WINDOWSOR/ESSEX REGION STORMWATER MANUAL

Originally Prepared by:
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Prepared for:

Essex Region Conservation Authority
sustaining the place for life

on behalf of the Windsor/Essex Region, consisting of the following municipalities:

Version	Date	Description	Author
0	Oct. 12/18	Final Draft for Stakeholder Review	Stantec
1	Dec. 6/18	1st Publication of SWM Standards Manual	Stantec
2	June 12/24	Amendment No. 1	Landmark

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Special thanks to all for their shared wisdom and insights that contributed to the manual.

V2 AMENDMENT NO.1

Note that the document name has been revised (i.e., condensed) to "Windsor-Essex Region Stormwater Manual". The rationale for this change is provided below.

All other changes made to the previous version of this document (i.e., Version 1) are identified by a "V2" (i.e., Version 2) superscript annotation at the start of the change. Where specific sections of text have been changed, a subscript annotation is also included at the end of the change. For example: ^{V2} All text within the "V2" start/end points represents new or edited information that has been changed from version 1. $_{V2}$

At the time of developing the *SWM Standards Manual*, stakeholders and technical contributors agreed that a consistent approach to stormwater analysis / management in this region was necessary – this was the primary objective of the *Manual*. A secondary objective was to weed-out approaches that were deemed unsuitable for the region. The *Manual* has been published for five years now and during this time, it has become evident that there are sections that are not clearly expressing the intent or application that was envisioned.

The *Manual*, in its prescriptive and conservative nature, has at times appeared to replace or discourage the practice of engineering judgement to solve a specific problem with a tailored solution. In many instances, it seems that the *Manual* is being viewed and adopted as a rigid standardized approach for all SWM project types and sizes. As a key emphasis to this amendment, the *Manual* should be viewed as a guide or helpful tool rather than a rule book or rigid standard.

Based on the foregoing, the document name has been revised to more appropriately represent its intent. As an added benefit, the name and its associated acronym (i.e., WERSM) are both simplified.

More specifically, this amendment is intended to provide additional guidance, clarification, review and revisions related to a number of items, which are outlined in the following table.

Amendment No.1 – Revision Summary Table

Section No.	Comments
Document	Revised document name to Windsor-Essex Region Stormwater Manual
Preface	Added text to clarify that this manual may not address all stormwater requirements. Replaced "standards" with "design criteria" and "manual".
1.2	Removed "standards" from heading. Replaced "standards" with "design criteria".
1.3	Added text to clarify the objective of the manual.
1.5.1	Revised Eq. 1.5.1.
1.5.2	Added text to clarify levels of consequence.

Section No.	Comments
3.0	Added text to promote simple methods to meet the design objective. Replaced "standards" with "design criteria". Added text to note that additional requirements may arise through consultation with the municipality and other agencies.
3.2.1.1	Added text regarding updated IDF data.
3.2.2.4	Added text to clarify sewer design method and intent.
3.2.2.5	Added text to clarify intent of HGL requirements.
3.2.2.7	Added text to include exceptions for reduction in C value. Revised C Values table
3.2.3.5	Added text to advise that specific roadway standards/requirements may apply.
3.3.1.3	Added text to clarify definition of downstream receiver(s).
3.3.1.4	Added text to clarify intent and acceptable methods of hydrologic analysis.
3.3.2	Added text to provide guidance on backwater conditions and clarify storage requirements.
3.3.2.1	Revised text to suit design intent. Added guidance for using enhanced storage as stress test.
3.3.2.2	Revised text to suit design intent. Added text regarding backwater conditions. Added guidance regarding proper use of the Modified Rational Method. Added guidance for estimating stress test additional volume using MRM.
3.3.2.7	Replaced "standard" with "manual".
3.3.5.3	Added tables to summarize AMMM levels at various return periods.
3.3.5.6	Added guidance regarding sewer outfall clearance depth.
3.3.5.7	Added guidance regarding ice thickness for submerged inlets/outlets.
3.3.5.8	Added guidance regarding uncontrolled areas.
3.3.5.9	Added guidance regarding interim SWMP where improvements to the receiver are planned.
3.6	Replaced "standards" with "design criteria".
3.6.1.5	Added clarification of design intent.
3.7.1.2	Added guidance in estimating flows for large agricultural watersheds.
3.7.2.2	Added clarification regarding SCS hydrograph method peaking factor.
3.7.3.1	Added clarification regarding the use of Eq. 3.7.3.1.
3.7.5	Added wet pond to Impervious % table.
3.7.6	Added guidance regarding deviations to typical depression storage depths.
3.7.7.3	Added dry AMC infiltration rate values. Added text to clarify design intent.
3.7.7.4	Added text to clarify recommended use of CN method and CN values.
3.7.7.5	Added dry AMC infiltration rate values. Added text to clarify design intent.
3.7.8.3	Added note regarding change to UST timeseries. Added text to clarify intent and application of the stress test.
3.7.9.6	Added guidance regarding dual drainage modelling.
3.9.1	Added additional information related to the August 2017 extreme event
3.9.2	Replaced "standard" with "manual".

Section No.	Comments
3.10.1	Added text to clarify definition of infill development from stormwater perspective.
3.10.1.3	Added note to reference section 3.10.1.5.
3.10.1.5	Added guidance regarding flexibility in level of SWM quantity control.
3.10.2.1	Added text to clarify intent provide guidance on assessment approach. Replaced "standards" with "design criteria".
3.11	Revised submission requirements.
Appendix B	Revised SCS Type II & Urban Stress Test (UST) storm distribution tables
Appendix C	Added CN value tables
Appendix D	Added Amendment No.1 Summary and Supplemental Information

PREFACE

This document outlines stormwater management [V2] design criteria [v2] for the Windsor/Essex Region. The document presents what is considered to be the best practice for the region, given the state of the science at this time. It is to be viewed as a living document, to be reviewed, updated and improved. At a minimum, the document is to be reviewed every 5 years.

The [V2] manual [v2] provide practical, and at times, fairly prescriptive design criteria. **However, the designer is solely responsible for stormwater design and has the flexibility to deviate from the specified guidance <u>provided that</u> the supporting rationale and technical merit meets the stormwater objectives of the manual to the satisfaction of the Municipality, the Conservation Authority (ERCA or LTVCA) and other approval agencies. [V2] The Municipality and Conservation Authority reserve the right to request additional information to satisfy requirements that are not specified in this document. [V2]**

The document is presented in six sections as outlined below, along with a brief description of the section content.

SECTION 1: INTRODUCTION

This section is for ALL STAKEHOLDERS. It discusses why we need the manual.

SECTION 2: PLANNING

This section is for ALL STAKEHOLDERS. It highlights the importance of proper drainage planning, and how decisions made at the planning stage impact the ultimate drainage function.

SECTION 3: DESIGN

This section is for CONSULTANTS and MUNICIPALITIES. It presents design [V2] criteria [v2] and requirements for stormwater design in the Windsor/Essex region.

SECTION 4 – PRIVATE DRAINAGE SYSTEMS

This section is for CONSULTANTS, MUNICIPALITIES and DEVELOPERS. It discusses the need for better coordination of municipal and private drain design as well as provides recommendations for private drainage system construction to mitigate basement flooding.

SECTION 5 – IMPLEMENTATION/CONSTRUCTION

This section is for CONSULTANTS, MUNICIPALITIES and DEVELOPERS. It discusses the requirements of proper implementation and construction of stormwater designs.

SECTION 6 – OPERATION AND MAINTENANCE

This section is for CONSULTANTS and MUNICIPALITIES. It discusses the minimum requirements for operation and maintenance manuals to support stormwater infrastructure.

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Abbreviations

AMC	Antecedent Moisture Conditions
BMP	Best Management Practice
CA	Conservation Authority
ECA	Environmental Compliance Approval
ERCA	Essex Region Conservation Authority
ESC	Erosion and Sediment Control
ETV	Environmental Technology Verification
GI	Green Infrastrucuture
ha	hectares
HGL	Hydraulic Grade Line
HWL	High Water Level
IDF	Intensity-Duration-Frequency
L/s	Litres per second
LID	Low Impact Development
LTVCA	Lower Thames Valley Conservation Authority
m	metres
m^3	cubic metres
m^3/s	cubic metres per second
mm	millimetres
MNRF	Ministry of Natural Resources and Forestry
MECP	Ministry of Environment, Conservation and Parks (formerly MOECC and MOE)
MOECC	Ministry of Environment and Climate Change
MTO	Ministry of Transportation Ontario
NRCS	National Resources Conservation Service
NWL	Normal Water Level
OGS	Oil/Grit Separator
RVC_T	Runoff Volume Control Target
SCS	Soil Conservation Service (now NRCS)
SWM	Stormwater Management
SWMF	Stormwater Management Facility
SWMP	Stormwater Management Practice
TSS	Total suspended solids
WQS	Water Quality Storm
WSEL	Water Surface Elevation

Glossary

1:5 year storm event (also referred to as 5-year storm	A storm event with a 1:5 year return period or 20% probability of occurrence in any given year.
Allowable release rate	A maximum specified flow rate at which development is allowed to discharge.
Antecedent moisture condition	The pre-storm soil moisture condition.
Backwater condition	A backflow condition or rise in water level which impacts conveyance capacity
Combined sewer	A combined sewer is a sewage collection system of pipes and tunnels designed to also collect surface runoff
Detention	The temporary storage of stormwater to control runoff discharge rates and promote settling of sediment.
Extended detention	A specified volume to be detained over a minimum 24-hour period for water quality purposes.
Freeboard	The depth measured from the water surface elevation to a specified reference point (e.g. manhole cover, building opening, pond bank)
Holistic approach	An approach that considers in the context of the overall watershed.
Hydraulic grade line	The surface or profile of water flowing in an open channel or a pipe flowing partially full. If a pipe is under pressure, the hydraulic grade line is that level water would rise to in a small, vertical tube connected to the pipe.
Hydrodynamics	The study of motion of liquids, and in particular, water. A hydrodynamic model is a tool able to describe or represent in some way the motion of water.
Hyetograph	A graphical representation of the distribution of rainfall over time.
Level of service	Level of service refers to the efficiency of the drainage system to capture and convey runoff away from the surface and buildings. In the context of drainage, level of service is described in terms of a return period.
Major	In the context of stormwater, major relates to a major storm event. For purposes of design, the major storm event is typically quantified as a 1:100 year storm event.
Minor	In the context of stormwater, minor relates to a minor storm event. For purposes of design, the minor storm event is typically specified for storm sewer sizing with a return period of 1:2 year or 1:5 year.
Obvert	Elevation at the highest point of the inner surface of a pipe (i.e. interior top of pipe)

Permanent pool	The body of water which remains in the stormwater management pond.
Private drainage system	A system of underground piping, sump pump, roof leaders, rear yard catch basins, sewage ejector pumps, etc. which convey stormwater and sewage flows from private property to the municipal sewer(s).
Receiver	The receiving drain, watercourse or sewer.
Retention	The permanent storage of stormwater to control runoff discharge rates and volume by promoting infiltration, evapotranspiration and re-use.
Return period	A return period, also known as a recurrence interval is an estimate of the likelihood of an event, such as an earthquake, flood or a river discharge flow to occur
Runoff	Surface water, from precipitation, that flow over the land surface.
Stormwater	Stormwater is the water from rain or melting snow that is not absorbed into the ground.
Subcatchment	An area of land where all surface runoff converges or is assigned to a single point along a drainage feature. E.g. a storm sewer manhole.
Watercourse	An open channel that conveys water to a larger watercourse or waterbody.
Watershed	An area of land that drains into a watercourse or waterbody

1.0 INTRODUCTION

1.1 BACKGROUND AND HISTORY

Land development increases both the amount (volume) of stormwater runoff, and the rate at which runoff occurs. A principal purpose of stormwater management (SWM) is to mitigate the potential for flooding to downstream landowners due to the hydrologic effects of development.

The physiology of the landscape that predominates much of the Windsor/Essex region imposes special challenges to the effective implementation of SWM. There are two characteristics of the region in particular that create special challenges – the nature of the terrain and the type of soils that predominate the area.

Most of Essex County is very flat in comparison to other regions of the province. Due to the limited land gradient, many of the receiving watercourses and trunk sewers flow near full during even moderate rainfall events. During extreme events many watercourses overflow. Most trunk sewers exhibit hydraulic grade lines that exceed the top of the sewer, and often match or exceed the surrounding ground elevations.

In addition to the flat topography, most of Essex County is underlain by impervious, clayey soils. This feature results in a larger percentage of the annual rainfall that the region receives being converted to runoff, as compared to areas that are underlain by sandy, pervious soils. The clayey soils create an additional challenge – clay soils limit the ability to infiltrate rainfall.

The focus of SWM has been evolving over the years. Prior to 1990, SWM focused on quantity control, to reduce post-development peak runoff rates to pre-development levels. In the early 1990s, the objective of SWM in Ontario was expanded to include quality control. Since 1990, many SWM facilities have been constructed in the region. Most SWM facilities that service larger developments (i.e., greater than a few lots) employ some form of pond. The ponds that were constructed prior to 1994 were normally dry ponds, intended to achieve quantity control. Those constructed since typically incorporate a permanent pool that is intended to provide an extended detention, quality control function.

1.2 [V2] WHY THE MANUAL IS NEEDED

In the Windsor/Essex region, the prescribed [V2] design criteria [v2] for stormwater management systems vary considerably from municipality to municipality. This leads to a wide range of variation in stormwater management designs, which results in inconsistent stormwater management measures. There is a need for regional [V2] design criteria [v2] to:

- provide a minimum standard and consistent level of service and protection of the environment throughout the region
- provide fair and equitable minimum standard for all

- address stormwater at the watershed scale without being limited by municipal boundaries.
- streamline the review process to eliminate re-submissions and re-design efforts

1.3 OBJECTIVE

The principal objective of this manual is to provide a clear, concise and consistent approach to stormwater design within the Windsor/Essex region. The proposed manual serves to identify the general policies and technical guidelines adopted by regulatory agencies. It provides direction to consulting firms and municipalities in the development and review of technical reports in support of new development. The manual is to be read in conjunction with the current Ministry of the Environment, Conservation and Parks 2003 Stormwater Management Planning and Design Manual (published under the former name, Ministry of Environment), and the current Ministry of Natural Resources and Forestry (MNRF) 2002 Natural Hazard Technical Guide. The manual is tailored to the local challenges of this region and in this respect, it is intended to supplement other applicable manuals/guides, and in some instances discourage practices that are not suited to this region.

The manual is intended to be prescriptive in certain aspects of design and establish minimum submission requirements that require absolutes. Notwithstanding, the designer shall continue to have flexibility in the design of stormwater management solutions and is solely responsible for their design for any given site.

[V2] This document is generally intended to support new development by addressing stormwater management in a consistent manner across Windsor-Essex. Large-scale studies such as but not limited to: Environmental Assessments, Secondary Plans, subwatershed studies, etc., may, and likely should, consider other factors and design criteria that are not included in this document. This document is not intended to be a cookbook or a comprehensive guide that addresses all stormwater management considerations. [V2]

1.4 WHY STORMWATER MANAGEMENT IS IMPORTANT

1.4.1 General

Land development results in increased rates and volumes of stormwater runoff. Without stormwater management measures, the impacts of development can lead to increased flooding, degradation of water quality and aquatic ecosystems, stream erosion and property damage. Left unmanaged, stormwater often eventually leads to major public expense in infrastructure to solve flooding or erosion problems.

1.4.2 In the Windsor/Essex Region

Stormwater management measures are being implemented by municipalities in the Windsor/Essex Region, however the approach has generally been isolated to individual developments and at the site planning level. This approach has the potential to lead to both inefficient and inconsistent implementation of stormwater management within a watershed. Stormwater management requires a holistic approach on a watershed scale that considers both stormwater management constraints and opportunities.

The Windsor/Essex region has relatively flat topography and poorly drained soils that create many challenges for effective management of stormwater. Many drainage systems in the region are affected by lake levels which can have a notable backwater effect. Moreover, the construction of stormwater management ponds in flat areas has artificially created a backwater condition that can surcharge sewer systems and potentially saturate home foundation walls. The lack of gradient has also contributed to a significant number of pumped outlets within the region.

1.5 UNDERSTANDING LEVEL OF SERVICE AND RISK

Level of service refers to the efficiency of the drainage system to capture and convey runoff away from the surface and buildings. In the context of drainage, level of service is described in terms of a return period – the likelihood that a storm event of specified magnitude will occur in any given year. For example, a 1:100 year storm event has a 1 in 100 or 1% chance of occurring in any given year. The return period can give a false sense of safety as a 1% chance is interpreted as an absolute rather than a statistical average.

To illustrate this point, the following table correlates return periods and probability of exceedance (or risk) over the design life.

Table 1.5 – Probability of Exceedance (Risk)

Return Period	Design Life					
	2	**5**	**10**	**25**	**50**	**100**
2	75%	97%	100%	100%	100%	100%
5	36%	67%	89%	100%	100%	100%
10	19%	41%	65%	93%	99%	100%
25	8%	18%	34%	64%	87%	98%
50	4%	10%	18%	40%	64%	87%
100	2%	5%	10%	22%	39%	63%

Risk (r) = $1 – (1-1/T)^{L}$, where T = return period and L = Design Life (MNRF, 2002)

For example, there is a 63% chance of exceeding a 1:100-year storm in the next 100 years. It should be acknowledged historical records used to derive return periods are often based on less than 100 years of data (61 years at Windsor Airport).

1.5.1 Defining Risk/Reliability

The acceptable risk for a project must ultimately consider the consequence of failure or capacity exceedance of the design. The minimum standard level of service has been defined herein as a 100-year design storm. This minimum standard does not guarantee that a given site will never flood but rather, it guides the design of mitigating measures to achieve a low risk of flooding.

Where an individual site's potential damages due to flooding are high, it is the practitioner's responsibility to design to a more conservative standard or to provide a sufficient emergency flow route in accordance with the proponent's site-specific needs.

The inverse of risk is reliability and is a standard term used in other engineering fields to define the design standard over the long-term or design life of the infrastructure. When evaluating infrastructure within a flood control perspective, reliability should be defined to clearly express the level of protection that the infrastructure is being designed to. As a specific example, the 1:100 year 24-hour rainfall amount based on Windsor Airport historical data is 108mm. A pond designed to this 1:100 return period has a 1% chance of exceeding its design high water level in any given year. However, the same pond has a 63% chance of exceeding its design high water level over the next 100 years. Conversely, the pond design can be said to have a reliability of 36% over of the next 100 years, where reliability and risk are defined per Eq. 1.5.1 below.

V2 **Eq. 1.5.1:** Risk = 1 – Reliability = $\left(1-\left(1-\frac{1}{T}\right)^{L}\right)$
where T = Return Period and L = Design Life

Assuming the same pond accounted for a minimum freeboard depth of 0.3 metres – as measured from the 1:100-year design water level to the top of bank and that said freeboard provided additional storage to contain a rainfall of 150mm, the return period would be 1:2,600 year and the pond would actually have a reliability of 96% over the next 100 years with regards to overtopping of the pond banks.

Defining flood risk in terms of reliability of the design provides a much clearer sense of the long-term level of protection or flood mitigation provided by the design and adjusts expectations on its performance.

1.5.2 Risk Assessment

The following definitions, adopted by MTO Highway Drainage Standards, provide general guidance on assessing the consequence of failure or capacity exceedance from the perspective of Public Safety, Traffic Delays, Damage due to Flooding, and Natural Habitat Impacts. These definitions do not include site-specific risks that may need to be considered, such as high consequence and damages due to loss of a specific high-cost crop or downtime costs of an industrial site due to delayed delivery of products, to name a few.

Low Consequence:

- Public Safety – failure or capacity exceedance is not a significant risk to public safety
- Traffic Delays – there would be no significant traffic delays as there are alternative routes
- Damage due to Flooding – flooding would be local or would be limited to unimproved rural lands that would not be adversely affected by the flooding
- Natural Habitat Impacts – any impacts will be temporary (i.e. fish habitat not permanently affected and vegetation damage will generally recover within two growing seasons)

Medium Consequence:

- Public Safety – failure or capacity exceedance is not a great risk to public safety
- Traffic Delays – there may be road closure(s) causing delay or detouring (nuisance)
- Damage due to Flooding – land uses such as croplands or parking will be flooded
- Natural Habitat Impacts – temporary impacts anticipated that may take more than two growing seasons to recover

High Consequence:

- Public Safety – failure or capacity exceedance represents a significant risk to public safety
- Traffic Delays – road closure(s) causing significant impact on traffic or emergency vehicles
- Damage due to Flooding – buildings will be flooded
- Natural Habitat Impacts – permanent damage anticipated, requiring mitigation and/or habitat compensation

The consequence of failure or capacity exceedance shall be determined for each of the four categories (Public Safety, Traffic Delays, Damage due to Flooding, Natural Habitat Impacts). The worst-case impact (low, medium, high) from the four categories shall be used for selecting the Return Period, [V2] storm distribution timestep or maximum sewer inlet time [V2], that shall guide design.

V2 The following provides some examples of Level of Consequence:

- A residential subdivision is typically designed with low consequence as infrequent short-term ponding on local streets does not have a significant impact on the public or property. It should be acknowledged that a passable depth of surface ponding (i.e., less than 0.3m) at a local street catch basin for a short duration (i.e., less than 20 min.) is not a significant impact to the public, but rather a minor inconvenience.
- A highway would typically be designed with high consequence as short-term ponding on the highway is a significant risk to public safety.
- An arterial roadway may warrant a medium consequence design as short-term ponding could affect traffic flow and/or restrict the number of passable lanes, resulting in some risk to public safety and traffic impacts due to delays and detouring.
- An arterial roadway may warrant a high consequence design at a major sag location, whereby surface ponding depths could make the road impassable, resulting in significant risk to public safety and/or significant impact to traffic. V2

1.6 STORMWATER PARADIGMS

The list below outlines general paradigm shifts that have occurred with respect to stormwater. Some of these paradigms may not coincide with the events or timelines experienced in our region. The brief history of past paradigms is intended to illustrate that solving one problem can create another more challenging problem. As such, history tells us that a cautious and progressive shift to a new paradigm is recommended.

1. Before sewers, stormwater and wastewater flowed along streets and in ditches. This led to unpleasant and toxic conditions of smell and disease.

2. The solution was to put this water in sewers (what we now call combined sewers) and discharge to the nearest river or lake.

3. This resulted in water quality issues with the receiving water and consequently with the drinking water that was being supplied by the same polluted waters. The solution was to collect and treat wastewater through one sewer (referred to as a sanitary sewer) and convey stormwater through a second/separate sewer (referred to as a storm sewer). Priority was placed on draining stormwater away from the landscape as quickly and efficiently as possible.

4. By the 1970s, it became evident that the efficient storm sewer systems had created an unforeseen problem in the form of downstream flooding and channel erosion. The solution was detention of stormwater via pond storage with controlled outflow equal to the pre-development condition.

5. By the 1980s, new technologies led to stormwater master planning to find the preferred solution to the watershed flooding problems.

6. New studies focused on evaluating stormwater pollution, and by the 1990s, the inclusion of stormwater quality control was the new standard. Water quality evolved from pollutant removal to assessment of the larger ecosystem at the watershed scale, however it was becoming apparent that regional solutions were difficult to implement and did not address water quality and erosion issues at the community level. It was realized that watershed health issues were a cumulative impact of numerous individual sites and that this is where the problems needed to be addressed.

7. The early 2000s began to look at lot level controls as sustainable green infrastructure (also known as low impact development) that would more closely mimic the natural hydrology of undeveloped land. Our province is currently in the midst of a paradigm shift to low impact development, mainly driven by regulatory requirements that were developed in other regions of the province.

8. The latest of stormwater challenges are dealing with climate change and creating resilient stormwater systems.

2.0 PLANNING

The focus of this manual is not on stormwater planning; however, planning is the first step of proper stormwater management and a necessary step to set objectives for stormwater design. The need for stormwater management is a direct result of land development. As a result, land use planning and stormwater design must be integrated to be most effective.

As stated in Ontario's 2014 Provincial Policy Statement (Policy 1.6.6.7), planning for stormwater management shall:

- **a) minimize, or, where possible, prevent increases in contaminant loads;**
- **b) minimize changes in water balance and erosion;**
- **c) not increase risks to human health and safety and property damage;**
- **d) maximize the extent and function of vegetative and pervious surfaces; and**
- **e) promote stormwater management best practices, including stormwater attenuation and re-use, and low impact development.**

2.1 IMPORTANCE OF WATERSHED STORMWATER PLANNING

Good planning provides a fundamental basis for addressing stormwater requirements efficiently and cost effectively. Stormwater planning should be undertaken by municipalities to provide economies of scale at the watershed level.

In the absence of stormwater planning at the watershed/subwatershed level, the land developer and consultant are often responsible for defining stormwater management objectives through pre-consultation with local agencies and municipalities. This approach may result in the following:

- Watershed/subwatershed ecosystem and water management issues and priorities may not be identified.
- Cumulative impacts of development on flooding, water quality, erosion, and baseflow cannot be assessed at the site level without significant engineering evaluations being undertaken by the development engineer related to the receiving drainage system.
- The identification of natural area linkages and wildlife corridors is best accomplished at the watershed/subwatershed scale.
- Regional stormwater management approaches and/or improvements to conveyance features cannot be evaluated.

In instances where a watershed plan does not exist, individual developments have been allowed to proceed on the basis that stormwater measures match pre-development peak flow conditions. The rationale supporting this approach is such that the post-development outflow will not exceed the pre-development outflow for the same proposed development area and thus should not create any adverse hydraulic impacts to the watershed (i.e., does not make things worse). The issue with this approach is that pre-development conditions cannot be properly assessed at the lot level scale of individual developments. **The potential consequence of this is incremental and cumulative negative impacts on the watershed**. Further discussion in section A-3.3.1.4 illustrates this point.

To ensure that development does not make things worse, it is imperative that pre-development conditions be evaluated on the watershed scale. **In the absence of watershed planning, interim measures shall be determined via pre-consultation with the Conservation Authority (ERCA or LTVCA) and Municipality.** Conservative assumptions may be warranted to simplify the watershed evaluations and make them manageable for small scale developments.

Undoubtedly, this will place a significant burden on development for which the only remedy is proper watershed planning. Ideally, watershed planning will not only ensure that development maintains existing conditions, but it can improve upon them and address existing issues with well planned development.

3.0 DESIGN CRITERIA

This section of the manual outlines V2 design criteria V2 and input parameters to provide clear and concise guidance to stormwater management practitioners and ensure a consistent approach to stormwater design within the Windsor/Essex region.

V2 A best effort was made to include as much guidance as possible with respect to design criteria; however, additional requirements may arise through consultation with the municipality and other agency approval authorities, such as those that may be within Consolidated Linear Infrastructure Environmental Compliance Approval agreements. V2

V2 As a helpful tip or reminder to all practitioners, the following is an adaptation from the foreword written by G. Colling, P. E. for practical design of sheet pile bulkheads handbook, applied to hydrology:

"The variable nature of rainfall and the uncertainty of factors governing the hydrological response, present problems of rainfall losses and runoff generation that cannot be solved with mathematical precision. It must never be overlooked that, in practice, one cannot expect to obtain from computations in hydrology, results of more than approximate accuracy in relation to reality.

To deal with a hydrological problem it is therefore necessary to have straightforward methods of design that are rapid and accurate enough to reveal the possible solutions of the problem in view within a relatively short time. These methods should avoid lengthy computations and should yield results that can be easily verified." V2

3.1 DUTY OF CARE

! The designer is solely responsible for stormwater design and has a duty of care to consider and account for site specific conditions that may warrant variations in design criteria and parameters compared to those provided in this manual. In such instances where variations are proposed, the proponent will need to provide technical justification for review and approval by the Conservation Authority and Municipality. It is strongly urged that any proposed variations be proposed/reviewed at the pre-consultation stage or otherwise as soon as they become apparent in order to mitigate re-design efforts in the event that the proposed variations are not accepted. A "!" symbol is denoted throughout this section to remind the practitioner to read and acknowledge this section 3.1 when following the guidance of this manual.

3.2 STORMWATER DRAINAGE SYSTEMS

3.2.1 Rainfall Intensity

3.2.1.1 *Design Storm Intensities:* The design storm intensity shall be calculated using Equation 3.2.1.1. See **Appendix A** for supplemental information in reference to this section.

Eq. 3.2.1.1: Intensity (mm/hr) = $\frac{a}{(T+b)^c}$ where T = time of concentration in minutes

Table 3.2.1.1 below summarizes Intensity-Duration-Frequency (IDF) curve parameters (a, b, c) based on 61 years (1946-2007) of historical rainfall data from Windsor Airport (Station No: 6139525).

Table 3.2.1.1 – IDF Curve Parameters

Parameters	Return Period (Years)					
	2	**5**	**10**	**25**	**50**	**100**
a	854	1259	1511	1851	2114	2375
b	7.0	8.8	9.5	10.2	10.6	11.0
c	0.818	0.838	0.845	0.852	0.858	0.861

V2 With new historical rainfall data from 2008 to 2016, a comparison was made to evaluate the impact of the new data on the foregoing IDF curve parameters. The evaluation concluded that the % change in rainfall amounts under various durations and return periods is statistically insignificant (see **Appendix D** for a detailed summary of data comparisons). As such, no change is warranted at this time. It is recommended that rainfall data be reviewed again at the next manual update, or when an Environment Canada IDF statistics update provides data beyond 2016. V2

3.2.2 Storm Sewer (Minor) System

The minor system, typically a storm sewer, consists of drainage works that convey flows from the design minor storm event. These systems offer quick and efficient drainage of urbanized areas to limit the inconvenience of stormwater ponding.

3.2.2.1 *Standard Return Period:* The standard for new municipal storm sewer (minor) system design is a 5-year return period. Where new storm sewers are proposed to connect to existing sewers designed to the historical 2-year design standard, the new storm sewers shall be sized to the new standard 5-year with appropriate flow control to limit the flow to the available capacity of the receiving storm system. See **Appendix A** for supplemental information in reference to this section.

3.2.2.2 *Custom Return Period:* For non-typical municipal minor system design, the design return period shall be based on applicable MTO, MNRF or other applicable drainage design standard (see **Appendix C** for reference). The Municipality and/or the CA have the discretion to specify a return period that is greater or lesser than the standard design storm.

3.2.2.3 *Rainfall Intensity:* Rainfall intensity for stormwater design shall be based upon 3-parameter IDF curves derived from Environment Canada's Windsor Airport rainfall data. (See **Appendix A** for further discussion). Refer to **Table 3.2.1.1** for IDF Curve parameters.

3.2.2.4 *Sewer Design Method:* Storm sewer networks can be designed using the Rational Method for storm catchment areas where the time of concentration does not exceed two times the appropriate maximum inlet time per **Graph 3.2.2.6**. Larger catchment areas require hydrologic/hydraulic modeling to verify/confirm the capacity of the sewer system. Design storm hyetographs are discussed in section 3.7.8.

V2 Design of storm sewers using a spreadsheet with the Rational Method and Manning's Equation remains a consistent and standardized approach to appropriately size sewers to meet the desired level of service (i.e., design return period) under uniform flow conditions (i.e., where the pipe slope (So) is equal to the hydraulic gradient (Sf)).

In applications where Sf is not equal to So (e.g., submerged sewer outfall with high tailwater conditions; or, major storm event exceeding sewer design capacity), the practitioner may need to calculate the Hydraulic Grade Line (HGL) with consideration to the requirements of section 3.2.2.5 below. This calculation can be done using a spreadsheet or modelling software. V2

3.2.2.5 *Hydraulic Grade Line (HGL) Analysis / Surface Ponding:* For storm sewer design, the hydraulic grade line shall not rise above an elevation equal to 0.3 metres below ground elevation. The HGL analysis shall consider backwater conditions and minor losses. By satisfying the foregoing HGL requirement, the resulting standard is that no surface ponding shall occur under the minor storm event, except as defined in section 3.3.2.6 where parking lot storage is deemed acceptable.

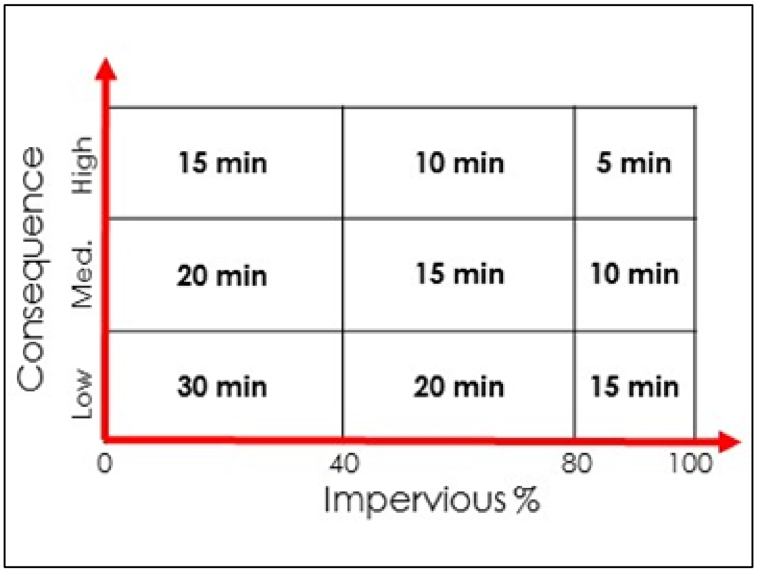
V2 This subsection serves as a typical minimum level of service that limits the frequency of nuisance ponding. For new development with new infrastructure, the preferred level of service would ideally maintain free flow conditions under the design event (i.e., HGL no higher than obvert of the sewer – no submergence). When frequent backwater conditions are anticipated to result in frequent sewer outfall submergence conditions with corresponding reduction in hydraulic gradient (i.e., reduction in available head differential across the overall sewer length), it may be warranted to increase the sewer sizing beyond the standard Manning's Equation uniform flow design in order to achieve the desired level of service. Based on the foregoing, it should be understood that the sewer sizing would be increased to accommodate a condition where HGL slope (Sf) needs to be flatter than the pipe slope (So). It should also be understood that new infrastructure should not typically be sized such that Sf is steeper than So under design conditions.

Notwithstanding, it is important to note that for existing sewers, the warrants for achieving the foregoing HGL requirements (i.e., recommended level of service) should be considered with the condition/age of the sewer as well as the cost/disruption of sewer replacement.

<u>Caution for the modeller:</u> Modelled HGL profiles can sometimes depict HGL slopes (Sf) that are steeper than the pipe slopes (So), which demonstrates that the sewers are undersized to convey the estimated flow. It should be acknowledged that a short reporting time step in the model simulation could result in HGL profiles that capture a very short duration where Sf > So. For residential storm sewer HGL assessments, a reporting timestep of 5 min is suggested to flatten out instantaneous short-duration peaks that do not represent a meaningful volume and correspondingly, do not result in a meaningful impact on the sewer's level of service. Conversely, when assessing highway drainage and potential roadway surface ponding, a short duration of ponding could be deemed a high risk and, in this instance, a small reporting timestep (e.g., 1 min. or less) would be critical to the assessment. The foregoing is another example of a model being 'fit for purpose'. V2

3.2.2.6 *Sewer Inlet Times:* Inlet times shall generally follow **Graph 3.2.2.6** as maximum permissible values, which are dependant on impervious level and consequence of exceedance (as defined in Section 1.5.2).

! Graph 3.2.2.6 – Maximum Inlet Times



With consideration to the fact that stormwater management is not a one size fits all science, the above graph is an important tool for practitioners to exercise judgment on acceptable level of service for varying goals and objectives. The impervious level allows for dampening of runoff response with increased perviousness, in a way that models cannot always mimic. The consequence allows the practitioner to adjust the level of service based on the consequence of short-term exceedances to the design capacity.

Where the selection of consequence level is unclear, the Conservation Authority and Municipality shall guide practitioners in assessing the proper consequence level.

3.2.2.7 *Runoff Coefficients (C value):* C values shall generally follow **Table 3.2.2.7** as minimum design values to be used. These values are to be used only for Rational Method peak flow calculations. A reduction to the overall C value shall not be made on account of disconnected roofs. See **Appendix A** for supplemental information in reference to this section. [V2] Exceptions to this requirement could be considered for soils in Hydrologic Group A or B (i.e., soils with high infiltration capacity). [V2]

! Table 3.2.2.7 – Minimum C Values for Standard 5-Year Sewer Design

Land Use	C value
Roof areas	0.95
V2 Asphalt, Concrete	0.90 – 0.95
Gravel	0.70
Grass – sandy soil	0.15
Grass – clay soil	0.20
Residential – Single family	0.60
Residential – Single family (lot size 500 m^2 or less)	0.70
Residential – Semi-detached	0.70
Residential – Townhouse / Row housing	0.80
Industrial / Commercial	0.90
V2 Wet Pond	1.0

3.2.3 Major System/Floodproofing

The major system consists of drainage features that convey flows during major storm events that occur less frequently. Typically, the major system consists of surface features such as roadways and overland swales that provide a pathway to safely convey runoff to the receiving outlet. The dual drainage concept is such that the minor system provides the convenient drainage for minor storm flows and the major system assists in conveying major storm flows in excess of the minor system capacity. The major system always exists, regardless of whether or not it is planned for.

3.2.3.1 *Standard Return Period:* The minimum standard for major system design is a 100-year return period. Refer to section 1.5 for discussion related to return periods, level of service and risk.

3.2.3.2 *Public Safety / Damage:* The depth and velocity of overland flow are to be limited to mitigate hazard to the public, erosion or other property damage. Refer to MNRF Flood Hazard Guide Figure 6-2, included in **Appendix C** of this document.

3.2.3.3 *Surface Ponding:* Surface ponding on roads and parking lots shall not exceed 0.3 metres in depth or less if required by the Municipality. For high traffic roadways (e.g., highways, arterial roads), lower depths may be required.

3.2.3.4 *Floodproofing Elevations:* The **minimum** lowest opening into all buildings shall be at least 0.3 metres above the Regulatory Flood Level or on-site calculated 100-year water storage elevation, whichever is greater. Additional floodproofing measures may be warranted based on Building Code requirements and/or site-specific risks and potential for damages. Refer to section 1.5 for discussion related to risk.

3.2.3.5 *Access Routes:* Driveways, walkways, and local roadways essential to ingress and egress should be 0.15 metres above the 100-year monthly mean water level, or 0.3 metres below the Regulatory Flood Level, whichever is greater. Provision for "dry" (no surface ponding) access routes above the regulatory level shall be provided for institutional buildings servicing the sick, elderly, young or disabled, or essential public services.

V2 The practitioner should also consider Municipal, County and Provincial roadway standards / requirements that may be applicable. V2

3.2.3.6 *Overland Flow Routes:* Failure to plan for a major system can result in flood damage. The dual drainage concept reinforces the need for proper major system design to ensure that there is an overland flow route with sufficient capacity to convey flows to a stormwater management facility, and/or directly to a sufficient outlet. Overland flow needs to be considered carefully to ensure that the major system does not inadvertently convey flows to an existing low point at an unknown location. When overland flow routing is achieved via roadways, road grading shall limit intermediate high points to no more than 0.25 metres (preferably 0.20 metres) to allow for overflow depth.

3.2.4 Inlet Capacity

3.2.4.1 *Inlet Capture:* Under typical conditions, the practitioner shall ensure that sufficient inlet capacity is available to capture the storm sewer design flows.

3.2.4.2 *Inlet Controls:* In certain situations, there may be merit in implementing catch basin inlet controls to limit inflow to the storm and/or combined sewer system. This approach can mitigate sewer surcharging conditions, attenuate peak flows and maintain a lower hydraulic grade line under larger, less frequent storm events. There are also drawbacks to consider, such as maintenance/operation concerns and a lower level of service in the form of more frequent surface ponding and a less efficient/convenient drainage system. The suitability of inlet controls should be discussed during **pre-consultation** with the Conservation Authority and the Municipality.

3.3 STORMWATER QUANTITY CONTROL

Stormwater quantity control can be described as temporary storage of runoff in ponds, depressions or underground pipes/structures. Quantity control can also be achieved by infiltration measures, although not typically practiced in the Windsor/Essex region due to predominately clay soils. Infiltration measures are generally ineffective for most of the region, unless a pervious soil stratum is artificially created. Refer to section 3.8 for additional discussion related to low impact development controls that promote infiltration measures as well as other volume reduction measures.

Stormwater quantity control is often necessary to mitigate impacts of urbanization and resulting increase runoff peak flow. Higher runoff peak flow from developments is detained and released at a controlled rate that the receiving watercourse or storm sewer can convey without adverse impacts.

3.3.1 Allowable Release Rate

3.3.1.1 *Watershed Study:* Ideally, the practitioner shall refer to the appropriate watershed planning study or drainage plan which should prescribe an allowable release rate for the watershed or subcatchments thereof. Hydrologic/hydraulic studies at the watershed scale should evaluate stormwater quantity control alternatives and determine the optimum balance of conveyance capacity versus detention requirements. Allowable release rates should always be prescribed on a **flow rate per hectare basis.**

3.3.1.2 *Pre-consultation:* In the absence of watershed planning studies or drainage plans, **pre-consultation with CA and municipalities is mandatory** to discuss and confirm an appropriate allowable release rate. The Municipality and/or Conservation Authority has the discretion to mandate a specific allowable release rate or to rely on the practitioner to determine an appropriate rate.

3.3.1.3 *Hydraulic Capacity Assessment:* It is recommended that watersheds adopt an allowable release rate based on the hydraulic capacity of the receiver(s). This approach relies upon a hydraulic analysis, which is objective and relatively certain or finite whereas an estimation of pre-development peak flow requires a hydrologic analysis which can be subjective, uncertain and variable based on a range of hydrologic parameters, buildout conditions and methods that can be used.

[V2] When assessing the capacity of the receiver(s), it should be understood that this is not limited to the immediate receiver, but also includes all downstream drainage features leading to a sufficient outlet (i.e., a location where the water level or flow rates will not damage downstream lands or roads – e.g., Detroit River). V2

3.3.1.4 *Hydrologic/Agricultural Discharge Rates:* When the hydraulic capacity assessment of the receiver(s) is deemed impractical, the Municipality and/or Conservation Authority may accept that the allowable release rate be determined based on;

1) V2 a hydrologic analysis V2 with due consideration to the supplementary information provided in **Appendix A** or;

2) a specified agricultural Drainage Coefficient used with the following discharge equation;

Eq. 3.3.1.4: Discharge (L/s) $= 0.116\ x\ Area\ (ha)\ x\ Drainage\ Coefficient\ (mm/day)$

In the absence of site-specific Drainage Coefficients, the following values are recommended:

- For the RVC_T 32mm Storm as defined in section 3.4.1.3:
  Maximum of 25 mm/day
- For storms exceeding the RVC_T up to the 100-year Storm:
  Maximum of 50 mm/day

V2 The manual previously recommended that a hydrologic analysis using the SCS Type II distribution be used to determine the allowable release rate for a site. The rationale for this recommendation was that the commonly used 'SWMM RUNOFF' modelling routine does not capture the observed 'sponge effect' in our region whereby soils will hold a significant amount of rain and significantly dampen high intensity rainfall. Thus, by limiting the rainfall intensity to the SCS Type II distribution (i.e., low intensity storm), the unrealistic peaky runoff response modelled from a peaky design storm input (i.e., Chicago storm) was mitigated.

However, recent watershed studies in the region have shown that when the SWMM hydrologic approach includes both the RUNOFF and GROUNDWATER routines, the latter allows the model to capture the 'sponge effect' and thus appropriately captures the dampening effect that undeveloped lands have on high intensity Chicago storms. Alternatively, the empirically derived SCS hydrograph with appropriate peaking factor for our region can also reasonably mimic the observed low agricultural runoff response when applying peaky Chicago storms into the model.

Notwithstanding the above, the objective of this subsection is to provide guidance to reasonably estimate the design flow for which the receiver was designed to convey. As an example, municipal drains are often designed to convey a flow based on a 2-year design return period. In this instance, the hydrologic analysis would estimate flows based on a 2-year storm.

In addition to modelling analysis, the Rational Method could also be used to estimate flows. However, it should be acknowledged that this method provides an apparent simplicity by lumping many hydrologic parameters into two variables:

- rainfall intensity (as a function of time of concentration); and,
- runoff coefficient (a single value to estimate the proportion of rainfall the becomes runoff).

Thus, careful consideration should be made in selecting these variables to reasonably mimic the attenuated runoff response from typical flat agricultural lands in our region. V2

3.3.2 Storage Requirements

To determine storage facility storage volume requirements, a practitioner must evaluate inflow versus outflow. A facility with gravity type flow controls (weir, orifice, pipe) will have a varying outflow rate based on the varying levels in the pond and the varying levels of the receiving drain/storm sewer. In our region with relatively flat lands, it is common to expect that the varying drain levels will create a backwater condition that effectively reduces the facility's outflow to some undetermined amount and for some undetermined period of time. Often times, the amount and duration are difficult to determine. V2 **Appendix D** provides guidance on how to reasonably account for backwater conditions in flow control design.

Where significant and prolonged backwater conditions are expected, a prudent approach is to assume a constant high backwater level. In instances where a reliable outflow cannot be achieved and/or the consequence of exceedance of the storage facility is high, the application of enhanced storage volume requirements would be appropriate. Refer to **Appendix D** for additional discussion related to storage requirements. V2

3.3.2.1 V2 *Enhanced Storage Requirements:* Enhanced storage requirements assume no release rate (i.e., a discharge rate of zero). The enhanced V2 100-year design storage volume to be provided is to be equivalent to the specified storage depth of runoff multiplied by the total catchment area. The specified storage depth varies based upon the Hydrologic Soil Group and impervious level and is to be calculated from the appropriate equation below. See **Appendix A** for details on the parameters and method used to calculate/develop the equations below.

! Designers shall refer to the discussion under Section 3.1 before using these equations.

For Hydrologic Soil Group A:

Eq. 3.3.2.1a: Storage Depth (mm) = $11 + 0.95x$ where x = impervious %, > 50%

For Hydrologic Soil Group B:

Eq. 3.3.2.1b: Storage Depth (mm) = $12 + 0.94x$ where x = impervious %, > 50%

For Hydrologic Soil Group C:

Eq. 3.3.2.1c: Storage Depth (mm) = $50 + 0.56x$ where x = impervious %, > 50%

For Hydrologic Soil Group D:

Eq. 3.3.2.1d: Storage Depth (mm) = $72 + 0.33x$ where x = impervious %, > 50%

V2 Using Storage Equations for Stress Test Volume Estimations:

Provided that the site could provide a reliable average release rate of 5 L/s/ha or larger, the storage depths calculated using the storage equations above would provide sufficient storage to contain the stress test runoff volume. Refer to **Appendix D** for discussion supporting this design guidance. V2

3.3.2.2 V2 *Normal Storage Requirements:* Where a reliable outflow can be expected (i.e., pumped outflow with backup power, backwater conditions are not present or have been reasonably accounted for), then the storage volume requirements can be determined with due consideration to the varying outflow rate in response to varying head differential (i.e., difference between headwater and tailwater levels) across the flow control element(s). V2

V2 **In theory**, a detailed hydrodynamic model can compute a hydrograph that measures the variation of both headwater (HW) and tailwater (TW) levels over time. While a model can predictably measure varying head differential conditions for the site, it is important to acknowledge that it only does this for the synthetic design storms that are typically simulated in the model. Thus, the tailwater condition in the model represents one or few rainfall distributions and soil conditions.

In practice, there are infinite spatial and temporal variations of rainfall as well as variable antecedent soil conditions that affect runoff and corresponding tailwater conditions. In many instances, simplifying assumptions can be made to deduce the complex and highly variable head differential down to a reasonable approximation of a single design head best fits the expected operating range of the flow control element.

Refer to **Appendix D** for guidance related to flow control design framework and practical examples. V2

Storage volume requirements are to be determined based on the most critical of the 100-year design storms as discussed in section 3.7.8 and tabulated in **Appendix B**.

V2 Use of Modified Rationale Method:

The Modified Rational Method (MRM) could potentially be used for storage volume calculations of specific sites up to 5 hectares provided that the practitioner can justify the assumption of a constant release rate. Refer to **Appendix D** for additional discussion regarding the proper use of the MRM. Where the MRM is deemed acceptable, the 100-year Runoff Coefficient (100-year C value) shall be based on the following equation; V2

Eq. 3.3.2.2: 100-year C value = $\frac{\text{Storage } Depth\ (from\ Section\ 3.3.2.1)}{108\ mm\ (100\ Year\ 24\ hour\ Rainfall)}$

Pre-consultation with the Municipality and Conservation Authority is mandatory to review V2 and approve the proposed use of the MRM V2 and the proposed design outflow.

V2 The stress test is simply the addition of 42mm rainfall spread over 24 hours and added to the design storm. This equates to a uniform increase in rainfall of 1.75mm/hr. Thus, a simple approach to estimate **additional** runoff from the Stress Test (i.e., Stress Test runoff volume less 100-year runoff volume) could be done using the following equation. V2

Eq. 3.3.2.2b: Stress Test Additional Runoff Volume (m^3) = 1.75 mm/hr x MRM Critical Duration (hrs) x Area (ha) x 10 (convert mm to m and ha to m^2)

3.3.2.3 *Minimum Freeboard Depth:* A minimum freeboard depth – as measured from the 100-year design high water level to the lowest building opening – should be at least 0.3 metres.

3.3.2.4 *Acceptable Risk:* The minimum freeboard depth requirement in the preceding section is a floodproofing measure based on a minimum standard level of service, which has been defined herein as a 100-year design storm. Refer to section 1.5 for further discussion on level of service and risk.

Where an individual site's potential damages due to flooding are high, it is the practitioner's responsibility to design to a more conservative standard or to provide a sufficient emergency flow route in accordance with the proponent's

site-specific needs. The Municipality and/or Conservation Authority may also, at their discretion, require a larger freeboard depth or other safeguards to minimize risk where appropriate (e.g., pond immediately adjacent to residential homes without a sufficient emergency flow route).

3.3.2.5 *Rooftop Storage:* Rooftop storage is not permitted due to lack of municipal control over the practice. Green roof infrastructure may be acceptable with supporting maintenance agreement and restrictive covenant with owner to prevent alteration to system. However, while green roof infrastructure may be acceptable and even encouraged where appropriate, the available storage capacity of any rooftop system will not be accounted for in the required available storage for any particular site/development since maintenance and prevention of alterations to the system cannot be guaranteed even with agreements in place.

3.3.2.6 *Parking Lot Storage:* Surface ponding on parking lots is prohibited for the first 32mm rainfall – defined as the RVC_T under section 3.8 and the Water Quality Storm (WQS) in **Appendix B**. All events up to the WQS shall be stored by stormwater practices other than parking lot surface storage (e.g. underground storage, surface swales/ponds, rain gardens, etc.). For storms exceeding the WQS, surface ponding on parking lots may be acceptable up to a maximum depth of 0.30 metres. Lower depths or "dry" (no surface ponding) may be warranted for institutional access or industrial operations. Refer to section 3.2.3.5 for access route standards.

