facility upgrades) should be designed to minimize runoff velocity and therefore reduce sediment transport to end-of-pipe facilities. Located within the Clarington Energy Business Park, the 400,000 tpy scenario Facility stormwater conveyance network should be designed to convey the 5-year precipitation event (Aecom, 2009).

It is unknown whether upgraded Facility area stormwater runoff would be routed to stormsewer infrastructure within the 140,000 tpy scenario Facility footprint. As such, it may be prudent to consider designing the 140,000 tpy scenario stormsewer infrastructure to the 400,000 scenario tpy 5-year precipitation event runoff. This pro-active design component is unlikely to bear significant financial burden yet may prevent conveyance capacity problems if the Facility were to be upgraded in the future.

2.3.2 End-of-Pipe Facilities

The increased capacity scenario would require a SWM pond with a larger storage capacity then that required for the 140,000 tpy scenario. All the minimum sizing and length to width ratios stated in Section 5.2.2.2 of the Report should still be followed under the 400,000 tpy scenario.

The SWM pond will also still need to provide at a minimum, 24 hours of drawdown to the 25 mm storm and ensure outlet discharges are below those derived from pre-development modeling in Section 3.4.3 of the Report.

For a site with approximately 55% imperviousness, the minimum permanent pool volume for a wet pond with an enhanced level of protection is 150 m^3/ha of drainage area. Similarly, the minimum extended storage volume for enhanced level of protection is 40 m^3/ha (MOE, 2003).

For the 12.4 ha Site, the minimum required extended detention storage is 496 m^3 which is considerably less than the 2131 m^3 of 400,000 tpy scenario post-development runoff generated during the 25 mm event. For this reason, the extended detention component of the SWM pond is expected to be sized to approximately 2131 m^3 or 172 m^3/ha. The recommended extended detention volume for the 140,000 tpy scenario was 1820 m^3 or 147 m^3/ha.

A conservative approach to designing the flood control volume for the stormwater pond is to assume full volumetric containment of the 100-year storm runoff event. Some stormwater above the permanent pool storage capacity would be discharged during the inflow of the flood control volume. However, to employ the above conservative approach, the total 100-year precipitation event volume was used. Based on the HEC-HMS model developed for this assessment, the 400,000 tpy scenario post-development total runoff volume for the 100-yr event was 8803 m^3. A summary of required stormwater pond storage volumes is provided in Table 2-3.

Table 2-3 - Required Pond Volumes for an Enhanced Level of Protection in a Wet Pond

Pond Volumes	Enhanced Level of Protection	
	Required Pond Volumes (m^3/ha)	Development Site Pond Volume (m^3)
Quality Control Criteria	80% SS removal	na
Permanent Pool	150	1860
Extended Storage*	25 mm event runoff	2131
Flood Control Volume	100 yr event runoff	6672 (8803-2131)
Total Stormwater Pond Volume	na	10663

*40 m^3/ha is the minimum required extended detention storage volume (MOE, 2003). The extended detention volume should ensure a minimum 24 hours of drawdown to the 25 mm precipitation event.

The 400,000 tpy scenario Facility Site plan (Figure 2-1) suggests that the 400,000 tpy scenario SWM pond will occupy approximately 2100 m^2. A pond with a total storage capacity in excess of 10,000 m^3 and minimum side wall slopes of 3 and 5 to 1 (See Section 5.2.2.2) would require a depth of over 10 m. It is recommended that the SWM pond be re-designed during detailed design to provide adequate length to width ratio and limit the overall pond depth to <3 m.

SWM ponds are regulated by the MOE through the C of A process. Any alterations to the 140,000 tpy scenario SWM pond will require that an amendment to the C of A be acquired.

2.3.3 SWM Facility Outlet

The 400,000 tpy scenario SWM end-of-pipe facility outlet would serve a similar roll to that in the 140,000 tpy scenario and therefore should be designed according to the same regulatory criteria. This suggests that a similar outlet structure, with altered elevation profile can be employed in the 400,000 tpy scenario design. This will ensure the minimum drawdown times and runoff retention capacities are maintained.

Similar to the 140,000 tpy scenario, the primary pond outlet structure is presented conceptually as a bottom-draw hickenbottom riser to discharge the extended detention volume estimated at 2131 m^3. The orifice opening configuration of the riser would ensure that stormwater receives at least a 24-hour drawdown period. The riser would inlet below the elevation of the permanent pool to reduce thermal impacts associated with discharging water from pond surfaces. The riser would also connect to a reversed slope pipe which would have an outlet invert at the elevation of the permanent pool. As such, the reversed slope pipe outlet would control discharge and ensure that the permanent pool elevation is maintained. The hickenbottom riser should be located 150 mm below the maximum expected ice depth to ensure continued functionality during winter conditions and 30 cm below the permanent pool elevation is the recommended riser height.