3.3.2.7 *Hybrid Detention Approach:* A hybrid detention approach accounts for both on-site and regional detention, which is commonly implemented for commercial/ industrial developments. At a minimum, this [V2] manual [V2] recommends that at least 50% impervious be accounted for routing and regional storage design. This minimum is based on the assumptions and rationale discussed in **Appendix A**. A greater impervious level may be warranted for site-specific building coverages and surface ponding constraints.

For this approach to be successful, the stormwater management plan must clearly define the flow control rate and storage volume required for the individual sites on a <u>per hectare basis (e.g., L/s/ha and m^3/ha)</u>. The stormwater plan shall also provide control elevations for buildings, roadways and overall property limits to ensure that runoff is contained to the overall site and that any overflow from on-site storage is directed via major system flow routes towards the regional facility.

3.3.3 Peak Flow Timing Issues

The implementation of detention storage to mitigate increased flow from urbanization can have a significant impact on peak flow. As urbanization increases, timing effects and superposition of prolonged outflows from detention facilities can have a cumulative impact on downstream discharge.

3.3.3.1 For smaller watersheds with a large proportion of existing or planned urbanization, a simple approach to deal with timing issues is to ignore any lag in flow routing throughout the watershed. In other words, peak outflows from the to-be developed subcatchments within the watershed should be assumed to coincide and sum up to an overall peak flow conveyed by the receiver. (See **Appendix A** for supplemental information)

3.3.3.2 For larger watersheds or watersheds with limited urbanization, it may be appropriate to account for basin lag and timing effects on overall peak flow. However, the practitioner and Municipality should have a clear understanding of the potential impact of future development on the watershed. (See **Appendix A** for supplemental information)

3.3.4 Volume Mitigation Issues

Development adds impervious surface, thus reducing infiltration and evapo-transpiration and increasing runoff from a given storm event. The additional runoff volume is typically addressed by detention storage which allows the development to maintain its pre-development release rate. However, the additional runoff volume from development increases flow duration which can lead to erosion and/or sedimentation problems downstream. (See **Appendix A** for supplemental information)

3.3.4.1 To the extent that is practical, stormwater management controls shall endeavor to reduce runoff volume created by development. Reference section 3.8 for guidance in this regard.

3.3.4.2 Increased volume can create or exacerbate flooding issues on pumped systems. The practitioner shall evaluate the potential impacts of additional volume on pumped systems. (See **Appendix A** for supplemental information)

3.3.5 Other Design Considerations

3.3.5.1 *Provisional Storage:* With uncertainty regarding potential future increases to the 100-year design storm due to climate change, it would be prudent for practitioners, municipalities and developers to consider provisions for potential future stormwater facility expansions to account for future increases in storage requirements.

3.3.5.2 *Multi-Use Facilities:* When applicable, consideration should be given to multi-use facilities such as depressed park areas that provide stormwater storage during infrequent flood events yet serve as recreational lands for the majority of the time. At a minimum, surface ponding in parkland should be limited to storms exceeding the minor 5-year storm, or greater, at the discretion of the Municipality.

3.3.5.3 *Outfall Conditions:* Outfall conditions are often an important factor in this region's stormwater design. It is difficult to determine the joint probability of both extreme rainfall and high lake levels (i.e., it is unknown what the probability of occurrence would be for both a 100-year storm event and concurrent 100-year lake level). Thus, designing to a specified level of service can vary significantly based on assumed lake/river levels.

To provide a consistent minimum standard, this manual recommends that outfall conditions be determined from maximum monthly mean levels based on annual maximums from 1918 to present. The minimum return period shall be selected based on the consequence of failure or capacity exceedance definitions in Section 1.5.2 and **Table 3.3.5.3** below. Designer to refer to discussion under Section 3.0 before using these values.

! **Table 3.3.5.3 – Outfall Condition Minimum Return Periods**

Consequence	Minimum Return Period (Years)
Low	5
Medium	10
High	25

V2 Refer to **Appendix D** for historical annual maximum monthly mean levels from 1918 to 2022 (inclusive) as well as lake and river station chart datums. V2

3.3.5.4 *Orifice Controls:* Past experience has shown that orifice plates used for flow control have, in some instances, been removed to eliminate the nuisance caused by frequent surface ponding. It is recommended that a short pipe section (2-3 times the orifice diameter) be used in lieu of orifice plate to mitigate the potential for tampering.

3.3.5.5 *Orifice Sizing:* Orifice diameters less than 100mm shall be only be permitted with proper protection against clogging, such as a perforated riser pipe and filtration measures to protect the orifice from debris. Alternatively, inlet control devices can be used in lieu of small orifice diameters to restrict low flows.

3.3.5.6 ^{V2} *Outfall Sewer Clearance Depth:* To the extent practical, a storm sewer outfall should be at least 0.3m above the bottom of a receiving watercourse. A lesser clearance depth may be acceptable to avoid pumping or to achieve minimum cover requirements, with the understanding that more frequent inspection and maintenance may be required to mitigate potential blockage. $_{V2}$

3.3.5.7 ^{V2} *Ice Thickness for Submerged Inlets/Outlets:* Ice thickness is typically not a concern for stormwater management pond inlets in the region. However, in instances where ice thickness is a concern, the MOE guidance and local historical climate data can be relied upon to easily estimate ice thickness. Refer to **Appendix D** for details. $_{V2}$

3.3.5.8 ^{V2} *Uncontrolled Areas:* When new development requiring stormwater management cannot practically collect and control all runoff from the site (e.g., a strip of grassed area adjacent to a roadway boulevard), consideration could be given to allowing a small area to runoff without control. In this instance, the uncontrolled flow should be subtracted from the allowable release rate determined for the overall site area and the resulting reduced rate should be used as the control rate for the remainder of the site. $_{V2}$

3.3.5.9 ^{V2} *Interim SWM Plan:* When preparing a SWM plan and/or undertaking a SWM design, there may be warrants for the practitioner to consider both interim and ultimate conditions. For example, an existing drain with limited capacity may require a small allowable release rate for a proposed development, which could be significantly smaller than a future (ultimate) allowable release rate based on planned future drain improvements with increased capacity. In this instance, the design should consider an interim SWM plan based on the existing allowable rate and an ultimate SWM plan based on the future (ultimate) allowable rate.

3.4 STORMWATER QUALITY CONTROL

Where stormwater is sometimes viewed as being as clean as rainwater, it can carry significant pollutants and have a significant negative impact on receiving watercourses. To mitigate adverse impacts from development, stormwater quality controls are applied – commonly referred to as *Stormwater Management Practices* (SWMPs). This section discusses the quality objectives and requirements for the region.

3.4.1 Standard Quality Objectives

3.4.1.1 As a minimum standard of quality control, suspended solid removal via settling, filtration or hydrodynamic separation is required. Surface water quality objectives and land use are to be considered when evaluating the potential impact of development on the receiving watercourse. **Pre-consultation** with the CA and the Municipality is required to identify any specific water quality objectives for the watershed and receiving watercourse(s) in question.

3.4.1.2 The MECP provides specific water quality storage requirements based on receiving waters as outlined in Table 3.2 of their 2003 SWM manual. The minimum standard protection level is "**Normal**" for our region, which is generally suitable where a stable downstream habitat has adapted to moderate sediment loading – a typical condition in our region due to extensive long-term agricultural practices. However, site-specific conditions may require "Enhanced" protection. **Pre-consultation** with the CA and the Municipality is necessary to confirm the protection level requirements.

3.4.1.3 The MECP's proposed Low Impact Development (LID) Stormwater Management Guidance Manual – Draft Version 2.0 dated November 2017 specifies a Runoff Volume Control Target (RVC_T) of 32mm for our region based upon the 90th percentile rainfall event. This storm is representative of a frequent event to be retained on-site or captured and treated prior to release. The RVC_T shall be distributed based on a Chicago 2-year 4-hour storm as defined in **Appendix B** with time interval per **Graph 3.7.8.1**.

3.4.1.4 For all stormwater Best Management Practices (BMPs), the proposed MECP guidance requires that 90% (RVC_T) of the total runoff volume be captured and treated, while maintaining an overall removal efficiency of 70% for normal protection. For enhanced protection, the overall removal efficiency shall be at least 80%. The treatment efficiency is based on long-term average suspended solids removal based on a typical particle size distribution provided in **Table 3.4.1.4** below. Site-specific particle size distributions may be required to suit individual site characteristics.

! Table 3.4.1.4 – Typical Particle Size Distribution

MOE 1994 Particle Size Distribution	
Particle Size (µm)	**% of Distribution**
< 20	20
20 – 40	10
40 – 60	10
60 – 130	20
130 – 400	20
400 – 4000	20

3.4.1.5 The OGS manufacturer shall measure Total Suspended Solids (TSS) removal efficiency based on the rainfall data provided in **Table 3.4.1.5** below. See **Appendix A** for supplemental information in reference to this section.

! Table 3.4.1.5 – Rainfall Intensity / Rainfall Volume Relationship

Rainfall	% of Total	Rainfall	% of Total	Rainfall	% of Total
mm/hr	**Volume**	**mm/hr**	**Volume**	**mm/hr**	**Volume**
2	N/A	9	3.6%	30	4.6%
3	13.2%	10	3.2%	35	3.8%
4	9.6%	11	2.8%	40	2.9%
5	7.5%	12	2.5%	45	2.4%
6	6.0%	15	6.6%	50	1.8%
7	4.8%	20	8.3%	>50	6.6%
8	4.1%	25	5.8%		

3.4.2 Customized Quality Objectives

Pre-consultation with the Conservation Authority and the Municipality to review of any applicable watershed planning studies, Source Protection Plans, etc. should be undertaken to identify any specific quality objectives for the development. Where discharges are close to a beach, a domestic water supply intake, an environmentally sensitive area or an area of concern, customized quality objectives may be required to target specific pollutants. For example, gas stations will require oil/grease and spill containment in addition to suspended solids removal. Areas where high nutrient loadings are of concern will require additional phosphorus removal measures. Industrial sites may require filtration measures to target specific metals, etc.

3.4.3 Other Design Considerations

Water Quality Treatment Units: Water quality units shall be selected from technologies which have been verified by the Canadian Environmental Technology Verification (ETV) program. A listing of Current Verified Technologies can be found at; http://etvcanada.ca/home/verify-your-technology/current-verified-technologies/

3.5 IN-STREAM EROSION CONTROL

Many watercourses within the region have relatively flat gradients and correspondingly low flow velocities whereby in-stream erosion is not expected to be a prominent issue. Nonetheless, increased flow duration from extended detention may have a cumulative impact on the receiver and eventually lead to erosion issues.

3.5.1.1 In the absence of watershed specific erosion control requirements, a minimum 24-hour detention of the 32mm RVC_T or water quality storm (WQS) is recommended as erosion control. Refer to **Appendix B** for the recommended WQS distribution. This standard will apply to most watercourses in the region and is typically easily achieved by virtue of the relatively low allowable release rate to the receiving watercourse.

3.5.1.2 Specific watercourses in the Windsor/Essex region may require more detailed evaluations of erosive index, erosion potential, tractive force or velocity-duration data and continuous modelling. Specific watercourses should be identified by the CA and municipalities and confirmed during **pre-consultation** or added to this manual.

3.6 STORMWATER BEST MANAGEMENT PRACTICES

Stormwater best management practices are extensively covered in the MECP guidance provided by the 2003 Stormwater Management Planning and Design Manual as well as the Draft No.2 of Low Impact Development (LID) Stormwater Management Guidance Manual dated November 27, 2017 and numerous supporting resources list therein. This section outlines a few key ^{V2} design criteria $_{V2}$ to consider in conjunction with those of the MECP guidance.

3.6.1.1 *Ease of Access:* SWM facility design shall include safe maintenance access and operation considerations. Access roads are required to all inlets, outlets, spillways and sediment forebay.

3.6.1.2 *Rooftop Storage:* Rooftop storage is not permitted, except for green roofs with conditions as outlined in section 3.3.2.5.

3.6.1.3 *Pond Grading:* Side slopes shall be no steeper than 6:1 slope within 3.0m on either side of the normal water level (NWL). Average slope from NWL to top of bank shall be no steeper than 5:1 (i.e., terraced grading combining both 3:1 and 7:1 is acceptable, provided it is outside of the 3.0m buffer surrounding the NWL as prescribed above).

3.6.1.4 *SWM Facility Inlets:* Inlet pipe inverts shall be set to the NWL or higher. Where there is a preference to submerged inlets to the facility, the last section of pipe only (i.e., pipe length from inlet manhole to waterbody) can be dropped below the NWL provided that the obvert of the pipe is set below the maximum anticipated thickness of ice.

3.6.1.5 *SWM Facility Minor Storm HGL:* Where the downstream receiver is a SWM storage facility, the minor storm water level shall not exceed the inflow sewer obvert (i.e., shall not create a backwater condition on the minor storm system). V2 This section is intended to limit backwater conditions from the SWM facility to infrequent storms exceeding the typical 5-year minor design storm. This is in keeping with the preferred level of service described in Section 3.2.2.5, where the 5-year HGL does not exceed the sewer obvert throughout the sewer system. The aim of this section is to minimize the frequency whereby water levels in the sewer will surcharge above private drain connections. V2

3.6.1.6 *Safety to Public:* Warning signage should be considered by the Municipality at pond access points to advise the public of the pond's function. An example warning sign is presented in Appendix C as referenced from the City of Pickering Standard Drawing P-1007.

3.6.1.7 *Anti-seepage Collars*: Anti-seepage collars or other approved impervious plug shall be installed on all outlet pipes or as directed by a geotechnical engineer.

3.6.1.8 *Sediment Drying Area:* A sediment drying area shall be designated for ease of future maintenance. The area should be sized for a minimum 10 years of estimated sediment accumulation assuming a height of 1.5m and slope of 5:1.

3.7 HYDROLOGIC AND HYDRAULIC ANALYSIS

Hydraulic analysis is relatively accurate when compared to hydrology. Pipe sizes are finite, drain sections are measurable and hydraulic capacity and grade calculations should not vary to a significant degree, if at all, from one practitioner to the next. Hydrologic analysis however is an inaccurate science that can vary tremendously. This section is intended to provide some consistency to the methodology and parameters used to perform hydrologic analysis in the region.

3.7.1 Use of Computer Programs

There are modelling computer programs that are hydrologic (i.e., measure how much rainfall becomes runoff and how often runoff occurs) and some that are hydraulic (i.e. measure how high water levels will rise and how fast stormwater drainage features can convey runoff). Many models are both hydrologic and hydraulic.

A computer model is a decision support tool. A model can analyze hydrologic and hydraulic conditions for various land uses and buildout scenarios however it cannot make decisions. Modelers should consider the level of detail required to make an informed decision. A higher level of detail should be driven by the need and benefit of achieving more reliable model results.

3.7.1.1 *Model Reliability:* There is a general tendency to view model results as inherently accurate. This may in part be due to the level of computational precision displayed by model results (e.g., the peak flow is calculated to be 1,219.852 L/s or the storage volume required is 15,938.149 m^3). While modeling software can certainly have a sound mathematical basis and perform complex algorithms, the "accuracy", or more perhaps more aptly defined "reliability", of the model output is a function of the user's skill and knowledge of the model software, which is relied upon to input parameters that will replicate actual conditions as closely as possible. Regardless of the user's skill and experience, the model is an estimation that does not warrant results to three decimal places. This manual recommends de-emphasizing precision and promoting better reliability.

The reliability of model output depends on the quality of the input data and the judgment of the modeler in making critical assumptions. When model inputs and assumptions have high levels of uncertainty, the results should be viewed with the same level of uncertainty. Performing reliable hydrologic modeling can be a challenge. Without gauged data to calibrate the model, the reliability of the model relies heavily on experience and professional judgment.

3.7.1.2 *Model Calibration:* This manual strongly recommends obtaining gauged data to assist modelers in building reliable models that can be relied upon as representing actual conditions. In this regard, it is recommended that a continuous gauged data program be implemented on a regional scale in cooperation between the Conservation Authorities and the various municipalities. An unreliable model can significantly over-estimate or under-estimate infrastructure needs, resulting in much greater costs (capital costs or damages) than the cost of collecting gauged data.

Stormwater modelling reference materials unanimously emphasize the need for calibrating and validating models to reliably reflect actual conditions. Even complex and detailed models can generate different results for the same project based on minor variations in model inputs.

The following are general guidelines for calibration/validation:

1. If gauged data is available, hydrologic parameters can be calibrated to fit observed data. It is typically a good idea to first match flow volumes, then match peaks and timing.
2. After changing hydrologic parameters to fit observed data from specified calibration events, the results should be checked (validated) against events not used in the calibration process.
3. Adjustments to hydrologic parameters should be limited to a reasonable range.

V2 Refer to **Appendix D** for guidance in estimating flows for large agricultural watersheds. V2

3.7.2 Runoff Estimation Methods

3.7.2.1 *Rational Method:* The Rational Method is most widely used in runoff estimation due to its simplicity. This method was derived for peak flow estimation and should only be used as such within the limitations of section 3.2.2.4. The Modified Rational Method is not acceptable for estimating storage, except as specified in section 3.3.2.2. See **Appendix A** for further discussion regarding the Rational Method.

3.7.2.2 *Unit Hydrograph Methods:* A unit hydrograph represents the runoff response of the drainage basin. There are many unit hydrographs methods that have been derived from gauged basins to correlate hydrograph parameters (peak flow, time to peak, recession limb) to basin characteristics (area, slope, roughness). While this manual does not seek to identify preference to a particular method, <u>it requires that the practitioner understand the relationships between the derived hydrograph and basin characteristics and whether those relationships are applicable and transferable to the basin being analyzed.</u>

For example: The standard SCS unit hydrograph is based on “rolling hills” topography and a corresponding short recession limb equal to 1.67 times the time to peak, which is certainly not the case in this region. In many areas within our region, the typical SCS peaking factor based on “rolling hills” would require adjustment from the default 484 (US units) conversion factor to account for flatter lands, as well as a corresponding lengthening of the recession limb. Suggested peaking factor and limb ratio values are presented in **Table 3.7.2.2** below.

! Table 3.7.2.2 – Suggested Peaking Factor and Limb Ratio

General Description	Peaking Factor	Limb Ratio (Recession to Rising)
Urban areas; steep slopes	575	1.25
Typical SCS	484	1.67
Mixed urban/rural	400	2.25
Rural, rolling hills	300	3.33
Rural, slight slopes	200	5.5
Rural, very flat	100	12

V2 When using the SCS unit hydrograph method and associated Peaking Factors (PF) as suggested in **Table 3.7.2.2** above, the following provides a suggested range of slopes to match the general descriptions:

- Very Flat = 0.5% or less (PF = 100)
- Flat = 0.5 to 2% (PF = 150)
- Slight Slopes = 2 to 5% (PF = 200)
- Rolling Terrain = 5 to 10% (PF = 300)

Recent hydrologic and hydraulic studies of very flat agricultural lands within the region have found that using the SCS unit hydrograph method with PF = 100 has provided very good results compared to observed data and other proven hydrologic methods. V2

3.7.2.3 *Kinematic Wave Model:* The kinematic wave model represents a more physical based approach to runoff estimation based on the application of fundamental laws of conservation of mass and momentum to describe free-surface flow over an idealized plane surface. While modeling efforts require more intensive inputs to sufficiently define the physical drainage characteristics of the watershed, this method provides a more accurate estimation of the actual runoff response in an ungauged watershed.

Care must be taken when defining the level of detail required for subcatchment delineation. As subcatchment size and flow lengths increase, the assumption of uniform sheet flow over a plane surface becomes less representative of actual sheet flow that concentrates into surface depressions and shallow flow pathways. In this case, the model assumption can lead to over-estimation of infiltration.

3.7.3 Time of Concentration

Time of concentration is defined as the travel time of runoff from the most hydraulically remote point in the contributing area to the specific outlet point of interest. Overland or sheet flow occurs in upper reaches of the contributing area over a short distance (typically in the range of 30m to 130m). Beyond this distance, flow tends to concentrate in rills and gullies as shallow concentrated flow which conveys flows to defined open channels or pipes as concentrated flow. Thus, time of concentration estimates are typically a sum of these three components as summarized by **Equation 3.7.3** below;

Eq. 3.7.3: Time of Concentration = $t_{sheet} + t_{shallow} + t_{concentrated}$

3.7.3.1 *Overland or Sheet Flow:* Overland flow travel time is commonly estimated using a version of the kinematic wave equation, a derivative of Manning's equation, given as Equation 3.7.3.1 below;

Eq. 3.7.3.1: t_{sheet} (min.) = $\dfrac{6.92\ L^{0.6}\ n^{0.6}}{I^{0.4}\ S^{0.3}}$

where: L = overland flow length, metres
n = Manning's roughness coefficient, see Table 3.7.4.1
I = rainfall rate, mm/hr (Eq. 3.2.1.1)
S = average slope m/m

V2 The above equation is iterative and requires an assumed time of concentration to calculate rainfall rate (I). The calculation is repeated with until the assumed time of concentration is equal to the resulting t_{sheet} value. V2

3.7.3.2 *Shallow Concentrated Flow:* Shallow concentrated flow travel time can be estimated using a relationship between velocity and slope as shown in Equation 3.7.3.2 below;

Eq. 3.7.3.2: $t_{shallow}$ (min.) = $\dfrac{L}{60\ k\ S^{0.5}}$

where: L = shallow flow length, metres
k = intercept coefficient, see Table 3.7.3.2
S = slope, %

Table 3.7.3.2 – Typical Intercept Coefficients for Eq. 3.7.3.2

Land Cover/Flow Regime	k
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213

Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow); alluvial fans in western mountain regions	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area (shallow concentrated flow); small upland gullies	0.619

3.7.3.3 *Concentrated or Channel Flow:* Concentrated flow travel time in open channels or pipes can be estimated using Manning's Equation to calculate average flow velocity. The travel time is estimated using Equation 3.7.3.3 below;

Eq. 3.7.3.3: $t_{concentrated}$ (min.) = $\frac{L}{60\ V}$

where: L = concentrated flow length, metres
V = Manning's velocity, m/s

3.7.4 Overland Flow Roughness

3.7.4.1 *Manning's Roughness Coefficients:* **Table 3.7.4.1** below provides typical roughness coefficients for hydrologic computations. For cultivated soils, the residue cover has a significant impact on the roughness coefficient and ultimately on the runoff response of the catchment. In the absence of gauged data to calibrate this parameter, it is suggested that conservative values be used in analysis.

! Table 3.7.4.1 – Typical Manning's Roughness Coefficients for Overland Flow

Surface	n
Smooth Asphalt/Concrete	0.013
Cultivated Soils - Residue Cover < 20%	0.06
Cultivated Soils - Residue Cover > 20%	0.17
Range (natural)	0.13
Grass - Short Prairie	0.15
Grass - Dense	0.24
Woods - Light Underbrush	0.40
Woods - Dense Underbrush	0.80

3.7.5 Impervious Level

3.7.5.1 Impervious percentages shall generally follow **Table 3.7.5.1** as minimum design values to be used. A reduction to the impervious level shall not be made to account for disconnected roofs. (See section A-3.2.2.7 of **Appendix A** for further discussion)

! Table 3.7.5.1 – Minimum Impervious Percentage

Land Use	Imp %
Residential – Single family	60
Residential – Single family (lot size 500 m² or less)	70
Residential – Semi-detached	70
Residential – Townhouse / Row housing	80
Industrial / Commercial	90
V2 Wet Pond	100

3.7.6 Depression Storage

3.7.6.1 Depression storage is defined as excess water which ponds on the land surface when the rainfall intensity exceeds the infiltration capacity of the soil during a storm event. The depression storage capacity of a drainage basin is usually expressed in terms of an equivalent average depth over the basin. Typical depression storage values are presented in **Table 3.7.6.1** below.

! Table 3.7.6.1 – Typical Depression Storage Depths

Land Cover	Storage Depth (mm)
Paved area	2.5
Flat roofs	2.5
Lawn	7.5
Wooded area	10.0
Open field	10.0

Given the flat topography in the region, depression storage may be an important model calibration parameter to adjust runoff volume. It would be reasonable to assume that some very flat areas within the region could store more than the typical values shown above. V2 Typical depths are usually recommended for new development; however, larger depths may be warranted when it is important to capture additional storage due to very flat topography, lack of subsurface drainage and vegetation cover. **Appendix D** provides some general guidance where larger values may be warranted. V2

3.7.7 Infiltration Losses

The ability for rainfall to infiltrate into the soil is a function of surface infiltration, soil porosity as well as the underlying soil percolation rate. Runoff occurs when either infiltration capacity or soil porosity is exceeded.

3.7.7.1 *Antecedent Moisture Conditions:* Infiltration parameters can vary depending on the type of antecedent moisture conditions (AMC). When performing continuous modelling, the infiltration parameters should be based on dry conditions given that the model will account for infiltration capacity loss during rainfall events and infiltration capacity recovery during inter-event periods. <u>For single event modelling, infiltration parameters should be based on dry conditions for minor system design and normal antecedent conditions for major system design.</u>

3.7.7.2 *Saturated Hydraulic Conductivity:* The Green-Ampt method's saturated hydraulic conductivity (Ks) parameter and the Horton method's minimum infiltration rate parameter (fmin) essentially represent the same value. There are numerous references and sometimes significant variability from one reference to the next. For consistency, this manual provides recommended values according to Musgrave (1955), which provides an upper and lower range for each soil group. Depending on the texture of the soil, the value represents the upper, middle or lower value within the range.

3.7.7.3 *Green-Ampt Method:* The Green-Ampt method is a theoretical based method that approximates the physical nature of infiltration losses. Typical Green-Ampt infiltration parameters are presented in **Table 3.7.7.3** below. **Appendix A** includes a table showing all soil types in the region as well as the corresponding texture and hydrologic group.

! **Table 3.7.7.3 – Typical Green-Ampt Infiltration Parameters**

Parameter	Hydrologic Group			
	A	B	C	D
Su (mm)	100	300	250	180
V2 Ks, dry (mm/hr)	25	13	5	3
Ks, normal – clay	7.6	3.8	1.3	0.5
Ks, normal - loam	9.5	5.7	2.5	1.0
Ks, normal - sand	11.4	7.6	3.8	1.3
IMD, dry (fraction)	0.34	0.32	0.26	0.21
IMD, normal (fraction)	0.17	0.16	0.13	0.10

V2 Infiltration parameters should be based on normal AMC for major system design and dry AMC for minor system design and stress test assessments. The values in **Table 3.7.7.3** are not intended to be strictly followed for applications such as model calibration. Refer to **Appendix D** for additional discussion. V2

3.7.7.4 *NRCS (SCS) Curve Number Method:* The curve number method has limitations as explained in **Appendix A** and should be used only as deemed appropriate by an experienced practitioner with a sound understanding of the methodology and its noted limitations.

V2 While the manual cautions the improper use of the CN Method and outlines its limitations, it was not intended to discourage its use when deemed appropriate. The method is best suited for agricultural runoff estimation but can also be applied in urban settings such as, but not necessarily limited to:

- where the objective is to estimate runoff volume (e.g., storage facility sizing);
- where the catchment area is primarily impervious.

Refer to **Appendix C** for CN Value Tables. V2

3.7.7.5 *Horton Method:* The Horton Equation is empirically based on an initial infiltration rate that gradually decreases (exponential decay) as soil becomes more saturated and converges to the soil's saturated hydraulic conductivity. Typical Horton infiltration parameters are presented in **Table 3.7.7.5** below. **Appendix A** includes a table showing all soil types in the region as well as the corresponding texture and hydrologic group.

! Table 3.7.7.5 – Typical Horton Infiltration Parameters

Parameter	Hydrologic Group			
	A	B	C	D
fmax, dry (mm/hr)	250	200	125	75
fmax, normal (mm/hr)	250	80	50	25
V2 fmin, dry (mm/hr)	25	13	5	3
fmin, normal - clay	7.6	3.8	1.3	0.5
fmin, normal - loam	9.5	5.7	2.5	1.0
fmin, normal - sand	11.4	7.6	3.8	1.3
k (1/hr)	4	4	4	4

V2 Infiltration parameters should be based on normal AMC for major system design and dry AMC for minor system design and stress test assessments. The values in

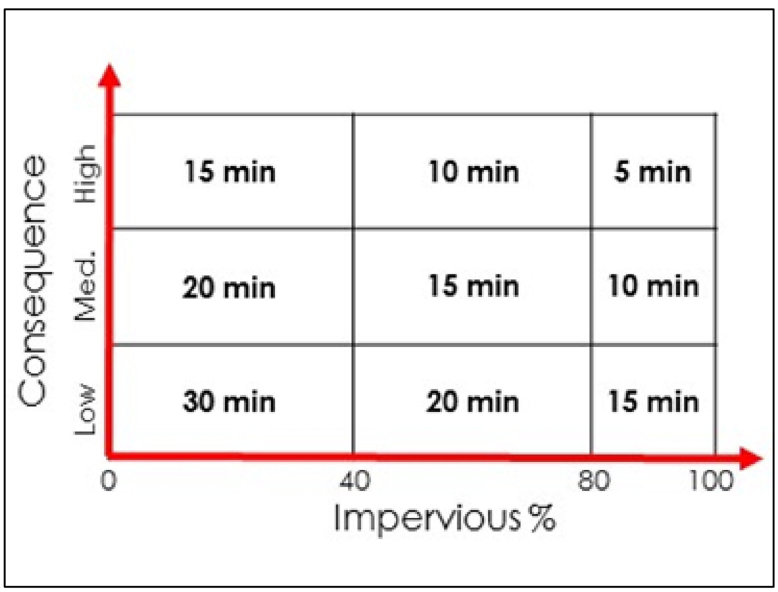
Table 3.7.7.5 are not intended to be strictly followed for applications such as model calibration. Refer to **Appendix D** for additional discussion. V2

3.7.8 Design Storm Distributions

Every storm has three Ds that are related to frequency. Depth, Duration and Distribution. Depth and Duration have well established frequency relationship (i.e., IDF curves). We do not have information on frequency of distributions. Hence, the selection of storm distribution must be made carefully and conservatively as this assumption can significantly affect the magnitude of the peak flow we are trying to estimate. For consistency in the region's approach, this manual recommends various design storms distributions, as provided in **Appendix B**, to evaluate both conveyance and storage requirements of a specific project.

3.7.8.1 *Conveyance Capacity:* To evaluate conveyance capacity of urban drainage systems, a Chicago 4-hour distribution shall be used. When using the Chicago 4-hour storm, the maximum timestep shall be based on **Graph 3.7.8.1** and dependant on impervious level and consequence of exceedance – as defined in Section 1.5.2). See **Appendix A** for supplemental information in reference to this section.

! Graph 3.7.8.1 – Maximum Timestep for Chicago 4-Hour Storm



The above graph is the same as Graph 3.2.2.6 – Maximum Inlet Times.

3.7.8.2 *Storage Requirements:* To evaluate stormwater storage facilities or pumped systems, both the Chicago 4-hour and SCS Type II 24-hour storm distributions shall be evaluated to determine the critical storage volume. See **Appendix A** for supplemental information in reference to this section.

3.7.8.3 *Climate Change Adaptation:* Stormwater infrastructure should be evaluated based on a **"stress test" event, herein defined as 150mm of rainfall** – representing a 39% increase compared to Windsor Airport's 100-year 24-hour rainfall of 108mm. Supporting discussion and rationale for the proposed increase is provided in section 3.9. The "stress test" storm shall be distributed as summarized below and specified in **Appendix B**:

- Rural Conditions: SCS Type II distribution
- Urban Conditions: Chicago 100-year 24-hour distribution with uniform distribution of the additional 42mm (i.e., additional 42mm spread evenly over the 24-hour period). [V2] Note: The storm timeseries in **Appendix B** has been revised as per the discussion in **Appendix D**. [V2]

The "stress test" storm is intended to assess the resiliency and vulnerability of the designed (or pre-existing) system. However, in instances where identified vulnerability and risk is deemed unacceptable to the Municipality and/or the CA, the design will need to be adjusted to mitigate the unacceptable risk. [V2] The stress test is intended to assess potential severe consequences due to exceedance of stormwater systems most affected by volume (i.e., stormwater ponds and pumped storm sewer systems). Moreover, it is not generally recommended to be applied on large scale watersheds where the spatial extents of the extreme rainfall are much smaller than the subject catchment area.

For new development [V2] where the stress test creates unacceptable risk [V2], the stress test event shall be contained within the site and maintained below the lowest building opening elevation of the site. [V2] This requirement is not intended to apply to all developments because the stress test is not considered as a Design Storm. Rather, it is to be considered as a "what would happen if this extreme storm occurs?" where design modifications would only be made to mitigate unacceptable risk.

Unacceptable risk related to the stress test is intended to correspond to the consequence of severe damage, rather than the consequence of any negative or undesired outcome. For example, the nuisance and access issues from surface ponding depths exceeding 0.3m; or surface ponding encroachment onto properties, are consequences that should generally be acceptable for extreme storms exceeding the 100-year floodproofing standard.

It should be considered that aversion to any risk and corresponding upsizing of storm infrastructure beyond design standards comes at an additional cost that should provide proportional value/benefit.

Refer to **Appendix D** for additional discussion supporting the adequacy of the current 100-year design storm as a typical floodproofing standard for new development. v2

3.7.8.4 *Watershed Drainage Studies*: For watershed scale drainage studies, the SCS Type II 24-hour storm distribution is recommended for rural conditions and the Chicago distribution for urbanized conditions. Both of these storm distributions have concentrated rainfall within the middle portion of the storm. For watersheds with both urban and rural conditions, it is recommended that both storms be evaluated. For larger watersheds with time of concentrations greater than 2 hours, it is also recommended that the lower intensity but more persistent AES 30% 12-hour storm be evaluated to assess the potential for superposition of subcatchment peak flows at the downstream reaches of the receiver. Where applicable, the evaluation of the Probable Maximum Storm may also be required. The latter two storms are defined in the MNRF River & Stream Systems: Flooding Hazard Limit Technical Guide dated 2002 (see **Appendix C** for reference).

3.7.8.5 *Allowable Release Rate:* Further to discussion in Section 3.3.1.4, when a hydrologic analysis is deemed appropriate to assess pre-development condition flow rates, the SCS Type II 24-hour storm shall be used.

3.7.9 Hydraulic Analysis

For certain applications, such as design of a small storm sewer system, the use of standard spreadsheet calculations using Manning's Equation and the Rational Method may suffice. However, hydraulic analysis of stormwater drainage systems will generally require modelling to evaluate the hydrodynamics of the system under the minor and major design storm events. The following section provides minimum requirements.

3.7.9.1 *Storm Sewer Hydraulic Grade Line (HGL):* Ideally, the hydraulic grade line would always be maintained below basement elevations, however this is impractical in most of the Windsor/Essex region due to limited gradient and the shallow sewer installations that are required to preserve fall. A typical acceptable level of service in this region requires that;

- the minor system HGL be maintained below ground elevations (i.e. no surface storage) and that;
- the major system HGL corresponds to;
    - a maximum surface ponding depth of 0.3 metres and;
    - a minimum 0.3 metres below building opening elevations.

More stringent HGL requirements may be required at the discretion of the Municipality and/or the CA based on known flooding issues or other site-specific conditions.

3.7.9.2 *Boundary Conditions:* Hydraulic grade line analysis must consider downstream boundary conditions of the downstream receiver. It is not acceptable to assume free outfall or normal flow depths condition without due consideration to the potential backwater conditions of the receiver. This is a particularly important design consideration in low lying areas near lakes and major watercourses as well as pumped systems where backwater conditions are most prominent and storm sewer surcharging is anticipated.

3.7.9.3 *SWM Facility Minor Storm HGL:* Where the downstream receiver is a SWM storage facility, the minor storm water level shall not exceed the inflow sewer obvert (i.e. shall not create a backwater condition on the minor storm system).

3.7.9.4 *Storm Sewer Manning's Coefficients:* Minimum roughness coefficient should follow Appendix C of the MTO Gravity Pipe Design Guidelines.

3.7.9.5 *Minor Losses:* Hydraulic analyses shall account for minor losses for inlet and outlet losses, bend losses and other appropriate losses.

3.7.9.6 [V2] *Dual Drainage Modelling:* Dual drainage modelling consists of modelling the conveyance capacity and interaction between both the minor system (i.e., the storm sewer) and the major system (i.e., overland flow – roadways).

New development designs in the region typically require 5-year storm sewer design and road grading to ensure overland routing towards a SWM pond. Thus, the objective of meeting required conveyance capacity for both minor and major systems can generally be achieved by a spreadsheet calculation (using Rational Method and Manning's Equation) and proper road grading design (i.e., dual drainage modelling is not necessary for new development). Generally, the region is flat and surface flow depths and velocities that may arise during a high-intensity rainfall are well below safety thresholds that would warrant modelling to estimate surface depths/velocities.

The foregoing does not preclude the use of dual drainage modelling, which may be warranted to address certain objectives and conditions that require an understanding of minor and major system interaction and performance. When dual drainage modelling is deemed necessary, it is important to acknowledge that not all dual drainage models are equal as the interaction between sewers and roadways can be represented in various ways – ranging from simple to complex. Refer to **Appendix D** for further discussion regarding dual drainage modelling methods. V2

3.8 LOW IMPACT DEVELOPMENT (LID) CONTROLS

3.8.1 MECP Guidance

The MECP (formerly MOECC) released Draft No.2 of its Low Impact Development (LID) Stormwater Management Guidance Manual dated November 27, 2017. The Draft document provides guidance on LID approaches as well as a comprehensive list of supporting resources related to LID from planning & design to construction and operation/maintenance.

The guidance describes the Runoff Volume Control Target (RVC_T), which is founded upon the principles of;

- *Maintaining the pre-development water balance and returning precipitation volume to the natural pathways of runoff, evapotranspiration and infiltration in proportions which are in keeping with the watershed conditions prior to development. The goal of maintaining the pre-development water balance shall be to ensure the ecosystem function and natural quality and hydrological characteristics of natural features, including aquatic habitat, baseflow, water quality, temperature, storage levels and capacity, and hydroperiods will be maintained and known impacts of urbanization are avoided.*

For the Windsor/Essex region, the specific RVC_T is 32mm. This volume control target is specific to the Windsor/Essex region based on the 90th percentile rainfall derived from an analysis of this region's historical hourly rainfall data. It supersedes and improves upon the previously used 25mm quality/erosion control volume, which represented the same 90 percent capture approach but more generally applied across the province.

3.8.1.1 To provide flexibility in the implementation of the RVC_T, a Control Hierarchy was developed as follows:

- Priority 1 (Retention): infiltration, evapotranspiration and or re-use. <u>The control volume does not become runoff</u>.
- Priority 2 (LID Volume Capture and Release): Utilize LID filtration. The control volume is filtered and released to the receiver at a <u>reduced rate and volume</u> (a portion may be lost via infiltration and/or evapotranspiration).
- Priority 3 (Other Volume Detention and Release): Other technologies which utilize filtration, hydrodynamic separation and/or sedimentation (to detain and treat runoff). The control volume is treated and released to the receiver at a <u>reduced rate</u>.

Refer to **Appendix A** or the MECP document itself for additional discussion regarding flexible treatment options for sites with restrictions (reference section 3.3.3.5 of the MECP guidance document).

3.8.2 Implementing LID in the Windsor/Essex Region

Retention of the specified 32mm of rainfall may prove challenging to implement for many parts of our region. While the approach certainly has merits, there remains concern that the shift to LID may prove impractical as a uniformly mandated approach to stormwater management in the Windsor/Essex region given the region's predominance of clay soils, lack of topographic relief, high groundwater and surface backwater conditions. The MECP acknowledges constraints and provides flexibility in control volume requirements, yet its Draft guidance implies that a significant effort and burden of proof (i.e. studies, monitoring, etc.) will be required to support the rationale that the priority 1 cannot be practically achieved. This manual proposes that the local Conservation Authorities, municipalities and practitioners have the best understanding of the region and are therefore best suited to determine the appropriate priority for the region.

This manual acknowledges the benefits of LID measures for peak flow attenuation, water quality and volume reduction and encourages its implementation **where it is expected to be beneficial**. In some instances, there is no significant benefit and potentially disadvantages to achieving the goal of maintaining the pre-development water balance. Refer to section 4.0 for further discussion related to a locally observed disadvantage to increasing infiltration.

It is well known in our region that many areas are not ideal for infiltration practices and cannot be relied upon to reduce conventional stormwater infrastructure costs. Moreover, with a non-infiltration LID measure such as rain harvesting, the designer cannot be assured that the homeowner will empty the collected rain before the next storm. Thus, the necessity for redundancy leaves the region to implement retention at a premium. Notwithstanding, the implementation of such practices as supplemental measures to conventional stormwater measures could potentially yield some tangible benefits. For example, high-intensity thunderstorms that overwhelm urban storm sewer systems and result in basement flooding are more likely to occur in the summer months where it is also more likely that;

- groundwater levels would be lower
- warmer/dryer conditions could result in shrinking/cracking of clay soils – thereby creating pathways for improved infiltration capacity
- less frequent rainfall and greater infiltration/evapotranspiration losses would allow for full drawdown of infiltration storage (i.e. full storage volume would be available).
- Dryer conditions would encourage the use of harvested rain (i.e. full storage volume would be available).

The resulting benefit would be peak flow attenuation, reduced runoff and improved resiliency to the overall system, provided that the increased infiltration does not impact utilities or property.

3.8.3 LID Design Considerations

3.8.3.1 More infiltration could direct water into sewer trenches which could increase existing basement flooding risk. Refer to section 4.0 for further discussion.

3.8.3.2 LID facilities should generally include pre-treatment to capture oils, debris and suspended solids.

3.8.3.3 The inspection and maintenance of numerous small scattered facilities could easily overwhelm local government staff with increasing budgetary constraints and challenges to meet current operation and maintenance demands.

3.8.3.4 Public should be educated on source controls and encouraged to undertake measures on their properties. This will take time and poses challenges with regards to maintenance, ownership and restrictive covenants to ensure measures are secured in perpetuity from one property owner to the next.

3.8.3.5 LID controls require pre-treatment which can be challenging in rights-of-way. Space can be limited in ROW and avoidance of LID facilities by utilities could be challenging.

3.8.3.6 Consideration should be given to soil amendment with compost or other organic matter to enhance infiltration, capture runoff pollutants, and reduce the adverse effects of soil compaction associated with construction.

3.8.3.7 Development planning and building practices should be in sync with LID (i.e. land use density, roof disconnects, etc.)

3.8.3.8 LID measures could potentially be used as a storage redundancy over and above the prescribed 100-year design standard, which could also serve as a climate change adaptive measure.

3.8.3.9 Refer to the Credit Valley Conservation Authority and the Toronto Region Conservation Authorities guidance documents on LID. (https://cvc.ca/low-impact-development/low-impact-development-support/stormwater-management-lid-guidance-documents/)

3.9 CLIMATE CHANGE

There remains a lack of clear and consistent guidance with regards to climate change and what this means to the stormwater practitioner in the context of rainfall amounts and distributions used for stormwater designs. Recent extreme events in our region combined with Provincial Policy Statement 2014 as well as MECP policy and expectations impose a need to consider the resiliency and vulnerability of stormwater infrastructure under increasing rainfall conditions. As further study and science evolves, it is hoped that the results will lead to clear guidance on climate change and its impacts on stormwater design standards. Until then, the practitioner must continue on with the most reliable information available.

3.9.1 Practical Guidance for the SWM Practitioner

"Theory can leave questions unanswered, but practice has to come up with something." – Mason Cooley.

The evidence presented in **Appendix A** and V2 **Appendix D** v2 suggests that recent extreme rainfall experienced in our region over the past few years has been related to prolonged rainfall and increased volume rather than increases to short-term intensities of 5 to 30 minutes. As such, the recommended approach to assess resiliency and vulnerability, <u>at this time and based on the region's current understanding of rainfall</u>, is to proceed slowly with an assumed 150mm rainfall amount as a defined "stress test" event. Refer to section 3.7.8.3 and **Appendix B** for storm distribution details.

Meanwhile, design standards should continue to rely upon the long-standing historical data provided by the Windsor Airport station. <u>The foregoing value of 150mm is not arbitrary but is also not derived to any particular level of certainty or defined confidence limit.</u> See **Appendix A** for a detailed discussion.

3.9.2 Beyond IDF Curves

IDF curves are an important tool for the stormwater practitioner. However, the complex problem of defining accurate IDF curves and adapting curves for climate change impacts is one of **many considerations** when designing resilient stormwater infrastructure or evaluating existing system vulnerabilities.

It is true that recent events exceeded 100-year rainfall and that the magnitude of rainfall led to flooding damage. However, it is **also the manner in which the rainfall is dealt with**, the limitations of our region and specified acceptable level of risk that contributes to flooding damage. It is important that these factors not be overlooked or substituted with the expectation that updated IDF curves and corresponding supersized infrastructure will solve all flooding problems. Perhaps more importantly, the cost of floodproof infrastructure is unlikely to be affordable or justified when compared to the expected cost of damages.

The objective of stormwater management from a flood control perspective is to **<u>mitigate (not prevent)</u> flooding damage** but to conversely and **<u>most certainly prevent loss of life</u>**. Surface flooding on a roadway or parking lot is typically not damaging at depths up to 0.3 metres. Road closures, while inconvenient, are also not typically damaging.

Of all the damage caused by recent extreme events in the region, basement flooding damage is the most significant as it not only carries an explicit repair cost but also results in significant emotional distress that is more difficult to quantify in terms of cost. This [V2] manual [V2] recognizes basement flooding as significantly damaging and dedicates a complete section (section 4.0) to the interaction between municipal infrastructure and private storm/sanitary systems. Mitigation measures at the lot level are believed to be the most practical approach to protect homes against flooding.

The region's flat topography significantly limits hydraulic gradient, particularly under high lake/river levels. Areas near waterbodies – lake, river or artificial pond for stormwater detention – are all subject to backwater conditions which limit stormwater conveyance capacity and/or necessitate pumping. Pumping stations are not typically designed to handle extreme events. Again, it is often not affordable or justified to do so when compared to the expected cost of damages, notwithstanding the large number of basement flooding damages due to vulnerable private drainage systems.

Most designs are based on a specified level of service, typically defined in the form of a return period (i.e. storm sewer designed to a 5-year event or a stormwater pond designed to a 100-year event). As discussed in section 1.5, the probability of exceedance (risk) over the design life of the infrastructure is more apt to define the reliability of the design.

Other design considerations include the impact of urbanization on stormwater **volumes**. This document evolves from the current **flow rate control** approach where post-development stormwater has been historically restricted to pre-development flow rates measured at the lot level scale and without due regard for increased volume. The recommended approach considers the carrying capacity of the receiving watercourse(s) and prescribes a more holistic watershed management approach that considers the cumulative impacts of small, incremental changes to the hydrologic cycle.

It is important for municipalities and practitioners to bear in mind that extreme rainfall is only one of many other factors that contribute to flooding. The identification of other flooding causes and targeted mitigation measures that address the root cause of flooding is paramount. One interesting example is included in **Appendix A**.

3.9.3 Climate Adaptation and Mitigation

As referenced from the Engineer's Canada National Guideline: Principles of Climate Adaptation and Mitigation for Engineers, the need to incorporate climate change through adaptation and resiliency considerations into engineering works can be realized through the following actions:

1. Listing the climate change predictions and potential impacts for the area where the project is located;
2. Discussing the aspects of the project the engineer believes could be impacted;
3. Detailing what has been done in the design to reduce those impacts;
4. Discussing the climate-relevant national, provincial, and municipal level codes, policies and bylaws establishing the level of acceptable risk, and identifying the client's level of risk tolerance;
5. Detailing what additional/revised operations and maintenance (O&M) and inspection procedures are recommended within the service life cycle of the project; and
6. Outlining policies and procedures to restore interruptions to service, loss of functionality or repair damages from extreme weather events.

3.10 INFILL AND EXISTING DEVELOPMENT

3.10.1 Infill Development

V2 The term 'infill development' may have a different meaning from a planning versus a stormwater management perspective. Within the context of stormwater management, infill development should consider any development within an area that was built-up before the need for stormwater management was recognized (or areas built to an older SWM standard). For these types of developments, the minimum objective is to ensure that the infill does not adversely impact the existing condition. Infill development should never make things worse; however, it may be impractical to improve or rectify an existing sub-standard condition. In some cases, and in lieu of a "Master Drainage Study" that could provide guidance in such special circumstances, pre-consultation discussions should take place with the Municipality, the Conservation Authority, and the Owner/Developer, to review any practical opportunities to make some minor improvements to the existing sub-standard conditions. Taking all factors into consideration, and depending on the risk of each unique situation and the tolerance of the existing condition, implementation of some practical minor improvements may be required. V2

The discussion below (in italics) is an excerpt of the MECP's (formerly MOE) 2003 SWM Guidelines;

Infill projects can range in size from a single lot to the complete redevelopment of significantly larger areas. Many forms of infill development can be more intensive than previous uses and *have higher levels of imperviousness (e.g., more pavement), runoff rates, and contaminant loading per unit of area. In many cases, areas surrounding the new infill development were built before the need for stormwater controls was recognized and are already experiencing stormwater management problems.* ***Although the development of single, individual infill sites may not have significant impacts, the development of many individual sites can have cumulative effects and exacerbate or create problems at the subwatershed and watershed level including flooding, erosion, or water quality degradation.*** **[Emphasis Added]**

Applying stormwater management practices in developed areas can be a challenge. Land availability and cost often limit stormwater management options in infill situations. Stormwater controls in infill situations are frequently implemented on private property and owners are responsible for their maintenance. Municipalities can generally require owners to maintain these controls; however, the proliferation of numerous, small, scattered facilities may be undesirable from a management and operations perspective.

3.10.1.1 An Infill Development Plan or Subwatershed Rehabilitation Plan is the preferred approach to address stormwater management requirements, particularly where significant growth is expected.

3.10.1.2 On-site SWM is generally preferred. Where on-site facilities are impractical or ineffective, financial contribution can be collected in lieu to fund stormwater management measures located elsewhere within the same subwatershed.

3.10.1.3 Where additions or expansions are proposed, the overall site should be considered and retrofitted as required to meet the current SWM quality and quantity control standards of this manual. V2 Some flexibility may be warranted as discussed in section 3.10.1.5 below. V2

3.10.1.4 Where reconstruction or rehabilitation projects do not alter the existing condition with regards to runoff peak flow and volume nor adversely impact the existing drainage system, the Municipality can, at their discretion, allow less than standard SWM measures to suit existing constraints. Such projects shall demonstrate a reasonable effort to implement practical SWM measures that will improve upon the existing condition.

3.10.1.5 V2 *Flexibility in Level of SWM Quantity Control:* Infill development SWM quantity control requirements are categorized under four levels: Normal, Exempt, Basic and Enhanced. The varying levels provide some flexibility to the Municipality and Conservation Authority with a means to apply tailored SWM requirements based on the size and nature of the proposed development. Refer to **Appendix D** for details. V2

3.10.2 Existing Development

3.10.2.1 The adoption of this manual may, in some instances, introduce more stringent SWM design criteria for future phases. Where existing developments of partial buildout are concerned, any proposed phases of development shall include an initial re-assessment of the existing SWM plan if applicable, an amendment to the SWM plan. It shall be acknowledged that amendments to the SWM plan may require retrofits to the existing stormwater facilities or construction of additional stormwater management measures to meet the new V2 design criteria V2. Following the initial re-assessment, future phases should consist of a simple review to confirm that the future phases are meeting the new SWM plan.