Subsequently, storage volumes exceeding the extended detention volume up to the flood control volume may be discharged additionally by a weir structure. The total flood control volume includes the 100-year event (8803 m^3). The combination of a primary hickenbottom riser and weir discharges must never exceed 0.5 m^3/s which equates to the pre-development peak discharge for the 100-year event.

In summary, the 400,000 tpy scenario SWM pond and outlet structures would be designed to ensure that post-development peak discharges would not exceed pre-development peak discharges for similar sized precipitation events. In addition, the SWM facility would provide at least 24-hours of drawdown to the 25 mm precipitation event. The final SWM pond and outlet configuration would be provided during detailed design of the 400,000 tpy scenario upgrades.

2.3.4 Offsite Stormwater Conveyance

The conveyance swale located immediately south of the Site alongside the CN Rail tracks will act as the receiver for all discharged stormwater from the Facility. Hydraulic modeling of the proximal reach of the CN Rail swale conducted in Section 3.4.3.1 of the Report indicated that the conveyance capacity of the watercourse was approximately 0.14 m^3/s. It was determined through existing condition stormwater runoff modeling that this was equivalent to the peak discharge of approximately the 5-yr precipitation event and that channel upgrades/widening may be necessary.

Future development of the Clarington Energy Business Park is expected to include centralization of SWM ponds (classified as phase II) and increased conveyance requirements for existing stormwater routing channels (Aecom, 2009). One of the facets of this centralized SWM system would be a large magnitude conveyance swale located in approximately the same CN Rail easement as the small vegetated swale described above. This centralized SWM swale would route stormwater from many developed properties to a large SWM pond south of the CN

Rail corridor. According to Aecom (2009), as development proceeds, SWM upgrades would be conducted as necessary. It is anticipated that this centralized SWM swale would be constructed prior to the 400,000 tpy scenario. This suggests that channel upgrades may not need to be addressed in this 400,000 tpy scenario assessment.

3.0 WATER SUPPLY AND WASTEWATER DISCHARGE

3.1 Water Supply Requirements

The maximum annual water demand for the 140,000 tpy scenario is estimated to be 42,000 m^3/yr or, assuming a continuous 365 day operation, 115,068 L/day or 1.3 L/s. Since precise technical specifications for the 400,000 tpy scenario are not currently available, a proration approach will be used to estimate the 400,000 tpy scenario water needs. Using a ratio of 2.86 (400,000 / 140,000) it was determined that the maximum annual water demand for the 400,000 tpy scenario would be approximately 120,120 m^3/yr or, assuming a continuous 365 day operation, 329,096 L/day or 3.8 L/s. The water needs of the 400,000 tpy scenario Facility would likely be less than a linear extrapolation from the 140,000 tpy scenario and therefore this assumption provides a conservative estimate.

Preliminary assessments (Jacques Whitford and Genivar, 2007a) assumed that a similar Facility processing up to 250,000 tpy would require approximately 100 L/s. The prorated 400,000 tpy scenario water demand is only 3.8 L/s or 3.8% of this estimated value suggesting that the Facility is considerably more water efficient than first anticipated. This study (Jacques Whitford and Genivar, 2007a) assumed that the 250,000 tpy Facility's water demand could be met by the exclusive use of one 300 mm watermain. Currently, there is a 300 mm watermain running alongside Osbourne Road that can be accessed for Facility water demands.

Considering the 400,000 tpy scenario Facility's water demand is relatively low, it is anticipated that water supply needs could be met through connection to the existing Osbourne Road watermain. A full hydraulic assessment should be carried out during detailed design to ensure the firewater and facility demands can be met. If water demands for the 400,000 tpy scenario Facility cannot be met through connection to the Osbourne Road watermain, a secondary connection to a 300 mm watermain approximately 3.5 km away would be necessary (Jacques Whitford and Genivar, 2007a).

Online firewater demand would be determined during the detailed design phase for the 400,000 tpy scenario upgrades.

3.2 Wastewater Discharge

The maximum annual wastewater discharge for the 140,000 tpy scenario Facility is proposed to be 3,000 m^3/yr or, assuming a continuous 365 day operation, 8219 L/day (0.1 L/s). This value represents almost exclusively sanitary discharge as there is expected to be very minimal to no industrial wastewater discharge from this Facility. The 140,000 tpy scenario Facility is proposed to have 33 full-time employees (Jacques Whitford, 2009). According to MMAH (1997) the average full-time employee (8-hour shifts, 5 days/week) generates approximately 125 L/day of sanitary wastewater which for 33 employees equals 4125 L/day. However, the 140,000 tpy Facility will receive refuse up to 6 days a week and a number of employee types will work 12 hour shifts (Jacques Whitford, 2009). These caveats would bring the total yearly wastewater discharge to approximately 3,000 m^3/yr.