V2 The intent of this requirement is to ensure that the SWM pond is sufficiently sized to meet current requirements adopted since the 1st publication of this regional *SWM Manual* dated December 2018 (i.e., higher imperviousness, 24-hour rainfall amount, etc..). This requirement also includes the assessment of minor/major drainage systems. The foregoing assessment can typically be achieved by a 1D model analysis or sewer HGL spreadsheet analysis combined with a review of existing and proposed grading to identify overland flow routes. This assessment should not typically require detailed dual drainage modelling analysis, although it may be warranted in certain instances. V2

3.10.2.2 SWM design for new development often includes assumed values for impervious level based on expected land use. Past practice has shown that residential development can significantly exceed assumed impervious level with the addition of sidewalks, driveways, patios, sheds, pools, etc. Actual impervious levels shall be measured as development progresses to verify/confirm that the assumed design values are being maintained. Conservative levels should be accounted for at the design stage to ensure that the actual levels do not exceed the assumed levels, thus mitigating costly retrofits to infrastructure. Storage facilities should account for additional storage, or at least allocate land for future expansion, to address deviations from assumed levels.

3.11 SUBMISSION REQUIREMENTS

V2 The following provides a fairly comprehensive list of items that may be required in a SWM design submission. Note that items not listed herein may also be required. At a minimum, the items noted with an asterisk (*) represent typical "must-haves" for any SWM design submission. V2

3.11.1 General

3.11.1.1 *Site Description:*

1) *Location – nearest roads, watershed & subwatershed
2) *Existing Conditions – land use on site & surrounding areas
3) *Proposed Conditions
4) *Drainage Area – for the site, tributary & watershed
5) Watercourses, Wetlands - present on site, and type (permanent or intermittent)
6) *Drainage patterns and ultimate drainage location/outfall

3.11.1.2 *Background Information:*

1) Watershed Plans
2) Sub-Watershed Plans
3) Master Drainage Plans (MDPs)
4) Other Previous Reports and Relevant SWM Requirements
5) Existing Models
6) Geotechnical Report

3.11.1.3 *Figures:*

1) *Location Plan
2) Legal Plan of Survey
3) Pre-Development Drainage Area Plan
4) Post-Development Drainage Area Plan
5) Proposed SWMF locations
6) Proposed Site Plan – grading, servicing and details
7) Erosion and Sediment Control Plan

3.11.2 Quality Control

3.11.2.1 *Design Criteria:*

1) *Level of Protection
2) *Drainage Area to Facility (ha)
3) *Percentage Impervious
4) SWM Facility Monitoring and Maintenance Requirements
5) Customized Quality Objectives

3.11.2.2 *Oil-Grit Separators (OGS):*

1) Approved Manufacturer
2) Model Number
3) Sizing Calculations Included
4) TSS Removal (%)
5) Annual Runoff Treated (%)
6) Sediment Storage Capacity

7) Total Storage Volume
8) Maximum Treatment Flow Rate
9) Particle Size Distribution and particle specific gravity used in sizing
10) Appropriate Lab Results and/or Field Study Results

3.11.2.3 *Wet Ponds/Wetlands/Hybrid:*

1) Permanent Pool Storage Requirements (m^3/ha)
2) Permanent Pool Storage Requirements (m^3)
3) Permanent Pool Volume Provided (m^3)
4) Extended Detention Storage Requirements (m^3/ha)
5) Extended Detention Storage Requirements (m^3)
6) Extended Detention Volume Provided (m^3)
7) Detention Time - minimum 24 hours
8) Inlet and Outlet Structure Details

3.11.3 Quantity Control

3.11.3.1 *Design Criteria:*

1) Runoff Coefficient or Impervious Calculations
2) *Allowable release rate (m^3/s)
3) *Design release rate (m^3/s)
4) SWMF Type
5) Stage vs Storage Table
6) *Outlet Design
7) *Total Active Storage Required (m^3)
8) *Total Active Storage Provided (m^3)

3.11.4 Hydrologic & Hydraulic Modeling

3.11.4.1 *Hydrologic Modeling:*

1) V2 Runoff method
2) V2 Infiltration method
3) Other hydrologic routines (e.g., groundwater, etc., if applicable)

3.11.4.2 *Hydraulic Modeling:*

1) Type of Hydraulic Model – 1D, 1D dual drainage, 1D minor with 2D major system
2) Hydraulic routing method

V2 Note: When hydrologic and hydraulic modelling parameters are consistent with the guidance in this manual, the reporting of the model analysis does not need to summarize these same parameters. The report should identify and rationalize the use of any parameters that deviate from the manual guidance. V2

3.11.5 Hydrogeology

1) Soils / Hydrogeology Report
2) Seasonal Groundwater Elevations
3) Pre & Post Development Water Budget
4) Special Construction Considerations and Recharge Measures

3.11.6 Construction Sediment Control

1) Sediment Control Plan
2) Sizing of Temporary Sediment Basins and details
3) Check dam locations and details
4) Silt fence location and details
5) Outlet location
6) 24-hour Extended Detention Calculations
7) Sequencing and Maintenance/Inspection schedule and notes

3.11.7 Other

1) Summary of model inputs and outputs V2 (upon request) V2
2) Schematic representation of pre and post development hydrologic models
3) Storm sewer design sheets
4) Storm sewer design drainage plan, showing areas and runoff coefficients
5) All final reports and plans signed and sealed
6) All drawings, calculations and model units shall be in **metric**.

4.0 PRIVATE DRAINAGE SYSTEMS

Given the limited land gradients that predominates the Windsor/Essex region, the function of both municipal and private drainage systems are often impacted by backwater conditions. This condition is not limited to areas near waterbodies – it is also common for most storm sewer systems in the region, which outlet to a pumped outfall, a stormwater management pond or an open channel with limited conveyance capacity.

Notwithstanding SWM efforts, recent rainfall events have revealed a potential shortcoming of the strategies that have been implemented in the region. Apart from the benefit of controlled peak runoff rates to downstream lands, ponds that are constructed in flatter areas such as the Windsor / Essex region, produce an undesirable effect that can potentially aggravate the flood risk to upstream lands – by elevating the hydraulic grade line in the upstream sewers and trench bedding. This phenomenon is particularly problematic in the Windsor / Essex region, due to the manner in which sewers have been traditionally installed, and the way that foundation and roof drainage has been provided in the region.

In the past 5 to 10 years, a potential incompatibility of urban SWM and building construction practices has become apparent to some engineers, particularly the developers of this manual. The following subsections address an unusual phenomenon that is believed to occur and recommend practices that are intended to mitigate a potentially unusual flood risk.

4.1 DISCUSSION OF LOCAL PHENOMENON

As noted, in most of the Windsor/Essex region, SWM systems that employ any type of pond inherently elevate upstream water levels. During infrequent events, when the rainfall duration and intensity is greatest, and water levels within the ponds approach the maximum design level, pond levels typically approach the grade of the surrounding lands. When this occurs, storm sewers typically surcharge to levels that greatly exceed the footing elevation of the buildings that line the streets. Moreover, sewer inverts are frequently installed above footing elevations.

A long-standing construction practice in the Windsor/Essex Region has been to bed new sewers in ‘sewer stone’ (graded clear stone). The use of sewer stone eliminates the need to compact the bedding and its ‘free-draining’ properties facilitates sewer construction in areas that exhibit a high groundwater table.

An undesirable characteristic of sewer stone is its ability to transmit groundwater efficiently. As a result of this property, and since the storm sewer system outlets directly to the SWM pond, the hydraulic grade line within the stone bedding of the storm sewers generally matches or exceeds the water level in the pond.

The fact that utility trenches typically cross within the road right-of-way compounds the problem. Since the bedding materials of each trench can and often do hydraulically interact, as the HGL or 'groundwater level' of the storm sewer bedding becomes elevated, the HGL of the sanitary sewer and watermain bedding material also becomes elevated to a comparable level. The potential problem arises when the service connections that connect to the building are taken into consideration.

Each housing unit has multiple service trenches that extend from the road right-of-way to the building envelope – a storm connection, a sanitary connection, a water service, and a hydro trench (incl. bell and cable tv). These service trenches are normally bedded in similar 'sewer stone' or sand that can efficiently transmit water to the building foundation. Although measures can be taken to effectively 'cut-off' this groundwater supply, measures have not always been either prescribed and/or properly implemented to mitigate this condition. Taking into consideration the flat terrain that predominates the region, it becomes evident how building foundation drains in even modern developments can become overwhelmed by groundwater during heavier rainfall events.

4.2 NEED FOR BETTER COORDINATION OF MUNICIPAL AND PRIVATE DRAIN DESIGN

The foregoing highlights a flooding phenomenon that is somewhat unique to the Windsor/Essex region that has not been effectively addressed in the past. **Without better coordination of the municipal and private components of urban drainage systems, flooding will continue to occur in the region, regardless of what SWM measures are implemented on the municipality portion of the system.**

The following sets out recommendations for improving the effectiveness of urban drainage in the Windsor / Essex region where conditions are suitable for the afore-mentioned phenomenon to occur.

4.2.1.1 It is imperative that private drainage systems be constructed to handle the expected backflow pressure conditions of the stormwater system and that private connection trenches be hydraulically disconnected from the main sewer trench. Homebuilders should install impervious trench plugs on all utility trenches on the building side of the lot line to mitigate subsurface flow of groundwater through granular bedding materials to the building foundations. A suitable impervious material should be used such as bentonite, Class A bedding, or compacted clay. Consideration should be given to installing two trench plugs on each utility to provide some redundancy.

4.2.1.2 It is strongly recommended that all homes be equipped with backflow prevention and reliable/durable sump pump systems. A sump pump with backup power should also be considered to mitigate potential basement flooding in the event of a power outage. Consideration could also be given to overflow routing of foundation drainage to a separate structure located in the garage for ease of access and emergency pumping via a backup sump pump or portable pump with generator.

4.2.1.3 Flooding issues commonly arise from deficient private drainage connections that re-introduce sump pump discharge back to the foundation drainage via cracks and pipe displacements. A secondary sump pump outlet to ground surface is recommended to ensure sump pump efforts are not lost.

4.2.1.4 All plumbing fixtures located in the basement level should be plumbed through a sewage ejector pump. The discharge piping should be installed such that the piping is raised above elevation of the ground outside of the structure before it exits the building. This measure will effectively prevent backflow of domestic sewage from the municipal sewer to the basement.

4.2.1.5 Stormwater can enter the sanitary system indirectly via loose joints, cracks in pipes and manholes, cleanouts or illicit drainage connections, causing sanitary sewer backup and flooding. The coincidence of backups with surface ponding can be indicative of significant inflow to the sanitary sewer via the manhole cover lift holes. Sealing manhole covers to mitigate inflow is recommended with due consideration to maintaining proper venting of the sanitary sewer system. New sanitary sewer design should endeavor to locate manholes away from low points.

4.2.1.6 High lake levels will naturally raise long-term groundwater levels in areas near waterbodies. Homeowners and homebuilders should be informed of this condition which should be carefully considered when deciding on backfill material surrounding the home, basement finish floor elevation and foundation design. Homeowners that choose to construct basement finish floor elevations below natural or stormwater management waterbody levels should expect frequent sump pump operation as well as sustained groundwater pressure on foundations which may require additional construction measures from both a structural and waterproofing perspective.

4.2.1.7 Consideration should be given to installing impervious trench plugs intermittently along the mainline sewers to mitigate the upstream piping of groundwater through the bedding material of the sewer.

4.2.1.8 Strapping of private drain pipes along the foundation walls should be prohibited. Private drain pipes should be installed away from the backfill zone.

5.0 IMPLEMENTATION/CONSTRUCTION

5.1 IMPLEMENTATION OF STORMWATER MANAGEMENT PLANS

There is often a disconnect between the approved SWM Plan outlined in a report and the construction drawings that are prepared to implement the SWM Plan. It is important to standardize a process in which recommendations of a SWM Plan are incorporated into the construction drawings.

5.1.1.1 All recommendations of a SWM Plan and design details of a SWM facility should be summarized on a SWM related construction drawing.

5.1.1.2 For phased development buildout, the individual phase construction drawings should be reviewed by the Municipality. Alternatively, the Municipality may request a letter of conformance from the Designer to confirm that the development is consistent with the SWM plan requirements.

5.1.1.3 Actual impervious levels for constructed phases should be reviewed to confirm that construction has proceeded in accordance with the design parameters used to size the stormwater facility. Should the actual impervious exceed design parameters, future buildout conditions or the stormwater management plan will need to be re-designed to suit. If the above retrofit measures are not feasible, the future development should not be allowed to proceed.

5.1.1.4 Whenever feasible, stormwater management facilities and green infrastructure should be established prior to development.

5.1.1.5 Upon completion of final grading of the facility, the Proponent shall complete a topographic survey of the facility. The survey shall be compared to the design SWM facility to verify that it has been constructed in accordance with the design. The Proponent shall provide the Municipality with a letter from the Professional Engineer(s) stating that it has been constructed in general conformance with the approved drawings. Record drawings and an electronic copy of the surveyed points data shall be enclosed with the letter.

5.2 EROSION AND SEDIMENT CONTROL (ESC) PLANS DURING CONSTRUCTION

The largest amount of sediments and pollutants will be coming off the site during construction. The construction will require control of significant amounts of sediment, not only during construction of buildings or municipal infrastructure but also during subsequent buildout of homes in a residential development. The quality control facilities that are designed as part of the completed stormwater management facility are designed for full build out with stabilized site conditions. These facilities are generally not capable of sufficiently addressing construction sediment and erosion controls.

5.2.1.1 Stormwater management submissions shall include an erosion and sediment control plan to mitigate construction sediment. The ESC plan is to be prepared by the Consultant and implemented by the Developer and Municipality. The plan should minimally include the following:

- Erosion and Sediment Control Plan during servicing construction **and home building.**
- Flow Management Plan
- Spill Control and Response Plan
- Landscaping Restoration Plan
- Dust Control
- Vehicle Tracking Control/Mud Mats
- Implementation and Quality Control of ESC Plan
- Inspection and Monitoring Plan

The Greater Golden Horseshoe Conservation Authorities (GGHCA) guideline titled "Erosion & Sediment Control Guidelines For Urban Construction" provides guidance for developing ESC plans.

6.0 OPERATION AND MAINTENANCE

It is important for owners to have a clear and concise operation and maintenance strategy to ensure that the intended design, performance and aesthetics of the stormwater management facility is implemented and maintained. This is often also required as a condition of an Environmental Compliance Approval (ECA). Operation and Maintenance procedures shall be provided by the Designer as a separate manual or drawing(s) for easy reference and implementation. These procedures shall be followed by the Developer during the maintenance period and by the Municipality upon final acceptance of the pond.

6.1 MINIMUM REQUIREMENTS

At a minimum, an operation and maintenance manual or drawing(s) shall include the following:

6.1.1 Facility Design Objectives and Functions

6.1.1.1 Summary of SWM objectives and functions: water quality, erosion and flood controls, process narrative to describe pond operation under various storm conditions (i.e. 25mm quality storm, minor storm and major storm), including stage/storage information.

6.1.1.2 Description of SWM features/structures and inspection requirements for same.

6.1.1.3 Facility design attributes: contributing area, impervious area, elevations/volumes for permanent pool, extended detention, active storage, release rates.

6.1.2 General Maintenance Activities

6.1.2.1 Periodic inspection is required to identify and schedule maintenance such as; debris and litter removal, sediment accumulation depth measurements, inlet/outlet repairs, pond bank and access road repairs, etc.

6.1.2.2 SWM ponds will generally require periodic vegetation maintenance. Grass cutting and weed control may be required to ensure that weeds and invasive species do not invade the pond banks. Site specific vegetation management measures shall be included as part of the manual or Landscaping Plan drawing. During the first two years of operation, inspections should be made after every significant storm (i.e. daily rainfall greater than 25mm) to ensure proper functioning. After this initial period, annual inspections may suffice unless site specific conditions warrant more frequent inspection.

6.1.2.3 Trash and debris shall be removed promptly to mitigate the potential for clogging of outlet pipes.

6.1.2.4 If oil/sheen is observed, it should be removed immediately by use of oil-absorbent pads or a professional with a vacuum truck. Special disposal requirements may apply.

6.1.2.5 Algal mats are prominent in stagnant conditions during summer months. If mats develop over 10% of the water surface, they should be removed using a rake and left to dry on the pond banks.

6.1.2.6 All SWM quality control measures require periodic maintenance for proper function.

6.1.3 Sediment Removal

6.1.3.1 Sediment removal frequency is dependent on many factors and can vary significantly. Removal shall be performed once the permanent pool volume equals the volume corresponding to a removal efficiency of 5% below the required treatment efficiency. SWM design calculations should identify the depth of sediment accumulation in the forebay that triggers the cleanout requirement.

6.1.3.2 Sediment accumulation rates are typically much larger during the construction period of a catchment area. Once a catchment area is fully developed and established, sediment accumulation rates tend to be significantly lower. For planning purposes only, the MECP provides typical annual sediment loading rates as outlined in Table 6.3 of their 2003 SWM manual.

6.1.3.3 Sediment to be properly handled and disposed of according to current regulations.

6.1.4 Monitoring and Reporting

6.1.4.1 Monitoring and reporting requirements as defined in the MECP's Environmental Compliance Approval (ECA) shall be included in the O&M manual.

6.2 ADDITIONAL REFERENCES

More detailed discussions regarding inspection and maintenance can be found in the MECP 2003 manual and TRCA Inspection and Maintenance Guide for Stormwater Management Ponds and Constructed Wetlands.

7.0 REFERENCES

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16. Ontario Ministry of Environment, Stormwater Management Planning and Design Manual, 2003.
17. Ontario Ministry of Natural Resources, River & Stream Systems: Flooding Hazard Limit Technical Guide, 2002.
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19. Ontario Ministry of Transportation, Highway Drainage Design Standards, 2008.
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APPENDIX A

Supplemental Information

A-3.2.1 Rainfall Intensity

A-3.2.1.1 *Understanding Rainfall versus Runoff Return Period:* As per OMAFRA drainage guidelines, "*the designer should understand the distinction between a storm with a 10-year return period and a flood with a 10-year return period, since a 10-year storm does not necessarily produce a 10-year flood. If a 10-year storm occurs when the ground is dry it will produce only a small flood. If the same size of storm occurs when the ground is wet it may produce double or triple the previous flood discharge. This is one of the difficulties in using the design storm concept. The true 10-year flood can only be determined by a frequency analysis of a large number of measured flood discharges in a watershed. Because such measurements are seldom available, the practice of estimating runoff from more readily available rainfall rates is widely used and accepted, and for this purpose it is often assumed that a 10-year storm will produce a 10-year flood*".

A-3.2.2 Storm Sewer (Minor) System

A-3.2.2.1 *Level of service:* Throughout the Essex Region, most municipalities specify the minor storm event as a 1 in 5 year (5-year) return period – a storm that has a 20% probability of occurring in any given year. These systems offer quick and efficient drainage of urbanized areas to limit the inconvenience of stormwater ponding.

Inconsistency: There has been inconsistency within the municipalities in the design intensities being used for sewer design, ranging from 2-year AES Windsor Airport to 5-year AES Windsor Airport to 5-year City of Windsor IDF curves. For a typical residential development with 20 min inlet time, the corresponding intensity varies from 52 mm/hr to 79 mm/hr. A standard approach to sewer design is preferred within the Windsor/Essex region.

A-3.2.2.3 *IDF curve fitting and impact on intensity equation:* The current AES Windsor Airport data is fitted to an IDF curve [i = a / (t+b)^c] where b is set to zero. When the actual b value is included the 5-year (20 min inlet time) intensity increases from 67.6 to 75.0 mm/hr – an 11% increase simply by providing a three-parameter curve fit to the rainfall data compared to the simplified two parameter curve fitting performed by AES.

A-3.2.2.7 A reduction in C value or impervious level, and subsequently in infrastructure sizing, should not be made on account of disconnected roofs. The rationale for this restriction is as follows:

Firstly, there is no guarantee that the roof leaders will be disconnected, particularly those that land onto paved areas and introduce liability to the municipality as well as risk of injury and inconvenience to the property owner.

Secondly, infiltration capacity in the region's predominantly clay soils is limited and should not be relied upon to infiltrate additional water from roofs. Under the design 5-year storm event, grassed areas are likely to be saturated, leaving roof water to flow overland into the storm sewer rather than infiltrate. It could be argued that roof water being directed to the surface introduces some lag and peak flow attenuation as compared to directly connected roofs. While this is true, current standard practice for storm sewer design already accounts for this attenuation via use of a 20 min. inlet time.

The above is not meant to discourage downspout disconnection. There are significant benefits to disconnecting roof leaders to reduce inflow to the private drainage system, maintain the natural hydrologic cycle and also promote stormwater infiltration / stormwater volume reduction on a long-term basis. Rather, the restriction is meant to promote that infrastructure be sized for the likely scenario that a 5-year design storm will completely saturate grass areas, which will no longer have capacity to infiltrate additional roof water.

A-3.3.1 Allowable Release Rate

A-3.3.1.4 It is generally acceptable to assume that a receiving open drain was designed to accommodate a 2-year undeveloped flow for the agricultural lands. However, in flat lands with average slope less than 0.5% and low hazard, municipal drains may only be designed to carry flows based on agricultural curves. These curves express discharge based on a Drainage Coefficient expressed in mm/day. Refer to OMAFRA Drainage Guide For Ontario Publication 29 for further details.

Individual versus holistic approach: The approach of treating individual development sites as an isolated catchment without consideration to the overall watershed hydrodynamics raises the following potential concerns:

1) On the surface, one might expect that individual developments restricted to a 2-year agricultural flow rate should sum up to equal the same 2-year agricultural flow for which the drain was designed. This is likely not the case. It is reasonable to expect that the hydrologic analysis of the drain design considered long lengths of overland flow across agricultural lands as compared to much shorter lengths across small individual properties. The result is shorter flow times and higher peak flows in the individual sites that sum up to a flow that exceeds the drain design.

2) Each designer may calculate pre-development flow rates using different methods with different hydrologic parameters and design storms, ultimately resulting in ambiguity, inconsistency and inequality in the calculated allowable release rate of individual sites.

3) From a hydrologic perspective, outflow from a developed site with a restricted 2-year pre-development flow rate is not the same as the pre-developed outflow. Although the peak outflow rate is designed to match under both pre and post-developed conditions, the shape of the outflow hydrograph can vary significantly. Development can significantly change the hydrology of the site (i.e. – quicker runoff response from sewer systems versus agricultural drainage, increase in runoff volume and subsequently a prolonged period of outflow from the site).

Example:

<u>Pre-Developed</u>

Area = 5 ha
Tc = 80 min.
C = 0.2
Q_{2yr} = 54 L/s
Q_{total} = 54 L/s

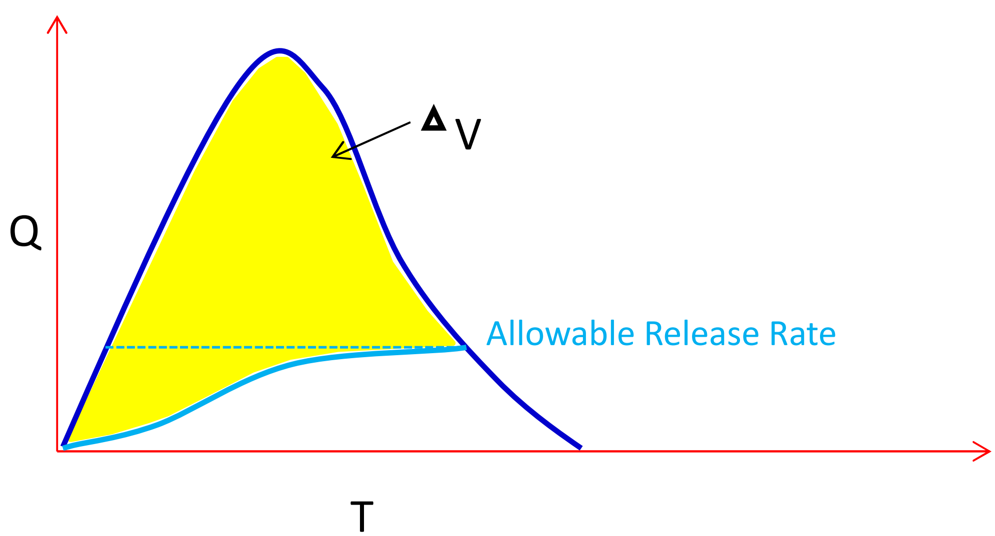
<u>Developed with Flow Restriction</u>

Area = 1 ha x 5
Tc = 40 min. for each site
C (allowable) = 0.2
Q_{2yr} = 18 L/s/site
Q_{total} = 89 L/s (165% of pre-developed flow)

A-3.3.2 Storage Requirements

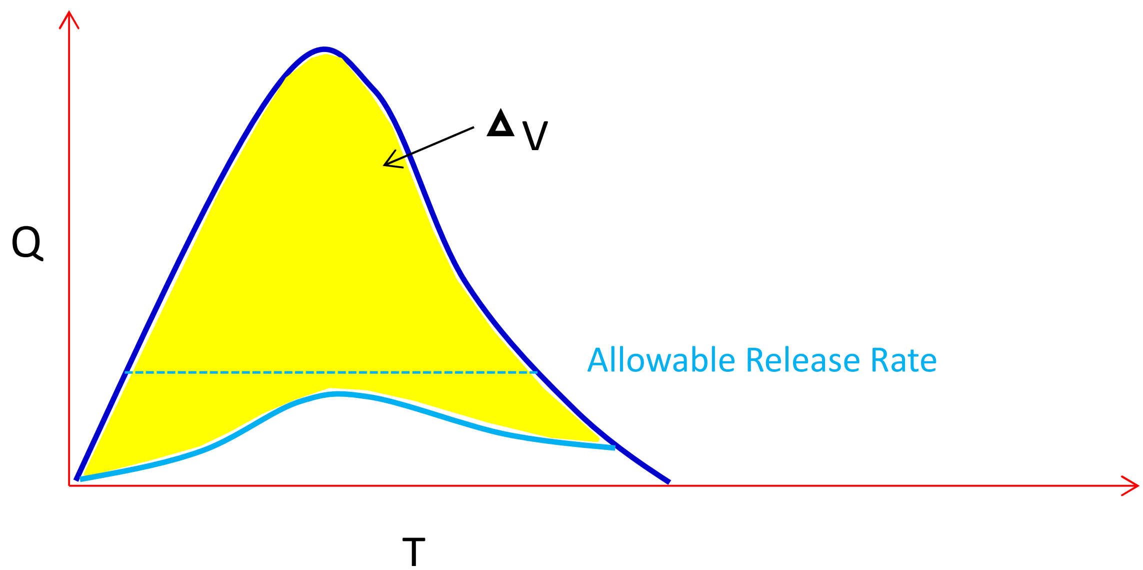
A-3.3.2.1 *Outflow from Storage Facility:* Gravity outflow from a storage facility is not constant and varies with storage levels.

Figure A-3.3.2.1a – Storage Volume for Gravity Outflow



Impact of Backwater Conditions: Gravity outflow from storage facility is reduced with decreasing head due to backwater conditions, resulting in additional storage required. The impact of backwater conditions can be impractical to analyze on a watershed scale and leave the practitioner without means of quantifying the reduced outflow from backwater conditions and resulting additional storage required. Where allowable release rates are small compared to the expected peak inflow to the facility, it can be reasonably conservative (i.e. not impractical) to assume a zero discharge.

Figure A-3.3.2.1b – Storage Volume for Gravity Outflow with Backwater Conditions



Development of Storage Equations: The standard storage equations were derived from PCSWMM modeling based on the following assumptions and input parameters:

- 100-year Rainfall (See Appendix B):
  - 108mm with SCS Type II 24-Hour Distribution
  - 81.6mm with Chicago 4-Hour Distribution
- Flow length: 40m
- Slope: 1%
- Depression Storage: 2.5mm impervious, 7.5mm pervious
- Manning's Roughness: 0.011 impervious; 0.24 pervious
- *Modified Green-Ampt Infiltration with following inputs;
  - Hyd. Group A: Su = 100, Ks = 9.5, IMD = 0.17
  - Hyd. Group B: Su = 300, Ks = 5.7, IMD = 0.16
  - Hyd. Group C: Su = 250, Ks = 1.3, IMD = 0.13
  - Hyd. Group D: Su = 180, Ks = 0.5, IMD = 0.10

*A Note on the Modified Green-Ampt Method used in PCSWMM:

> *The Modified Green-Ampt method changes the original Green-Ampt procedure by not depleting moisture deficit in the top surface layer of soil during initial periods of low rainfall as was done in the original method. This change can produce more realistic infiltration behavior for storms with long initial periods where the rainfall intensity is less than the soil's saturated hydraulic conductivity.*

A-3.3.2.7 Hybrid Detention Approach: The hybrid approach of both regional and on-site storage reduces the size of storm sewers as compared to having no on-site controls. However, this approach must define the proper limit of on-site storage requirements that can be effectively and practically managed without forcing less economical storage costs onto the individual sites.

Individual sites can provide surface storage at relatively small cost. However, if underground storage is also required, it will typically be more costly than adding the same storage volume in a regional pond. **At a minimum, this standard recommends that at least 50% impervious be accounted for routing and regional storage design.** For example: an industrial site with 90% impervious would consider at least 50% impervious for routing and regional storage design and the remaining 40% impervious for on-site storage.

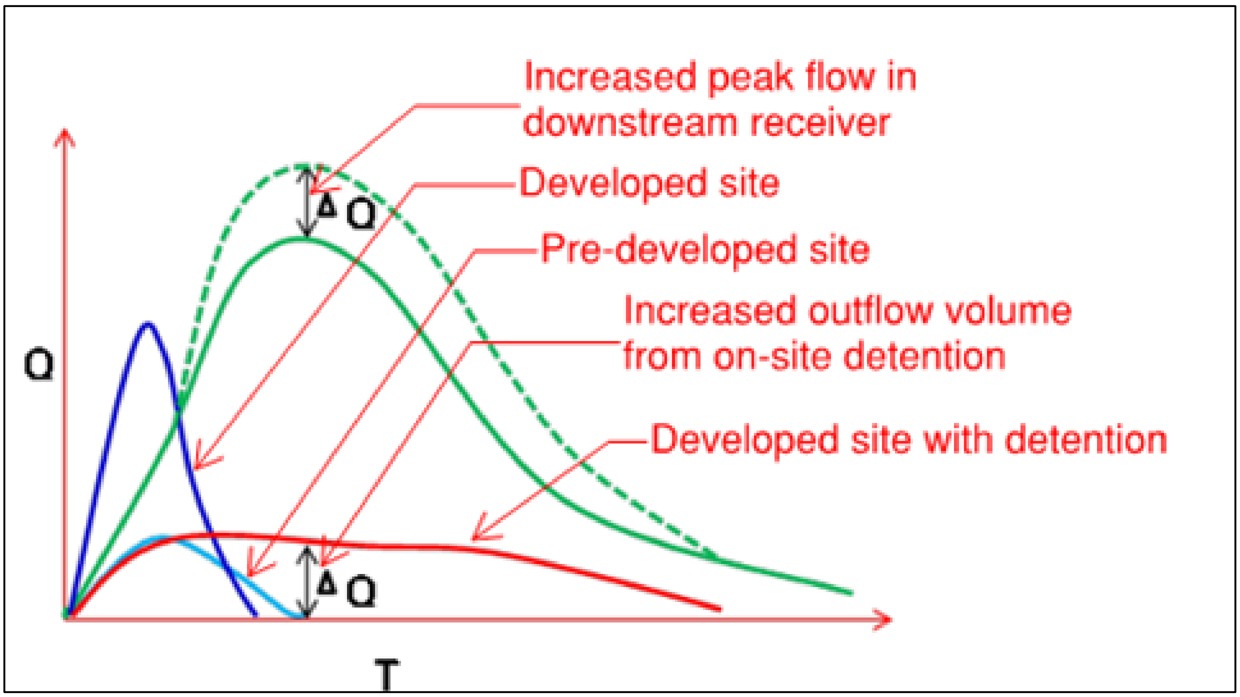
The recommended 50% impervious for routing and regional storage is derived assuming a maximum 50% lot coverage and minimum 10% landscaping (pervious) coverage, leaving 40% as potential parking lot coverage. Conservatively, we can assume that much of the landscaping area will be contributing runoff under the 100-year event, resulting in 50% (40% parking lot + 10% landscaping) of the 100-year rainfall or 54mm (50% x 108mm) to be stored on-site. Given that the first 32mm is required to be stored by stormwater practices other than parking lot surface storage (e.g. underground storage, surface swales/ponds, rain gardens, etc.), the remaining 22mm would be required to be stored on the parking surface. Assuming typical prismatic storage surfaces (i.e. Volume = Area / 3) with a maximum 0.3m storage depth at the catch basin, the required surface storage coverage is estimated to be 22% (i.e. 0.022m x 3 / 0.3m) or approximately half of the total parking lot area. Accounting for grading constraints which limit the full use of the parking area for storage, this is believed to be a reasonable assumption of parking lot surface storage capacity.

A-3.3.3 Peak Flow Timing Issues

A-3.3.3.1 While conservative, this approach is likely to be reasonable for many drains within the Windsor/Essex region. Given the flat topography in most of the region, conveyance capacity is limited. The limited hydraulic gradient often makes conveyance capacity improvements a non-viable option and thus, large detention facilities are the default stormwater approach. Large detention facilities and small release rates equals a long duration of outflow where any lag between subcatchments (say 1 to 2 hours) becomes insignificant when compared to several days of outflow. And while the outflows from each detention facility will vary over time based on their stage-outflow relationship, it would be reasonable to expect that stormwater facilities may require pumped outflow to a shallow receiving drain and thus the outflow will be constant, prolonged and immediate.

A-3.3.3.2 An example to illustrate timing issues with future development is the following: A residential development is being proposed at the downstream reach of a large watershed which is largely agricultural lands. The downstream reach of the receiving drain outlets directly to the lake. Given the proximity of the development to the lake, it would be reasonable to allow the development to proceed without stormwater quantity control. As shown in **Figure A-3.3.3.2**, a hydraulic impact assessment would show that unrestricted flow from the proposed development will drain ahead of the watershed peak flow and thus will not increase the peak flow to the receiving drain.

Figure A-3.3.3.2 – Timing Effect of Stormwater Detention

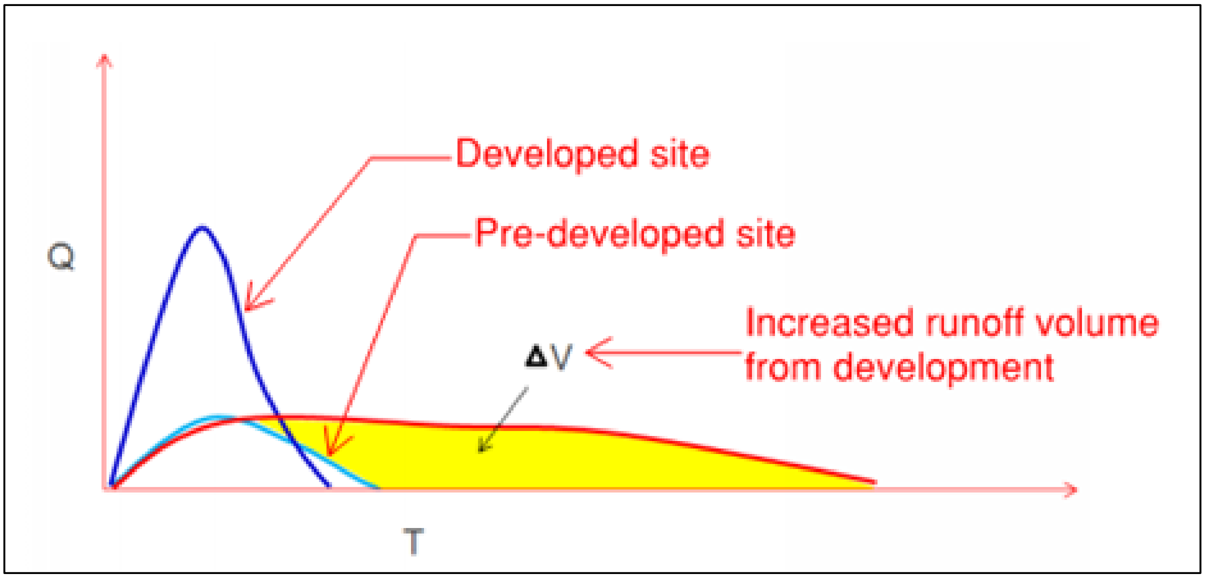


Conversely, the figure shows that adding stormwater detention to restrict flows to the pre-development condition will actually increase the overall peak flow to the drain given that the prolonged outflow, while no greater than the pre-development peak flow rate, will now increase the overall peak flow to the receiving drain. <u>Initial Conclusion: the development should NOT have stormwater detention.</u>

However, the foregoing conclusion is only valid under the condition that the largely agricultural lands remain undeveloped and the hydrologic response of the watershed does not change. What happens when the largely agricultural lands now begin to slowly develop? After 10 years of upstream development, the watershed's hydrologic response (both volume and timing) has changed. Runoff reaches the drain much faster, even though these developments have detention facilities and restricted release rates. The unrestricted flow from the subject downstream development may now increase the overall peak flow in the downstream receiver. **Revised Conclusion: Stormwater planning at the watershed level is necessary to avoid adverse impacts from urbanization.**

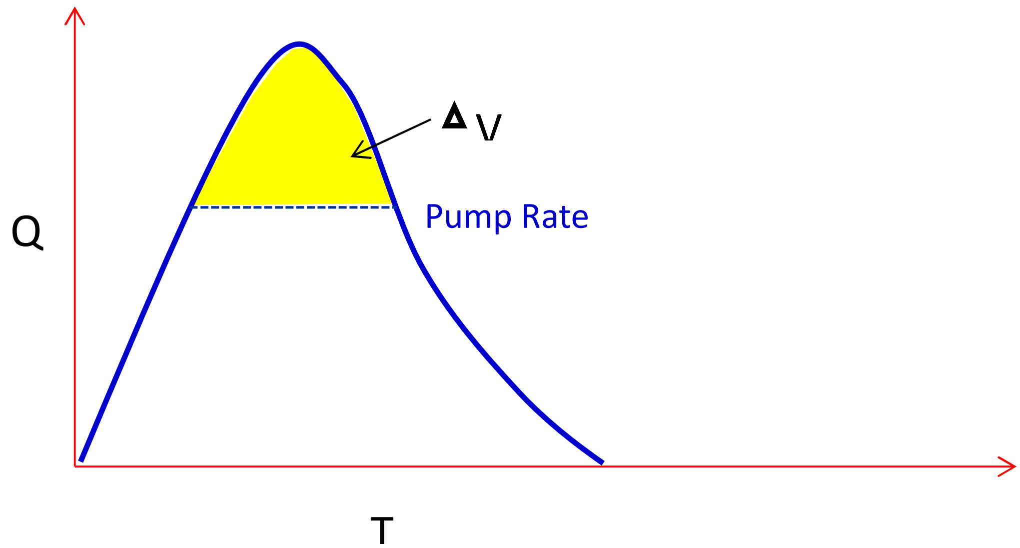
A-3.3.4 Volume Mitigation Issues

Figure A-3.3.4 – Increased Runoff Volume from Development



A-3.3.4.2 Pumped Drainage Systems: Pump design capacity varies depending on the specific requirements of a site. Often, the pump rate is designed to handle more frequent storm flows with the expectations that infrequent storms will produce peak flows that temporarily exceed the pump rate. When this occurs, temporary storage is required upstream of the pump.

Figure A-3.3.4.2 – Storage Volume for Pumped Outflow



A-3.4.1 Standard Quality Objectives

A-3.4.1.5 Current practice calculates OGS removal efficiencies based on a weighted calculation that accounts for rainfall intensity variation as a function of total annual rainfall volume. In other words, the historical hourly rainfall data at Windsor Airport suggests that 60.7% of the total rainfall volume occurs from rainfall intensities of 2 mm or less. Based on site characteristics, a 2 mm/hr rainfall is then converted to a corresponding flow rate and a removal efficiency is then calculated for said flow rate. Assuming a removal efficiency of 90% at the calculated flow rate, the relative efficiency is then 60.7% x 90% = 54.6% of the required 70% removal requirement for normal protection. The above process is repeated for various rainfall intensities ranging from 1 mm/hr to 25mm/hr and the relative efficiencies at each rainfall intensity are summed up to produce the overall removal efficiency.

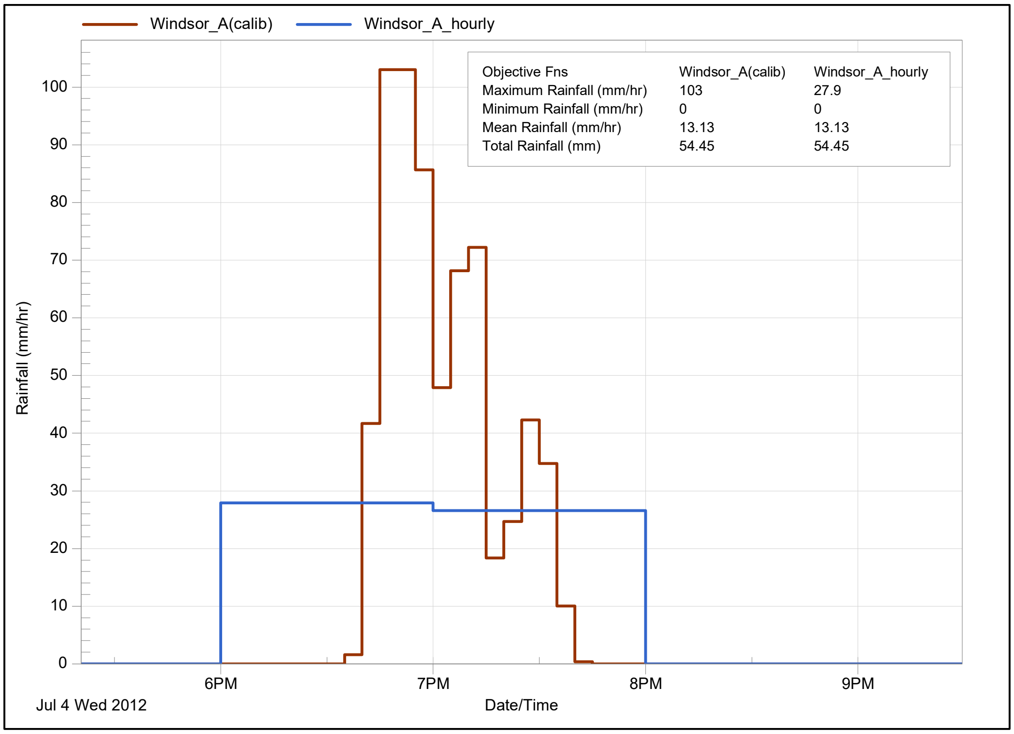
Two concerns are apparent with the above methodology;

1. As illustrated in **Graph A-3.4.1.5** below, the hourly rainfall data dampens actual intensities experienced from short duration high intensity storm events and;

2. rainfall intensities of 2mm/hr are not likely to produce runoff and/or provide the energy required to washoff pollutant buildup.

To address the above concerns, this Manual recommends that the Windsor Airport Tipping Bucket rainfall data – data set available from 2008 to 2015 with no data available in 2009 and 2010 – be used to determine the rainfall intensity / rainfall volume relationship and that the rainfall intensities of 2mm/hr or less be excluded from said rainfall analysis. *<u>These adjustments yield **Table 3.4.1.5** to be used by OGS manufacturers to estimate TSS removal efficiencies.</u>*

Graph A-3.4.1.5 – Hourly versus 5-minute Rainfall Intensity



A-3.7.2 Runoff Estimation Methods

A-3.7.2.1 The Rational Method can be useful and appropriate provided the user understands its underlying assumptions and limitations.

<u>Assumptions:</u>

- Runoff coefficient assumes a constant proportional rainfall loss throughout the storm
- The time of concentration should be taken as the hydraulically furthest point of the catchment – at which point the entire catchment is contributing runoff
- Uniform rainfall intensity for the duration of the storm – which is equal to the time of concentration or greater.

<u>Limitations:</u>

- The runoff coefficient (C) attempts to account for many variables that influence how much rainfall becomes runoff. Thus, selection of the C value relies heavily on judgment and can be somewhat subjective. Moreover, the Rational Method assumes a constant C value whereas runoff volume can vary significantly based on antecedent moisture conditions as well as rainfall intensity and duration.

- The assumption that the storm intensity will be spatially and temporally uniform over a catchment area for the duration of the storm (equal or greater than time of concentration) should be limited to small catchments. It is recommended that the use of the Rational Method for sizing of conveyance systems be limited to smaller catchment areas as prescribed in section 3.2.2.3.

- The furthest point of a watershed may require a time concentration to be estimated across a large agricultural field which in turn will reduce the overall time of concentration at the downstream end. When this situation occurs, the user should verify that the smaller downstream developed area with smaller time of concentration does not produce a larger flow.

A-3.7.7 Infiltration Losses

Table A-3.7.7 – Soil Types in Essex County

Texture	Symbol	Name	Acreage	Hydrologic Group
Clay Soils	Bc	Brookston Clay	250,000	D
	Toc	Toledo Clay	17,500	D
	Cc	Clyde Clay	2,500	D
	Jc	Jeddo Clay	3,500	D
	Cac	Caistor Clay	13,500	C
	Pc	Perth Clay	9,000	C
Clay Loams	Pcl	Perth Clay Loam	8,000	C
	Cacl	Caistor Clay Loam	2,500	C
	Bcl	Brookston Clay Loam	30,000	D
Silt Loam	Tos	Toledo Silt Loam	1,000	D
Loams	Bg	Burford Loam	3,700	A
	Bg-s	Burford Loam Shallow Phase	5,300	A
	Hl	Harrow Loam	4,000	A
	Fl	Farmington Loam	2,000	B
	Pl	Parkhill Loam	5,000	C
	P-r	Parkhill Loam Red Sand Spot Phase	5,000	C

Texture	Symbol	Name	Acreage	Hydrologic Group
Fine Sandy Loams	Tfs	Tuscola Fine Sandy Loam	6,000	C
	Cdl	Colwood Fine Sandy Loam	7,000	C
Sandy Loams	Hs	Harrow Sandy Loam	3,500	A
	Fsl	Fox Sandy Loam	5,300	A
	Bel	Berrien Sandy Loam	16,000	C
	C-s	Caistor Sand Spot Phase	1,500	C
	B-s	Brookston Clay Sand Spot Phase	18,000	D
	Was	Wauseon Sandy loam	3,000	C
Sands	Gs	Granby Sand	1,000	C
	Bes	Berrien Sand	8,000	C
	Ps	Plainfield Sand	1,700	A
	Es	Eastport Sand	2,500	A
Misc. Soils	B.L.	Bottom Land	7,300	-
	Ma	Marsh	7,000	-
	Ml	Muck	1,700	-

A-3.7.7.4 The SCS Curve Number (CN) Method was originally intended to predict volume of runoff from daily rainfall. The rationale behind the method is that when infiltration depth is small compared to soil saturation, runoff is proportionally small compared to rainfall. Today, the method has been extended to perform more hydrologic analysis than it was originally intended for and with that comes several cautions:

- The method was based on average conditions and thus may not be accurate for historical events.
- The curve number is based on daily empirical data and was not intended to consider varying rainfall duration and intensity.
- The standard initial abstraction $Ia = 0.2S$ is derived from agricultural watersheds and may overestimate losses for impervious areas and underestimate losses due to depression storage.
- Most importantly, the proportional nature of the equation translates to a proportional increase in infiltration as the rainfall intensity increases, thus applying the highest rate of infiltration during the highest peak rainfall intensity. This limitation can lead to significant over-estimation of infiltration and corresponding under-estimation of peak flow, particularly for analysis of high-intensity thunderstorms.

A-3.7.8 Design Storm Distributions

A-3.7.8.1 Typical models cannot be expected to mimic all of the complexity of a real-world conditions. For example, short-term minor detention from rear yard ponding would likely not be captured in a model of residential development. Thus, a very short-time step with corresponding very high intensity rainfall would produce higher model peak flows than actual conditions where dampening of short duration cloudbursts would be expected to occur. Moreover, where consequence of exceedance is lower, the additional effort to define the minutia of actual conditions in a model is not justified.

A-3.7.8.2 The Chicago 4-hour distribution represents a high intensity thunderstorm and is used to assess the conveyance capacity of an urban system as well as localized surface ponding. Conversely, the SCS Type II storm distributed at 2-hour intervals is used to evaluate volumetric capacity of storage elements and pumped systems. However, depending on the discharge rate of the storage element or pump, the Chicago storm can sometimes produce the critical storage volume requirements.

A-3.8 Low Impact Development (LID) Controls

This section presents the discussion provided in MECP's Draft No.2 of its Low Impact Development (LID) Stormwater Management Guidance Manual dated November 27, 2017 regarding flexible treatment options for sites with restrictions:

The RVC_T acknowledges that retention (Control Hierarchy Priority 1) or Volume Capture and Release (Control Hierarchy Priority 2) may not be feasible for every site as a result of site-specific constraints. For all sites, regardless of perceived restrictions (i.e. constraints), the proponent should attempt to comply with the appropriate volume control alternative as described above. The Runoff Volume Control Target (RVC_T) acknowledges that volume control is achievable on these sites via re-use and evapotranspiration practices even when partial or no infiltration is possible.

The constraints which may result in the application of alternatives to the above prescribed volume targets include:

a) *Shallow bedrock† and Karst;*

b) *High groundwater† or areas where increased infiltration will result in elevated groundwater levels which can be shown to impact critical utilities or property;*

c) *Swelling clays or unstable sub-soils;*

d) *Contaminated soils (i.e. Brownfields);*

e) *High Risk Site Activities including spill prone areas;*

f) *Prohibitions and or restrictions per the approved Source Protection Plans and where impacts to private drinking water wells cannot be appropriately mitigated;*

g) Flood risk prone areas or structures and/or areas of high inflow and infiltration (I/I) where wastewater systems (storm and sanitary) have been shown through technical studies to be sensitive to groundwater conditions that contribute to extraneous flow rates that cause property flooding / sewer back-ups and where LID BMPs have been found to be ineffective;

h) For existing Linear Developments where reconstruction is proposed and where available surface and subsurface areas is not available based on a site-specific assessment completed by a qualified person. Areas where private property is susceptible to flooding from high groundwater levels;

j) Surface water dominated or dependant features including but not limited to marshes and/or riparian forest wetlands which derive the all or a majority of their water from surface water, including streams, runoff, and overbank flooding. Surface water dominated or dependant features which are identified through approved site specific hydrologic or hydrogeologic studies, and/or Environmental Impact Statements (EIS) may be considered for a reduced volume control target. Pre-consultation with the MECP and local agencies is required;

k) Existing urban areas where risk to life, human health, property or infrastructure has been is identified and substantiated by a qualified person through an appropriate area specific study and where the risk cannot be reasonably mitigated per the relevant design guidelines;

l) Water reuse feasibility study has been completed to determine non-potable reuse of stormwater for onsite or shared use. Potable reuse may be considered on case specific basis.

† May limit infiltration capabilities if bedrock and groundwater is within 1m of the proposed facility invert per Table 3.4.1 of the LID Stormwater Planning and Design Guide (2010, V1.0 or most recent). Detailed assessment or studies are required to demonstrate infiltration effects and results may permit relaxation of the minimum 1m offset.

The two alternatives identified for sites with restrictions (i.e. constraints) are:

Alternative #1 – Reduced Runoff Volume Control Target

For site with restrictions, the proponent attempts to comply with the following conditions:

a) Achieve at least 75% volume control from all impervious surfaces for the runoff generated by the geographically specific 90th percentile rainfall event (Figure 3.1.2).

b) Options considered and presented should examine the merits of relocating project elements to address, varying soil conditions and other constraints across the site.

c) Not applicable for sites which directly discharge to a watercourse (See Section 3.3.3.6)

Alternative #2 – Maximum Extent Possible (MEP)

a) For site with restrictions, the proponent attempts to comply with the following conditions: Achieve volume control to the maximum extent possible (MEP). In regards to Alternative #2, the maximum extent possible (MEP) is defined as the maximum achievable volume control, using all known, available and reasonable approaches, including the methods as described within this manual, given the site restrictions. The specific scope of MEP may be negotiated between the relevant parties and / or subject to the jurisdiction of the relevant municipality, local or provincial agency.

b) Options considered and presented should examine the merits of relocating project elements to address, varying soil conditions and other constraints across the site.

c) Not applicable for sites which directly discharge to a watercourse. (See Section 3.3.3.6)

A-3.9 Climate Change

The discussion below (in italics) is an excerpt of the MECP's Policy Review of Municipal Stormwater Management in the Light of Climate Change, published April 5, 2016 (Updated April 6, 2016):

> *Overall, the municipalities need better tools to manage stormwater and to build municipal stormwater systems that are resilient and adaptive to climate change to better protect the environment. Currently no province-wide inventory is available for municipal stormwater systems to gauge the size of the problem or to compare any achieved progress on system condition or vulnerability to climate change.*
>
> *Resilient systems for municipal stormwater management are systems that strengthen the treatment train approach already established in the SWM Manual by building in resiliency to climate change. This would be accomplished by developing technical guidance for source control such as under the SWM Manual, but also by developing a MOE policy framework that could clarify and encourage municipalities and others to plan and act on resiliency for climate change.*
>
> *A MOE policy vision for resilient systems for municipal stormwater management may include, for example, the following considerations:*
>
> - *Include both source control (lot, neighbourhood) and conventional stormwater management.*
> - *Reduce the generation of stormwater by building communities that interfere less with the natural water cycle.*
> - *Reuse stormwater and recognize stormwater as a resource (e.g. for flushing toilets, landscape watering).*

- *Recycle the municipal stormwater back into the natural water cycle, with careful regard for water quality and quantity cumulative impacts on watersheds and groundwater.*
- *Include data collection and vulnerability assessment for the existing conventional stormwater management systems to assist in adaptation decisions by municipalities.*
- *Include long term planning for municipal stormwater management including a systematic approach to adaptation and assessment of the cumulative impacts on the watershed.*
- *Include tracking the progress of climate change adaptation, in particular source control, across the province as part of public education.*

There would be environmental and possible fiscal benefits to municipalities through adopting this approach. Source control can be expected to reduce the volume of stormwater that will be directed from private properties to municipal stormwater management infrastructure. Source control can have water quality benefits related to stormwater by treating, managing or reusing stormwater on properties or nearby on road rights of way where rain falls.

Information is required on the inventory and status of conventional systems that extensively exist in many municipalities across Ontario today. Municipalities need to examine the vulnerability of their conventional stormwater management systems to climate change and how they can be improved to increase the resiliency and adaptation to the uncertainties and extremities of climate change.

While it may be possible in some cases for municipalities to manage much of the run-off from private properties (residential, businesses) by source control facilities on the road right of way, this may be very costly or not always possible to do so. Municipalities need the cooperation of the property owners in order to effectively manage stormwater at the source. The MOE believes that better results can be achieved for source control when individuals, businesses, ministries, agencies and others collaborate and cooperate. As such, policies, guidance, public education and incentives are preferred over prescriptive mandatory regulations and legislation.

- *A MOE policy framework is needed to support resilient municipal stormwater management systems and adaptation to climate change and other identified stressors for new and existing developments.*
- *Data collection and information management systems are necessary to track the inventory, condition and performance of stormwater systems in order to assess Ontario's vulnerability to climate change and aid adaptive decision-making for infrastructure renewal.*

> *Resilient systems for municipal stormwater management can be accomplished in Ontario through MOE collaboration with municipalities and industry in developing policy, technical guidance, and municipal tools, all of which will assist municipalities to make informed decisions about climate change adaptation for their stormwater management systems.*

A-3.9.1 Practical Guidance for the SWM Practitioner

Again, the recommended 150mm rainfall amount as a "stress test" is not derived from a theoretical basis, nor is it derived to any level of certainty or defined confidence limit. Rather, it is based on a pragmatic observation that the recent extreme events have relatively large areas (greater than 25 square kilometres) with approximated rainfall amounts between 125mm and 150mm (refer to radar rainfall maps in Appendix C). While both storms measured larger amounts in localized areas, it is deemed impractical and currently unjustified to increase rainfall amounts by 100% to match said isolated areas that measured extreme rainfalls over 200mm. A rainfall of 150mm appears to encapsulate a significant spatial extent (greater than 25 square kilometres) from the recent events as well as match 2090 projections from both the MTO IDF Tool (148.8mm) and Regional IDF Study (147.1mm). These studies, as well as the recent extreme events are further discussed in the remainder of this section.

For perspective, the recommended 150mm rainfall over 24 hours translates to a 1:2,620 year storm based on Windsor Airport's historical data. Perhaps more clearly expressed as risk over a design life of 100 years, the 108mm rainfall has a 63% chance of occurring whereas the 150mm rainfall has a 4% chance of occurring.

Regional IDF Study – A Comparison of Future IDF Curves for Southern Ontario

Study Objectives

Extreme rainfall statistics in the form of intensity-duration-frequency (IDF) curves are used extensively in the design of water management infrastructure and policies. The IDF study and technical report prepared by Dr. Coulibaly et al (July 2015) aimed to understand the limitations and applicability of different techniques for updating IDF statistics in light of climate change.

Study Conclusions

The study highlighted the complexity of the development of future IDF projections and the various sources of uncertainty involved. <u>Ultimately, based on the study results, it was recommended that further study is needed before major change in infrastructure design standards. Further study is recommended to include the analysis of nonstationarity of extreme rainfall and development of regional IDF statistics.</u>

Given the uncertainly in future IDF curves, it was also recommended that weight-of-evidence approaches be used. IDF curves may form part of the evidence based approach for adaptation to extreme precipitation risk as well as historical extremes and information on the resiliency (i.e. thresholds and vulnerabilities) of stormwater systems.

The trend analysis results indicated that there is no significant trend with a slight apparent decreasing trend for short duration storms. The study recommended that trend results should be taken with caution in part because there is no objective way to discriminate trends among climatic trends, anthropogenic caused changes and sampling variability.

Windsor Airport IDF Curves

Standard practice in the region has generally relied upon Windsor Airport curves for design – whether it be Rational Method sewer design or creating storm distributions for hydrologic modeling. The Windsor Airport offers the most robust historical dataset (61 years from 1946-2007) as compared to the Harrow station (28 years from 1966-1989 & 2001-2007) and Point Pelee station (22 years from 1975-1993 & 2002-2004).

Table A-3.9.1a showing Windsor Airport extreme rainfall trends from 1995-2007 illustrates a decreasing trend from 1995 to 2007 across almost all durations and return periods.

Table A-3.9.1a – Windsor Airport Extreme Rainfall Trends 1995-2007

Duration	% Change in Rainfall					
	2 Year	*5 Year*	*10 Year*	*25 Year*	*50 Year*	*100 Year*
5 min	*-2.0%*	*-2.2%*	*-2.3%*	*-2.4%*	*-2.4%*	*-2.4%*
10 min	*-1.7%*	*-2.3%*	*-2.6%*	*-2.8%*	*-3.0%*	*-3.1%*
15 min	*-2.0%*	*-2.5%*	*-2.8%*	*-3.0%*	*-3.1%*	*-3.2%*
30 min	*-3.7%*	*-3.7%*	*-3.7%*	*-3.7%*	*-3.7%*	*-3.7%*
1 hour	*-4.9%*	*-3.2%*	*-2.5%*	*-1.9%*	*-1.5%*	*-1.2%*
2 hour	*-4.9%*	*-2.7%*	*-1.7%*	*-0.8%*	*-0.2%*	*0.2%*
6 hour	*-4.0%*	*-3.3%*	*-3.1%*	*-2.8%*	*-2.6%*	*-2.5%*
12 hour	*-3.4%*	*-1.8%*	*-1.1%*	*-0.4%*	*-0.1%*	*0.3%*
24 hour	*-2.2%*	*-1.5%*	*-1.2%*	*-0.9%*	*-0.7%*	*-0.5%*

Table A-3.9.1b showing Windsor Airport extreme rainfall trends from 1995 to 2015 continues to illustrate a decreasing trend for short-duration events from 5min to 30min duration for nearly all return periods. The trends illustrate an increasing trend in 1 hour, 2 hour, 12 hour and to a lesser extent the 24 hour durations.

Table A-3.9.1b – Windsor Airport Extreme Rainfall Trends 1995-2015[1]

Duration	% Change in Rainfall					
	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
5 min	-1.7%	-2.7%	-3.1%	-3.6%	-3.8%	-4.0%
10 min	0.2%	-0.9%	-1.3%	-1.8%	-2.1%	-2.3%
15 min	0.3%	-0.5%	-0.9%	-1.2%	-1.4%	-1.6%
30 min	-0.1%	-0.1%	-0.1%	-0.1%	-0.1%	-0.1%
1 hour	-1.2%	0.7%	1.6%	2.4%	2.8%	3.2%
2 hour	-1.8%	0.5%	1.6%	2.6%	3.2%	3.6%
6 hour	-0.8%	-0.5%	-0.4%	-0.3%	-0.2%	-0.1%
12 hour	-0.4%	0.8%	1.3%	1.8%	2.1%	2.4%
24 hour	0.2%	0.3%	0.4%	0.5%	0.5%	0.5%

Note 1: The extreme rainfall data for Windsor Airport is only available up to and including 2007. Data for subsequent years was derived from raw tipping bucket data that has not been verified to be accurate and that is missing most of the rainfall that occurred in 2009 and 2010.

Conclusion: Short-term durations events are slightly trending downwards, thus showing no evidence to increasing IDF curves for stormwater conveyance design. As mentioned in the regional study, trend results should be taken with caution in part because there is no objective way to discriminate trends among climatic trends, anthropogenic caused changes and sampling variability. Sampling variability in particular is demonstrated in section A-3.9.2.

MECP on Considering Climate Change

Dated October 2017, the MECP published a guide titled "Considering climate change in the environmental assessment process". Most pertinent to the Windsor/Essex Region SWM Standards Manual is *the Intensity Duration Frequency Curves* section under Appendix A and the referenced *Drainage Information* section under Appendix C. Said section provides a list of information about, and tools for, generating intensity duration frequency curves, which are available through:

- Ministry of the Environment and Climate Change
  AR4:A1B. Dynamically-downscaled climate projections with the PRECIS model under A1B emissions scenario, projected rainfall intensity-duration-frequency curves and daily and hourly time series data for climate change impact assessment. Climate projections are provided via the Ontario Climate Change Data Portal.

- Ministry of Transportation
  The IDF Curve Lookup is a web-based application provided by the Ontario Ministry of Transportation (MTO) for the purpose of retrieving intensity-duration-frequency curves.

MECP – Ontario Climate Change Data Portal

The technical report for the Ontario Climate Change Data Portal (CCDP) notes the following:

> *The projected IDF curves and the up-to-date project results as well as all associated data have been made publicly available at Ontario Climate Change Data Portal (Ontario CCDP): http://ontarioccdp.ca. The Ontario CCDP is developed with care and believed to be reliable, but mechanical or human errors remain a possibility. The IEESC [Institute for Energy, Environment and Sustainable Communities, University of Regina] accepts NO responsibility for any inaccuracies or omissions in the data, nor for any loss or damage directly or indirectly caused to any person or body by reason of, or arising out of, any use of Ontario CCDP. All IDF curves presented in the report and posted on this Portal at this time are calculated using the original model outputs of hourly precipitation. While all the revealed changes in the projected precipitation or IDF curves are possible from a physics or climate change science perspective, one should use these curves with extreme caution for practical applications. Further investigation is being undertaken to calibrate these IDF curves using historical data. These IDF curves will be updated once they are calibrated. Alternatively users can always download the hourly precipitation data and calculated their own IDF curves with appropriate corrections.*

Under the report acknowledgments the following is stated:

> *This project has received funding support from the Ontario Ministry of the Environment. Such support does not indicate endorsement by the Ministry of the contents of this material.*

Conclusion: The CCDP provides uncalibrated IDF curves to be used with extreme caution. Moreover, while the MECP references the CCDP under the guide titled "Considering climate change in the environmental assessment process", it does not endorse the contents of the material. This reference is deemed inappropriate to be used in our region at this time.

MTO IDF Curve Lookup System

The University of Waterloo was commissioned by the Design and Contract Standards Office of the Ministry of Transportation of Ontario (MTO) to update the intensity–duration–frequency (IDF) curves that are used to estimate design storms for drainage infrastructure.

The IDF Curve Lookup tool uses the Waterloo Multiple Physiographic Parameter Regression (WATMAPPR) model (Seglenieks 2009), which is based on the square grid technique (Solomon et al. 1968), to estimate the curve parameters. The square grid technique uses UTM 10 km grid squares as elementary subcatchments. The premise of the tool is that local and regional topography strongly influenced local climate. Thus, topographic parameters are useful interpolators of surface fields of interest, such as temperature, runoff and, in this case, IDF curve AB parameters.

Detailed information about the system notes the following:

> *This project does not address the spatial variability of time trends for extreme precipitation in Ontario. The analysis combines the datasets from all stations and determines their collective historical trend. The projections are extrapolations based on past trends and assume that the rate of change will stay constant. This serves two purposes. For now, the extrapolations provide a better projection of future precipitation extremes than a stationary model. In the future, the extrapolation will serve as a baseline for forecasts that incorporate both climatological factors and local variability.*

The tables below illustrate the minor variance in rainfall depth across the region as measured by the MTO IDF curve tool at 9 locations – Windsor Airport, LaSalle, Tecumseh, Belle River, ERCA office, Amherstburg, Harrow, Kingsville, Leamington. In summary, there is little variance across the region but there is a notable increase in rainfall amounts (e.g. 24 hr rainfall of 108mm at Windsor Airport versus 137mm for the region based on MTO IDF curves).

Table A-3.9.1c – Present Day (Ref. 2010) Rainfall Depths for 9 Locations Throughout the Region

	100 Year Rainfall Depths (mm)			
	10 min	15 min	30 min	24 hour
Average	30.8	34.8	42.8	137.3
Maximum	31.0	35.0	43.1	138.2
Minimum	30.6	34.5	42.5	136.4
Variance	0.4	0.5	0.6	1.8

Table A-3.9.1d – Projected 2090 Rainfall Depths for 9 Locations Throughout the Region

	100 Year Rainfall Depths (mm)			
	10 min	15 min	30min	24 hour
Average	31.7	35.9	44.4	148.3
Maximum	31.9	36.1	44.7	148.8
Minimum	31.5	35.6	44.1	146.4
Variance	0.4	0.5	0.6	2.4

Conclusion: The MTO IDF curve tool considers datasets from all stations within the region. It is difficult to ascertain how the tool is combining the datasets. Moreover, the assumption that future rate of change will remain constant to that of past trends somewhat contradicts the previously mentioned warning from the regional IDF study which noted that trend results should be taken with caution in part because there is no objective way to discriminate trends among climatic trends, anthropogenic caused changes and sampling variability. Put simply, the trend results may be related to climate change and/or may be a matter of spatial variability of extreme events. A perfect example of sampling variability affected by spatial variation of extreme storm events is illustrated in section A-3.9.2.

Comparison of IDF Curves

From a SWM practitioner's perspective, there are generally two parameters of importance when considering climate change and its potential impacts to IDF curves. They are, the 5 min to 30 min rainfall timestep that will define peak rainfall intensity of the design storm rainfall hyetograph and the 24 hour rainfall depth that will define the design rainfall amount. The return periods of interest are generally limited to the minor and major storm events, typically 5-year and 100-year return periods, respectively. That being said, the following table summarizes rainfall intensities and amounts from the various sources discussed in this section. The Regional Study values are based on the 50% percentile curves.

Table A-3.9.1e – Windsor Airport Rainfall Comparison

Duration	5-year Rainfall Intensity (mm/hr)			100-year Rainfall Intensity (mm/hr)		
	EC 2007	MTO 2010	Regional IDF Study 2030	**EC 2007**	MTO 2010	Regional IDF Study 2030
5min	**143.2**	180.6	-	**227.1**	299.3	-
10min	**103.3**	111.3	-	**160.8**	184.4	-
15min	**87.9**	83.8	86.4	**142.5**	138.9	172.7
30min	**58.7**	51.6	56.9	**98.0**	85.6	101.1
Duration	5-year Rainfall Depth (mm)			100-year Rainfall Depth (mm)		
	EC 2007	MTO 2010	Regional IDF Study 2030	**EC 2007**	MTO 2010	Regional IDF Study 2030
24 hour	**68.0**	82.8	65.2	**107.9**	137.2	104.5

Table A-3.9.1f – Projected 2090 Windsor Airport Rainfall Comparison

Duration	5-year Rainfall Intensity (mm/hr)			100-year Rainfall Intensity (mm/hr)		
	EC 2007	MTO 2090	Regional IDF Study 2090	**EC 2007**	MTO 2090	Regional IDF Study 2090
5min	**143.2**	188.2	-	**227.1**	306.9	-
10min	**103.3**	116.7	-	**160.8**	189.8	-
15min	**87.9**	88.2	106.6	**142.5**	143.3	229.7
30min	**58.7**	54.8	68.6	**98.0**	88.7	141.3

Duration	5-year Rainfall Depth (mm)			100-year Rainfall Depth (mm)		
	EC 2007	MTO 2090	Regional IDF Study 2090	**EC 2007**	MTO 2090	Regional IDF Study 2090
24 hour	**68.0**	93.6	77.8	**107.9**	148.8	147.1

Conclusion: While the three sources display some consistencies, the overall impression from the above tables is that the IDF curves are variable and uncertain. To re-iterate one of the key regional IDF study recommendations, it was recommended that further study is needed before major change in infrastructure design standards. These standards recommend that, for now, the long-standing historical rainfall data from Windsor Airport continue to be used for design standards. Notwithstanding, the recent extreme events experienced in the region warrant some consideration with regards to assessing resiliency and vulnerabilities of existing stormwater systems and future designs under extreme rainfall events. This is discussed in the following section.

Historical Extremes

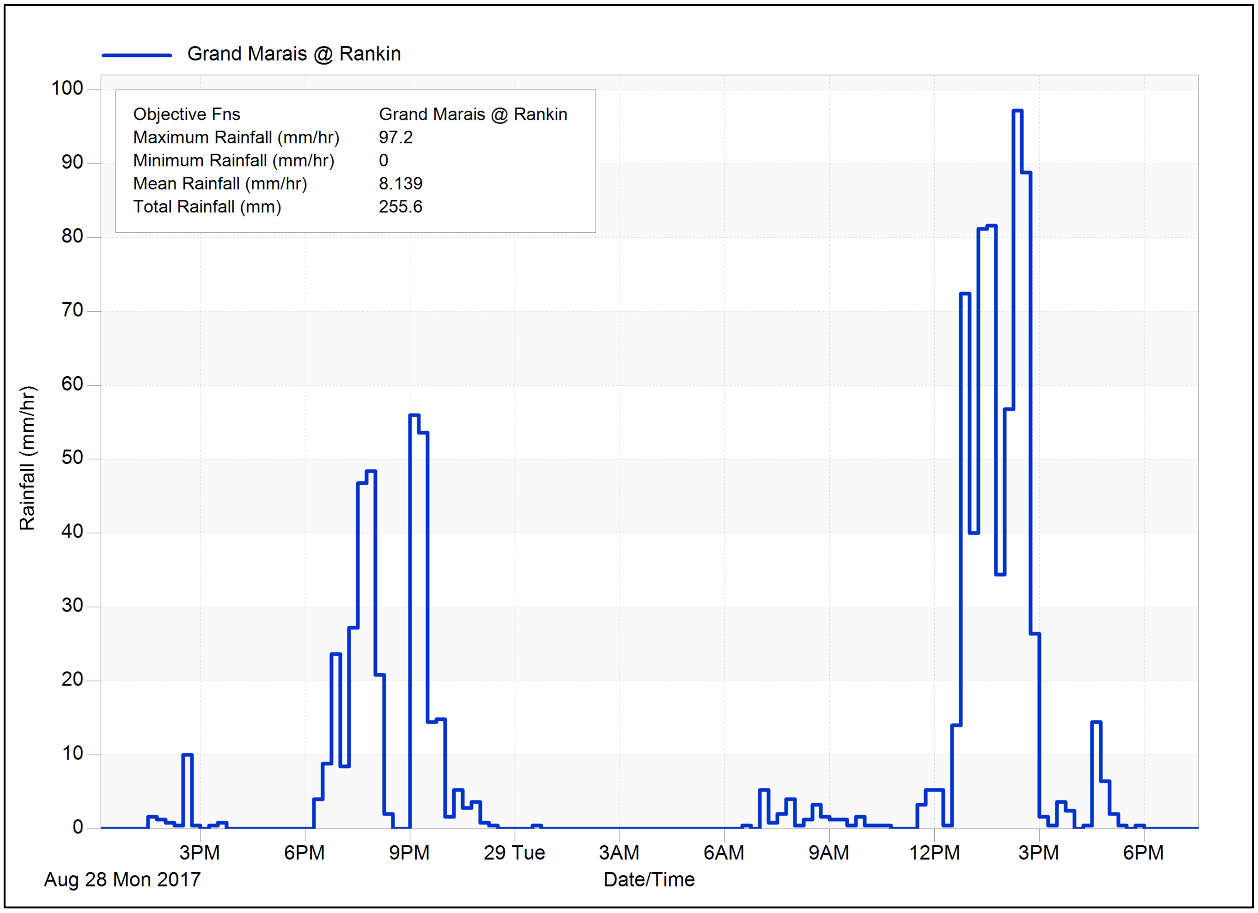
From our region's historical rainfall dataset (included in Appendix C – references), the Point Pelee and Harrow stations have combined for three occurences where 24 hour rainfalls exceeded Windsor Airport's 1:100 year rainfall of 108mm – Point Pelee station recorded 114mm (1989) and Harrow station recorded 121mm (1966) and 263mm (1989).

Recent rainfall data from flooding events in the Tecumseh, Lakeshore and Windsor areas suggest that northern areas of the region have also experienced greater than 108mm in 24 hours. Included in Appendix C are uncalibrated radar rainfall maps of these events with available rain gauge measured depths for comparison. While there is a level of uncertainty with radar estimates, and sometimes with rain gauge estimates, the maps are helpful for the purposes of assessing spatial variation of rainfall over the region. As illustrated in the maps, the core of these three storm events does not pass over the Windsor Airport climate station – a condition that demonstrates the spatial variability of the recent extreme events as well as the sampling variability of the historical rainfall record at Windsor Airport.

The maps also show 24 hour rainfall amounts between 125mm and 150mm over a significant aerial extent (i.e. the large rainfall amounts are not isolated micro-bursts of small spatial extent measured at one individual rain gauge).

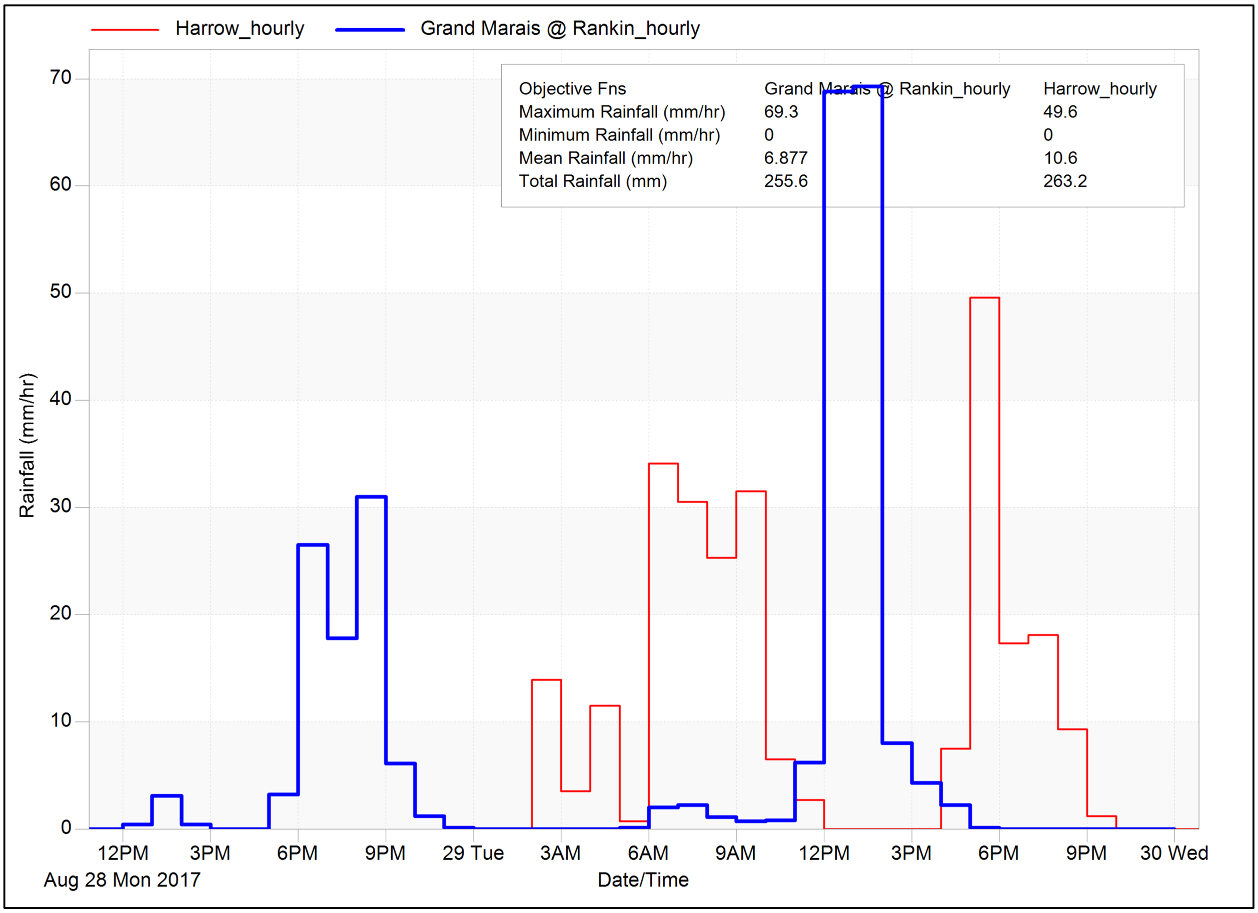
Graph A-3.9.1a depicts the rainfall hyetograph for the August 2017 storm as recorded by ERCA's Grand Marais rain gauge @ Rankin (GMr Gauge as shown on rainfall maps). This location captured the eye of the storm and generally represents the most intense rainfall that was experienced. It is believed that this storm intensity exceeded all known historical records of extreme events in the region, including the Harrow storm of 1989. While the storm significantly exceeded a 1:100 year return period for all durations from 1 hour to 24 hour, the 15 min rainfall was equivalent to a 1:8 year storm and the 30 min equivalent to a 1:68 year. Therefore, while the storm was severe, the short-term intensity of this extreme event did not exceed the current 1:100 year intensity as defined by Windsor Airport rainfall data.

Graph A-3.9.1a – August 2017 Storm Measured from Grand Marais Rain Gauge @ Rankin



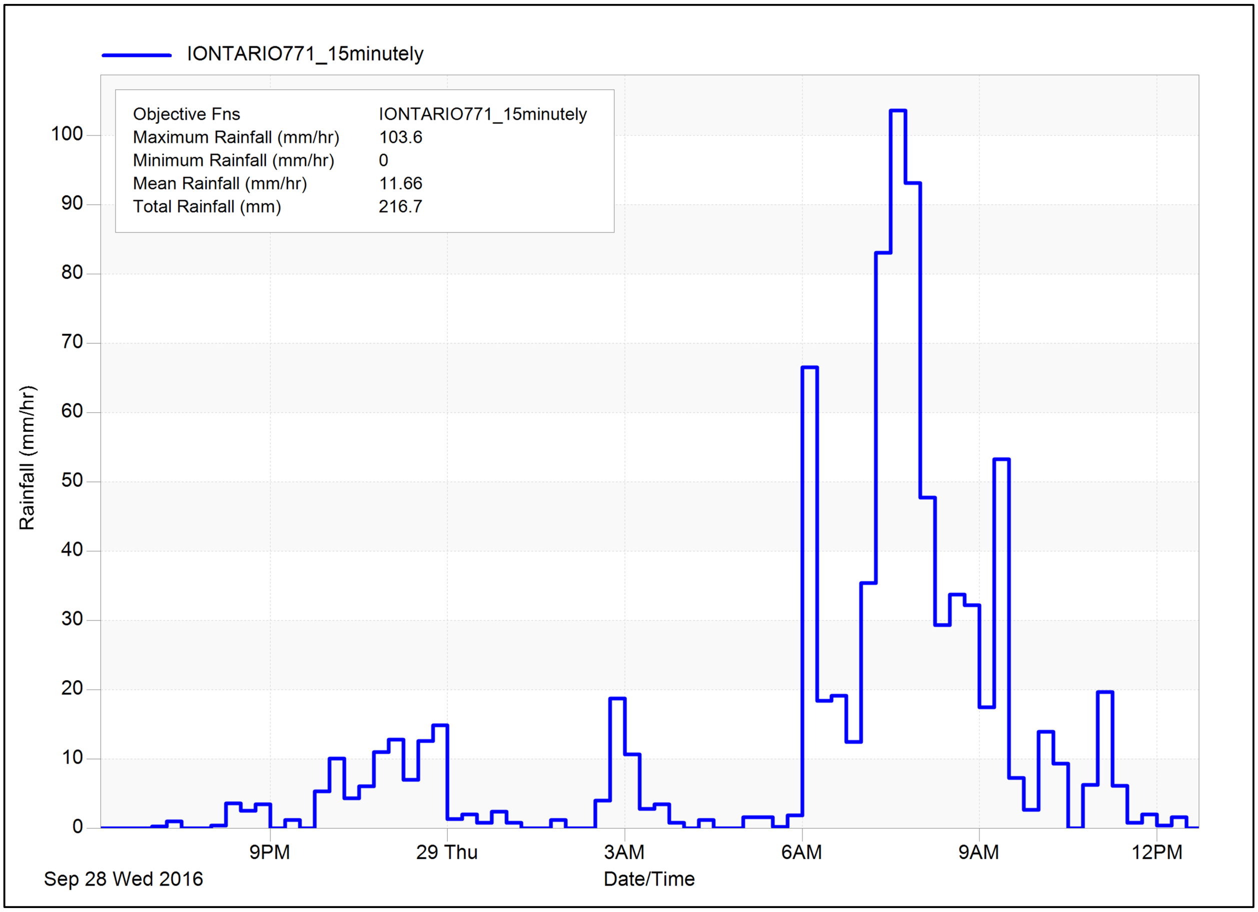
Graph A-3.9.1b depicts the rainfall hyetograph for August 2017 storm as recorded by ERCA's the Grand Marais rain gauge @ Rankin Graph compared to the July 1989 as recorded by Environment Canada's Harrow Station rain gauge. The Harrow storm has been transposed for comparison purposes.

Graph A-3.9.1b – August 2017 Storm Measured from Grand Marais Rain Gauge @ Rankin Compared to July 1989 Harrow Station Rain Gauge (Transposed for Comparison)



Graph A-3.9.1c depicts the rainfall hyetograph for the September 2016 storm as recorded a private gauge in the vicinity of Revland and St. Thomas – approximately 1,100 metres northwest of Manning and Tecumseh intersection. This storm also significantly exceeded a 1:100 year return period for all durations from 1 hour to 24 hour. The 15 min rainfall was equivalent to a 1:11 year storm and the 30 min equivalent to a 1:103 year. Again, while the storm was severe, the short-term intensity of this extreme event was less than or relatively equal to the current 1:100 year intensity as defined by Windsor Airport rainfall data.

Graph A-3.9.1c – September 2016 Storm Measured @ IONTARIO771 Rain Gauge Near Manning & Tecumseh



The extreme storms in September 2016 and August 2017 both appeared to stall over the Windsor, Tecumseh and Lakeshore regions. It is said that these extreme events may have been the result of two separate storms tracking from different directions and combining to create a phenomenon that seemingly dropped all of the storm's moisture over the region. The above commentary is speculative at this time. Further study from qualified experts in the field of both meteorology and climatology is recommended to provide guidance on how these extreme events fit within the context of stormwater design based on single station historical rainfall data.

Conclusion: As recommended in the regional IDF study discussed in section 3.9.2, further study is needed before major change in infrastructure design standards. Moreover, the suggested phenomenon that appeared to have stalled both extreme events in not well understood, at least not by the technical contributors of these standards. It would be a logical next step to attempt to better understand the conditions that led to these recent extreme events and the probability of recurrence for similar conditions. Has this occurred previously? Did a similar phenomenon lead to the extreme rainfall of the 1989 Harrow Storm?

As further studies and science evolves to provide clear guidance on stormwater design standards, the recurring extreme events of the last couple of years warrant that the current standards account for some level of rainfall increase to evaluate the resiliency and vulnerabilities of stormwater systems.

A-3.9.2 Beyond IDF Curves

Climate change may have had implications on the severity of the extreme events experienced in September 2016 and August 2017. However, there exists many other causes of flooding. One example of this is illustrated by the September 2016 storm event.

Graph A-3.9.2 depicts the rainfall hyetograph for the September 2016 storm as recorded a private gauge (IONTARIO771) in the vicinity of Revland and St. Thomas – approximately 1,100 metres northwest of Manning and Tecumseh intersection compared to the rainfall recorded by the City of Windsor's Wellington PS gauge. Recall, the rainfall at the private gauge exceeded the 1:100 year rainfall amounts for most durations.

The rainfall measured at the Wellington PS was much less intense with a 1:2 year return period across all duration except for the 12 hour and 24 hour where return periods were 1:4 year and 1:9 year, respectively. <u>These two rainfall hyetographs at opposite ends of the City clearly demonstrate the spatial variation of the September 2016 storm.</u> This spatial variation, combined with the information provided by another private gauge recording near the Wellington gauge, shines an important light on the perception that climate change is the culprit for all recent flooding.

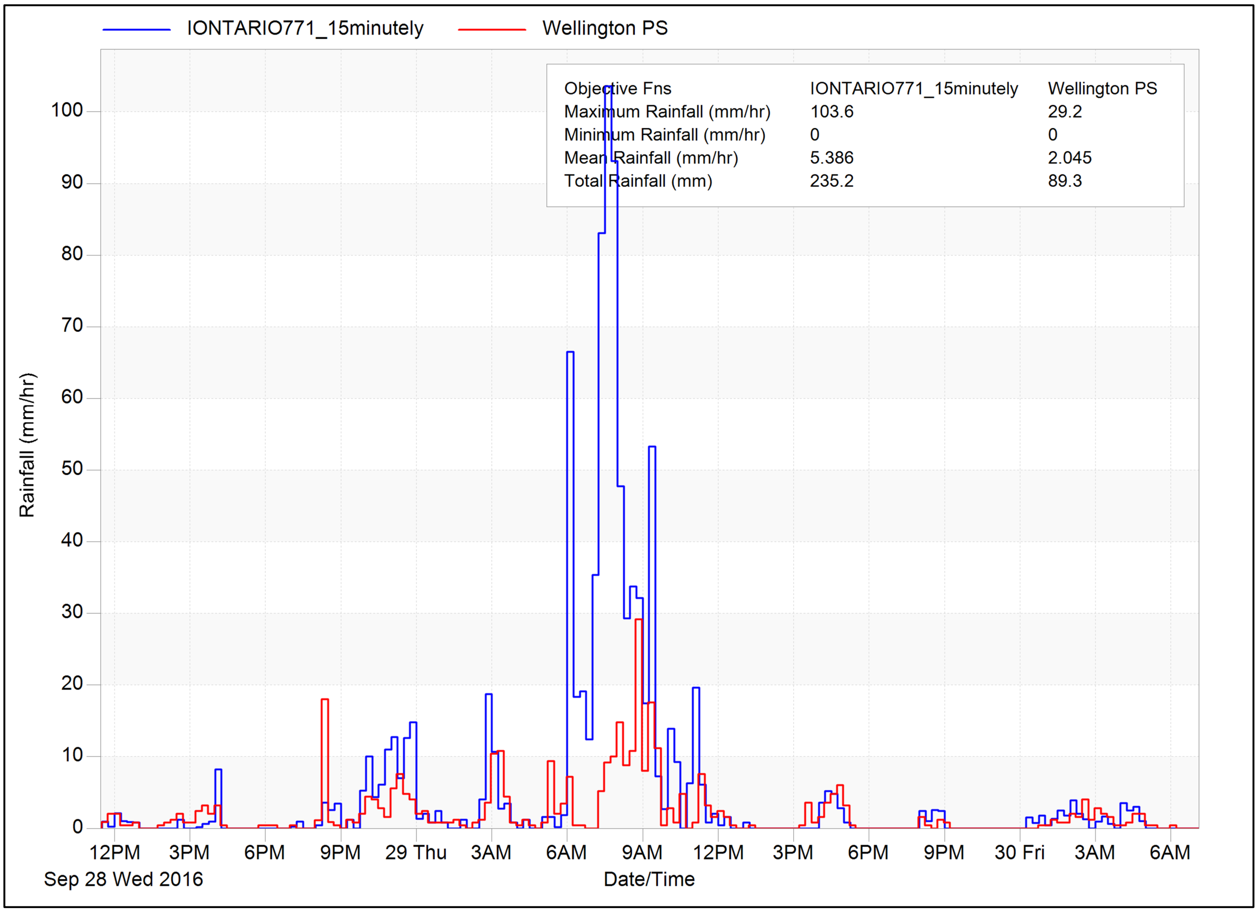
The private gauge recording is located near Randolph and Totten, approximately 1.2 kms away from the Wellington gauge. From 12pm on September 28th to 7am on September 30th the Wellington gauge measured 89mm whereas the private (CoCoRaHS) gauge measured 86mm in that time.

CoCoRaHS stands for **Co**mmunity **Co**llaborative **Ra**in, **H**ail and **S**now – a grassroots volunteer network of backyard weather observers of all ages and backgrounds working together to measure and map precipitation (rain, hail and snow) in their local communities.

The private CoCoRaHS gauge measures volume only and records daily amount at approximately 7am. At 7:21am on the 29th, the gauge measured 39.6mm and the observer noted that it was "pouring.." At 7:39am on the 30th, the gauge measured 46.5mm and noted the following; "State of Emergency in the Neighbourhood due to Flooding. Right now is a RainPause". Assuming a similar distribution between the two gauges – which can be reasonably assumed given the proximity of the gauges combined with the close match in rainfall volume –

the area of the private gauge experienced relatively low intensity storm which is not typically conducive of flooding.

Graph A-3.9.2 – IONTARIO771 Rain Gauge Near Manning & Tecumseh Compared to City of Windsor Wellington Gauge Near Crawford and Tecumseh



At a cursory level, the foregoing strongly suggests that flooding in the west side of the City was not related to extreme rainfall or climate change. It is speculated that residents experiencing flooding in the west side of the City may have equated their rain to the media reporting of the much more intense and noteworthy rainfall at the east side of Windsor and Town of Tecumseh. Thus, fueling the bias that climate change is the culprit of all recent flooding.

APPENDIX B

Design Storm Distributions

1. Water Quality Storm
2. Chicago 2-year 4-hour Storm
3. Chicago 5-year 4-hour Storm
4. Chicago 10-year 4-hour Storm
5. Chicago 25-year 4-hour Storm
6. Chicago 50-year 4-hour Storm
7. Chicago 100-year 4-hour Storm
8. SCS Type II 24-hour Storm
9. Urban Stress Test Storm

WATER QUALITY STORM

CHICAGO 2-YEAR 4-HOUR
Adjusted Depth = 32.0 mm (90% percentile runoff volume control target)

Time h:mm	5min Rain mm/hr
0:00	1.68
0:05	1.77
0:10	1.88
0:15	1.99
0:20	2.13
0:25	2.28
0:30	2.46
0:35	2.68
0:40	2.94
0:45	3.26
0:50	3.67
0:55	4.21
1:00	4.94
1:05	6.00
1:10	7.67
1:15	10.65
1:20	17.28
1:25	40.48
1:30	94.95
1:35	37.90
1:40	21.47
1:45	14.71
1:50	11.11
1:55	8.91
2:00	7.44
2:05	6.39
2:10	5.60
2:15	4.99
2:20	4.50
2:25	4.11
2:30	3.78
2:35	3.50
2:40	3.26
2:45	3.06
2:50	2.88
2:55	2.72
3:00	2.58
3:05	2.45
3:10	2.34
3:15	2.23
3:20	2.14
3:25	2.06
3:30	1.98
3:35	1.91
3:40	1.84
3:45	1.78
3:50	1.72
3:55	1.67
4:00	0.00

Time h:mm	10min Rain mm/hr
0:00	1.73
0:10	1.94
0:20	2.21
0:30	2.57
0:40	3.10
0:50	3.94
1:00	5.47
1:10	9.16
1:20	23.89
1:30	71.41
1:40	18.09
1:50	10.01
2:00	6.91
2:10	5.30
2:20	4.31
2:30	3.64
2:40	3.16
2:50	2.80
3:00	2.51
3:10	2.29
3:20	2.10
3:30	1.94
3:40	1.81
3:50	1.69
4:00	0.00

Time h:mm	15min Rain mm/hr
0:00	1.78
0:15	2.13
0:30	2.70
0:45	3.72
1:00	6.21
1:15	16.41
1:30	57.83
1:45	11.58
2:00	6.48
2:15	4.53
2:30	3.51
2:45	2.88
3:00	2.45
3:15	2.14
3:30	1.91
3:45	1.72
4:00	0.00

Time h:mm	20min Rain mm/hr
0:00	1.83
0:20	2.39
0:40	3.52
1:00	7.32
1:20	48.91
1:40	12.79
2:00	6.11
2:20	3.97
2:40	2.98
3:00	2.40
3:20	2.02
3:40	1.75
4:00	0.00

Time h:mm	30min Rain mm/hr
0:00	1.96
0:30	3.21
1:00	8.21
1:30	37.80
2:00	5.51
2:30	3.20
3:00	2.30
3:30	1.81
4:00	0.00

2-YEAR DESIGN STORMS

CHICAGO 4-HOUR
Depth = 37.7 mm

Time h:mm	5min Rain mm/hr
0:00	1.98
0:05	2.09
0:10	2.21
0:15	2.35
0:20	2.51
0:25	2.69
0:30	2.90
0:35	3.16
0:40	3.47
0:45	3.85
0:50	4.33
0:55	4.96
1:00	5.82
1:05	7.07
1:10	9.04
1:15	12.54
1:20	20.36
1:25	47.69
1:30	111.86
1:35	44.65
1:40	25.30
1:45	17.33
1:50	13.09
1:55	10.50
2:00	8.77
2:05	7.53
2:10	6.60
2:15	5.88
2:20	5.31
2:25	4.84
2:30	4.45
2:35	4.12
2:40	3.84
2:45	3.60
2:50	3.39
2:55	3.20
3:00	3.04
3:05	2.89
3:10	2.75
3:15	2.63
3:20	2.52
3:25	2.42
3:30	2.33
3:35	2.25
3:40	2.17
3:45	2.09
3:50	2.03
3:55	1.96
4:00	0.00

Time h:mm	10min Rain mm/hr
0:00	2.04
0:10	2.28
0:20	2.60
0:30	3.03
0:40	3.66
0:50	4.64
1:00	6.45
1:10	10.79
1:20	28.15
1:30	84.13
1:40	21.31
1:50	11.80
2:00	8.15
2:10	6.24
2:20	5.07
2:30	4.29
2:40	3.72
2:50	3.30
3:00	2.96
3:10	2.69
3:20	2.47
3:30	2.29
3:40	2.13
3:50	2.00
4:00	0.00

Time h:mm	15min Rain mm/hr
0:00	2.10
0:15	2.52
0:30	3.18
0:45	4.38
1:00	7.31
1:15	19.33
1:30	68.13
1:45	13.64
2:00	7.63
2:15	5.34
2:30	4.14
2:45	3.40
3:00	2.89
3:15	2.53
3:30	2.25
3:45	2.03
4:00	0.00

Time h:mm	20min Rain mm/hr
0:00	2.16
0:20	2.81
0:40	4.15
1:00	8.62
1:20	57.62
1:40	15.07
2:00	7.19
2:20	4.68
2:40	3.51
3:00	2.83
3:20	2.38
3:40	2.06
4:00	0.00

Time h:mm	30min Rain mm/hr
0:00	2.31
0:30	3.78
1:00	9.68
1:30	44.53
2:00	6.49
2:30	3.77
3:00	2.71
3:30	2.14
4:00	0.00

5-YEAR DESIGN STORMS

CHICAGO 4-HOUR
Depth = 49.5 mm

Time h:mm	5min Rain mm/hr
0:00	2.44
0:05	2.58
0:10	2.73
0:15	2.91
0:20	3.12
0:25	3.36
0:30	3.65
0:35	3.99
0:40	4.41
0:45	4.92
0:50	5.59
0:55	6.46
1:00	7.66
1:05	9.42
1:10	12.20
1:15	17.18
1:20	28.20
1:25	64.52
1:30	139.58
1:35	60.83
1:40	35.06
1:45	23.95
1:50	17.96
1:55	14.28
2:00	11.81
2:05	10.06
2:10	8.75
2:15	7.74
2:20	6.94
2:25	6.29
2:30	5.76
2:35	5.30
2:40	4.92
2:45	4.59
2:50	4.30
2:55	4.05
3:00	3.83
3:05	3.63
3:10	3.45
3:15	3.29
3:20	3.14
3:25	3.01
3:30	2.89
3:35	2.78
3:40	2.67
3:45	2.58
3:50	2.49
3:55	2.41
4:00	0.00

Time h:mm	10min Rain mm/hr
0:00	2.51
0:10	2.82
0:20	3.24
0:30	3.82
0:40	4.67
0:50	6.02
1:00	8.54
1:10	14.69
1:20	38.85
1:30	107.72
1:40	29.51
1:50	16.12
2:00	10.93
2:10	8.25
2:20	6.62
2:30	5.53
2:40	4.76
2:50	4.18
3:00	3.73
3:10	3.37
3:20	3.08
3:30	2.83
3:40	2.63
3:50	2.45
4:00	0.00

Time h:mm	15min Rain mm/hr
0:00	2.58
0:15	3.13
0:30	4.02
0:45	5.66
1:00	9.76
1:15	26.72
1:30	88.40
1:45	18.73
2:00	10.21
2:15	6.99
2:30	5.33
2:45	4.31
3:00	3.64
3:15	3.15
3:30	2.78
3:45	2.49
4:00	0.00

Time h:mm	20min Rain mm/hr
0:00	2.66
0:20	3.53
0:40	5.34
1:00	11.61
1:20	75.35
1:40	20.75
2:00	9.59
2:20	6.07
2:40	4.47
3:00	3.55
3:20	2.95
3:40	2.54
4:00	0.00

Time h:mm	30min Rain mm/hr
0:00	2.86
0:30	4.84
1:00	13.11
1:30	58.69
2:00	8.60
2:30	4.82
3:00	3.39
3:30	2.64
4:00	0.00

10-YEAR DESIGN STORMS

CHICAGO 4-HOUR
Depth = 57.0 mm

Time h:mm	5min Rain mm/hr
0:00	2.74
0:05	2.90
0:10	3.09
0:15	3.29
0:20	3.53
0:25	3.81
0:30	4.15
0:35	4.54
0:40	5.03
0:45	5.63
0:50	6.41
0:55	7.43
1:00	8.85
1:05	10.92
1:10	14.21
1:15	20.11
1:20	33.09
1:25	74.91
1:30	157.73
1:35	70.81
1:40	41.12
1:45	28.10
1:50	21.03
1:55	16.68
2:00	13.76
2:05	11.68
2:10	10.14
2:15	8.94
2:20	8.00
2:25	7.24
2:30	6.61
2:35	6.08
2:40	5.63
2:45	5.24
2:50	4.91
2:55	4.61
3:00	4.35
3:05	4.12
3:10	3.91
3:15	3.73
3:20	3.56
3:25	3.40
3:30	3.26
3:35	3.14
3:40	3.02
3:45	2.91
3:50	2.81
3:55	2.71
4:00	0.00

Time h:mm	10min Rain mm/hr
0:00	2.82
0:10	3.19
0:20	3.67
0:30	4.34
0:40	5.33
0:50	6.92
1:00	9.89
1:10	17.16
1:20	45.47
1:30	122.80
1:40	34.61
1:50	18.86
2:00	12.72
2:10	9.54
2:20	7.62
2:30	6.34
2:40	5.44
2:50	4.76
3:00	4.24
3:10	3.82
3:20	3.48
3:30	3.20
3:40	2.96
3:50	2.76
4:00	0.00

Time h:mm	15min Rain mm/hr
0:00	2.91
0:15	3.55
0:30	4.57
0:45	6.49
1:00	11.33
1:15	31.33
1:30	101.26
1:45	21.94
2:00	11.86
2:15	8.06
2:30	6.11
2:45	4.92
3:00	4.13
3:15	3.56
3:30	3.14
3:45	2.81
4:00	0.00

Time h:mm	20min Rain mm/hr
0:00	3.01
0:20	4.01
0:40	6.13
1:00	13.52
1:20	86.55
1:40	24.32
2:00	11.13
2:20	6.98
2:40	5.10
3:00	4.03
3:20	3.34
3:40	2.86
4:00	0.00

Time h:mm	30min Rain mm/hr
0:00	3.23
0:30	5.53
1:00	15.30
1:30	67.63
2:00	9.96
2:30	5.51
3:00	3.85
3:30	2.97
4:00	0.00

25-YEAR DESIGN STORMS

CHICAGO 4-HOUR
Depth = 67.0 mm

Time h:mm	5min Rain mm/hr
0:00	3.15
0:05	3.34
0:10	3.55
0:15	3.80
0:20	4.08
0:25	4.41
0:30	4.81
0:35	5.28
0:40	5.86
0:45	6.58
0:50	7.50
0:55	8.73
1:00	10.44
1:05	12.93
1:10	16.90
1:15	24.01
1:20	39.58
1:25	88.69
1:30	182.17
1:35	84.02
1:40	49.16
1:45	33.62
1:50	25.13
1:55	19.87
2:00	16.35
2:05	13.84
2:10	11.98
2:15	10.55
2:20	9.42
2:25	8.50
2:30	7.74
2:35	7.11
2:40	6.57
2:45	6.11
2:50	5.71
2:55	5.36
3:00	5.05
3:05	4.78
3:10	4.53
3:15	4.31
3:20	4.11
3:25	3.93
3:30	3.76
3:35	3.61
3:40	3.47
3:45	3.34
3:50	3.22
3:55	3.11
4:00	0.00