It is anticipated that the advances in automation, expected to occur between the construction of the 140,000 tpy scenario and the 400,000 tpy scenario Facility's, would allow the 400,000 tpy scenario Facility to operate without requiring addition staff beyond the 33 necessary for the 140,000 tpy scenario. As a result, wastewater discharges for the 400,000 tpy scenario could be as low at those for the 140,000 tpy scenario. However, to provide a conservative estimate, a prorated maximum wastewater discharge was calculated. Using the proration factor of 2.86 (400,000/140,000) a maximum annual wastewater discharge of 8580 m^3/yr was estimated or, assuming a continuous 365 day operation, 23,507 L/day (0.27 L/s).

A preliminary assessment (Jacques Whitford and Genivar, 2007a) concluded that connecting a 450 mm gravity drain to the existing 1800 mm municipal sewer located north of the CN Rail tracks on Osbourne Road would be capable of conveying 63 L/s to the Courtice Water Pollution Control Plant located south of the proposed development. This capacity would be more than adequate to handle even the conservative case scenario for wastewater discharges from the 400,000 tpy scenario Facility.

APPENDIX E

Sample Erosion and Sediment Control Plan Checklist

Erosion and Sediment Control Requirements – Report.

ESC Plan Requirements – Report	Check
Project Descriptions: Brief description of the nature and purpose of the land disturbing activity. Also include the legal description of the property and a reference to adjacent properties and landmarks.	☐
Condition of Existing Site: Description of the land use, site topography, vegetation, and drainage of the site under existing conditions.	☐
Condition of Existing Receiving Water: Description of local receiving waters such as watercourses and lakes (e.g. warm water fisheries, cold water fisheries; aquatic habitat use, confined or unconfined valley).	☐
Adjacent Areas and Features: Description of neighbouring areas, such as residential and commercial areas, reserves, natural areas, parks, storm sewers, and roads that might be affected by the land disturbance.	☐
Soils: A description of soils on the site, including erodibility, and grain size analysis. This description should include a summary of the soils/geotechnical report for the site.	☐
Critical Areas: Description of areas within the development site that have potential for serious erosion or sediment problems.	☐
Permanent Stabilization: Description of how the site will be stabilized after construction is completed. This will require a phasing plan (to be provided on the ESC Plan drawing) of the stripped area to be reseeded and the expected time of stabilization.	☐
Design Details of Erosion and Sediment Control Measures: The supporting calculations and design details of the sediment control measures. Specifically for ESC ponds - calculations and details include permanent pool and extended detention volumes, pond sizing volume, and calculations for the pond outlet and emergency overflow outlet.	☐
Record Keeping Procedure: Include sample inspection and maintenance forms. Maintenance Record keeping procedure including name/designate of the personal who will keep the inspection and maintenance record.	☐
Stockpile Details: Stockpile details to include the height and volume at each proposed location.	☐
Emergency Contact: Provide a list of emergency and non-emergency contacts (e.g. owner, site supervisor)	☐
Stamped and Signed: ESC document/report must be stamped and signed by a Professional Engineer.	☐

Erosion and Sediment Control Plan Requirements - Drawing(s)

ESC Plan Requirements - Drawing(s)	Check
General Items: • Site address including application number (e.g. SP or T number) • Key map including site boundary limits • A legend identifying ESC measures • Drawing scale • North arrow • Location of any existing or proposed building(s) or structure(s) on the site	☐
Existing Contours: Existing elevation of the site at 0.5-1.0 m intervals to determine drainage patterns. Spot elevations may also be required. Extend existing contours to beyond property limit by a minimum of 30 meters.	☐
Existing Vegetation: Location of any trees, shrubs, grasses, and unique vegetation to be preserved or removed. Tree hoarding area(s) to be clearly shown.	☐
Water Resources Location(s): Location of any water body such as wetlands, lakes, rivers, streams, or drainage course on or adjacent to the site.	☐
Regional Storm Flood Plain and Fill Regulated Areas: Regional flood line level, fill regulated line and reference to relevant hydraulic model cross-section where applicable.	☐
Critical Areas: Area within or near the proposed development with potential for serious erosion or sediment problems.	☐
Proposed Contours/Elevation: Proposed changes in existing elevation contours for each stage of grading. A cut/fill plan showing existing and proposed contours. Spot elevation for proposed conditions should also be illustrated.	☐
Site Boundary Limits and Limits of Clearing and Grading: Site boundary limits and the limits of all proposed land disturbing activities.	☐
Existing and Proposed Drainage Systems: Location and direction of any existing/proposed storm drainage system (e.g. storm sewers, swales, ditches, etc.) and overland flow drainage patterns within and adjacent to the site.	☐
Limits of Clearing and Grading: A line defining the boundary of the area to be disturbed.	☐
Stockpile and Berm Data: Stockpile and/or berm locations, size and the diversion route of the runoff. Consideration will include proximity to existing homes	☐
Erosion and Sediment Control Measures Locations and Details: Location and details for all ESC measures proposed with notes provided to direct their timing/phasing such that there is an appropriate level of protection provided during all stages of construction (e.g. Sediment fence should be installed prior to any land disturbing activities).	☐

Source: GGHACA, 2006.