Time h:mm	10min Rain mm/hr
0:00	3.24
0:10	3.68
0:20	4.25
0:30	5.04
0:40	6.22
0:50	8.12
1:00	11.69
1:10	20.45
1:20	54.26
1:30	142.97
1:40	41.39
1:50	22.50
2:00	15.10
2:10	11.27
2:20	8.96
2:30	7.43
2:40	6.34
2:50	5.54
3:00	4.92
3:10	4.42
3:20	4.02
3:30	3.69
3:40	3.41
3:50	3.17
4:00	0.00

Time h:mm	15min Rain mm/hr
0:00	3.35
0:15	4.10
0:30	5.31
0:45	7.61
1:00	13.42
1:15	37.46
1:30	118.42
1:45	26.21
2:00	14.06
2:15	9.49
2:30	7.14
2:45	5.73
3:00	4.79
3:15	4.12
3:30	3.62
3:45	3.23
4:00	0.00

Time h:mm	20min Rain mm/hr
0:00	3.46
0:20	4.65
0:40	7.17
1:00	16.07
1:20	101.49
1:40	29.07
2:00	13.18
2:20	8.19
2:40	5.94
3:00	4.67
3:20	3.85
3:40	3.29
4:00	0.00

Time h:mm	30min Rain mm/hr
0:00	3.72
0:30	6.46
1:00	18.21
1:30	79.54
2:00	11.77
2:30	6.44
3:00	4.45
3:30	3.42
4:00	0.00

50-YEAR DESIGN STORMS

CHICAGO 4-HOUR
Depth = 73.9 mm

Time h:mm	5min Rain mm/hr
0:00	3.39
0:05	3.60
0:10	3.83
0:15	4.10
0:20	4.42
0:25	4.78
0:30	5.22
0:35	5.74
0:40	6.38
0:45	7.18
0:50	8.21
0:55	9.58
1:00	11.48
1:05	14.27
1:10	18.71
1:15	26.68
1:20	44.13
1:25	98.55
1:30	200.17
1:35	93.46
1:40	54.82
1:45	37.46
1:50	27.94
1:55	22.05
2:00	18.10
2:05	15.29
2:10	13.21
2:15	11.60
2:20	10.34
2:25	9.31
2:30	8.47
2:35	7.77
2:40	7.17
2:45	6.66
2:50	6.22
2:55	5.83
3:00	5.49
3:05	5.18
3:10	4.91
3:15	4.67
3:20	4.45
3:25	4.25
3:30	4.07
3:35	3.90
3:40	3.75
3:45	3.61
3:50	3.47
3:55	3.35
4:00	0.00

Time h:mm	10min Rain mm/hr
0:00	3.50
0:10	3.97
0:20	4.60
0:30	5.48
0:40	6.78
0:50	8.89
1:00	12.87
1:10	22.70
1:20	60.47
1:30	157.69
1:40	46.14
1:50	24.99
2:00	16.69
2:10	12.40
2:20	9.83
2:30	8.12
2:40	6.92
2:50	6.02
3:00	5.34
3:10	4.79
3:20	4.35
3:30	3.98
3:40	3.68
3:50	3.41
4:00	0.00

Time h:mm	15min Rain mm/hr
0:00	3.61
0:15	4.43
0:30	5.78
0:45	8.32
1:00	14.82
1:15	41.74
1:30	130.87
1:45	29.15
2:00	15.53
2:15	10.42
2:30	7.80
2:45	6.24
3:00	5.19
3:15	4.46
3:30	3.90
3:45	3.48
4:00	0.00

Time h:mm	20min Rain mm/hr
0:00	3.73
0:20	5.04
0:40	7.84
1:00	17.79
1:20	112.29
1:40	32.35
2:00	14.55
2:20	8.97
2:40	6.47
3:00	5.06
3:20	4.17
3:40	3.55
4:00	0.00

Time h:mm	30min Rain mm/hr
0:00	4.02
0:30	7.05
1:00	20.19
1:30	88.10
2:00	12.97
2:30	7.02
3:00	4.82
3:30	3.69
4:00	0.00

100-YEAR DESIGN STORMS

CHICAGO 4-HOUR
Depth = 81.6 mm

Time h:mm	5min Rain mm/hr
0:00	3.71
0:05	3.94
0:10	4.20
0:15	4.50
0:20	4.85
0:25	5.25
0:30	5.73
0:35	6.31
0:40	7.03
0:45	7.92
0:50	9.07
0:55	10.59
1:00	12.72
1:05	15.84
1:10	20.81
1:15	29.71
1:20	49.12
1:25	108.91
1:30	218.23
1:35	103.42
1:40	60.97
1:45	41.72
1:50	31.11
1:55	24.53
2:00	20.12
2:05	16.98
2:10	14.65
2:15	12.86
2:20	11.44
2:25	10.30
2:30	9.36
2:35	8.58
2:40	7.91
2:45	7.34
2:50	6.85
2:55	6.42
3:00	6.04
3:05	5.70
3:10	5.40
3:15	5.13
3:20	4.88
3:25	4.66
3:30	4.46
3:35	4.27
3:40	4.10
3:45	3.95
3:50	3.80
3:55	3.67
4:00	0.00

Time h:mm	10min Rain mm/hr
0:00	3.83
0:10	4.35
0:20	5.05
0:30	6.02
0:40	7.47
0:50	9.83
1:00	14.28
1:10	25.26
1:20	67.16
1:30	172.68
1:40	51.34
1:50	27.82
2:00	18.55
2:10	13.75
2:20	10.87
2:30	8.97
2:40	7.63
2:50	6.63
3:00	5.87
3:10	5.26
3:20	4.77
3:30	4.37
3:40	4.03
3:50	3.74
4:00	0.00

Time h:mm	15min Rain mm/hr
0:00	3.95
0:15	4.87
0:30	6.36
0:45	9.19
1:00	16.45
1:15	46.45
1:30	143.67
1:45	32.45
2:00	17.25
2:15	11.53
2:30	8.62
2:45	6.87
3:00	5.71
3:15	4.89
3:30	4.28
3:45	3.81
4:00	0.00

Time h:mm	20min Rain mm/hr
0:00	4.09
0:20	5.54
0:40	8.65
1:00	19.77
1:20	123.48
1:40	36.02
2:00	16.15
2:20	9.92
2:40	7.13
3:00	5.56
3:20	4.57
3:40	3.88
4:00	0.00

Time h:mm	30min Rain mm/hr
0:00	4.41
0:30	7.78
1:00	22.45
1:30	97.06
2:00	14.39
2:30	7.74
3:00	5.30
3:30	4.04
4:00	0.00

SCS TYPE II 24-HOUR DESIGN STORMS

		Unit Rainfall Depth = 1 mm	100-Year Depth = 108 mm	Rural Stress Test Depth = 150 mm	5-Year Depth = 68.0 mm	V2 2-Year Depth = 53.4 mm
Time h:mm	**Rain %**	**2hour Rain mm/hr**	**2hour Rain mm/hr**	**2hour Rain mm/hr**	**2hour Rain mm/hr**	**2hour Rain mm/hr**
0:00	0	0.000	0.00	0.00	0.00	0.00
2:00	2	0.010	1.08	1.50	0.68	0.53
4:00	3	0.015	1.62	2.25	1.02	0.80
6:00	3	0.015	1.62	2.25	1.02	0.80
8:00	4	0.020	2.16	3.00	1.36	1.07
10:00	6	0.030	3.24	4.50	2.04	1.60
12:00	48	0.240	25.92	36.00	16.32	12.82
14:00	16	0.080	8.64	12.00	5.44	4.27
16:00	6	0.030	3.24	4.50	2.04	1.60
18:00	4	0.020	2.16	3.00	1.36	1.07
20:00	3	0.015	1.62	2.25	1.02	0.80
22:00	3	0.015	1.62	2.25	1.02	0.80
0:00	2	0.010	1.08	1.50	0.68	0.53

^{V2} URBAN STRESS TEST STORM

CHICAGO 100-YEAR 24-HOUR (108 mm) + UNIFORM DISTRIBUTION OF ADDITIONAL 42 mm Depth = 108 mm + 42 mm = 150 mm			
Time h:mm	**15min Rain mm/hr**	**Time h:mm**	**15min Rain mm/hr**
0:00	2.41	12:15	4.42
0:15	2.43	12:30	4.24
0:30	2.45	12:45	4.08
0:45	2.47	13:00	3.94
1:00	2.49	13:15	3.82
1:15	2.51	13:30	3.71
1:30	2.53	13:45	3.61
1:45	2.56	14:00	3.52
2:00	2.59	14:15	3.44
2:15	2.61	14:30	3.37
2:30	2.65	14:45	3.31
2:45	2.68	15:00	3.25
3:00	2.71	15:15	3.19
3:15	2.75	15:30	3.14
3:30	2.80	15:45	3.09
3:45	2.84	16:00	3.05
4:00	2.89	16:15	3.01
4:15	2.95	16:30	2.97
4:30	3.01	16:45	2.93
4:45	3.09	17:00	2.90
5:00	3.17	17:15	2.87
5:15	3.26	17:30	2.84
5:30	3.36	17:45	2.81
5:45	3.49	18:00	2.78
6:00	3.63	18:15	2.76
6:15	3.80	18:30	2.73
6:30	4.01	18:45	2.71
6:45	4.27	19:00	2.69
7:00	4.61	19:15	2.67
7:15	5.05	19:30	2.65
7:30	5.66	19:45	2.63
7:45	6.56	20:00	2.61
8:00	8.04	20:15	2.59
8:15	10.84	20:30	2.58
8:30	18.04	20:45	2.56
8:45	48.06	21:00	2.54
9:00	145.42	21:15	2.53
9:15	34.36	21:30	2.52
9:30	19.10	21:45	2.50
9:45	13.36	22:00	2.49
10:00	10.42	22:15	2.48
10:15	8.66	22:30	2.47
10:30	7.50	22:45	2.45
10:45	6.67	23:00	2.44
11:00	6.06	23:15	2.43
11:15	5.58	23:30	2.42
11:30	5.20	23:45	2.41
11:45	4.90	0:00	0.00
12:00	4.64		

APPENDIX C

Reference Material

1. Agricultural Drain Components: *OMAFRA Design and Construction Guidelines, June 1986 – Page 14*
2. MNRF River & Stream Systems: Flooding Hazard Limit Technical Guide dated 2002 – Page 42: Rainfall Distributions (Table D-6)
3. MTO Road Surface Drainage Systems: *MTO Drainage Design Standards, January 2008 – SD-1*
4. MTO Bridge and Culvert Crossings: *MTO Drainage Design Standards, January 2008 – WC1*
5. Temporary Works during Construction: *MTO Drainage Design Standards, January 2008 – TW-1*
6. City of Pickering Drawing P-1007 – Stormwater Management Pond Warning Sign
7. Radar Rainfall Maps: Figures 1 & 2
8. Environment Canada IDF Data: Windsor A, Harrow CDA & Point Pelee CS Stations
9. MNRF River & Stream Systems: Flooding Hazard Limit Technical Guide dated 2002 – Page 27a: Flood Plain Stability Charts (Figure 6-2)
10. [V2] Curve Number Tables: Haestad Methods Inc., Stormwater Conveyance Modeling and Design – Pages 125 to 128

Table 1. Recommended Criteria for the Design of Agricultural Drain Components.

Component	Recommended Design Criteria	Method
Open Channel - Low hazard, flooding could wash out culverts, erode field corners, cause minor damage or interruption to farm operations.	2 year storm	In flat lands, average slope less than 0.5% use Agricultural curves. In rolling lands average slope more than 0.5% use Rational Method or SCS peak flow method.
Field culverts - low hazard, flooding could erode driving surface	2 year storm	Rational Method[b].
Residence or major agricultural culverts - low hazard, flooding could wash out culvert, isolate farm residence or farm commercial operation	5-10 year storm	Rational Method[b].
Township road culvert - replacement[a]	5-10 year storm	Rational Method[b].
County Road culvert replacement[a]	10-25 year storm	Rational Method[b].
MTC highway culvert - replacement[a]	see Table 2	Rational Method[b]. If area exceeds 25 km^2 use MTC Index Flood Method or hydrologic model.

[a] Obtain Road Authority's current criteria prior to design.
[b] Other hydrologic design methods may be used where appropriate.

TABLE D-6
RAINFALL DISTRIBUTIONS (PERCENT)

Type	Reference	Storm Duration	HOURS											
			1	2	3	4	5	6	7	8	9	10	11	12
Probable Maximum Storm	Small Dams	6 Hour	8	9	11	49	15	8	-	-	-	-	-	-
HAZEL	Ministry of Natural Resources	12 Hour	3	2	3	6	8	6	11	6	6	25	18	6
TIMMINS	Ministry of Natural Resources	12 Hour	8	10	6	1	3	10	23	10	12	6	7	4
Return Period Storms	SCS II	24 Hour 2 Hour Increment	2	3	3	4	6	48	16	6	4	3	3	2
	AES, 30% Southern Ontario	12 Hour	15	25	22	14	12	8	9	1	0	0	0	0
	AES, 30% Northern Ontario	12 hour	8	17	15	14	18	14	6	3	3	1	1	0
	AES, 70% Southern Ontario	12 hour	3	5	7	10	14	10	12	11	9	9	5	5
	AES, 70% Northern Ontario	12 Hour	3	5	7	10	13	12	8	10	11	11	5	5

Note: A.E.S. 30% distributions represent 70% of all storms for which the accumulated hourly rainfall was equal or less than shown. Consequently, only 30% of the storms had higher accumulated rainfall.

SD –1 Design Flows for Surface Drainage Systems

SCOPE

This standard identifies the minimum Design Flows that shall be used for the sizing of road surface drainage systems. The selected Design Flow shall be applied to size the minor and major drainage systems (piped and surface flow) for various MTO road types. This standard provides the hydrologic basis for all Surface Drainage Standards (SD-2 to SD-13).

DESIGN REFERENCES

MTO Drainage Management Manual (1997), Chapter 3.

1. HYDROLOGY

1.1 Design Flows

Design Flows for the Minor and Major highway drainage systems are as follows:

Design Flow for Minor System and Major System		
Functional Road Classifications	**Drainage System Type**	**Design Flow**
Freeway Arterial (Urban)	Minor System	10-Year
	Major System	100- Year
Arterial (Rural) Collector (Urban and Rural)	Minor System	10-Year
	Major System	100-Year
Local Road (Urban and Rural)	Minor System	5-Year
	Major System	-
Depressed Roadways (see SD-7)	Minor System	25-Year
	Major System	100-Year

1.2 Local External Catchment Areas Draining to the Highway Right-of-Way (Proposed Highway)

1.2.1 Either an overland flow route (swale, ditch or realigned watercourse) or a storm sewer system shall convey the external runoff from the point of interception to the receiving watercourse.

1.2.2 For a proposed highway, the capacity of the conveyance system shall be sufficient to convey the Major System Design Flow.

1.2.3 For an existing highway subject to modification, the capacity of the conveyance system shall be sufficient to ensure the following:

- No increase in flood risk to properties, adjacent to the highway right-of-way including significant natural areas/habitats; and
- The design of the overland flow route adheres to the requirements for the design of Roadside Ditches (Standard SD-9).

2. HYDRAULICS

The hydraulic standards are addressed in Standards SD-2 to SD-13.

3. PHYSICAL CHARACTERISTICS

There are no Physical Characteristics specific to this Standard.

4. COMMENTARY

As part of the design process the following shall be addressed:

- Include downstream capacity constraints in the design of the Minor System and the Major System. The analysis should extend as far downstream as the change in flow may have an impact on downstream erosion potential or flood risk.
- Allow for future road widening when sizing the Minor System and the Major System.
- Include interception of groundwater as a component of the conveyed flows.
- Ensure that the drainage system accommodates conveyance of the Major System Design Flow.

WC-1 Design Flows (Bridges and Culverts)

SCOPE

This standard identifies the minimum Design Flows for the sizing of bridges and culverts for flow conveyance on Regulated and non-Regulated Watercourses. It also identifies the requirement for accommodating the Regulatory Flow on Regulated Watercourses, and for determining the maximum allowable increase in flood elevations upstream of a bridge or culvert. This standard provides the hydrologic basis for all water crossing standards, WC-1 to WC-13.

DESIGN REFERENCES

Canadian Highway Bridge Design Code (CHBDC, 2000).
Exceptions to the Canadian Highway Bridge Design Code, CAN/CSA-S6-00 For Ontario, June 2002

1. HYDROLOGY

This standard addresses the Design Flow requirements for standard road classifications and low volume roads.

1.1 Standard Road Classifications

1.1.1 As a minimum, bridges and culverts of Provincial Highways shall be designed to the criteria shown in the following table, except as outlined in Section 1.1.2 to Section 1.1.4 of this standard:

Design Flow Return Period for Bridges and Culverts - Standard Road Classifications

Functional Road Classification	Return Period of Design Flows (Years) [1,2,3]		Check Flow for Scour
	Total Span less than or equal to 6.0 m	Total Span greater than 6.0 m	
Freeway, Urban Arterial	50	100	130% of 100 year
Rural Arterial, Collector Road	25	50	115% of 100 year
Local Road	10	25	100% of 100 year

Note:
1. The listed design flows apply to roads under the jurisdiction of the Ministry of Transportation.
2. The Fish Passage Design Flow for culverts is defined in Standard WC-12 Fish Passage Requirements Through Culverts
3. Sometimes referred to as Normal Design Flow

1.1.2 On Regulated Watercourses the Regulatory Flow shall be calculated in all cases where Floodline Mapping is available, where there is a potential risk to public safety, or where there is potential damage to adjacent properties, as applied in Section 2.3 of this standard.

1.1.3 The criteria may be modified in exceptional cases, such as for unusually large structures, or for vital routes which must remain useable during Regulatory Flow conditions. Use of Regulatory Flow criteria in the latter case shall be justified by a cost-benefit analysis.

1.1.4 If the road classification is likely to be upgraded or downgraded within 5 years of construction, the Return Period shall be based on the future classification.

1.2 Low Volume Roads

Design Flow Return Periods for Bridges on Low Volume Roads were developed to achieve economies without compromising safety. These requirements apply only to bridges. Culverts shall be designed in accordance with Section 1.1 of this standard.

1.2.1 As a minimum, bridges shall be designed to accommodate the Design Flow without damage to the structure or approaches. Relief Flow over the road shall be in accordance with Standard WC-13 Relief Flow (Bridges and Culverts). Drainage facilities for Low Volume Roads shall be designed to the criteria shown in the following table, except as provided in Section 1.2.2 to 1.2.5 of this standard:

Design Flow Return Period for Bridges on Low Volume Roads

		Return Period of Design Flow (Years)(1,2)	
Road Function	**Vulnerability**	**Total Span less than or equal to 6.0 m**	**Total Span greater than 6.0 m**
Collector and Arterial	High	25	50
	Low	25	50
Local	High	10	25
	Low	10	25
Resource Access	High	5	10
	Low	5	10
Recreation	High	5	10
	Low	2	2

Notes:
1 The listed design storms apply to roads under the jurisdiction of the Ministry of Transportation.
2 Sometimes referred to as Normal Design Flow

1.2.2 The Check Flow need not be considered.

1.2.3 On Regulated Watercourses the Regulatory Flow shall be calculated in all cases where Floodline Mapping is available, where there is a potential risk to public safety, or where there is potential damage to adjacent properties, as applied in Section 2.3 of this standard.

1.2.4 Low water crossings, which accommodate the Design Flow but overtop during more severe flooding, may be considered as an alternative, but not for Collector or Arterial Roads.

1.2.5 The Return Period should be determined by the owner in order to establish the acceptable length of time the structure is impassable. Where required, approval shall be sought from other agencies having jurisdiction.

1.2.6 The hydrology criteria may be modified in exceptional cases, such as for unusually large structures or for vital routes which must remain useable during more severe storm conditions. Use of a more severe design storm in the latter case shall be justified by a cost-benefit analysis.

1.3 Channel Realignment or Diversion

Channel realignment or channel restoration upstream or downstream of a water crossing that will alter the storage or discharge characteristics upstream of the crossing, shall be designed to meet the design standards of the crossing. As a minimum the combined capacity of the watercourse and floodplain shall convey the 25-year Design Flow. The main channel is to be designed to a lower Design Flow such that a stable channel is maintained.

2. HYDRAULICS

2.1 Design Flow and Upstream Water Surface Elevations

The existing and proposed upstream water surface elevations shall be calculated for Design Flow identified in Sections 1.1.1 and 1.2.1 of this standard and shall be used for the design of the Water Crossing.

2.2 Range of Flows and Upstream Water Surface Elevations

The existing and proposed upstream water surface elevations shall be calculated for Design Flows with Return Periods ranging from 5 years to 100 years, where the estimated water surface elevations will be used for assessing impacts on Rating Curves upstream of the water crossing.

2.3 Regulatory Flow and Upstream Water Surface Elevations

The existing and proposed upstream water surface elevations shall also be calculated for Regulated Watercourses where the Regulatory Flow estimate is required.

2.4 Check Flow

The Return Period for the Check Flow is identified in Section 1.1.1 of this standard. The Check Flow shall be used for scour analysis to assess structural integrity where required.

2.5 Winter Flow Condition

The Winter Flow Depth shall be used to evaluate icing conditions where required.

3. PHYSICAL CHARACTERISTICS

There are no physical characteristic standards applicable to Design Flows (Bridges and Culverts).

4. COMMENTARY

- The decision whether there would be any risk to public safety or potential damage to adjacent properties as a result of change in flood elevations shall be determined in consultation with the Municipality, Conservation Authority or the Ministry of the Natural Resources given their responsibilities under the Conservation Authorities Act and Lakes and Rivers Improvement Act.
- In the case where a drainage system that is not subject to regulations for conveyance or flood protection (e.g. municipal drain) is being conveyed under the highway, the design approach shall be followed for the protection of the highway.

- Where, through consultation with the Conservation Authority and/or MNR, there is an increase in flood elevation on private land that will adversely impact the landowners, an agreement will be made with the affected landowners.
- Design Flows for water crossings shall normally be based on existing runoff conditions, but, at the request of the municipality concerned, and subject to the Ministry's cost sharing policies, may be based on runoff conditions anticipated 20 years from the time of design.
- Upstream water surface elevations are calculated for all design storms in recognition that any increase in flood elevation may represent an increase in flood risk.
- Assessment of the Check Flow is not normally required if the structure is designed to the larger Regulatory Flow criteria.
- Performance of culverts on fish migration routes shall be checked with the Standard WC-12, Fish Passage through Culverts.
- The calculation of upstream elevations for a range of Design Flows under existing and proposed conditions is to be used to evaluate the impact of the structure on the upstream Rating Curve. If there is a negative impact, based upon the effect on private property or drainage systems, it may be necessary to change the proposed opening size to mitigate potential impacts.

TW-1 - Return Period of Design Storms for Temporary Works

SCOPE

This standard identifies the return periods for the sizing of temporary drainage facilities (e.g. bridges, culverts, diversion channels and diversion pipes) during construction. It also includes the return period associated with the design of temporary erosion control basins.

DESIGN REFERENCES

MTO Drainage Management Manual (1997), Chapter 6

1. HYDROLOGY

The Return Period for the design of drainage measures required during construction shall be assessed independently for each project. The contributing factors affecting the choice of a Return Period depend on the length of the construction period and include the potential consequences in terms of public safety, traffic delays, property damage due to flooding, and environmental impacts.

1.1 Consequence of Failure or Capacity Exceedance Definitions

The following definitions apply to assessing the consequence of failure or capacity exceedance from the perspective of Public Safety, Traffic Delays, Damage due to Flooding, and Natural Habitat Impacts.

- Low:
  - Public Safety – failure or capacity exceedance is not a significant risk to public safety
  - Traffic Delays – there would be no significant traffic delays as there are alternative routes
  - Damage due to Flooding –flooding would be local or would be limited to unimproved rural lands that would not be adversely affected by the flooding
  - Natural Habitat Impacts – any impacts will be temporary (i.e. fish habitat not permanently affected and vegetation damage will generally recover within two growing seasons)
- Medium:
  - Public Safety – failure or capacity exceedance is not a greater risk to public safety
  - Traffic Delays – there may be road closure causing delay or detouring (nuisance)
  - Damage due to Flooding – land uses such as croplands or parking will be flooded
  - Natural Habitat Impacts – temporary impacts anticipated that may take more than two growing seasons to recover
- High:
  - Public Safety –failure or capacity exceedance represents a significant risk to public safety
  - Traffic Delays – road closure causing significant impact on traffic or emergency vehicles
  - Damage due to Flooding – buildings will be flooded
  - Natural Habitat Impacts – permanent damage anticipated, requiring mitigation and/or habitat compensation

For each type of measure (e.g. culvert, by-pass channel) the consequence of failure or capacity exceedance shall be determined for each of the four categories (Public Safety, Traffic Delays, Damage due to Flooding, Natural Habitat Impacts). The worst case impact (low, medium, high) from the four categories shall be used for selecting the Return Period that shall guide design.

1.2 Return Period for Bridges Culverts, Diversion Channels and Diversion Pipes

The minimum Return Period for temporary drainage works shall be as follows.

Minimum Minor Return Period For Temporary Drainage Works			
Duration of Construction	**Return Period (Years)**		
	Consequence:		
	Low	**Medium**	**High**
Less than 2 months	2	2	2
Up to 4 months	2	5	5
Up to 8 months	5	5	10
Up to 12 months	5	5	20
Up to 18 months	5	10	25
Greater than 18 months	10	10	25

1.3 Return Period for Temporary Erosion Control Basins

A 25 mm design storm with a duration of three hours shall be used to size temporary erosion control basins.

2. HYDRAULICS

There are no standards specific to Hydraulics.

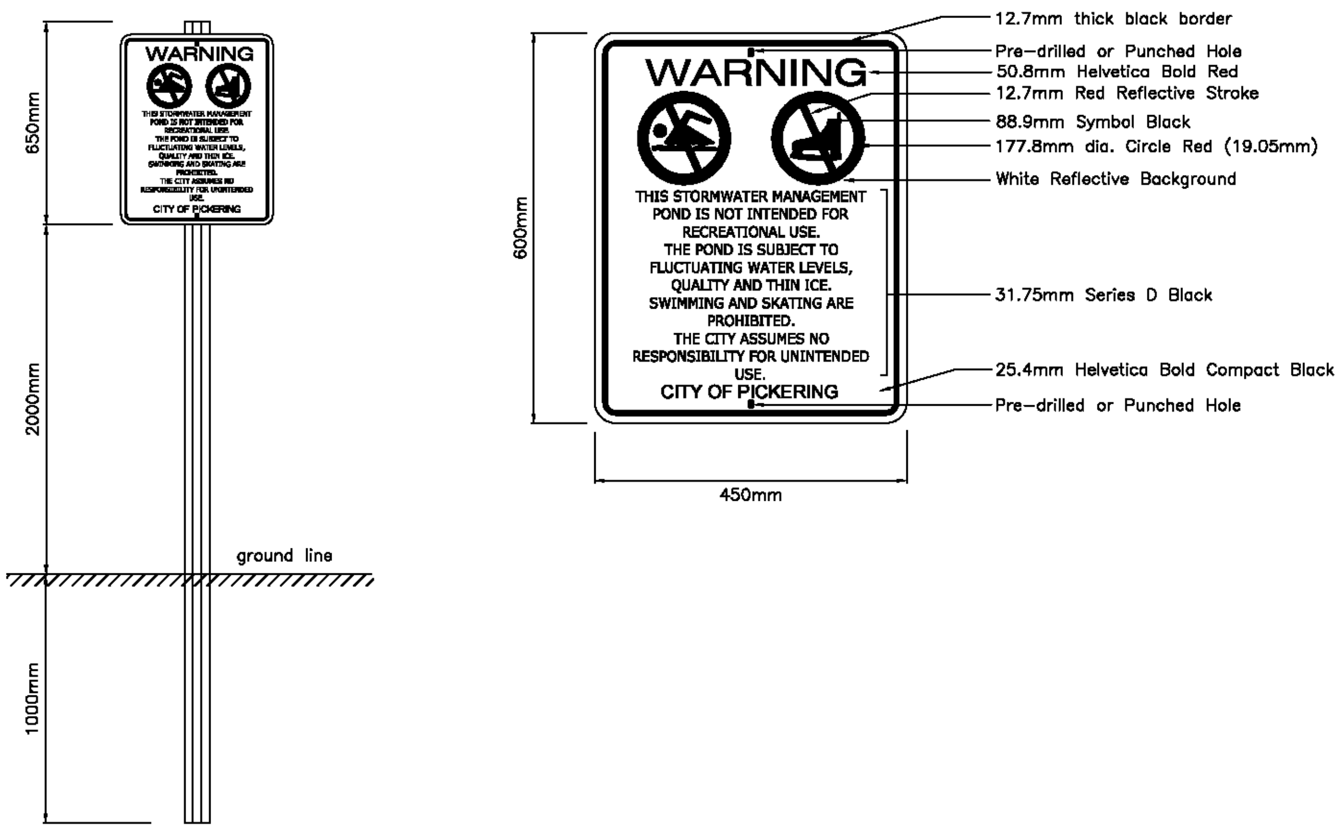
3. PHYSICAL CHARACTERISTICS

Temporary Erosion Control Basins shall be sized to include both a Permanent Pool and Live Storage. The Permanent Pool shall have a capacity of 125 cubic metres per hectare of upstream catchment area, while the Live Storage shall be large enough to contain the runoff generated by the 25 mm design storm noted in Standard TW-1 (1.3).

4. COMMENTARY

- This standard does not apply where dam and pump methodologies are used to divert streamflow from a construction site.
- The following table illustrates the method of determining the consequence of failure or capacity exceedance. The Worst Case from Column 1 to 4 will be used for establishing the Return Period for temporary drainage works.

Typical Measures	**Consequences**				
	Public Safety (1)	**Traffic Delays (2)**	**Damage due to Flooding (3)**	**Natural Habitat Impacts (4)**	**Worst Case from Column 1 to 4**
Temporary Culvert/Bridge	LOW	MEDIUM	LOW	LOW	MEDIUM
Diversion Channel	LOW	LOW	HIGH	MEDIUM	HIGH
Note: The above ratings are for illustrative purposes only.					



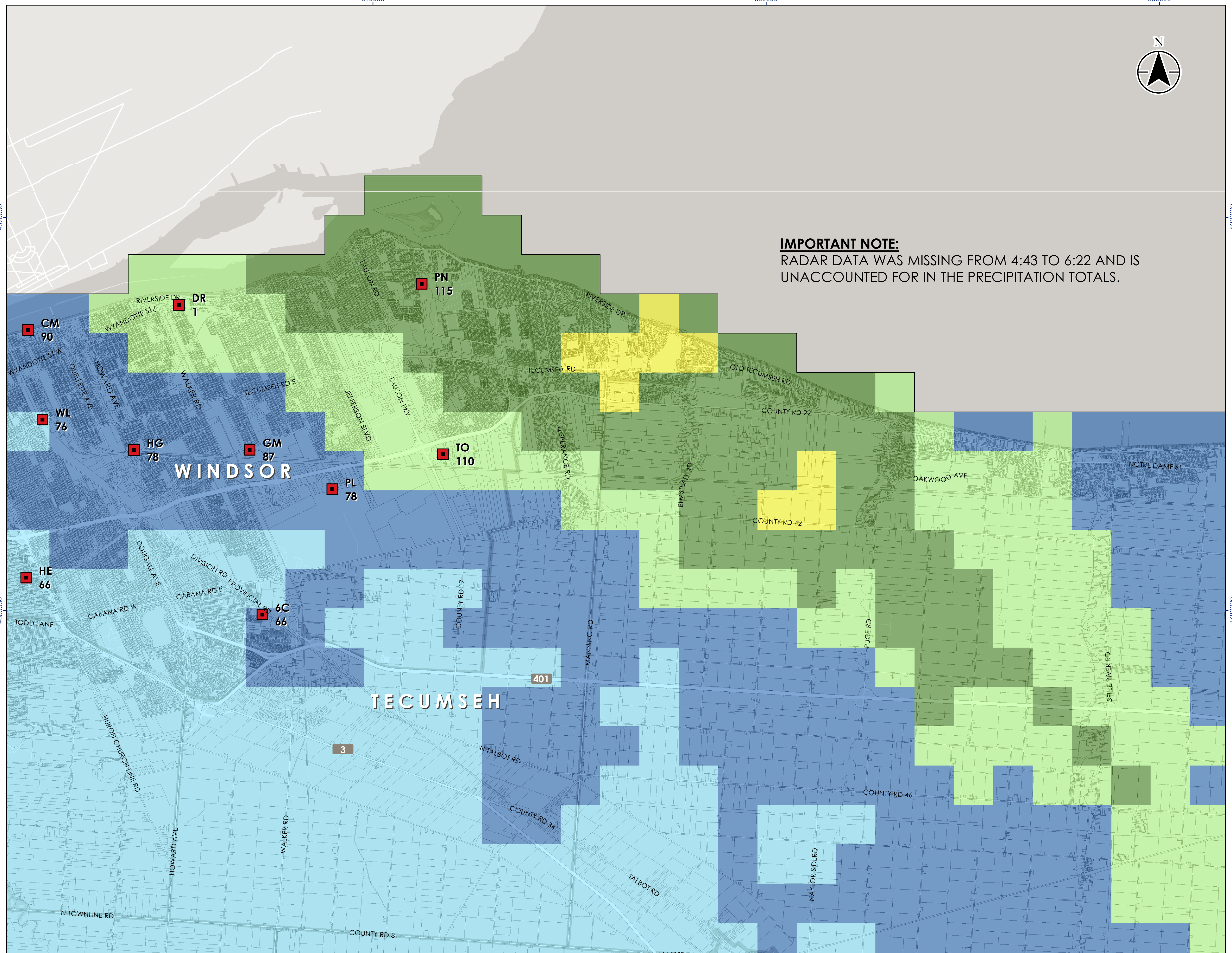
SIGN REQUIREMENTS
SIGN(S) MUST BE PLACED AT ALL POND ENTRANCES.

SIGNAGE FACE
SIGN(S) TO BE MANUFACTURED USING REFLECTIVE FINISH (ENGINEER GRADE)
1/8" THICK ALUMINUM PANEL (2" RADIUS CORNERS), WITH TOP AND BOTTOM MOUNT HOLES.

MOUNTING
SIGN(S) TO BE MOUNTED TO 3.65m U–CHANNEL GALVANIZED STEEL POST.

All dimensions are in millimetres unless otherwise noted.

City of Pickering	Planning & Development Department	
DRAWN P. HELGESEN	STORMWATER MANAGEMENT POND WARNING SIGN	REVISION NO.
APPROVED		DATE
DATE JANUARY 2012		P–1007



Stantec

Legend

Analysis Area

Rain Gauge and Observed Precipitation Amount (mm)

Precipitation (mm)

- 175 - 206
- 150 - 175
- 125 - 150
- 100 - 125
- 75 - 100
- 50 - 75
- 25 - 50

< 25 mm not shown

Rain Gauge Key

6C	6th Concession PS
AM	Ambassador PS
CM	CMH Woods PS
DR	Drouillard PS
GM	Grand Marais PS
GMr	Grand Marais @ Rankin
HE	Huron Estates PS
HG	Howard Grade Separation PS
LE	Leffler PS
LR	Lou Romano WRP
PL	Pillette PS
PN	Pontiac PS
TO	Twin Oaks PS
WL	Wellington PS

0 2 4 km

1:100,000 (At original document size of 11x17)

Notes

1. Coordinate System: NAD 1983 UTM Zone 17N
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2016.
3. Rainfall amounts shown are estimated from uncalibrated radar rainfall data and may not accurately represent the actual amount of rainfall that occurred. Radar data was obtained from the National Centers for Environmental Information (NCEI), NEXRAD Level-III Digital Precipitation Rate (DPR) product, Detroit, Michigan radar station (Station ID: KDTX).

Project Location
Essex County

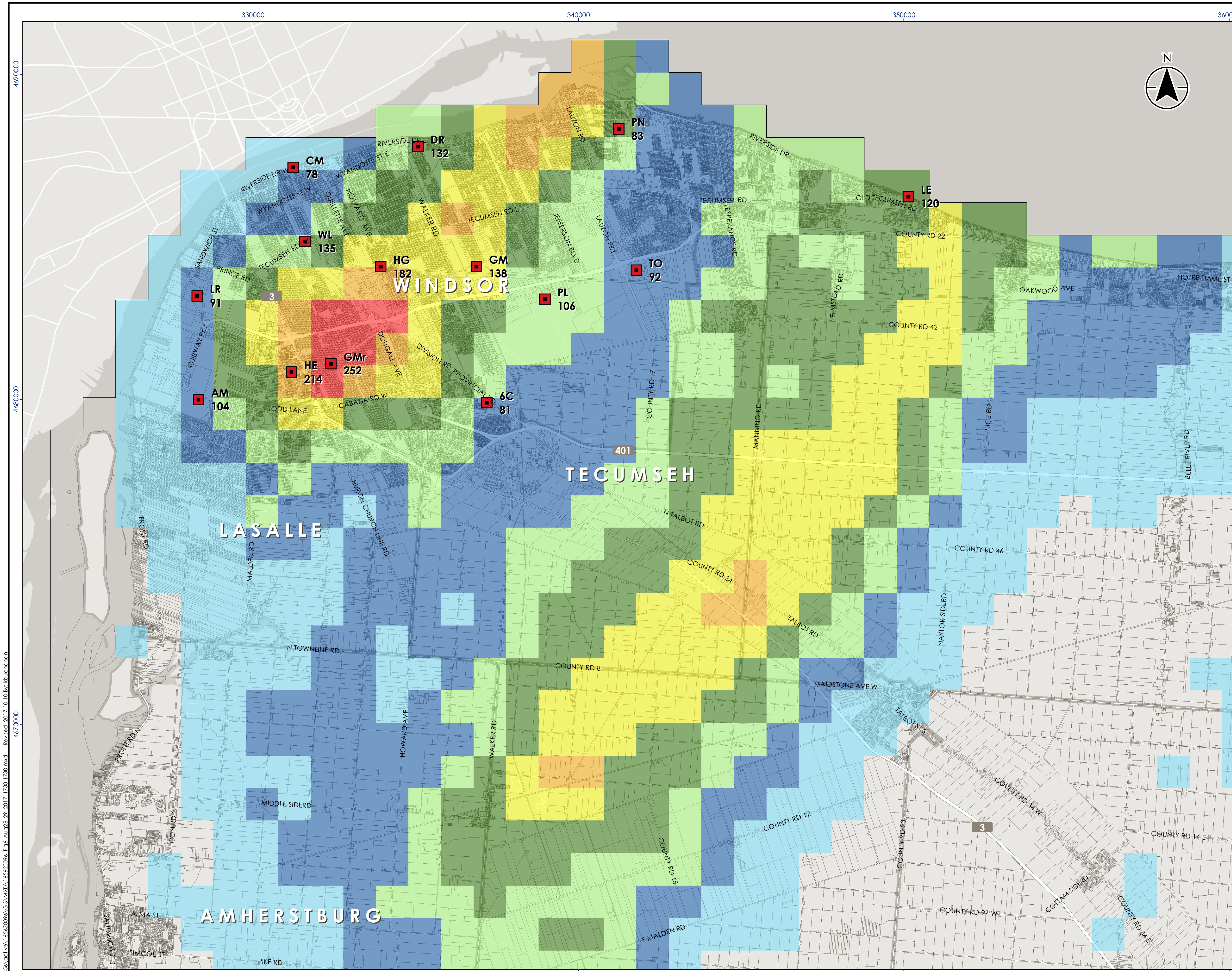
Prepared by KDB on 2017-10-10

Project
UNCALIBRATED RADAR ANALYSIS

Figure No.
1

Title
24-Hour Rainfall Amounts
Sept. 28 & Sept. 29, 2016; 18:00 to 18:00

V:\01656\active\165620096\GIS\MXD\165620096_Fig7_Sept28_29_2016_1800-1800.mxd Revised: 2017-10-10 By: kbuchanan



Stantec

Legend

Analysis Area

Rain Gauge and Observed Precipitation Amount (mm)

Precipitation (mm)

- 175 - 206
- 150 - 175
- 125 - 150
- 100 - 125
- 75 - 100
- 50 - 75
- 25 - 50
- < 25 mm not shown

Rain Gauge Key

6C	6th Concession PS
AM	Ambassador PS
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DR	Drouillard PS
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LE	Leffler PS
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PL	Pillette PS
PN	Pontiac PS
TO	Twin Oaks PS
WL	Wellington PS

0 2 4 km

1:120,000 (At original document size of 11x17)

Notes

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Project Location
Essex County

Prepared by KDB on 2017-10-10

Project
UNCALIBRATED RADAR ANALYSIS

Figure No.
2

Title
24-Hour Rainfall Amounts
Aug. 28 & Aug. 29, 2017; 17:30 to 17:30

idf_v3-30_2022_10_31_613_ON_6139525_WINDSOR_A
Environment and Climate Change Canada
Environnement et Changement climatique Canada

Short Duration Rainfall Intensity-Duration-Frequency Data
Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée

Gumbel - Method of moments/Méthode des moments

2022/10/31

===

WINDSOR A ON 6139525

Latitude: 42 17'N Longitude: 82 58'W Elevation/Altitude: 189 m

Years/Années : 1946 - 2016 # Years/Années : 66

===

****

Table 1 : Annual Maximum (mm)/Maximum annuel (mm)

****

Year Année	5 min	10 min	15 min	30 min	1 h	2 h	6 h	12 h	24 h
1946	10.7	14.2	15.0	22.6	29.0	42.7	46.2	49.3	54.9
1947	9.7	18.8	23.4	27.9	37.8	38.1	54.4	61.0	71.4
1948	7.1	7.9	9.9	11.7	15.0	16.0	30.7	40.1	44.2
1949	11.9	19.3	22.4	29.0	47.0	51.8	54.1	57.9	71.6
1951	5.8	8.9	13.2	18.5	26.9	34.3	38.1	44.7	53.8
1952	7.4	13.2	15.0	19.0	30.7	37.1	43.9	46.7	47.0
1953	20.1	20.8	21.1	32.5	40.6	51.1	52.3	54.9	55.1
1954	8.9	13.7	16.0	20.6	24.6	27.4	52.1	66.3	67.3
1955	8.4	9.4	12.4	18.8	21.3	34.0	41.7	41.9	60.2
1956	8.4	11.7	12.2	15.5	23.4	30.7	35.6	39.9	44.2
1957	10.9	19.8	26.7	46.5	52.8	57.4	96.8	100.3	100.3
1958	7.9	11.7	11.9	15.0	23.1	29.5	35.1	38.9	40.4
1959	11.7	16.3	17.5	21.3	21.6	34.0	40.6	60.5	65.5
1960	7.6	10.7	12.4	16.8	29.5	34.5	36.6	43.9	54.4
1961	11.7	17.3	22.6	35.8	38.1	43.2	43.2	43.2	45.7
1962	14.0	21.8	24.4	34.0	54.6	64.3	64.8	64.8	64.8
1963	13.0	20.1	27.2	40.9	43.9	45.0	45.0	45.0	56.9
1964	13.2	17.5	22.4	27.2	27.2	28.7	33.5	36.6	39.4
1965	7.9	12.7	13.7	15.7	22.6	27.2	32.8	55.1	58.9
1966	14.5	18.8	23.1	32.5	33.0	47.0	57.1	64.5	67.3
1967	13.7	20.1	21.8	24.9	26.4	26.4	41.1	62.2	72.6
1968	10.2	15.2	17.5	25.9	36.8	38.1	60.2	77.2	78.2
1969	11.2	19.0	20.8	27.7	27.7	36.1	57.1	57.1	57.1
1970	9.4	12.2	16.5	17.0	22.6	29.5	29.7	36.6	39.6

1971	9.4	17.8	25.7	35.1	35.1	35.1	43.9	43.9	43.9
1972	9.9	12.2	14.5	19.6	25.9	28.7	31.0	31.2	39.1
1973	12.7	18.0	22.6	27.9	30.0	30.2	33.5	37.8	40.4
1974	15.0	26.2	39.4	41.1	45.0	49.5	49.8	49.8	49.8
1975	8.4	14.2	21.1	24.9	25.1	31.7	40.6	44.7	52.8
1976	9.1	13.2	14.0	16.5	22.9	27.4	34.3	35.3	40.4
1977	7.1	10.9	14.0	16.5	25.4	29.2	29.2	31.7	41.4
1978	8.8	10.5	11.8	20.4	21.1	21.1	22.5	28.6	31.0
1979	10.4	16.6	24.9	32.0	48.3	52.6	55.3	60.8	61.2
1980	14.1	17.2	25.0	35.6	45.3	45.6	46.3	79.8	80.0
1981	-99.9	16.7	23.3	26.0	32.0	45.6	77.3	81.7	92.3
1982	7.7	10.7	13.2	18.9	27.3	28.3	28.3	40.4	49.9
1983	15.0	16.5	22.1	32.3	38.7	45.4	62.1	62.1	82.0
1984	6.0	8.8	11.3	17.5	17.7	21.0	32.6	34.9	37.2
1985	11.2	13.3	13.9	18.7	24.6	39.6	58.4	59.2	59.2
1986	8.0	12.7	18.0	19.4	20.7	32.0	37.7	47.7	88.6
1987	11.9	16.5	19.8	24.5	29.9	36.1	39.1	41.6	52.8
1988	7.0	8.8	12.5	12.7	12.9	14.4	28.0	32.3	33.0
1989	7.4	11.9	17.5	21.2	27.0	36.3	48.3	61.7	71.8
1990	11.4	16.0	18.8	20.5	22.4	26.4	41.9	52.2	70.6
1991	5.6	9.6	12.9	25.7	37.2	40.5	40.5	40.7	43.2
1992	6.5	9.8	12.0	16.9	25.7	29.8	34.4	34.4	45.8
1993	7.0	9.6	10.5	11.2	17.2	23.9	28.7	30.6	44.7
1994	8.3	11.3	14.6	23.8	30.0	43.2	51.3	51.5	80.7
1995	9.7	17.2	24.3	40.5	56.7	58.9	63.0	63.0	63.6
1996	13.5	15.4	16.8	18.7	18.7	19.1	40.2	40.4	46.3
1997	7.9	11.5	15.6	17.5	21.8	30.6	38.2	39.9	41.7
1998	7.3	12.7	13.9	15.7	16.4	26.8	31.4	36.2	57.4
1999	9.3	13.3	16.5	20.8	21.0	22.2	23.4	24.8	29.8
2000	7.6	11.2	13.1	20.4	26.4	31.0	51.8	89.0	94.6
2001	6.1	10.2	12.2	12.8	14.3	17.2	24.1	38.1	48.4
2002	6.9	9.1	10.8	14.4	17.2	17.4	29.6	31.7	43.2
2003	7.2	10.0	12.2	14.4	14.8	14.8	22.7	33.5	34.6
2004	13.3	15.7	18.6	20.4	22.1	33.2	35.8	37.3	53.7
2005	10.5	16.9	24.0	25.8	26.0	26.0	29.8	30.6	41.2
2006	10.6	18.3	23.6	26.6	35.7	51.3	53.1	53.3	66.9
2007	8.0	15.1	18.7	30.9	48.6	48.8	50.4	55.8	57.6
2008	9.7	18.5	26.7	35.9	37.4	38.0	47.2	65.7	75.3
2012	11.0	20.8	26.6	41.2	54.2	54.4	54.4	54.8	59.8
2013	10.8	18.6	25.3	28.1	43.8	54.7	57.0	57.0	58.1
2014	11.1	18.6	23.7	32.8	38.1	38.9	63.0	73.6	80.3
2015	11.9	18.2	19.9	37.2	41.4	50.0	64.6	73.4	73.4
2016	9.9	14.5	17.7	23.0	24.0	32.9	42.3	56.3	78.8
# Yrs. Années	66	67	67	67	67	67	67	67	67
Mean Moyenne	9.9	14.7	18.4	24.4	30.2	35.6	43.9	50.1	57.4
Std. Dev. Écart-type	2.8	4.0	5.7	8.4	10.9	11.6	13.8	15.7	16.5
Skew. Dissymétrie	0.90	0.29	0.78	0.63	0.70	0.32	1.01	0.85	0.55
Kurtosis	4.42	2.62	4.14	2.72	2.82	2.66	5.09	3.70	2.77

*-99.9 Indicates Missing Data/Données manquantes

Warning: annual maximum amount greater than 100-yr return period amount
Avertissement : la quantité maximale annuelle excède la quantité
pour une période de retour de 100 ans

Year/Année	Duration/Durée	Data/Données	100-yr/ans
1953	5 min	20.1	18.6
1957	6 h	96.8	87.3
1957	12 h	100.3	99.4
1974	15 min	39.4	36.3

****

Table 2a : Return Period Rainfall Amounts (mm)
Quantité de pluie (mm) par période de retour

****

Duration/Durée	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans	#Years Années
5 min	9.5	11.9	13.6	15.6	17.1	18.6	66
10 min	14.1	17.6	19.9	22.8	25.0	27.2	67
15 min	17.5	22.5	25.8	30.0	33.2	36.3	67
30 min	23.0	30.4	35.3	41.6	46.2	50.7	67
1 h	28.4	38.0	44.4	52.5	58.4	64.4	67
2 h	33.7	43.9	50.7	59.3	65.6	71.9	67
6 h	41.6	53.8	61.9	72.1	79.7	87.3	67
12 h	47.5	61.4	70.6	82.2	90.8	99.4	67
24 h	54.7	69.3	79.0	91.2	100.3	109.3	67

****

Table 2b :

Return Period Rainfall Rates (mm/h) - 95% Confidence limits
Intensité de la pluie (mm/h) par période de retour - Limites de confiance de 95%

****

Duration/Durée	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans	#Years Années
5 min	113.7	143.2	162.7	187.3	205.6	223.7	66
	+/- 7.4	+/- 12.4	+/- 16.8	+/- 22.6	+/- 27.1	+/- 31.6	66
10 min	84.4	105.5	119.4	137.0	150.1	163.1	67
	+/- 5.2	+/- 8.8	+/- 11.9	+/- 16.1	+/- 19.2	+/- 22.4	67
15 min	69.9	90.0	103.3	120.2	132.6	145.0	67
	+/- 5.0	+/- 8.4	+/- 11.4	+/- 15.4	+/- 18.4	+/- 21.4	67
30 min	46.0	60.9	70.7	83.1	92.3	101.5	67
	+/- 3.7	+/- 6.2	+/- 8.4	+/- 11.3	+/- 13.6	+/- 15.8	67
1 h	28.4	38.0	44.4	52.5	58.4	64.4	67
	+/- 2.4	+/- 4.0	+/- 5.5	+/- 7.3	+/- 8.8	+/- 10.2	67
2 h	16.8	22.0	25.3	29.6	32.8	36.0	67

	+/- 1.3	+/- 2.1	+/- 2.9	+/- 3.9	+/- 4.7	+/- 5.4	67
6 h	6.9	9.0	10.3	12.0	13.3	14.5	67
	+/- 0.5	+/- 0.9	+/- 1.2	+/- 1.6	+/- 1.9	+/- 2.2	67
12 h	4.0	5.1	5.9	6.9	7.6	8.3	67
	+/- 0.3	+/- 0.5	+/- 0.7	+/- 0.9	+/- 1.1	+/- 1.2	67
24 h	2.3	2.9	3.3	3.8	4.2	4.6	67
	+/- 0.2	+/- 0.3	+/- 0.3	+/- 0.5	+/- 0.6	+/- 0.6	67

****

Table 3 : Interpolation Equation / Équation d'interpolation: R = A*T^B

R = Interpolated Rainfall rate (mm/h)/Intensité interpolée de la pluie (mm/h)
RR = Rainfall rate (mm/h) / Intensité de la pluie (mm/h)
T = Rainfall duration (h) / Durée de la pluie (h)

****

Statistics/Statistiques	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans
Mean of RR/Moyenne de RR	41.4	52.9	60.6	70.3	77.4	84.6
Std. Dev. /Écart-type (RR)	40.0	50.3	57.1	65.8	72.2	78.6
Std. Error/Erreur-type	11.7	16.4	19.5	23.4	26.3	29.2
Coefficient (A)	24.6	31.7	36.4	42.4	46.8	51.2
Exponent/Exposant (B)	-0.709	-0.707	-0.706	-0.706	-0.705	-0.705
Mean % Error/% erreur moyenne	10.3	12.0	12.7	13.4	13.8	14.2

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Environment Canada/Environnement Canada

Short Duration Rainfall Intensity-Duration-Frequency Data
Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée

Gumbel - Method of moments/Méthode des moments

2012/02/09

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HARROW CDA AUTO ON 6133362
(composite)
Latitude: 42 2'N Longitude: 82 54'W Elevation/Altitude: 191 m

Years/Années : 1966 - 2007 # Years/Années : 28

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Table 1 : Annual Maximum (mm)/Maximum annuel (mm)

****

Year Année	5 min	10 min	15 min	30 min	1 h	2 h	6 h	12 h	24 h
1966	11.9	18.3	20.3	32.0	33.0	39.4	74.7	114.0	121.4
1967	9.7	15.0	19.3	26.2	26.7	29.5	45.2	46.7	48.5
1969	13.5	21.1	25.4	49.0	58.4	61.5	61.5	66.3	68.8
1970	8.4	10.7	15.2	27.2	31.7	33.5	38.9	47.2	50.0
1971	10.2	10.9	14.2	21.6	27.4	28.4	29.7	29.7	33.8
1972	6.6	9.1	10.9	16.5	21.6	25.4	32.8	50.3	64.3
1973	5.1	6.3	8.6	14.5	27.7	35.8	50.8	56.4	56.9
1974	6.3	7.6	8.4	9.4	12.7	16.5	25.9	34.3	35.1
1976	4.6	6.3	8.1	9.9	15.5	15.7	32.0	37.6	38.9
1977	8.1	13.7	17.8	19.8	22.9	23.9	31.0	35.8	59.4
1978	6.0	12.0	15.6	21.2	21.3	21.3	26.7	33.3	35.2
1979	6.0	12.0	14.2	15.8	16.6	16.6	29.5	38.5	38.5
1980	13.0	19.4	23.3	29.9	37.2	39.7	39.8	48.5	56.8
1981	17.8	19.0	21.6	22.3	24.0	24.7	27.1	34.8	51.4
1982	10.8	16.8	23.7	26.0	29.0	29.0	29.0	37.2	37.2
1984	10.2	14.3	17.7	26.1	28.7	35.5	36.2	36.6	36.6
1985	10.2	17.2	20.8	26.7	26.8	26.9	28.6	30.8	53.2
1986	12.0	20.7	25.2	34.6	39.4	44.7	50.5	50.5	54.1
1987	8.5	12.8	16.9	20.2	31.3	38.6	60.6	82.7	89.6
1988	11.7	19.0	25.0	37.2	53.1	53.2	53.2	54.2	54.2
1989	8.7	17.2	24.7	37.6	49.6	80.0	133.6	187.7	263.2
2001	5.6	7.8	9.4	13.2	18.4	23.2	28.6	41.2	41.2
2002	6.4	10.8	12.6	14.2	16.0	16.2	28.8	37.4	38.8

2003	7.4	11.8	14.2	18.0	21.2	25.0	36.6	38.2	39.4
2004	8.4	11.8	12.8	22.2	29.4	39.6	42.4	42.4	42.4
2005	10.4	13.0	14.0	15.4	16.4	18.4	24.6	29.0	44.4
2006	10.0	14.8	20.0	27.0	33.2	33.4	33.8	34.0	45.8
2007	7.4	11.6	12.6	16.2	19.2	21.4	44.0	45.6	45.8
# Yrs. Années	28	28	28	28	28	28	28	28	28
Mean Moyenne	9.1	13.6	16.9	23.2	28.2	32.0	42.0	50.7	58.7
Std. Dev. Écart-type	3.0	4.3	5.5	9.2	11.3	14.6	21.9	32.2	44.1
Skew. Dissymétrie	0.83	0.07	0.06	0.84	1.15	1.60	2.99	3.34	4.05
Kurtosis	4.35	2.40	2.15	4.09	4.41	6.45	14.03	15.40	20.86

*-99.9 Indicates Missing Data/Données manquantes

Warning: annual maximum amount greater than 100-yr return period amount
Avertissement : la quantité maximale annuelle excède la quantité
pour une période de retour de 100 ans

Year/Année	Duration/Durée	Data/Données	100-yr/ans
1989	2 h	80.0	77.9
1989	6 h	133.6	110.7
1989	12 h	187.7	151.7
1989	24 h	263.2	197.1

****

Table 2a : Return Period Rainfall Amounts (mm)
Quantité de pluie (mm) par période de retour

****

Duration/Durée	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans	#Years Années
5 min	8.6	11.2	13.0	15.2	16.8	18.5	28
10 min	12.9	16.7	19.2	22.4	24.7	27.1	28
15 min	16.0	20.8	24.1	28.1	31.2	34.2	28
30 min	21.7	29.8	35.2	42.0	47.0	52.0	28
1 h	26.3	36.3	42.9	51.3	57.5	63.7	28
2 h	29.6	42.6	51.1	61.9	70.0	77.9	28
6 h	38.4	57.8	70.6	86.8	98.8	110.7	28
12 h	45.5	73.9	92.7	116.5	134.2	151.7	28
24 h	51.5	90.5	116.3	148.9	173.1	197.1	28

****

Table 2b :

Return Period Rainfall Rates (mm/h) - 95% Confidence limits

Intensité de la pluie (mm/h) par période de retour - Limites de confiance de 95%

****

Duration/Durée	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans	#Years Années
5 min	103.4	135.0	155.9	182.4	202.0	221.5	28
	+/- 12.2	+/- 20.5	+/- 27.7	+/- 37.3	+/- 44.6	+/- 52.0	28
10 min	77.4	100.2	115.2	134.3	148.4	162.4	28
	+/- 8.8	+/- 14.7	+/- 19.9	+/- 26.9	+/- 32.1	+/- 37.4	28
15 min	63.9	83.4	96.3	112.6	124.6	136.6	28
	+/- 7.5	+/- 12.6	+/- 17.0	+/- 23.0	+/- 27.5	+/- 32.0	28
30 min	43.4	59.6	70.4	84.0	94.1	104.1	28
	+/- 6.2	+/- 10.5	+/- 14.2	+/- 19.2	+/- 22.9	+/- 26.7	28
1 h	26.3	36.3	42.9	51.3	57.5	63.7	28
	+/- 3.8	+/- 6.5	+/- 8.8	+/- 11.8	+/- 14.1	+/- 16.5	28
2 h	14.8	21.3	25.6	31.0	35.0	39.0	28
	+/- 2.5	+/- 4.2	+/- 5.7	+/- 7.6	+/- 9.1	+/- 10.6	28
6 h	6.4	9.6	11.8	14.5	16.5	18.5	28
	+/- 1.2	+/- 2.1	+/- 2.8	+/- 3.8	+/- 4.6	+/- 5.3	28
12 h	3.8	6.2	7.7	9.7	11.2	12.6	28
	+/- 0.9	+/- 1.5	+/- 2.1	+/- 2.8	+/- 3.3	+/- 3.9	28
24 h	2.1	3.8	4.8	6.2	7.2	8.2	28
	+/- 0.6	+/- 1.1	+/- 1.4	+/- 1.9	+/- 2.3	+/- 2.7	28

****

Table 3 : Interpolation Equation / Équation d'interpolation: R = A*T^B

R = Interpolated Rainfall rate (mm/h)/Intensité interpolée de la pluie (mm/h)
RR = Rainfall rate (mm/h) / Intensité de la pluie (mm/h)
T = Rainfall duration (h) / Durée de la pluie (h)

****

Statistics/Statistiques	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans
Mean of RR/Moyenne de RR	37.9	50.6	59.0	69.5	77.4	85.2
Std. Dev. /Écart-type (RR)	36.5	47.0	53.9	62.7	69.2	75.6
Std. Error/Erreur-type	10.8	11.4	12.2	13.4	14.3	15.3
Coefficient (A)	22.7	32.2	38.4	46.2	52.0	57.7
Exponent/Exposant (B)	-0.704	-0.653	-0.633	-0.617	-0.608	-0.601
Mean % Error/% erreur moyenne	9.7	8.2	7.7	7.4	7.2	7.1

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Environment Canada/Environnement Canada

Short Duration Rainfall Intensity-Duration-Frequency Data
Données sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée

Gumbel - Method of moments/Méthode des moments

2012/02/09

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POINT PELEE CS ON 613P001
(composite)
Latitude: 41 57'N Longitude: 82 31'W Elevation/Altitude: 176 m

Years/Années : 1975 - 2004 # Years/Années : 22

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Table 1 : Annual Maximum (mm)/Maximum annuel (mm)

****

Year Année	5 min	10 min	15 min	30 min	1 h	2 h	6 h	12 h	24 h
1975	7.9	14.0	14.7	15.5	31.0	34.3	34.3	34.3	45.5
1976	9.1	13.2	17.0	20.8	22.4	32.3	37.8	48.3	49.3
1977	10.2	17.3	22.9	34.5	51.6	51.6	53.6	53.6	53.6
1978	4.6	9.1	11.4	16.0	16.0	21.2	23.9	34.5	37.0
1979	8.1	12.1	15.2	16.6	16.8	16.8	26.1	49.2	58.6
1980	11.7	17.8	18.8	25.7	35.0	37.3	41.6	41.8	70.9
1981	8.6	13.8	15.8	18.8	22.1	22.7	29.8	34.4	50.9
1982	13.4	18.8	24.9	33.8	34.7	35.9	35.9	36.4	36.4
1983	8.5	11.3	15.0	21.6	28.1	32.5	37.5	44.6	54.8
1984	12.5	17.2	18.7	19.9	22.2	25.7	29.4	33.0	33.0
1985	9.9	12.4	17.0	19.9	19.9	21.7	29.6	29.8	29.9
1986	7.4	9.9	13.3	21.9	24.0	37.7	48.2	48.4	51.0
1987	10.5	15.2	18.8	18.8	29.6	38.0	73.1	81.1	91.4
1988	7.7	9.6	10.6	13.4	15.6	18.8	29.5	37.0	40.0
1989	14.3	20.4	30.6	51.0	63.2	85.8	102.5	110.5	113.6
1990	12.2	14.1	16.7	23.3	36.7	50.5	77.9	106.3	106.4
1991	8.6	14.9	15.4	18.0	21.2	26.4	40.1	57.0	58.8
1992	6.9	9.4	12.5	22.2	36.5	55.0	75.7	83.4	85.8
1993	14.3	20.4	21.0	21.2	21.2	24.3	26.1	31.4	38.2
2002	17.6	22.4	23.2	23.4	23.4	30.2	41.8	49.0	51.6
2003	7.6	8.6	9.6	12.6	15.6	20.8	49.4	56.0	56.0
2004	13.0	19.2	23.6	41.8	61.0	63.4	65.4	65.4	72.6

# Yrs. Années	22	22	22	22	22	22	22	22	22
Mean Moyenne	10.2	14.6	17.6	23.2	29.4	35.6	45.9	53.0	58.4
Std. Dev. Écart-type	3.1	4.1	5.2	9.3	13.7	16.8	20.8	23.2	23.0
Skew. Dissymétrie	0.56	0.22	0.70	1.73	1.38	1.51	1.29	1.40	1.12
Kurtosis	3.39	2.34	3.75	6.19	4.59	5.76	4.43	4.56	3.93

*-99.9 Indicates Missing Data/Données manquantes

****

Table 2a : Return Period Rainfall Amounts (mm)
Quantité de pluie (mm) par période de retour

****

Duration/Durée	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans	#Years Années
5 min	9.7	12.4	14.2	16.5	18.2	19.9	22
10 min	13.9	17.6	20.0	23.0	25.3	27.5	22
15 min	16.7	21.3	24.3	28.1	31.0	33.8	22
30 min	21.7	29.9	35.4	42.3	47.4	52.5	22
1 h	27.2	39.3	47.4	57.5	65.1	72.5	22
2 h	32.8	47.7	57.5	70.0	79.2	88.4	22
6 h	42.5	60.8	73.0	88.4	99.8	111.1	22
12 h	49.2	69.7	83.2	100.3	113.1	125.7	22
24 h	54.6	75.0	88.5	105.5	118.1	130.7	22

****

Table 2b :

Return Period Rainfall Rates (mm/h) - 95% Confidence limits
Intensité de la pluie (mm/h) par période de retour - Limites de confiance de 95%

****

Duration/Durée	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans	#Years Années
5 min	116.4	149.2	170.8	198.2	218.6	238.7	22
	+/- 14.2	+/- 23.9	+/- 32.3	+/- 43.6	+/- 52.2	+/- 60.8	22
10 min	83.5	105.4	119.9	138.2	151.7	165.2	22
	+/- 9.5	+/- 16.0	+/- 21.6	+/- 29.1	+/- 34.8	+/- 40.6	22
15 min	66.9	85.2	97.3	112.5	123.8	135.1	22
	+/- 7.9	+/- 13.3	+/- 18.0	+/- 24.3	+/- 29.1	+/- 33.9	22
30 min	43.4	59.9	70.8	84.6	94.8	105.0	22
	+/- 7.2	+/- 12.1	+/- 16.3	+/- 22.0	+/- 26.3	+/- 30.6	22

1 h		27.2	39.3	47.4	57.5	65.1	72.5	22
	+/-	5.3 +/-	8.9 +/-	12.0 +/-	16.2 +/-	19.3 +/-	22.5	22
2 h		16.4	23.8	28.8	35.0	39.6	44.2	22
	+/-	3.2 +/-	5.4 +/-	7.3 +/-	9.9 +/-	11.8 +/-	13.8	22
6 h		7.1	10.1	12.2	14.7	16.6	18.5	22
	+/-	1.3 +/-	2.2 +/-	3.0 +/-	4.1 +/-	4.9 +/-	5.7	22
12 h		4.1	5.8	6.9	8.4	9.4	10.5	22
	+/-	0.7 +/-	1.2 +/-	1.7 +/-	2.3 +/-	2.7 +/-	3.2	22
24 h		2.3	3.1	3.7	4.4	4.9	5.4	22
	+/-	0.4 +/-	0.6 +/-	0.8 +/-	1.1 +/-	1.4 +/-	1.6	22

****

Table 3 : Interpolation Equation / Équation d'interpolation: R = A*T^B

R = Interpolated Rainfall rate (mm/h)/Intensité interpolée de la pluie (mm/h)
RR = Rainfall rate (mm/h) / Intensité de la pluie (mm/h)
T = Rainfall duration (h) / Durée de la pluie (h)

****

Statistics/Statistiques	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans
Mean of RR/Moyenne de RR	40.8	53.5	62.0	72.6	80.5	88.4
Std. Dev. /Écart-type (RR)	40.3	50.8	57.8	66.7	73.3	79.8
Std. Error/Erreur-type	9.0	12.8	15.4	18.7	21.1	23.6
Coefficient (A)	24.3	33.1	38.8	46.1	51.5	56.8
Exponent/Exposant (B)	-0.704	-0.684	-0.675	-0.667	-0.663	-0.660
Mean % Error/% erreur moyenne	8.3	10.9	12.3	13.6	14.3	14.9

FIGURE 6-1: FLOOD PLAIN STABILITY CHART FOR HUMANS – 1

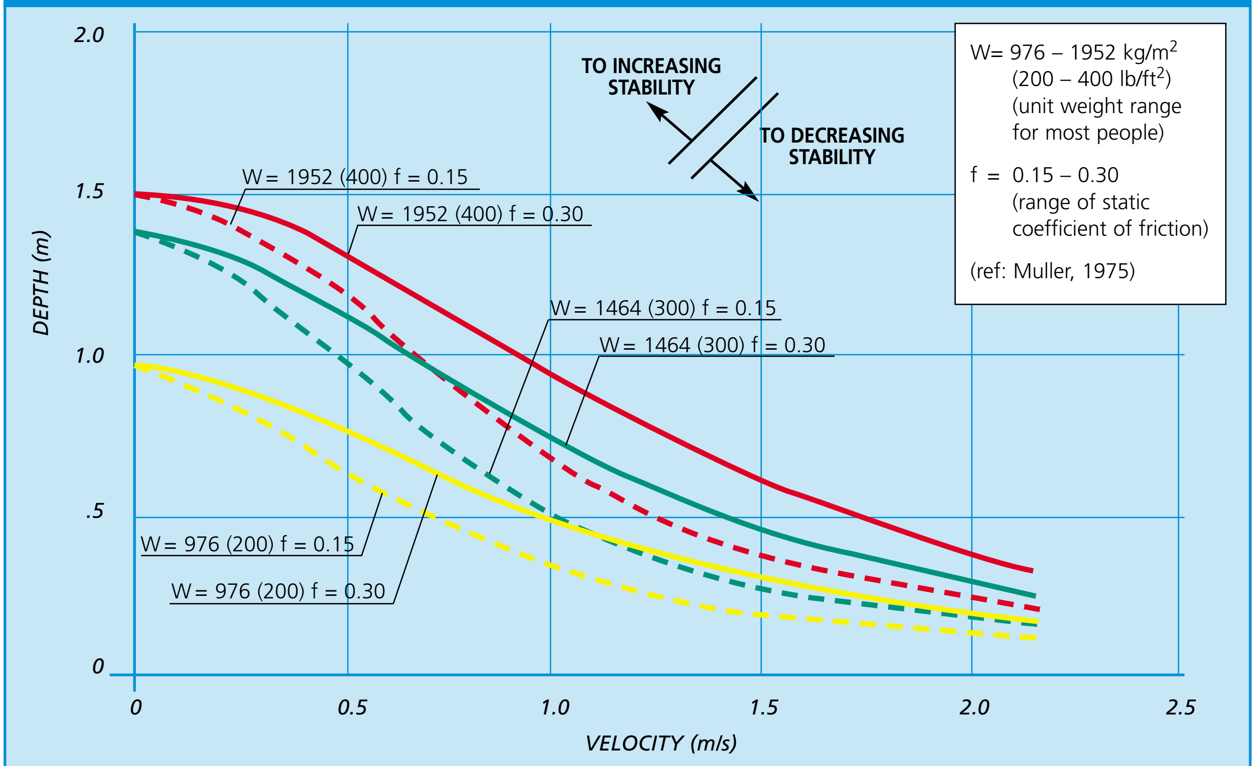


FIGURE 6-2: FLOOD PLAIN STABILITY CHART FOR HUMANS – 2

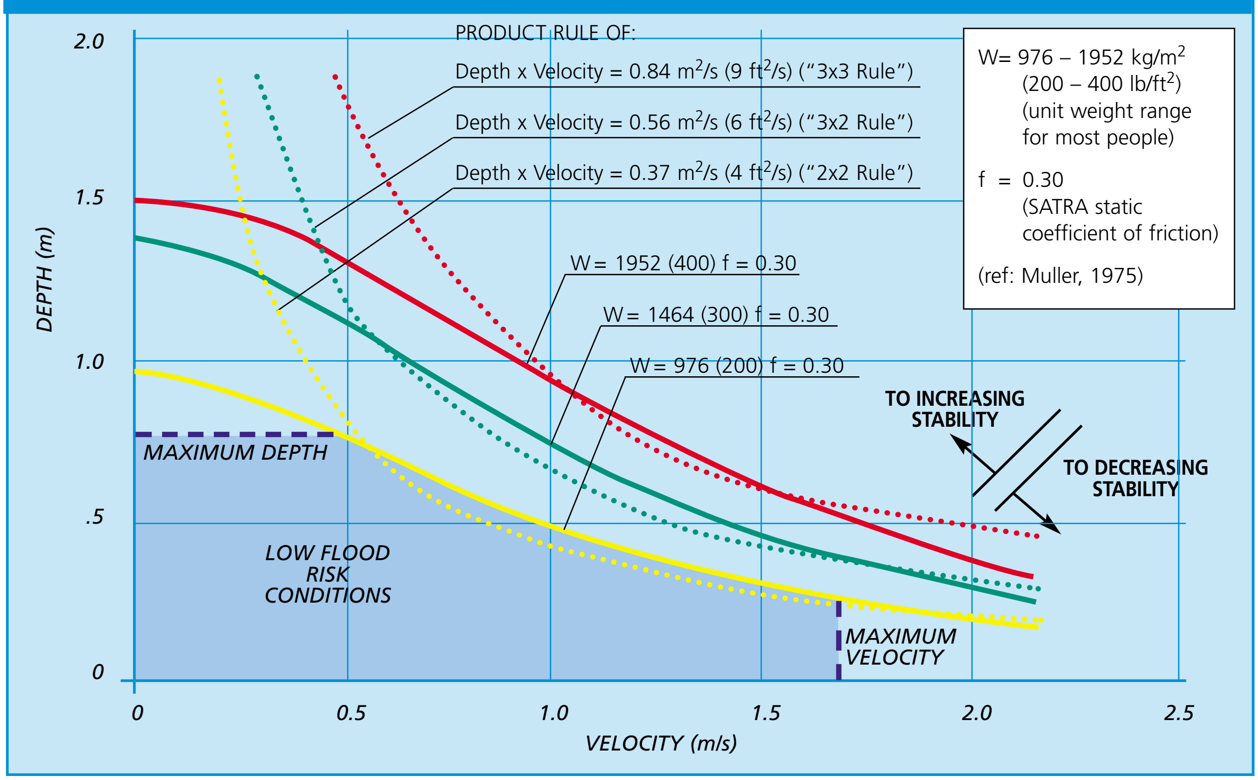


Table 5.4 Curve numbers (CN) and constants for the case $I_a = 0.2S$ (from Mockus, 1972)

(1)	(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)
CN for Condition II	*CN* for Conditions I	*CN* for Conditions III	S Values[a] (in.)	Curve[a] Starts Where P = (in.)	*CN* for Condition II	*CN* for Conditions I	*CN* for Conditions III	S Values[a] (in.)	Curve[a] Starts Where P = (in.)
100	100	100	0	0.00	61	41	78	6.39	1.28
99	97	100	0.101	0.02	60	40	78	6.67	1.33
98	94	99	0.204	0.04	59	39	77	6.95	1.39
97	91	99	0.309	0.06	58	38	76	7.24	1.45
96	89	99	0.417	0.08	57	37	75	7.54	1.51
95	87	98	0.526	0.11	56	36	75	7.86	1.57
94	85	98	0.638	0.13	55	35	74	8.18	1.64
93	83	98	0.753	0.15	54	34	73	8.52	1.70
92	81	97	0.870	0.17	53	33	72	8.87	1.77
91	80	97	0.989	0.20	52	32	71	9.23	1.85
90	78	96	1.11	0.22	51	31	70	9.61	1.92
89	76	96	1.24	0.25	50	31	70	10.0	2.00
88	75	95	1.36	0.27	49	30	69	10.4	2.08
87	73	95	1.49	0.30	48	29	68	10.8	2.17
86	72	94	1.63	0.33	47	28	67	11.3	2.26
85	70	94	1.76	0.35	46	27	66	11.7	2.35
84	68	93	1.90	0.38	45	26	65	12.2	2.44
83	67	93	2.05	0.41	44	25	64	12.7	2.55
82	66	92	2.20	0.44	43	25	63	13.3	2.65
81	64	92	2.35	0.47	42	24	62	13.8	2.76
80	63	91	2.50	0.50	41	23	61	14.4	2.88
79	62	91	2.66	0.53	40	22	60	15.0	3.00
78	60	90	2.82	0.56	39	21	59	15.6	3.13
77	59	89	2.99	0.60	38	21	58	16.3	3.26
76	58	89	3.16	0.63	37	20	57	17.0	3.41
75	57	88	3.33	0.67	36	19	56	17.8	3.56
74	55	88	3.51	0.70	35	18	55	18.6	3.71
73	54	87	3.70	0.74	34	18	54	19.4	3.88

a. For *CN* in column 1

Table 5.5 Runoff curve numbers for urban areas (Mockus, 1969)[a]

Cover Description	Average Percent Impervious Area[b]	Curve Numbers for Hydrologic Soil Group			
Cover Type and Hydrologic Condition		**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[c]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious area only)[d]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1 to 2 in. sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre (506 m^2) or less (town houses)	65	77	85	90	92
1/4 acre (1,012 m^2)	38	61	75	83	87
1/3 acre (1,349 m^2)	30	57	72	81	86
1/2 acre (2,023 m^2)	25	54	70	80	85
1 acre (4,047 m^2)	20	51	68	79	84
2 acres (8,094 m^2)	12	46	65	77	82
Developing urban areas					
Newly graded area (pervious areas only, no vegetation)[e]		77	86	91	94
Idle lands (CNs are determined using cover types similar to those in Table 5.6)					

a. Average runoff condition, and $I_a = 0.2S$.

b. The average percent impervious area shown was used to develop the composite *CN*s. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a *CN* of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

c. *CN*s shown are equivalent to those of pasture. Composite *CN*s may be computed for other combinations of open space cover type.

d. Composite *CN*s for natural desert landscaping should be computed using Figure 2.3 or 2.4 (in TR-55) based on the impervious area percentage ($CN = 98$) and the pervious area *CN*. The pervious area *CN*s are assumed equivalent to desert shrub in poor hydrologic condition.

e. Composite *CN*s to use for the design of temporary measures during grading and construction should be computed using Figure 2.3 or 2.4 (in TR-55) based on the degree of development (impervious area percentage) and the *CN*s for the newly graded pervious areas.

Table 5.6 Runoff curve numbers for cultivated agricultural lands[a] (Mockus, 1969)

Cover Description			Curve Numbers for Hydrologic Soil Group			
Cover Type	Treatment[b]	Hydrologic Condition[c]	A	B	C	D
Fallow	Bare soil	--	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

a. Average runoff condition, and $I_a = 0.2S$.

b. Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

c. Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including density and canopy of vegetative areas, amount of year-round cover, amount of grass or close-seeded legumes in rotations, and degree of surface roughness. "Poor" indicates that there are factors that impair infiltration and tend to increase runoff. "Good" indicates that there are factors that encourage average and better than average infiltration and tend to decrease runoff.

Table 5.7 Runoff curve numbers for other agricultural lands[a] (Mockus, 1969)

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover Type	**Hydrologic Condition**	**A**	**B**	**C**	**D**
Pasture, grassland, or range-continuous forage for grazing[b]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow-continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush—brush-weed grass mixture with brush the major element[c]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[d]	48	65	73
Woods-grass combination (orchard or tree farm)[e]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[f]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[d]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots		59	74	82	86

a. Average runoff condition, and $I_a = 0.2S$

b. *Poor:* less than 50% ground cover or heavily grazed with no mulch. *Fair:* 50 to 75% ground cover and not heavily grazed. *Good:* more than 75% ground cover and lightly or only occasionally grazed

c. *Poor:* less than 50% ground cover. *Fair:* 50 to 75% ground cover. *Good:* more than 75% ground cover

d. Actual curve number is less than 30; use $CN = 30$ for runoff computations

e. *CN*s shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the *CN*s for woods and pasture.

f. *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning. *Fair:* Woods are grazed but not burned, and some forest litter covers the soil. *Good:* Woods are protected from grazing, and litter and brush adequately cover the soil

Table 5.8 Runoff curve numbers for arid and semiarid rangelands[a] (Mockus, 1969)

Cover Description		Curve Numbers for Hydrologic Soil Group:			
Cover Type	**Hydrologic Condition[b]**	**A[c]**	**B**	**C**	**D**
Herbaceous-mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen-mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper-pinyon, juniper, or both; grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub-major plants include saltbush, greasewood, creosote-bush, blackbrush, bursage, palo verde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

a. Average antecedent moisture condition, and $I_a = 0.2S$. For range in humid regions, use Table 5.7.

b. *Poor:* less than 30% ground cover (litter, grass, and brush overstory). *Fair:* 30 to 70% ground cover. *Good:* more than 70% ground cover.

c. Curve numbers for group A have been developed for desert shrub.

APPENDIX D

Amendment No.1 Summary and Supplemental Information

Item No.	Item	Section No.	Comments
1	Backwater Conditions	3.3.2	Provided guidance with respect to designing flow controls and determining tailwater conditions.
2	Dual Drainage Modelling	3.7.9.6	Provided guidance regarding various modelling approaches that can be used to suit specific SWM designs, including modelling of interaction between catch basins and storm sewers.
3	Infill Development	3.10.1	Provided guidance regarding flexible SWM criteria to be considered for expansions of pre-existing development.
4	Pond Reassessments	3.10.2.1	Provided guidance and clarifications to simplify pond reassessments.
5	Storage Requirements	3.3.2	Revised reliable outflow conditions as 'normal' (standard) storage requirements and zero discharge storage calculations as 'enhanced' storage requirements where more conservative storage volume is warranted.
6	Stress Test	3.7.8.3	Clarified intent and application of the Stress Test.
		3.7.8.3	Reviewed mathematical inconsistencies between Chicago 100-year 4-hour and Stress Test rainfall hyetographs.
		3.7.8.3	Reviewed recommendation to include a new Stress Test storm with 40% increase in rainfall intensity.
		3.3.2	Reviewed requirement to assess Stress Test when storage volumes are estimated assuming zero discharge.
		3.3.2	Provided guidance to facilitate analysis of the Stress Test storm using the Modified Rational Method.
7	Water Quality	3.4.1	Clarified level of protection, particle size distribution and ETV certication requirements.
8	Design Intent	3.0	Provided foreword related to variable nature of hydrology and straightforward design methods.
		3.2.2.4	Provided clarification related to sewer design method and intent.
		3.2.2.7	Noted exceptions to this requirement.
		3.2.3.5	Noted considerations to Municipal, County and Provincial roadway standards / requirements.
		3.3.1.4	Provided additional context and clarifications related to hydrologic analysis for determining allowable release rates.
		3.3.2.2	Provided additional context related to the MRM and Eq. 3.3.2.2.
		3.7.7.4	Provided clarification regarding the use of CN Method.
		3.9.1	Added context related to spatial and temporal variability of the August 2017 extreme event
9	Clearance Depth	3.3.5.6	Provided guidance regarding recommended clearance from new storm sewer inverts to watercourse bottom.
10	Depression Storage	3.7.6.1	Reviewed wooded area value and discuss adjustment of depression storage to model poor drainage conditions.
11	Ice Thickness	3.3.5.7	Provided guidance regarding ice thickness considerations with respect to submerged inlets.
12	Infiltration Parameters	3.7.7.3	Clarified infiltration parameters to be applied for Stress Test evaluations.
		3.7.7.3	Reviewed recommendation to apply normal antecedent conditions to both minor and major system design.
13	Model Validation	3.7.1.2	Provided general guidance regarding flow estimates for large scale agricultural watersheds.

Item No.	Item	Section No.	Comments
14	Model Inputs / Outputs	3.11	Clarified submission requirements related to model inputs and outputs.
15	MRM Max Area	3.3.2.2	Reviewed maximum area criteria for Modified Rational Method (MRM).
16	Outfall Conditions	3.3.5.3	Provided clarification on how to determine return period water levels for outfall conditions and revise link to historical levels.
17	Phasing	3.3.5.10	Clarified considerations for development phasing.
18	Rational Flow Rate	3.3.1.4	Provided guidance on use of Rational Method to determine allowable release rate.
19	Sewer HGL	3.2.2.5	Clarified sewer HGL analysis requirement.
		3.6.1.5	Clarified sewer HGL analysis requirement.
20	Capacity of Receiver(s)	3.3.1.3	Clarified need to confirm the allowable release rate to the immediate receiver and all affected downstream receivers.
21	Timestep	1.5.2	Clarified level of consequence to ease implementation of Graph 3.2.2.6 and 3.7.8.1.
22	Uncontrolled Areas	3.3.5.8	Provided guidance regarding release rates from sites with both controlled and uncontrolled areas.
23	IDF Curves / Rainfall Tables	Appendix B	Revised the % rainfall for the SCS Type II Table - currently only sums to 94%.
		Appendix B	Added the SCS Type II 2-year 24-hour distribution.
		3.2.1.1	Reviewed IDF curve information.
24	Tsheet Calc	3.7.3.1	Clarified that Tsheet calculation in Eq 3.7.3.1 is an iterative process.
25	CN Values	3.7.7.4	Provided guidance with respect to SCS CN values for existing conditions in our region.
26	SWMF Inperviousness	3.2.2.7	Added SWM facilities to Table 3.2.2.7 (C value) and Table 3.7.5.1 (%imp).
		3.7.5.1	Added SWM facilities to Table 3.2.2.7 (C value) and Table 3.7.5.1 (%imp).
27	Formatting	3.3.2	Revised numbering in section 3.3.2.
28	Risk Equation	1.5.1	Revised Eq. 1.5.1: Reliability = 1 - Risk
29	Allowable Rate	3.3.1.4	Revised section 3.3.1.4(1) to specify typical 2-year return period.
30	C Values	3.2.2.7	Revised Asphalt, Concrete, Roof Areas from a C value of 0.95 to a range of 0.90 to 0.95
31	Checklist	3.11	Reviewed checklist of items to be included in a submission.

Landmark Engineers Inc.

COMPARISON OF RAINFALL IN 1995 VERSUS 2007 & 2016

2016 compared to 1995

Duration	Duration	% Change in Rainfall					
		2 Year	*5 Year*	*10 Year*	*25 Year*	*50 Year*	*100 Year*
0.083	5 min	*-1.0%*	*-2.2%*	*-2.8%*	*-3.3%*	*-3.6%*	*-3.9%*
0.167	10 min	*0.8%*	*-0.3%*	*-0.8%*	*-1.2%*	*-1.5%*	*-1.7%*
0.25	15 min	*0.7%*	*-0.2%*	*-0.7%*	*-1.1%*	*-1.3%*	*-1.5%*
0.5	30 min	*-0.2%*	*-0.2%*	*-0.2%*	*-0.2%*	*-0.3%*	*-0.3%*
1	1 hour	*-1.8%*	*-0.2%*	*0.5%*	*1.1%*	*1.5%*	*1.8%*
2	2 hour	*-2.3%*	*-0.4%*	*0.5%*	*1.3%*	*1.7%*	*2.1%*
6	6 hour	*-1.4%*	*-1.4%*	*-1.3%*	*-1.3%*	*-1.3%*	*-1.3%*
12	12 hour	*-0.7%*	*0.4%*	*0.8%*	*1.3%*	*1.5%*	*1.7%*
24	24 hour	*0.2%*	*0.4%*	*0.5%*	*0.6%*	*0.7%*	*0.8%*

2007 compared to 1995

Duration	Duration	% Change in Rainfall					
		2 Year	*5 Year*	*10 Year*	*25 Year*	*50 Year*	*100 Year*
0.083	5 min	*-2.0%*	*-2.2%*	*-2.3%*	*-2.4%*	*-2.4%*	*-2.4%*
0.167	10 min	*-1.7%*	*-2.3%*	*-2.6%*	*-2.8%*	*-3.0%*	*-3.1%*
0.25	15 min	*-2.0%*	*-2.5%*	*-2.8%*	*-3.0%*	*-3.1%*	*-3.2%*
0.5	30 min	*-3.7%*	*-3.7%*	*-3.7%*	*-3.7%*	*-3.7%*	*-3.7%*
1	1 hour	*-4.9%*	*-3.2%*	*-2.5%*	*-1.9%*	*-1.5%*	*-1.2%*
2	2 hour	*-4.9%*	*-2.7%*	*-1.7%*	*-0.8%*	*-0.2%*	*0.2%*
6	6 hour	*-4.0%*	*-3.3%*	*-3.1%*	*-2.8%*	*-2.6%*	*-2.5%*
12	12 hour	*-3.4%*	*-1.8%*	*-1.1%*	*-0.4%*	*-0.1%*	*0.3%*
24	24 hour	*-2.2%*	*-1.5%*	*-1.2%*	*-0.9%*	*-0.7%*	*-0.5%*

Compare 1995 to 2016 Rain Data

Duration	Duration	Rainfall Amounts																	
		2 Year			5 Year			10 Year			25 Year			50 Year			100 Year		
		1995	2016	Change	1995	2016	Change	1995	2016	Change	1995	2016	Change	1995	2016	Change	1995	2016	Change
0.083	5 min	9.6	9.5	-1.0%	12.2	11.9	-2.2%	13.9	13.6	-2.8%	16.1	15.6	-3.3%	17.8	17.1	-3.6%	19.4	18.6	-3.9%
0.167	10 min	14.0	14.1	0.8%	17.6	17.6	-0.3%	20.1	19.9	-0.8%	23.1	22.8	-1.2%	25.4	25.0	-1.5%	27.7	27.2	-1.7%
0.25	15 min	17.3	17.5	0.7%	22.5	22.5	-0.2%	26.0	25.8	-0.7%	30.4	30.0	-1.1%	33.6	33.2	-1.3%	36.8	36.3	-1.5%
0.5	30 min	23.0	23.0	-0.2%	30.5	30.4	-0.2%	35.4	35.3	-0.2%	41.7	41.6	-0.2%	46.3	46.2	-0.3%	50.9	50.7	-0.3%
1	1 hour	28.9	28.4	-1.8%	38.1	38.0	-0.2%	44.2	44.4	0.5%	51.9	52.5	1.1%	57.6	58.4	1.5%	63.2	64.4	1.8%
2	2 hour	34.5	33.7	-2.3%	44.1	43.9	-0.4%	50.5	50.7	0.5%	58.5	59.3	1.3%	64.5	65.6	1.7%	70.4	71.9	2.1%
6	6 hour	42.2	41.6	-1.4%	54.6	53.8	-1.4%	62.8	61.9	-1.3%	73.1	72.1	-1.3%	80.8	79.7	-1.3%	88.4	87.3	-1.3%
12	12 hour	47.9	47.5	-0.7%	61.2	61.4	0.4%	70.0	70.6	0.8%	81.2	82.2	1.3%	89.5	90.8	1.5%	97.7	99.4	1.7%
24	24 hour	54.6	54.7	0.2%	69.0	69.3	0.4%	78.6	79.0	0.5%	90.6	91.2	0.6%	99.6	100.3	0.7%	108.4	109.3	0.8%
				-0.6%			**-0.5%**			**-0.4%**			**-0.3%**			**-0.3%**			**-0.2%**

Compare 2007 to 2016 Rain Data

Duration	Duration	Rainfall Amounts																	
		2 Year			5 Year			10 Year			25 Year			50 Year			100 Year		
		2007	2016	Change	2007	2016	Change	2007	2016	Change	2007	2016	Change	2007	2016	Change	2007	2016	Change
0.083	5 min	9.4	9.5	1.1%	11.9	11.9	0.0%	13.6	13.6	-0.5%	15.8	15.6	-1.0%	17.4	17.1	-1.3%	18.9	18.6	-1.5%
0.167	10 min	13.7	14.1	2.5%	17.2	17.6	2.0%	19.5	19.9	1.8%	22.5	22.8	1.6%	24.6	25.0	1.5%	26.8	27.2	1.4%
0.25	15 min	17.0	17.5	2.8%	22.0	22.5	2.4%	25.3	25.8	2.2%	29.5	30.0	2.0%	32.5	33.2	1.9%	35.6	36.3	1.8%
0.5	30 min	22.2	23.0	3.6%	29.4	30.4	3.6%	34.1	35.3	3.6%	40.1	41.6	3.6%	44.6	46.2	3.6%	49.0	50.7	3.6%
1	1 hour	27.5	28.4	3.3%	36.9	38.0	3.1%	43.1	44.4	3.1%	50.9	52.5	3.1%	56.7	58.4	3.0%	62.5	64.4	3.0%
2	2 hour	32.8	33.7	2.7%	42.9	43.9	2.4%	49.6	50.7	2.2%	58.1	59.3	2.1%	64.3	65.6	2.0%	70.6	71.9	1.9%
6	6 hour	40.5	41.6	2.6%	52.8	53.8	2.0%	60.8	61.9	1.8%	71.1	72.1	1.5%	78.6	79.7	1.4%	86.2	87.3	1.3%
12	12 hour	46.2	47.5	2.8%	60.1	61.4	2.2%	69.2	70.6	2.0%	80.8	82.2	1.7%	89.4	90.8	1.6%	98.0	99.4	1.5%
24	24 hour	53.4	54.7	2.5%	68.0	69.3	2.0%	77.6	79.0	1.7%	89.8	91.2	1.5%	98.9	100.3	1.4%	107.9	109.3	1.3%
				2.7%			**2.2%**			**2.0%**			**1.8%**			**1.7%**			**1.6%**

KEY TAKEAWAY: THE TABLES ABOVE DEMONSTRATE THAT THE 2016 RAINFALL DATA IS MORE INTENSE THAN PREVIOUS 2007 RAINFALL DATA. HOWEVER, THE 2016 DATA IS VERY SIMILAR TO 1995.

COMPARISON OF RAINFALL IN 1995 VERSUS 2007 – % Change vs Standard Deviation

Compare 2007 to 2016 Rain Data

Duration	Duration	Rainfall Amounts																	
		2 Year			5 Year			10 Year			25 Year			50 Year			100 Year		
		2007	2016	Change	2007	2016	Change	2007	2016	Change	2007	2016	Change	2007	2016	Change	2007	2016	Change
0.083	5 min	9.4	9.5	1.1%	11.9	11.9	0.0%	13.6	13.6	-0.5%	15.8	15.6	-1.0%	17.4	17.1	-1.3%	18.9	18.6	-1.5%
0.167	10 min	13.7	14.1	2.5%	17.2	17.6	2.0%	19.5	19.9	1.8%	22.5	22.8	1.6%	24.6	25.0	1.5%	26.8	27.2	1.4%
0.25	15 min	17.0	17.5	2.8%	22.0	22.5	2.4%	25.3	25.8	2.2%	29.5	30.0	2.0%	32.5	33.2	1.9%	35.6	36.3	1.8%
0.5	30 min	22.2	23.0	3.6%	29.4	30.4	3.6%	34.1	35.3	3.6%	40.1	41.6	3.6%	44.6	46.2	3.6%	49.0	50.7	3.6%
1	1 hour	27.5	28.4	3.3%	36.9	38.0	3.1%	43.1	44.4	3.1%	50.9	52.5	3.1%	56.7	58.4	3.0%	62.5	64.4	3.0%
2	2 hour	32.8	33.7	2.7%	42.9	43.9	2.4%	49.6	50.7	2.2%	58.1	59.3	2.1%	64.3	65.6	2.0%	70.6	71.9	1.9%
6	6 hour	40.5	41.6	2.6%	52.8	53.8	2.0%	60.8	61.9	1.8%	71.1	72.1	1.5%	78.6	79.7	1.4%	86.2	87.3	1.3%
12	12 hour	46.2	47.5	2.8%	60.1	61.4	2.2%	69.2	70.6	2.0%	80.8	82.2	1.7%	89.4	90.8	1.6%	98.0	99.4	1.5%
24	24 hour	53.4	54.7	2.5%	68.0	69.3	2.0%	77.6	79.0	1.7%	89.8	91.2	1.5%	98.9	100.3	1.4%	107.9	109.3	1.3%
				2.7%			**2.2%**			**2.0%**			**1.8%**			**1.7%**			**1.6%**

Compare 2007 to 2016 Rain Data

Duration	Duration	Rainfall Amounts																	
		2 Year			5 Year			10 Year			25 Year			50 Year			100 Year		
		2007	2016	StdDev	2007	2016	StdDev	2007	2016	StdDev	2007	2016	StdDev	2007	2016	StdDev	2007	2016	StdDev
0.083	5 min	9.4	9.5	0.04	11.9	11.9	0.00	13.6	13.6	-0.03	15.8	15.6	-0.06	17.4	17.1	-0.08	18.9	18.6	-0.10
0.167	10 min	13.7	14.1	0.09	17.2	17.6	0.09	19.5	19.9	0.09	22.5	22.8	0.09	24.6	25.0	0.09	26.8	27.2	0.10
0.25	15 min	17.0	17.5	0.08	22.0	22.5	0.09	25.3	25.8	0.10	29.5	30.0	0.10	32.5	33.2	0.11	35.6	36.3	0.11
0.5	30 min	22.2	23.0	0.10	29.4	30.4	0.13	34.1	35.3	0.15	40.1	41.6	0.17	44.6	46.2	0.19	49.0	50.7	0.21
1	1 hour	27.5	28.4	0.08	36.9	38.0	0.11	43.1	44.4	0.12	50.9	52.5	0.14	56.7	58.4	0.16	62.5	64.4	0.17
2	2 hour	32.8	33.7	0.08	42.9	43.9	0.09	49.6	50.7	0.09	58.1	59.3	0.10	64.3	65.6	0.11	70.6	71.9	0.12
6	6 hour	40.5	41.6	0.08	52.8	53.8	0.08	60.8	61.9	0.08	71.1	72.1	0.08	78.6	79.7	0.08	86.2	87.3	0.08
12	12 hour	46.2	47.5	0.08	60.1	61.4	0.09	69.2	70.6	0.09	80.8	82.2	0.09	89.4	90.8	0.09	98.0	99.4	0.09
24	24 hour	53.4	54.7	0.08	68.0	69.3	0.08	77.6	79.0	0.08	89.8	91.2	0.08	98.9	100.3	0.08	107.9	109.3	0.08
				0.08			**0.08**			**0.09**			**0.09**			**0.09**			**0.10**

KEY TAKEAWAY: THE TABLES ABOVE DEMONSTRATE THAT THE % CHANGE IS STATISTICALLY INSIGNIFICANT. RECOMMEND MAINTAINING CURRENT IDF CURVES.

HOW TO DETERMINE TAILWATER CONDITIONS FOR FLOW CONTROL DESIGN

SWM designs typically include stormwater quantity control and a corresponding controlled allowable release rate. The practitioner is required to consider the impact of tailwater conditions when sizing the flow control element.

In theory, a detailed hydrodynamic model can compute a hydrograph that measures the variation of both headwater (HW) and tailwater (TW) levels over time. While a model can predictably measure varying head differential conditions (i.e., difference between HW and TW) for the site, it is important to acknowledge that it only does this for the synthetic design storms that are typically simulated in the model. Thus, the tailwater condition in the model represents one or few rainfall distributions and soil conditions.

In practice, there are infinite spatial and temporal variations of rainfall as well as variable antecedent soil conditions that affect runoff and corresponding tailwater conditions. In many instances, simplifying assumptions can be made to deduce the complex and highly variable head differential down to a reasonable approximation of a single design head best fits the expected operating range of the flow control element.

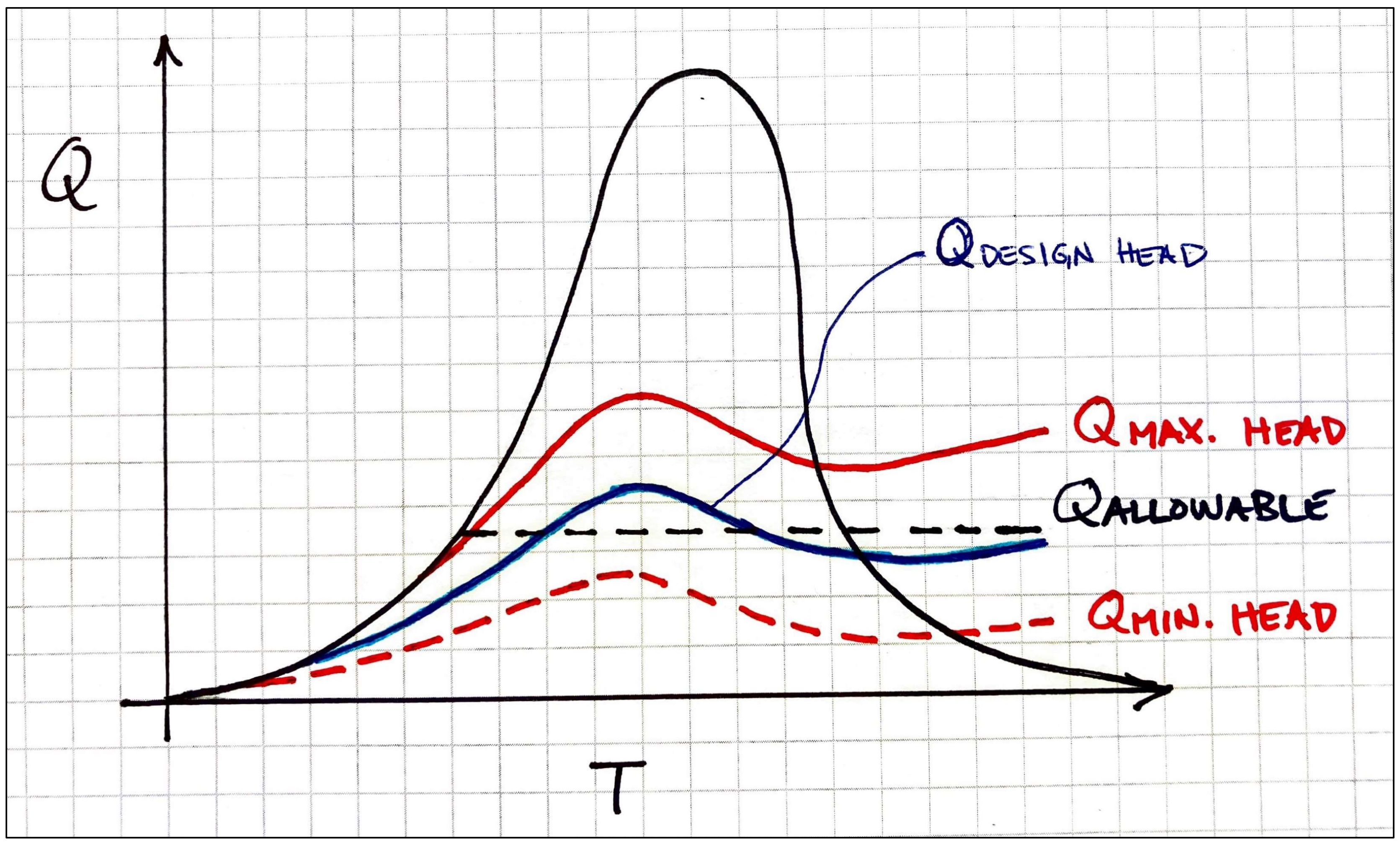
KEY TAKEAWAY:

- HYDRODYNAMIC MODELLING CAN BE MORE ACCURATE TO PREDICT RUNOFF AND ASSOCIATED MAX. WATER LEVELS, BUT ONLY FOR THE SNAPSHOT CREATED BY THE FEW SYNTHETIC RAINFALL EVENTS BEING MODELLED.
- ACTUAL WATER LEVELS IN DRAINAGE SYSTEMS WILL VARY SIGNIFICANTLY BASED ON ACTUAL RAINFALL AND SOIL CONDITIONS.

FLOW CONTROL DESIGN FRAMEWORK

Generally, the design framework of the flow control design is to best fit the allowable release rate. When designing a static flow control element (e.g., a pipe or orifice), the varying discharge rate can exceed allowable rate for a period of time to offset the period of time when the discharge rate is less than the allowable, as depicted in the graph below.

As a simple approach to implement this concept, the practitioner must determine an average head based on assumed design operating ranges for both headwater and tailwater conditions. When assuming max. or min. head levels, the discharge rate can be too restricted or not restricted enough.



KEY TAKEAWAY: WITH SOME EXCEPTIONS, TYPICAL FLOW CONTROL DESIGN SHOULD CONSIDER A VARYING OUTFLOW RATE THAT AVERAGES TO THE SPECIFIED ALLOWABLE RATE (SEE $Q_{\text{DESIGN HEAD}}$ IN GRAPH ABOVE).

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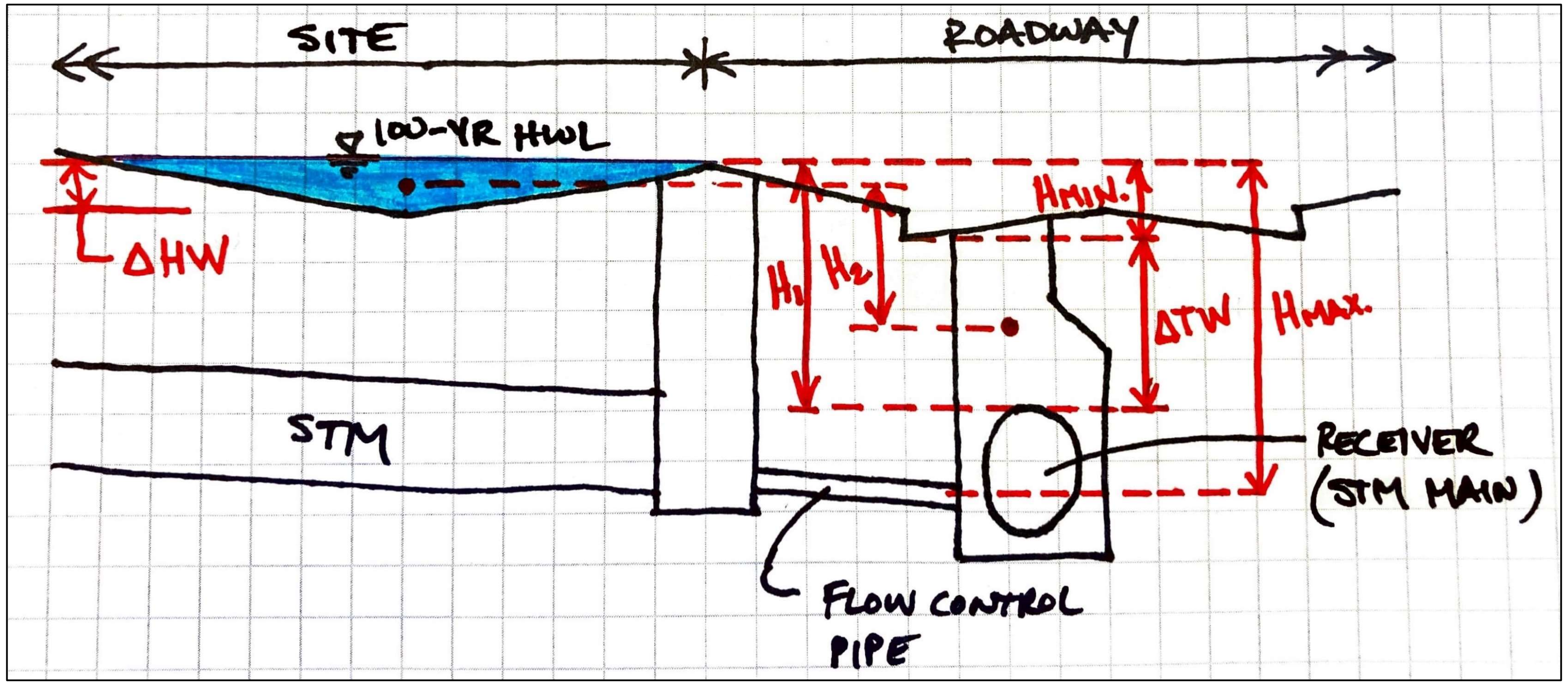
EXAMPLE: DESIGN FRAMEWORK APPLIED TO COMMERCIAL SITE

In this example, the commercial site outlets to a storm sewer within the road right-of-way. To the right is a sketch denoting various head values that can be assumed for flow control design purposes.



- The maximum head (H_{max}) ensures that the site discharge will not exceed the allowable rate. However, this amount of head is an extreme that is unlikely to be reached under most design conditions in the region.
- The minimum head (H_{min}) considers a high tailwater condition. This small amount of head may occur for a short period during a storm but would normally be larger on average. Sizing a flow control using H_{min} would result in a larger flow control and a corresponding discharge rate that could be excessively high during lower tailwater conditions.
- The headwater range (ΔHW) is predominantly going to vary at the surface level for a commercial site where the bulk of the storage is typically provided by the parking lot. The predominant tailwater range (ΔTW) for a storm sewer can reasonably be assumed to vary from the obvert up to ground level. Discharge may exceed the allowable rate when tailwater is below the obvert, however when this occurs, the sewer has available capacity to accommodate the larger flow. When the storm sewer is flowing at its design capacity (i.e., flowing full), the site discharge using a head equal to H_1 release rate will be no greater than the allowable rate.
- H_1 & H_2 represent typical suggested depths for flow control design. H_1 represents a larger head and correspondingly smaller flow control element, which may be more appropriate for certain receivers (e.g., receivers that: are impacted by surcharge conditions; have limited conveyance capacity; have been specifically designed to receive an allowable rate under a major 100-year design storm, etc..). H_2 represents an average of both headwater and tailwater design ranges, which more aptly implements the average design head concept illustrated in the previous slide.

*This example is intended to provide general guidance and does not apply to all site specific conditions.

EXAMPLE: COMMERCIAL SITE FLOW CONTROL DESIGN TO RECEIVING MUNICIPAL DRAIN

In this example, the commercial site is very small compared to the size of the receiver drainage area and it would be impractical to undertake a detailed hydrologic/hydraulic capacity assessment of the receiving drain to determine the potential impact of tailwater conditions on the site outlets. Some conservative simplifying assumptions can be made to provide reasonable criteria for flow control design.

Flow controls can be designed based on an average head within a reasonably assumed operating range, whereby:

a. Max. operating head = On-site High Water Level (HWL) minus assumed water level in drain equal to half the full drain depth*;

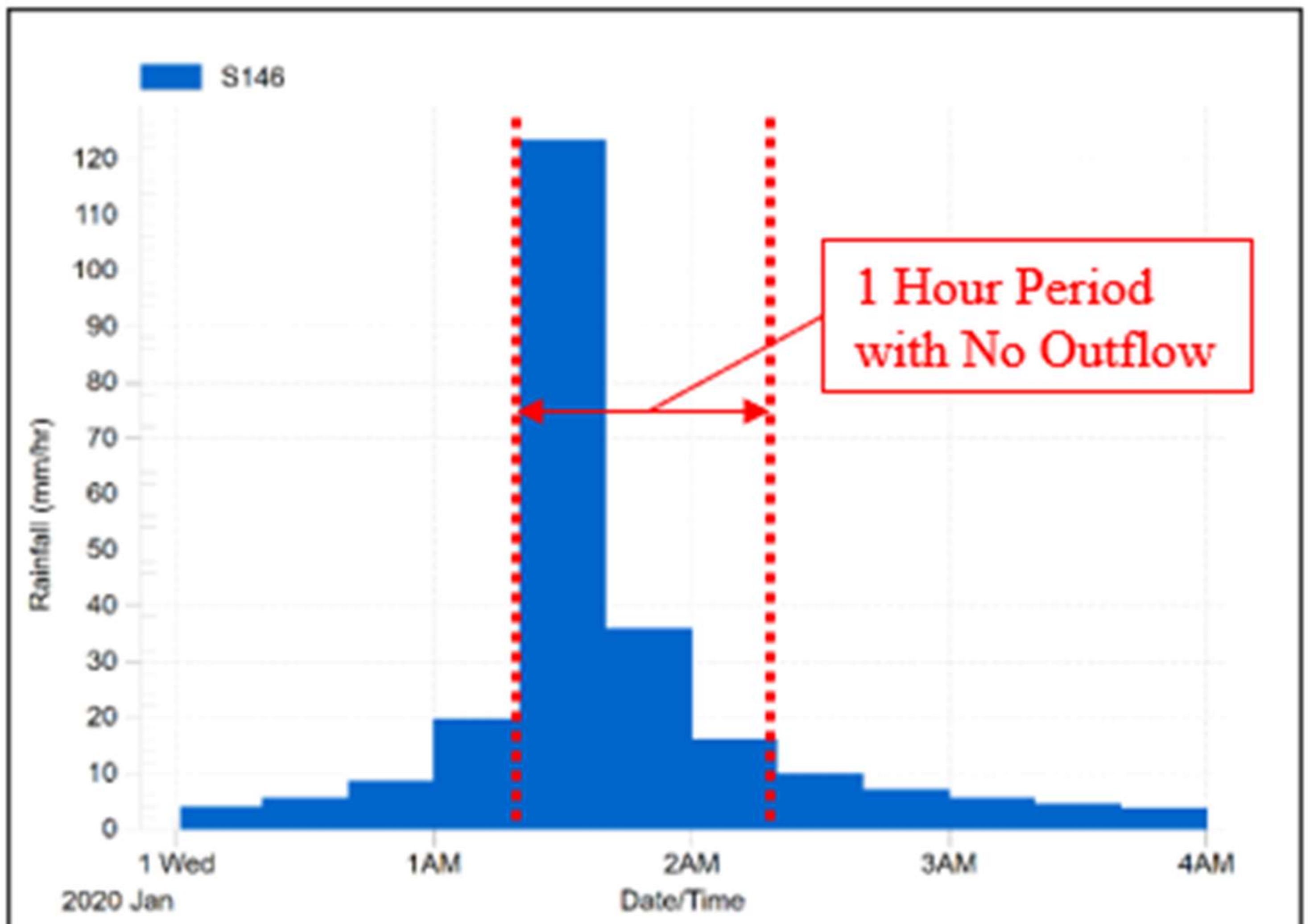
b. Min. operating head = On-site HWL minus assumed water level in drain equal to full drain depth. If less than 0.3m, then a minimum head of 0.3m would generally be deemed appropriate; and,

c. Design head = average of max. & min. operating head

* This assumption should be justified or revised to suit the receiver. In this example, a typical flow rate was estimated based on a simple calculation of the contributing area multiplied by a typical agricultural drainage rate of 25mm/day, and the Manning's equation was used to estimate the corresponding flow depth in the receiving open drain.

EXAMPLE: ACCOUNTING FOR SIGNIFICANT CONVEYANCE CAPACITY CONSTRAINTS AND PROLONGED BACKWATER CONDITIONS

When significant conveyance capacity constraints exist, tailwater elevations can rise quickly and be sustained as the storm subsides and the storm sewer takes some time to drawdown excess runoff stored at the surface. Under these conditions, a practitioner may wish to account for a period of time during the peak rainfall and some assumed time afterwards where the site discharge is limited. A simple conservative assumption would be to assume no outflow for a certain period, as illustrated below. The result is additional storage required on site, which is equal to the assumed period of no outflow multiplied by the allowable rate.



- Additional Storage Volume due to Backwater (m^3) = Q_{allow} (m^3/s) x 3,600 seconds

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REQUIREMENT TO ASSESS STRESS TEST WHEN STORAGE VOLUMES ARE ESTIMATED ASSUMING ZERO DISCHARGE

Section 3.3.2 of the manual provides storage equations that estimate storage depths assuming no discharge. Naturally, this approach provides a conservative storage volume, to varying degrees depending on the actual discharge rate. Intuitively, one might expect that the conservative simplifying assumption of zero discharge could provide a sufficient amount of excess storage volume to offset the additional runoff from a Stress Test event. Of course, the validity of this conclusion depends on the actual discharge rate.

The table at the top right summarizes the critical 100-year storage depths assuming no discharge. Note that aforementioned equations are a linear best fit of these depths at varying imperviousness. The table below it summarizes the Stress Test storage depths with an assumed constant release rate of 5 L/s/ha, which is the rate required to ensure that the Stress Test storage depths are less than the 100-year (no discharge) depths.

*Note: the Strest Test storage depths shown were derived from the same input parameters as the 100-year storage depths except for: IMD values based on dry conditions as per Table 3.7.7.3; and, infiltration rates based on dry conditions (see Item 12 for further discussion).

KEY TAKEAWAY: PROVIDED THAT THE SITE COULD PROVIDE A RELIABLE AVERAGE RELEASE RATE OF 5 L/S/HA OR LARGER, THE STORAGE DEPTHS CALCULATED USING THE STORAGE EQUATIONS IN SECTION 3.3.2 (I.E., 100-YEAR WITH NO DISCHARGE) WOULD PROVIDE SUFFICIENT STORAGE TO CONTAIN THE STRESS TEST RUNOFF VOLUME.

100-YEAR 24-HOUR SCS STORM (NO DISCHARGE)

Imp %	Depth (mm)			
	Soil A	Soil B	Soil C	Soil D
50	58.7	59.5	77.9	89.1
60	68.2	68.8	83.5	92.4
70	77.7	78.1	89.0	95.8
80	87.1	87.4	94.6	99.1
90	96.4	96.5	100.1	102.4

STRESS TEST (DISCHARGE = 5 L/s/ha)

Imp %	Depth (mm)			
	Soil A	Soil B	Soil C	Soil D
50	43	43	62	74
90	94	94	98	100

DIFF: Stress Test - 100YR

Imp %	Depth (mm)			
	Soil A	Soil B	Soil C	Soil D
50	-15.7	-16.5	-15.9	-15.1
90	-2.4	-2.5	-2.1	-2.3

PROPER USE OF MODIFIED RATIONAL METHOD WITHIN ITS LIMITATIONS

The Manual currently accepts that the Modified Rational Method be used for sites less than 2 hectares with the use of a 100-year Runoff Coefficient (100-year C value) based on equation 3.3.2.2.

The main concern with the Rational Method is the potential error in estimating the rainfall to runoff relationship using a single parameter (i.e., C value). This relationship will vary based on numerous factors, such as: soil characteristics, ground cover, ground slope, depression storage, antecedent moisture conditions, rainfall intensity and rainfall amount. Possibly the largest potential source of error is the increase in runoff of pervious area for large rainfall amounts that exceed the soil's saturation point. Eq. 3.3.2.2 is provided to capture this phenomenon. For example, a typical pervious C value of 0.2 would be adjusted to 0.25 for a 100-year storm based on several hydrologic references and a 50% impervious area would have a corresponding 100-year C value of 0.60. However, when using Eq. 3.3.2.2, the excess pervious area runoff generated from large rainfall events is taken into account and the corresponding C value is 0.82.

In addition to the runoff volume concern mitigated by the application of Eq. 3.3.2.2, the other significant potential error with the MRM is the assumed constant release rate being subtracted from the rainfall volume to estimate storage volume required.

The maximum area of 2 hectares was prescribed with the understanding that small sites of this size would typically have surface storage rather than SWM facilities, whereby the upstream water level would quickly reach the surface and produce an outflow similar to the design release rate. The backwater conditions discussion in Section D-3.3.2 herein correlate to this idea where an assumed constant release rate provides a reasonable fit to the expected variation in outflow due to real-world hydrodynamics.

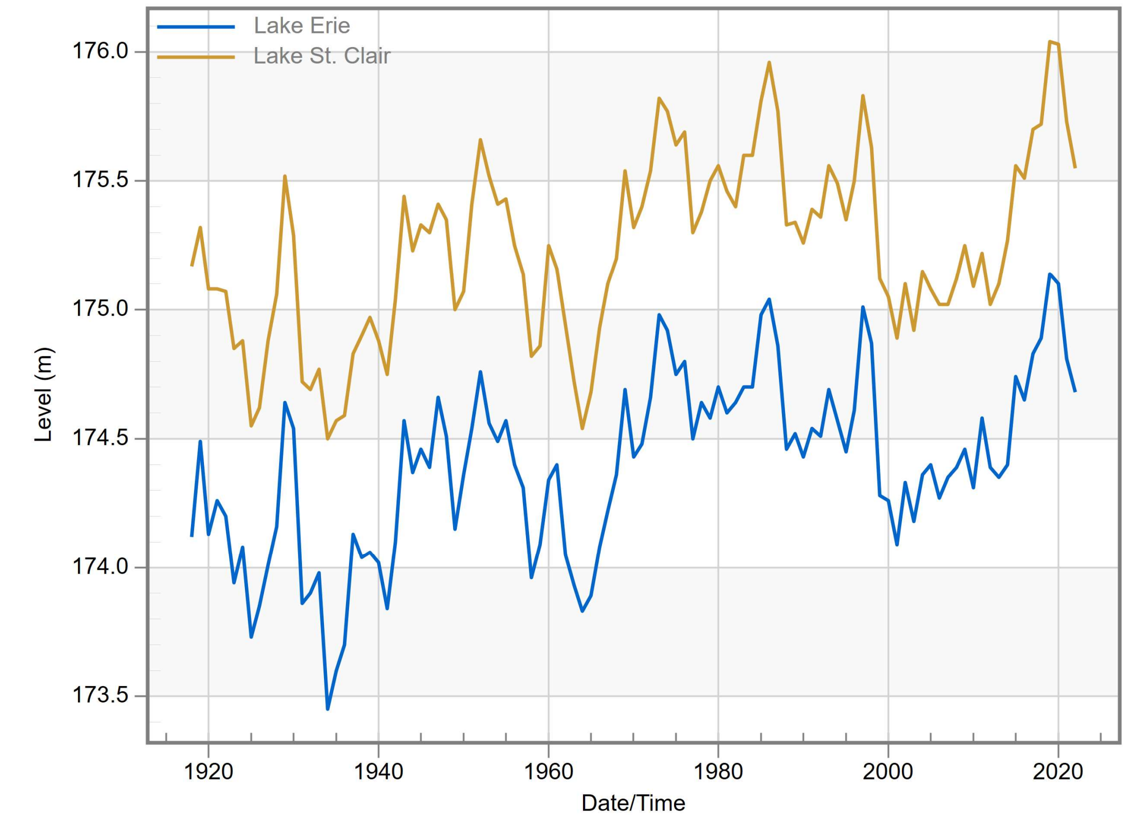
Conversely, a 10 hectare residential subdivision with a wet pond storage facility and gravity outlet would have an outflow that gradually increases with the gradual increase in water level as the pond fills during the storm event. In this instance, the use of MRM is not recommended given that a constant release rate is not a valid assumption, which would result in underestimated storage volume.

KEY TAKEAWAY: THE MRM COULD POTENTIALLY BE USED FOR CERTAIN SITES UP TO 5 HECTARES PROVIDED THAT THE PRACTITIONER CAN JUSTIFY THE ASSUMPTION OF A CONSTANT RELEASE RATE AND MUNICIPALITY AND/OR ERCA IS PRE-CONSULTED TO REVIEW AND APPROVE THE USE OF MRM FOR THE SPECIFIC SITE.

The table below provides various levels based on historical annual maximum monthly mean levels from 1918 to 2022 (inclusive).

ANNUAL MAXIMUM MONTHLY MEAN LEVELS IN METRES (1918-2022)

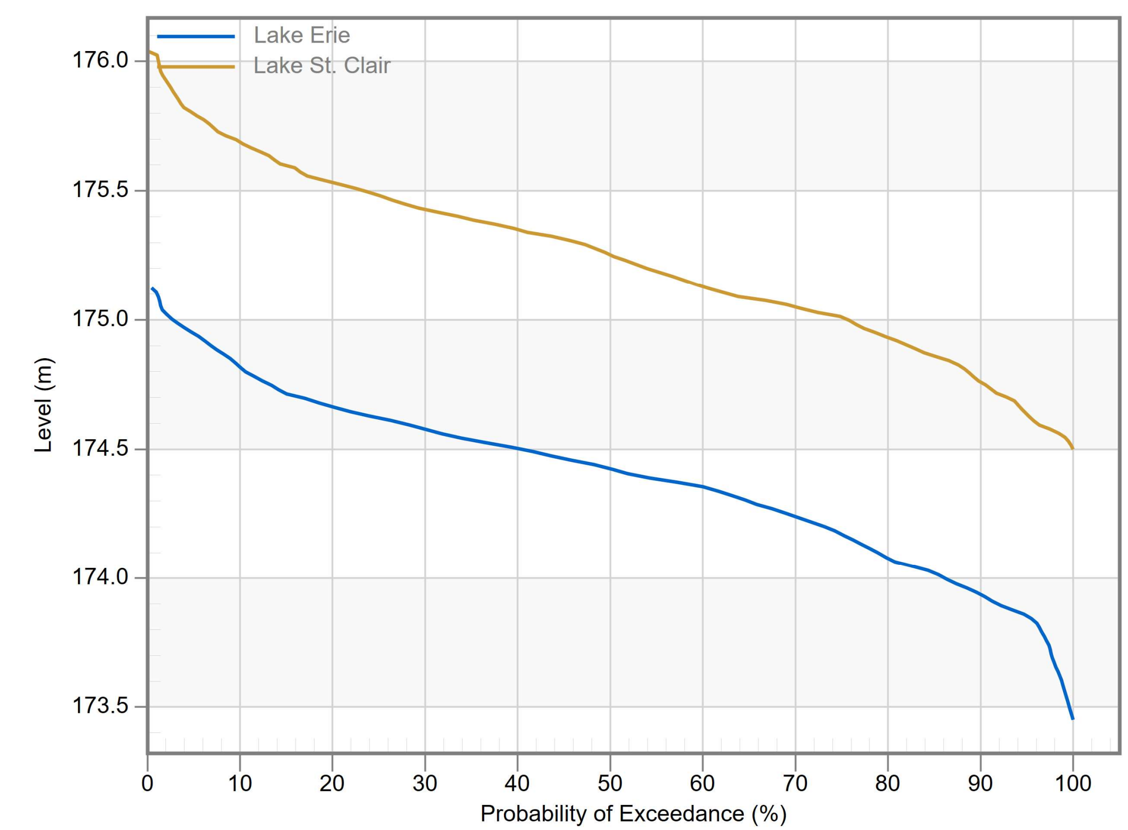
Level	Lake Erie	Lake St. Clair
5-Year	174.68	175.54
10-Year	174.82	175.70
25-Year	174.98	175.82
Average	174.40	175.24
Maximum	175.14	176.04
Minimum	173.45	174.50
Chart Datum	173.50	174.40



When considering outfall conditions along the Detroit river, adjustments to the above listed lake levels can be made based on the difference between lake and river station chart datums (see table below).

CHART DATUMS FOR CANADIAN STATIONS ALONG DETROIT RIVER

Station Name	Station ID	Datum (m)	Lat.	Long.
Tecumseh	11975	174.28	42.341	-82.931
LaSalle	11985	173.93	42.237	-83.107
Amherstburg	11995	173.87	42.144	-83.114
Bar Point	12005	173.50	42.062	-83.115



KEY TAKEAWAY: LAKE WATER LEVELS HAVE BEEN ESTIMATED FOR VARIOUS RETURN PERIODS. CHART DATUMS HAVE BEEN PROVIDED TO INTERPOLATE LEVELS ALONG THE DETROIT RIVER.

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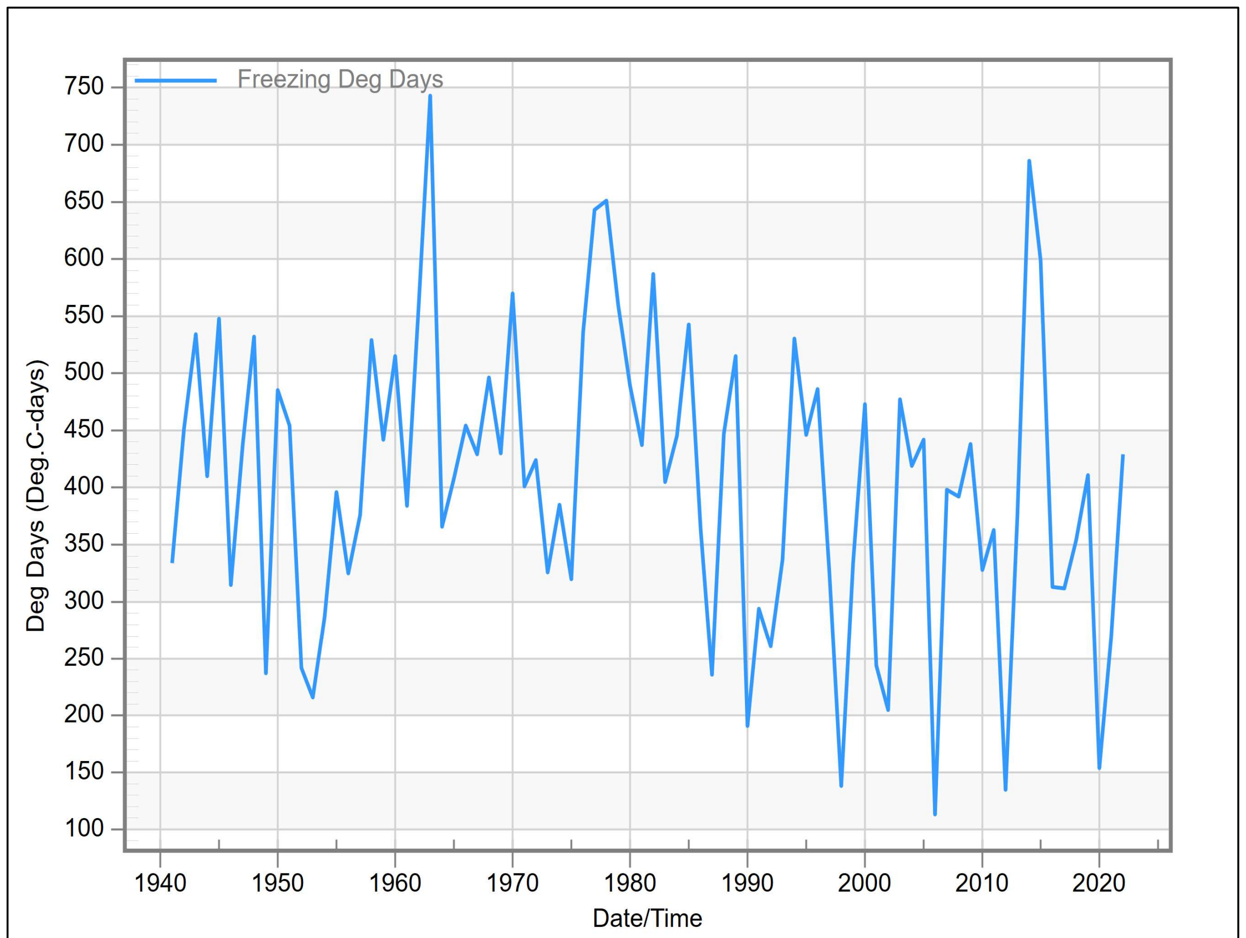
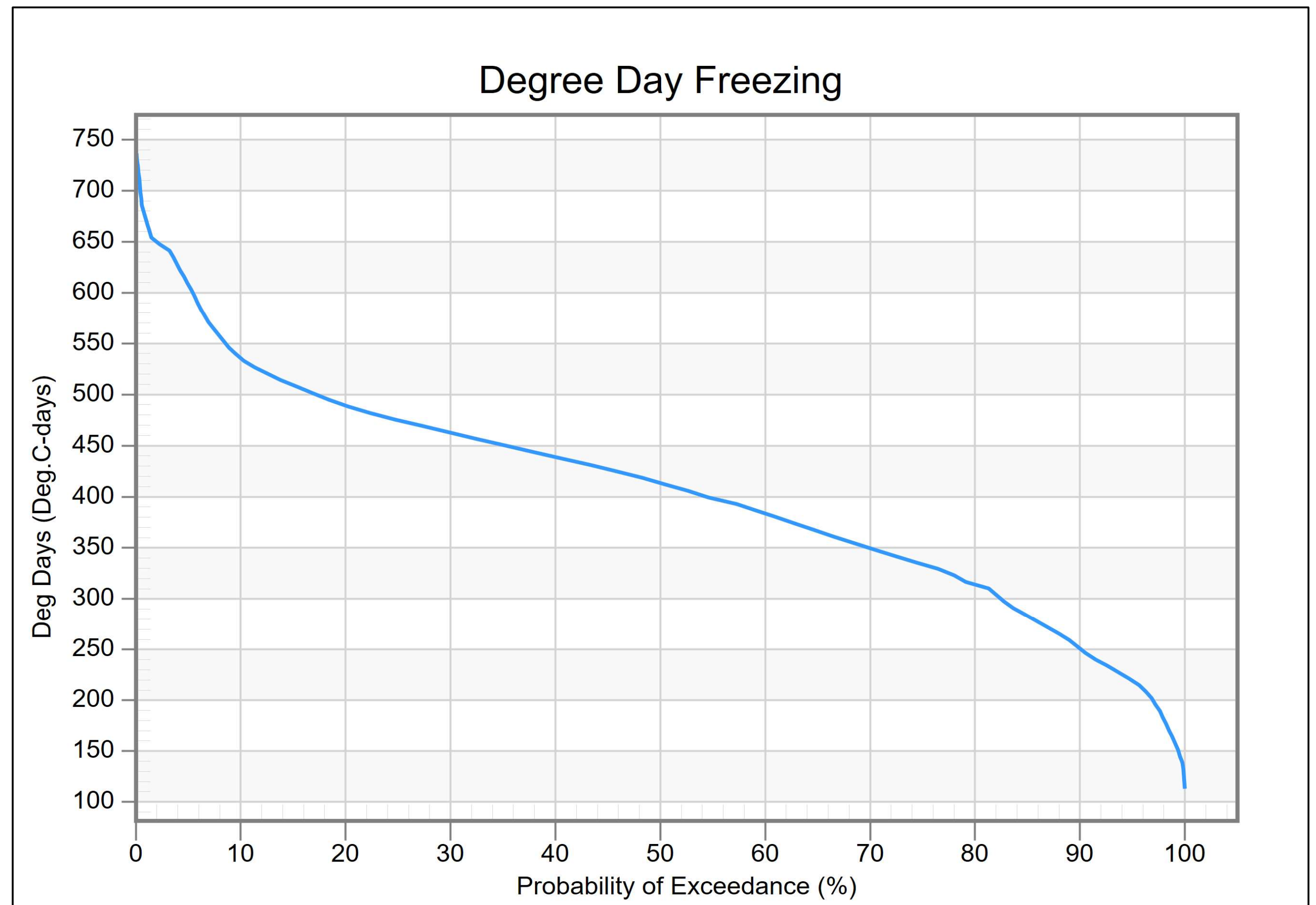
APPLICATION OF ICE THICKNESS CALCULATIONS IN WINDSOR-ESSEX REGION

The 2003 MOE SWM guidelines provide guidance related to estimating ice thickness. From historical climate data from Windsor Airport (shown in graphs below), the annual Freezing Degree Days sums up to a maximum of 740 and an average of 410. From MOE guidance, the manual suggests that a typical pond design would consider a coefficient of 17 – corresponding to maximum and average ice thickness of 460mm and 340mm, respectively. Thus, a SWM pond inlet of 600mm dia. inlet is unlikely to ever get fully blocked. It should be acknowledged that the required conveyance capacity for a snowmelt and/or winter rain event is much smaller than the design capacity based on a typical 5-year design storm (i.e., only partial flow area is needed to drain the sewers). Moreover, the warmer water from these events would melt the ice formation at the inlet and increase the flow area as the event progressed.





KEY TAKEAWAY: BASED ON THE FOREGOING, ICE THICKNESS IS TYPICALLY NOT A CONCERN FOR SWM POND INLETS. HOWEVER, IN INSTANCES WHERE ICE THICKNESS IS A CONCERN, THE MOE GUIDANCE AND THE CLIMATE DATA BELOW CAN BE RELIED UPON TO EASILY ESTIMATE ICE THICKNESS.

ESTIMATING FLOWS FOR LARGE AGRICULTURAL WATERSHEDS

When attempting to estimate flows for large agricultural watersheds, the subjective nature of hydrology can result in a high degree of variability. Whether the estimate is derived from sophisticated modelling tools or a simple Rational Method calculation, the estimated flow can range significantly depending on the methodology used and the input parameters selected.

The Canard River and Big Creek Flood and Fill Line Study prepared by The Proctor and Redfern Group date 1982 plotted 100-year flood flows derived from stream gauge data and determined a best fit line: $Q = 2.05\,A^{0.80}$, where A is area in sq. km.

As a comparison, 100-year flood flows were extrapolated from available Environment Canada stream gauges for Ruscom River (45 years), Canard River (36 years) and Little River (27 years). The result show a reasonably low variability to support the use of these rates as guidance when estimating agricultural flows from relatively flat watershed with clay soils.

KEY TAKEAWAY: WHEN ATTEMPTING TO ESTIMATE FLOWS FOR LARGE AGRICULTURAL WATERSHEDS, THE SUBJECTIVE NATURE OF HYDROLOGY CAN RESULT IN A HIGH DEGREE OF VARIABILITY. GUIDANCE IS PROVIDED HEREIN TO REDUCE THE VARIABILITY.

Watershed 100-Year Flow Data

Watershed	Ruscom	Canard	Little
Area (Sq. Km)	103.5	186.8	44.9
Gauged Flow (L/s/ha)	9.9	6.1	12.5
Canard Study Flow (L/s/ha)	8.1	7.2	9.6

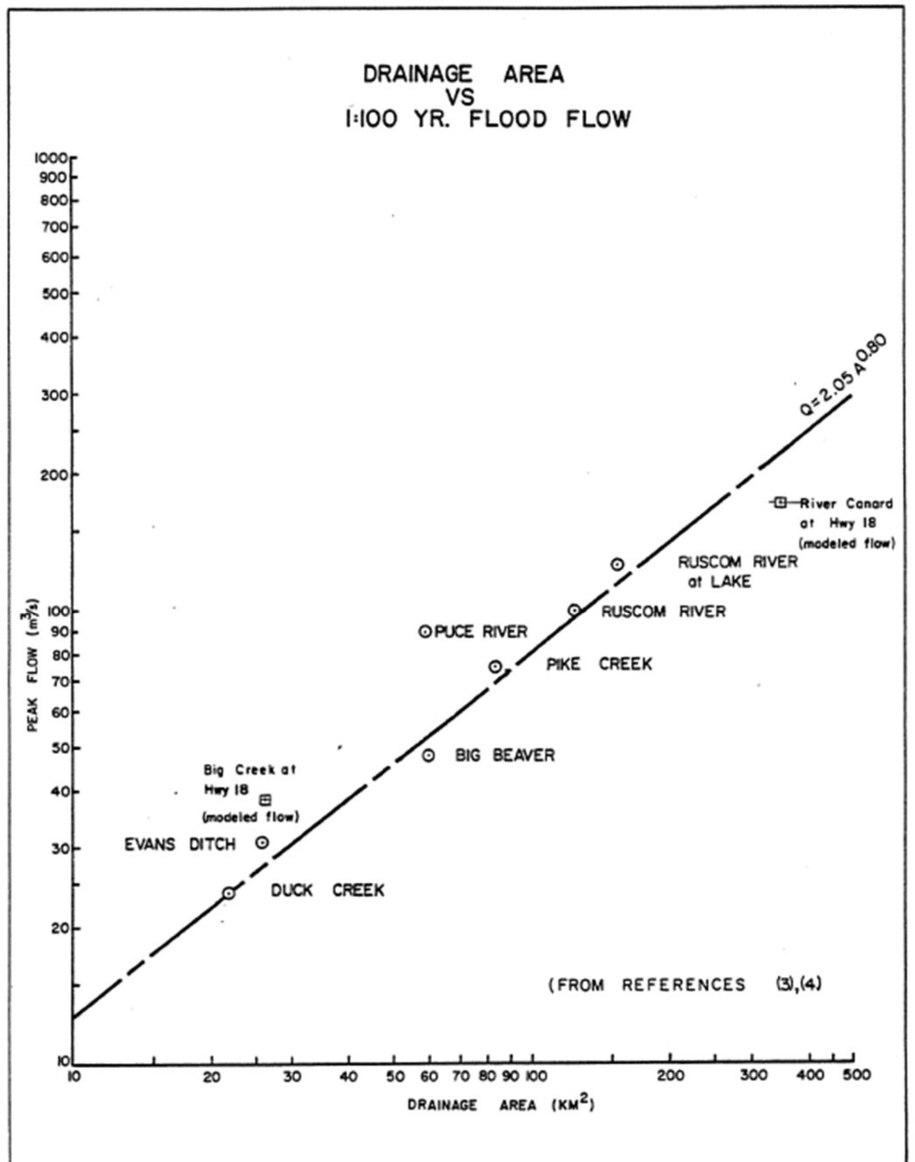


FIGURE 2.2

ATYPICAL DEPRESSION STORAGE VALUES

Depression storage values should be reasoned based on the catchment's ability to hold water. These values affect both runoff volume and rate, making it an important modelling parameter to adjust during model calibration. Typical design values are usually recommended for new development, however, larger values may be warranted when it is important to capture additional storage due to very flat topography, lack of subsurface drainage and vegetation cover. The following provides some general guidance where larger values may be warranted:

- A typical design value of 7.5mm would be applied for newer residential lots with minimum 2-3% grading requirements for front and rear yard and a rear yard catch basin to collect and direct runoff to the sewer main. However, larger residential properties and/or properties with flatter grades may warrant a value of 10 to 15mm. Properties without rear yard drainage may warrant a value of 15 to 30mm, depending on the size of the rear yard.
- A typical design value of 10mm would be applied for flat agricultural land and drained open fields. Very flat land and/or lands with notable local depressions may warrant larger values, for example:
    - A very flat agricultural area may warrant a value of 15 to 20mm;
    - A very flat open field without tile drainage may warrant a value of 20 to 30mm;
- In addition to flat topography, wooded areas can also provide a notable amount of rainfall loss via interception and absorption. These amounts are typically added to depression storage values when accounting for rainfall losses in hydrologic modelling analysis. Depending on the topography and density of vegetation, values for wooded areas may range between 10 to 40mm.

When characterizing topography, the following provides a suggested range of associated slopes:

- Very Flat = 0.5% or less
- Flat = 0.5 to 2%
- Slight Slopes = 2 to 5%
- Rolling Terrain = 5 to 10%

KEY TAKEAWAY: TYPICAL DESIGN VALUES ARE USUALLY RECOMMENDED FOR NEW DEVELOPMENT, HOWEVER, LARGER VALUES MAY BE WARRANTED WHEN IT IS IMPORTANT TO CAPTURE ADDITIONAL STORAGE DUE TO VERY FLAT TOPOGRAPHY, LACK OF SUBSURFACE DRAINAGE AND VEGETATION COVER.

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INFILTRATION PARAMETERS SHOULD BE ADJUSTED TO SUIT ACTUAL CONDITIONS DURING CALIBRATION

Minimum infiltration rates (f_{min}) or saturated conductivity rates (K_s), as listed in the Standards (see table on right), are conservatively low to provide a factor of safety for new design applications. They are not intended to be strictly followed for applications such as model calibration. During model calibration, engineering judgement should vary the infiltration based on antecedent moisture conditions that take into account the conditions that existed at the time of the calibration event. This is particularly critical when calibrating a highly pervious watershed.

Minimum Infiltration / Saturated Conductivity (mm/hr)

Soil Type	Hydrologic Group			
	A	B	C	D
clay	7.6	3.8	1.3	0.5
loam	9.5	5.7	2.5	1.0
sand	11.4	7.6	3.8	1.3

Based on model calibration efforts for a number of recent watershed studies in the region, it has become apparent that the actual hydrologic response from agricultural lands during high-intensity thunderstorms occurring in typical dry summer conditions is significantly dampened and runoff volumes are much lower than what the standard design infiltration parameters would estimate. These studies found that adjusting Hydrologic Group D clay soil conductivity rates to a relatively high value of 3 to 5 mm/hr provided a good fit under the foregoing conditions. While this rate is as much as 10 times that for a typical, conservative standard rate for clay soils, it is very reasonable to expect that very dry clay will experience shrinking and exhibit fissures that would significantly increase infiltration rates. Conversely, calibrating to very wet conditions in the spring or fall found that a very low rate of 0.1 to 0.2 mm/hr provided a better fit to observed runoff data. Here again, the predictable behavior of clay swelling under wet conditions explains the results observed by calibration.

KEY TAKEAWAY: DESIGN INFILTRATION RATES ARE CONSERVATIVELY LOW TO PROVIDE A FACTOR OF SAFETY FOR NEW DESIGN APPLICATIONS. THEY ARE NOT INTENDED TO BE STRICTLY FOLLOWED FOR APPLICATIONS SUCH AS MODEL CALIBRATION.

DESIGN CONDITIONS

Section 3.7.7.1 of the manual recommends that infiltration parameters should be based on dry antecedent conditions for minor system design and normal antecedent conditions for major system design. The following provides the rationale supporting this recommendation:

The minor system is designed for convenient and efficient conveyance of flows produced by the minor design storm. For urban drainage (e.g., storm sewer design), it is typically the high-intensity thunderstorms that produce the largest peak flows and these storms typically occur in the summer months with dry antecedent conditions. The major system is designed to safely convey and/or store major storm flows to mitigate flooding. As such, there is merit in conservatively assuming a normal antecedent condition for all storm types related to major storm design.

The manual currently does not specify an antecedent moisture condition (AMC) for the stress test event. It is recommended that the stress test consider a dry AMC. It was after all, consideration of the September 2016 and August 2017 extreme events that elicited the stress test and both of these storms had dry AMCs.

MIN. INFILTRATION / SATURATED CONDUCTIVITY RATES

The manual's recommended minimum infiltration / saturated conductivity rates (according to Musgrave, 1955) are low compared to observed infiltration rates under dry conditions. This amendment recommends that new minimum infiltration / saturated conductivity rates be applied for dry conditions and for minor system design, with the following values as recommended in the 1997 MTO Drainage Manual:

- Soil Group A – 25 mm/hr;
- Soil Group B – 13 mm/hr;
- Soil Group C – 5 mm/hr;
- Soil Group D – 3 mm/hr.

KEY TAKEAWAY: INFILTRATION PARAMETERS SHOULD BE BASED ON DRY ANTECEDENT CONDITIONS FOR MINOR SYSTEM DESIGN AND NORMAL ANTECEDENT CONDITIONS FOR MAJOR SYSTEM DESIGN. LARGER INFILTRATION RATES ARE RECOMMENDED FOR DRY CONDITIONS AND FOR MINOR SYSTEM DESIGN.

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IS A 40% INCREASE IN RAINFALL INTENSITY JUSTIFIED?

A 40% increase in rainfall has been applied in recent master drainage plans within the region. Landmark's interpretation of the recent major rainfall events as well as the findings from the 2015 Regional IDF Study for Southern Ontario do not support an increase in rainfall intensity. The recent major events of September 2016 and August 2017 did not exceed current 100-year design rainfall intensity at the most intense gauge readings.

Moreover, the 2015 Regional IDF Study trend analysis results indicated that there is no significant trend with a slight apparent decreasing trend for short duration storms. More importantly, the study recommended that; **"trend results should be taken with caution in part because there is no objective way to discriminate trends among climatic trends, anthropogenic caused changes and sampling variability."** [Emphasis Added]

Landmark's interpretation of this key statement is that we simply don't know if rainfall trends are the result of natural climate variability, man-made causes or variability of storm paths and spatial distribution relative to a given historical rain gauge. Based on the above, an increase in rainfall intensity does not seem warranted.

CURRENT 100-YEAR DESIGN STANDARDS ARE CONSERVATIVE

It should be acknowledged that the 100-year 4-hour Chicago design storm is a synthetic storm distribution that is very peaky and inherently conservative in the manner in which it is derived (i.e., the Chicago storm is a theoretical frequency distribution, not observed in nature, and gives the most intense storm possible for any given frequency and duration, independent of the specific storm). For example, the intense thunderstorm that produces a 100-year rainfall of 218 mm/hr over 5 min is unlikely to be the same storm that produces a 100-year rainfall amount of 108mm over 24 hours. High-intensity short-duration rains tend to be isolated cloudburst events, which are unlikely to be concurrent with long-duration rainfall intensities. Moreover, the estimation of runoff produced by this design rainfall will typically apply the recommended design infiltration parameters, which are conservatively low to provide a factor of safety in design.

Therefore, we believe that the combination of these two conservative design inputs provides a reasonable and sufficient level of service for sizing of conveyance elements, with few exceptions where consequences of design exceedance are severe.

KEY TAKEAWAY:

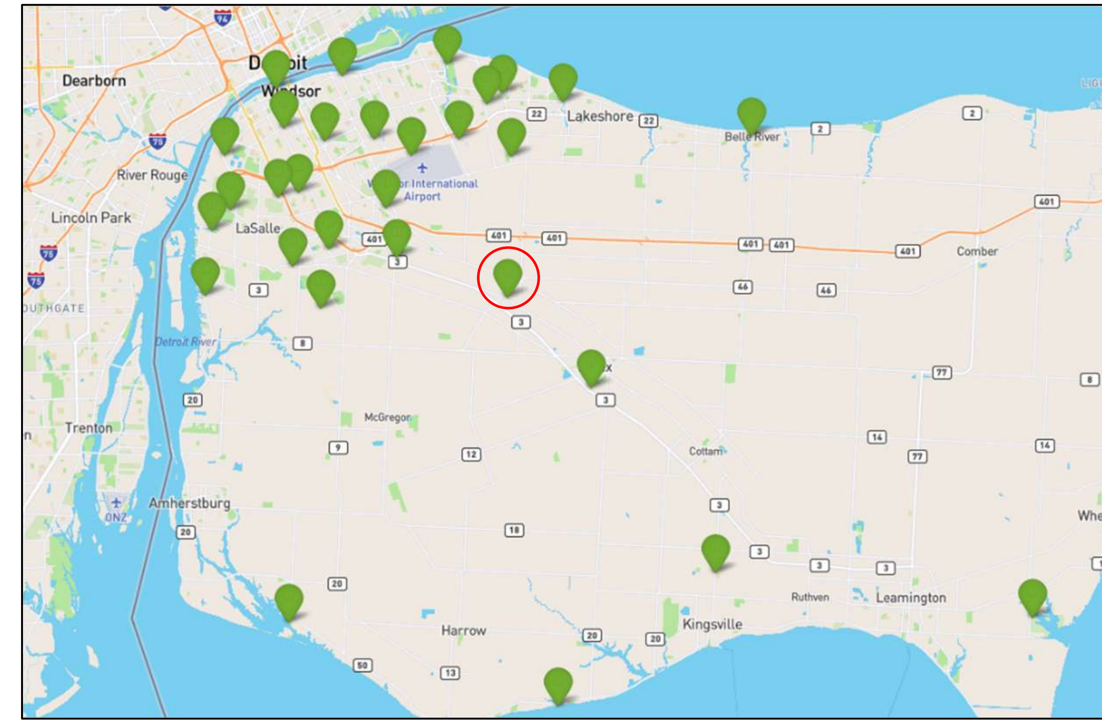
- INCREASE IN RAINFALL INTENSITY SEEMS UNWARRANTED AT THIS TIME.
- THE CHICAGO DESIGN STORM IS A SYNTHETIC RAINFALL DISTRIBUTION WHICH GIVES THE MOST INTENSE STORM POSSIBLE FOR A GIVEN FREQUENCY AND DURATION.
- INFILTRATION PARAMETERS ARE CONSERVATIVELY LOW TO PROVIDE A FACTOR OF SAFETY IN DESIGN.

SAMPLING VARIABILITY

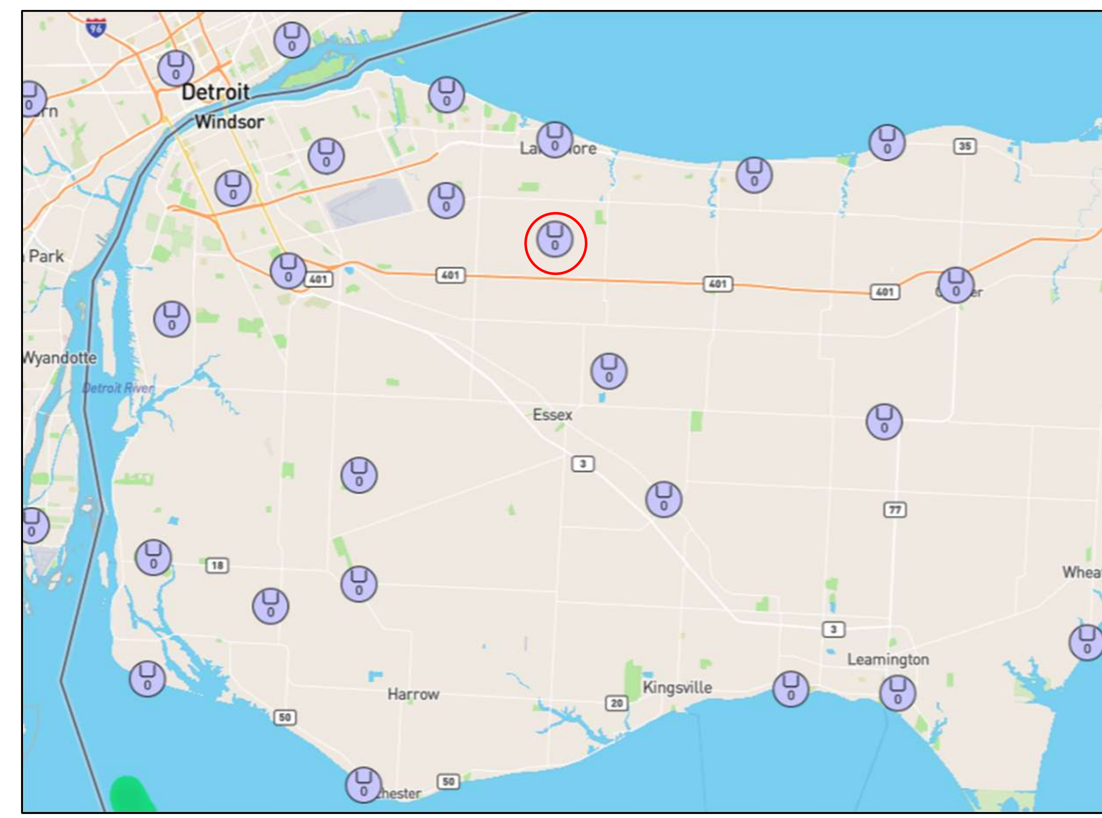
Over the last decade, the network of rain gauges in the Essex region has growth significantly. The screenshots herein provide a view of the extensive network of public and private gauges. Clearly, rainfall can now be sampled from numerous rain gauges and high intensity rainfall from isolated cloudbursts are more readily captured by the extensive gauge network. While more information can be a good thing, it can also create confusion. For example, it is possible that a 100-year rain event can be recorded at one of the numerous gauges every year, however, this does not mean that the frequency of historical rainfall intensities and durations are significantly changing.

KEY TAKEAWAY: THE PROBABILITY OF A SPECIFIC RAINFALL INTENSITY AND DURATION SHOULD NOT BE DETERMINED FROM VARIOUS GAUGES. RATHER, IT SHOULD BE DETERMINED BY ANALYZING RAIN DATA OCCURRING AT THE SAME GAUGE OVER TIME (E.G., WINDSOR AIRPORT).

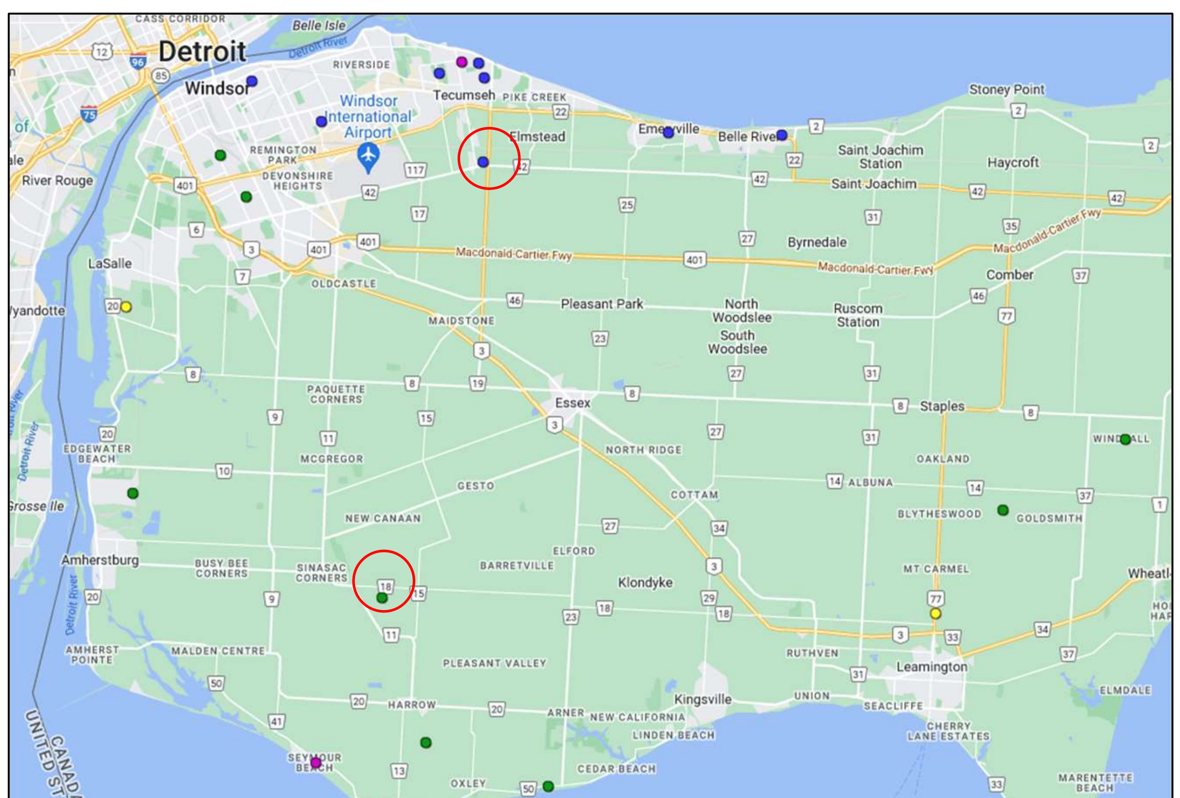
CAMPBELL CLOUD RAIN GAUGE NETWORK



WEATHER UNDERGROUND PRIVATE RAIN GAUGE NETWORK



COCORAHS PRIVATE RAIN GAUGE NETWORK



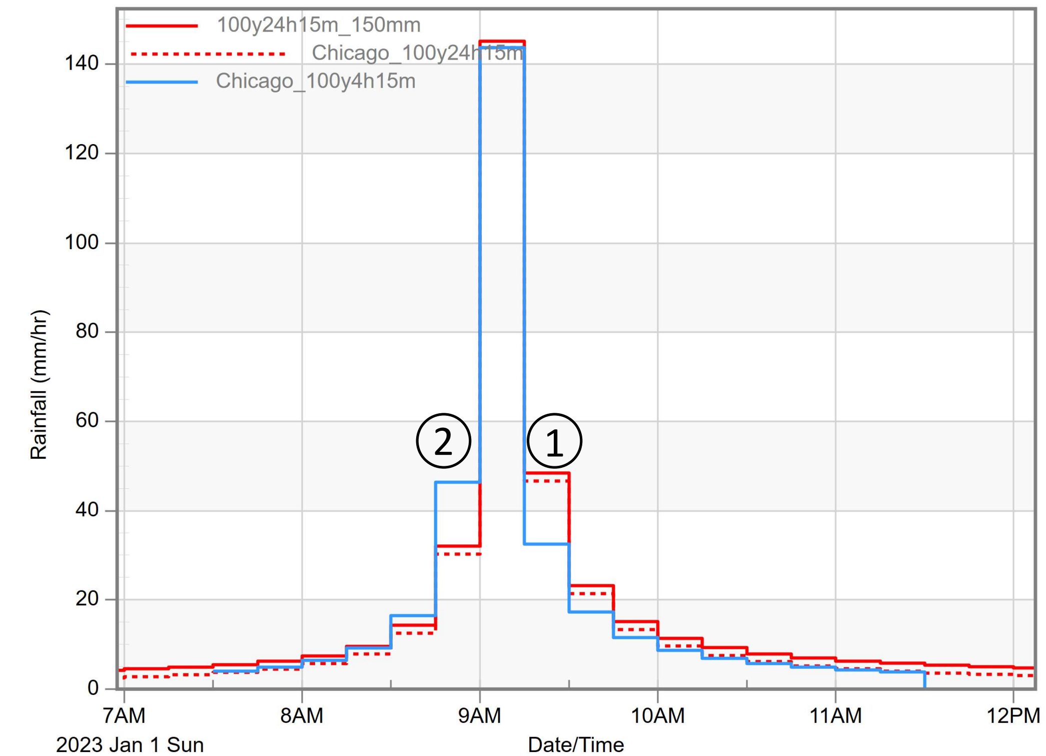
REVIEW OF MATHEMATICAL INCONSISTENCIES BETWEEN CHICAGO 100-YEAR 4-HOUR AND URBAN STRESS TEST RAINFALL HYETOGRAPH

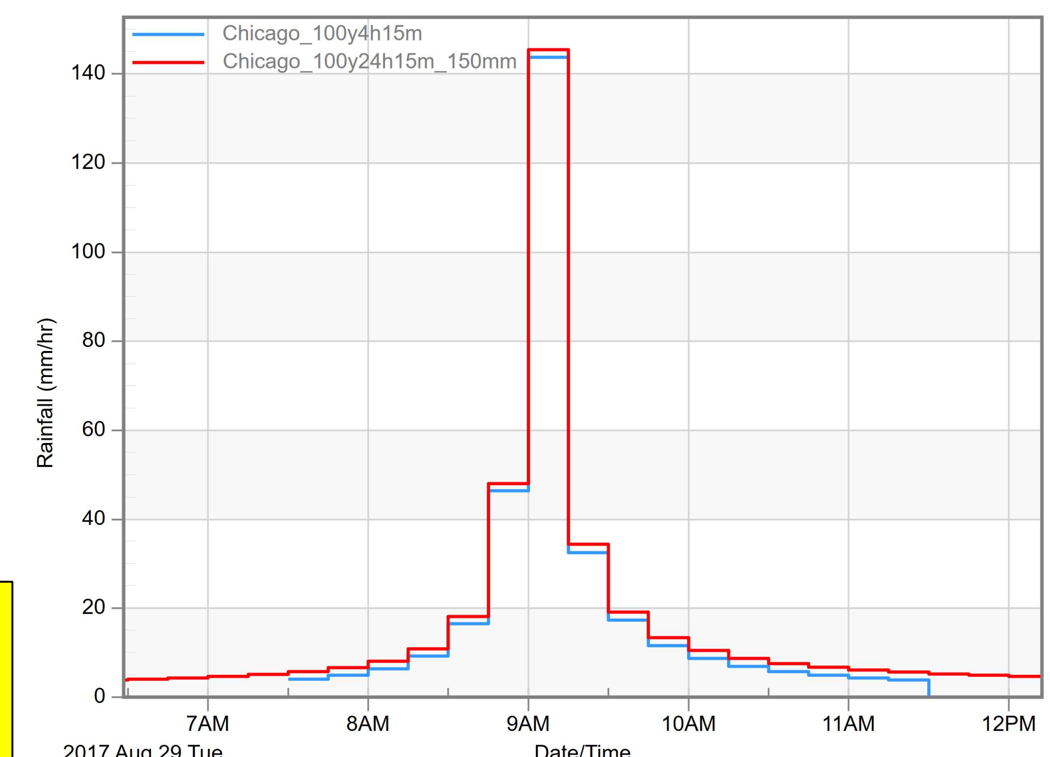
It has been concluded from a recent watershed study in this region that the Chicago 100-year 4-hour storm produced more critical results than the Urban Stress Test (UST, 100y24h15m_150mm) event. As previously mentioned, the application of the Stress Test was to simply to add 42mm of rainfall spread over 24 hours (i.e., a uniform rainfall intensity increase of 1.75mm/hr). As such, the 4-hour rainfall intensities were intended to be slightly smaller (i.e., 1.75mm/hr less) than the Stress Test at each rainfall interval in the storm hyetograph.

Theoretically however, the Chicago distribution methodology places the peak rainfall intensity at the prescribed ratio of time before the peak compared to the total storm time. In our region, a ratio (r) of 0.38 is commonly used. Once the peak rainfall interval is placed, the subsequent rainfall intervals alternate between 'before' and 'after' the peak until the full hyetograph is derived. The alternating sequence begins either 'before' or 'after' the peak depending on the location of the time to peak relative to the hyetograph's rainfall interval start/end times.

For example, the 24-hour distribution second rainfall interval is placed after the peak interval (labelled on top right graph as 1) whereas the 4-hour distribution places the second rainfall interval before the peak (labelled on top right graph as 2). This difference results in an additional 6mm of rainfall occurring before the peak rainfall interval during the 4-hour distribution as compared to the 24-hour, which explains how the 4-hour storm would derive more critical results.

By using a ratio of 0.3758, the location of the peak relative to the time interval is the same for both the 4-hour and 24-hour storms and as such, the UST encapsulates the 4-hour storm (as shown in the bottom right graph).





KEY TAKEAWAY:

- CONCEPTUALLY, THE UST SHOULD INHERENTLY PRODUCE LARGER FLOWS THAN THE CHICAGO 100-YEAR 4-HOUR (I.E., THE LATTER RAINFALL HYETOGRAPH SHOULD HAVE BEEN FULLY EMBEDDED WITHIN THE UST HYETOGRAPH). HOWEVER THE CHICAGO DISTRIBUTION METHODOLOGY CREATED HYETOGRAPHS THAT SOMEWHAT DEVIATED FROM THE INTENT.
- REMEDY: REVISE UST TO MEET ORIGINAL INTENT (USE r=0.3758).

DUAL DRAINAGE MODELLING – IS IT NECESSARY?

Important Reminder: A model should always be created to meet the objective.

Dual drainage modelling consists of modelling the conveyance capacity and interaction between both the minor system (i.e., the storm sewer) and the major system (i.e., overland flow – roadways).

New development designs in the region typically require 5-year storm sewer design and road grading to ensure overland routing towards a SWM pond. Thus, the objective of meeting required conveyance capacity for both minor and major systems can generally be achieved by a spreadsheet calculation (using Rational Method and Manning's Equation) and proper road grading design (i.e., dual drainage modelling is not necessary for new development). Generally, the region is flat and surface flow depths and velocities that may arise during a high-intensity rainfall are well below safety thresholds that would warrant modelling to estimate surface depths/velocities.

The foregoing does not preclude the use of dual drainage modelling, which may be warranted to address certain objectives and conditions that require an understanding of minor and major system interaction and performance. When dual drainage modelling is deemed necessary, it is important to acknowledge that not all dual drainage models are equal as the interaction between sewers and roadways can be represented in various ways – ranging from simple to complex. The following two methods will be further discussed within this manual:

- Method 1: No Inlet Capacity Restriction
- Method 2: Dual Orifices Representing Grate and Lead Separately

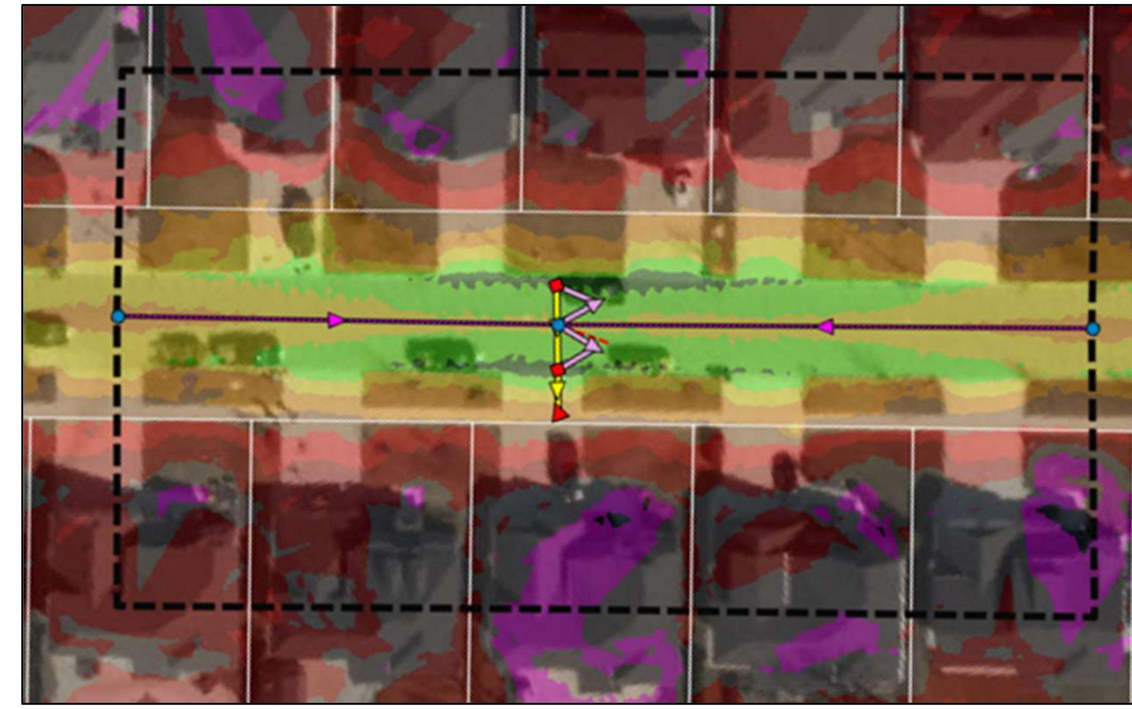
KEY TAKEAWAY: A MODEL SHOULD BE FIT FOR PURPOSE. NEW DEVELOPMENTS, PARTICULARLY THOSE IN THE PREDOMINANT FLAT TERRAIN IN OUR REGION, CAN TYPICALLY BE DESIGNED WITHOUT ANY MODELLING (DUAL DRAINAGE OR OTHERWISE).

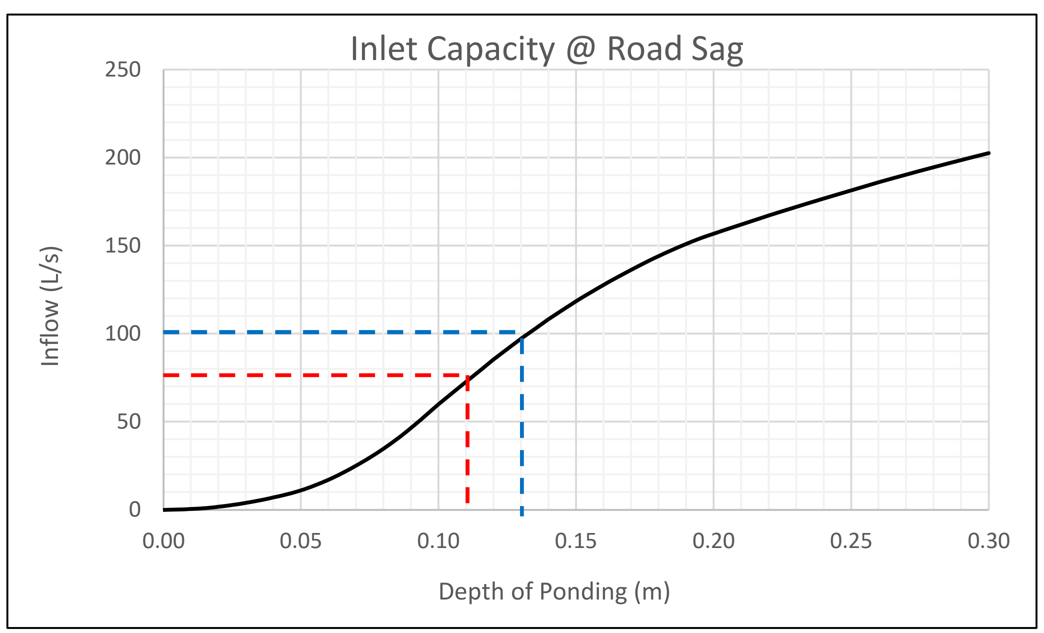
METHOD 1: NO INLET CAPACITY RESTRICTION

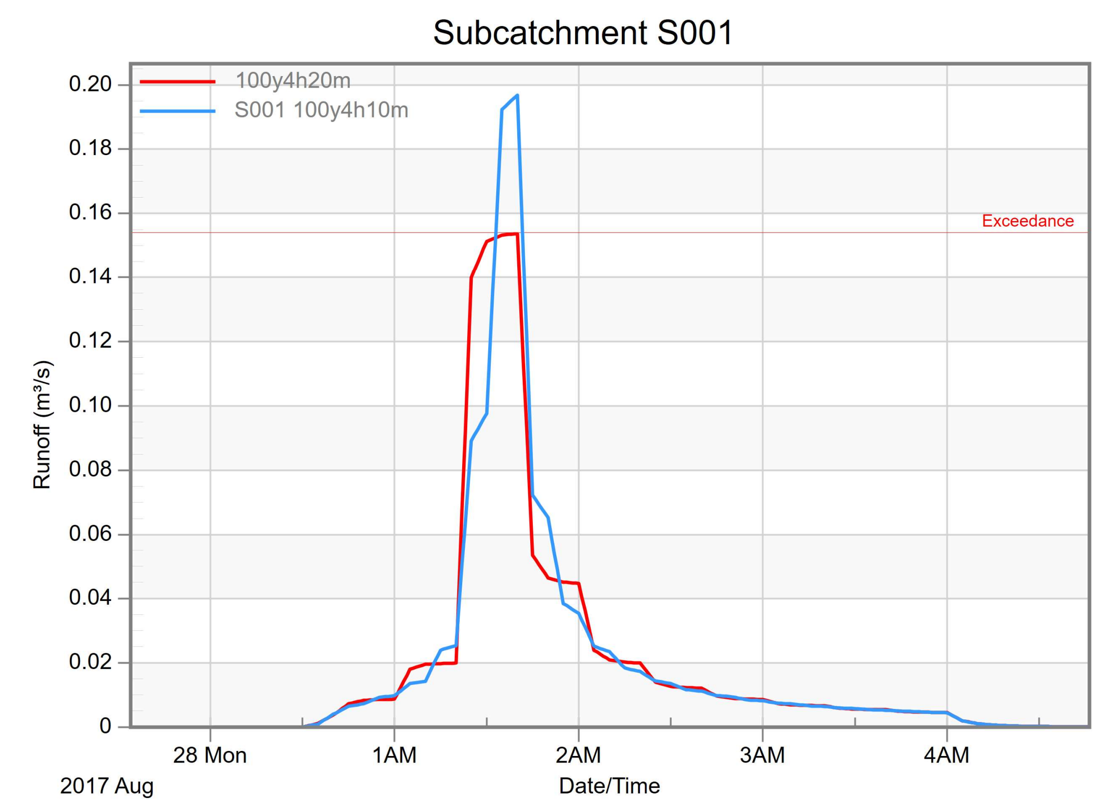
This method assumes that there is no capacity restrictions between the minor and major systems. The model interaction typically consists of larger bottom orifices which assume no inlet capacity limitations and simulate an unrestricted flow between the surcharged sewer to the road surface. This is a simplifying assumption that suits many applications in this region for two main reasons:

- Flat terrain requires numerous road sags*;
- A portion of urban runoff is drained directly to the sewer via private drain pipes that collect roof and rear yard runoff.

*Assuming that stormwater from the front half of roofs is directed onto the surface, the catchment area for a set of single catchbasins can be estimated as 90m maximum length by 50m width, as illustrated on top right plan. At 80% impervious, the 100-year flow is estimated to be approximately 200 L/s based on a 10min timestep, as shown on bottom right graph (i.e., 100 L/s to each CB). As shown below, the inlet capacity of a standard CB grate requires a depth of ponding of approximately 0.13m to convey 100 L/s.







KEY TAKEAWAY: CATCH BASINS AT ROADWAY SAGS PROVIDE SUFFICIENT INLET CAPACITY TO CAPTURE 100-YEAR PEAK FLOWS. METHOD 1 IS APPROPRIATE FOR DEVELOPMENTS WHERE CATCH BASINS ARE PREDOMINANTLY AT ROADWAY SAGS.

METHOD 2: DUAL ORIFICES REPRESENTING GRATE AND LEAD SEPARATELY

This method represents the catchbasin grate as a bottom orifice equal to the opening area of the grate. This orifice connects the roadway to a catchbasin node. A separate orifice is added to represent the CB lead, which connects the catchbasin node to the sewer.

This method more realistically represents the interaction between the sewers and roadway. Inherently, this method must also include a more detailed subcatchment discretization to separate flows drained to catchbasins and flows drained to private drain connections (PDCs) along the sewer.

It should be acknowledged that standard drainage practices in this region contain and collect runoff at the individual lot level. Disconnecting roof downspouts is generally recommended, however, runoff from rear yard downspouts is collected by a lot's rear yard rain and drained directly to the sewer. Front yard roof lead disconnections are less common, particularly around paved areas where the potential for ice formation and risk of slip and fall injuries is undesirable. Thus, a significant portion of urban runoff drains to the sewer via PDCs, whereas catchbasin drainage is generally limited to roadways and front yards.

KEY TAKEAWAY:

- METHOD 2 IS MORE REALISTIC BUT REQUIRES SIGNIFICANT ADDITIONAL MODELLING EFFORT THAT MAY PROVIDE LITTLE BENEFIT WHEN CATCH BASINS ARE PREDOMINANTLY LOCATED AT ROADWAY SAGS.
- METHOD 2 MAY BE WARRANTED IN CERTAIN CONDITIONS (E.G., WHEN SURFACE DEPTHS/VELOCITIES ARE IMPORTANT TO ESTIMATE AND WHERE A SIGNIFICANT NUMBER OF CATCH BASINS ARE ON-GRADE (I.E., WHERE THE ROADWAY CONTINUES TO SLOPE ACROSS THE CATCH BASIN AND BYPASS FLOW OCCURS UNDER LARGER GUTTER FLOWS).

SPATIAL VARIABILITY

The figure on the right depicts a square kilometre grid showing 24-hour rainfall amounts from 28 August 2017 6pm to 29 August 2017 6pm, with corresponding colour gradient as defined in the legend. The figure also includes locations of rain gauges within or near the Turkey Creek watershed.

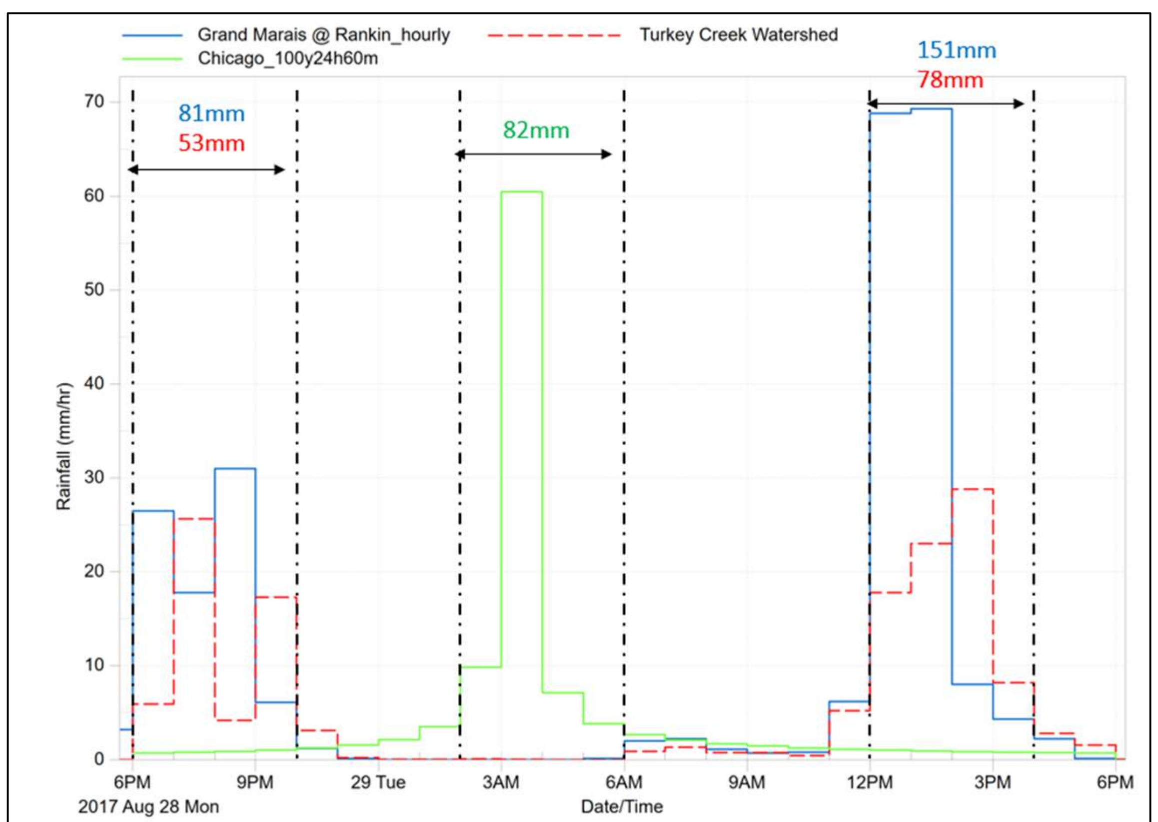
KEY TAKEAWAY: THE AUGUST 2017 EVENT MEASURED ~250MM OF RAINFALL IN 24 HOURS AT THE GRAND MARAIS AND RANKIN RAIN GAUGE, WHEREAS THE TURKEY CREEK WATERSHED AVERAGED ABOUT 150MM.

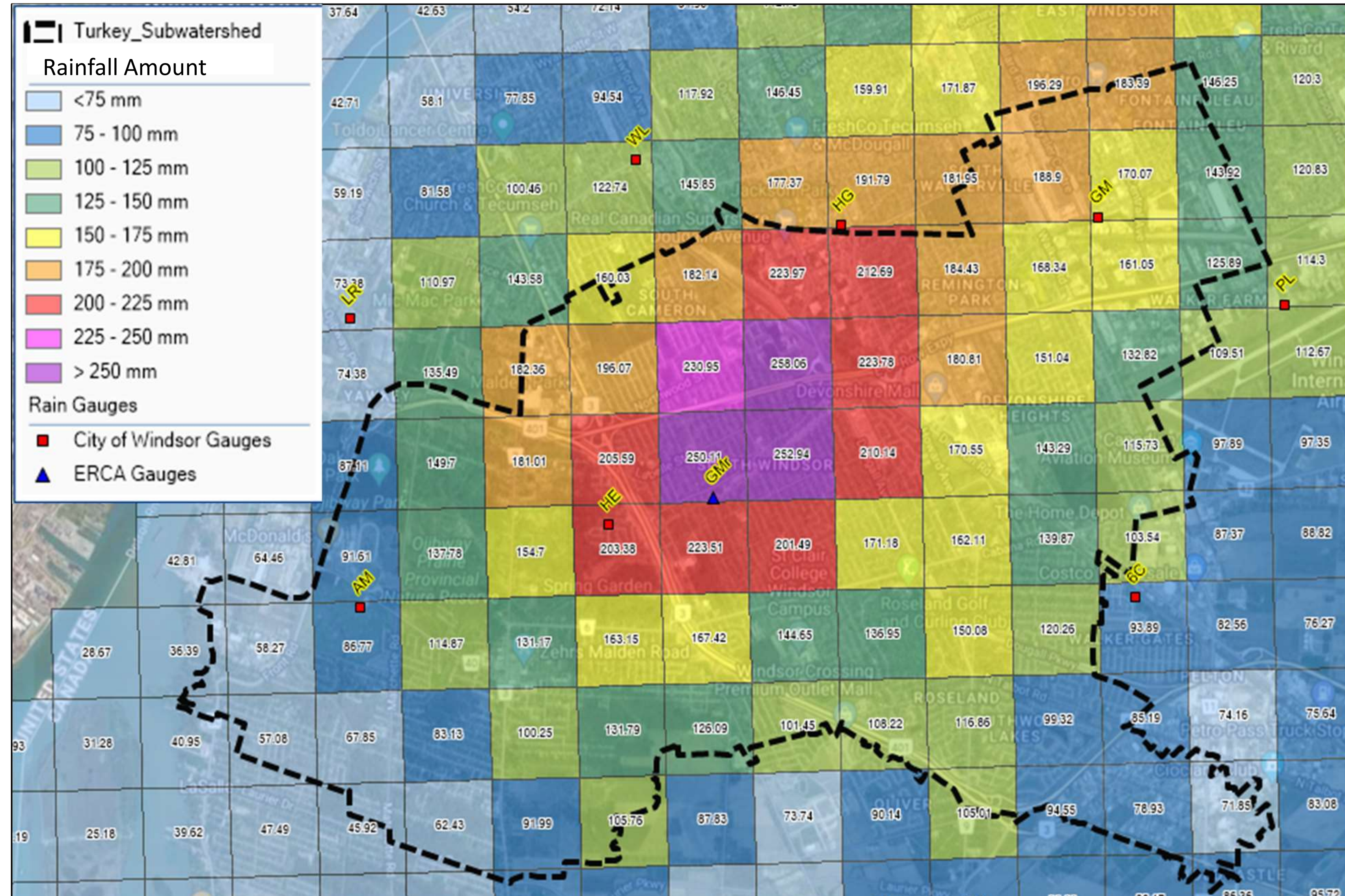
TOTAL 24-HOUR RAINFALL:

Grand Marais @ Rankin = 250mm

Turkey Creek Watershed = 148mm

Chicago 100-Year 24-Hour = 108mm





TURKEY CREEK WATERSHED 24-HOUR RAINFALL – AUGUST 28-29, 2017

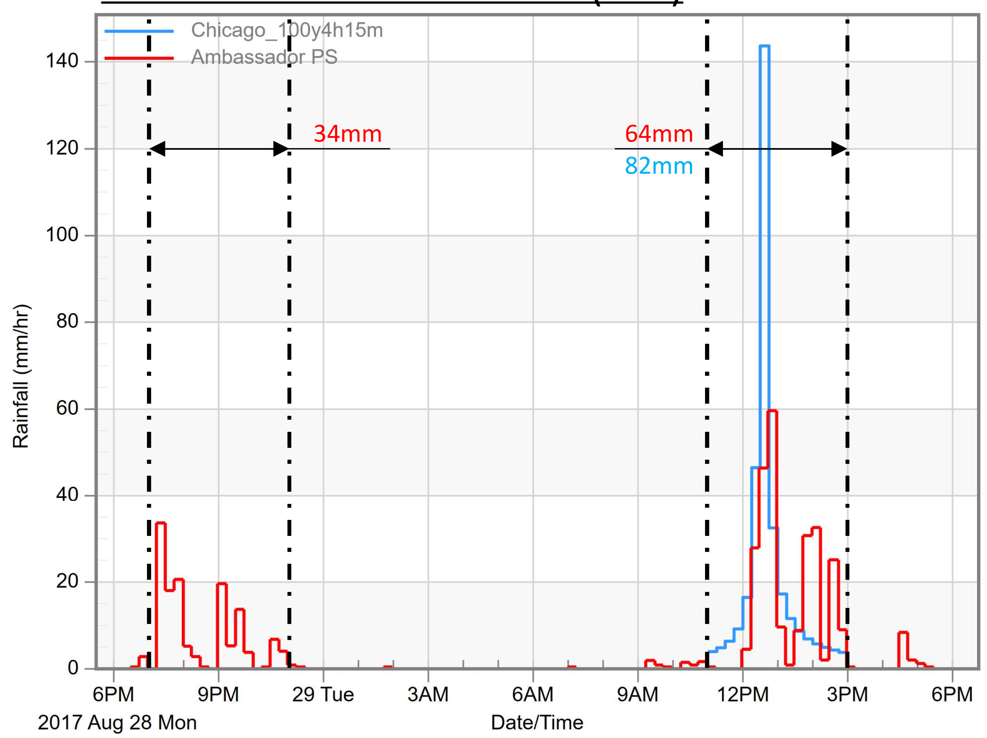
TEMPORAL VARIABILITY

As depicted by the graph on the left, the majority of the extreme 24-hour rainfall in August 2017 actually occurred as two separate storms with 14 hours in between – one Monday evening generally between 6pm & 10pm and the second Tuesday afternoon generally between 12pm and 4pm.

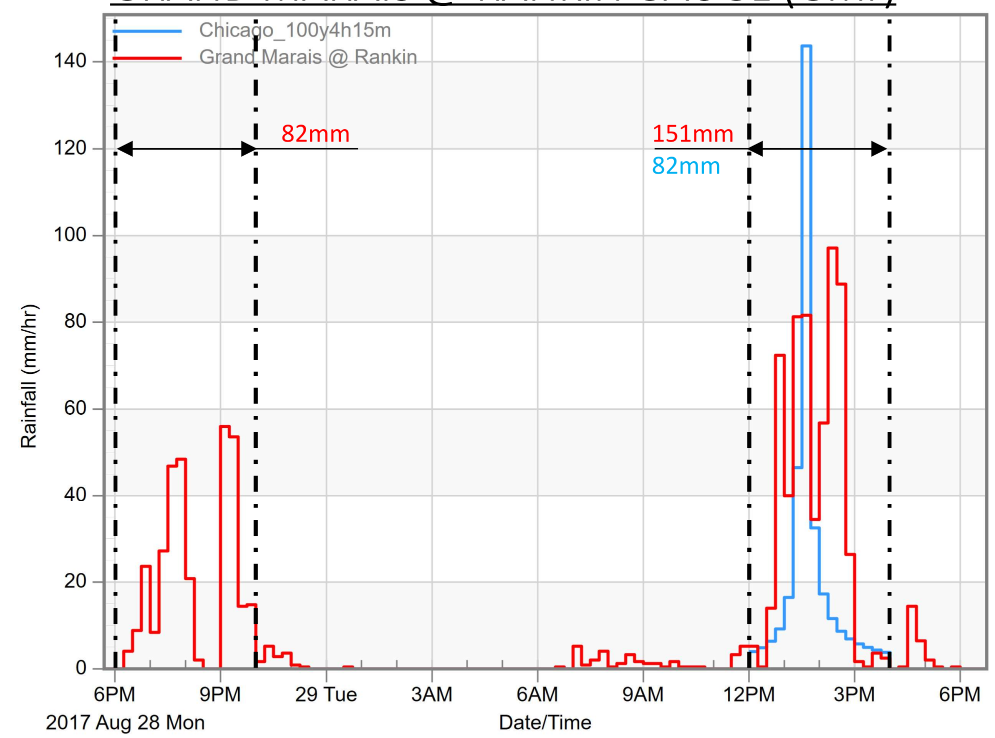
KEY TAKEAWAY: VIEWED SEPARATELY, THE DESIGN CHICAGO 4-HOUR RAINFALL IS MORE INTENSE AND VOLUMINOUS THAN EACH OF THE TWO RAINFALL EVENTS AVERAGED OVER THE TURKEY CREEK WATERSHED. THE GRAND MARAIS GAUGE CAPTURED THE MORE LOCALIZED INTENSE RAINFALL.

Landmark Engineers Inc.

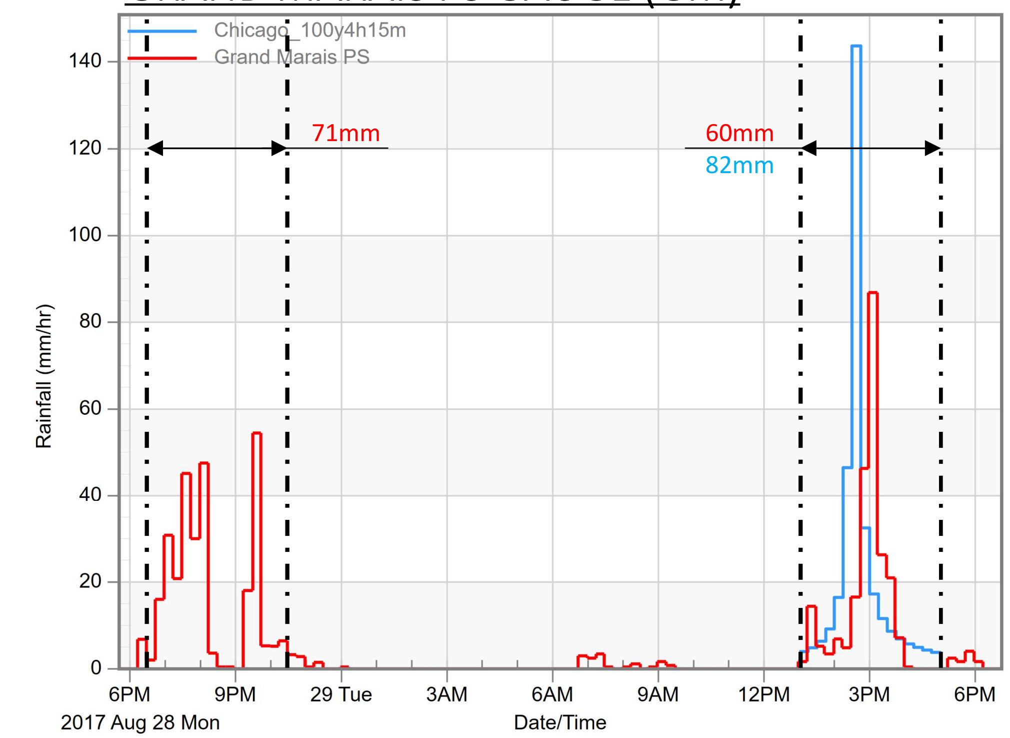
AMBASSADOR PS GAUGE (AM)



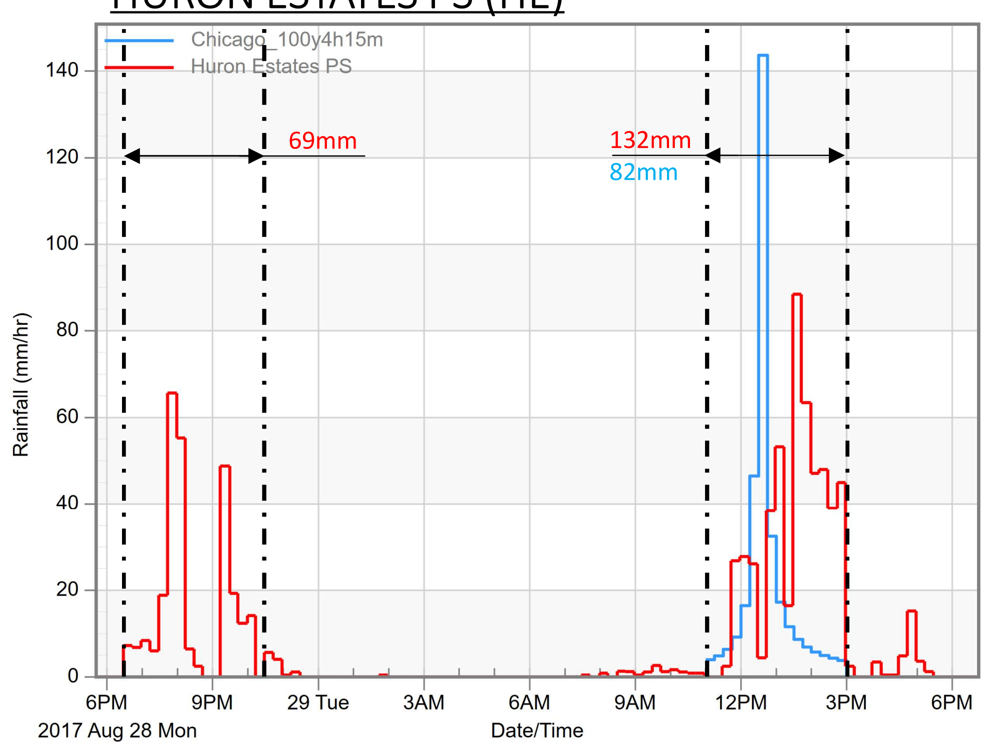
GRAND MARAIS @ RANKIN GAUGE (GMr)



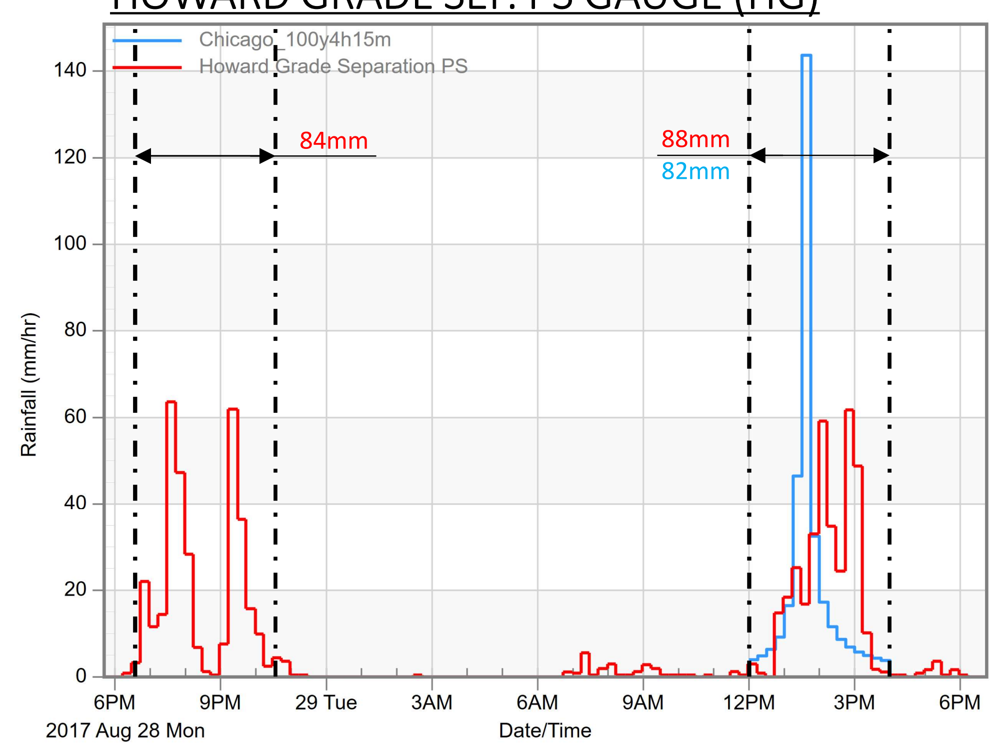
GRAND MARAIS PS GAUGE (GM)



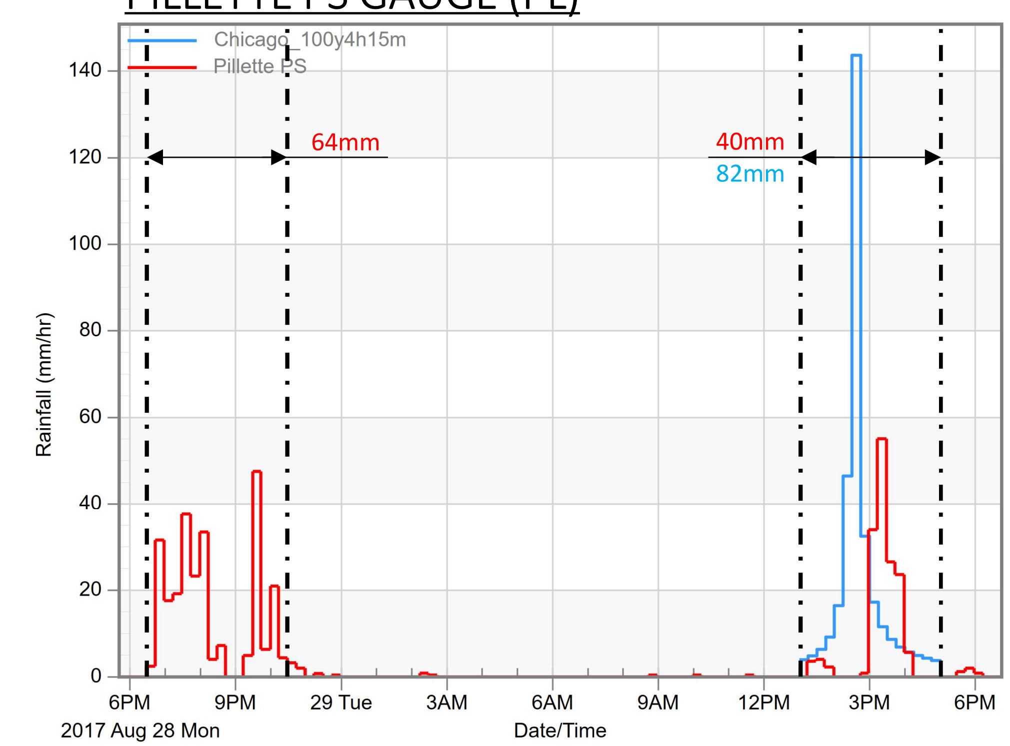
HURON ESTATES PS (HE)



HOWARD GRADE SEP. PS GAUGE (HG)



PILLETTE PS GAUGE (PL)



KEY TAKEAWAY: AS DEPICTED BY THE GRAPHS, THE TWO DISTINCT AUGUST 2017 RAINFALL EVENTS WERE NOT MORE INTENSE THAN THE STANDARD CHICAGO 100-YEAR 4-HOUR DESIGN STORM. MOREOVER, ON AVERAGE, THE DESIGN STORM VOLUME EXCEEDS THE VOLUME OF THE LARGER AUGUST 29TH AFTERNOON STORM EVENT.

DEFINING 'INFILL DEVELOPMENT' FROM SWM PERSPECTIVE

The term 'infill development' may have a different meaning from a planning versus a stormwater management perspective. Within the context of stormwater management, infill development should consider any development within an area that was built-up before the need for stormwater management was recognized (or areas built to an older SWM standard). For these types of developments, the minimum objective is to ensure that the infill does not adversely impact the existing condition. Infill development should never make things worse, nor should it be required to improve or rectify an existing sub-standard condition - unless there is a practical opportunity and a willingness to do so on the part of the developer, municipality, or relevant partners/stakeholders.

FLEXIBILITY IN LEVEL OF SWM QUANTITY CONTROL TO SUIT DIFFERENT SITES

Infill development SWM quantity control requirements are categorized under four levels: Normal, Exempt, Basic and Enhanced. The varying levels provide some flexibility to the Municipality with a means to apply tailored SWM requirements based on the size and nature of the proposed development.

Infill SWM – Normal

This SWM level will generally apply as the default requirement for all infill development. Allowable rates are based on the capacity of the receiver. For normal SWM measures, storage requirements are based on an assumed constant release rate. While it is understood that this assumption does not represent the actual varying outflow of a gravity outlet, it provides a consistent and equitable standard for all properties, irrespective of the varying conveyance capacity and hydrodynamics of the local sewer under any given storm event. Moreover, it is a practical standard that does not place undue burden on individual properties – properties that should be considered as infill development from a stormwater perspective.

Infill SWM – Exempt

This SWM level will generally apply to minor infill development activities such as: the paving of existing gravel parking lots; and small parking lot expansions and/or building additions of less than 5% of the total property area. In these instances, no specific SWM requirements are recommended, although the implementation of SWM measures should be encouraged to the extent that is practical. This level is applicable to substantially developed properties and is not applicable to low impervious properties that attempt to phase development into small pieces to avoid SWM measures.

Infill SWM – Basic

This SWM level will generally apply to infill development activities on properties with: existing imperviousness in excess of 60%; and, where a 'Normal' level of SWM is not practical (i.e., new storage cannot be reasonably accommodated via surface storage in parking lots and/or grassed depressions). A 'Basic' level of SWM is intended to meet the minimum objective of ensuring that the proposed development does not adversely impact the existing condition. The basic level of SWM is achieved by providing storage equivalent to the excess volume created by the proposed increase in impervious area.

Infill SWM – Enhanced

This SWM level will generally apply to infill developments where the consequences of storage exceedance (and subsequent spills onto adjacent lands) would pose an unacceptable risk. To be clear, our definition of 'unacceptable risk' means that the consequence of exceedance is likely to cause severe damage. It does not mean that the consequence of any negative or undesired outcome is unacceptable. Examples of this would include: the nuisance and access issues associated with surface ponding depths exceeding 0.3 metres; or minor flood damage from surface ponding encroachment onto buildings or vehicles. Such consequences are generally considered acceptable for extreme storms exceeding the 100-year floodproofing standard.

To achieve an 'Enhanced' level of SWM, an additional storage volume requirement due to Backwater (over and above the 'Normal' level requirements) is recommended:

- Additional Storage Volume due to Backwater (m^3) = Q_{allow} (m^3/s) x Assumed Duration of Zero Outflow (see section D-3.3.2 herein). Suggested durations for assumed zero outflow would typically be: 1 hour for typical 2-year minor system designs; 30 min. for more efficient drainage systems; 2 hours for sluggish drainage systems. *Site specific conditions may warrant a different assumed duration.

As an added measure, the 'Enhanced' level of SWM should also consider the potential to contain the additional storage volume from the Stress Test; to the extent that is practical (i.e., the additional storage can be reasonably accommodated via surface storage).